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Assessment of the Role of Drop Panel on Post Yield Performance of Flat Plate Framing System

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Huge respect and love for my Father for his unwavering support and guidance even at miles away, last but not least respect for my Mother for her unconditional love.

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ABSTRACT

Flat plate is among the commonly used flooring systems, simplicity in construction formwork set up and architectural freedom made this system very ideal and most practiced. Its use is limited to resisting vertical loading, its role as a primary lateral load resisting system is pulled back by its poor performance during cyclic earthquake loading. This poor performance; development of brittle punching shear hinge at a slab-column connection is the main reason for the system to be prohibited for use as a primary lateral load resisting system. Flat plate framing system uses a beamless plate for vertical load transfer and bracing of building. Connection capacity of this framing system is very small owing to the fact that beams are absent at connections. Unbalanced moment at the slab-column connection compromises the shear capacity at the shear critical region. This compromised shear capacity will result in a brittle punching shear failure during an earth quake loading. To overcome this connection failure, moment-shear interaction at slab-column connection needs to be properly modelled and be accounted for in analysis and design. Using eccentric shear model to analytically model moment-shear interaction at a slab-column connection, the role of drop panels in post yield response of flat plate framing system acting as a primary lateral load resisting system is studied. For this purpose 3-storey and 6-storey buildings were used as model buildings. A static nonlinear pushover analysis is used to compare post yield performances of these building models with and without drop panels. Out puts of nonlinear analysis showed that a drop panel helped avoid a brittle punching shear failure at slab-column connection. Drop panels on both model buildings have made yielding of flexural reinforcement to occur before punching shear failure, thus insuring a global ductile failure. Both the 3-story and 6-story model buildings have shown an increased base shear coefficient and roof drift ratio due to drop panels at slab-column connection. It is learnt that drop panels did not change the early stiffness of both building models. It is also learnt that early strength loss and sudden loss of stability on model buildings were improved by using drop panels at slab-column connection.

Key words: pushover analysis, eccentric shear model, drop panel, punching shear

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CHAPTER ONE - Introduction

In this chapter, overall performance of flat plate framing system and its response at a slab-column connection during cyclic earthquake loading is viewed as an introduction. An analytical model that helps model a close response of slab-column connection in flat plate framing system is overviewed. Also in this chapter, the role of drop panel in enhancing the lateral performance of slab-column connection is discussed. At last, a type of analysis method that is believed to adequately yield in a closer response of flat plate framing system during earthquake loading is given an introduction.

1.1 Overview

Flat plate framing system is a beamless configuration that a flat plate is directly connected to a supporting column. Vertical occupancy loading is transferred to supporting columns through strip of slabs acting as beams. The depth of the slab is relatively small and flexible. This makes flat plate framing system possess a weak bracing effect and leads them to a higher drift during earthquake loading.

Besides a higher drift, this framing system shows an early slab-column connection failure. Few building design codes including Eurocode, does not allow using flat plate framing system as a primary earthquake resisting system. The main reason for this is that, flat plate framing system experience a brittle punching shear failure at the slab-column connection during cyclic earthquake loading. A slab-column connection in flat plate framing system is weak owing to its small depth. These weak slab-column connection results in a connection unbalanced moment to compromise shear capacity at the connection and lead to a connection brittle failure. Thus, a moment-shear interaction has to be checked at the connection to avoid early brittle failure.

A moment-shear interaction at a slab-column connection is closely captured by eccentric shear model. During an earthquake loading, not all unbalanced moment at the slab-connection is resisted by flexure action. A portion of the unbalanced moment is resisted by a shear action (Wight and G. Macgregor 2012). Eccentric shear model accounts for moment-shear interaction by splitting the unbalanced moment to be resisted by two actions; flexure action and eccentric shear action. Flexure reinforcement is provided within a transfer width equalling $C_2 + 3h$ to resist portion of the unbalanced moment through flexure action. The other portion of the unbalanced moment is resisted by eccentric shear action at a shear critical region.

A shear critical region at a slab-column connection is subjected to two shear forces; one coming from vertical occupancy loading and the other from portion of unbalanced moment as eccentric shear. The connection needs to have enough shear capacity not only for the vertical gravity loading but also for the portion of the unbalanced moment coming as eccentric shear. If there is not enough shear capacity at the slab-column connection, the gravity shear and eccentric shear together might pass the connection shear capacity and lead to a brittle shear failure.

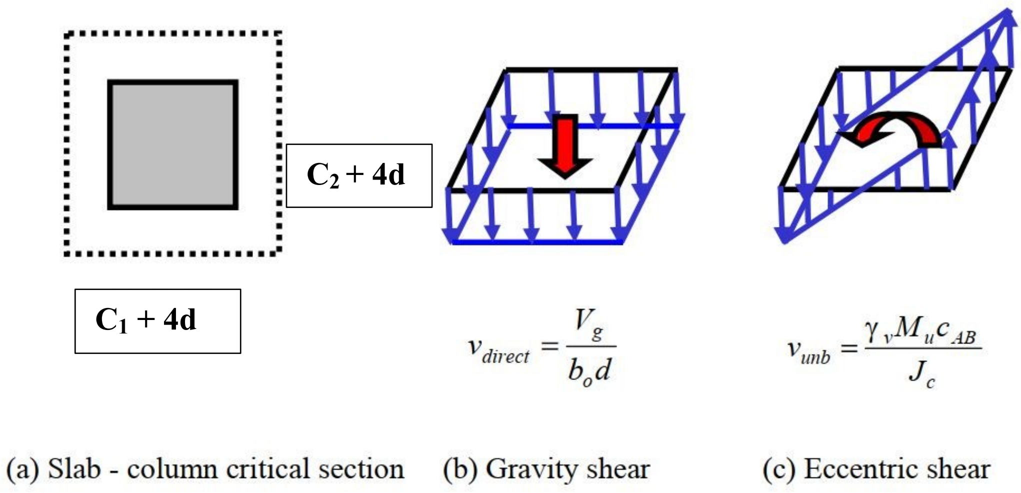


Figure 1. 1 : Slab-Column Critical Section and Shear Stress Distributions (Kang, Rha and Wallace 2003)

Slab-column connection needs to have an ample reserve of shear capacity to overcome problems with brittle punching shear failure. A way to check if a reserve capacity at a connection is large enough to avoid punching failure is see its shear ratio. Higher shear ratio has a small reserve of shear capacity and leads to a connection problem unless additional rehabilitation methods are used.

A way to reduce a shear ratio at a connection is to increase a shear resistance at shear critical region. Increasing the slab depth will definitely increase the shear resistance, but if the depth increase is for the whole slab width it might not reduce the shear ratio as the induced shear force is increased by the additional dead weight of the slab. Increasing the slab depth at only around the column will definitely help reduce the shear ratio by increasing a shear resistance at the critical region without significantly increasing the induced shear force. Drop panels are very much ideal to reduce connection shear ratio by increasing shear resistance at a shear critical region without adding a significant dead load to the slab.

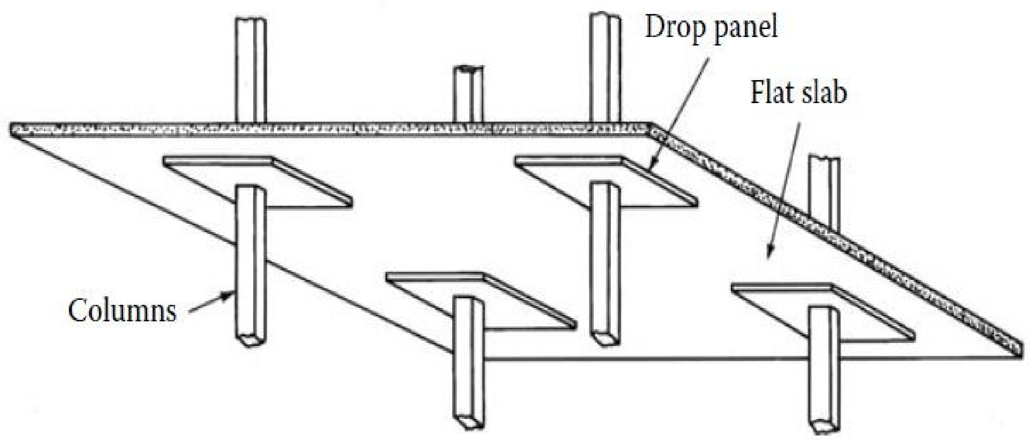


Figure 1. 2 : Flat Slab Framing System (With Drop Panel) **(**Taranath 2010)

Two building models with three and six stories, both having a similar regular plan and elevation layout are used to see the effect of drop panels in post yield stage. The post yield assessment helps tell if drop panels could overcome a brittle connection failure. The model buildings are designed for a gravity loading and a lateral loading equalling quarter of the buildings mass.

To closely capture post yield response of model buildings, a static nonlinear analysis is chosen. As the model buildings are in medium story range with regular plan and elevation layout, first mode is likely to be influential. For model buildings with an influential first mode, pushover analysis will yield in a closer response result.

1.2 Statement of the Problem

It is desired to see the influence of a drop panel in post yield performance of flat plate framing system acting as a primary earth quake resisting system. The major performance lack of flat plate framing system; development of a brittle punching shear hinge at a slab-column connection is to be examined if a drop panel helps avoid it. A desirable type of failure; yielding of flexural reinforcement in the transfer width prior to punching shear failure at a slab-column shear critical region is to be seen if prevailed under the role of a drop panel.

Overall performance of a flat plate system with drop panel is to be examined by a static nonlinear pushover analysis for model buildings with regular plan and elevation layout.

1.3 Objective of the Research

1.3.1 General Objective

The general objective of this research is to see the role of a drop panel in enhancing the post yield performance of flat plate framing system acting as a primary earthquake resisting system.

1.3.2 Specific Objective

The research will seek to see in details the role of drop panel in overcoming performance lacks of flat plate framing used as a primary earthquake resisting system. Some specific objectives to be addressed in the research work are listed below.

- See if a drop panel help bring up the desirable failure; yielding of flexural reinforcement in the transfer width occurring before punching shear failure at a shear critical region around the slab-column connection.
- See in details how a capacity curve is affected by the introduction of a drop panel.
- See if there is an increase in base shear capacity, global ductility and reserve capacity in flat plate framing system due to drop panels.
- Also, see the type of hinges formed and their distribution over the building structure.

1.4 Scope

Post yield performance of a flat slab system will be examined by using a nonlinear static pushover analysis. In the analysis material nonlinear property will only focus on one cycle stiffness degradation. Hysteretic behaviour of materials will not be modelled.

Analysis will only be done for lateral loading case. There will be no combined gravity and lateral load modelling.

Only regular building frames both in elevation and plan layout are chosen for different story range. As the response of irregular frames in elevation and plan layout is very much complex and hard to capture, it's pre-believed that seeking a good response from irregular flat slab frame system is almost impossible.

As the chosen system is regular both in elevation and plan layout, it is believed that contribution of higher modes is insignificant. As such a conventional pushover analysis method will be used. Higher mode contribution and torsional effect in post yield performance evaluation is not included.

1.5 Organization of the Research

Chapter 1: In this chapter, overall performance of flat plate framing system and its response at a slab-column connection during cyclic earthquake loading is viewed as an introduction. An analytical model that helps model a close response of slab-column connection in flat plate framing system is overviewed. Also in this chapter, the role of drop panel in enhancing the lateral performance of slab-column connection is discussed. At last, a type of analysis method that is believed to adequately yield in a closer response of flat plate framing system during earthquake loading is given an introduction.

Chapter 2: This section includes a review on few past researches conducted to study lateral performance of flat plate framing system and discussion on some technical aspects used in this research work to study post yield performance of flat plate framing system.

Chapter 3: This section discusses the reason behind for the selection of a model building with a specific configuration type. Dimensioning of model buildings and material used in model buildings is also discussed.

Chapter 4: In this chapter a detailed background on the method of analysis and how it will yield a closer response result to an actual response is discussed. Later in the section a modelling technique that would capture the close behaviour of primary components and a mathematical model that best describes a slab-column connection performance is discussed in details with relevant equations.

Chapter 5: This section includes parameters and input values used for the pushover analysis. Equations discussed the previous chapter are used to compute the corresponding parameters and input values. Response of model buildings both in the pre and post yield states are discussed in details with representative graphs. To help discuss the overall ultimate capacity of specimen buildings, a base shear to top storey displacement is plotted and initial hinge formations and their type are traced and identified.

Chapter 6: Over all conclusions drawn from the research work are summarized on this chapter.

2 CHAPTER TWO - Literature Review

Flat plate framing systems have long been used as a main structural component in building construction. They have been preferred for numerous advantages they have on architectural design freedom and construction simplicity. Thus, their performance needs to be boosted to cope with their demand by safe guard occupancy and go in harmony with design Codes.

This section includes a review on few past researches conducted to study lateral performance of flat plate framing system and discussion on some technical aspects used in this research work to study post yield performance of flat plate framing system.

2.1 Past studies on lateral performance of flat plate framing system

Numerous researches have been conducted to boost lateral performance of flat plate framing systems. In these researches, modelling and analysis techniques have been developed to closely capture the lateral performance of flat plate framing system. Also, in these researches overall performance of flat plate framing system is carefully studied and possible failure modes with brittle regions are identified.

In the reviewed research works, possible remedies are put forward to boost lateral performance of flat plate framing system. Their feasibility of use is identified for ranges of seismic zones.

Selected research works conducted on the lateral performance of flat plate framing system are reviewed. The driving idea behind the research works, modelling and analysis techniques used and their conclusion remarks are included below.

(Thomas H.-K. Kang, Changsoon Rha and John W. Wallace 2003) conducted a research to see a lateral response of a flat plate framing system with shear studs at slab-column connection. Two types of specimen buildings were put to a test, each were one third scale representations of typical flat plate frames constructed in moderate-to–high seismic zone in the United States. One of the specimen models was a conventionally reinforced concrete flat plate and the other was a nominally reinforced flat plate with post-tensioning tendons. The specimens were subjected to a gravity load and increasing intensity of uniaxial base acceleration histories on the shake table at Earthquake Engineering Research Center at UC Berkeley's Richmond Field Station. On the research work, shear studs were used to help increase the slab-column connection shear capacity and sufficiently resist the increase in punching shear coming from portion of the unbalanced moment. Possible failures at the slab-column connection were considered in the research. Load deformation curve for all connection failure types are constructed based on data's obtained from shake table test. Computer analysis results based on this load deformation curves were found to closely match the experimental findings. Columns are analytical modelled by accounting cracked stiffness. To analytical model flat plates, an effective slab width model with an effective width factor α of 0.8 for the RC specimen and 0.65 for the PT specimen were used. A cracking factor β of 0.33 for the RC specimen and 0.5 for the PT specimen were also used to analytical model flat plates. Computer analysis results based on these analytical models have shown a good correspondence to an experimental response for low-to-moderate levels of shaking. Slab-column punching failures occurred during the tests. Lateral drift ratios of 3% and 4% were achieved for the RC and PT frames respectively. Little loss of lateral load capacity was observed. Overall, the results indicate that the slab-column frames can be designed to have sufficient drift capacity to be used as a non-participating frame, or as a primary lateral force resisting system in low-to-moderate seismic regions.

A research conducted by (Jinkoo Kim and Taewan Kim 2008), lateral performance of flat plate framing systems not designed for seismic load were investigated. As an analysis method, capacity spectrum method based on ATC-40 and nonlinear dynamic analyses were used. Two specimen buildings with three and six story, where the three story model building only designed for gravity loading and the six story model building designed for both gravity and wind load were used. To analytical model specimen buildings, effective beam width ratio based on formulas from Luo and Durrani (1995) is used. A punching shear failure at the slab-column connection is modelled based on the procedures of Heuste and Wight (1999). Analysis results from capacity spectrum method show that the inter story drift responses of the non-seismic designed flat plate model structures subjected to earthquake with return period $2{,}400$ years were less than the limit states defined in system level. On the contrary, results based on reliability-based procedure of FEMA $355F$ $(2000b)$ shows that the model structures failed to satisfy the performance criteria for seismic safety.

(Virote BOONYAPINYO, Nuttawut INTABOOT, Pennung WARNITCHAI and Nguyen Hong TAM 2004) have conducted a research to evaluate the seismic performance of post tensioned slab-column connection frame only designed for gravity and wind loading. A nonlinear pushover analysis using SAP-2000 is carried out to see the ultimate capacity of model building. An equivalent frame model with an explicit transverse torsional member is used to model slab-column connection. Several factors are included in the analysis model such as P-Delta effect, strength and stiffness contribution from masonry infill and foundation flexibility. A nine story post tensioned flat plate frame building in Bangkok is used as a specimen model. Results from nonlinear push over analysis showed that the specimen building possess relatively low lateral stiffness, low lateral strength capacity, and poor inelastic response characteristics. The evaluation also showed that the slab-column frame combined with the shear wall system and drop panel can increase the strength and stiffness significantly.

(Sang Whan Han, Chang Seok Lee and Hyun Wook Kwon 2013) have done research to see the seismic performance of flat plate framing system designed only for gravity loading. A specimen building with three and seven stories were used. A hysteretic behaviour of slab-column connection is analytical modelled based on outputs from a shake table test. Plate is modelled using effective width method and columns are modelled using fibre section model to account for hysteretic behaviour. A slab-column connection is modelled by accounting the interaction of moment and shear. Eccentric shear model is used to analytically model moment-shear interaction. Two possible joint failures at the slab-column connection are accounted for in analytical model. A nonlinear pushover and nonlinear dynamic time history analysis were used to see the seismic performance of the specimen buildings. In the analysis result, nonlinear static pushover analyses indicated that the strength and drift capacities of flat plate frames are significantly affected by shear ratio. An increase in shear ratio has made the lateral strength and drift capacities to decrease significantly.

A research was conducted by (Thomas H.-K. Kang, John W. Wallace and Kenneth J. Elwood 2009) to propose an improved modelling technique for capturing the nonlinear behaviour of flat plate framing systems based on analytical and experimental findings. As a specimen building, two storey reinforced concrete and post tensioned slab-column frames were used. The modelling technique used in this research accounted for slab flexural yielding, slab flexural yielding due to unbalanced moment transfer and loss of slab-column moment transfer capacity due to punching shear failure. Column was modelled using a fibre model and plate is modelled using an effective beam width model. Slab-column connection is analytical modelled by accounting the moment-shear interaction and all possible joint failure modes. Inputs for back bone curve of joint spring were based on experimental finding. Hysteretic behaviour of slab-column connection is accounted for in the modelling. Computer analysis result based on the above modelling techniques has showed a close resemblance to an experimental finding. Thus it was concluded on the research that the proposed modelling techniques were adequate enough to capture seismic performance of flat plate framing system.

2.2 Eurocode and Flat Plate Framing System

On the guideline of Eurocode-8, it is clearly stated that the use of flat plate framing system as a primary lateral load resisting system is not covered. Guidelines available are on design and detailing of flat plates used as vertical load resisting system.

As flat plate framing systems experience a brittle slab-column connection failure during an earthquake loading, they are labelled as non-suitable framing system for the use as a primary earthquake resisting system in Eurocode-8.

Eurocode has given a coefficient β in computation for an induced shear stress to account for eccentric support loading. This coefficient is put to use to reduce connection shear ratio for the use in non-seismic participation of flat plates.

$$V_{Ed} = \beta \frac{V_{Ed}}{u_1 d} \qquad \text{(Eq. 2.1)}$$

Where:

V_{Ed} is induced shear stress

β is coefficient accounting for eccentric support loading

u_1 is length ofcontrol perimiter

d is mean effective depth of slab

2.3 Nonlinear pushover analysis

A pushover analysis is to incrementally apply a deformation at top story of centre of mass of specimen buildings and see what force is required and keep track of formation of hinges along the way. By the end of pushover analysis, a vital analysis output as a graph plot of bottom story shear to top story deformation is obtained. This plot is a capacity curve and helps tell the global ultimate capacity of the specimen buildings with location of hinge formation.

A plot of base shear to top story deformation, which is referred as a capacity curve helps tell the global ductility of the specimen buildings and a drift capacity. When this curve is combined with a specific spectrum curve, an output which helps predict the performance point could be obtained.

Using a specific spectrum curve based on a local condition, a top story deformation value could be read from the spectrum graph based on the period of the specimen buildings for the influential lower mode. Using this top story deformation value, a point on a capacity curve could be located and may represent the performance level of the specimen buildings.

As a general good practice, the performance point should be located in the middle portion of the platu of the capacity curve. This location usually results in a stable and efficient design. When checking for performance level of the rehabilitated flat plate specimen buildings with drop panel, it is required that the performance point is made sure to lie in the middle portion of the platu of the capacity curve.

2.4 Pushover Analysis Using ETABS 2016

ETABS offers two ways of defining pushover load case. The first is force controlled; that is to push the structure to a certain defined force level. The second is displacement controlled; that is to push the structure to a specified displacement (Habibullah and Pyle 1998).

To do a force controlled pushover analysis, loading on the structure should be pre-known. This pushover load case is less preferred and unsuitable for most cases.

Displacement controlled pushover analysis is preferred over force controlled as it is the most practiced method of analysis and gives computational efficiency in most cases (Oguz 2005).

In displacement controlled analysis the specimens are pushed passed their target displacement by as much as 150% and see what force is needed to do so.

In displacement controlled analysis using ETABS 2016, the program offers three ways of defining loading cases. This are uniform acceleration, user defined lateral load pattern and specified mode shape. The first, uniform acceleration is a loading case where acceleration is loaded as a load at a joint preferably at top story of centre of mass of the structure. The other, user defined lateral load pattern is to define a load case based on a desired static load pattern. The last, specified mode shape is to load the dominant mode shape of the structure as loading case.

In the modal analysis, loading is as force on the joint that is proportional to the product of a specified mode shape times its circular frequency squared (ω^2) times the mass tributary joint.

The other loading pattern, a uniform distribution, consisting of lateral forces at each level proportional to the total mass at each level may be used as a pushover loading pattern.

Accounting more than one lateral loading pattern help avoid uncertainties and varied distribution of response in the structure that arise from intense and cyclic earthquake loading as elements start to yield and stiffness characteristics change.

While using ETABS 2016 to do the nonlinear static analysis, member unloading after formation of hinges is as to ‘’Unload Entire Structure’’, as this method is good in computational efficiency and widely used in practice. Also positive increments will be saved in plotting capacity curve, as the whole intention of this research is to see the force deformation curve without points of hinge unloading (Oguz 2005). Including hinge unloading is thought to be useless in this research work.

As it is intended to develop a capacity curve at the end of the analysis, pushover analysis results are saved at multiple stages. Only positive increments are saved, including negative increments could create confusion. Conjugate displacement control is used in the displacement controlled analysis as it gives a reliable result and helps with problems that arise from convergence.

Sudden strength loss is to be avoided in the entire analysis as it complicates the analysis and gives unreliable result. Thus negative slope is limited to no steeper than 10% of its initial elastic stiffness.

In the pushover analysis the structure is pushed monolithically until a target displacement is passed. A displacement for the force deformation curve of the specimen buildings should range from 0% to 150% of the target displacement. A control node where later a story deformation is reported shall be place at the top story of centre of mass of specimen buildings.

The following general sequence of steps is involved in performing a nonlinear static pushover analysis using ETABS 2016.

1. Create a model with the conventional technique.
2. Create a user defined hinge properties and assign them to elements where a plastic hinge is likely expected.
3. Define a pushover load case as discussed in the previous paragraphs. Also, include a nonlinear dead load case where it will be used as a starting load for the pushover analysis.
4. Run the pushover Load Cases.
5. Review the pushover results: Plot the pushover curve, the deflected shape showing the hinge states, and print or display any other results that may help in result interpretation.

2.5 Point Hinge Definition

To define a plastic hinge in a computer based analysis, two options are available. These options available in ETABS 2016 are program default and user defined properties. Care should be given in the selection of the option that would represent the close plastic response of members. Program default uses an averaged value of commonly used members to assign plastic hinges. This makes the default assignment method not able capture effects that come from difference in plastic hinge length and effects of transverse reinforcement (Inel and Ozmen 2006). On the contrary user defined hinge properties give the option for the designer to assign a desired hinge data and accounts for different factors.

In plastic property assignment, plastic curve of force-displacement (force-deformation) is used for axial force and shear under force degree of freedoms. Plastic curve of moment-curvature is used for moment and torsion under moment degree of freedom (Computer and Structures, INC 2017).

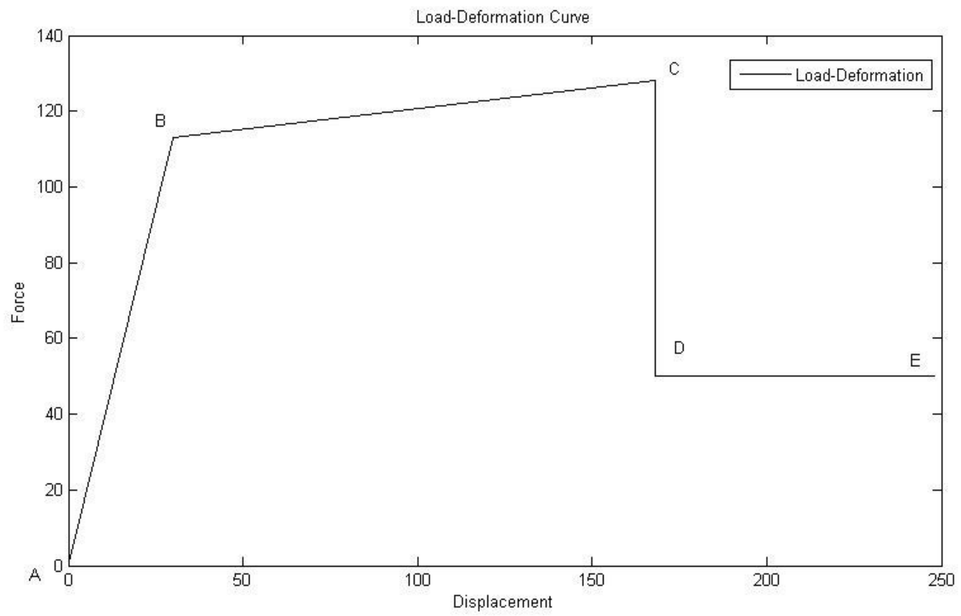


Figure 2. 1: Force Displacement Curve

A force deformation graph shown on figure 2.1 will be adopted to characterize pre and post yield stiffness of components of the specimen building.

Plastic deformation beyond point A will be on the frame element and accounts for elastic deformation, deformation beyond point B will be added to the elastic deformation and take place at the plastic hinge. As the hinges unload elastically, it does so without any plastic deformation.

When using ETABS 2016 for analysis, hinge deformation in the program will consider no deformation values at point B. Deformation values beyond point B will be used in post yield analysis. The displacement at point B will be subtracted from deformation at point C, D and E (Computer and Structures, INC 2017).

Scaling factor for force/moment and displacement/deformation values is used. A yield moment is used to normalize force/moment values so that point B on the curve would have a value equalling one.

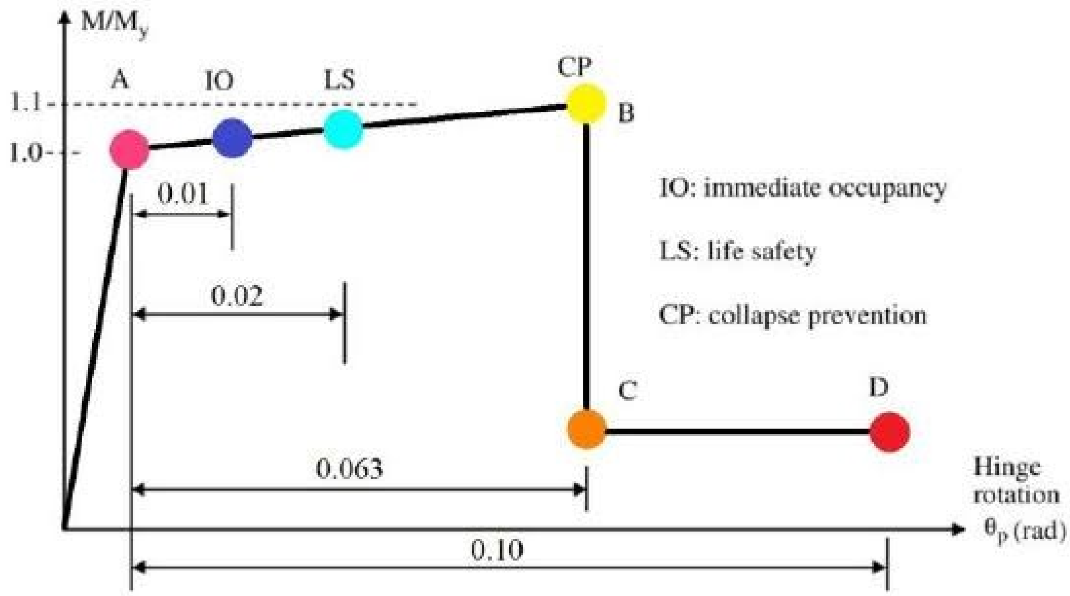


Figure 2. 2: Normalized Force Deformation Curve (Online image resource)

2.6 Shear Force at Slab-Column Connection and Its Interaction with Moment

A conventional beam-column frame transfers an unbalanced moment to a vertical column through horizontal beams. This is an obvious phenomena explained by the fact that, a load prefers a stiffer region to travel. Beams are relatively stiffer than slabs as they have larger depth, thus a floor load tends to travel to a beam from a supporting slab then to a vertical column.

In this end, an unbalanced moment and a connection shear force are adequate carried by a relatively stiffer beam. Any possible increase in a connection shear force due to moment–shear interaction during cyclic earthquake loading is also adequately resisted by a stiffer beam.

A flat framing system has a different case. As this framing system is beamless, unbalanced slab moment is transferred to a supporting column through a strip of slab acting as a beam. The convention slab depth in flat plate framing system is considered as small compared to a depth of a beam in beam-column frame system. As a result, section capacity of strip of slab acting as a beam in flat plate framing system is small. Any stress and interaction of moment and shear at a slab-column connection needs to be accounted for in an earthquake analysis and design

During a cyclic earthquake loading, unbalanced moment at a slab-column connection in flat plate framing system is resisted by two mechanisms; flexure action and punching shear. A portion of the unbalanced moment is resisted by flexure rebar within a transfer width at the connection. The rest of the unbalanced moment is resisted by a punching shear action at the connection (Kang, Rha and Wallace 2003).

In this respect a slab-column connection needs to be able to resist both shear forces that come from a conventional vertical slab shear load and a shear force from portion of unbalanced slab moment.

To closely model the actual phenomena at a slab column connection, a good analytical model that can capture moment-shear interaction has to be developed. To this end, eccentric shear model is used to analytically model moment-shear interaction at a slab-column connection.

There are two types of shear failures in flat plate system, a one way shear failure and a two way or punching shear failure. A one way punching shear failure takes place in the whole slab width and two-way shear failure takes place around slab-column connection. Two way shear resistance is usually lower than one way shear resistance (Wight and G. Macgregor 2012). In this research, only a two way shear failure at slab-column connection of flat plate framing system is studied.

ATC-40 dictates in its guide lines that the shear and moment transfer strength of the slab-column connection should be calculated by considering the combined action of flexure, shear and torsion in the slab at the connection with the column. The same principle is used in this research when analysing specimen buildings for lateral force.

(Wight and G. Macgregor 2012) discusses the interaction of shear and moment at joint using an eccentric shear model. This interaction should be considered while doing a lateral load analysis. The interaction should also be considered while trying to mitigate joint problems that come from cyclic earthquake loading.

Flat plate frame model should account potential yielding in flexure due to unbalanced moment transfer from the slab to the column within the slab flexural transfer width equalling: (Kang, Wallace and Elwood 2009)

$C_2 + 3h$

Where:

C_2	is column width perpendicular to applied loading
h	is slab thickness

2.7 Eccentric Shear Model

Eccentric shear model is an analytical model that helps closely model the interaction of moment and shear at a slab-column connection. During a cyclic lateral loading, a portion of an unbalanced moment is resisted by punching shear action at the slab-column connection. Eccentric shear model quantifies an induced shear force at the connection due to direct slab shear force and shear force from portion of unbalanced moment. A connection shear capacity has to be strong enough to resist both shear forces computed in eccentric shear model to avoid a brittle punching shear failure.

A flat plate model should also account for potential punching failure both prior to and after yielding of slab flexural reinforcement. In this end eccentric shear model is able to model slab-column connection and all potential performance requirements.

Transfer of unbalanced moment to column in flat plate framing system is assumed to occur through two mechanism; flexure action and eccentric shear. Portion of the unbalanced moment $\Upsilon_f M_{unb}$ is resisted by flexure action and the other portion $\Upsilon_v M_{unb}$ is resisted by eccentric shear (Kang, Wallace and Elwood 2009).

Portion of the unbalanced moment transferred through flexure $(\Upsilon_f M_{unb})$ must have a reinforcement placed at a width equalling $C_2 + 3h$. The remaining fraction of unbalanced moment $(1 - \Upsilon_f)M_{unb} = \Upsilon_v M_{unb}$ is transferred in eccentric shear on the slab critical section defined to extend $2d$ from the face of the column.

The resulting maximum shear stress accounting for both direct slab shear and shear from portion of unbalanced moment is shown below:

$v_u = v_{dir} + v_{unb} = \frac{v_g}{b_o d} + \frac{\Upsilon_v M_u C}{J_c}$ (Eq. 2.2)

Where:

v_{dir}	is gravity shear stress on the critical section
b_o	is width of critical section
d	is effective depth
v_g	is gravity force to be trnasfered from the slab to the column
v_{unb}	is shear stress on the section due to unbalanced moment being transfered in eccentric shear
c	is distance from the centroid of the critical section to the perimiter of the critical section
J_c	is polar moment of inertia of the critical section

v_u has to be less than the nominal shear stress capacity of the critical section.

$v_u \le v_n$

A fraction of unbalanced moment $\Upsilon_f M_{unb}$ is transferred by flexure over a transfer width of $C_2 + 3h$ centered on the column.

$\Upsilon_f = \frac{1}{1+\frac{2}{3}*\sqrt{\frac{b1}{b2}}}$ (Eq. 2.3)

$\Upsilon_v = 1 - \Upsilon_f$

Where:

$b1$	is width of the critical section parallel to the loading direction
$b2$	is width of critical section transverse to b1

Equation for polar moment of inertia for interior and exterior slab-column connection is shown below;

Interior;

$$Jc = 2\left[\frac{b1^3 d}{12} + \frac{db1^3}{12}\right] + 2\,(b_2\,d)\left(\frac{b1}{2}\right)^2 \qquad \text{(Eq. 2.4)}$$

Exterior

$$Jc = \left[\frac{b_2 d^3}{12} + \frac{db2^3}{12}\right] + 2\,(b_1\,d)C^2 \qquad \text{(Eq. 2.5)}$$

Where:

$b_1 = c_1 + 2\left(\frac{d}{2}\right)$ is length of the sides of the shear perimeter perpendicular to the axis of bending

$b_2 = c_2 + 2\left(\frac{d}{2}\right)$ is length of the sides of the shear perimeter parallel to the axis of bending

c_2 is width of column perpendicular to the axis of bending

c_2 is width of column parallel to the axis of bending

2.8 Failure at Slab-Column Connection

Flat plate framing system shows poor performance when subjected to lateral loading. One of the performance issues, slab column connection weakness is investigated in details in this research work. This weakness, at a slab-column connection is a brittle punching shear failure and a flexural reinforcement yielding within transfer width of the flat plate.

Portion of an unbalanced moment designated to be resisted by connection shear and the direct slab shear force form occupancy should be lower than the joint shear capacity. If not, a joint punching shear failure could happen before yielding of flexural reinforcement in the transfer width. As a mitigation method to avoid the brittle punching failure at slab-column connection, there should be a reserve shear capacity at the connection to adequately resist eccentric shear. One option to do this is to provide drop panels at the slab-column connection. The detail significance of drop panels is discussed in subsequent sections.

A slab-column connection provided with an adequate reserve of joint shear capacity by placing a drop panel at the connection will delay the presence of a brittle shear failure and brings yielding of flexural rebar first. With the introduction of drop panels at slab-column connections to boost the joint shear capacity, a ductile failure is guaranteed.

Several failure modes could be observed in laterally loaded flat plate framing system. Among them is development of flexural yield line across the full width of the slab. In this case, slab-column connection is strong enough to transfer the unbalanced moment and shear that develop due to the development of the flexural yield lines.

But if the slab-column connection is not strong enough to support the unbalanced moment and shear that develop for a flexural yield lines across the slab, slab-column will fail first.

Reinforced concrete slab-column frames tend to yield at approximately 1.5% lateral drift. A gravity shear ratio less than 0.4 is generally required to ensure displacement ductility ratios greater than one.

In slab-column connection failure, three failure modes could be present:

1. Failure of slab rebar; yielding of slab flexural reinforcement followed by punching shear failure. This flexural yielding occurs in the transfer width of $C_2 + 3h$.
2. Failure by punching shear; punching shear failure prior to flexural yielding of slab flexural reinforcement. Failure occurs in the critical region of $2d$ from the face of the column. This failure is a brittle type and less desirable.
3. Flexural yielding of rebar’s in the column strip. This segment is assumed to take 100% of the earthquake loading.

Punching shear failure at slab-column connection prior to slab reinforcement yielding occurs as the unbalanced moment proportioned to eccentric shear plus direct shear passes the limiting value of resistance connection punching stress.

2.9 Nominal Shear Capacity of Slab-Column Connection

Nominal shear capacity of each plate in the specimen building is quantified based on Eurocode guide lines.

The nominal shear capacity of plates is used to compute a shear ratio. Connection shear ratio is a ratio of induced shear force to nominal capacity. This ratio value helps tell the type of joint failure that may occur.

Shear resistance according to Eurocode should be checked along a defined control perimeter $U1$ at a distance $2d_{eff}$ from the loaded area.

$$d_{eff} = \frac{dy+dz}{2} \qquad \text{(Eq. 2.6)}$$

Where:

dy is effective depth in the longitudal direction

dz is effective depth in the transverse direction

Design punching shear stress in accordance to Eurocode is as follow;

$$V_{Rd,c} = C_{Rd,c}K(100\,\rho\,fck)^{\frac{1}{3}} \geq Vmin \qquad \text{(Eq. 2.7)}$$

Where:

fck is characteristic compressive strength of concrete in Mpa

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2, \text{d in mm}$$

$$\rho = \sqrt{\rho y\;\rho z} \leq 0.02$$

where ρ is a mean value of rebar ratio in the twoorthogonal direction taken within a width of C + 6d

$$C_{Rd,c} = \frac{0.18}{\Upsilon c}$$

$$Vmin = 0.035K^{3/2}\,fck^{1/2}$$

3 CHAPTER THREE - Model Building

3.1 Description of Model Building

This section discusses the reason behind for the selection of a model building with a specific configuration type. Dimensioning of model buildings and material used in model buildings is also discussed.

3.2 Background for Model Building

As it is to be briefly discussed in the coming sections, specimen buildings are in medium story range and have a regular plan and elevation layout so that they will have a dominant first mode response when subjected to earthquake loading.

As a good design practice, structures should reside to a simple and regular structural configuration to have their response to a tuned level. Flat plate framing systems posses a small lateral stiffens and are prone to excessive drift and excitation when subjected to a ground shaking. If their structural configuration is made irregular and if their exist abundance of indirect load paths, their response will be even more out of tune and result in disaster failure. Thus a regular and direct load path frame configuration of flat plate framing system should be considered for further rehabilitation with drop panel.

Response of irregular flat plate buildings is very unpredictable and requires additional lateral resisting mechanism to help contain their response. If additional lateral resisting mechanisms are used in flat plate framing system, the assumption of having this framing system as a primary lateral load resisting system is lost.

Flat plate framing system shows weak lateral performance even in regular configuration, having an irregular configured flat plate frame system would result in an even disastrous response. Thus this research promotes rehabilitation of a regularly configured flat plate framing system with drop panels. It is believed that rehabilitating of irregular configured flat plate framing system with drop panel is almost impossible.

3.3 Structural Configuration of Model Building

As research specimen buildings, two flat plate building models are chosen from medium story range to help show the extent of their lateral performance capacity. As such a three and six story building models are chosen.

Both models have a direct vertical load path and meet the requirements of regularity both in plan and elevation. This configuration property and the fact that the model buildings are in a medium story range, their response is largely influenced by the first mode and results in a tuned excitation.

Both building models are to be studied for their response as a flat plate and a flat slab with drop panels at all slab column connection. As for the lay out both model buildings have a three bay by three bay plan layout with bay width of $4m$ and all stories in both models have a height of $3m$.

The model buildings are assumed to have a fixed end base connection and are modelled in a computer model accordingly.

Building models are assumed to serve for a conventional residential use and loading on the buildings is according to Eurocode's guideline.

3.3.1 Member Sizing of Model Building

Computing lateral load based on Eurocode guideline is impossible as there is no guideline on what coefficient to use for flat plate framing system. It is clearly stated on Eurocode guide line on earthquake design that it does not cover performance of flat plate framing system acting as a primary lateral load resisting system. Eurocode only give design and detailing guidelines of flat plate framing systems serving mainly as a vertical load resisting system.

For this research work, flat plate framing system need to be designed to resist some amount of lateral load so as during a pushover analysis the columns would not yield before connection yielding is observed.

If columns are not designed to resist some amount of lateral load, they might yield before other critical regions yield and result in the overall building stiffness reduction and give unrealistic capacity curve.

In a conventional design, columns are made to be stiffer than other structural element as they need to resist both vertical and horizontal loads.

In this end, model buildings are designed to resist laterally quarter of their total mass. Member sizing such as column size is based on this rough but conservative approach. All other performance checks such as stability and plate deformation are done conventionally.

Global stability is checked for the applied lateral load and the result came out below Eurocode safety limits. Check for local stability is found non-important as all vertical columns are stocky.

As for the flat plate thickness, a thickness that satisfies a deflection limit of $L/200$ for long term nonlinear cracked analysis is used. Accordingly plate thickness of $150mm$ is used.

Flexural reinforcement for slab is based on a finite element analysis for the applied vertical loading.

Equivalent frame method is used to analytical model flat plate framing system. Thus, to comply with equivalent frame method a planner model is used for a further nonlinear push over analysis. Accordingly axis-B is used for both specimen buildings.

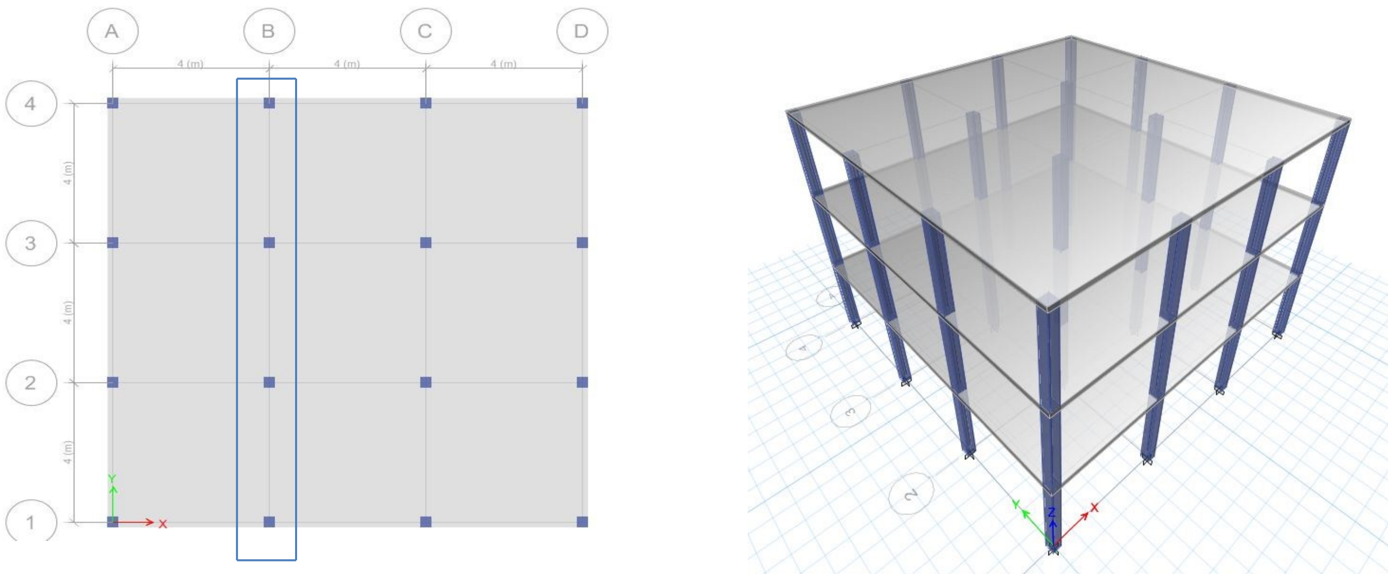
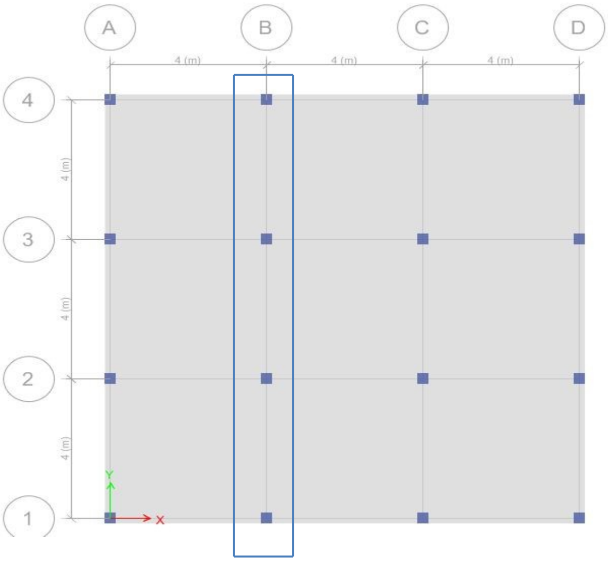


Figure 3. 1 : Plan and 3D of 3-Storey Model Building (ETABS 2016)



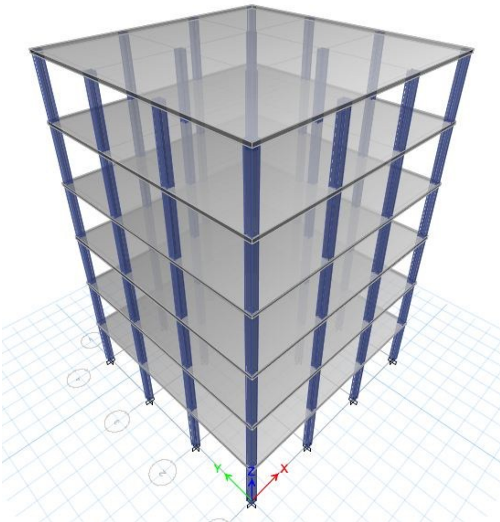


Figure 3.2 : Plan and 3D of 6-Storey Model Building (ETABS 2016)

4 CHAPTER FOUR - Modelling and Analysis

In this chapter a detailed background on the method of analysis and how it will yield a closer response result to an actual response is discussed. Later in the section a modelling technique that would capture the close behaviour of primary components and a mathematical model that best describes a slab-column connection performance is discussed in details with relevant equations.

4.1 Method of Analysis

One of the key goals in engineering is to capture a close response of a structure when subjected to loading, to do so choosing the right method of analysis is primary concern. The chosen method of analysis should be able to closely yield a closer response result to the actual response.

The end result that is needed for final remarks and conclusions in this research is a post yield performance and nonlinear properties that come from material yielding. To do so, a simple and most practiced method of analysis is chosen to meet the research objectives. This method of analysis is called nonlinear pushover analysis.

4.1.1 Why Chose Pushover Analysis?

As a method of analysis, nonlinear static pushover analysis is chosen in this research work as it is intended to see the post yield performance of specimen buildings. This post yield performances include; types of hinges, enhancement of capacity curve with the introduction of drop panel and overall failure mode of specimen buildings. It should be clear that elastic analysis is not able to capture all this performances.

Though elastic analysis is simple and most practiced, it lacks the capacity to capture a nonlinear portion of a material behaviour. It also comes with lots of uncertainties and limitations in predicting the close ultimate capacity of members. On the contrary nonlinear analysis closely captures nonlinear portion of a material and closely predicts the ultimate capacity of members.

As the whole plan of this research work is to see the nonlinear property of flat plate framing system that come from material yielding, pushover analysis is believed to closely capture this nonlinear property.

Pushover analysis is better suited for non-tall structures; buildings with medium height. A reason for this is that, tall structures are flexible and exhibit dominancy of higher modes while non-tall structures have dominancy of lower modes. Only lower modes are closely captured by a pushover analysis. Nonlinear response of tall structure could only be captured by a dynamic nonlinear analysis (krawinkler 1998).

The research specimen buildings are all in medium story range and their influential mode is of lower primary mode. Pushover analysis is suited for such building types to closely capture their nonlinear response.

Pushover analysis is suitable in capturing overloading in potentially brittle elements such as columns and connection (krawinkler 1998). As this research work is aiming to expose slab-column connection shear overloading, the writer of this research strongly believes that static pushover analysis is the most suitable method of analysis.

Nonlinear pushover analysis greatly helps capture if there exists story mechanism and overloading in brittle members in flat plate framing system (Oguz 2005). As the main goal of this research is to closely capturing the post yield performance of flat plate framing system that involves story mechanism and connection brittle failure due to shear overloading, pushover analysis is likely to yield a closer response result.

Building specimens used in this research have a regular plan and elevation layout with uniform mass distribution. This makes the specimen buildings to have their large mass participate in the primary mode. This implies that first mode is likely to be influential.

Static pushover analysis is ideal for structures that have influential first mode. Which in this case all specimen buildings can be analysed using pushover analysis as the have an influential first mode.

As a static pushover analysis is going to be used as an analysis method, participation of higher modes should be made sure non-influential. Influential higher modes make the pushover analysis less credible (Fema-356).

One way to make sure that specimen buildings have an influential primary mode is to deal with their configuration in the early conceptual design phase and keep the layouts as simple as possible. In this regard, specimen buildings in this research work meet the conceptual design goal for simple and influential first mode building response.

Other way to check that higher modes are non-influential is to run two response spectrum analysis; one with multiple modes with 90% mass participation modes included and the other by only considering the first mode. Influence of higher modes shall then be discarded if a story shear at any floor level from multiple mode analysis is lower than 1.3 times the shear force of first mode analysis (Fema-356).

4.1.2 Computer Based Analysis

As modelling and analysis are very important aspects of the research, the choice of a computer program to do this is even important. A reliable and most acknowledge computer program to handle a sensitive nonlinear analysis is chosen from number of options. In this respect ETABS 2016 from CSI is used for modelling and analysis.

ETABS 2016 offers powerful tools to model structures for nonlinear analysis and run a pushover load case. Output results are very composed and offer a clear overview of the ultimate performance of a structure. Graph plots and hinge tracking are viewed with a good graphics interface. This helps eases analysis interpretation.

In this research work the most practised method and computationally efficient method called displacement control pushover analysis is used.

As FEMA-356 dictates to use two types of loading patterns for push over analysis, modal and triangular load pattern are used. This approach helps reduce response uncertainties that arise from joints unloading due to hinge formation and changing of the dynamic response of the structure.

According to FEMA-356 at least two vertical distribution of lateral load shall be applied, one pattern selected from modal pattern loading and the other from uniform distribution.

From a modal pattern, a vertical distribution proportion to the shape of the fundamental mode in the direction is considered. Use of this distribution shall be permitted only when more than 75% of the total mass participates in this mode. As all specimen buildings in this research have their large mass participate in the fundamental mode, the use of modal pattern with vertical distribution proportional to fundamental mode is found appropriate.

In load pattern definition for pushover analysis of specimen buildings, modal and user defined lateral load distribution is chosen over uniform load pattern as this load pattern underestimates deformation values and is less reliable pattern definition.

Uniform load pattern is good in response prediction in the elastic range, but its response prediction is very overestimating in inelastic range, as such it is less likely to capture representative response in plastic state (Oguz 2005).

Modal load pattern based on the first mode of specimen buildings and a predefined lateral load pattern based on story mass of specimen building is chosen as a pattern definition in the pushover analysis. A final result from the two patterns that closely reside to actual post yield performance is adopted for final remark.

4.2 Modelling

In any engineering analysis, modelling is the most important part and the one that requires a great deal of attention. In this respect, a computer model that closely resembles the actual properties of components of the specimen buildings is used. Both the elastic and plastic portions of the components of the specimen buildings are carefully quantified and are used in the model. Analytical model for the slab-column connection property in the specimen buildings that closely resembles the actual response is given a great deal of attention and a best fit analytical model is used accordingly.

4.2.1 Member Modelling

FEMA-356 designation of components of building as primary or secondary element is based on their significant contribution in lateral force resistance and contribution of preventing collapse. Based on this designation, both columns and flat plate in the specimen building are considered as primary elements.

In the modelling, vertical primary seismic elements of a flat plate framing system are all modelled as line elements.

Flat plates are modelled as shell elements. These plates are refined to a $1mX1m$ rectangular mesh size to help yield in a closer response. Connection rigidity in slab-column connection could be accounted for in a computer model by using a rigid panel zone at the connection (Park, Han and Kee 2008). As such, a rigid panel zone is used at slab-column connection of the computer models of the specimen buildings.

Behaviour of components in the specimen building whether they are deformation controlled or force controlled is decided based on their force-deformation curve. This helps set up the backbone curve as it tells how components will ultimately fail.

Primary Components are classified as deformation controlled or force controlled based on their load deformation curve. Type 1 curve in figure 4.1 represents a deformation controlled component if only $e > 2g$. Type 2 curve in the same figure could be termed as deformation controlled if only $e \geq 2g$. If the above conditions are not met, component is likely to have brittle failure and is represented by type 3 curve in figure 4.1 as a force controlled component.

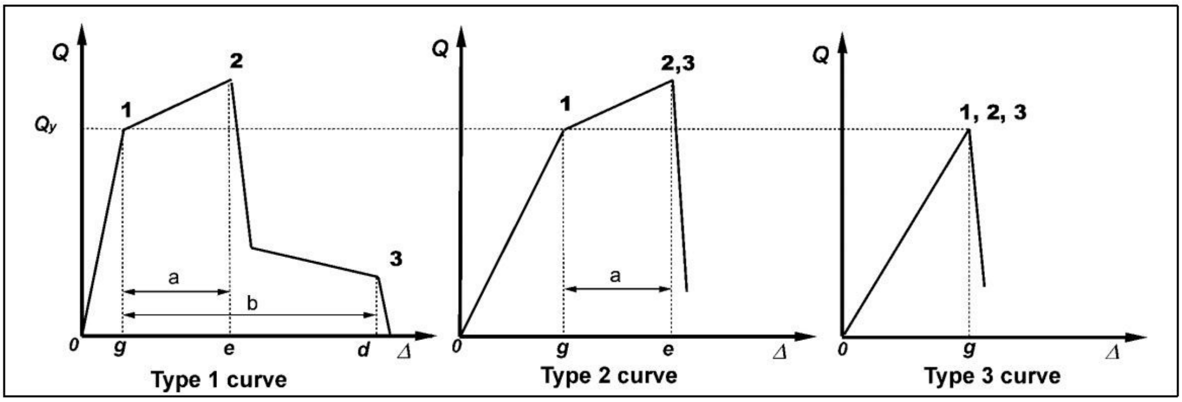


Figure 4. 1 Component Force to Deformation Curves (FEMA-356)

In the analysis, primary elements are assigned as line elements. The analysis program initially gives these elements a linear material property, later in the analysis a nonlinear material property to these elements is assigned by using point hinges at the end node of each elements. These end nodes are points where maximum flexure and shear is likely expected.

A user defined hinge property to be assigned to elements is based on both material and member property.

4.2.1.1 Plastic Hinge

One of the methods used to assign a nonlinear property to components of specimen buildings is to use plastic hinges at locations where they are expected to develop first. This plastic hinges are placed where the member is likely to yield first. Effect of plastic hinges in the analysis is only prevailed in the post yield response of members.

In the analysis, plastic regions are assigned to element ends as point hinges. Although ETABS 2016 offers other option such as links to assign plastic regions, point hinge is used in the computer based modelling and analysis.

As the specimen buildings are not to be designed as a code based ductile building, they would not have sufficient column tie reinforcement. Poorly detailed column hinges could only be captured by user defined hinge assignment (Inel and Ozmen 2006).

User defined plastic hinge enables the user to account effects of different hinge length and confinement effect due to transverse reinforcement (Inel and Ozmen 2006). In this end, user defined plastic hinge is more preferred and used in this research.

For the purpose of quantifying moment and curvature relation for both elastic and plastic regions of members of the specimen building, a computer program developed at the University of Toronto based on a modified compression field theory called Response2000 is used.

To capture post yield response of an element in the specimen buildings, plastic hinges are placed at locations on the element where a desired effect is to likely to attain a yielding value. In this respect, members of the specimen building are assigned plastic hinges at each end nodes.

A slab-column connection shear failure that occurs due to cyclic earthquake loading is a brittle type and does not show signs of ductility. In the computer modelling, ductility is not considered in the slab-column connection shear hinge and even on any other element where shear hinge is expected (Inel and Ozmen 2006).

In the analysis, assigned hinges will only affect the response and performance of structures after yield point. Their effects come in place during the nonlinear analysis and affect the plastic response of the test specimen buildings.

Refining hinges will yield in a steeper downfall of load deformation curve and helps capture a brittle failure. As this research seeks to find possible connection shear failures, hinges are refined in the computer based analysis. Hinge override helps in avoiding steeper negative slope and hinges are discretised in relative length.

Hinge subdivision help capture better results when two hinge types are present at the same point. Recommended hinge sub division ranges from 0.02 to 0.25. In this research hinge subdivision of 0.12 is used

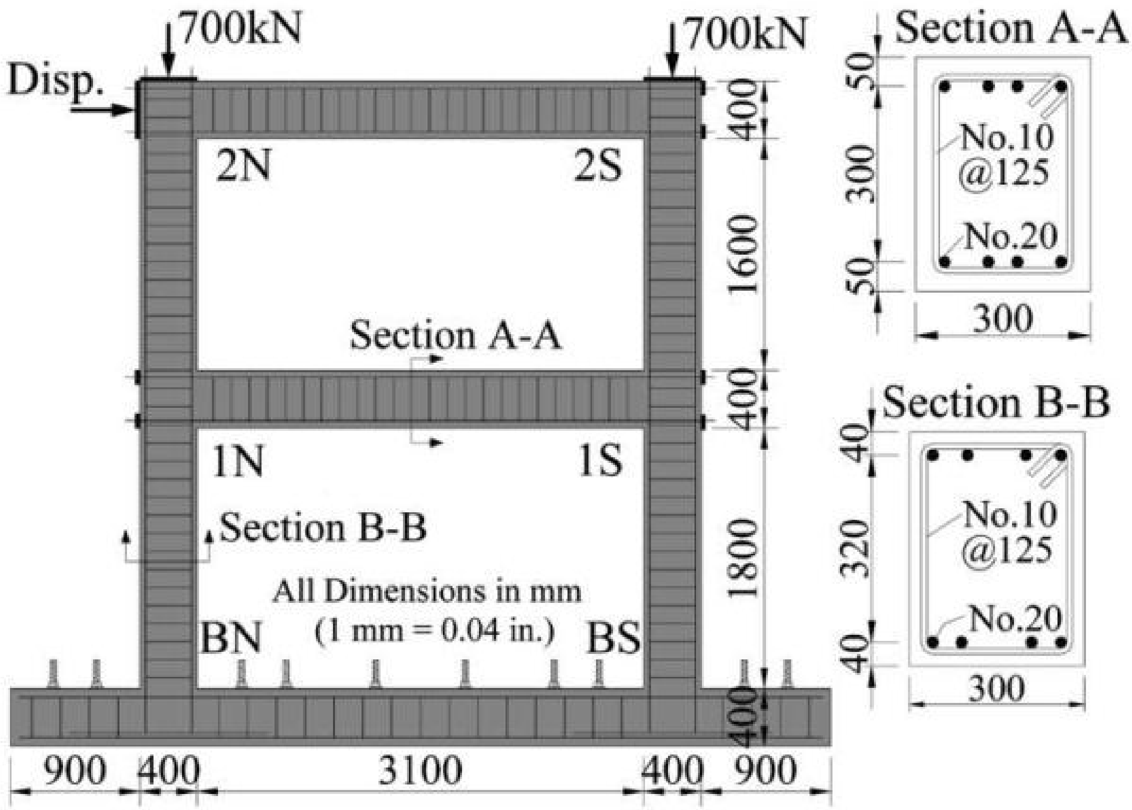
Degrees of freedom that are not assigned plastic hinges will remain elastic for the entire analysis and will not show any plastic property.

To assess performance levels, occupancy level designations will be adopted as necessary from FEMA-356 tables. This performance designation does not have any effect in the entire analysis result. Acceptance criteria for component of a structure shall be referred from table 6-7 and table 6-14 of FEMA-356.

4.3 Verification of a Computer Based Pushover Analysis

Static nonlinear push over analysis for model buildings is conducted using a finite element based computer program. Outputs of a computer analysis should closely resemble to an actual response. To see how close a computer modelling and analysis results are to an actual response, a computer response outputs need to be compared to an experimental response output for closeness.

To do so, computer analysis result of a pushover analysis is compared to a response result of an experimental 2-storey frame built and examined by *Vecchio and Emara (1992).* The experimental frame is a 2-storey beam-column connection frame with cross-section sizes as shown on figure 4.2.



No.10 rebar equals 50mm^2 in area

No.20 rebar equals 300mm^2 in area

Figure 4. 2: Experimental 2-Storey Frame **(**Vecchio and Emara, 1992)

The experimental frame was laterally pushed on the top storey to a displacement of $155mm$ and a plot of top storey displacement to base shear was recorded.

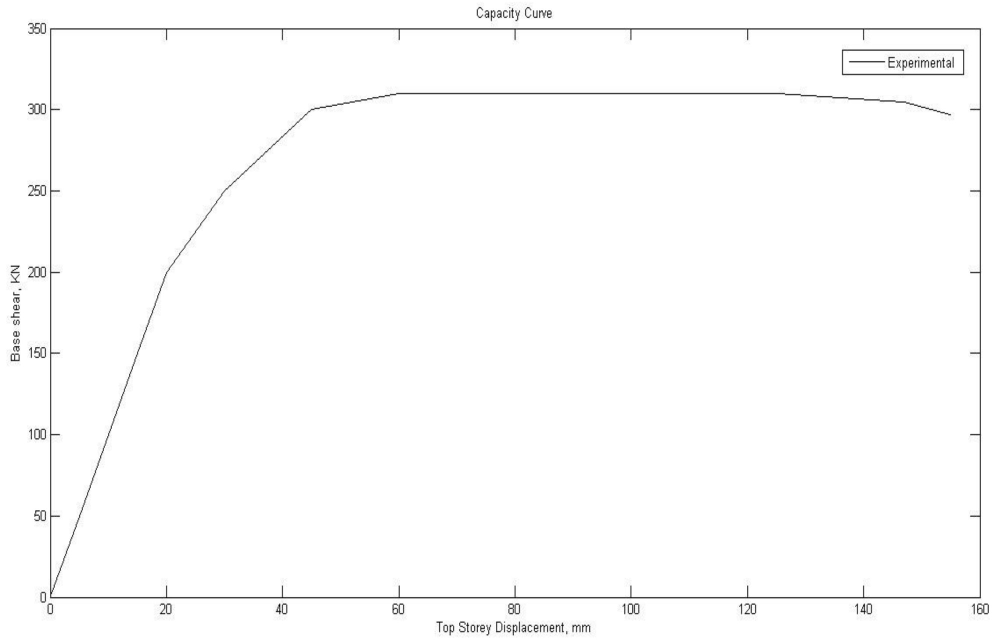


Figure 4. 3: Capacity Curve for an Experimental Frame

4.3.1 Comparison of Experimental Results to a Computer Output

Computer based modelling and analysis was conducted using ETABS 2016, in the modelling members stiffness was reduced by half to account for possible cracking. Moment Point hinges were placed on members were a possible moment yielding is likely to occur. Base connection on the computer model is assumed to be of a fixed end connection. A user defined hinge is used to assign point hinges. To help in computing moment-curvature for each member, Response-2000 developed at the University of Toronto is used.

A push over load case based on the first mode shape of the 2-storey frame is defined and used in the analysis. A plot of top story displacement to a base shear is generated for the computer model and plotted alongside the experimental result to ease in comparison.

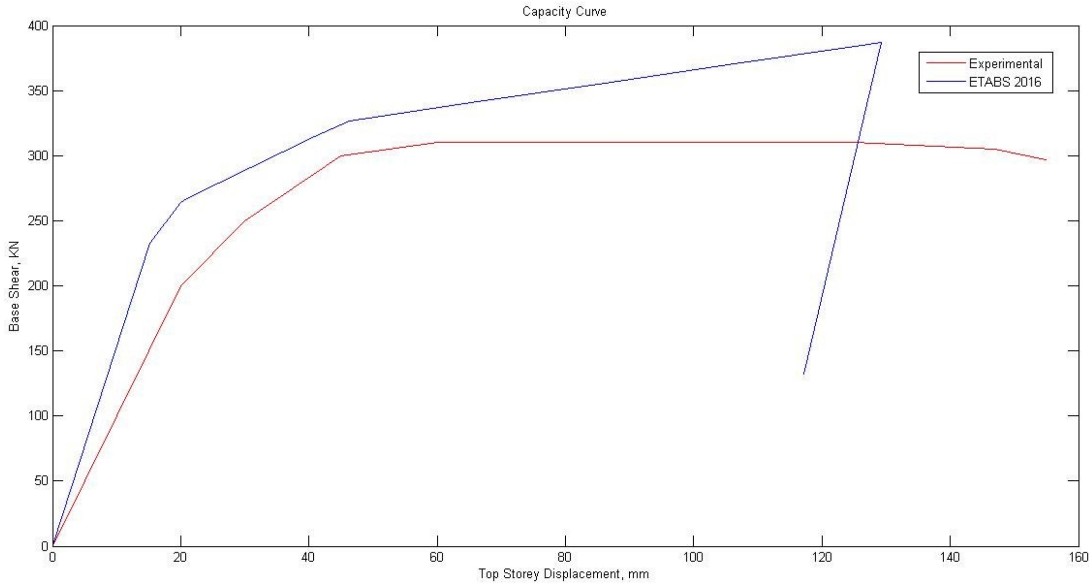


Figure 4. 4: Capacity Curve for Both Experimental Frame and Computer Model

Based on figure 4.4, a computer model was able to closely represent the performance of the experimental frame. Maximum top storey deformation and yielding base shear were closely captured by computer model. It is seen on the figure that initial stiffness of the computer model is slightly higher than the experimental frame. This could be explained by the fact that a fixed end base connection in the computer model could not closely model the actual connection of the experimental frame. Perhaps, a spring end connection model could have yielded in a closer result.

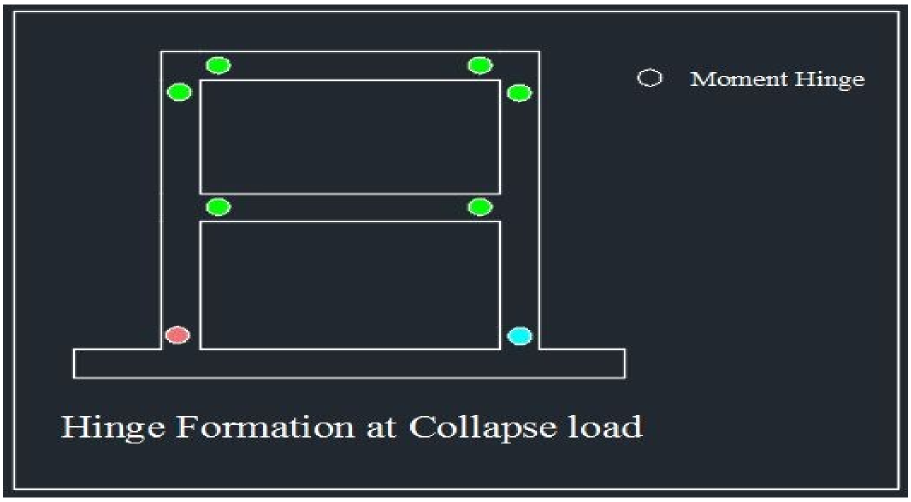


Figure 4. 5: Hinge Formation at Collapse Load of an Experimental Frame

4.4 Analytical Modelling

To closely capture the actual response of flat plate framing system, an appropriate analytical model plays a major role. A slab-column connection in flat plate framing system receives an additional shear force other than gravity shear force and undergoes a severe punching shear stress during a cyclic earthquake loading. This added shear stress comes from the unbalanced moment and gives an idea on how to propose an analytical model that accounts the interaction of shear force and moment at slab-column connection. (Han, Lee and Kwon 2013) The following sub topics give a discussion on an analytical model that best describes the interaction of shear and moment at a connection.

4.4.1 General Back Ground

Flat plate framing system serving as a main lateral force resisting system shows a large development of shear force at a slab-column connection as a punching type during a cyclic earthquake. This is due to the interaction of moment and shear force at the connection and this interaction needs to be accounted for in the analysis and design so as to avoid a brittle punching shear failure (Kang, Rha and Wallace 2003).

Unbalanced moment in the flat plate is transferred to a supporting column through a strip of slab portion. This unbalanced moment is not fully resisted by a flexure action of the slab strip, rather a portion of the unbalanced moment is resisted by a shear action at the slab-column connection. This brings out the fact that flat plate needs an additional punching shear capacity to meet the shear demand (Wight and Macgregor 2012).

To come up with an efficient way to cover the additional shear demand, an accurate analytical model that accounts for moment-shear interaction and a model that gives a quantitative data on how high the shear resistance should be to avoid a punching shear failure is proposed. In this respect eccentric shear model is used as an analytical model to account for moment-shear interaction at a connection.

If the moment-shear interaction at a connection is not fully accounted for in the design, there could be a possible punching failure at the connection. As time goes on more punching shear failures could take place in a single plate, this ultimately leads for the whole plate to crush down to a lower plate and lead to a progressive collapse.

As a good engineering practice in flat plate framing system, it is very well advised to have a continuous slab bottom reinforcement at the slab column connection to helps avoid a joint failure that leads to a further progressive collapse (Kang, Rha and Wallace 2003).

Continuing of some bottom reinforcement through the column joint will assist in the transfer of force and provide some resistance to collapse by catenary action in the event of punching shear failure. Bars could be considered continuous if they have proper lap splice, mechanical couplers or are developed beyond the support (ATC-40 Report 1996).

The above paragraph further reassures that, if a flat plate system is used as a primary lateral load resisting system its slab-column joint could experience a punching stress and lead to a progressive collapse. As a good practice, properly developed bottom rebar at column slab joint may avoid a progressive collapse.

In the analysis model, a progressive collapse is not explicitly modelled. As such a continuous bottom rebar is assumed to be provided for all specimen models at the slab-column connection.

Concrete confinement in column core due to column ties is accounted for in the model. Accordingly the confinement is accounted by increasing the compression strain of concrete.

Maximum compressive strain of confined concrete is referred from ATC-40, where

$$\epsilon_{cu} = 0.02\ for\ {}^{s}/_{db} \leq 8 \qquad \text{(Eq. 4.1)}$$

$$\epsilon_{cu} = 0.005\ for\ {}^{s}/_{db} \leq 16 \qquad \text{(Eq. 4.2)}$$

Where:

s	is longitudinal spacing of confining transverse reinforcement
db	is diameter of longuitdnal reinforcement

Values in-between could be interpolated.

A plastic hinge used in the model to account the plastic state of components of the specimen building is given a defined hinge length and assigned as a point hinge.

Plastic hinge length is determined using ATC-40 recommendation;

$lp = \frac{h}{2}$ (Eq. 4.3)

Where:

h is section depth in the direction of loading

$\theta p = (\Phi u - \Phi y)lp$ (Eq. 4.4)

Where:

θp is plastic hinge rotation

Material inputs for the modelling of specimen buildings are referred form Eurocode. In accordance to Eurocode provision, ultimate concrete compression strain is referred from table 2.1. Concrete tensile strength is read from table 2.1 of Eurocode for a corresponding cylindrical strength. Thus for $C20/25$, a tensile strength of $2.2Mpa$ is taken. Rebar ultimate strain is taken as 12%.

To cope with the additional demand in slab-column connection punching shear that come from moment-shear interaction, a drop panel with a specific dimension is used for the specimen buildings.

Accordingly, drop panels used to make a flat slab framing system should have a total depth of a least 1.25 times the thickness of the slab adjacent to the drop panel.

$Ht \geq 1.25 * h$

Where:

Ht is total thickness at joint, including drop panel

h is thickness of slab adjucent to drop panel

4.4.2 Equations of Analytical Model

To capture a closer response of flat plate framing system, a good representative analytical model is of a prior concern. To this end, a representative analytical model for slab, column and slab column connection is defined below.

4.4.2.1 Slab Model

Conventional flat plate modelling techniques used to model vertical occupancy loading such as equivalent frame method could not be used to model lateral loading in flat plate framing system as they give far less accurate response results (Park, Han and Kee 2008). In this respect, modified lateral load modelling technique by Banchik (1987) that is proofed to give closer response results is used for specimen buildings.

Slab is modelled using effective beam width model with the slab width reduced by an effective beam width coefficient α. A factor β is used to account for cracking (Banchik 1987).

$$\text{effective slab width} = L_2\ \alpha\ \beta \qquad \text{(Eq. 4.5)}$$

Where:

L_2 is transverse slab width

α is effictive beam width coefficient

β is stiffness reduction factor, $\frac{1}{3}$ is used in computation

To compute effective beam width coefficient, the following equations are used:

For interior span:

$$\alpha = \left(\frac{5C1}{L2} + \frac{L1}{4L2}\right)\frac{1}{1-V^2} \qquad \text{(Eq. 4.6)}$$

For exterior span:

$$\alpha = \left(\frac{3C1}{L2} + \frac{L1}{8L2}\right)\frac{1}{1-V^2} \qquad \text{(Eq. 4.7)}$$

Where:

$C1$ is column dimension in the loading direction

V is poisson ratio of concrete

$L2$ is slab width transverse to loading

$L1$ is slab width in the direction of loading

4.4.2.2 Column Model

Column is modelled as a line element with point plastic hinges at moment critical regions. Moment-curvature for both elastic and plastic segment of column accounting for confinement due to transverse reinforcement is computed using Response2000 and fed in to back bone of plastic hinge.

4.4.2.3 Slab-Column Connection Model

The most critical region in flat plate framing system is the slab-column connection. This connection undergoes sever punching shear stress during an earthquake loading as portion of an unbalanced moment starts to compromise the connection shear capacity. A model that closely predicts the actual response of slab-column connection helps in the process of providing a mitigation method. Slab-column connection is modelled by accounting the interaction of shear and moment at the joint.

Eccentric shear model used to analytical model a moment-shear interaction at a slab-column connection in a planner frame computer based nonlinear analysis has shown a close match to a response of an experimental result in a research conducted by (Han, Lee and Kwon 2013). In this respect eccentric shear model is used to analytically model slab-column connection in planner frame nonlinear analysis of model buildings.

To account for connection rigidity, a rigid end zone factor of 1 is used in the computer model for a length of half of the columns width.

At a slab-column connection, two types of failures are modelled based on eccentric model. These are shear dominant failure and flexure dominant failure.

4.4.2.3.1 Shear Dominant Failure

Shear dominant failure takes place as the summation of the portion of the unbalanced moment carried by eccentric shear and direct shear force from occupancy passes the capacity of the critical regions shear capacity.

Slab-column connection that does not have a reserve shear capacity to resist an added shear demand that comes from unbalanced moment, may fail in a brittle manner. This failure can be avoided by boosting the joint shear capacity by introducing a drop panel at the connection.

The unbalanced moment (M_{unb}) at the connection corresponding to the punching shear capacity can be estimated using the following equation (Han, Lee and Kwon 2013). This equation is obtained by rearranging eccentric shear model equation.

$$M_{unb} = \left(V_{cr,d} - V_{ed}\right) \frac{J_c}{C\Upsilon_v} \quad \text{(Eq. 4.8)}$$

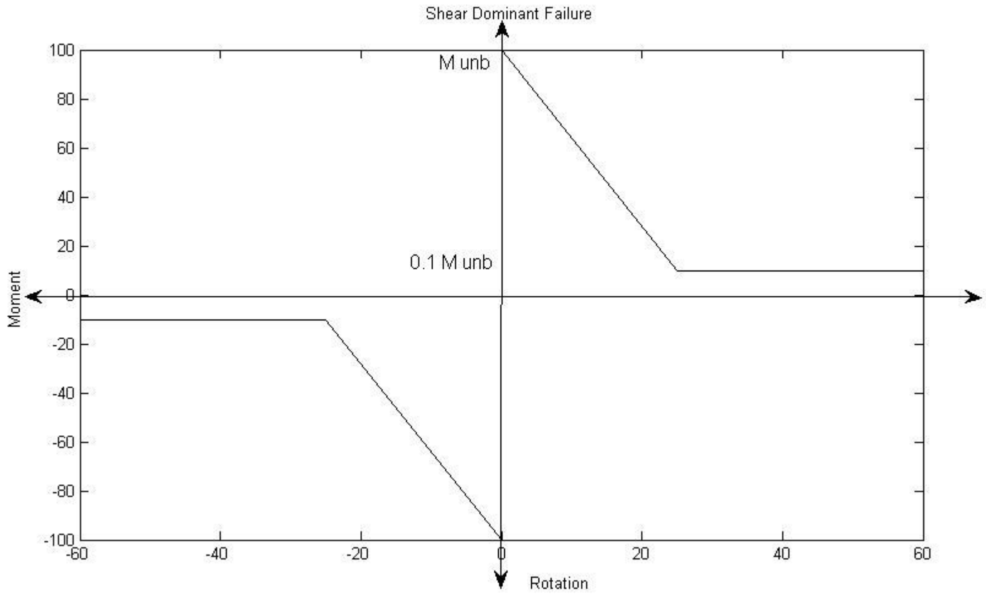


Figure 4. 6: Back Bone Curve for Shear Dominant Failure Mode (Han, Lee and Kwon 2013)

Figure 4.6 represents a typical shear dominant failure at a slab-column connection in flat plate framing system. In the figure, M_{unb} represents a maximum moment that induces a punching shear failure. This type of failure is so sudden that a reserve capacity beyond yielding has a step down fall.

4.4.2.3.2 Flexure Dominant Failure

The slab–column connection fails in a ductile manner when the M_{unb} calculated from the connection punching shear capacity corresponding to shear dominant failure is greater than M_{unb} corresponding to slab reinforcement yielding.

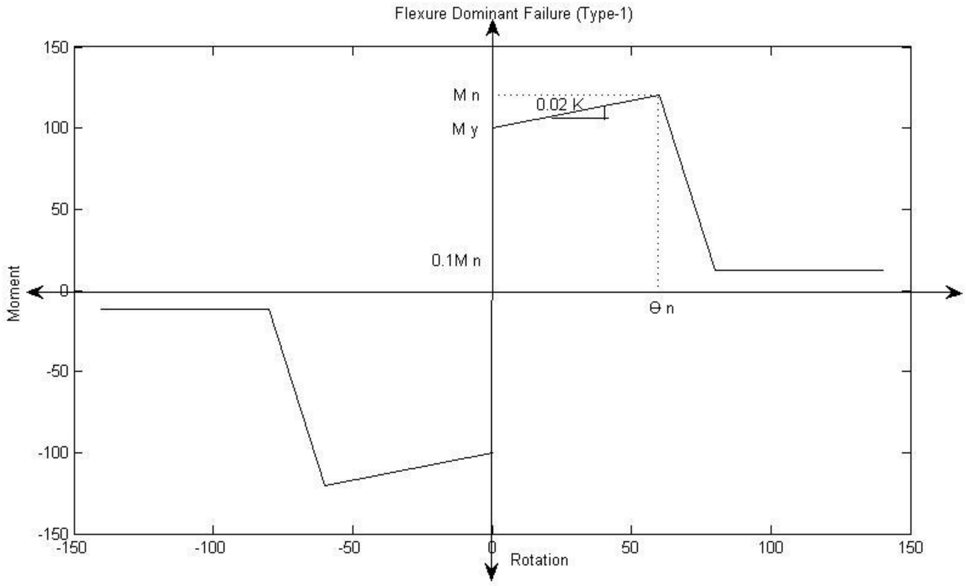


Figure 4. 7: Back Bone for Flexure Dominant Failure (Mode-1) (Han, Lee and Kwon 2013)

Figure 4.7 represents a back bone curve for flexure dominant failure. M_y and M_n in the figure represents a yielding and ultimate moment capacity of a flexure dominant failure in flat plate framing system respectively. Post yield stiffness on the back bone curve could be represented by a reduced elastic stiffness K. θ_n on the figure represents a punching shear failure inducing rotation.

Punching shear failure after yielding of flexural reinforcement could happen as a shear inducing drift capacity is reached as shown on figure 4.7.

While computing flexural capacity of an interior slab-column connection, positive and negative bending of the two adjacent transfer width slabs should be considered.

M_{unb} corresponding to slab reinforcement yielding is computed using the following equation:

$M_{unb} = My^+ + My^-$, for interior connection (Eq. 4.9)

$M_{unb} = My^+$, for exterior connection (Eq. 4.10)

Where:

My^+ is positive slab yielding moment

My^- is negative slab yielding moment

Flexural resistance values of both My^{+} and My^{-} are computed using a chart from designers guide to Eurocode 2. Chart used to calculate flexural resistance is shown on the last page under Annex A section.

The ultimate moment corresponding to flexure controlled failure is computed using the following equation:

$$Mn^{+} = 0.85f_{ck}\ a\ b\ (d - \frac{a}{2}) \quad \text{(Eq. 4.11)}$$

$$Mn^{-} = 0.85f_{ck}\ a\ b\ (d - \frac{a}{2}) \quad \text{(Eq. 4.12)}$$

A value for a is found by solving the following quadratic equation:

for Mn^{+};

$$(0.85f_{ck}\ l)a^2 + (0.003Es\ Ast - Asb\ Fy)a - (0.003Es\ Ast\ \beta\ d') = 0 \quad \text{(Eq. 4.13)}$$

for Mn^{-};

$$(0.85f_{ck}\ l)a^2 + (0.003Es\ Asb - Ast\ Fy)a - (0.003Es\ Asb\ \beta\ d') = 0 \quad \text{(Eq. 4.14)}$$

Where:

β is a factor relating the depth of the equivalent rectangular compression stress block to the neutral axis depth, equaling 0.85

f_{ck} is characteristics compressive strength of concrete

l is slab width

Es is elastic modulus of reinforcement

Ast is area of top reinforcement

Asb is are of bottom reinforcement

Fy is yielding strength of reinforcement

d' is distance from extreme compression fibre to centroid of longitudinal reinforcement

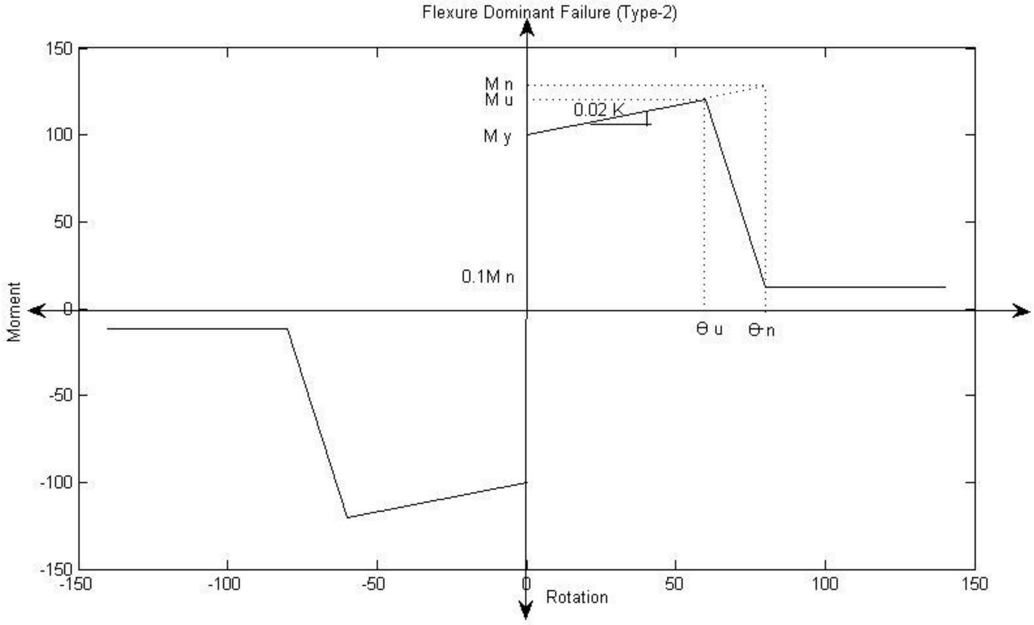


Figure 4. 8: Back Bone Curve for Flexure Dominant Failure (Mode-2) (Han, Lee and Kwon 2013)

Figure 4.8 represents a flexure dominant failure. In the figure, punching shear failure after yielding of flexural reinforcement is due to slab-column connection reaching its shear capacity.

In flexural dominant failure mode, connection failure occurs after slab reinforcement yields. Beyond yielding, the connection fails by reaching either the drift capacity (θ_u) or punching shear strength, whichever occurs first.

To provide an equation for calculating θ_u, an equation proposed by Han is used. The equation for drift capacity θ_u causing punching shear failure is proposed by conducting a rigorous analysis with respect to $\frac{Vc}{Vg}$ for both interior and exterior connection.

$\theta u = 0.048 - \frac{0.048Vg}{Vc}$, for interior connection (Eq. 4.15)

$\theta u = 0.035 - \frac{0.035Vg}{Vc}$, for exterior connection (Eq. 4.16)

Where:

Vc is Nominal shear force capacity

Vg is Gravity shear force

5 CHAPTER FIVE - Performance Assessment of Model Building

This section includes parameters and input values used for the pushover analysis. Equations discussed in the previous chapter are used to compute the corresponding parameters and input values. Response of model buildings both in the pre and post yield states are discussed in details with representative graphs. To help discuss the overall ultimate capacity of specimen buildings, a base shear to top storey displacement is plotted and initial hinge formations and their type are traced and identified.

5.1 Analysis Input Parameters

Both the 3-storey and 6-storey specimen buildings have similar bay width and plan configuration, thus thickness of the plate is similar for both specimen buildings and a corresponding plate property stays the same.

Using the analytical model discussed in the previous chapter with the corresponding equations, inputs for analysis and back bone curve are shown below.

Using Eq. 2.7, nominal shear capacity for both flat plate and flat slab is computed and shown in table 5.1 and table 5.2 respectively. Induced shear stress along the critical region is computed using the conventional method.

Table 5. 1: Critical Region Shear Capacity and Induced Shear Stress for Flat Plate (Without Drop Panel).

Nu	**Location**	$\boldsymbol{V_{Rd,c}\ (MPa)}$	$\boldsymbol{Ved\ (MPa)}$	$\boldsymbol{d(mm)}$	**Shear ratio (%)**
1	*Interior*	0.57	0.503	120	88
2	*Exterior*	0.45	0.42	120	93

Values of Ved in table 5.1 are induced shear stress computed for a respective tributary area.

Table 5. 2: Critical Region Shear Capacity and Induced Shear Stress for Flat Slab (With Drop Panel).

Nu	**Location**	$V_{Rd,c}$ (MPa)	Ved (MPa)	$d(mm)$	**Shear ratio (%)**
1	*Interior*	0.44	0.17	220	38.6
2	*Exterior*	0.44	0.14	220	32

Portion of the unbalanced moment carried by eccentric shear and polar moment of inertia are computed using Eq. 2.3, Eq. 2.4 and Eq. 2.5 for both flat plat and flat slab and are shown in table 5.3 and table 5.4 respectively..

Table 5. 3: Portion of Unbalanced Moment Carried by Eccentric Shear, Polar moment of Inertia and Centroid of Shear Perimeter for Flat Plate (Without Drop Panel).

Nu	**Location**	Υ_v	$C_c(m)$	$J_c(m^4)$
1	*Interior*	0.4	0.39	0.038
2	*Exterior*	0.36	0.39	0.025

Table 5. 4: Portion of Unbalanced Moment Carried by Eccentric Shear, Polar Moment of Inertia and Centroid of Shear Perimeter for Flat Slab (With Drop Panel).

Nu	**Location**	Υ_v	$C_c(m)$	$J_c(m^4)$
1	*Interior*	0.4	0.59	0.243
2	*Exterior*	0.36	0.59	0.145

Back bone curve values for shear controlled failure mode are computed using Eq. 4.8 and put in table 5.5 and table 5.6 for both flat plate and flat slab models respectively.

Table 5. 5: Input for Back Bone Curve of Shear Controlled Failure Mode for Flat Plate (Without Drop Panel).

Nu	**Location**	$M^{+}{}_{unb}(KN\ m)$	$0.1\ M^{+}{}_{unb}(KN\ m)$	θ' **(rad)**	θ'' **(rad)**
1	*Interior*	16.32	1.6	0.0001	0.01
2	*Exterior*	5.34	0.53	0.0001	0.01

Table 5. 6: Input for Back Bone Curve of Shear Controlled Failure Mode for Flat Slab (With Drop Panel).

Nu	**Location**	$M^{+}{}_{unb}(KN\ m)$	$0.1\ M^{+}{}_{unb}(KN\ m)$	θ'**(rad)**	θ'' **(rad)**
1	*Interior*	278	27.8	0.0001	0.01
2	*Exterior*	204.8	20.48	0.0001	0.01

Back bone curve values for flexure controlled failure mode are computed using Eq. 4.9 and are put in table 5.7 and table 5.8 for both flat plate and flat slab models respectively.

Table 5. 7: Input for Back Bone Curve of Flexure Controlled Failure Mode for Flat Plate (Without Drop Panel).

Nu	**Location**	$M_{\ unb}(KN\ m)$	$M_{n}(KN\ m)$	θ_{n} **(rad)**	θ_{u} **(rad)**
1	*Interior*	33.8	45.9	0.015	0.0058
2	*Exterior*	11.4	18.8	0.02	0.0013

Table 5. 8: Input for Back Bone Curve of Flexure Controlled Failure Mode for Flat slab (With Drop Panel).

Nu	**Location**	$M_{\ unb}(KN\ m)$	$M_{n}(KN\ m)$	θ_{n} **(rad)**	θ_{u} **(rad)**
1	*Interior*	44.6	74.5	0.018	0.029
2	*Exterior*	17	32.7	0.022	0.023

Back bone curve input for moment hinge in column model is shown on table 5.9. Response2000 is used to compute moment-curvature curve of column by accounting core confinement.

Table 5. 9: Back Bone Curve Input for Model Building Column.

Point on curve	M/SF	Rotation (rad)
B	1	0.00173
C	1.35	0.0365
D	0.07	0.045
E	0.07	0.06

*Moment $SF = 53.9KN.m$

**Rotation $SF = 1$

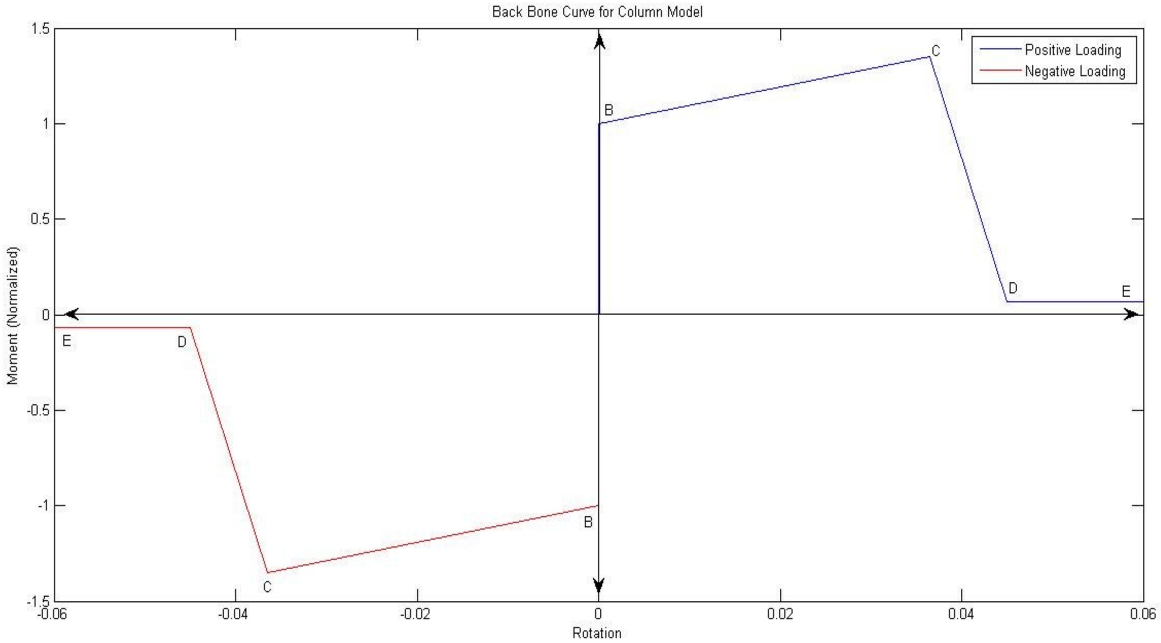


Figure 5. 1: Back Bone Curve for Moment Hinge in Column Model

5.2 Response of Model Building

This sub section includes discussion on the global stability of specimen buildings in relation to a code based stability index value. Tracing of brittle punching shear hinge formation and how a drop panel has improved connection shear problem is discussed. At last, the overall performance enhancement of drop panels in flat plate specimen buildings is discussed with the aid of a capacity curve.

5.2.1 Global Stability of Model Buildings

As flat plate framing systems are beam less, they possess a weak bracing configuration. This leads to a large drift and makes the whole building prone to stability issues. Thus, it's a prior concern that model buildings are globally stable and inter storey drifts are within a safe limit.

In this regard global stability of both the 3-storey and 6-storey specimen buildings is checked for a code based stability index. Stability index equation and limiting values are referred from Eurocoad-8. Based on the stability index equation, inter storey drift value should be below 0.3 so as to insure stability of specimen buildings. The global stability is checked for an earthquake load applied laterally as quarter of the model buildings mass.

$$\theta = \frac{P_{tot}\ d_r}{V_{tot}\ H} \qquad \text{(Eq. 5.1)}$$

Where:

θ is inter storey drift sensitivity coefficient

P_{tot} is total gravity load considered in sesmic condition

d_r is design inter storey drift

V_{tot} is total sesmic storey shear

H is inter storey height

Based on the above stability index equation, sensitivity coefficients are computed at each storey for both building models. As such, maximum values of 0.054 and 0.178 for 3-storey and 6-storey model buildings respectively are computed. These values assure that both model buildings are well within the stability range.

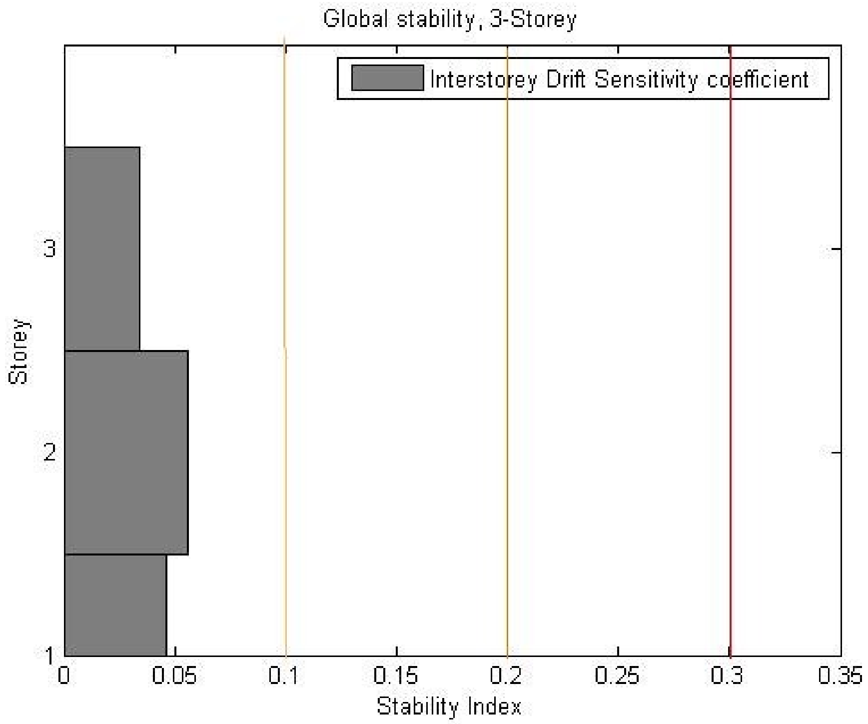


Figure 5. 2 : Global Stability Index for 3-Storey Model Building

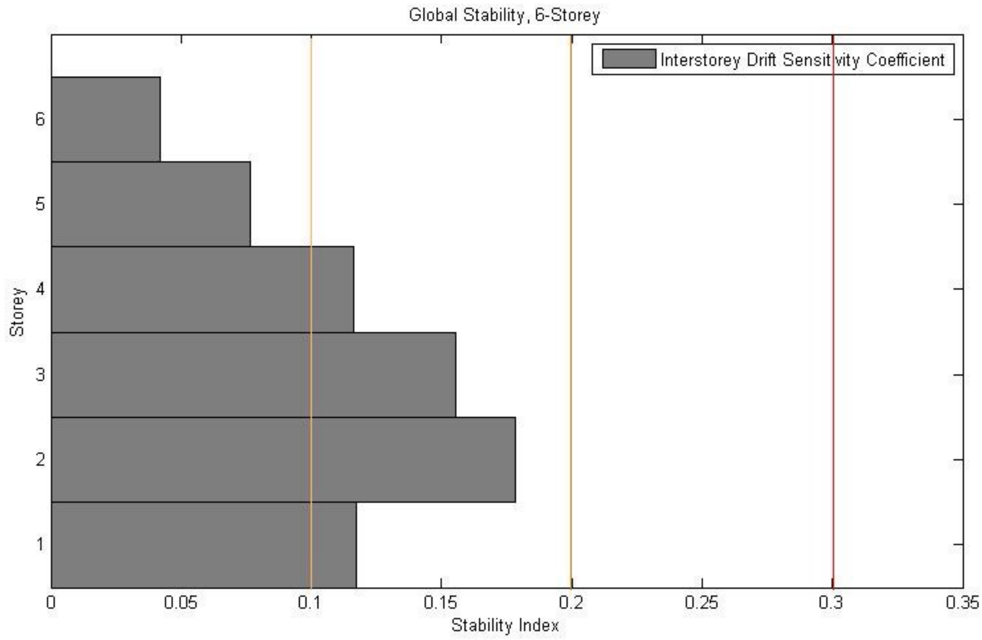


Figure 5. 3 : Global Stability Index for 6-Storey Model Building

5.2.2 Hinge Formation, Type and Distribution

Outputs of the pushover analysis were able to clearly identify possible weak spots on the flat plate frame as hinge. All possible hinges at global failure are identified with their type and location. The type of hinge that formed initially is identified with its corresponding drift ratio and its influence on the overall response of model building is assessed.

5.2.2.1 Model buildings Without Drop Panel

Both flat plates of the 3-storey and 6-storey building models without drop panel have a shear ratio of 88% and 93% for interior and exterior connection respectively. This shear ratio values have given little shear reserve capacity to carry the additional shear force coming from portion of the unbalanced moment. Thus, slab column connections at model buildings without drop panel are likely to experience a brittle connection failure.

Pushover analysis for model buildings without drop panel has shown that, slab-column connection shear dominant failure has occurred before flexure reinforcement yielding at the transfer width.

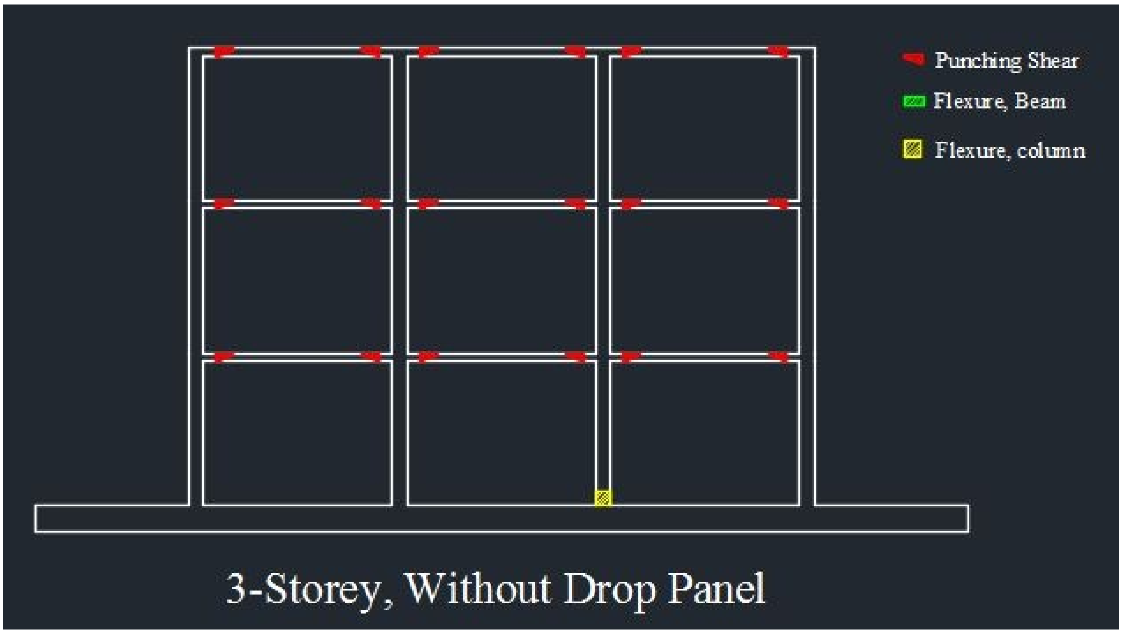


Figure 5. 4 : Hinge Formation for 3-Storey Model Building at Collapse Load (Without Drop Panel)

Based on outputs of the pushover analyse on the 3-storey model building without drop panels, all slab-column connections have punching shear hinges prior to flexural hinges at the transfer width. No slab flexure hinges were observed even at ultimate failure of the 3-storey building model. Only one bottom column flexure hinge is observed at ultimate failure.

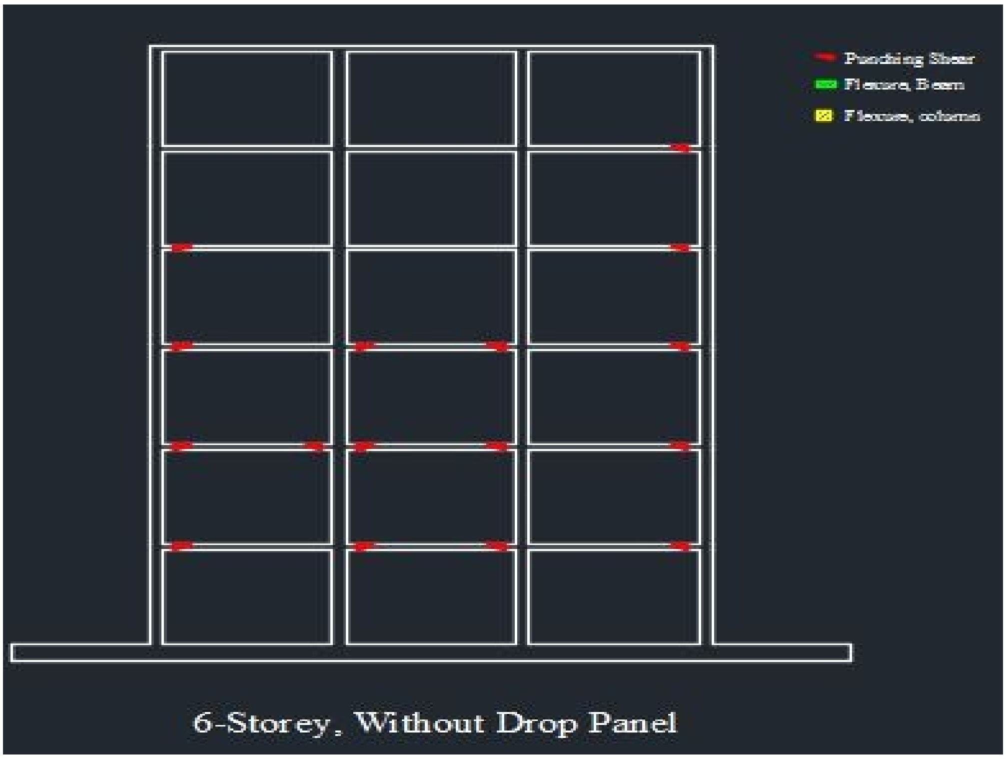


Figure 5. 5 : Hinge Formation for 6-Storey Model Building at Collapse Load (Without Drop Panel)

Also, outputs of the pushover analyse on the 6-storey model building without drop panels, all slab-column connections have punching shear hinges prior to flexural hinges at the transfer width. No slab flexure hinges were observed even at ultimate failure of the 6-storey building model. And no flexure column hinges were observed at ultimate failure.

5.2.2.2 Model Buildings with Drop Panel

From the response of model buildings without drop panel, it is learnt that a brittle punching shear failure has occurred before flexure reinforcement yielding at the transfer width. This failure is due to high value of shear ratios at the connection.

One way to lower shear ratio at the connection and enhance connection shear capacity is to use a drop panel. Both building models were checked for response enhancement after putting a drop panel of depth $0.1m$. The drop panels have a width of $0.5m$ from the face off the columns.

As such, shear ratios were reduced to 38.6% and 32% for interior and exterior connection respectively of both the 3-storey and 6-storey model buildings by introducing a drop panel at the connection. This shear ratio values have given the slab-column connection an ample reserve capacity to handle the additional shear force coming from portion of the unbalanced moment.

With a reduced shear ratio values due to drop panels at the slab-column connection, brittle punching shear failures are exceeded by flexural reinforcement yielding at the transfer width. Thus, ductility at the slab-column connection is insured by using drop panels.

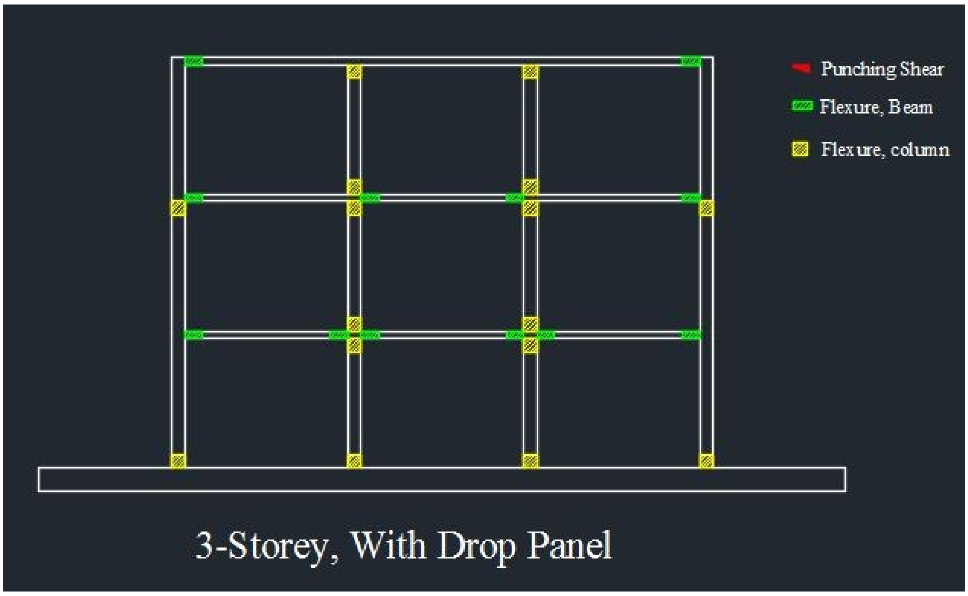


Figure 5. 6 : Hinge Formation for 3-Storey Model Building at Collapse Load (With Drop Panel)

Based on hinge formation tracing for the 3-storey model building with drop panel above, no punching shear hinges were observed at slab-column connection even at ultimate collapse. Multiple flexural reinforcement yielding is observed at all stories in the transfer width. All bottom base columns and few upper columns have formed flexure hinges at ultimate collapse. It can be said that, drop panels have made the slab-column connection to have a ductile response and made sure flexure controlled failure come before punching shear failure.

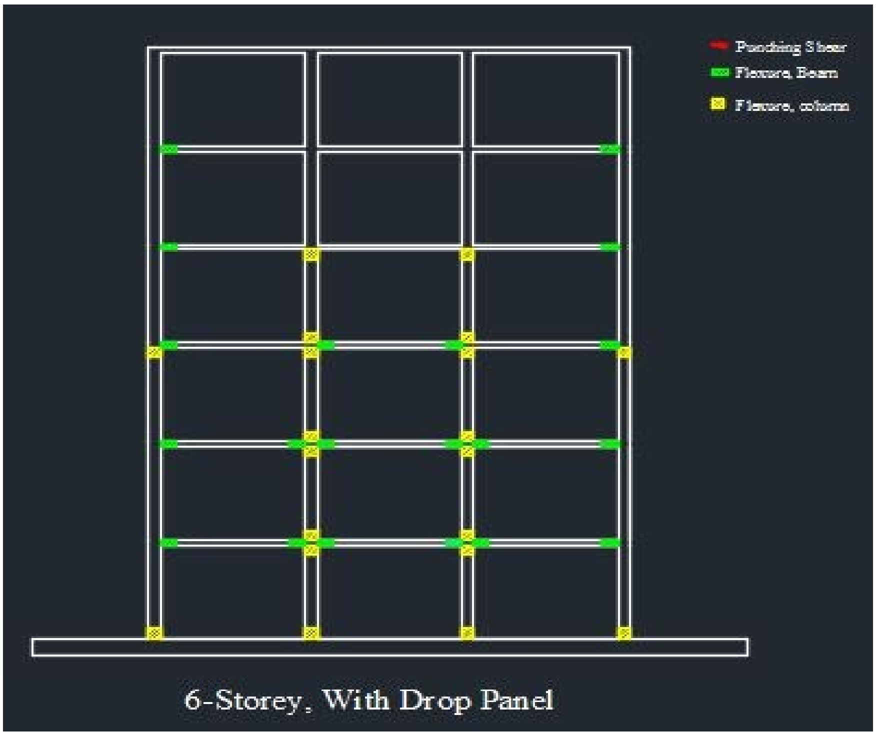


Figure 5. 7 : Hinge Formation for 6-Storey Model Building at Collapse Load (With Drop Panel)

Here also for the 6-storey model building with drop panel, hinge formation showed that no punching shear hinges were present at slab-column connection even at ultimate collapse. Multiple flexural reinforcement yielding at the transfer width were observed at all stories except the top storey. All bottom base columns and few upper columns have formed flexure hinges at ultimate collapse. It can be said that, drop panels have made the slab-column connection to have a ductile response and made sure flexure controlled failure exceeds punching shear failure.

5.2.3 Base Shear to Top Story Displacement

One vital out of a pushover analysis is a plot of base shear to top story displacement, this plot is a called a capacity curve and helps elaborate the pre and post yield performance of model buildings. Capacity curve for both categories of building models, with and without drop panels were obtained at the end of the pushover analysis. Using the graphs, global response; ultimate capacity and initial stiffness were carefully studied. It is then learnt that, drop panels have significantly enhanced the global performance of model buildings.

5.2.3.1 Model Building without Drop Panel

Both the 3-storey and 6-storey model buildings without drop panels have shown a limited base shear and drift capacity. The 3-storey model building has shown a small drift capacity after yielding, but the yielding hinge is of brittle shear, the drift after yielding could not be accounted for global ductility. The 6-storey model building has suddenly lost stability and analysis was terminated without any drift beyond yielding, this is a typical brittle performance without any post yield drifts.

Overall, it is learnt that both model buildings without drop panel have shown small base shear capacity with no or little post yield drift. For the larger story; 6-storey model building, its lateral response without drop panel is a sudden strength drop with brittle nature.

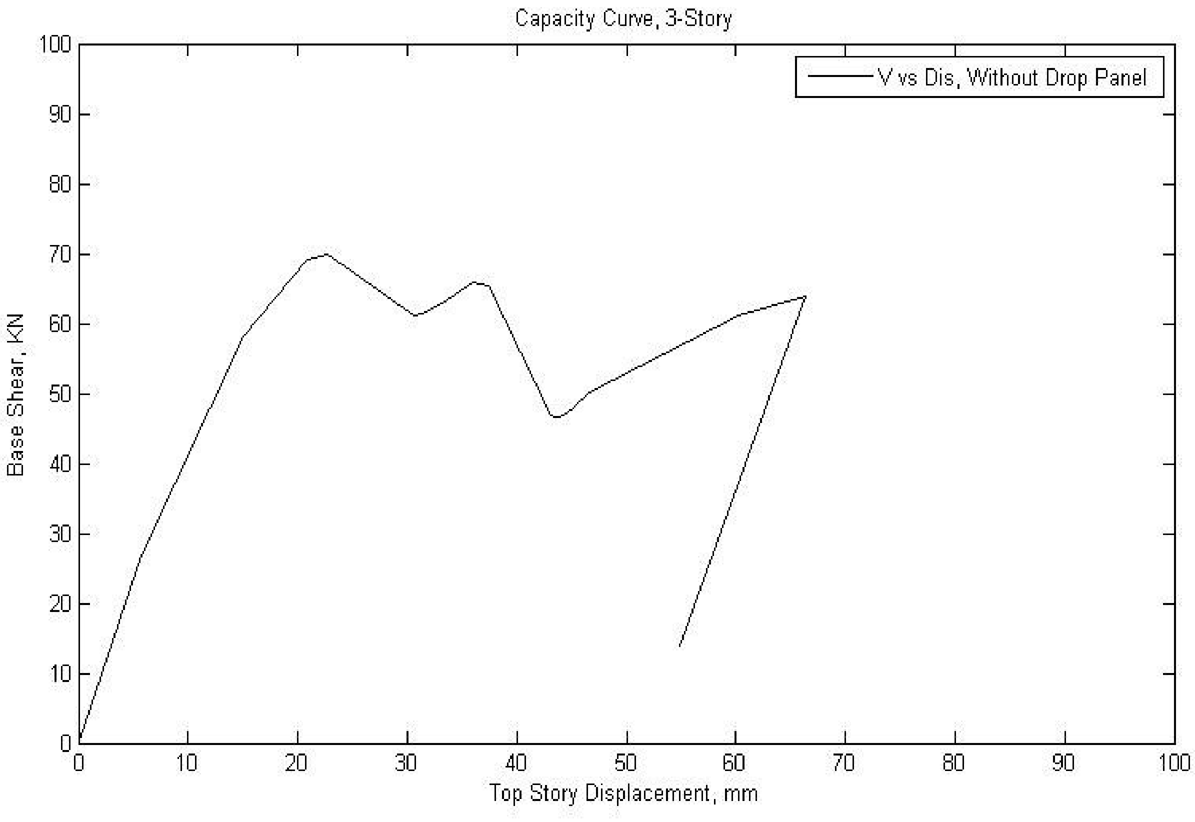


Figure 5. 8 : Capacity Curve for 3-Storey Model Building (Without Drop Panel)

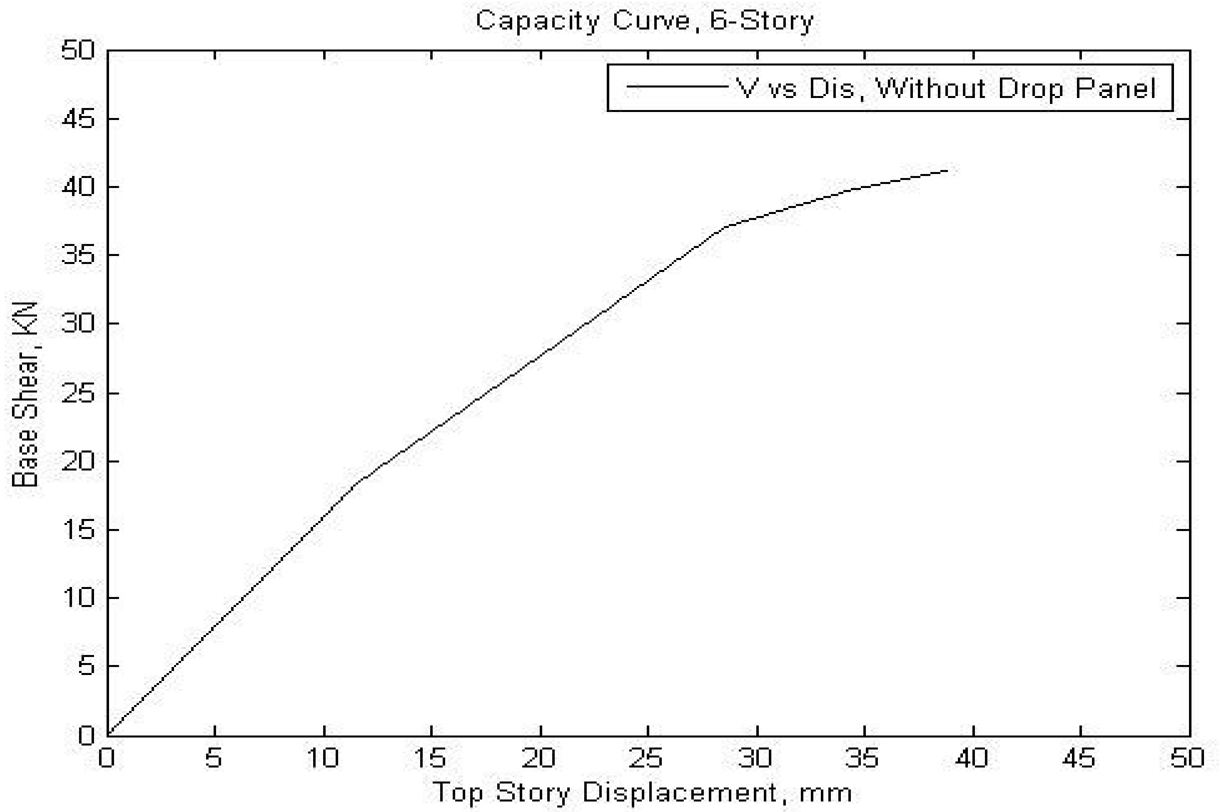


Figure 5. 9 : Capacity Curve for 6-Storey Model Building (Without Drop Panel), failure by loss of stability

5.2.3.2 Model Building with Drop Panel

After running a pushover analysis for both model buildings with drop panels, a response has shown a significant increase in base shear and drift capacity. Initial yielding on the capacity curve for both model buildings is due to yielding of flexural reinforcement and drift after yielding is large enough to ensure a global ductility.

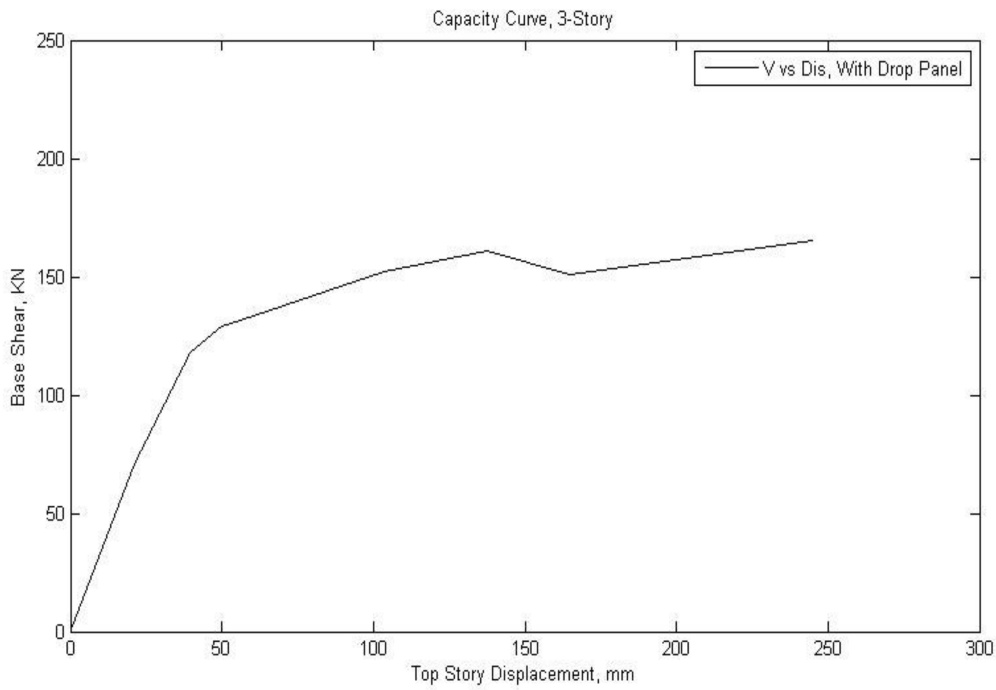


Figure 5. 10 : Capacity Curve for 3-Storey Model Building (With Drop Panel)

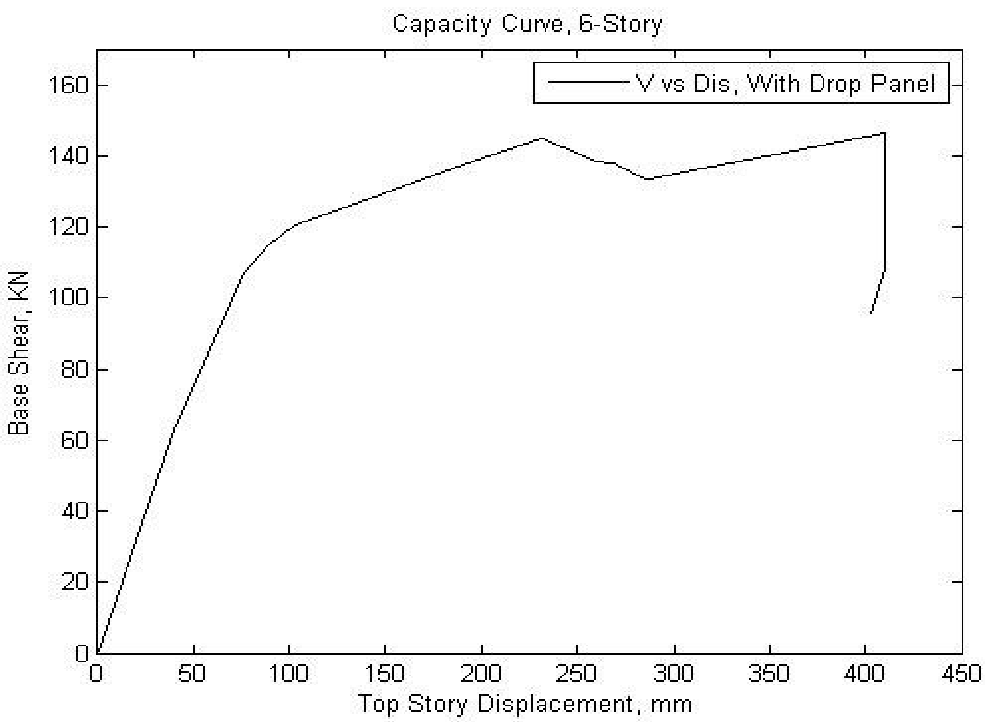


Figure 5. 11 : Capacity Curve for 6-Storey Model Building (With Drop Panel)

5.2.4 Performance Enhancement of Drop Panels on Model Buildings

The critical weakens of flat plate model buildings without drop panels is that, a brittle punching shear exceeds a ductile flexural reinforcement yielding during a cyclic earthquake loading. This performance, without drop panel has shown a reduced capacity in both base shear and top story drift.

Addition of drop panel to flat plate model buildings has significantly reduced the slab-column shear ratio and in return increased the base shear and drift capacity. Yielding of flexural reinforcement in the transfer width has exceeded a brittle punching shear failure for model buildings with drop panels and even at ultimate collapse no connection shear hinges were observed.

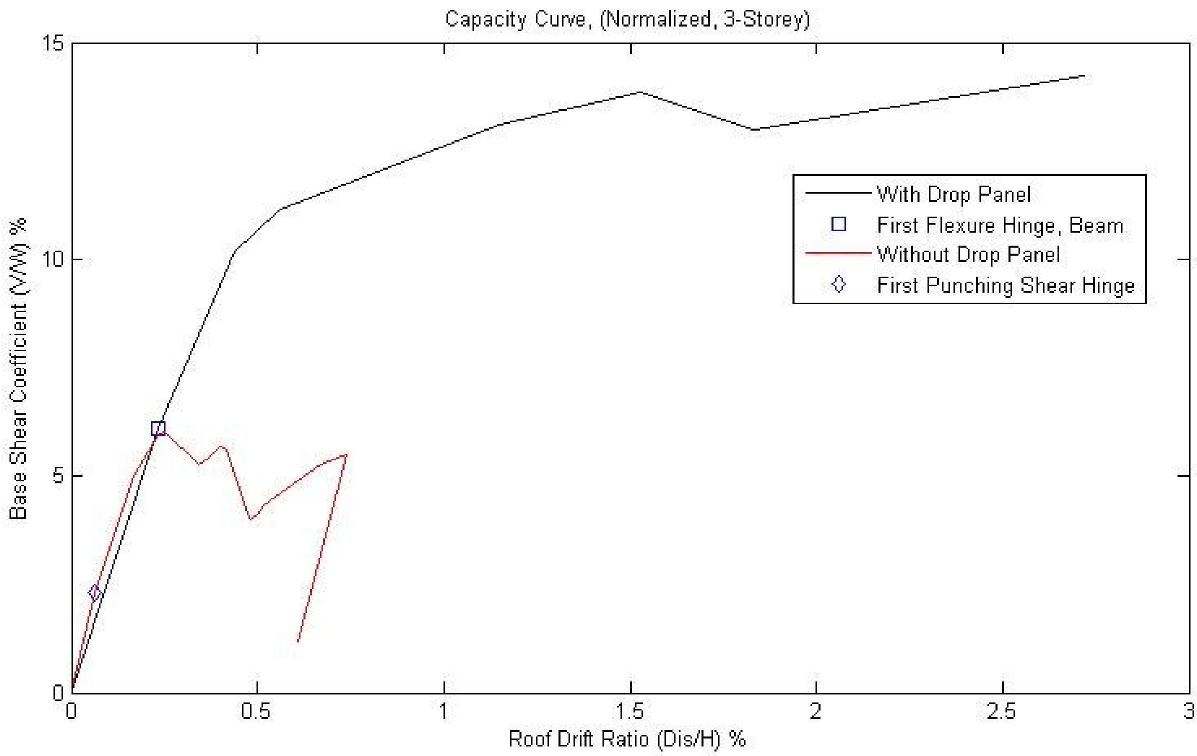


Figure 5. 12 : Performance Enhancement of Drop Panels on the 3-Storey Model Building

For the 3-storey model building as shown in figure 15.12, a drop panel has increased the base shear coefficient from 6.01% to 14.2% . Also, a drop panel has helped increase the roof drift ratio from 0.73% to 2.7%. Initial elastic stiffness is barely influenced by addition of drop panels at slab-column connection.

The first hinge that occurred on the 3-storey model building without drop panel is a brittle punching shear hinge. It has occurred early on the capacity curve at a roof drift ratio of 0.06% and a base shear coefficient of 2.3%. This shows that a 3-storey model building without drop panel is more likely to experience an early and brittle slab-column connection failure if used as a primary earthquake resisting system.

The first hinge that occurred in the 3-storey model building with drop panels is a flexure hinge at a slab transfer width. It occurred at a roof drift ratio of 0.23% and a base shear coefficient of 6.1%. This shows that a drop panel was able to cater an early connection shear failure and insured a ductile connection failure.

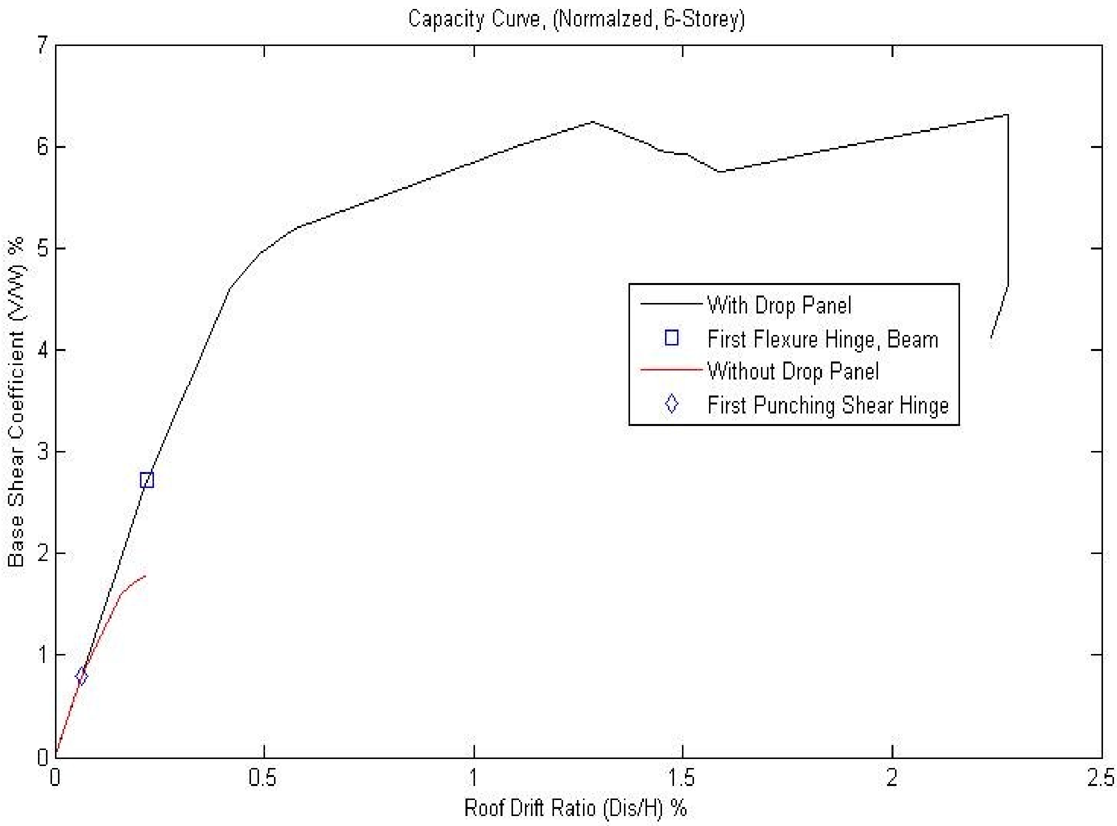


Figure 5. 13 : Performance Enhancement of Drop Panels on the 6-Storey Model Building

6-storey model building without drop panel has suddenly lost strength and had a very small drift capacity. As seen on figure 15.13 the sudden strength loss was so early, its base shear coefficient barely passed a 1% value. The model building without drop panel possesses no reserve capacity.

A drop panel on the 6-storey model building has increased the base shear coefficient from 1.8% to 6.3%. Its roof drift ratio has also increased from 0.2% to 2.27%.

Figure 5.13 shows that the first hinge formed on the 6-storey building model without drop panel was a punching shear hinge that occurred early on the capacity curve. The first shear hinge occurred at a base shear coefficient of 0.8% and roof drift ratio of 0.06%. all successive hinges were punching shear hinges, the whole building model experienced an early and sudden strength loss.

Also on the figure a drop panel used on the 6-storey model building has made yielding of flexural reinforcement to exceed the brittle punching failure. The first hinge, that is a yielding of flexural reinforcement in the transfer width had occurred at a base shear coefficient of 2.7% and at a roof drift ratio of 0.22%.

A drop panel on the 6-storey model building has significantly increased the base shear capacity. It has also helped cater problems with slab-column connection, all slab hinges were yielding of flexural reinforcement within the transfer width. No punching shear hinges were present even at ultimate collapse. Sufficient deformation was observed beyond yield point due to drop panels on the model building; this is explained by the fact that a reduced connection shear ratio due to drop panels has substituted all brittle connection failures with ductile flexural yielding and made the whole building model possess sufficient global ductility. The model building with drop panel has also shown a reserve capacity beyond point C on the capacity curve.

6 CHAPTER SIX – CONCLUSION AND RECOMMENDATION

6.1 Conclusion

It is clearly seen that a drop panel has created a significant performance changes both in pre and post yield segment of the load deformation curve of the model buildings. Over all conclusions drawn from the research work are summarized below.

- It is learnt that initial elastic stiffness has not shown a considerable change in both 3-storey and 6-storey model buildings due the introduction of drop panels.
- Both model buildings without drop panel had a sudden and early strength loss while model buildings with drop panel had shown an extended ductility capacity and a strength loss that came at a later time.
- It is learnt that shear ratio at a shear critical slab-column connection is significantly reduced in both model buildings due to drop panels. A reduction in shear ratio due drop panel has enabled the slab-column connection to have enough reserve of shear capacity to handle the additional shear force coming from portion of the unbalanced moment.
- Due to drop panels, both building models had no punching shear hinges. All slab hinges were due to yielding of flexural reinforcement in the transfer width. Thus, drop panel has brought flexure controlled failure before punching controlled failure.
- Drop panel has significantly increased yielding base shear in both building models. Both building models with drop panel have shown sufficient drift capacity beyond yielding. No sudden strength loss was observed in both models with drop panel and reserve capacity was guaranteed with the introduction of drop panels.
- In 3-storey and 6-storey model buildings with drop panel, no punching shear hinges were observed even at ultimate collapse.
- Addition of drop panel on the 3-storey building model has increased the base shear coefficient from 6.01% to 14.2% and the roof drift ratio from 0.73% to 2.7%. Also on the 6-storey model building, drop panel has increased the base shear coefficient from 1.8% to 6.3% and the roof drift ratio from 0.2% to 2.27%.

- The first brittle punching shear hinges on the 3-storey model building without drop panel has occurred at a base shear ratio of 2.3% and at a roof drift ratio of 0.06%. For the 6-storey model building without drop panel the first brittle punching shear hinge has occurred at 0.8% and roof drift ratio of 0.06%.
- For both model buildings with drop panels the first hinges were yielding of flexural reinforcement and the hinges took place at higher base shear and roof drift ratio.
- Both building models with drop panel have shown a sufficient drift capacity beyond yield point. Thus, a drop panel has insured an adequate global ductility.
- For model buildings without drop panel, the strength loss and overall global failure is due to slab-column punching shear failure. While for the model buildings with drop panel, strength loss comes at an extended drift compared to the models without drop panel and no connection punching shear failure is observed. Thus it can be concluded that incorporating drop panel in flat plate framing system can help overcome slab-column punching failure.
- It is also seen that shear ratio has a direct influence on the lateral response of a flat plate framing system. Flat plate framing systems with higher value of shear ratio tend to exhibit a punching shear failure at a slab-column connection while flat slab framing system with lower shear ratio due to drop panel seem not to have slab-column connection punching shear failure.
- Drop panel can help decrease connection shear ratio and in return increase the yielding base shear capacity and avoid punching shear failure at slab-column connection.

6.2 Recommendation for Further Work

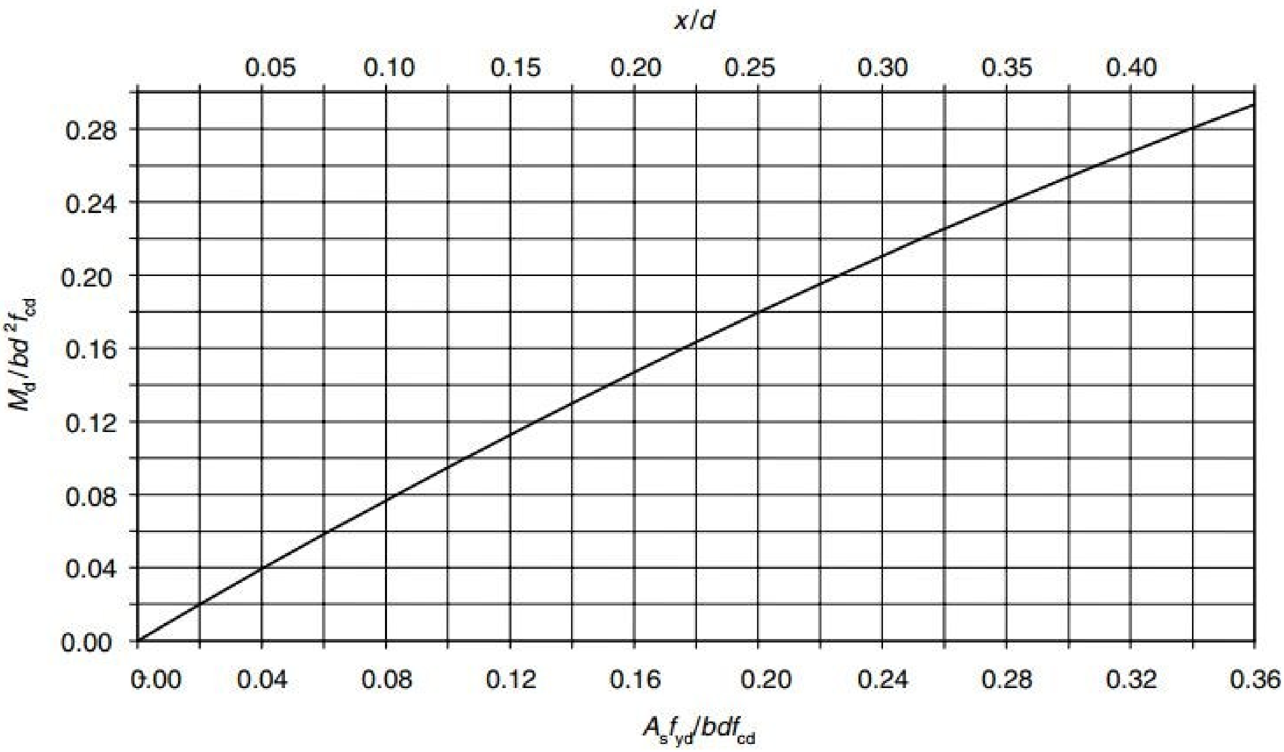
The writer strongly believes that this research work could help give insights and initial ideas on future research works conducted on post yield performance of flat plate framing system and their possible remedial for performance lacks. In this end, initial ideas for future research work are put forward by the writer.

- For a more exact and in depth post yield performance assessment of flat plate framing system, a more accurate nonlinear dynamic time history analysis could be used as a method of analysis and for member hinge modelling fibre hinges could be used. Rather than using a one cycle stiffness degradation to model material nonlinearity, a hysteric pinch model could be used to yield in a better result.

- A parametric study could be done on the depth of a drop panel that will yield in the highest shear ratio with no punching shear failure occurring before yielding of flexural reinforcement.
- Other rehabilitation elements at a slab-column connection such as shear studs and post tension tendon could be assessed for possible performance enhancement.

ANNEX

Annex A Design Chart for Singly Reinforced Sections



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