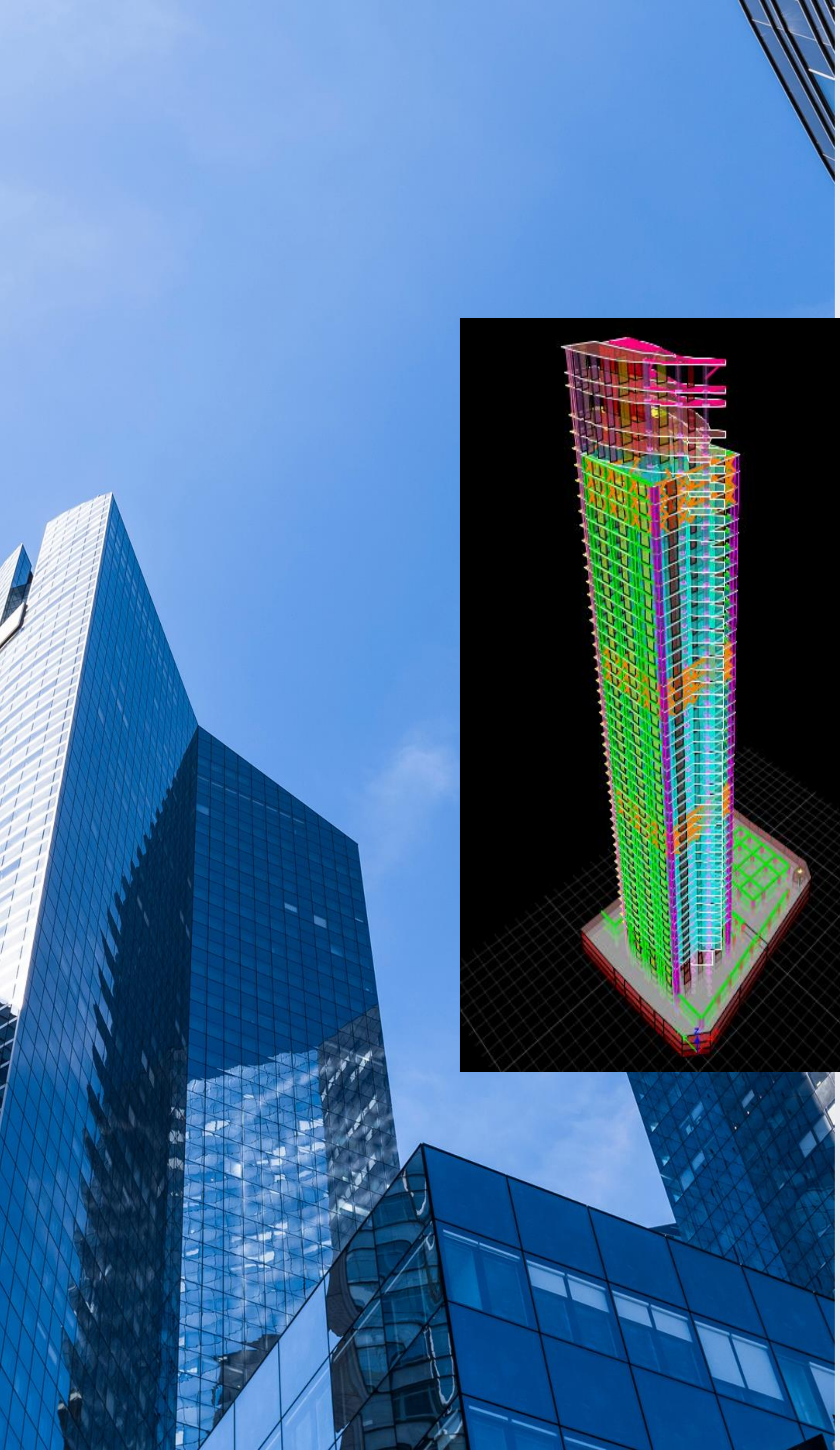
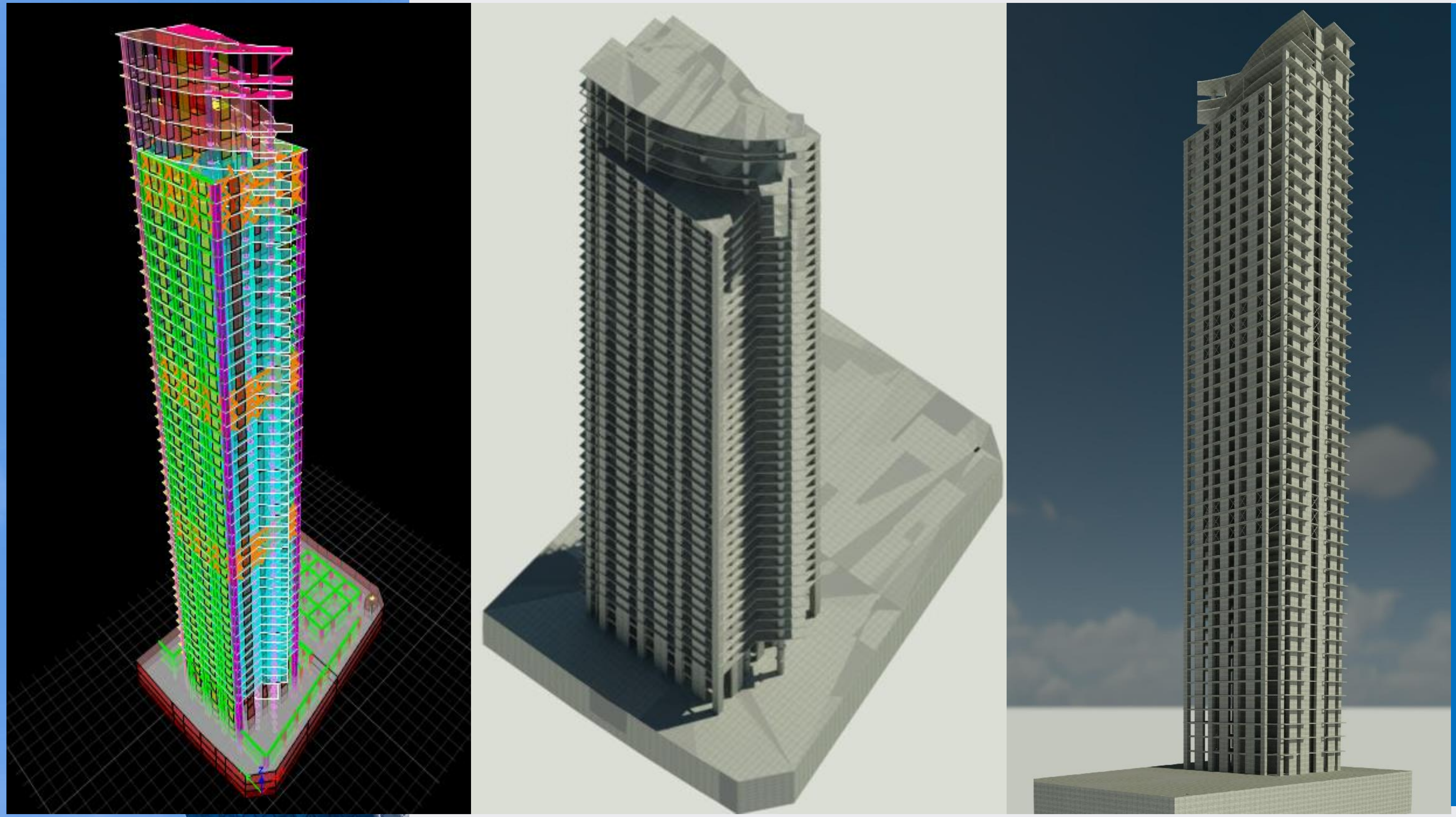




G+47 High rise Executive Tower located at Arizona

Designed by: Er. Kareemul Islam (Structural Engineer)
Guided and Checked By: Er. Aman Kumar (Director @STRUCTUREX)

Design Methodology

Structure member design means selection of a cross section that will safely and economically resist the applied force.

LRFD method is used for the design of the structure. In this method, each member of the structure is designed to satisfy Serviceability and collapse criteria.

Economic usually means minimum weight (lightest cross sectional shape, minimum amount of steel).

There are three different philosophies to design any structure.

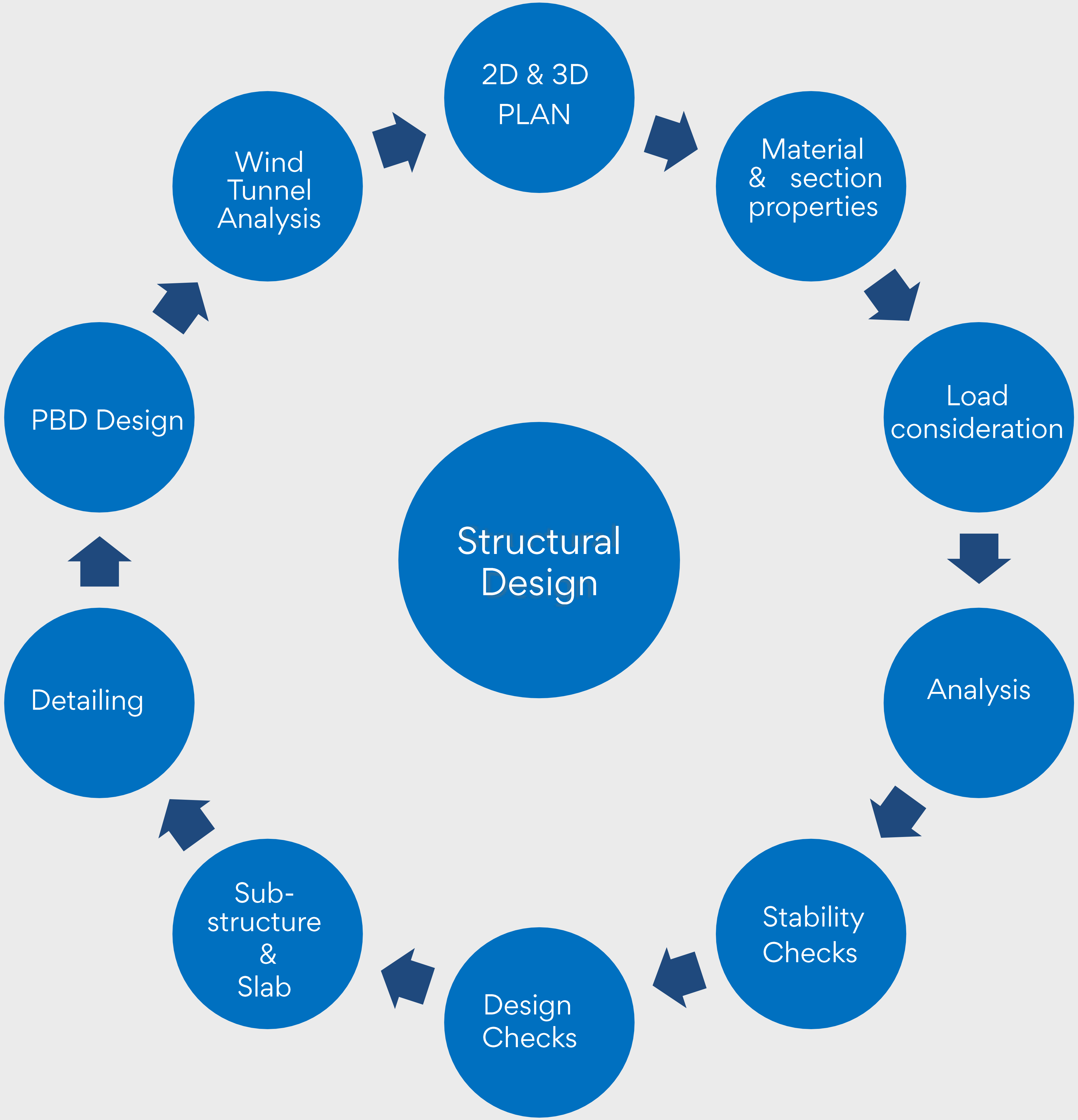
Allowable Stress Design(ASD).

Plastic Design.

Load and Resistance Factor Design (LRFD).

Structural Plan

Taking Architectural plan as base, the column positions are decided to provide proper structural frame without disturbing the parking plan. Column orientation is taken to provide maximum rigidity along spans and also proper Resistance to lateral (wind& earthquake) loads.


Structural Design
2D & 3D PLAN
Material & section properties
Load consideration
Analysis
Stability Checks
Design Checks
Sub-structure & Slab
Detailing
PBD Design
Wind Tunnel Analysis

Outline

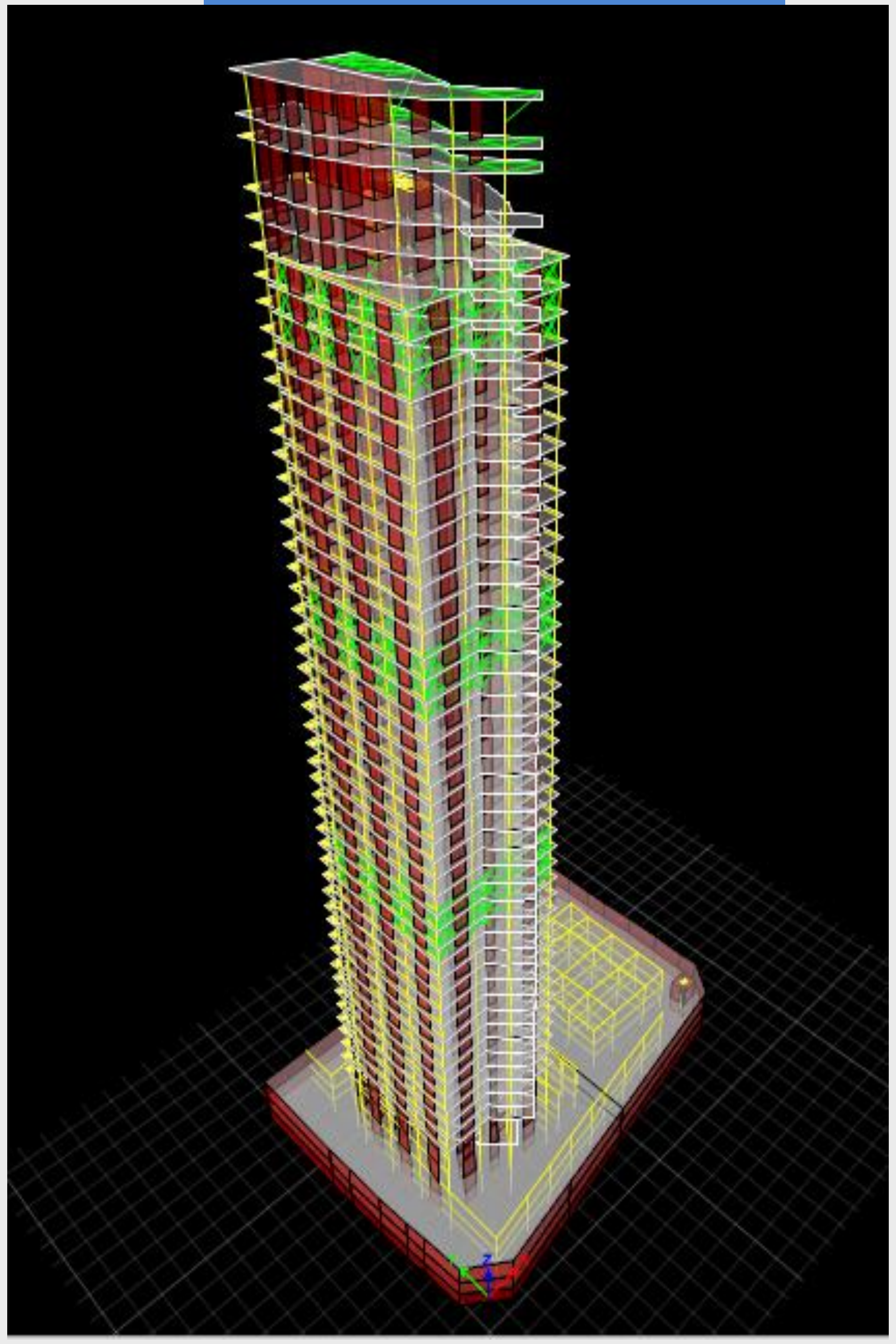
1. Code Design Methodology to the Structural Design
2. Description of the Project worked on
3. Determination of the dimensions of the structural elements
4. ETABS (Super Structure Design)
5. ETABS model
6. Earthquake & Wind loads
7. SAFE (Sub Structure Design)
8. Design of Footings
9. Design of Columns Beams Slabs and Beams
10. Lateral Stability Systems

Outline

- Commercial Building G+47 stories + 3 basement for parking with GL.
- F'c = 35Mpa for horizontal element (Slabs and Beams)
- F'c = 50 Mpa for Vertical element (Columns, Walls, and Foundations)
- Fy = 400 Mpa (longitudinal reinforcement)
- Fyt =240 Mpa (transversal reinforcement)
- Qa = 300 KN/m^2 (Bearing capacity of soil)
- Height of the basement 3.5&5.3m (Vary)
- Height of the typical stories 3.6m

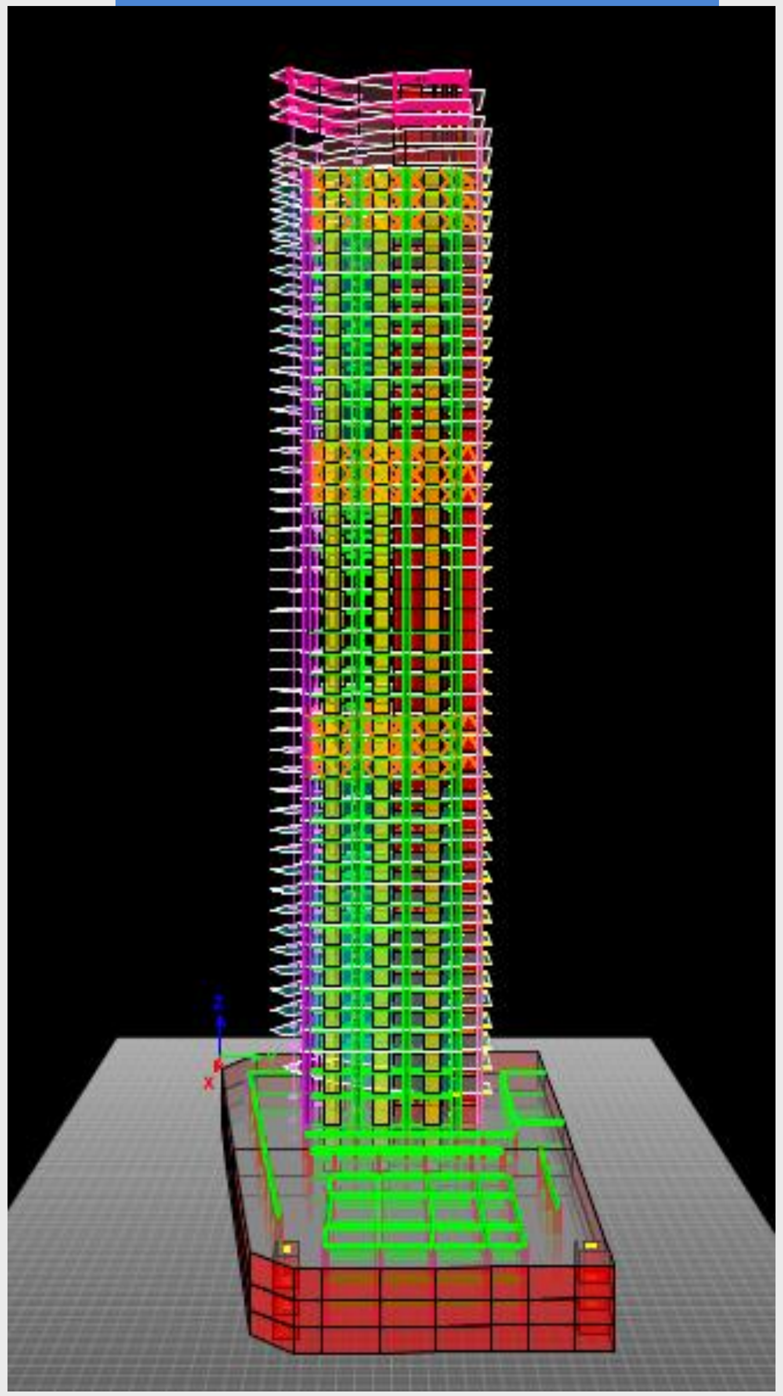
Project Information

Location of Building	Arizona
Building Type	Commercial Purposes
Number of Floors	3B + G + 47 Floors (3 BASEMENT WITH GROUND FLOOR)
Typical Height of Floor to Floor	3.60 m & 4.3 m (Vary)
Total Height of Building	185.00 m
Length of Building	71.90 m (With Basement) 27.40m (Tower)
Width of Building	66.0m (With Basement) 35.1m (Tower)
SBC of Soil	300 KN/m^2



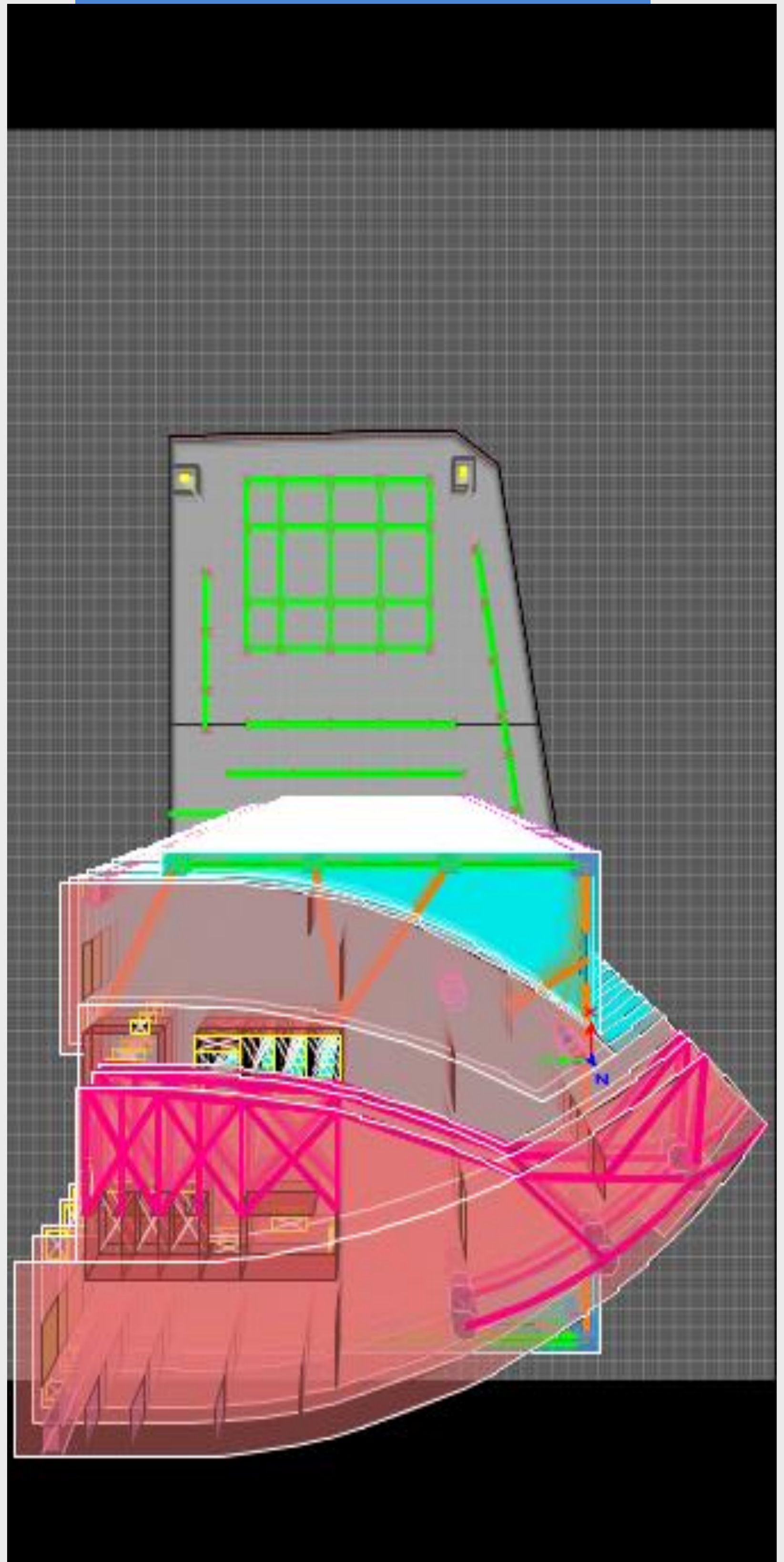
Project Information

Loads Considered	Dead, Live, Wind & Seismic loads (ASCE 7-16 & UBC-97)
Analysis Performed	FEM, Modal Analysis , Response Spectrum Analysis, P-Δ Analysis (with modified stiffness)
Checks	Serviceability, Stability, Irregularity & Modal checks (as per ACI-318 & UBC 97)
Design & Detailing	As per AC1-314
Foundation System	Raft + Pile Foundation Allowable Settlement=50mm
Foundation Checks	Ground bearing pressure (GBP), Settlement, Punching Shear, Crack width & Reinf. Soil Structure interactions
Software Program Used	ETABS, SAFE, Spread sheets & Auto cad CSI Detailer



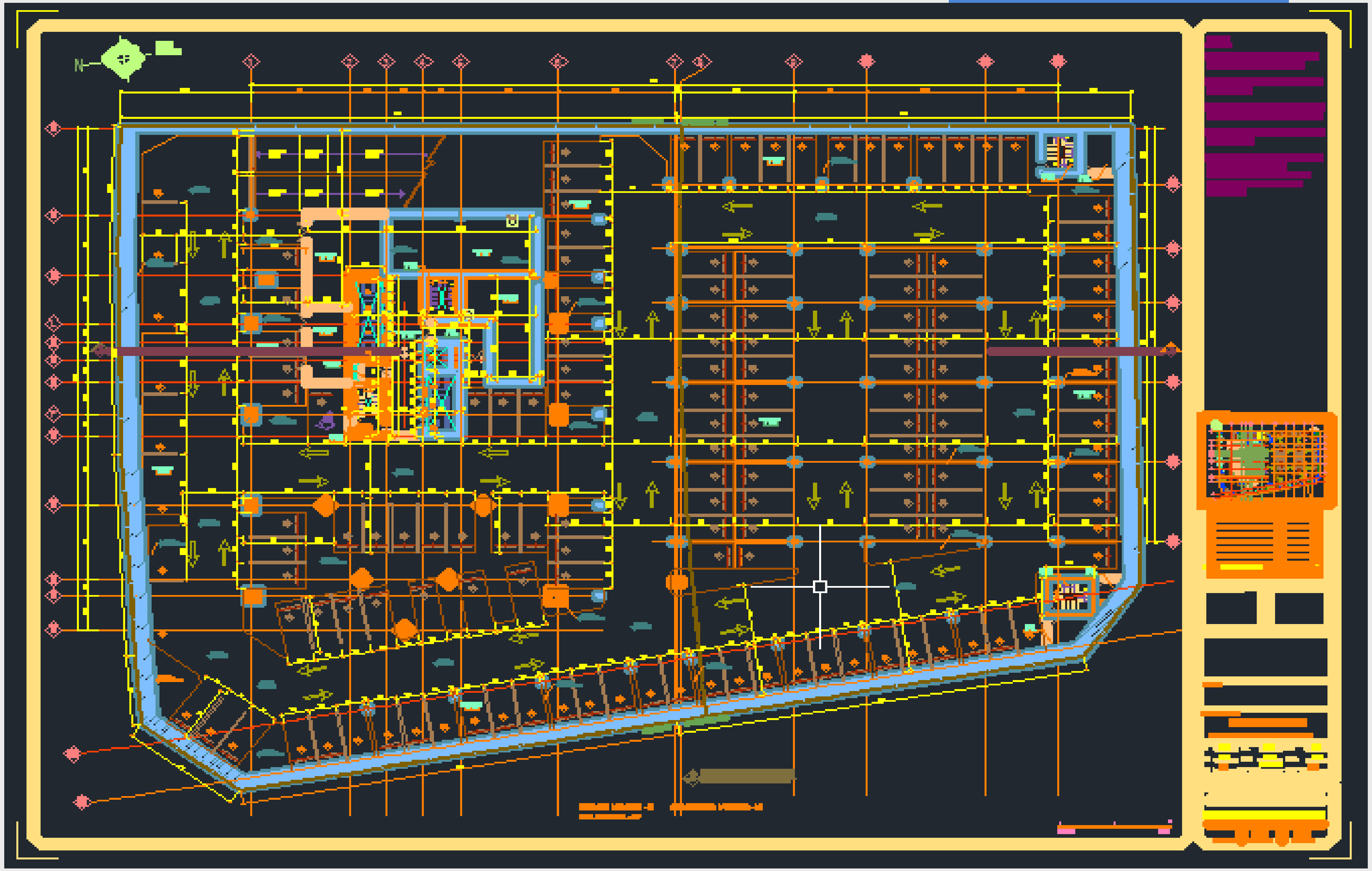
Tower Design with different Seismic systems

- Shear wall earthquake system, Special frame earthquake system, Dual system (shear wall with special frame)
- Choose the correct structural system from a lot of structural forms, floors systems with collaboration with design software modeling techniques and designed codes
- Viscous damper earthquake resistant
- Benefits of using outrigger in high rise tower design
- Show the effect of sub-modulus parameters (foundation type & bearing capacity of soil) on the structure response of tall buildings (story displacement, story drift, foundation settlement, and column forces).



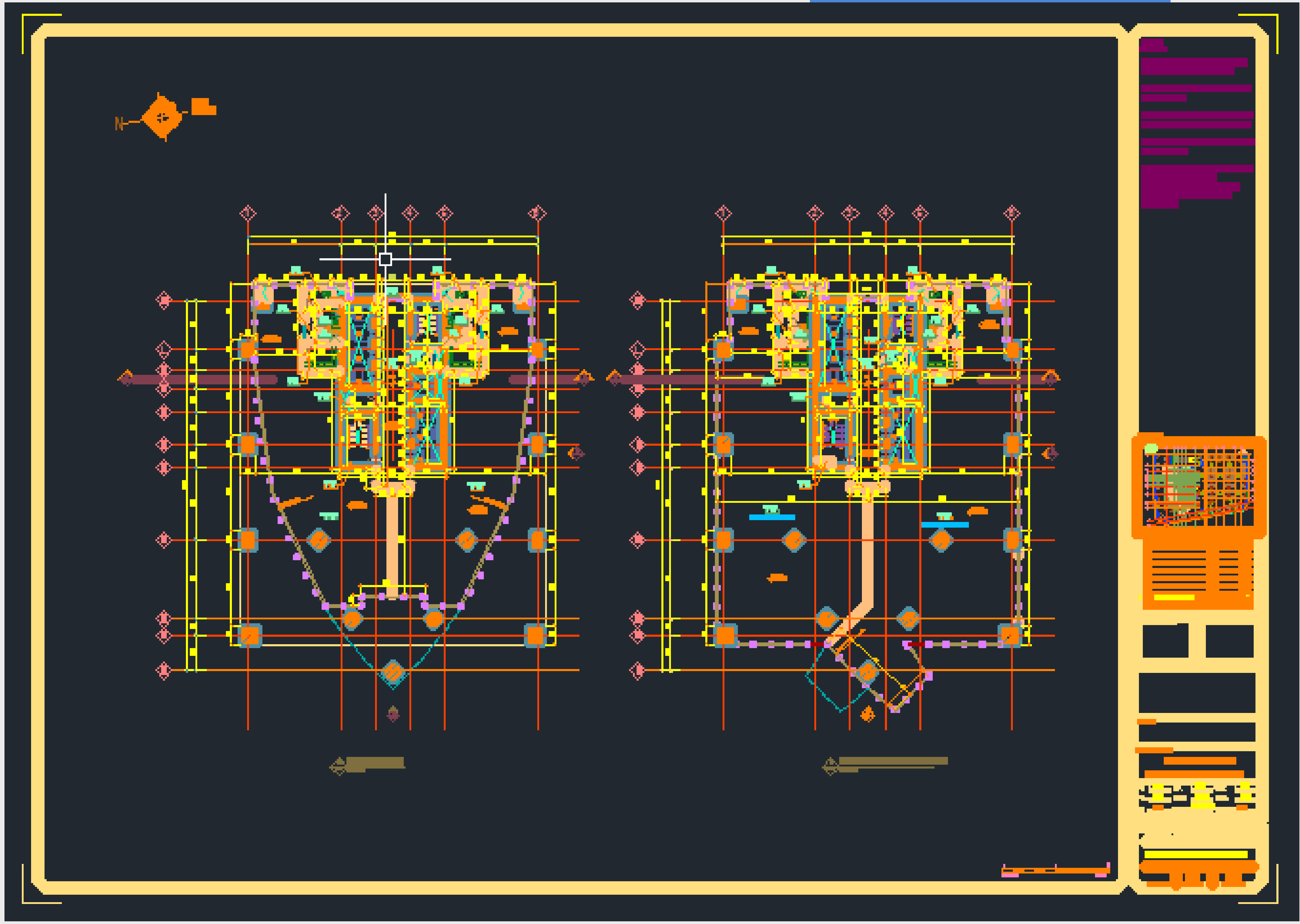
Architectural Drawings

Typical Basement Plan



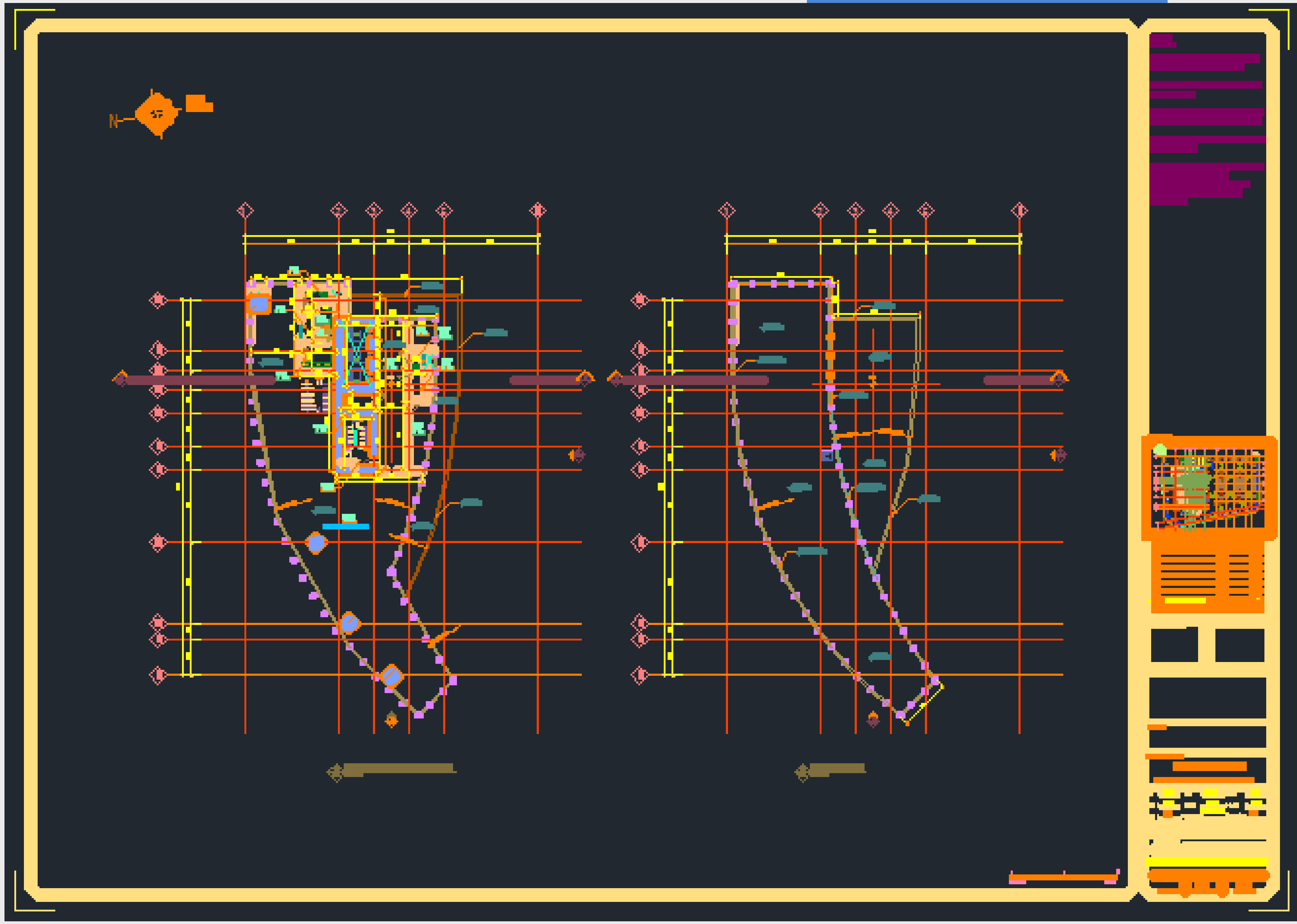
Architectural Drawings

Typical Floor Plan



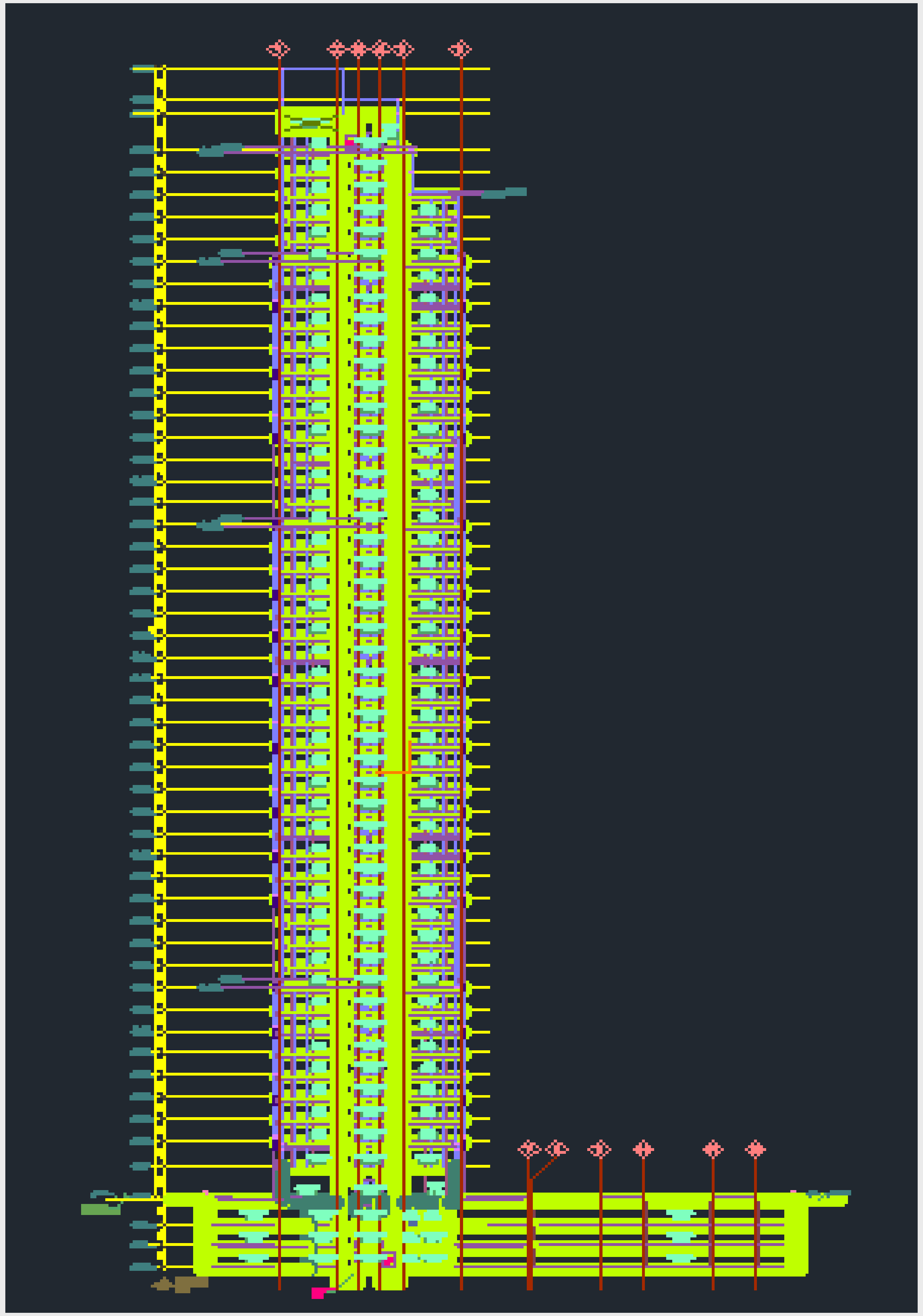
Architectural Drawings

Typical Floor Plan



Architectural Drawings

Typical Elevation



Description of the studied Building

The building used in this study was designed for Arizona site of US. The structural system used to resist lateral forces is Special RC shear wall, Outrigger System & Dampers in Three-dimensional model of the studied building is shown in Figure 1. Floor plans are shown in Figure 2. More details are shown in Table 1.

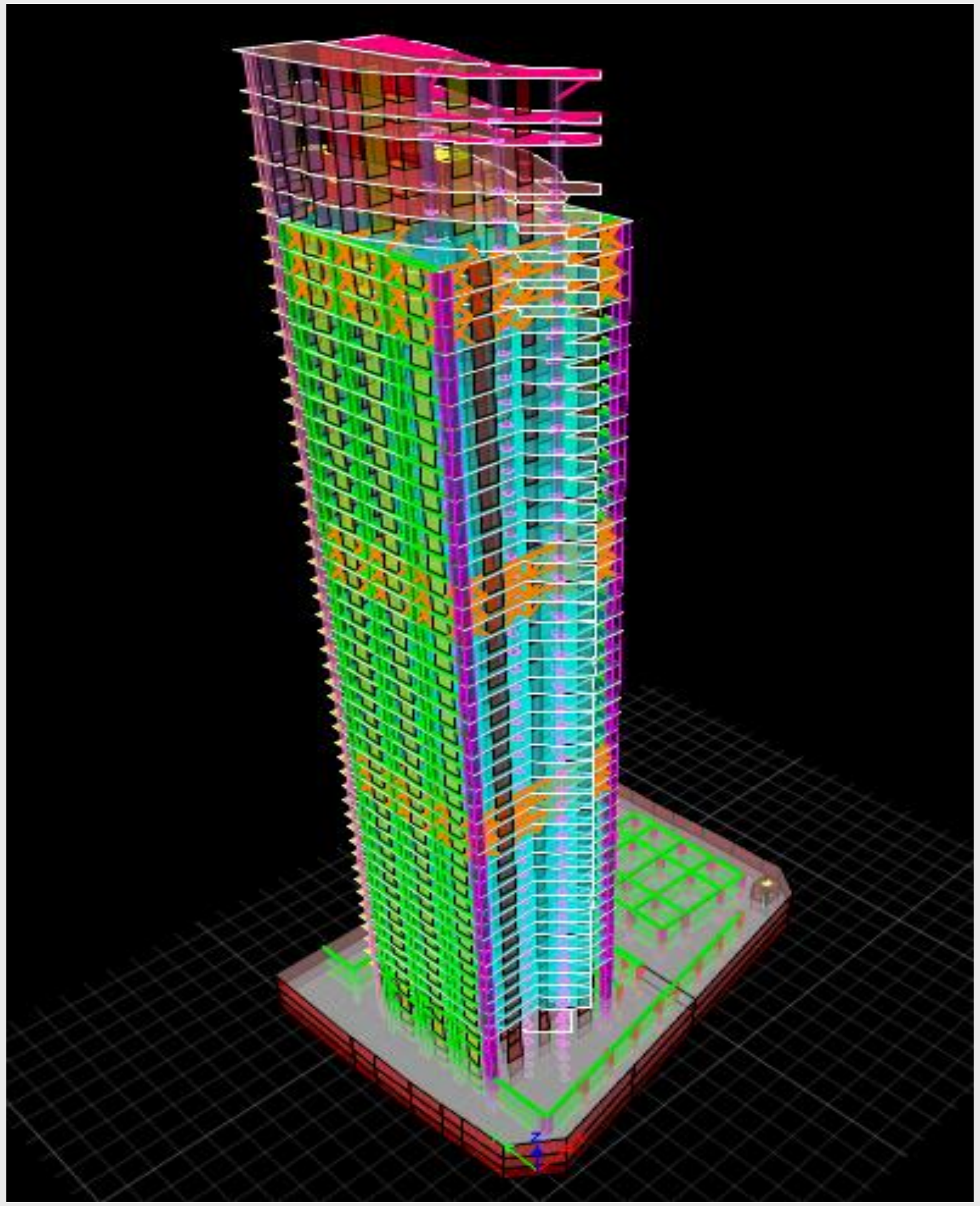


Figure 1. Three-dimensional model of the studied building.

Structural Framing

Typical Plan

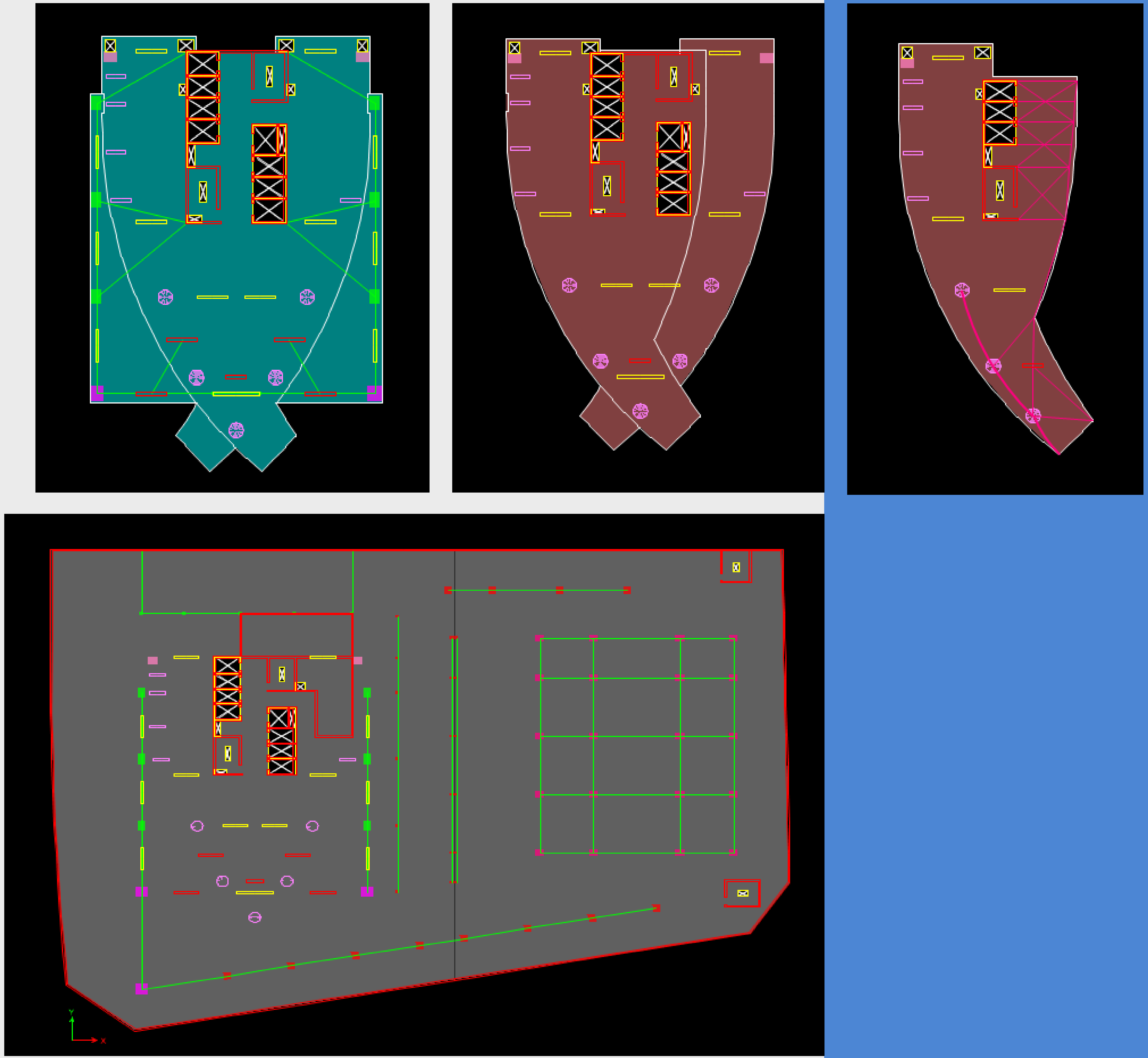


Figure 2. Three-dimensional model of the studied building.

Description of the Project

- Assume thickness of shear walls
- Assume all columns & beams
- As per Stability Systems
- We will use the American code for the structural design

CHECKS TO BE TAKEN

STEP-1 EXTRACT STRUCTURE FRAMING FROM ARCHITECTURAL DRAWINGS.

Taking Architectural plan as base, the column positions are decided to provide proper structural frame without disturbing the parking plan. Column orientation is taken to provide maximum rigidity along spans and also proper Resistance to lateral (wind& earthquake) loads.

STEP-2 GENARATES MATHMATICAL MODEL IN ETABS

- Check the materials concrete, rebar, steel etc
- Prominently these are Concrete fck & Steel fy, fu. Once the material properties are entered, define Beam, Column sizes in the Frame section list of ETABS.
- Check section sizes of framing members.

STEP-3 REMOVE ALL ERRORS AND CHECK INSTABILITY .

- Check errors joints to joints, joints to frames , joints to shells, frames overlap , shell overlaps etc.
- Check the instability/warning

Step-4 perform modal analysis as per self weight free vibration conditions (all checks must be taken & optional)

THREE FUNDAMENTAL TRANSLATIONAL NATURAL PERIODS,

TX1 = ASSOCIATED WITH ITS HORIZONTAL TRANSLATIONAL OSCILLATION ALONG X

TY1 = ASSOCIATED WITH ITS HORIZONTAL TRANSLATIONAL OSCILLATION ALONG Y
TZ1 = ASSOCIATED WITH ITS ROTATIONAL OSCILLATION ALONG Z

Rz value should be less than 10%. In mode 1&2

- Guidelines for fundamental time-period = 0.1 * N (N= No of levels)
- The fundamental lateral natural periods of the building in the two principal plan directions are away from each other by at least 10 percent of the larger value. (Optional Check)
- The natural period of fundamental torsional mode of vibration shall not be exceed 0.9 times the smaller of the natural periods of the fundamental translational modes of vibration in each of the orthogonal directions in plan. (Optional Check)
- The first three modes together contribute at least 65 present mass participation factor in each principal plan direction. (Optional Check)
- Another check the submission of all the modes at least should be cross 90%.

NOTE- This provision is not applicable to buildings with large podiums in the lower story's.

Step-5 perform gravity analysis

- (Calculation of DL, SDL,LL, OHT, LMR etc. & check gravity deflection as per desired codes).S/350

Step-6 perform linear static analysis (equivalent static method)

- Check Load combination
- Check the mass source
- Live load reduction factor
- Check Static Seismic Forces as per Code Provisions
- Check Base shear as per Code Provisions
- Check deflection against EQ allowed = H/250

Step-7 Perform Dynamic Analysis

- Check that the Response spectrum is correctly scaled (Scale factor = (Ex / Spec x) x Existing Scale factor)

  As per ASCE 7-16 FX or FY of RS max. > = 0.85 x calculated base shear (EX or EY)
- Check Vertical Earthquake Effects (Full EQ load of one direction plus 30% of EQ load of other directions)
- Check the mass participation factor in first three modes & all modes (Optional)
- Check that the participation factor is compliant to codes or not.
- Check load combinations
- Check Dynamic Seismic Forces as per Code Provisions
- Frequency Check
- (NO of modes all 90%, Max. allowable frequency is 33 Hz)
- Maximum deflection against EQ allowed = H/250
- Inter storey drift ratio should be less than **ΔM = 0.7R ΔS** times the story height.
- Story Stiffness
- We need to take here stiffness of upper story should not be much greater than the stiffness of story below.
- Eccentricity & Accidental Torsion
- The eccentricity between the centre of mass and the geometric centroid of the building at that not exceed 15% of the overall building width along each principal axis considered at each level.
- Check the Pounding effect & Expansion Joint

Step-8 All Irregularities checks (plan & elevation irregularities)

Table 5 Plan Irregularities

i. Torsion
ii. Re-entrant Corners
iii. Openings in Floor Slabs
iv. Out-of-plane Offsets
v. Non-parallel Frames

A building is said to be torsional irregular, when,

- The maximum horizontal displacement of any floor in the direction of the lateral force at one end of the floor is more than 1.5 times its minimum horizontal displacement at the far end of the same floor in that direction; $\Delta max > 1.5 \Delta min$

- The natural period corresponding to the fundamental torsional mode of oscillation is more than those of the first two translational modes of oscillation along each principal plan directions

- In torsion ally irregular buildings, when the ratio of maximum horizontal displacement at one end and the minimum horizontal displacement at the other end is the range of 1.5 -2.0

- The floor slabs are stiff in their own plane (this happens when its plan aspect ratio is less than 3)
- The maximum plan aspect ratio (L,/B,) of the overall building shall not exceed 5.0. In case of an L shaped building, Z, and B, shall refer to the respective length and width of each leg of the building.

ii) Re-Entrant Corners

- A building is said to have a re-entrant corner in any plan direction, when its structural configuration in plan has a projection of size greater than 15 percent of its overall plan dimension in that direction.

iii) Openings-in Floor Slabs

In buildings with discontinuity in their in-plane stiffness, if the area of the geometric cut-out is,

- Less than or equal to 50 percent, the floor slab shall be taken as rigid or flexible depending on the location of and size of openings; and

- More than 50 percent, the floor slab shall be taken as flexible.

iv) Out of Plane Offsets

Conditions shall be satisfied, if the building is located in Seismic Zones III, IV and V:

- Lateral drift shall be less than 0.2 percent in the story having the offset and in the story below; and

- The forces and moments due to earthquake effects in the elements connecting the two vertical elements with out of plane offset elements, the vertical element supporting the offset, and connections shall be enhanced by a factor of at least 2.5.

Non Parallel Frames

- To check if the structure needs to take in consideration the orthogonal effects that the seismic loads make.
- To take into consideration the orthogonal effects - response spectrum
- Full EQ load of one direction plus 30% of EQ load of other direction

Vertical Irregularities

- Stiffness (Soft story)
- Mass
- Geometry
- In Plane Discontinuity
- Strength (Weak Story)
- Floating Columns
- Modes of Oscillation

i) Stiffness Irregularity (Soft Story)

- A soft story is a story whose lateral stiffness is less than that of the storey above.
- The inter-storey drift shall be limited to 0.2 percent in that storey and all story below, if any, with stiffness irregularity.
- Lateral translational stiffness of any storey shall not be less than 70 percent of that of the storey above.

ii) Mass Irregularity

- Mass irregularity shall be considered to exist, when the seismic weight of any floor is more than 150 percent of that of the floors below.

iii) Vertical Geometry Irregularity

- Vertical geometric irregularity shall be considered to exist, when the horizontal dimension of the lateral force resisting system in any storey is more than 130 percent of the storey below.

iv) In Plane Discontinuity

- In-plane discontinuity in vertical elements which are resisting lateral force shall be considered to exist, when in-plane offset of the lateral force resisting elements is greater than 20 percent of the plan length of those elements.

- In buildings with in-plane discontinuity and located in Seismic Zones II, the lateral drift of the building under the design lateral force shall be limited to 0.2 percent of the building height;

- Buildings with in-plane discontinuity shall not be permitted in Seismic Zones III, V and V.

v) Strength Irregularity (Weak Story)

- A weak storey is a storey whose lateral strength is less than that of the storey above.

vi) Floating Columns

- Such columns are likely to cause concentrated damage in the structure.

- This feature is undesirable, and hence should be prohibited, if it is part of or supporting the primary lateral load resisting system.

vii) Modes of Oscillation

- The first three modes together contribute at least 65 percent mass participation factor in each principal plan direction.

- The fundamental lateral natural periods of the building in the two principal plan directions are away from each other by at least 10 percent of the larger value.

NOTE- This provision is not applicable to buildings with large podiums in the lower story's.

Step-9 check lateral stability system

Dual System

- Buildings with dual system consist of moment resisting frames and structural walls (or of moment resisting frames and bracings) such that both of the following

conditions are valid:

- Two systems are designed to resist total design lateral force in proportion to their lateral stiffness, considering interaction of two systems at all floor levels; and
- Moment resisting frames are designed to resist independently at least 25 percent of the design base shear.

STEP-10 APPLY WIND ANALYSIS

- The maximum deflection against wind allowed is H/500.
- Check Base shear as per Code Provisions
- Check story drift
- If the frequency of the building is less than 1Hz then wind dynamic analysis needs to be done by considering the guest factor.
- Check Wind Tunnel Test as per CTBUH Code Provision
- Considered R-Wind for Wind Simulation as per CTBUH Code Provision
- Performance-based Wind Design (PBWD) of Building Structures

STEP-11 CHECK P-DELTA ANALYSIS

- Second order analysis is used in the analysis where the lateral sway caused by secondary moments are important. Second order effects or P delta effect is used in those structures in which the structural elements are subjected under the effect of external compressive loads.

STEP-12 APPLY CREEP & SHRINKAGE ANALYSIS

- Check the elastic deflection
- Check the creep deflection
- Axial shortening of columns effects

STEP-13 APPLY AUTOSEQUENCE ANALYSIS

- Check the performance of a structure with the various loads applied in a single step differs significantly from that when the loads are applied in stages.
- Check the deflection

STEP-14 APPLY BUCKLING ANALYSIS

- Calculates buckling modes
- Check the buckling factor
- Stable the structure, buckling occurs lateral to the direction of load application which is the suddenly failure.

Step-15 apply non linear static analysis (pushover analysis)

Nonlinear Static (Pushover) Analysis Procedures

- The Capacity Spectrum Method (ATC 40, FEMA 356)
- The Displacement Coefficient Method (ATC 40, FEMA 356)
- The FEMA 440 Improved NSPs (FEMA 440, ASCE 41-06/13)

Determine the capacity or pushover curve total seismic force against deflection

Determine the :

- Capacity curve
- Demand Curve
- Performance point.

Check the effects of Material Nonlinearities (Effect of material properties)

- Cracking in concrete
- Crushing in concrete
- Yielding in steel
- Creep and shrinkage (time-dependent)

- Check the performance of the structures where is subjected to gravity loading and a monotonic displacement-controlled lateral load pattern which continuously increases through elastic and inelastic behavior until an ultimate condition is reached.

Step-16 Time History Analysis

- Scaling base shear performed.
- To determine the structure response with respect to ground motion records.
- Check the combined story response plots (to display the max. drift or displacement or shear over turning moments)
- Check the response spectrum curves
- Check the structure performance with respect to ground motion records.

Step-17 Apply Vibration Analysis

- Considered this building in an area of high seismicity and high winds needs to be carefully designed to ensure the adequate stiffen a building in order to reduce the dynamic response under wind loading. However, this has the effect of increasing the seismic base shear that is attracted. By adding supplementary damping to the structure, it is possible to reduce the flexural stiffness of the building to minimize seismic base shear, and at the same time control the wind response.

Suitable dampers to be considered :

- Viscous Dampers
- Base Isolation
- Tuned Mass Damper etc.

Checks to be taken:

- Story shear
- Base shear
- Story drift.
- Point Displacement
- Mode shapes

- To make structure stable against seismic & wind and reduces reinforcement hence make structure economical.

STEP-18 PERFORMANCE BASED DESIGN (PBD ANALYSIS)

- Actual forces that appear on structures during earthquakes are much higher than the design forces specified in the Standard.

— Design philosophy adopted

Minor shaking (<DBE)	No damage to SE	No damage to NSE
Moderate shaking (DBE)	No damage to SE	Some damage to NSE
Severe shaking (MCE)	No collapse	Damage to NSE

Check the effects of Material Nonlinearities (Effect of material properties)

- Cracking in concrete
- Crushing in concrete
- Yielding in steel
- Creep and shrinkage (time-dependent)

- Check the structure performance with respect to ground motion records.

STEP-19 MEMBERS & CONNECTIONS DESIGN

- All the structures elements need to well within the strength design limit criteria.

- Members and connections of reinforced and prestressed concrete structures shall be designed (as per code provision) such that premature failure does not occur due to shear or bond.

- Some provisions for appropriate ductile detailing of RC members are code provision.

- To resist large displacement during EQs without collapse.

STEP-20 FOUNDATION SYSTEM ANALYSIS AND DESIGN

- Check the materials concrete, rebar, steel etc prominently these are Concrete fck & Steel fy, fu
- Check section sizes of footings as per reaction forces.
- Consider suitable foundation system as per reaction forces & SBC of soil
- Check load combinations
- Check deflections
- Elastic Deflection
- Immediate cracked deflection
- Long-term cracked deflection accounting for creep and shrinkage
- Total deflection
- Check soil pressure
- Check punching shear
- Check crack width
- Check UPLIFT by Non linear analysis & Overturning
- Design the foundation system as per code provisions

STEP-21 SLAB SYSTEM ANALYSIS AND DESIGN

- Check the materials concrete, rebar, steel etc. prominently these are Concrete fck & Steel fy, fu
- Check section sizes of slabs as per reaction forces.
- Consider suitable slab system as per loadings (waffle, ribbed, flat, solid slabs etc.)
- Check load combinations
- Check deflections
- Elastic Deflection
- Immediate cracked deflection
- Long-term cracked deflection accounting for creep and shrinkage
- Total deflection
- Check punching shear
- Check crack width
- Design the slab system as per code provisions

Step-22 CSI detailer to structure detailing and drawing generations

- To extract drawings
- Detailed views
- Bills of materials (BOM)
- Bills of quantities (BOQ)
- Reinforcement Details for slabs, beams, columns, and walls, grouping similar beams and columns in the process.
- Rebar Schedules.
- Rebar Cages
- Detailing Rules

Step-23 Revit Structure And Tekla Structure (Optional)

- To modeling, detailing and extract all sheets like erection , shop drawings & fabrications etc.
- To extract CNC data as per need for fabricator.
- In Revit structure import dxr file perform the detailing and extract the sheets.
- In Tekla structure import by extension integrator perform the detailing and extract the sheets.
- In RCDC import MDB file perform the design and detailing and extract the sheets structure can also optimize in RCDC.

Building Materials

Material	Properties
Rebar	Fyt240, Fy400
Grade of Concrete	Fc 35MPa, Fc50MPa
Non Structural Walls	Light Weight Block
Glazing Panel	Glass Sheets

Section Properties

Frame Section	**Beam**	400mmx550mm
		650mmx900mm
	Column	750mmx900mm
		1000mmx1300mm
		1400mmx1400mm
Wall Section **Shear wall & Core walls**	**Shear Wall**	250mm thick
		300mm thick
Slab Section	**General Slab**	250 thick mm
	Parking Slab	225 thick mm
	Staircase Slab	230 thick mm
	Service Slabs	220 thick mm

Slab Loadings

[ASCE Table C3.1-1 b Minimum Design Dead Loads (Kn/m2)

Loads on Slabs	Dead Load ASCE Table C3.1-1 b	Live Load ASCE Table 4.3-1
General Slab Asphalt block (51 mm), 13-mm mortar	1.44KN/m^2	3KN/m^2
Staircase Slab Solid flat tile on 25-mm mortar base	1.10KN/m^2	3KN/m^2
Parking Slab Solid concrete fill 30mm	1.2KN/m^2	4KN/m^2
Terrace Ceramic or quarry tile (19mm) on 13 mm mortar bed	1.53KN/m^2	1.5KN/m^2
Bath/WC Solid flat tile on 25 mm mortar base	1.10KN/m^2	2KN/m^2
Service Slab Marble & Mortar on stone-concrete fill	1.58KN/m^2	2.0KN/m^2
Other Slabs Terrazo (25mm) on stone-concrete fill	1.53KN/m^2	2.0KN/m^2
OHT	16.0KN/m^2	0.75KN/m^2
LMR-1	7.0KN/m^2	0.75KN/m^2
LMR-2	5.6KN/m^2	0.75KN/m^2

Frame Loads

ASCE Table C3.1-1 b

1.	Typical Height of the beam	75 mm
2.	Density of the Block work including finishing	10KN/m3
3.	Thickness of the block work	200 mm
4.	External Wall Load Calculation	(Thickness of wall) x (Height of Floor-Depth of Beam) x (Density of Material) = 0.2 x (4.0-0.75) x 10 = 6.5 KN/ m
5.	Internal Wall Load Calculation	(Thickness of wall) x (Height of Floor-Depth of Beam) x (Density of Material) = 0.15 x (4.0-0.75) x 10 = 4.8 KN/ m
6.	Parapet Wall Load Calculation	(Thickness of wall) x (Height of Floor-Depth of Beam) x (Density of Material) = 0.2 x 1.0 x 10 = 2.0 KN/ m
7.	Loads on projections which is window glazing	Loads on projections which is window glazing Loads= 5KN/m

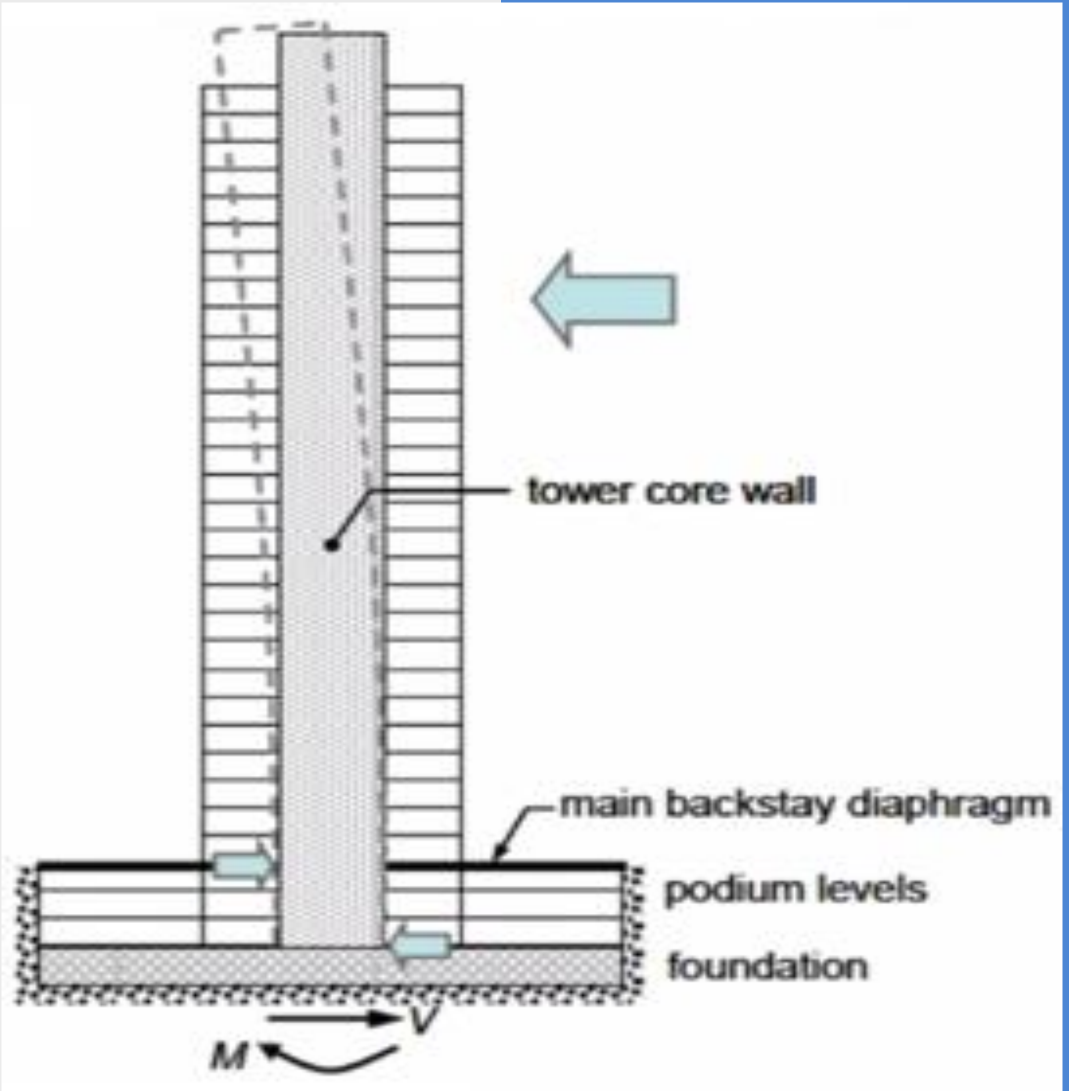
Podium & Backstay effects

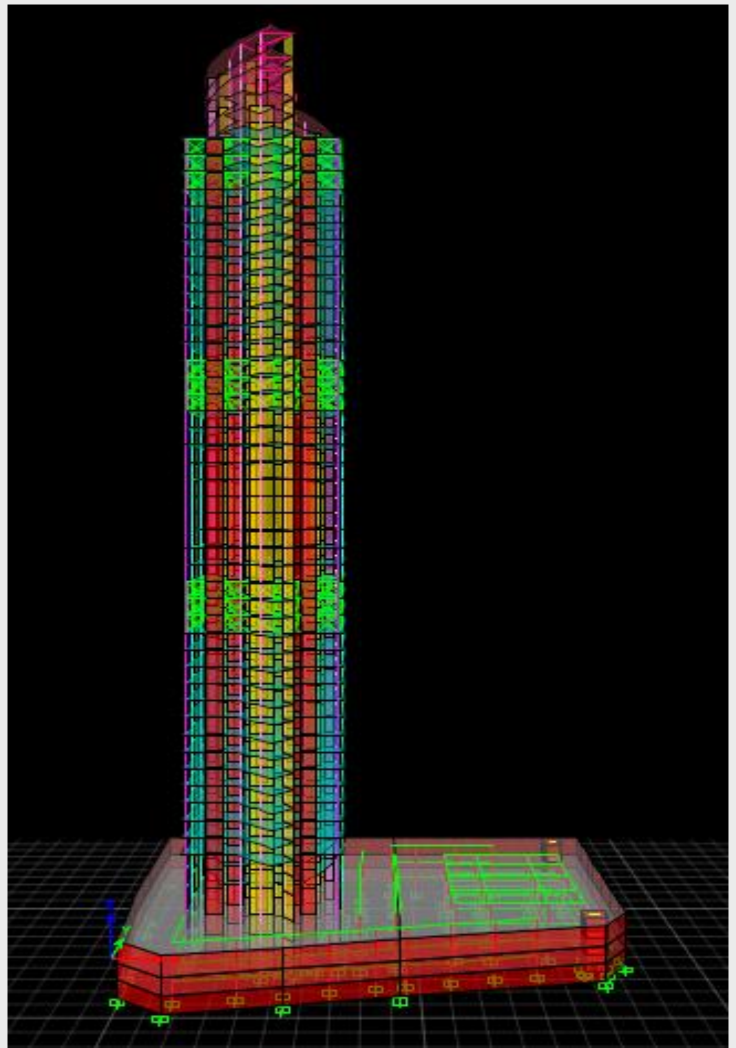
- Any lower part of a tall building structure that contains additional seismic-force resisting elements in comparison to the tower above, can be considered a podium.

Elements of podium
- The reinforced concrete perimeter walls at the below-grade levels.
- Floor diaphragms at the below-grade levels.
- Supporting soils (Passive Earth pressure).

- Backstay effect is the transfer of lateral forces from the seismic-force resisting elements in the tower into additional elements that exist within the podium.

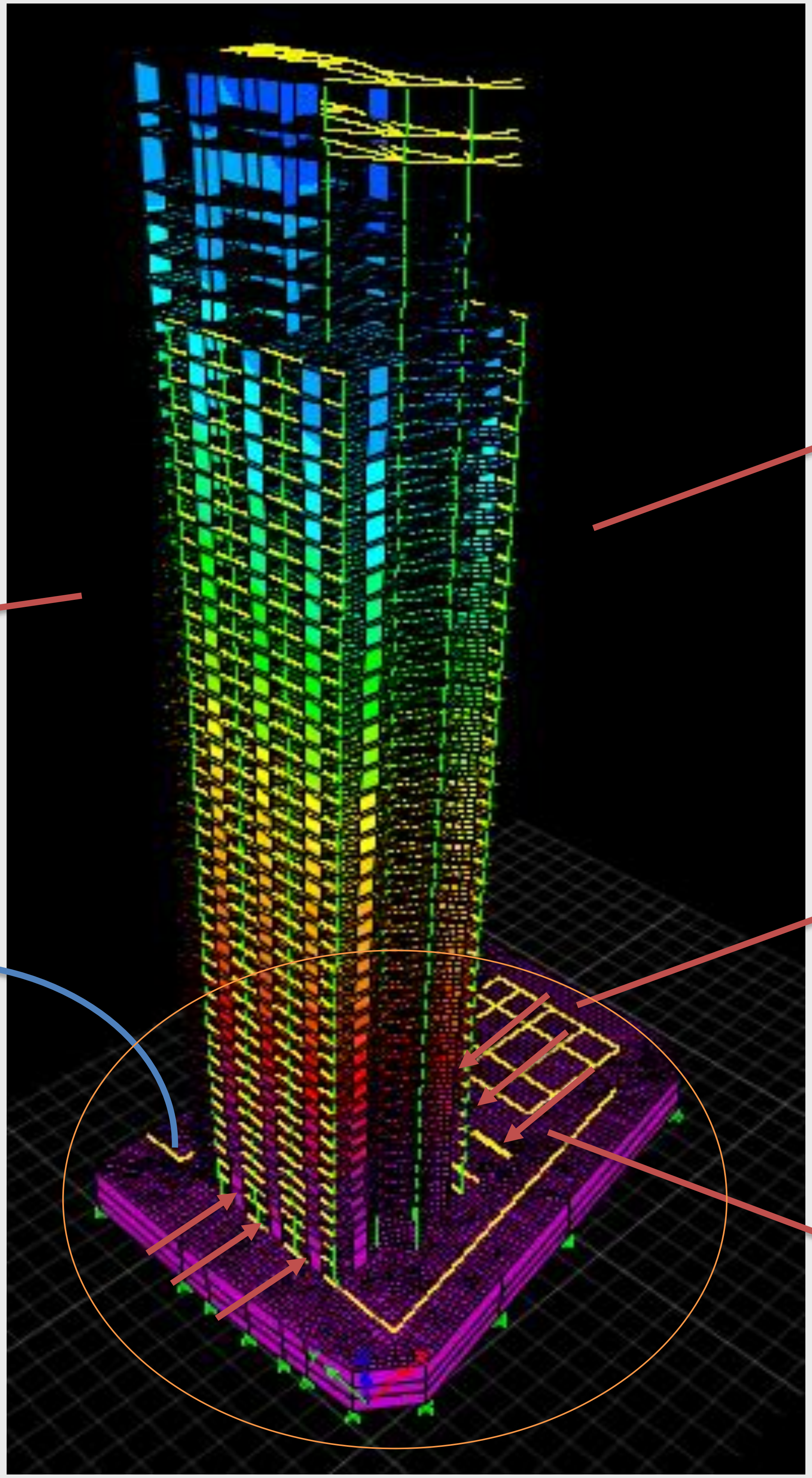
- This effect imposes different boundary conditions (restraints) on the tower





Reference:
Modeling and acceptance criteria for seismic design and analysis of tall buildings", PEER/ATC 72-1:2010

Podium & Backstay effects



Movement of Main Tower

Backstay Effect

Trying to resist lateral movement of tower

Causes increase in stresses in podium elements

Two critical stage of cracking is considered

Resistance by podium floor

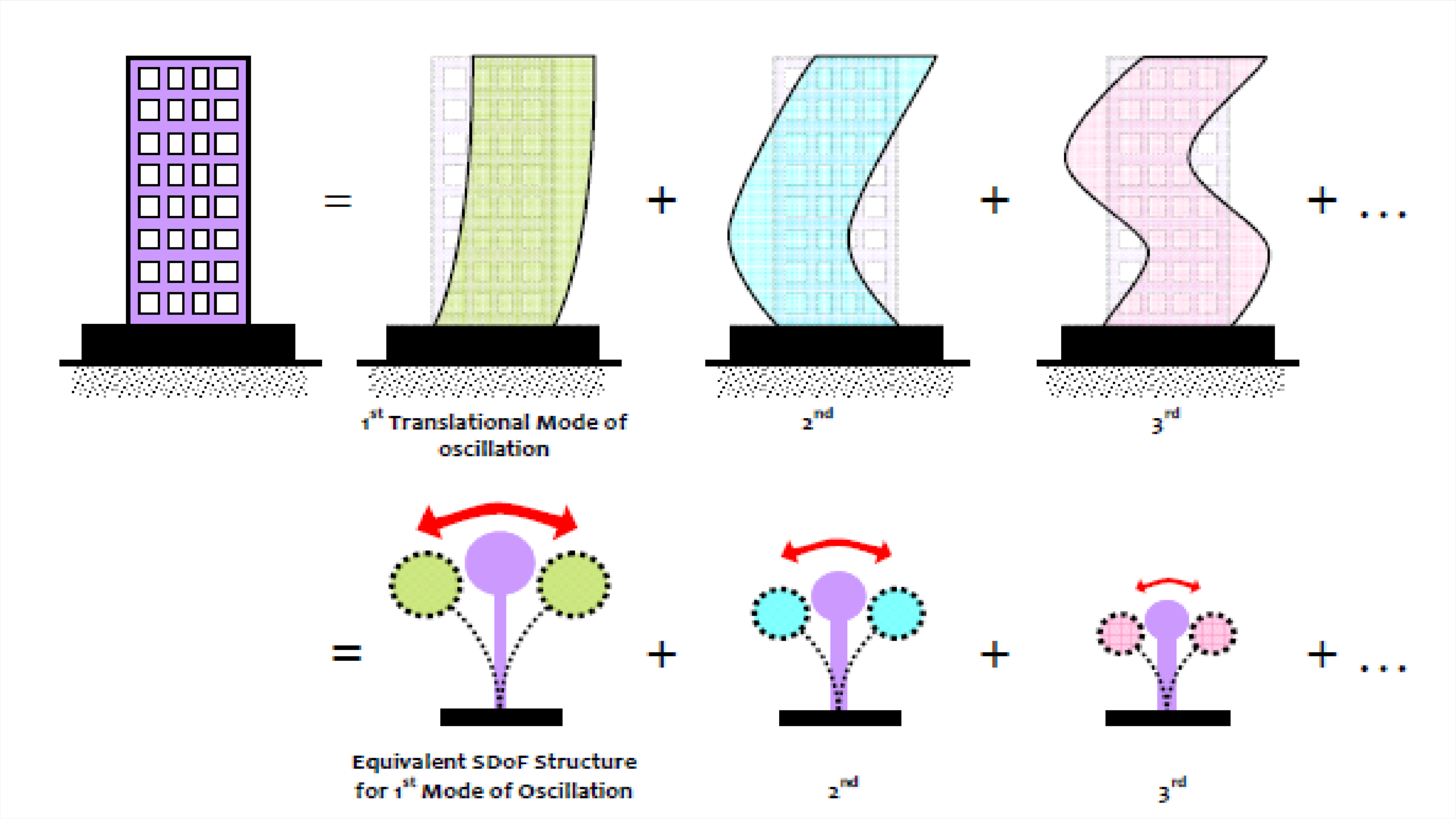
Modal Analysis

- The modal analysis determines the inherent natural frequencies of vibration
- Each natural frequency is related to a time period and a mode shape
- Time Period is the time it takes to complete one cycle of vibration
- The Mode Shape is normalized deformation pattern
- The number of Modes is typically equal to the number of Degrees of Freedom
- The Time Period and Mode Shapes are inherent properties of the structure and do not depend on the applied loads
- The Modal Analysis should be run before applying loads any other analysis to check the model and to understand the response of the structure.

In structural engineering, modal analysis uses the overall mass and stiffness of a structure to find the various periods at which it will natural time period.

Modal analysis helps to determine the vibration characteristics(natural frequencies and mode shapes) of a structure or component, showing the movement of different parts of the structure.

Modal Analysis



Modal Analysis Results

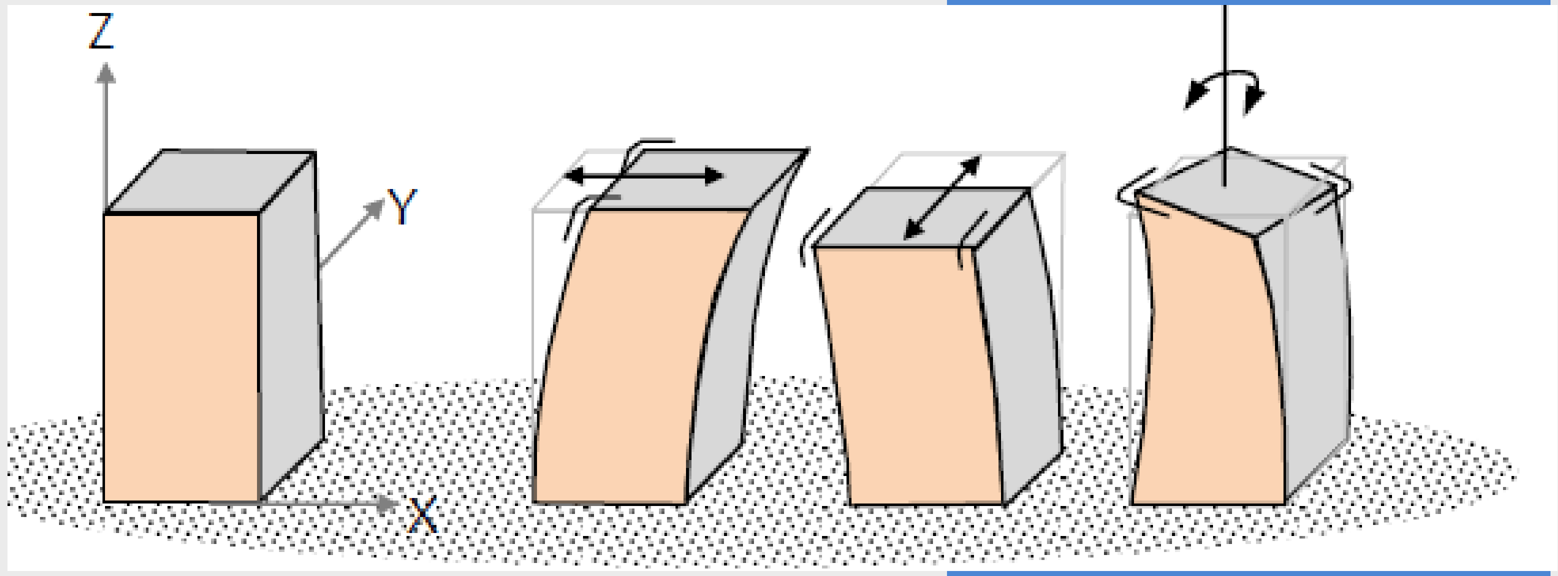
- Guidelines for fundamental time-period be 0.1 * N (N= No of levels). [ASCE Cl: 12.8-8]
  Therefore 0.1 x 53 = 5.3 < 4.6, Hence OK

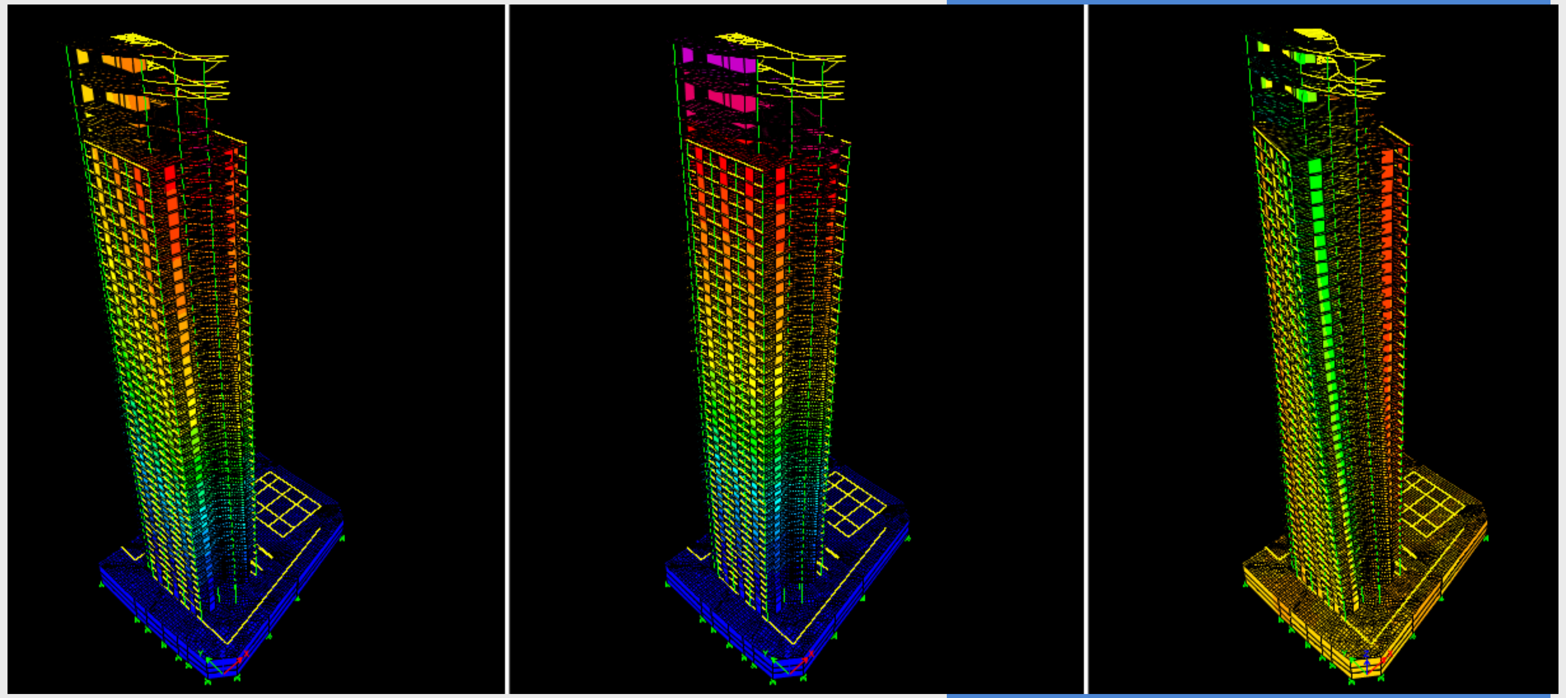
  THREE FUNDAMENTAL TRANSLATIONAL NATURAL PERIODS,

  TX1 = ASSOCIATED WITH ITS HORIZONTAL TRANSLATIONAL OSCILLATION ALONG X

  TY1 = ASSOCIATED WITH ITS HORIZONTAL TRANSLATIONAL OSCILLATION ALONG Y
  TZ1 = ASSOCIATED WITH ITS ROTATIONAL OSCILLATION ALONG Z

- The submission of all the modes at least should be cross 90%, , Hence OK [ASCE Cl: 12.9.1.1]





Case	Mode	Period (sec)	UX	UY	SUM UX	SUMUY	RZ	SUMRZ
Modal	1	4.603	0.3307	0.2524	0.3307	0.2524	0.0347	0.0347
Modal	2	3.899	0.1436	0.2959	0.4743	0.5483	0.0331	0.0678
Modal	3	3.481	0.1724	0.0505	0.6467	0.5988	0.356	0.3238
Modal	60	0.039	0.0077	0.0027	0.9998	0.9999	0.0002	0.9977

Gravity Deflection

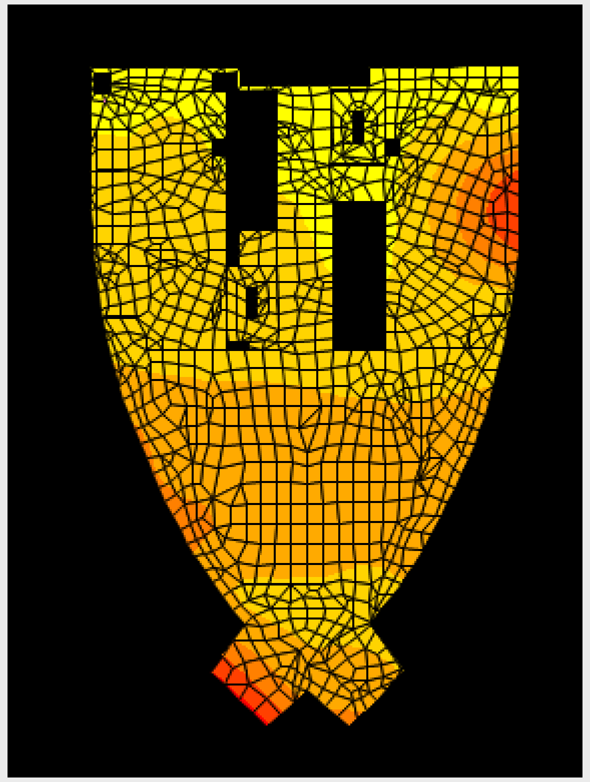
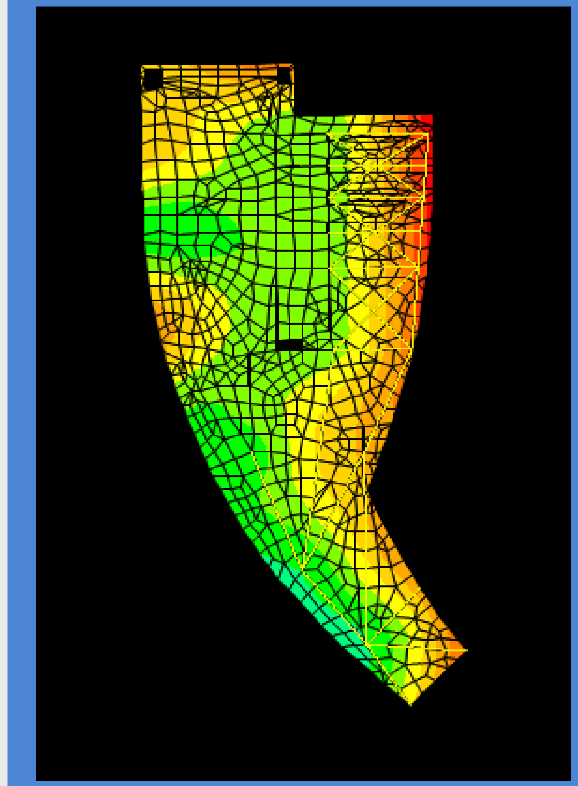
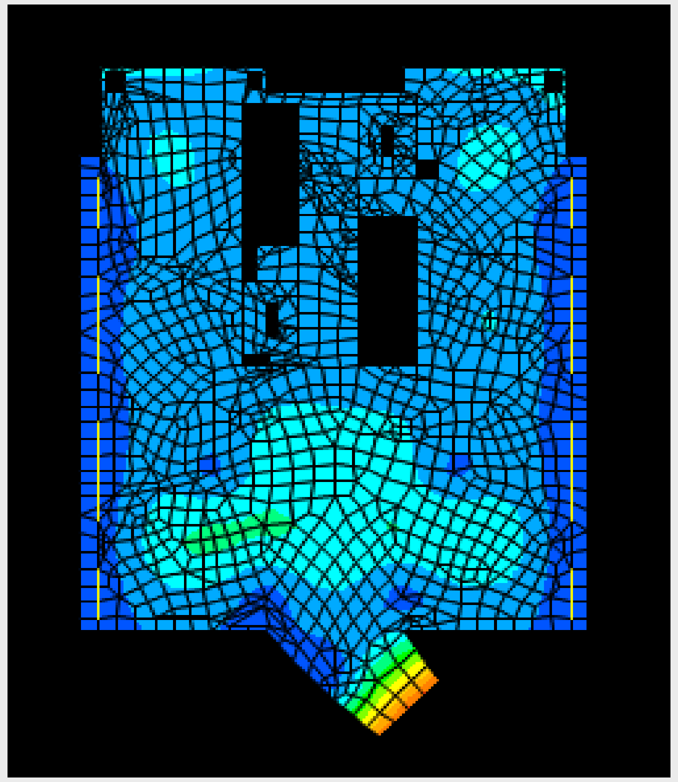
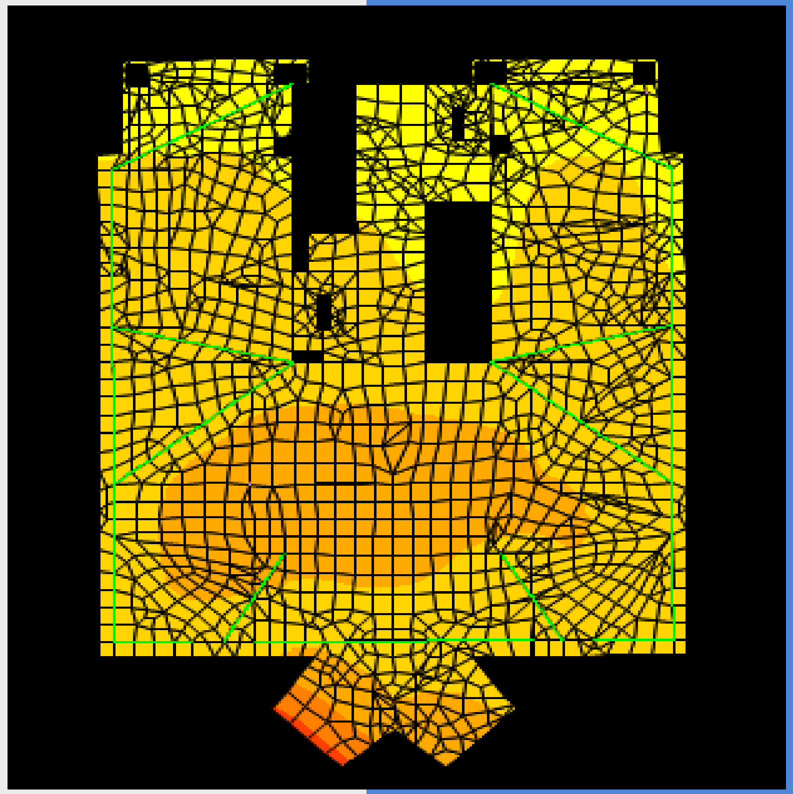
- Max. allowed span/350 (or) 20mm

- Maximum deflection= 13.57mm
- Near Column deflection = 10.127mm
- Axial shortening = 3.443mm

- Max. Deflection Axial shortening x Creep coefficient = 3.443 x 3 = 10.329mm

- Span/350= 5.425 x 1000/350 = 15.5mm

Therefore maximum deflection is less

Earthquake Design

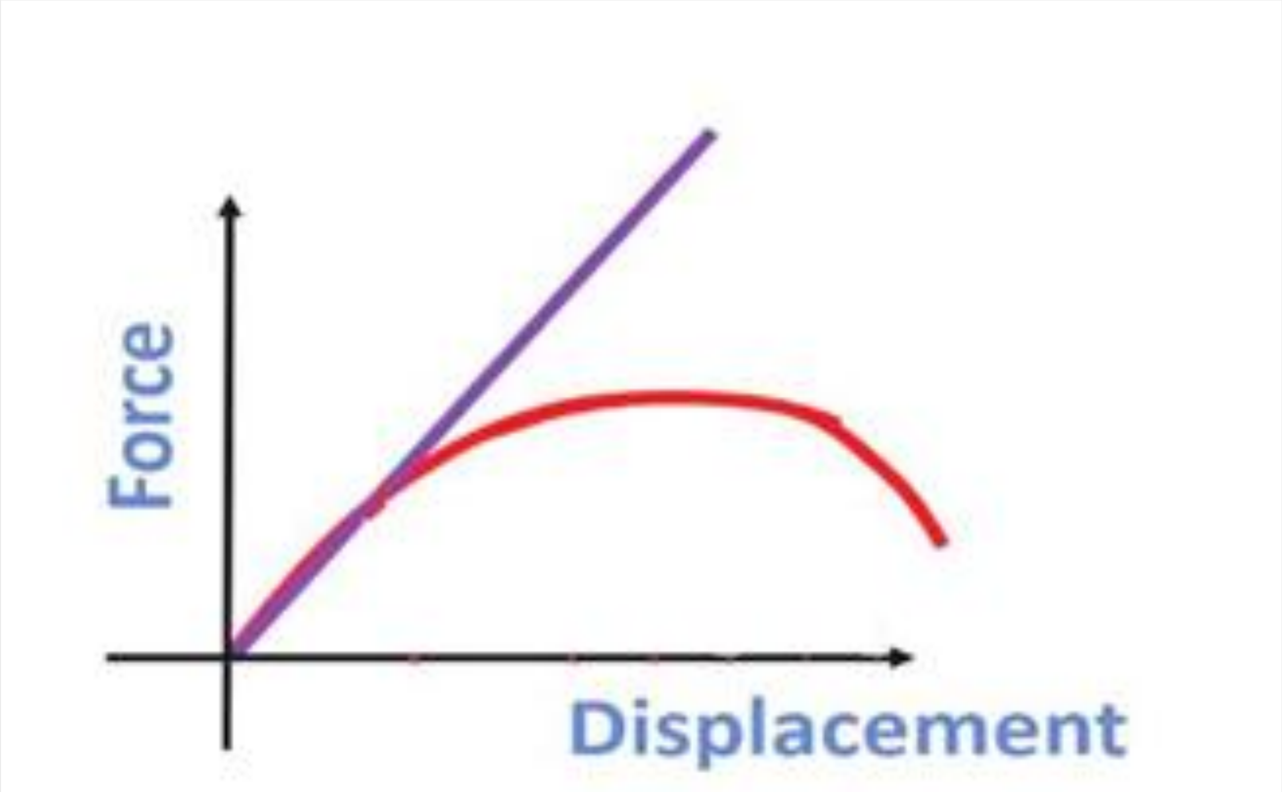
SEISMIC ANALYSIS

LINEAR ANALYSIS

- Equivalent Static Analysis
- Dynamic Analysis
  - Response spectrum analysis
  - Modal time history analysis

NON-LINEAR ANALYSIS

- Nonlinear Static Analysis (Pushover analysis)
- Nonlinear Dynamic or Time History Analysis

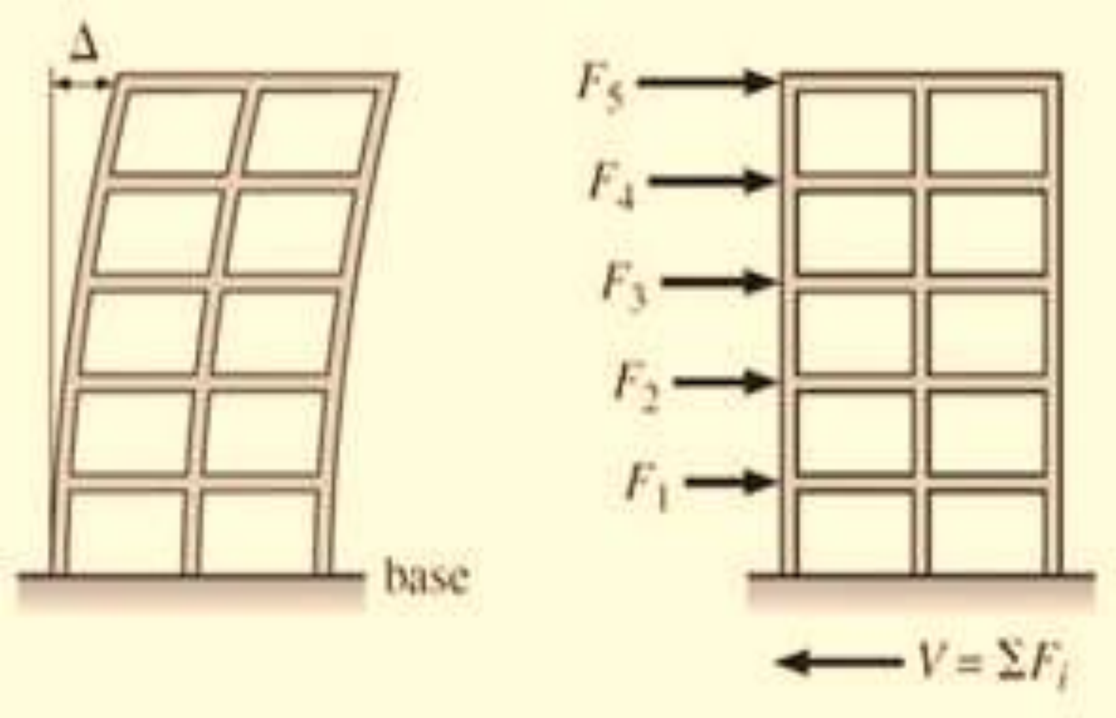


Earthquake Design as per ASCE7-16

Regular < 73m Static
Regular > 73m Dynamic
Irregular < 21m Static
Irregular > 21m Dynamic
Irregular number of stories 5 and above Dynamic
Eccentricity between center of mass and rigidity > 20% Dynamic

Static Analysis Base Shear= V = Cs.W ASCE (Cl. 12.8-1)

Where,
Cs= Seismic response coefficient
W= Seismic weight of the building



Specification

Site class	: B (S1=0.2 & Ss=0.5)
Long-Period Transition Period	: 8
Site coefficient, Fa	:0.9
Site coefficient, Fv	:0.8
Calculated coefficients	
SDS = (2/3) * Fa * Ss	:0.12
SD1 = (2/3) * Fv * S1	:0.2667
Response Modification, R	: 6 (Table 12.2-1)
System Over strength, Omega	: 2.5 (Table 12.2-1)
Deflection Amplification, Cd	: 5 (Table 12.2-1)
Occupancy Importance, I	: 1.25 (Table 1.5-1)
Ct(ft)	:0.02: 0.75 (As per ASCE (12.8-2)

Earthquake Design
Stability Checks

ASCE 7-16 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern EQX according to ASCE 7-16, as calculated by ETABS.

Direction and Eccentricity

Direction = X

Structural Period

Period Calculation Method = Program Calculated

Coefficient, Ct [ASCE Table 12.8-2]	C_t = 0.02ft
Coefficient, x [ASCE Table 12.8-2]	x = 0.75
Structure Height Above Base, h_n	h_n = 643.37 ft
Long-Period Transition Period, T_L [ASCE 11.4.5]	T_L = 8 sec

Factors and Coefficients

Response Modification Factor, R [ASCE Table 12.2-1]	R = 6
System Overstrength Factor, Ω0 [ASCE Table 12.2-1]	Ω_0 = 2.5
Deflection Amplification Factor, Cd [ASCE Table 12.2-1]	C_d = 5
Importance Factor, I [ASCE Table 1.5-2]	I = 1.25
Ss and S1 Source = 0.75	
Mapped MCE Spectral Response Acceleration, Ss[ASCE 11.4.2]	S_s = 2.29g
Mapped MCE Spectral Response Acceleration, S1 [ASCE 11.4.2]	S_1 = 0.869g
Site Class [ASCE Table 20.3-1] = B - Rock	
Site Coefficient, Fa [ASCE Table 11.4-1]	Fa = 0.9
Site Coefficient, Fv [ASCE Table 11.4-2]	Fv = 0.8

Seismic Response

MCE Spectral Response Acceleration,
S_{MS} [ASCE 11.4.4, Eq. 11.4-1] — $S_{MS} = F_a S_S$ — $S_{MS} = 2.061g$

MCE Spectral Response Acceleration,
S_{M1} [ASCE 11.4.4, Eq. 11.4-2] — $S_{M1} = F_v S_1$ — $S_{M1} = 0.6952g$

Design Spectral Response Acceleration,
S_{DS} [ASCE 11.4.5, Eq. 11.4-3] — $S_{DS} = 2/3\, S_{MS}$ — $S_{DS} = 1.374g$

Design Spectral Response Acceleration,
S_{D1} [ASCE 11.4.5, Eq. 11.4-4] — $S_{D1} = 2/3\, S_{M1}$ — $S_{D1} = 0.463467g$

Equivalent Lateral Forces

Seismic Response Coefficient, CS [ASCE 12.8.1.1, Eq. 12.8-2]

$$C_S = \frac{S_{DS}}{\left(\frac{R}{I}\right)}$$

[ASCE 12.8.1.1, Eq. 12.8-3]

$$C_{S,max} = \frac{S_{D1}}{T\left(\frac{R}{I}\right)}$$

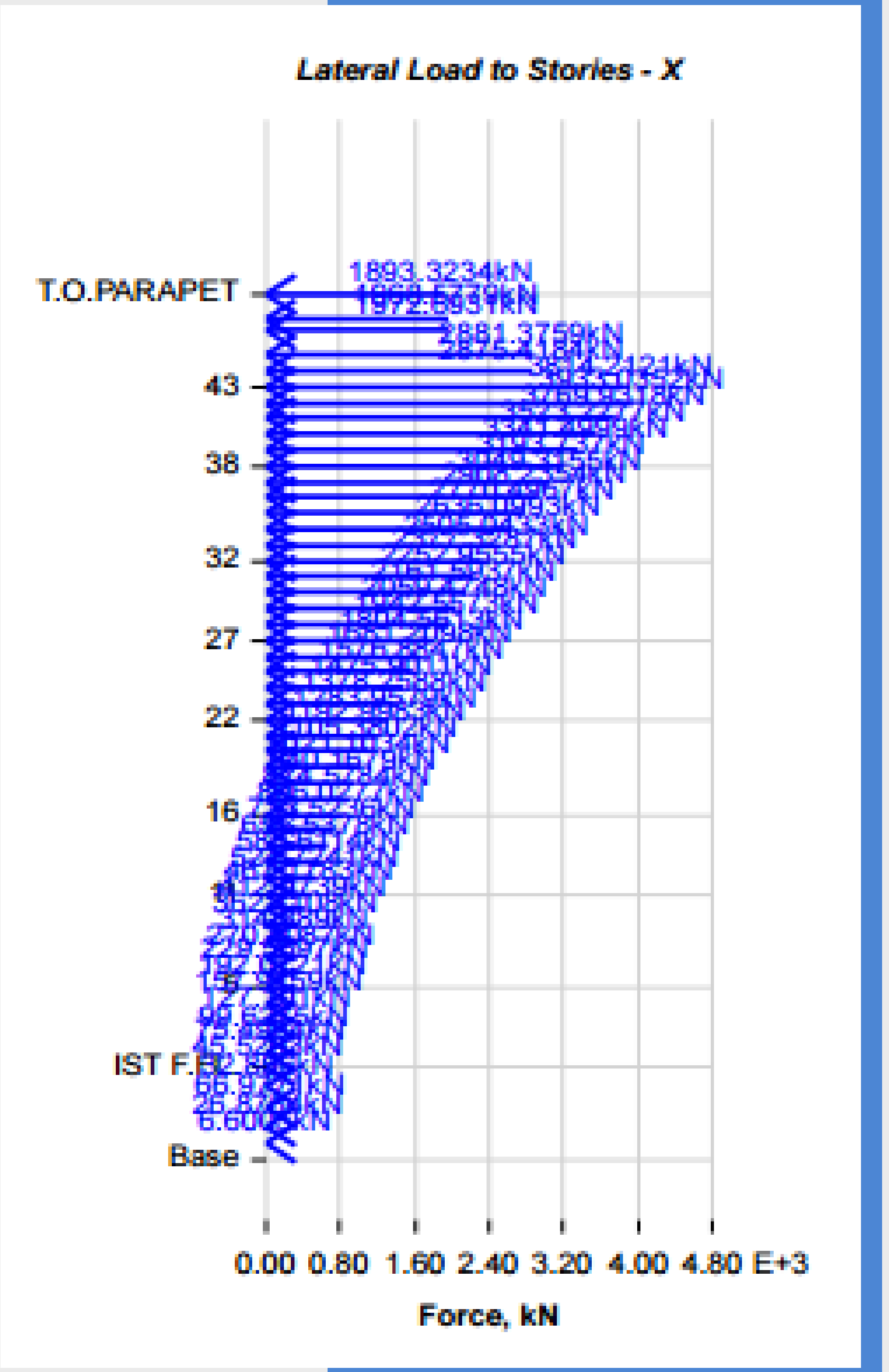
[ASCE 12.8.1.1, Eq. 12.8-5]

$$C_{S,min} = max\ (0.044\ S_{DS}\ I, 0.01\ ) = 0.07557$$

[ASCE 12.8.1.1, Eq. 12.8-6]

$$C_{S,min} = 0.5 \frac{S_1}{\left(\frac{R}{I}\right)} \text{ for } S_1 = 0.6g$$

$$C_{S,min} \le C_s \le C_{S,max}$$



Applied Story Forces

Calculated Base Shear

Direction	Period Used (sec)	Cs	W (KN)	V b (KN)
X	3.577	0.090521	86539.2207	78358.3836

Earthquake Design

Stability Checks

ASCE 7-16 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern EQY according to ASCE 7-16, as calculated by ETABS.

Direction and Eccentricity

Direction = Y

Structural Period

Period Calculation Method = Program Calculated

Coefficient, Ct [ASCE Table 12.8-2]	Ct = 0.02ft
Coefficient, x [ASCE Table 12.8-2]	x = 0.75
Structure Height Above Base, h_n	hn = 643.37 ft
Long-Period Transition Period, TL [ASCE 11.4.5]	TL = 8 sec

Factors and Coefficients

Response Modification Factor, R [ASCE Table 12.2-1]	R = 6
System Overstrength Factor, Ω_0 [ASCE Table 12.2-1]	Ω_0 = 2.5
Deflection Amplification Factor, C_d [ASCE Table 12.2-1]	C_d = 5
Importance Factor, I [ASCE Table 1.5-2]	I = 1.25
Ss and S_1 Source = 0.75	
Mapped MCE Spectral Response Acceleration, Ss[ASCE 11.4.2]	Ss = 2.29g
Mapped MCE Spectral Response Acceleration, S_1 [ASCE 11.4.2]	S_1 = 0.869g
Site Class [ASCE Table 20.3-1] = B - Rock	
Site Coefficient, Fa [ASCE Table 11.4-1]	Fa = 0.9
Site Coefficient, Fv [ASCE Table 11.4-2]	Fv = 0.8

Seismic Response

MCE Spectral Response Acceleration,
S_{MS} [ASCE 11.4.4, Eq. 11.4-1] $S_{MS} = F_a S_S$ $S_{MS} = 2.061g$

MCE Spectral Response Acceleration,
S_{M1} [ASCE 11.4.4, Eq. 11.4-2] $S_{M1} = F_v S_1$ $S_{M1} = 0.6952g$

Design Spectral Response Acceleration,
S_{DS} [ASCE 11.4.5, Eq. 11.4-3] $S_{DS} = 2/3\ S_{MS}$ $S_{DS} = 1.374g$

Design Spectral Response Acceleration,
S_{D1} [ASCE 11.4.5, Eq. 11.4-4] $S_{D1} = 2/3\ S_{M1}$ $S_{D1} = 0.463467g$

Equivalent Lateral Forces

Seismic Response Coefficient, CS [ASCE 12.8.1.1, Eq. 12.8-2]

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)}$$

[ASCE 12.8.1.1, Eq. 12.8-3]

$$C_{s,max} = \frac{S_{D1}}{T\left(\frac{R}{I}\right)}$$

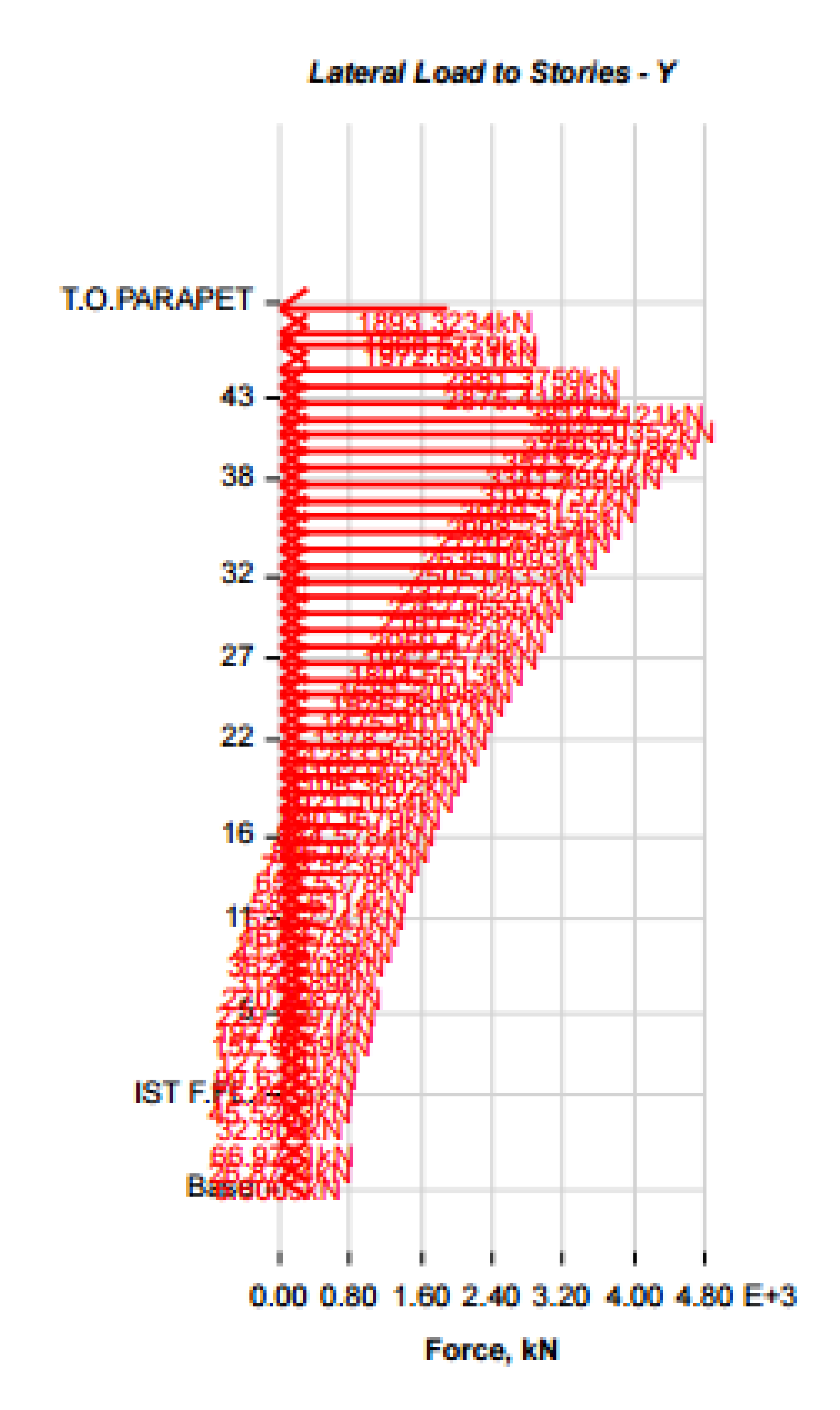
[ASCE 12.8.1.1, Eq. 12.8-5]

$$C_{s,min} = max\ (0.044\ S_{DS}\ I, 0.01\ ) = 0.07557$$

[ASCE 12.8.1.1, Eq. 12.8-6]

$$C_{s,min} = 0.5\frac{S_1}{\left(\frac{R}{I}\right)}\ for\ S_1 = 0.6g$$

$$C_{s,min} \le C_s \le C_{s,max}$$



Applied Story Forces

Calculated Base Shear

Direction	Period Used (sec)	Cs	W (KN)	V b (KN)
Y	3.577	0.090521	86539.2207	78358.3836

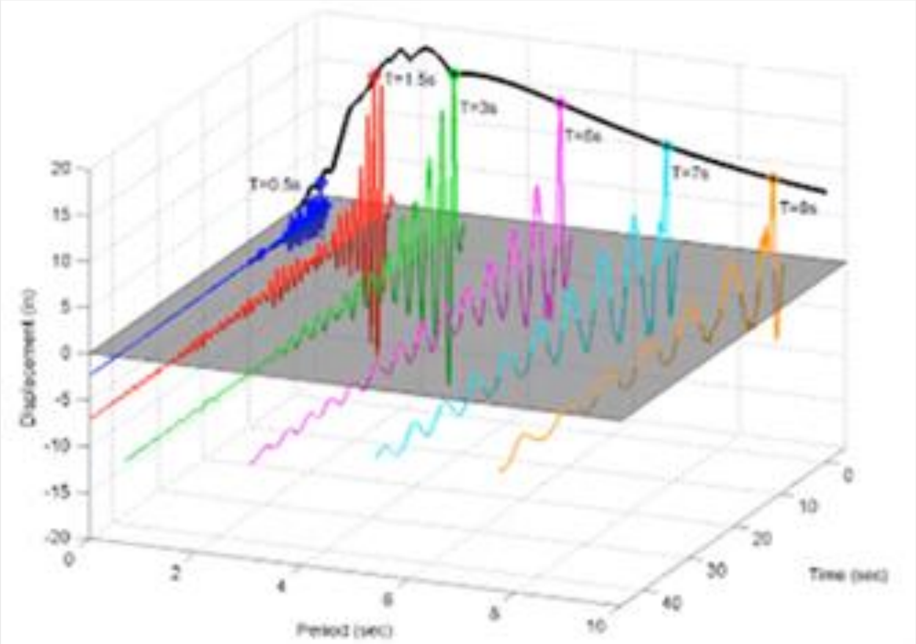
Linear Dynamic Analysis

These methods take into account the ground conditions like soil type & zone sensitivity of an structure & predicts behavior of a building under the action of Earthquake forces.

Response Spectrum Method:

Simplified method of dynamic analysis for buildings [ASCE Ch: 12.9]

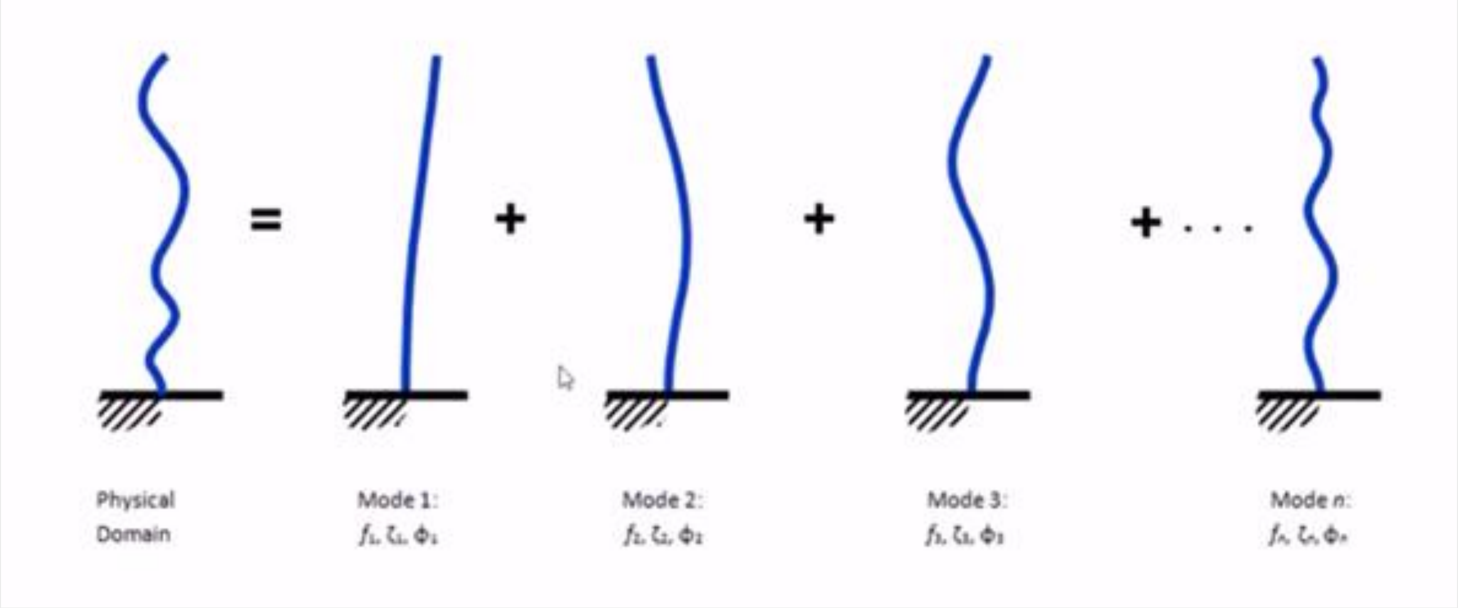
Response spectrum method can be carried out for any building using Design response or site specific response spectrum.



Dynamic Analysis Performed on High-rise Buildings, Tall and Irregular Structures

Response of structures under seismic loadings

Response (Displacements, Drifts, Forces and Stresses)



Response-spectrum analysis (RSA) is a linear-dynamic statistical analysis method which measures the contribution from each natural mode of vibration to indicate the likely maximum seismic response of an essentially elastic structure. Response-spectrum analysis provides insight into dynamic behavior by measuring pseudo-spectral acceleration, velocity, or displacement as a function of structural period for a given time history and level of damping. It is practical to envelope response spectra such that a smooth curve represents the peak response for each realization of structural period.

Response-spectrum analysis is useful for design decision-making because it relates structural type-selection to dynamic performance. Structures of shorter period experience greater acceleration, whereas those of longer period experience greater displacement. Structural performance objectives should be taken into account during preliminary design and response-spectrum analysis.

Damping and RSA:

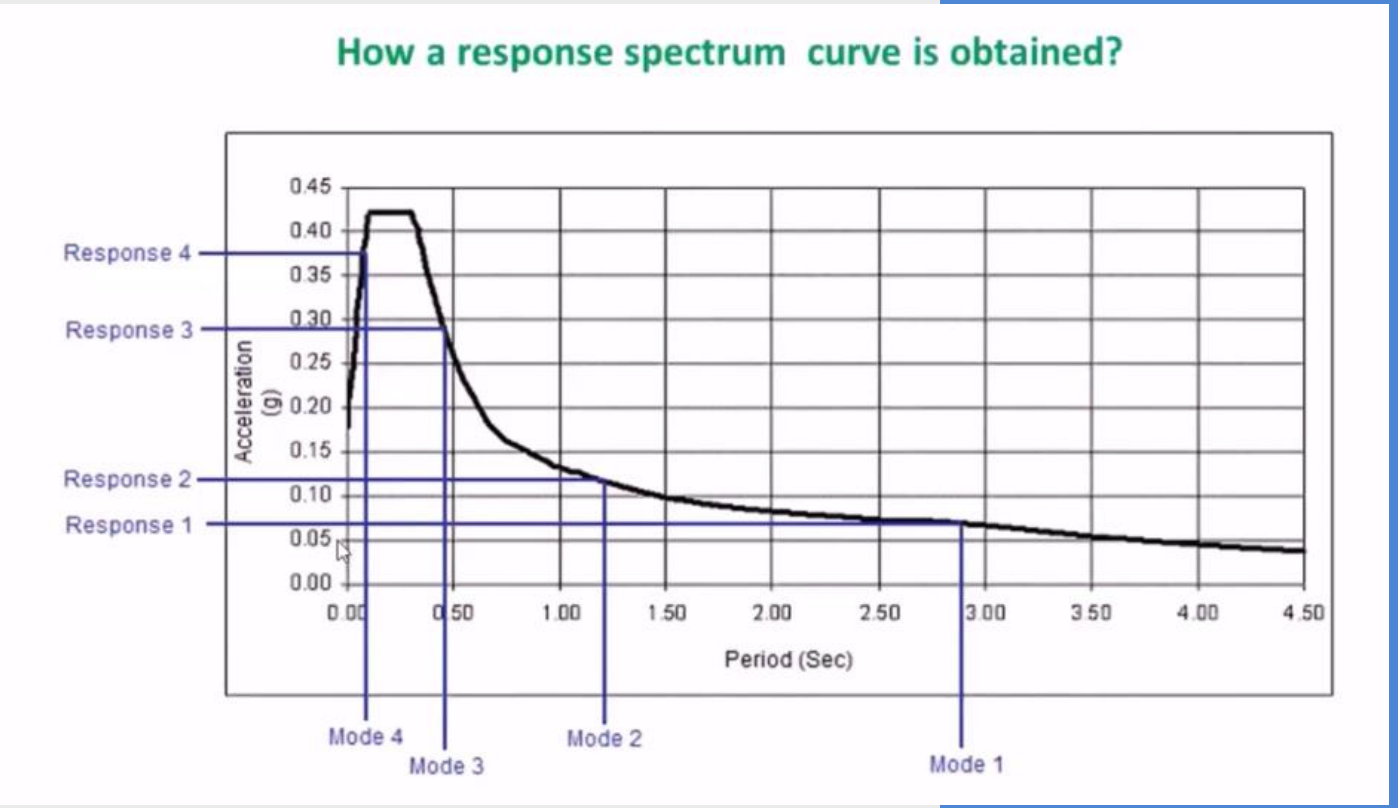
RSA provides insight into how damping affects structural response. A family of response curves may be developed with variable levels of damping. As damping increases, response spectra shifts downward.

The International Building Code (IBC) is based on 5% damping. This accounts for incidental damping from hysteretic behavior, which is not explicitly modeled during RSA.

- Provides the maximum seismic response of a structure By measuring the natural mode of vibration

Need for Response spectrum method

- The time history data is not available
- Determine the peak seismic response of a building
- Find accurate results for structural design applications



Modal Combination & Directional Combination:

Peak responses of each mode are combined to give total building response. So the responses from different mode must be combined.

We have two combination section in RSA, one is in Modal Combination, Another one in Directional Combination.

Here Combination ways for Modal Combination:

1. Absolute Sum of All the Peak Responses together (This is not very realistic as it assumes all peak responses occur at same time). Let's say we just have 2 modes to combine in a Response Spectrum analysis, From equations we end up with an equation like the following:

$$R^2 = R1^2 + 2*\varepsilon*R1*R2 + R2^2$$

Now, one extreme is where epsilon=1. This is basically just summing up the modal response:

$$R^2 = R1^2 + 2*R1*R2 + R2^2 =(R1+ R2)^2$$

$$R=R1+R2$$

$$R= |R1| + |R2| \text{ (in practice)}$$

Which is very conservative.

Modal Combination & Directional Combination:

2. Square Root Sum of the Squares (SRSS) (This can be used, however when terms are squared all values are positive and there is no correspondence between response results like that would be in time history analysis.) Let's say we just have 2 modes to combine in a Response Spectrum analysis. Now, consider ε =0. Then we have the following:

$$R^2 = R1^2 + R2^2$$

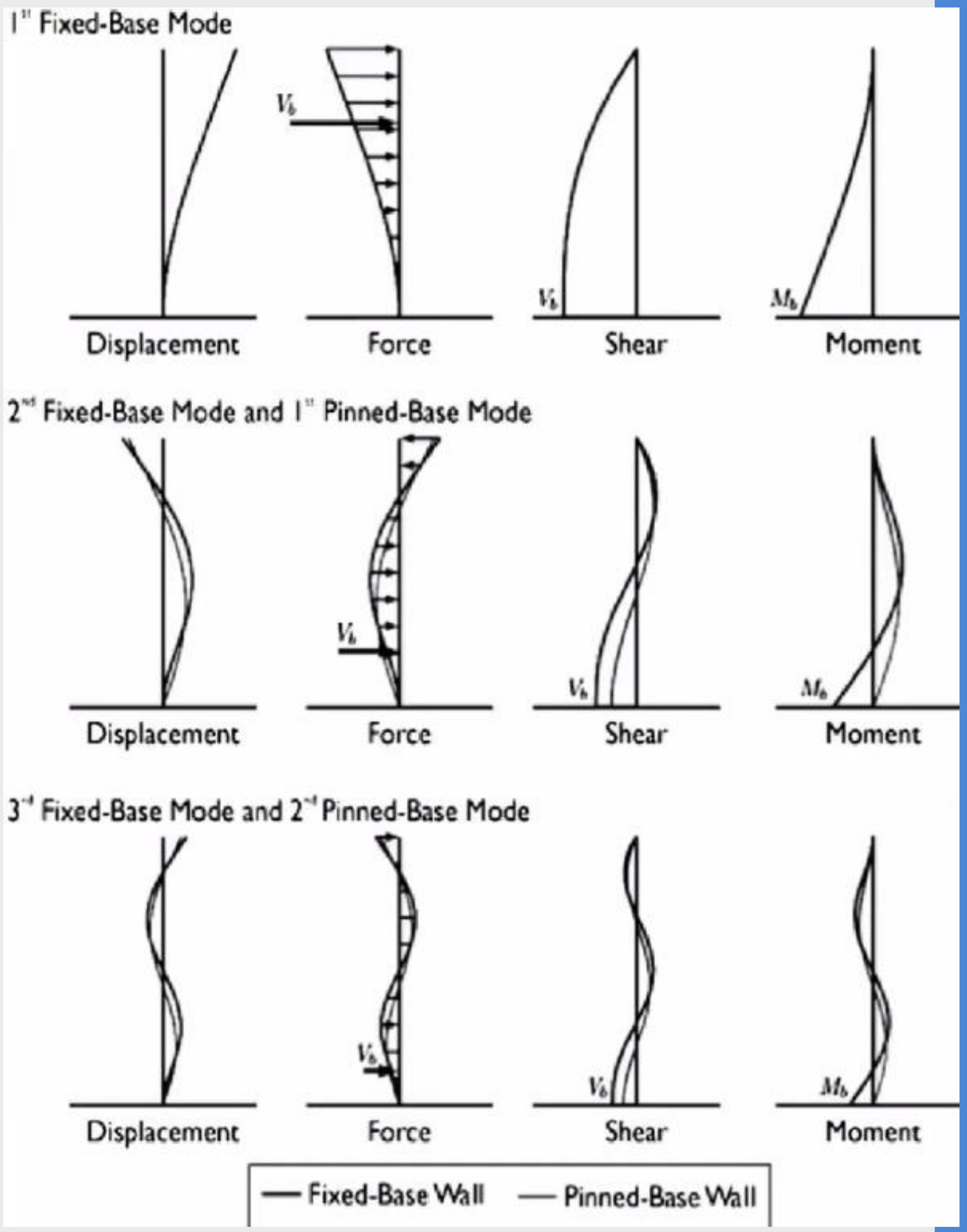
R = SRSS($R1^2 + R2^2$), This is the SRSS method. Basically, another way to think of it is that we don't account for modal interaction (interaction of modes 1 and 2) since we neglect the R1*R2 term and just take the SRSS.

- For structures with well separated frequencies, SRSS provides a good estimate of total peak response.

3. Complete Quadratic Combination(CQC) (This is commonly used). The CQC uses value of epsilon in between 0 to 1 (0 < epsilon/damping co-eff < 1) to calculate the total response, so we get somewhere in-between. In other words, CQC account for some interaction of modes when the modes are closely spaced.

- CQC is used for structures having closely spaced frequencies. This is **default & preferred** modal combination.

4. General Modal Combination technique (GMC) is used to combine the modal results. This is the same as the complete modal combination procedure described by Equation 3.31 in Gupta (1990). The GMC method takes into account the statistical coupling between closely-spaced Modes similarly to the CQC method, but also includes the correlation between modes with rigid-response content



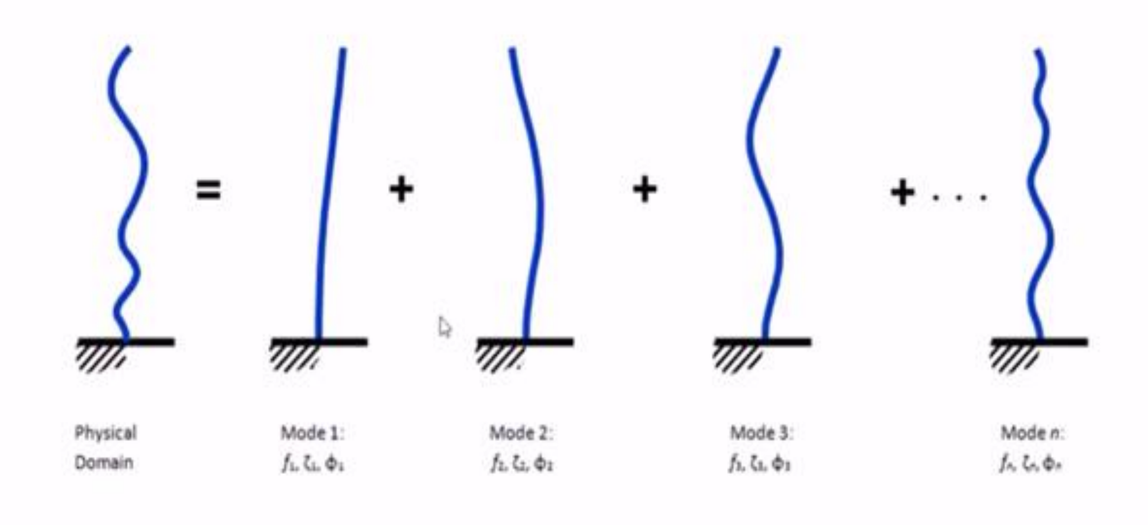
Directional Combination: For each displacement, force, or stress quantity in the structure, modal combination produces a single, positive result for each direction of acceleration. These directional values for a given response quantity are combined to produce a single, positive result. That's why we need to apply the directional combination type to specify directional scale factor (drif) to be used.

Here Combination types for Directional Combination:

1. Select, DCT as Absolute to combine the directional results by taking the sum of their absolute values. This method is usually over-conservative.

2. Select DCT as SRSS to combine the directional results by taking the square root of the sum of their squares. This method is invariant with respect to coordinate system, i.e., the results do not depend upon your choice of coordinate system when the given response-spectrum curves are the same. This is the recommended and default method for directional combination. Dy. Wilson explains that combined directional effects id be accounted for more effectively by using an alternative method in which the SRSS combination of two 100-percent spectra analyses is applied in any direction, or along either orthogonal axis. This method is valid because design forces and results are independent of the reference system used. Further, this method also accounts for independent and simultaneous ground motions which occur normal to those along the principal direction.

3. The Complete Quadratic Combination in 3 directions CQC3 method is the full expansion of SRSS method. Its generic expression accounts for the model correlation coefficient among vibration modes and the critical angle 6 between the seismic excitation and the structural axis. It offers the most critical orientation of the ground motion components.



Conclusion: From the discussion on Modal Combination Method, Directional Combination Type we can understand that CQC is default and preferable for Modal Combination and SRSS is default and preferable for Directional Combination in usual practice.

Three are different methods to combine different modal cases responses in to a response spectrum curve.

ABSSUM-Absolute Sum Method

In this method where the peak response in each model case summed up to formed a single response curve but there is unlikely to happen as a peak response curve but there is unlikely to happen as a peak response model case occurring a different time-period.

SRSS-Square Root of Sum of Squares

In this method we will give the almost accurate response of the structure but the main drawback of this method is that it cannot give the correct result if the frequency is very closely to space.

CQC-Complete Quadratic Combination

In this method we will give correct response even the frequency are very closely to space.

We have to adopt CQC method.

Seismic Dynamic Base Shear

If Building is irregular & Height is more than >21 m than we consider dynamic base shear

Building height is 185m, so required to do dynamic base shear analysis.

Scale factor = (Ex / Spec x) x Existing Scale factor.

As per ASCE 7-16 FX or FY of RS max. > = 0.85 x calculated base shear (EX or EY)

	Un cracked Model	Service Model	Strength Model
EQX	5974.04	5974.04	5974.04
EQY	4646.48	4646.48	4646.48
Spec-X	5974.04	5974.04	5974.04
Spec-Y	4646.48	4646.48	4646.48
Scale factor for Spec-X	1971.35	2136.48	2551.26
Scale factor for Spec-Y	1630.49	1777.23	2007.14

Maximum Displacement against Earthquake

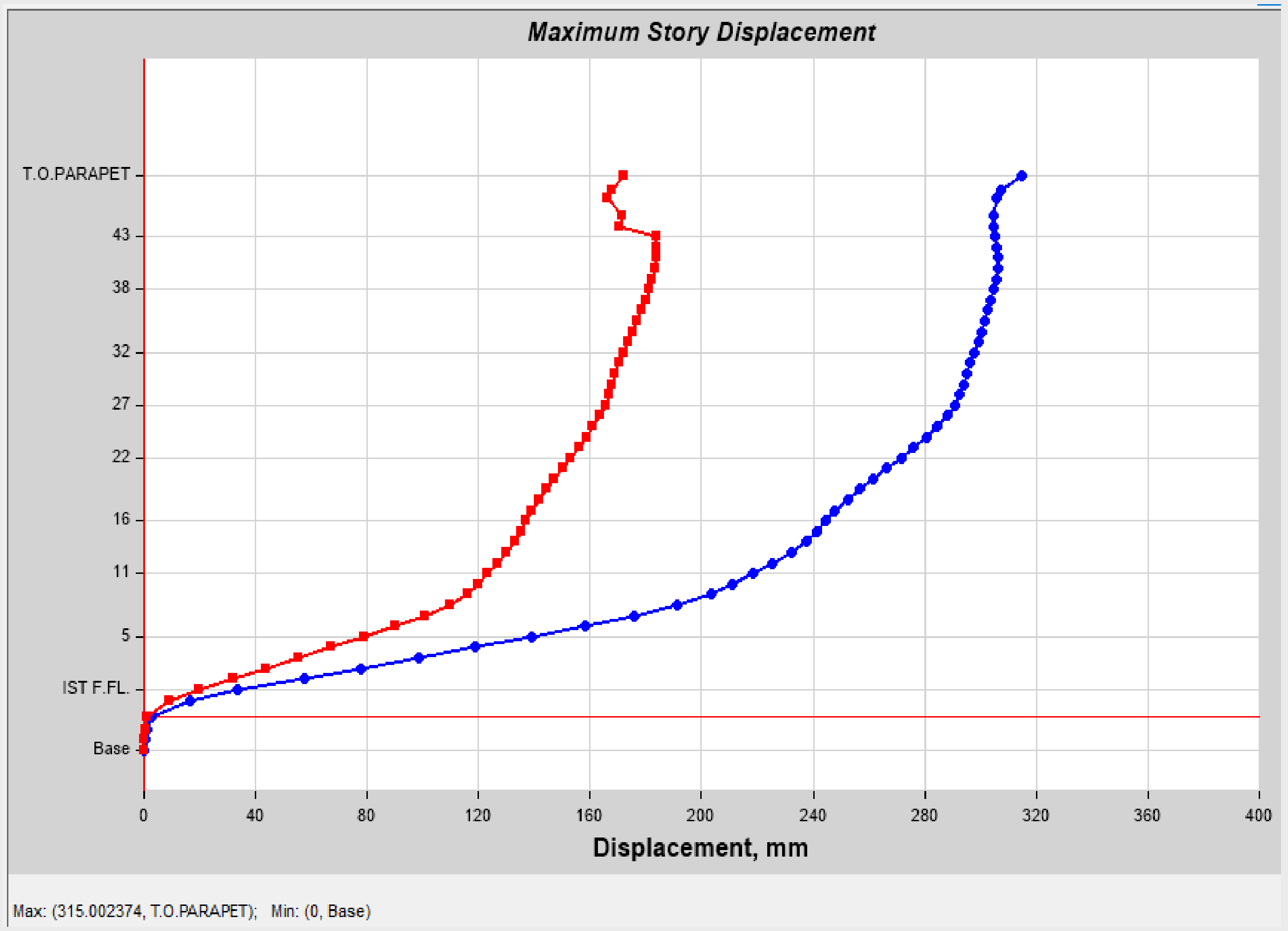
Maximum deflection against EQ allowed = H/250.

Max. allowed is H/250 = (180 x 1000)/250 =720 mm

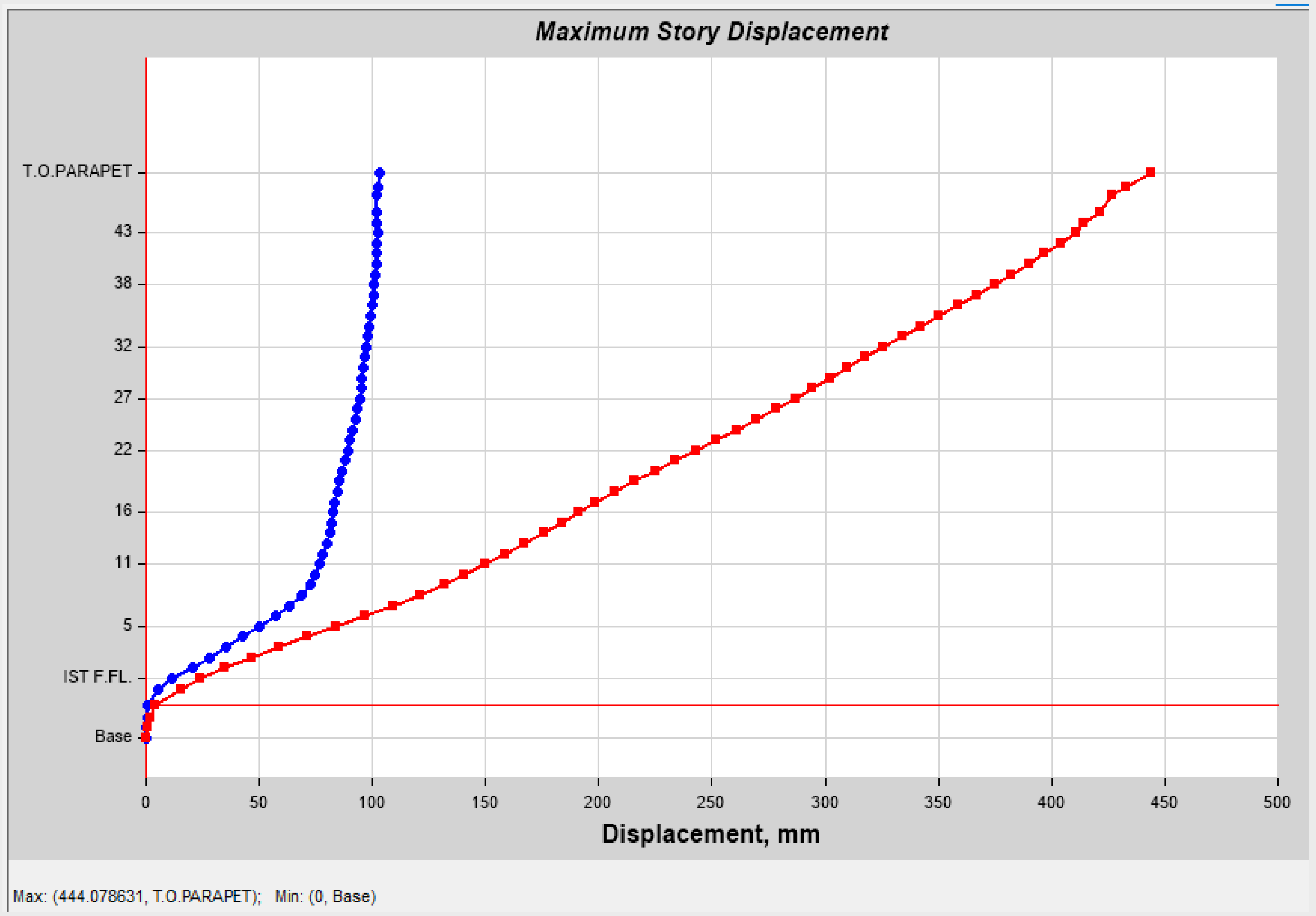
ETABS values

Spec X = 315.98 mm which is less than max. allowed hence it is safe

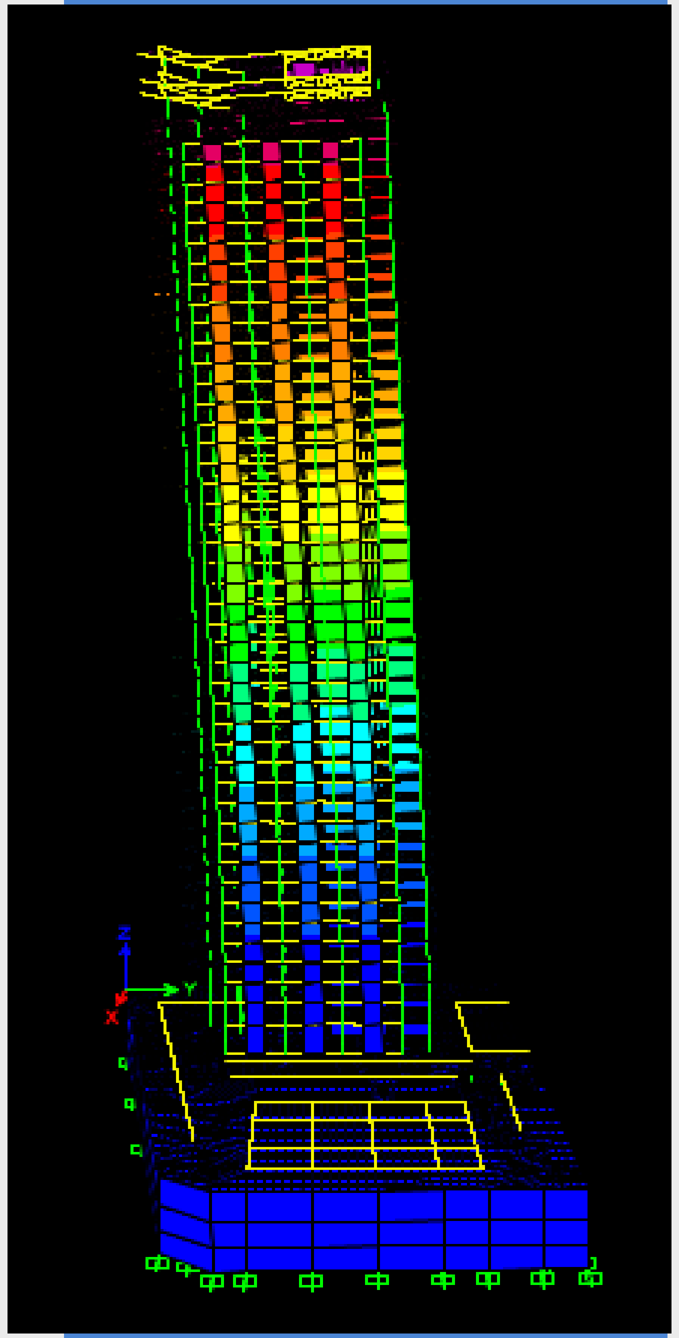
Spec Y = 444.80 mm which is less than max. allowed hence it is safe



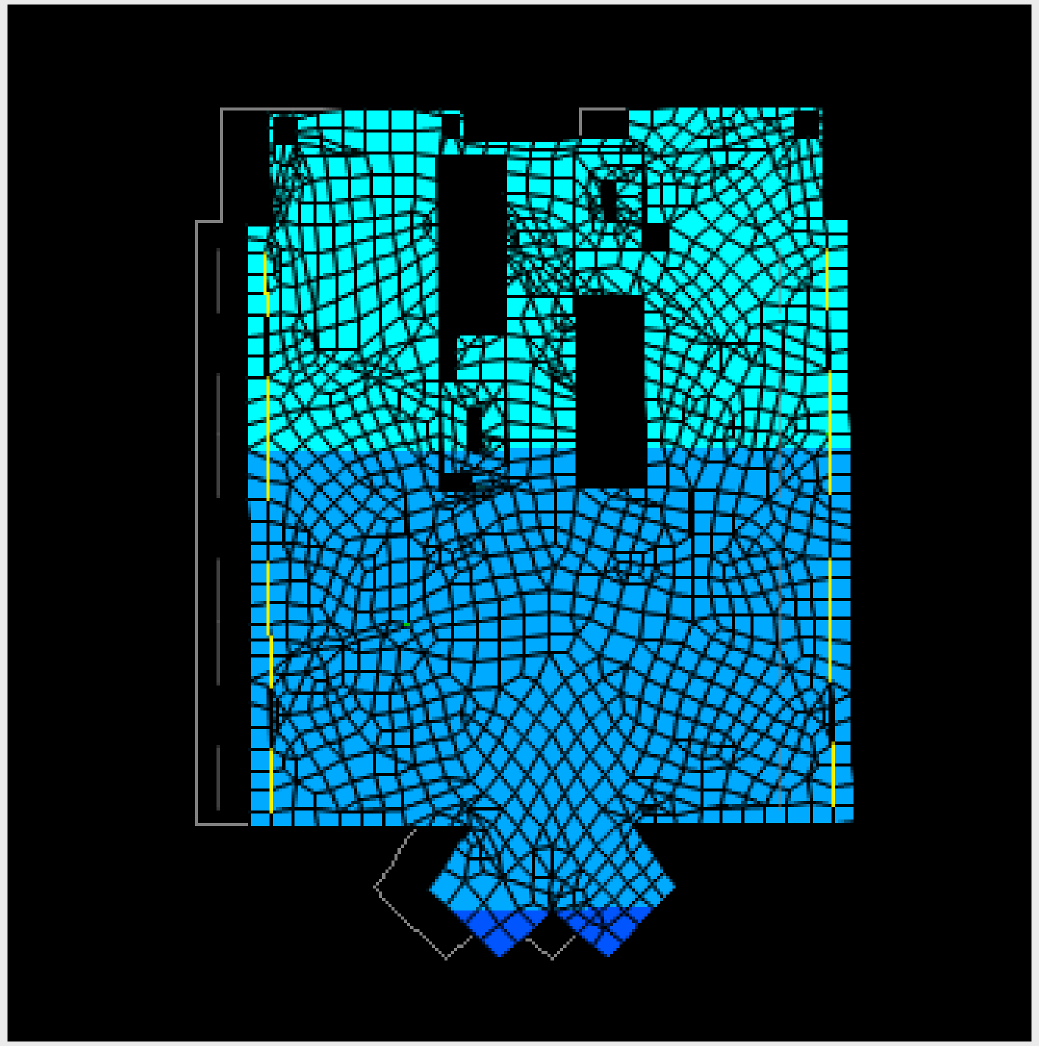
Maximum Displacement=315mm



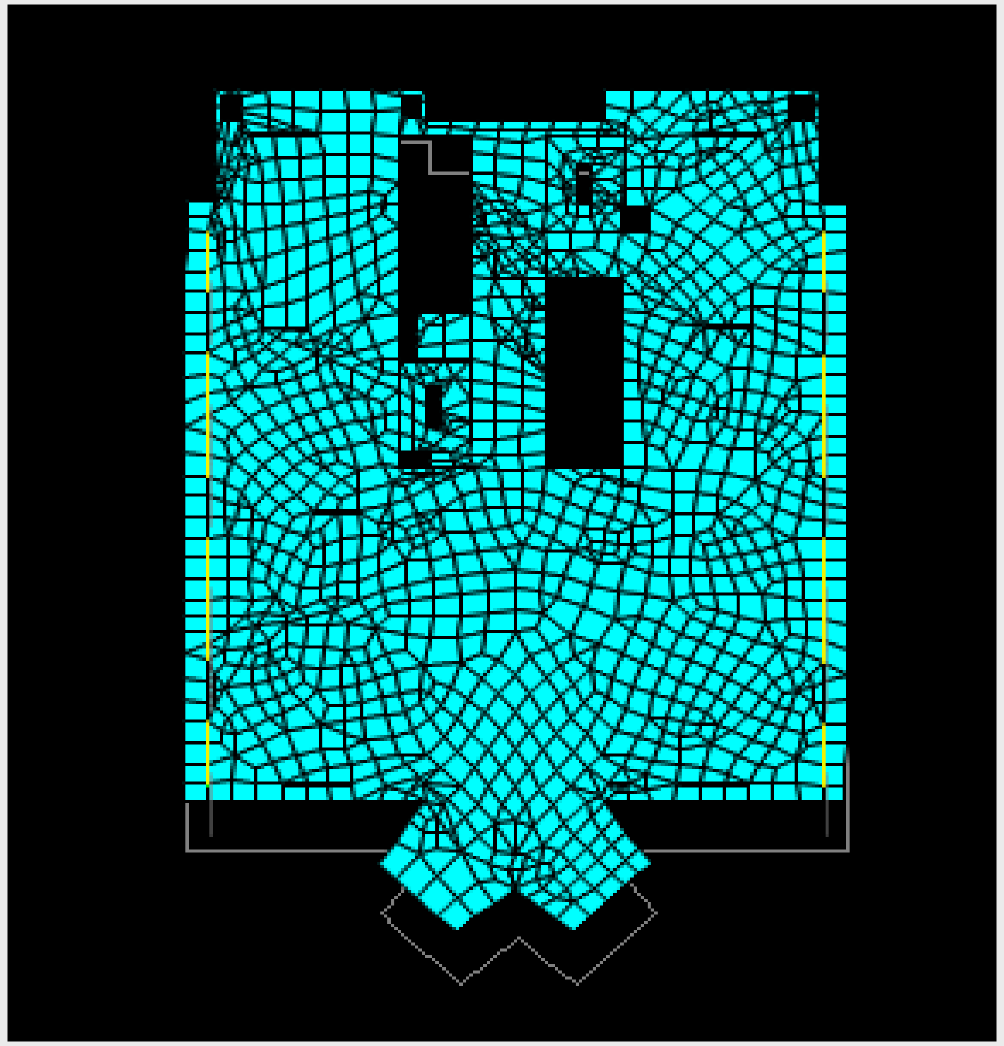
Maximum Displacement=444mm



Lateral Deflection



X Direction



Y Direction

Inter Story Drift

The maximum inelastic response is defined as:

$$\Delta_M = 0.7R\Delta_S$$

Where

R is the structural system coefficient

For structures with a period less than 0.7 seconds, the maximum story drift is limited to

$$\Delta_M \leq 2.5\% \ (T \leq 0.7 \text{ seconds}).$$

Where h is the story height.

For structures with a period greater than 0.7 seconds, the maximum story drift is limited to:

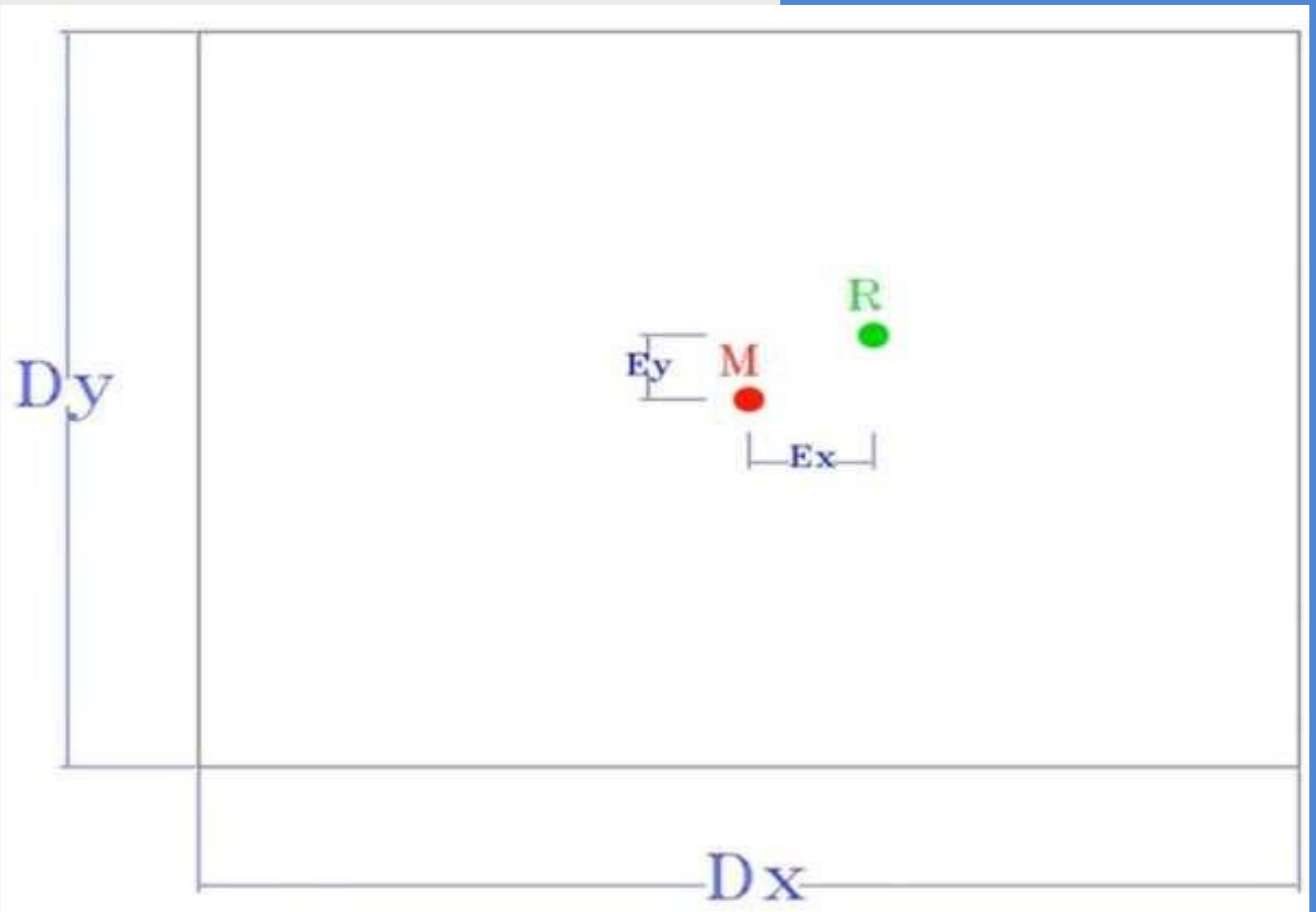
$$\Delta_M \leq 2\% \ (T \geq 0.7 \text{ seconds}).$$

Timeperiod-5.164
Max. story drift SpecX=0.002442
=0.002442X0.7X6X100
=0.94017
0.94 which is below 2% so are safe for X dir.

Max. story drift Specy=0.001724
=0.001724X0.7X6X100
=0.66374
0.66 which is below 2% so are safe for Y dir.

ECCENTRICITY CHECK

Building dimension along direction (X) = 88.5m
XCM = 40.28
XCR = 34.97
E = 40.28-34.97 = 5.31
5.31/88.5 X 100 = **6.01% Hence OK**



Building dimension along direction (Y) = 65.5m YCM = 37.16
YCR = 36.99
E = 37.16-36.99 = 0.18
0.18/65.5 X 100 = **0.3% Hence OK**

[ASCE Cl: D.3.1]

- The eccentricity between the center of mass and the geometric centroid of the building at that level shall not exceed 15% of the overall building width along each principal axis considered at each level.

$$Ex \leq 15\ \% * DX$$
$$Ey \leq 15\ \% * Dy$$

According to Code

Torsional Amplification Factor

[ASCE Cl: 12.8.4.3]

D max / D avg > 1.2
Torsional Irregularity exists

D max / D avg > 1.4
Extreme Torsional Irregularity exists

D max = maximum drift (R1 or R2)
D avg = (R1 +R2)

Story max./ avg drift in Spec-X= 1.27
Story max./ avg displacement in Spec-X= 1.215
=(1.215/1.2)2
=1.025
=0.05 x 1.025
=0.05125

Story max./ avg drift in Spec-y= 1.29
Story max./ avg displacement in Spec-X= 1.32
=(1.32/1.29)2
=1.04
=0.05 x 1.04
=0.052

$$A_x = \left[\frac{\delta_{max}}{1.2\,\delta_{avg}}\right]^2$$

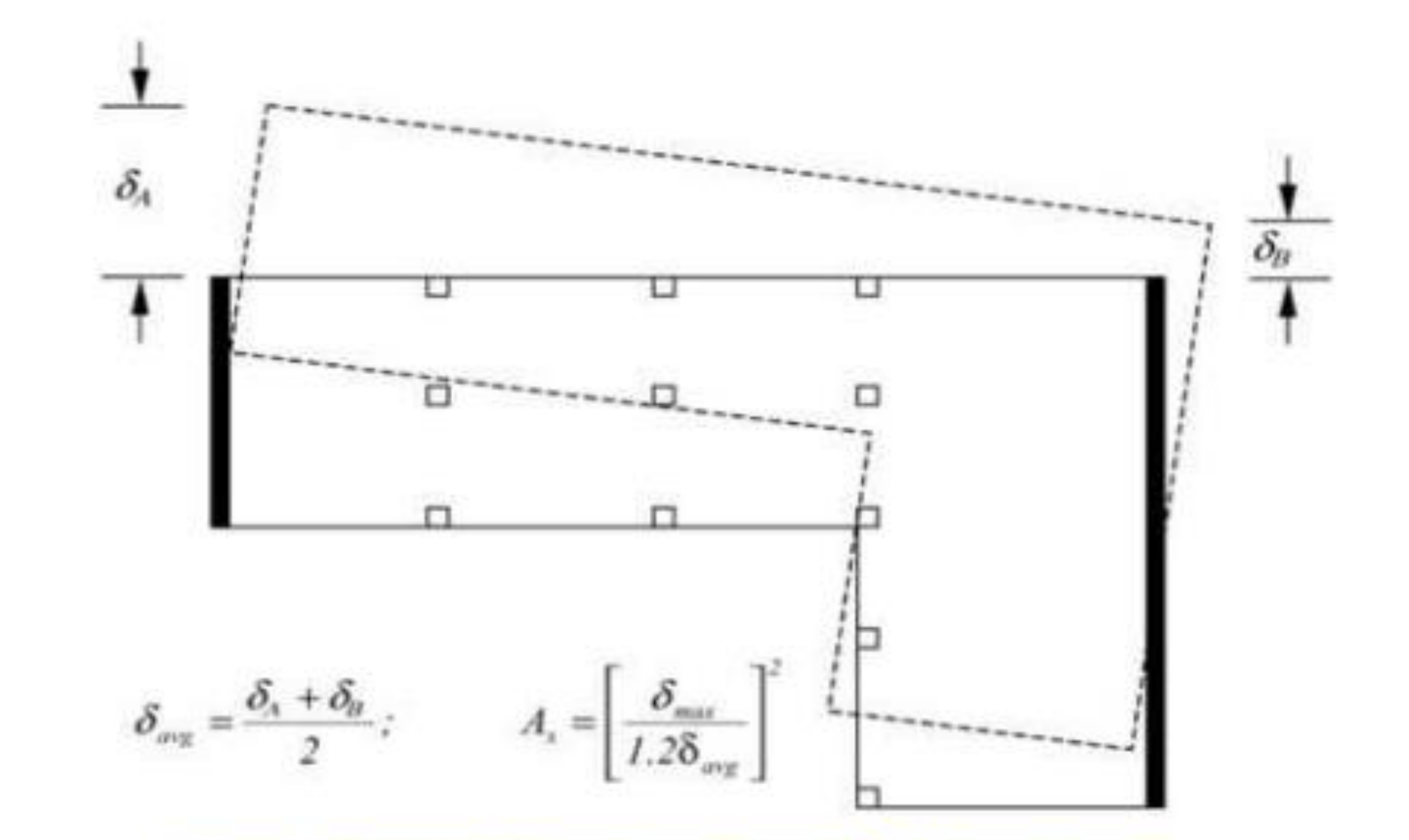
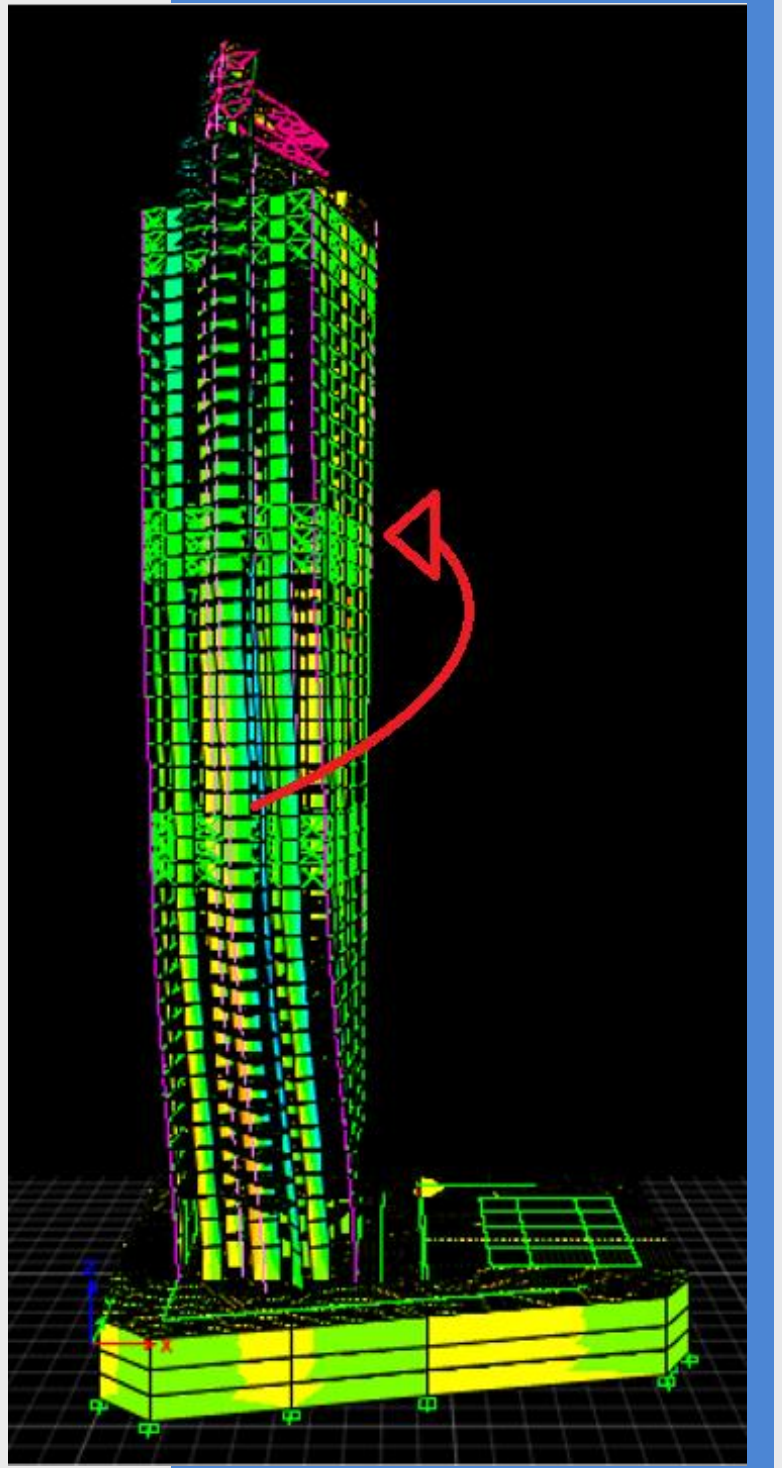


FIGURE 12.8-1 Torsional Amplification Factor, A_x

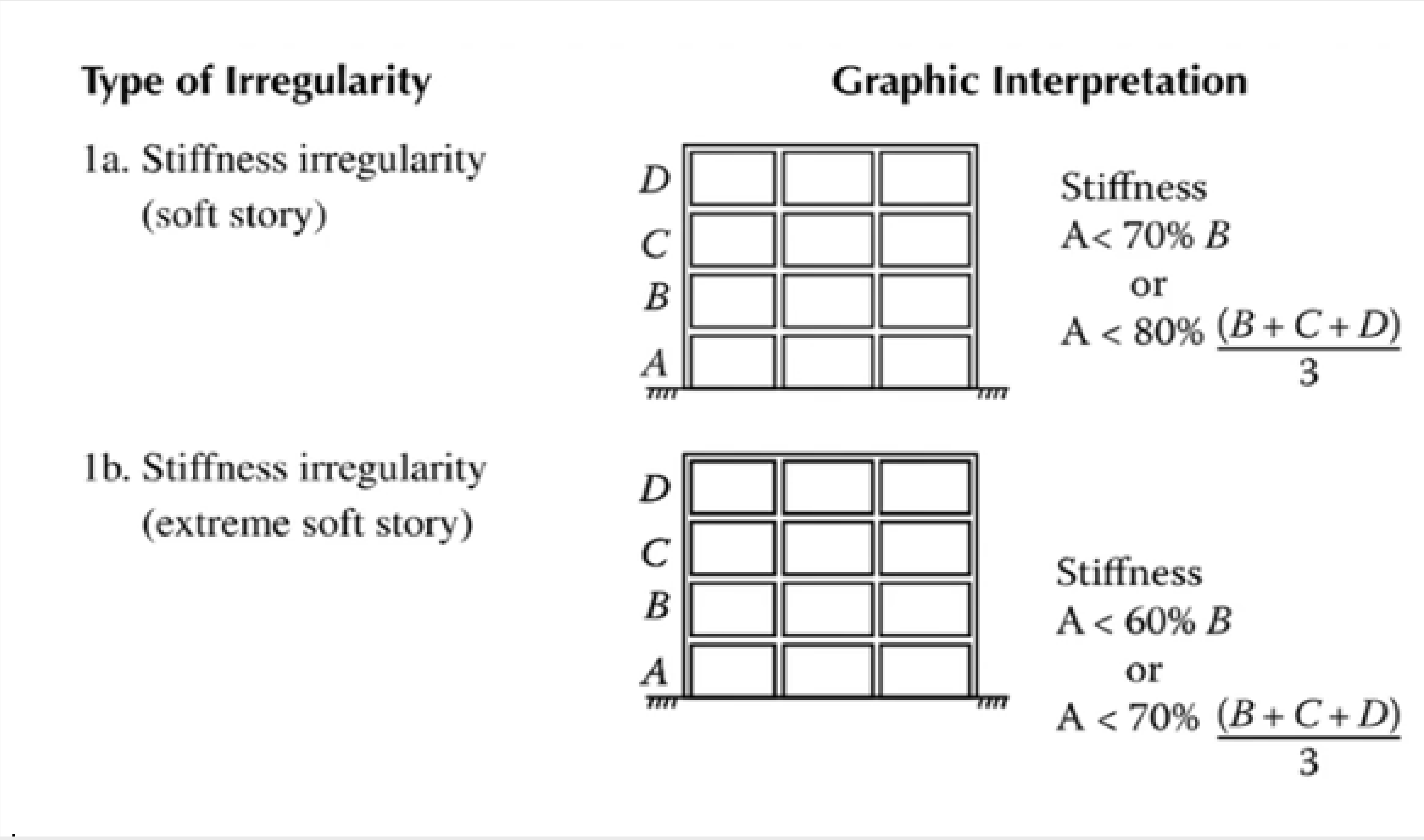


Soft Story

[ASCE Table 12.6-1]

Story Stiffness SpecX
A= 138675103.00
B= 72924702.51
C= 24811168.36
D= 3657013.60
A<70%B
=190 > 70% (OK)

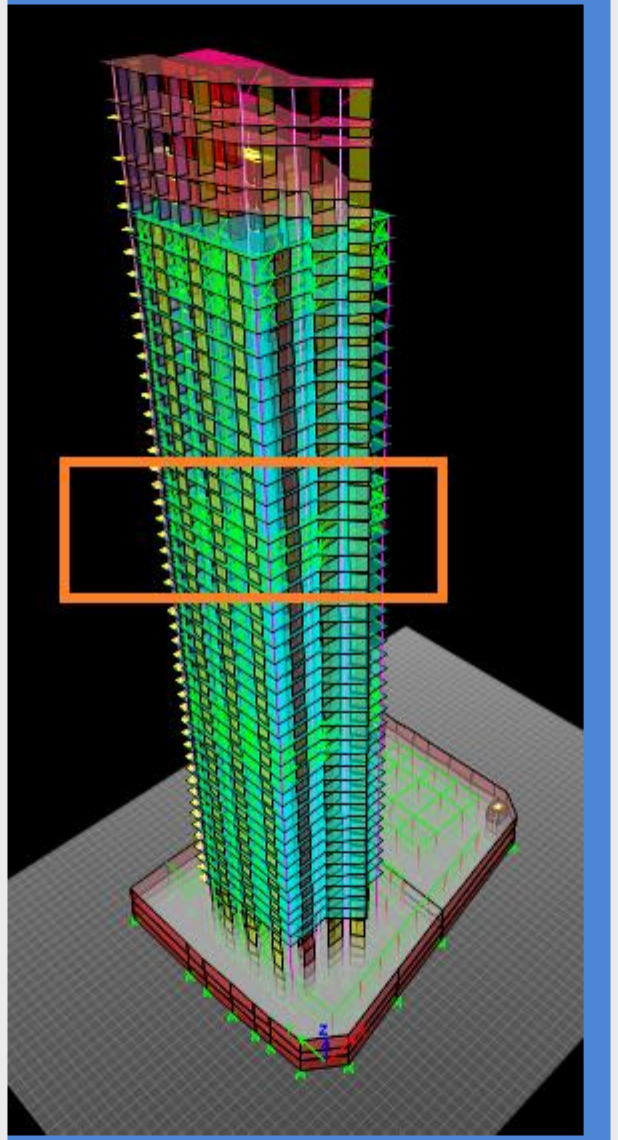
A<80%(B+C+D)/3
A avg= 33797628.15
=410> 80% (OK)



Story Stiffness SpecY
A= 109054208.00
B= 53605863.31
C= 24817594.37
D= 5207631.87
A<70%B
=203 > 70% (OK)

A<80%(B+C+D)/3
Aavg= 27877029.85
=391> 80% (OK)

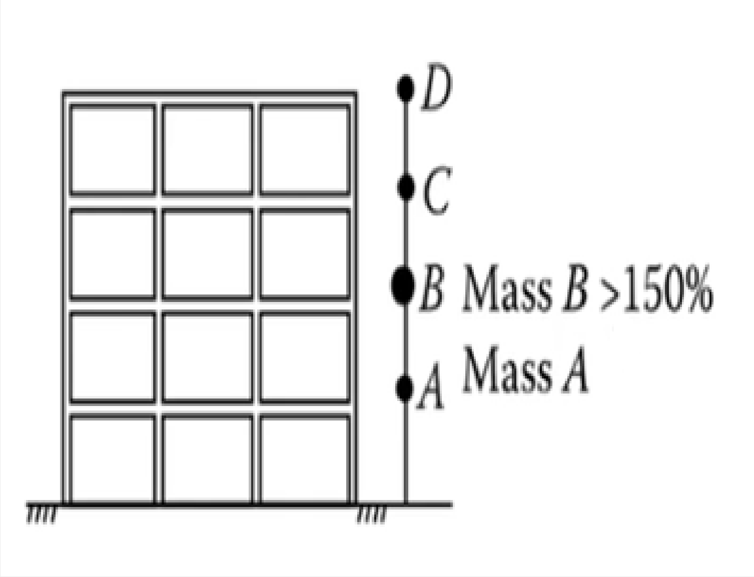
A soft story is a storey whose lateral stiffness is less than that of the storey above.



Mass irregularity shall be considered to exist, when the seismic weight of any floor is more than 150 percent of that of the floors below.

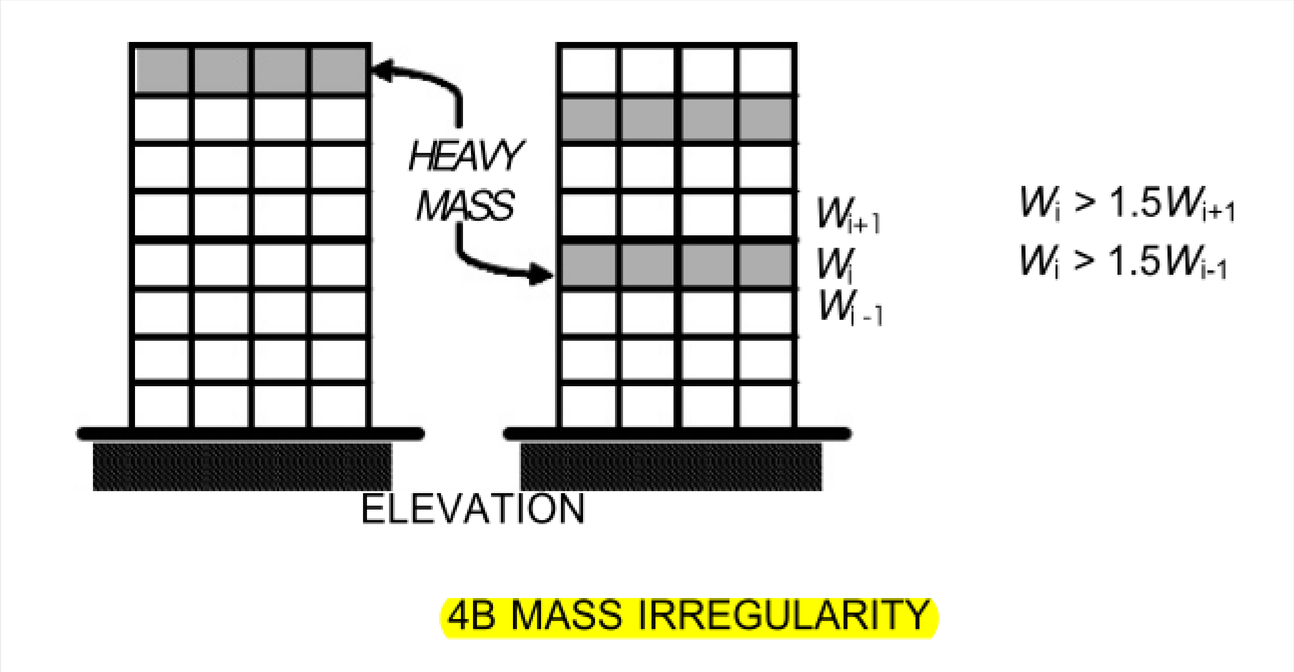
Heavy Story

[ASCE Table 12.6-1]



Mass in Spec-X

A= 6146148.81
B= 6257045.81
B< 150
=98% < 150% (OK)



Mass in Spec-Y

A= 6146148.81
B= 6257045.81
B< 150
=98% < 150% (OK)

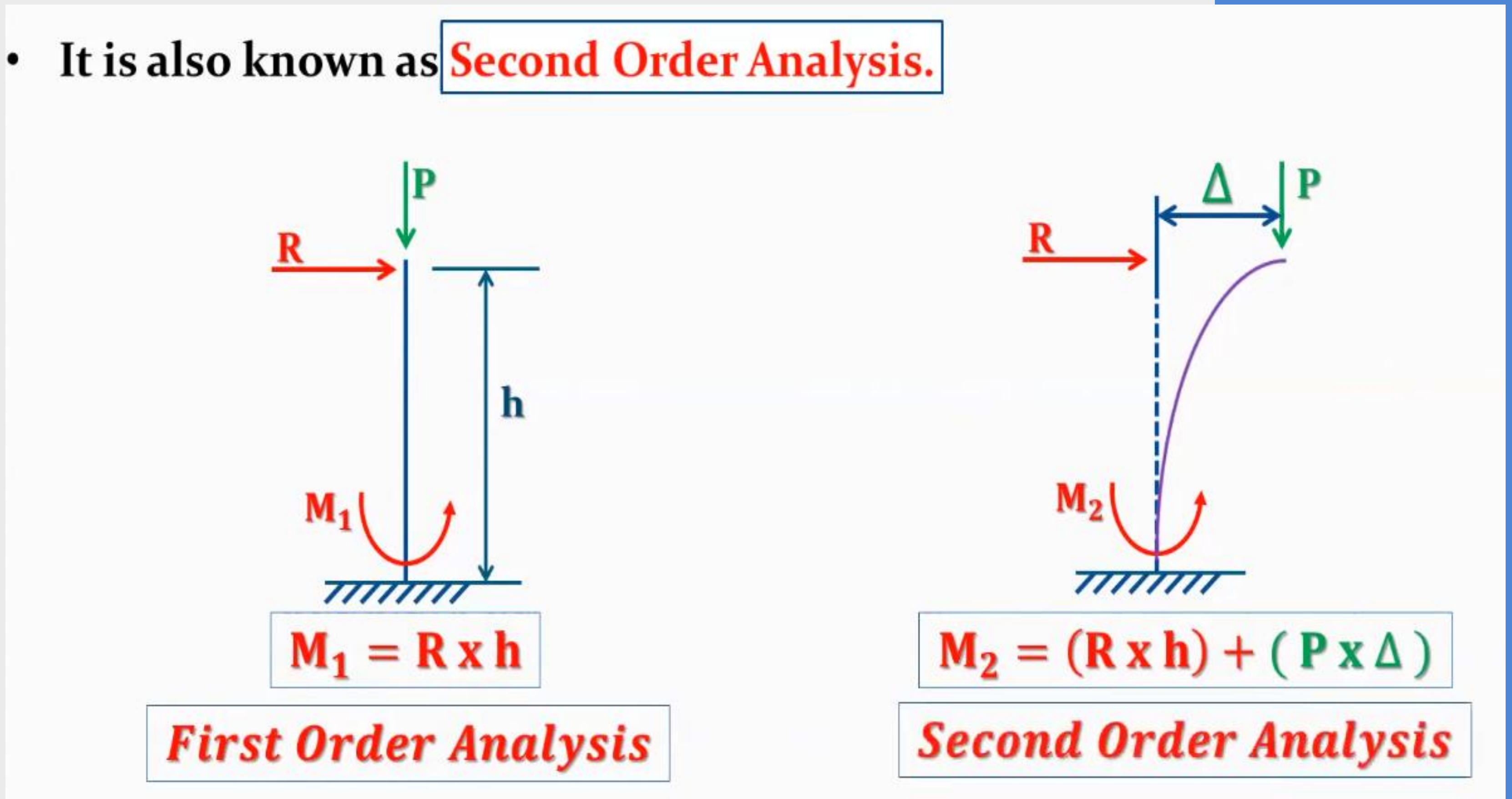
P-DELTA ANALYSIS CHECK
[ASCE Cl: 12.8.7]

Check P Delta effect according the ACI code

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d} \quad , \quad \theta max = \frac{0.5}{\beta C_d} \leq 0.25$$

If $\Theta_{max} > 0.25$ the structure is potentially unstable and shall be redesigned. (We need to include P-delta)

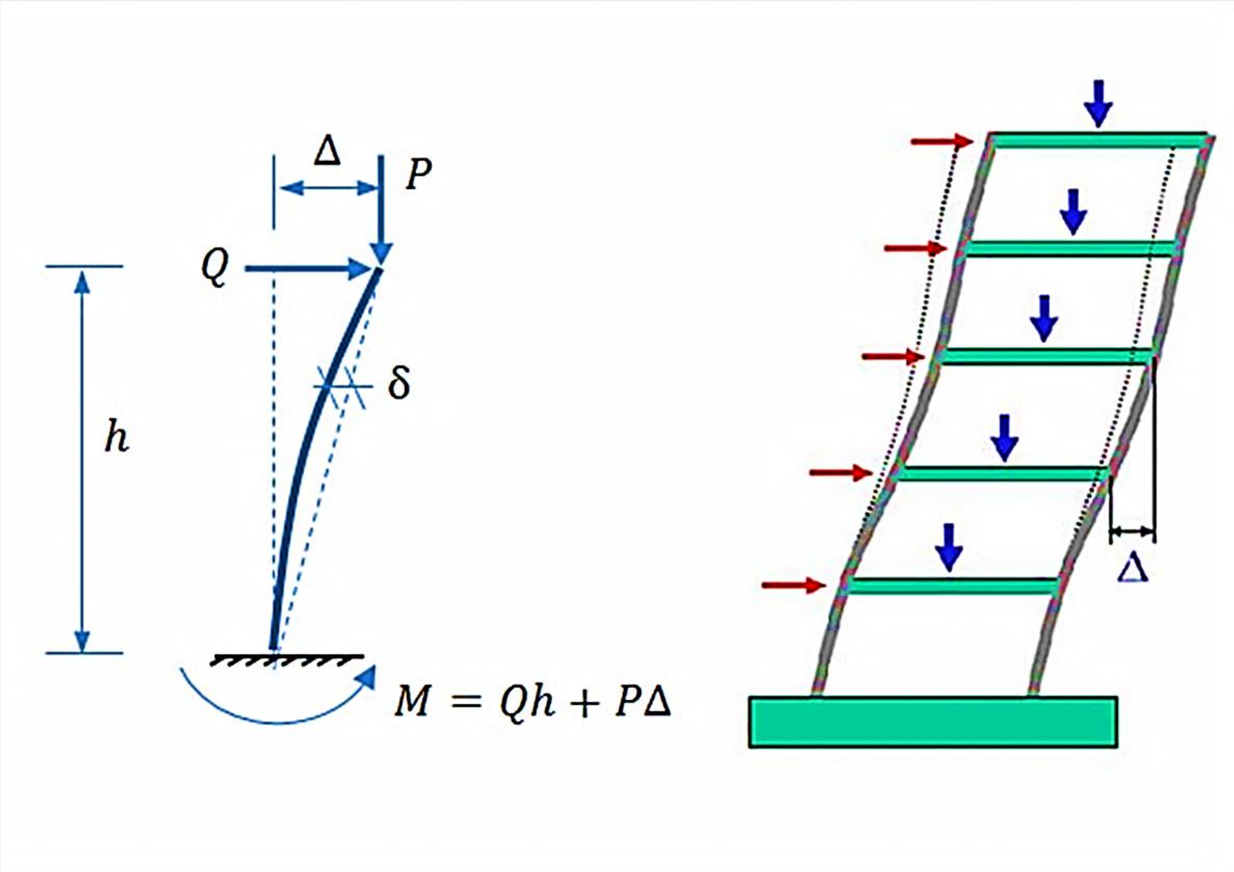
If $\Theta \leq 0.1$ the structure is stable (No P-delta)



Define ⟶ P-Delta options

Iterative based on loads

Load pattern	scale factor
DEAD	1.2
SDL	1.2
LIVE	0.5



WIND ANALYSIS

[ASCE Ch: 26.0]

WIND SPEED	= 200 mpa	[ASCE fig. C26.14-2]
EXPOSURE TYPE	= B	[ASCE Cl: 26.7]
TOPOGRAPHICAL FACTOR, kzt	= 1.0	[ASCE Cl: 26.8.1]
Ground elevation Factor	= 1	
GUST FACTOR	= 0.85	[ASCE Cl: 26.11]
DIRECTIONALITY FACTOR, Kd	= 0.85	[ASCE Cl: 26.6]

Maximum Displacement against Wind

The maximum deflection against wind allowed is H/500.

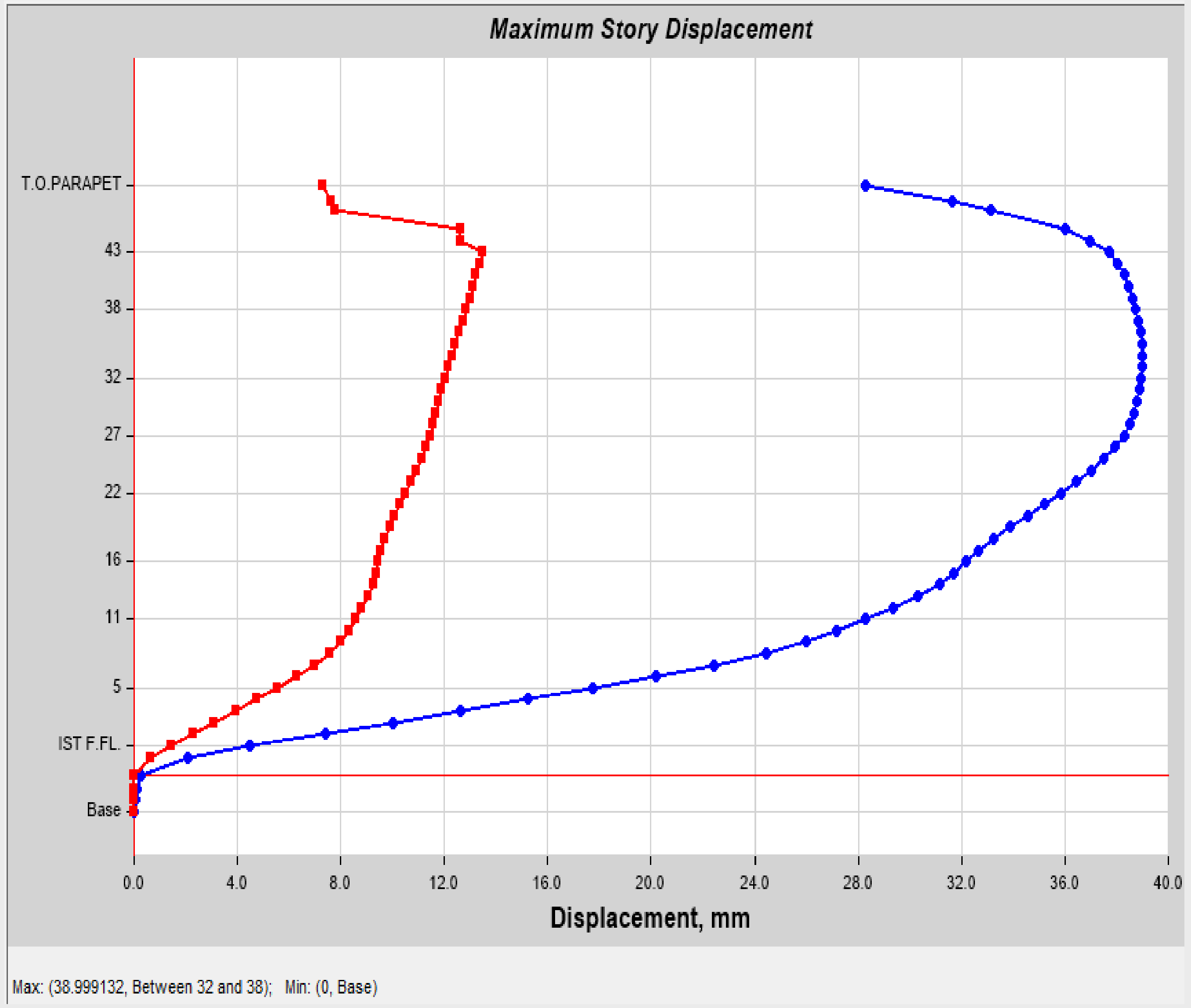
Max. allowed is H/500 = (180 x 1000)/500 =360 mm

ETABS values

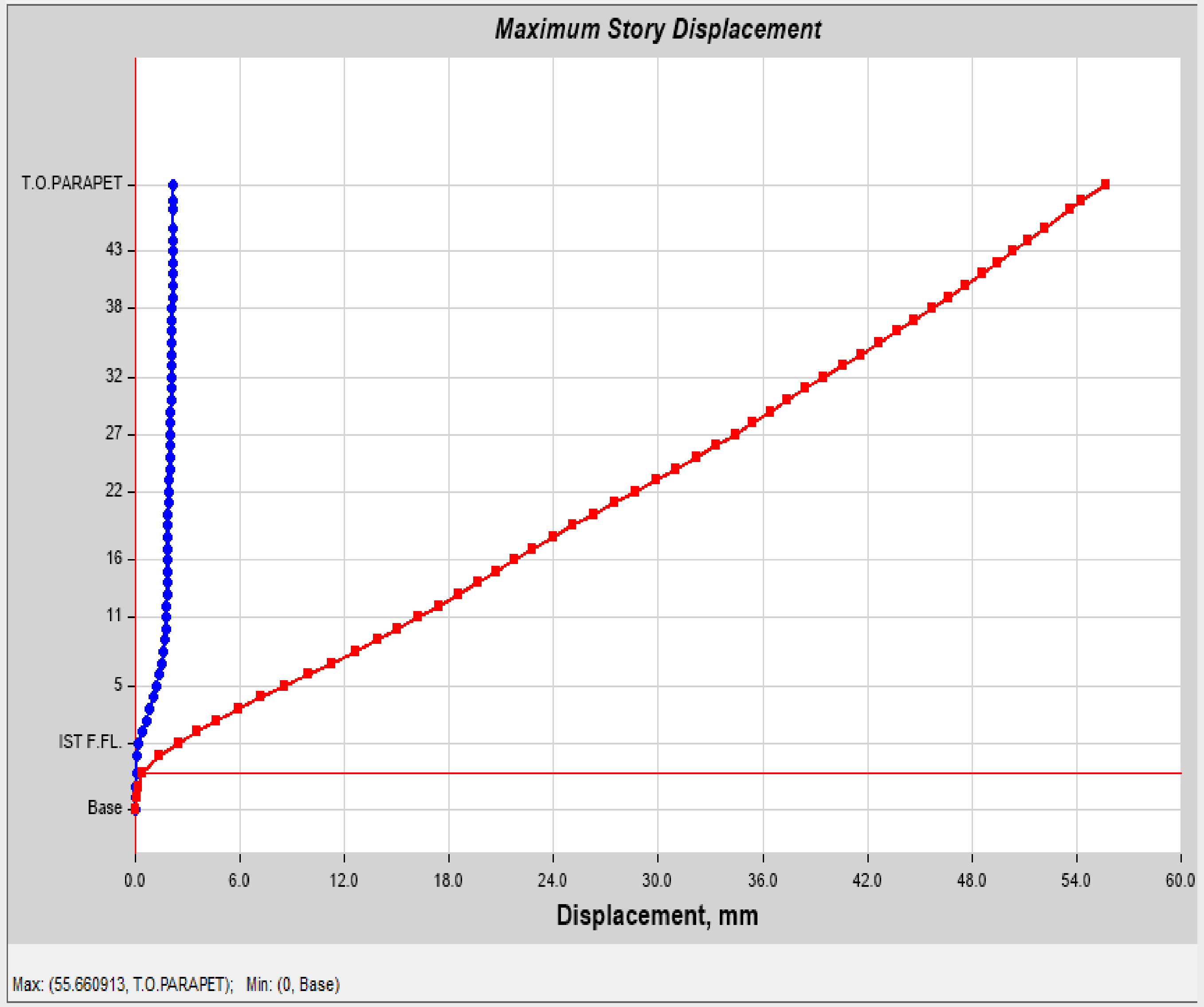
WLX = 38.2 mm which is less than max. allowed hence it is safe

WLY = 55.7 mm which is less than max. allowed hence it is safe

WIND ANALYSIS



Maximum Displacement WLX=38.2mm



Maximum Displacement WLY=55.7mm

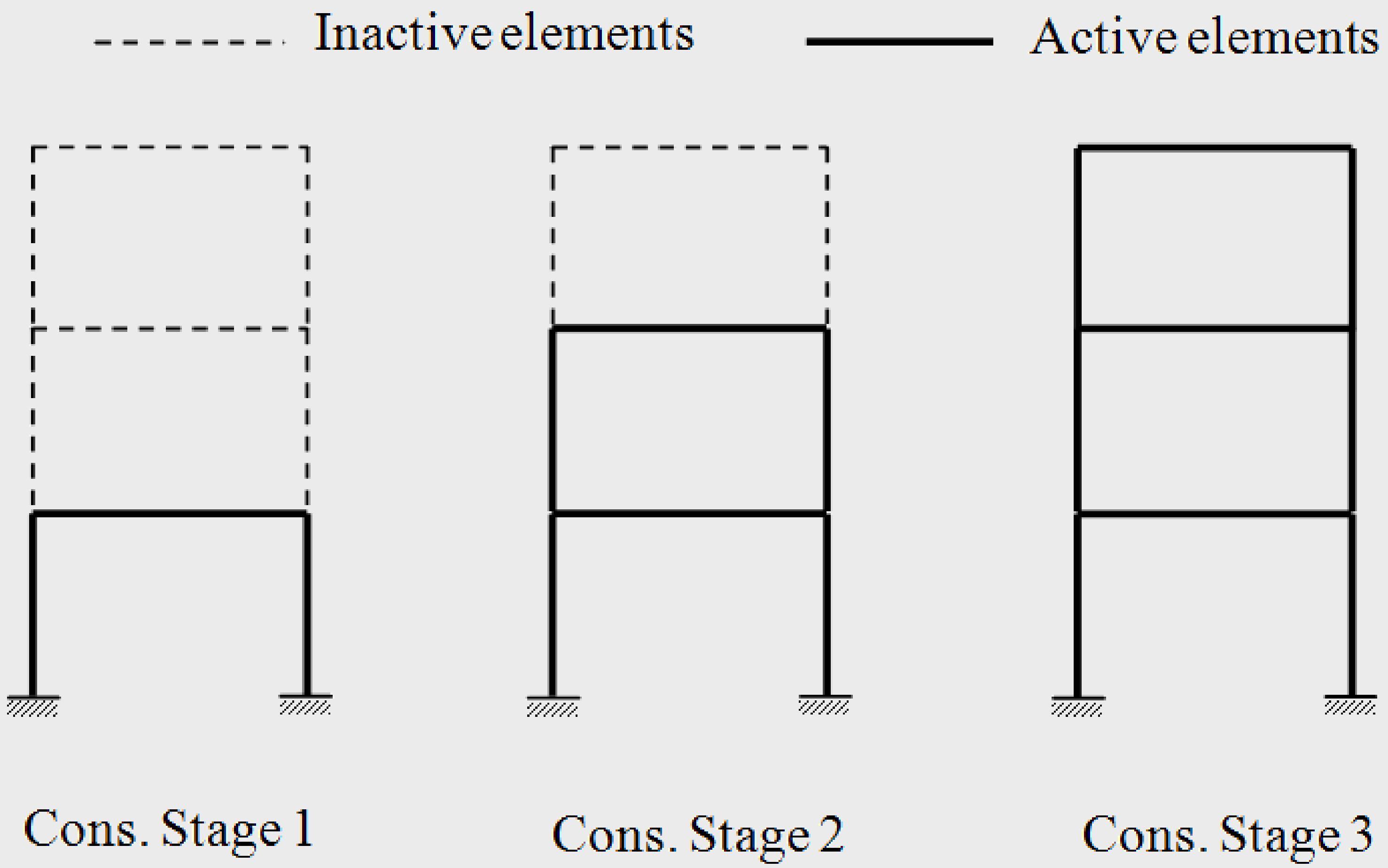
CONSTRUCTION SEQUENCE ANALYSIS (CSA)

CSA is a nonlinear analysis approach in which the structure is analyzed at various stages corresponding to the construction sequence and the partial required loads are applied sequentially at every stage.

The performance of a structure with the various loads applied in a single step differs significantly from that when the loads are applied in stages. The phenomenon known as Sequential Construction Analysis is used to analyze the structure at each story.

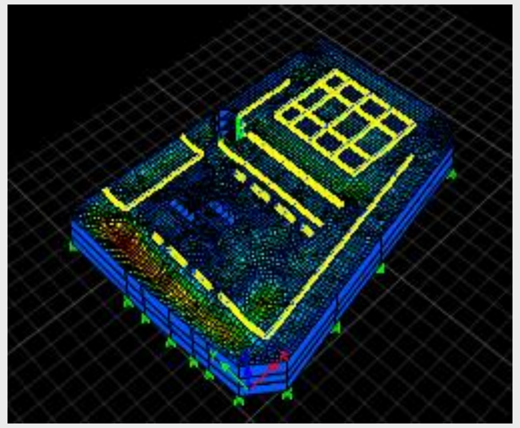
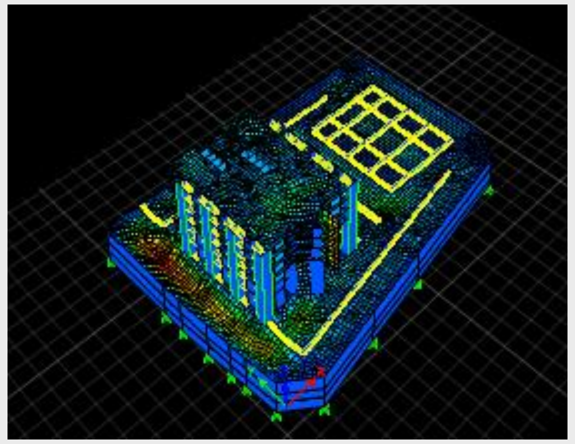
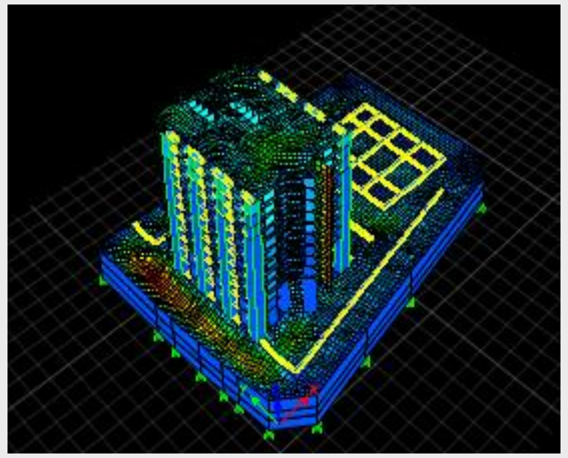
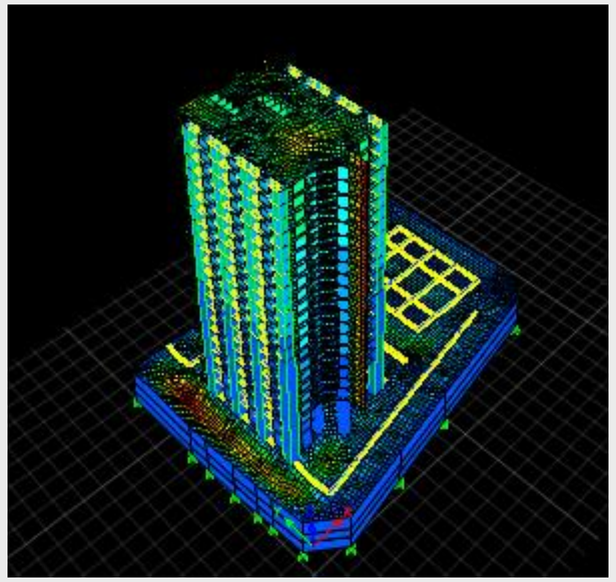
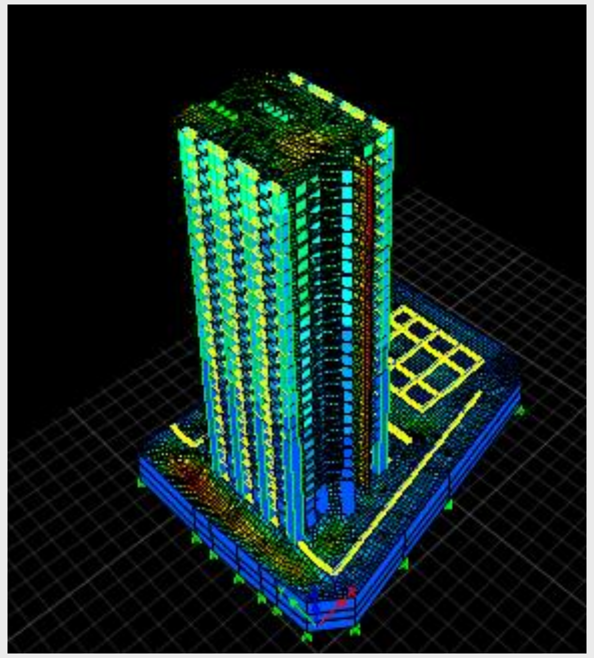
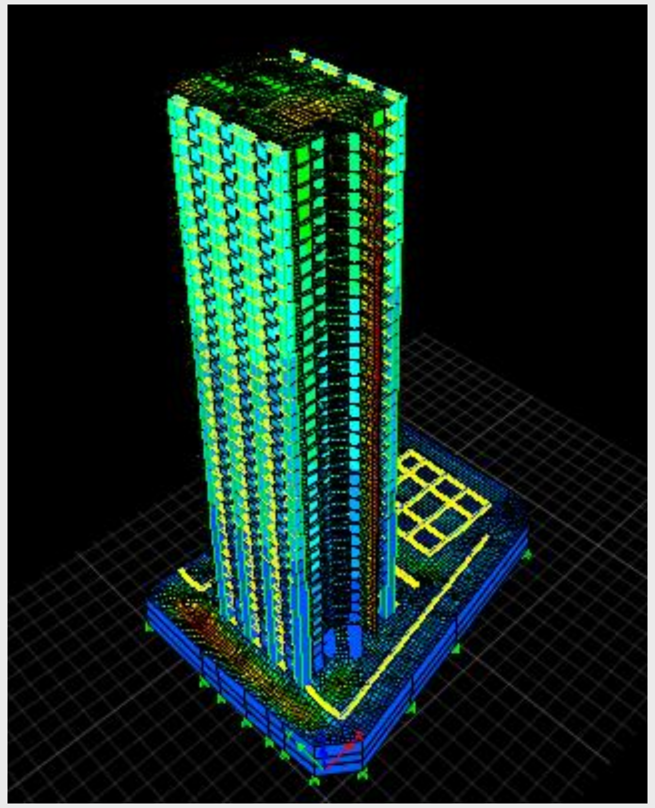
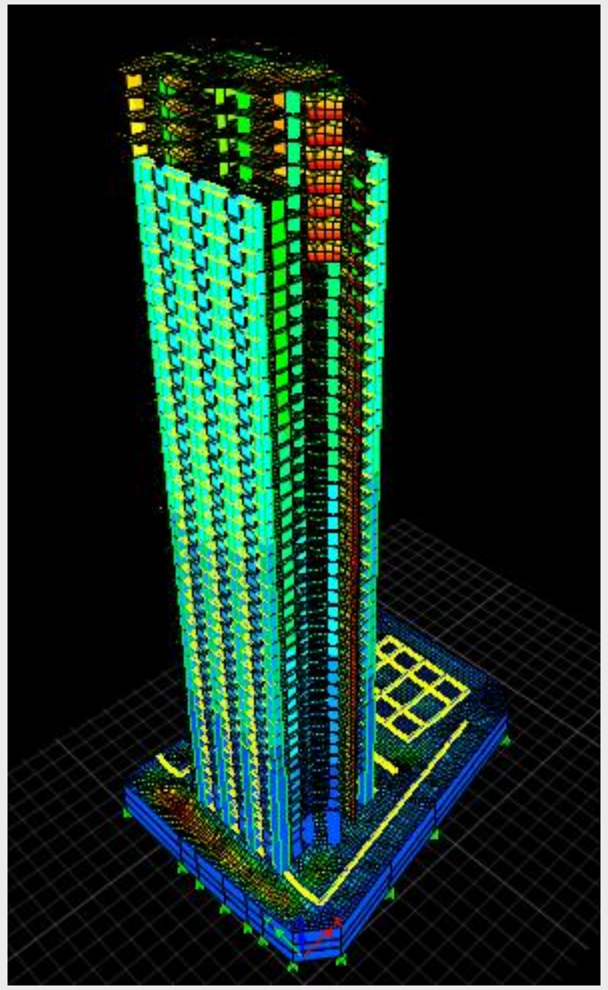
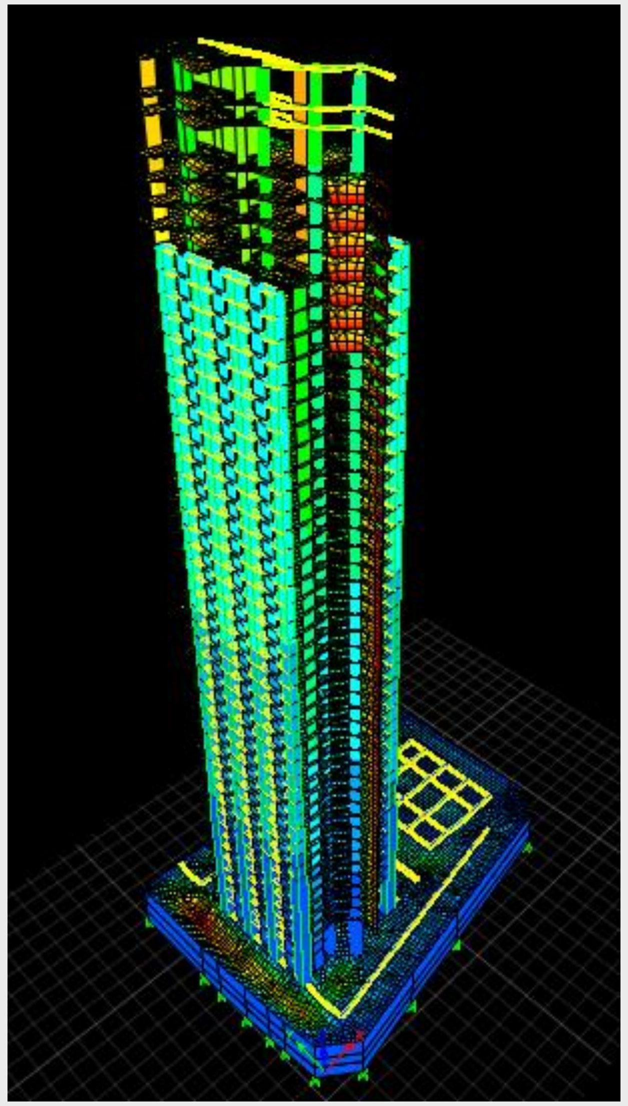
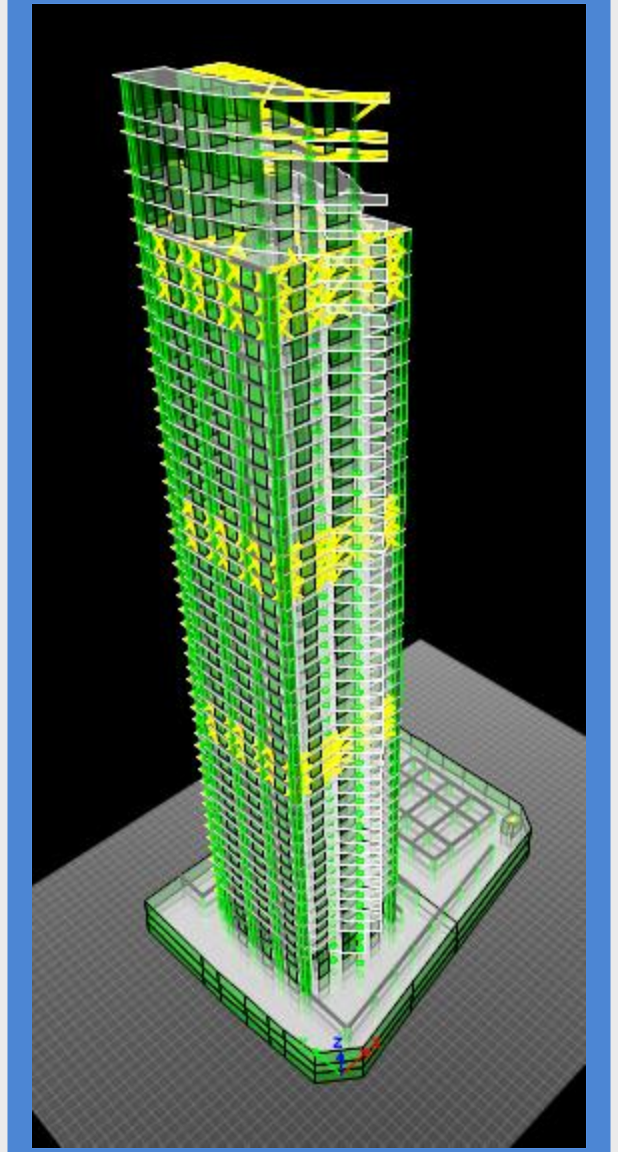
Time Dependent effects

- Creep
- Shrinkage
- Concrete Strength, fc
  - Modulus of elasticity



CONSTRUCTION SEQUENCE ANALYSIS (CSA)

Determined the creep & auto-sequence deflection
Span/350

CREEP & SHRINKAGE ANALYSIS

Determined the creep & auto-sequence deflection
Span/350

Creep, along with **shrinkage** (decrease in direct strains over time) and aging (change in elastic modulus with age), are time-dependent material behaviors which may be applied to concrete objects using staged-construction analysis.

The deflection including the effects of temperature, creep & shrinkage occuring after erection of partition & the application of finishes should not normally exceed span/350 or 20mm which ever is less.

The control of deflection

1) Limit of strength
2) Limit of serviceability

Serviceability

1) Cracking
2) Drift
3) Shrinkage
4) Ground bearing pressure

structure does not fall, structure become serviceable, will lock not good.

In which we have considering load combination: (DL+ LL) for serviceability for unfactored

Shorter span/350 or 20mm which ever is less (does not this value exceed)

Check for deflection on more deflected slab like cantilever slabs, large span slabs, there check it.

To control it by increase slab depth & beam depth.

Determined the creep & auto-sequence deflection
Span/350

SPAN/350= 14.28mm

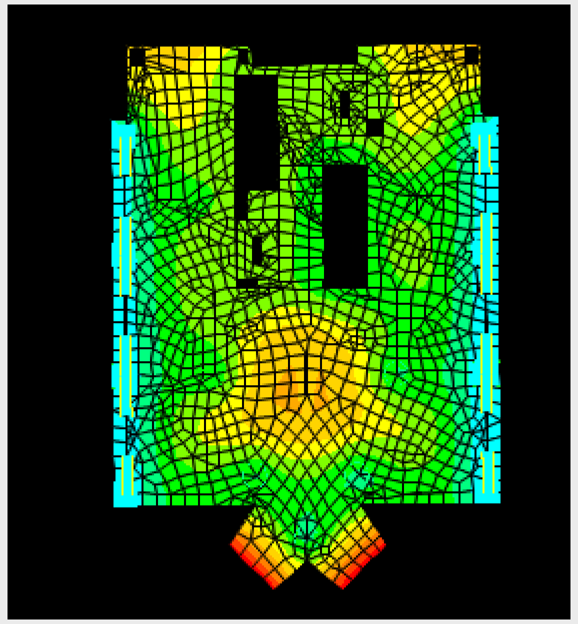
SHORTER SPAN=5m

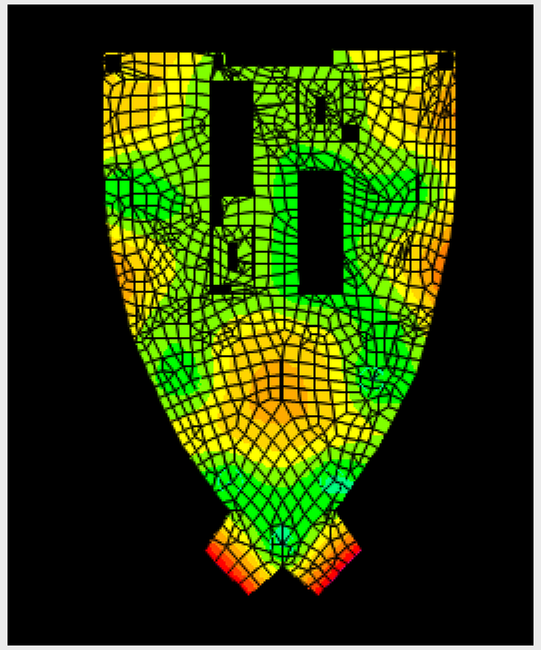
Virtual deflection= 13.468mm

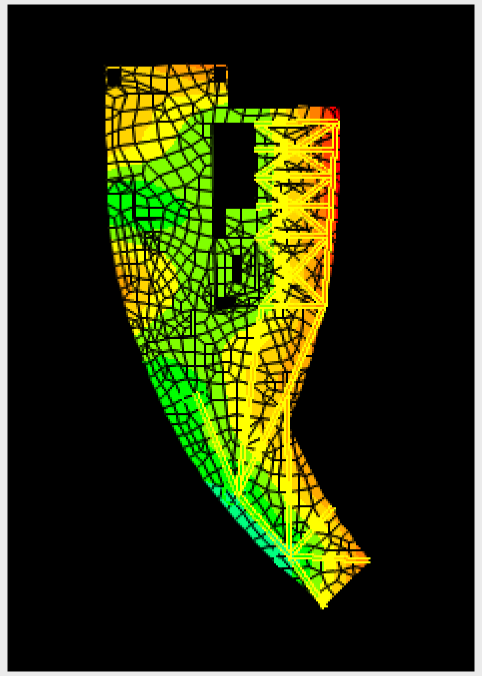
Axial shortening of columns= 3.570mm

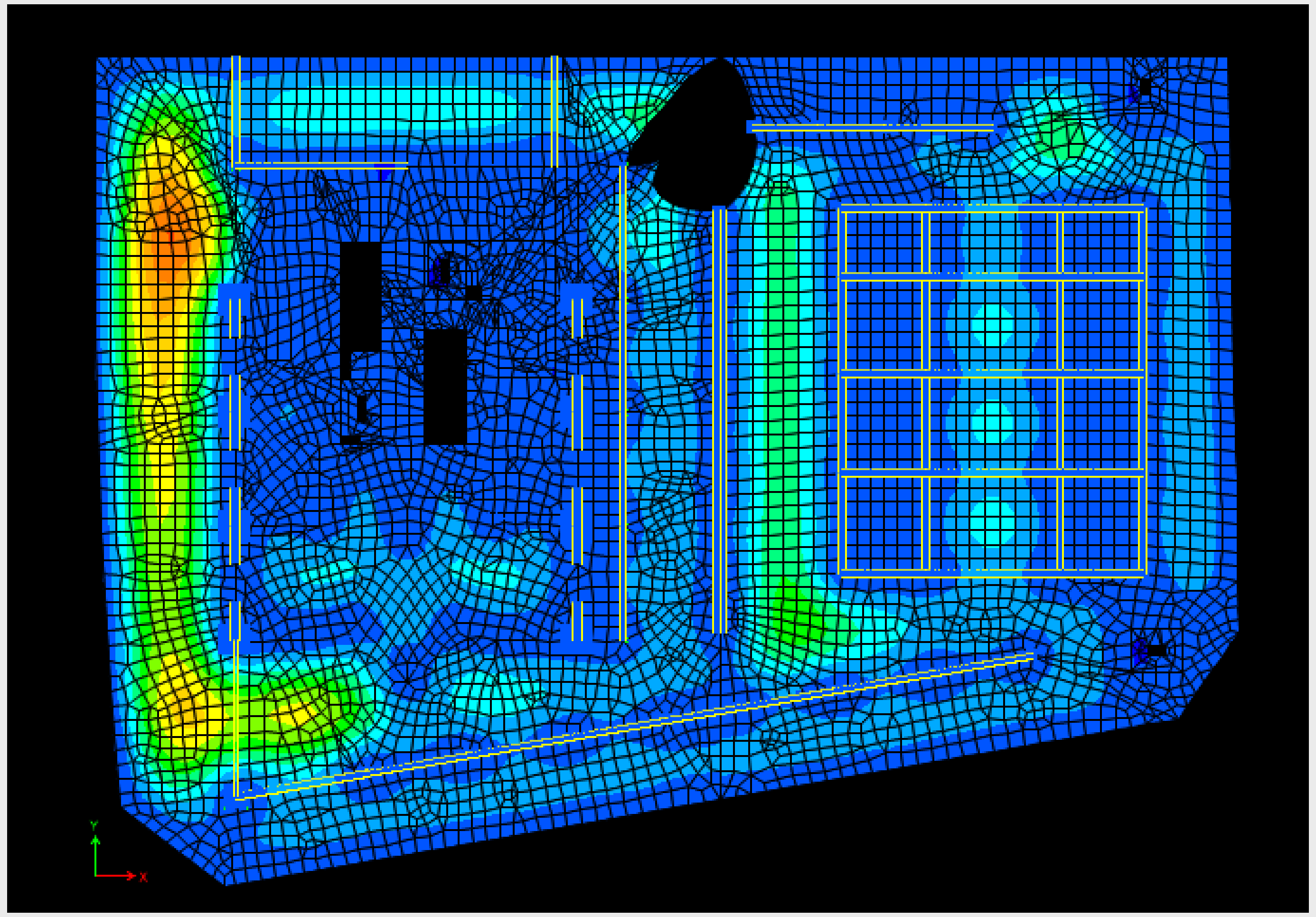
Real deflection= 9.898mm

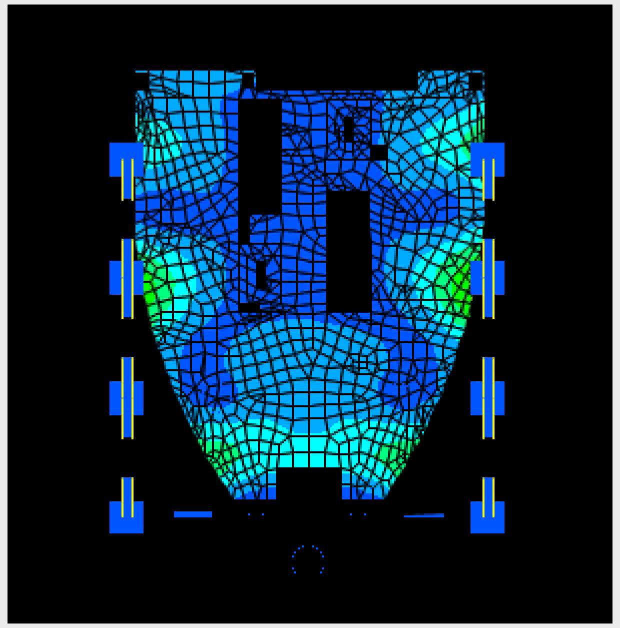
For creep for 1 year=1.065x 1.1= 1.1715

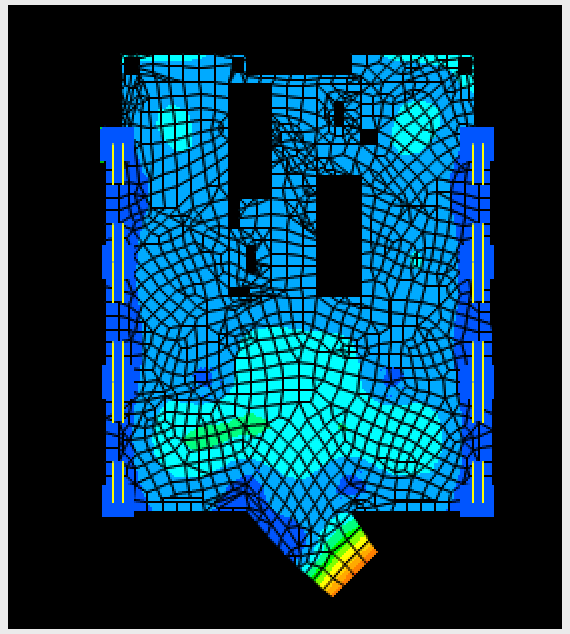










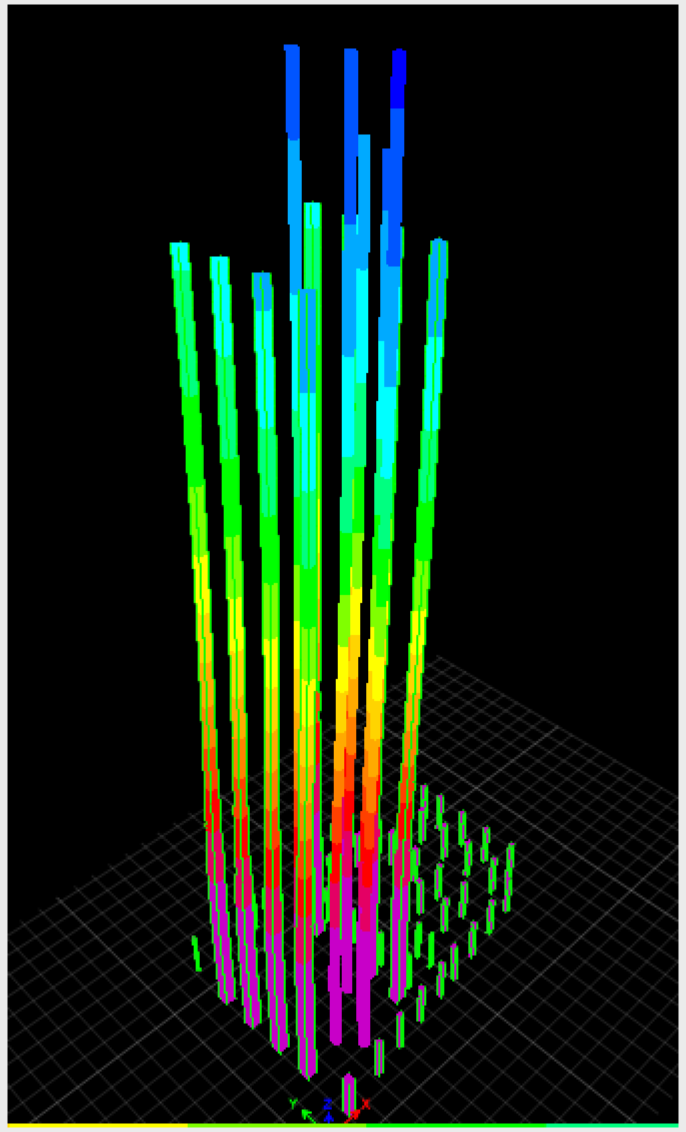


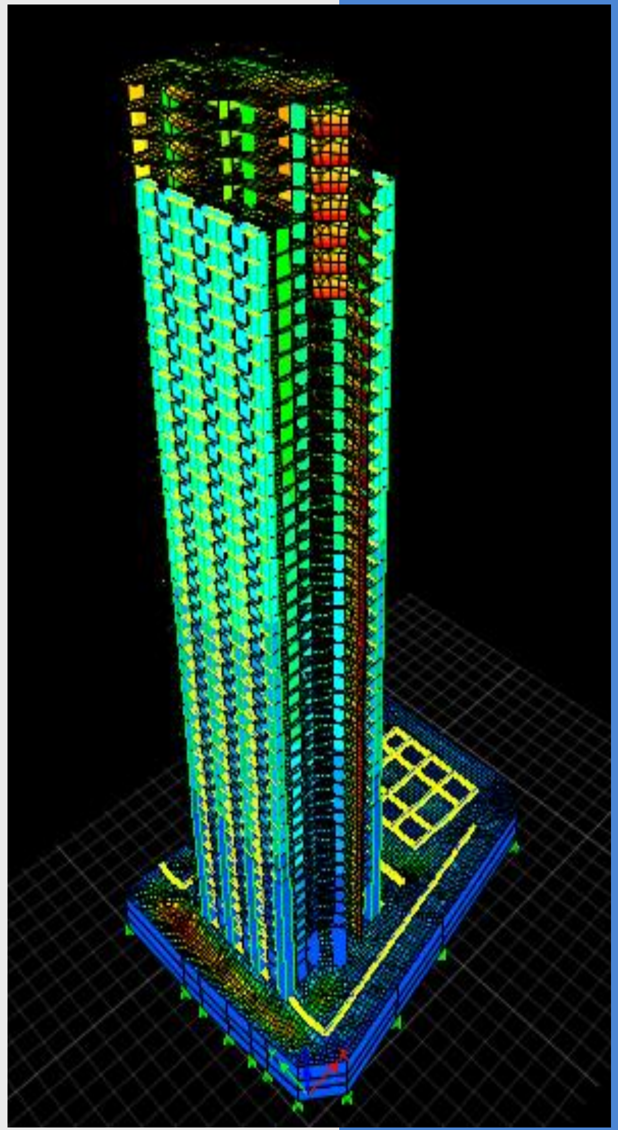
Axial shortening of columns in tall buildings

In tall buildings, columns carry huge loads which effectively compress the column along its axis. This is commonly known as 'axial shortening'. The axial column displacements generate significant forces within the structural elements they support, and can also cause failure of surrounding non-structural elements.

Causes of axial shortening

1. Elastic deformation (Δi): Instantaneous deformation that depends on the magnitude of load, strength of concrete and age of concrete at the time of load application.
2. Shrinkage (Δs) and creep (Δc): Time and deformation dependent on concrete properties, amount of reinforcement, volume/surface ratio, ambient humidity conditions, magnitude of sustained load (affects creep only) and loading history. As a rough guide, 40% of the inelastic deformation takes place within the first 28 days, while after three to six months, 60% and 70% of the total deformation will occur, increasing to 90% after two years.



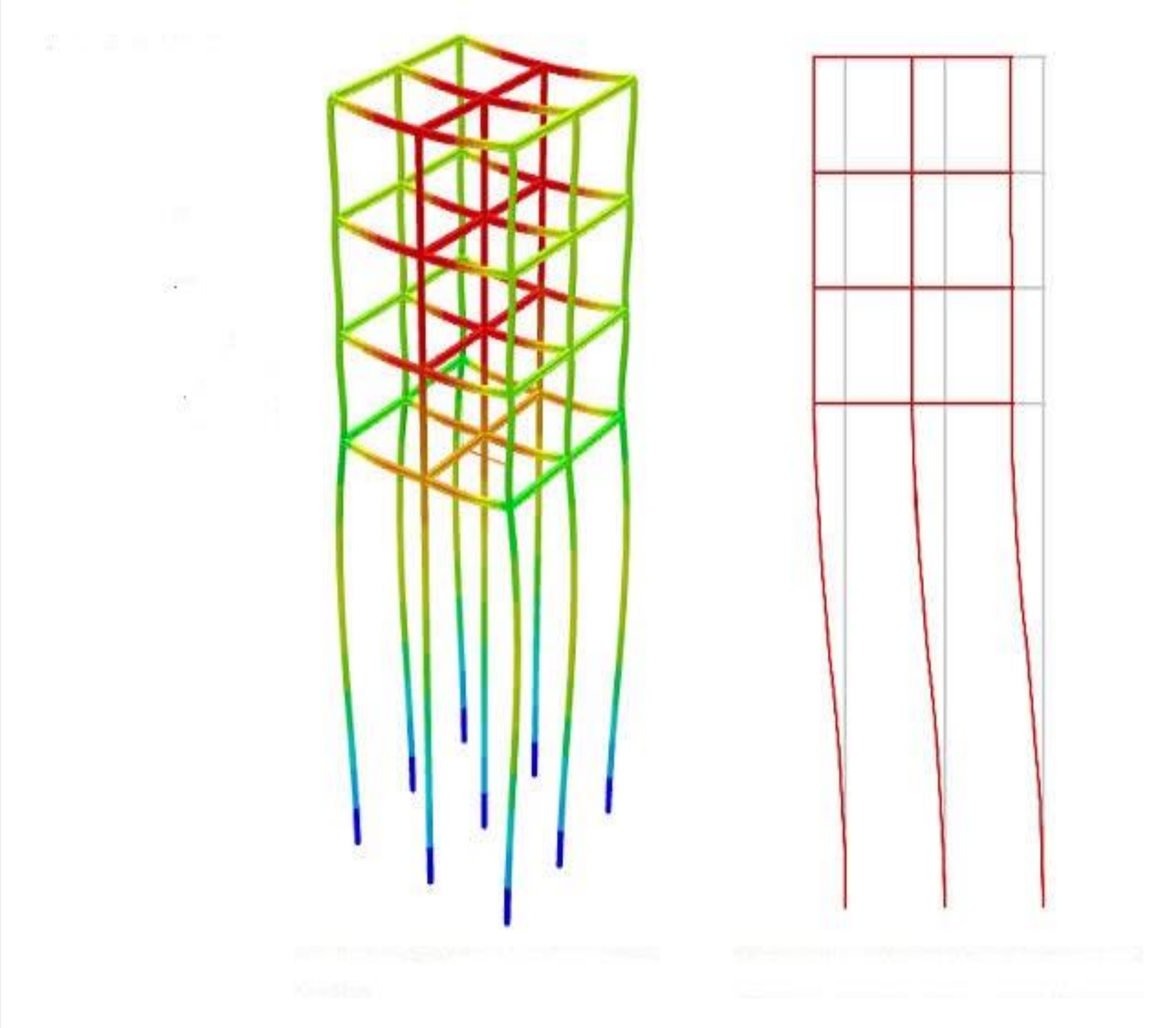


Buckling Analysis

Buckling occurs physically when a structure becomes unstable under a given loading configuration, and mathematically when a bifurcation occurs in the solution to equations of static equilibrium. The two primary means for performing buckling analysis include Eigenvalue and Nonlinear buckling analyses. Buckling must be explicitly evaluated for each set of loads considered because, unlike natural frequencies, buckling modes are dependent upon a given load pattern. When evaluating buckling, any number of load cases may be defined, each of which should specify loading, convergence tolerance, and the number of modes to be found. Since the first few buckling modes may have similar factors, we recommend finding a minimum of six modes.

Modify Buckling analysis to include P-Delta important for incremental buckling loads

P-Delta load + BF X applied load = buckling load



Buckling Analysis

Buckling modes calculated 1

Buckling Factor X Applied load = Buckling Load

Loads applied x Buckling factor = Buckling loads

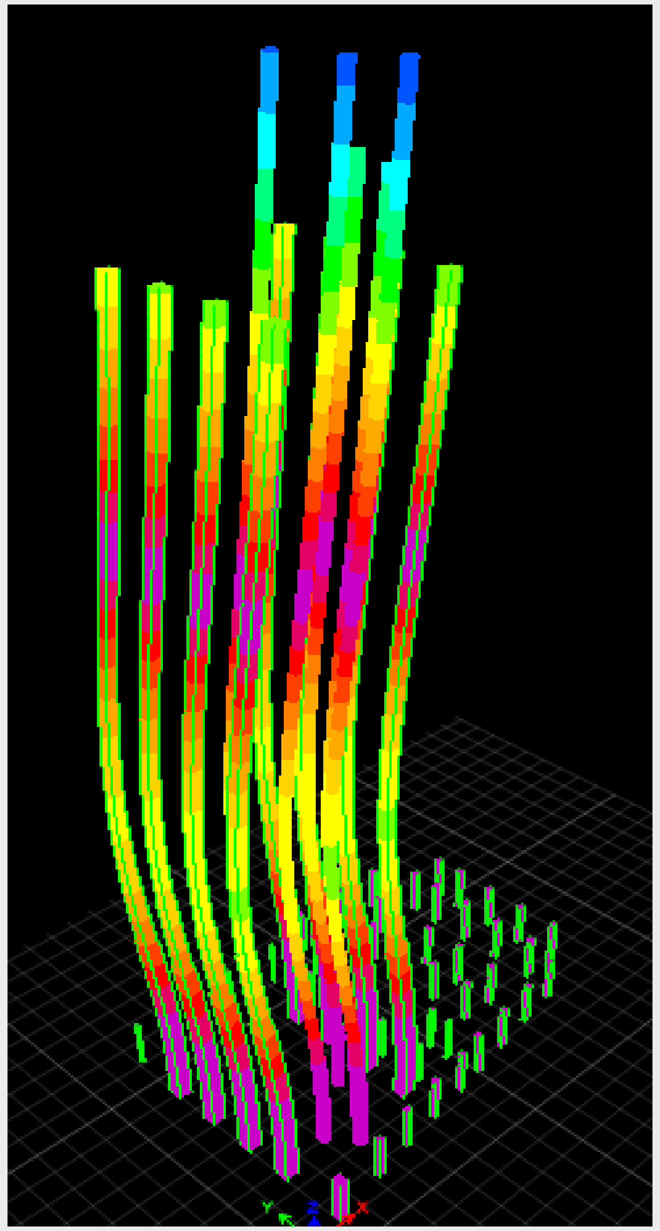
For mode 1 = 3.281

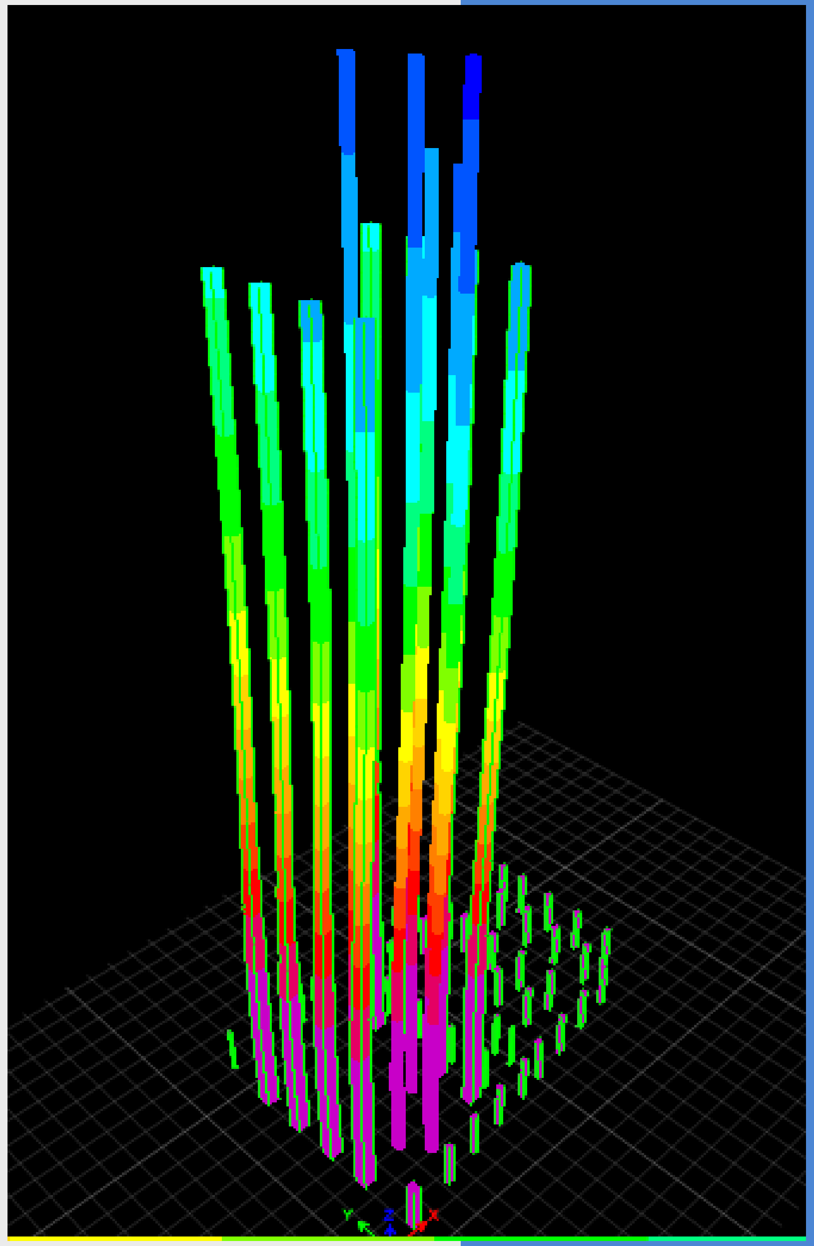
3.281 x (DL + LL) = Buckling load

P-Delta load + BF X applied load = buckling load (DL+ SDL + 0.5 LL) + 3.281 X (DL+LL) = buckling load

APPLIED LOAD UP = BUCKLING FACTOR DOWN

TABLE: Buckling Factors		
Case	Mode	Scale Factor
Buckling	1	1
Buckling	2	1.21
Buckling	3	1.119
Buckling	4	1.132
Buckling	5	1.392
Buckling	6	1.311





Time History Analysis

Time-history analysis provides for linear or nonlinear evaluation of dynamic structural response under loading which may vary according to the specified time function. Dynamic equilibrium equations, given by $K\,u(t) + C\, {}^{d}/_{dt}\, u(t) + M\, d^2/_{dt}\, u(t) = r(t)$, are solved using either modal or direct-integration methods. Initial conditions may be set by continuing the structural state from the end of the previous analysis.

This method calculates the response of a structure subjected to earthquake excitation at every instant of time (hence the name Time History). Various seismic data are necessary to carry out the seismic analysis i.e. acceleration, velocity, displacement data, etc., which can be easily procured from seismograph data's analysis for any particular earthquake. It is an important technique for structural seismic analysis especially when the evaluated the structural response is nonlinear.

Time function

CSI Software handles the initial conditions of a time function differently for linear and **nonlinear** time-history **load cases.**

- **Linear cases** always start from zero, therefore the corresponding time function must also start from zero.

- **Nonlinear cases** may either start from zero or may continue from a previous case. When starting from zero, the time function is simply defined to start with a zero value. When analysis continues from a previous case, it is assumed that the time function also continues relative to its starting value. A long record may be broken into multiple sequential analyses which use a single function with arrival times. This prevents the need to create multiple modified functions.

Non linear cases two types

- Fast non linear analysis (FNA)
- Direct integration time history analysis

Fast Nonlinear Analysis (FNA)

Fast Nonlinear Analysis (FNA) is a modal analysis method useful for the static or dynamic evaluation of linear or nonlinear structural systems. Because of its computationally efficient formulation, FNA is well-suited for time-history analysis, and often recommended over direct-integration applications. During dynamic-nonlinear FNA application, analytical models should:

- Be primarily linear-elastic.
- Have a limited number of predefined nonlinear members.
- Lump nonlinear behavior within link objects.

Direct-integration time-history analysis

Direct-integration time-history analysis is a , nonlinear dynamic analysis method in which the equilibrium equations of motion are fully integrated as a structure is subjected to dynamic loading. Analysis involves the integration of structural properties and behaviors at a series of time steps which are small relative to loading duration. The equation of motion under evaluation is given as follows:

$$M\ddot{u}(t) + C\dot{u}(t) + Ku(t) = F(t)$$

Comparison between FNA and direct-integration time-history analyses

Nonlinear **modal** time-history analysis, also known as Fast Nonlinear Analysis (FNA), is generally more accurate and efficient than direct-integration **time-history** analysis. The accuracy of FNA depends upon the sufficiency of suitable mode shapes, similar to how direct integration requires small enough time steps to accurately characterize dynamic behavior.

Damping is handled differently between these two analysis methods. FNA limits proportional damping at the frequency extremes to 0.99995 that of critical, while direct integration uses i**mass- and stiffness-proportional damping** n which damping at very low and very high frequencies may exceed critical.

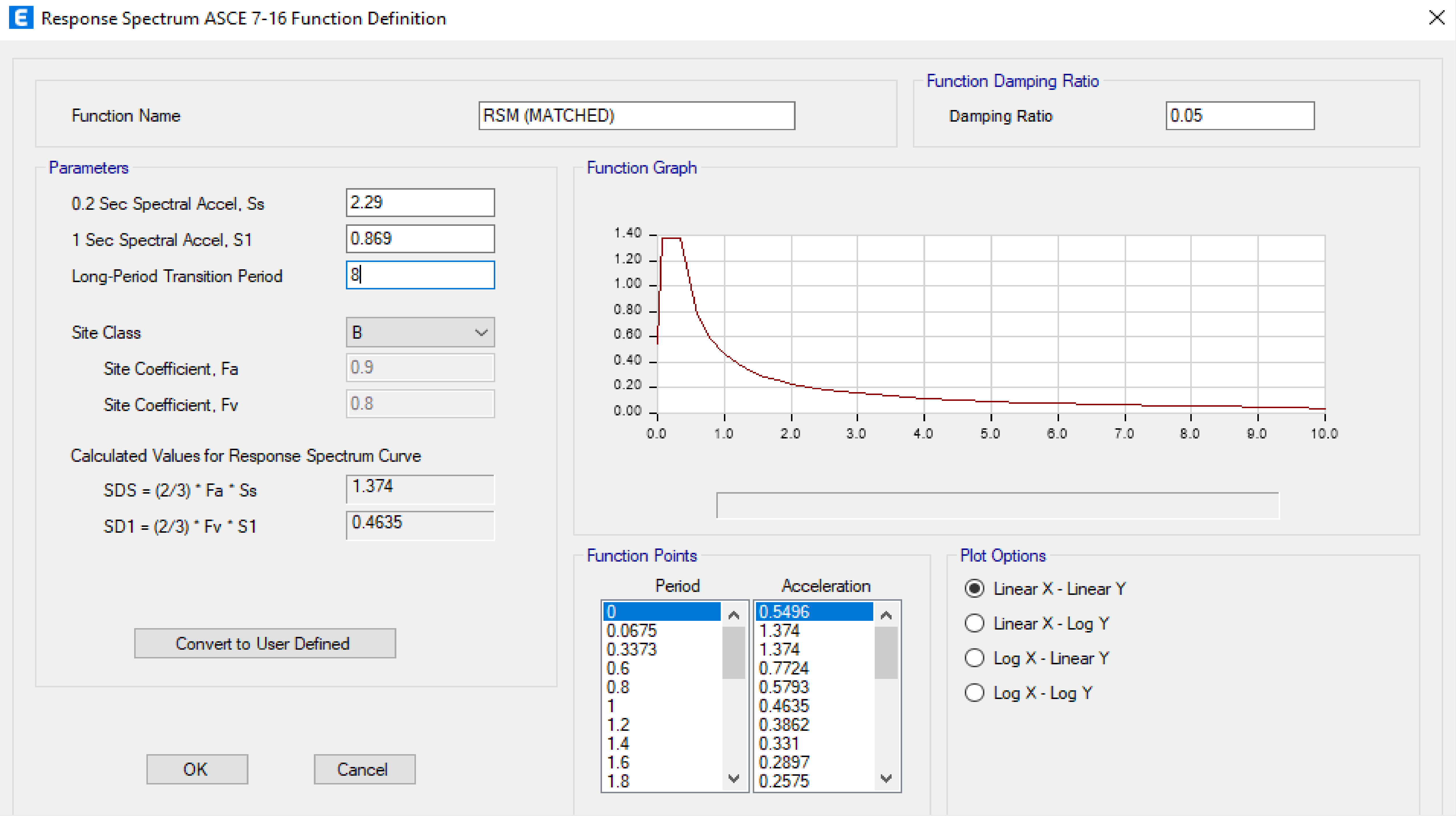
We recommend using the default Convergence tolerance (1e-4) during FNA application, and for direct integration, a tolerance equal to or less than 1e-3.

Results may be sensitive to physical parameters, loading conditions, and the analytical technique applied, especially with irregular structures and advanced nonlinear systems. Since FNA is an accurate and efficient analysis method, it may be worthwhile to apply this technique to a series of models which simulate variable computational scenario. For example, foundation springs and substructure may be included, then omitted, to provide a comparison study.

Time History Analysis

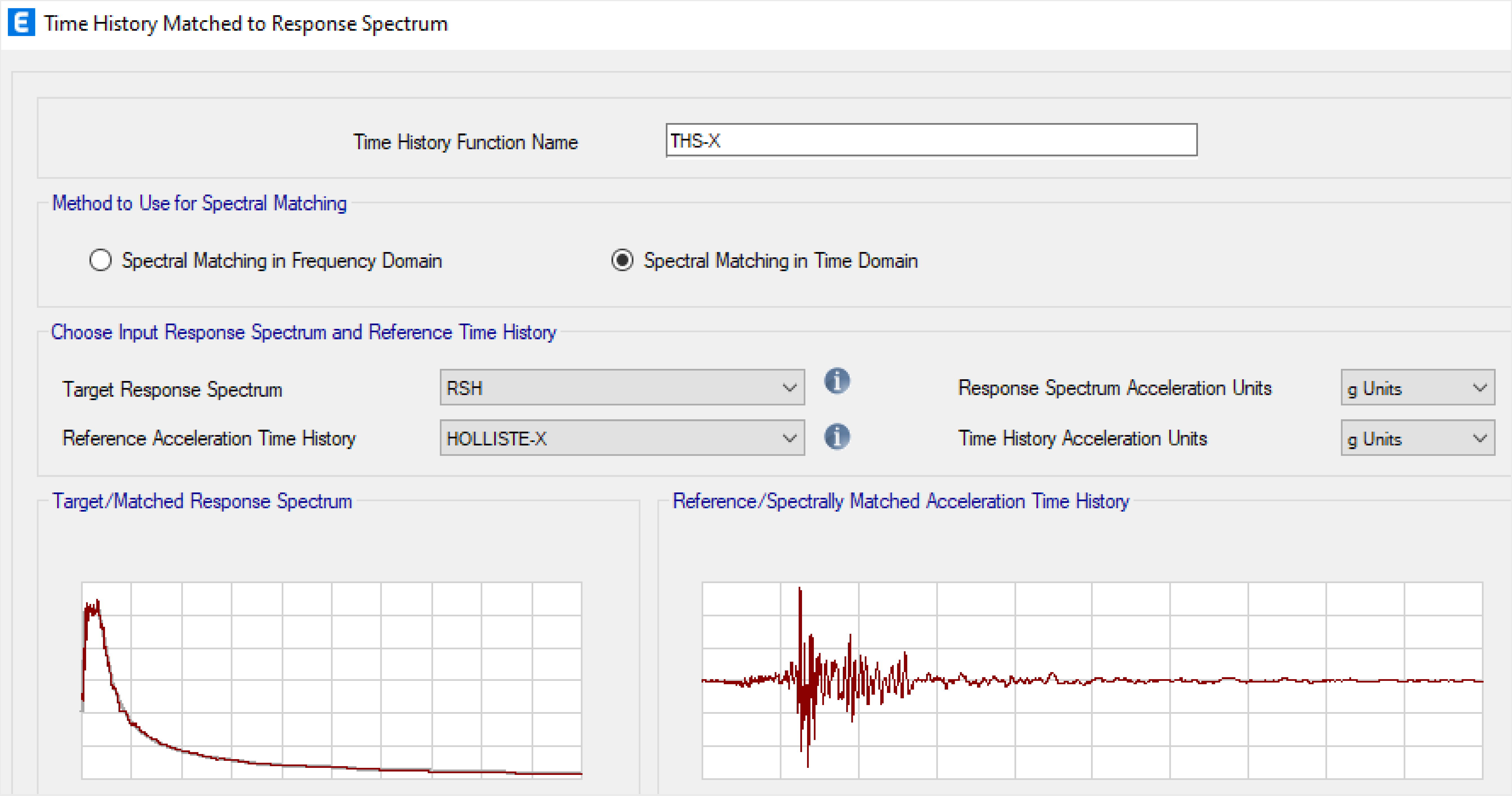
- Provides for linear or nonlinear evaluation of dynamic structural response under loading which may vary according to time function.
- Define the target response spectrum
- Matching the time history with target response spectrum
- As per ASCE 7-16-Scaled time history has to be greater than target response spectrum from 0.2T to 1.5T [ASCE Cl: C16.2.3.1]

Linear Model



Time History Analysis

- As per ASCE 7-16-Scaled time history has to be greater than target response spectrum from 0.2T to 1.5T [ASCE Cl: C16.2.3.1]



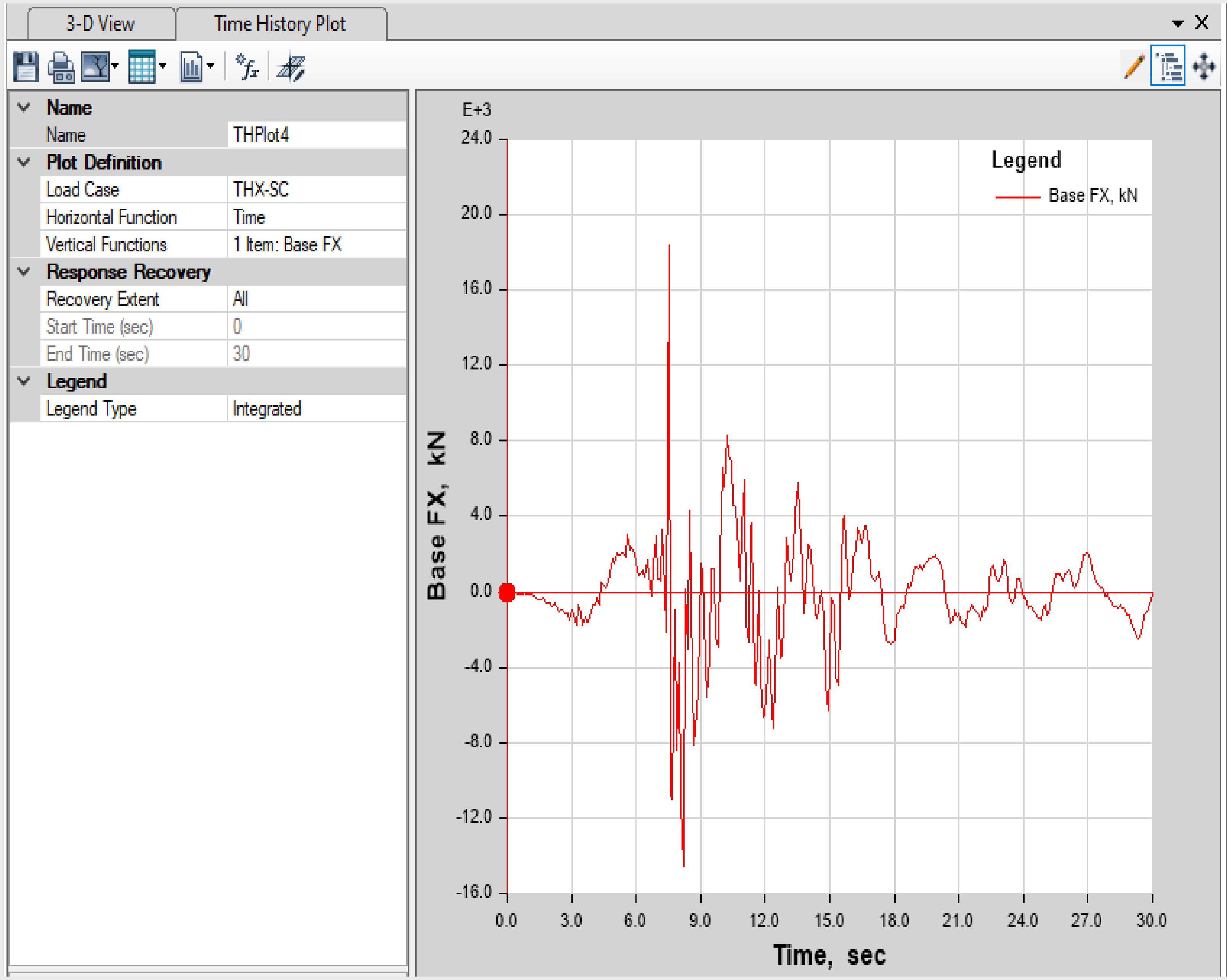
Time History Analysis

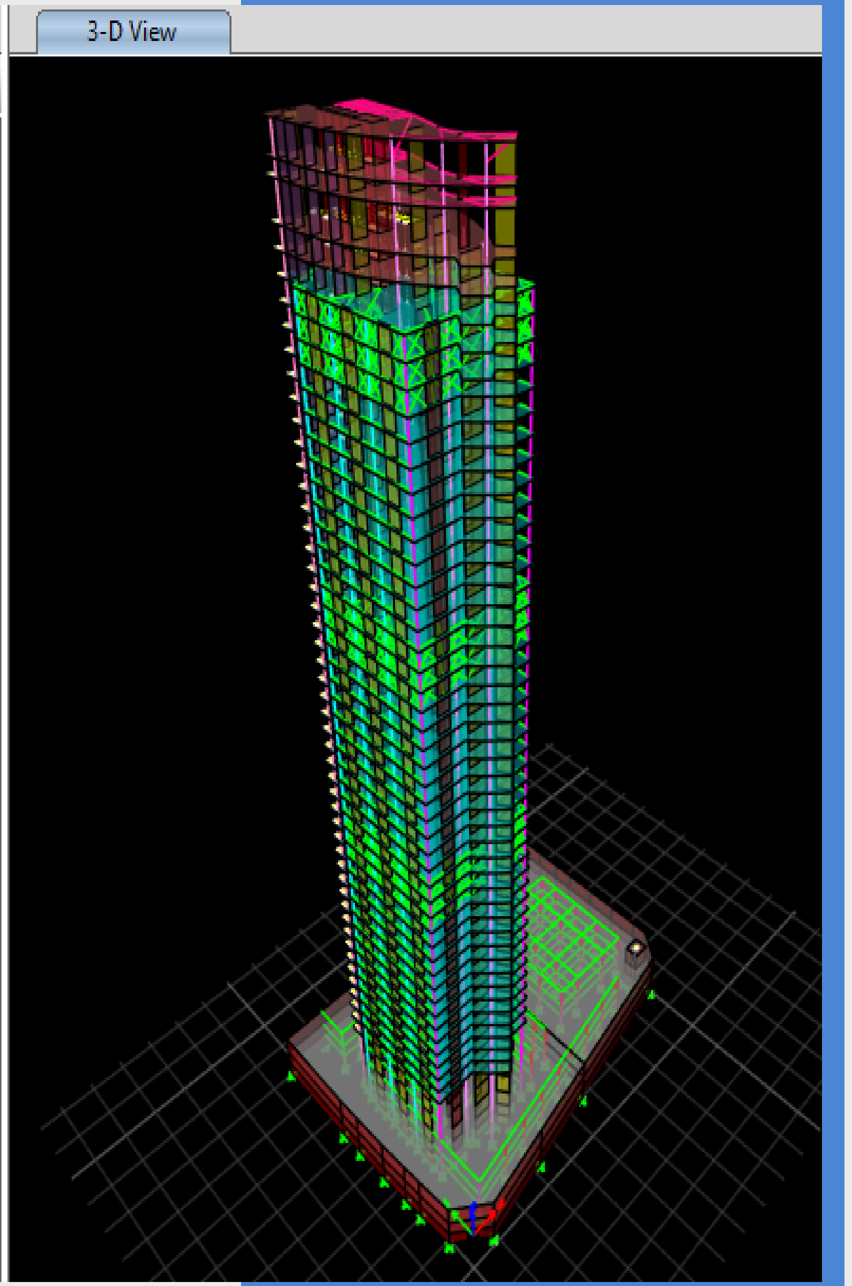
Base Shear

.Scale factor = (Ex / Spec x) x Existing Scale factor.

As per ASCE 7-16 FX or FY of THX max. > = 0.85 x calculated base shear (EX or EY)

Load Cases	Base Shear
EQ-X	72145.8763
EQ-Y	72396.7837
THX-X	71142.2374
THY-Y	71236.7249

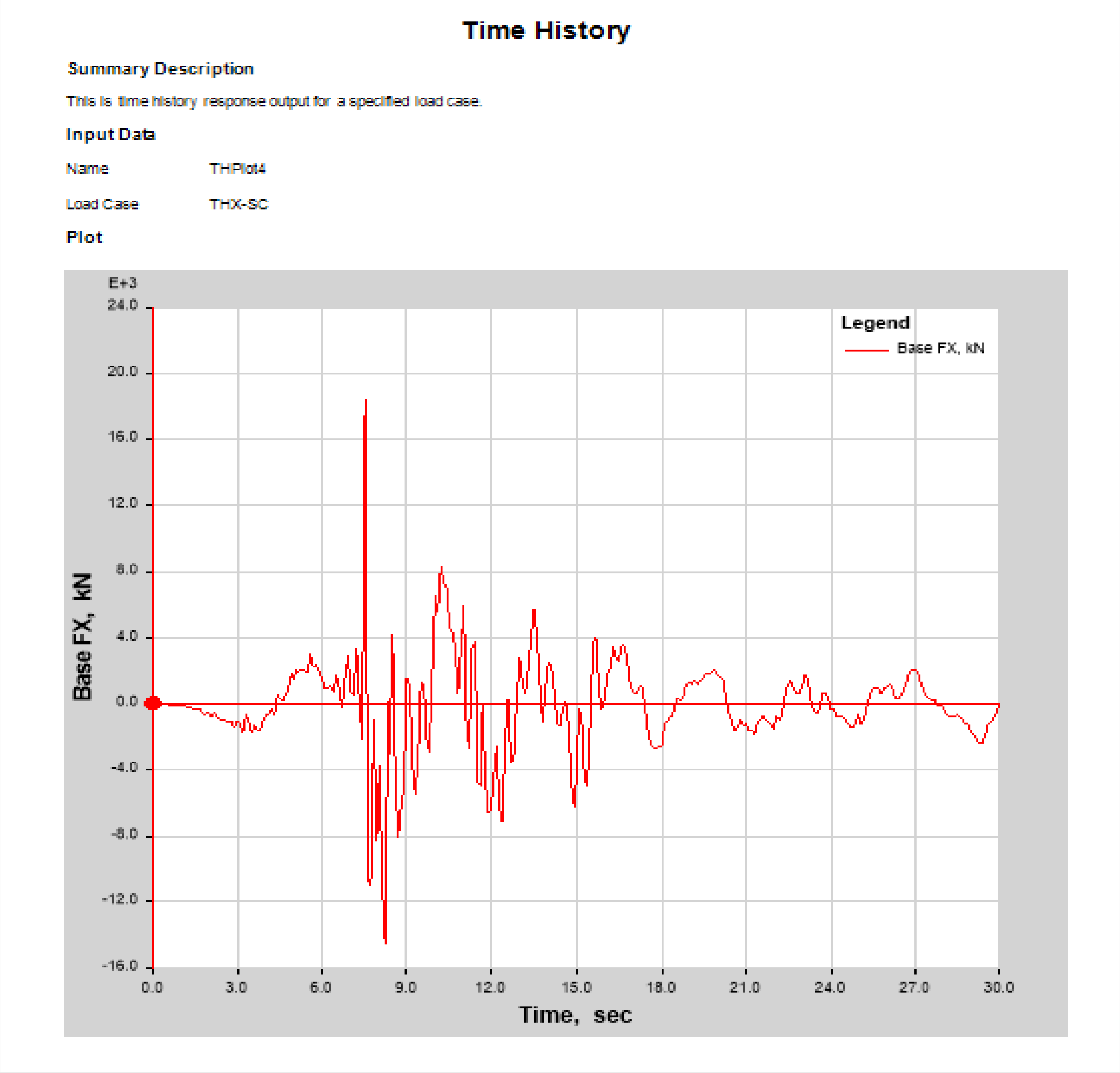


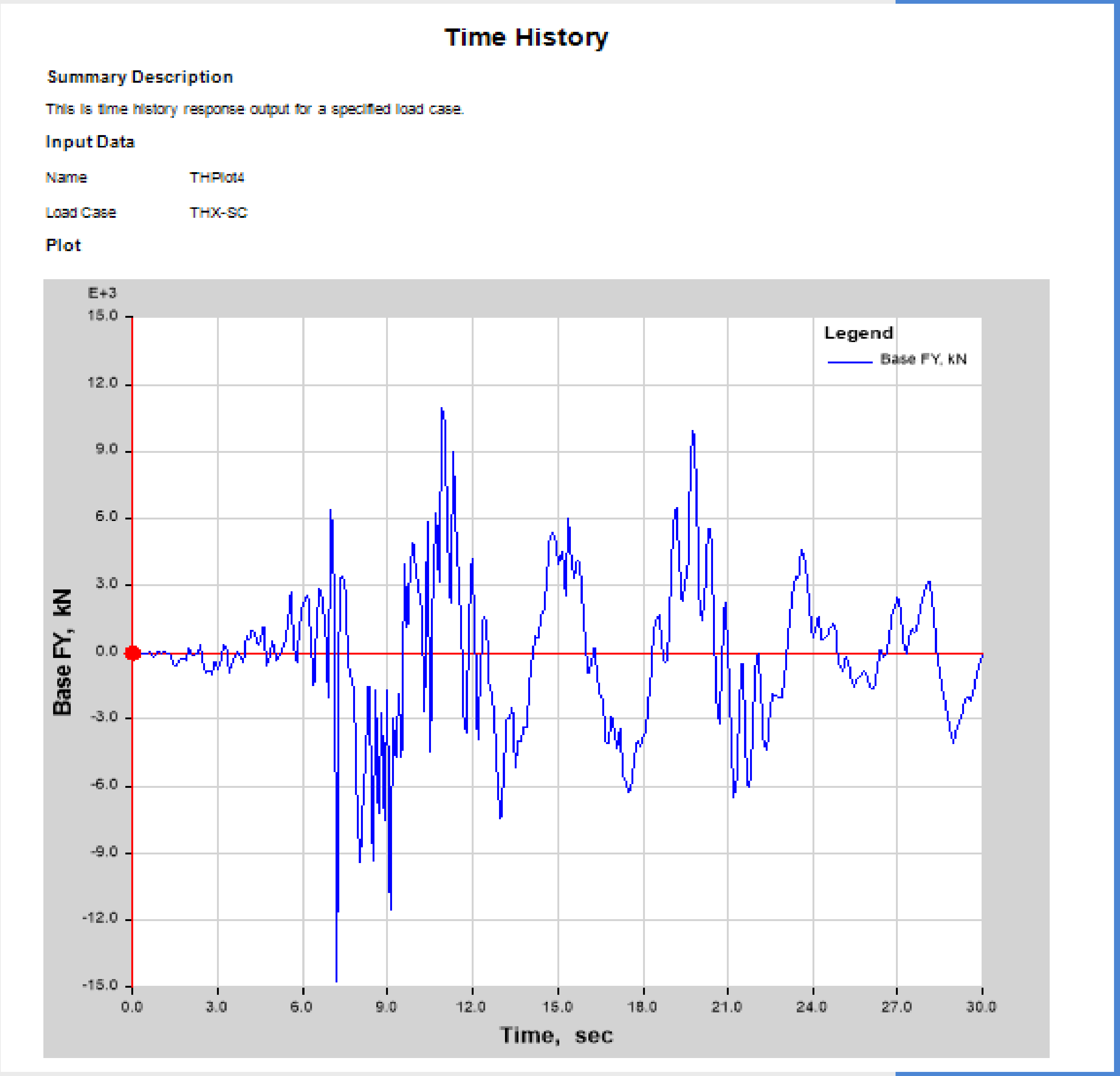


Time History Analysis

Base FX
Base Shear Max.=18407.34KN
Time=7.5 sec.

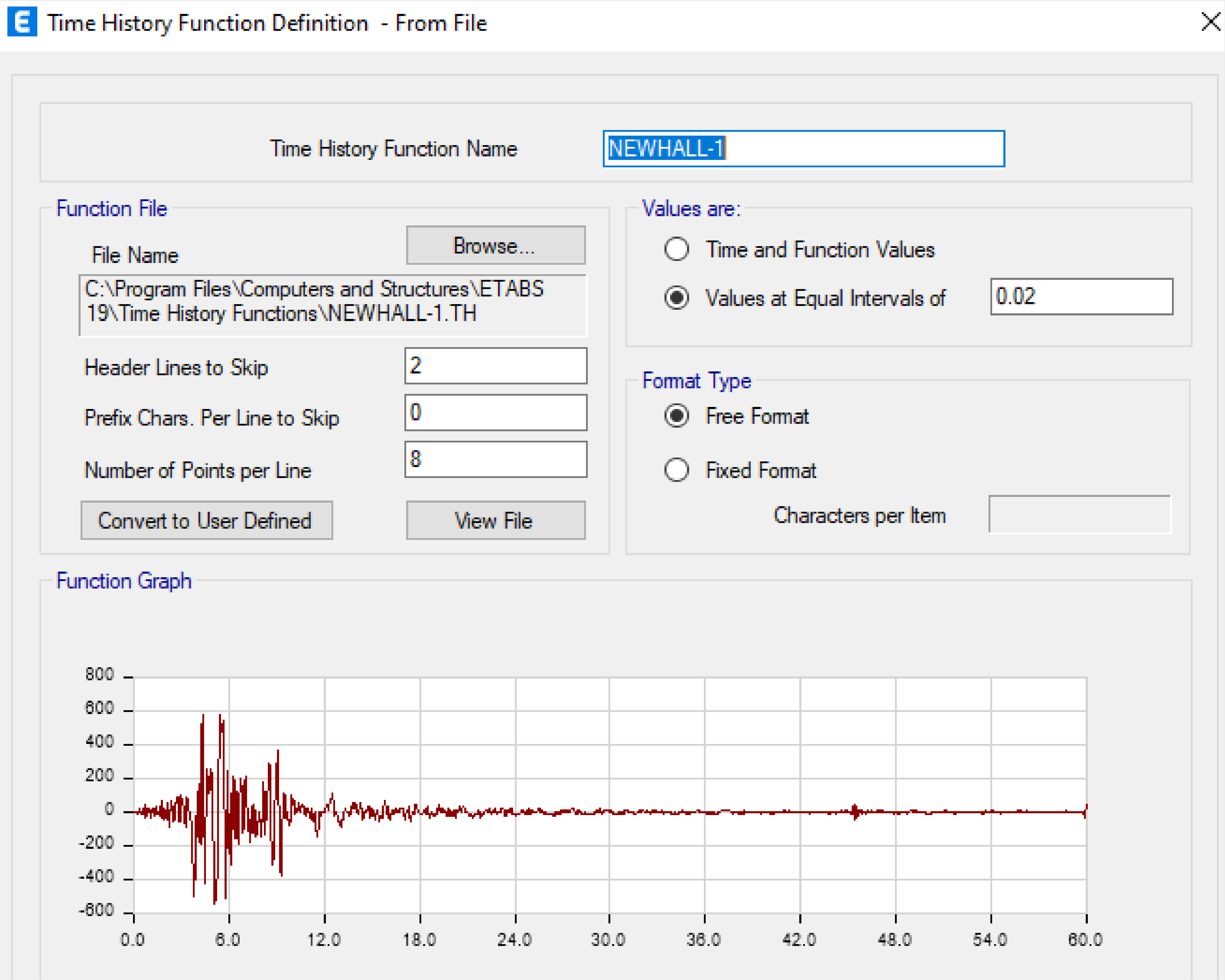
Base FX
Base Shear Max.=10948.72KN
Time=10.9 sec.





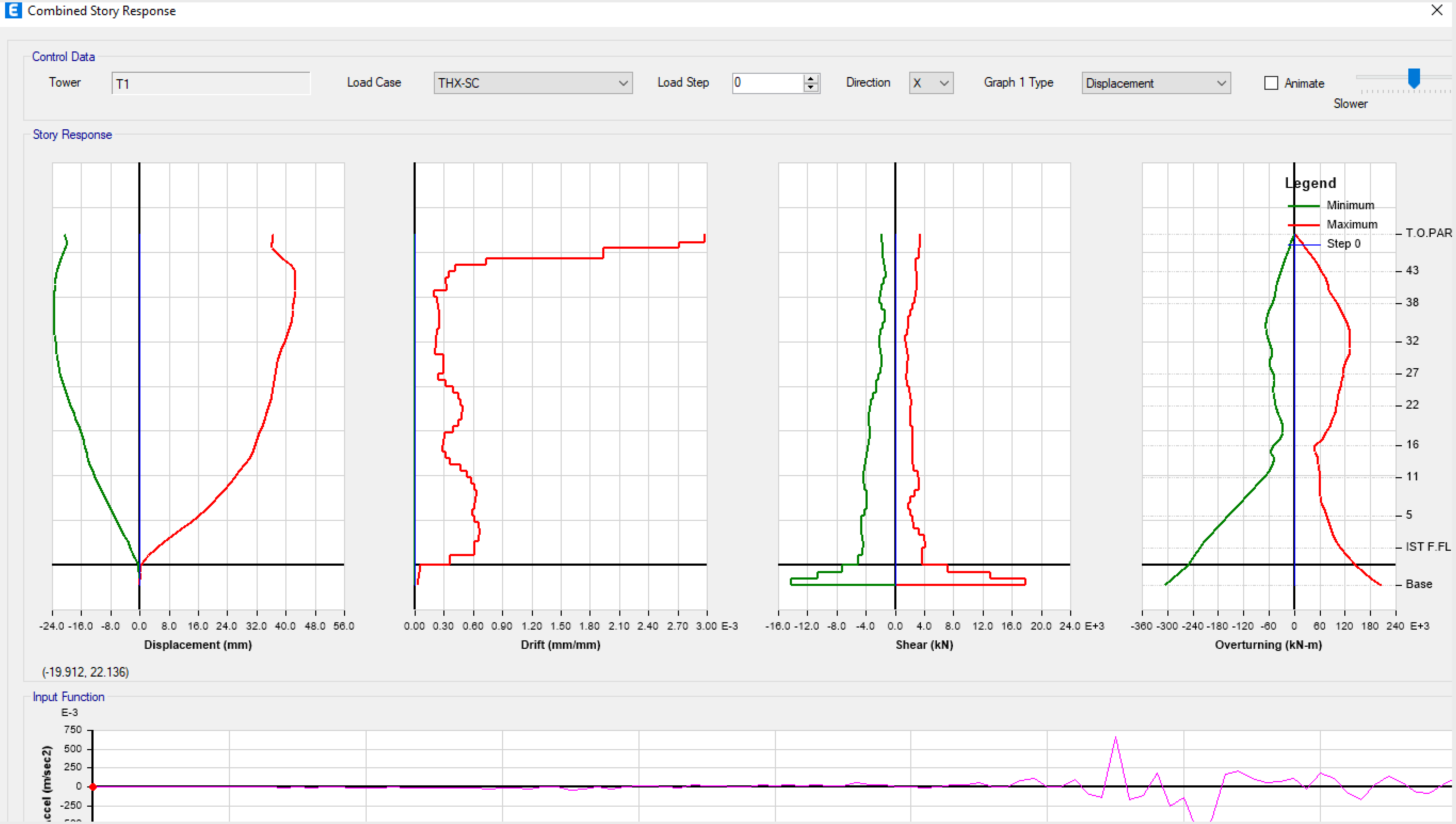
Time History Analysis

Non Linear FNA



Time History Analysis

- We can see the various effects on seismic waves on our building model graphs each graph represents the variation effects for different stories
- As per ASCE code the maximum story drift value not exceed is ΔM = 0.7R ΔS

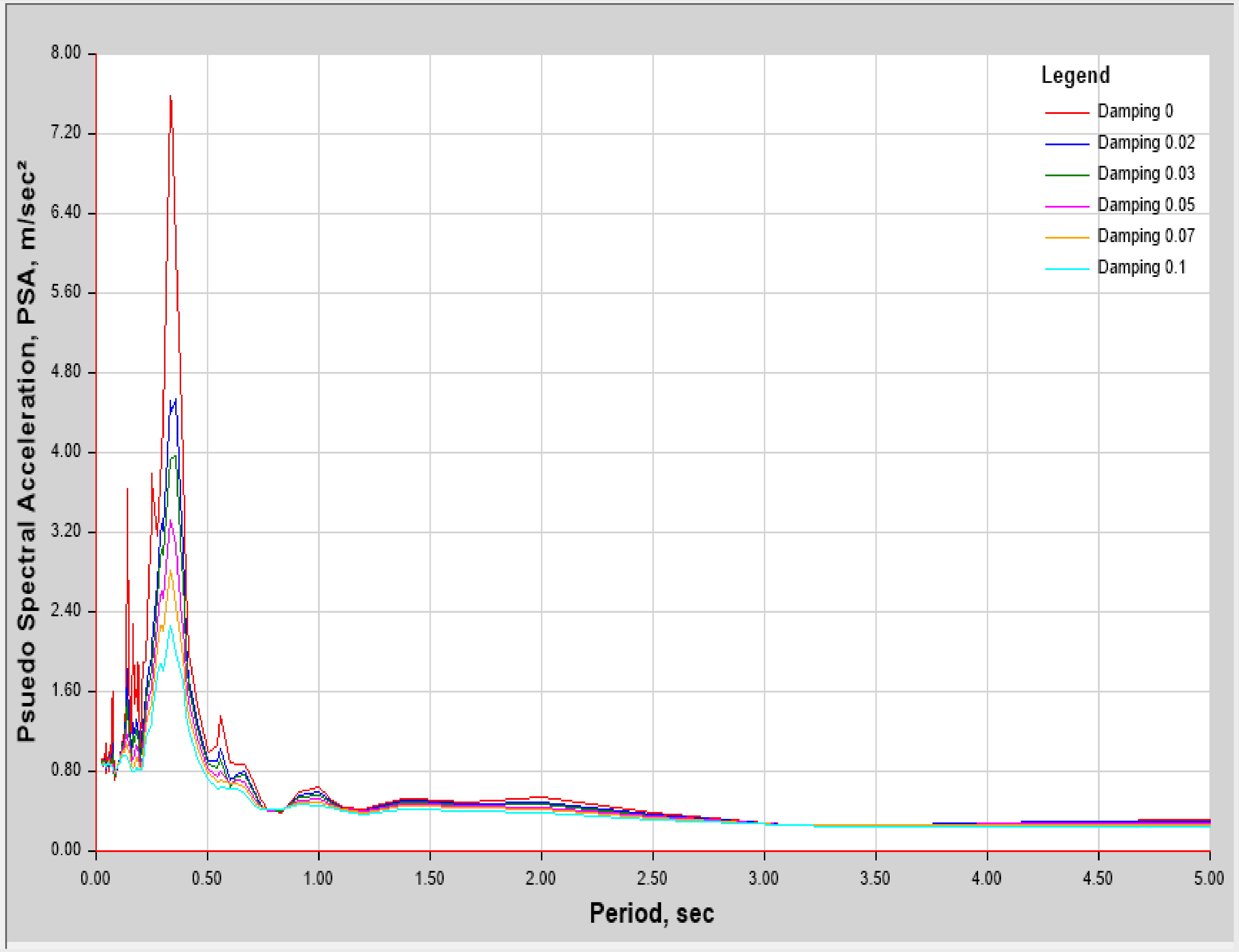


Time History Analysis

For THX in X direction we can see the max. response here 0.33 sec, 7.585018 value of PSA for min. damping i.e. 0 damping for maximum damping it is very less for damping factor 0.

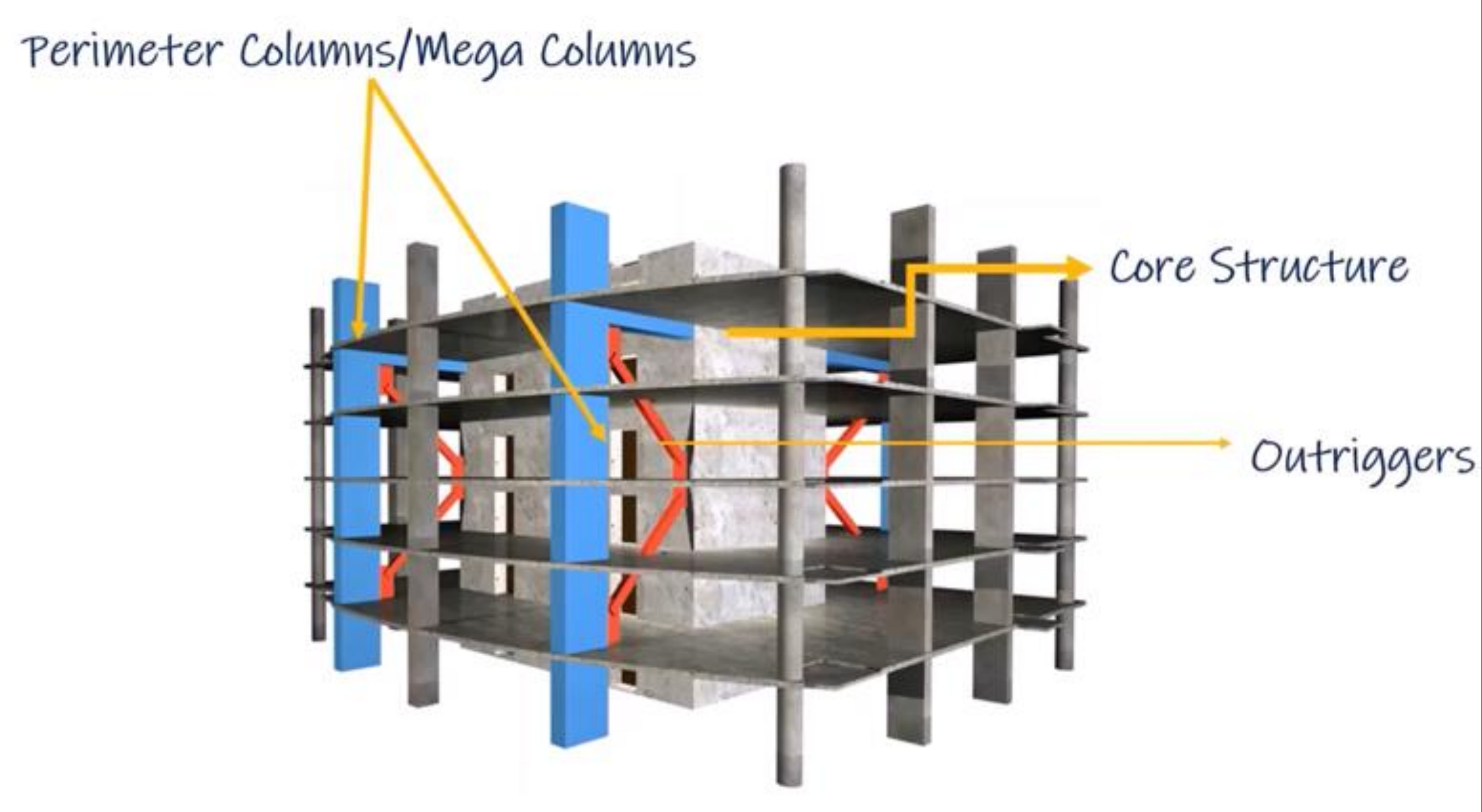
Name	
Name	RSFromTH1
Show	
Time History Case	THX-SC
Story	T.O.PARAPET
Joint Label	124
Coordinate System	Global
Response Direction	X
Vertical Axis	
Item Plotted	PSA
Scale Type	Linear
Units	Current Units
Horizontal Axis	
Item Plotted	Period
Scale Type	Linear
Widening (%)	0
Frequencies/Periods	
Include Default	Yes
Include Structural	Yes
Include User	No
User Values	None
Damping	
Damping Ratios	0; 0.02; 0.03; 0.05; 0.07; 0.1
Legend	
Legend Type	Integrated

Story
The story at which the time history results are obtained.



Outrigger Structural System

- The *outrigger truss* is a simple trusses spanning over the full height of that story and across the full width of building.
- The role of the outrigger trusses is to make the columns act together in resisting overturning moments acting on the building.
- The pairs of columns generate couples of axial tension and axial compression to counter the overturning moments; this reduces the overall bending effects in columns.



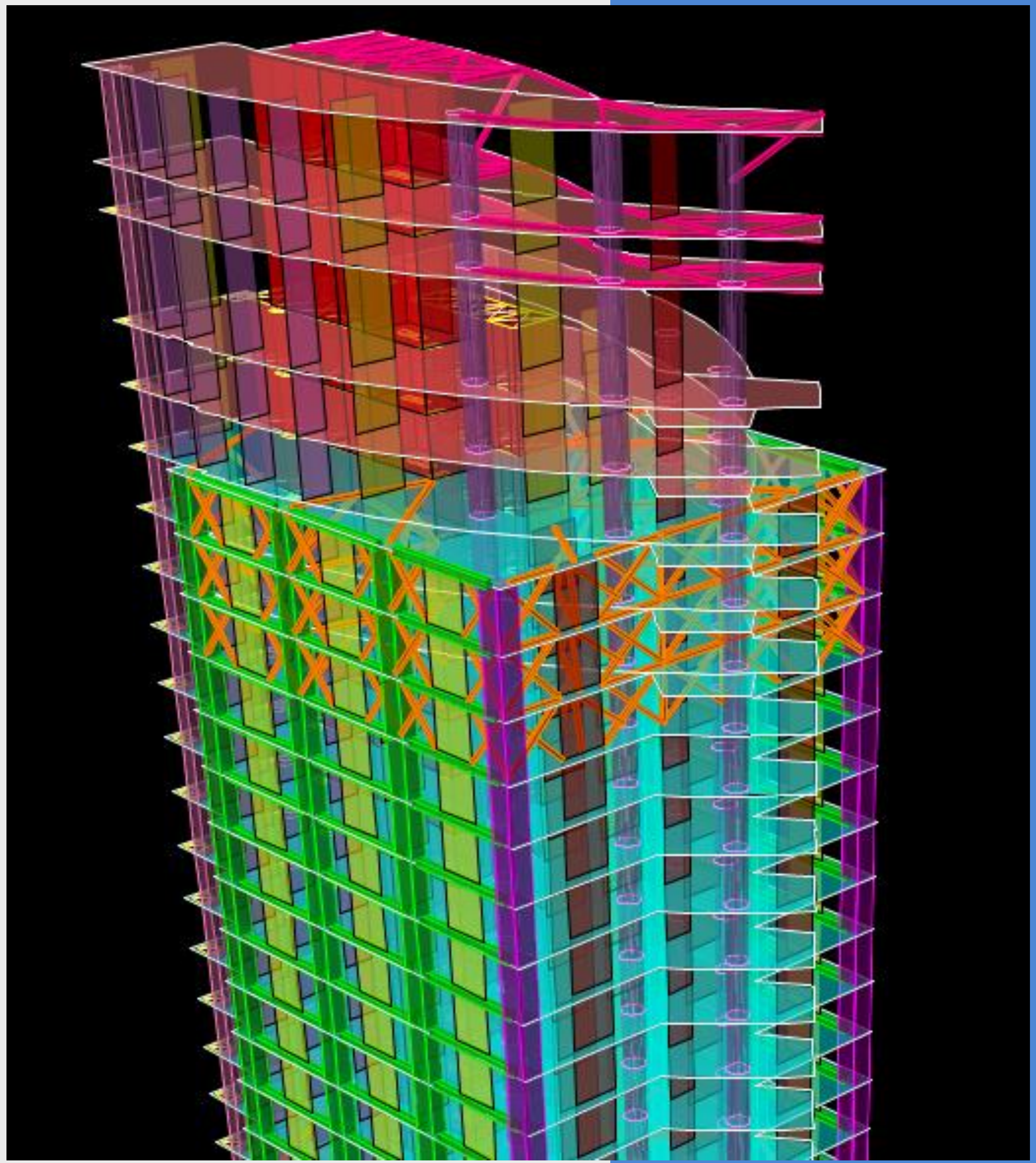
- Interior Lateral structural systems provided to improve the overturning stiffness and strength of high-rise buildings.

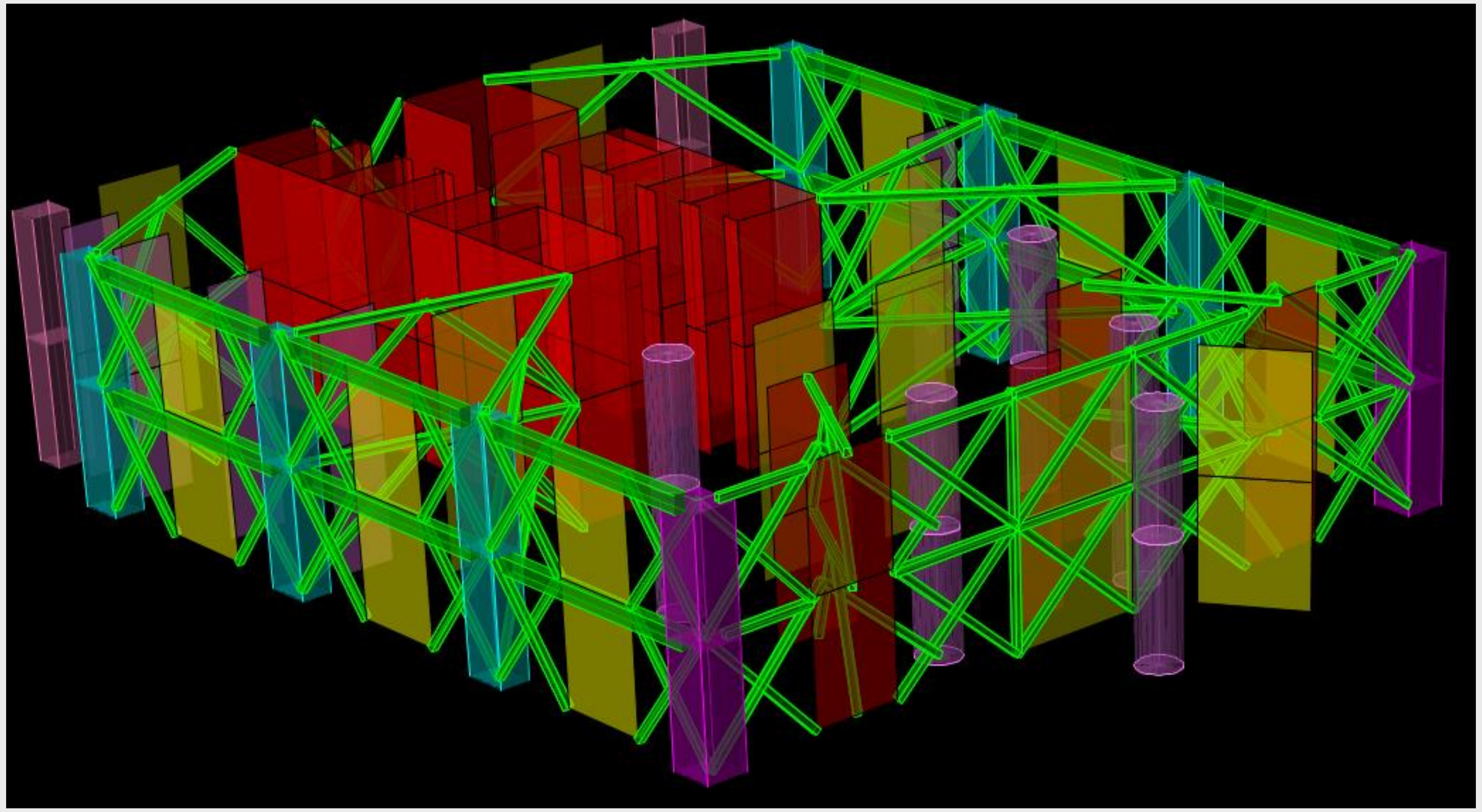
Outrigger Structural System

Outrigger are rigid horizontal structures designed to improve building overturning stiffness and strength by connecting the core to closely spaced outer columns

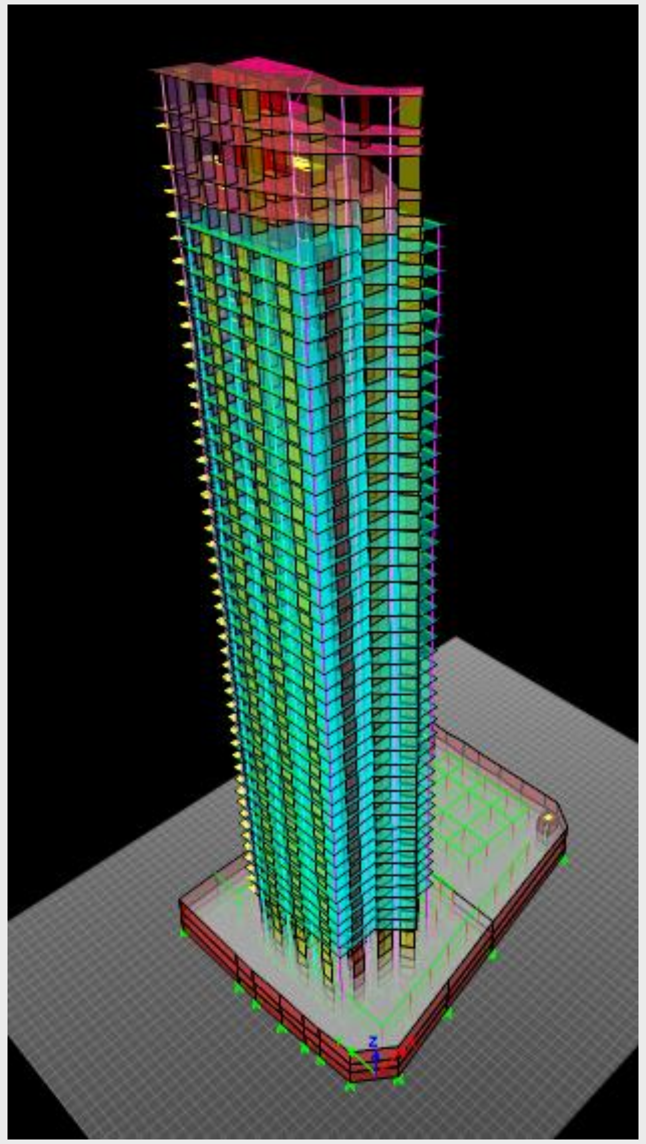
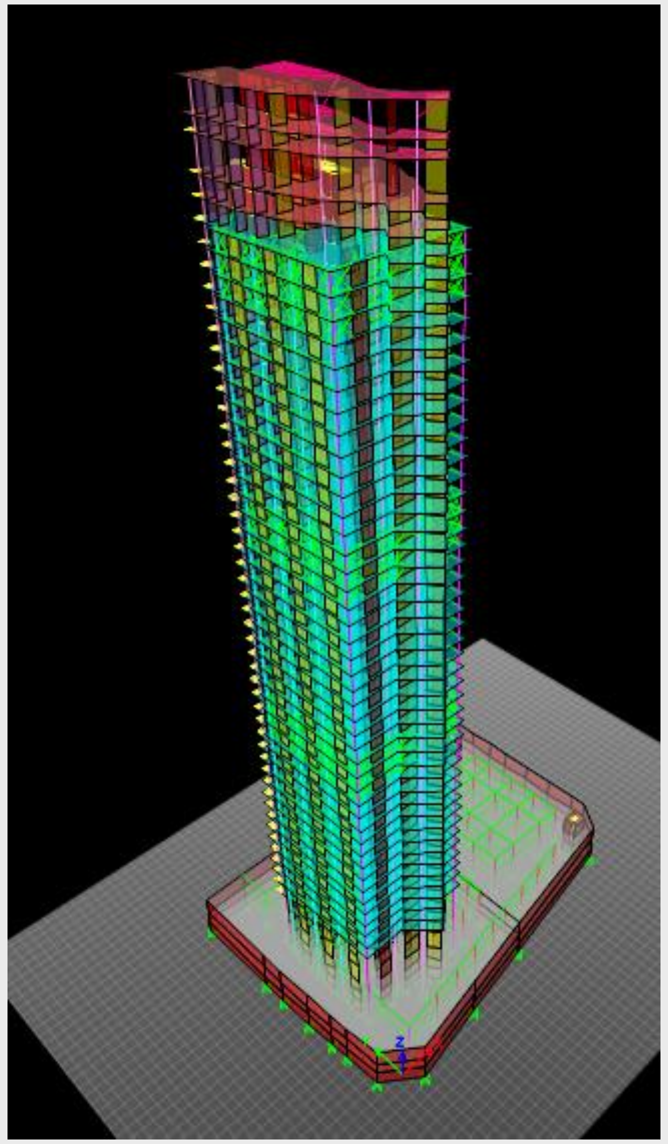
This form consists of central core either braced frames/ shear walls with horizontal cantilever “outrigger” trusses or girders connecting the core to outer columns and makes the structure to behave as partly composite cantilever

The outriggers are in form of walls in reinforced concrete building and trusses in steel structures. Multilevel outrigger systems can provide up to five times the moment resistance of a single outrigger system.

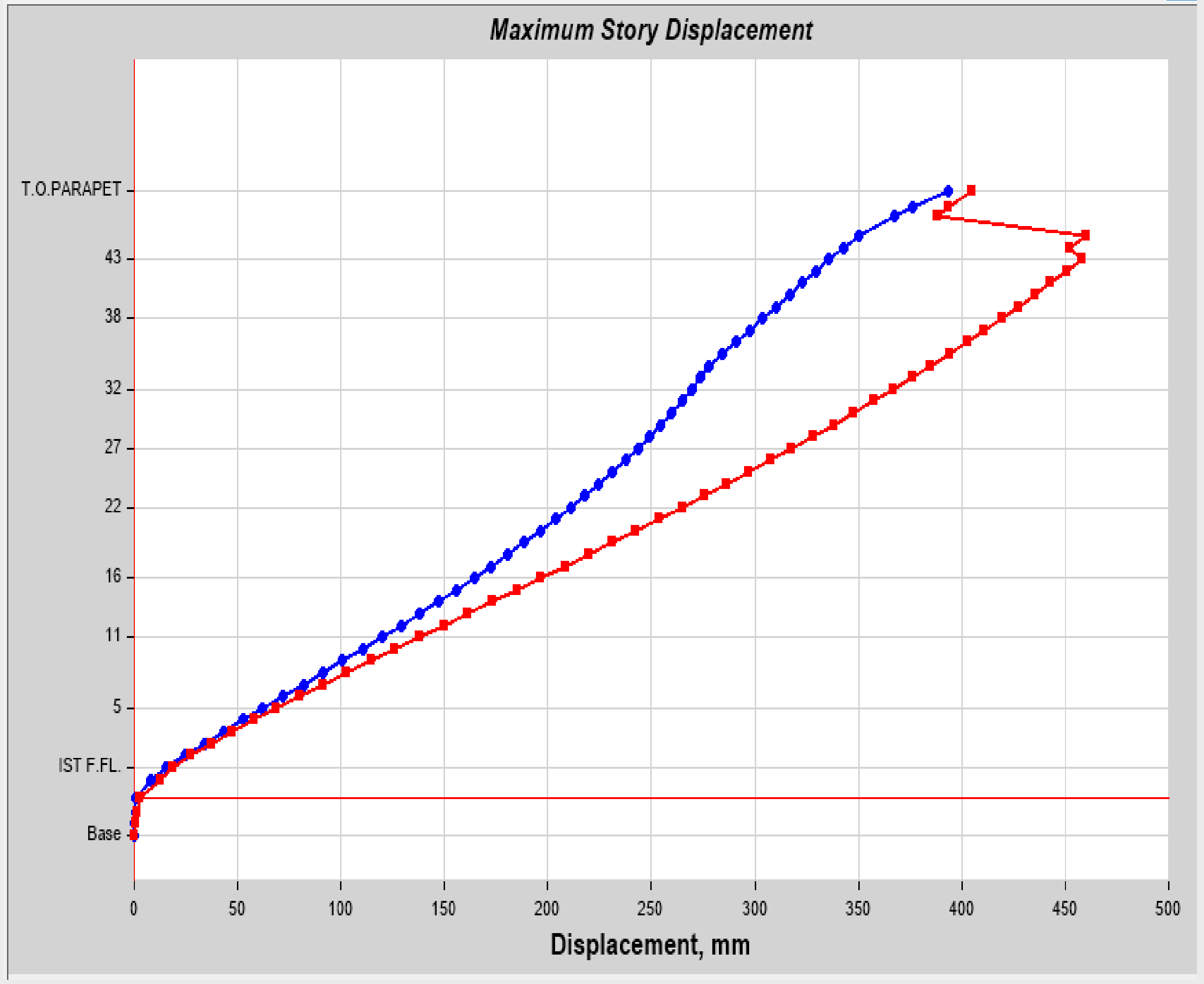
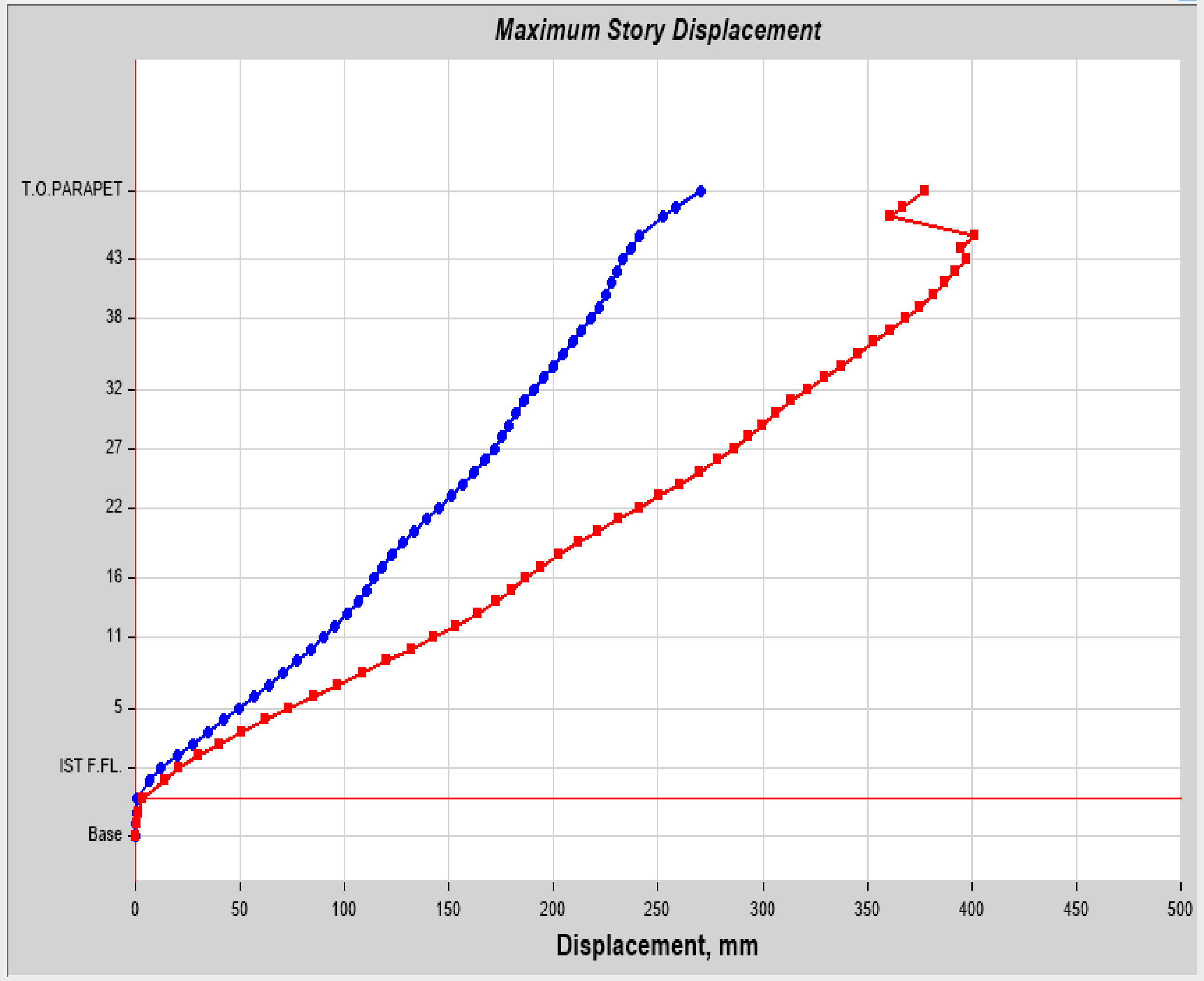




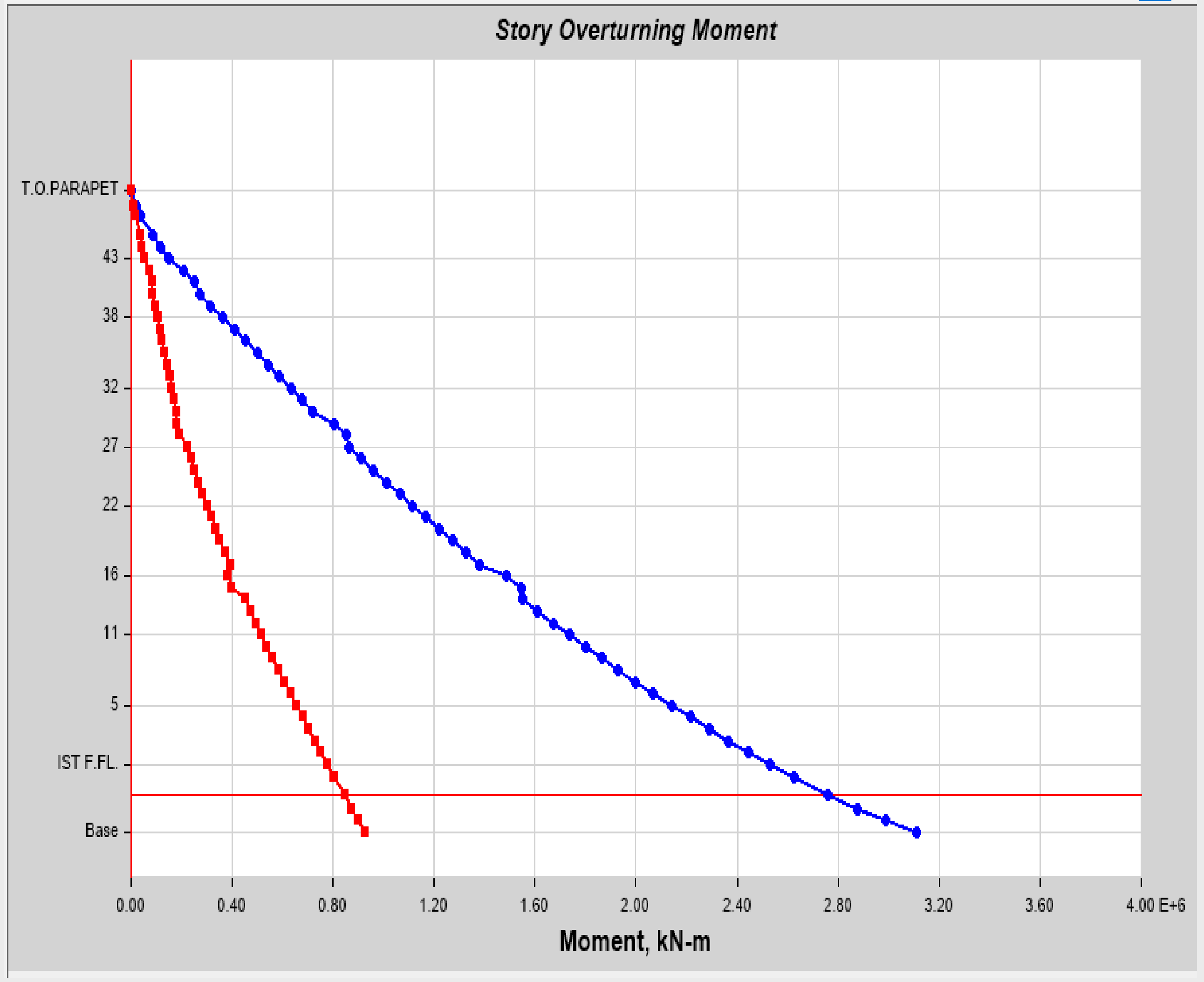
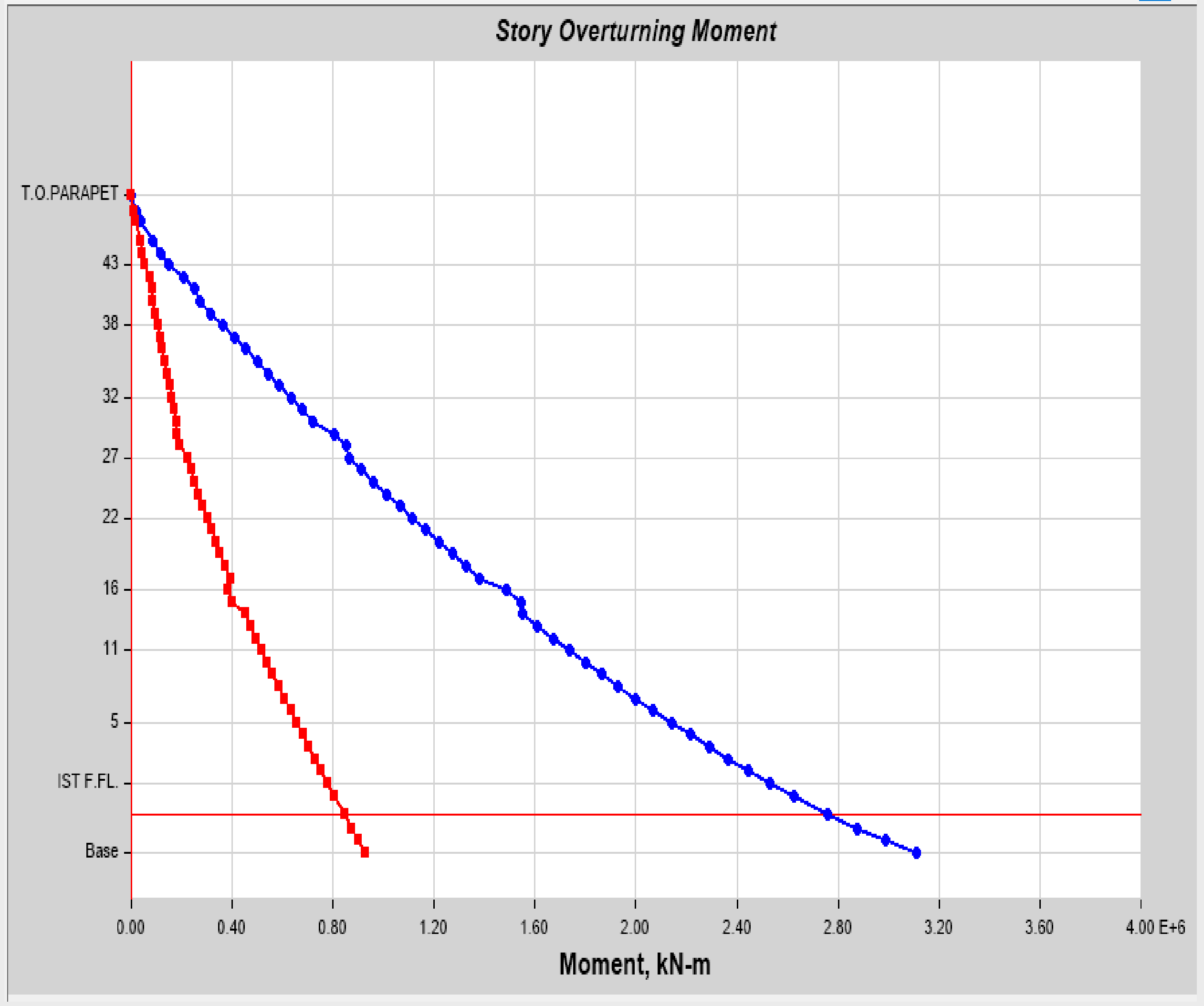
Compare Outrigger Structural System
Vs
Conventional Structural System

Conventional System	Outrigger System
	

Compare Outrigger Structural System
Vs
Conventional Structural System

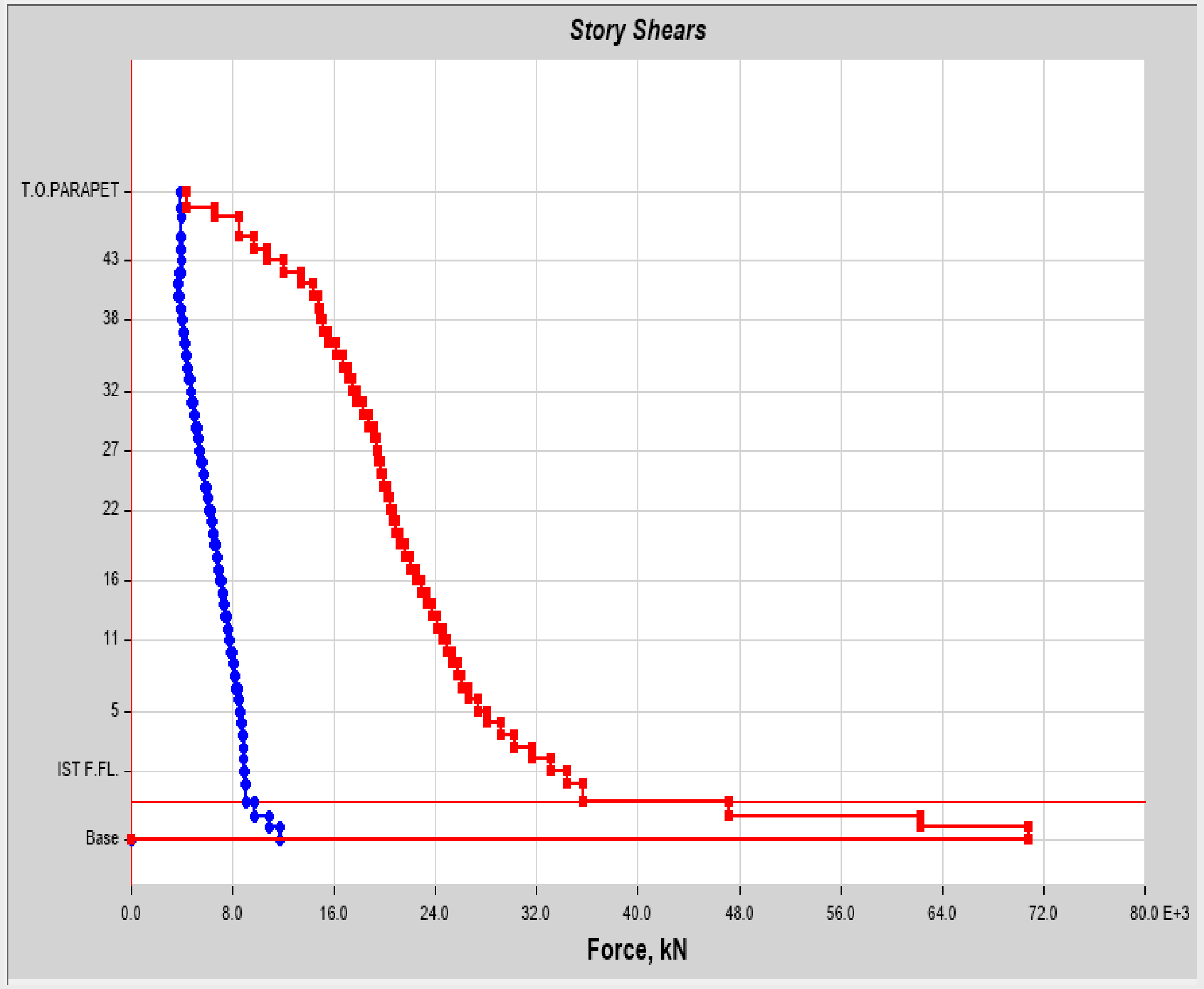
Conventional System	Outrigger System
	
Maximum displacement=659mm SpecY	Maximum displacement= 401mm SpecY

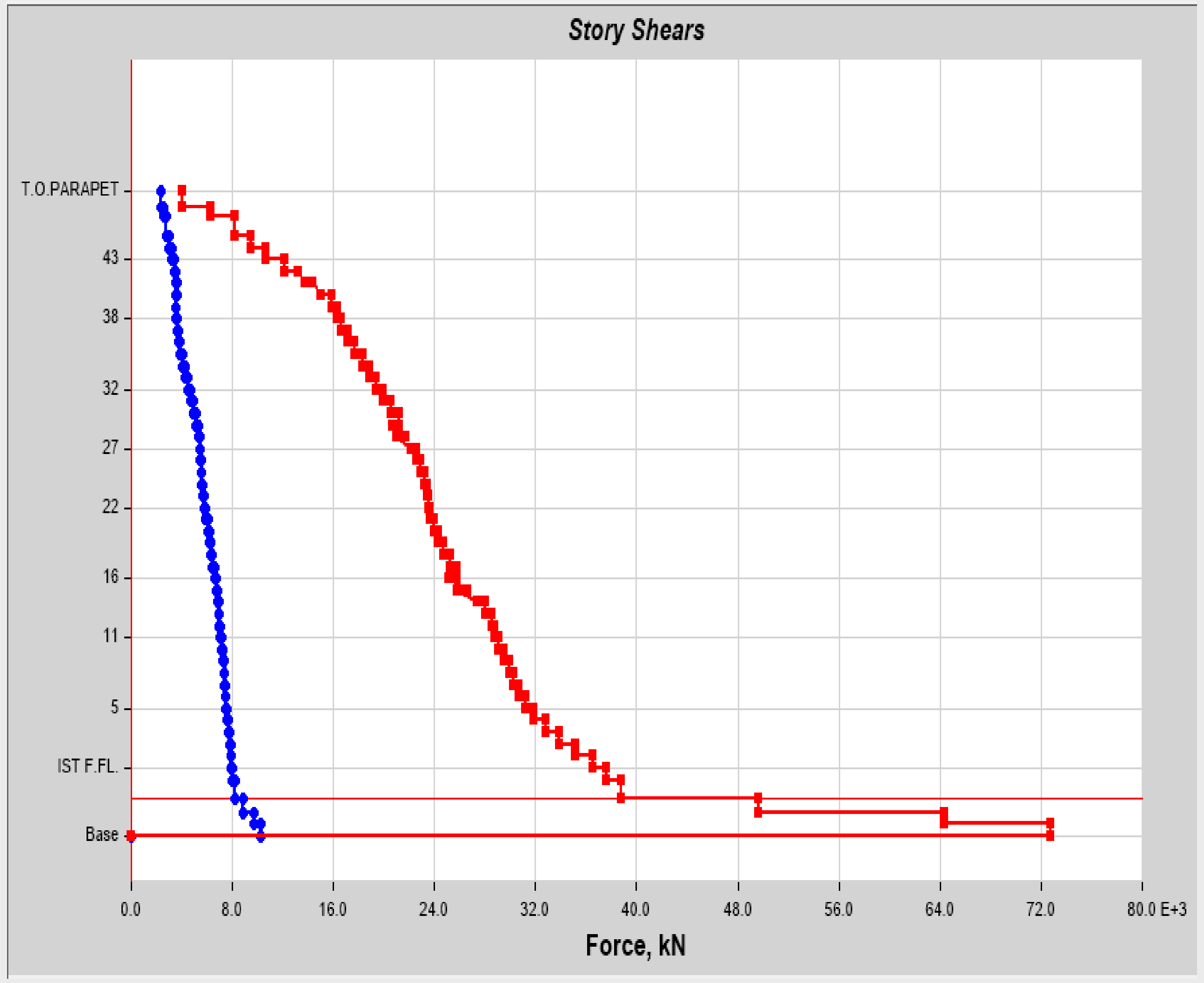
Compare Outrigger Structural System
Vs
Conventional Structural System

Conventional System	Outrigger System
	
Overturning moment=2454867 SpecY	Overturning moment=3110924 SpecY

Compare Outrigger Structural System
Vs
Conventional Structural System

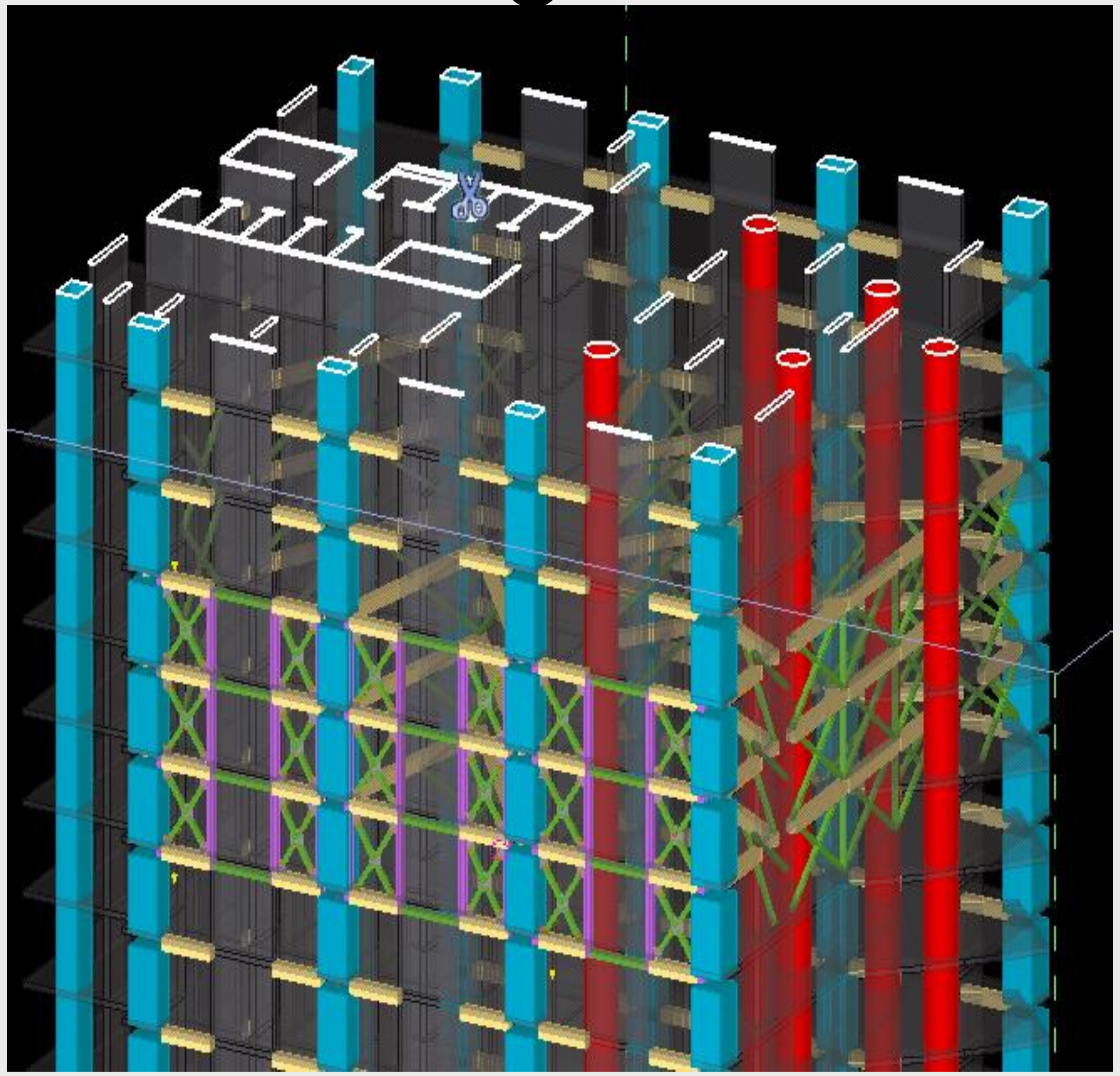
Conventional System	Outrigger System

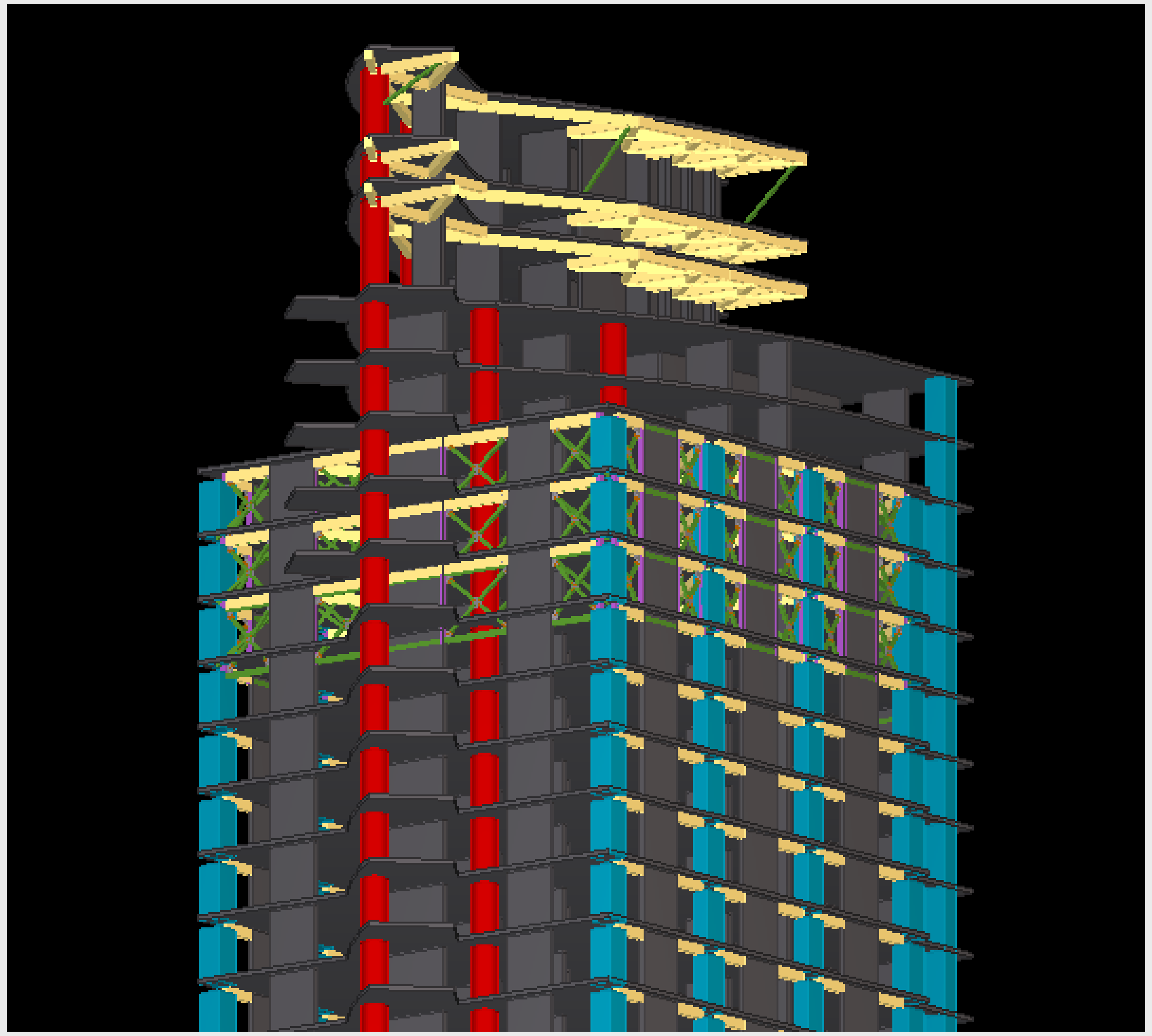


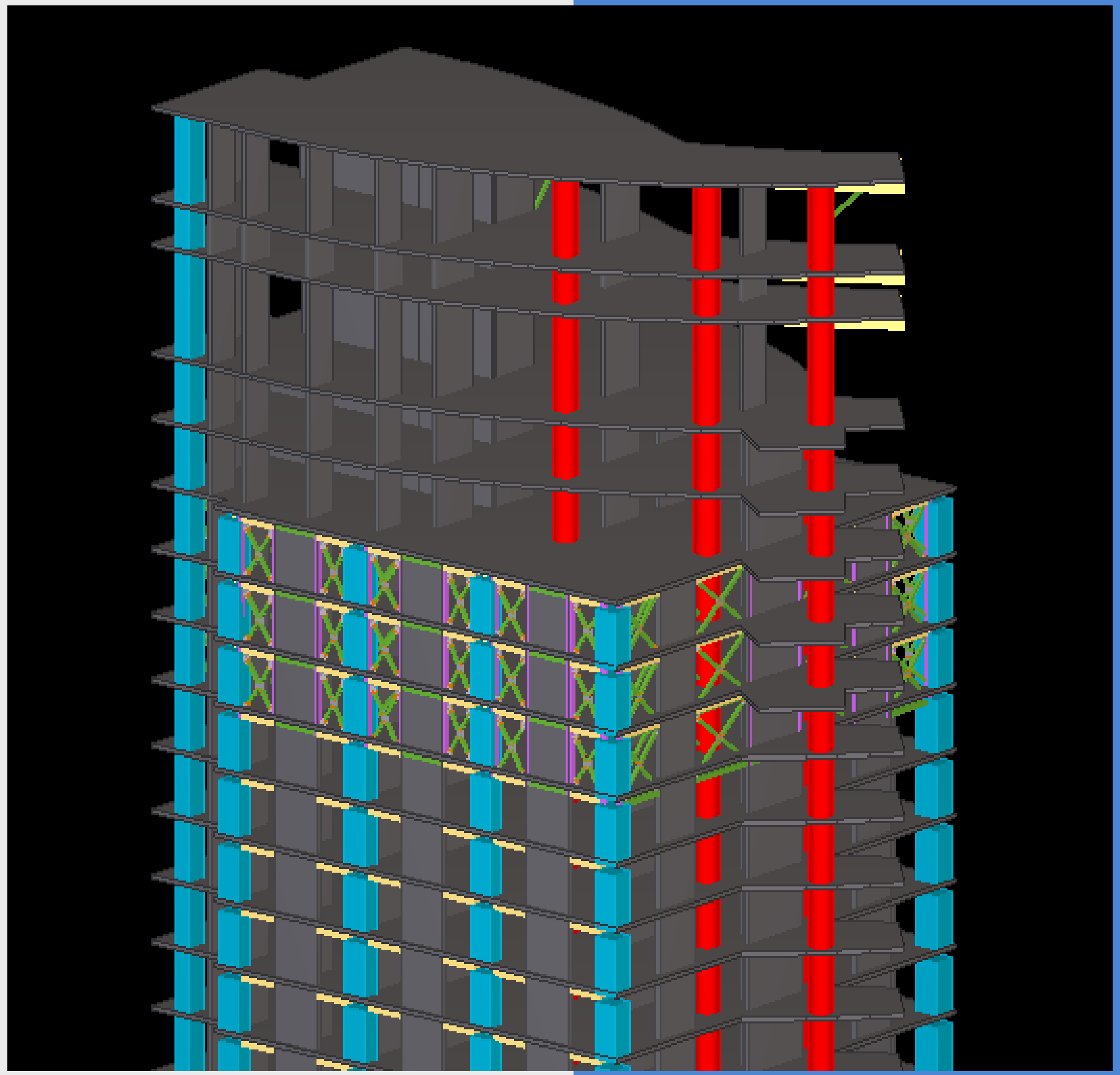


Story Shear=70867 KN SpecY	Story Shear=72769KN SpecY

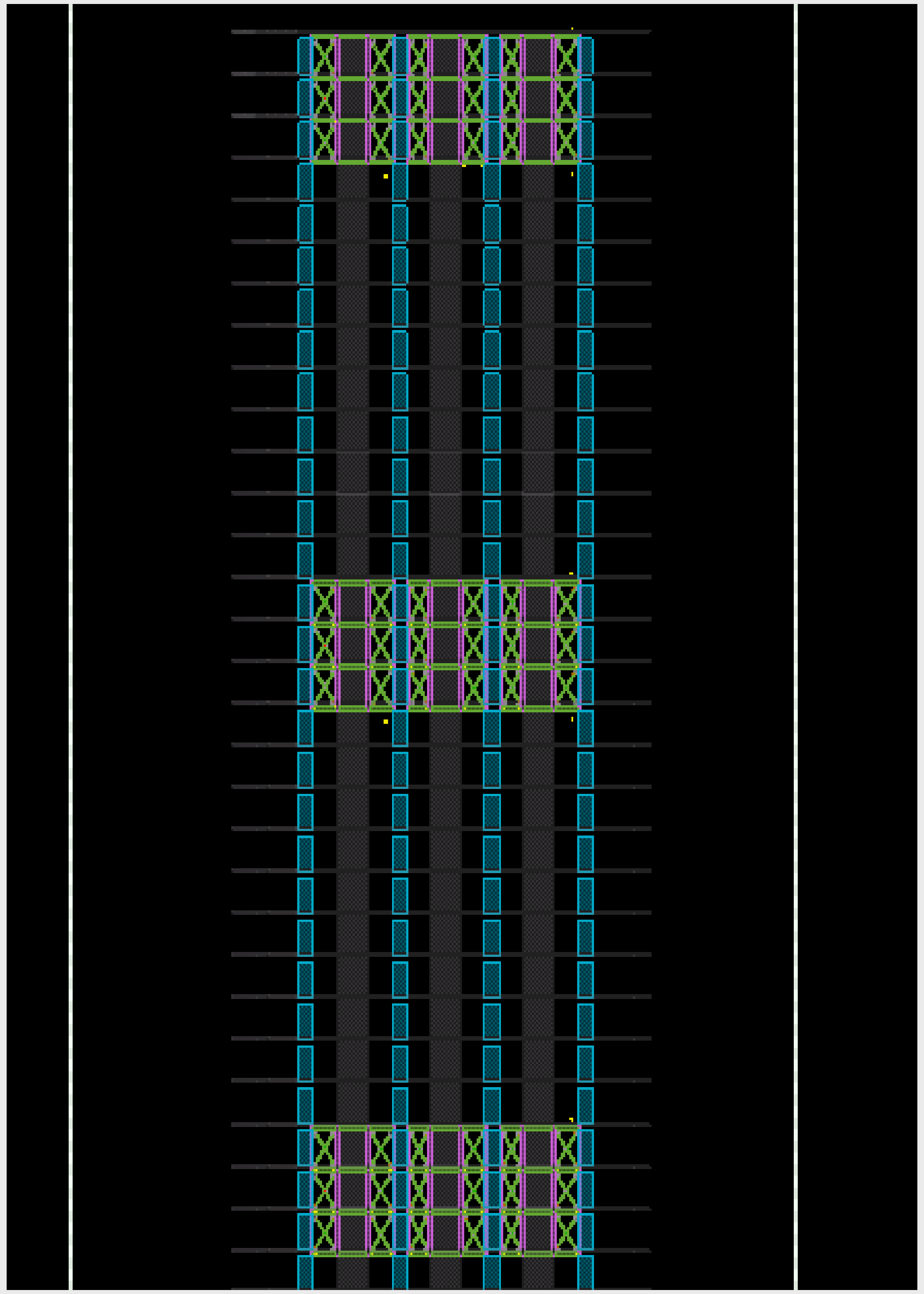
Outrigger Structural System Connection Arrangement

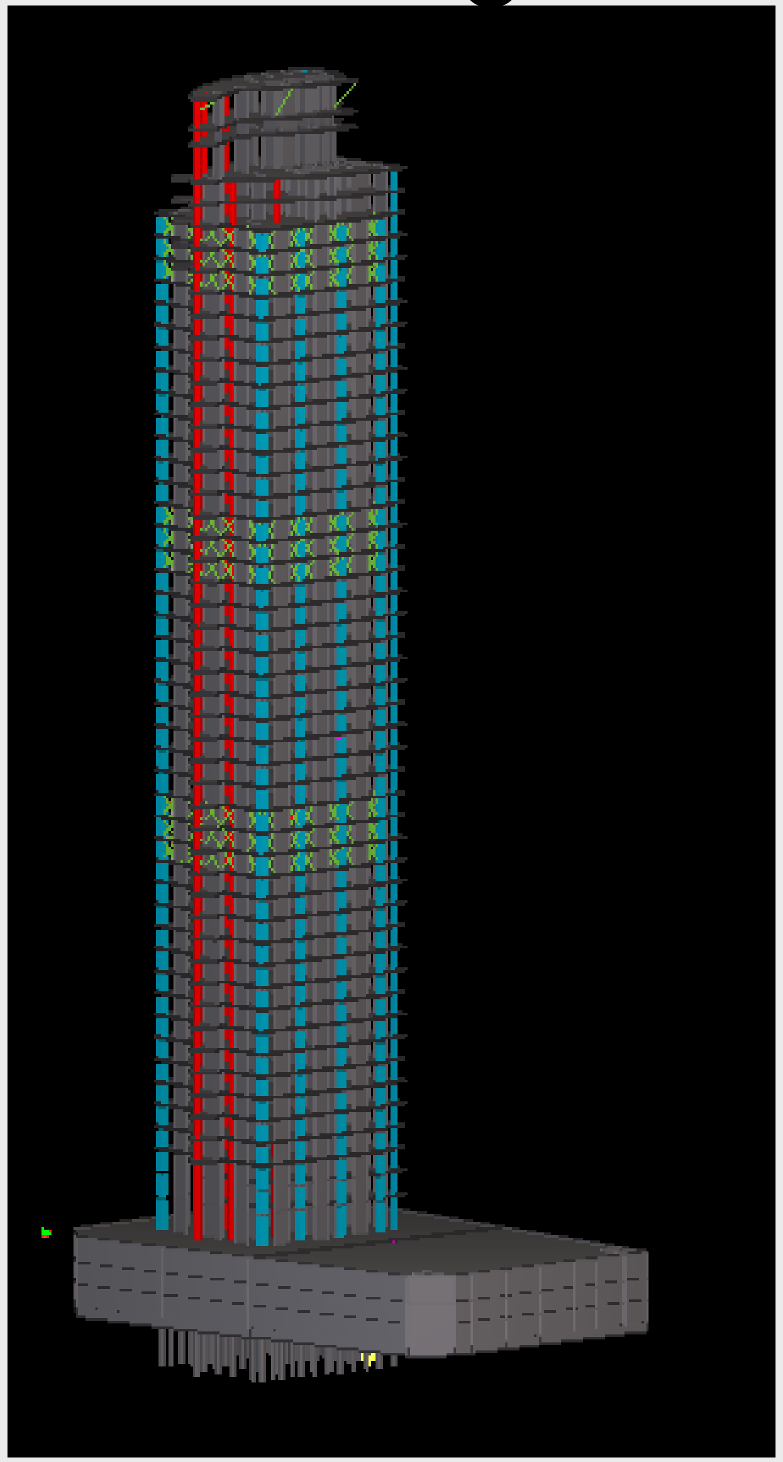


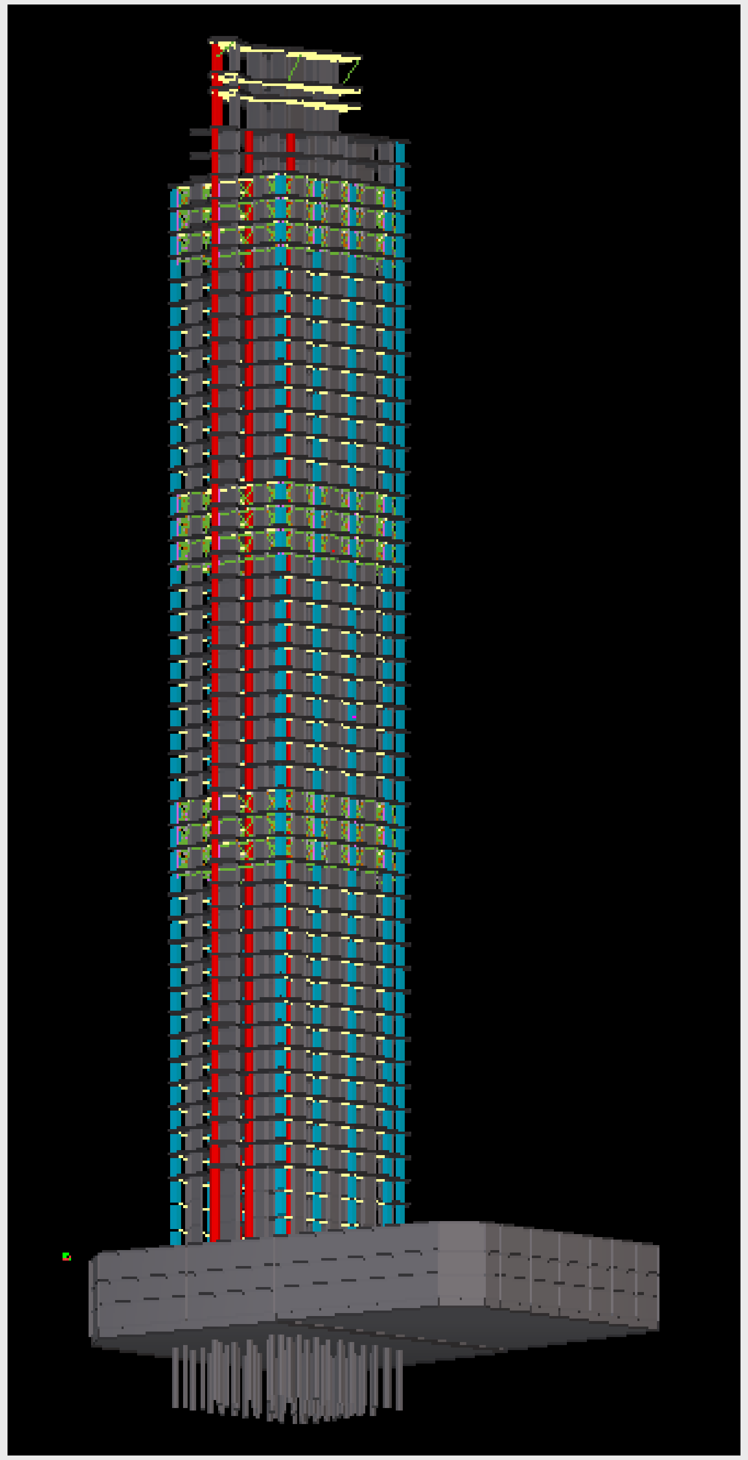




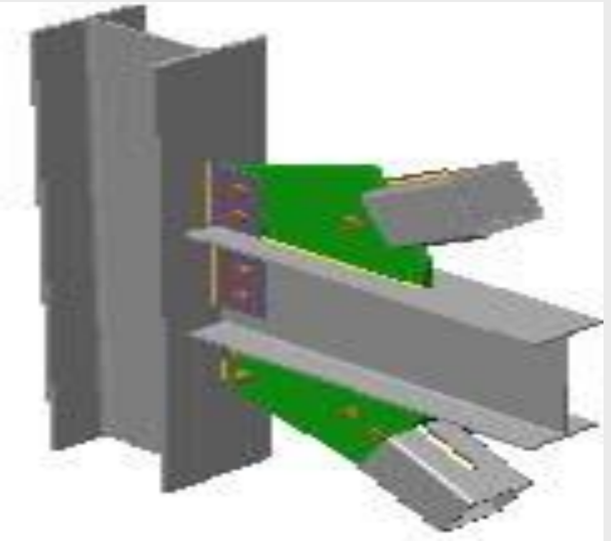
Outrigger Structural System
Connection Arrangement



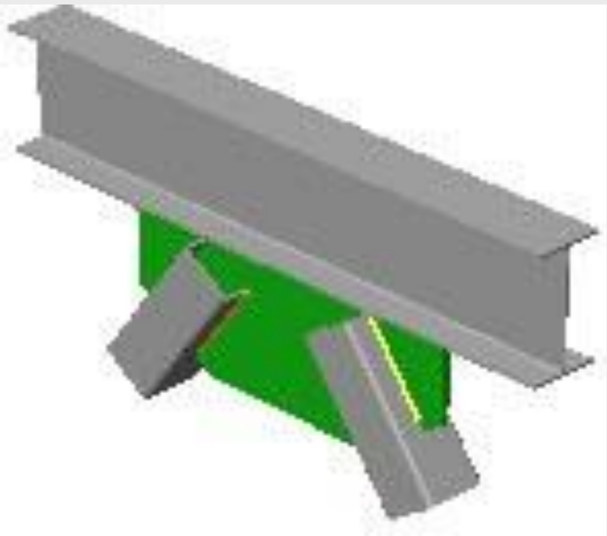




Outrigger Structural System Connection Arrangement

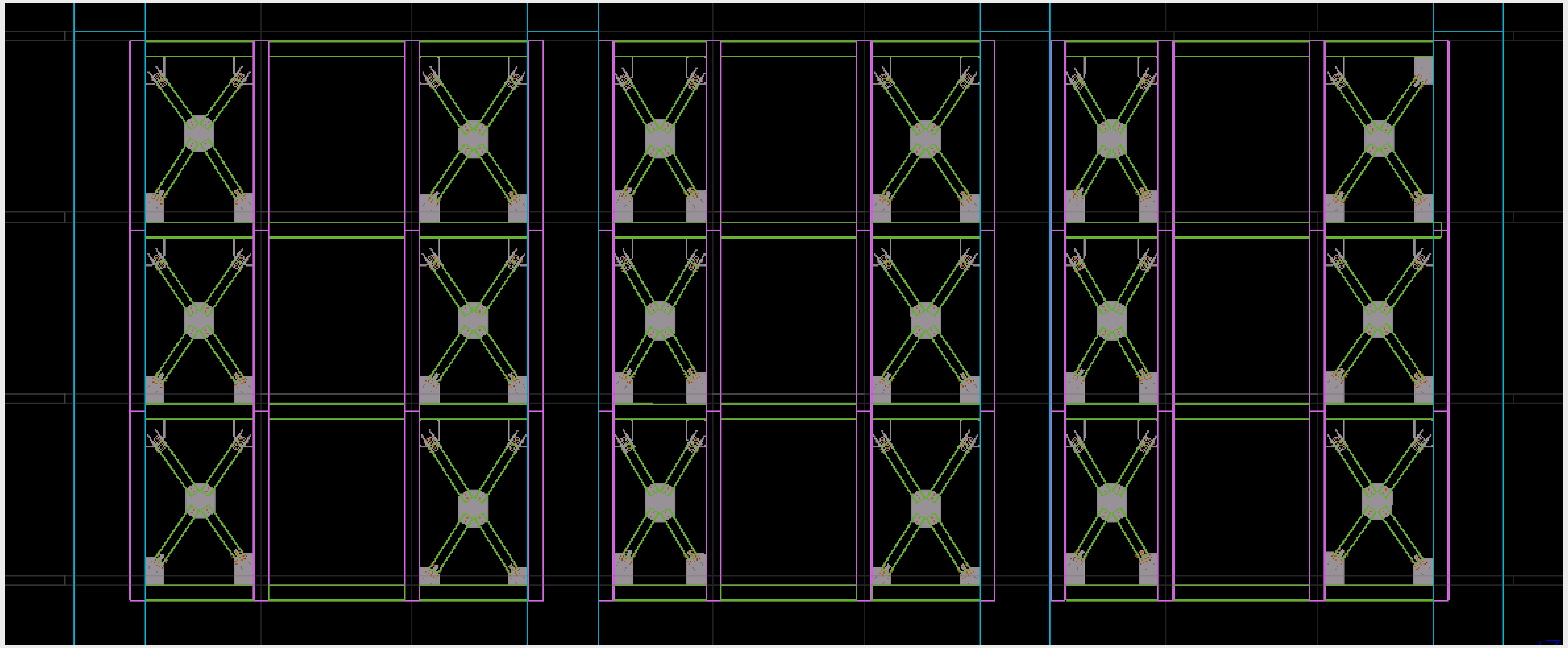


CBB: Column-Beam-Brace



CVR: Chevron Brace

VXB: Vertical X Brace



❖ **Gusset connection -Primarily transfers axial force but also minor moments and shear forces**

Vibration Analysis

[ASCE Ch: 18.0]

Damping Plays important role in design of Earthquake Resistant Structures, which reduces the response of the structure when they are subjected to lateral loads. There are many different types of dampers in use. In the present study Fluid Viscous dampers (FVD) are used to evaluate the response of RC buildings.

The main task of a structure is to bear the lateral loads and transfer them to the foundation. Since the lateral loads imposed on a structure are dynamic in nature, they cause vibrations in the structure. In order to have earthquake resistant structures, fluid viscous dampers have been used.

The viscous fluid dampers (VFD) are the more applied tools for controlling responses of the structures. These tools are applied based on different construction technologies in order to decrease the structural responses to the seismic excitation.

TYPES OF DAMPERS

Dampers are classified based on their performance of friction, metal (Flowing), Viscous, Viscoelastic, shape memory alloys (SMA) and mass dampers. About the advantages of using dampers we can infer to high energy absorbance, easy to install and replace them as well as coordination to other structure members.

- Friction dampers
- PVD Damper
- Pall Friction Damper
- Metallic Dampers
- Lead Injection Damper (LED)
- Shape Memory Alloy (SMA)
- Viscous Dampers
- Mass Damper
- Lead Rubber Damper LRM and Rubber Damper HDRB
- Regulatory Mass Damper TMD
- Passive Seismic Controlling System

Viscous Dampers

In this damper, by using viscous fluid inside a cylinder, energy is dissipated. Due to ease of installation, adaptability and coordination with other members also diversity in their sizes, viscous dampers have many applications in designing and retrofitting.

These types of dampers are connected to the structure in three ways:

- Damper installation in the floor or foundation (in the method of seismic isolation).
- Connecting dampers in stern pericardial braces.
- Damper installation in diagonal braces.

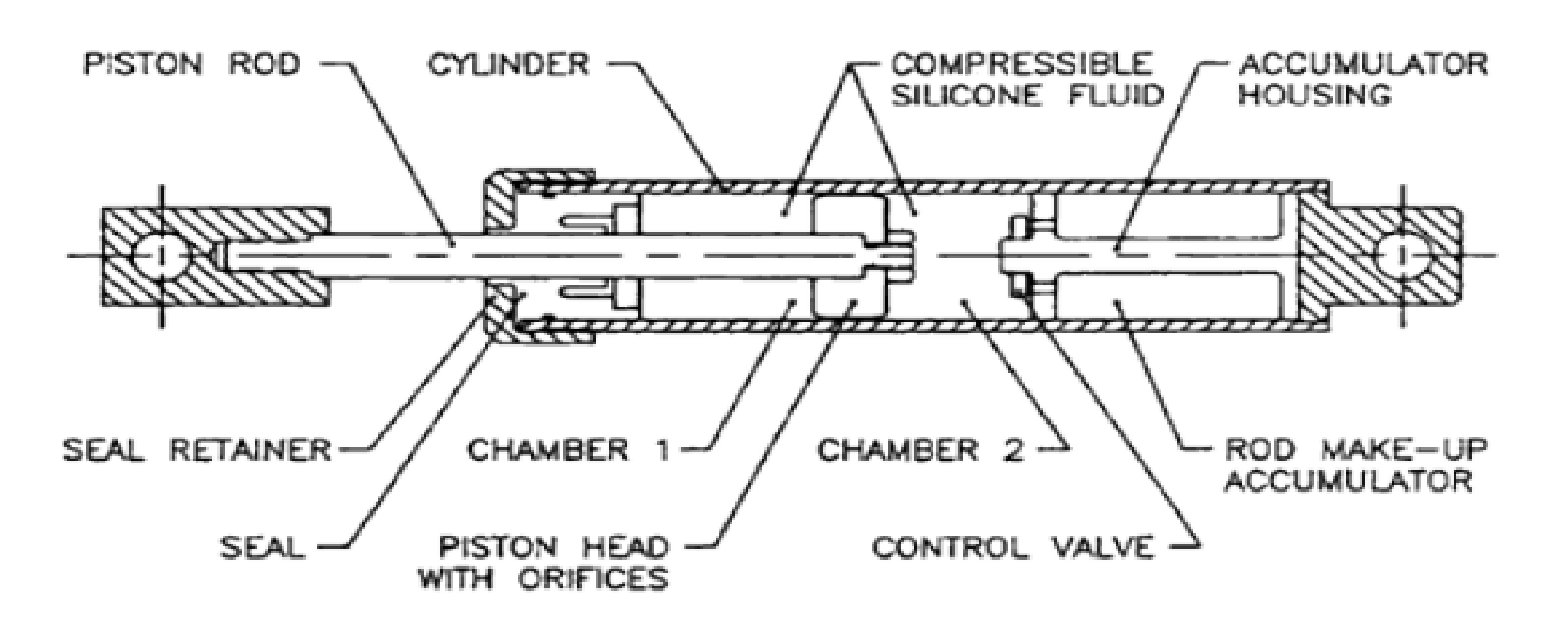


Figure a: Longitudinal Section of Viscous Damper

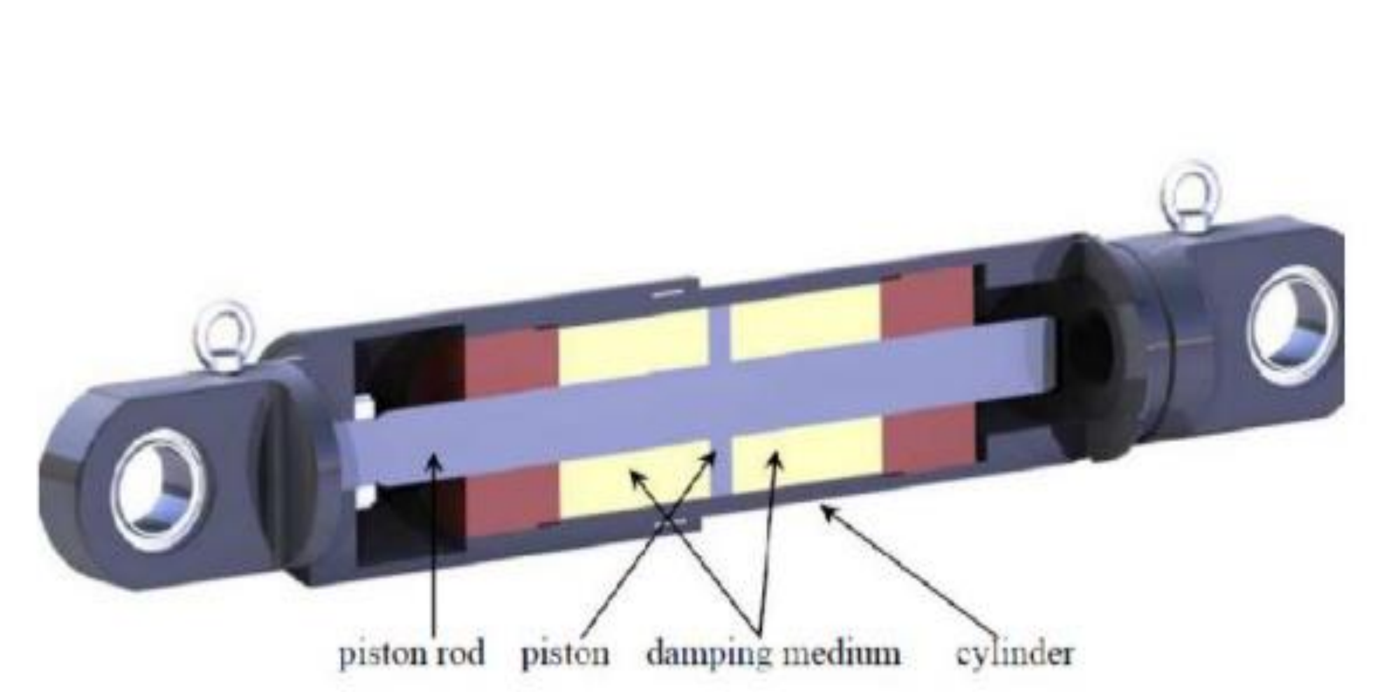


Figure b: Fluid Viscous Damper Cross-section.

MODELLING OF DAMPERS

The dampers used in modelling these buildings are from Taylor Devices Inc. made in USA. They provide two types of FVD with data that can be used in ETABS for modelling of structure. They are: -

1. Fluid viscous dampers & lock-up devices clevis – clevis configuration.
2. Fluid viscous dampers & lock-up devices clevis – base plate configuration.

Any one of these can be used in the structure, since it is easy to fit FVD with base plate is selected here for modelling of structure here. The details of fluid viscous dampers & lock-up devices clevis – base plate configuration are as shown below.

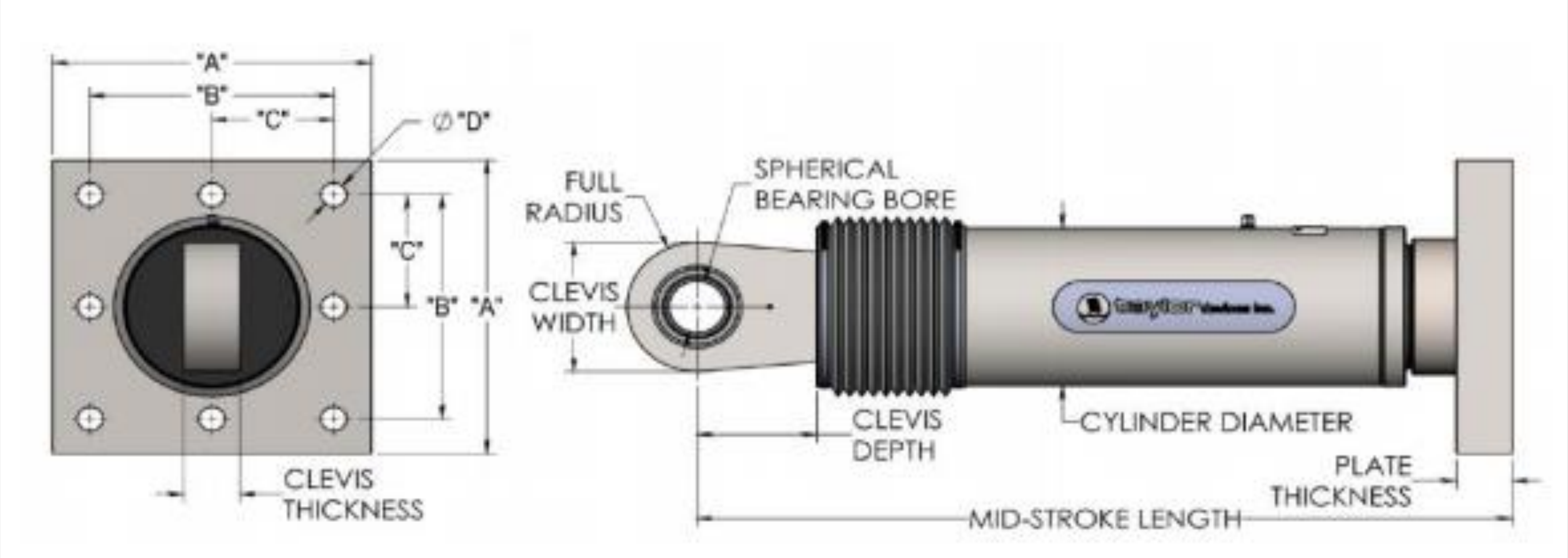


Figure c : Fluid viscous dampers & lock-up devices clevis – base plate configuration.
Courtesy: Taylor Devices.

Viscous Dampers

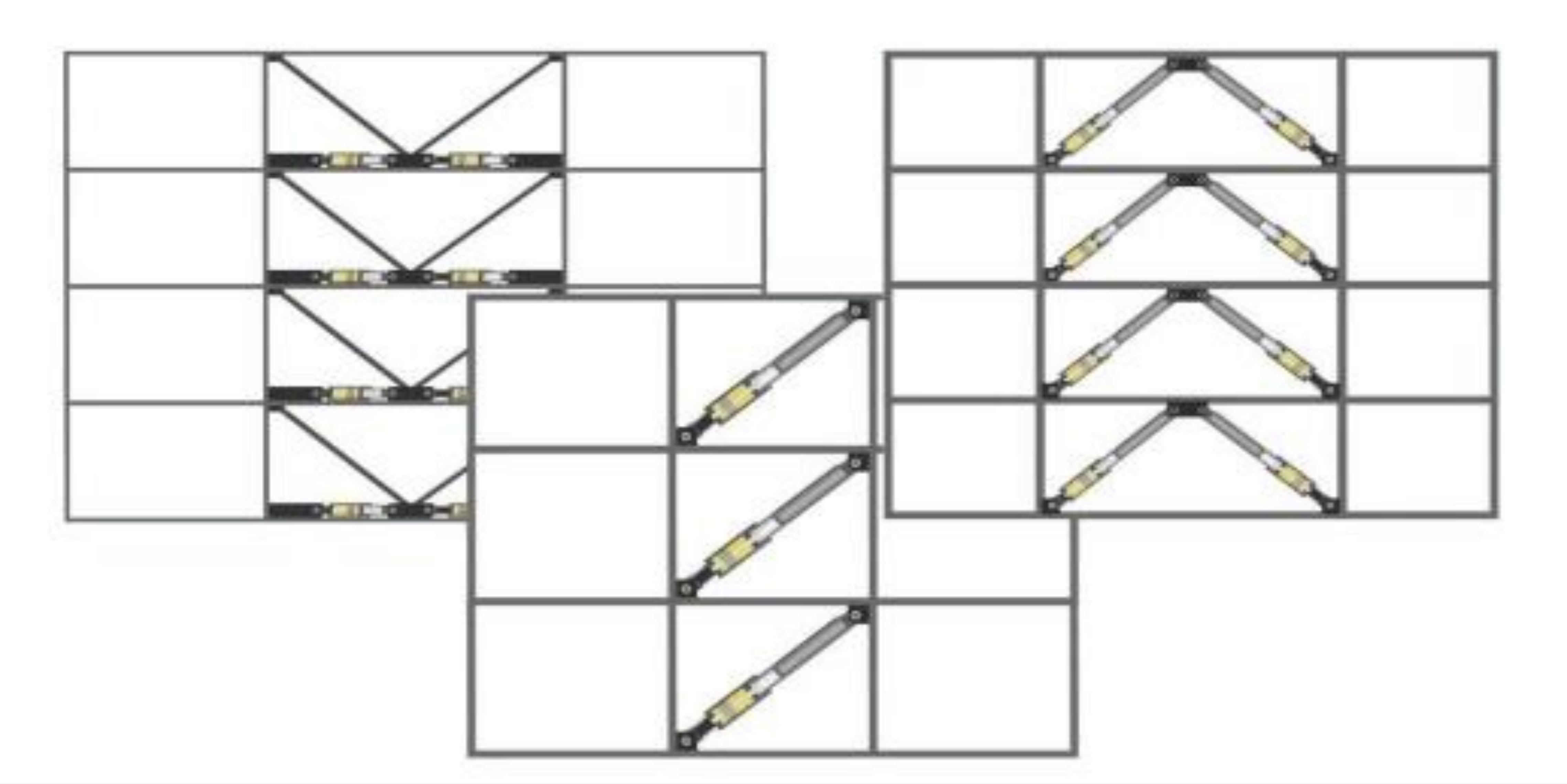


Figure 12: Viscous Damper Installation Methods

Viscous Dampers

Table 5 : FVD with Different Capacities Force(kN).

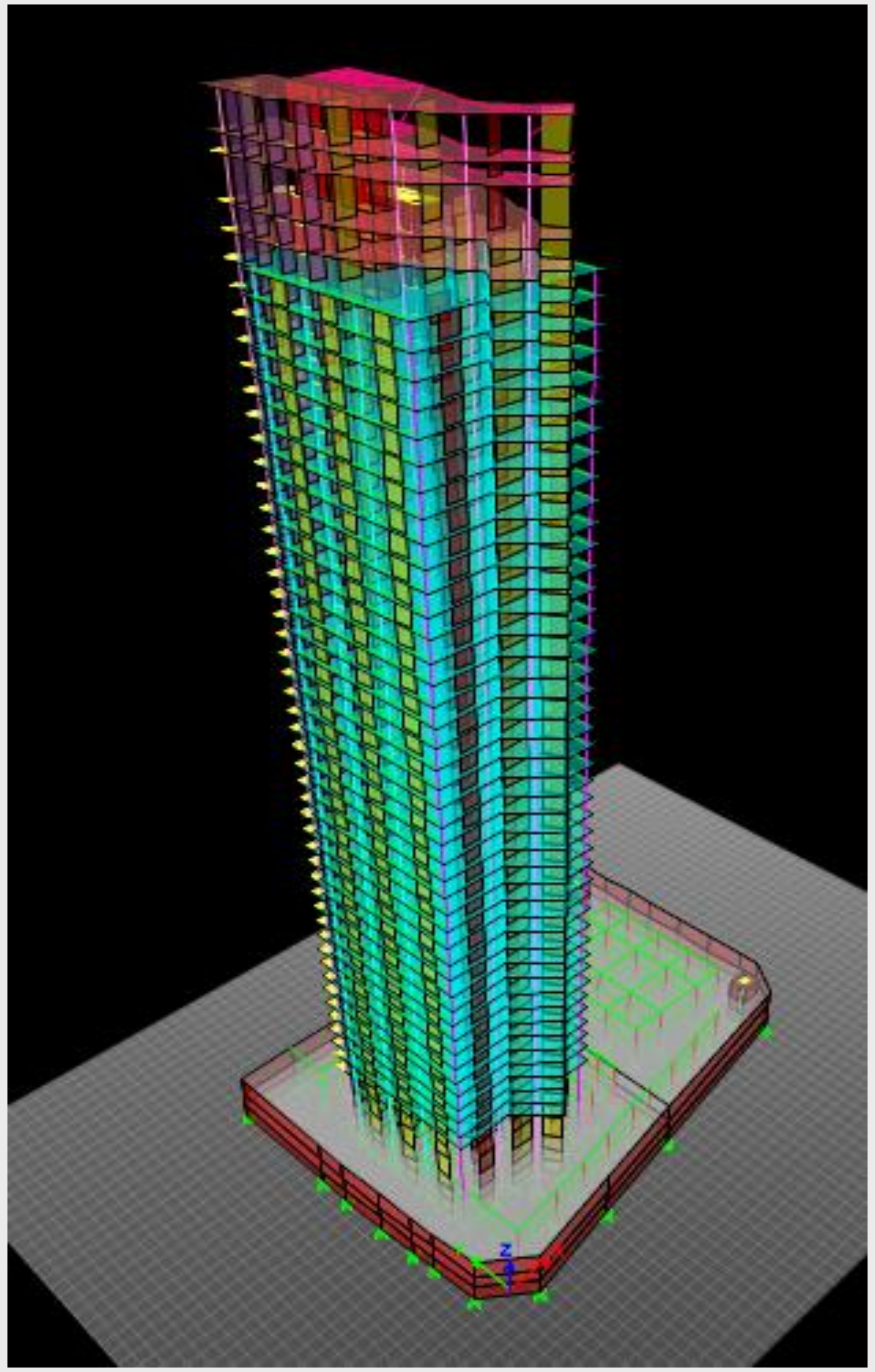
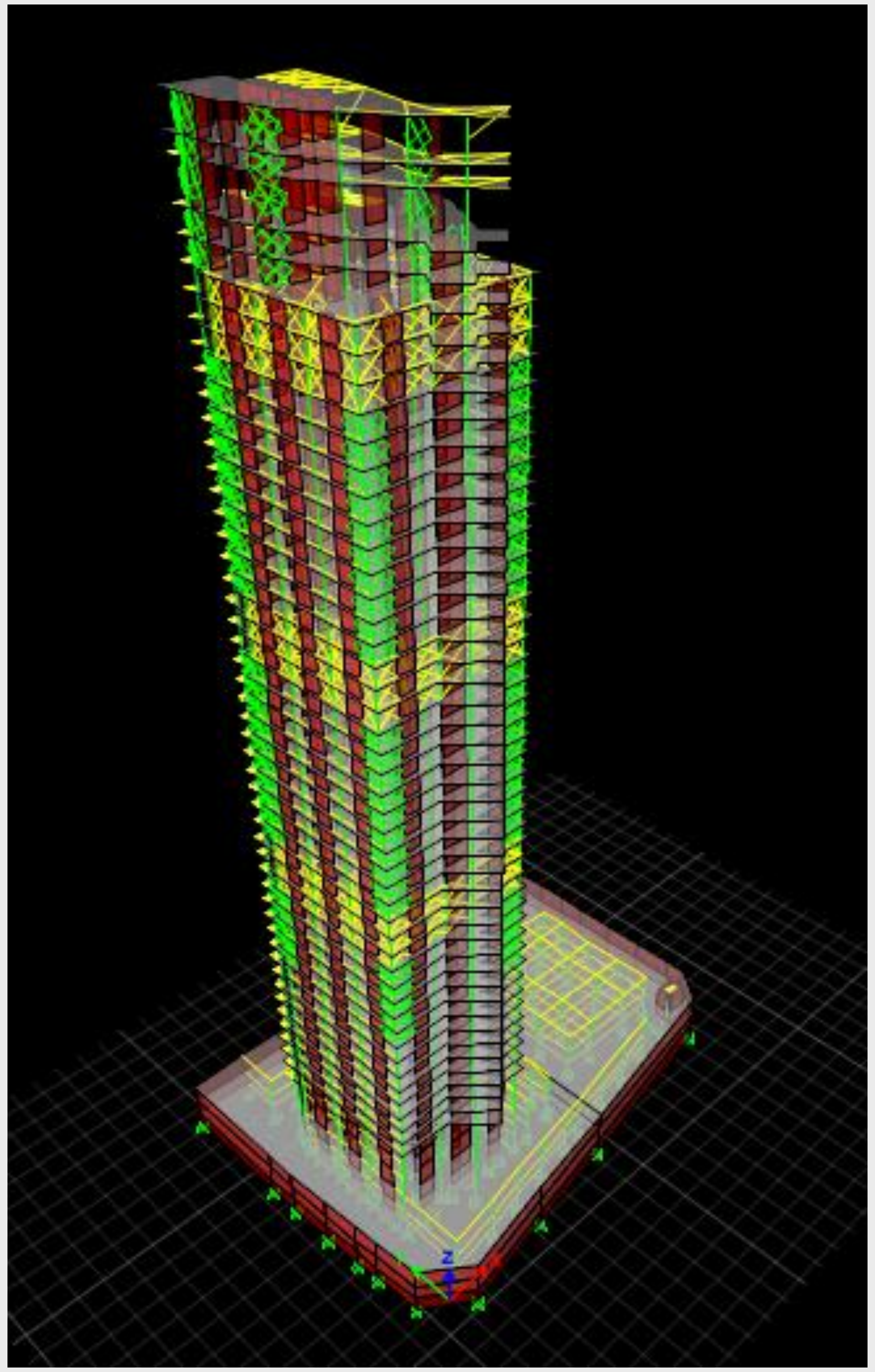
FORCE (kN)	TAYLOR DEVICES MODEL NUMBER	SPHERICAL BEARING BORE DIAMETER (mm)	MID-STROKE LENGTH (mm)	STROKE (mm)	CLEVIS THICKNESS (mm)	MAXIMUM CLEVIS WIDTH (mm)	CLEVIS DEPTH (mm)	BEARING THICKNESS (mm)	MAXIMUM CYLINDER DIAMETER (mm)	WEIGHT (kg)
250	17120	38.10	787	±75	43	100	83	33	114	44
500	17130	50.80	997	±100	55	127	102	44	150	98
750	17140	57.15	1016	±100	59	155	129	50	184	168
1000	17150	69.85	1048	±100	71	185	150	61	210	254
1500	17160	76.20	1105	±100	77	205	162	67	241	306
2000	17170	88.90	1346	±125	91	230	191	78	286	500
3000	17180	101.60	1441	±125	117	290	203	89	350	800
4000	17190	127.00	1645	±125	142	325	273	111	425	1088
6500	17200	152.40	1752	±125	154	350	305	121	515	1930
8000	17210	177.80	1867	±125	178	415	317	135	565	2625

Fluid viscous dampers with different forces can be used for different types of buildings, since structure modelled is of low height; smaller devices were used to start analysis. This tabular data can be fed in program as shown below.

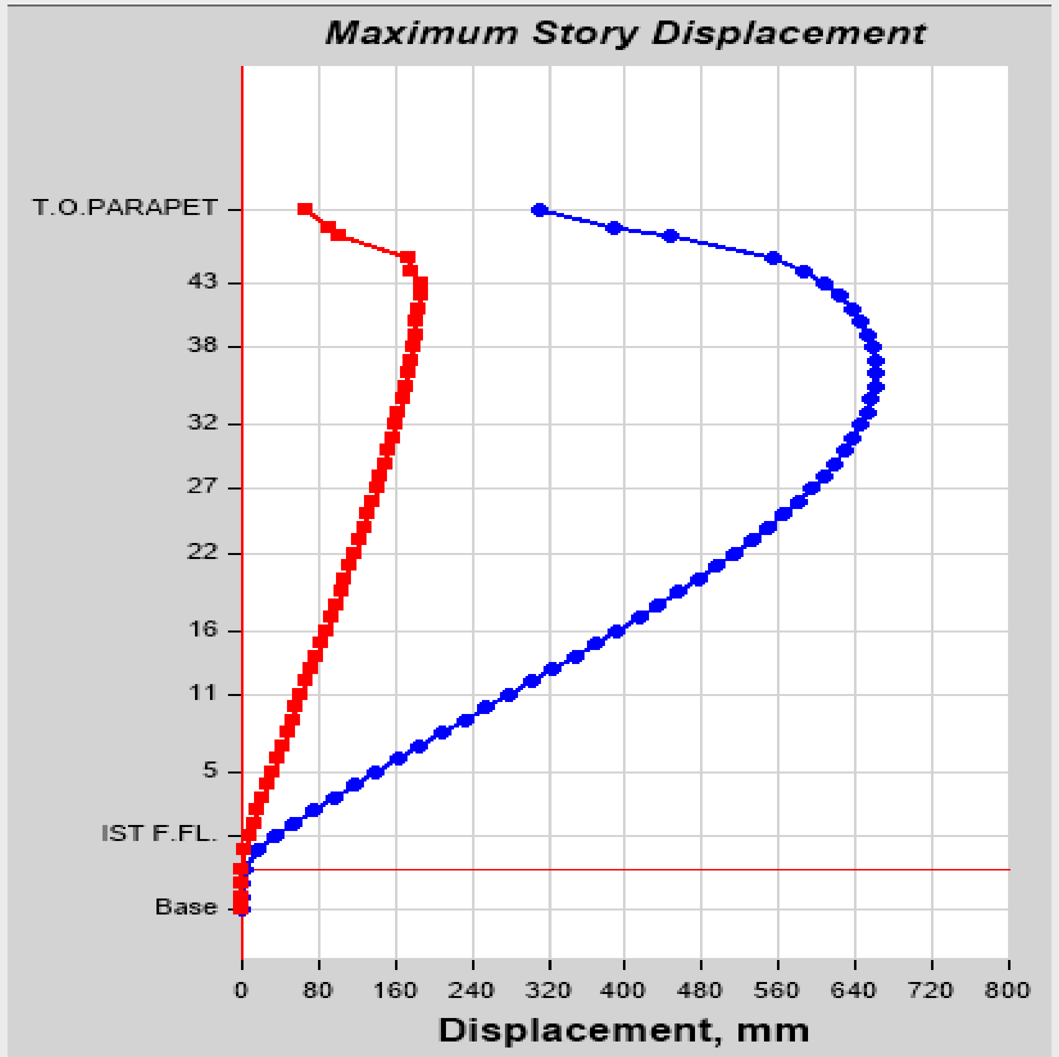
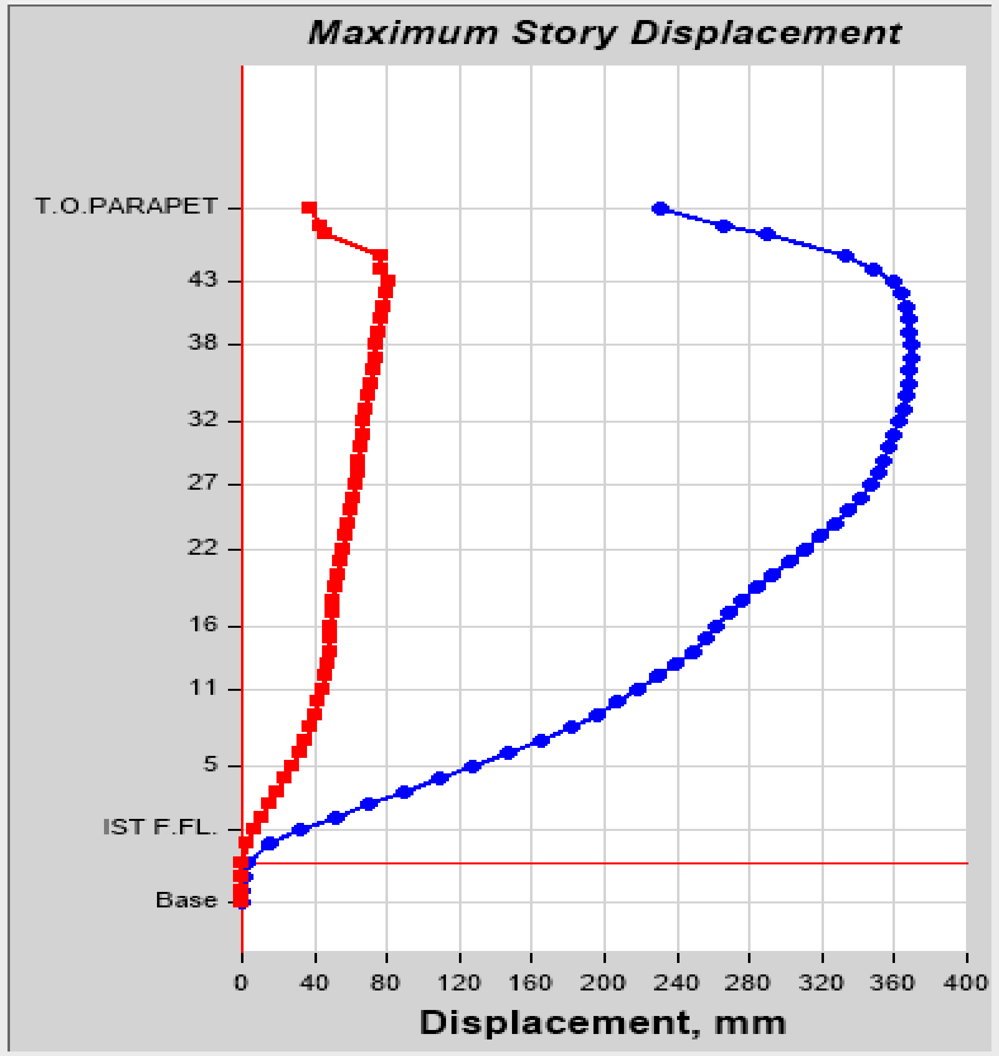
FVD is added to structure after defining in Link properties by adding a new Damper Exponential in Link Property Data.

ETABS MENU=> Define=> Link Properties=> Add new Link=> Link Property Data.

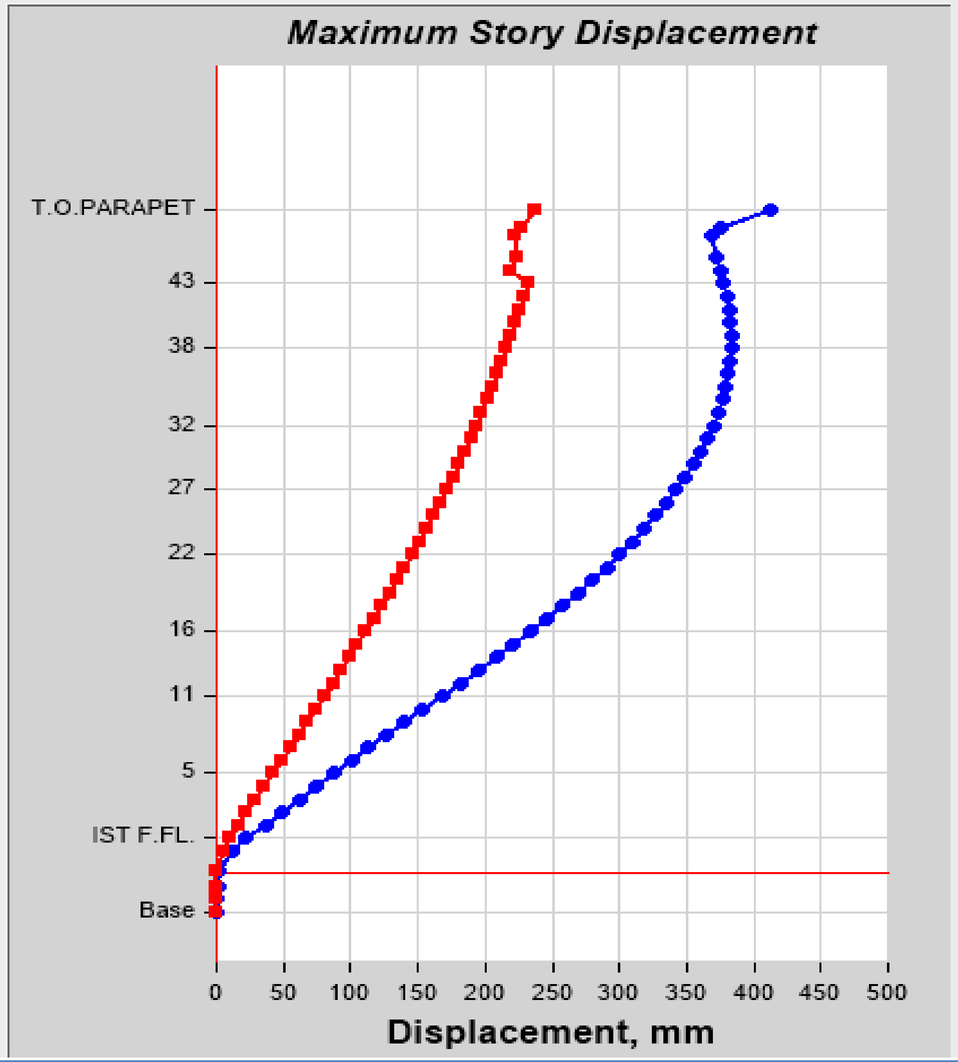
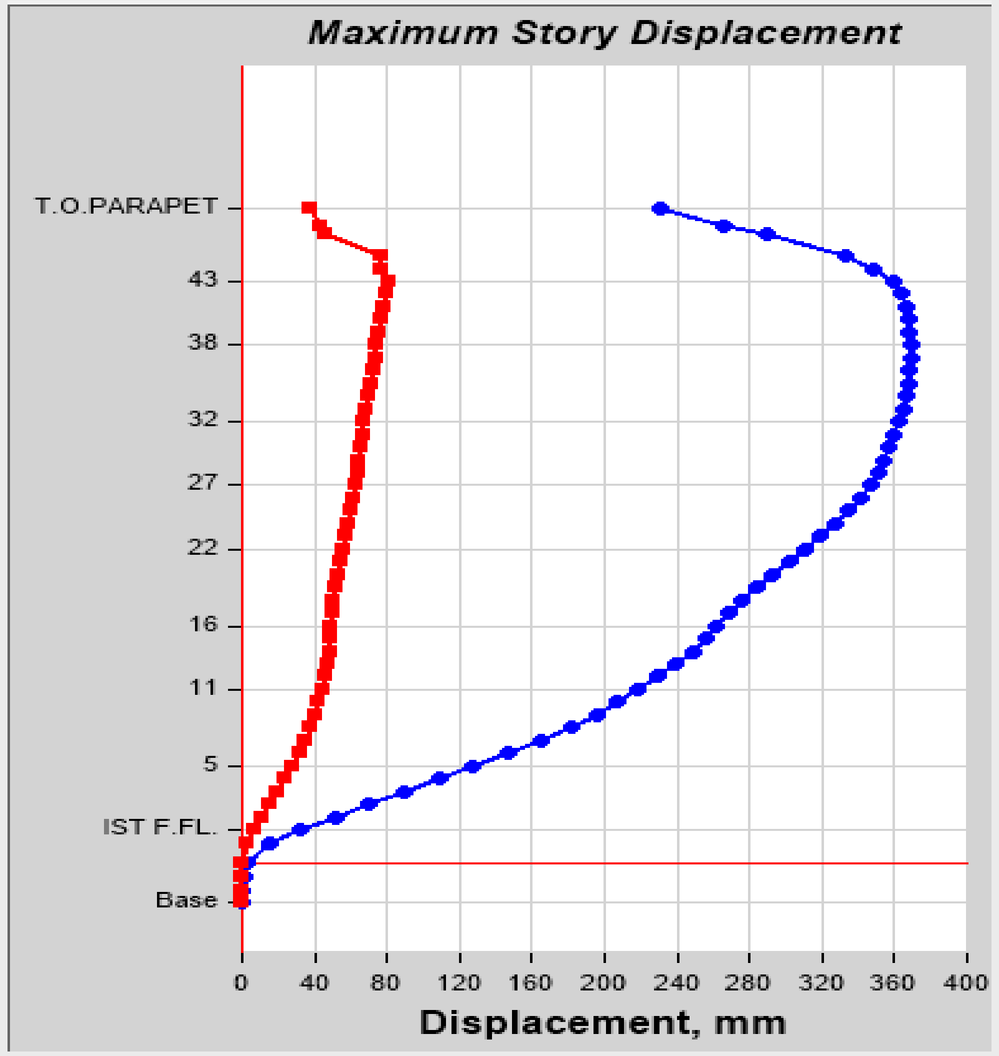
Compare Outrigger + FVD Structural System
Vs
Conventional Structural System

Conventional System	Outrigger + FVD System
	

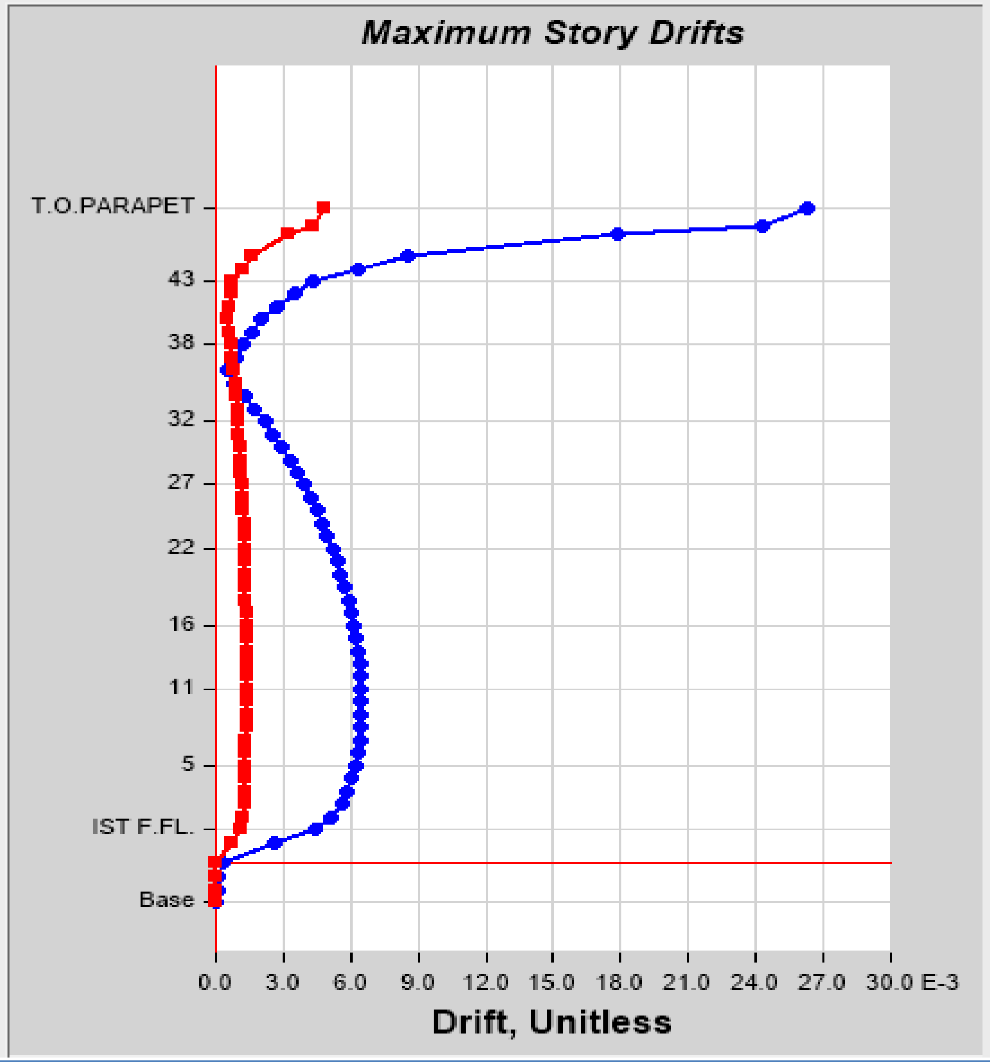
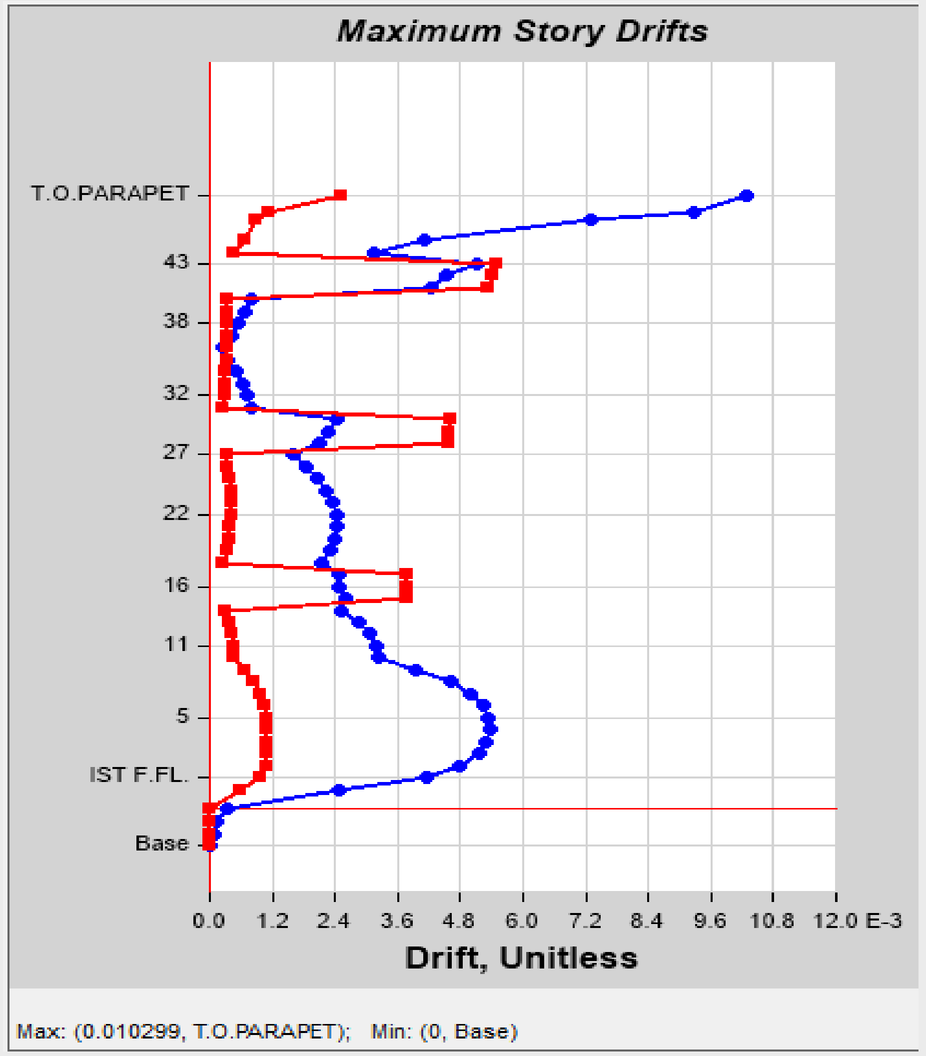
Compare Outrigger + FVD Structural System
Vs
Conventional Structural System

Conventional System	Outrigger + FVD System
	
Lateral Displacement=660mm EQX	Lateral Displacement=368mm EQX

Compare Outrigger + FVD Structural System
Vs
Conventional Structural System

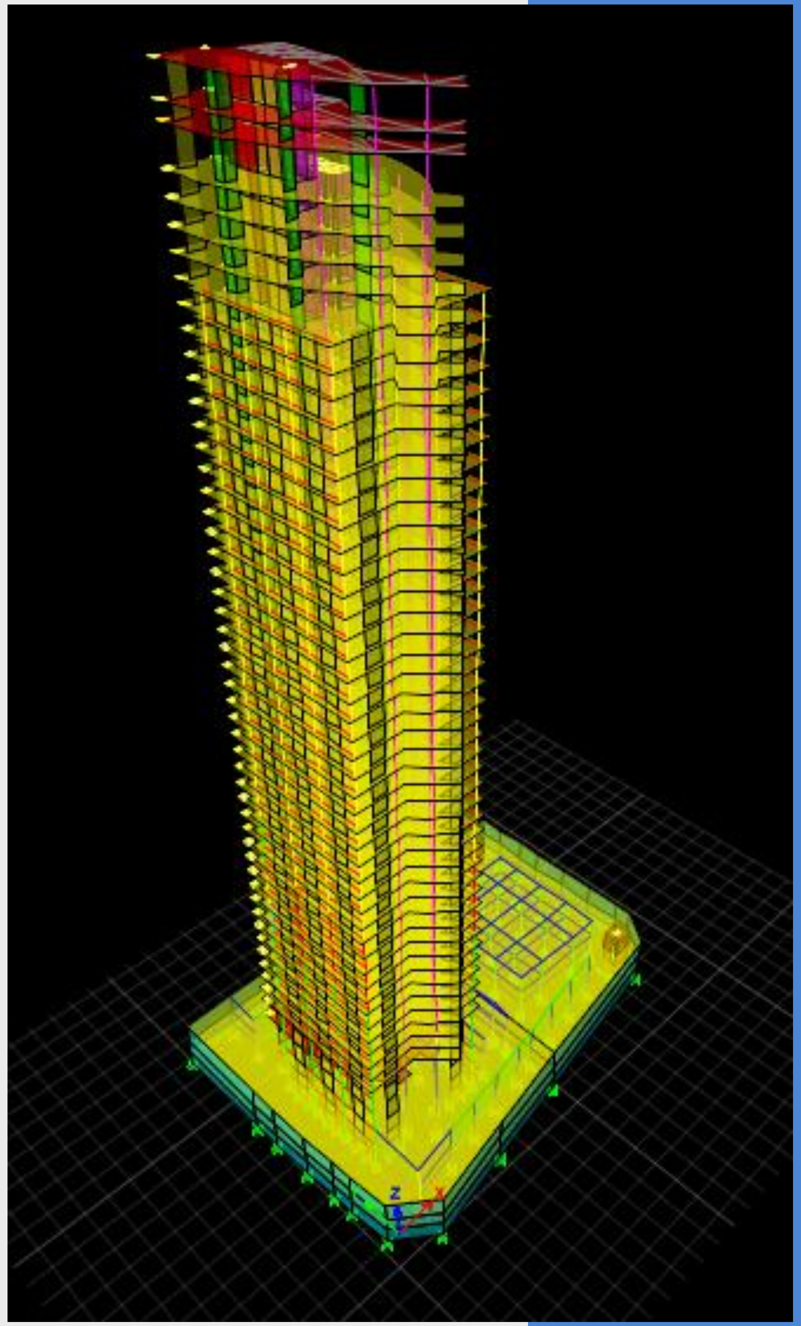
Conventional System	Outrigger + FVD System
	
Lateral Displacement=412mm SpecX	Lateral Displacement=315mm SpecX

Compare Outrigger + FVD Structural System
Vs
Conventional Structural System

Conventional System	Outrigger + FVD System
	
Max. Story Drift=0.026251 SpecX	Max. Story Drift=0.010299 SpecX

STRUCTURE MEMBERS & CONNECTIONS DESIGN

- All the structures elements need to well within the strength design limit criteria.
- Members and connections of reinforced and prestressed concrete structures shall be designed (as per code provision) such that premature failure does not occur due to shear or bond.
- Some provisions for appropriate ductile detailing of RC members are code provision.
- To resist large displacement during EQs without collapse.

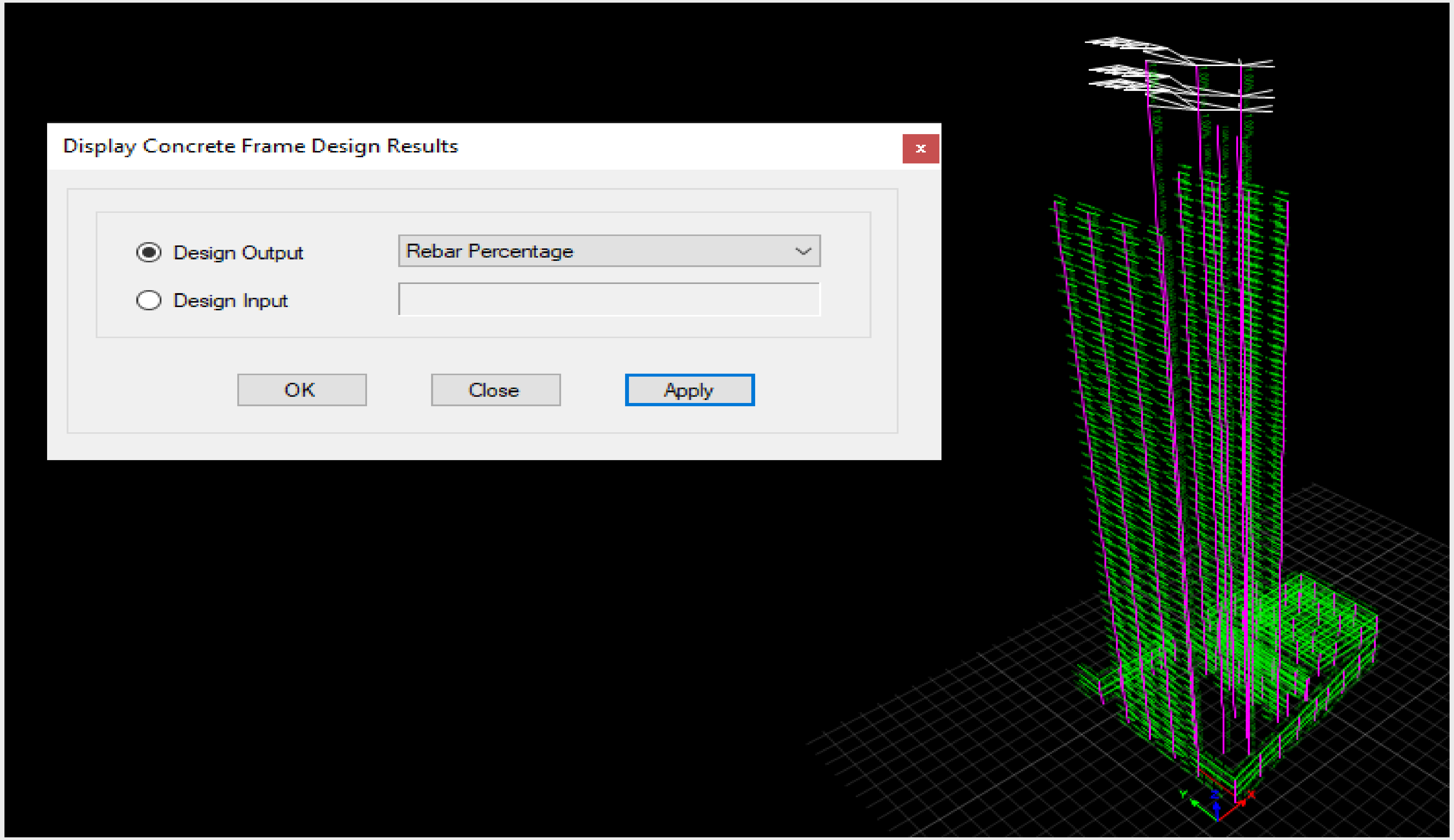


Frame Design

- All the structures all section sizes are very economic and maximum rebar % for column.
- elements need to well within the strength design limit criteria

Code recommends the minimum reinforcement for column and beam should be **1.0%** and **0.2%** respectively a good thing about in ETABS is that even in the reinforcement required by column comes out to be less than 1.0% like 0.6% even than it will be report as 1.0% it takes recommendation as per codes seriously

- Check Rebar percentage



Frame Design

The PMM interaction ratio

The demand/capacity ratio (D/C) **PMM** ratio is the sum of the bending moment demand/capacity ratio. A **PMM** value greater than one will be used here to indicate overstressed members.

The PMM interaction ratio must not be greater than 1.If it is greater means the columns are over stressed and must increase the section of column.

Check PMM

All columns represented by purple color and the purple color ratio is (0.9-1) so its safe.

Check B-C Capacity

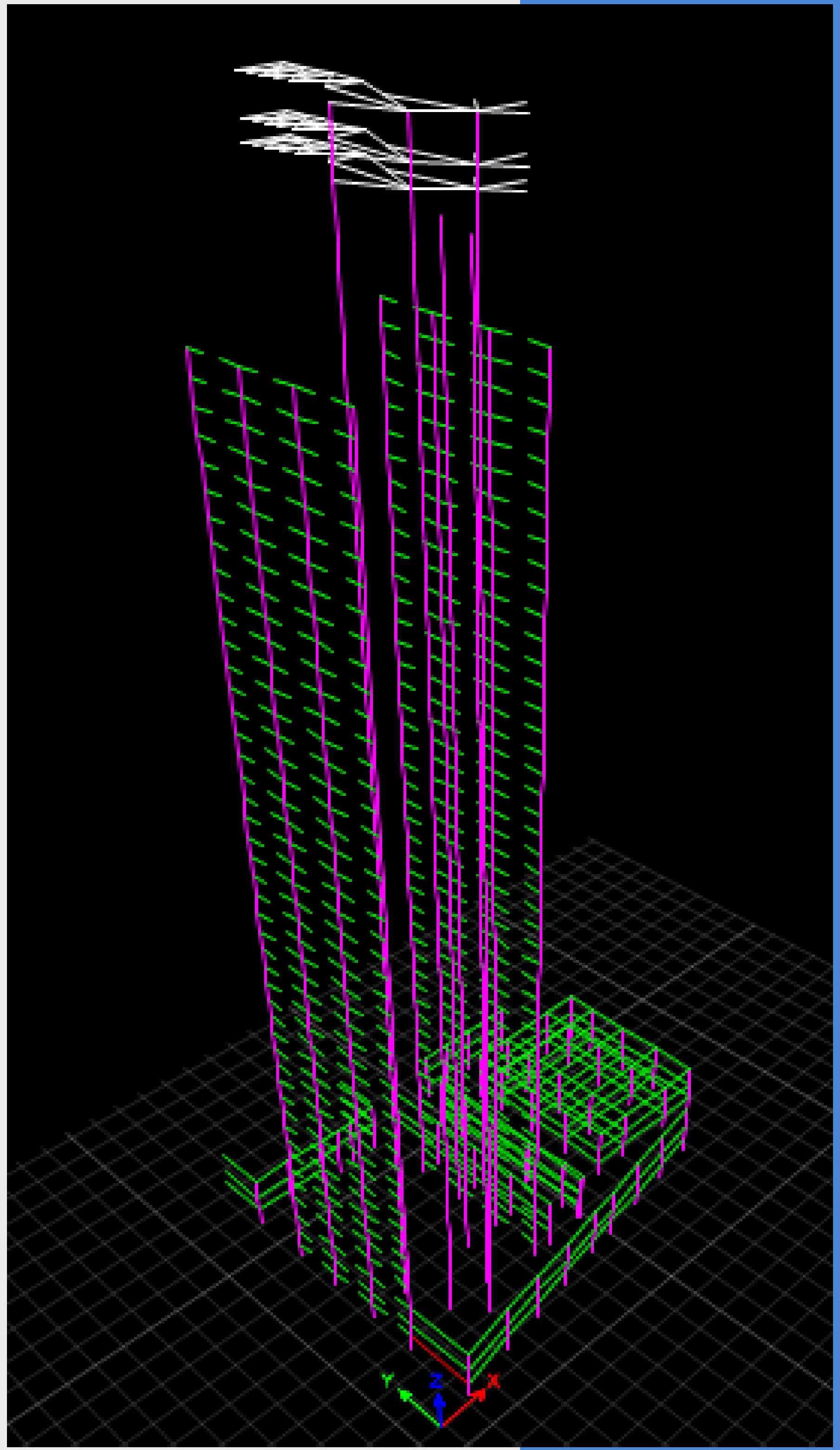
The Beam and Column capacity ratio must be less than 1.If its greater than 1 we must increase the section sizes or increase the the rebars values.

The ratios are less than 1 its is safe.

Check C-B Capacity

The Column and Beam capacity ratio should be greater than 1.If its less than 1 we must increase the section sizes or increase the the rebars values.

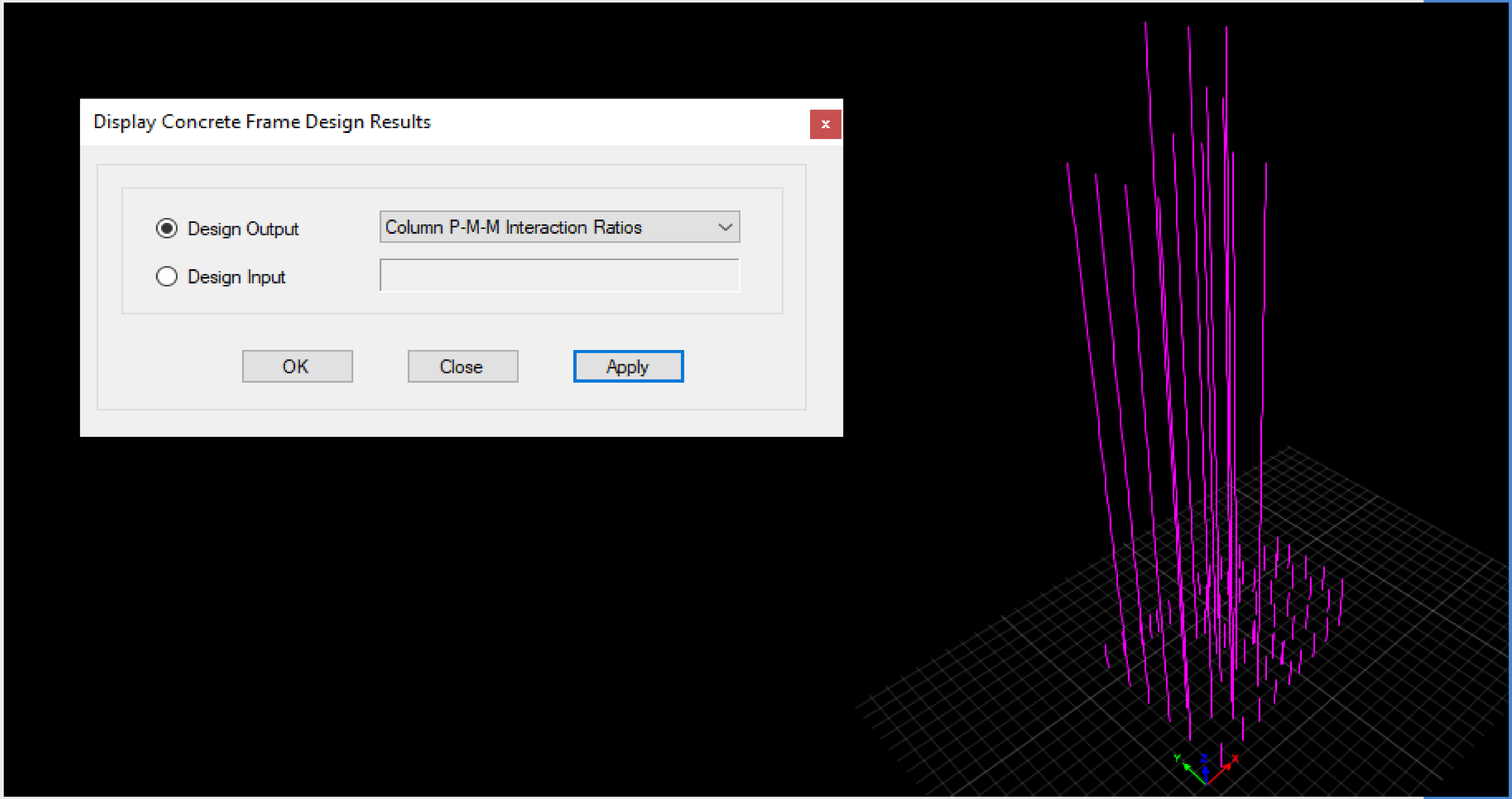
The ratios are greater than 1 its is safe.



Frame Design

Check PMM

All columns represented by purple color and the purple color ratio is (0.9-1) so its safe.



Frame Design

The Interaction surface for section

All columns represented by purple color and the purple color ratio is (0.9-1) so its safe.

Interaction Surface for Section CIR 2500MM (ACI 318-14) Station 3.5 m

Display Options

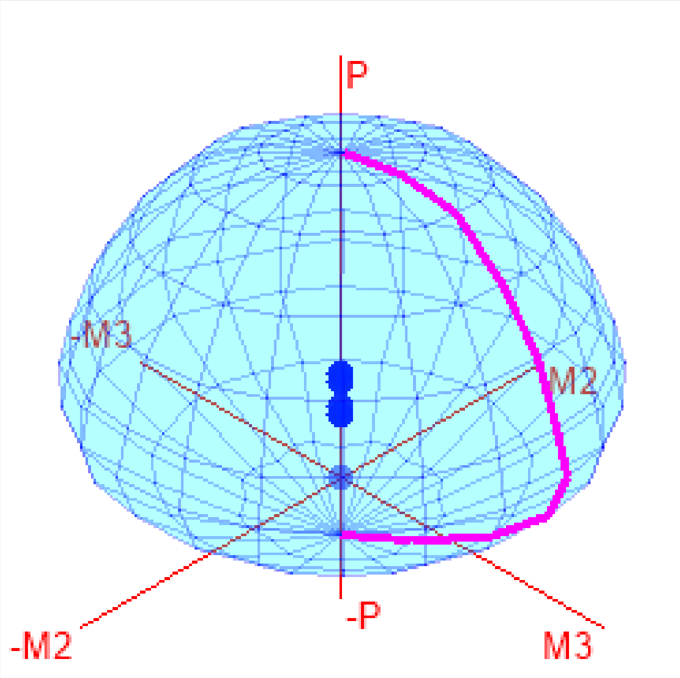
- (●) Show Design Code Data
  - (●) Include Phi
  - ( ) Exclude Phi
  - ( ) Exclude Phi and Increase Fy
- ( ) Show Fiber Model Data

Curve Data

Point	P kN	M2 kN-m	M3 kN-m
1	85021.2829	0	0
2	85021.2829	0	9030.2819
3	81482.4278	0	16546.8625
4	70240.194	0	23561.0382
5	57479.1081	0	28855.9448
6	43625.2967	0	32108.1676
7	32824.395	0	34199.4471
8	20872.1869	0	32356.6543
9	6631.0914	0	24041.8991
10	-6819.2768	0	12411.9546
11	-18158.8572	0	0

All | Curve #1 0 deg

3D Interaction Surface

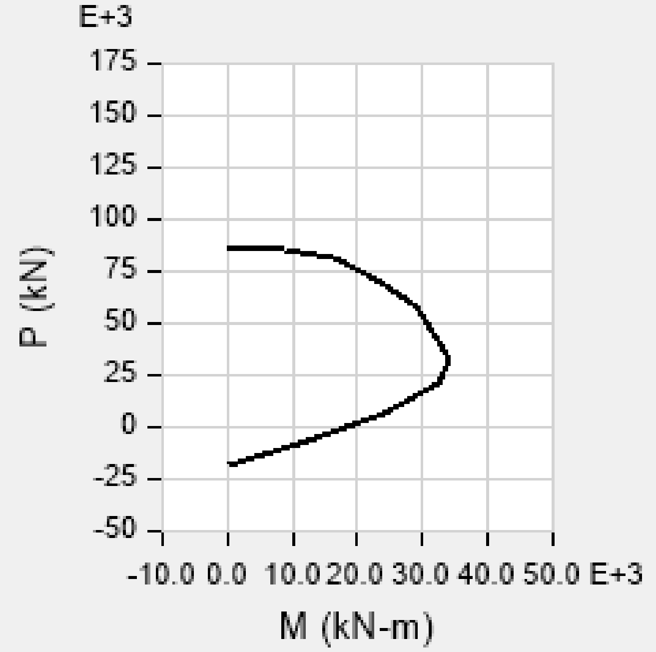


Plan 315 deg

Elevation 35 deg

3D | MM | PM3 | PM2

Current Interaction Curve



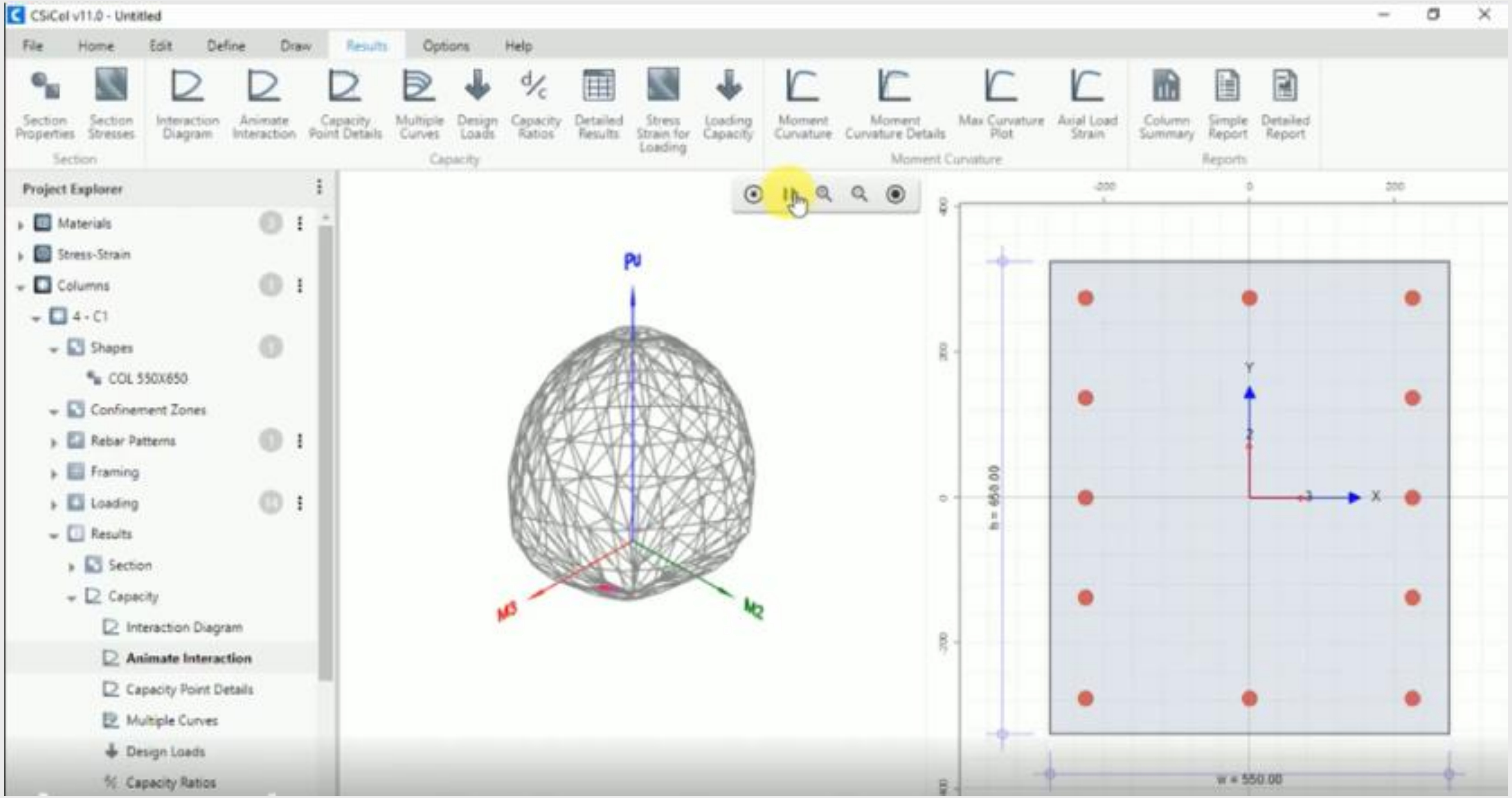
☐ Superimpose Dashed Fiber Curve

Note: Compression is positive in this form.

Done

Frame Design

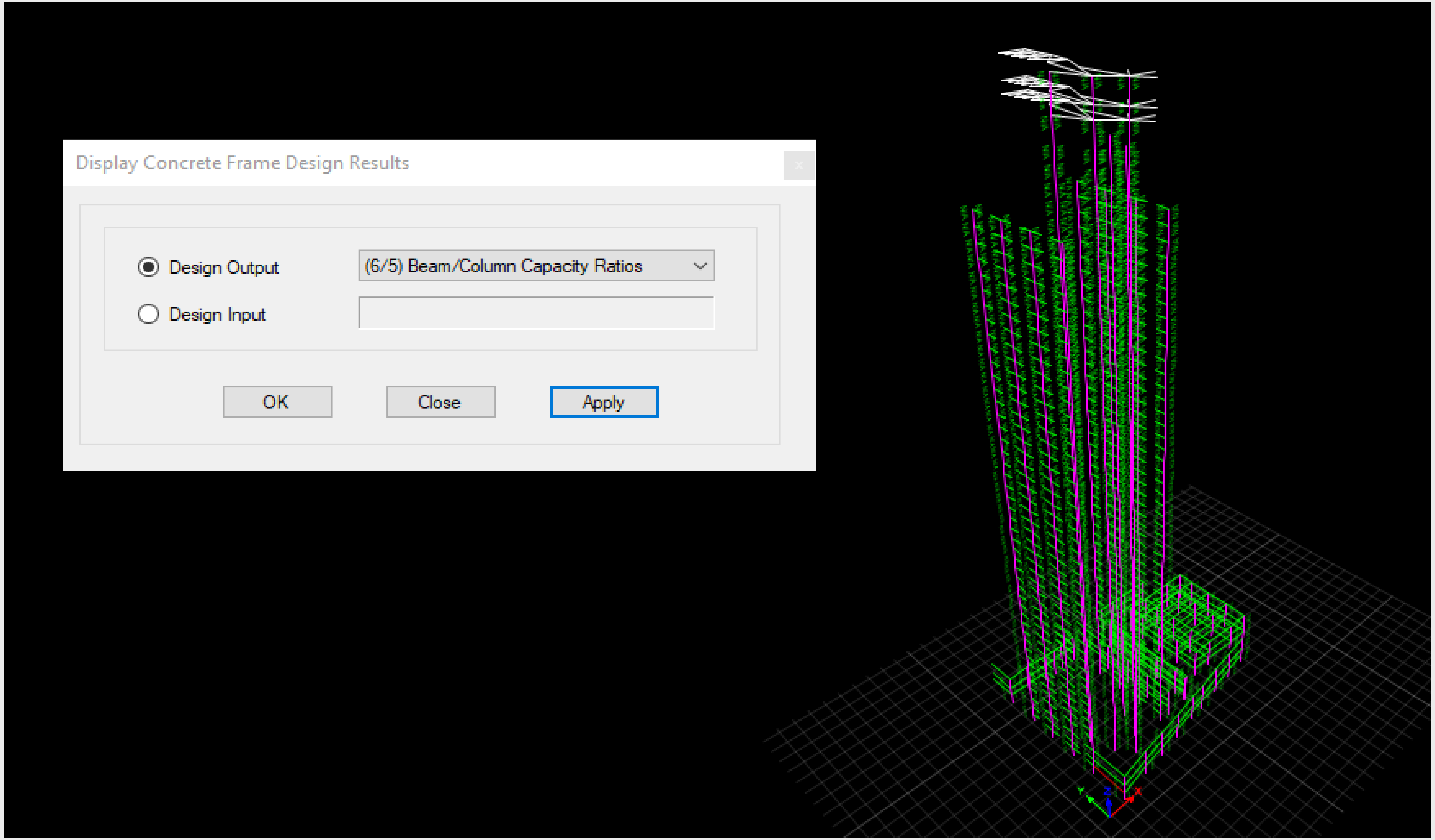
The Interaction surface for section by CSI COL



Frame Design

Check B-C Capacity

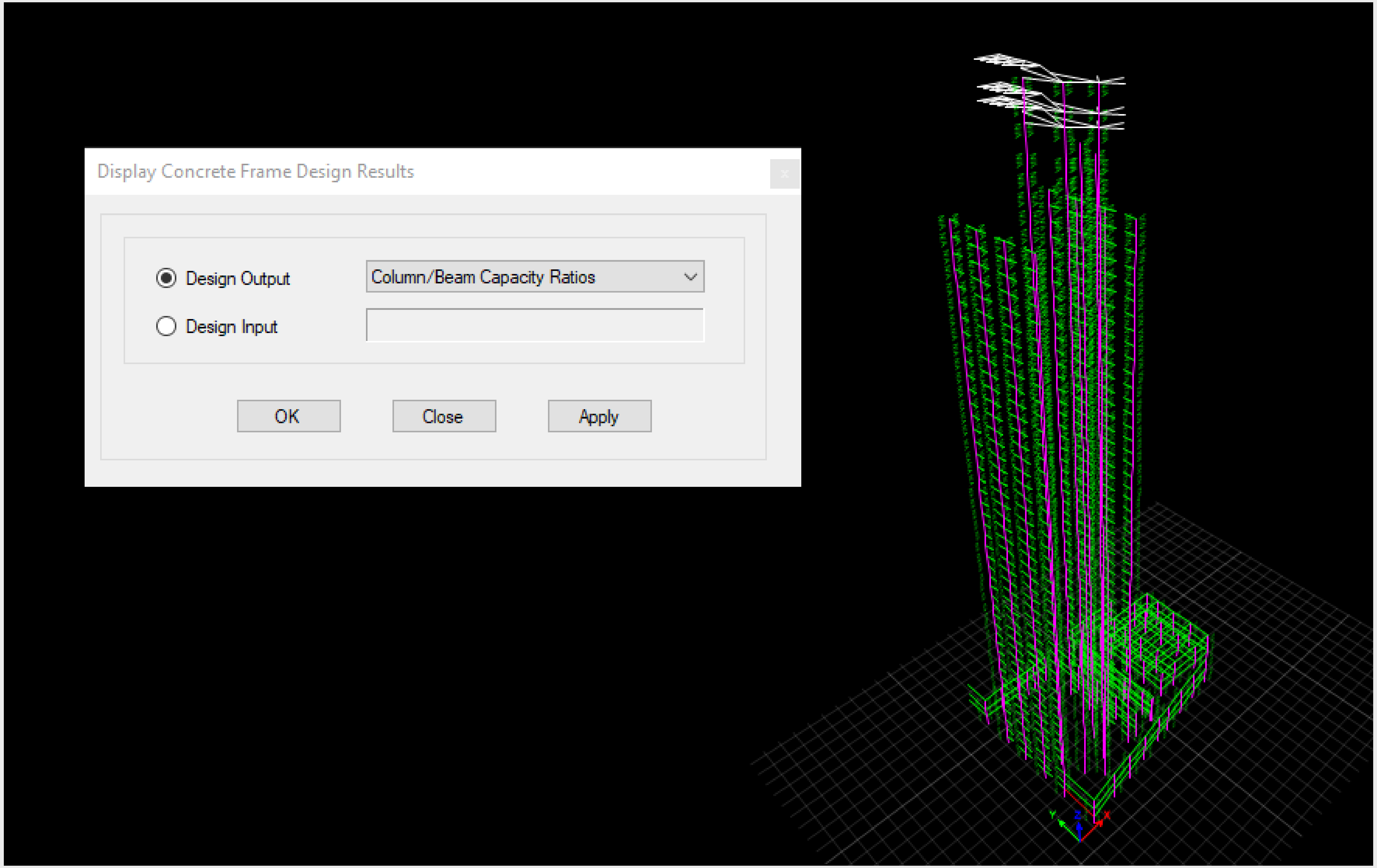
The ratios are less than 1 its is safe



Frame Design

Check C-B Capacity

The ratios are greater than 1 its is safe.

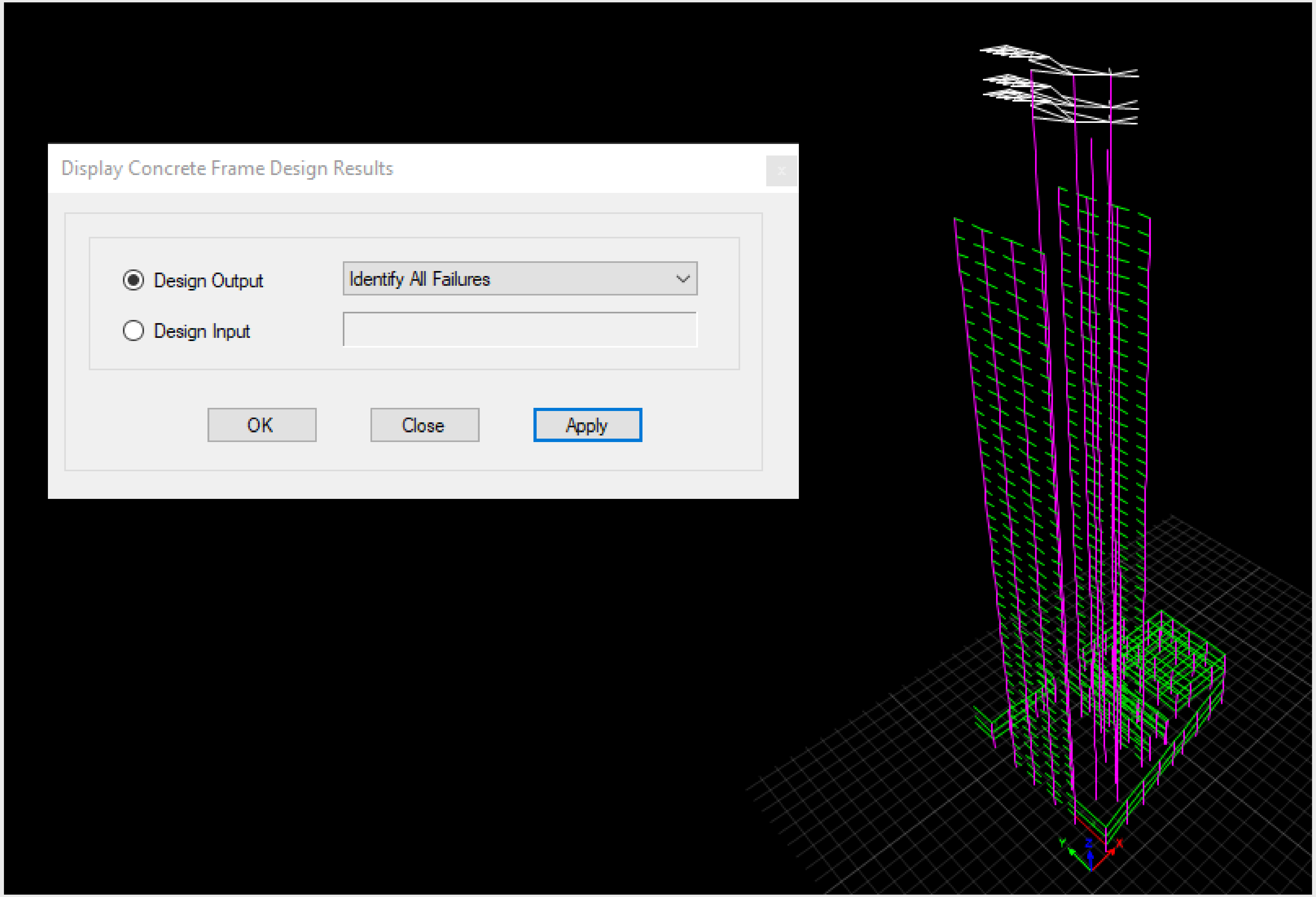


Frame Design

Identity member failure

All members are passed.

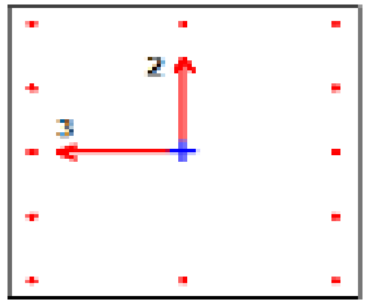
All sections size are very economic and maximum rebar % for column.



COLUMN SECTION DESIGN

ETABS Concrete Frame Design

ACI 318-14 Column Section Design



Column Element Details (Summary)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF	Type
3RD BASE F.FL.	C133	104	COL 900X1000	DCon22	2869.7	3500	0.403	Sway Special

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
900	1000	60	27.3

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
24855.58	27.58	1	413.69	413.69

Design Code Parameters

Φ_T	Φ_{CTied}	$\Phi_{CSpiral}$	Φ_{Vns}	Φ_{Vs}	Φ_{Vjoint}	Ω_0
0.9	0.65	0.75	0.75	0.6	0.85	2

Axial Force and Biaxial Moment Design For P_u , M_{u2} , M_{u3}

Design P_u kN	Design M_{u2} kN-m	Design M_{u3} kN-m	Minimum M2 kN-m	Minimum M3 kN-m	Rebar Area mm²	Rebar % %
1132.3998	19.3777	-65.8935	0	0	9000	1

Axial Force and Biaxial Moment Factors

	C_m Factor Unitless	δ_{ns} Factor Unitless	δ_s Factor Unitless	K Factor Unitless	Effective Length mm
Major Bend(M3)	0.388701	1	1	1	2869.7
Minor Bend(M2)	0.339619	1	1	1	2869.7

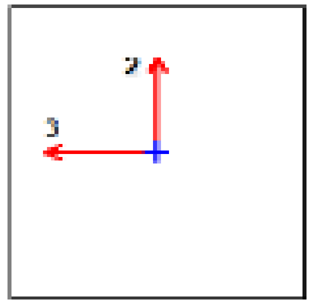
Shear Design for V_{u2} , V_{u3}

	Shear V_u kN	Shear ΦV_c kN	Shear ΦV_s kN	Shear ΦV_p kN	Rebar A_v /s mm²/m
Major, V_{u2}	141.7007	0	141.7007	141.7007	485.86
Minor, V_{u3}	25.0233	0	25.0233	25.0233	96.01

BEAM SECTION DESIGN

ETABS Concrete Frame Design

ACI 318-14 Beam Section Design



Beam Element Details (Summary)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF	Type
IST BASE F.FL.	B740	387	BEAM 550X650	DCon20	10029.3	10527.8	0.658	Sway Ordinary

Section Properties

b (mm)	h (mm)	b_f (mm)	d_s (mm)	d_{ct} (mm)	d_{cb} (mm)
550	650	550	0	60	60

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
24855.58	27.58	1	413.69	413.69

Design Code Parameters

Φ_T	Φ_{CTied}	$\Phi_{CSpiral}$	Φ_{Vns}	Φ_{Vs}	Φ_{Vjoint}
0.9	0.65	0.75	0.75	0.6	0.85

Design Moment and Flexural Reinforcement for Moment, M_{u3}

	Design -Moment kN-m	Design +Moment kN-m	-Moment Rebar mm²	+Moment Rebar mm²	Minimum Rebar mm²	Required Rebar mm²
Top (+2 Axis)	-336.2983		1601	0	1082	1601
Bottom (-2 Axis)		0	0	0	0	0

Shear Force and Reinforcement for Shear, V_{u2}

Shear V_{u2} kN	Shear ΦV_c kN	Shear ΦV_s kN	Shear V_p kN	Rebar A_v /S mm²/m
270.7071	212.2536	58.4536	40.9304	319.32

Torsion Force and Torsion Reinforcement for Torsion, T_u

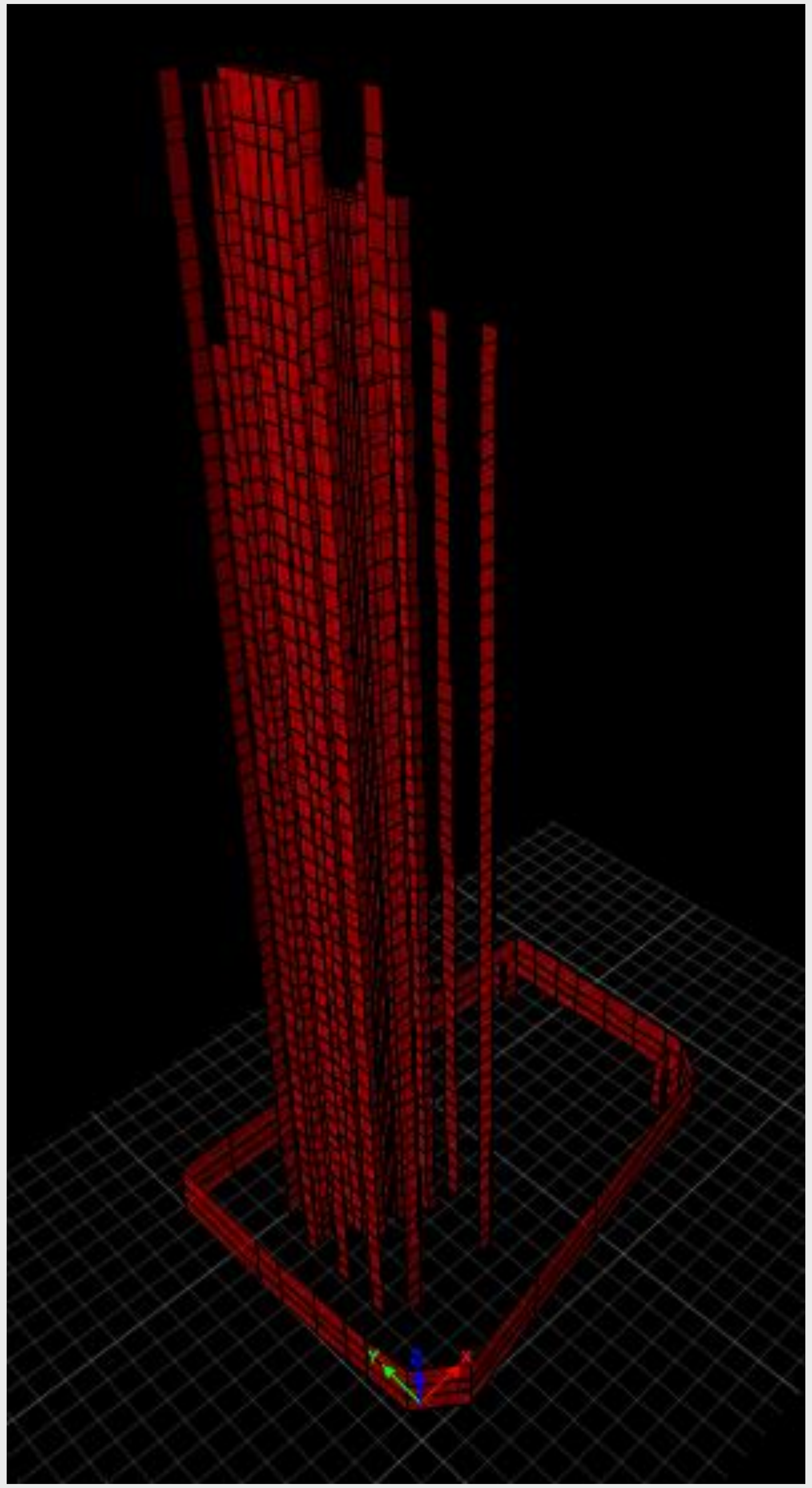
Φ*T_u kN-m	Tth kN-m	Tcr kN-m	Area A_o cm²	Perimeter, p_h mm	Rebar A_t /s mm²/m	Rebar A_l mm²
85.9882	17.4161	69.6644	2199.1	2044.4	630.12	1288

Shear wall & Core wall Design

- Excellent against lateral loads
- Can be used for column/wall
- Must to have for high rise building

- Minimum thickness : 150mm
- Minimum steel : 0.0025%

Lets design it we can see that piers are design for lift shaft now each time design pier, pier is another name of column nothing different it can design one by one here we can see it can design it the shear wall and provide the reinforcement .



WALL SECTION DESIGN

ETABS Shear Wall Design

ACI 318-14 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (mm)	Centroid Y (mm)	Length (mm)	Thickness (mm)	LLRF
3RD BASE F.FL.	P115	33000	18500	3000	300	0.4

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
24855.58	27.58	1	413.69	413.69

Design Code Parameters

Φ_T	Φ_C	Φ_V	Φ_V (Seismic)	IP_{MAX}	IP_{MIN}	P_{MAX}
0.9	0.65	0.75	0.6	0.04	0.0025	0.8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 mm	Left Y_1 mm	Right X_2 mm	Right Y_2 mm	Length mm	Thickness mm
Top	Leg 1	31500	18500	34500	18500	3000	300
Bottom	Leg 1	31500	18500	34500	18500	3000	300

Flexural Design for P_u, M_{u2} and M_{u3}

Station Location	Required Rebar Area (mm²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P_u kN	M_{u2} kN-m	M_{u3} kN-m	Pier A_g mm²
Top	2250	0.0025	0.0023	DWal22	5343.0613	-9.9439	-18.3214	900000
Bottom	2250	0.0025	0.0023	DWal22	5402.4404	4.9676	-30.949	900000

Shear Design

Station Location	ID	Rebar mm²/m	Shear Combo	P_u kN	M_u kN-m	V_u kN	ΦV_c kN	ΦV_n kN
Top	Leg 1	750	DWal20	8682.4746	-29.7722	5.8628	706.4217	1264.8971
Bottom	Leg 1	750	DWal20	8778.9656	-50.2922	5.8628	712.5447	1271.0201

Boundary Element Check (ACI 18.10.6.3, 18.10.6.4)

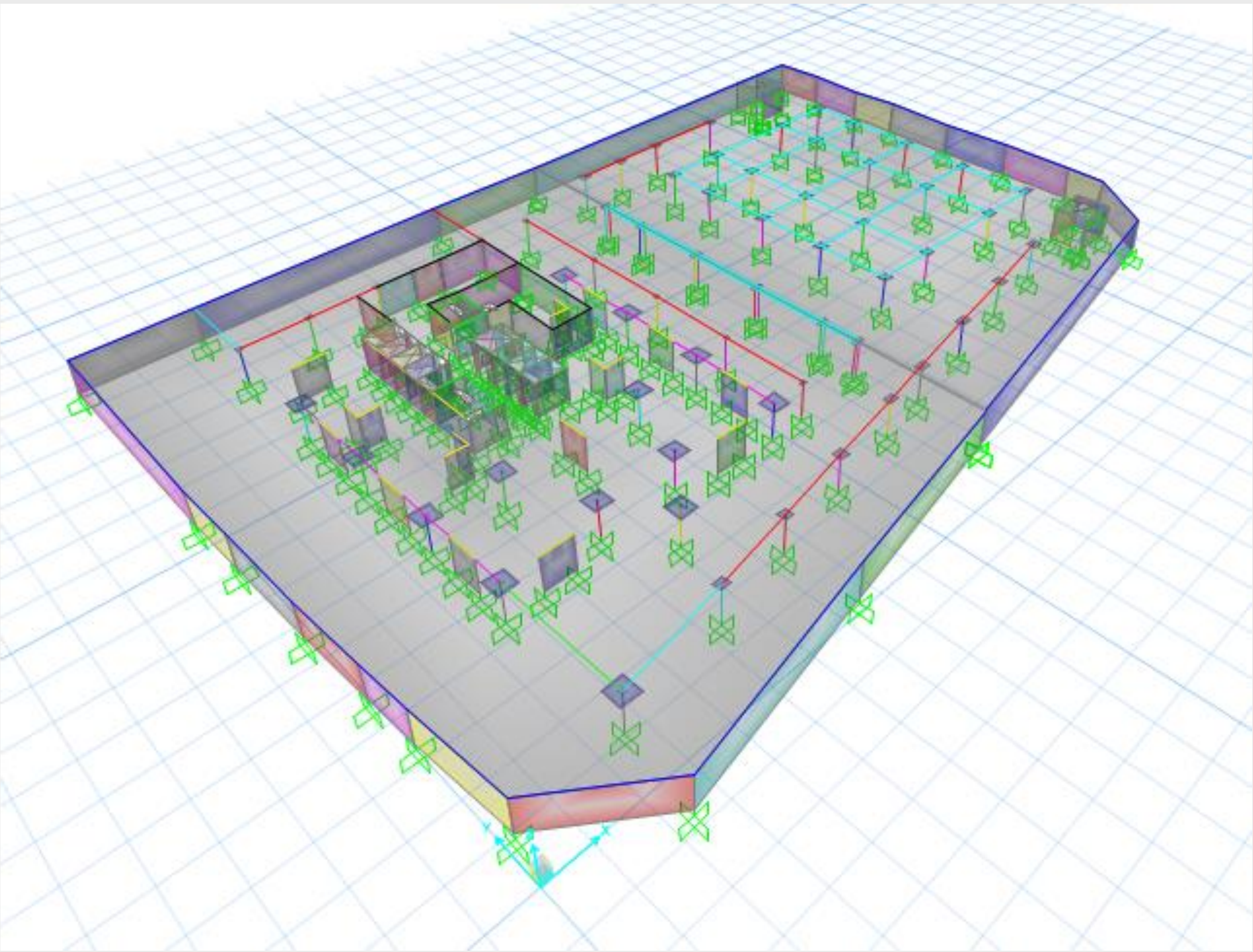
Station Location	ID	Edge Length (mm)	Governing Combo	P_u kN	M_u kN-m	Stress Comp MPa	Stress Limit MPa	C Depth mm	C Limit mm
Top–Left	Leg 1	1879.8	DWal11	8682.4746	-29.7722	9.71	5.52	2179.8	666.7
Top–Right	Leg 1	1879.8	DWal11	8682.4746	-29.7722	9.58	5.52	2179.8	666.7
Bottom–Left	Leg 1	1903.2	DWal11	8778.9656	-50.2922	9.87	5.52	2203.2	666.7
Botttom–Right	Leg 1	1903.2	DWal11	8778.9656	-50.2922	9.64	5.52	2203.2	666.7

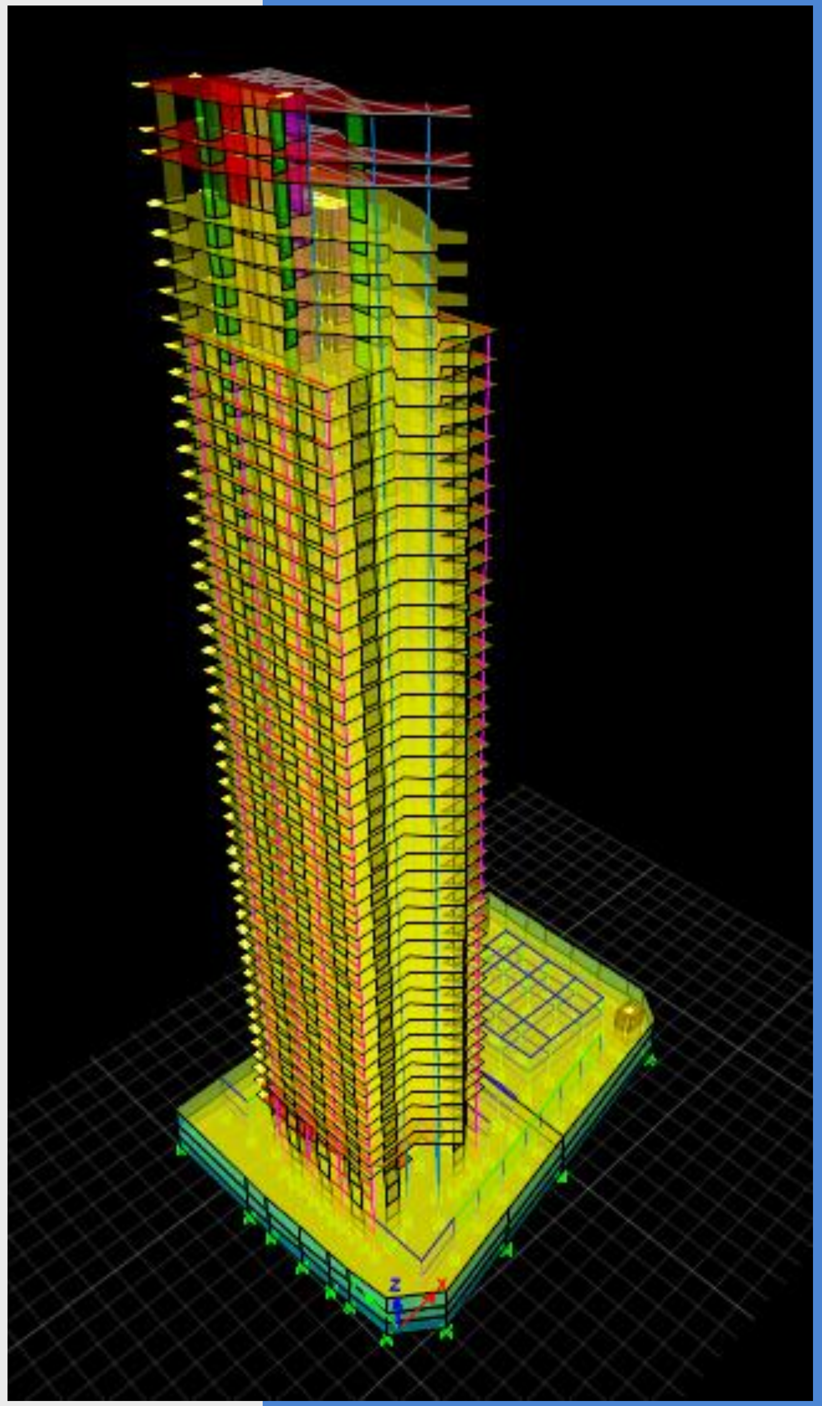
SLAB Design

Slab can be analyze and design with the help of SAFE software and slab spread sheets in which we can easily found out slab reinforcement for any length of span.

For **shell thin** in which neglect the shear deformation generally used in normal slabs on the other hand **shell thick** consider shear deformation The thick plate formulation captures both shear and bending deformations whereas the thin plate the formulation is based only on bending deformations and neglects shear deformation generally used in Raft slabs, foundation pads, transfer slab.

Check for deflection more deflected slab like cantilever slabs, large span slab, there check it, to control it by increase slab depth and beam depth.





Short and Long term deflections

Three types of **deflections** are available, including:

1. Elastic Deflection
2. Immediate cracked deflection
3. Long-term cracked deflection accounting for creep and shrinkage

For Elastic Deflection

Section **uncracked** used

Load Combination = 1D (Dead) +1L (Live) +1SD (Super Dead)

For Immediate (Short term) cracked deflection

Section **cracked** used

Load Combination = 1D (Dead) +1L (Live) +1SD (Super Dead)

For Long term deflection

Section **cracked** used (with creep and shrinkage)

Load Combination (Sustained)= 1D (Dead) +0.25 L (Live) +1SD (Super Dead) → we consider 25 % live load as sustained load.

But with doing so, we are missing with rest of 75% live load deflection.

For Accommodation of 75% live load

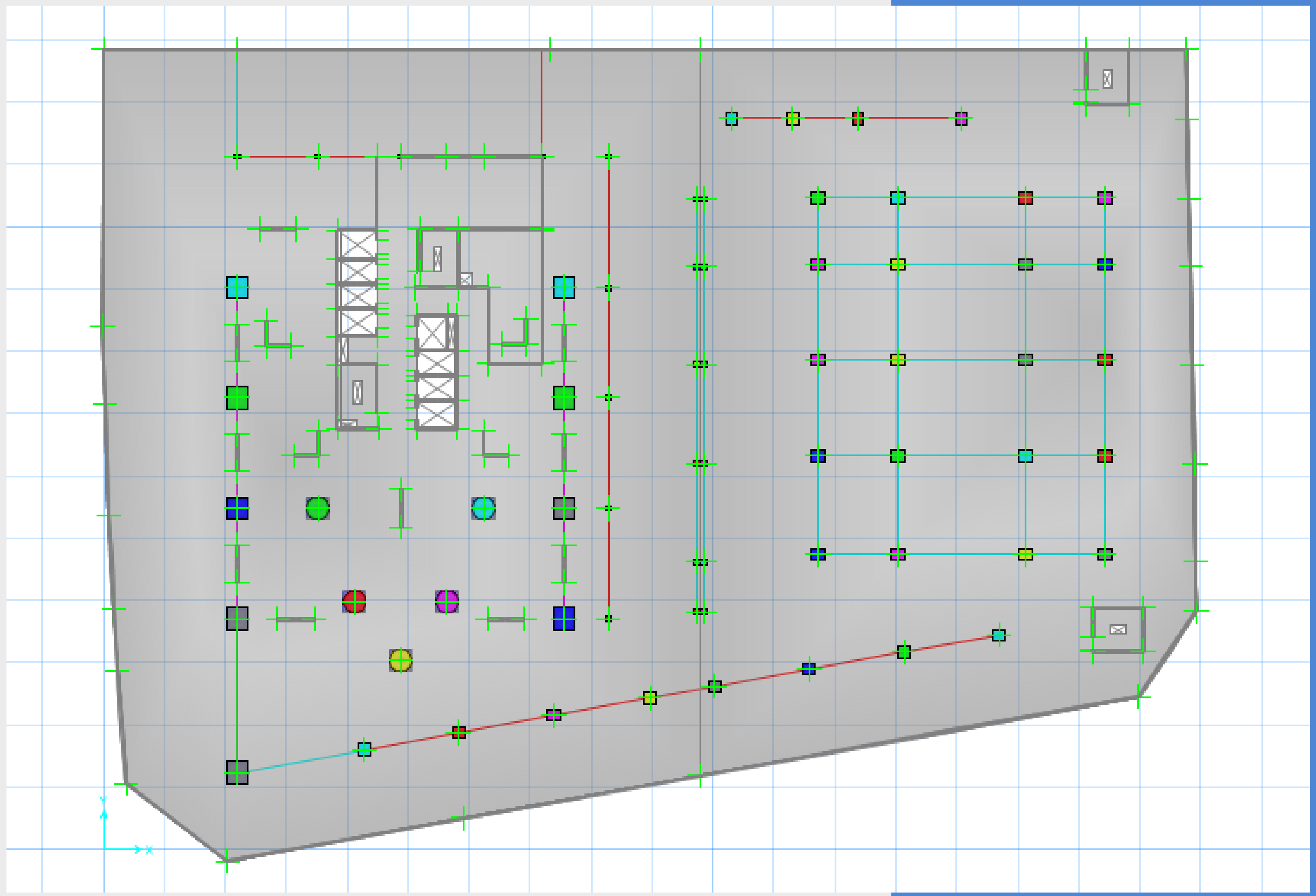
Long term Deflection Combination

= (1D +0.25 L +1SD {with C & Sh}) +[1D +1L +1SD] -[1D +0.25L +1SD]

Cracked Deflections

Immediate **- no Shrinkage or Creep**

Long Term **- Sustained Loads**
with
Creep & Shrinkage



(DEAD+SDEAD+0.25*LIVE) w/ Creep & Shrinkage

+

(DEAD+SDEAD+LIVE)

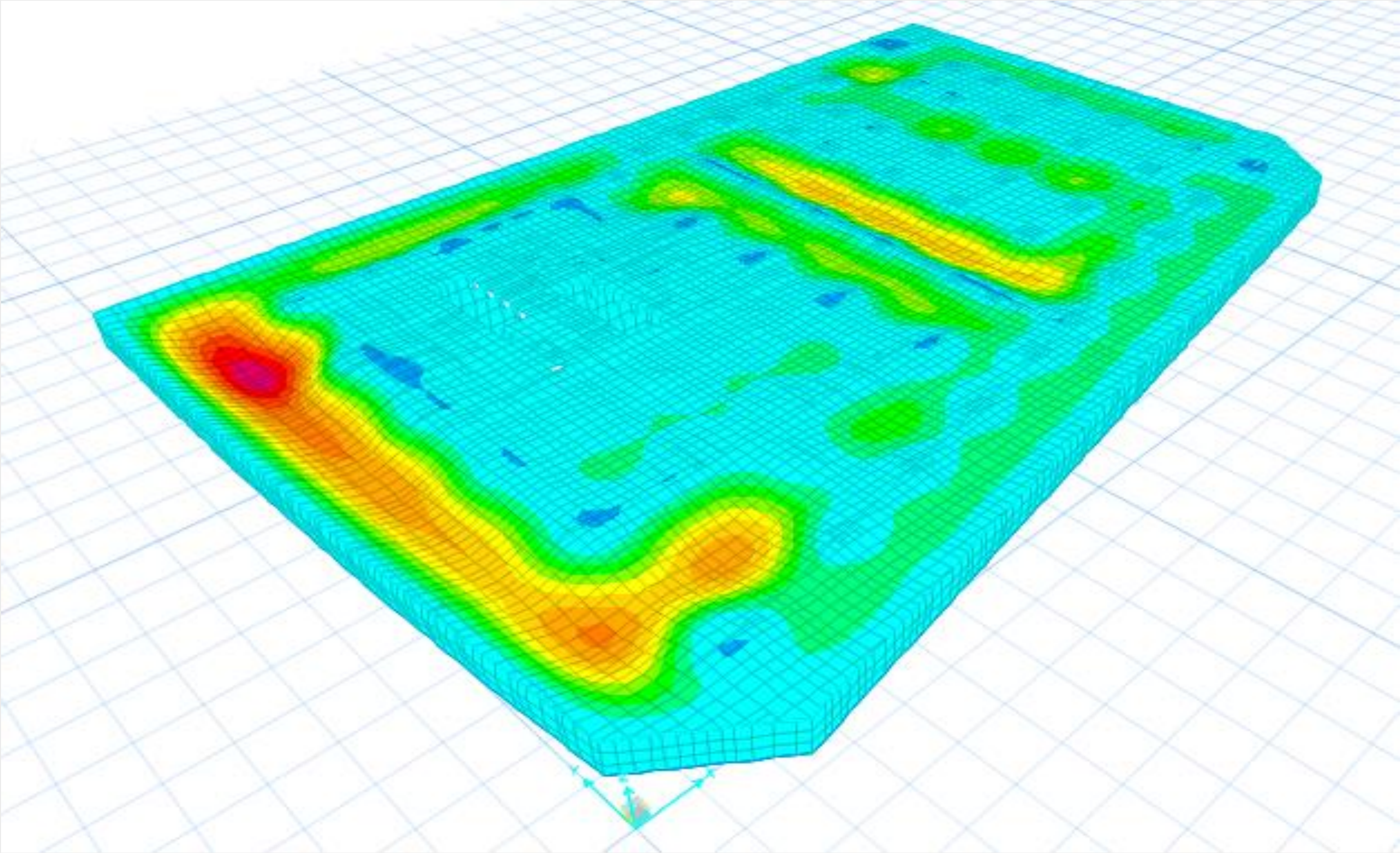
-

(DEAD+SDEAD+0.25*LIVE)

Allowable Deflections:

Immediate Deflection = Span /360

Long term deflection = Span /250

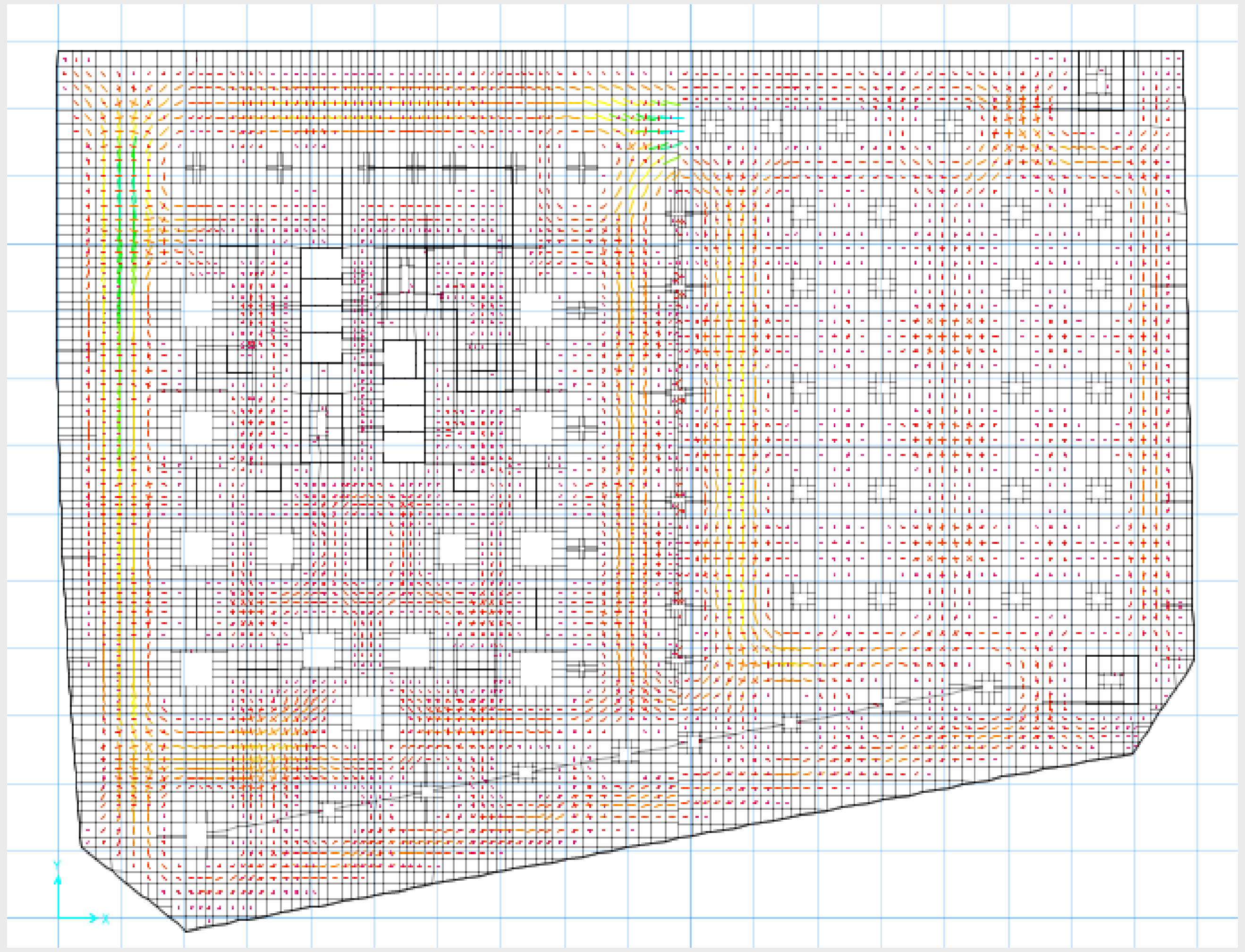


Max. deflection = 18mm

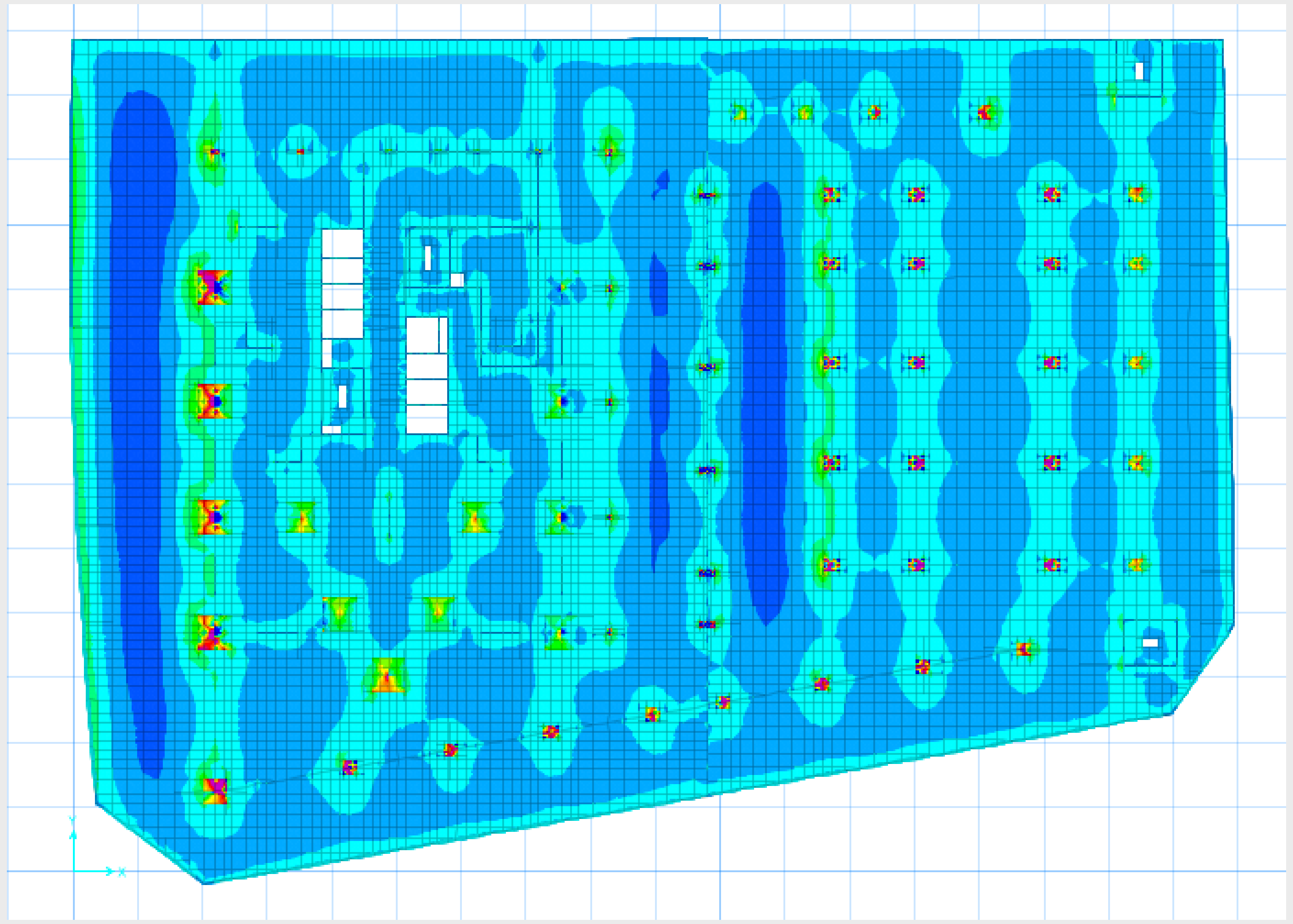
Crack Width

Immediate Deflection = Span /360

Long term deflection = Span /250



Crack Width =0.25



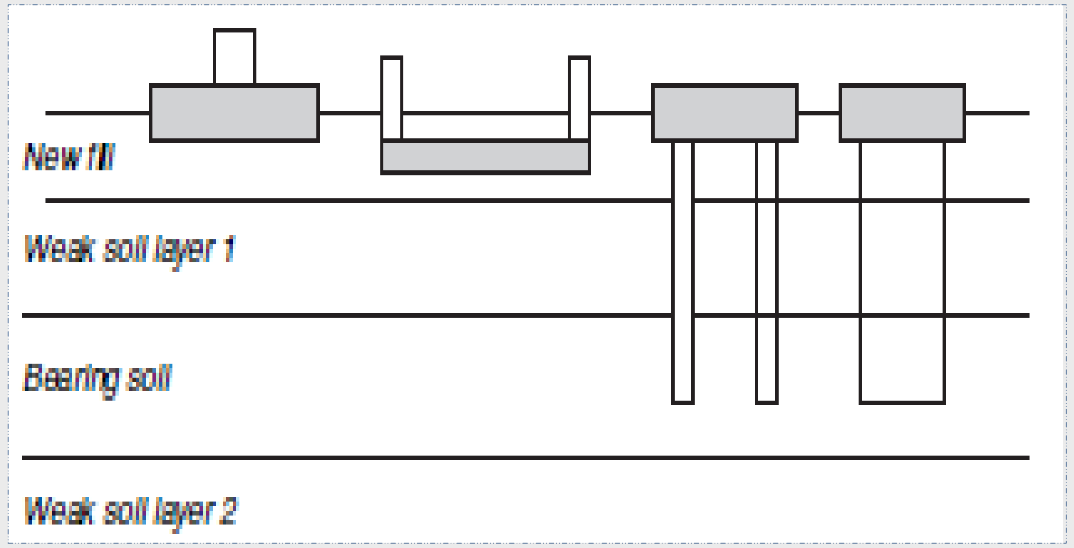
Resultant stresses

Footing

All engineered construction resting on the earth must be carried by some kind of interfacing element called a foundation. The foundation is the part of an engineered system that transmits to, and into, the underlying soil or rock the loads supported by the foundation and its self-weight. The resulting soil stresses except at the ground surface are in addition to those presently existing in the earth mass from its self-weight and geological history. The term superstructure is commonly used to describe the engineered part of the system bringing load to the foundation, or substructure. The term superstructure has particular significance for buildings and bridges; however, foundations also may carry only machinery, support industrial equipment (pipes, towers, and tanks), act as sign bases, and the like. For these reasons it is better to describe a foundation as that part of the engineered system that interfaces the load-carrying components to the ground. It is evident on the basis of this definition that a foundation is the most important part of the engineering system.

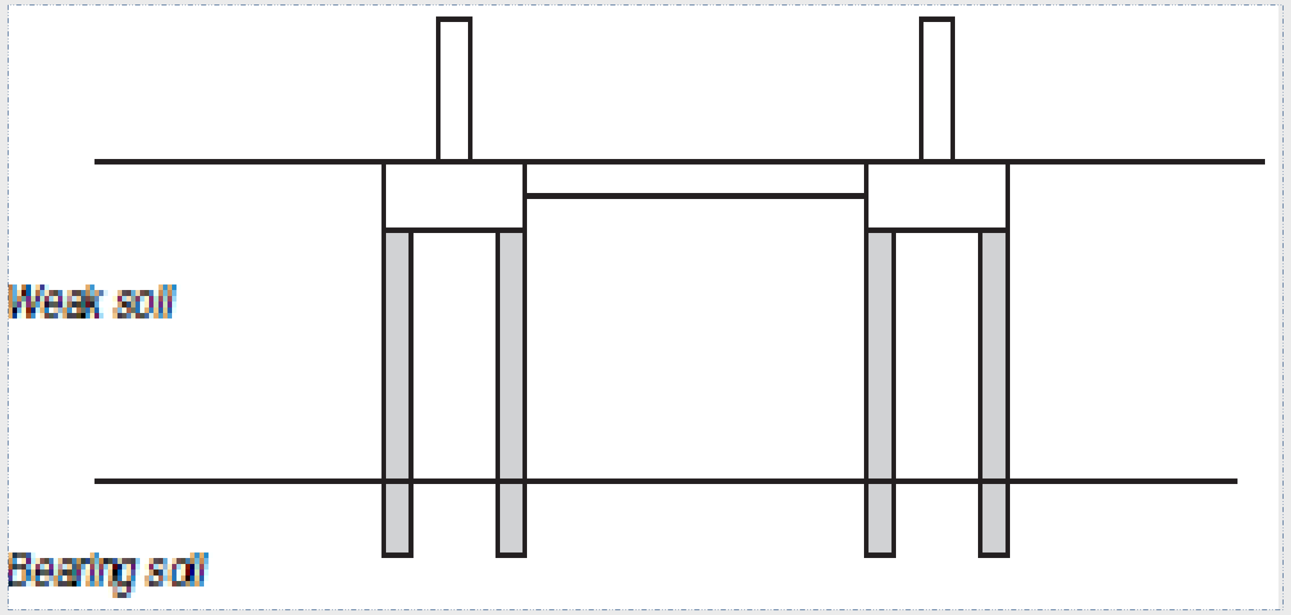
A mat foundation is a large concrete slab used to interface one column, or more than one column in several lines, with the base soil. A mat foundation may be used where the base soil has a low bearing capacity and/or the column loads are so large that more than 50 percent of the area is covered by conventional spread footings. It is common to use mat foundation for deep basements both to spread the column loads to a more uniform pressure distribution and to provide the floor slab for the basement.

The geotechnical engineer needs to compute the bearing capacity of the soil immediately below the footing. If the bearing capacity is adequate, settlement needs to be computed. Settlement can be immediate or long term. Both immediate and long-term settlements should be computed.



Footing

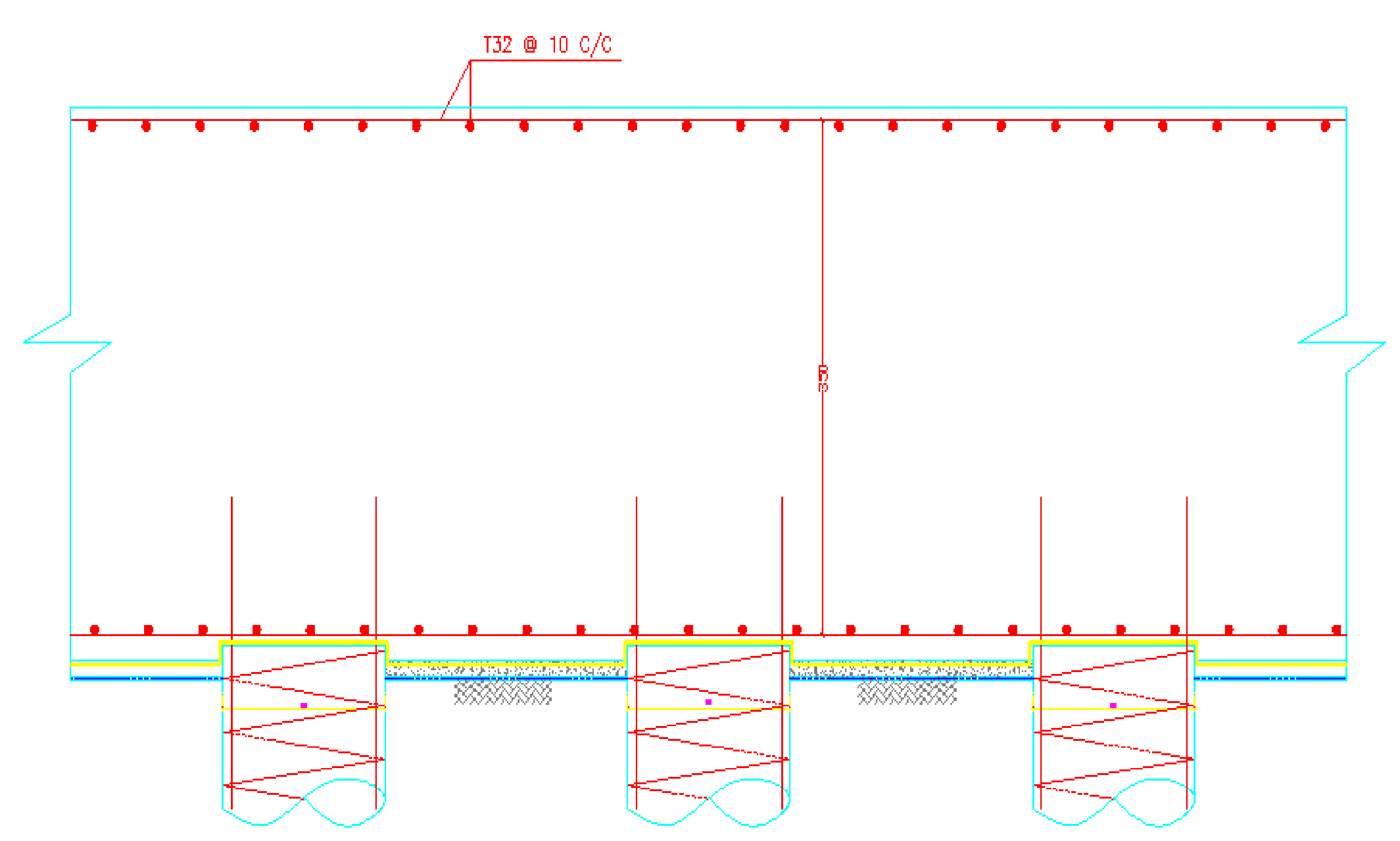
Piles are used primarily in areas where near-surface soil conditions are poor. They are made of timber, concrete, or steel and are located in clusters. The piles are driven down to strong soil or rock at a predetermined depth, and each cluster is then covered by a cap or reinforced concrete. A pile may support its load either at the end or by skin friction along its entire length. The number of piles in each cluster is determined by the structural load and the average load-carrying capacity of each pile in the cluster. A timber pile is simply the trunk of a tree stripped of its branches and is thus limited in height. A concrete pile, on the other hand, may be of an reasonable length and may extend below groundwater level as well. For extremely heavy or tall buildings, steel piles, known as H-piles because of their shape, are used. H-piles are driven through to bedrock, often as far as 30 m (100 ft) below the surface. H-piles can be driven to great depths more easily than piles made of wood or concrete; although they are more expensive, the cost is usually justified for large buildings, which represent a substantial financial investment.



Pile Specifications

Pile Data from Geotechnical Report
Pile Diameter/width (D) =1000MM & 900MM
Safe load of pile (Capacity) = 660KN
Length of Pile (L)= 10M
Spring constant (K=EA/L)

Due to SBC is low, and settlement is more than our allowable limit
Preferred Raft + Pile Foundation



Geotechnical Report for Foundation Recommendations

Residential Project up to G+2
Minimum SBC: 8Tonne/m2 or 120 kN/m2

Commercial Multi-storey Building
Soil Test is Must

Geotechnical Report for Foundation Recommendations

To determined during logging based on the standard practices prescribed by BS 5930:2015 as well as the values of the unconfined compressive strength obtained in the laboratory, based on which the final strength descriptions were adopted. The adopted scale of rock strength based on the unconfined compressive test and as per BS-5930 standard requirements is seen in table-2

Table-1: Relative Density of Granular Soils Based on SPT N Values.

Term	SPT N-values (blows/300 mm penetration)
Very loose	0 to 4
Loose	4 to 10
Medium dense	10 to 30
Dense	30 to 50
Very dense	over 50

Table-2: Strength Scales of Rocks Cores Based on Standard UCS Test Value

Term	Unconfined Compressive Strength (MN/m^2)
Extremely weak	0.60 – 1.0
Very weak	1.0 to 5.0
Weak	5.0 to 25
Medium Strong	25 to 50
Strong	50 to 100
Very strong	100 to 250
Extremely strong	> 250

Geotechnical Report for Foundation Recommendations

Table -4: General Stratigraphy

Depth*	Description
0.00 to 0.50 m	Brown, silty, fine to medium grained, calcareous **SAND**.
0.50 to 1.85/2.35 m	Medium Dense to very dense, locally dense, brown, silty, fine to medium grained, calcareous **SAND**.
1.85/2.35 m to 12.13 m (End of BH)	Extremely weak to weak, very thinly to thinly bedded, brown, **SANDSTONE,** partially weathered, very closely to closely spaced, horizontal to sub horizontal fractures/Very dense poorly cemented SAND.

*Depth is related to existing ground level.

5.2 Groundwater

At the time of investigation, ground water table level encountered below the existing ground surface is shown in table no. 5; variations in ground water table are anticipated due to seasonal and tidal fluctuations or by artificial induced effects. Therefore reconfirmation is recommended prior to any works related to the ground water regime.

Table: 5 – Groundwater level records

Borehole ID	BH depth (m)	Groundwater Level (m, below EGL)
BH-01	12.0	4.60
BH-02		4.50
BH-03		4.40

Geotechnical Report for Foundation Recommendations

6.1.4 Bearing Capacity of Shallow Foundations

Table-6(a): Allowable Bearing Pressure for Strip / Isolated / Pad foundation

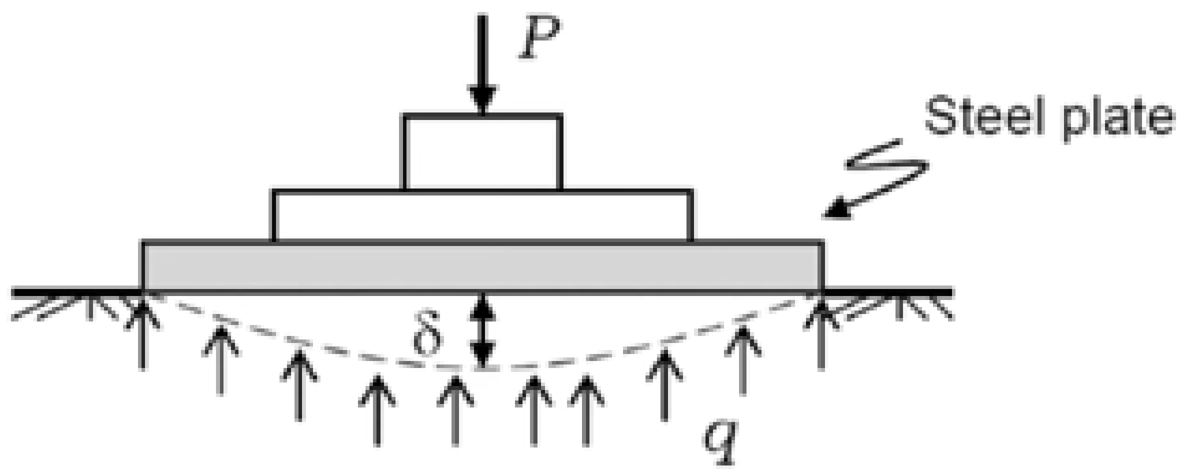
Foundation Type	Depth of foundation Below EGL(m)	Maximum Foundation Width(m)	Allowable bearing Pressure (kN/m^2)	Modulus of Subgrade Reaction (kN/m^3)
Shallow (Strip / Isolated) Limiting settlement of 25.0mm	1.0	3.0	150	18000
	2.0		250	30000
	3.0		365	43800
	4.0		500	60000

Table-6(b): Allowable Bearing Pressure for raft foundation

Foundation Type	Depth of foundation Below EGL(m)	Size of Foundation (m)	Allowable bearing Pressure (kN/m^2)	Modulus of Subgrade Reaction (kN/m^3)
Shallow (Raft / Basement Raft) Limiting settlement of 50.0mm	3.00 - 4.00	43x50	650	39000
	6.00 - 7.00	43x50	700	42000

Geotechnical Report for Foundation Recommendations

Factor of safety	3	-
Bearing capacity of soil	300	Kn/m2
Allowable settlement	50	mm
Allowable settlement	0.05	Kn/m2
Bearing capacity u	900	Kn/m2
K=p/A	18000	Kn/m3



$$k_s = \frac{q}{\delta}$$ ← Soil pressure / ← Settlement

$\delta = 1$ *in.* (25 *mm.*)

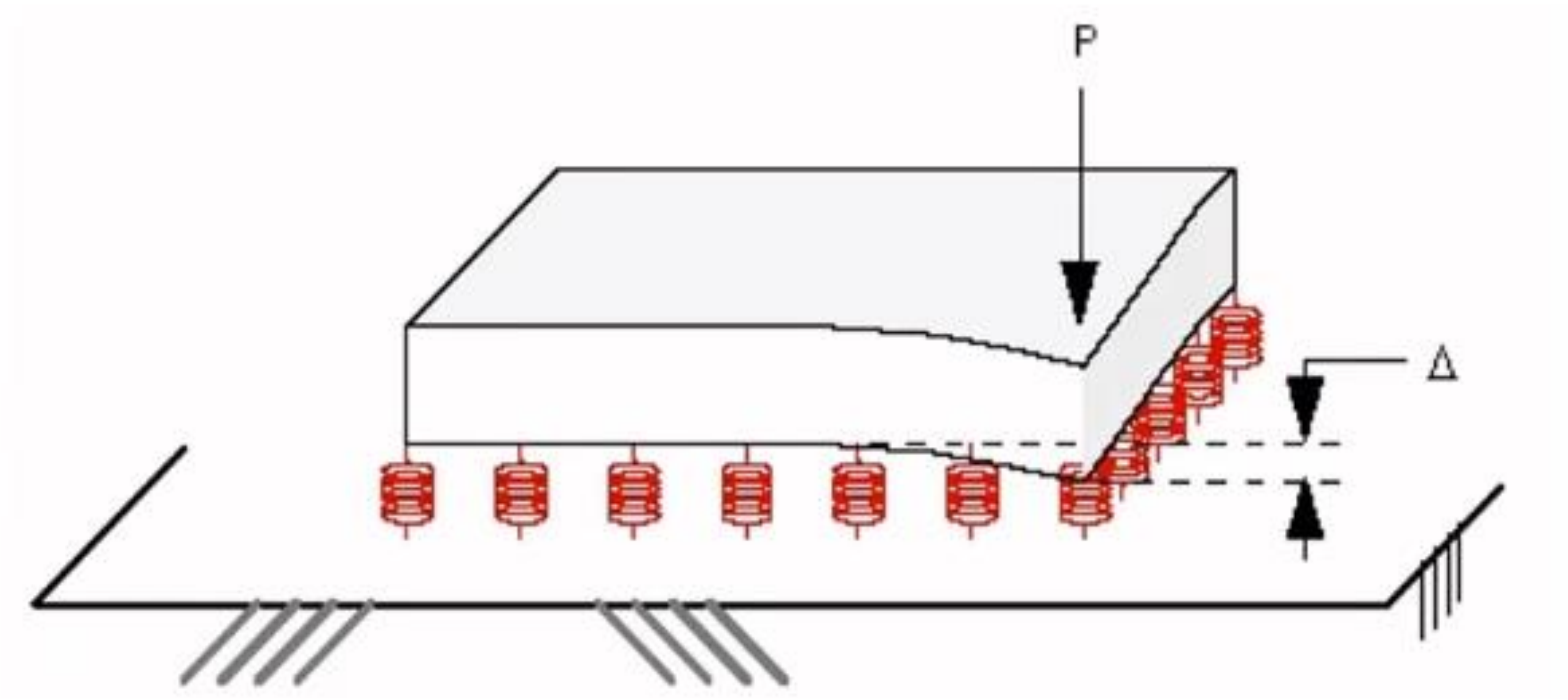
K_s = Modulus of subgrade reaction or
Subgrade modulus or
Coefficient of subgrade reaction

where q = soil pressure, and delta = 1 inch (assumed allowable settlement)

JOSEPH E. BOWLES, Foundation analysis and design. Fifth edition. Page 501-505

Geotechnical Report for Foundation Recommendations

The reactive pressure to resist a load is thus proportional to the spring deflection (which is a representation of slab deflection) and k (Figure 2):



$$P = k\Delta$$

where: P = reactive pressure to support deflected slab

k = spring constant = modulus of subgrade reaction

Δ = slab deflection

Typically, the modulus of subgrade reaction is estimated from other strength/stiffness tests, *however, in situ values can be measured using the plate bearing test.*

Conclusions

The both approaches are working fine in the design field. Bowl's Approach modulus of subgrade (ks) gives conservative and satisfactory values for the design.

The Basic approach is to conservative and on extreme safe side and it follows the basic philosophy spring constant.

Recommendations

You can use Bowl's Approach modulus of subgrade (ks) and it is also safe to use.

Foundation Systems

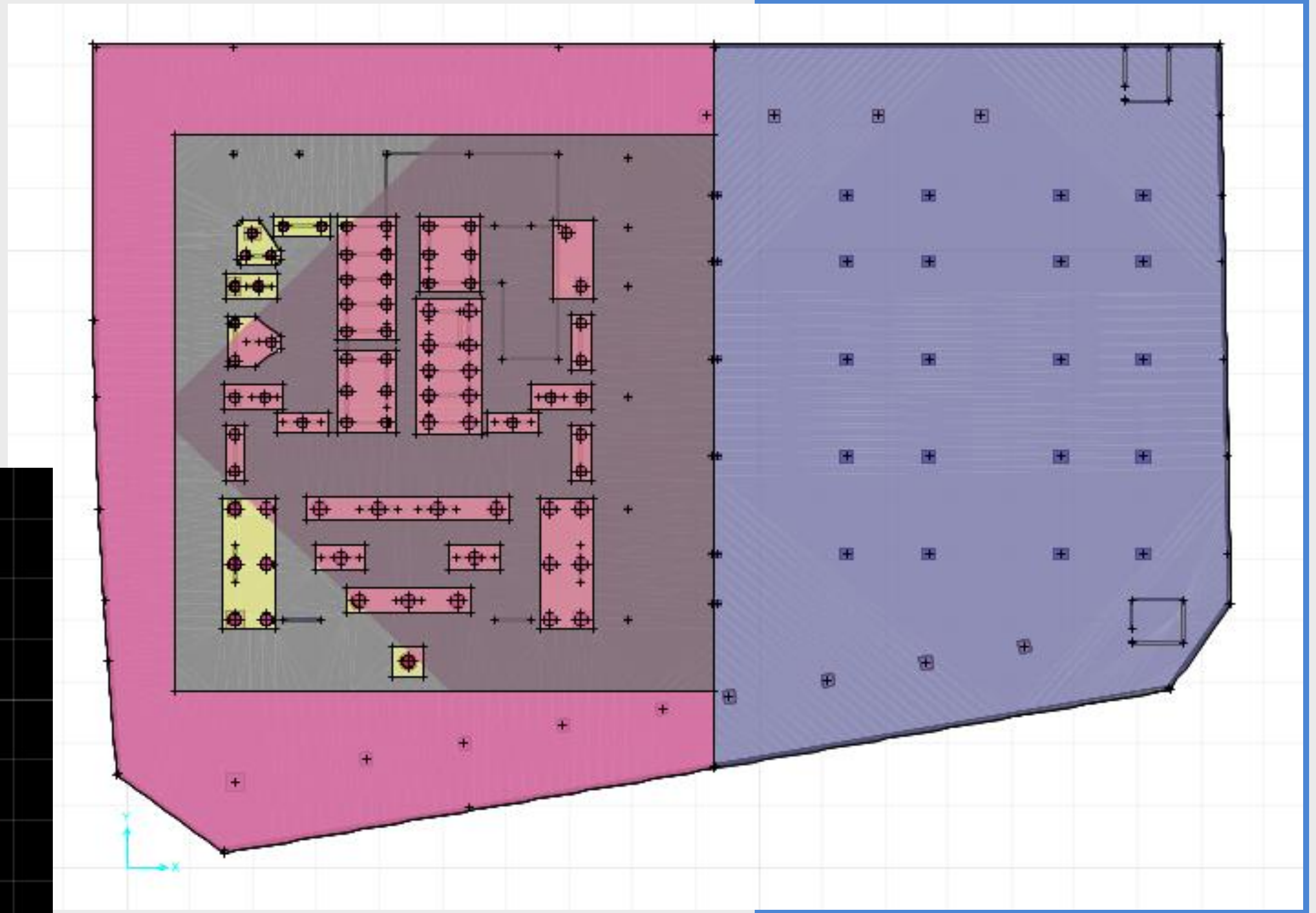
- Foundation system Descriptions

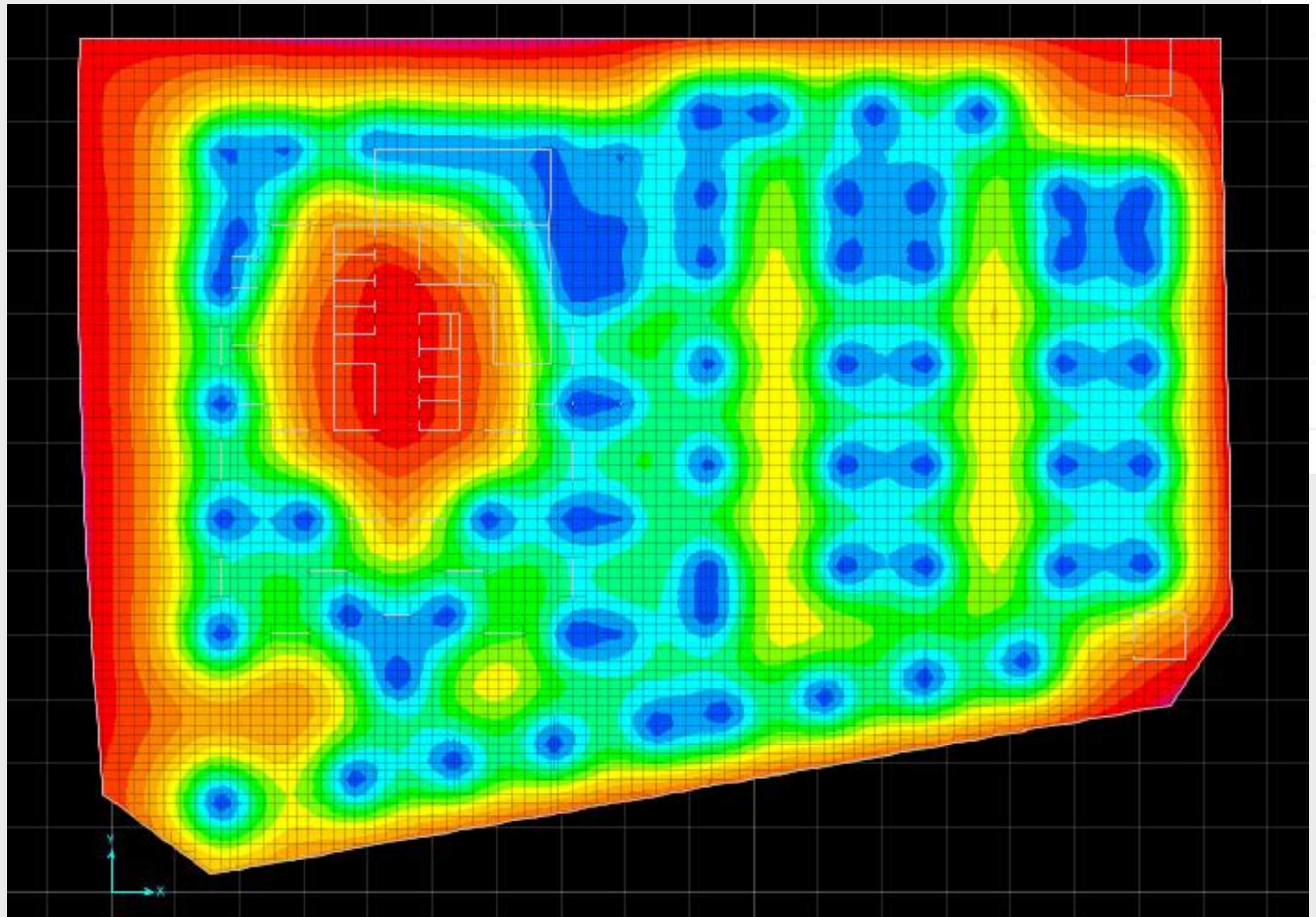
  Raft and Raft+Pile Footing of size 90m x 65m with depth of 1.4m & 1.5 m and grade of Fc 25 & Fc 30 provided.
  Drop of size 3400mm x 1800mm
  Beam of 400mm x 900mm

Commercial Building G+47 stories + 3 basement for parking with GL.

Material Grades
- F'c = 25Mpa
- F'c = 30 Mpa
- Fy = 500Mpa
- Fyt = 415Mpa
- Qa = 300 KN/m²

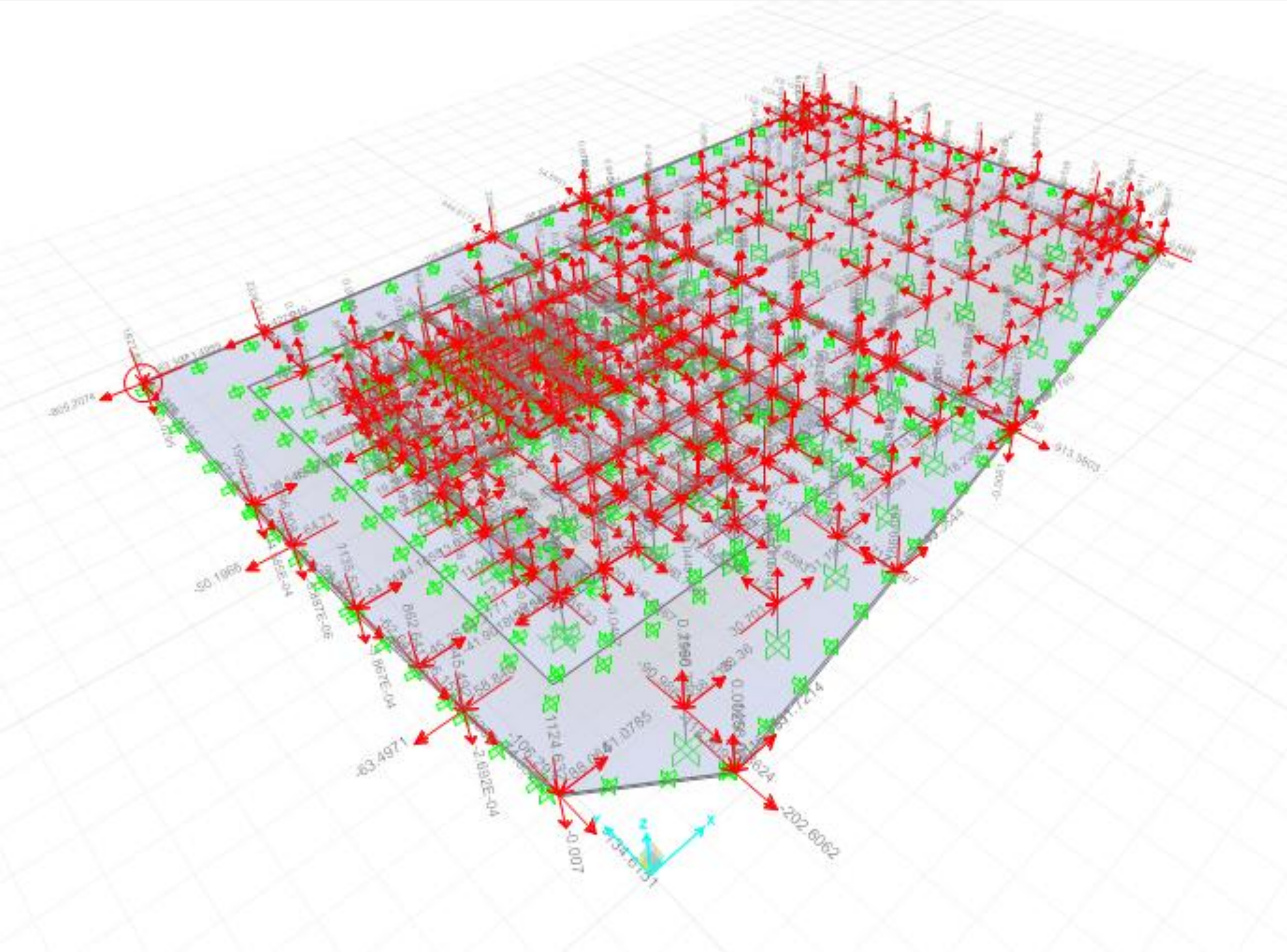
(Bearing capacity of soil)

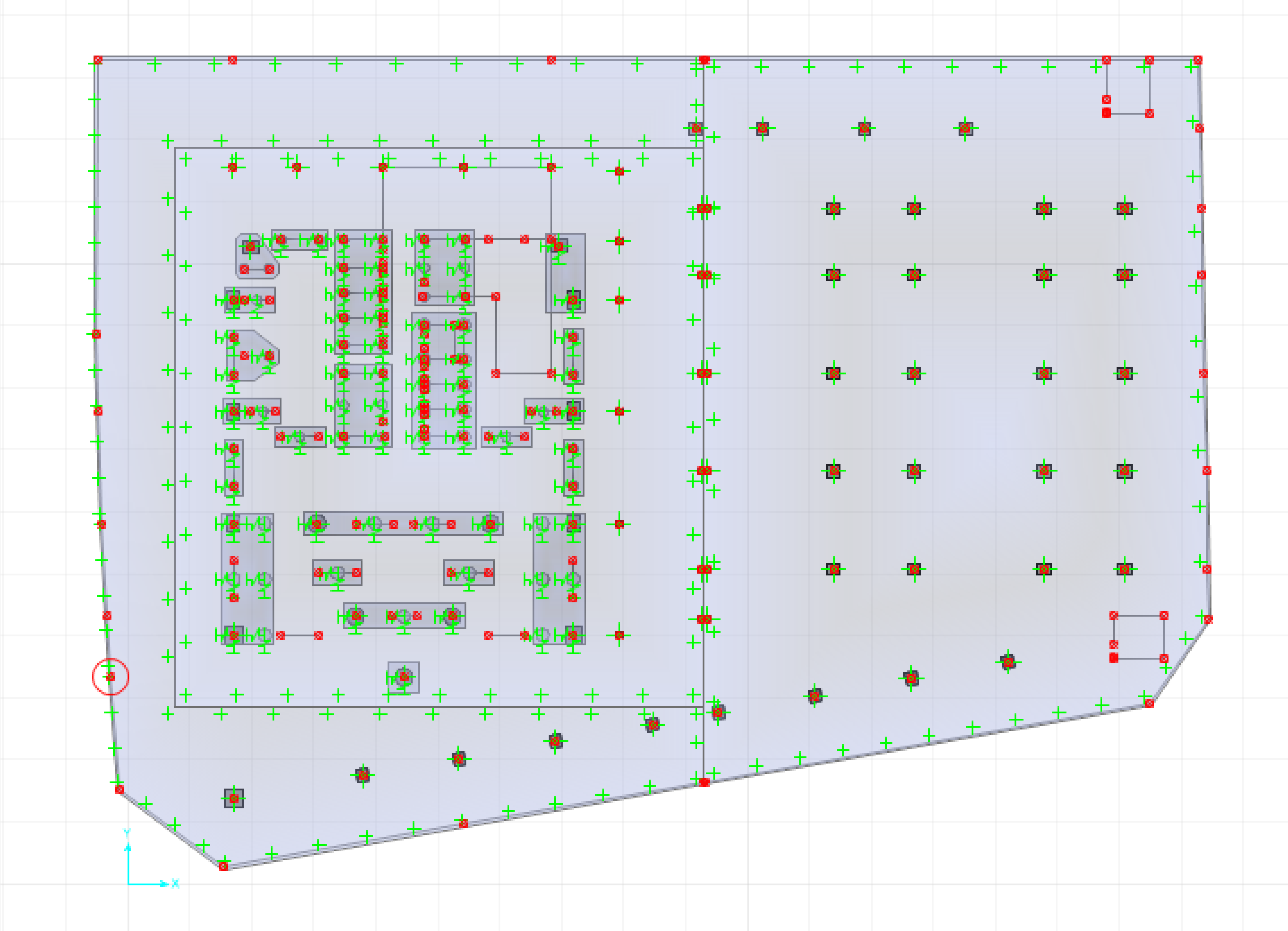




Foundation Systems

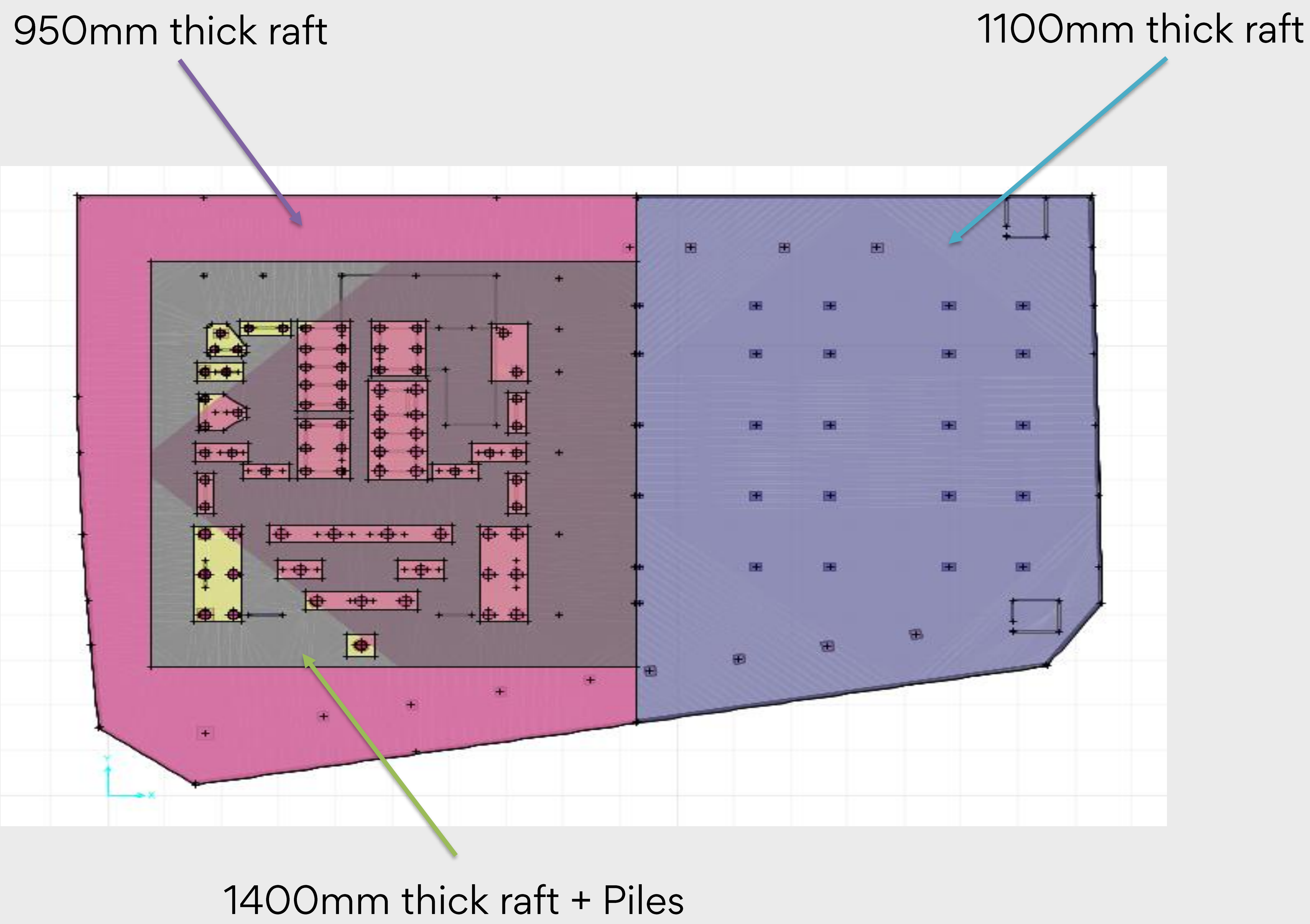
Base Reactions





Load Case/Combo	FX	FY	FZ	MX	MY	MZ
Dead	0.00001609	-0.0046	525477.84	5371202.72	-10599787.00	-2.81
Live	6.782E-07	-0.0007	55735.93	567263.87	-1128568.00	-0.67
Live>3	6.696E-07	-0.0007	26800.18	247148.04	-536506.12	-0.08
WX	0.0026	-13019.6153	0.00	1235702.47	45609.18	250.34
WY	-6216.3309	-0.0002	0.00	579.47	-5§77736.55	-104.14
SPEC-X MAX.	5980.9973	1699.1763	0.00	4156755.07	441428 27	97836.68
SPEC-Y MAX.	1336.7905	4737.7779	0.00	405315.29	122604.61	-3.48

Footing Layout



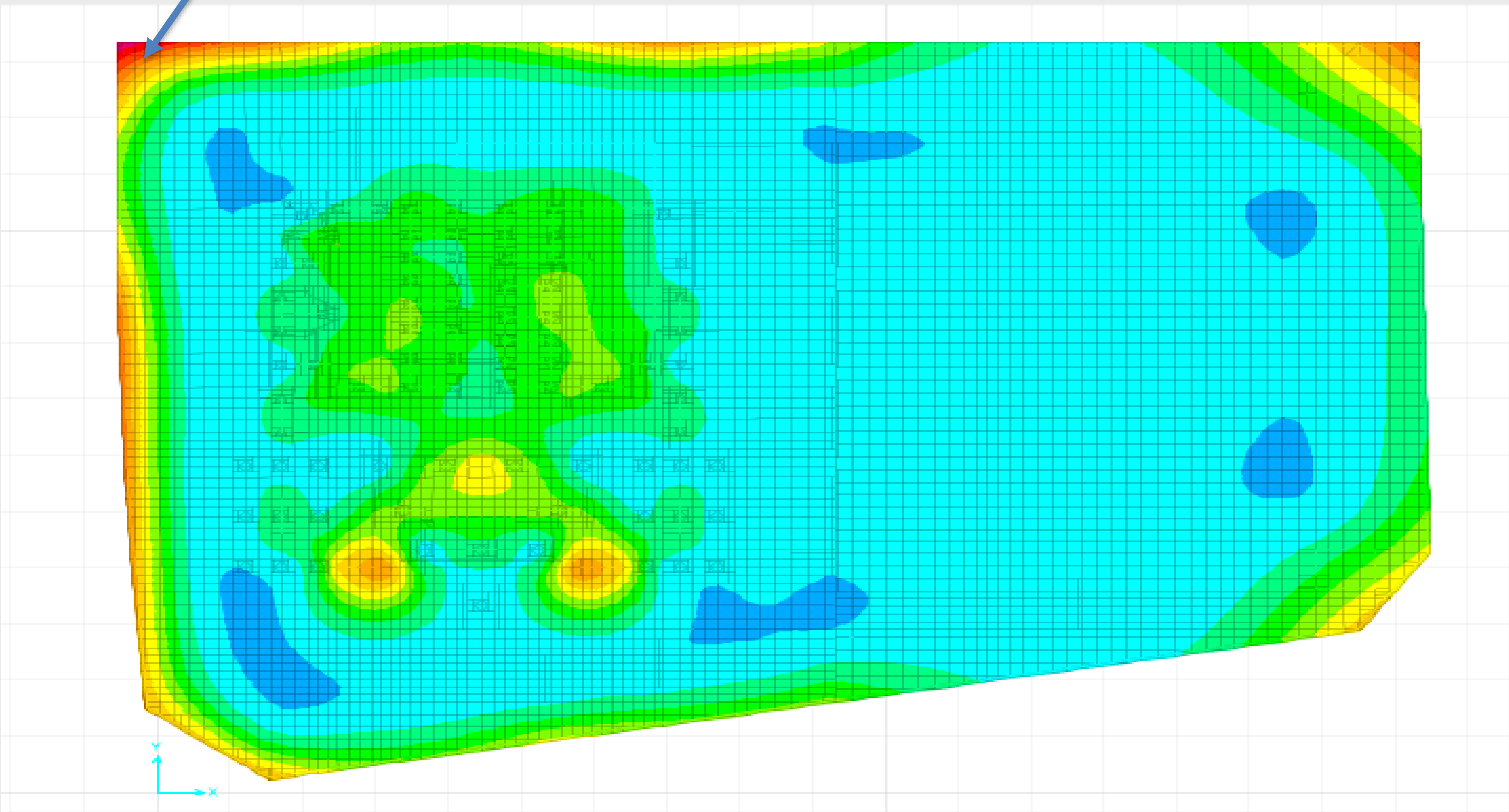
Foundation checks

Settlement

Here maximum settlement for dead load & live load =34.44mm
< allowable settlement=50mm

Hence OK

34.44mm



Foundation checks

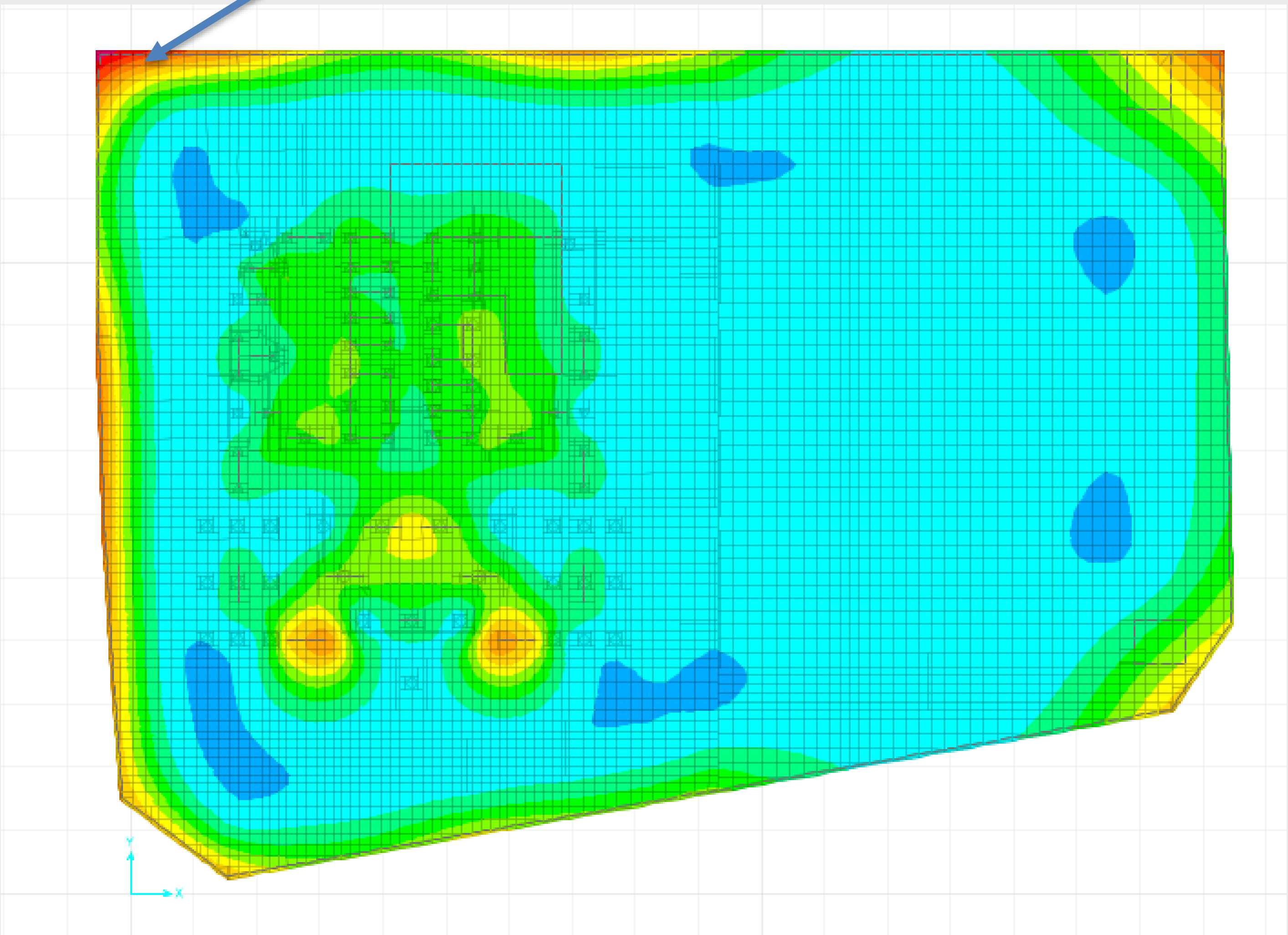
Ground Bearing Pressure

The allowable soil bearing is 300 KN/M² and allowable settlement is 50mm
Subgrade modulus = (Safe bearing capacity of soil / Allowable settlement)
= 300/ 0.050 = 6000 KN/M³

Here maximum GBP = 172.21 KN/M² for dead load & live load < Allowable bearing pressure = 300 KN/M²

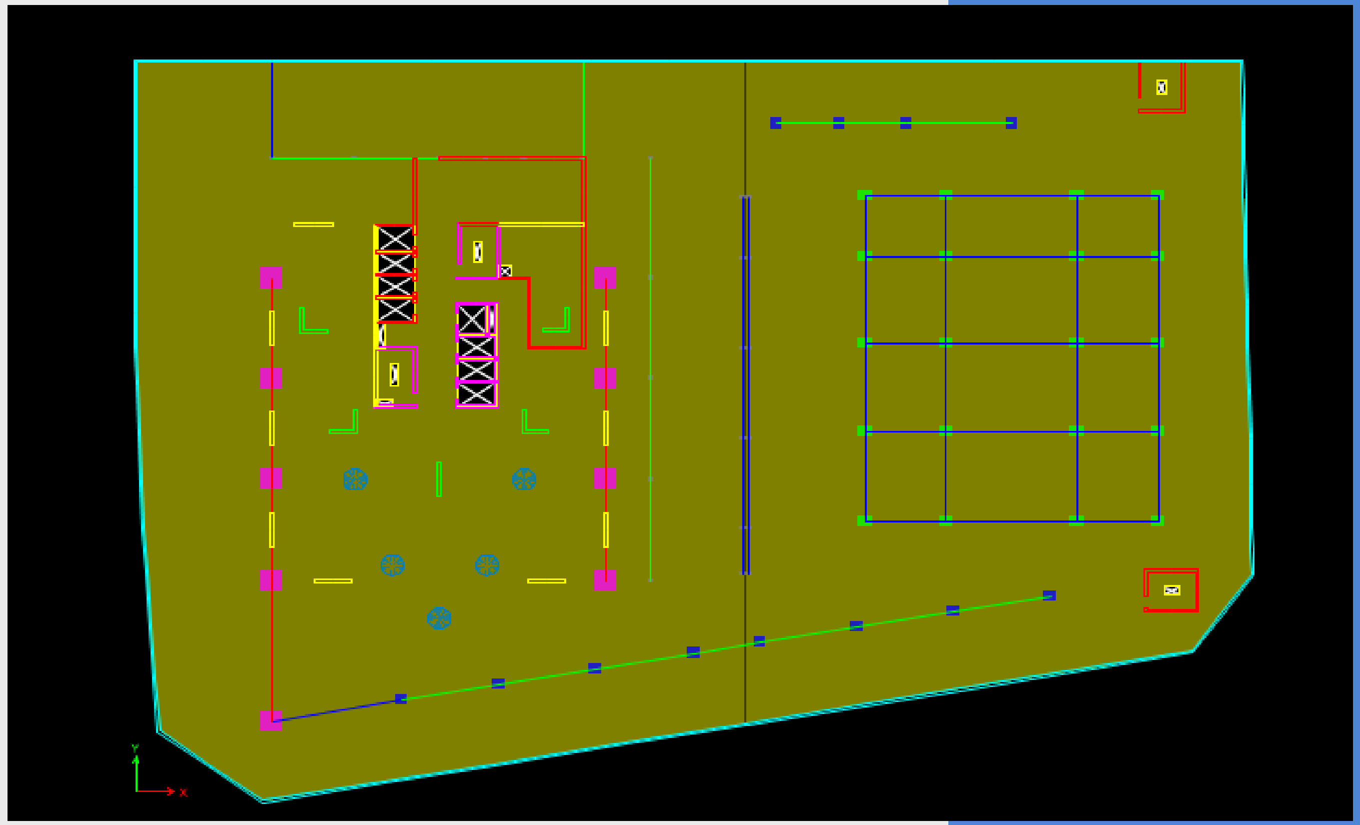
Hence OK

172.21 KN/M²



Foundation checks

Service Load Combination	Strength Load Combination
1DL +1LL	1.5DL + 1.5LL
1 DL +1 SPEC-X	1.5 DL + 1.5 SPECX
1 DL + 1 SPEC-Y	1.5 DL + 1.5 SPECY
1DL ± 1WX	1.5 DL ± 1.5 WX
1DL ± 1WY	1.5 DL ± 1.5 WY
1 DL + 0.8LL + 0.8 SPECX	1.2DL + 1.2 LL + 1.2 SPECX
1 DL + 0.8LL + 0.8 SPECY	1.2DL + 1.2LL + 1.2 SPECY
1DL + 0.8LL ± 0.8 WX	1.2DL + 1.2LL + 1.2 WX
1DL + 0.8LL ± 0.8 WY	1.2DL + 1.2LL + 1.2 WY



Foundation checks

Under the envelope of service load combination
One way shear can be checked d from the face of
Face of column.
Where d is the effective depth of footing.
Permissible shear stress

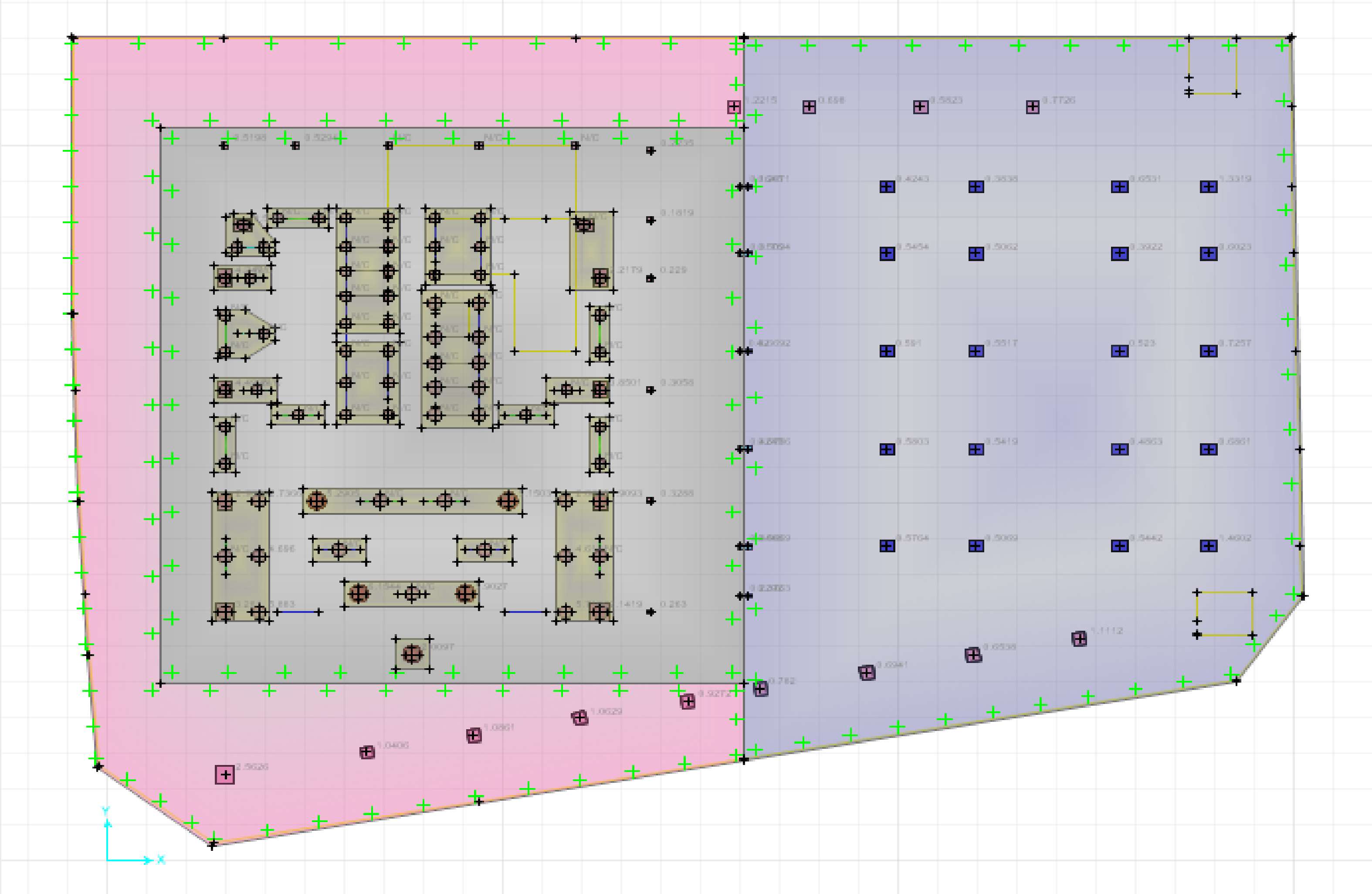
Ks x tc
Ks = (0.5 + βc)
βc = shorter side of col./longer side shear wall
= 0.230/2
= 0.115

Ks = (0.5 + 0.115)
Ks = 0.6

τc = 0.25√fck
= 0.25√25
= 0.25x5
= 1.25

Ks τc = 0.6 X 1.25
= 0.75 N/mm2 stresses value

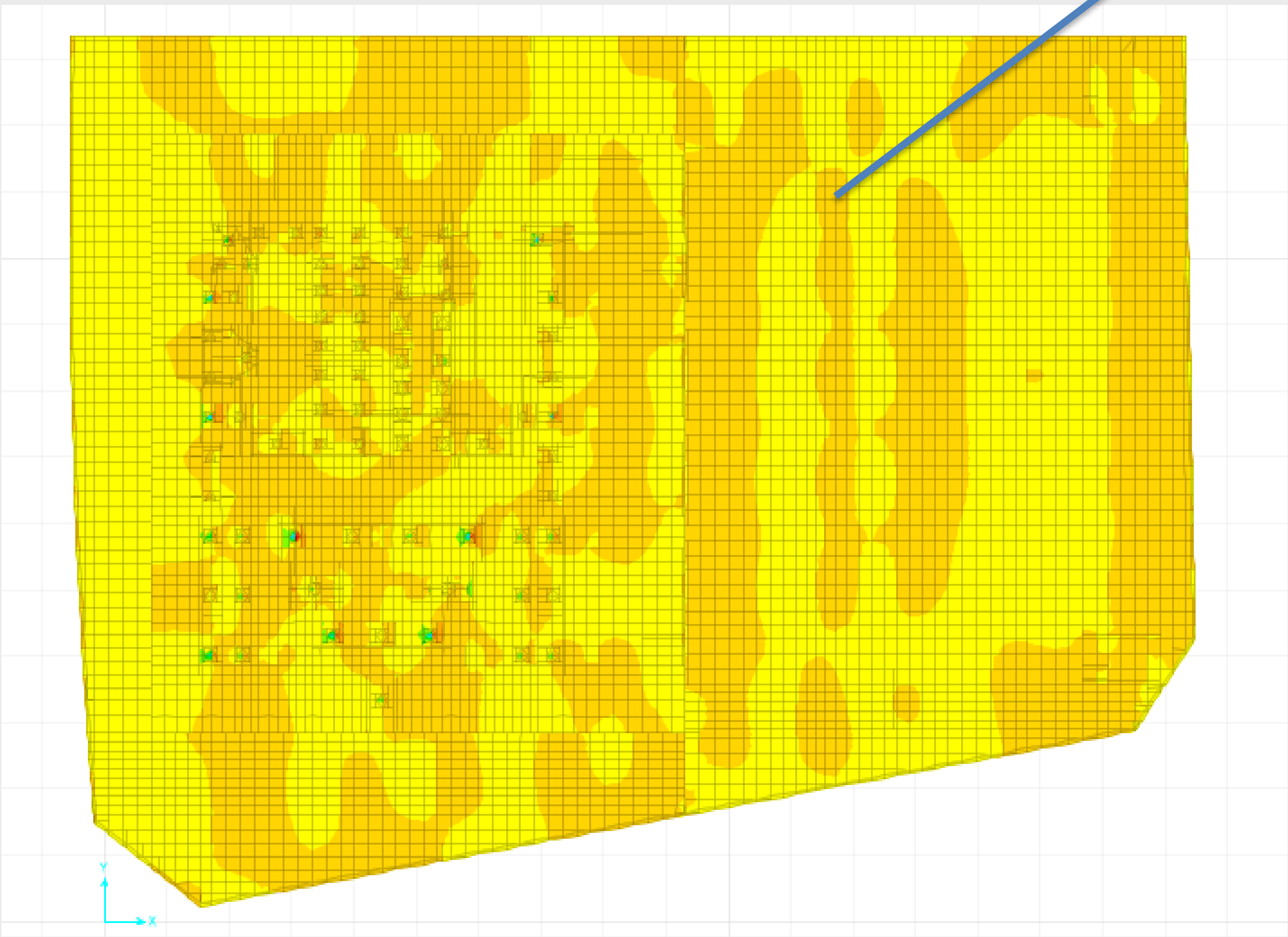
Punching Shear



Foundation checks

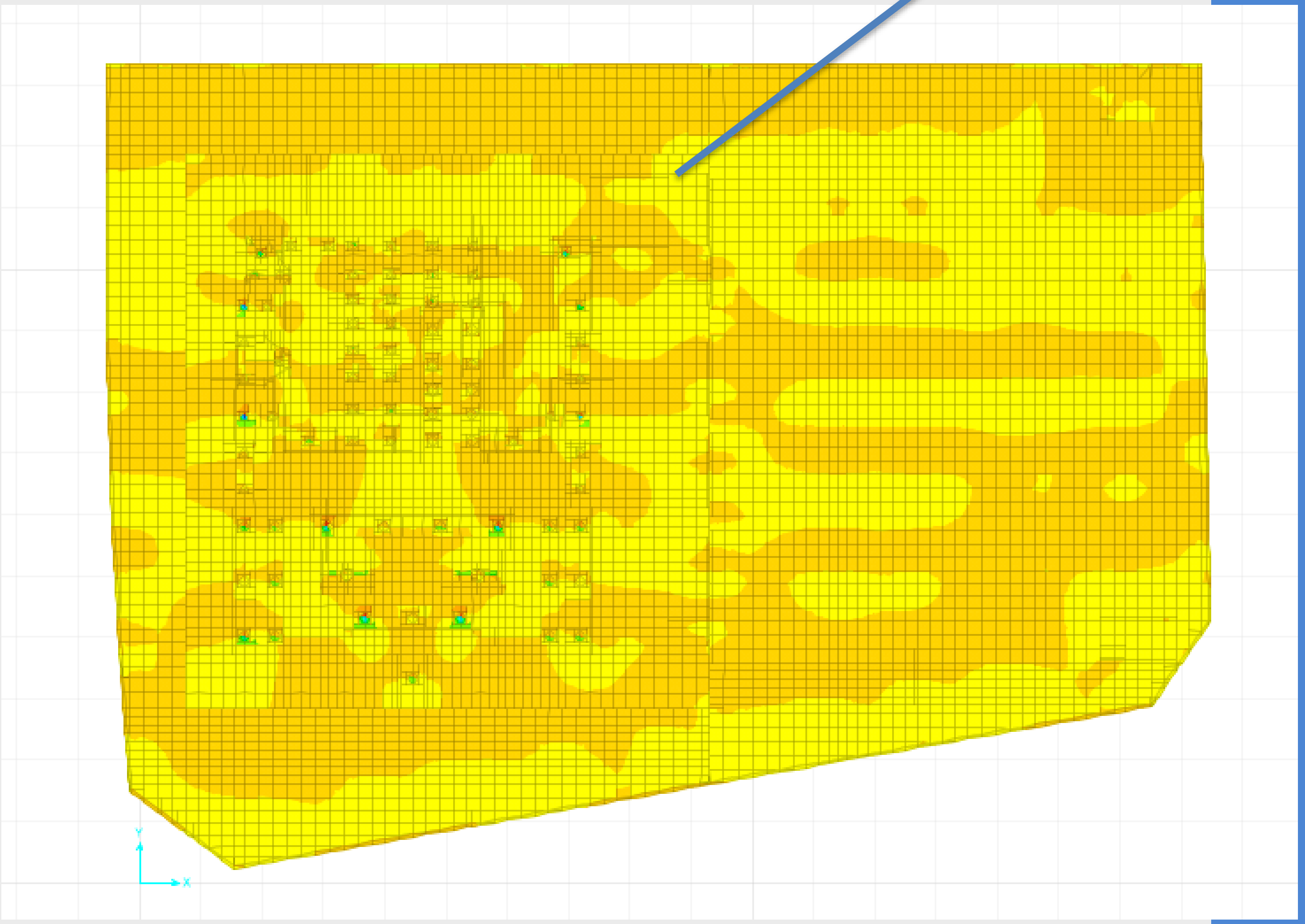
STRESS ALONG S13

0.52N/m2



STRESS ALONG S23

0.64N/m2



Max stress is 0.52N/mm2 & 0.64N/mm2 which is less than 0.75 N/mm2

Hence OK

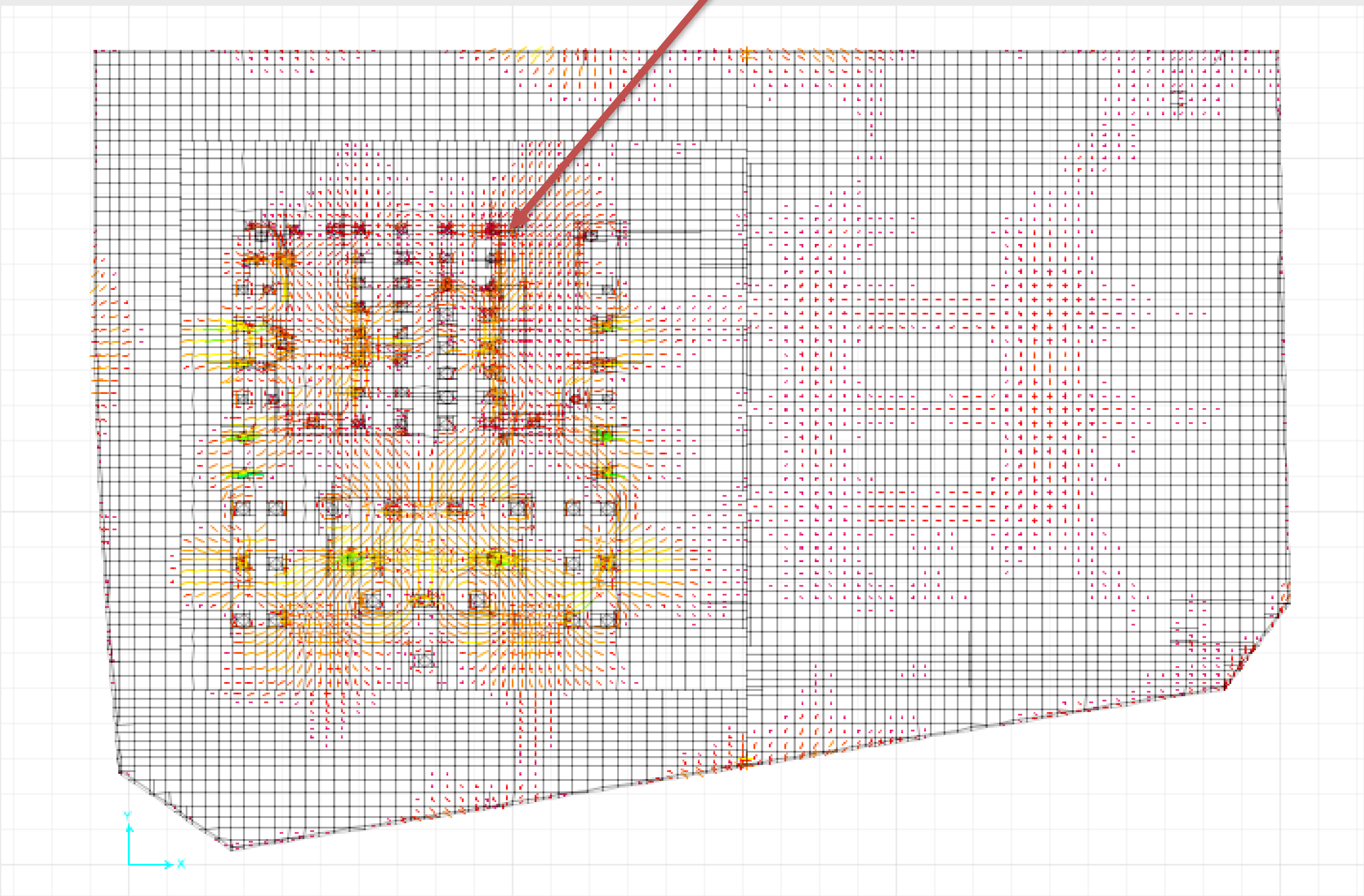
Foundation checks

Crack width Check

Moment in service load envelope and area of steel is used to calculate crack width

The maximum value of crack width is 0.0125 < 0.2mm
For moderate exposure conditions

0.0125 mm



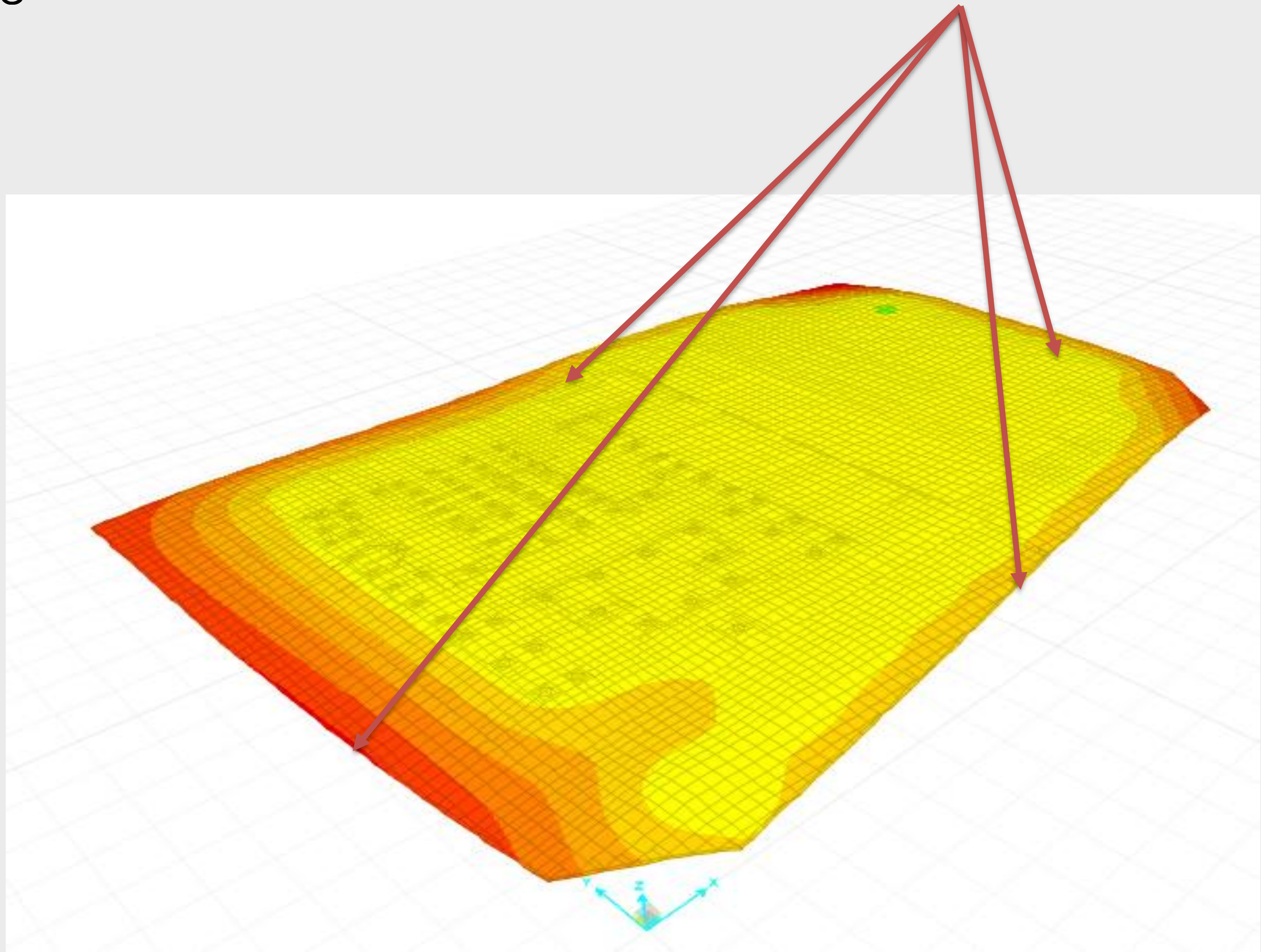
Permissible crack width = 0.2 mm , Hence it is Safe

Foundation checks

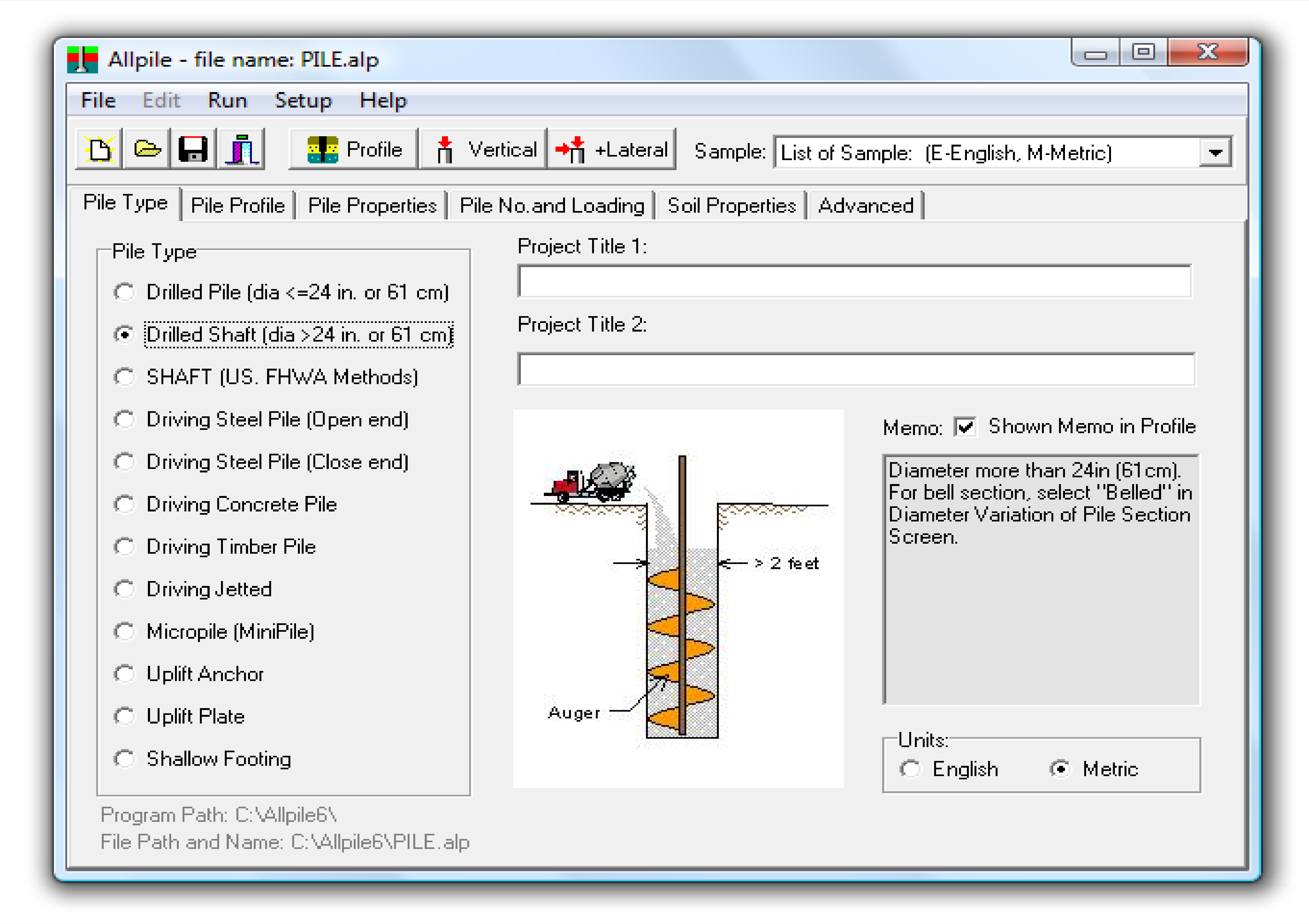
Uplift by non linear analysis and check overturning

No one portion coming in tension

Hence OK

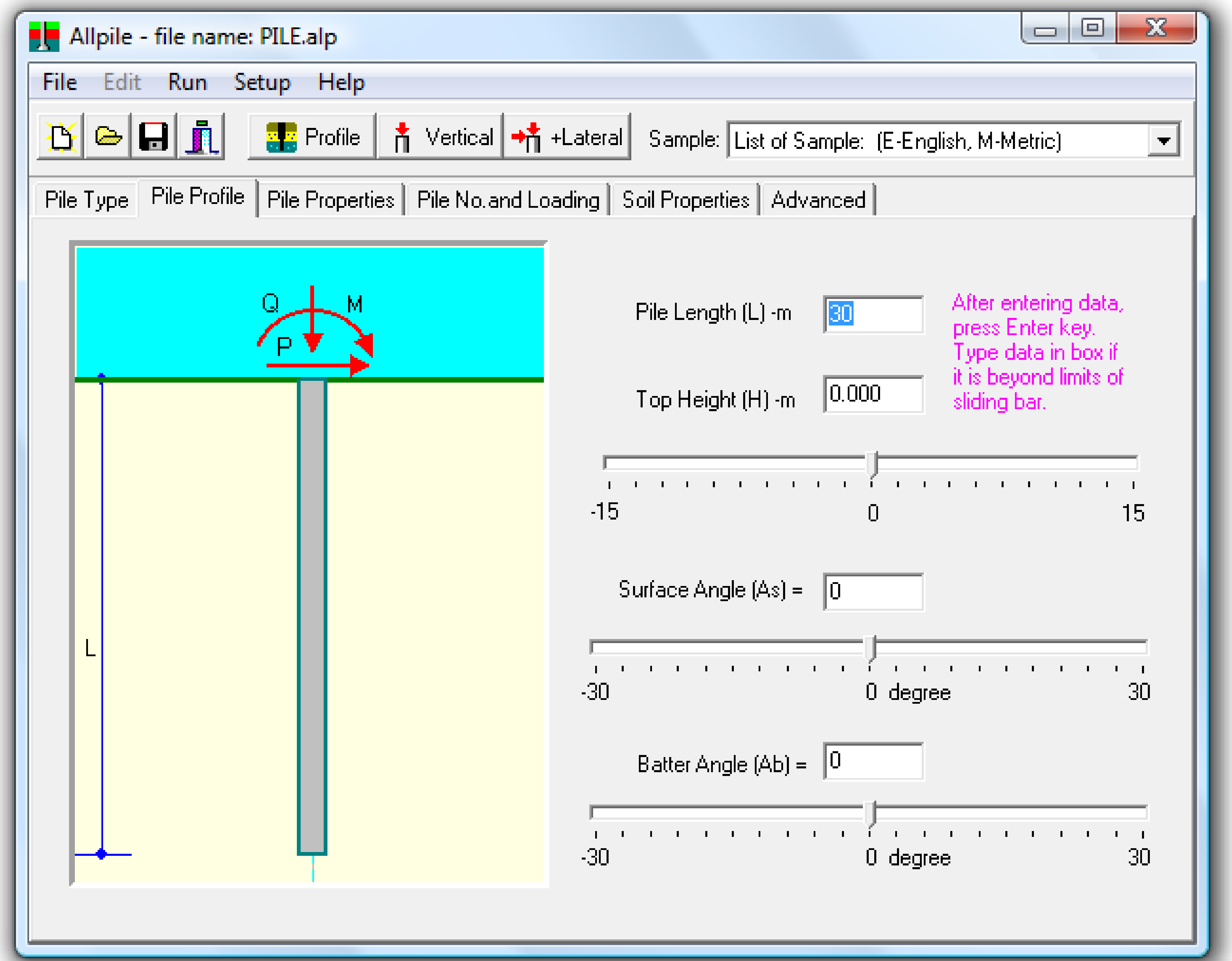


All Pile

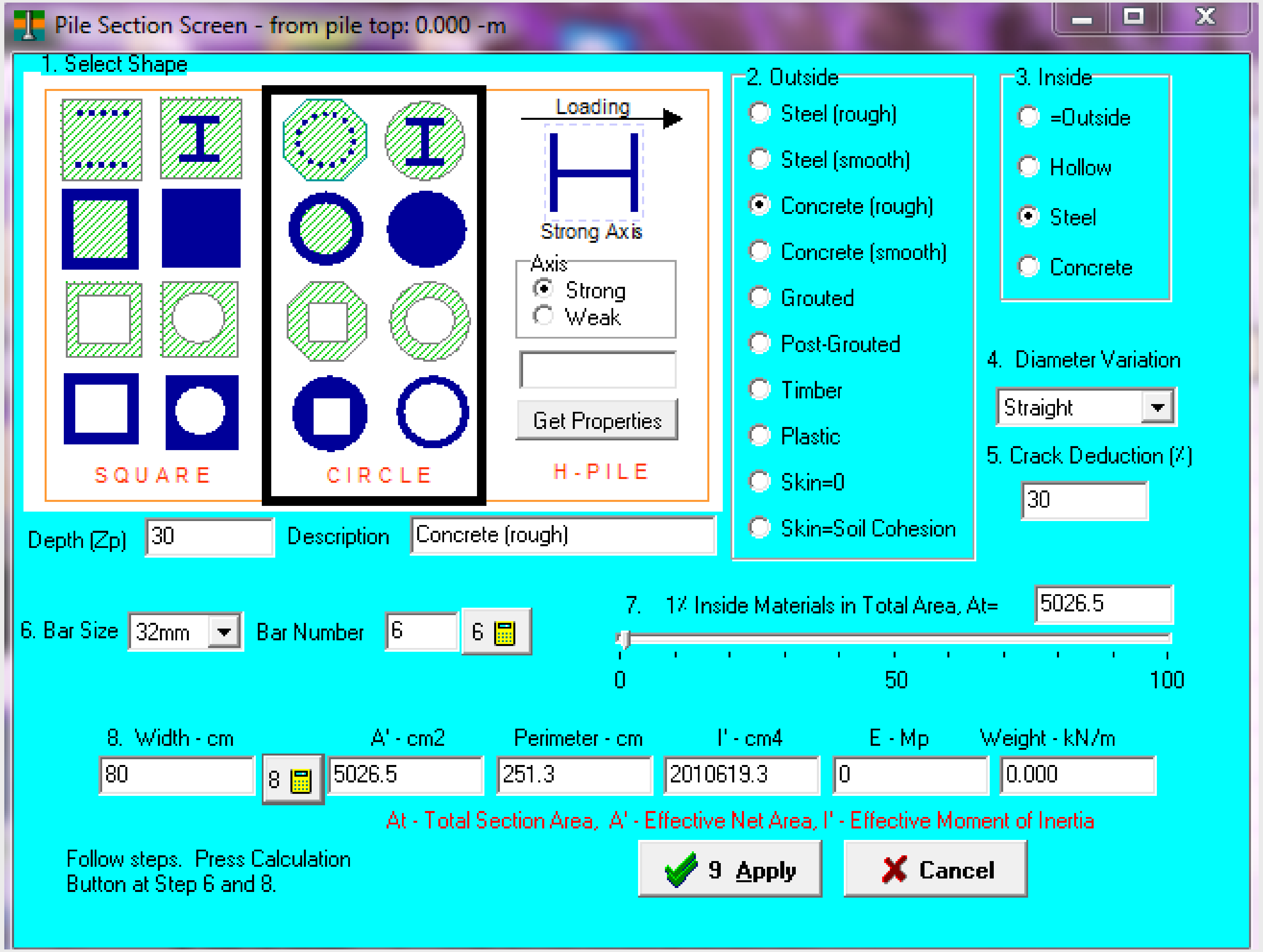


Allpile - file name: PILE.alp
File
Edit
Run
Setup
Help
Profile
Vertical
+Lateral
Sample:
List of Sample: (E-English, M-Metric)
Pile Type
Pile Profile
Pile Properties
Pile No.and Loading
Soil Properties
Advanced
Pile Type
Drilled Pile (dia <=24 in. or 61 cm)
Drilled Shaft (dia >24 in. or 61 cm)
SHAFT (US. FHWA Methods)
Driving Steel Pile (Open end)
Driving Steel Pile (Close end)
Driving Concrete Pile
Driving Timber Pile
Driving Jetted
Micropile (MiniPile)
Uplift Anchor
Uplift Plate
Shallow Footing
Project Title 1:
Project Title 2:
> 2 feet
Auger
Memo:
Shown Memo in Profile
Diameter more than 24in (61cm). For bell section, select "Belled" in Diameter Variation of Pile Section Screen.
Units:
English
Metric
Program Path: C:\Allpile6\
File Path and Name: C:\Allpile6\PILE.alp

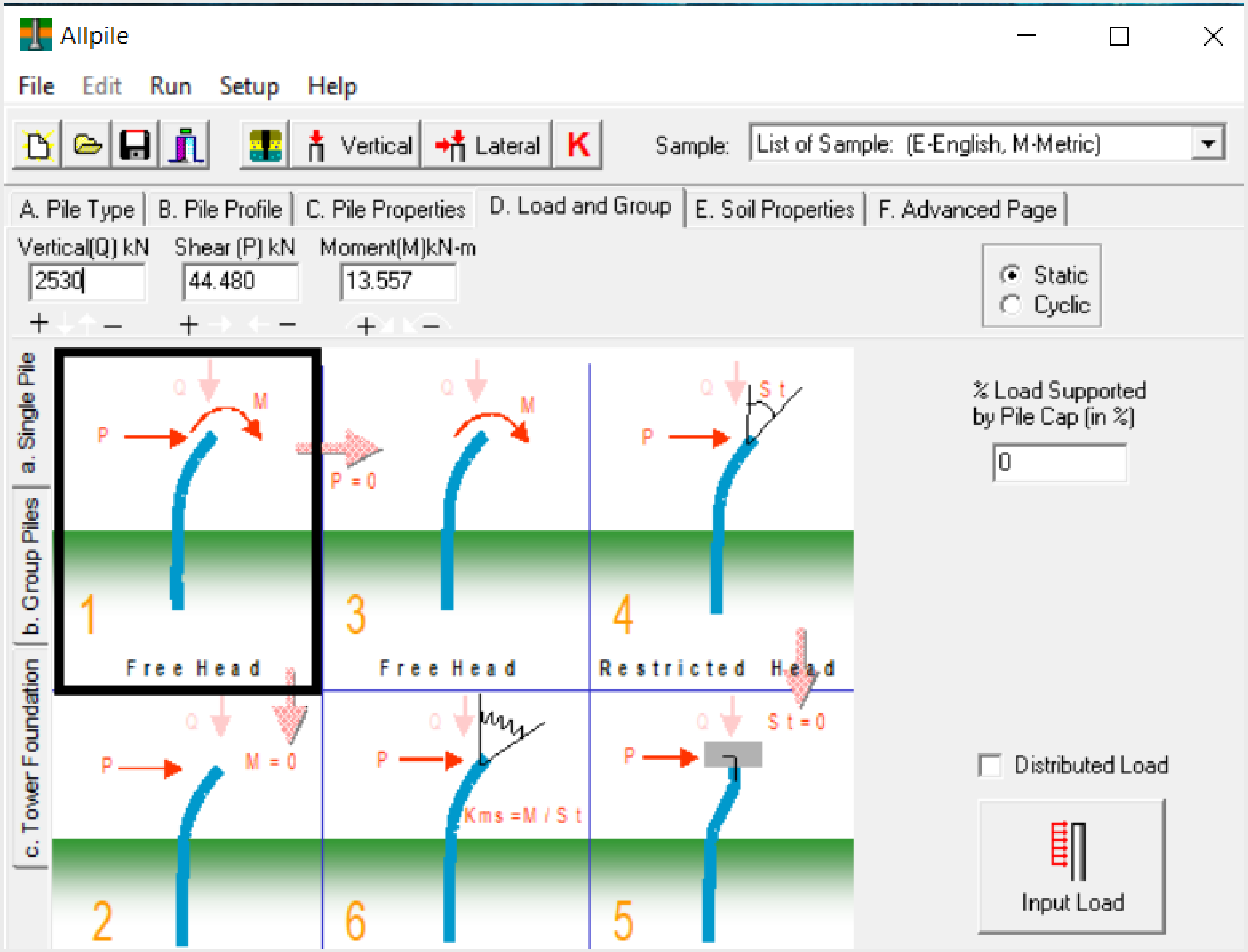
All Pile


Allpile - file name: PILE.alp
File Edit Run Setup Help
Profile
Vertical
+Lateral
Sample: List of Sample: (E-English, M-Metric)
Pile Type
Pile Profile
Pile Properties
Pile No.and Loading
Soil Properties
Advanced
Q
M
P
L
Pile Length (L) -m
30
Top Height (H) -m
0.000
After entering data, press Enter key. Type data in box if it is beyond limits of sliding bar.
-15
0
15
Surface Angle (As) =
0
-30
0 degree
30
Batter Angle (Ab) =
0
-30
0 degree
30

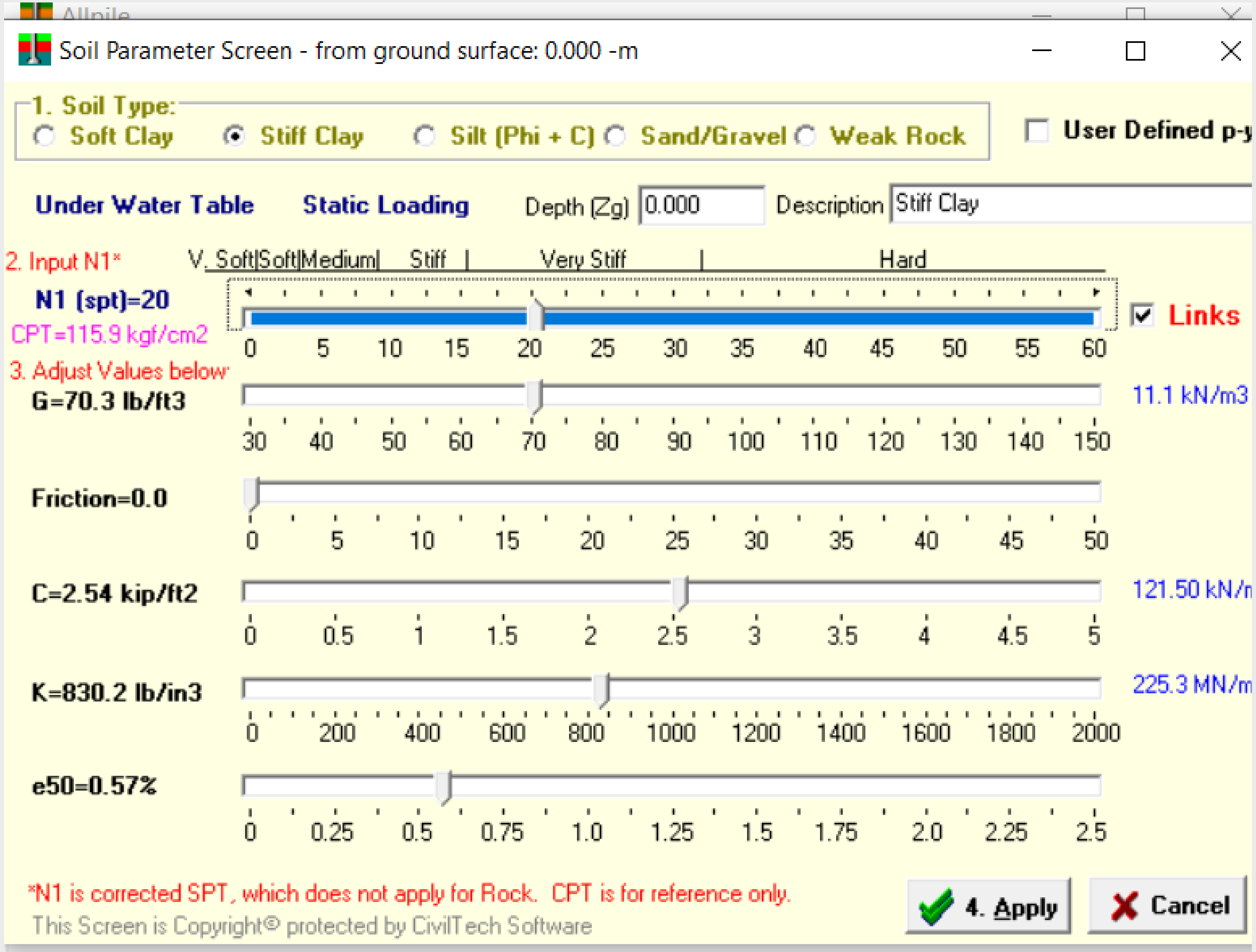
Pile Properties



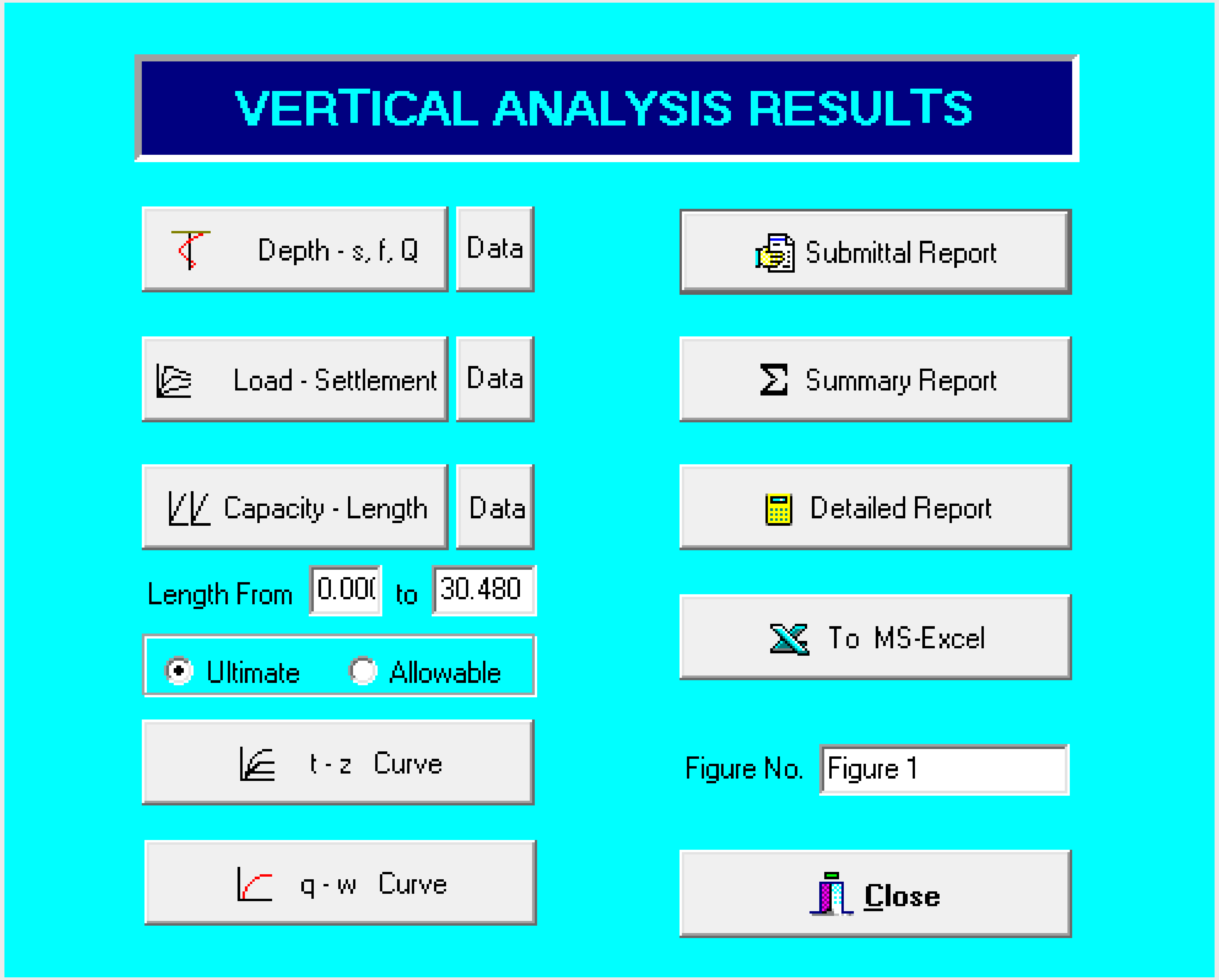
Pile Loading



Soil Properties



Summary output



CSI Detailer

STRUCTURAL DETAILING AND DRAWING GENERATION

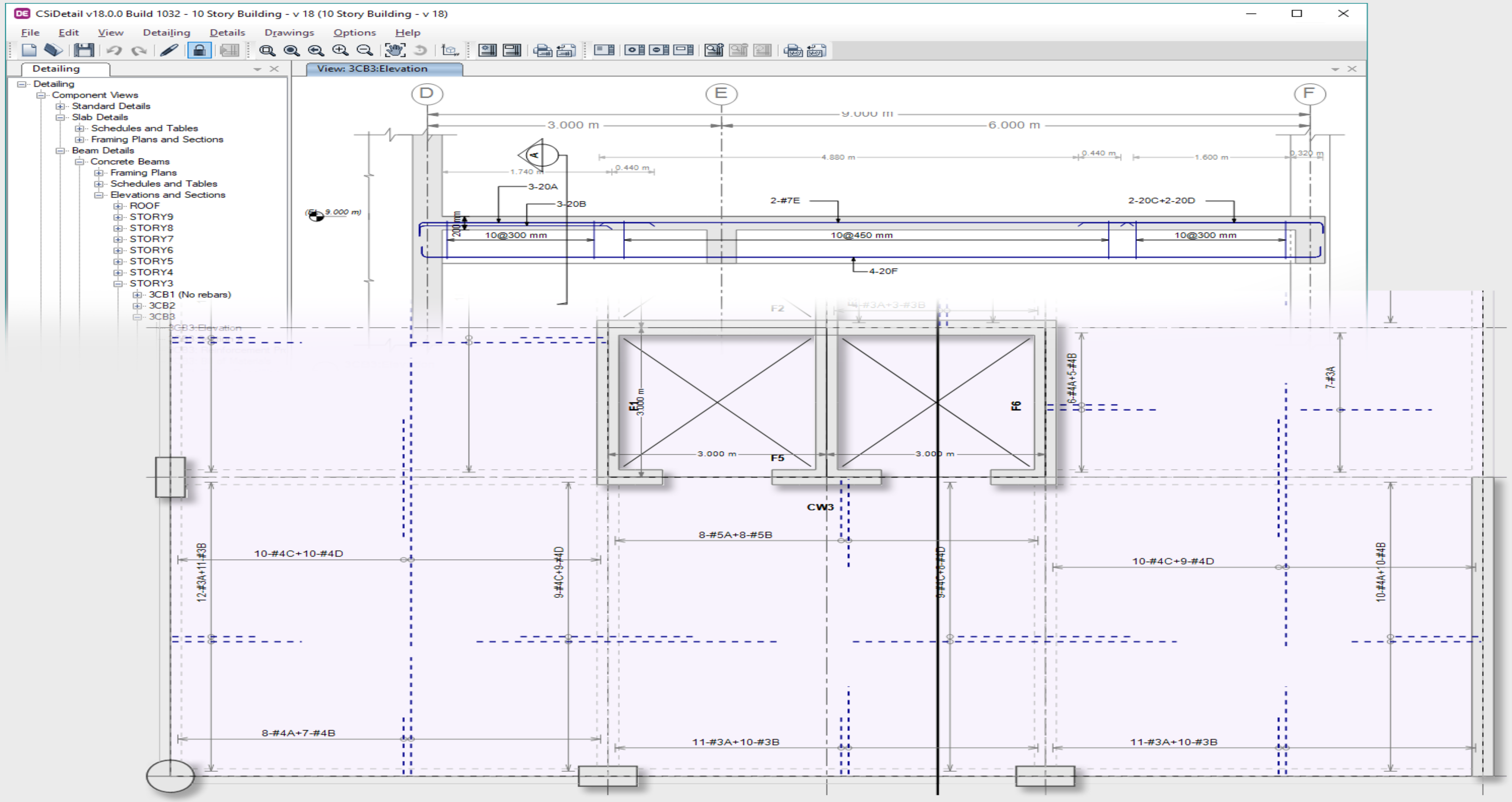
Overview

CSI Detail is an integrated and interactive software product for generating detailing output, such as detailed views, drawings, bills of materials (BOM), and bills of quantities (BOQ) from ETABS models.

CSI Detail creates a detailed model of the structure, based on its analysis model and the design process carried out in ETABS. It automatically generates views and drawings that can be used as the basis for preparing final engineering drawings, as well as BIM files for use in BIM tools. Detailing can be carried out both for concrete and steel buildings.

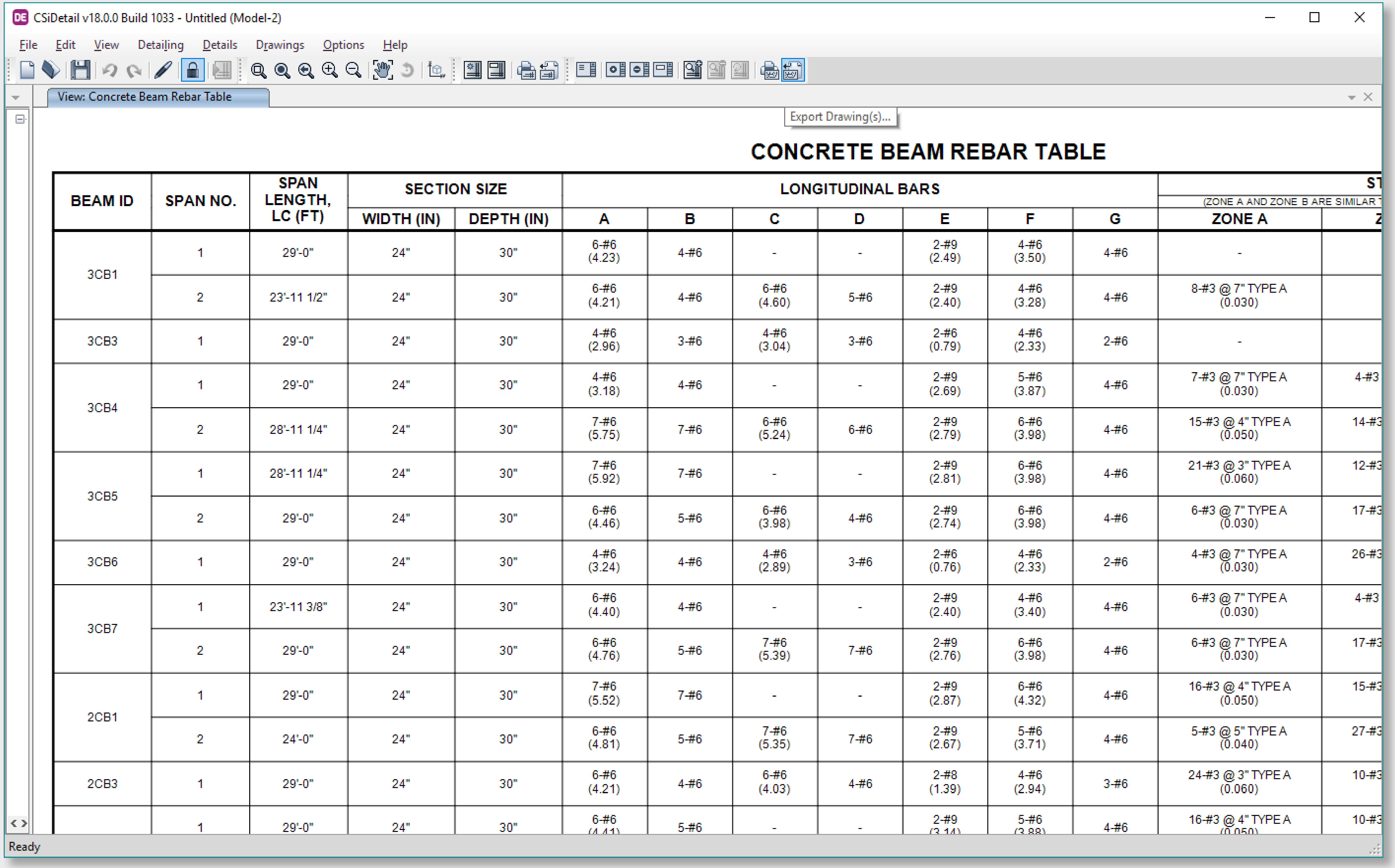
REINFORCEMENT DETAILS

CSI Detail generates reinforcement details for slabs, beams, columns, and walls, grouping similar beams and columns in the process.



REBAR SCHEDULES

Reinforcement details are presented in the form of tables and schedules for groups of components, as well as drawn in plans, elevations, and sections for individual elements.

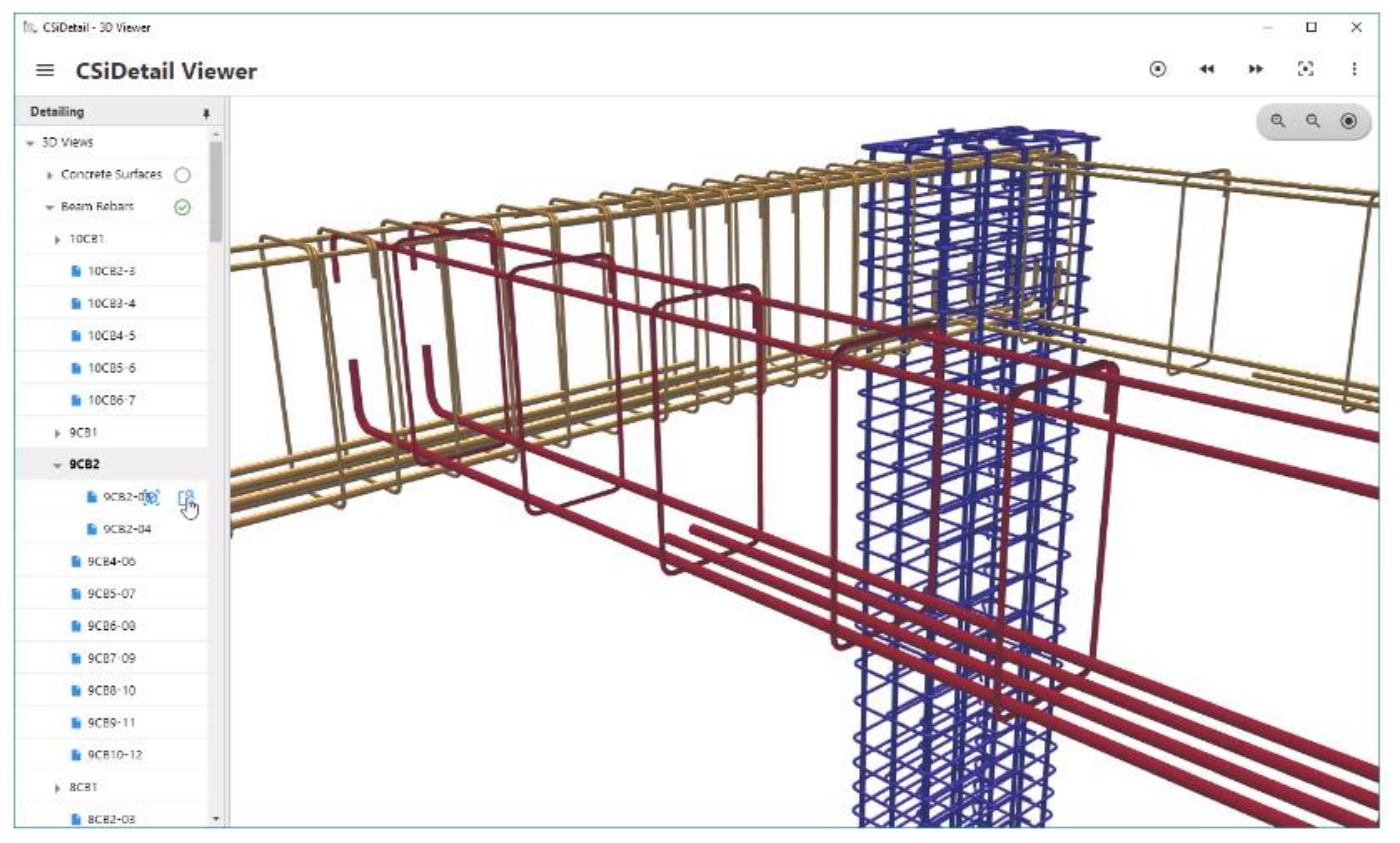


CONCRETE BEAM REBAR TABLE

BEAM ID	SPAN NO.	SPAN LENGTH, LC (FT)	SECTION SIZE		LONGITUDINAL BARS							ST… (ZONE A AND ZONE B ARE SIMILAR …	
			WIDTH (IN)	DEPTH (IN)	A	B	C	D	E	F	G	ZONE A	Z…
3CB1	1	29'-0"	24"	30"	6-#6 (4.23)	4-#6	-	-	2-#9 (2.49)	4-#6 (3.50)	4-#6	-	
	2	23'-11 1/2"	24"	30"	6-#6 (4.21)	4-#6	6-#6 (4.60)	5-#6	2-#9 (2.40)	4-#6 (3.28)	4-#6	8-#3 @ 7" TYPE A (0.030)	
3CB3	1	29'-0"	24"	30"	4-#6 (2.96)	3-#6	4-#6 (3.04)	3-#6	2-#6 (0.79)	4-#6 (2.33)	2-#6	-	
3CB4	1	29'-0"	24"	30"	4-#6 (3.18)	4-#6	-	-	2-#9 (2.69)	5-#6 (3.87)	4-#6	7-#3 @ 7" TYPE A (0.030)	4-#3
	2	28'-11 1/4"	24"	30"	7-#6 (5.75)	7-#6	6-#6 (5.24)	6-#6	2-#9 (2.79)	6-#6 (3.98)	4-#6	15-#3 @ 4" TYPE A (0.050)	14-#3
3CB5	1	28'-11 1/4"	24"	30"	7-#6 (5.92)	7-#6	-	-	2-#9 (2.81)	6-#6 (3.98)	4-#6	21-#3 @ 3" TYPE A (0.060)	12-#3
	2	29'-0"	24"	30"	6-#6 (4.46)	5-#6	6-#6 (3.98)	4-#6	2-#9 (2.74)	6-#6 (3.98)	4-#6	6-#3 @ 7" TYPE A (0.030)	17-#3
3CB6	1	29'-0"	24"	30"	4-#6 (3.24)	4-#6	4-#6 (2.89)	3-#6	2-#6 (0.76)	4-#6 (2.33)	2-#6	4-#3 @ 7" TYPE A (0.030)	26-#3
3CB7	1	23'-11 3/8"	24"	30"	6-#6 (4.40)	4-#6	-	-	2-#9 (2.40)	4-#6 (3.40)	4-#6	6-#3 @ 7" TYPE A (0.030)	4-#3
	2	29'-0"	24"	30"	6-#6 (4.76)	5-#6	7-#6 (5.39)	7-#6	2-#9 (2.76)	6-#6 (3.98)	4-#6	6-#3 @ 7" TYPE A (0.030)	17-#3
2CB1	1	29'-0"	24"	30"	7-#6 (5.52)	7-#6	-	-	2-#9 (2.87)	6-#6 (4.32)	4-#6	16-#3 @ 4" TYPE A (0.050)	15-#3
	2	24'-0"	24"	30"	6-#6 (4.81)	5-#6	7-#6 (5.35)	7-#6	2-#9 (2.67)	5-#6 (3.71)	4-#6	5-#3 @ 5" TYPE A (0.040)	27-#3
2CB3	1	29'-0"	24"	30"	6-#6 (4.21)	4-#6	6-#6 (4.03)	4-#6	2-#8 (1.39)	4-#6 (2.94)	3-#6	24-#3 @ 3" TYPE A (0.060)	10-#3
	1	29'-0"	24"	30"	6-#6 (4.41)	5-#6	-	-	2-#9 (3.14)	5-#6 (3.88)	4-#6	16-#3 @ 4" TYPE A (0.050)	10-#3

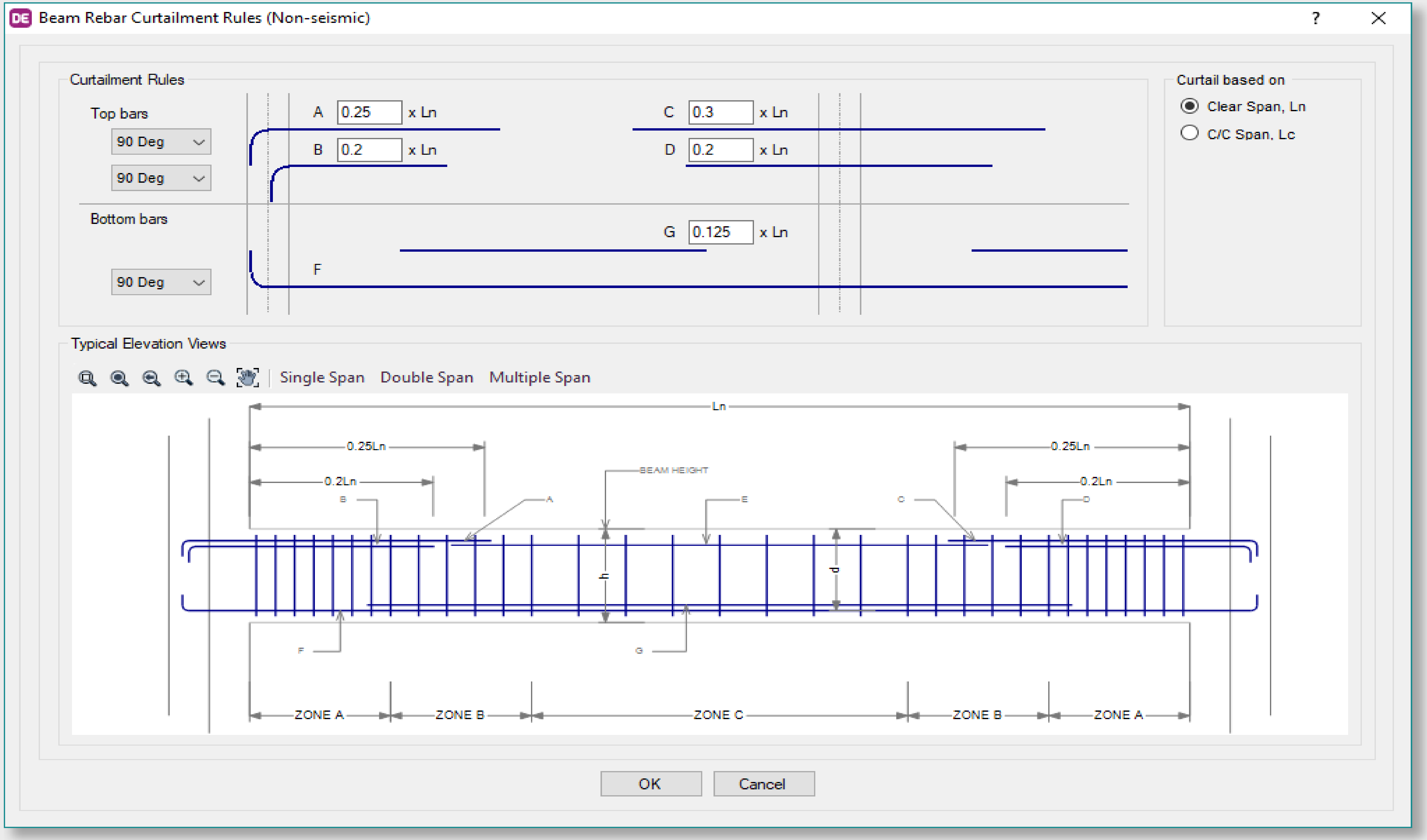
REBAR CAGES

The rebar cage of individual components or the entire structure can be viewed in an interactive 3D model, showing individual rebars in true 3D coordinates.



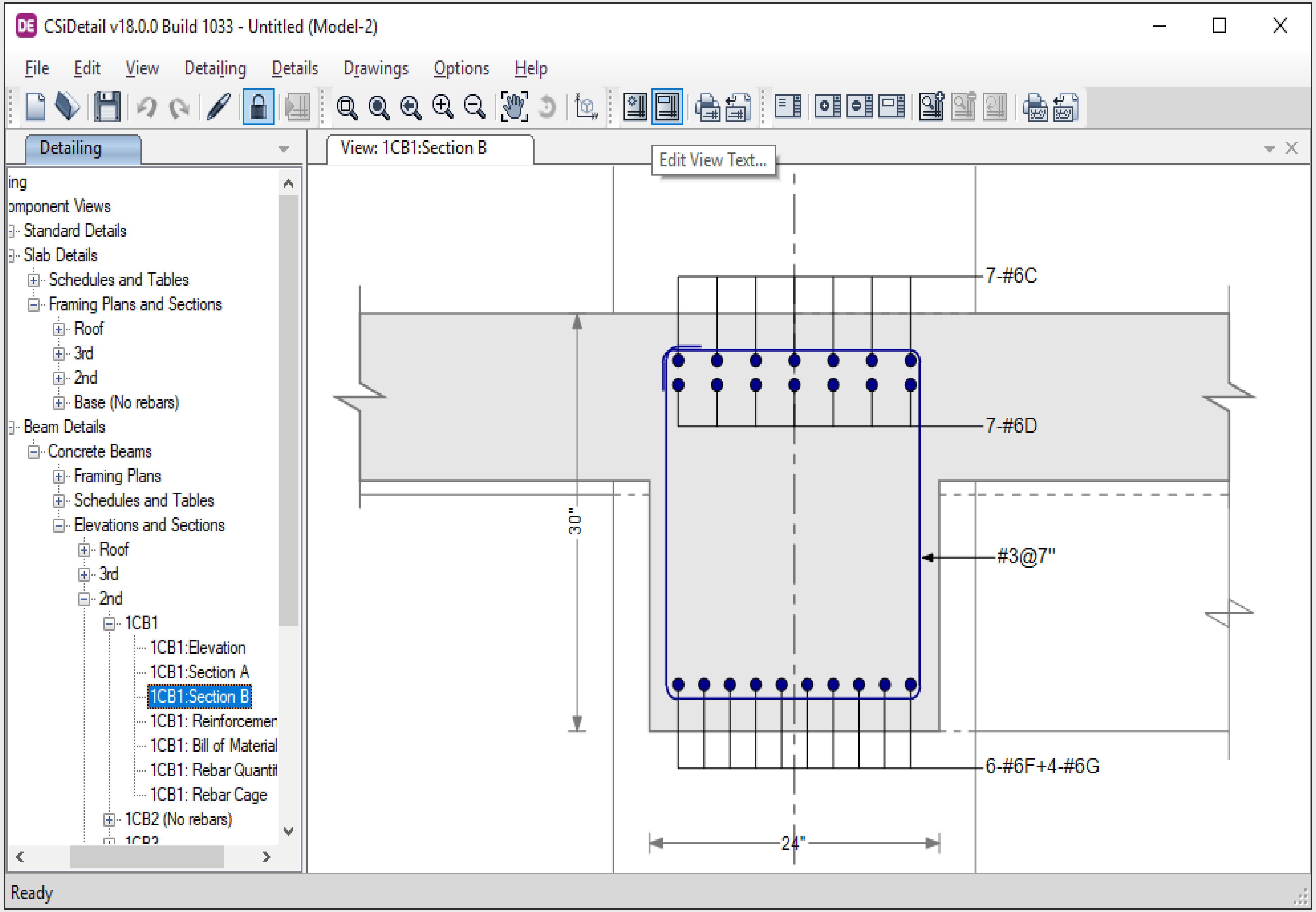
DETAILING RULES

Detailing is carried out based on an extensive set of preferences and rules to control bar-size selection, spacing, curtailment and placement.

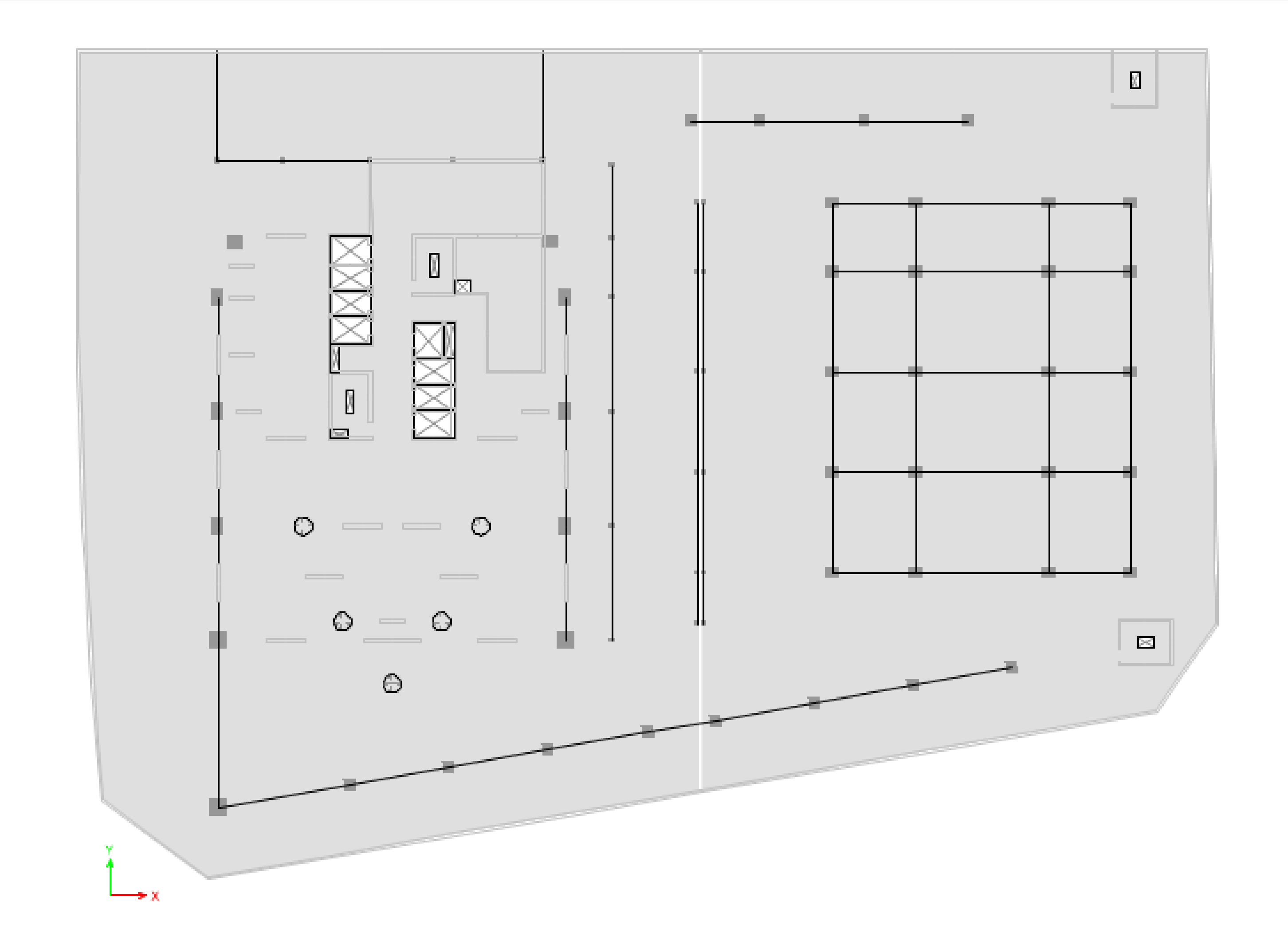


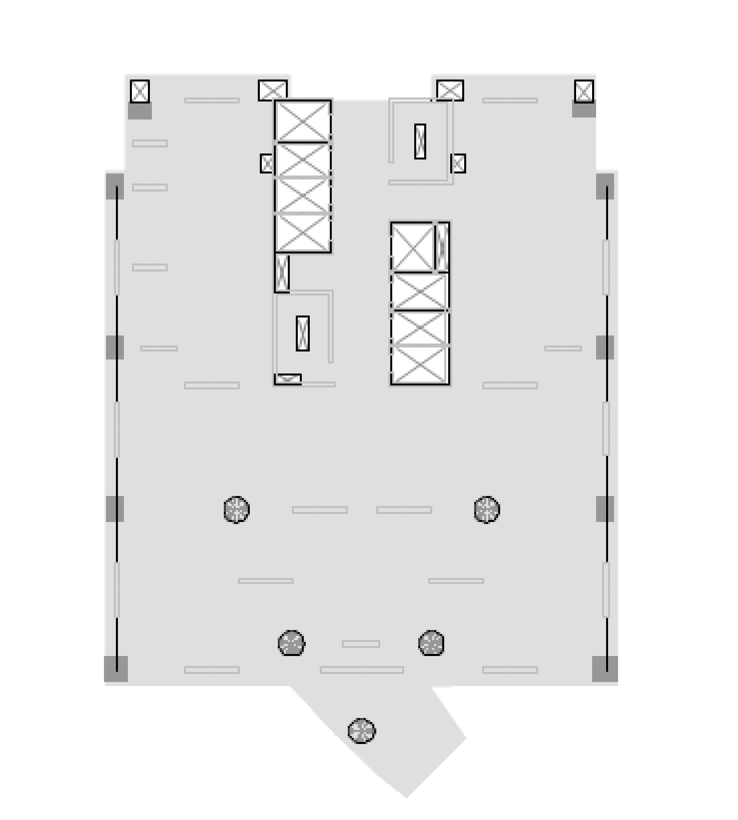
EDITING CAPABILITIES

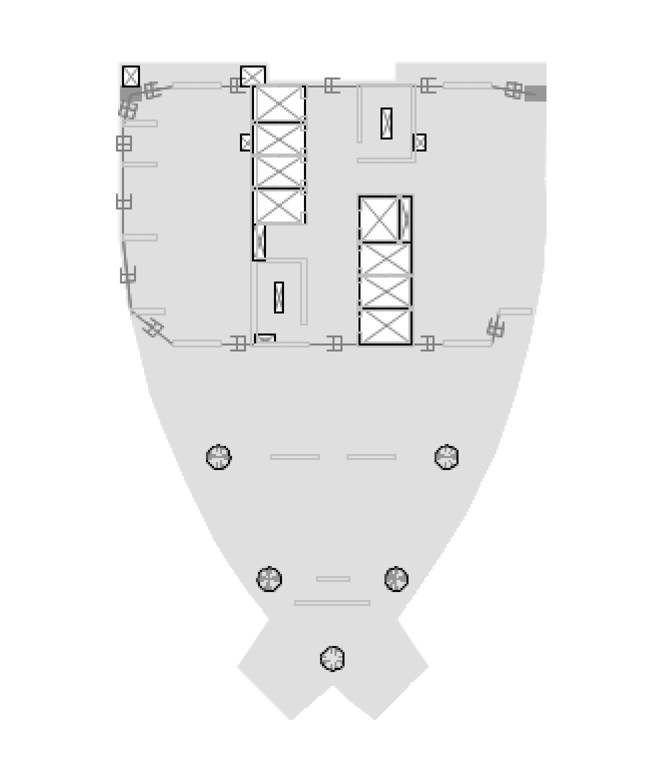
CSI Detail has extensive capabilities for editing reinforcement.
Generated views can be edited or formatted as needed.

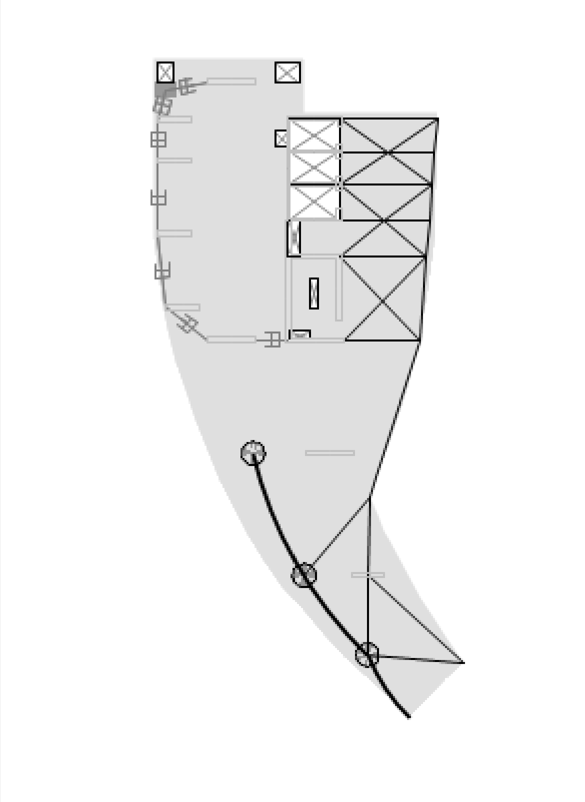


Erection Drawings

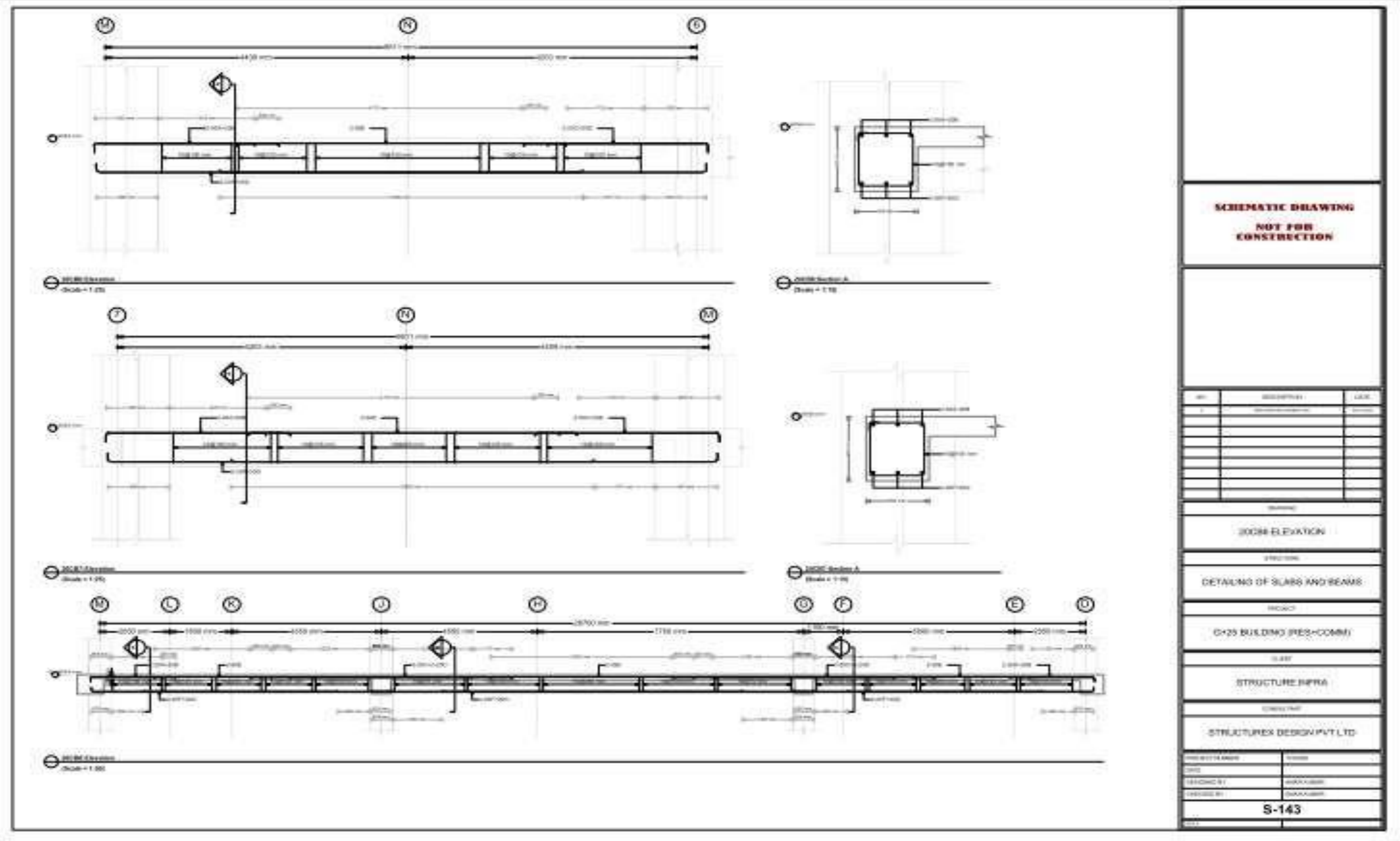




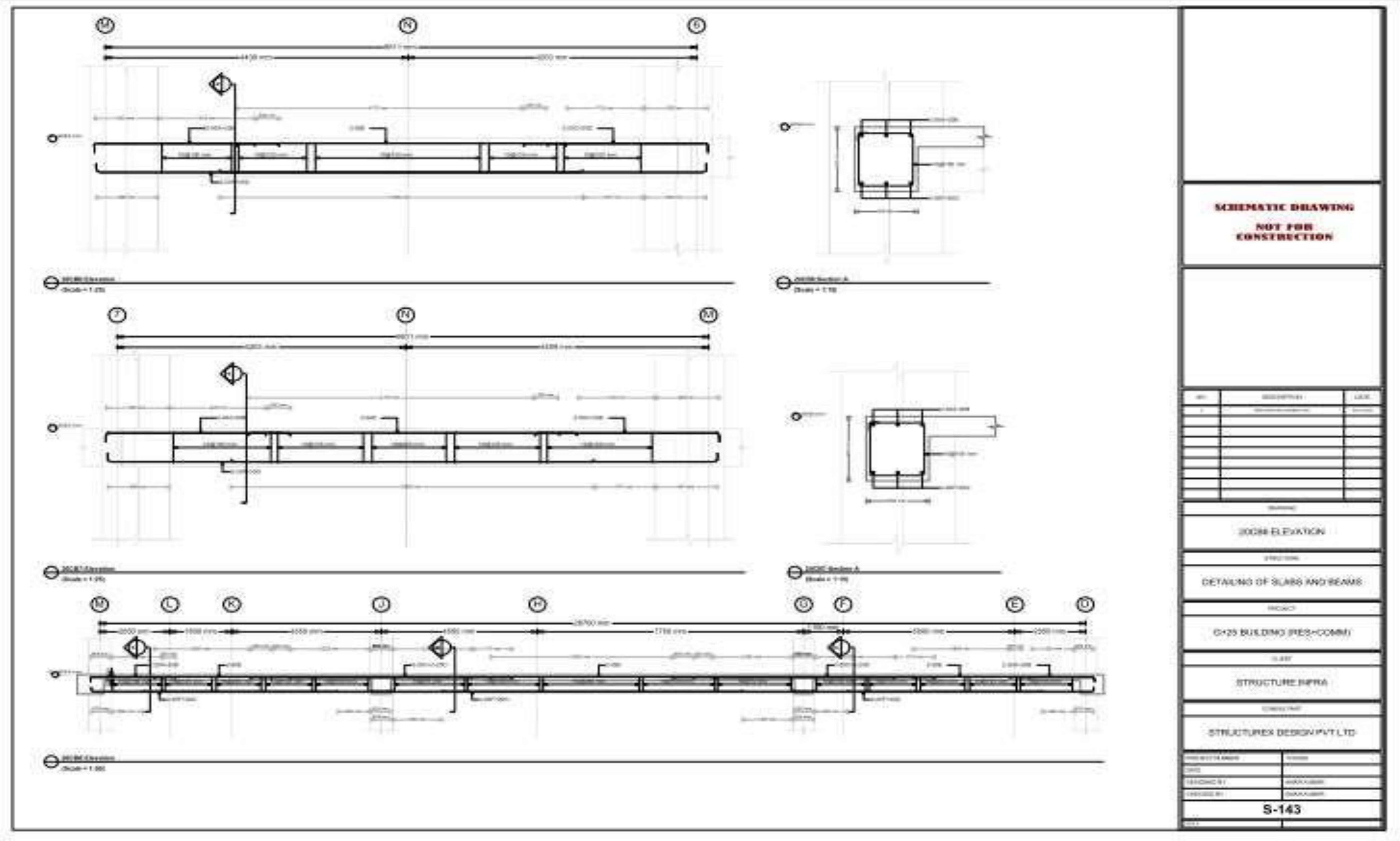




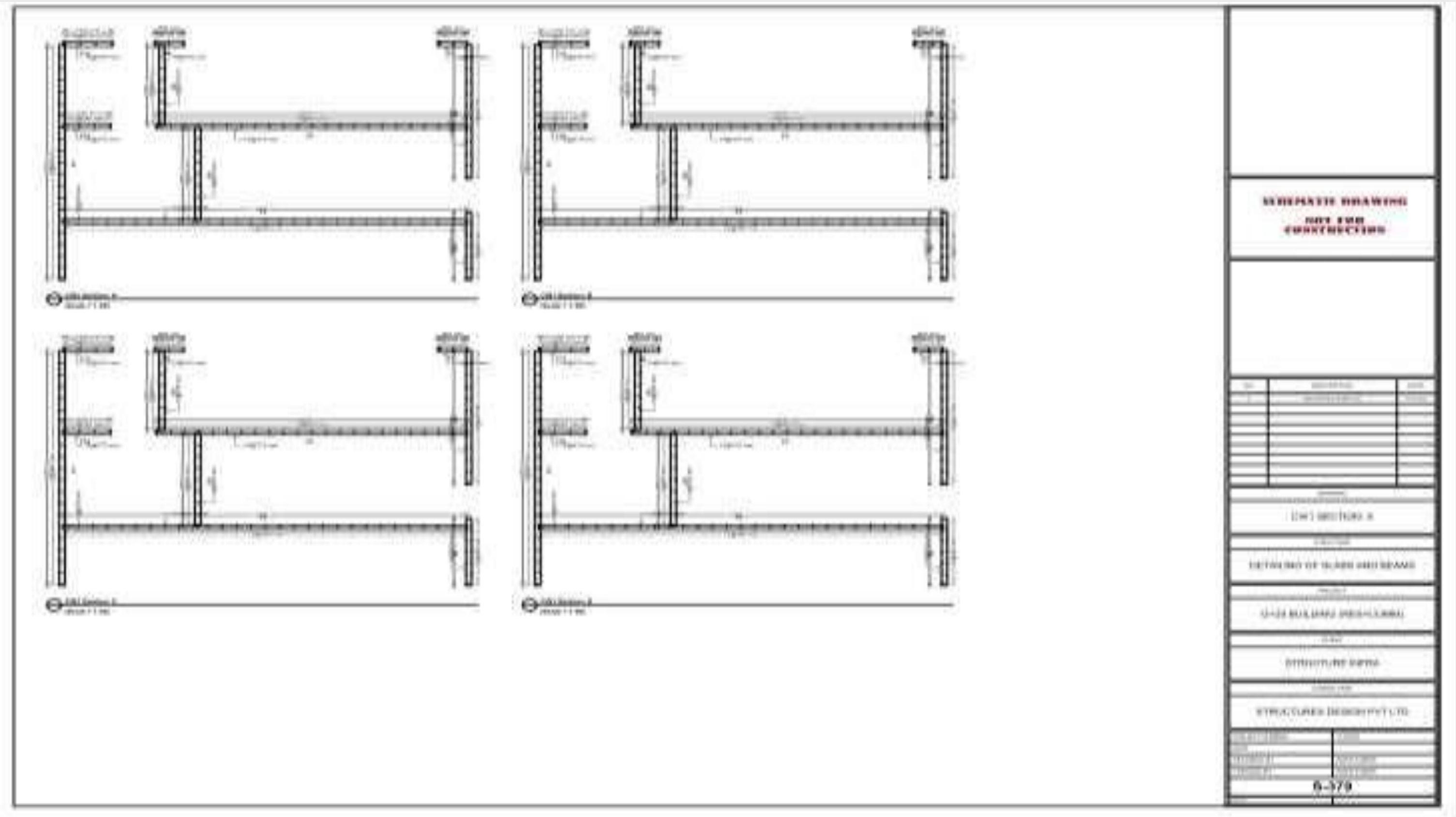
Beam Detailing

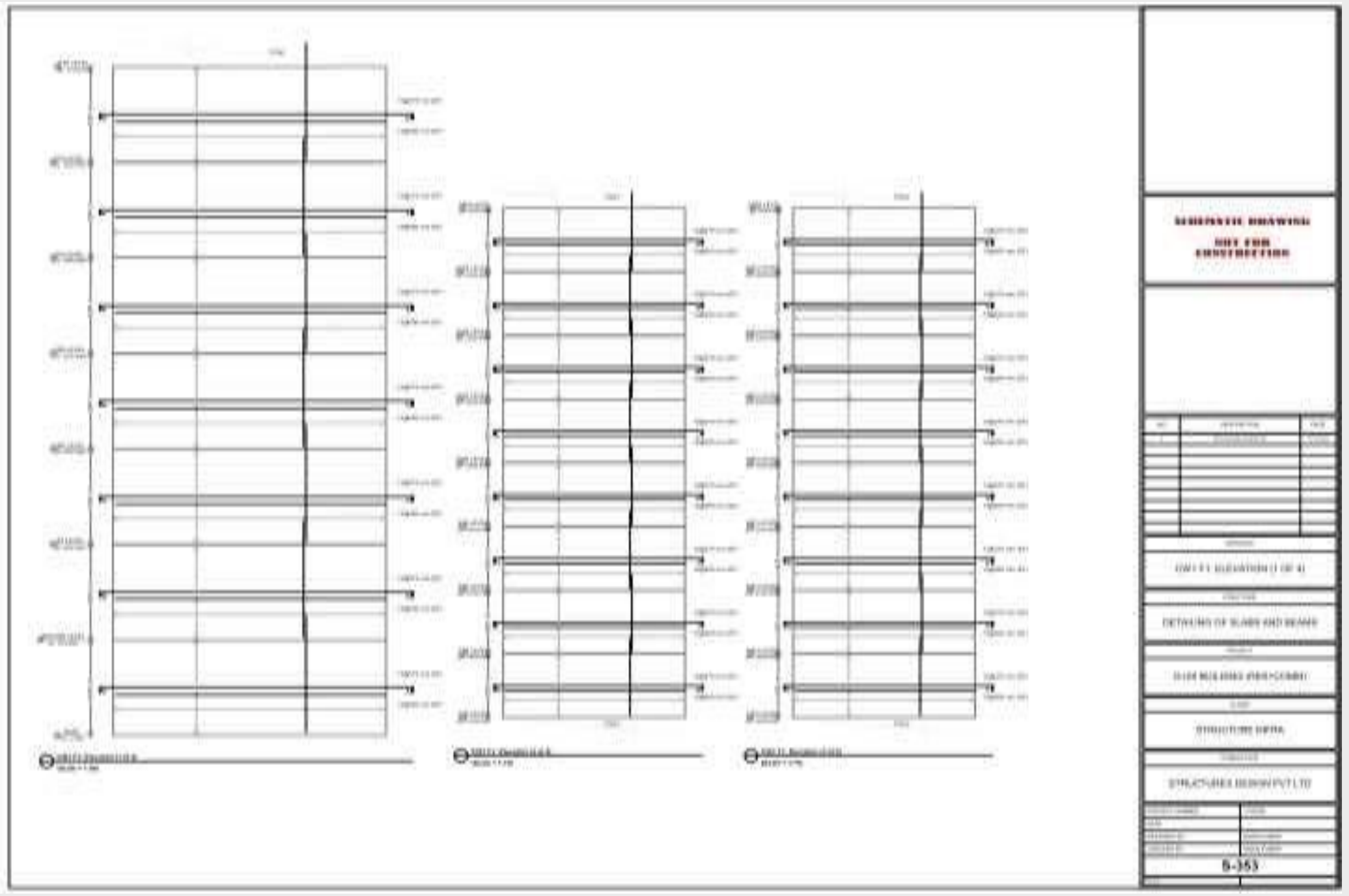


Column Detailing



Wall Detailing

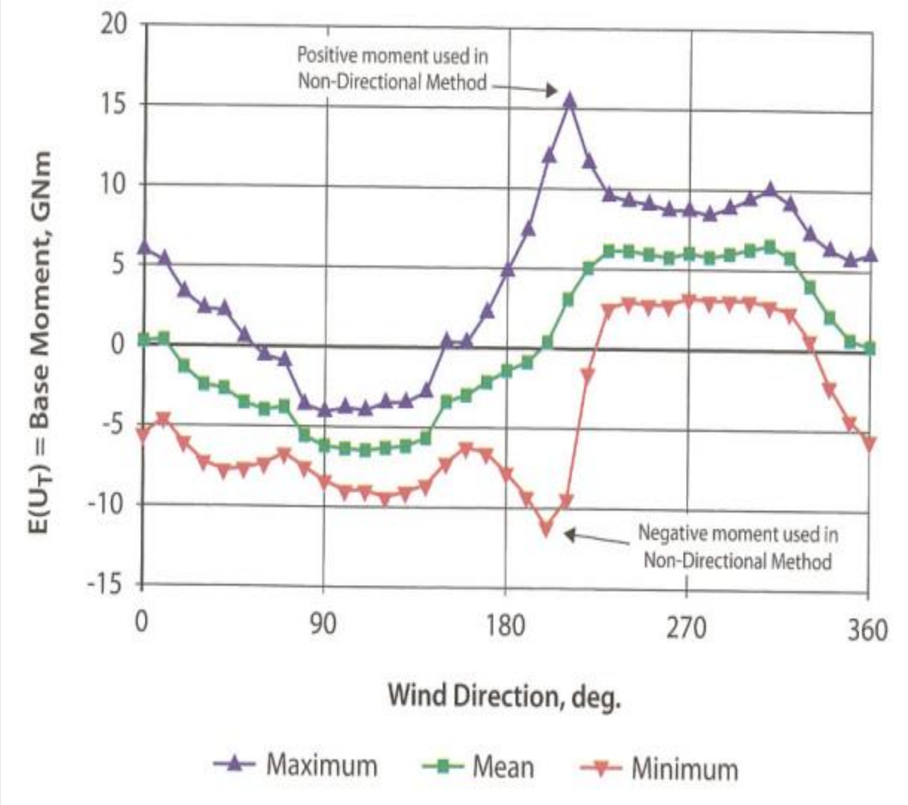


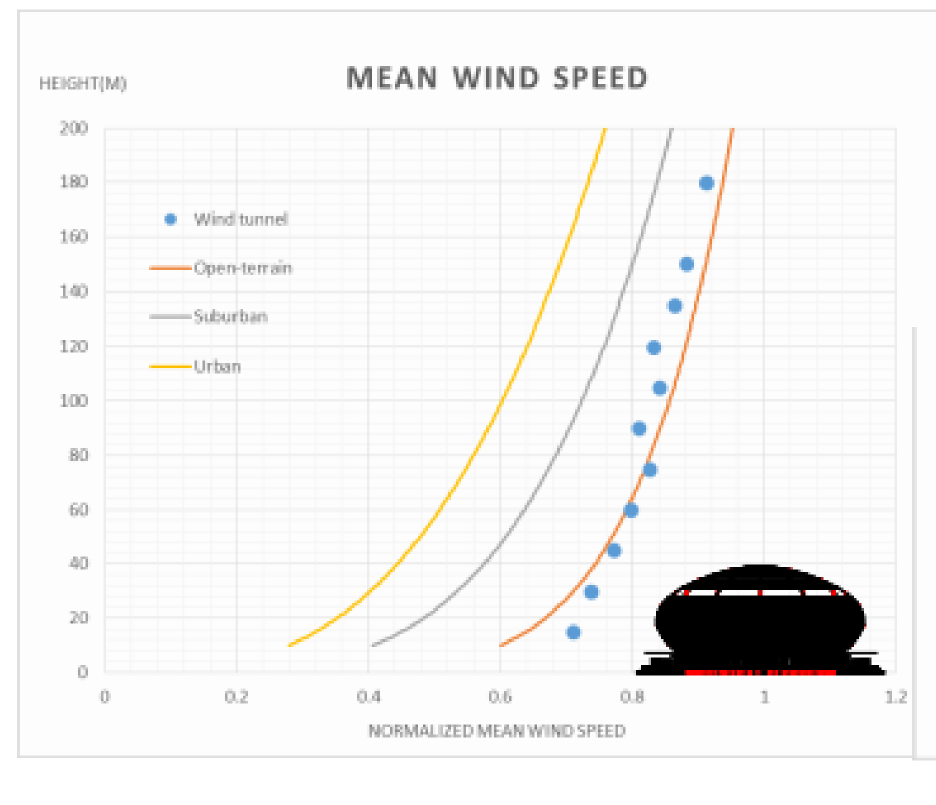


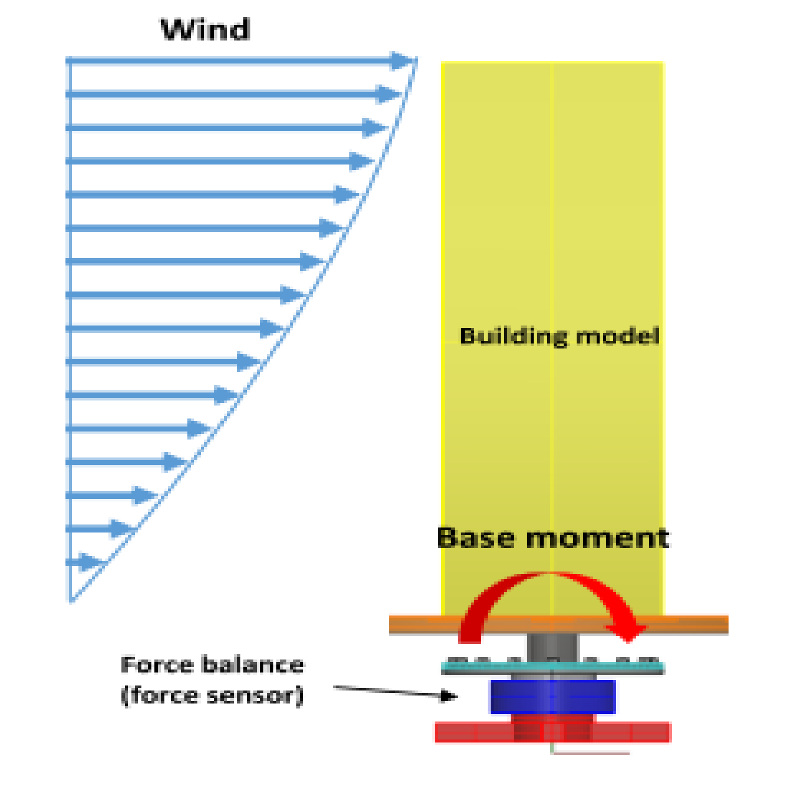
Wind Tunnel Test

Scaled model of proposed development with its surrounding

Pneumatic tubes inside pressure model, RWDI







Wind Tunnel Test

WIND TUNNEL PROCEDURE: A procedure for determining wind loads on buildings and other structures, in which pressures and/or forces and moments are determined for each wind direction considered, from a model of the building or other structure and its surroundings, in accordance with Chapter (ASCE 7-16 Cl.31.) & ASCE 49

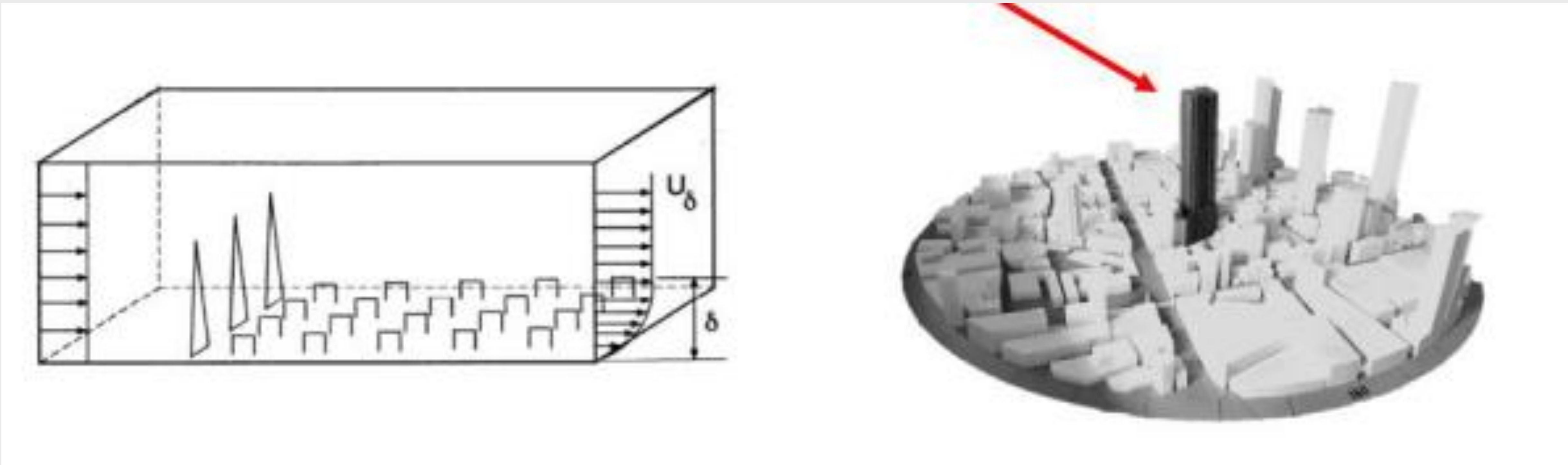
Wind Tunnel Testing of
High-Rise Buildings CTBUH Technical Guides

Measurement of

Force
Pressure
Wind Speed

Natural Wind Simulation

Scaled Model



Wind Tunnel Test

(Atmospheric boundary layer wind tunnel)

- It has been Jointly developed between Thammasart University and Asian Institute of Technology to serve both academic research and commercial purposes.

- 2.5m by 2.5m and 22m (width by height by length)

- Equipped with a 160 kW electric motor
- Capable of producing wind speed of 25 m/s

Atmospheric boundary layer: The lowest layer of troposphere that is directly influenced by the presence of earth surface.

Wind Climate Analysis

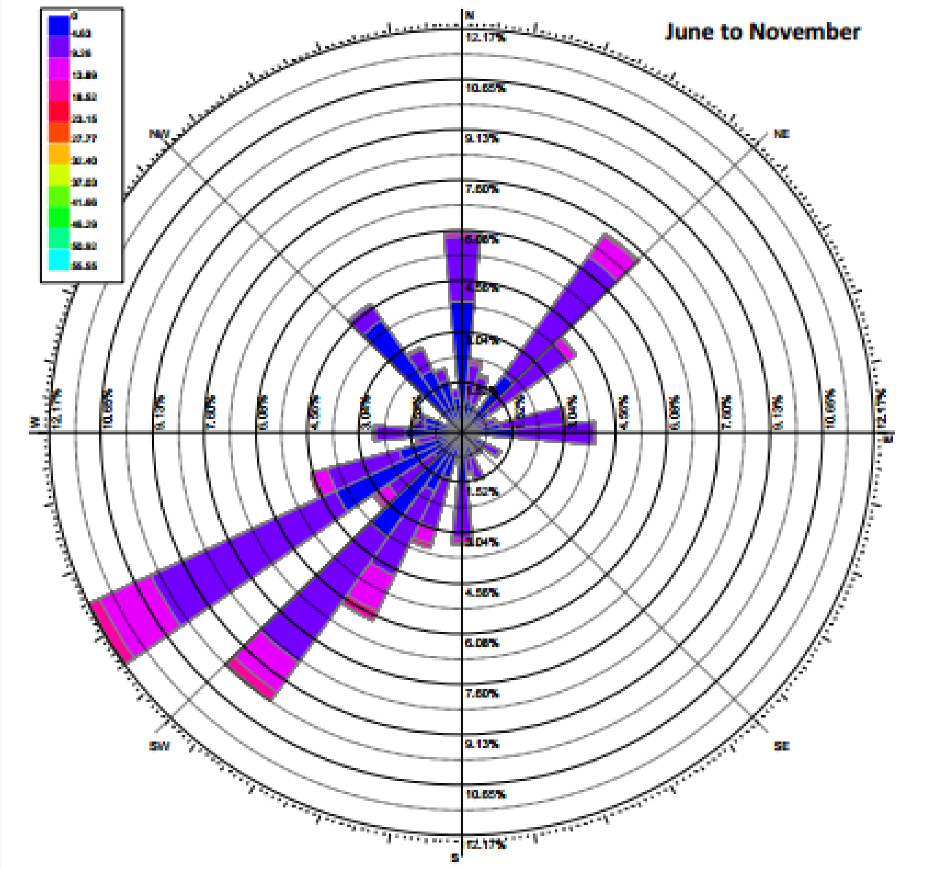
The wind climate model is derived from the analysis of meteorological data and mapped to wind tunnel data to predict wind loads and responses.

In order to obtain reliable and accurate wind characteristics at the specific site, the wind model is combined with terrain analysis and this will be the target wind properties for the wind tunnel test.

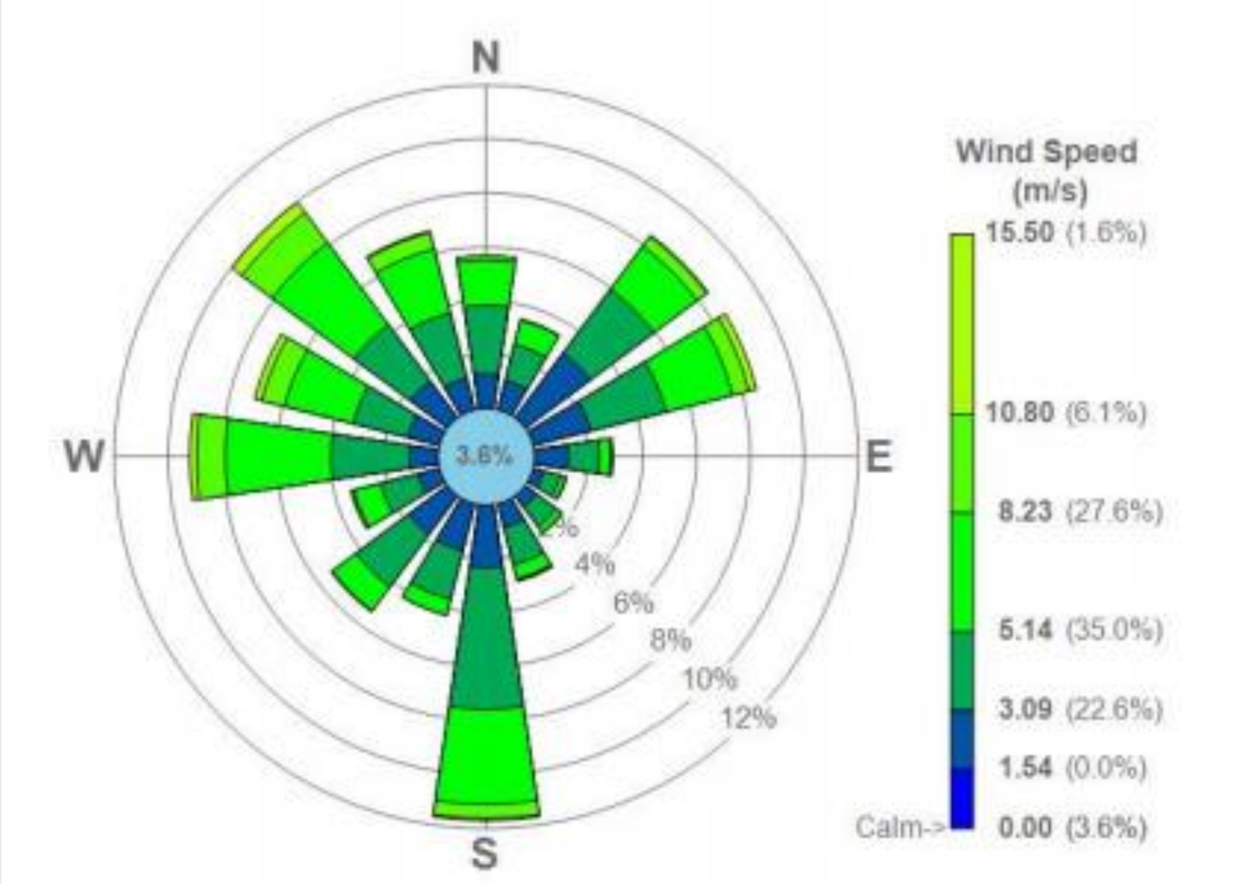
The example of wind speed and turbulence intensity profiles is shown in the next slide.

A wind rose is a graphic tool used by meteorologists to give a clear view of how wind speed and direction are typically distributed at a particular location.

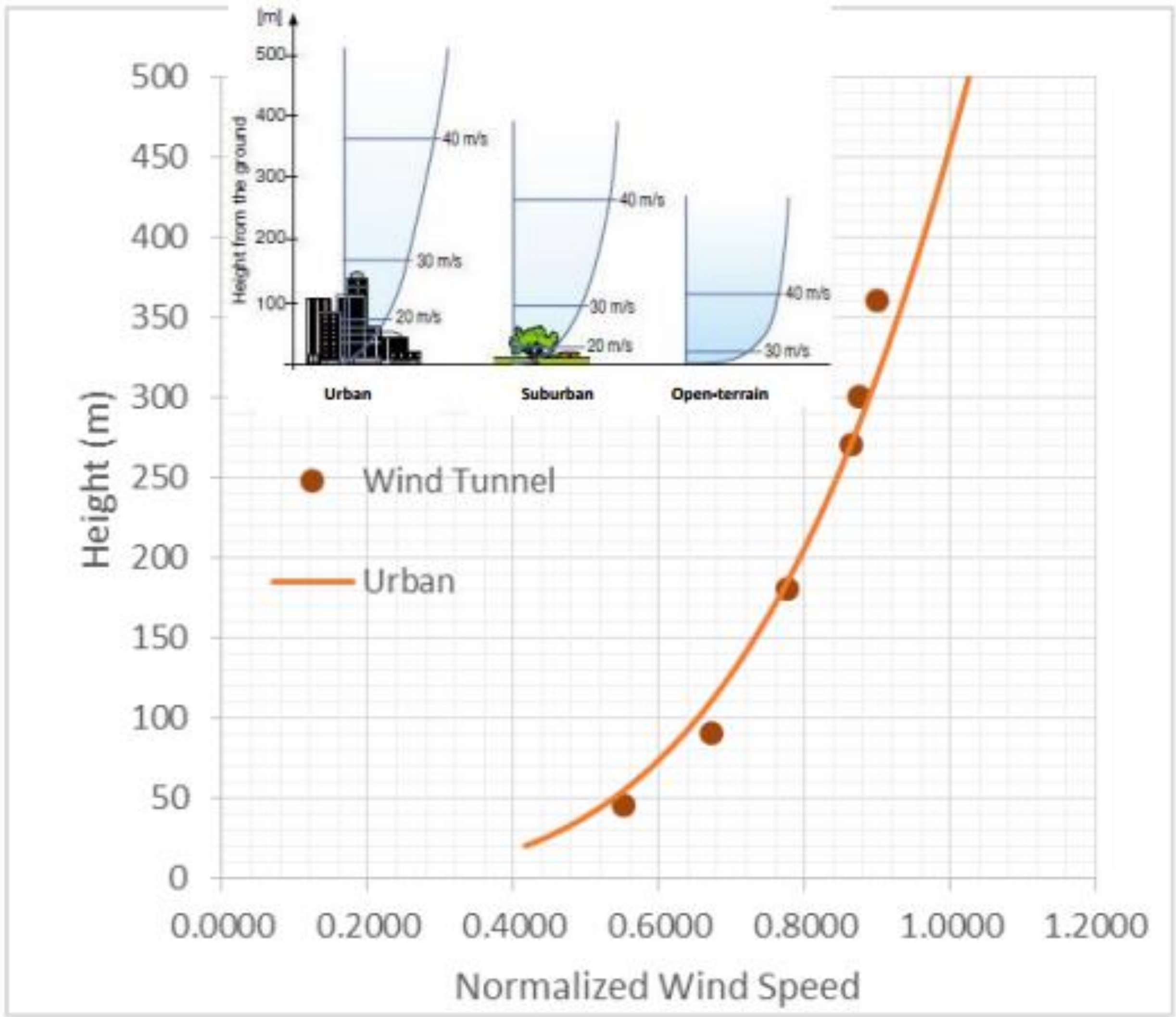
Using a polar coordinate system of gridding, the frequency of winds over a time period is plotted by wind direction, with color bands showing wind speed ranges. The direction of the longest spoke shows the wind direction with the greatest frequency.

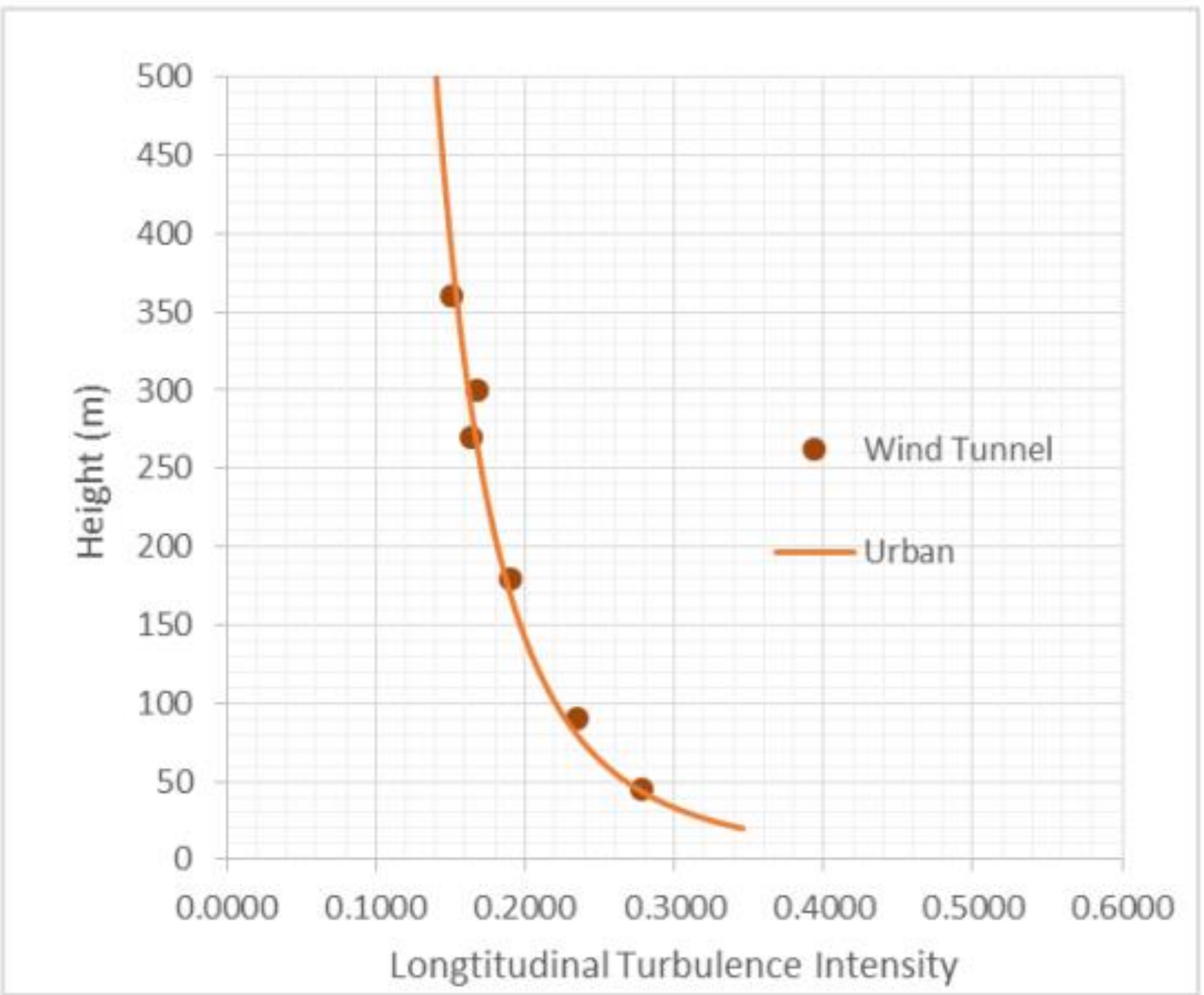


Wind rose taken from 10 years wind speed historical data



Wind Profile Simulation

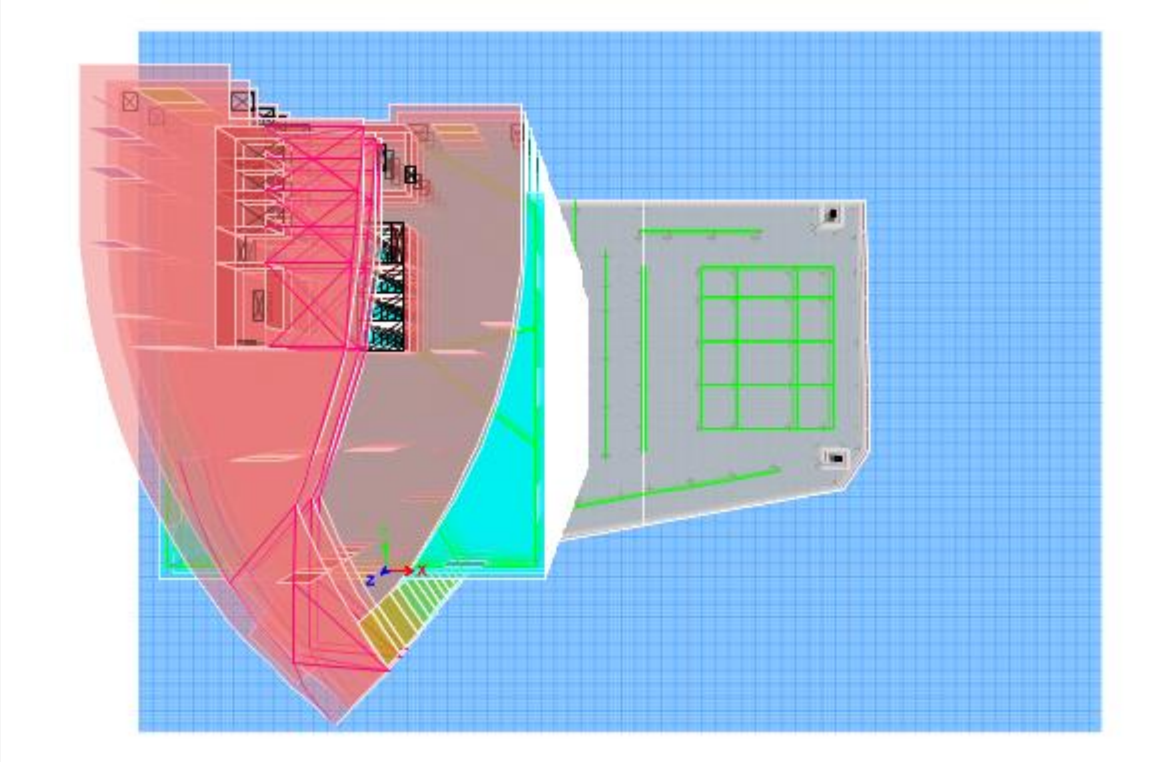


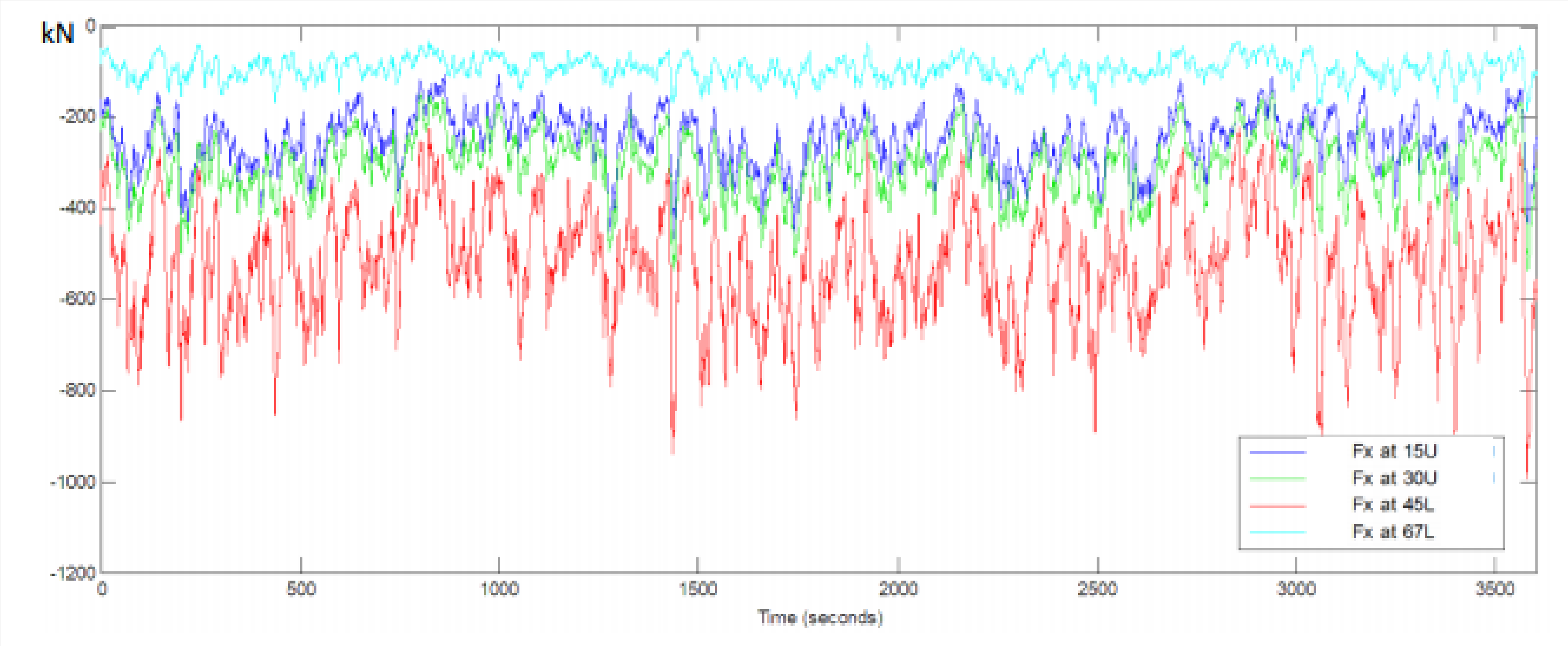


Testing Results

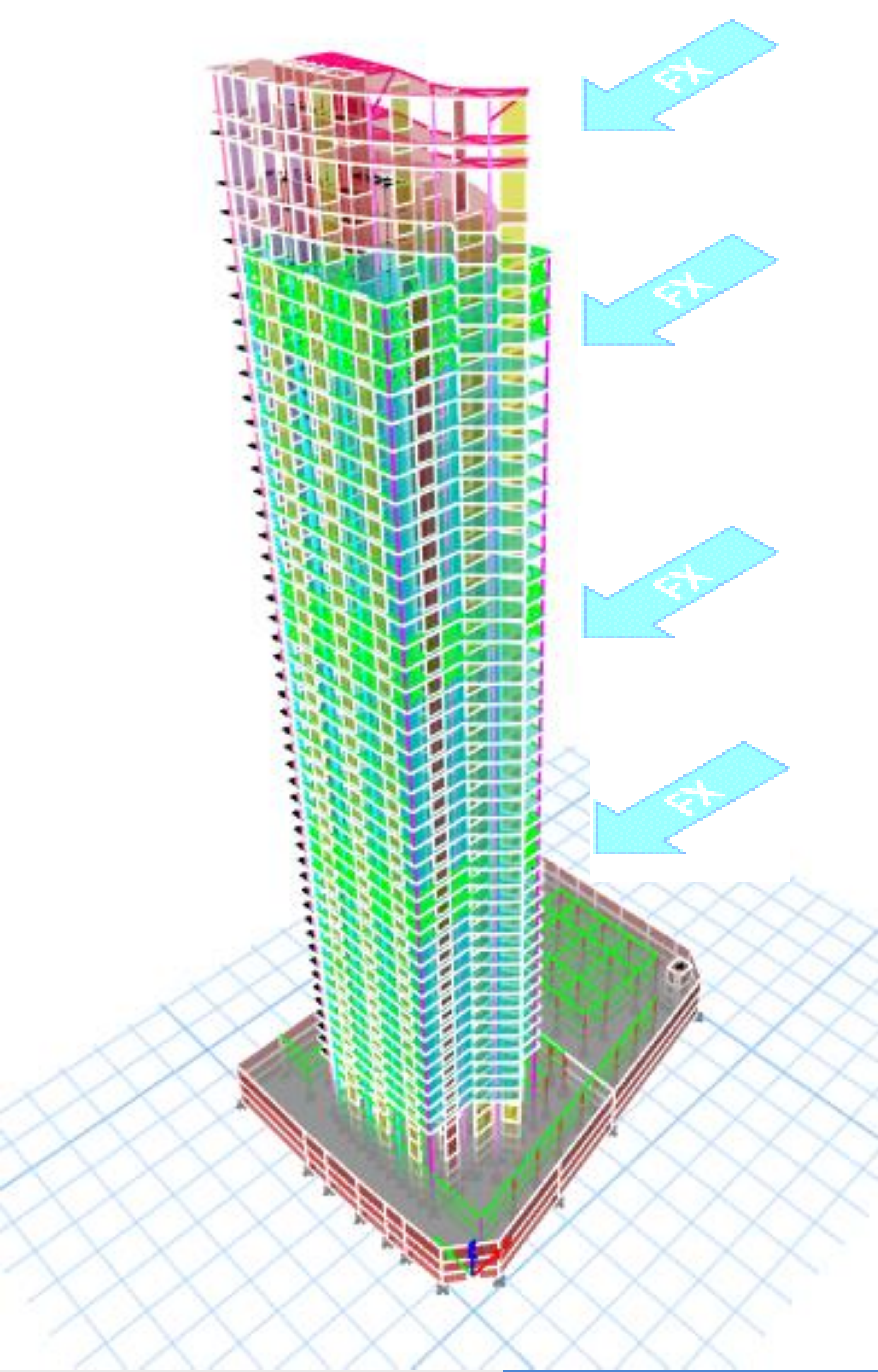
Wind load obtained from wind tunnel test can be either point loads or area pressure loads depending on which technique being used.

- Point loads
- Area pressure loads



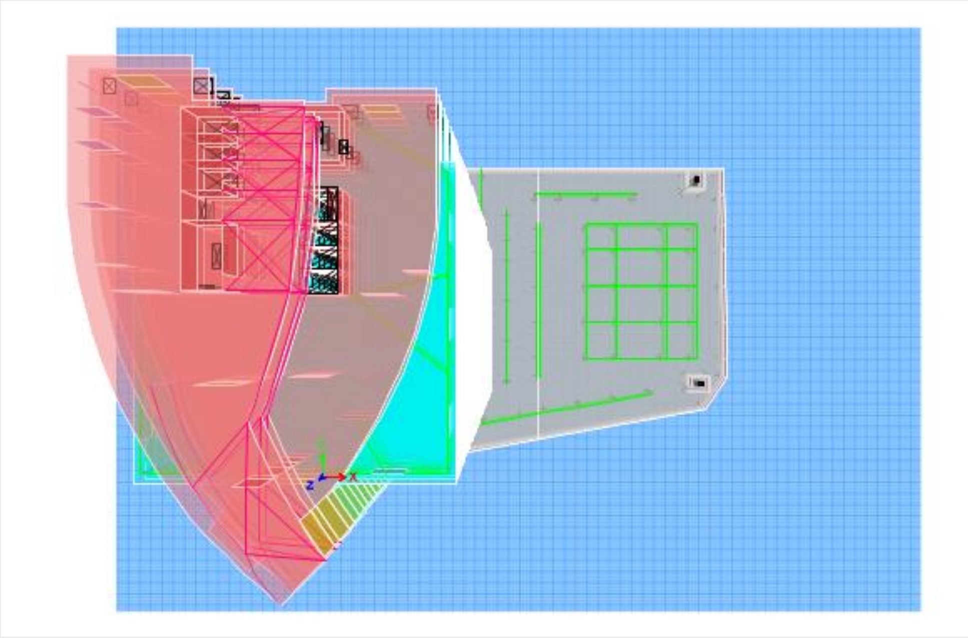


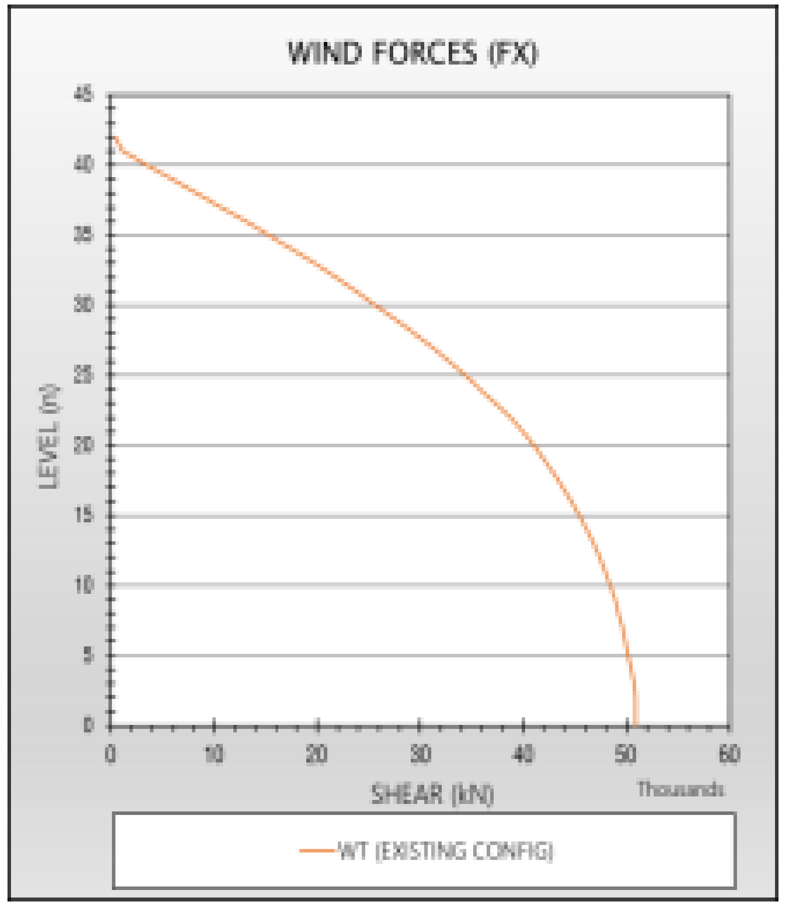
1 hour span of time history point loads at different elevations

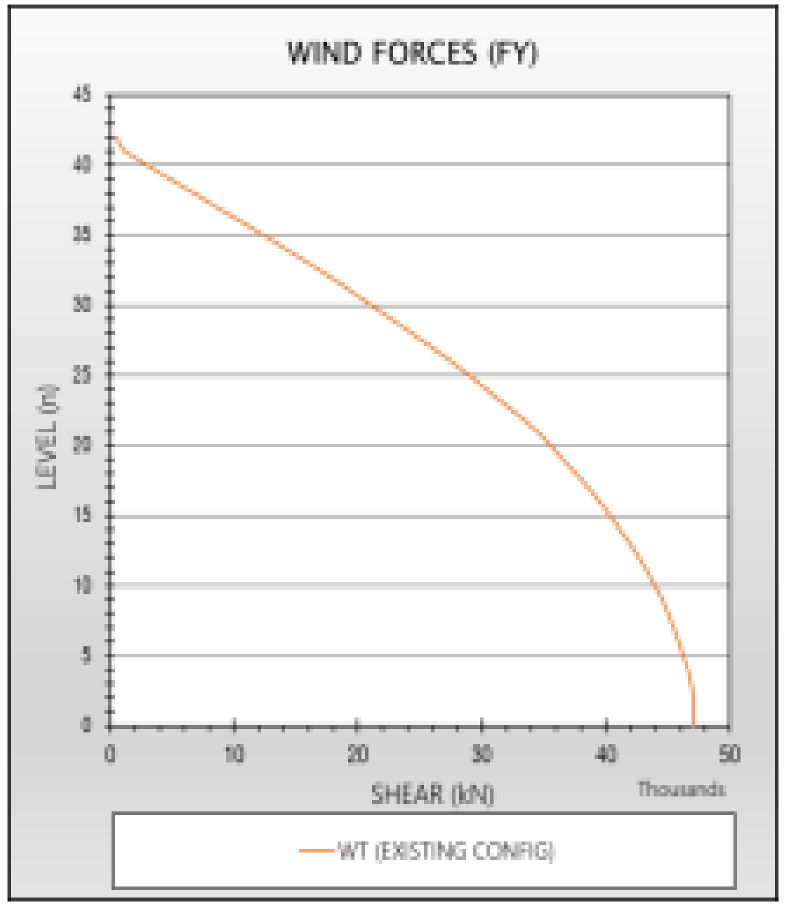


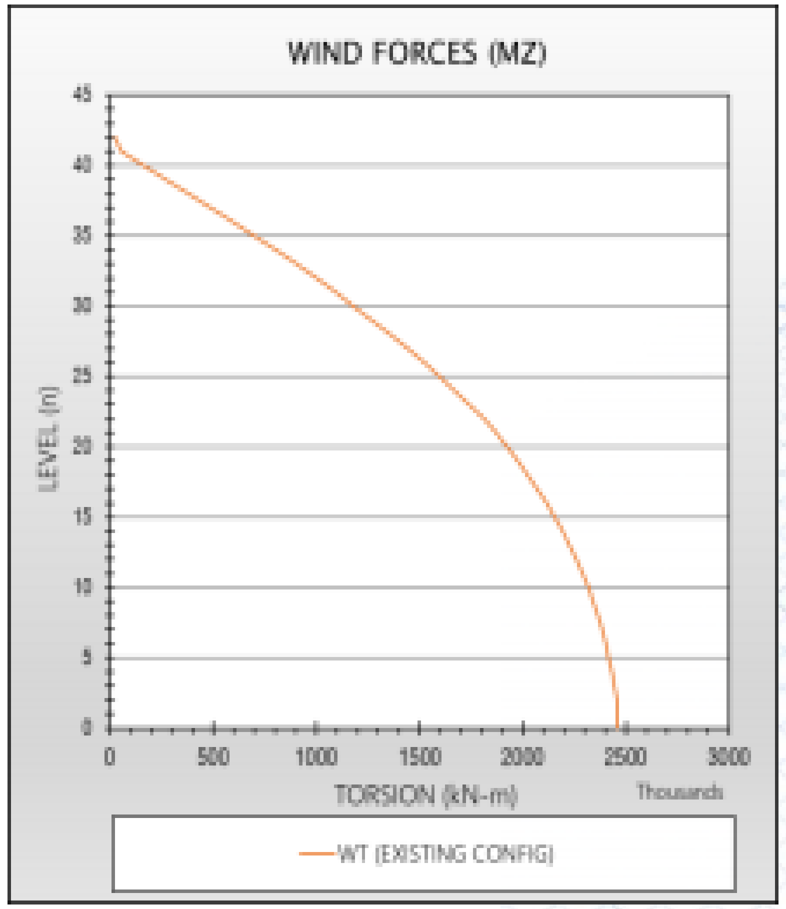
Testing Results

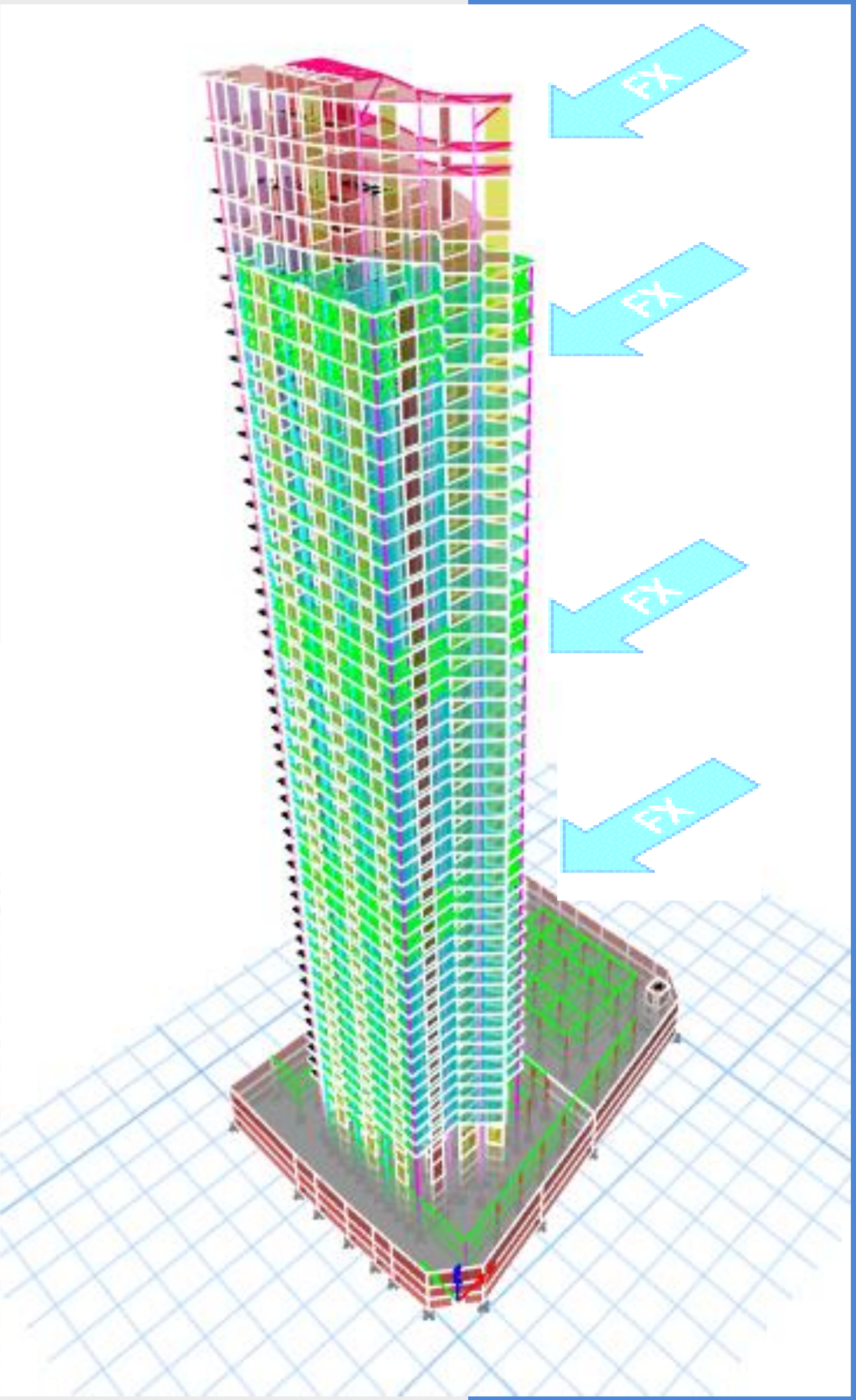
Floor-by-floor equivalent-static wind forces including dynamic effects based on provided structural properties.











Wind Tunnel Test

Detail of measured wind functions

- Design wind information
- Axis system
- Point load calculation
- Base moments and point load coefficients

Point Load Calculation

The Floor-by-floor loads for shear forces (X-and Y-axes) and torsional moment (about Z-axis) at level i are calculated from the point load coefficients and base moment time history functions for each wind direction.

$$f x,i,\theta\ (t) = p x,i \times m\ y,\theta\ (t)$$
$$f y,i,\theta\ (t) = p y,i \times m\ x,\theta\ (t)$$
$$m\ z,i,\theta\ (t) = p z,i \times m\ z,\theta\ (t)$$

Load time functions at level i for wind direction θ kN
for both Fx, Fy
kNm for Mz

Point load coefficients at level i
kN/MNm for both Fx, Fy
kNm/MNm for Mz

Time history of base moments for wind direction θ MNm for both Fx, Fy
MNm for Mz

Wind Tunnel Test

Overview of load definition

Time history functions
3 components (X-,Y-,and Z-axes) of base moment for 36 wind directions
Total number of time history function will be **108 functions**

Load patterns
3 components (X-,Y-,and Z-axes) of point load coefficients
Total number of load pattern will be **3 patterns**

Load cases
3 components (X-,Y-,and Z-axes) of load being applied simultaneously for each wind direction
Total number of load case will be **36 cases**

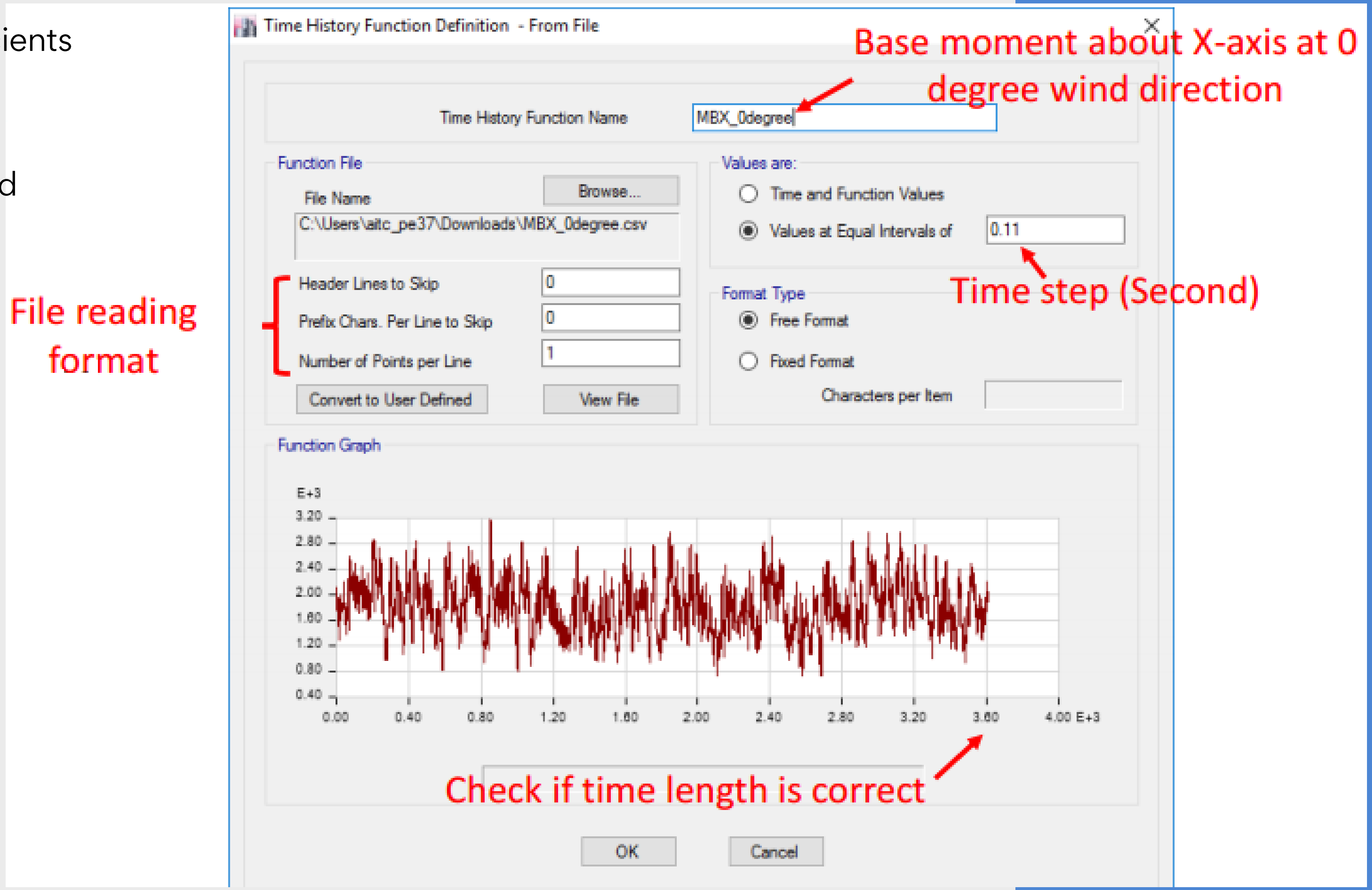
Load combinations
Compliance with structural standard code

Define time history functions

3 Base moment (MX, My, and MZ) time history functions for all 36 wind directions will be assigned in **Define> Time History Function Definition**

These base moments will be then multiplied by point load coefficients to obtain Floor-by Floor loads in Load Cases... later

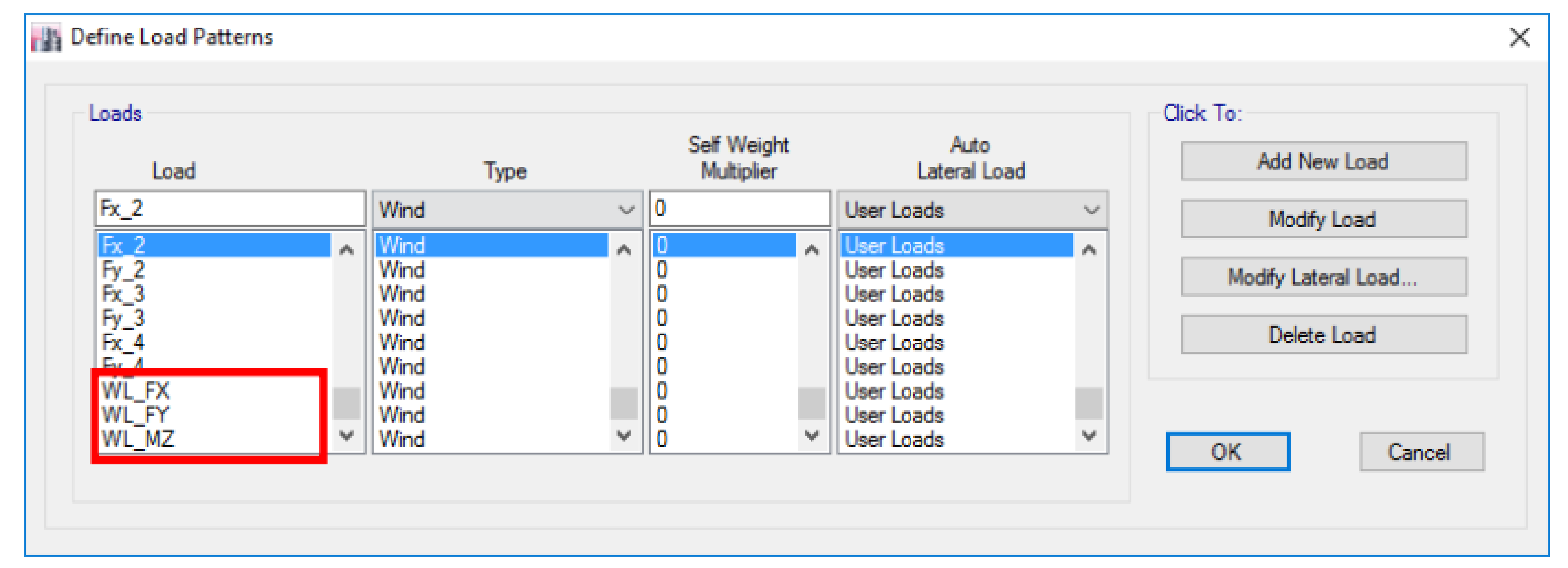
Therefore, total number of 108 functions will be added in this step.



Wind Tunnel Test

Define load patterns

Load distributions along the height or load
patterns can be defined by inputting **point load
coefficients** for 3 components (X-, Y- and Zaxes). Therefore, there will be **3 load patterns**
(each for single direction).



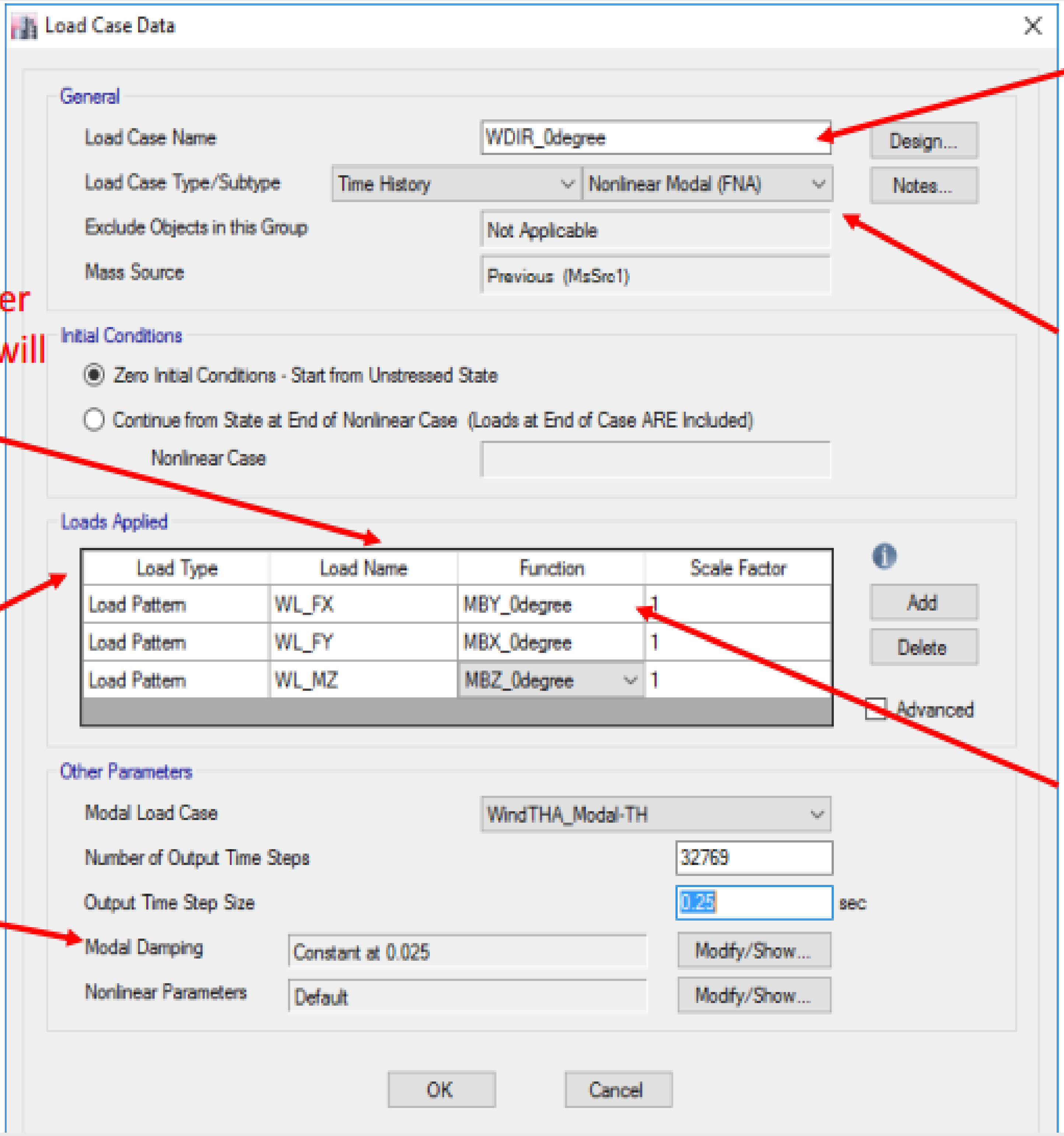
Wind Tunnel Test

Define load cases

Select the **Load pattern** pre-defined earlier
3 components (FX, FY, and MZ) of load will be considered simultaneously

Load type will be **Load pattern**

Damping ratio, typically in the range of 1.5 to 2.5% (for structural design)



There will be **36 load cases** according to number of wind direction considered

Analysis method can be performed by **modal superposition** or **direct-integration**, and both can be **linear** or **nonlinear**.

Select the **load function** pre-defined earlier
For example, the base moment about Y-axis produces shear force in X direction

Wind Tunnel Test

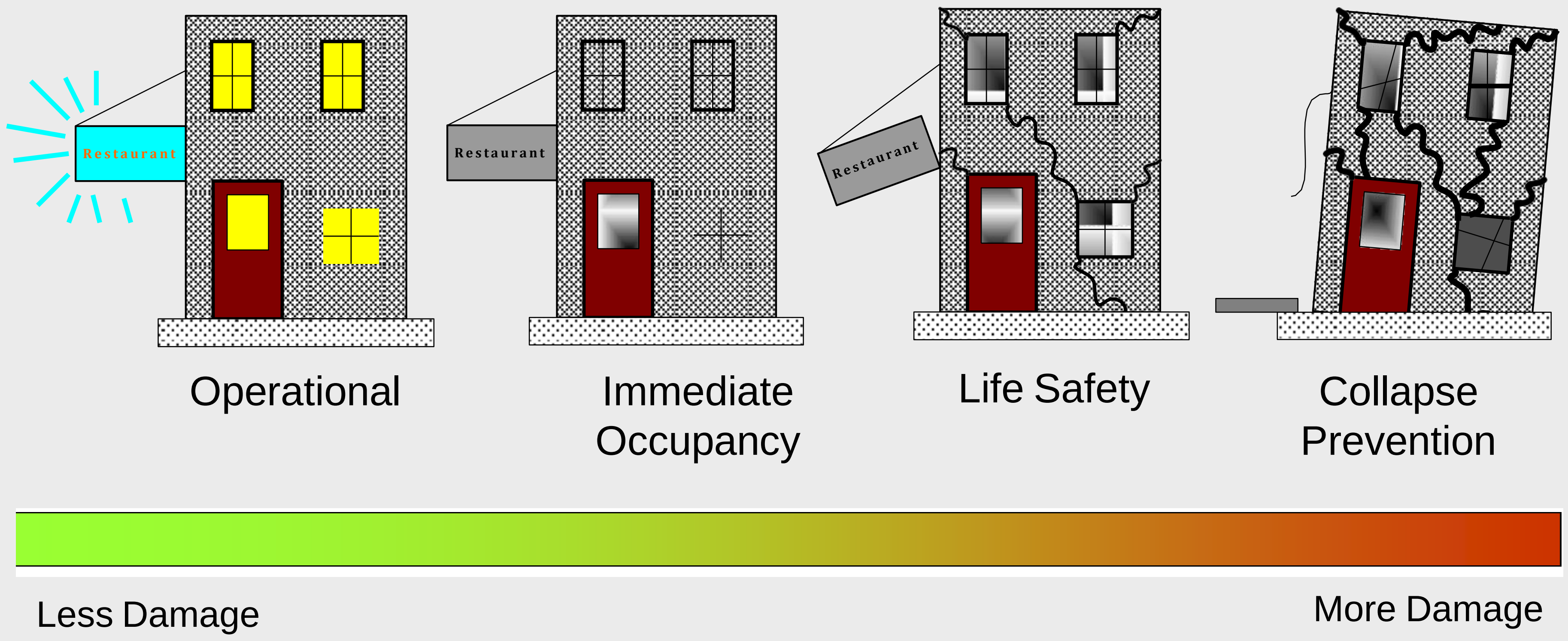
- Magnitude unit: MNm (base moments)
- Sampling length: Typically 1 hour long
- Time step: 0.11s

Maximum deflection allowed against wind is H/500.
Px = 199 mm < 180 X 1000/500 = 360 mm
Py = 188 mm < = 360 mm ……… Safe

											Wind Tunel Values for 50 years						
1		Log 2				Log 3				Log 4				Log 5			
FY(KN)	MZ (KN-m)	Story	Fx (KN)	FY(KN)	MZ (KN-m)	Story	Fx (KN)	FY(KN)	MZ (KN-m)	Story	Fx (KN)	FY(KN)	MZ (KN-m)	Story	Fx (KN)	FY(KN)	MZ (KN-m)
-6	2036	T.O.PARAPET	-72	155	784	T.O.PARAPET	79	135	2036	T.O.PARAPET	115	-6	-1365	T.O.PARAPET	115	75	-1365
90	9579	ROOF	-249	437	3689	ROOF	281	484	9579	ROOF	444	90	-6423	ROOF	444	214	-6423
55	8152	46	-248	471	3139	46	257	437	8152	46	395	55	-5466	46	395	207	-5466
27	5126	45	-178	353	1974	45	168	280	5126	45	254	27	-3437	45	254	137	-3437
21	5047	44	-175	362	1944	44	162	277	5047	44	245	21	-3384	44	245	138	-3384
18	4959	43	-172	362	1910	43	158	272	4959	43	237	18	-3325	43	237	137	-3325
15	4867	42	-169	358	1874	42	153	268	4867	42	230	15	-3263	42	230	136	-3263
10	4761	41	-168	355	1833	41	147	264	4761	41	222	10	-3192	41	222	137	-3192
7	4645	40	-164	350	1789	40	143	259	4645	40	214	7	-3115	40	214	136	-3115
4	4526	39	-162	342	1743	39	137	253	4526	39	206	4	-3035	39	206	134	-3035
-1	4394	38	-161	327	1692	38	129	248	4394	38	196	-1	-2946	38	196	134	-2946
-5	4251	37	-160	314	1637	37	122	243	4251	37	186	-5	-2851	37	186	133	-2851
-8	4106	36	-157	307	1581	36	117	236	4106	36	178	-8	-2753	36	178	131	-2753
-11	3948	35	-153	304	1521	35	113	230	3948	35	171	-11	-2648	35	171	129	-2648
-16	3782	34	-150	308	1457	34	108	226	3782	34	163	-16	-2536	34	163	130	-2536
-22	3614	33	-146	311	1392	33	103	221	3614	33	156	-22	-2423	33	156	130	-2423
-24	3434	32	-142	307	1323	32	99	214	3434	32	149	-24	-2303	32	149	128	-2303
-27	3248	31	-139	301	1251	31	94	206	3248	31	142	-27	-2178	31	142	124	-2178
-27	3071	30	-140	293	1183	30	90	197	3071	30	135	-27	-2059	30	135	120	-2059
-26	2925	29	-143	285	1127	29	88	188	2925	29	131	-26	-1962	29	131	115	-1962
-24	3034	28	-148	287	1168	28	91	191	3034	28	136	-24	-2035	28	136	115	-2035
-36	4693	27	-227	436	1807	27	139	291	4693	27	208	-36	-3147	27	208	175	-3147
-33	2186	26	-133	260	842	26	73	158	2186	26	107	-33	-1466	26	107	102	-1466
-33	1970	25	-129	250	758	25	70	148	1970	25	102	-33	-1321	25	102	96	-1321
-32	1752	24	-124	240	675	24	67	137	1752	24	96	-32	-1175	24	96	91	-1175
-34	1533	23	-120	232	590	23	64	129	1533	23	90	-34	-1028	23	90	87	-1028
-36	1316	22	-117	227	507	22	60	122	1316	22	84	-36	-882	22	84	84	-882
-38	1104	21	-114	221	425	21	57	115	1104	21	79	-38	-740	21	79	81	-740
-40	899	20	-111	217	346	20	53	108	899	20	73	-40	-602	20	73	79	-602

Performance Based Design of Buildings

PERFORMANCE LEVELS



Performance Based Design of Buildings

Performance Based Design is a process of designing structures for predictable performance for initially considered loads. This approach is used to design a new building or to evaluate an existing structure. In performance based design, structural engineers identify the specific performance of the structures in consultation with the owner initially and then proceed with the design or evaluation of the existing structure.

Performance based design is widely used in connection with the seismic loads. Evaluation of the existing structure or designing structure of against probable earthquake load finalized initially is done. However, in modern design now, performance based design issued in wind designs, earthquake designs, blast analysis and design and progressive collapse analysis.

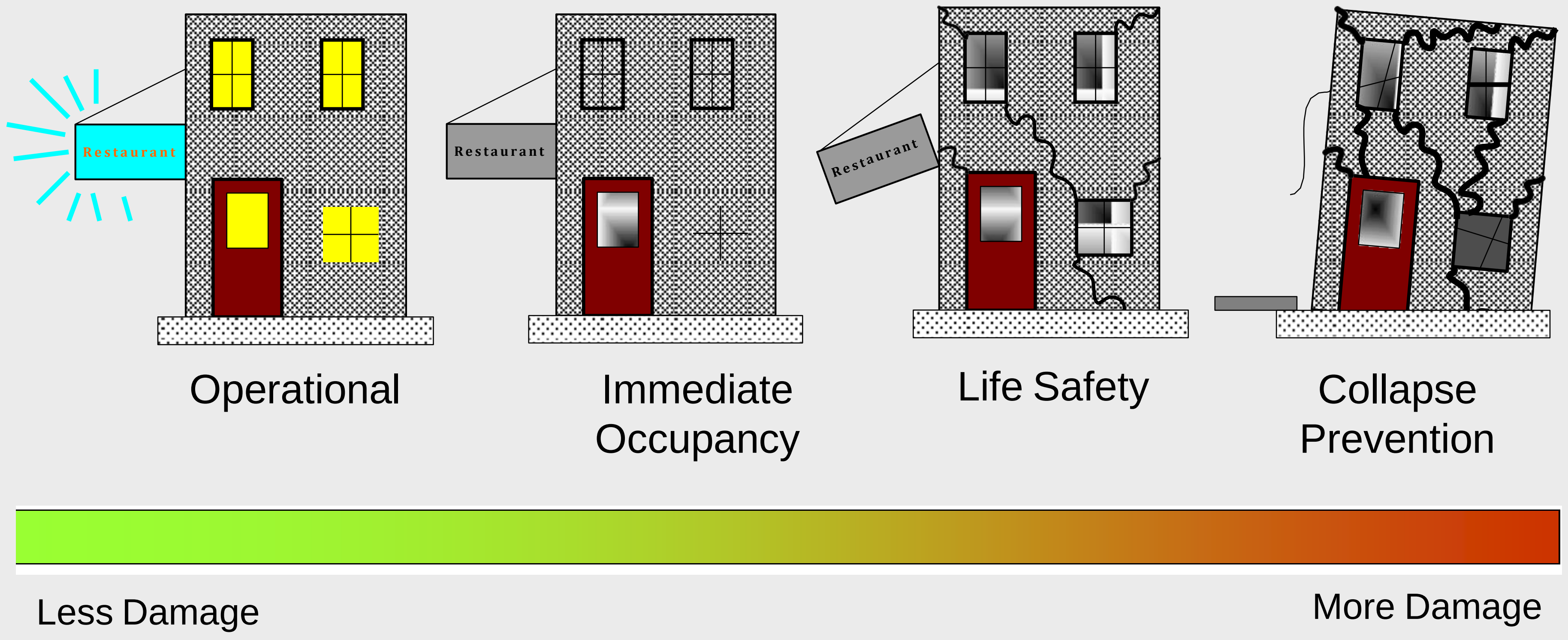
Further, performance-based design of structures considers the behavior of the nonstructural members also.
Due to the dynamic behavior to be considered in the loads such as winds and blast loads structural engineers have considered these types of design now.
If this concept is further explained, initially we select an occupancy level that the structure to behave of an initially finalized load with the owner. For example, consider a 7.5 magnitude earthquake. Then we consider the relevant load/acceleration values for performance based seismic design. Now we know the inputs and outputs. So we modify the structural stiffness to used these loads and outputs. This can be identified as the performance based seismic design of buildings.

Performance Based Design of Buildings

The basic concept of performance based seismic design is to provide engineers with the capability to design buildings that have a predictable and reliable performance in earthquakes.

Thus the Performance-based seismic design is a process that permits design of new buildings or upgrade of existing buildings with a realistic understanding of the risk of life, occupancy and economic loss that may occur as a result of future earthquakes.

Performance-based design begins with the selection of design criteria stated in the form of one or more performance objectives. Each performance objective is a statement of the acceptable risk of incurring specific levels of damage, and the consequential losses that occur as a result of this damage, at a specified level of seismic hazard.



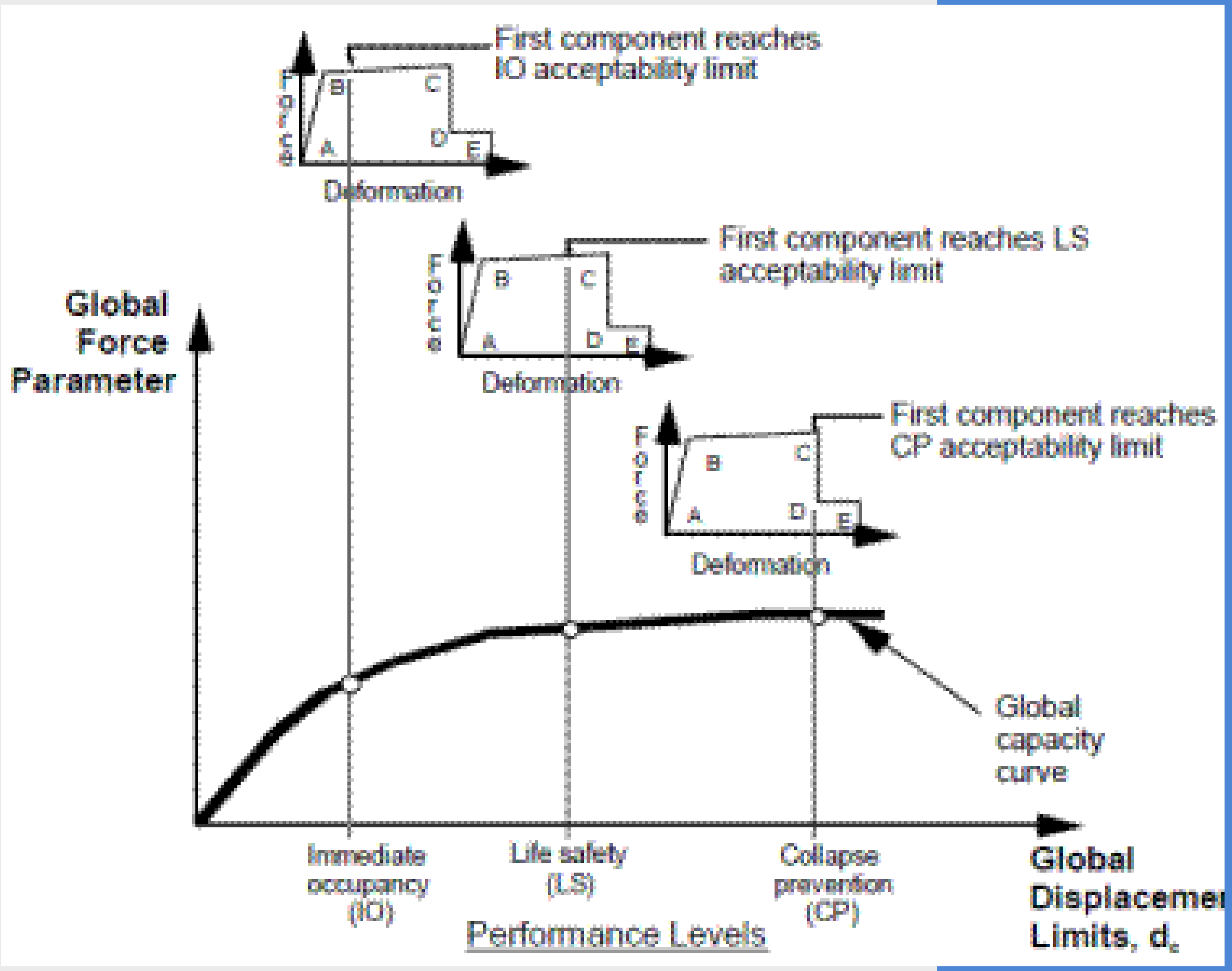
Performance Based Design of Buildings

in summary, the following steps can be identified in the performance-based building design of a new building.
Establish the performance objectives for specific inputs to be considered.
Proceed with the initial design

Check whether the desired outcomes are met.
The deformation of the structure is evaluated in terms of a drift of monitoring the behavior of structural and nonstructural elements. limitations of the drift have been specified by the guidelines such as FEMA 273 and FEMA 356.
The applicable limitations have provided not only for vertical elements but also the other elements.

The guidelines given in FEMA 356 can be used to define the performance levels. It can be specified in terms of the latera deflection/ drift of the building or as the formation of the hinges.

The occupancy level has three states defined based on the rotation of the element. They are specifying according to the rotation of the hinge. The following figure extracted from the FEMA 356 indicates the formation of occupancy levels as per the global displacement.



Performance Based Design of Buildings

As specified in FEMA 356, there are three occupancy levels.

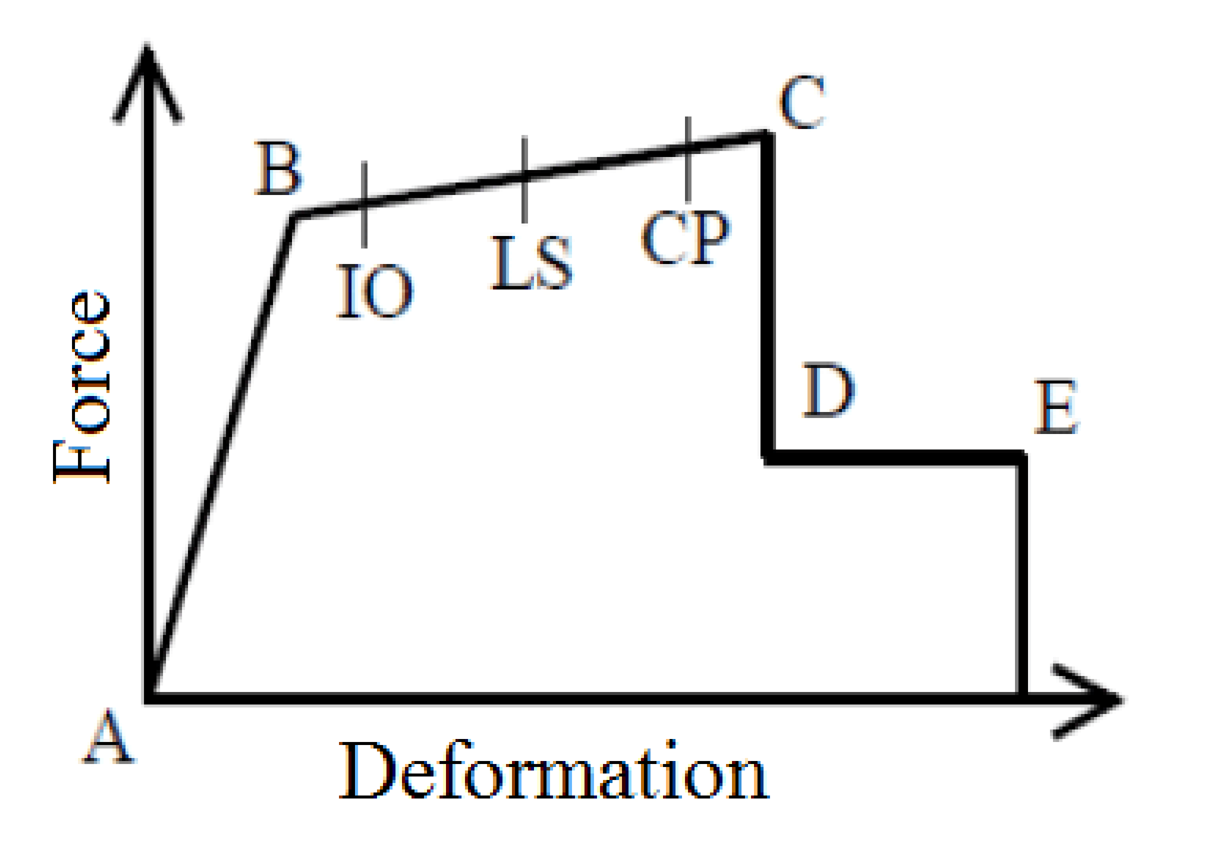
1. Immediate Occupancy
2. Lift Safety
3. Collapse Prevention

The occupancy levels are defined as discussed previously also based on the deformation of the structure. When the occupancy levels are decided for structural elements, it is done based on the rotation of the hinge.
Though the behavior of the elements is indicated as the above figure, in structural analysis, a more simplified behavior is considered. The displacement force cure indicated in the ETABS/Sap2000 structural analysis soft manual is indicated as the following figure.

The variation of each occupancy level is indicated in the above figure. Further, the collapse does not mean they fully collapse. There is a certain stiffness in the structural member after the loss of its stiffness suddenly as indicated by the range C-D-E of the above figure.

The behavior of the element is expressed in terms of the rotation of the hinge. The hinge rotation can be indicated in the following figure. It indicates the relative rotation and the levels in which the hinge indicates each occupancy level.

The above figure was extracted from the ETABS/Sap2000 software.



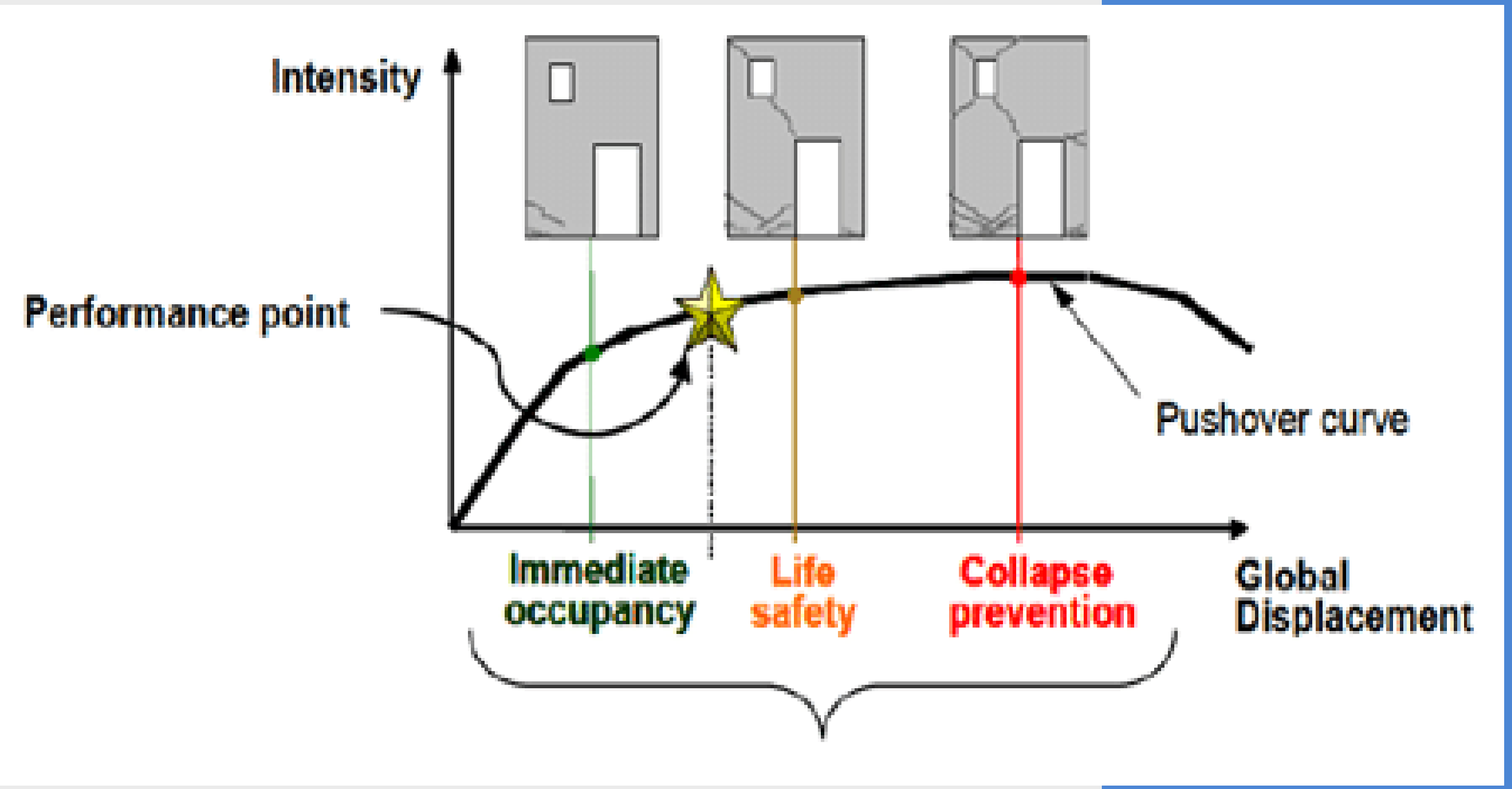
Performance Based Design of Buildings

Occupancy levels are defined to identify the behavior of the structural elements and for knowing their conditions. Occupancy levels represent the condition of the element from the operation level to the failure.

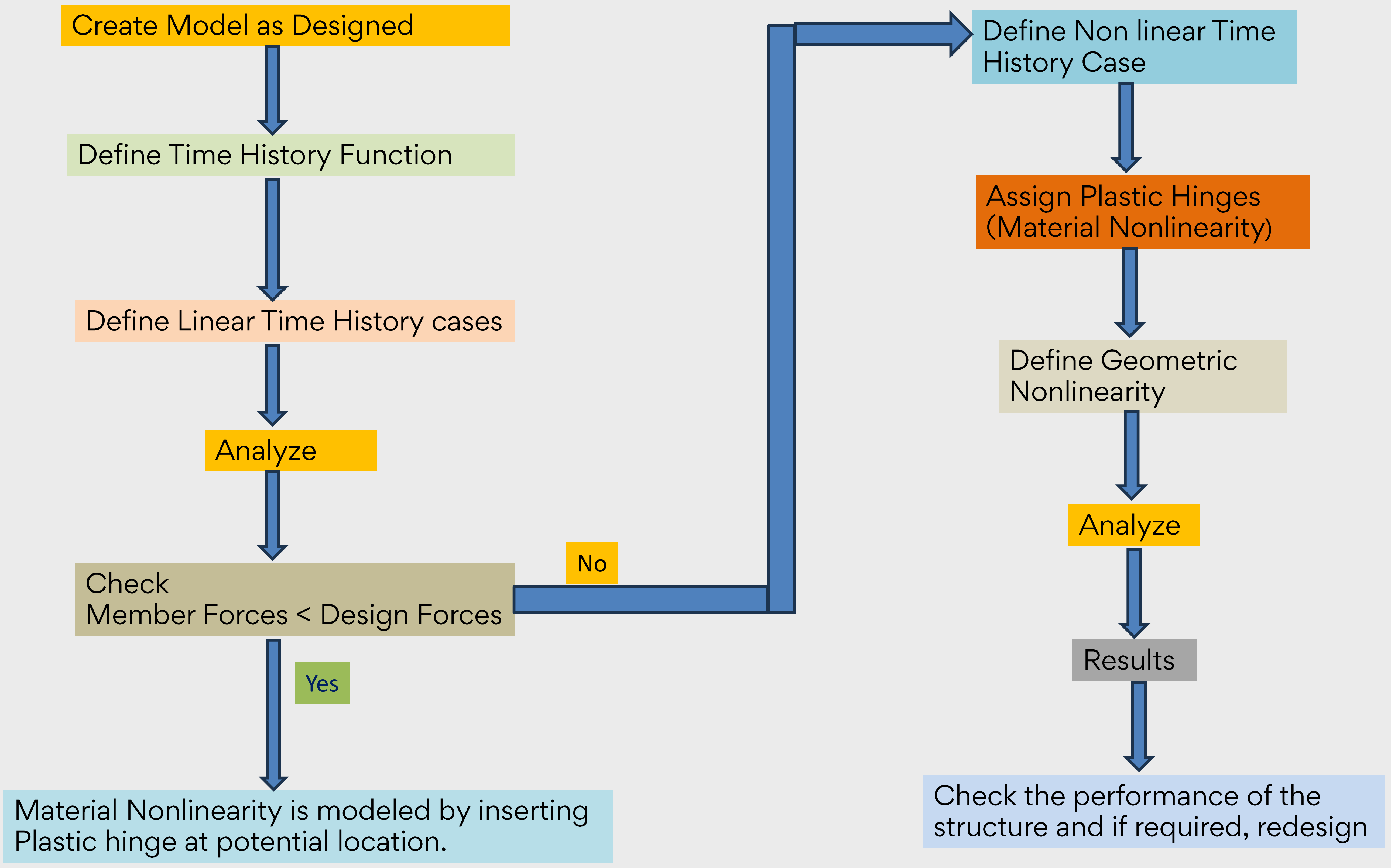
POSSIBLE STEPS FOR PERFORMANCE BASED DESIGN

1.PERFORM INITIAL DESIGN.
2.DETAIL DEFORMATION-CONTROLLED COMPONENTS.
3.RUN AN ELASTIC SLE ANALYSIS USING EFFECTIVE STIFFNESS.
4.PERFORM MCE NONLINEAR TIME-HISTORY ANALYSIS USING EXPECTED STRENGTHS.
5.VERIFY THAT RESULTS MEET THE ACCEPTANCE CRITERIA

PERFORMANCE-BASED DESIGN CAN BE APPLIED TO ANY TYPE OF LOADS, BUT IS TYPICALLY SUITABLE
AND TARGETED FOR EARTHQUAKE LOADS



Performance Based Design of Buildings



Performance Based Design of Buildings

Practical Approaches for Nonlinear Modeling of Structures

Defining Inelastic Behavior at

Nonlinear Modeling of Materials → Material Level → Fiber Modeling Approach

Nonlinear Modeling of Cross-sections → Cross-section Level

Nonlinear Modeling of Members → Member Level

Cross-section Level, Member Level → Plastic Hinge Modeling Approach

Performance Based Design of Buildings

Plastic Hinge Modeling Approach

Defining Inelastic Behavior at

- Nonlinear Modeling of Cross-sections → Cross-section Level
  - Moment-Curvature Plastic Hinges
  - Shear Hinges
  - Axial Hinges
  - Torsion Hinges
- Nonlinear Modeling of Members → Member Level
  - Moment-Rotation Plastic Hinges
  - Shear Hinges
  - Axial Hinges
  - Torsion Hinges

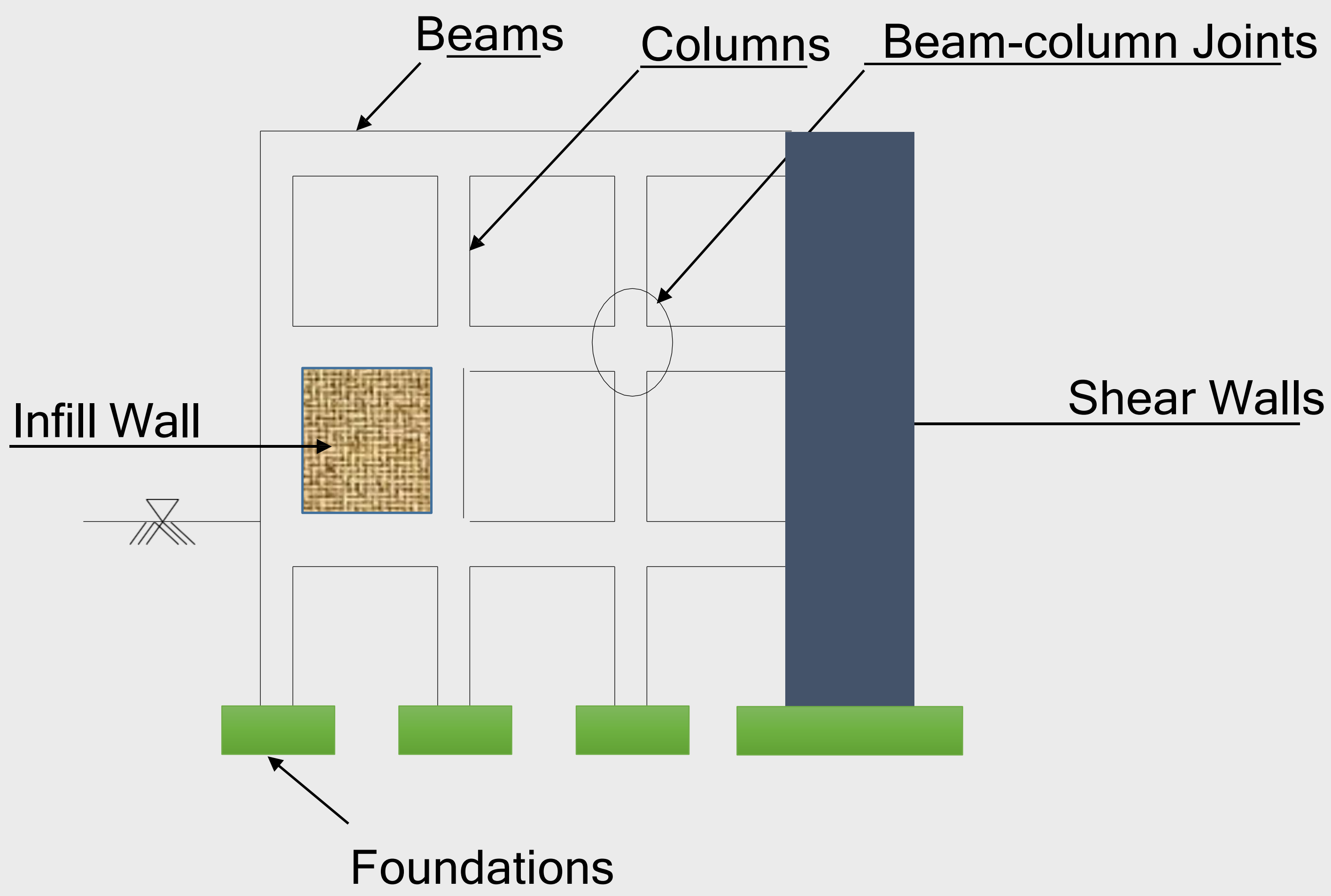
Plastic Hinge Modeling Approach

Plastic Hinge Elements

- Flexural Hinges
  - Moment-Curvature Plastic Hinges
  - Moment-Rotation Plastic Hinges
- Shear Hinges
- Axial Hinges
- Torsion Hinges

Performance Based Design of Buildings

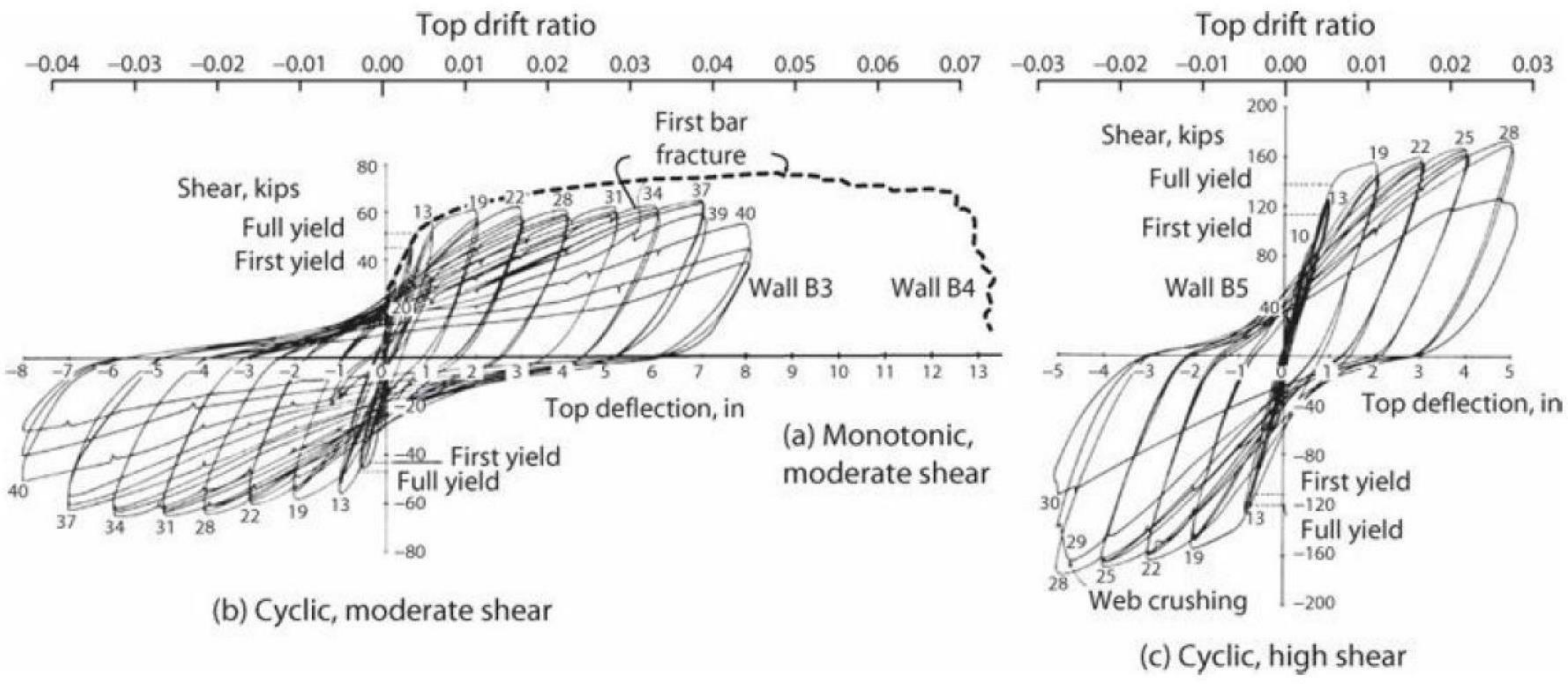
Nonlinear Models for Building Components



- Truss – Yielding and Buckling
- 3D Beam – Major direction Flexural and Shear Hinging
- 3D Column – P-M-M Interaction and shear Hinging
- Panel Zone – Shear Yielding
- In-Fill Panel – Shear Failure
- Shear Wall – P-M-Shear Interaction
- Springs – Foundation and Soil Modeling

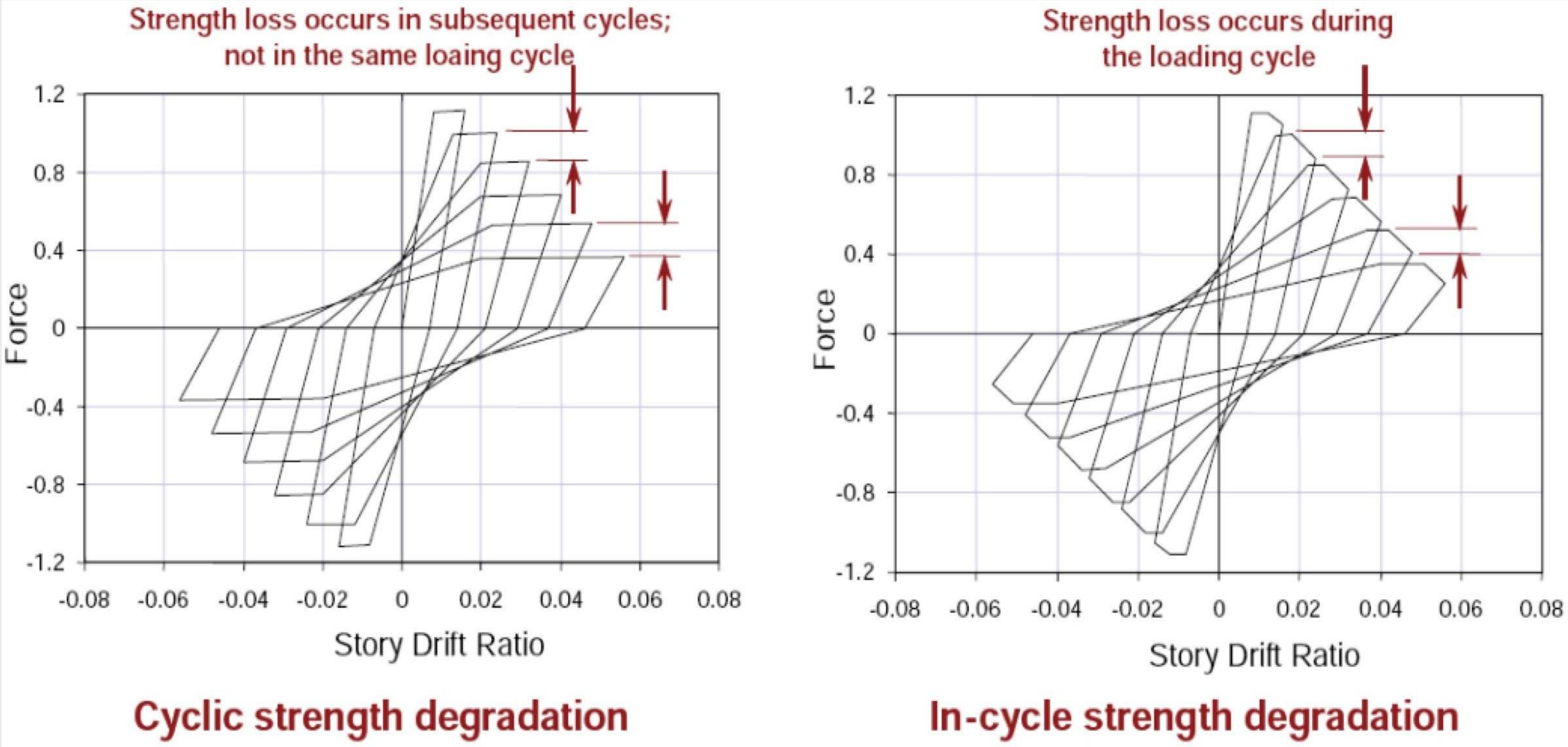
Performance Based Design of Buildings

Load-displacement relationship of shear walls



Performance Based Design of Buildings

Phenomena Associated with Cyclic Action-Deformation Curves



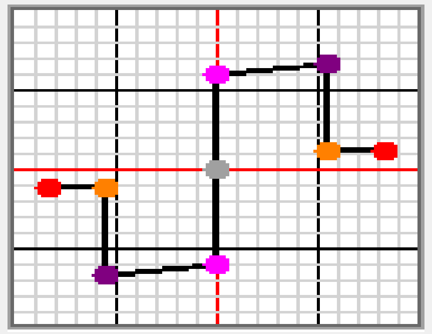
Performance Based Design of Buildings

ASCE 41 MOMENT HINGE

Frame Hinge Property Data for B18H1 - Moment M3

Displacement Control Parameters

Point	Moment/SF	Rotation/SF
E-	-0.2	-6
D-	-0.2	-4
C-	-1.12	-4
B-	-1	0
A	0	0
B	1	0
C	1.12	4
D	0.2	4
E	0.2	6



Symmetric

Load Carrying Capacity Beyond Point E

Drops To Zero

Is Extrapolated

Scaling for Moment and Rotation

		Positive	Negative	
Use Yield Moment	Moment SF	1397.9167	1397.9167	kip-ft
Use Yield Rotation (Steel Objects Only)	Rotation SF	0.00767	0.00767	

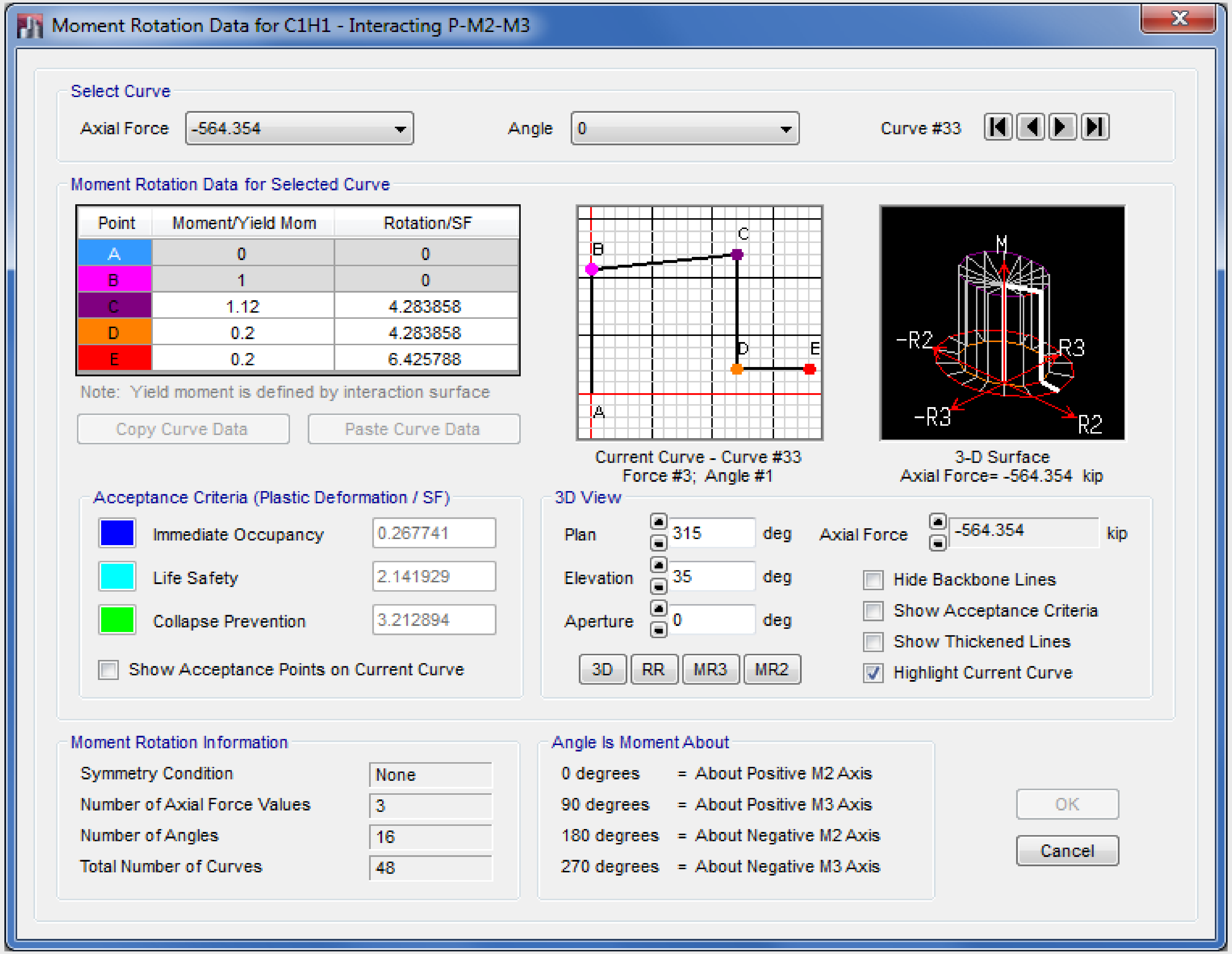
Acceptance Criteria (Plastic Rotation/SF)

	Positive	Negative
Immediate Occupancy	0.25	-0.25
Life Safety	2	-2
Collapse Prevention	3	-3

Show Acceptance Criteria on Plot

Performance Based Design of Buildings

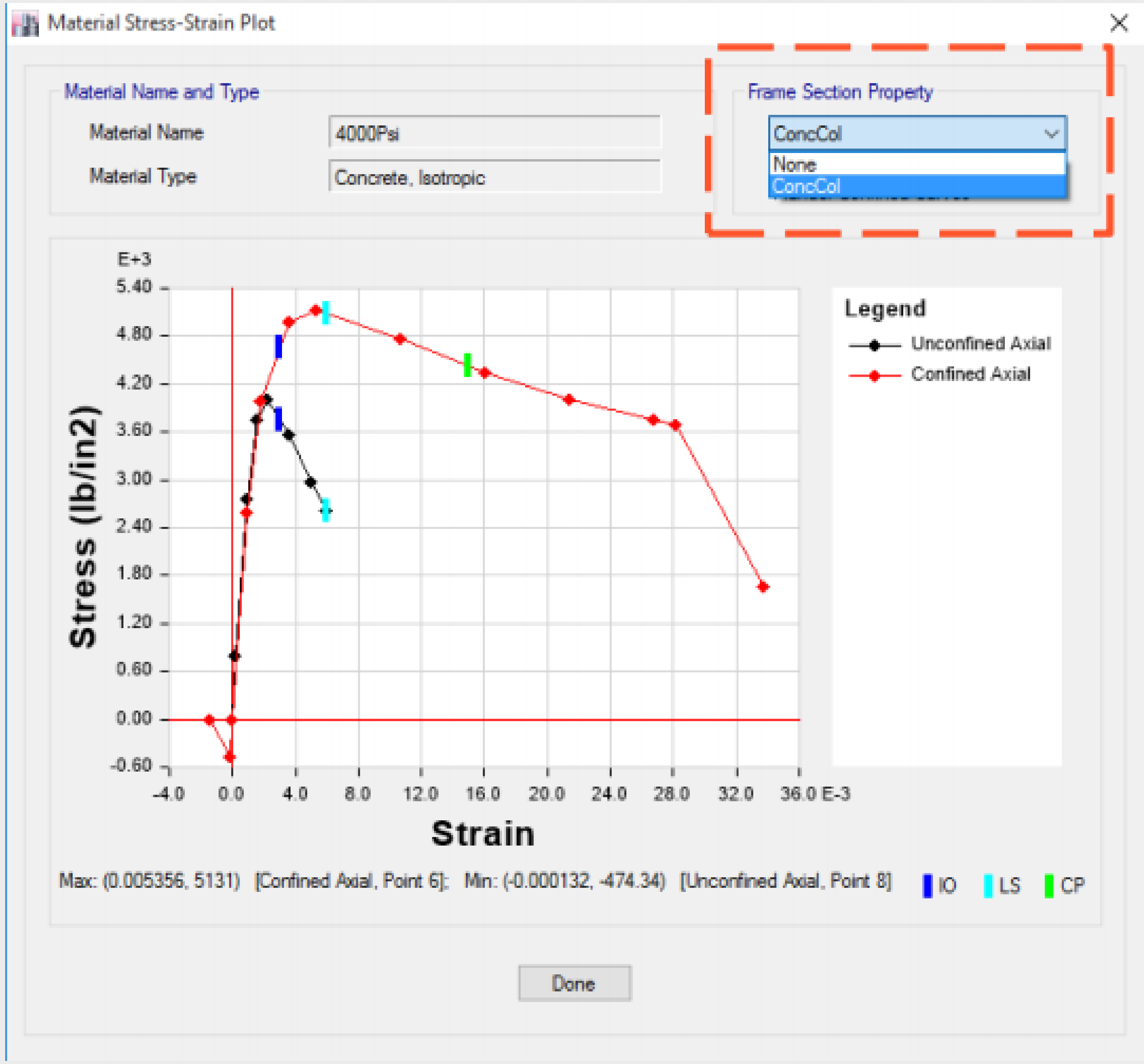
FEMA PMM HINGE



Performance Based Design of Buildings

Mander Confined and Unconfined Concrete Model in Tension and Compression

A shear stress-strain curve is computed internally from the direct stress-strain curve. The assumption is made that shearing behavior can be computed from tensile and compressive behavior acting at 45° to the material axes using Mohr's circle in the plane.



Performance Based Design of Buildings

STRENGTH vs. DEFORMATION

ELASTIC STRENGTH DESIGN - KEY STEPS

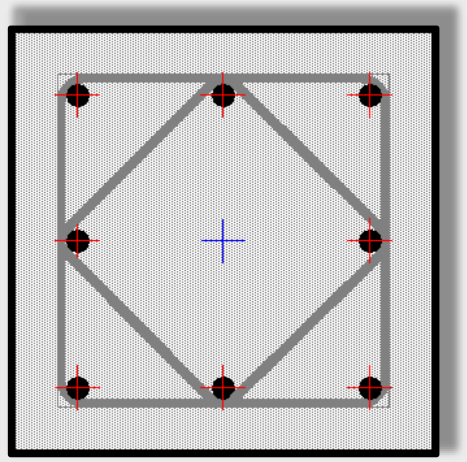
CHOSE DESIGN CODE AND
EARTHQUAKE LOADS DESIGN CHECK
PARAMETERS STRESS/BEAM MOMENT
GET ALLOWABLE STRESSES/ULTIMATE–
PHI FACTORS CALCULATE STRESSES –
LOAD FACTORS (ST RS TH) CALCULATE
STRESS RATIOS

INELASTIC DEFORMATION BASED DESIGN -- KEY STEPS

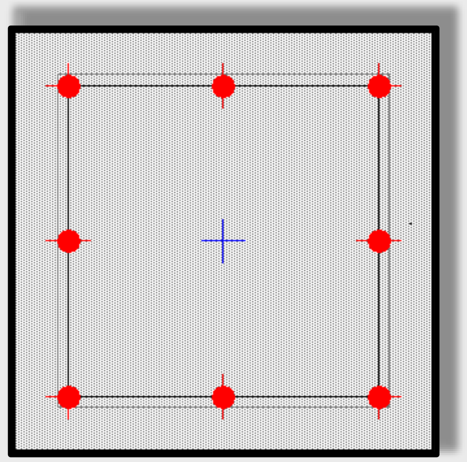
CHOSE PERFORMANCE LEVEL AND DESIGN
LOADS – ASCE 41 DEMAND CAPACITY MEASURES
– DRIFT/HINGE ROTATION/SHEAR GET
DEFORMATION AND FORCE CAPACITIES
CALCULATE DEFORMATION AND FORCE
DEMANDS (RS OR TH) CALCULATE D/C RATIOS
– LIMIT STATES

Performance Based Design of Buildings

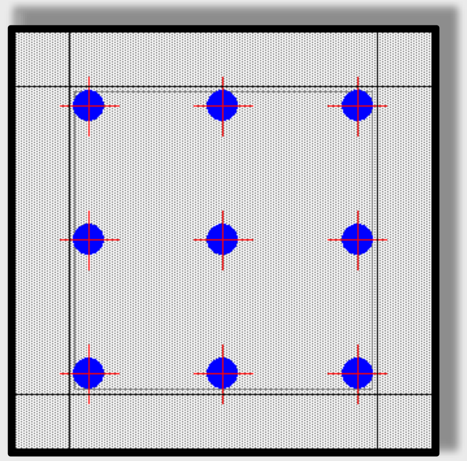
CONCRETE COLUMN FIBER HINGE MODEL



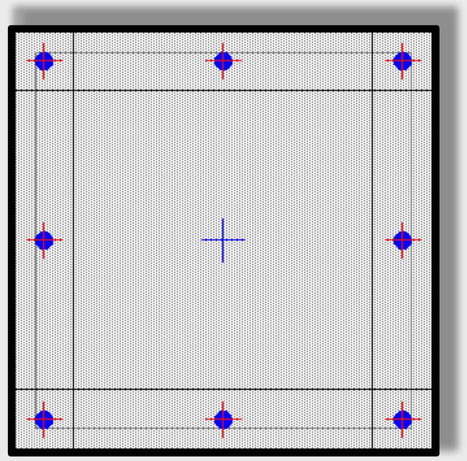
Reinforced Concrete Column



Steel Rebar Fibers



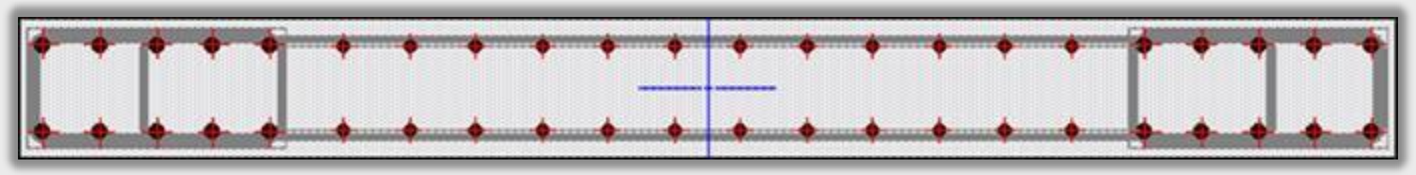
Confined Concrete Fibers



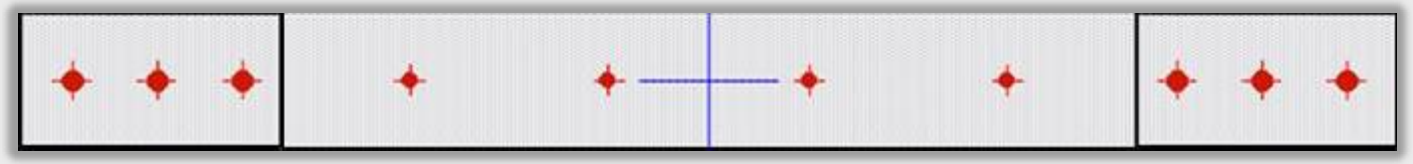
Unconfined Concrete Fibers

Performance Based Design of Buildings

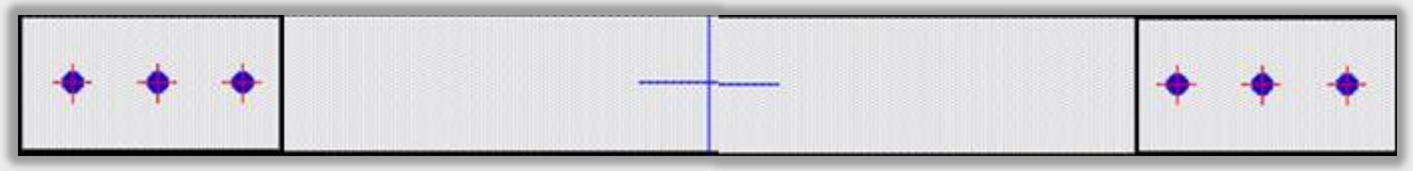
SHEAR WALL FIBER HINGE MODEL



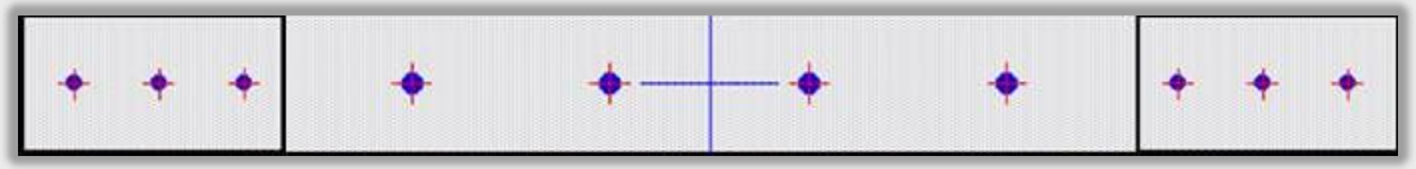
Reinforcement Layout



Steel Fibers



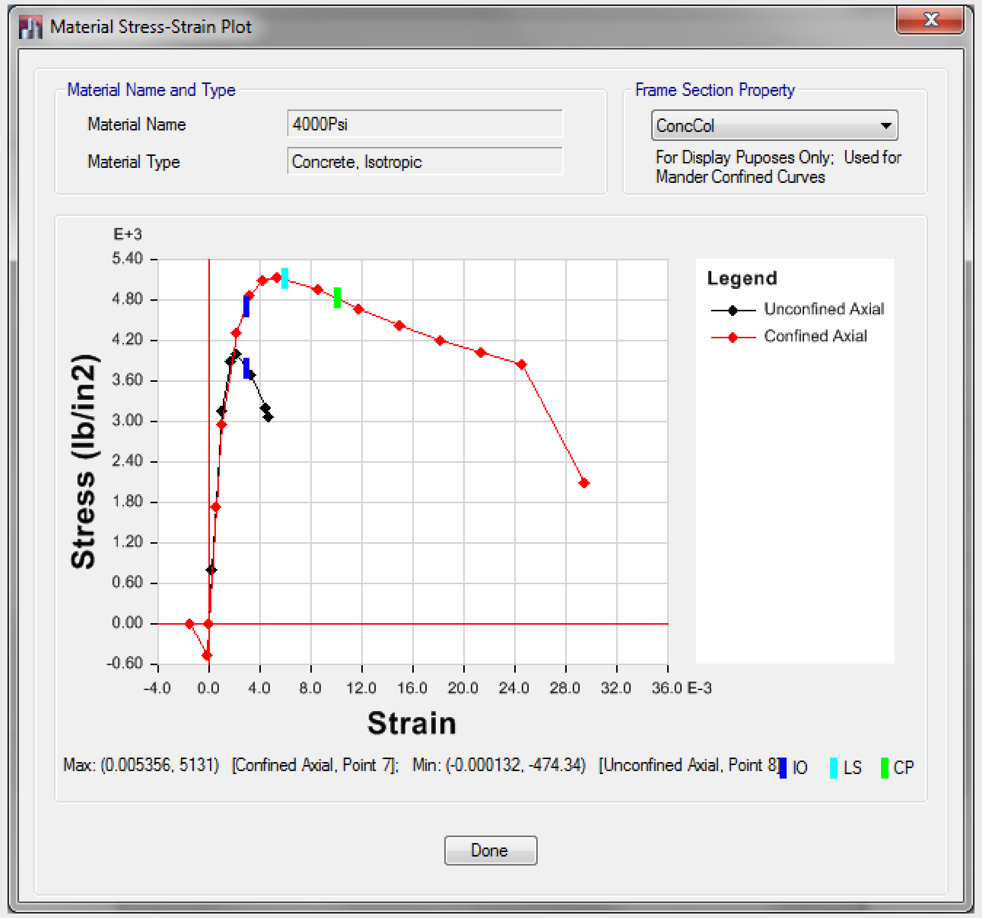
Confined Concrete Fibers



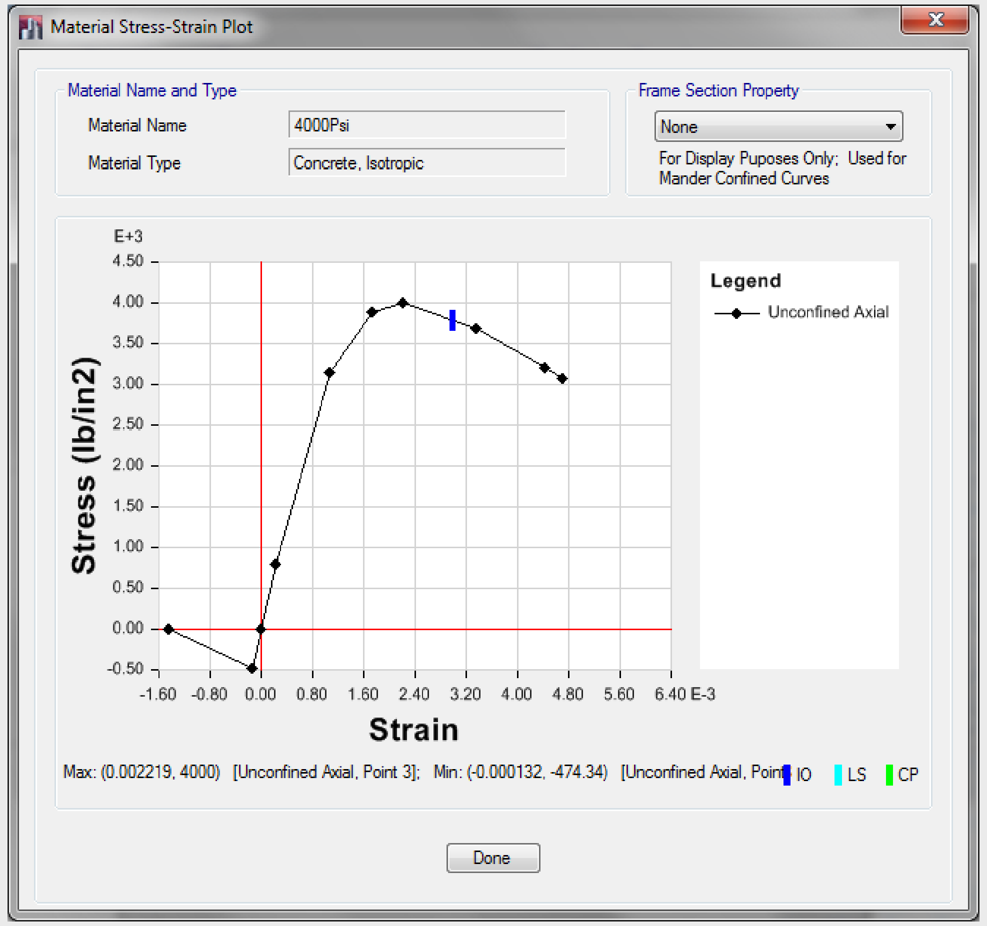
Unconfined Concrete Fibers

Performance Based Design of Buildings

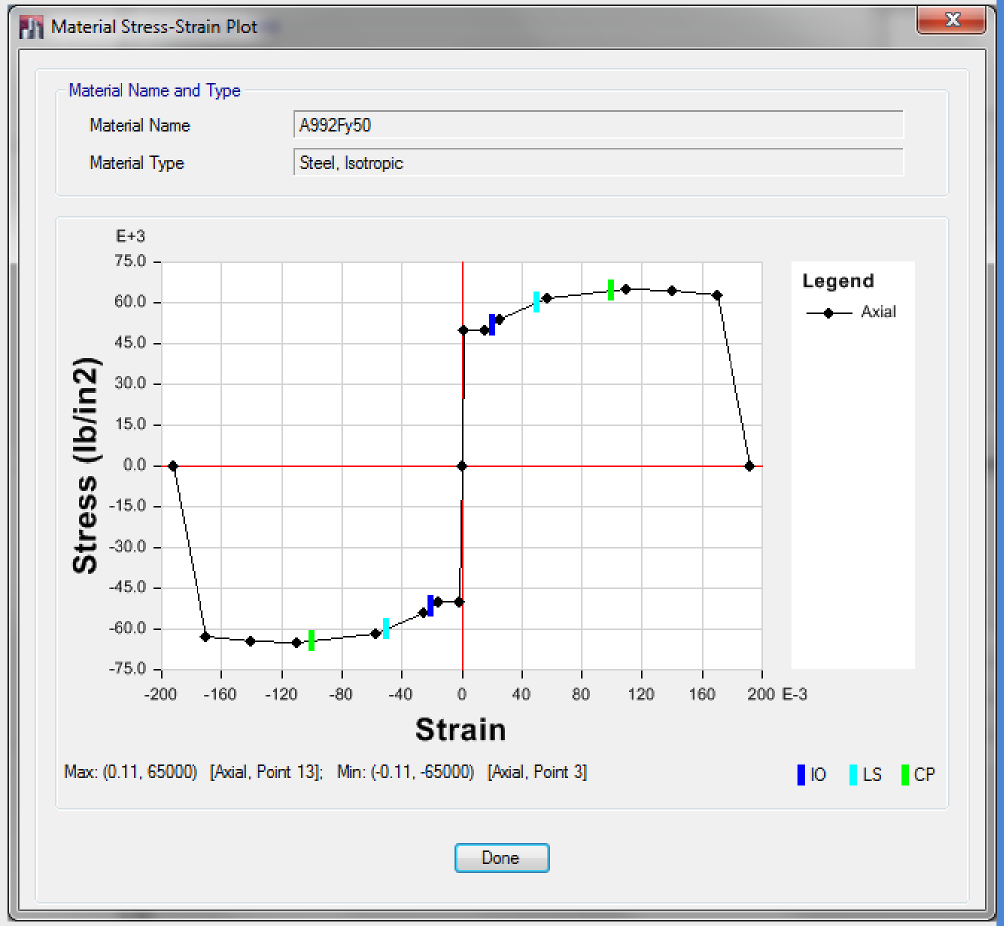
MATERIAL STRESS-STRAIN CURVES



Unconfined and Confined Concrete (Compared)



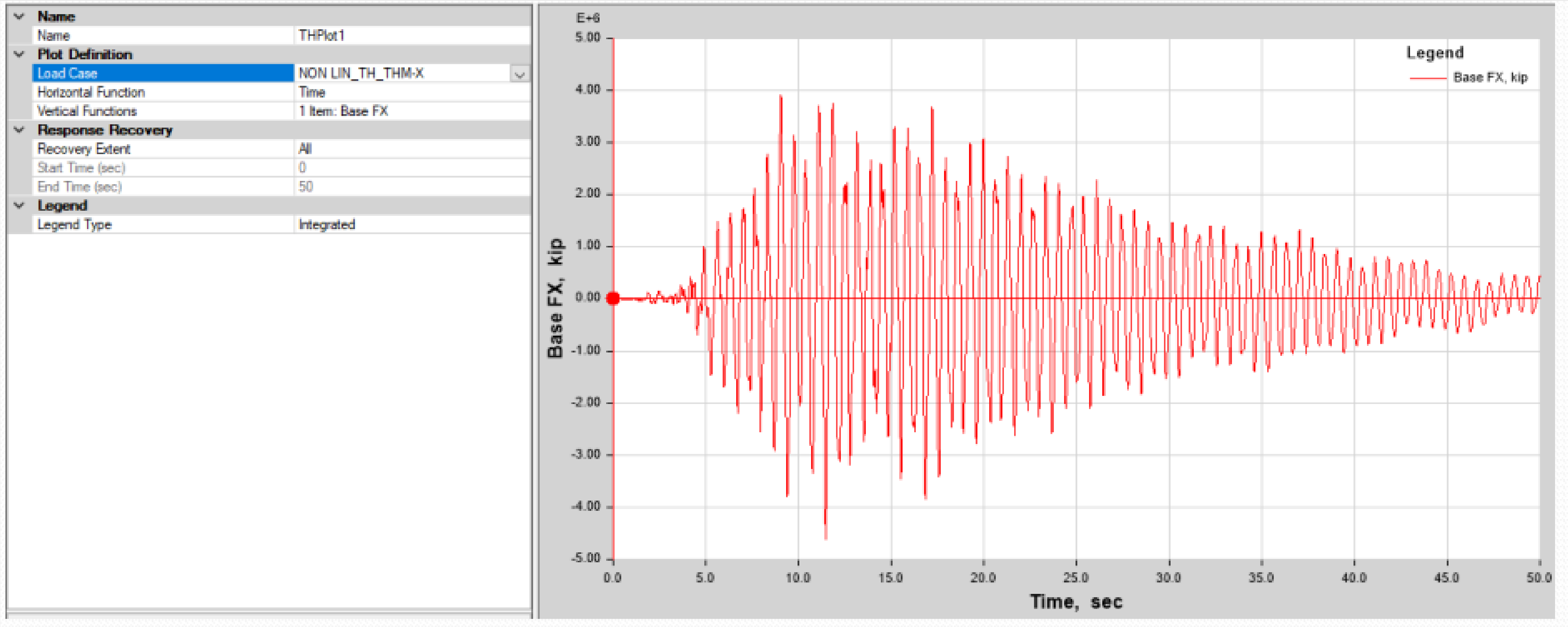
Confined Concrete



Steel

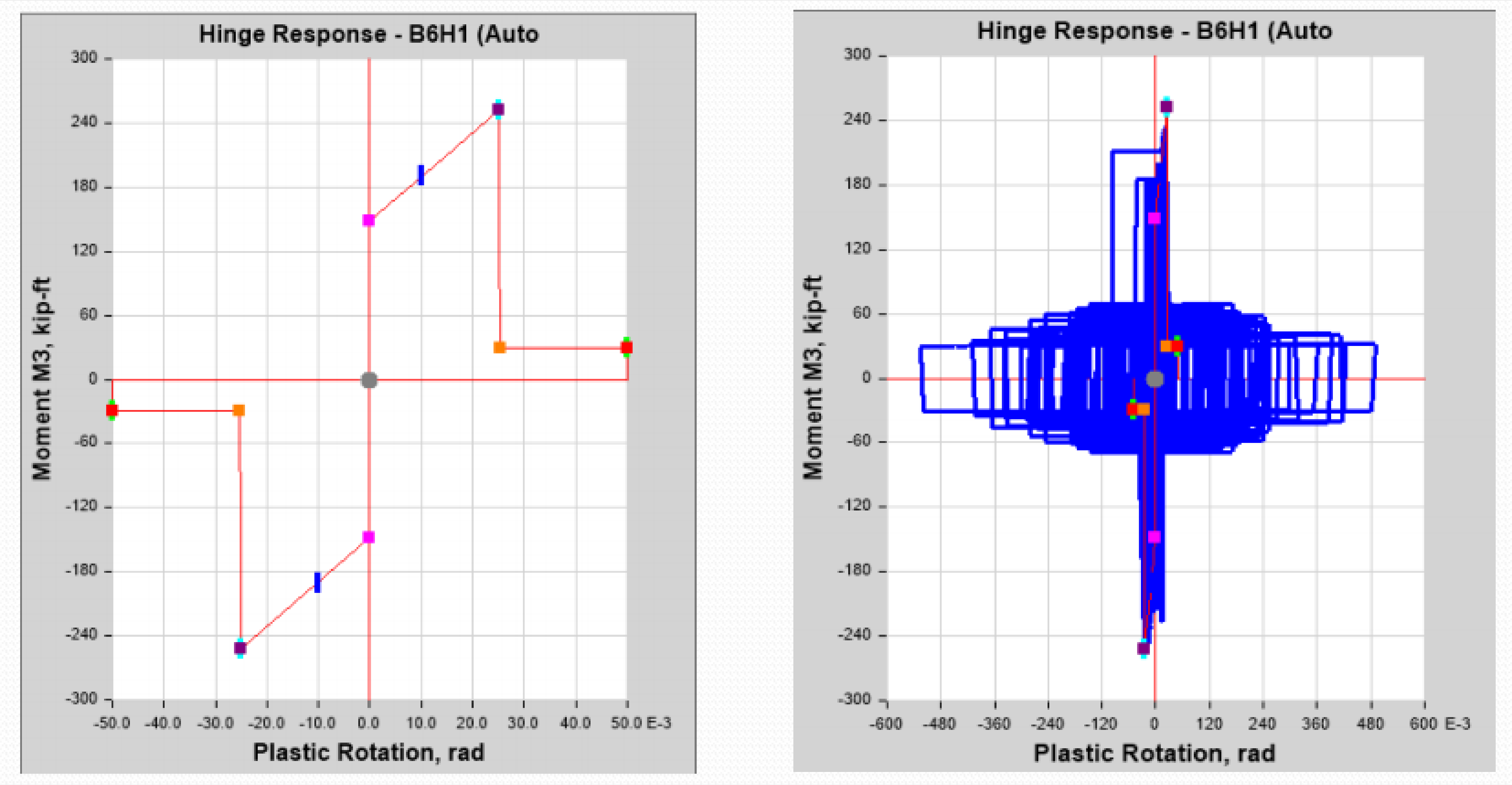
Performance Based Design of Buildings

Results



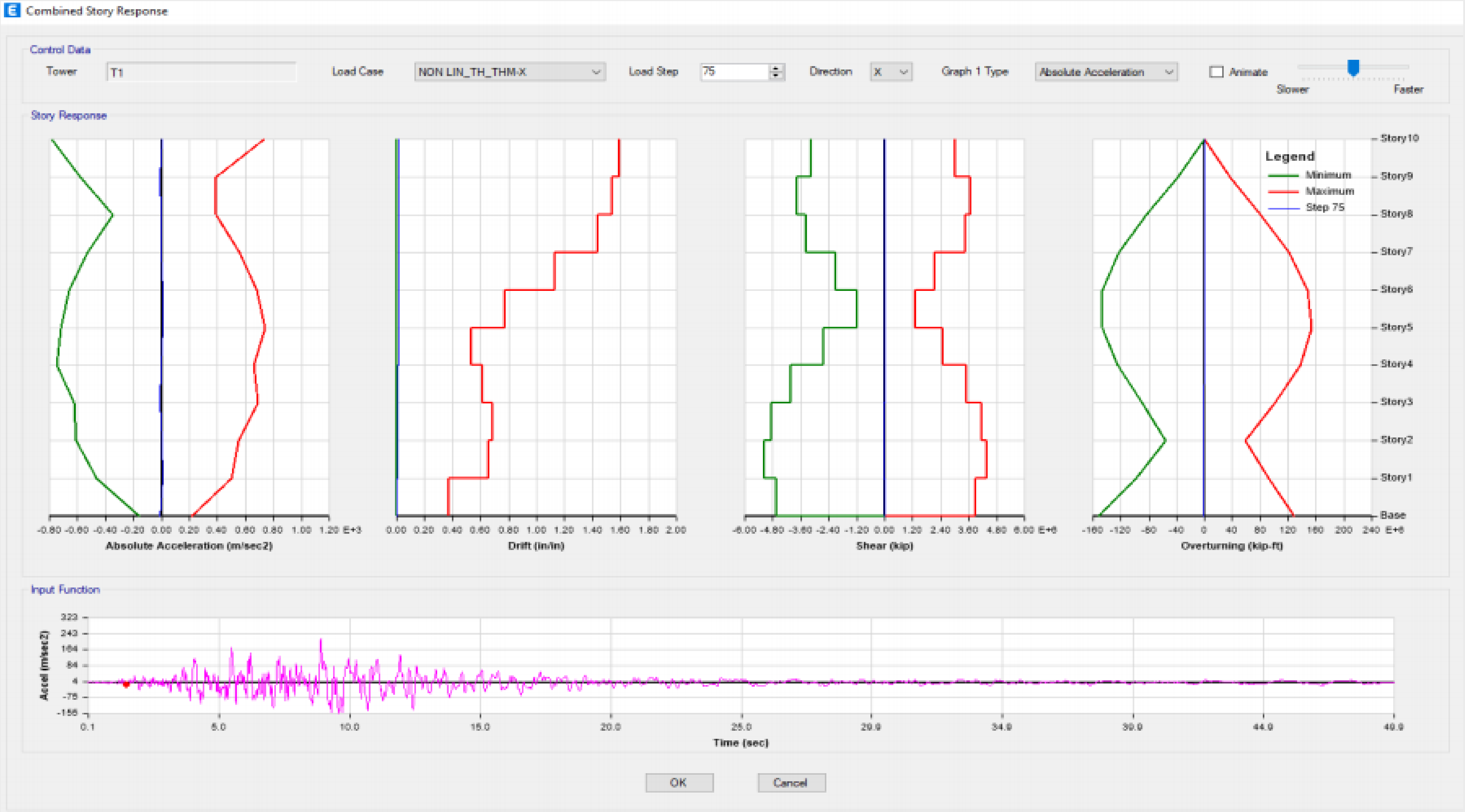
Performance Based Design of Buildings

Results



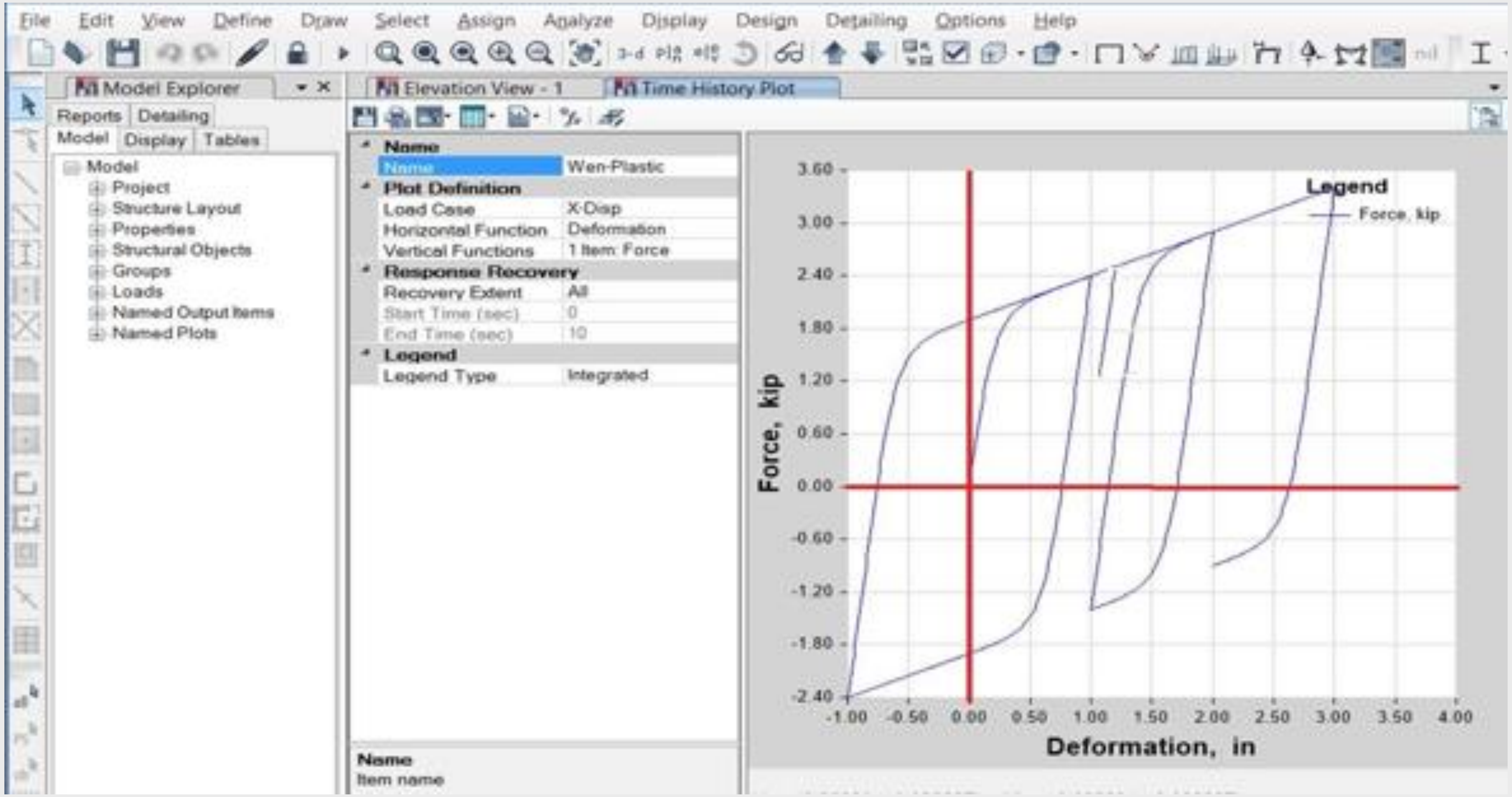
Performance Based Design of Buildings

Results



Performance Based Design of Buildings

HYSTERETIC BEHAVIOR

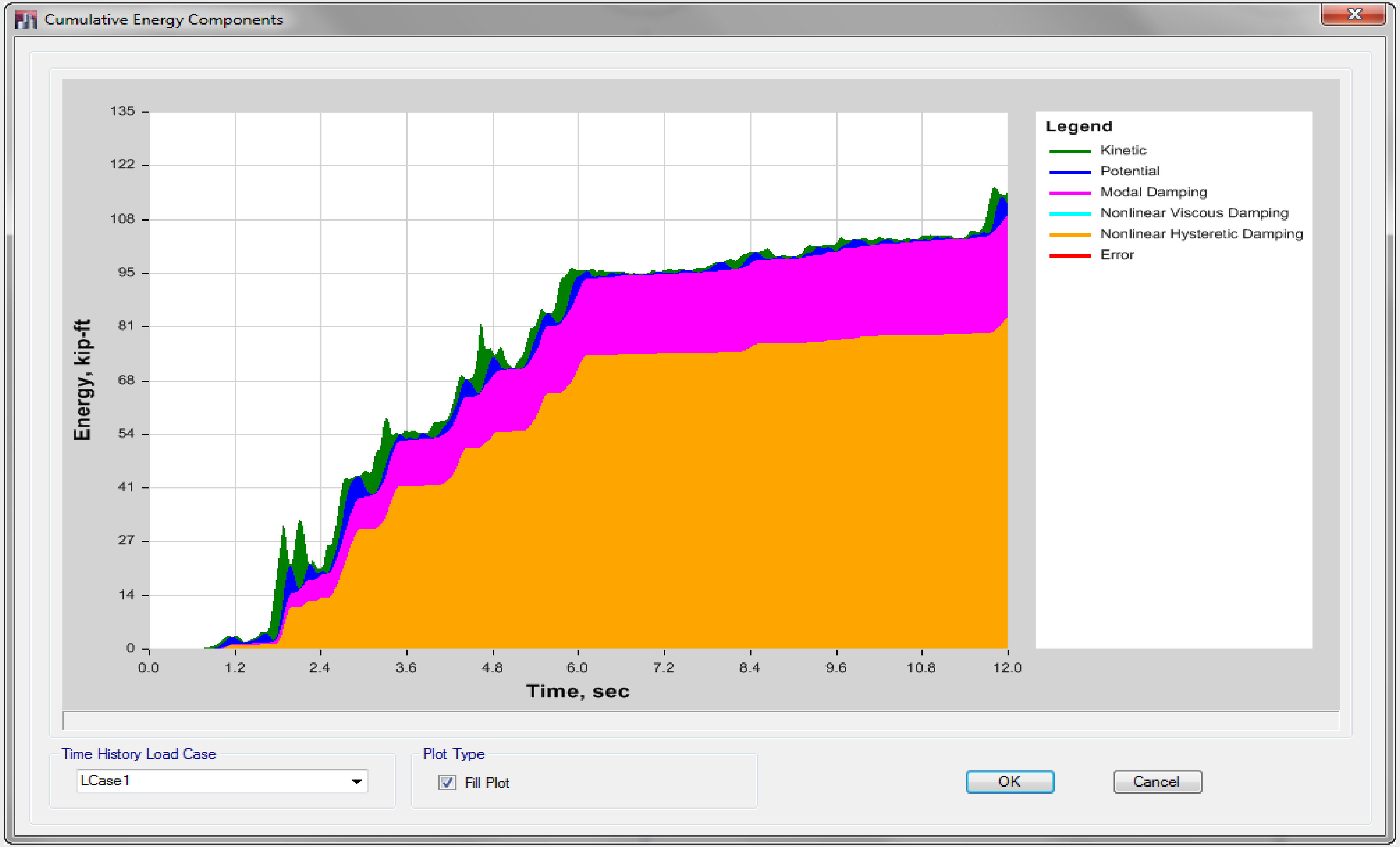


Performance Based Design of Buildings

Energy diagram

Performance Based Design of Buildings

ENERGY DIAGRAM w/ HYSTERETIC



Performance Based Design of Buildings

Conclusion

The basic concept of performance based seismic design is to provide engineers with the capability to design buildings that have a predictable and reliable performance in earthquakes.

Thus the performance-based seismic design is a process that permits design of new buildings or upgrade of existing buildings with a realistic understanding of the risk of life, occupancy and economic loss that may occur as a result of future earthquakes.

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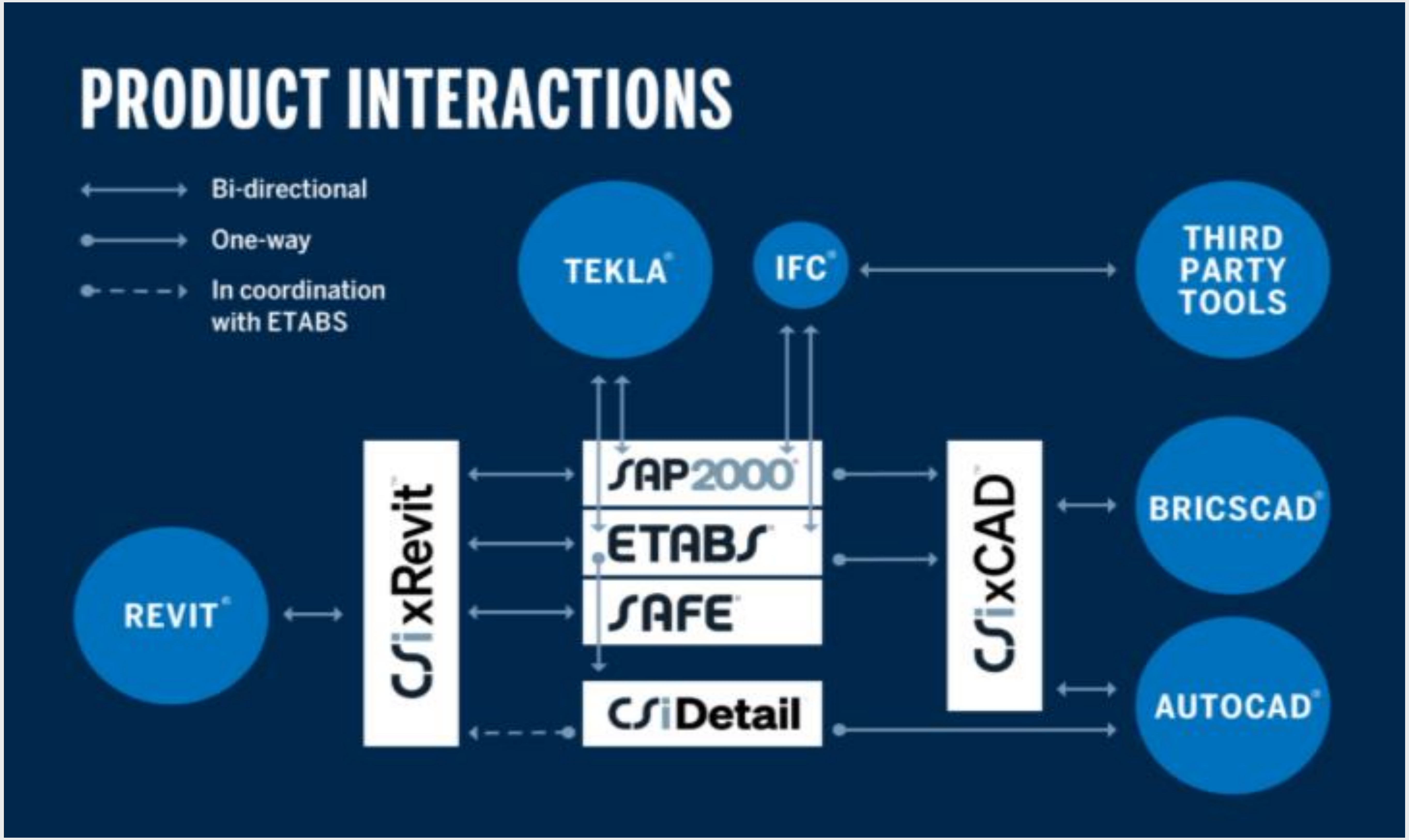
PRODUCT INTERACTIONS

CSI Detail offers numerous product interactions with various CSI software and third-party products.



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