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"STUDY ON PROGRESSIVE COLLAPSE ANALYSIS FOR TSUNAMI HAZARDS"

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ABSTRACT- *Tsunami — In Japanese "tsu" means harbor and "nami" means waves, so tsunami can be summed up as a harbor waves or seismic waves or tidal waves. They are water waves which are caused by submarine earthquakes. Wave movement with large velocity triggered but strong Earthquake at the sea bed is generally the primary cause of tsunami. It's a natural event on which a man has no control, neither can he control the damage caused by it. Occurrence of such natural calamities causes severe damage to the structure as well as to human lives too. Occurrence of tsunami (like the one during Indian Ocean tsunami on December 26, 2004, the Sumatra earthquake in 2004 or the one during the Tohoku earthquake in Japan in 2011) caused devastating damages to the coastal structures and tremendous casualties. Such deadliest mega event made the coastal community realize the need for the preparedness against it. Also most of the tsunamis are earthquake induced tsunamis; hence it is necessary that coastal structures should be designed against both earthquake and tsunamis.*
The community of structural engineer is well aware about the consequences of accidental and rare events like Earthquake, Tsunamis, Fire, Tornado and Blast which leads to building failure and loss of life and property. Among them Progressive collapse is one of the most devastating types of building failures, which most often lead to costly damages, multiple injuries, and possible loss of life. This paper deals with the hazards associated with the progressive collapse triggered due to tsunami forces. In the present study an attempt has been made to give recommendations and suggestions for the structures to be built upon the sea and large water bodies. For this G+5 hypothetical building was modeled in ETABS-2015 and the building was scrutinized by performing Non-Linear Static Analysis while following U.S. General Services Administration (GSA) document and Unified Facility Criteria – Department of Defense (UFC – DoD), USA guidelines along with FEMA-356 code of conduct. . From the study it was found that as we move away from the Tsunami forces along Y-direction the criticality of Storey goes to Storey 2, 3 and 5 and maximum deflection is in storey5 with 25.67% and 18.26% as compared to Storey 1 along Y and X direction respectively.

Keywords: *Progressive collapse, displacements, General Services Administrations, Unified Facility Criteria, hinges formation.*

1. INTRODUCTION

A) Introduction to Tsunami: For a country like India there are varieties of natural and manmade disasters. Among natural hazards, the most notable ones are earthquakes, cyclones, floods, droughts and landslides. Tsunami is an additional concern of safety after the super tsunami of 26^{th} December, 2004. And as a matter of fact India has a very long coastal line from the Kutch in Gujarat to West Bengal and the various Island groups that are exposed seas. Also the majority of the coastal areas fall under the moderate Seismic Zones III with some parts of Gujarat and the entire Andaman & Nicobar Islands coming under the most severe Seismic Zone V. Hence, it is important that one should prepare any mitigation and preparedness measures taken up in these coastal areas should also consider the multi-hazard nature of these areas. It is also important to note that the entire coast line is not uniform in terms of intensities of various hazards. The hazard intensities of earthquakes, floods and cyclonic storms in the west coast and the east coast vary widely.

Various Codes and Guidelines for Tsunami
Before the Indian Ocean Tsunami, it was assumed that there was no need for the design of structures against tsunami-induced forces. This situation was due to the assumption that tsunamis are "rare" events, with significantly large return periods. The devastation that may be caused by a tsunami of a large magnitude can be catastrophic as demonstrated by the 2004 Indian Ocean event which induced significant structural damage on infrastructure, killing over 3,00,000 people and leaving an estimated 1.5 million homeless [15]

List of few codes are as below
- The City and County of Honolulu Building Code (CCH, 2000);
- The minimum Design Loads for Buildings and Other Structures; SEI/ASCE 7-02 (ASCE 7, 2002);
- The Federal Emergency Management Agency- "Coastal Construction Manual" (FEMA CCM, 2000);

- Federal Emergency Management Agency- “Guidelines for Design of Structures for Vertical Evacuation from Tsunamis” (FEMA P646, 2008);
- Tsunami draft code- 26/-8/2016
  Tsunami Forces On Structures

There are three parameters essential for defining the magnitude and application of designing forces on structures:
a) Flow direction;
b) Inundation depth; and
c) Flow velocity.
These parameters mainly depend on:
1) Run-up height of tsunami and arrival time;
2) Coastal topography; and
3) Roughness of the coastal inland
B) Introduction Progressive Collapse
Progressive collapse is defined as an extent of damage or collapse that is disproportionate to the magnitude of the initiating event. Progressive collapse is a phenomenon that involves the damage of a structural element resulting in the collapse of a disproportionately large part of the structure or the entire structure.

Based on this definition, most catastrophic structural failures may be classified under progressive collapse. Progressive collapse is primarily an issue of vertical load- carrying capacity of a structure. However, the design of elements of a building may not depend only on vertical loads, but also on lateral loads from actions such as wind or earthquake. Beams, columns or joints of a framed structure may have a larger load-bearing capacity due to the design to wind or seismic actions. These elements would have higher capacities to confine damage to the initially affected area, and consequently prevent progressive collapse. It has been observed from a series of research that, continuity for offering alternate path and stability when a load-bearing element is lost is the best way to prevent progressive collapse of a structure.

The General Service Administration (GSA) has presented practical guidelines for the design to reduce the progressive collapse potential of federal buildings. The Department of Defense (DOD) has also presented a guideline for the new and existing buildings i.e. Unified Facilities Criteria- Department of Defense. These guidelines recommend the alternate path method for analyzing a structure for its progressive collapse vulnerability.

The alternate path analysis method involves removal of a column from four specified locations, which are then analyzed. The limits states of the elements are then checked and failed elements are removed. Subsequent analysis results in the redistribution of loading to the adjacent elements. The analysis is repeated until stability is achieved or the whole building fails.

2. LITERATURE REVIEW

Nayak S, Reddy M, Madhavi R, Dutta S [1] in their paper “Assessing tsunami vulnerability of structures designed for the seismic loading” concluded that when tsunami enters the shallow water i.e. approx. 30m the speed of tsunami waves diminishes.

They reported that for the multiple column effect, forces generally increase due to the blockage of flow by adjacent columns. They also found that seismic base shear force is greater than the tsunami force up to critical height and less than the tsunami force after critical height. Further they quoted that, critical height for a particular zone decreases with increase of response reduction factor R. It was found that critical heights obtained by considering seismic base shear calculated according IS 1893(Part1), ASCE7-05 and Eurocode8 and tsunami load obtained by FEMA55, CCH lies in the range of 1.2m–2.63 m for a masonry structures.

Yoshinobu Tsuji, et al [2] conducted a field surveyed and interrogated people of on over 40 islands of Flores during December 29, 1992 to January 5, 1993. After completing their survey they published the report in the form of paper titled “Damage to Coastal Villages due to the 1992 Flores Island Earthquake Tsunami”. After the survey they found the height of the tsunami was only 2.5-3.2 m above mean sea level. Maximum run-up was 2.3m. Palm trees forest was effective in dissipating the tsunami energy, there by mitigating the human toll. People were more swiftly killed by the tsunamis than by the earthquake Ian Robertson[3] researched on “Structural Analysis of Selected Failures Caused by the 27 February 2010 Chile Tsunami” and found that the highest inundation heights recorded by ITST (International Tsunami Survey Teams) groups were generally in the 10–12 meter range (EERI 2010).He also found that for well-defined structural element failures at sites where inundation depth was measured, it was easy to evaluate the hydrodynamic loading required to cause these failures and derive estimated lower bound flow velocity overland during the event. He specifically mentioned that tsunami debris impacts and foundations deserve greater attention in research and design provisions greater attention in research and design provisions.

Azadbakht M and Yim S [4] reported a paper on “Estimation of Cascadia Local Tsunami Loads on Pacific Northwest Bridge Superstructures” and found that, on average, a large seismic event in the Cascadia Subduction Zone (CSZ) occurs once every 500 years.They developed numerical model and concluded that the maximum tsunami horizontal and downward vertical loads occurred approximately simultaneously. The magnitudes of the tsunami horizontal and downward vertical loads were significantly affected by the water free-surface elevation at the seaside of the bridge cross section. The initial impact of a tsunami on a bridge superstructure did not lead to a significant uplift force when the bridge cross section had a seaward slope. Analyses of a deck-girder bridge with a closed railing system showed average increases of 33, 15, and 77% in the maximum tsunami horizontal, downward vertical, and uplift forces, respectively, compared with the corresponding open rail system.

Gary Y. K. Chock [5], researched on “Design for Tsunami Loads and Effects in the ASCE 7-16 Standards’ and found that the public safety risk has been only partially mitigated through warning and preparedness of evacuation. He also established that structural design of buildings and structures include requirements for the following tsunami effects:

- Unbalanced lateral forces at initial flooding;
- Buoyant uplift based on displaced volume; and
- Residual water surcharge loads on elevated floors;
- Drag Forces, per drag coefficient Cd for structure size and element shape;
- Lateral impulsive forces of tsunami bores:
- Hydrodynamic pressurization by stagnated flow
- Shock pressure effect of entrapped bore impulse;
- Poles, passenger vehicles, medium boulders are always applied;
- Shipping containers and boats apply if structure is in proximity to hazard zone; and Extraordinary impacts of ships only where in proximity to Risk Category III and IV structures;
- Local scour and soil pore pressure softening effects on the foundation; and
- General erosion
- Probabilistic offshore tsunami amplitude maps and tsunami design zone inundation maps were given to establish the basis of design;
- Procedures for tsunami inundation analysis utilize the design map values of offshore tsunami amplitude or the run up and inundation limit from the tsunami design zone map;
- Structural loading and analysis techniques for determining building performance are in turn calculated from the site’s basis of design parameters of inundation depth and flow velocity;

Ian Nicol Robertson [6] in his paper “Prototypical Building Design for Tsunami Loading” developed a prototypical building which was designed for various seismic design categories and soil conditions in accordance with IBC 2006 and ACI 318-08.

Based on the results of his study, the following conclusions were drawn:

- Multi-story reinforced concrete residential and office buildings can be designed to survive the tsunami flow scenarios assumed in this study. They can therefore provide refuge through vertical evacuation.
- Tsunami design resulted in less than 8% increase in reinforcing steel weight and less than 3% increase in concrete volume for the buildings.
- Special moment-resisting beam-column frame systems designed for high seismic conditions may not require any upgrading to resist tsunami loads.
- Out-of-plane shear of structural walls due to tsunami loads may require the addition of shear reinforcement in the form of headed studs. Alternatively the wall thickness may need to be increased.
- Debris impact due to a shipping container will likely exceed the shear and bending capacity of individual columns. It was recommended that the building be designed to prevent progressive collapse in the event of column failure.

Abdullah Keyvani [7] in his paper “Progressive Collapse of RC Frames Due to Heavy Impact Loads of Tsunami” quoted that progressive collapse is a relatively rare event, as it requires both an abnormal loading to initiate the local damage and

a structure that lacks adequate continuity, ductility and redundancy to resist the spread of damage. He compared the results of two analytical models with the experimental results. One of them was common and simplified Idealized Component Load-Deformation curve based on the FEMA 356, and the other was FEH method. Material nonlinearity models, which were able to demonstrate the specifics of progressive collapse such as catenary action, axial-moment (P-M) interaction of beams during catenary action, and reflecting the stages of progressive col-lapse, considered as proper model for progressive collapse analysis. Comparison curves demonstrated the predominating capability of FEH method in verifying the experimental results.
In addition, FEH method was able to consider the P-M interaction in beam elements due to the catenary action and tensile forces in beams. In contrast, FEMA 356 hinges were so conservative in comparison with the experimental data, and were not able to provide some comparable results such as strain response details. Also FEH method was neither sophisticated for research and/or practical purposes, nor unreliable. This method can be used as one of the proper equivalents for material nonlinearities especially for progressive collapse analysis.

Ian N. Robertson et al [8] researched and published paper titled "Lessons from Hurricane Katrina Storm Surge on Bridges and Buildings". After the survey and analysis they drew the following conclusions.

- Many engineered structures in the Katrina inundation zone experienced only nonstructural damage at the lower levels due to the storm surge and storm wave action. A number of structures experienced significant structural damage due to the effects of the coastal inundation.

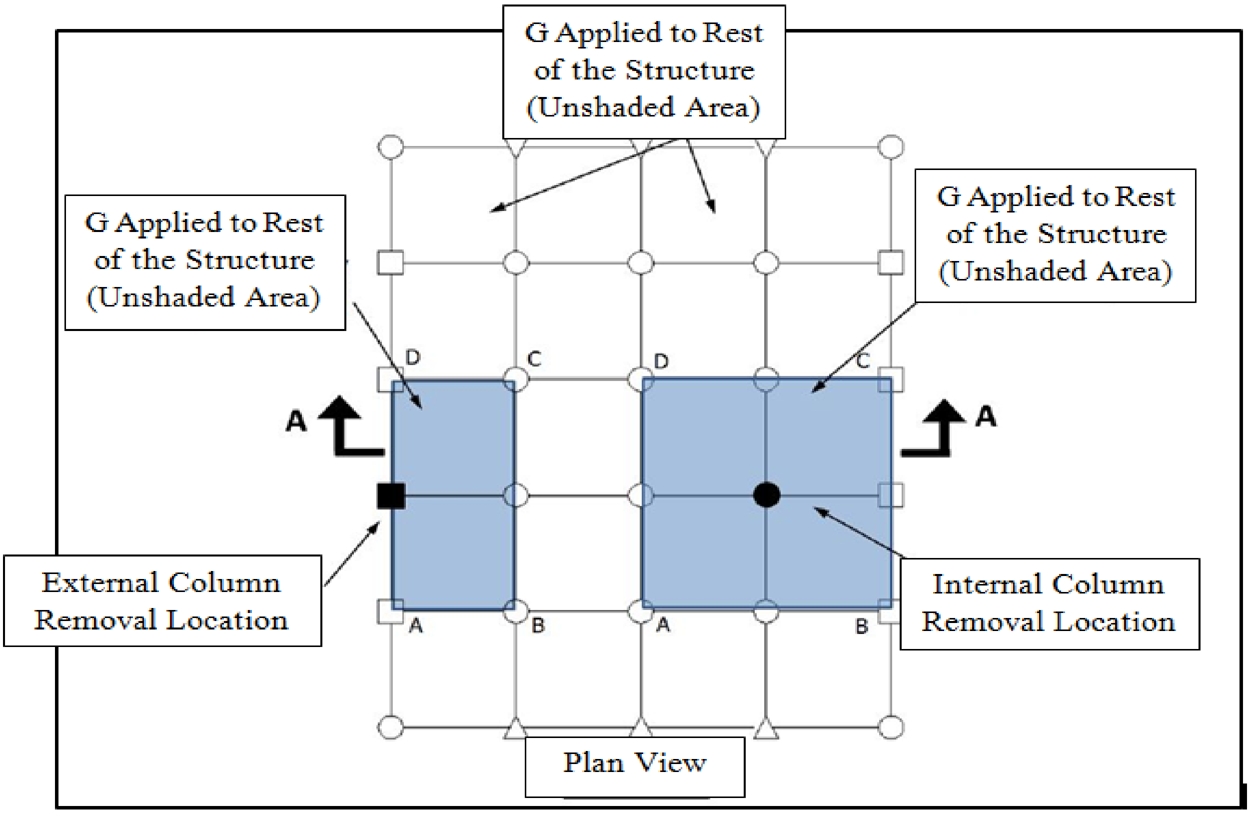
- Bridge decks and structural floor systems submerged during coastal inundation were subjected to significant hydraulic loads, including hydrostatic uplift due to buoyancy, which was amplified by the effect of entrapped air, and hydrodynamic uplift due to vertical wave action.

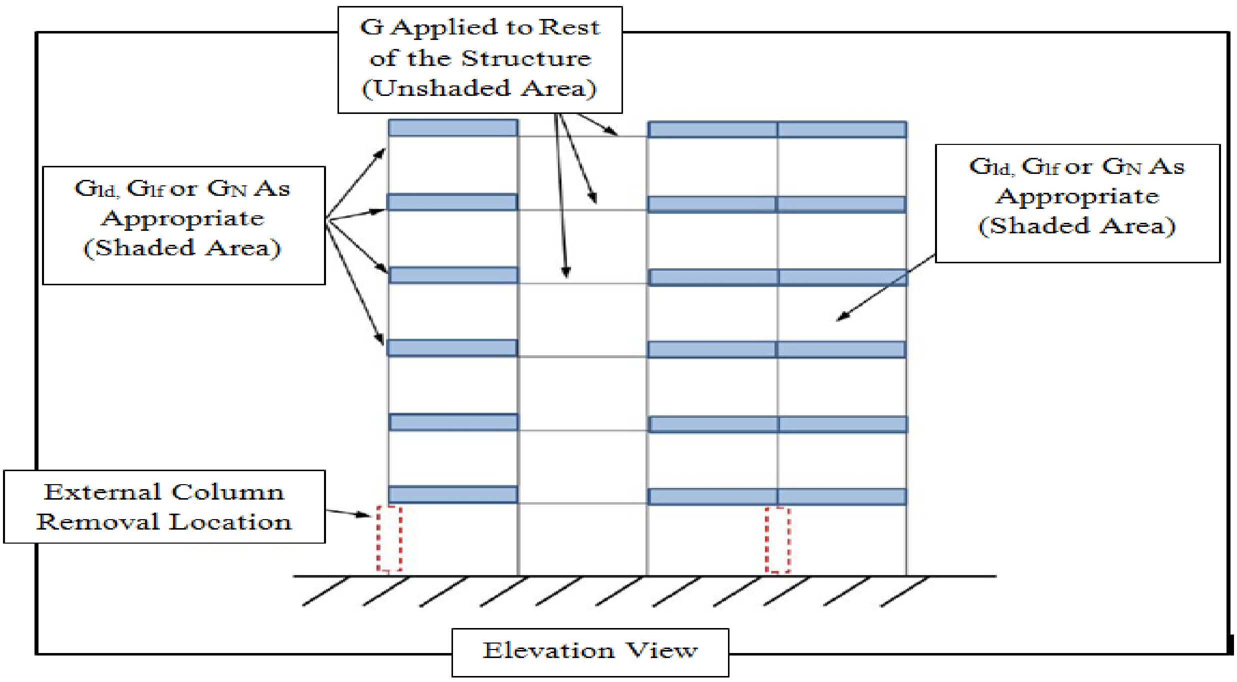
- Deck segments of low-level bridges in regions subjected to coastal inundation should be restrained against uplift and provide with shear keys designed to resist all anticipated lateral loads, ignoring the contribution of gravity-induced friction. Bulkheads and blocking should be designed to allow air to escape from below the deck, thereby reducing the volume of trapped air when submerged.
- Restraint systems for floating structures such as barges should be designed to permit water elevation changes anticipated during the design event. The restraint systems should also be designed for the lateral loads induced by the surge and wave action.
- Standard shipping containers should be considered as the design debris in many developed coastal areas.
- Multistory buildings should be designed for progressive collapse prevention in the event of unforeseen damage to individual structural elements at the lower levels.
- Building and bridge foundations must be designed to accommodate scour induced by the surge and wave action. Scour results from both shear-induced particulate transport and liquefaction-induced soil flow.

3. STRUCTURAL MODELLING

A) Methods of progressive collapse analysis:
1. Linear static Method
2. Nonlinear static method
3. Nonlinear dynamic method.

<u>Loads</u>
Analyze the model with two separate load cases:
1) To calculate the deformation-controlled actions QUD,
2) To calculate the force-controlled actions QUF. Apply the gravity loads to the model using the load cases for deformation-controlled actions and force-controlled actions as defined below.
Load Case for Deformation Controlled Actions. To calculate the deformation-controlled actions, simultaneously apply the following combination of gravity loads:
<u>Increased Gravity Loads for Floor Areas Above Removed Column or Wall.</u>





Apply the following increased gravity load combination to those bays immediately adjacent to the removed element and at all floors above the removed element as shown in above figures.

$G_{LD} = \Omega_{LD}$ [1.2 D + (0.5 L or 0.2 S)]

Where G_{LD} = Increased gravity loads for deformation-controlled actions for Linear Static analysis

D = Dead load including façade loads (kN/m^2)

L = Live load

S = Snow load (kN/m^2)

Ω_{LD} = Load increase factor for calculating deformation- controlled actions for Linear Static analysis; use appropriate value for framed or load-bearing wall structures.

Gravity Loads for Floor Areas Away From Removed Column or Wall.

Apply the following gravity load combination to those bays not loaded with GLD as shown in Figure 1 and Figure 2.

G = 1.2 D + (0.5 L or 0.2 S)

Where G = Gravity loads.

Load Case for Force-Controlled Actions Q_{uf}

Increased Gravity Loads for Floor Areas Above Removed Column or Wall.

Apply the following increased gravity load combination to those bays immediately adjacent to the removed element and at all floors above the removed element as shown in Figure 1 and Figure 2.

$G_{LF} = \Omega_{LF}$ [1.2 D + (0.5 L or 0.2 S)]

Where G_{LF} = Increased gravity loads for force-controlled actions for Linear Static analysis

D = Dead load including façade loads (kN/m^2)

L = Live load

S = Snow load kN/m^2

Ω_{LF} = Load increase factor for calculating force-controlled actions for Linear Static analysis.

Gravity Loads for Floor Areas Away From Removed Column or Wall.

Use Equation G = 1.2 D + (0.5 L or 0.2 S)

Where G = Gravity loads.

Determine the load G and apply it to the structure.

Analysis Procedure

The static linear elastic analysis approach may be used to assess the potential for progressive collapse. Linear Procedure, as suggested in GSA Guidelines, is not intended for predicting the detailed response or damage state that a building may experience when subjected to the instantaneous removal of a primary vertical element. However, a linear static Procedure may be used for determining the potential for progressive collapse (i.e., a high or low potential for progressive collapse).

The potential for progressive collapse can be determined by the following procedure as given in GSA guideline.

Step 1: The components and connections of both the primary and secondary structural elements shall be analyzed for the case of an instantaneous loss in primary vertical support. The applied downward loading on the structure for static analysis purposes is 2(DL + 0.25LL) and for dynamic analysis purposes is (DL+.25LL)

Step 2: The results from analyses performed in step 1 shall be evaluated by utilizing the analysis criteria specified by GSA guideline.

Details of the Structure taken.

Story Height at ground floor	3500 mm
Story Height at typical floor	3000mm
No. of Bays in X-Direction	4
No. of Bays in Y-Direction	4
Bay Width in X-Direction	5000 mm
Bay Width in Y-Direction	5000 mm
Column Size	500 X 500 mm^2
Beam Size	300 X 600 mm^2
Slab Thickness	200 mm
External wall thickness	230 mm
Number of Storey	5
Fck (Characteristic Strength of Concrete)	35 N/mm^2
Fy (Yield Strength of Steel)	500 N/mm^2

Loads on Structure.

Typical Storey slab dead load	5 kN/m^2
Typical Storey slab live load	3 kN/m^2
Frame load on external beam	11.73 kN/m

Tsunami Load Calculations

Buyout Force	F=ρgV
Hydrodynamic Force	F=0.5×ρ×Cd×B×h(u^2)
Surge Force	F=1.5(Hydrodynamic force)
Impact Force	F=C×u× (km)˜0.25
Hydrostatic Force	F=0.5×ρ×g×b×H

Notations used for Tsunami Design Load Evaluation	
ρ= density of salty water (kg/m^3)	g=gravitational acceleration (m/sec^2)
h= surge height (m)	B= width of column (m)
A_f= tsunami exposed area = B × h (m2)	C_d= drag co-efficient = 2
R= design run-up height (m)	Z= height of ground at site (m)
d_t= impact time of object (sec)	m= mass of impacting object (kg)
γ= specific weight of salty water = ρ × g(N)	V= area of floor × h (m3)
u= tsunami velocity in m/s	Cu= uplift force co-efficient = 3

Buoyant Force	32.12 kN/m
Hydrodynamic Force.	250 kN/m
Surge Force	375 kN/m
Impact Force-1 (A box Shipping Container)	1207.4 kN
Impact Force-2 (Wooden log)	147.4 kN
Hydrostatic Force	1.08 kN/m
Hydrostatic Uplift Force	0.8 kN/m

Application of Force

Name of Force	Location
Buoyant Force	On floor area in upward direction.
Hydrodynamic Force.	Uniform load on column up to Tsunami height.
Surge Force	Uniform load on column up to Tsunami height.
Impact Force-1 (A box Shipping Container)	As point load on leading edge of structure.(Joint forces)
Impact Force-2 (Wooden log)	As point load on leading edge of structure. (Joint forces)
Hydrodynamic Uplift Force	On floor area in upward direction.

Load Combinations taken

1. Fs
2. Fd
3. Fd +Fs
4. Fd + F_{I-1}
5. Fd + F_{I-2}
6. DL
7. LL
8. 1.05 DL + 1.275 LL + 1.275 (F_{I-1} +Fd)
9. 1.05 DL + 1.275 LL + 1.275 (F_{I-2} +Fd)
10. 1.05 DL + 1.275 LL + 1.275 (F_{I-1} + Fd+ Fs)
11. 1.05 DL + 1.275 LL + 1.275 (F_{I-2} + Fd+ Fs)
12. 0.9 DL + 1.3 (F_{I-1} + Fd)
13. 0.9 DL + 1.3 (F_{I-2} + Fd)
14. 0.9 DL + 1.3 (F_{I-1} + Fd + Fs)
15. 0.9 DL + 1.3 (F_{I-2} + Fd + Fs)

4: Step by Step Procedure.

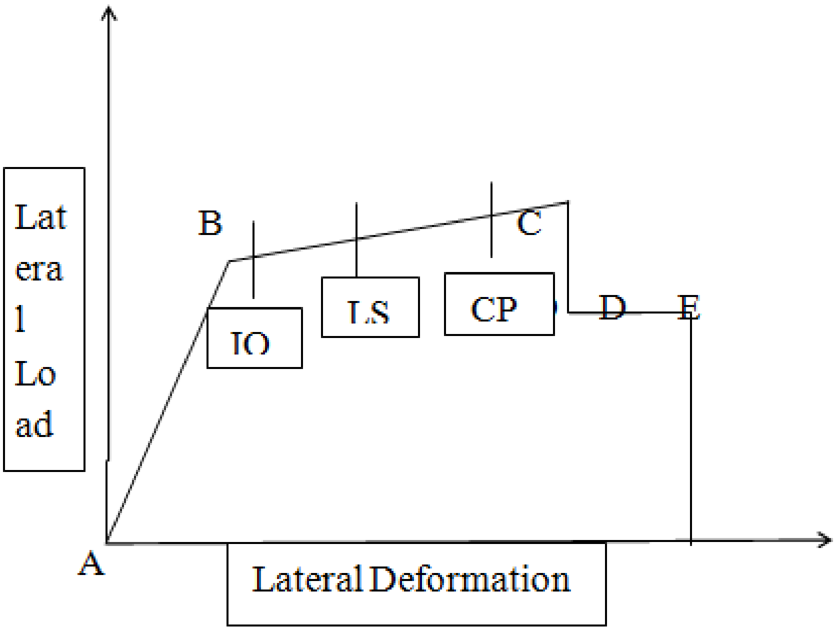


Figure 1 Force Displacement Curve

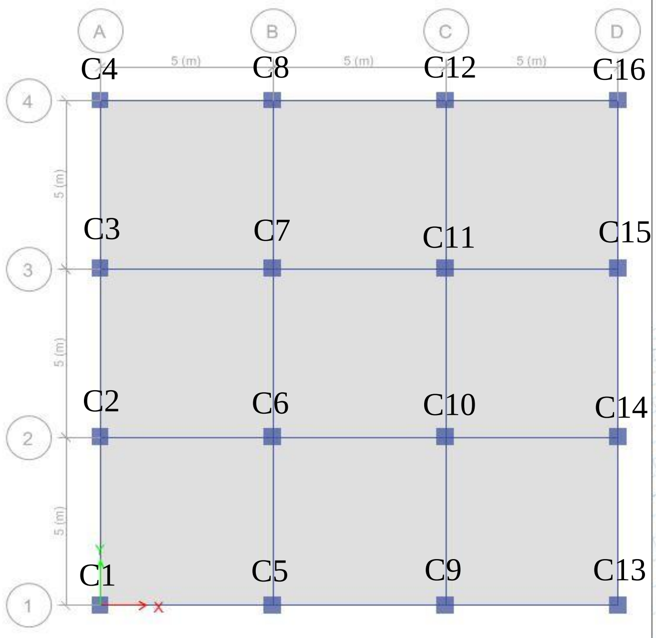
In the present study, the nonlinear analysis is carried out in software ETABS 2015. ETABS follows the ASCE 41-13 guidelines for default hinge properties. For each degree of freedom there is a force displacement (moment-rotation) curve, that gives yield value and plastic deformation following yield. Fig. 3 shows the force displacement curve with values at five points A-B-C-D-E.

In the curve shown in fig.3 point A is origin point. Point B shows yield condition. Point C is the ultimate capacity. After reaching ultimate capacity at point C, strength suddenly decreases and reaches to point D having some residual strength. Point E is the final deformation under residual strength. Intermediate points IO (immediate occupancy), LS (life safety), and CP (collapse prevention) shows capacity at 20%, 50% and 90% of ultimate capacity.
The stepwise procedure to perform nonlinear analysis for progressive is illustrated below:

Step 1: Prepare finite element models in ETABS. Perform concrete design and determine the reinforcement to be provided in members.
Step 2: Define hinge properties for beams and columns. The default hinge property is available in the software. Apply default plastic hinges or user defined plastic hinges to beams and columns.

Step 3: Define nonlinear case. Use P-Delta option in nonlinear case. Define required load pattern as specified in GSA guideline. Set required number of saved steps.

Step 4: Create column loss scenario and perform the static nonlinear analysis.
Step 5: Observe the hinge formation pattern.
Step 6: Determine vertical displacement corresponding to the applied load in each step. Plot the graph of load v/s displacement which shows the failure pattern.



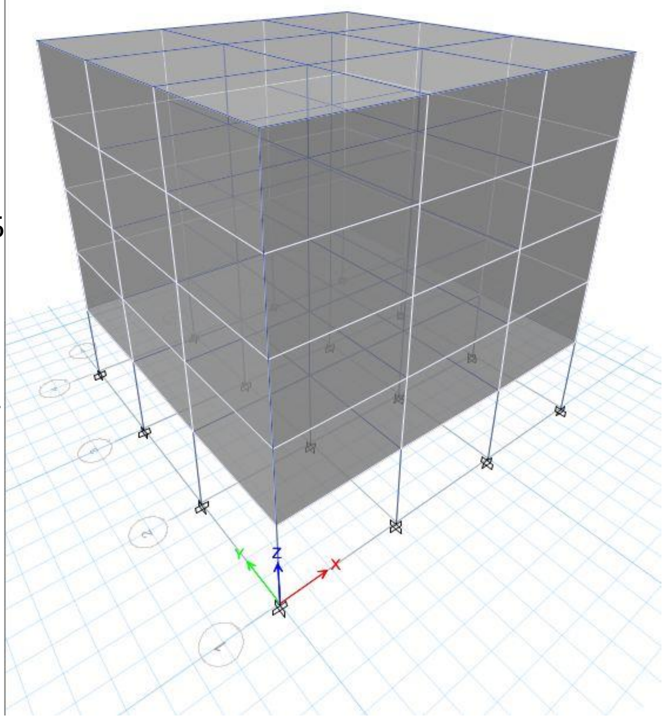


Figure 2 Plan and 3D View of Structure

Single Column Removal Cases.

Two Column Removal Case.

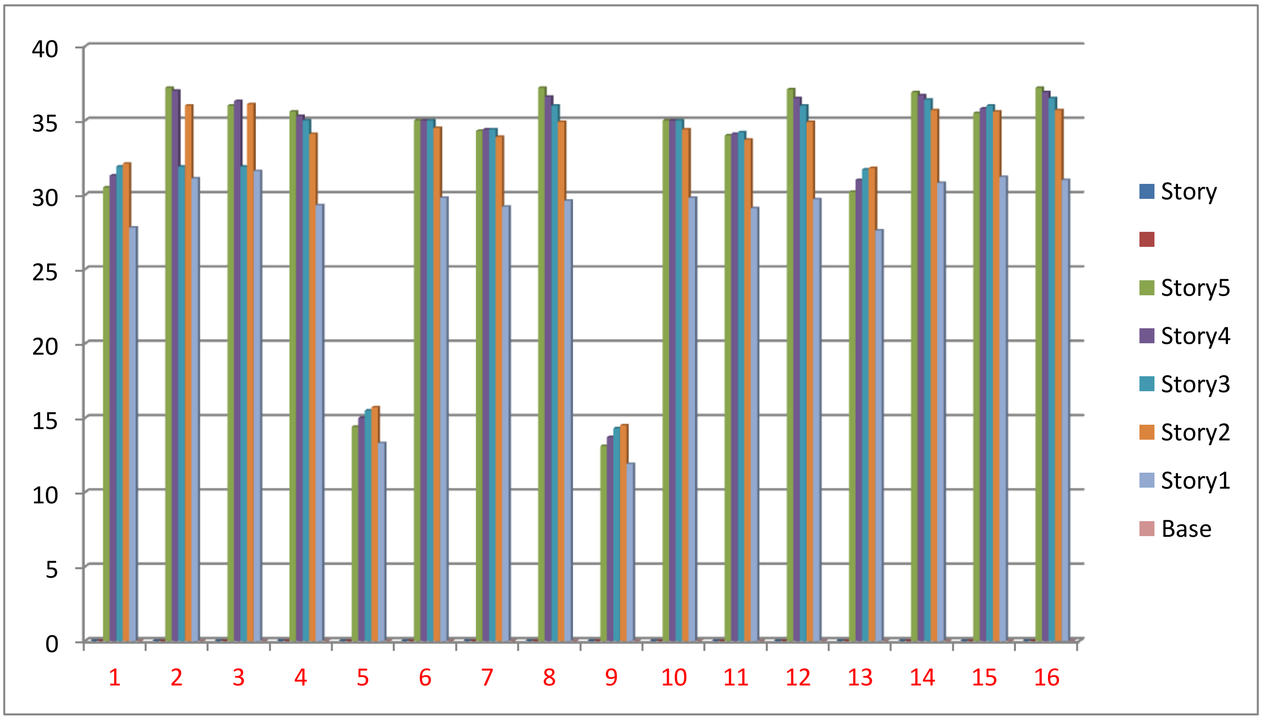
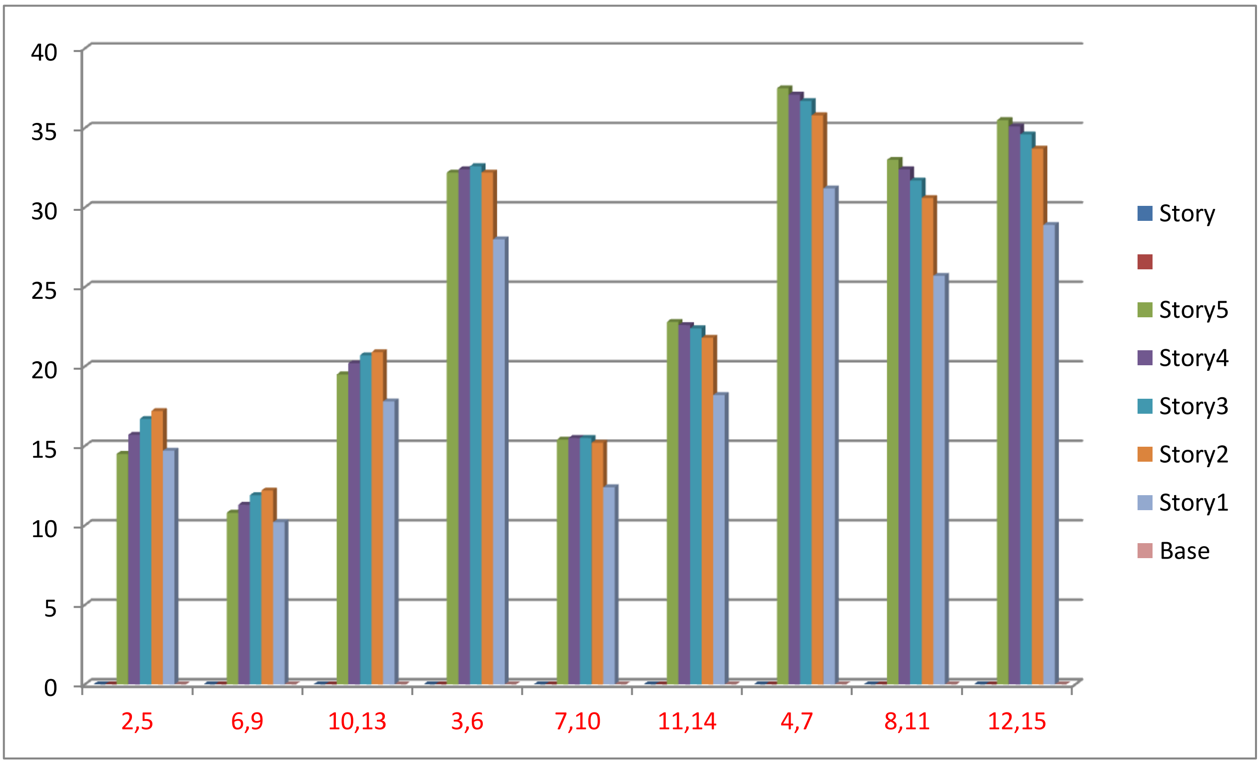


Figure 3 Single Column Removal Case

4. CONCLUSIONS



Column Number

Figure 4 Two Column Removal

- As we move away from the Tsunami forces along Y-Direction the criticality of Storey goes to Storey 2,3 and 5
- Along X-Direction the displacement in Storey goes to 2, 3 and 5.
- In two column removal cases from resultant direction (Diagonal cases) if both exterior columns of (column 2, 5 and column 12, 15) near end and far end from Tsunami forces the maximum displacement occurs at Storey 2 and Storey 5.
- Near end and far end from Tsunami forces the maximum displacement occurs at Storey 2 and Storey 5.
- Also in two column removal cases if both the internal columns are removed (column 7 and 10) the maximum Storey displacement will be at Storey 3.
- Also in case of one internal and one external column removal case (column 10 and 13) Storey 2 is critical.
- As in case of single column removal (near to Tsunami force, {column 1, 5, 9, 13}) the potential of progressive collapse to start is at Storey 2.
- From the graphs it can be concluded that in case of single column removal (collapse) the maximum deflection is in Storey 5 with 25.67% and 18.26% as compared to Storey 1 along Y and X direction respectively.

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