CERTIFICATION

ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

As members of the Examining Board of the Final M.Sc. Open Defense, we certify that we have read and evaluated the thesis prepared by: **Yohannes Lisanework** entitled: **Construction Stage Analysis Effect of Reinforced Concrete Buildings.**

And recommended that it be accepted as fulfilling the thesis requirement for the degree of: **Master of Science in Civil Engineering (Structural)**

Chairman	Signature	Date
Advisor	Signature	Date
Internal Examiner	Signature	Date
External Examiner	Signature	Date

ABSTRACT

The design of buildings is highly dependent on soft-wares so as to best represent the structure, reduce errors, efforts and time consumption. SAP, the most known soft-ware, enables the analysis and design of structures.

Although the software has two features, one enabling the analysis of structures once the members are fully represented (modeled) and they are completely loaded; and the other enabling the analysis of structures segment by segment as per the interest of the designer following the real construction sequence; designers do not have more practice, clear well studied guidance for the selection of either methods.

The non-linearity considered here is that caused by the stages only. The material and geometric non-linearity are not considered. The creep and shrinkage effects are also excluded to simplify the task.

This work is compiled of five chapters being the first one dealing on the introduction of the paper work. Literature review is dealt on the second chapter. The third chapter deals on the methodology followed for this work. The analysis results of both methods for the five buildings of rises of G+5, G+10, G+15, G+20 and G+25 are presented and discussed in the fourth chapter. The fifth chapter gives conclusion and recommendation.

Key words: *Complete Analysis Method (CA), Construction Stage Analysis Method (CSA), Non-linear Analysis.*

ACKNOWLEDGMENT

I want to give thanks to my GOD who helped me to do so including this project work.

A lot of effort is made by my advisor Dr. Ing. Adil Zekaria starting from the title selection to the up to the completion of the work. Thus, my gratitude goes to my advisor for all his help, treatment, advice and guidance.

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TABLE OF CONTENTS

Contents Pages

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREVIATIONS

CA= Complete Analysis

CSA= Construction Stage Analysis

DL= Dead Load

LL= Live Load

EQx= Earth Quake along X direction

EQy= Earth Quake along Y direction

Comb= Combination

1. INTRODUCTION

Different reasons may be mentioned for structures not to be completed once. For reinforced concrete structures the most unavoidable reason is the time needed for the hardening of concrete mass. The financial capacities of clients and the disputes between clients and contractors are also some of the reasons for such delay.

Now days, a number of buildings are constructing everywhere in Ethiopia. Although there are steel made buildings, most of the constructions are made of reinforced concrete material. There are also some design offices with experienced designers handling this huge-nation building mission.

“The structural self-weight, external loads, boundary conditions and materials are depended on stages during the construction process and their variations are overlooked in conventional design which is nothing but a limitation of conventional design procedure.” Yousuf Dinar, 2014

The design practice taking place is usually based on the assumption that the building is constructed and completed once and exposed to the final loadings. Such analysis method, which is called Complete Analysis or One-Step Analysis, is far from the truth. Practically, buildings are constructed floor by floor with clear gap of time and difference in loadings. Especially, this condition is true for buildings made of reinforced concrete.

Staged construction Analysis or Construction Stage Analysis refers to the analysis of structures step by step as in the actual construction process is done .Floors are constructed sequentially one after the other. Hence, they are also loaded step by step as it occurs practically.

The SAP version 14 is used to model buildings of different rises and analyze by the traditional complete analysis and the stage by stage loading consideration analysis method.

This paper tries to investigate the internal response variation of reinforced concrete buildings when analyzed using the traditional complete construction analysis method and the construction stage analysis method for three buildings of different number of stories.

1.2 Back Ground

Many works are not found under the title mentioned on this paper. In 1978 SC Chakrabarti together with his friends conducted a research under the title "*effect of sequence of construction in the analysis of multi storied building frame* "and found a remarkable observation. As per their observation under the self-weight loads, end moments created on beams by the construction stage analysis are reduced from 50% to 80% of that of the One Step analysis. And also the top column moments decreased while the bottom column moments increased when analyzed by construction stage method at each joint.

In January 2014, the thesis named as "*Chronological Construction Sequence Effects on Reinforced Concrete and Steel Buildings"*, was studied by Yousuf Dinar considering only self- weights of buildings.

He handled the analysis of six framed 3D buildings of constant story number difference using ETABS 9.7.2 and discussed on the internal response variation by the two analysis methods on critical members and joints. He added a foundation less column starting from the first story for clear observation of the response and found that for a column axial force, critical beam moment, shear and displacement values are higher when analyzed by CSA than by CA.

Buildings are actually constructed stage by stage. Hence, they need to be investigated through this analysis. More works on buildings under construction stage analysis should better be done to well study their effects and reach at important conclusions.

1. 3 Objectives

The general objective of this paper is to assess the internal response variation of reinforced concrete buildings analyzed by the conventional complete analysis and the real stage construction analysis.

The specific objectives are;

- To expand the awareness of construction stage analysis being it is real.

- To calculate the axial, flexure, shear and displacement of buildings considering stage construction.
- To compare the construction stage analysis result to the complete analysis response.
- To recommend on the effect of the construction stage analysis method.

1.4 Limitations

- This study is done by taking five reinforced concrete buildings of heights 22m, 37m, 52m, 67m and 79m. Thus, the maximum height of the buildings is 80m. This may not represent too high buildings.
- To simplify the task, the selected buildings are symmetric and regular both in plan and in elevation. Thus, their representation to irregular buildings might be questioned.
- Walls, floor finishes practically may be constructed starting from any story and continue randomly. But for this work it is assumed that they are constructed from the ground floor to the top.
- For clarity, ease of work and minimizing complexity, creep, shrinkage and aging effects of the concrete are not here considered.
- Each floor as constructed is assumed to be loaded by service load (start giving service - as is in the real world now-a-days) although the whole structure is not completed. Thus, every floor is loaded by live loads as form works supporting upper floor are removed from this floor.

2. LITERATURE REVIEW

2.1 Methods of Analysis

Analysis of structures refers to the process which enables determine the amount, direction and shape of the structural responses because of the design loads.

The structural responses such as axial force, shear force, moment, torsion, deflection and rotation show how the structure behaves when loaded.

There are a number of methods of analysis. Broadly analysis methods are classified as Static and Dynamic.

Static Analysis

Dynamic effects of loads on structures can be neglected when the deformation, velocity and acceleration are small. In such a case the analysis method used is Static.

Dynamic Analysis

This method is applicable for cases that deformation, velocity and acceleration are considerable.

Stiffness: is the property of a structure which characterizes the response of the structure to the applied load. Stiffness is affected by cross-sectional shape, material used and support condition.

Shape: since stiffness is a function of the moment of inertia of the cross section, structures of different shapes of same area have different stiffness values. The stiffness of a rectangular section, circular, I, channel…are different.

Material: The elasticity of different materials is different. The elasticity of steel and concrete are not the same; thus, differing in the stiffness values.

Support Condition: The resistance of a fixed supported beam to bending moment is higher than that of the simply supported. Thus, stiffness is affected by support type.

Static analysis can further be divided in to Linear and nonlinear.

Linear Static Analysis: A structure which retains its stiffness after loading is considered as Linear. For a linear structure, the Stress-Strain graph is linear. Here the assumption is that the shape of the material is not changed assuming small deformation and the material type is not changed.

$Ku = r$

Where, K- stiffness

u- displacement

r- loading

Nonlinear Static analysis: Because of any factors mentioned above, if the structure fails to retain its initial stiffness after loading, the stress-strain graph no longer remains linear. Thus, the structural properties are changed during loading and through time.

Material nonlinearity refers to a property of a material having no stress-strain linearity. I. e, the change in load is not proportional to the change in deformation. A nonlinear material fails to return back to its initial unstressed shape: to mean, it develops permanent deformation.

Geometric non linearity concerns to the change in the load-deformation characteristics of the structure because of the change on the shape of the member as a result of large deformation. Even if each stage is linear, the fact that structure changes from one stage to another is taken as nonlinear.

Nonlinear static analysis is important for the analysis of structures with material, geometric non linearity, to investigate staged construction with time dependent material behavior.

Staged construction: considers the changes in the structure, aging, creep, shrinkage of the structural elements.

2.2 Construction stage analysis effect on simple frame

Consider a two-story 2D frame with story height 1.5m and bay width 2m loaded to dead loads of 5KN/m as shown.

One can load the frame with the stated loading fully at a time. As a result, the particles of the reinforced concrete frame respond to the loads at a time.

Another one can also apply loads to the first story of the frame first as the second story is not constructed yet; and then to the second story.

The responses are different for the two cases. For instance, the positive bending moments on the beams are higher when the frame is loaded gradually than suddenly; while, the negative moments are lower.

When the frame is loaded suddenly, each particle starts to respond at the same time. Thus, the internal response of a single particle doesn't influence another particle sequentially. The effect of the first floor is also transferred to the second being they are constructed once. But, when gradually loaded, the particles at the second story influence the previously influenced particles at the first floor. This is to mean that, the material within the first floor has already changed its length, orientation resulting change in the stiffness leading to the nonlinearity because of gradual loading.

The joint (connection) fixity of frame elements to each other for the complete method of analysis model and construction analysis model are different in that the fixity for the CA model is once set and not changed throughout.

But for the CSA model the fixity of a given joint is changed from stage to stage (generally, increasing) as the given beam-column joint at the first stage is well- stiffened by another stage members added to that point.

This indicates that there is clearly change in the stiffness of members as there is change in the support type (connection type) from stage to stage. Thus, because of change in the stiffness, there is Non linearity on the structure modeled for CSA.

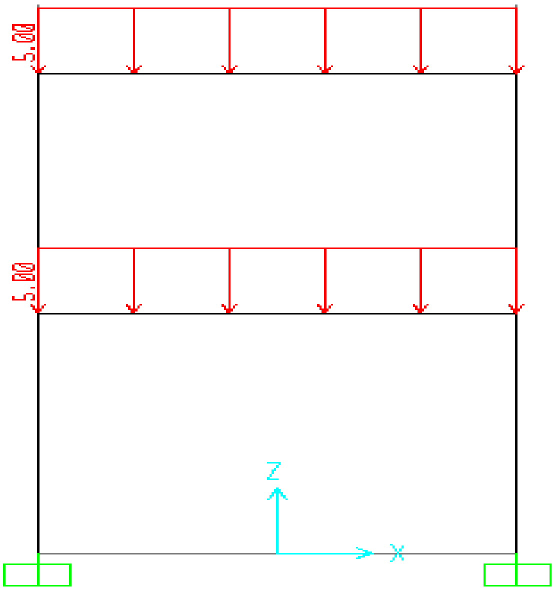


Figure 2-1 Complete Analysis Model

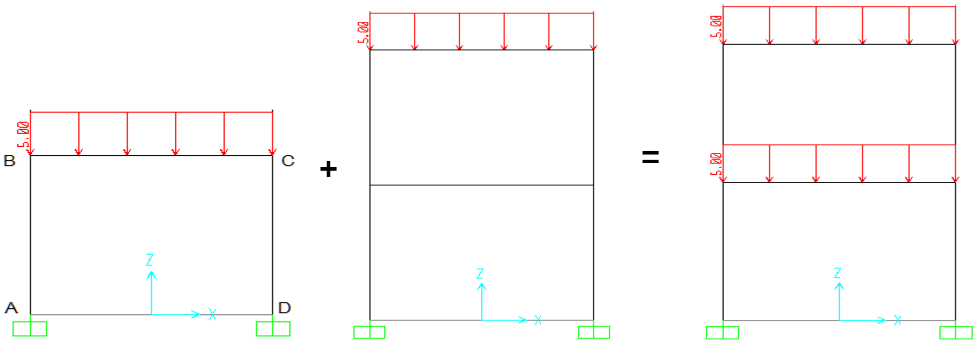


Figure 2-2 Stage Analysis Model

Using slope-deflection method of analysis:

$M_{ij} = 2EI/L_{ij}\,(2\theta_i+\theta_j-3\Delta/L_{ij}) + FEM_{ij}$

$M_{AB} = 2EI/L_{AB}\,(2\theta_A+\theta_B-3\Delta/L_{AB}) + FEM_{AB}$

$M_{BA} = 2EI/L_{AB}\,(2\theta_B+\theta_A-3\Delta/L_{AB}) + FEM_{BA}$

$M_{BC} = 2EI/L_{BC}\,(2\theta_B+\theta_C-3\Delta/L_{BC}) + FEM_{BC}$

$M_{CB} = 2EI/L_{BC}\,(2\theta_C+\theta_B-3\Delta/L_{BC}) + FEM_{CB}$

$M_{CD} = 2EI/L_{CD}\,(2\theta_C+\theta_D-3\Delta/L_{CD}) + FEM_{CD}$

$M_{DC} = 2EI/L_{CD}\,(2\theta_C+\theta_D-3\Delta/L_{CD}) + FEM_{DC}$

Equilibrium at joints

Joints B and C

$M_{BA}+M_{BC}=0$

$M_{CB}+M_{CD}=0$

Fixed End Moment of segments AB and CD is zero because there is no load on the span.

$FEM_{BC}=-5*2^2/12=-1.67$

$FEM_{CB}=5*2^2/12=1.67$

Solving the above equilibrium equations, $\theta_B = -0.271/EI$ and $\theta_C = 0.136/EI$

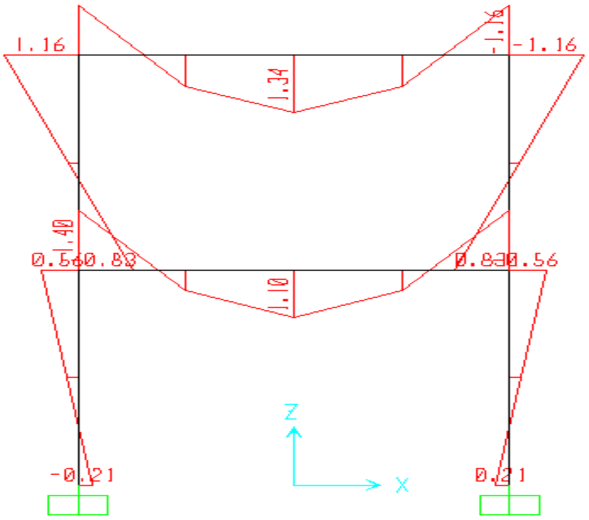
Thus,

M_{AB}= -0.361KNm M_{CB}= 1.107KNm

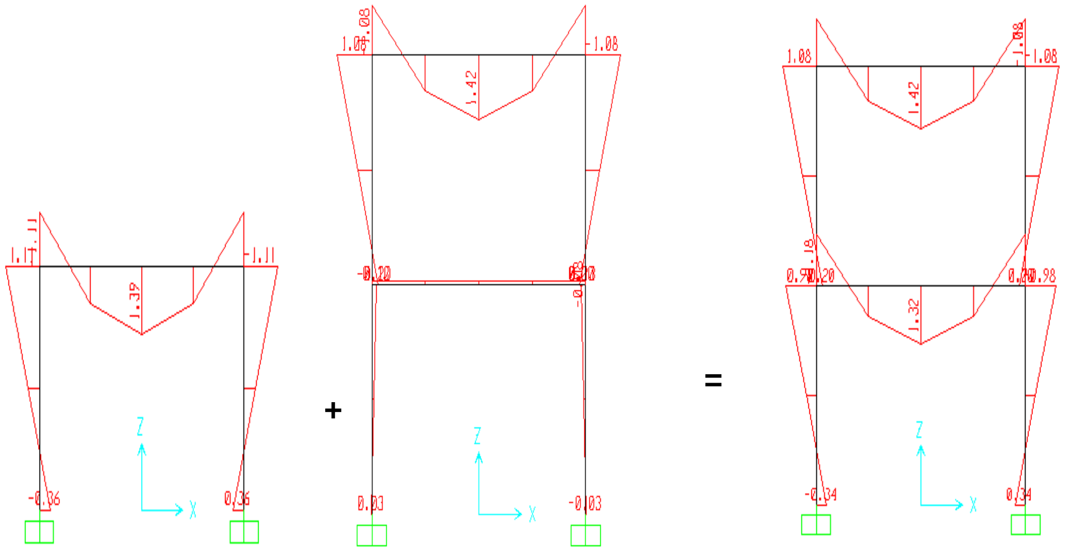
M_{BA}= 1.107KNm M_{BC}=-1.107KNm

M_{CD}= -1.107KNm M_{DC}= 0.361KNm

All the moments are thus calculated similarly and from this the moment diagram is drawn.



a.CA results



b.CSA results

Figure2-3 CA and CSA analysis results for a simple 2D frame

Support moments at the first story and second story by CA analysis method are 1.40KNm and 1.16KNm respectively. Whereas, support moments by CSA analysis method are 1.18KNm and 1.08KNm respectively.

Span moments at the first story and second story by CA analysis method are 1.10KNm and 1.34KNm respectively. Whereas; span moments by CSA analysis method are 1.32KNm and 1.42KNm respectively.

This indicates that, CA analyzed support moments become greater than CSA analyzed support moments but CSA analyzed span moments become greater than CA analyzed span moments.

3. METHODOLOGY

Thorough material collection and reading on the construction analysis is first done. Selection and modeling of reinforced buildings of stories G+5, G+10, G+15, G+20 and G+25 has followed. The use of important software, SAP V14.0.0 advanced, for the modeling and analysis is implemented. The analysis is handled with the consideration of earth quake. No material and geometric variation is considered.

Mainly the focus is on the deflection, flexural, axial and shear response variation of the structures when analyzed by CA and CSA. The out-put values are tabulated and drawn by graphs using Microsoft Word 2010 and Excel 2010.

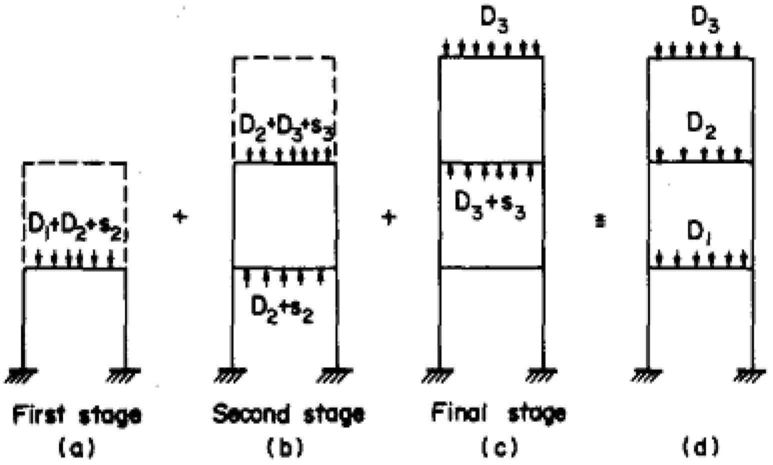
3.1 Modeling structures on SAP

Models used so far to represent the CSA and Conditions Considered

Basically, the construction stage analysis models follow the principle that floors/structural elements are constructed stage by stage. Here are three different models used in three papers to represent the construction stage analysis and their considerations are also described as below.

1. Model used for the "Effect of Sequence of Construction in the Analysis of Multi Storied Building Frame" prepared by S.C. CHAKRABARTI and his friends, 1978

In this study, a two dimensional, 11 storied, three- bay frame with 38.6m total height, 15.8m total width and 9 storied, two- bay frame with 28.2m total height, 7m total width are selected. Wind load and seismic load have been used in addition to self-weight. The reinforced concrete columns and slabs are considered to be constructed stage by stage. But the other nonstructural elements such as walls, floor finishes are assumed to be built after the structural element is completed.



Pattern of analysis for simulation of construction sequence.

Figure 3-1 CSA model number one

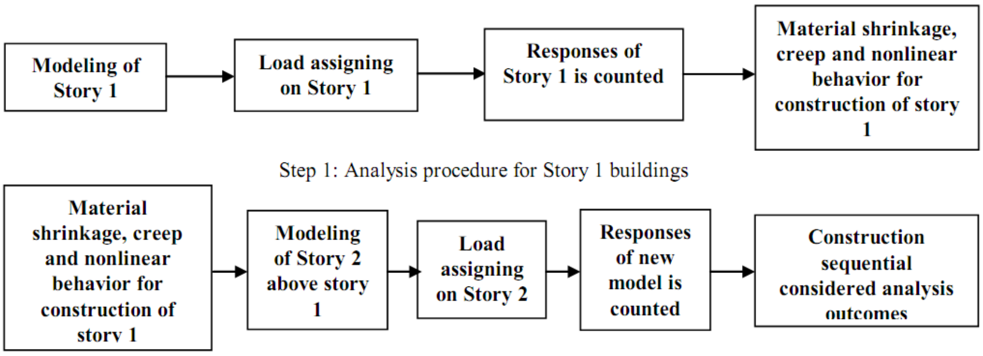
In this work, the floor constructed before is assumed to attain its strength as the construction of the next floor is started. Loads applied to the first floor are the weight of the first floor slab and beam weight (D1), the weight of the second floor slab (D2) carried by the first floor by form works and the weight of the shutterings (S2) . When the third floor is started to be constructed, the shutterings are then removed from first floor to the second to support the third floor and the load is named as S3.

Thus, load at the first floor is D1+D2+S2 and at the second floor D2+D3+S3. But, when the second floor is loaded, the load at the first floor reduces to D1 because the shuttering is removed from the first floor and placed at the second floor. The net stress developed at the frame as the second floor is constructed is the algebraic summation of the stresses of the single storyed frame and the two storyed frame. In the same way, the final stress on the structural members is the net stress of all the stresses created at each structure by each stage

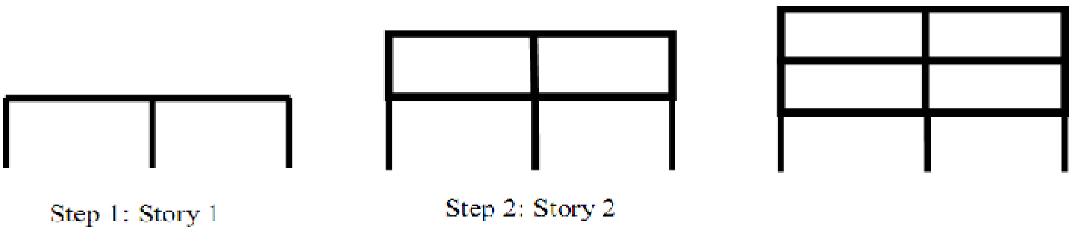
2. Model used for the “Chronological Construction Sequence Effects on Reinforced Concrete and Steel Buildings” prepared by Yousuf Dinar and his friends, 2014

For this model, buildings with Story 5, Story 10, Story 15, Story 20, Story 25 and Story 30 with their respective total height of 15m, 30m, 45m, 60m, 75m and 90m are selected. Each building story height is 3m and bay width of 5m.

Three dimensional analysis models are formulated to account the time dependent effects of creep, shrinkage, concrete-stiffness variation with time, sequential loading and foundation settlement. In plane rigidity of floors are assumed. Seismic load is considered according to UBC94 with seismic coefficient of seismic zone 2, moderate risk rated arena of Bangladesh. The buildings are first designed and their analysis results are considered.



Procedure followed in the analysis



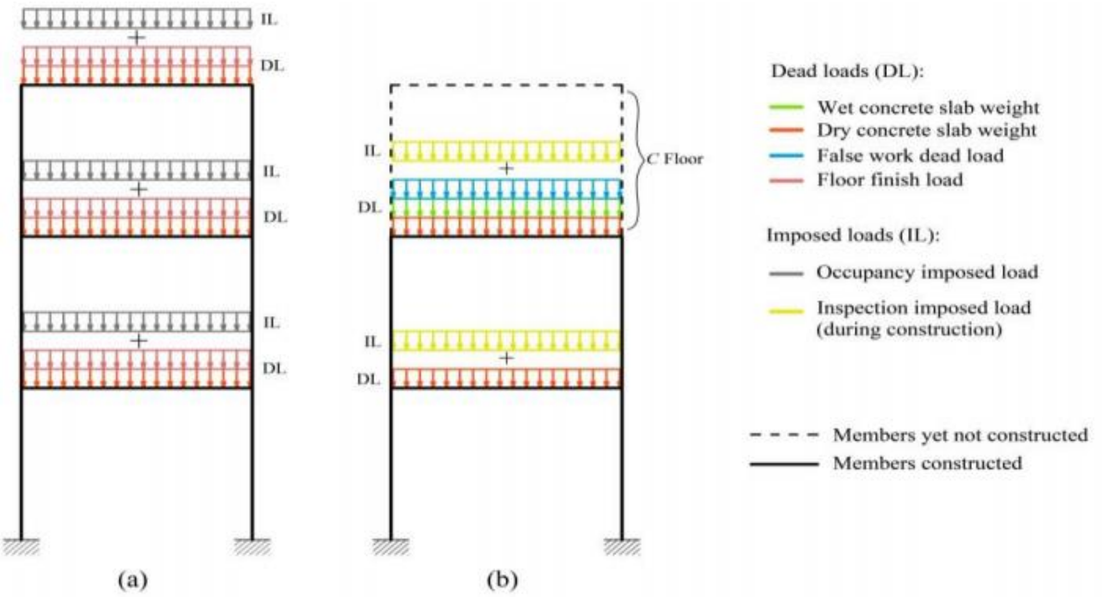
Construction sequential analysis

Figure 3-2 CSA Model number two

3. Model used for the "Construction Stage Analysis of RCC Frames." Prepared by K.M. Pathan, 2014

In this model two building of G+5 and G+7 with bay width along both directions of 4m are used. By varying the bay width in to 5m and 6m, different stiffness values are tested. The story height is of 3m.

The floor below (C-1) carries the self weight of the above floor (C) under construction by means of form work. The construction live load on the wet floor (C) is also transferred to the floor below. The C floor resists the above mentioned loads in addition to its self weight and inspection load on the C-1.



(a) Conventional Analysis; (b) Construction Stage Analysis.

Figure 3-3 CSA model number three

The loads considered are:

Dead Loads: Self weight of columns and beams, self weight of dry concrete slab (unit weight=25 KN/m3), Self weight of wet concrete slab (unit weight=26 KN/m3), Form work dead load=500 N/m2

Imposed Loads: Inspection live load on floor slab being constructed = 750 N/m2, Inspection live load on floor slab under construction=750 KN/m2.

All the three models take the consideration that each floor is constructed stage by stage.

Model 1 considered the self weight of the the building and is modeled as 2D. In model 2, an intermediate lower most column is removed so as to visualize the difference easily and the creep and shrinkage effects are considered in model 2,

Loads on supporting dry floors (loads from the wet floor, form work of the wet floor, construction live loads) are removed as each floor is constructed and became dry.

Model used in this paper

This project is based on the linear static analysis for complete analysis models and nonlinear static analysis for segmental analysis models. Six buildings having story of G+5, G+10, G+15, G+20 and G+25 are selected for this study. The G+5 building model has 3 bays along both X and Y axes of length 4 meters each with ground floor height of 3.6 m and the rest stories of height 3 m. There are two G+10 building models having 5 bays along both X and Y axes where one is without shear wall and the other with shear wall. The G+15 building and G+15 building model has 7 bays along both X and Y axes. The G+20 building model has 9 bays along both X and Y axes. The G+25 building has 11 bays along both X and Y axes. For all the five buildings, foundation column length is taken as 3 meters, the ground floor column as 3.6 meters, the rest story height as 3 meters, the beam length as 4 meters, the slab thickness as 15 cm and shear wall varying between 20 cm and 40 cm.

Proportionality of the buildings

The selected five buildings have constant story interval. The buildings are ground floor plus: FIVE, TEN, FIFTEEN, TWENTY and TWENTY FIVE. The corresponding plan sizes of the buildings are selected so as the height to width ratio of each building is fixed between 1.7 and 1.8.

Material and Sizing

The standard grades of materials selected for this work are C-25 for concrete and S-300 for reinforcing bars as in the EBCS 2/1995 code. Unit weight of normal concrete 24KN/m^3, of wet concrete 25KN/m^3, of steel 77KN/m^3.The five buildings selected are symmetric both horizontally and vertically. All beam and column members have rectangular shape. Beams and columns are well connected to each other. Sizes of columns and beams resulted from safe and economical design is as summarized here.

Table 3.1 Beam and Column Cross sectional size

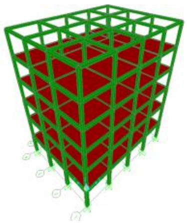
Building	Found. CL	Floor CL	Floor BM	Top-tie BM	(Shear wall)
G+5	450*450	400*400	250*300	250*250	No
G+10	550*550	500*500	300*350	250*250	No

G+10	500*500	450*450	300*350	250*250	Yes
G+15	1100*1100	1000*1000	300*450	250*300	Yes
G+20	1300*1300	1250*1250	350*500	250*300	Yes
G+25	1550*1550	1500*1500	400*500	250*300	Yes

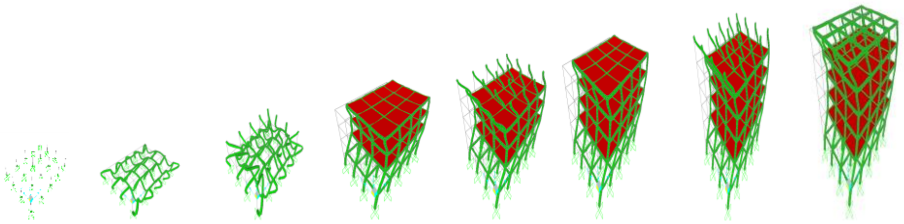
Units are in mm

The cross sectional sizes of columns at upper stories are lower than the sizes shown for corresponding building although not indicated in the table.

Each floor is made rigid in X-Y plane (using diaphragm). Shear walls are included in this study. Material and geometric non linearity is not included. Only the construction stage non linearity because of the step by step construction is considered. For the purpose of staged construction analysis, so as to observe the stability, columns of the same level are taken as one stage and beams and slabs of the same level are taken as another stage.



a) Five - story building CA model



Stage1 St 2 St 3 … St 8 St 9 St 10 St 11 St 12

b) Five - story building CSA model

Figure 3-4: Models for Stages of Construction

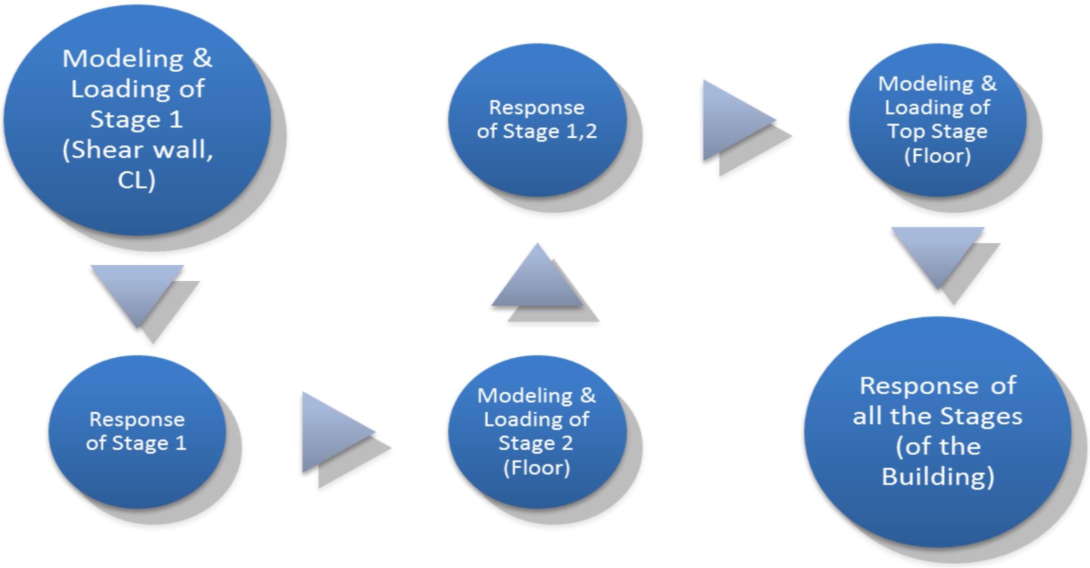


Figure 3.5 Construction Stage Analysis Step

For this model in the first stage, foundation columns and shear walls (if any) at the same level are considered as Stage 1. In this stage, their self-weight is applied to them.

After the structures in the first stage have responded, beams and slabs are constructed as Stage 2. In this stage, self-weight, lateral load, construction load, weight of wet slab from the above floor and weight of the supporting form work are applied.

As the structure has responded for the loads till this stage, next stage continues with the construction of columns and shear walls. In such a manner is the final stage becoming with the overall response of the structure.

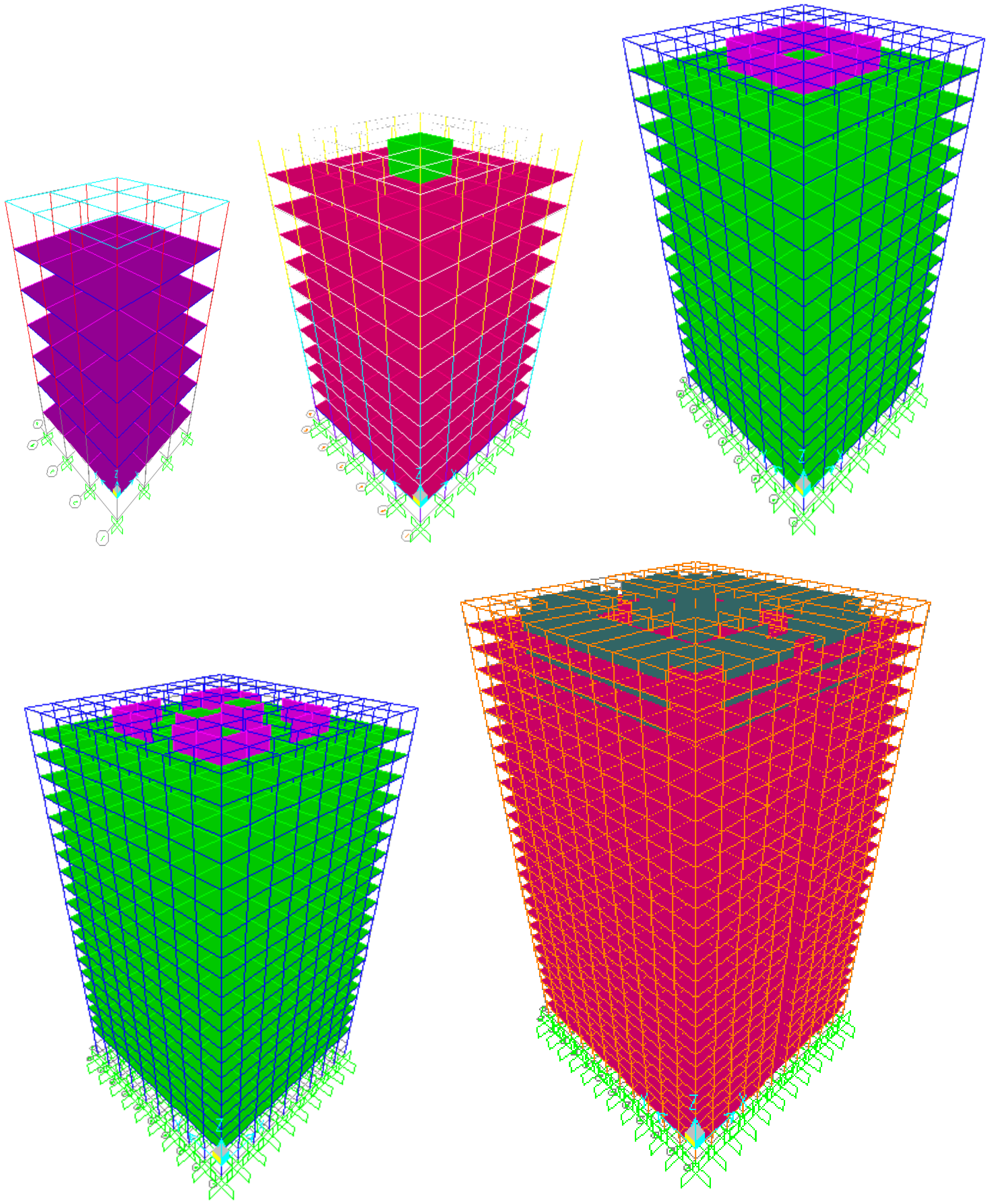


Figure 3.6 Model for G+5, G+10, G+15, G+20 and G+25 buildings

3.2 Loading structures on SAP

The self-weight of the buildings is considered by providing self-weight multiplier value ONE. All the slabs are loaded uniformly by 1.2 KN/m^2 dead load(floor finish), 3 KN/m^2 (according to EBCS 1, 1995 Category B C1) live load,. Form work 0.5KN/m^2, construction/inspection load of amount 0.75 KN/m^2 and upper floor wet slab weight of amount 3.75KN/m^2 to be resisted by the lower dry slab are considered for CSA case only. And the beams are uniformly loaded by dead partition load of amount 10 KN/m. Wet floor weights of upper floor slabs are transferred to the immediate lower floor (starting from the first floor) and are removed as the next floor is hardened. Earth quake is included with in the loads. Load cases are combined to give the combinations considered in the EBCS 2, 1995 code and two models for every building type are taken; one for complete analysis and another segmental analysis.

The load types used in the load pattern are DEAD, LIVE and the EARTH QUAKE in both X and Y directions and in both positive and negative directions.

For the complete analysis, the following load combinations are used:

Comb 1= 1.3 DL+1.6 LL

Comb 2=0.75 (Comb 1) + EQx

Comb 3=0.75 (Comb 1) - EQx

Comb. 4=0.75 (Comb 1) +EQy

Comb 5=0.75 (Comb 1) – Eqy

Comb 6=0.75 (Comb 1) + EQx (with 0.05 eccentricity)

Comb 7=0.75 (Comb 1) - EQx (with 0.05 eccentricity)

Comb. 8=0.75 (Comb 1) +EQy (with 0.05 eccentricity)

Comb 9=0.75 (Comb 1) – Eqy (with 0.05 eccentricity)

Comb found.= DL+LL, for foundation design

Comb ENV= max/min of (Comb 1 to Comb. 9)

For the staged construction analysis the following load combinations are used: Staged construction analysis CSA1, CSA2, CSA3, CSA4, CSA5, CSA6, CSA7, CSA8, CSA 9, CSA found., CSA envelope= max/min of (CSA1, CSA 2,…, CSA 9) for each stage members are added, objects are loaded by the load combinations described above.

When using SAP, the CSA combinations are defined in the Load Combination as shown below because SAP doesn’t list the load combinations in the load cases disabling the use of ENVELOPE for maximum and minimum values.

Comb CSA 1= CSA 1

Comb CSA 2= CSA 2

Comb CSA 3= CSA 3

Comb CSA 4= CSA 4

Comb CSA 5= CSA 5

Comb CSA 6= CSA 6

Comb CSA 7= CSA 7

Comb CSA 8= CSA 8

Comb CSA 9= CSA 9

Comb CSA foundation = CSA foundation

Comb CSA ENVELOPE= max/min (comb CSA 1, comb CSA 2, comb CSA 3, comb CSA 4, comb CSA 5, comb CSA 6, comb CSA 7, comb CSA 8, comb CSA 9)

Earth quake

Earth quake and Wind load are two most important Lateral load types. According to the EBCS, both the mentioned load types are not expected to take place at the same time. Thus, the dominating one is selected. For this study, Earth quake is considered as a dominating lateral load. The eccentricity ratio for additional torsion as explained in the EBCS 8,1995 is taken as 0.05 of floor length perpendicular to the earth quake direction. Thus, earth quake is tested at +/-0.05 times span length other than at the center of stiffness.

There are more options to feed Earth quake data to SAP. Of these the Eurocode 8/2004 is selected. The eccentricity ratio, ground acceleration ag, spectrum type, ground type, lower bound factor Beta, behavior factor q and correction factor lambda are required.

For this project work, the accidental torsional effect is represented by +/- 0.05 eccentricities. The ground type is selected as type C (deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters). Ground acceleration for ADAMA as revised in the unpublished EBCS is 0.15g. With the assumption of surface-wave Magnitude exceedence to 5.5, type 1 Spectrum is selected.

As stated in the EBCS 8, 1995, up to 80m high buildings, their fundamental period may be calculated by:

$T_1=C_1H^{3/4}$ where,

T_1- fundamental period of building in seconds

C_1=0.075 for reinforced concrete moment-resisting frames

H- height of the building above the base in meters

λ = correction factor, equal to 0.85 if $T_1 \leq 2T_c$ and the building has more than two storeys otherwise equal to 1

For type 1 spectrum soil type C, T_c=0.6

Behavior factor for partition walls… is 2 from Eurocode 8,2004

.recommended lower bound factor beta is 0.2

Table 3.2 Summary of Earth quake data

Building	T	2Tc	Correction factor	Lower bound factor	Behavior factor
G+5	0.75	1.2	0.85	.0.2	2
G+10	1.12	1.2	0.85	0.2	2
G+15	1.45	1.2	1	0.2	2
G+20	1.76	1.2	1	0.2	2
G+25	2.00	1.2	1	0.2	2

Coordinate System on SAP 2000 v 14

The global coordinate system and the displacement directions followed in this study are that of the SAP 2000 v14 software as indicated in the SAP manual. Directions opposite to the indicated are taken written in negative value.

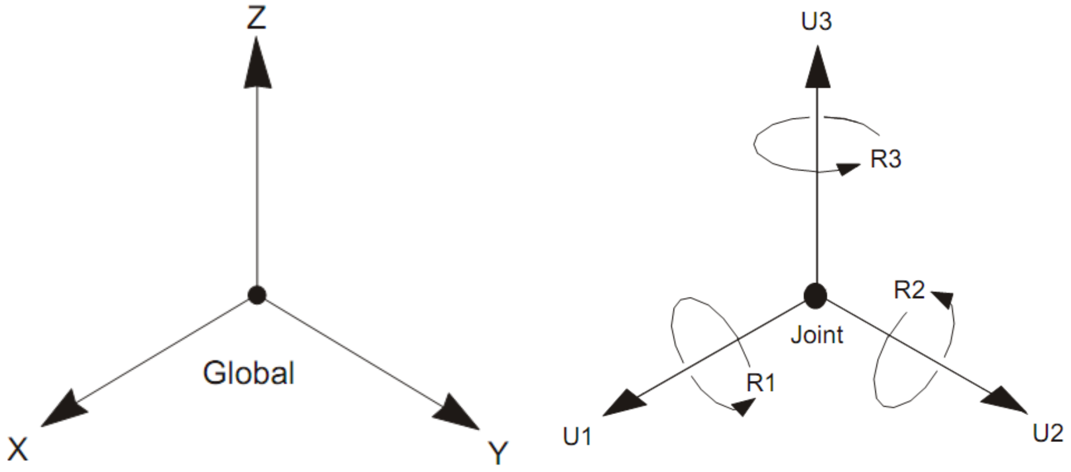


Figure3-5 Coordinate axis and displacement directions

Force along the line of action of the axis is Axial and moment about the line of action is Torsion. (1-direction)

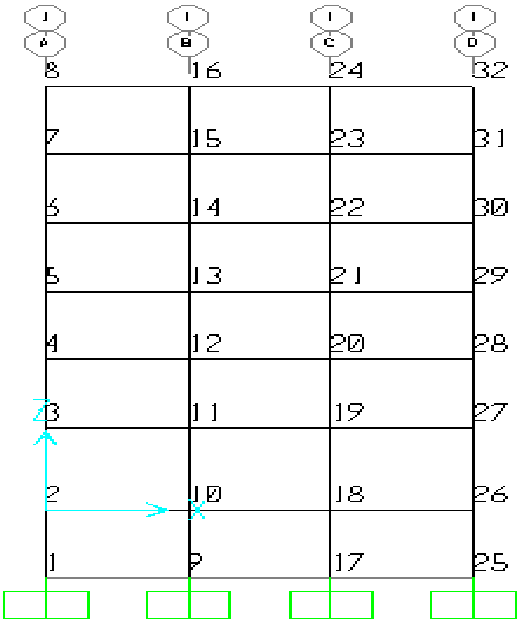
Moment about a perpendicular axis to the line of action of the element other than the Z- axis is M3. (2-direction) for columns about Y- axis

Moment about a perpendicular axis to the line of action of the element other than the Z- axis is M2. (3-direction) for columns about X- axis

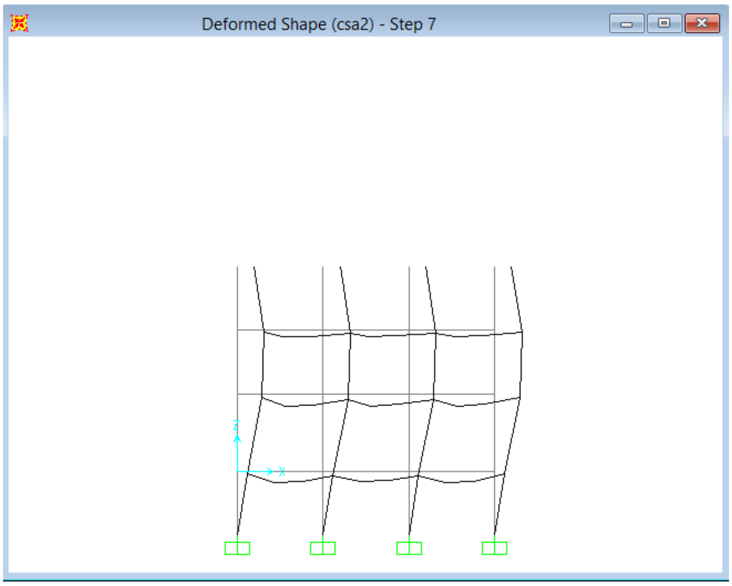
4 ANALYSIS RESULTS AND DISCUSSION

Each modeled building is analyzed and designed first. Then the analysis result for absolute maximum and absolute minimum for both complete analysis and construction stage analysis methods are compared after each cross section is made safe and economical.

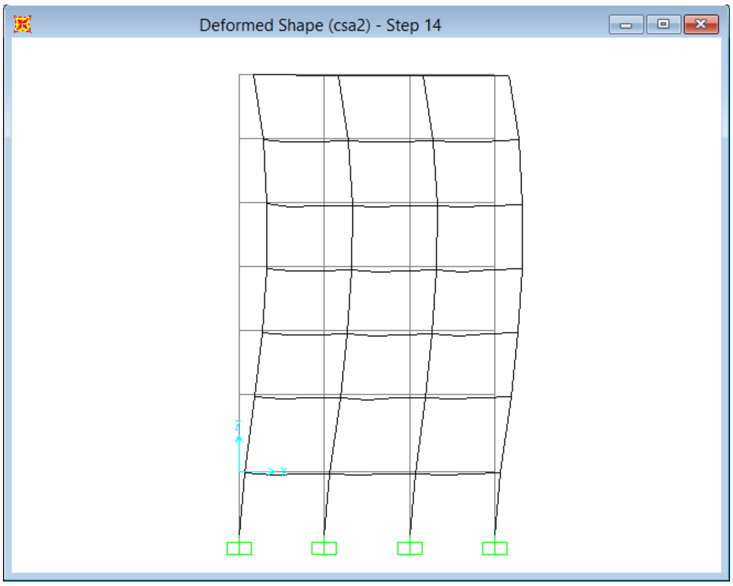
4.1 Horizontal deflection (U1) of an XZ frame



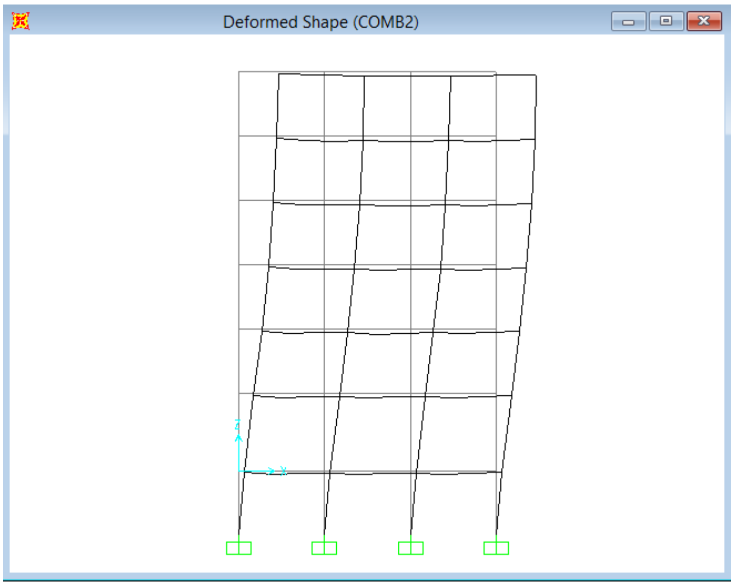
a. Frame layout with the joint no.s on the XZ ,Y=0 frame of G+5 building



b. Deflected shape of G+5 building for CSA 2 at stage



c. Deflected shape of G+5 building for CSA 2 at stage 14



d. Deflected shape of G+5 building for CA 2

Figure 4-1 Frame layout and deflected shapes at stages on the XZ ,Y=0 frame of G+5 building

Here the front most XZ frames of the buildings are selected for comparison. Within each frame, there are vertical column axes in which each axis contains beam-column joints which are named from the footing to the top most joint accordingly.

For each building an XZ frame at the front plane (Y=0) is taken.

The frame shown in figure 4-13 is located on the XZ plane at Y=0, It has four column axes along Z-axis in which, two of them axes A and B are taken for comparison. It is assumed that the effect of axes C and D can be observed from the considered axes A and B. Similarly, the rest buildings from G+10 to G+25 have 6,8,10 and 12 column axes with corresponding 13,18, 23 and 28 number of joints. Therefore, 3(from Axis A to Axis C), 4(from Axis A to Axis D), 5(from Axis A to Axis E) and 6(from Axis A to Axis F) column axes are taken for study.

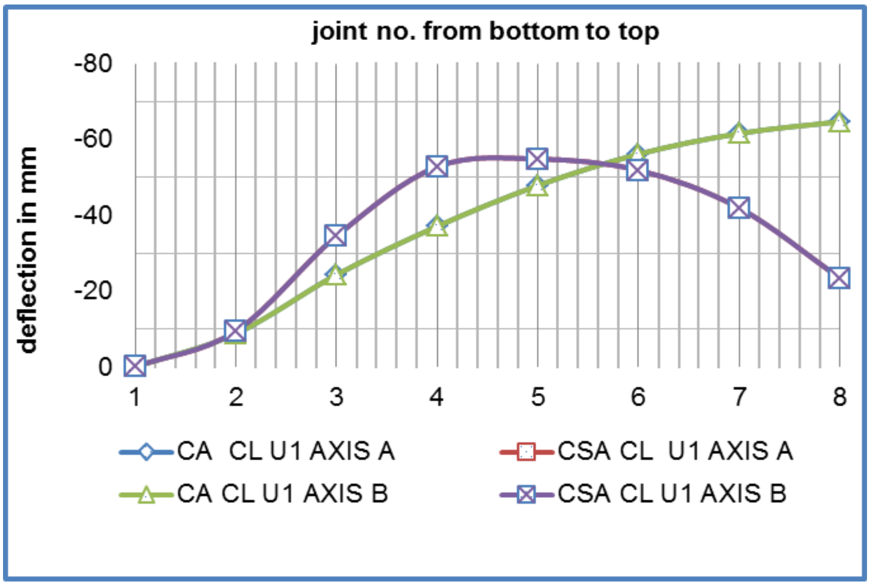


Figure 4-2 Horizontal deflection U1 of G+5 building

As indicated in the figure 4-1, every column axis along the Z axis starting from the foundation column has eight joints. These joints are named in this figure from 1-8. The negative sign in the lateral deflection U1 indicates that the deflection is to the left side.

The figure shows that lateral deflection below the sixth joint are larger as analyzed by CSA than CA. But from the sixth to the eighth joint CA value is larger. At the fourth joint, CSA value is -52.87 mm whereas, CA value is -37.12 mm. At the top joint, CSA value decreases to -23.40 mm but CA value increases to -64.79 mm.

The maximum CSA value occurs at joint 5 with value of -54.88 mm and the maximum CA value occurs at joint 8 with value of -64.79 mm.

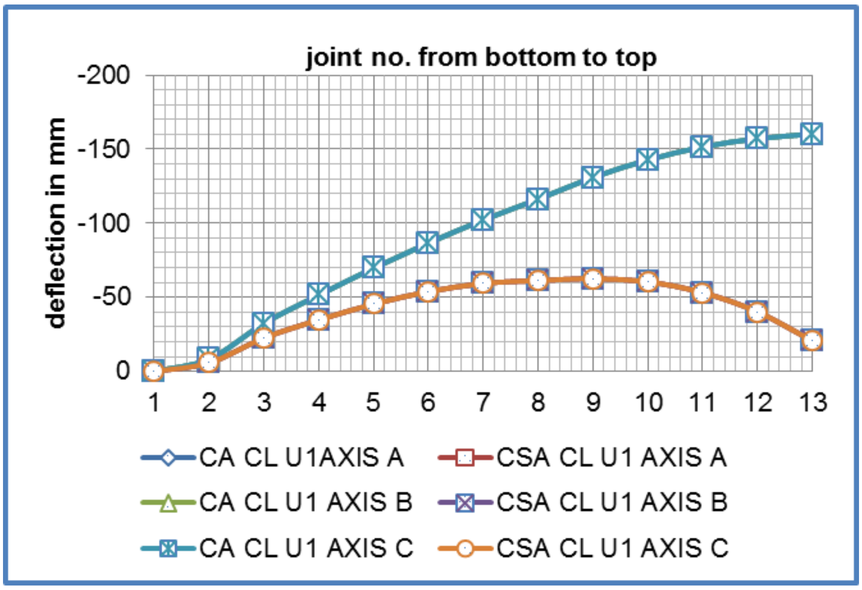


Figure 4-3 Horizontal deflection U1 of G+10 building with out shear wall

Each column axis on the XZ frame has 13 joints and a single frame consists of six column axes in which three of them are shown in the figure as Axis A, Axis B and Axis C. As it can be seen in the figure 4-2, the maximum CSA value is -62.61 mm at joint 9. But at top joint, its value decreases to -20.59 mm. The CA value at top joint is -160.25 mm.

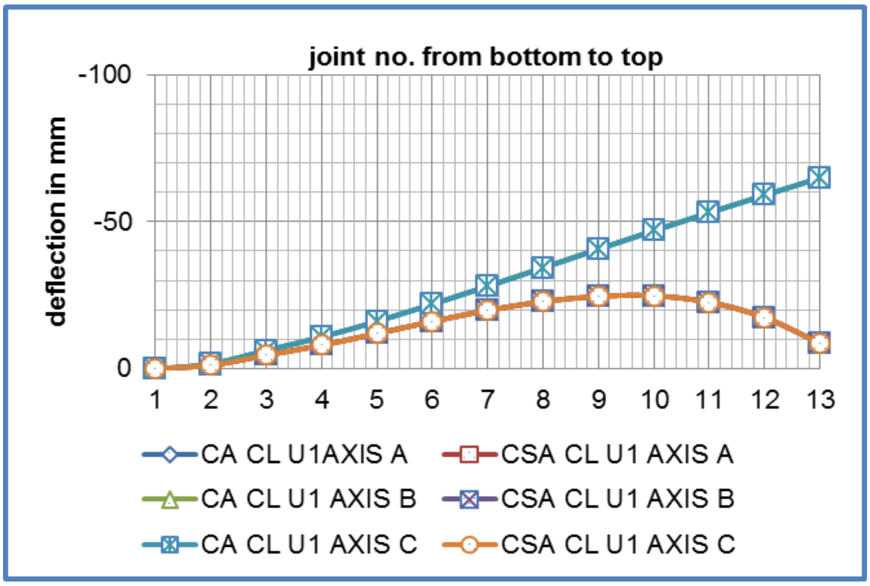


Figure 4-4 Horizontal deflection U1 of G+10 building with shear wall

Each column axis on the XZ frame has 13 joints and a single frame consists of six column axes in which three of them are shown in the figure as Axis A, Axis B and Axis C. As it can be seen in the figure 4-3, the maximum CSA value is -24.84 mm at joint 10. But at top joint, its value decreases to -8.67 mm. The CA value at top joint is -64.92 mm.

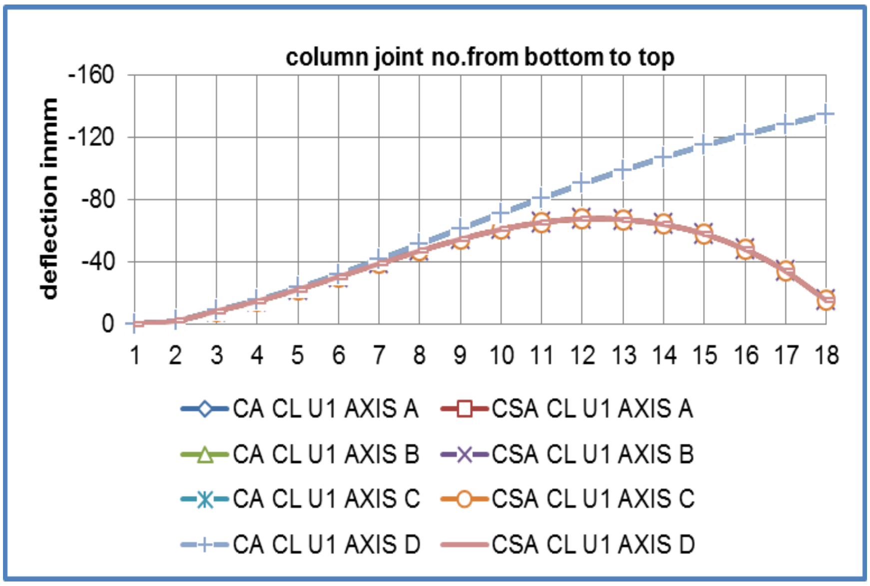


Figure 4-5 Horizontal deflection U1 of G+15 building

Each column axis on the XZ frame has 18 joints and a single frame consists of eight column axes in which four of them are shown in the figure as Axis A, Axis B, Axis C and Axis D. For the G+15 building in the figure 4-4, the maximum CSA value is -67.39 mm at joint 12. But at top joint, its value decreases to -14.84 mm. The CA value at top joint is -134.45 mm.

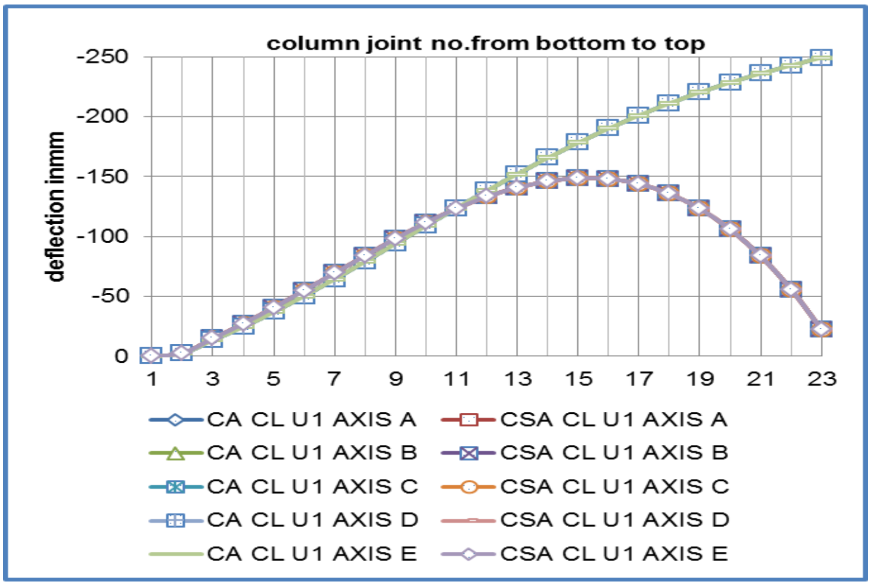


Figure 4-6 Horizontal deflection U1 of G+20 building

Each column axis on the XZ frame has 23 joints and a single frame consists of ten column axes in which five of them are shown in the figure as Axis A, Axis B, Axis C, Axis D and Axis E. For the G+20 building in the figure 4-5, the maximum CSA value is -148.71 mm at joint 15. But at top joint, its value decreases to -22.39 mm. The CA value at top joint is -249.28 mm.

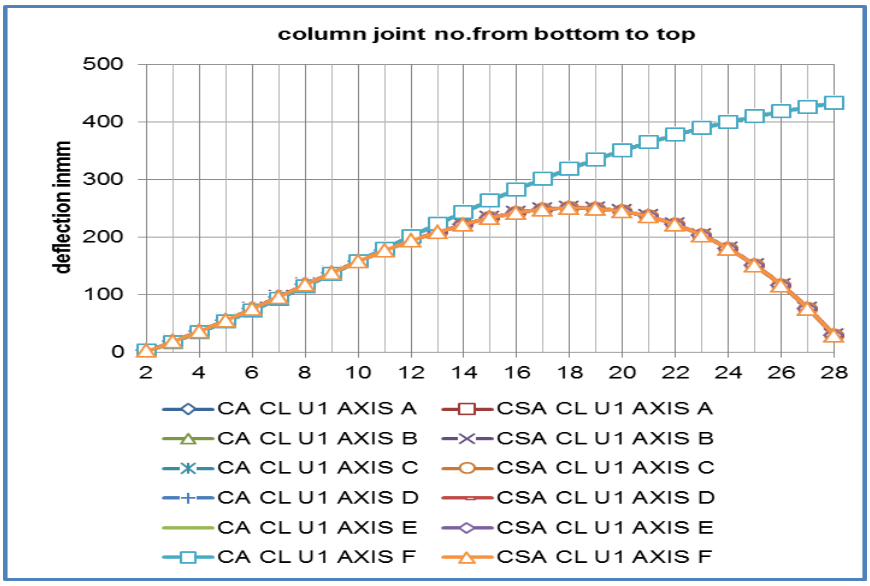


Figure 4-7 Horizontal deflection U1 of G+25 building

Each column axis on the XZ frame has 28 joints and a single frame consists of twelve column axes in which six of them are shown in the figure as Axis A, Axis B, Axis C, Axis D, Axis E and Axis F. For the G+25 building in the figure 4-6, the maximum CSA value is 250.56 mm at joint 18. But at top joint, its value decreases to 28.99 mm. The CA value at top joint is 432.77 mm.

For all the five buildings, the lateral deflection U1 as analyzed by the CA method and by the CSA method has clear difference. CA graph is an increasing graph from the footing to the top joint. But the CSA graph looks like a parabola opening downwards

For the complete analysis method, the lateral loads are applied at stories at the same time. This condition results increasing lateral deflection from bottom to top. The vertical load, which gives resistance to lateral deflection, on joints becomes less as going to top(going to top means reducing number of stories lying on the immediate story), resulting increased lateral deflection.

For the construction stage analysis method also the lateral loads are applied at stories but not applied at the same time. When lateral load is applied to a stage with rising columns on that story, the lower end of the column (where lateral load is applied) and upper end (where lateral load is not applied; as the floor at this level is not constructed yet) do not have the same deflection; as the lower end of the column is displaced to one side, the upper end is also displaced to the same side with lesser value.

For the CSA case, lower joints have lesser deflection as they are near to the fixed support. Joints near to the top floor also have lesser deflection because they have less or no stories above them where lateral loads are applied. Thus, joints nearly far from both ends of column axes have larger lateral deflection. It should be remembered that upper joint is deflected after the lower joint is deflected.

Thus, it is reasonable to have an increasing graph for CA deflection throughout the joints and a nearly parabola opening downwards deflection graph for CSA.

4.2 Horizontal deflection (U2) of an XZ frame

This is the horizontal deflection of frame joints taken for U1, located in the XZ frame along the Y- axis.

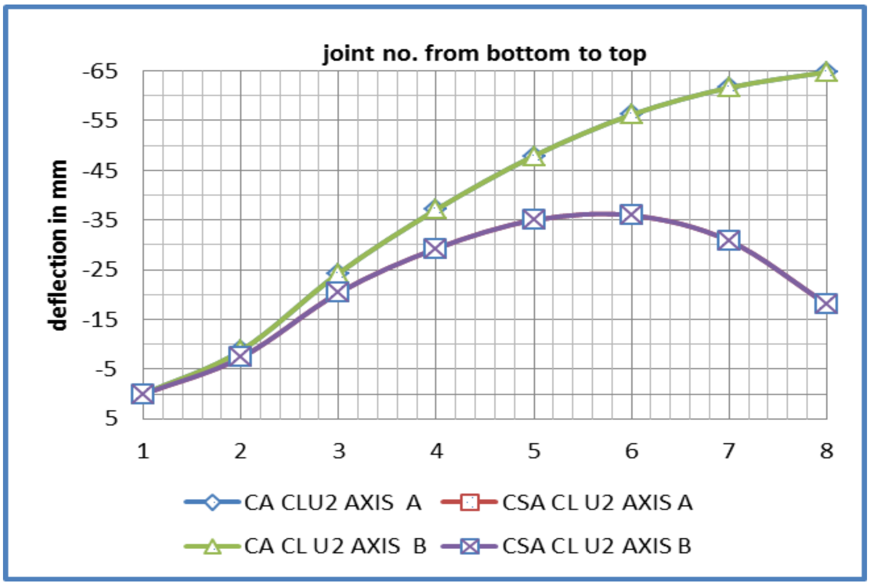


Figure 4-8 Horizontal deflection U2 of G+5 building

At the top joint as shown, the lateral deflection in the Y-direction of a joint in the XZ plane is -18.10 mm when analyzed by CSA and -64.79 mm when analyzed by CA.

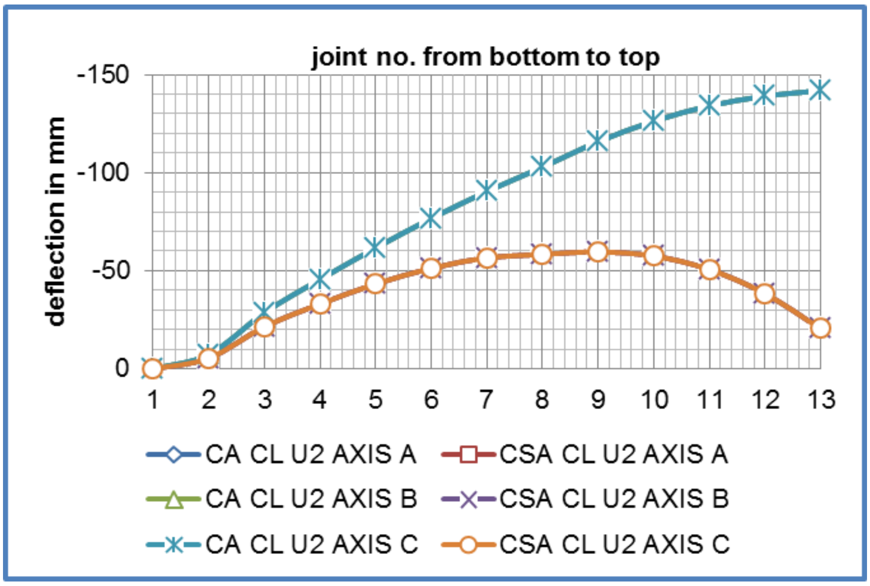


Figure 4-9 Horizontal deflection U2 of G+10 building with out shear wall

Throughout the joints, CSA value is dominated by CA value. We can read the lateral deflection at top joint as -20.47 mm for CSA and -141.87 mm for CA.

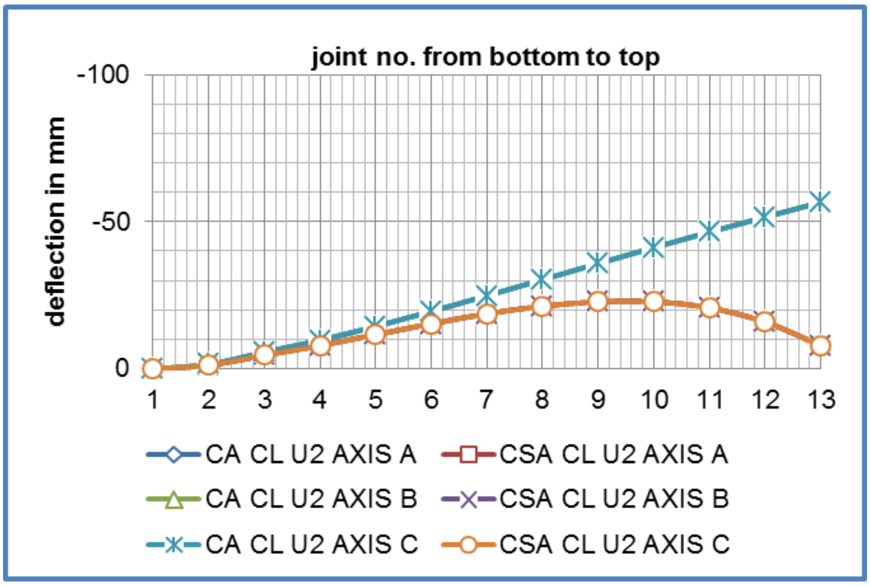


Figure 4-10 Horizontal deflection U2 of G+10 building with shear wall

As there is shear wall in this model, although the shape doesn't change, the value is reduced for CSA to -7.95 mm and for CA -56.61 mm.

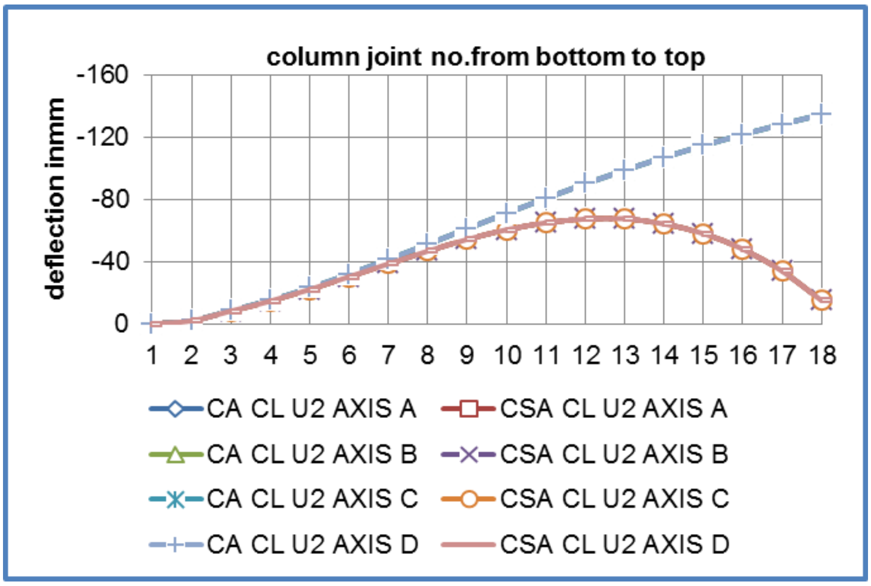


Figure 4-11 Horizontal deflection U2 of G+15 building

In this figure, top joint deflects by -14.86 mm as a result of CSA analysis and deflects by -134.45 mm as a result of CA analysis.

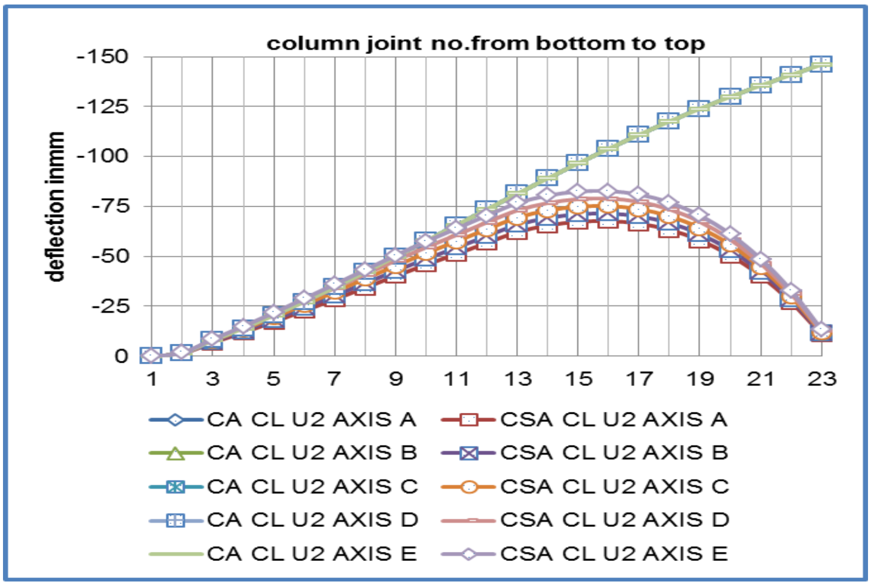


Figure 4-12 Horizontal deflection U2 of G+20 building

For all the five column axes in the XZ plane, CA is increasing but CSA decreasing as approaching to the top joint.

Thus, at the top joint, CSA deflection is -13.26 mm and CA deflection is -146.09 mm.

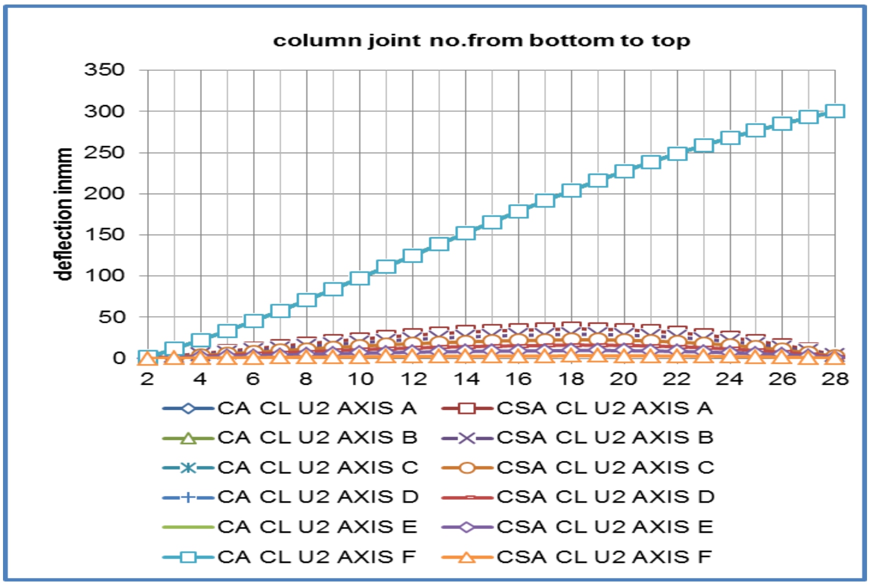


Figure 4-13 Horizontal deflection U2 of G+25 building

For this model, the gap between the two analysis methods is wide. At the top joint CSA results 0.366 mm deflection but CA results 300.30 mm deflection.

The reason why such CA deflection graph and CSA deflection graph resulted is as explained for the U1 deflection above.

4.3 Vertical deflection (U3) of an XZ frame

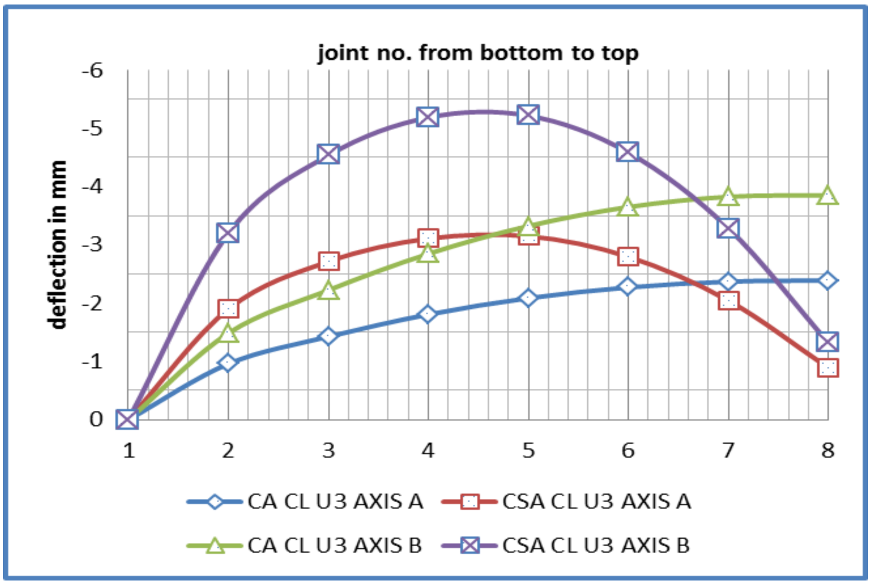


Figure 4-14 Vertical deflection U3 of G+5 building

In this figure, the down ward deflection for two vertical axes is shown. For the CSA and CA analysis methods, the vertical deflection U3 at the top floor along axes A and B are different (see joint no. 8).

At the intermediate joints for both axes A and B, CSA dominates the deflection value but at the top joint CA dominates.

At the fifth joint along axis B, CSA value is -5.22mm but CA value is -3.22 mm. But at the top joint, CSA value decreases to -1.32 mm while CA value increases to -3.85 mm.

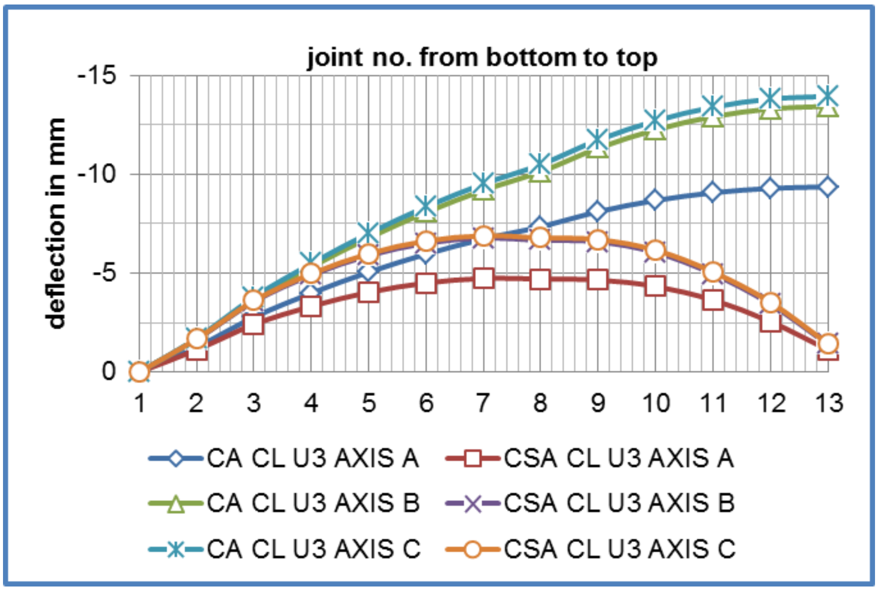


Figure 4-15 Vertical deflection U3 of G+10 building with out shear wall

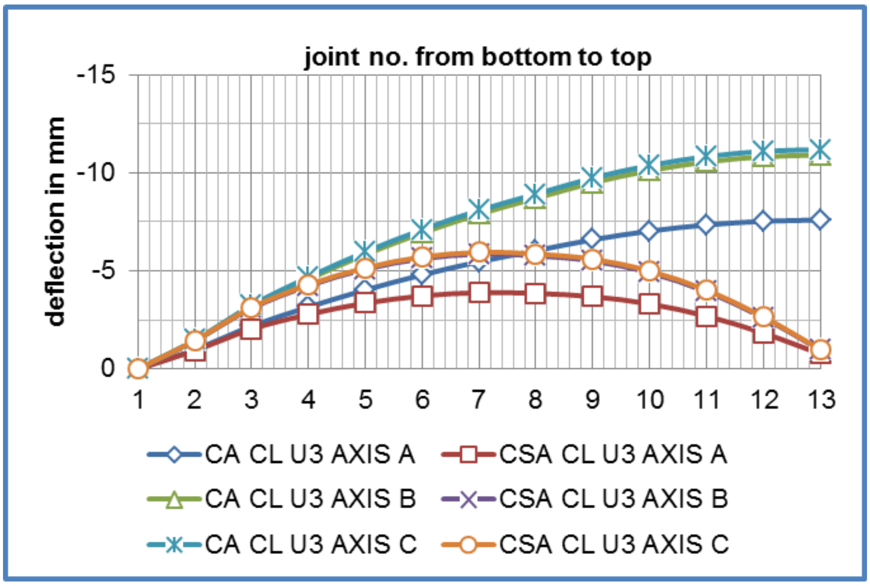


Figure 4-16 Vertical deflection U3 of G+10 building with shear wall

Here almost all the joints attain maximum vertical deflection as the building is analyzed by the CA method. CSA results a -0.99 mm deflection while CA results a -11.18 mm deflection.

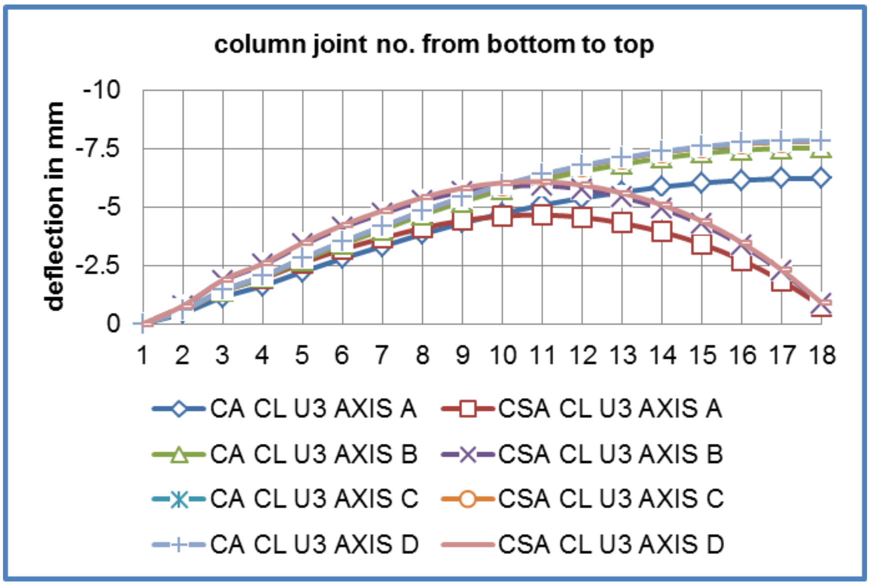


Figure 4-17 Vertical deflection U3 of G+15 building

For this figure, it is obvious that the CA values are gradually increasing and the CSA values open downwards as parabola. At top of axis D, the vertical deflecion by CA is -7.85 mm but by that of CSA is -0.90 mm.

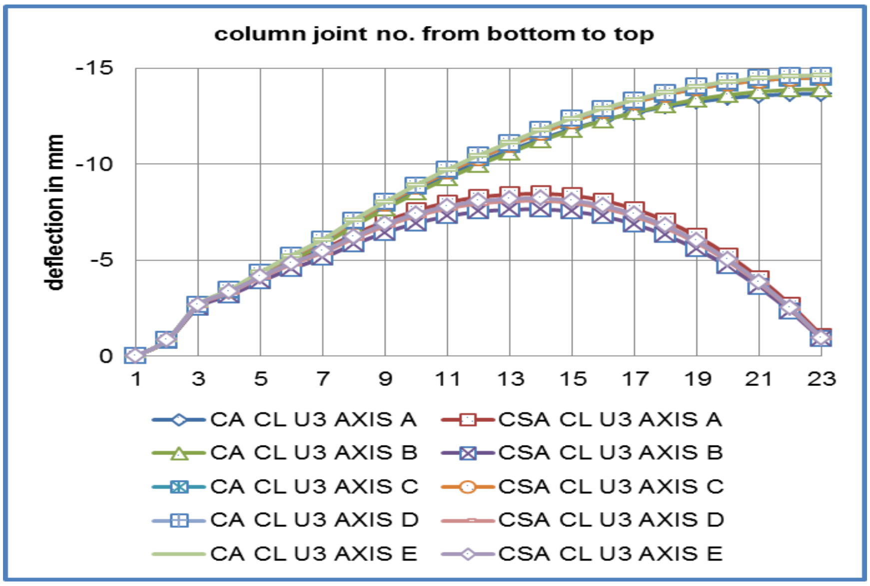


Figure 4-18- Vertical deflection U3 of G+20 building

Maximum CA at the top of the story is -14.64 mm and max CSA at the top story of the building is -0.96 mm.

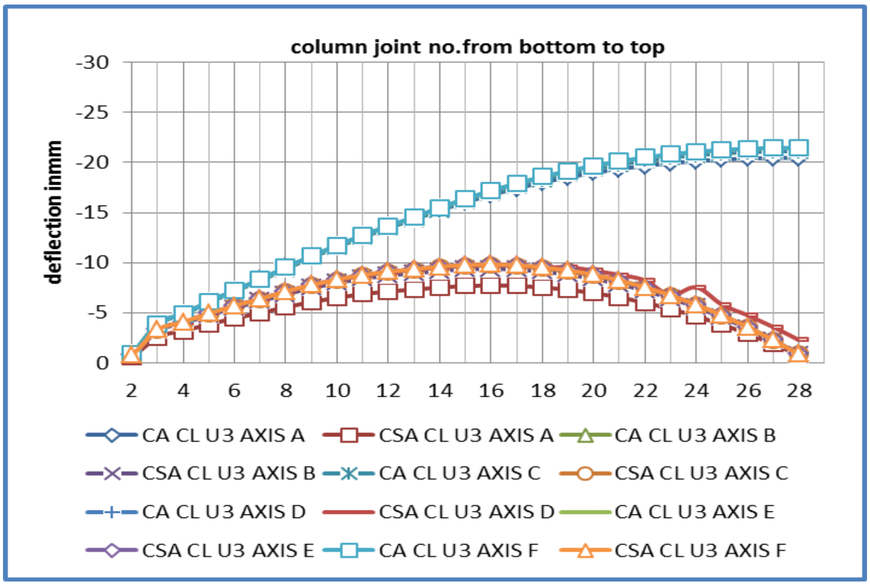


Figure 4-19 Vertical deflection U3 of G+25 building

In this figure also the CA analysis method continues to dominate the vertical deflection value. As a result, there is a 20.6 mm difference between the analysis methods.

Here the reason for the result and shape of the graphs of analysis methods remains the same to the previous U1 and U2 deflections except the fact that the deflection in this case is mainly caused by the vertical loading.

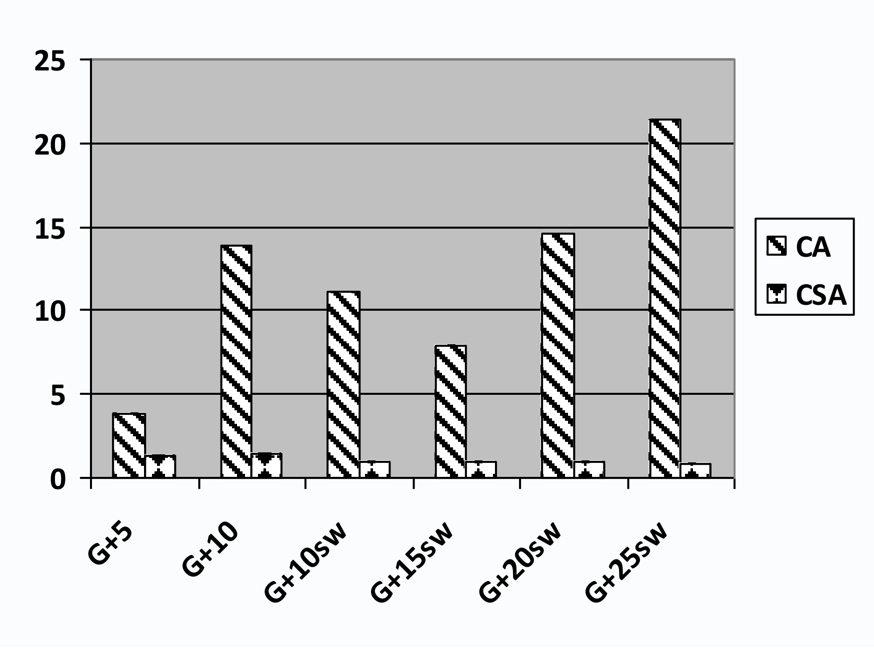


Figure 4-20 Vertically downwards deflection U3 of the six buildings at their top joints

Units are in mm. ‘sw’ stands for ‘with shear wall’. As can be seen in the figure, using CA analysis method exaggerates the deflection and as the number of story is increasing, the CA value is increasing while the CSA value is decreasing.

Remember that taking shear wall reduces the value and the way how the shear wall is arranged also affects the result.

4.4 Rotation (R2) about Y axis

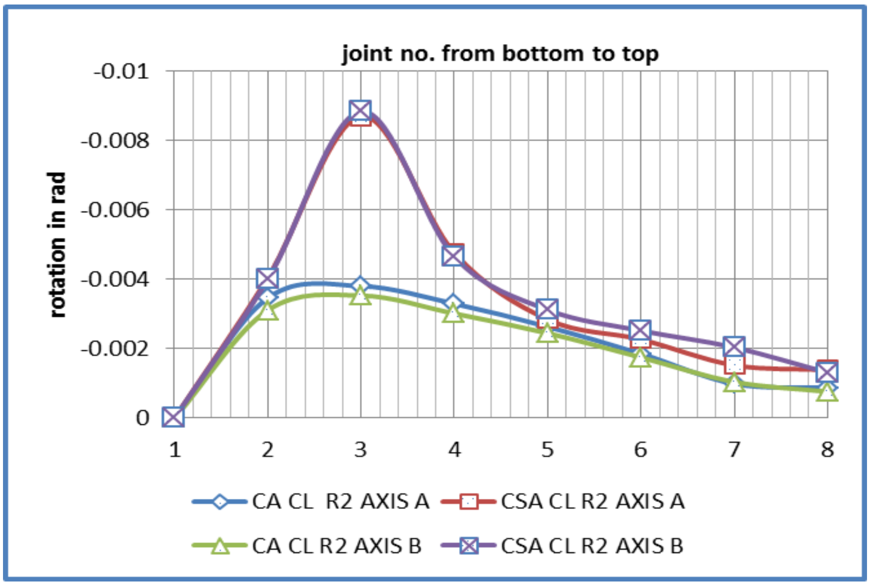


Figure 4-21 Rotation of G+5 building

This is the rotation of joints on XZ plane about the Y axis. For both the analysis methods along both axes, the graphs have a shape similar to a parabola opening downwards. Throughout the joints CSA value dominates.

At the mid height of the building we can read a 0.005309 difference between CSA and CA.

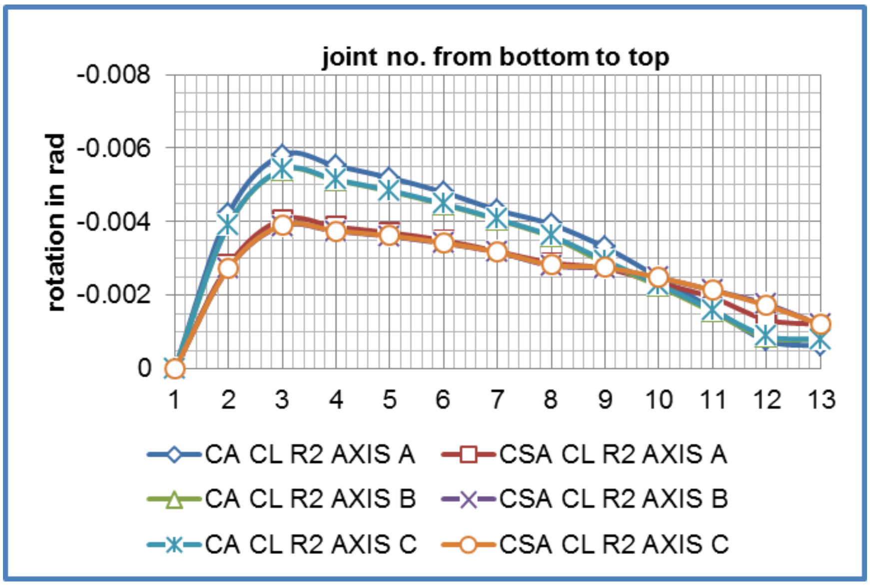


Figure 4-22 Rotation R2 of G+10 building with out shear wall

As shown on the figure, almost for all points, rotation about the Y axis for each joint by CA is larger than by CSA. Especially, from the 11th joint upwards, CSA values are larger than CA values resulting a 0.000403 rad difference.

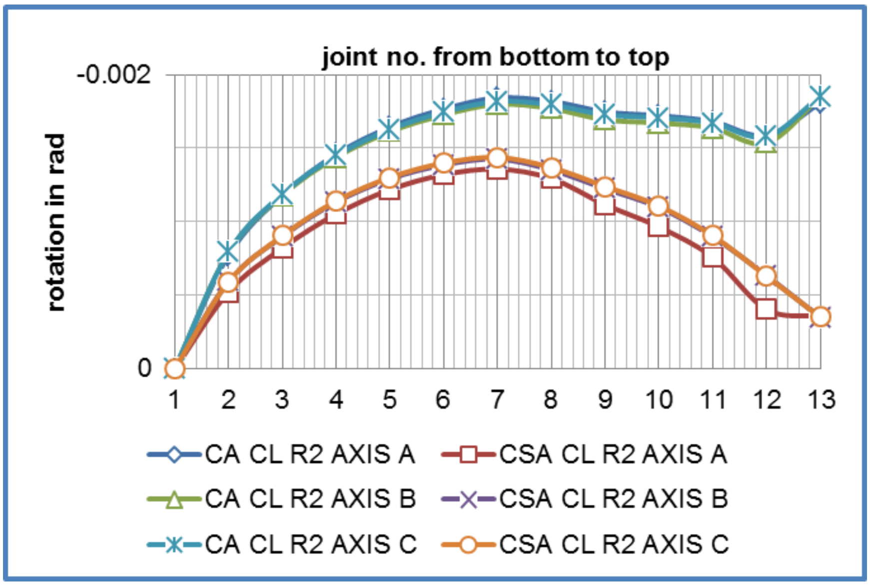


Figure 4-23 Rotation R2 of G+10 building with shear wall

As this building is stiffer because of shear wall, the rotational values are less as compared to the upper G+10 building without shear wall. Throughout the building, the joints are rotated about the Y axis to their maximum values as analyzed by CA than by CSA.

We can read a -0.001854 rad CA value and a -0.000352 rad CSA value.

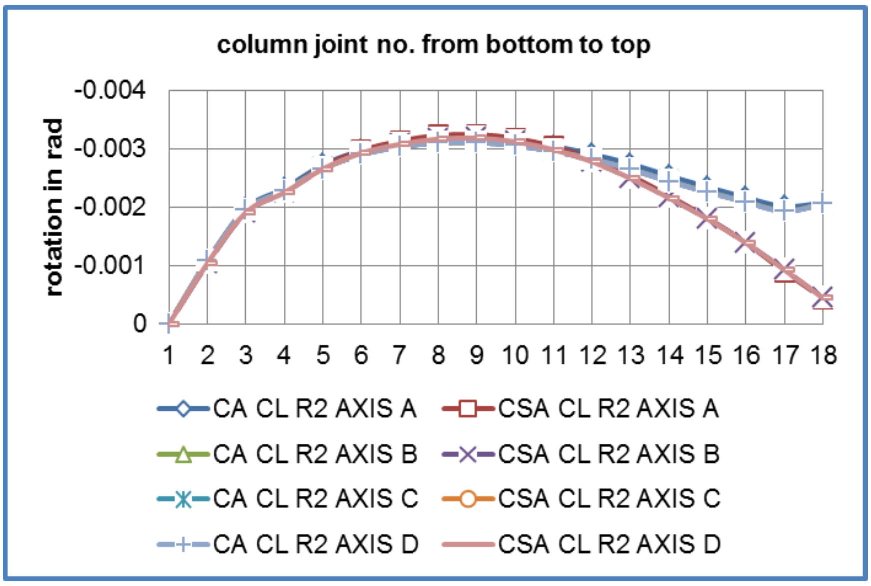


Figure 4-24 Rotation R2 of G+15 building

CA value becomes larger than CSA starting from the 12th joint up. Thus, at the 18th joint CA is -0.002055 rad and CSA is -0.000439 rad.

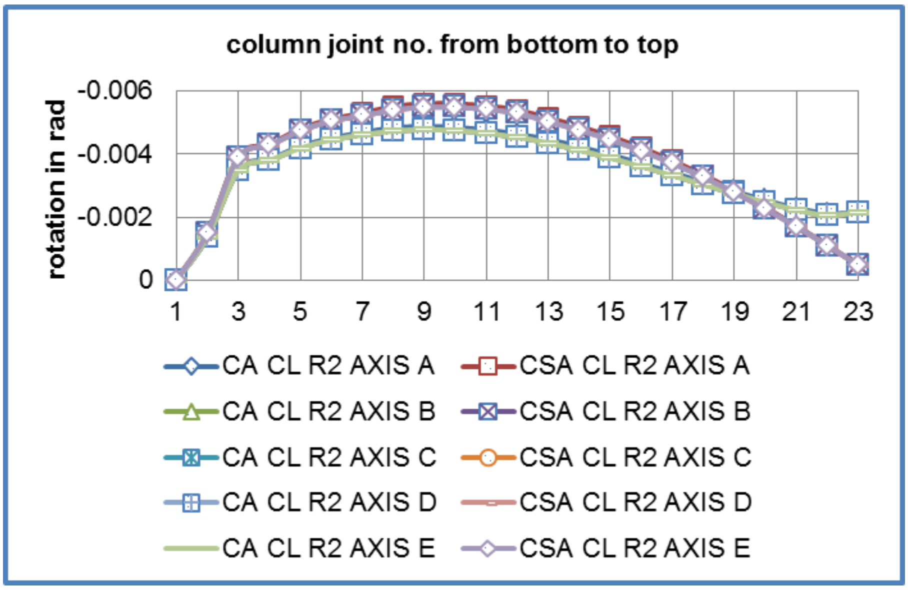


Figure 4-25 Rotation of G+ 20 building

Values of both analysis methods are almost the same. At the top floor the CA value is larger by 0.001674 rad.

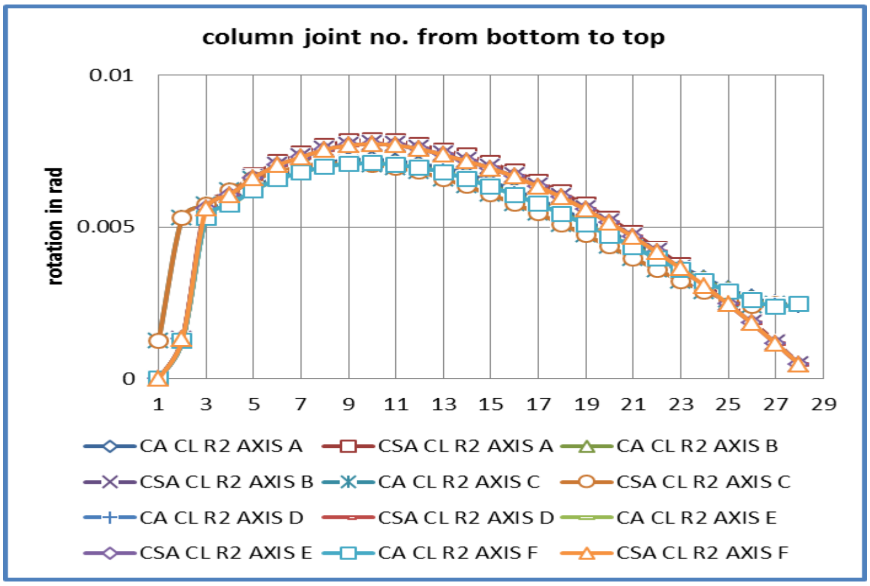


Figure 4-26 Rotation of G+ 25 building

Starting from the 23rd joint up to the last 28th joint, CA value is larger to CSA value by 0.001975 rad.

Now the shape for the rotation of joints about the Y-axis is almost the same for both CA and CSA methods. CA rotational value is dominating CSA for all the six models.

Here what we can observe is that there is exceess rotation resulted from unpractical analysis method of the model (CA method). As numbers of story are increasing, the difference between the analysis methods is also increasing.

4.5 Bending Moment M3 on Beams

For the bar graphs drawn, for both moment and shear, the first graph at each beam represents the CA value and the second graph represents the CSA value.

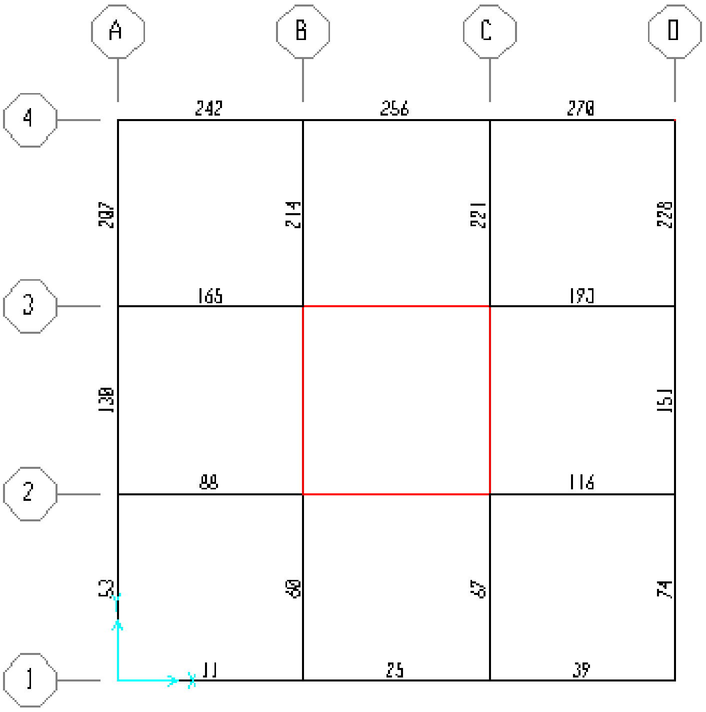


Figure 4-27 Frame layout and beam no. on the XY plane at Z=9.60m of G+5 building

As shown on the layout, the three beams on axis 1 named in the SAP model as 11, 25 and 39 are respectively named on the figures below as 1, 2 and 3. Such naming is also followed for all the buildings. The naming order is from left to right. All the beams for all the six models are located at the mid height of their total height.

A continuous beam has two negative (support) moments at the start (left) and end (right) segments; and a single positive (span) moment at the interior segment.

For this study, only the start/negative/support moment and the middle/positive/span moment are taken.

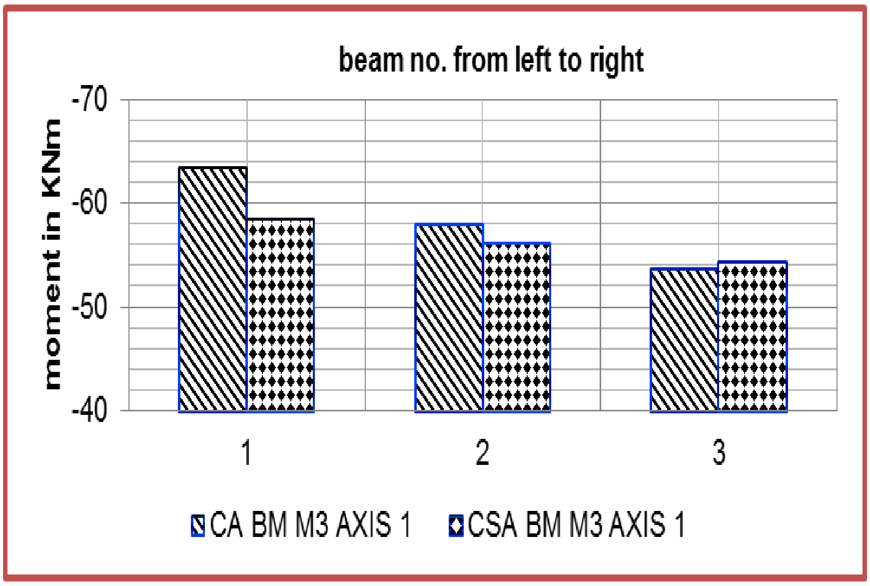


Figure 4-28 Left Negative moments of three beams of G+5 of building

- As you go from the first beam to the third beam, generally, the moments on the beams have decreasing trend.
- The CA moment is slightly larger than CSA with maximum difference of 5.02 KNm.

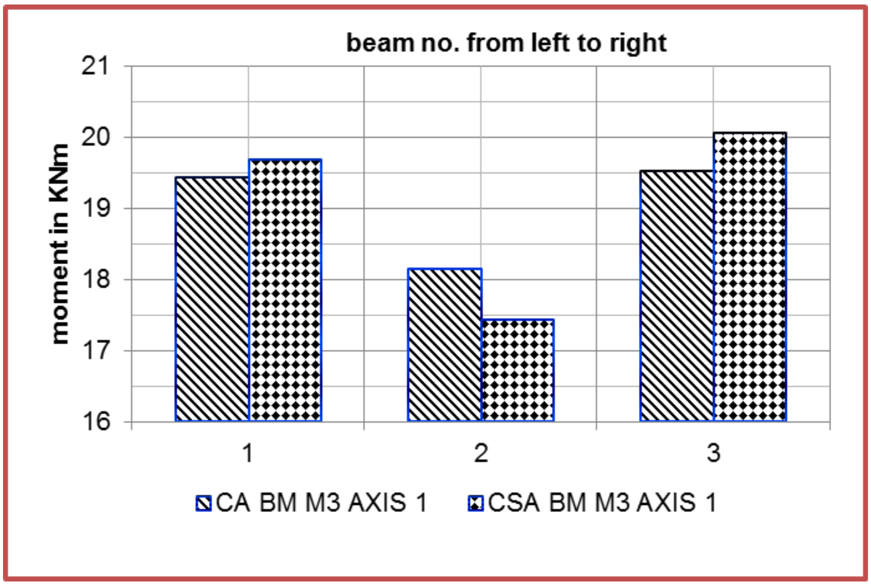


Figure 4-29 Positive moments of three beams of G+5 building

Positive moments resulted from CSA are larger at the exterior beams than at the interior beam as compared to CA. But the exceedence of CA at the interior bam is larger than the exceedence of CSA at the exterior beams. Thus, at the interior beam CA value exceeeds CSA value by 0.5 KNm.

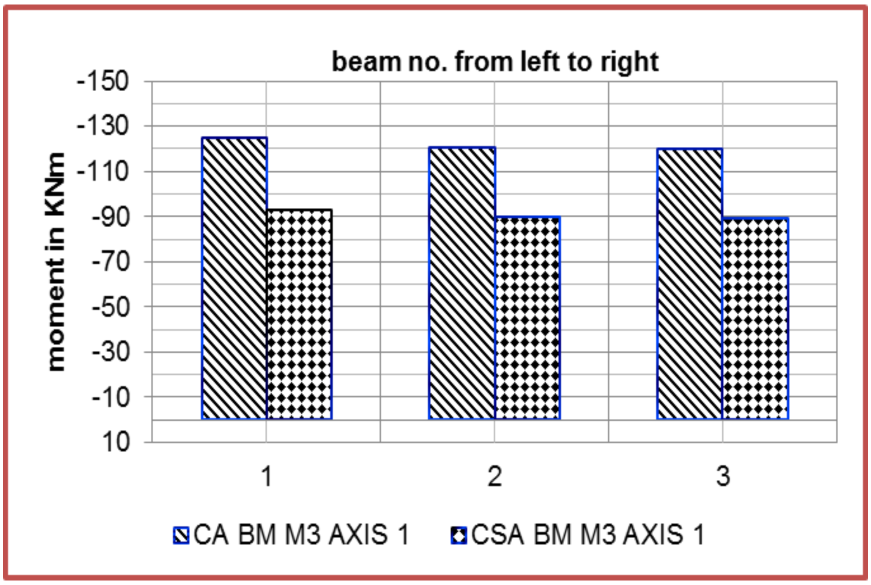


Figure 4-30 Left Negative Moment for beams of G+10 without shear wall

The moment resulted by the CA is larger than the CSA. At maximum, the CA moment, at beam 2, is larger than the CSA by 31.25 KNm.

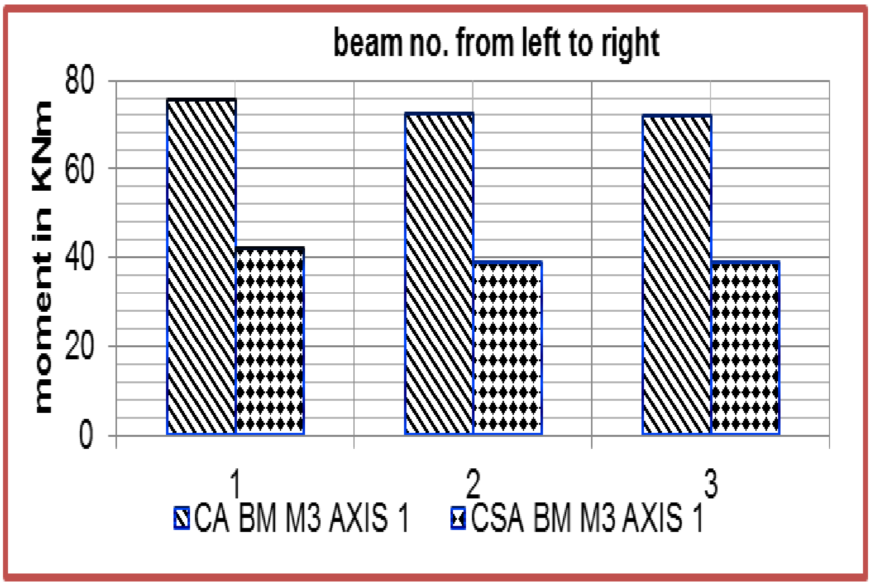


Figure 4-31 Positive Moment for beams of G+10 without shear wall

In this figure, positive moments are larger when analyzed by the CA than CSA with a maximum value of 33.41 KNm.

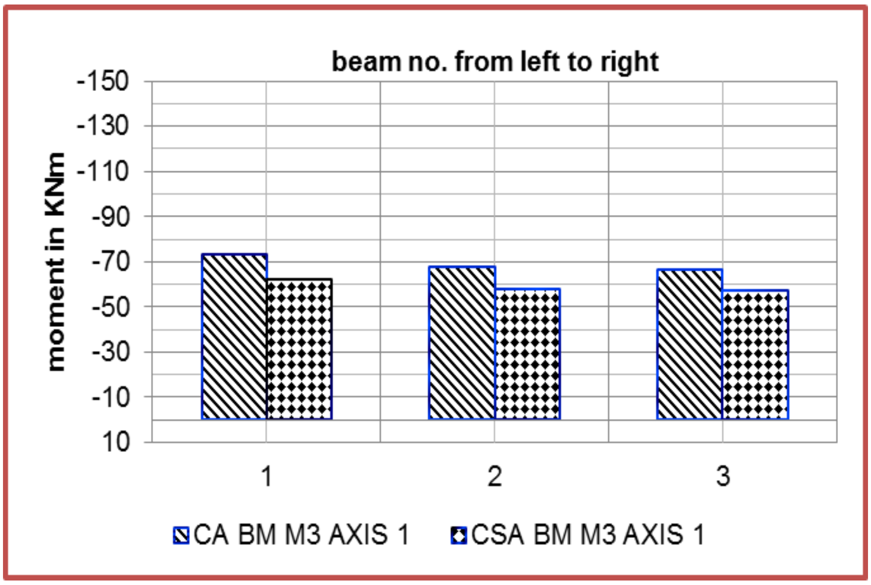


Figure 4-32 Negative Moment for beams of G+10 with shear wall

At the interior beam, CA exceeds CSA by 9.9 KNm moment value.

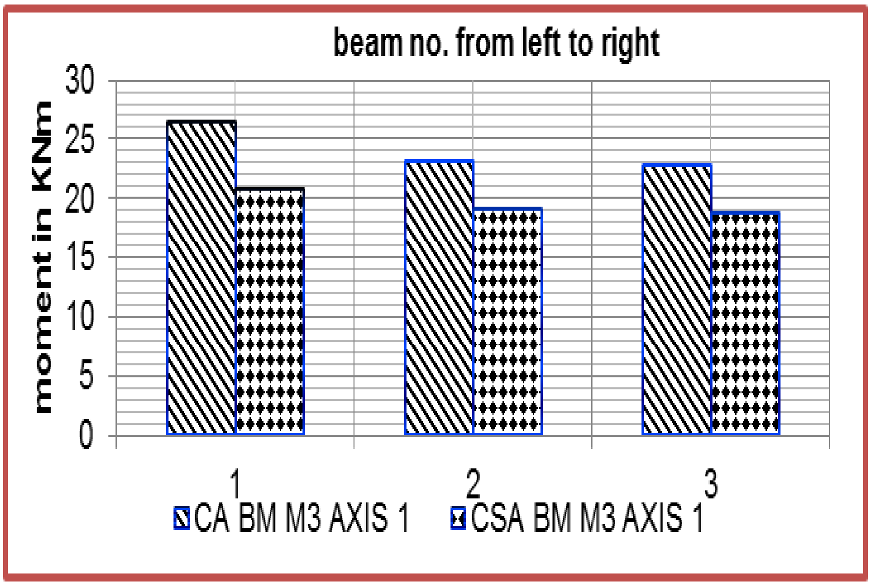


Figure 4-33 Positive Moment for beams of G+10 with shear wall

Maximum moment difference between the CA and CSA results occurring at the interior beam is 4.04 kNm.

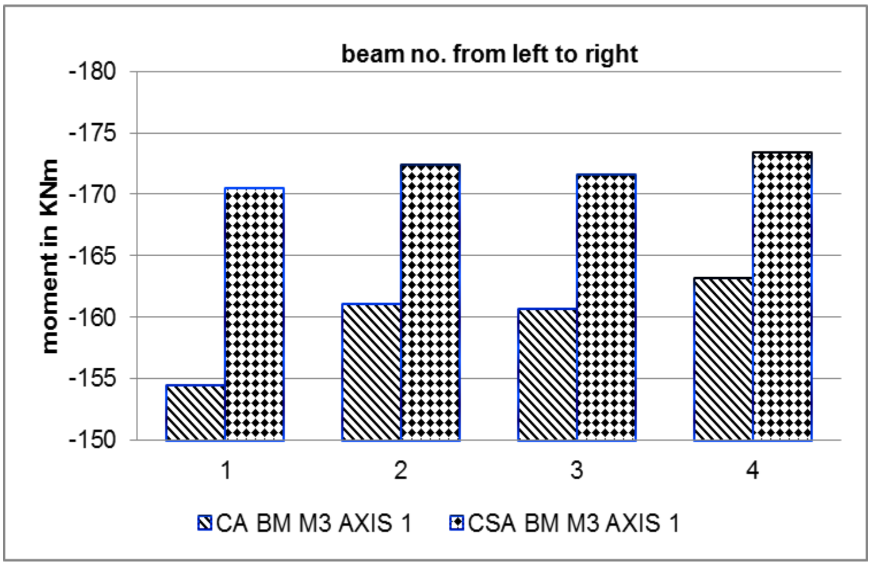


Figure 4-34 Left Negative moments of four beams of G+15

CA negative moments are smaller than CSA negative moments.There is atleast 16KNm difference b/n the moments by CSA and by CA.

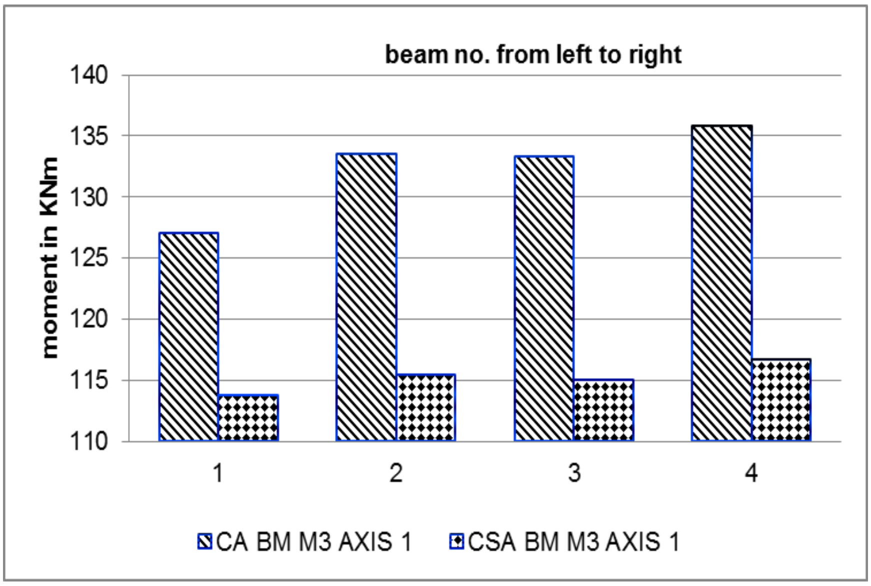


Figure 4-35 Positive Moments for beams of G+15 building

This figure shows that CA result is larger than CSA by 19 KNm.

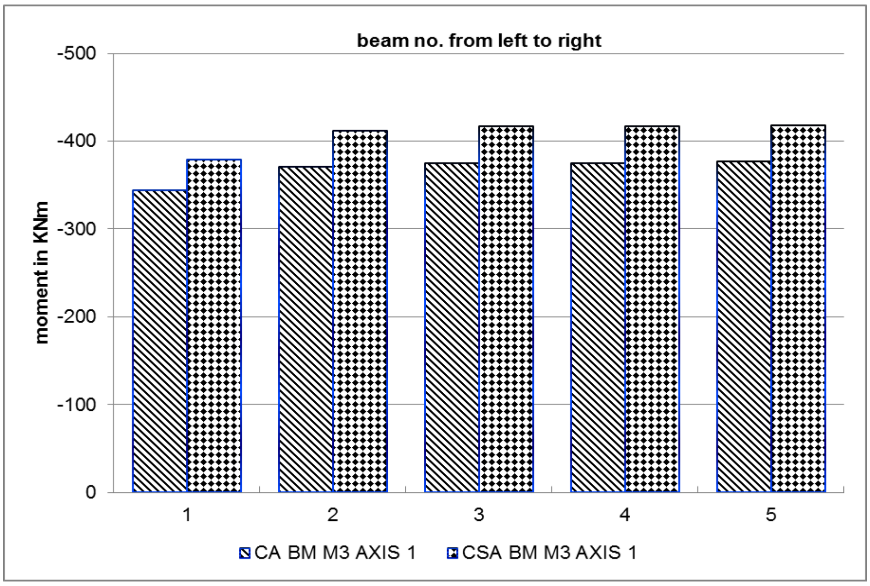


Figure 4-36 Left Negative moments of five beams of G+20

Maximum moment of 41.87KNm difference is observed at one of the beams by CSA analysis on CA analysis.

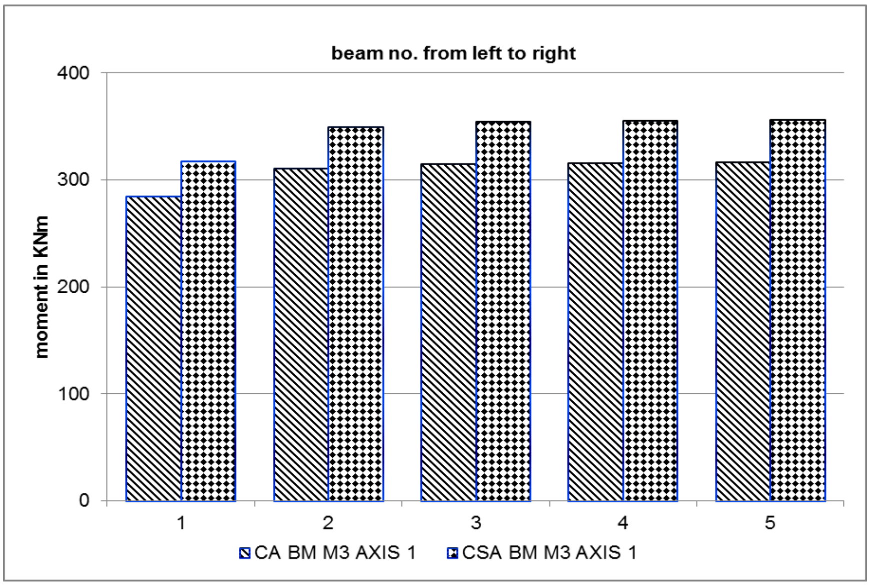


Figure 4-37 Positive Moments for beams of G+20 building

The positive moments here are larger for CSA case than for CA case with a minimum value of 39KNm.

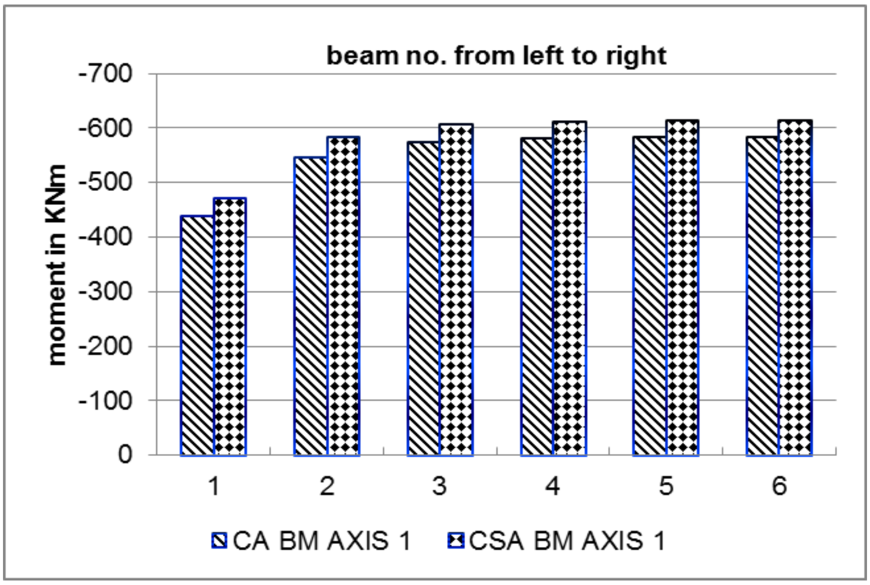


Figure 4.37 Right Negative moments of four beams of G+25

The moment difference between the two analysis methods is increasing as the building story number increases. In this figure, the CSA moments are larger than the CA moments with a maximum difference of 30.55 KNm.

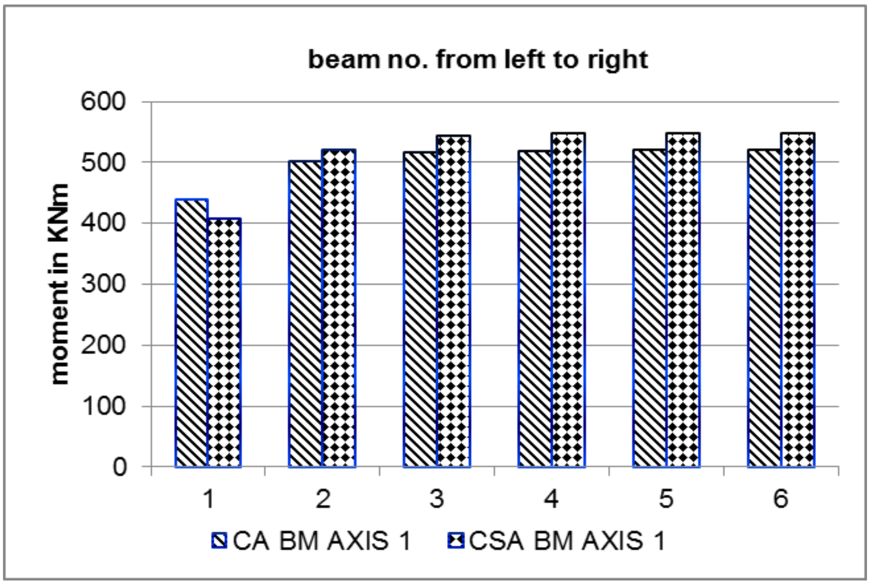


Figure 4.38 Positive Moments for beams of G+25 building

This figure may summerize the case that as number of story increases the moments on beams analyzed by CSA method become the governing moments than those analyzed by the CA method. For almost all beams in this figure, moments calculated by CSA are greater than calculated by CA by an amount of 28.15 KNm.

Therefore, moments resulted from CA analysis are larger till G+15 building and moments resulted from CSA analysis are larger for G+20 and G+25.

4.6 Beam Shear force

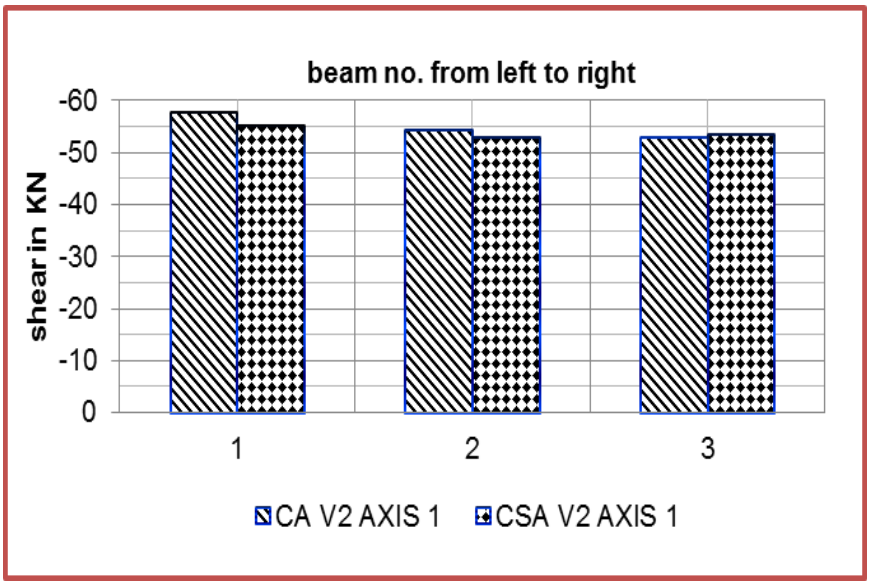


Figure 4-389 Shear values for three beams of G+5 building

As it is shown on the figure above, almost there is no value difference between the two analysis methods.

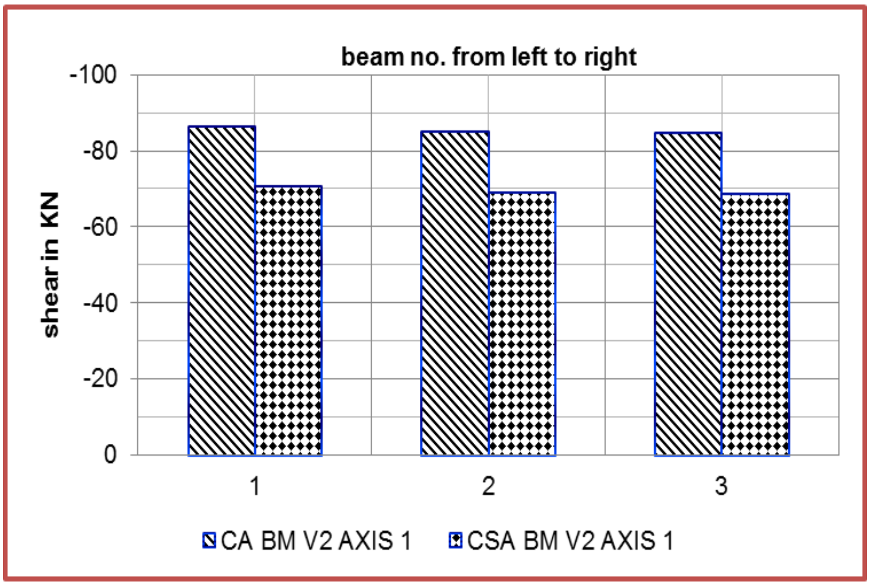


Figure 4-39 Shear values for three beams of G+10 building without shear wall

Shear values are larger for CA than CSA. We can observe a minimum of 15 KN shear difference between CA and CSA.

This figure explains the condition that for buildings of less story number, shear is dominated by the CA method than CSA method.

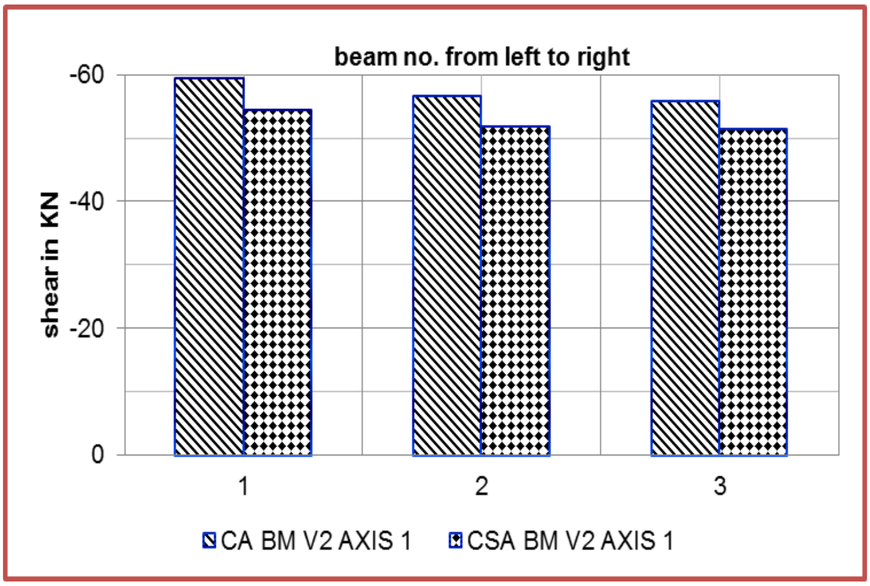


Figure 4-41 Shear values for three beams of G+10 building with shear wall

Here the shear values are less as the building is made stiffer by shear wall. CA is larger than CSA not more than 5KN.

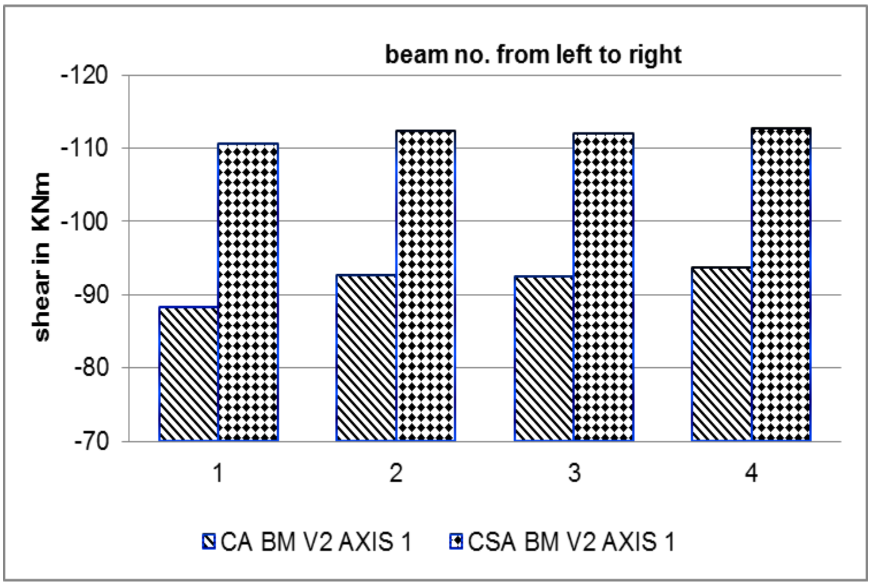


Figure 4-42 Shear value for four beams of beams G+15 building

For three of the beams shear values for CSA are larger than for CA. There is a maximum of 19 KN shear difference between the two analysis methods.

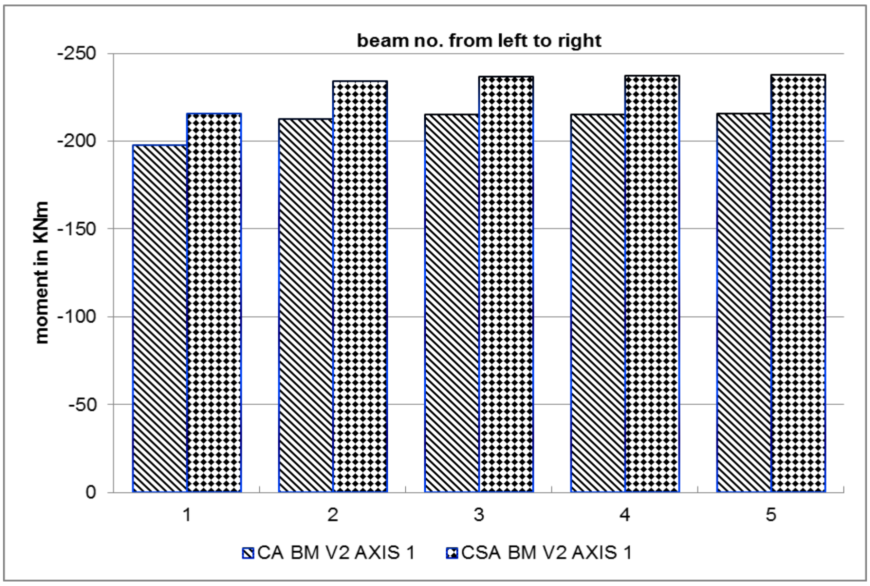


Figure 4-40 Shear value for five beams of beams G+20 building

There is at least 22 KN shear force difference between the CSA and CA analysis methods. Here we can observe that it is time for the CSA method to exceed CA analysis method as the story number becomes more.

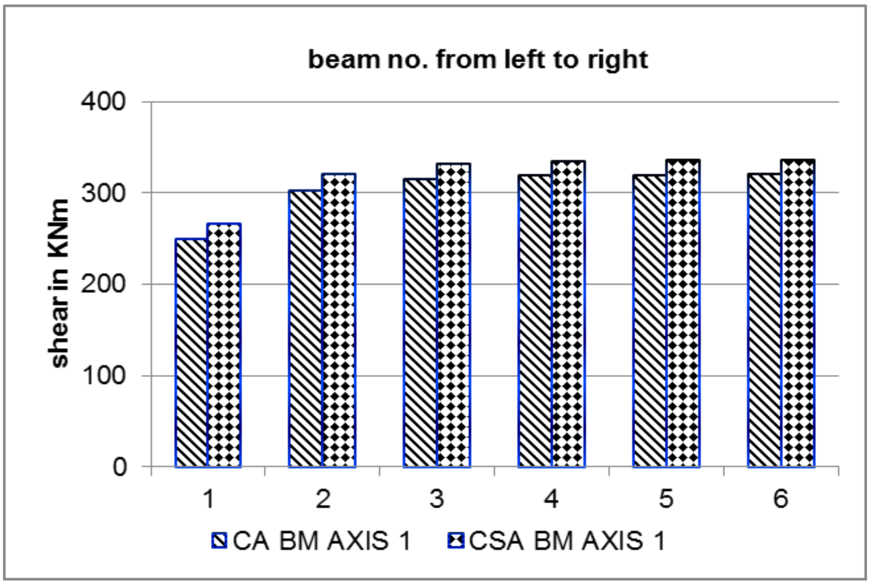


Figure 4-414 Shear value for five beams of G+25 building

For all the beams now CSA method of analysis results are larger than CA. CSA value is exceeding CA not by more than 16 KN.

The trend for shear force on beams is also the same to moments. As the story becomes more, the CSA shear result becomes larger than CA result. This questions the safety of high rise buildings.

4.7 Axial force of Columns

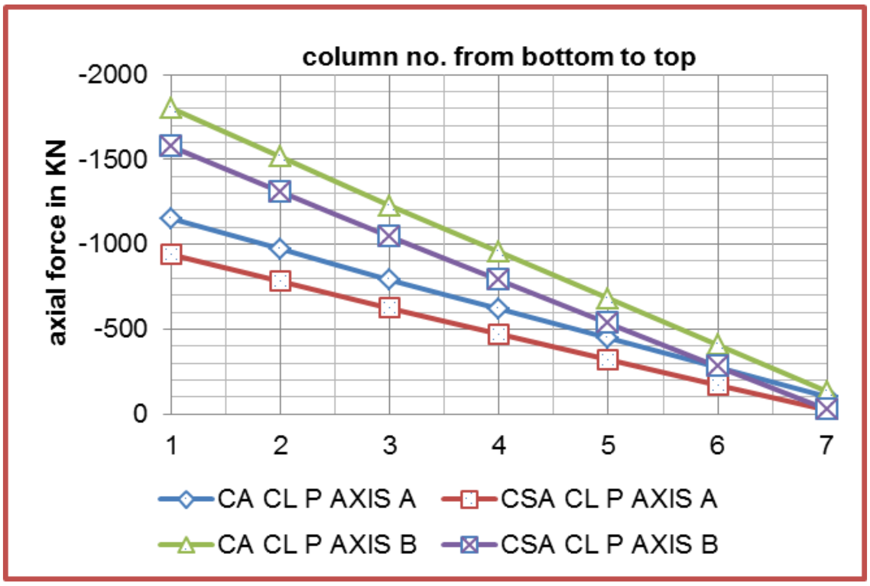


Figure 4-45 Axial forces on columns of first frame G+5

Each column lying along same vertical axis is named from 1 to 7 from bottom to top.

The axial force of CA is larger than CSA being their values increasing as going to the interior column axis of the frame. At the foundation column there is a 226.5 KN exceedence of CA value to CSA at the interior vertical column.

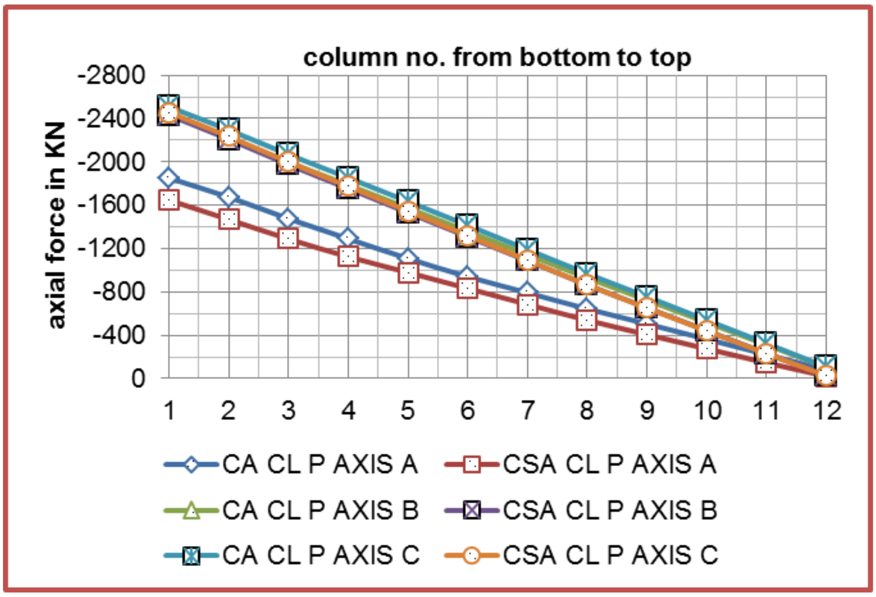


Figure 4-426 Axial force on columns on XZ, Y=0 plane G+10 without shear wall

The axial force by CA is larger than the CSA value.This difference becomes smaller as one goes from the exterior axis to the interior. At the exterior column axis, foundation column, CA value is 203.75 KN larger than CSA value.

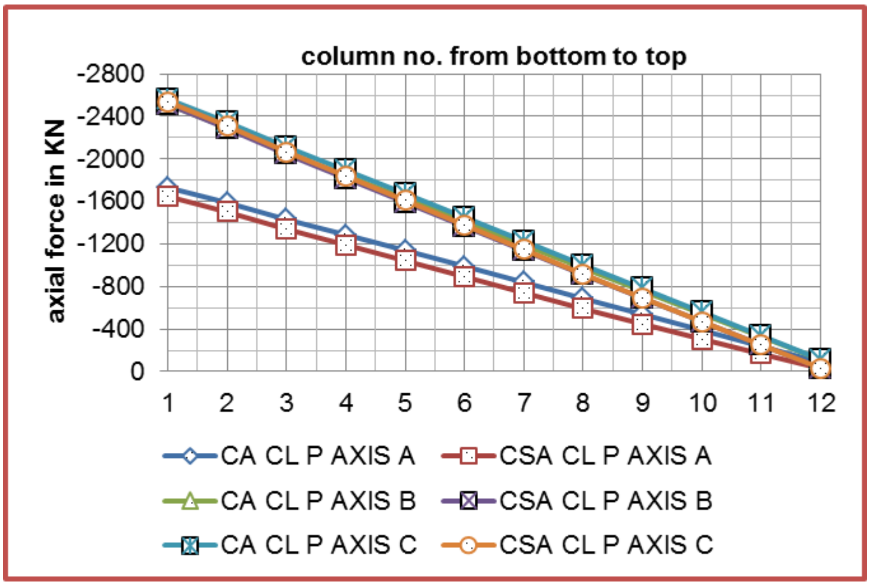


Figure 4-437 Axial force on columns on XZ, Y=0 plane G+10 with shear wall

For this building the maximum axial force difference between CA and CSA is 83 KN.

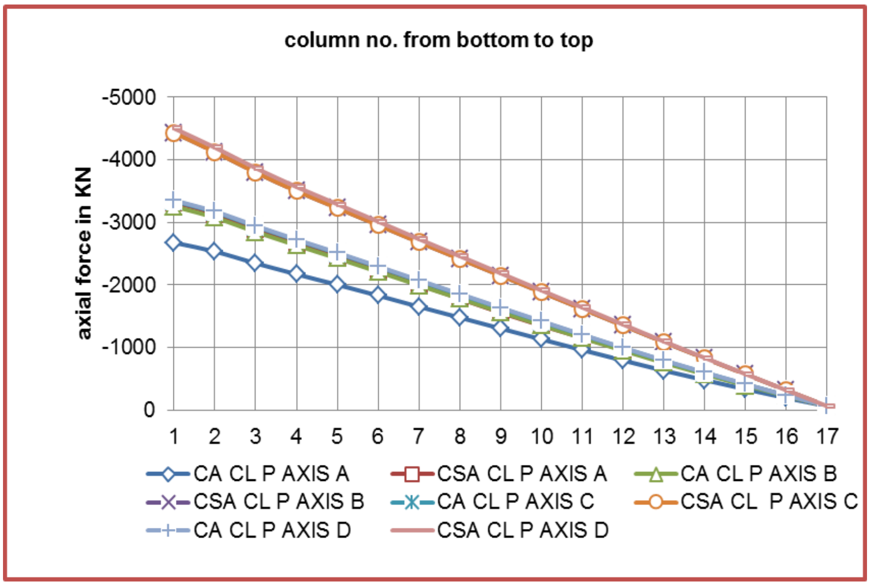


Figure 4-448 Axial force for columns of G+15 building

Axial force by CSA is higher than by CA by more than 1000KN.

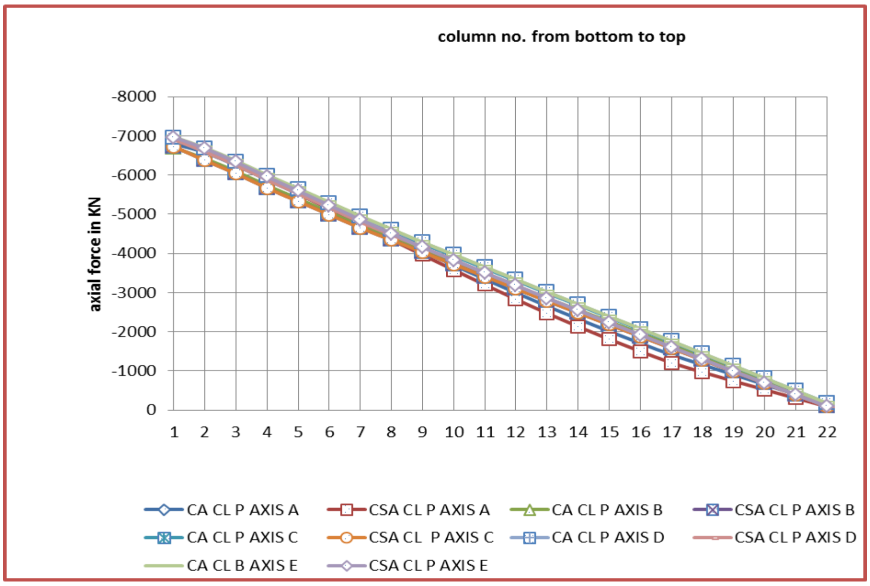


Figure 4-49 Axial Force for columns of G+20 building

There is very less axial force difference between the two analysis methods. But still CSA is larger than CA by 30KN.

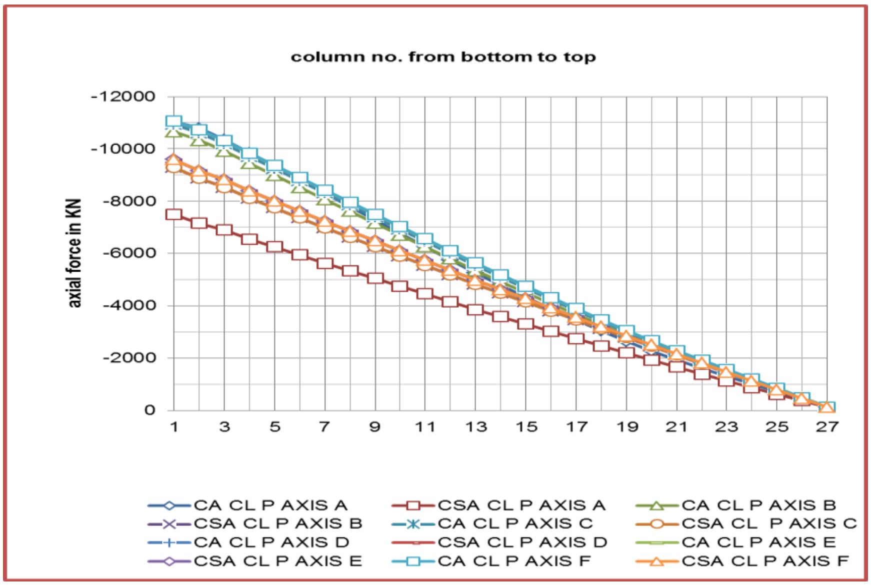


Figure 4-50 Axial Force for columns of G+25 building

For this building, CA axial force is exceeding CSA by 1500 KN.

4.6 Moment on Columns

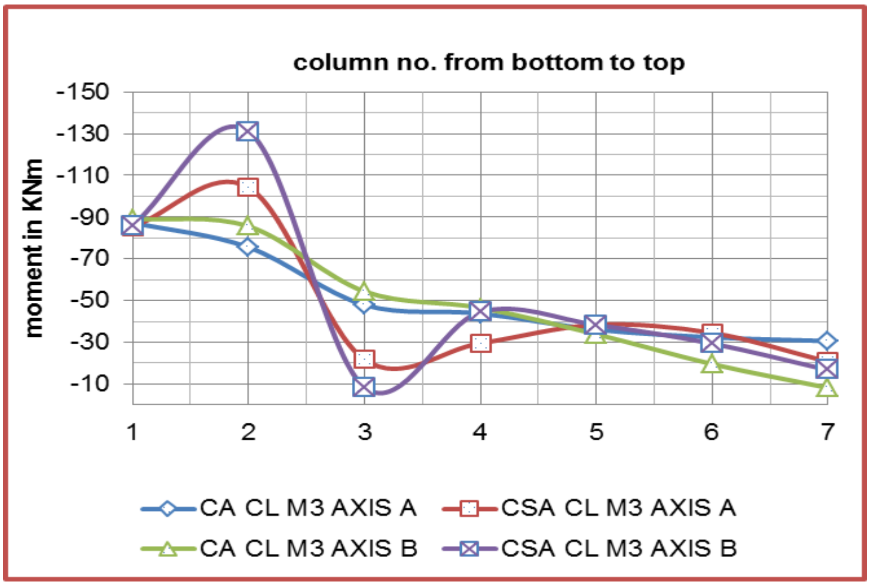


Figure 4-51 Moment for columns of G+5 building

The CSA result is larger than the CA especially at the second column with a maximum difference of 50 KNm. At the third joint, CA exceeds CSA by the same amount.

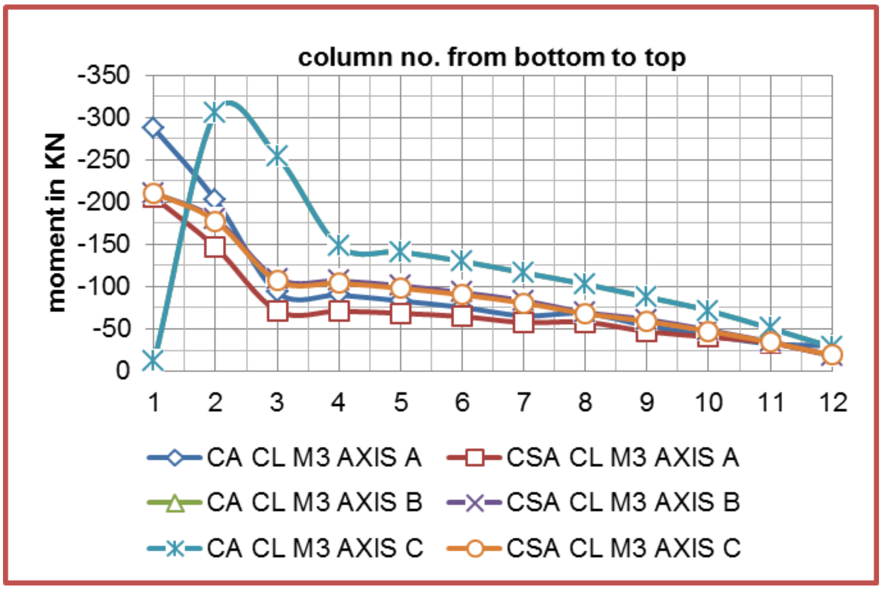


Figure 4-52 Moment for columns of G+10 building without shear wall

There is a clear exceedence of CA values on CSA values.

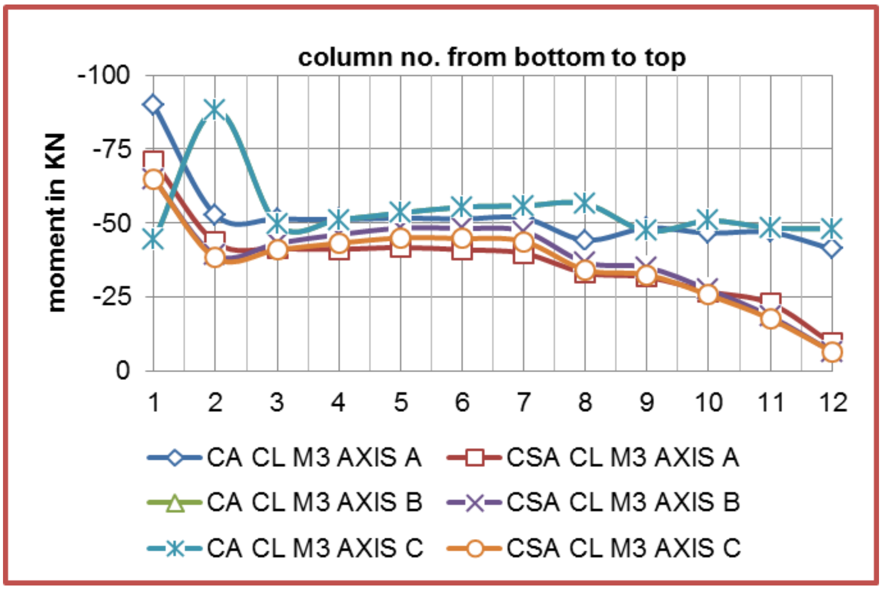


Figure 4-53 Moment for columns of G+10 building with shear wall

Here also CA is dominating.

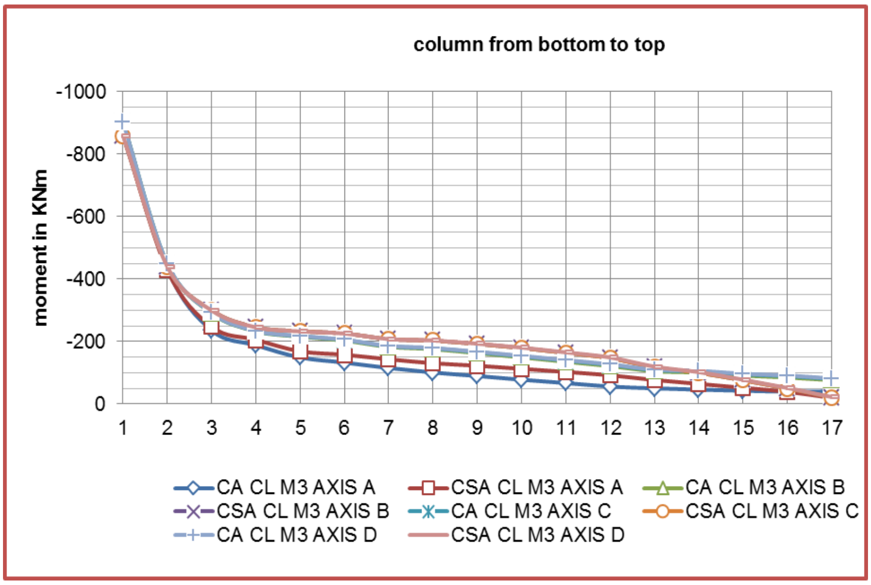


Figure 4-54 Moment for columns of G+15 building

For this building, CSA is now dominating.

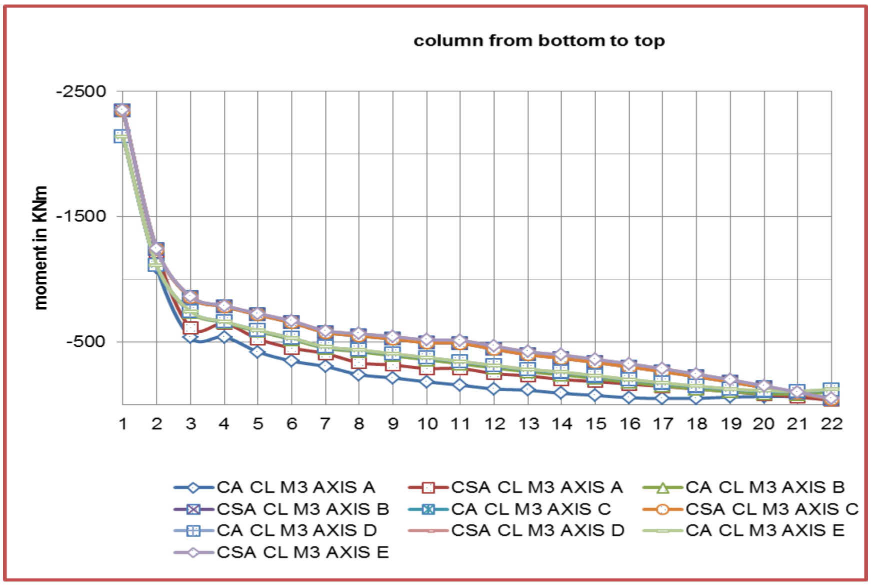


Figure 4-455 Moment for columns of G+20 building

Moment by CSA is greater to moment by CA even by more than 200 KNm.

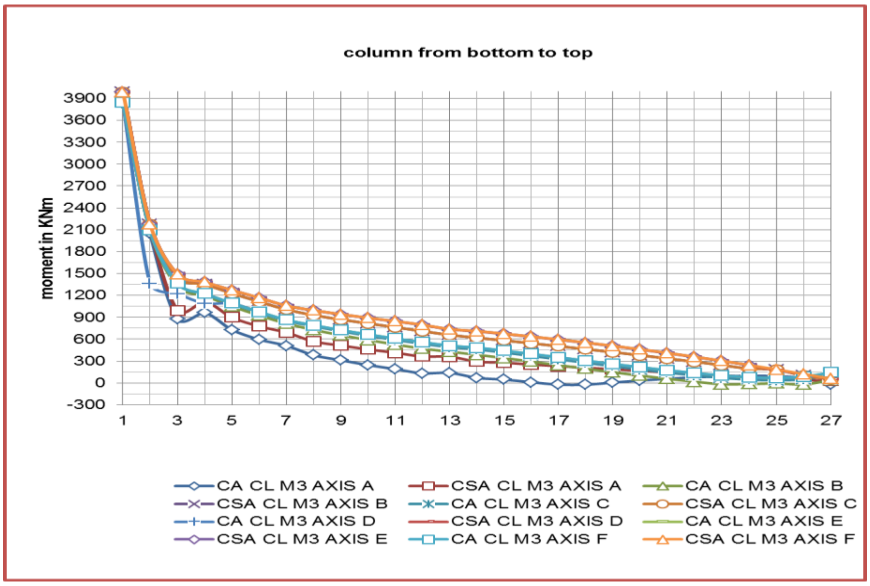


Figure 4-466 Moment for columns of G+25 building

For this building, CSA is larger than CA where the difference reaches 140 KNm.

Moments on columns show an increasing trend as the building becomes higher when analyzed by the CSA method. The influence of CA continues to the G+15 building. But, for the G+20 and G+25 buildings CSA method affects the moment in the columns.

5. CONCLUSION AND RECOMMENDATION

There is definitely considerable difference between the analysis results of CA and CSA methods.

For all buildings, the vertically downward deflection because of the complete analysis method is exaggerated. In the G+5 building CA method result indicates a 2.53mm additional down ward deflection of the last story than the CSA method. This result is also exaggerated for the G+25 building more than eight times resulting in a downward variation of 20.61mm.

Rotational value R2 is also greater as analyzed by the complete analysis method than the construction stage analysis method. The CSA rotational value for the G+5 building exceeds the CA rotational value by 0.000549 rad but for the rest buildings, CA becomes dominating. Thus, for the G+25 building the CA rotational value exceeds the CSA rotational value by 0.001975 rad.

Moments of beams and columns, Shear forces of beams and axial forces of columns are larger as analyzed by CA than CSA for the buildings of smaller story number. As the building story number increases, the mentioned CSA internal responses become larger than the CA.

For the G+5 and G+10 buildings, the CA method moment value is larger by around 35KNm for both positive and negative moments on beams. But beam moments for G+20 and G+25 buildings, the CSA method values are larger by around 25KNm.

This condition is true in that the complete analysis method assumes the construction of buildings and application of loading at the same time for all stories. But, the practical Construction analysis method considers the construction of buildings and application of loading step by step resulting a step by step response of structures to the applied loads.

Thus, I do not recommend the use of CA analysis results for design works. Especially, for high-rise buildings, their safety as designed by the CA result may be questioned. But for conditions that enforce to use the CA method only, the internal responses by CSA may be approximated from CA using factors that can be traced from this paper.

This work is performed on six buildings with number of stories varying from six to twenty six only. The irregularity of the buildings both in plan and in elevation is not included for this work. Thus, further researches on high rise buildings including the irregularities (plan, elevation), shear wall and partition wall arrangements and the creep and shrinkage effects may give broader and more reliable results.

The use of different slab types with different support conditions may also be recommended for further study.

More research may also help the development of some standard which enables the estimation of CSA method results from CA method results (as CSA method is tedious and time consuming requiring the exact load definition of each stage) for different conditions such as number of stories, function (purpose) of the building, plan and elevation irregularity, seismic zone, shear wall location and so on.

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APPENDIX A JOINT DISPLACEMENTS FOR EACH STAGE/STEP OF G+5 BUILDING

For appendices B and C, the joint numbers and column numbers are taken from axis A1, Y=0 from bottom to top and the values are results of nonlinear analysis for CSA1

Joint	StepType	StepNum	U1	U2	U3	R2
Text	Text	Unitless	mm	mm	mm	Radians
1	Step	1	0	0	0	0
1	Step	2	0	0	0	0
1	Step	3	0	0	0	0
1	Step	4	0	0	0	0
1	Step	5	0	0	0	0
1	Step	6	0	0	0	0
1	Step	7	0	0	0	0
1	Step	8	0	0	0	0
1	Step	9	0	0	0	0
1	Step	10	0	0	0	0
1	Step	11	0	0	0	0
1	Step	12	0	0	0	0
1	Step	13	0	0	0	0
1	Step	14	0	0	0	0
2	Step	1	0	0	-0.019366	0
2	Step	2	-5.66E-17	-2.159E-16	-0.131495	0.00062
2	Step	3	-5.47E-17	-2.29E-16	-0.177972	0.00062
2	Step	4	-1.80E-16	-2.275E-16	-0.451247	0.000823
2	Step	5	-1.79E-16	-2.339E-16	-0.481849	0.000823
2	Step	6	5.714E-17	-2.392E-16	-0.752164	0.000785

Joint	StepType	StepNum	U1	U2	U3	R2
Text	Text	Unitless	mm	mm	mm	Radians
2	Step	7	6.068E-17	-2.507E-16	-0.782766	0.000785
2	Step	8	6.14E-17	-1.355E-16	-1.057263	0.000804
2	Step	9	5E-17	-1.319E-16	-1.087865	0.000804
2	Step	10	3.271E-16	1.191E-17	-1.368359	0.000811
2	Step	11	3.038E-16	-1.077E-18	-1.398961	0.000811
2	Step	12	6.053E-16	2.365E-16	-1.686246	0.00082
2	Step	13	6.311E-16	2.136E-16	-1.716848	0.00082
2	Step	14	5.51E-16	4.828E-16	-1.899722	0.000824
3	Step	1	0	0	0	0
3	Step	2	0	0	0	0
3	Step	3	6.108E-18	-4.582E-17	-0.06042	0
3	Step	4	-4.70E-16	-1.606E-16	-0.401186	0.000349
3	Step	5	-4.73E-16	-1.81E-16	-0.45015	0.000349
3	Step	6	1.909E-16	8.548E-17	-0.880992	0.000544
3	Step	7	1.938E-16	5.536E-17	-0.929955	0.000544
3	Step	8	1.935E-16	2.78E-16	-1.368129	0.000515
3	Step	9	1.569E-16	2.901E-16	-1.417093	0.000515
3	Step	10	1.066E-15	7.925E-16	-1.864739	0.000535
3	Step	11	9.907E-16	7.486E-16	-1.913703	0.000535
3	Step	12	2.002E-15	1.642E-15	-2.372252	0.000544
3	Step	13	2.076E-15	1.581E-15	-2.421215	0.000544
3	Step	14	2.035E-15	2.347E-15	-2.713268	0.00055
4	Step	1	0	0	0	0
4	Step	2	0	0	0	0
4	Step	3	0	0	0	0

Joint	StepType	StepNum	U1	U2	U3	R2
Text	Text	Unitless	mm	mm	mm	Radians
4	Step	4	0	0	0	0
4	Step	5	-6.37E-18	-3.212E-17	-0.058646	0
4	Step	6	9.619E-16	5.295E-16	-0.560031	0.000481
4	Step	7	9.574E-16	4.837E-16	-0.62836	0.000481
4	Step	8	7.964E-16	5.499E-16	-1.236921	0.000707
4	Step	9	7.443E-16	5.576E-16	-1.30525	0.000707
4	Step	10	2.572E-15	1.186E-15	-1.927468	0.000682
4	Step	11	2.448E-15	1.114E-15	-1.995798	0.000682
4	Step	12	4.264E-15	2.809E-15	-2.633169	0.000711
4	Step	13	4.376E-15	2.714E-15	-2.701498	0.000711
4	Step	14	4.483E-15	3.86E-15	-3.107829	0.000716
5	Step	1	0	0	0	0
5	Step	2	0	0	0	0
5	Step	3	0	0	0	0
5	Step	4	0	0	0	0
5	Step	5	0	0	0	0
5	Step	6	0	0	0	0
5	Step	7	-9.00E-18	-6.814E-17	-0.078012	0
5	Step	8	-5.30E-16	-5.914E-17	-0.757796	0.000483
5	Step	9	-5.99E-16	-7.637E-17	-0.845491	0.000483
5	Step	10	2.617E-15	2.121E-16	-1.639371	0.000713
5	Step	11	2.434E-15	1.219E-16	-1.727066	0.000713
5	Step	12	5.239E-15	2.772E-15	-2.540935	0.000692
5	Step	13	5.399E-15	2.634E-15	-2.628629	0.000692
5	Step	14	5.656E-15	4.162E-15	-3.148021	0.000719

Joint	StepType	StepNum	U1	U2	U3	R2
Text	Text	Unitless	mm	mm	mm	Radians
6	Step	1	0	0	0	0
6	Step	2	0	0	0	0
6	Step	3	0	0	0	0
6	Step	4	0	0	0	0
6	Step	5	0	0	0	0
6	Step	6	0	0	0	0
6	Step	7	0	0	0	0
6	Step	8	0	0	0	0
6	Step	9	-8.99E-17	-4.838E-17	-0.097377	0
6	Step	10	4.765E-15	-1.896E-16	-0.963047	0.000493
6	Step	11	4.511E-15	-2.955E-16	-1.070107	0.000493
6	Step	12	8.26E-15	3.104E-15	-2.056882	0.000728
6	Step	13	8.485E-15	2.917E-15	-2.163942	0.000728
6	Step	14	8.978E-15	4.879E-15	-2.795003	0.00067
7	Step	1	0	0	0	0
7	Step	2	0	0	0	0
7	Step	3	0	0	0	0
7	Step	4	0	0	0	0
7	Step	5	0	0	0	0
7	Step	6	0	0	0	0
7	Step	7	0	0	0	0
7	Step	8	0	0	0	0
7	Step	9	0	0	0	0
7	Step	10	0	0	0	0
7	Step	11	-3.24E-16	-1.305E-16	-0.116743	-2.85E-20

Joint	StepType	StepNum	U1	U2	U3	R2
Text	Text	Unitless	mm	mm	mm	Radians
7	Step	12	3.452E-15	4.042E-15	-1.175863	0.000504
7	Step	13	3.769E-15	3.802E-15	-1.302289	0.000504
7	Step	14	4.797E-15	5.926E-15	-2.04242	0.000849
8	Step	1	0	0	0	0
8	Step	2	0	0	0	0
8	Step	3	0	0	0	0
8	Step	4	0	0	0	0
8	Step	5	0	0	0	0
8	Step	6	0	0	0	0
8	Step	7	0	0	0	0
8	Step	8	0	0	0	0
8	Step	9	0	0	0	0
8	Step	10	0	0	0	0
8	Step	11	0	0	0	0
8	Step	12	0	0	0	0
8	Step	13	4.398E-16	-2.962E-16	-0.136109	4.62E-20
8	Step	14	2.39E-15	1.892E-15	-0.887514	-0.000075

APPENDIX B COLUMN FORCES FOR EACH STAGE/ STEP FOR CSA1 OF G+5 BUILDING

Frame	**OutputCase**	**StepType**	**StepNum**	**P**	**M3**
Text	Text	Text	Unitless	KN	KN-m
1	csa1	Step	1	-18.954	0
1	csa1	Step	2	-73.827	-9.299
1	csa1	Step	3	-96.572	-9.299
1	csa1	Step	4	-230.306	-12.3592
1	csa1	Step	5	-245.282	-12.3592
1	csa1	Step	6	-377.567	-11.7792
1	csa1	Step	7	-392.543	-11.7792
1	csa1	Step	8	-526.875	-12.0699
1	csa1	Step	9	-541.851	-12.0699
1	csa1	Step	10	-679.117	-12.1702
1	csa1	Step	11	-694.093	-12.1702
1	csa1	Step	12	-834.684	-12.3091
1	csa1	Step	13	-849.66	-12.3091
1	csa1	Step	14	-939.154	-12.3683
2	csa1	Step	1	0	0
2	csa1	Step	2	0	0
2	csa1	Step	3	-22.745	1.854E-16
2	csa1	Step	4	-77.792	-19.8409
2	csa1	Step	5	-92.768	-19.8409
2	csa1	Step	6	-223.698	-22.8035
2	csa1	Step	7	-238.674	-22.8035
2	csa1	Step	8	-372.173	-23.1042
2	csa1	Step	9	-387.149	-23.1042
2	csa1	Step	10	-523.483	-23.9644
2	csa1	Step	11	-538.459	-23.9644
2	csa1	Step	12	-678.146	-24.6889
2	csa1	Step	13	-693.122	-24.6889
2	csa1	Step	14	-782.172	-25.0469
3	csa1	Step	1	0	0
3	csa1	Step	2	0	0
3	csa1	Step	3	0	0

Frame	OutputCase	StepType	StepNum	P	M3
Text	Text	Text	Unitless	KN	KN-m
3	csa1	Step	4	0	0
3	csa1	Step	5	-14.976	5.121E-17
3	csa1	Step	6	-69.529	-16.952
3	csa1	Step	7	-84.505	-16.952
3	csa1	Step	8	-216.271	-20.1376
3	csa1	Step	9	-231.247	-20.1376
3	csa1	Step	10	-366.249	-20.4372
3	csa1	Step	11	-381.225	-20.4372
3	csa1	Step	12	-519.514	-21.3389
3	csa1	Step	13	-534.49	-21.3389
3	csa1	Step	14	-622.866	-21.6716
4	csa1	Step	1	0	0
4	csa1	Step	2	0	0
4	csa1	Step	3	0	0
4	csa1	Step	4	0	0
4	csa1	Step	5	0	0
4	csa1	Step	6	0	0
4	csa1	Step	7	-14.976	1.866E-16
4	csa1	Step	8	-70.055	-18.1995
4	csa1	Step	9	-85.031	-18.1995
4	csa1	Step	10	-217.783	-21.6049
4	csa1	Step	11	-232.759	-21.6049
4	csa1	Step	12	-369.251	-22.3418
4	csa1	Step	13	-384.227	-22.3418
4	csa1	Step	14	-471.66	-23.0699
5	csa1	Step	1	0	0
5	csa1	Step	2	0	0
5	csa1	Step	3	0	0
5	csa1	Step	4	0	0
5	csa1	Step	5	0	0
5	csa1	Step	6	0	0
5	csa1	Step	7	0	0
5	csa1	Step	8	0	0

Frame	OutputCase	StepType	StepNum	P	M3
Text	Text	Text	Unitless	KN	KN-m
5	csa1	Step	9	-14.976	5.636E-16
5	csa1	Step	10	-70.493	-18.5785
5	csa1	Step	11	-85.469	-18.5785
5	csa1	Step	12	-219.183	-22.2353
5	csa1	Step	13	-234.159	-22.2353
5	csa1	Step	14	-320.516	-22.1921
6	csa1	Step	1	0	0
6	csa1	Step	2	0	0
6	csa1	Step	3	0	0
6	csa1	Step	4	0	0
6	csa1	Step	5	0	0
6	csa1	Step	6	0	0
6	csa1	Step	7	0	0
6	csa1	Step	8	0	0
6	csa1	Step	9	0	0
6	csa1	Step	10	0	0
6	csa1	Step	11	-14.976	-3.148E-16
6	csa1	Step	12	-70.923	-18.9342
6	csa1	Step	13	-85.899	-18.9342
6	csa1	Step	14	-170.247	-23.2316
7	csa1	Step	1	0	0
7	csa1	Step	2	0	0
7	csa1	Step	3	0	0
7	csa1	Step	4	0	0
7	csa1	Step	5	0	0
7	csa1	Step	6	0	0
7	csa1	Step	7	0	0
7	csa1	Step	8	0	0
7	csa1	Step	9	0	0
7	csa1	Step	10	0	0
7	csa1	Step	11	0	0
7	csa1	Step	12	0	0
7	csa1	Step	13	-14.976	3.527E-16
7	csa1	Step	14	-23.695	-12.2686