Performance Based Design of Cold-formed Steel Frames

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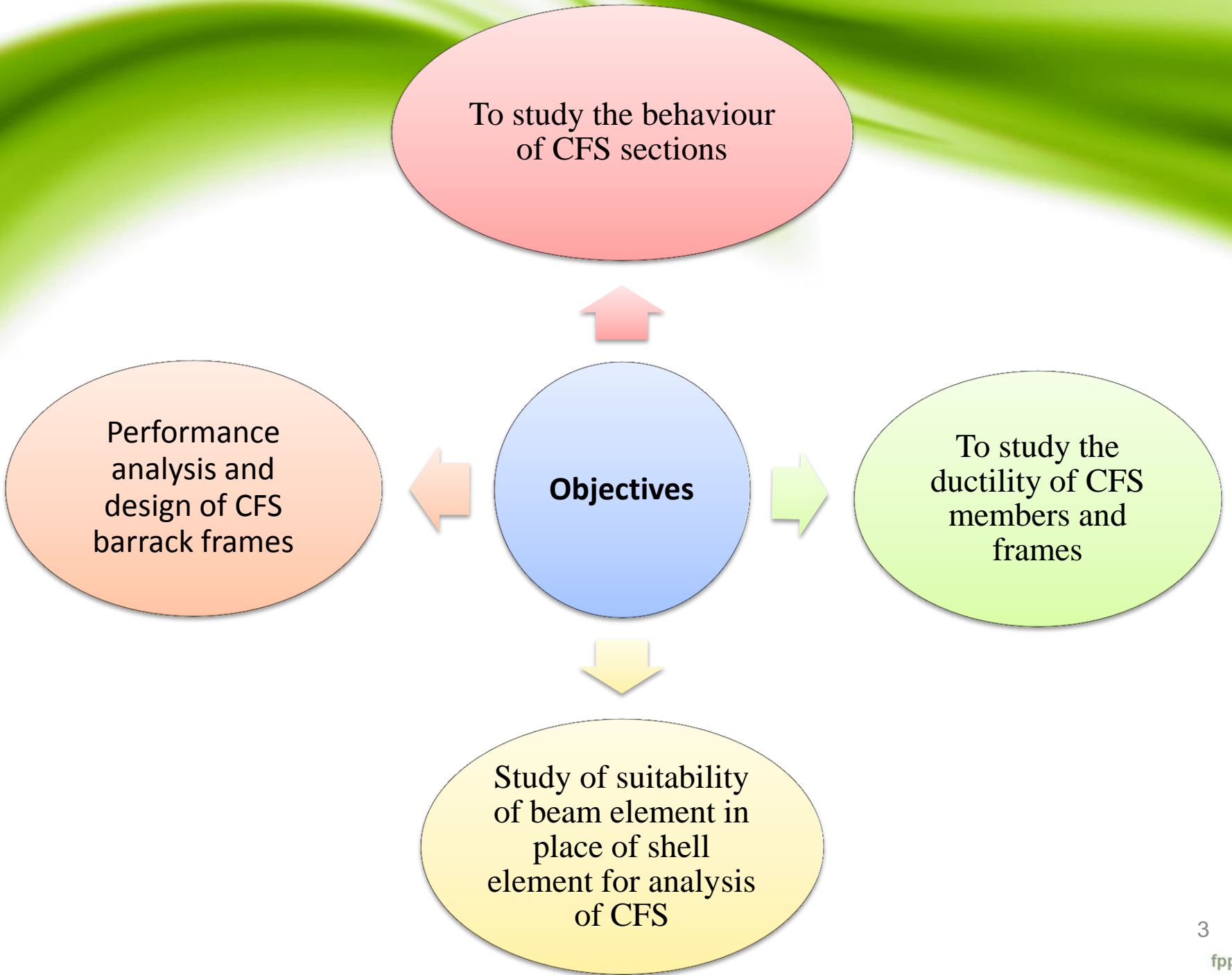
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Overview

❑ Introduction

❑ Objectives of the study

❑ Performance based design

❑ Behaviour of cold-formed steel

❑ PBD of CFS Barrack frames

❑ Conclusions

❑ Future Scope

To study the behaviour of CFS sections

Performance analysis and design of CFS barrack frames

Objectives

To study the ductility of CFS members and frames

Study of suitability of beam element in place of shell element for analysis of CFS

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Introduction

- CFS frame is semi-rigidly connected frame.
- Becoming famous structural system in European countries, Australia, Canada, US and some Asian countries for low residential constructions.
- The reasons for growing applications are:

  i) high strength to weight ratio

  ii) high stiffness

  iii) easy and fast erection and installation
- Being light in weight, it can replace timber construction in EQ regions.

Fig: Six floors CFS framed structure in United States

- But there is still a refrain among construction engineers and designers to use it at large scale.

- CFS sections are thin-walled sections which are very prone to sectional buckling – local buckling, distortional buckling, lateral-torsional buckling.

- Thin-walled sections being slender and semi-rigidly connected at joints gives very low ductility.

- So, there is need for performance evaluation of CFS structures at global and local level.

- Till now, many researches have been done to evaluate the performance of CFS and to improve its ductility.

Performance Based Design

Performance Based Design

- Relatively new concept of design in civil engineering.

- Goal- To produce structures that have predictable seismic performance under multiple levels of earthquake intensity.

- In order to do so, it is important that the behaviour of the structures is targeted in advance, both in elastic as well as the inelastic ranges of deformation.

- The deformation of member strength hierarchy, failure mechanism and structure strength become the primary elements of a performance-based design procedure.

- Design criteria are expressed in terms of performance objectives - lateral deflections, inter-storey drifts, element ductility.

- So, PBD is defined as a selection of design criteria and structural system such that at the specified levels of ground motion and with defined levels of reliability, the structure will not be damaged beyond certain limit states or other useful limits.
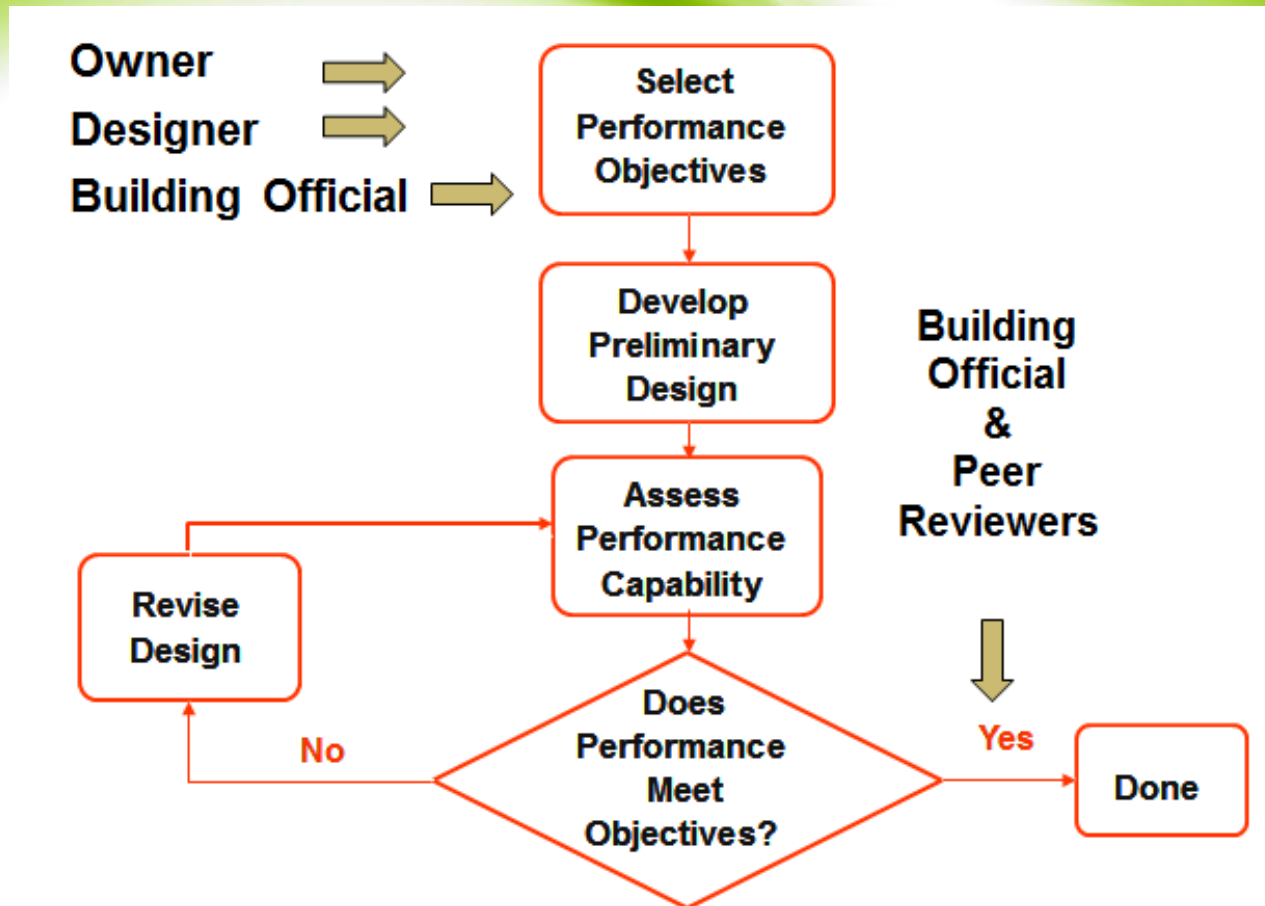
Fig: Algorithm for PBD

- So, first thing to be decided is performance objective.

- It comprises two parts, a design hazard level and a design performance level.

- The design hazard level is a quantification of the severity and character of ground shaking that a structure has to resist.

- The design performance is a quantification of the permissible types and distribution of damage to the structure, given that design hazard level is experienced.

Performance Objectives
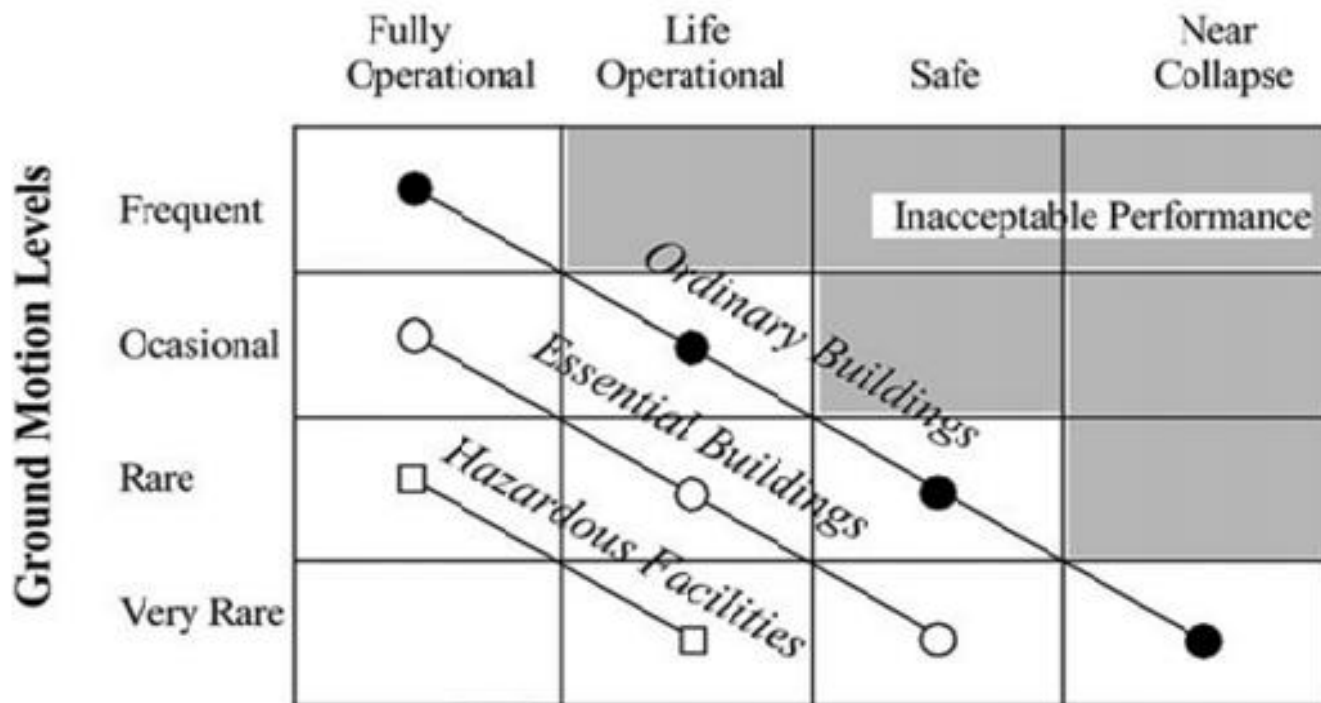
- Next step is the design process which we can do using code based procedure.

- Next very important step is performance evaluation.

- One of the methods proposed by ATC-40: Capacity Spectrum Method

Capacity Curve

Capacity Spectrum


Standard Format (Sa vs T)

ADRS Format (Sa vs Sd)
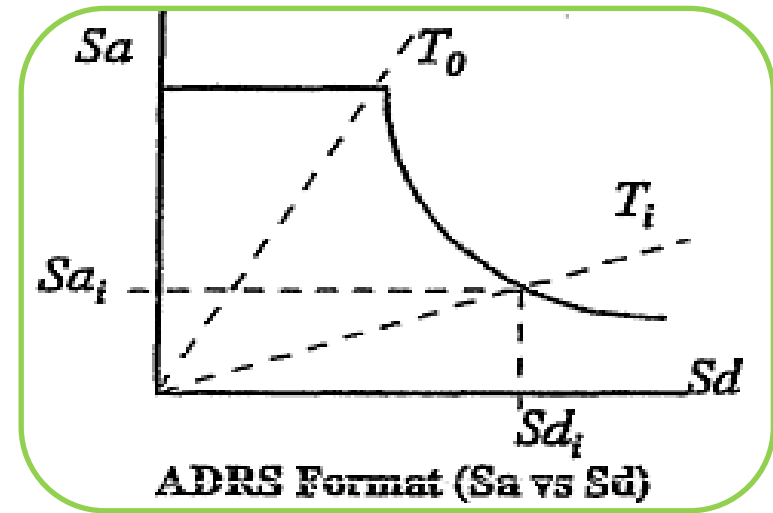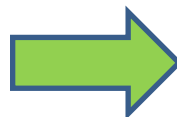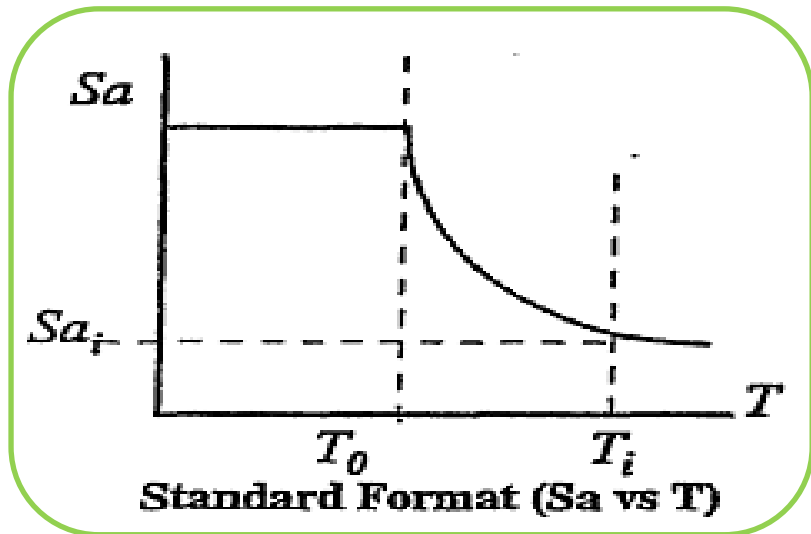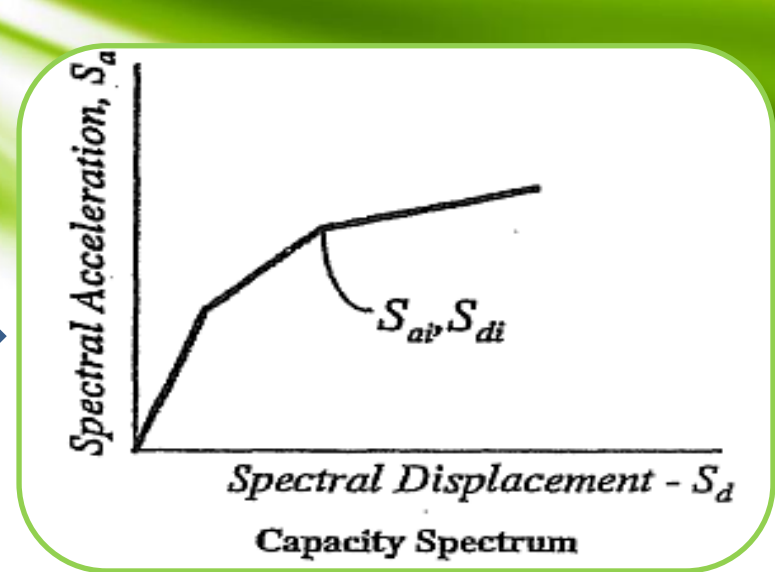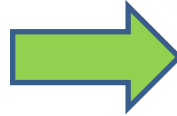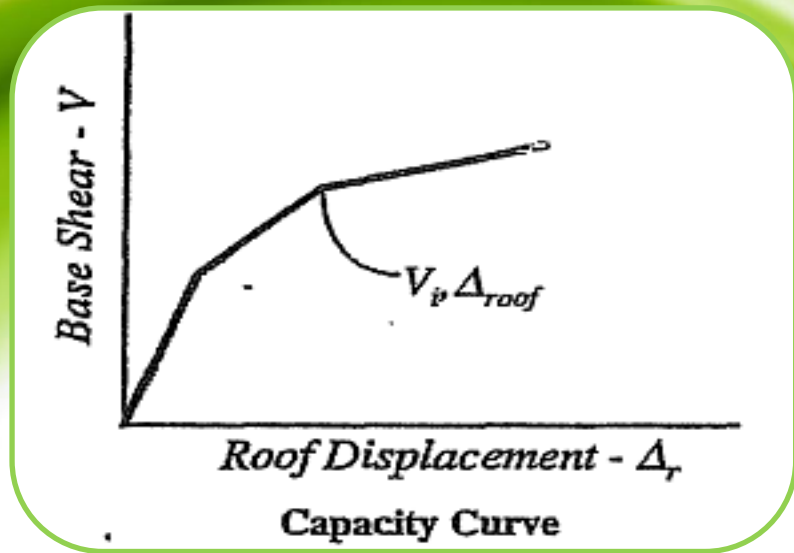
$$S_a = \frac{V/W}{\alpha_1}, \quad S_d = \frac{\Delta_{roof}}{PF1\emptyset_{roof1}}$$

$$PF_1 = \frac{\sum_{i=1}^{N} w_i\emptyset_{i1}/g}{\sum_{i=1}^{N}(w_i\emptyset_{i1}{}^2)/g}$$

$$\alpha_1 = \frac{(\sum_{i=1}^{N} w_i\emptyset_{i1}/g)^2}{(\sum_{i=1}^{N}\frac{w_i}{g})(\sum_{i=1}^{N}(w_i\emptyset_{i1}{}^2)/g)}$$

- Now we have obtained capacity of the spectrum in ADRS format.

- Next thing we require is the demand.

- Demand is obtained from the response spectra.

- Demand curve is obtained from traditional response spectrum by incorporating effective damping from hysteresis curve.

- If the reduced demand spectrum intersects the capacity spectrum at initially assumed displacement, then it is the performance point.

$$S_d = \frac{1}{4\pi^2} S_a T^2$$
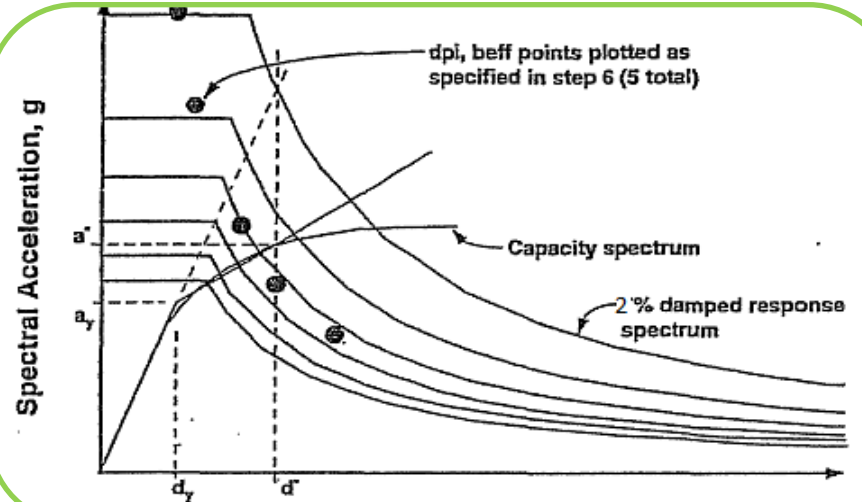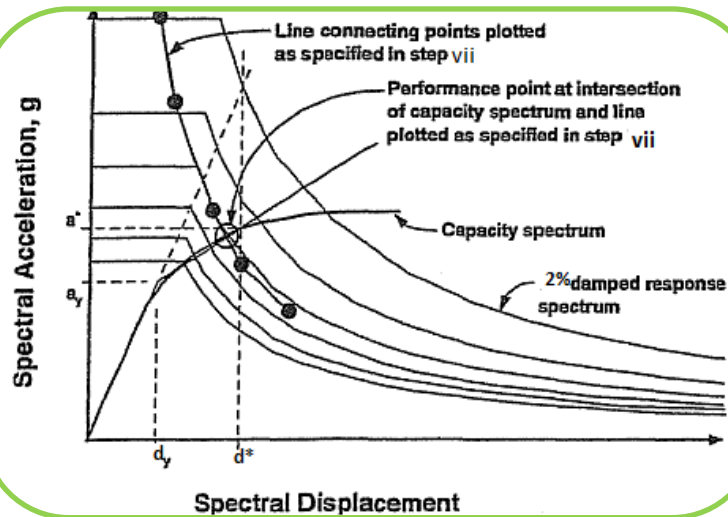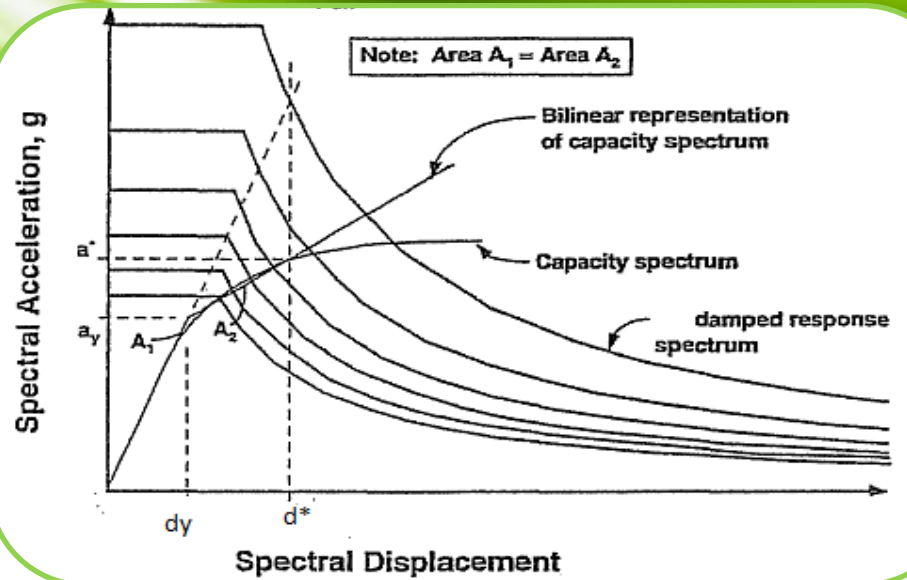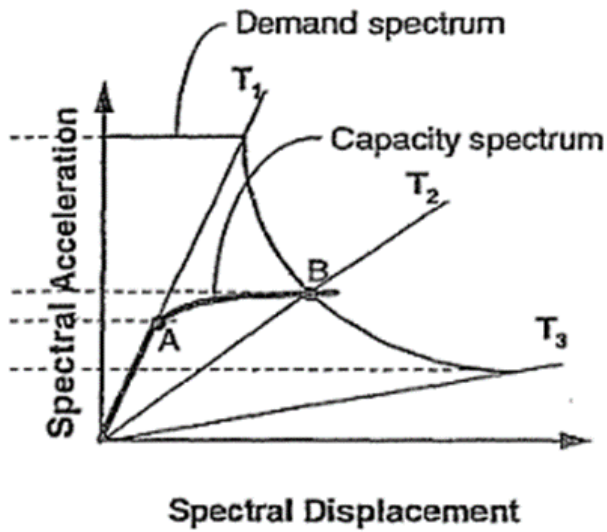
Behaviour and Performance of CFS

- CFS sections are fabricated from thin sheets by cold-rolling or press-braking.

- The mechanical properties of cold-formed sections are sometimes substantially different from those of the steel sheet, strip, plate, or bar before forming.

- This is because the cold-forming operation increases the yield point and the tensile strength and at the same time decreases the ductility.
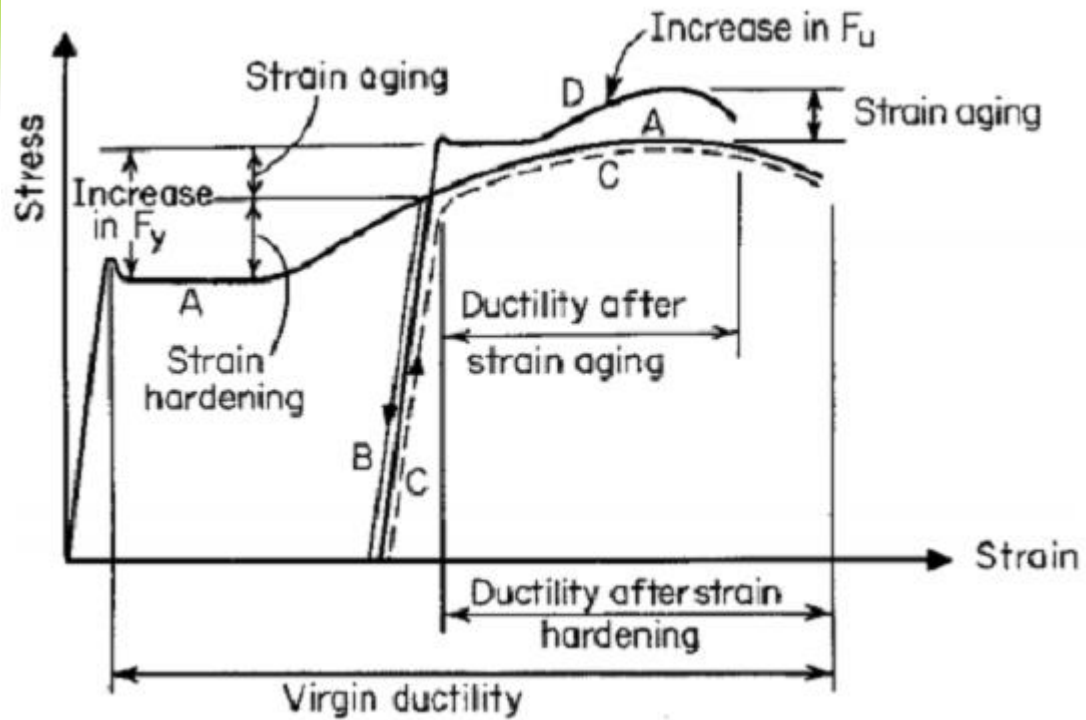
Fig: Stress-strain curve of CFS in tension

- Cold formed steel elements are either stiffened or unstiffened.
- An element which is supported by webs along both its longitudinal edges is called a stiffened element.
- An unstiffened element is one, which is supported along one longitudinal edge only with the other parallel edge being free to displace.

Stiffened element

Unstiffened element
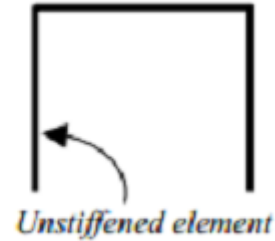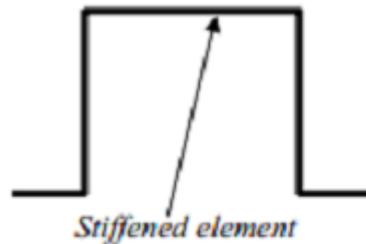
Post-buckling behaviour

- Two-dimensional compressed plate under different edge conditions will not fail like the one-dimensional members such as columns when the theoretical critical local buckling stress is reached.

- The plate will continue to carry additional load by means of the redistribution of stress in the compressive elements after local buckling occurs.

- This is a well-known phenomenon called post-buckling strength of plates.
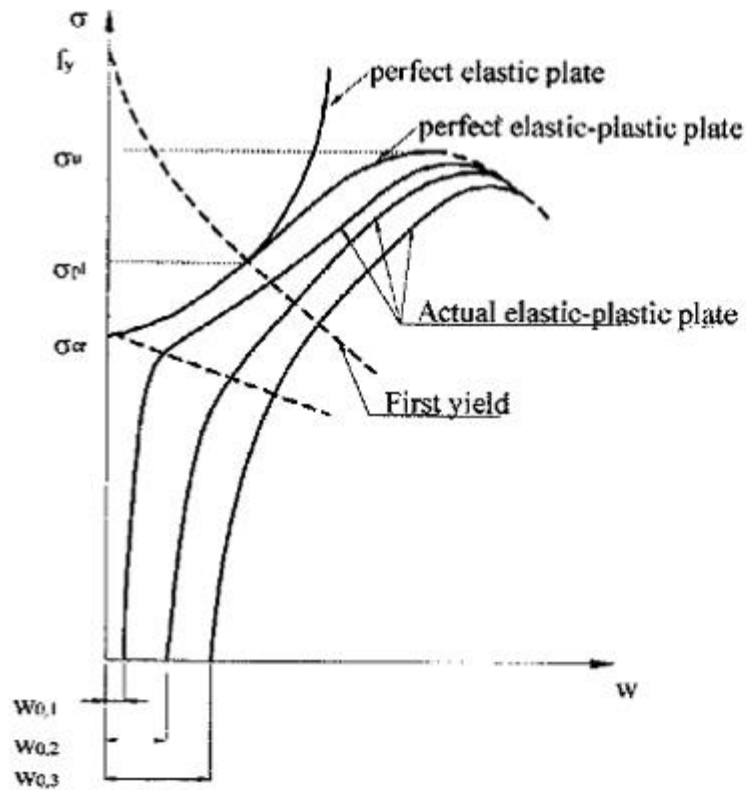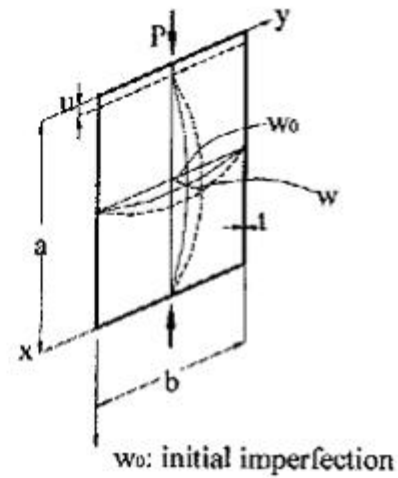
Post-buckling behaviour of CFS

General buckling modes

Different buckling modes in column

Different buckling modes in beam

(a) Buckled shape
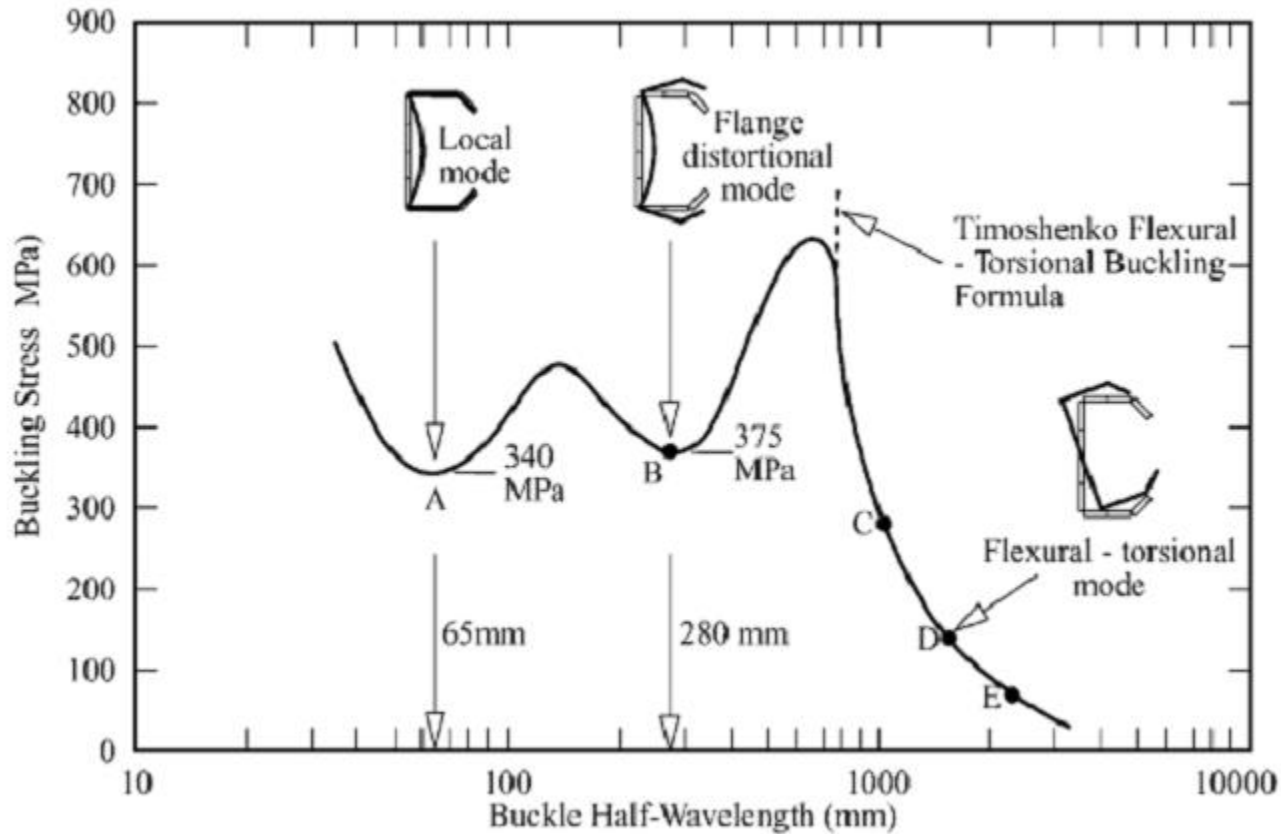
(b) Stress distribution

(c) Effective width

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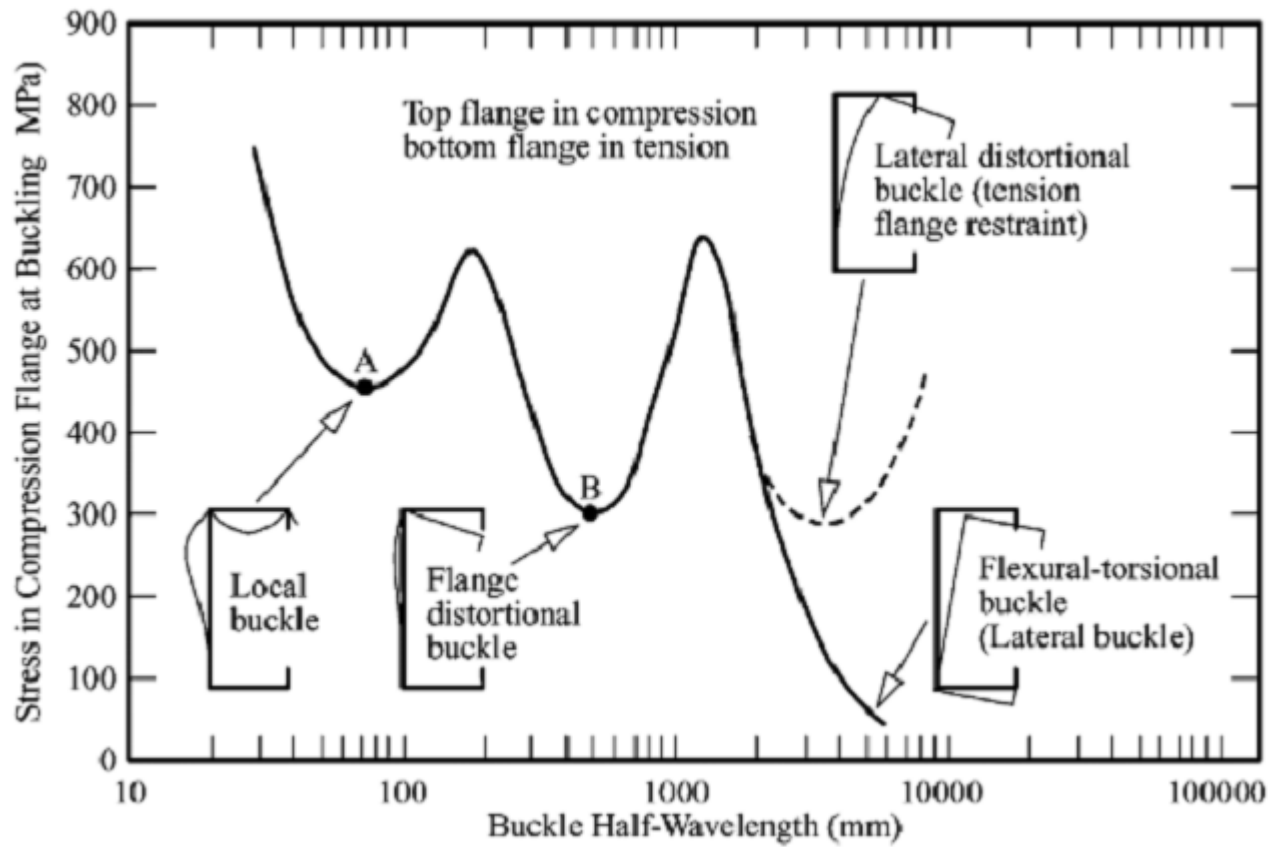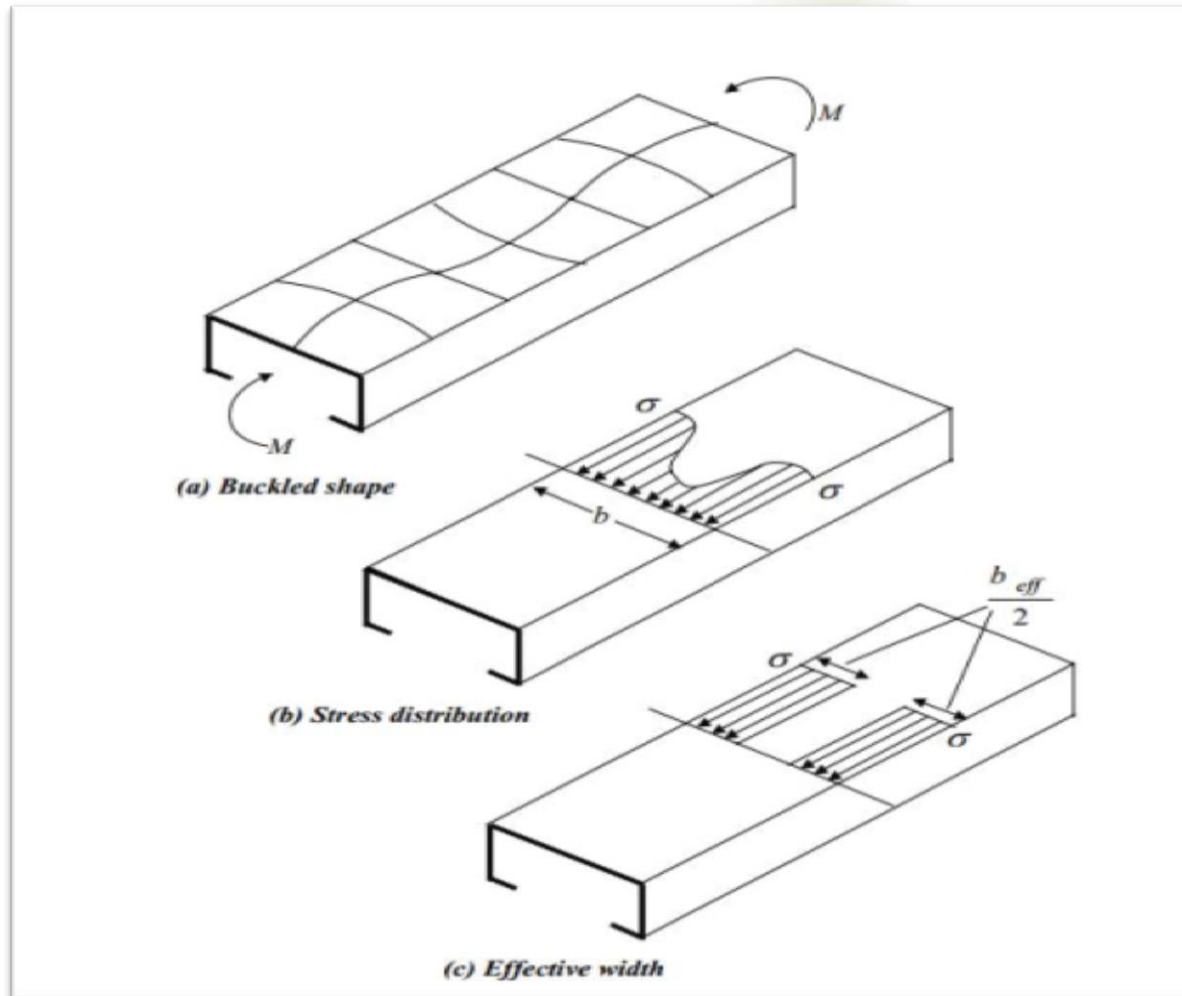
Seismic performance and ductility

- CFS are considered to be non-ductile and hence plastic design is not allowed.

- Consequence – R = 1; low dissipative, elastic design

- No design and specification code for CFS has discussed about its seismic performance.

- AISI and AS/NZS codes give specifications for cyclic and fatigue loads only.

- EN code keep CFS under category of low dissipative structures.

Improving performance of CFS

- Researchers have opined that there is a need to develop CFS frames in the same way as for hot-rolled steel.

- They also suggested that in earthquake resistant design, the required ductility of MRFs is mainly provided by the beams, while columns and connection elements are expected to remain elastic.

Beam sections
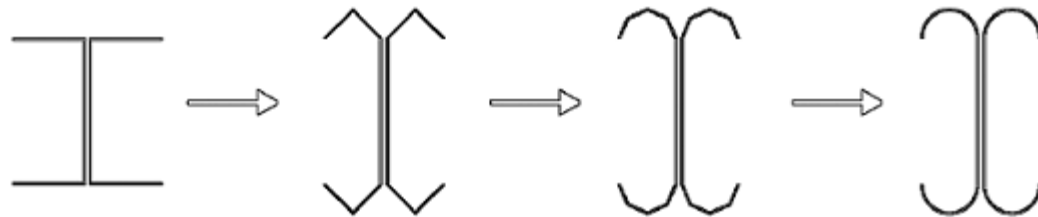
- The bent elements could support each other by producing in-plane stiffness when out-of plane deformations (local buckling) are initiated.

- This arching action can delay local buckling. In addition to the increased buckling strength, curved flange beam sections have the following advantages:

a) Higher ductility than flat flange sections due to reduction in cold work effects during the forming process; sharper corners result in higher cold work effects.

b) Increased bending strength, both Mp (plastic moment) and My (yielding moment).

- To check this:
- Finite element modelling was done with following parameters:

a) Element type : S4R5 homogenous shell element

b) Material : Bilinear stress-strain behaviour with Fy = 240 MPa, Fu= 370 MPa, E = 210 GPa, Es=E/100 (second modulus after yielding), μ (poisson's ratio) = 0.3

All beams are simply supported and laterally braced.

Flat flange beam sections

Label	thickness	Web-height	Flange-width	Lip-length	Length
FFB1-1,2,3	1,2,3	200	100	25	2000
FFB2-1,2,3	1,2,3	270	135	30	2700

Table 5-2: Curved beam flange section

label	thickness	Web height	Flange radius	Flange-angle	length
CFB1-1,2,3	1,2,3	200	40	180	2000
CFB2-1,2,3	1,2,3	270	53	180	2700

All dimensions are in mm

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Results of non-linear buckling analysis:

- For flat flange beams, the predominant buckling mode was flange buckling.

- For the analysed curved flange beams, the predominant buckling mode was web buckling.
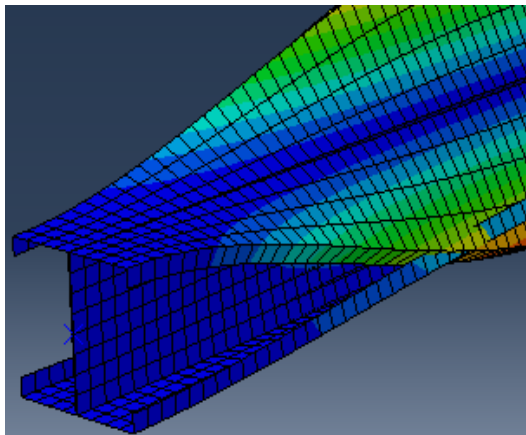
a) b)

Buckling modes in a) FFB -Flange buckling and b) CFB - Distortional and web-buckling

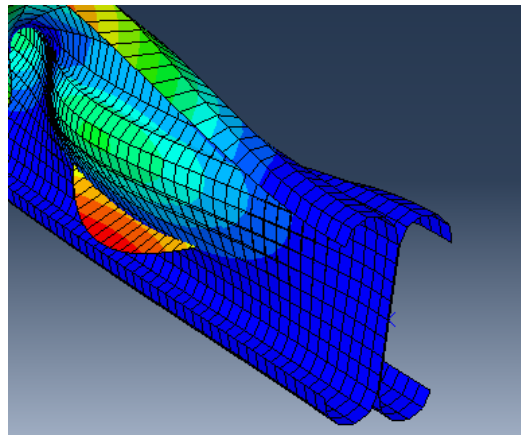Results of non-linear inelastic analysis:

- M_{cr1} (First eigen value-1st critical moment))/M_p value of flat beam is less than 1.

- Value of M_{cr1}/M_p for CFB sections is > 1.

- So, thicker CFB sections are advised to be used to avoid buckling before plasticisation.

- As can also be seen from graph that CFB sections provide more ductility than FFB.

Moment-deflection

Chart: Moment KNm (y-axis, 0 to 50) vs vertical_displacement (mm) (x-axis, 0 to 80).

Legend:
- CFB1-2
- FFB1-2
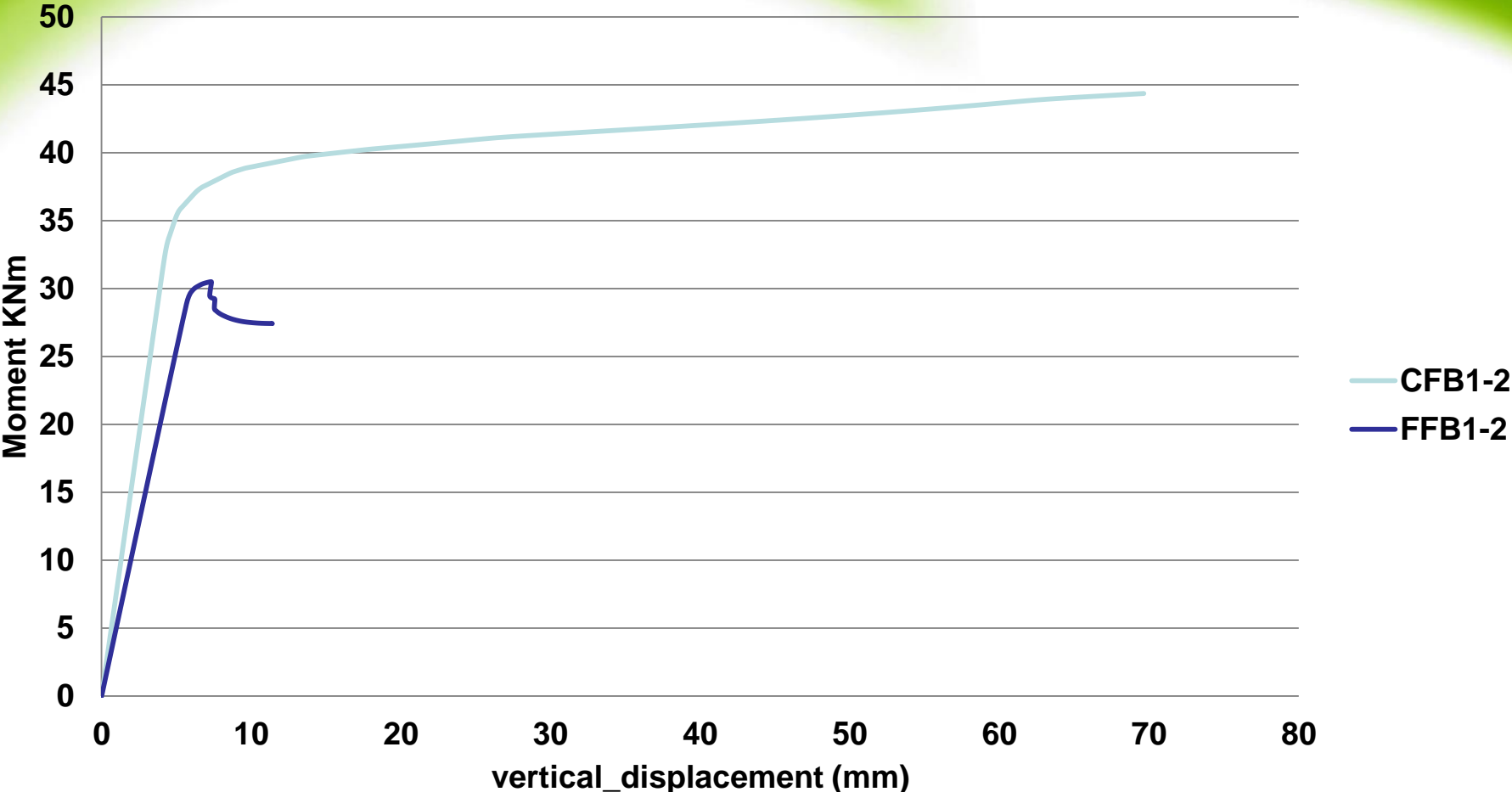
- <u>Validity of the result</u>: Result is matching with paper "Ductile moment-resisting frames using cold-formed steel sections: An analytical investigation" by A. Bagheri Sabbagh (2011) where they have proposed that using curved flange beam increases the ductility.

Column section:

- Local and flange-distortional buckling governs the behaviour of column.

- Local buckling occurs long before the global buckling and many times column not even achieves its global buckling

- Local and distortional buckling changes into plastic mechanism and column fails.

- Hence it is very essential to increase the load carrying capacity of column so that failure of column occurs at its overall buckling load or even higher.

label	Size (mm)	Thickness (mm)	Length (mm)	F_y (N/mm²)	F_u (N/mm²)
H-1	200	2	3000	240	370
H-2	200	2	3000	622	770
H-3	200	2	3000	707	827

H-1 is mild steel; H-2 and H-3 are high strength steel. All the members are simply supported.

Load vs axial deflection

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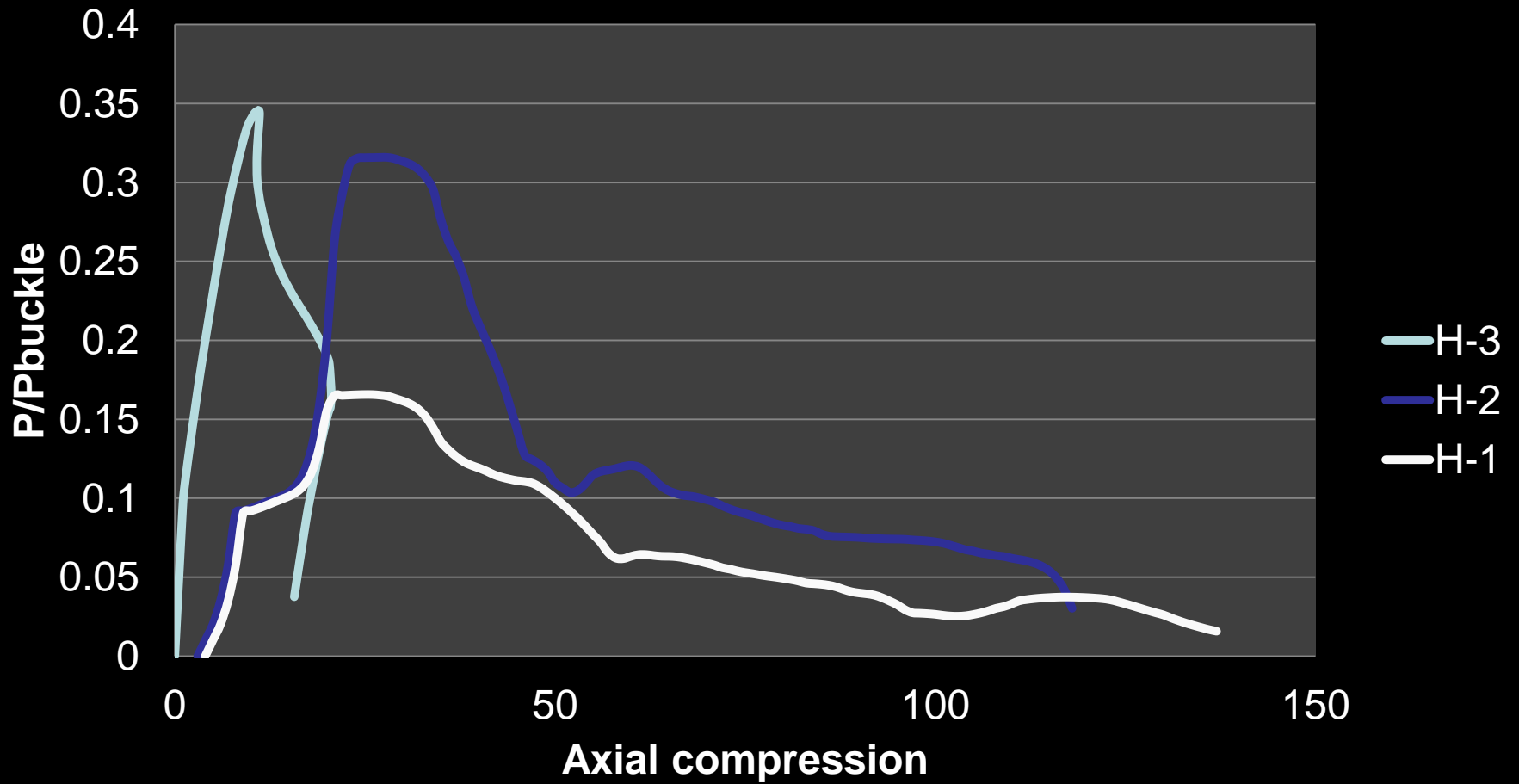
Results:

- i) High strength steel gives higher capacity .

- overall buckling load do not depend on strength of steel.

- ii) If D/t ratio of hollow section steel is further reduced, high capacity performance can be achieved.


Validity of the result: E. Ellobody and Ben Young (2005) in their paper proposed to use high strength column which gave more strength than mild steel and code specifications.

Column-floor connection study

- Beam-column connections of CFS frames were studied at IIT Bombay using cantilever method and double cantilever methods. (Talikoti)

- Here, column base connection is studied.

- The four node general purpose shell element was used to model the column and base plate while an eight node brick element was used to model the concrete floor.

- The material model used for the column and base plate was elastic-plastic with strain hardening $E = 2 \times 10^5$ N/mm^2, $F_y = 210$ N/mm^2, $F_u = 310$ N/mm^2, $E_{st} = E/100$, while the concrete is modelled using M20 concrete and $v = 0.2$.
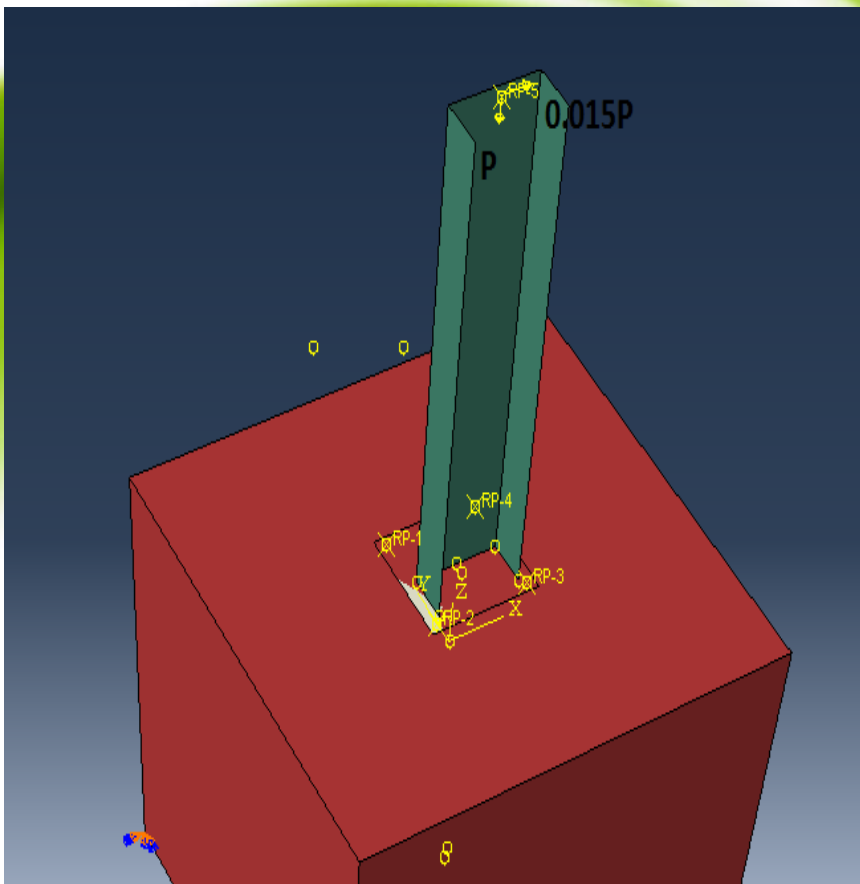
Moment-rotation curves for column base
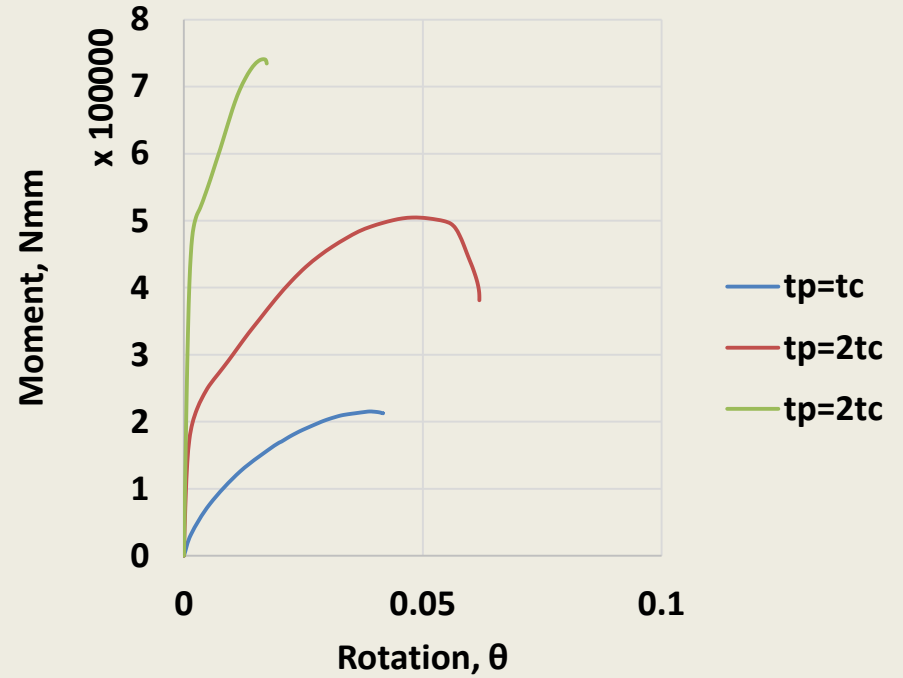
Column-floor connection study

t_p/t_c	Initial stiffness (Nmm/rad)
1	1.147×10^7
2	2.43×10^8
3	4.46×10^8

PBD of CFS Barrack frames

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- Barrack frames are constructed for army. US corps of engineers has developed a manual for design and construction of CFS barrack frames: TI-809-07.
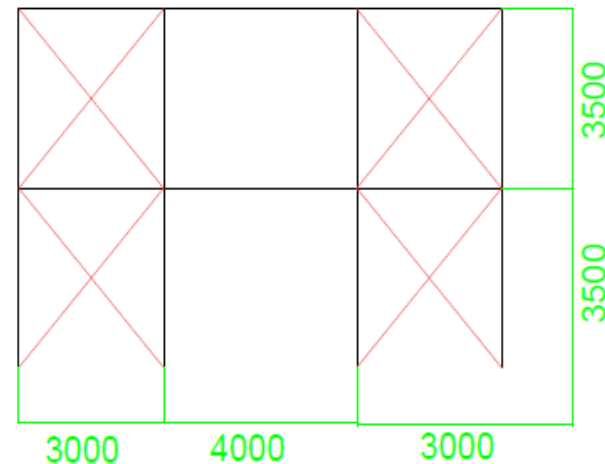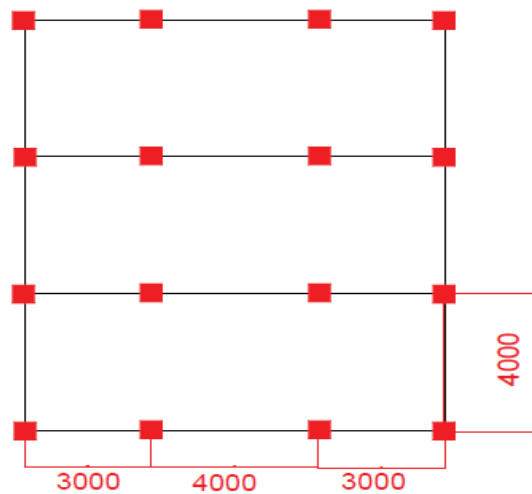
Plan and Elevation (all dimensions are in mm)

Analysis of Frames

- Modelling and analysis of frames using shell elements is time consuming.

- Modelling is done using open section beam element B32OS in ABAQUS which has seventh degree of freedom "warping".

- It has been verified in AISI research report RP 03-02 (2003), that this beam element predicts the behaviour of CFS almost same as shell elements.

Cross-section (Channel sections)	Dimension (mm)
Section_1	300 x 150 x 25 x 2
Section_2	300 x 150 x 25 x 3
Section_3	200 x 100 x 20 x 2

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Modal Analysis

ODB: frequecy_ana.odb   Abaqus/Standard 6.10-1   Tue Apr 21 22:19:14 India Standard Time 2015
Step: Step-1
Mode   1: Value = 1219.0   Freq = 5.5567   (cycles/time)

ODB: freq_2storey.odb   Abaqus/Standard 6.10-1   Wed Apr 22 19:14:27 India Standard Time 2015
Step: Step-1
Mode   1: Value = 520.11   Freq = 3.6297   (cycles/time)

ODB: frq_3sto_brac.odb   Abaqus/Standard 6.10-1   Wed Apr 22 20:41:15 India Standard
Step: Step-1
Mode   1: Value = 259.52   Freq = 2.5639   (cycles/time)

Frame Time period	ABAQUS result (Time period in seconds)	TI 809-07 (Time period in seconds)
1 storey	0.18	0.186
2 storey	0.275	0.31
3 storey	0.39	0.42

PBD of 2-storey frame

Type of frame	Cold form steel braced frame
Soil type	Rocky
Live load	Roof: 0.75 KN/m^2, Other floors: 4 KN/m^2
Response reduction factor, R	2.5
Zone factor	III
Concrete slab thickness on each floor	150 mm

Beam (lipped channel)	250 x 80 x 20 x 3.15 mm
Outer column (lipped channel)	100 x 60 x 15 x 2 mm
Inner column (lipped channel)	300 x 100 x 20 x 3 mm
Bracings (Flat)	50 x 2 mm

Pushover_2storey

Chart: Base Shear, N (y-axis, 0 to 120000) vs Roof displacement, mm (x-axis, 0 to 600)
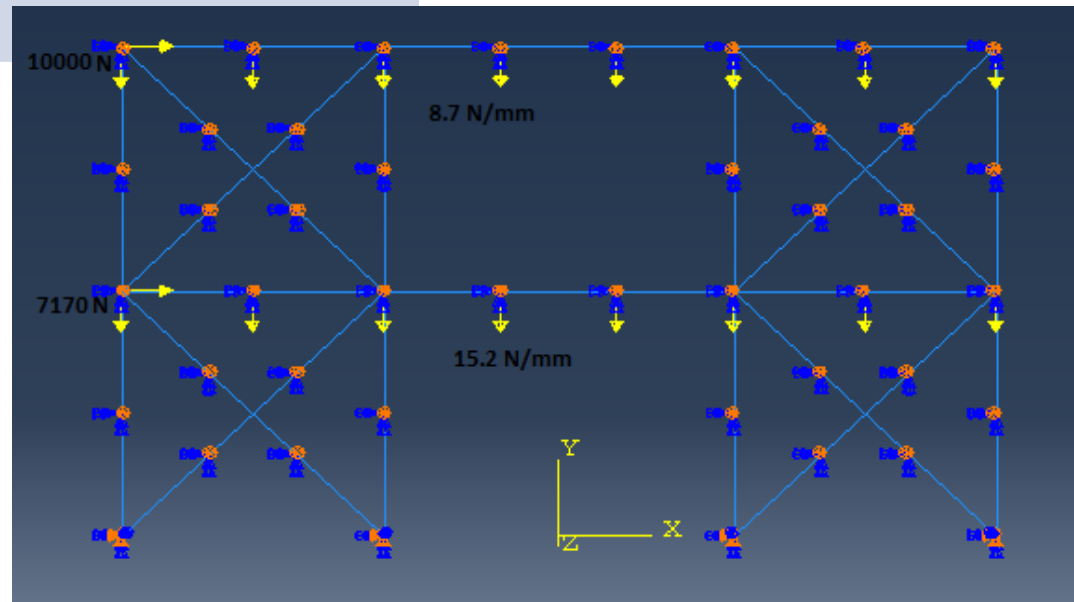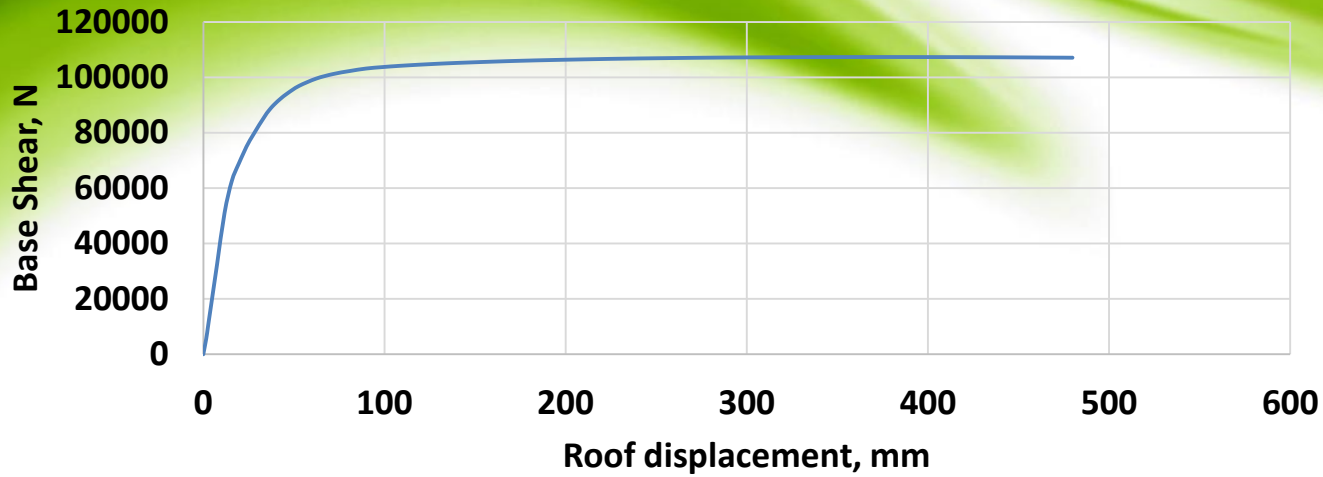
Type	MCE
Zone	III
Soil strata	Rocky
Near source	< 2 km

RS_Z-III_MCE

Chart: A, m/s/s (y-axis, 0 to 10) vs Time period, sec (x-axis, 0 to 5) — RS_Z-III_MCE
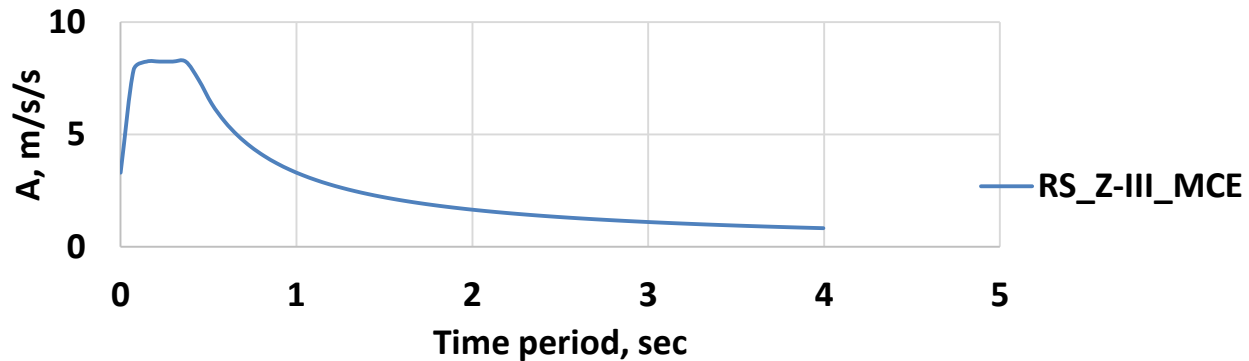
- ***Result***

- Performance point obtained is (31.9, 4.84).

- Displacement of 1st storey = 31.9*1.175*0.717 = 26.87 mm

- Displacement of roof = 31.9*1.175*1 = 37.48 mm

- Drift ratio = 37.48/7000 = 1/186 = 0.53% which corresponds to **Life safety** condition as mentioned in FEMA-356 (figure 5-2).

- Corresponding arc length is 30.43 and step corresponding to this in ABQUS shows that three braces and two base columns have yielded.

S, Mises
Multiple section points
(Avg: 75%)
+2.483e+02
+2.276e+02
+2.069e+02
+1.862e+02
+1.655e+02
+1.448e+02
+1.241e+02
+1.035e+02
+8.276e+01
+6.207e+01
+4.139e+01
+2.070e+01
+9.426e-03

ODB: push_2.odb   Abaqus/Standard 6.10-1   Tue May 05 23:03:44 India Standard Time 2015
Step: Step-2
Increment   30: Arc Length =   30.76
Primary Var: S, Mises

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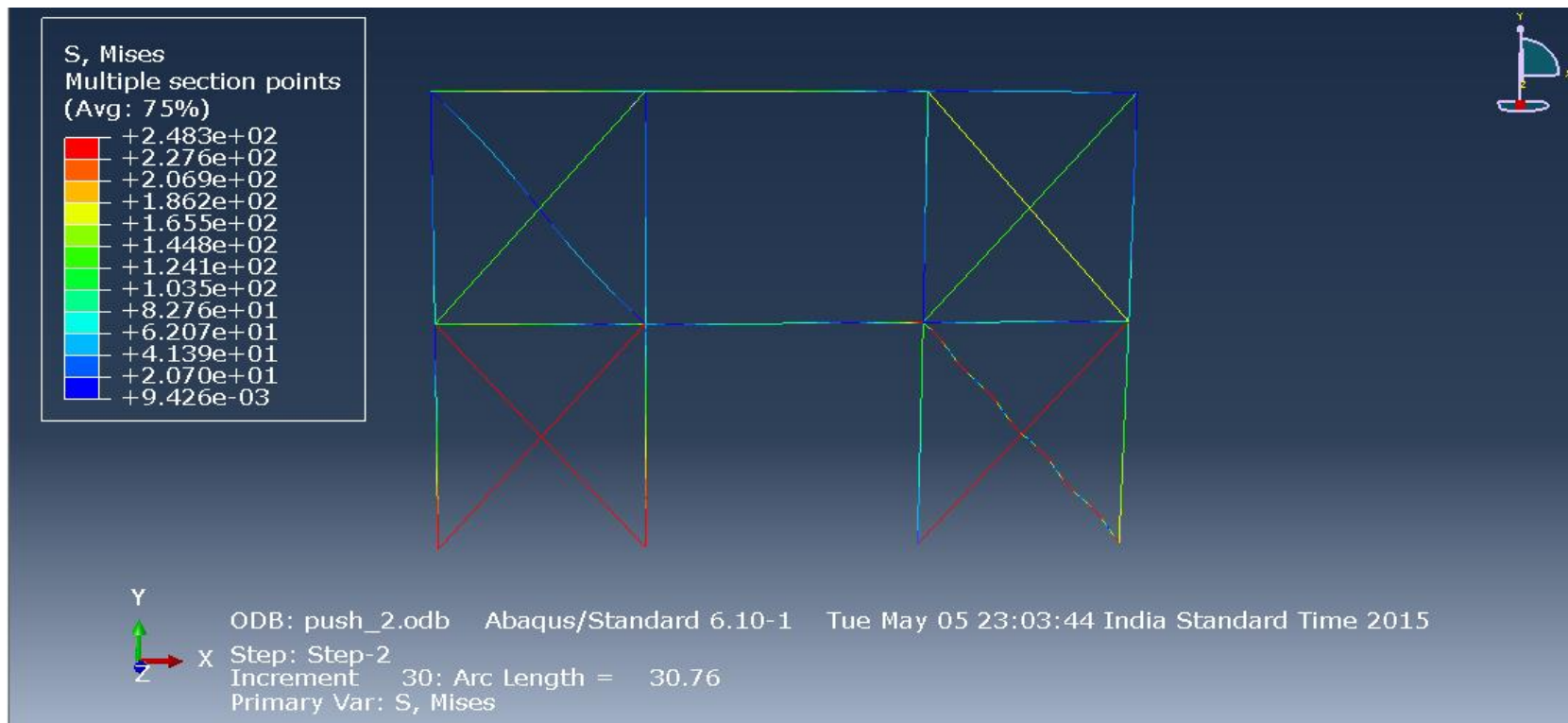
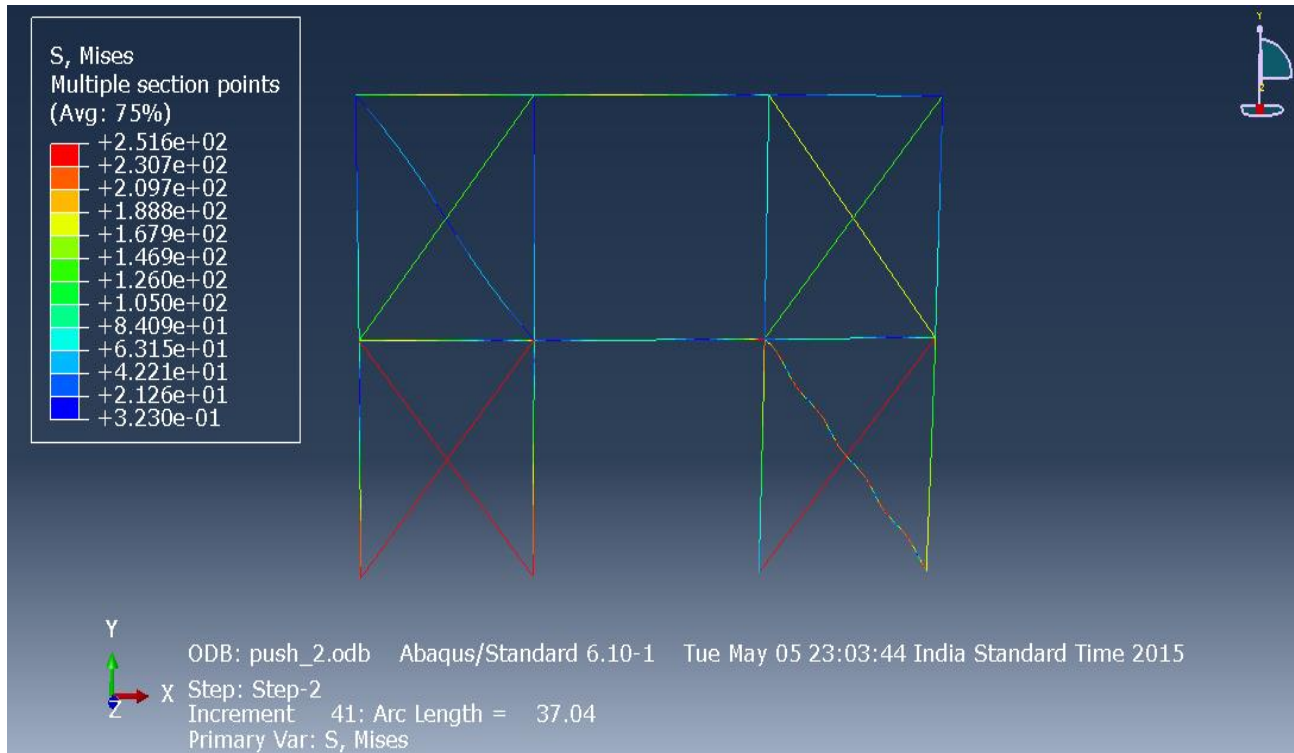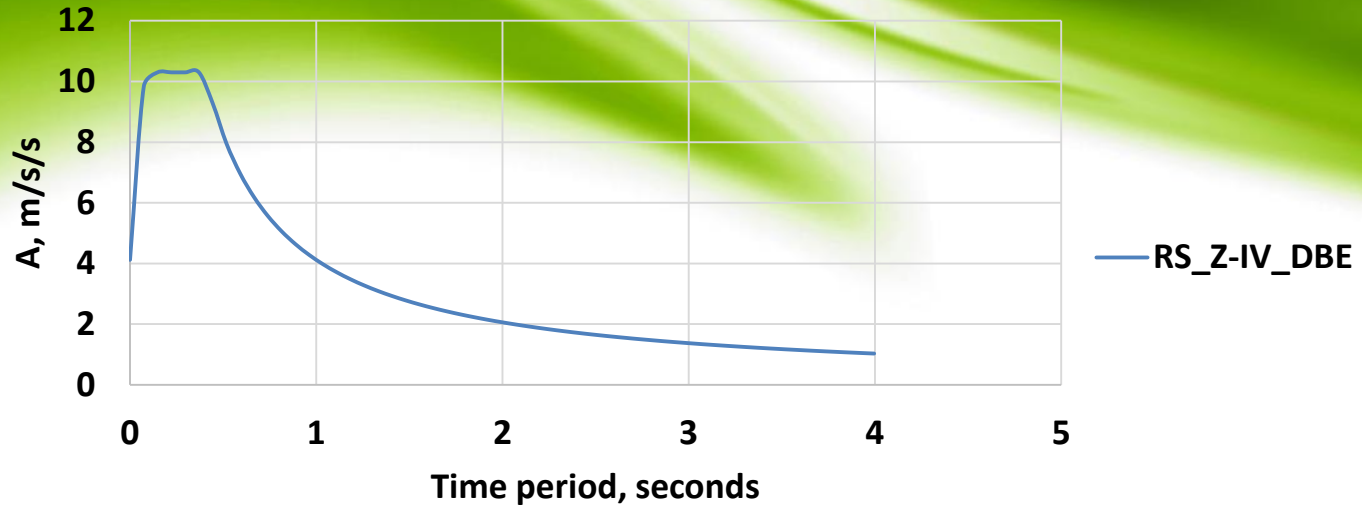| Elements	Type	Structural Performance Levels		
		Collapse Prevention S-5	**Life Safety S-3**	**Immediate Occupancy S-1**
Steel Moment Frames	Primary	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Hinges form. Local buckling of some beam elements. Severe joint distortion; isolated moment connection fractures, but shear connections remain intact. A few elements may experience partial fracture.	Minor local yielding at a few places. No fractures. Minor buckling or observable permanent distortion of members.
	Secondary	Same as primary.	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Same as primary.
	Drift	5% transient or permanent	2.5% transient; 1% permanent	0.7% transient; negligible permanent
Braced Steel Frames	Primary	Extensive yielding and buckling of braces. Many braces and their connections may fail.	Many braces yield or buckle but do not totally fail. Many connections may fail.	Minor yielding or buckling of braces.
	Secondary	Same as primary.	Same as primary.	Same as primary.
	Drift	2% transient or permanent	1.5% transient; 0.5% permanent	0.5% transient; negligible permanent

- <u>The same frame is analysed for DBE zone IV earthquake.</u>
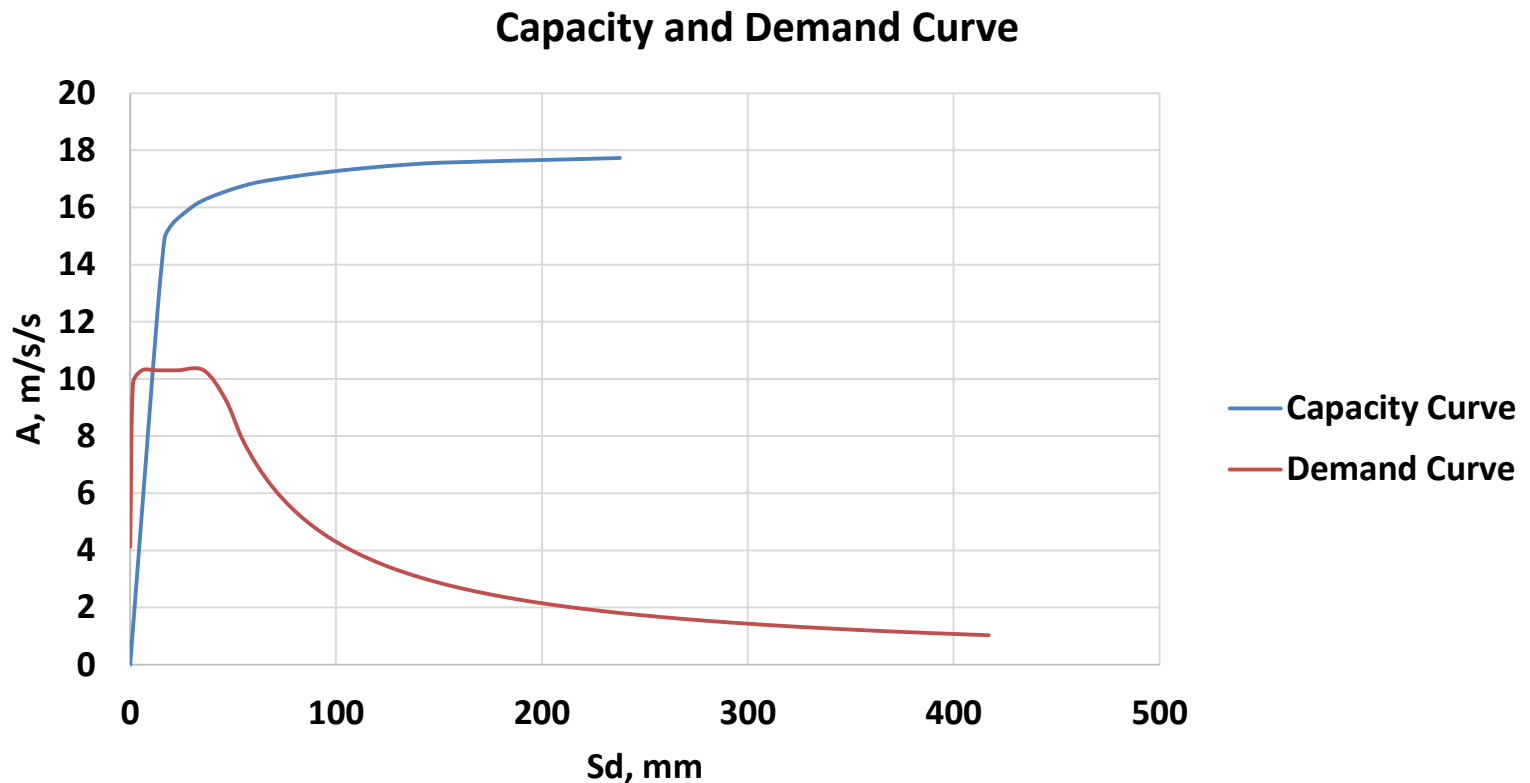
<u>Result</u>

- Performance point is (37.4053, 5.0628).

- Displacement of 1st storey = 37.4053*1.175*0.717 = 31.51 mm

- Displacement of 2nd storey = 44 mm.

- Drift ratio = 44/7000 = 1/159 = 0.628 % which comes under Life safety condition as per FEMA-356.

- Corresponding arc length is 37.04. At this step, bracings of base storey and two columns have yielded.

RS_Z-IV_DBE

A, m/s/s vs Time period, seconds

RS_Z-IV_DBE

S, Mises
Multiple section points
(Avg: 75%)
- +2.516e+02
- +2.307e+02
- +2.097e+02
- +1.888e+02
- +1.679e+02
- +1.469e+02
- +1.260e+02
- +1.050e+02
- +8.409e+01
- +6.315e+01
- +4.221e+01
- +2.126e+01
- +3.230e-01

ODB: push_2.odb   Abaqus/Standard 6.10-1   Tue May 05 23:03:44 India Standard Time 2015
Step: Step-2
Increment   41: Arc Length =   37.04
Primary Var: S, Mises

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Design problem: The two storey frame on rocky strata under design earthquake in zone IV be designed such a way that any members of the frame doesn't yield.
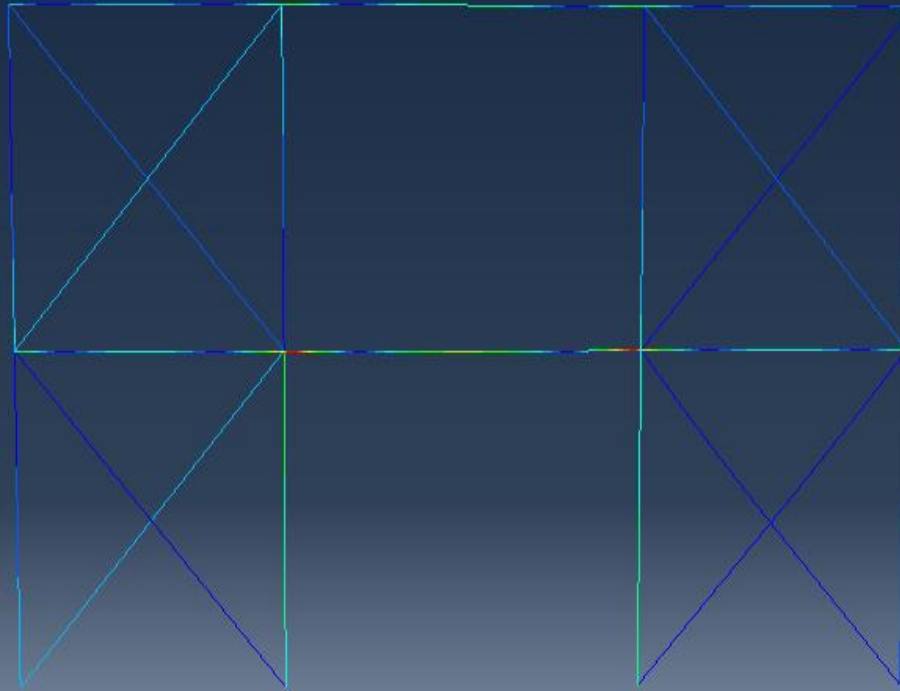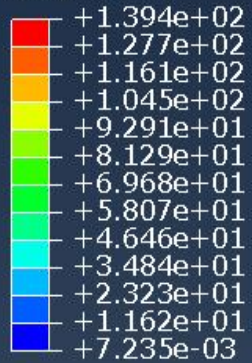
• To achieve this performance bracing area is increased from 50 x 2 mm to 100 x 6 mm.

Capacity and Demand Curve

Result:

- Performance point is (11, 10.30).

- Demand curve intersect capacity curve within elastic range. Since structure is in elastic range, its period of vibration obtained from response spectrum corresponding to ordinate of performance point should match with its fundamental time period. Corresponding to 10.30 m/s/s acceleration time period is 0.222 second while fundamental time period of 2 storey frame from table 7-1 is 0.275 seconds. Hence, above assertion is proved.

- Roof displacement is 13 mm.

- Drift ratio is $13/7000 = 1/538 = 0.18\% < 0.5\%$. Hence structure is under **Immediate occupancy** condition.
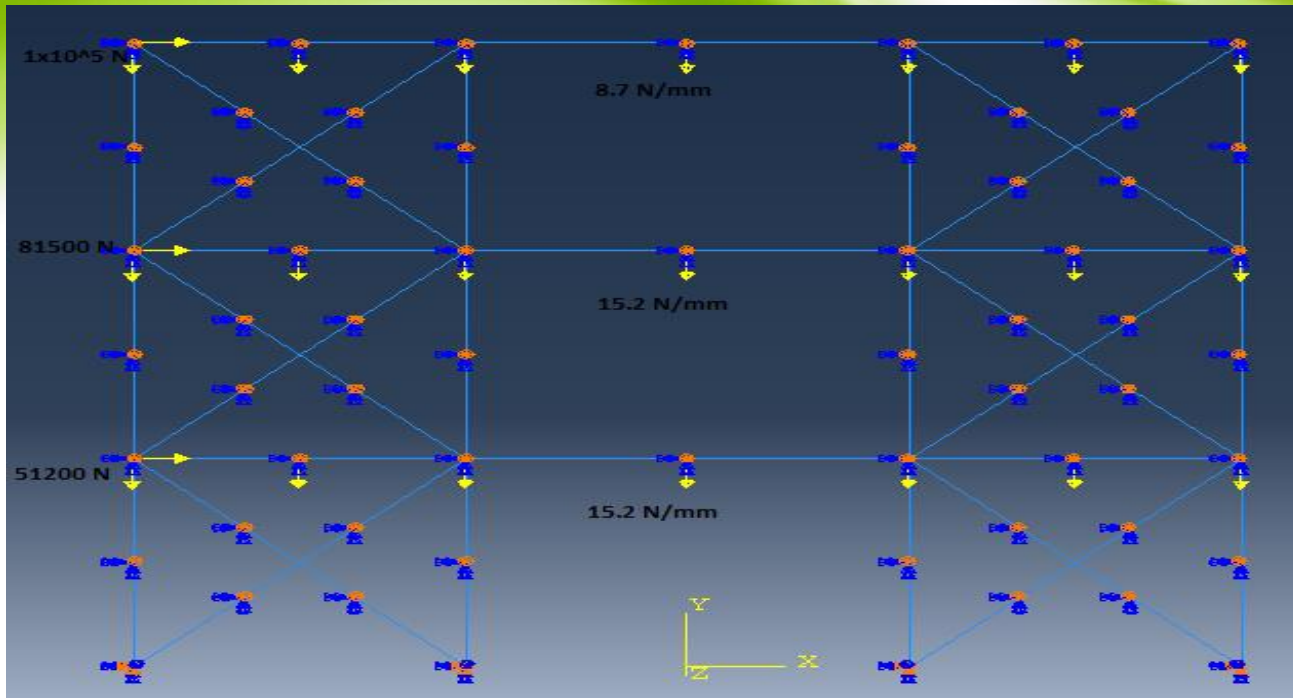
S, Mises
Multiple section points
(Avg: 75%)
+1.394e+02
+1.277e+02
+1.161e+02
+1.045e+02
+9.291e+01
+8.129e+01
+6.968e+01
+5.807e+01
+4.646e+01
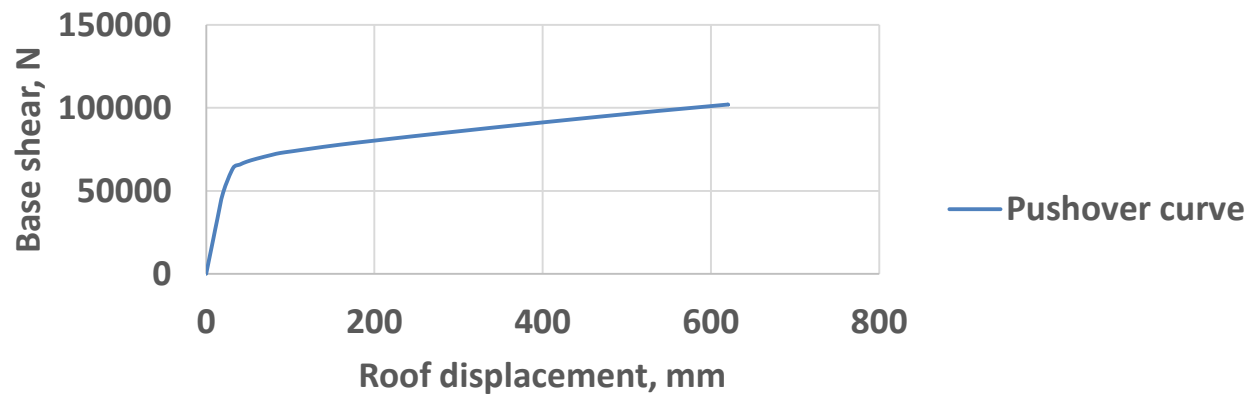+3.484e+01
+2.323e+01
+1.162e+01
+7.235e-03

ODB: push_2storey.odb   Abaqus/Standard 6.10-1   Tue May 26 20:02:03 India Standard Time 2015
Step: Step-2
Increment   38: Arc Length =   7.548
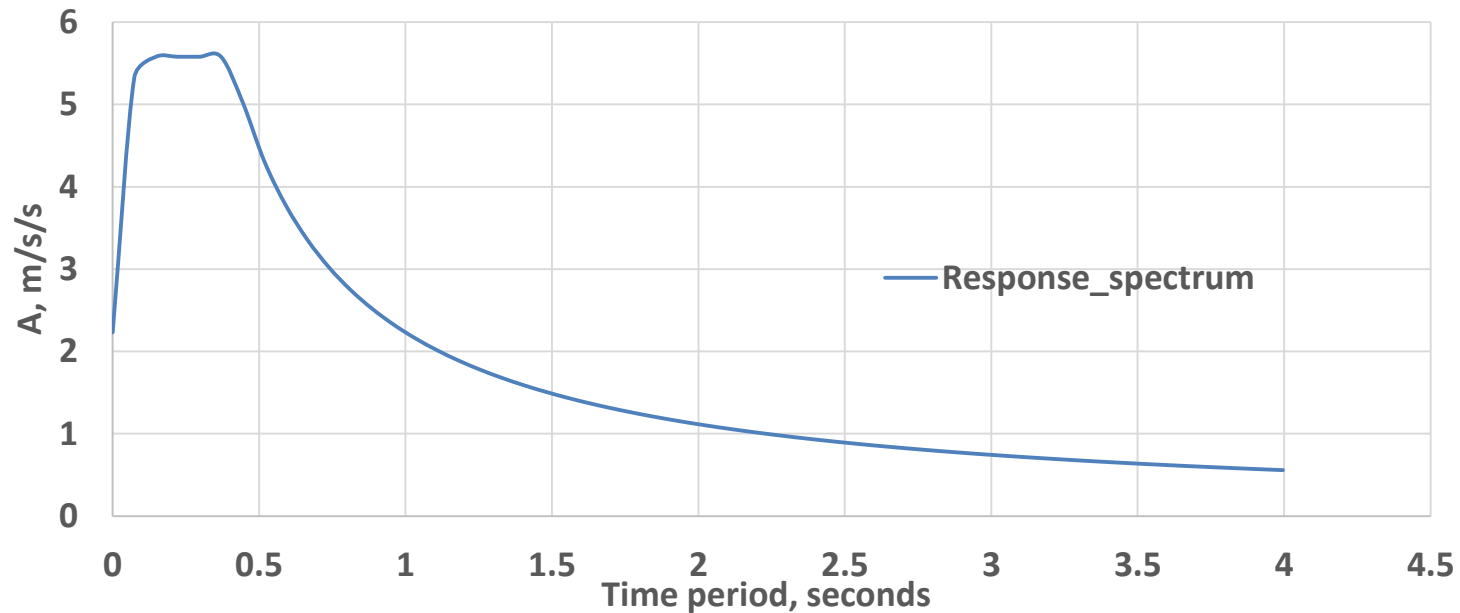Primary Var: S, Mises

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- Three storey frame

Pushover curve

Three storey frame is subjected to the earthquake of following parameters:

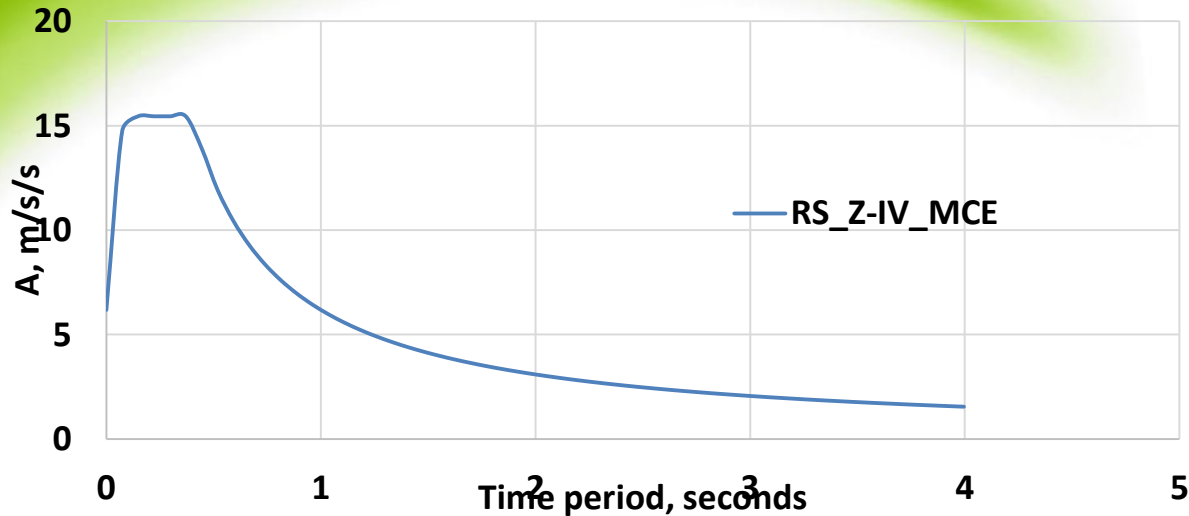| Type	DBE
Zone	III
Soil strata	Rocky

Response_spectrum

- *__Result__*:

- Performance point is (31.433, 2.31).

- Roof displacement = 39.45 mm.

- Drift ratio = 39.45/10500 = 0.37% < 0.5%. Hence, frame is under **immediate occupancy** condition.

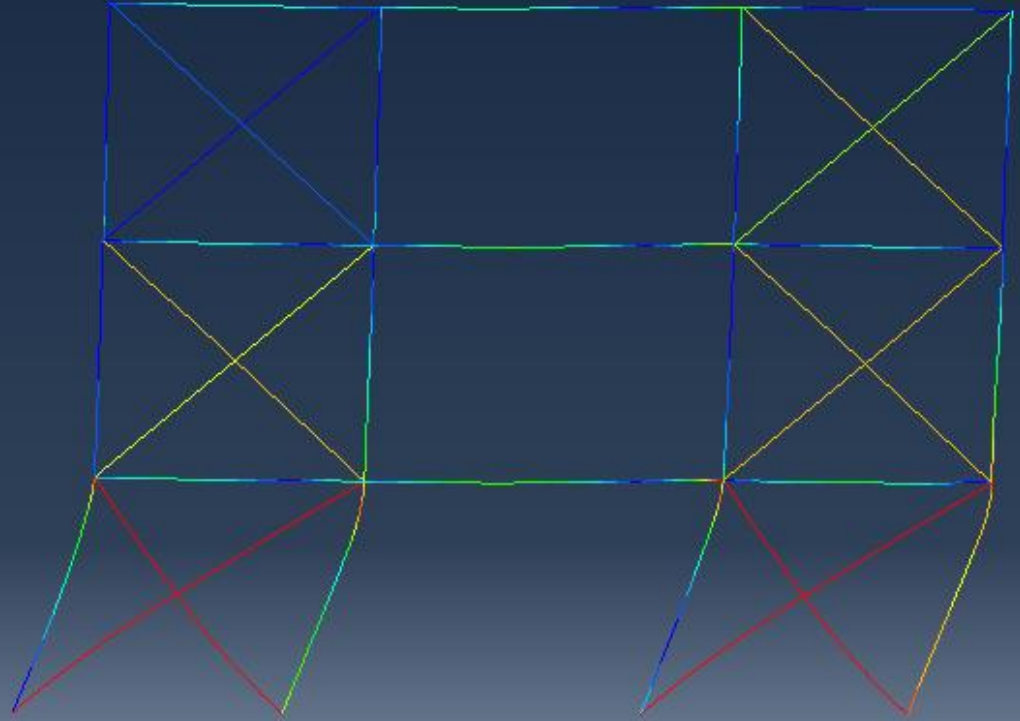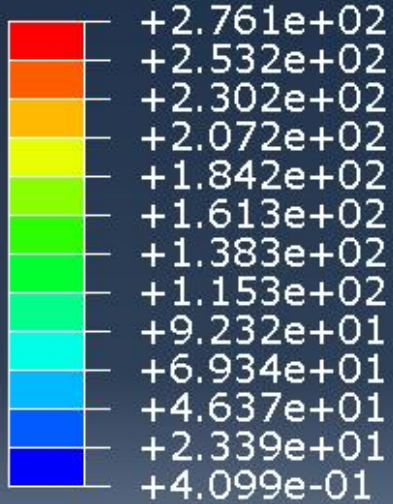- From figure 5-19, it can be seen that only bracings of base storey have yielded.
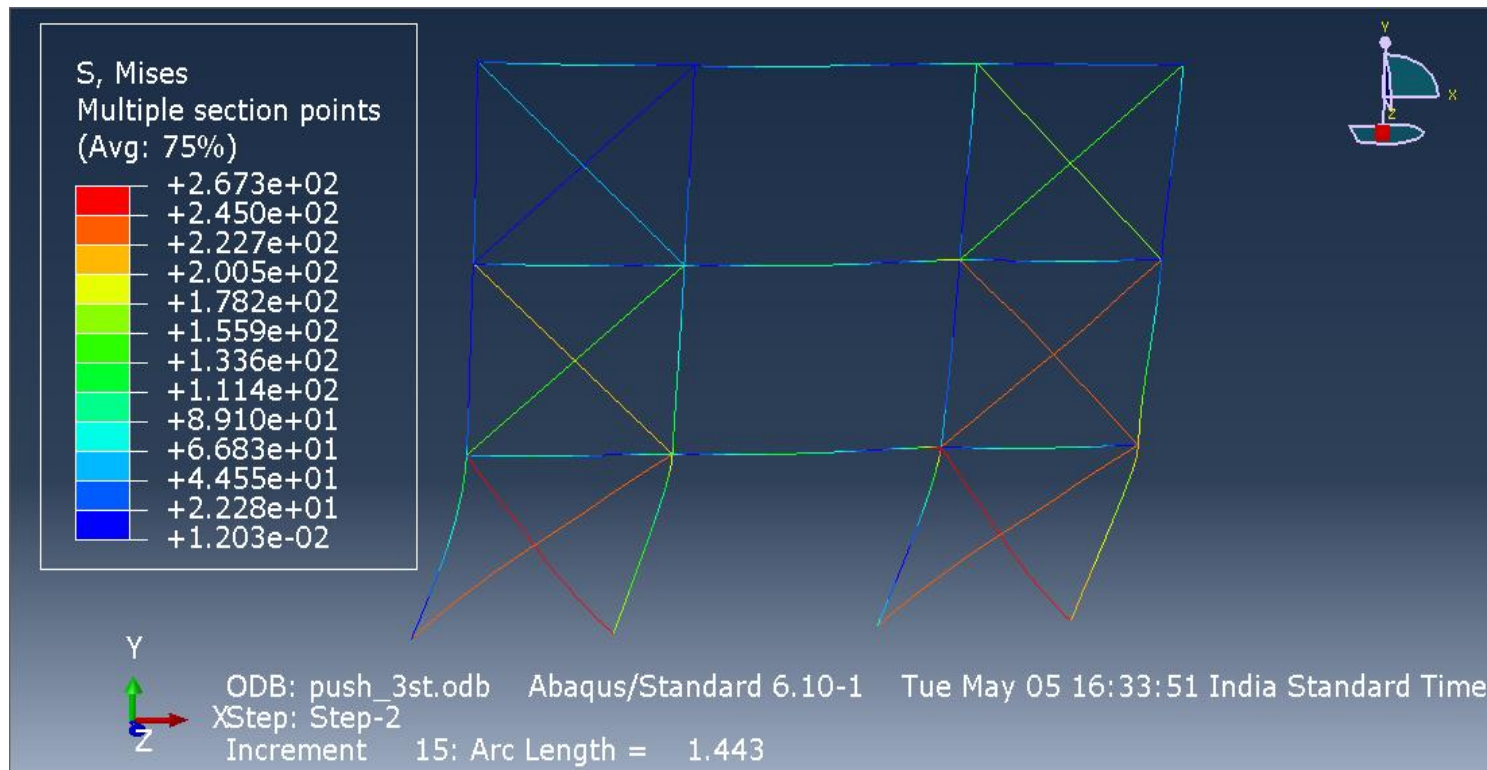
Type	MCE
Zone	IV
Soil Strata	Rocky

RS_Z-IV_MCE

1. Performance point is (159.74, 2.82).
2. Roof displacement = 200.47 mm.
3. Drift ratio = 200.47/10500 = 1.91 % almost equal to 2%.

This structure is near to **collapse prevention** condition, bracings have failed and base columns have yielded.

Frame with semi-rigid beam-column and column base joint:

- Semi-rigid frames are modelled using spring element at beam-column joint.
- . In the same way at the base of the frame, spring element is defined to make base of the frame semi-rigid with the base plate.
- At beam column joint, spring element of rotational stiffness 9×10^7 N mm/mm is defined whereas at the base, spring element of rotational stiffness 10×10^7 N mm/mm is defined.
- Fundamental time period of the frame is 0.376 seconds.

pushove-curve_semi-rigid
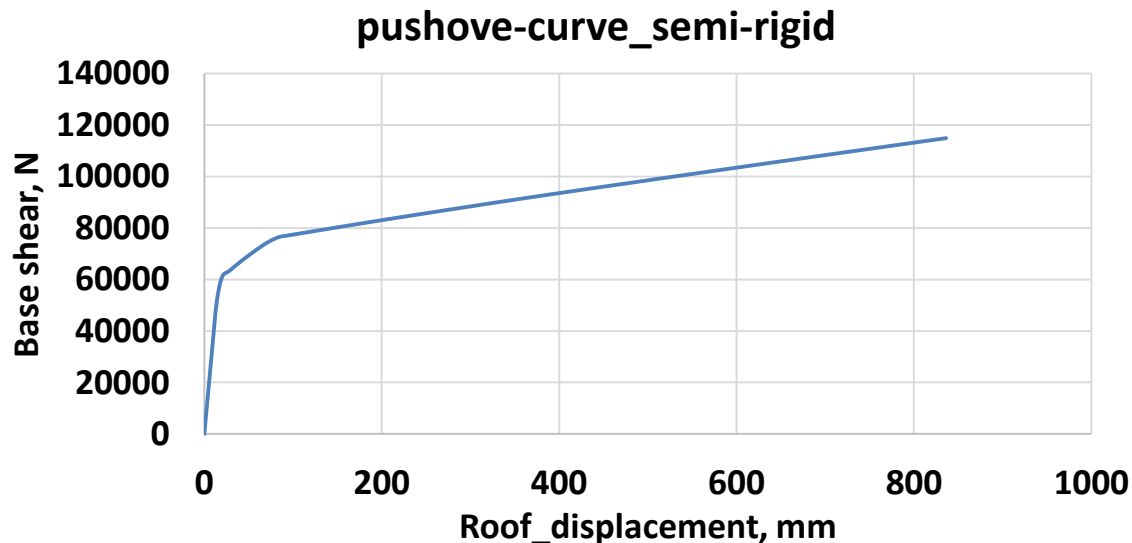
- The two storey semi-rigid frame is subjected to the design earthquake in zone IV. Soil strata is assumed to be rocky.

- ***Result***:

- Performance point is (53.92, 4.18).

- Displacement of 1st storey = 53.92*1.247*0.582 = 39.132 mm.

- Displacement of roof = 67.238 mm.

- Drift ratio = 67.238/7000 = 0.96% > 0.5% which is under **life safety** condition as per FEMA-356.

- As shown , all bracings have yielded.


S, Mises
Multiple section points
(Avg: 75%)
+2.380e+02
+2.181e+02
+1.983e+02
+1.785e+02
+1.586e+02
+1.388e+02
+1.190e+02
+9.915e+01
+7.932e+01
+5.949e+01
+3.966e+01
+1.983e+01
+1.461e-03

ODB: push_semi_2st.odb   Abaqus/Standard 6.10-1   Wed May 06 19:29:04 India Standard Time 2015
Step: Step-2
Increment   13: Arc Length =   193.6
Primary Var: S, Mises
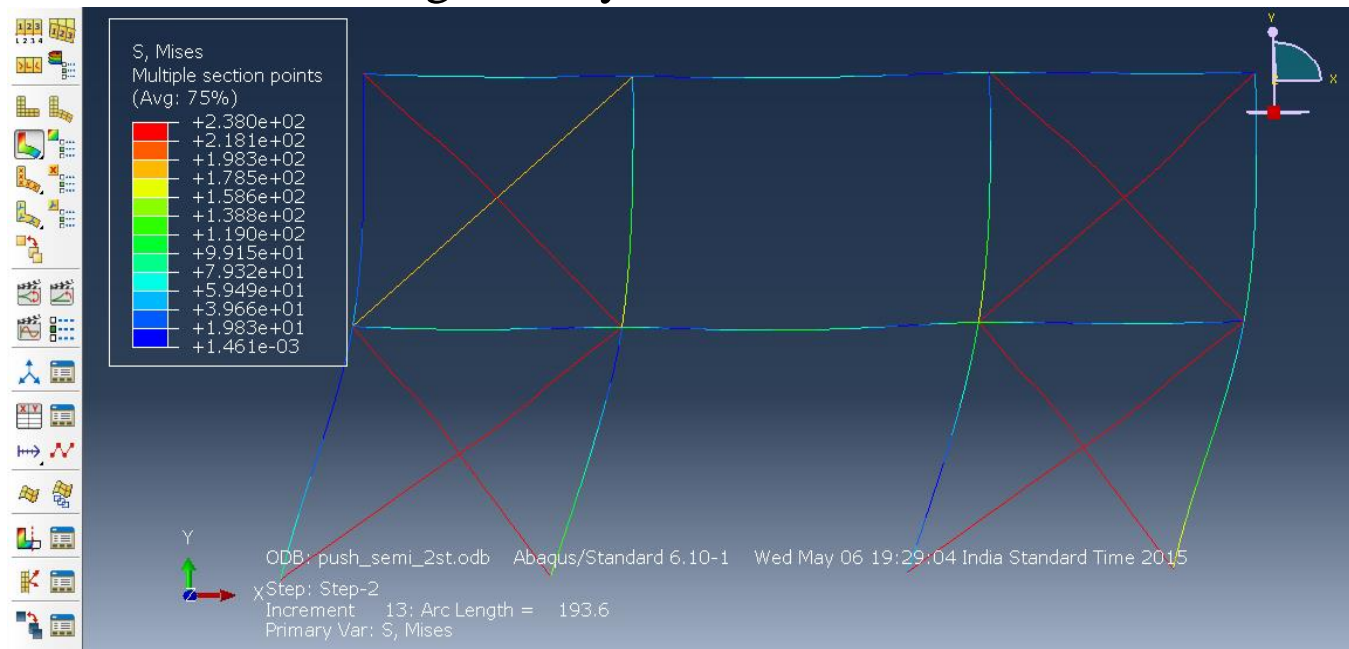
Design problem: This frame to be designed in such a way that under DBE and zone IV earthquake it falls in immediate occupancy condition (drift ratio <0.5%).

To achieve the performance mentioned in design problem, area of the braces are increased from 50 x 2 mm to 75 x 3 mm and braces are installed in the mid-bay of the frame.
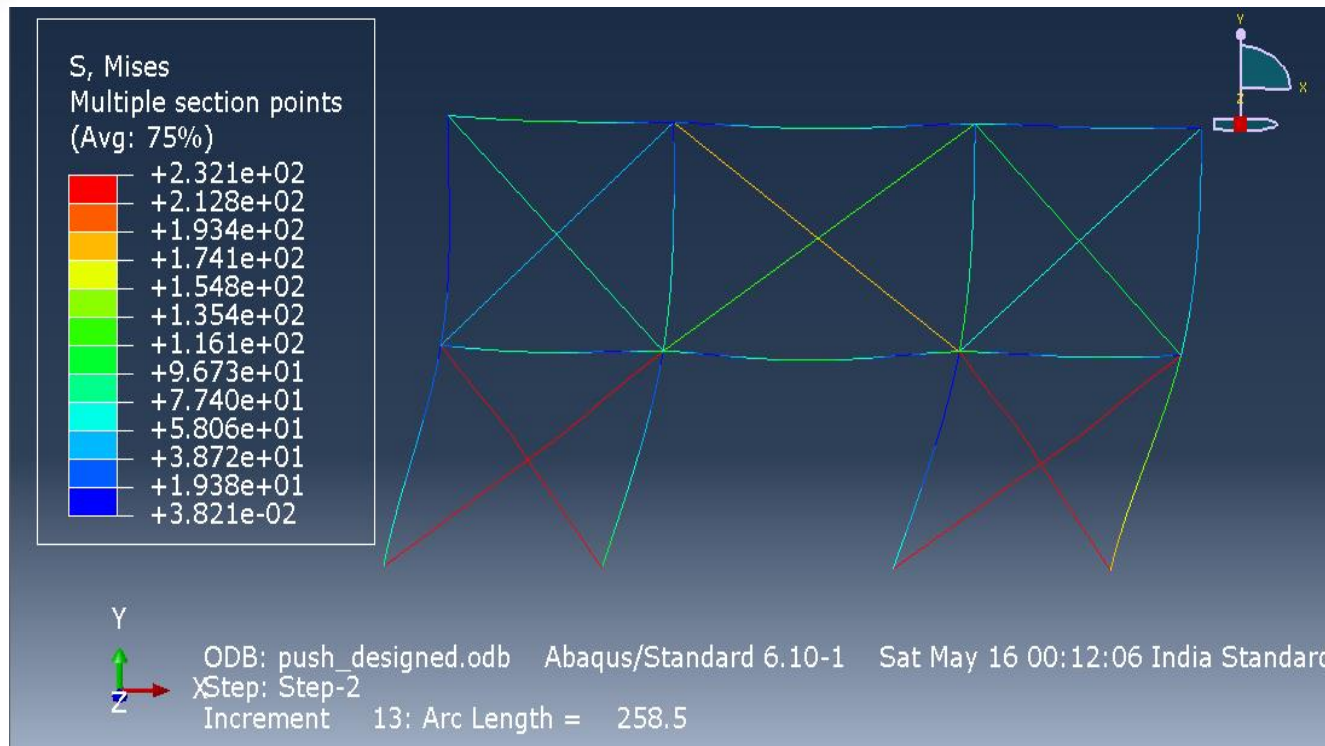
Drift ratio = 0.48%

IO conditions

Conclusions

- Thin walled cold-formed structures as appeared to be very slender and non-dissipative because of its sectional buckling phenomenon, can be made more ductile and dissipative.

- It is concluded that use of CFS in seismic region can be made possible by enhancing its performance.

- Column base joint stiffness increases with increasing plate thickness.

- Beam element B32OS can be fairly used for analysis of CFS frames.

- ATC-40 procedure works well for CFS as the performance from the visuals of ABAQUS matches with the criteria given in FEMA-356.

- Performance of semi-rigid frames for same demand is different from rigid frames. Drift ratio of semi-rigid frame is more than that of rigid frame.

Future scope

- Using the hinge properties of the CFS members and incorporating it in any finite element programming to obtain pushover curve of the CFS frame.

- PBD can be performed using other methods like coefficient methods given in ATC-40 and it can be checked if the design is possible with performance based plastic design.

- Here, planar frame is analysed and designed. Three dimensional frame can also be analysed and designed with the same method.

- Pushover analysis is done with the fact that first mode of vibration has major contribution. If other modes also have significant participation, other methods of pushover analysis can be done.

- Hysteresis behaviour of CFS members and beam-column joint can be done and energy dissipation can be observed.

- Effect of shape memory alloy as bracings on performance of CFS structures can be studied.

Thank You

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