

دوره جامع آموزش اصول محاسبات بهسازی لرزه ای و طراحی عملکردی سازه ها

- مبانی طراحی عملکردی (اصول و کاربردها)
- طراحی عملکردی در آیین نامه ها
- آشنایی با سیستم های نوین کنترل ارتعاشات لرزه ای
- حل مثال عملی با استفاده از نرم افزارهای CSI و SeismoSoft

مدرس: پروفسور حسن مقدم

به همراه تدریس نرم افزار توسط تیم مدرسین موسسه ۸۰۸

شروع دوره: ۵ تیر ماه ۹۷
مکان: دانشگاه صنعتی شریف، سالن انجمن فارغ التحصیلان
ویژه مهندسين عمران شاغل در صنایع فعال در حوزه مقاوم سازی
و سازمان های مرتبط و دانشجویان

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SeismoSoft در ایران
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تلفن تماس: ۰۲۱۸۸۲۷۲۶۹۴

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جزوه بخش نرم‌افزاری دوره جامع آموزش اصول محاسبات لرزه‌ای و طراحی عملکردی سازه‌ها

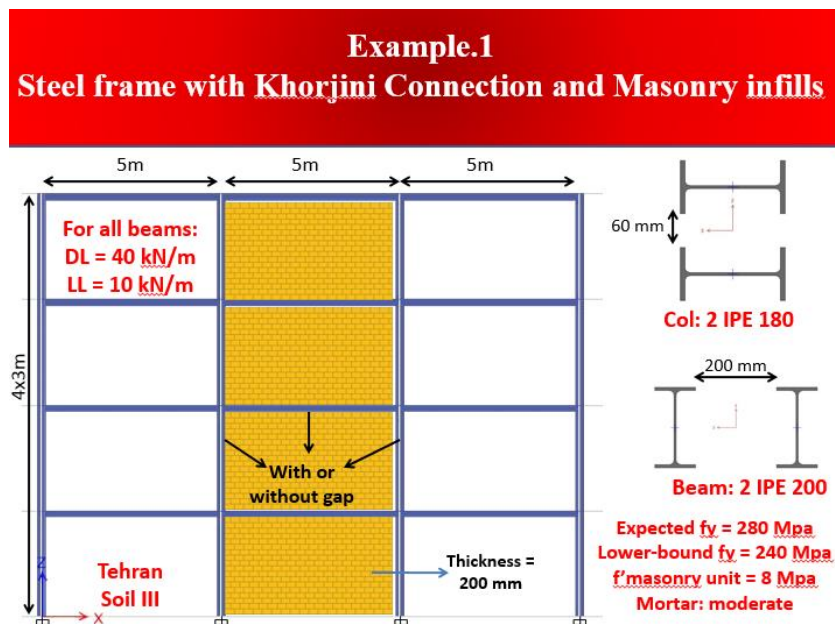
در این بخش از دوره که توسط دکتر موسوی از مدرسین موسسه ۸۰۸ برگزار می‌شود، به حل تمرینات زیر پرداخته خواهد شد:

۱. قاب ساده خرچینی + میانقاب آجری

قاب ۴ طبقه سه دهانه فرضیات بار و ابعاد هندسی تا حد امکان مشابه نمونه ۱۰۱۵ / محل بنا در تهران، خاک نوع ۳، دهانه‌ها ۵ متر، ارتفاع همه طبقات ۳ متر، بار مرده و زنده طبقات به ترتیب برابر ۴ و ۱ تن بر متر/ تیرها دابل آی پی ای ۲۰ / اتصالات خرچینی سنتی با نبشی بالا و پایین و با فرض سختی دوران صفر/ ستون‌ها پا باز دابل آی پی ای ۱۸ / یک دهانه از سه دهانه دارای میانقاب‌های آجری به کلفتی ۲۰ سانتیمتر در همه طبقات/ مقاومت فشاری آجر ۸ نیوتن بر م م مربع و ملات متوسط.

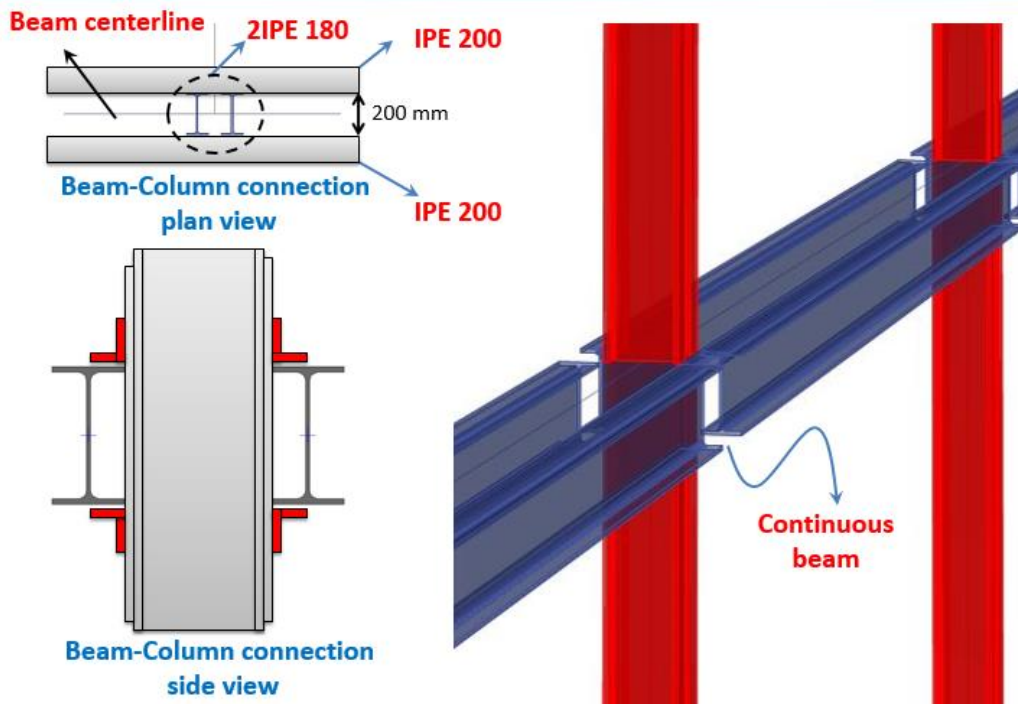
خروجی‌ها:

۱. جابجایی هدف در خطر ۱ و ۲
۲. مدل میانقاب/ مقاومت، سختی، نمودار غیرخطی
۳. وضعیت لولای خمیری در میانقاب در خطر ۱ و ۲
۴. دوران اتصال خرچینی در خطر ۱ و ۲
۵. نمودار رانش
۶. نمودار دوران اتصال برحسب جابجایی بام
۷. نمودار گریز میانقاب برحسب جابجایی بام





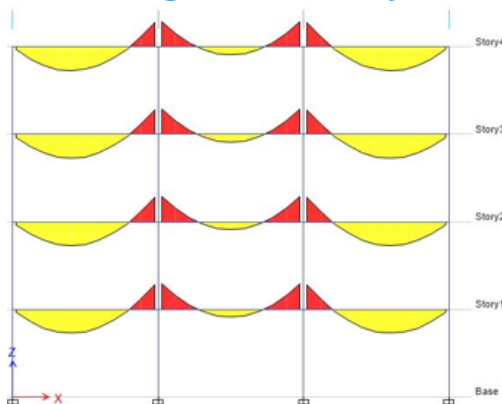
Example.1



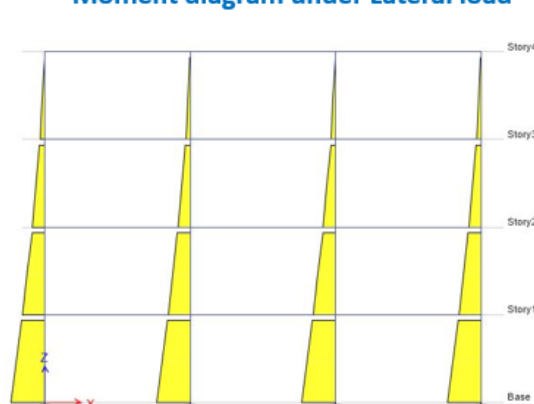
Example.1

Let's assume the bare frame with no infill

Moment diagram under Gravity load



Moment diagram under Lateral load



Bare frame would be unstable in the case of pinned base columns.

The beams are continuous but get no moment from lateral loads .

This can be modeled with a fixed beam-column connection but with a panel zone with zero flexural stiffness.



Example.1

This type of column sections are not covered in ASCE 41-13 so we use Code 360

Although the beams would not get moment from lateral loads, significant moments could be generated due to the gravity loads as well as the infill. So plastic hinges are defined at the middle of the beams and beam ends.

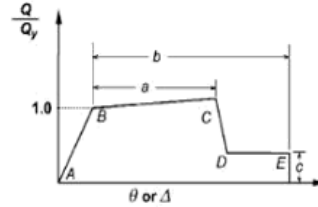


FIG. 9-1. Generalized Force-Deformation Relation for Steel Elements or Components

From ETABS, obtain $PCL=1070$ [kN](#)

From ETABS gravity analysis most column are force-controlled. Still we would define P-M3 Plastic hinges for columns

Define the behavior for $P/PCL = 0.2$ and assign the parameters for $P/PCL < 0.2$

Define the behavior for $P/PCL = 0.45$ and assign the parameters for $0.2 < P/PCL < 0.5$

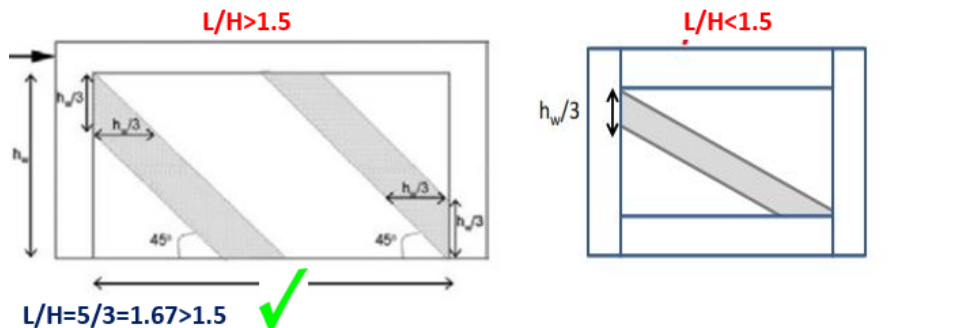
Define the behavior for $P/PCL = 0.5$ and assign all parameters zero or very small values

ETABS would automatically interpolate between different P/PCL and also accounts for P/PCL on moment capacity of the column

Per Code 360, rotation of the [khorjini](#) connections with top and bottom angles should be limited to [0.01 rad](#) and [0.02 rad](#) for LS and CP criteria, respectively.

Example.1

Infill modeling per ASCE 41-13



Moderate mortar

$$f'_b = 8 \text{ MPa} \rightarrow f'_m = 2.5 \text{ to } 2.96 \text{ Mpa} = 2.7 \text{ MPa} \rightarrow \text{Expected } f'_m = 1.3 \times 2.7 = 3.5 \text{ MPa}$$

$$E_m \approx 400f'_m = 400 \times 3.5 = 1400 \text{ MPa}$$

This is the horizontal component of the ultimate capacity of both struts

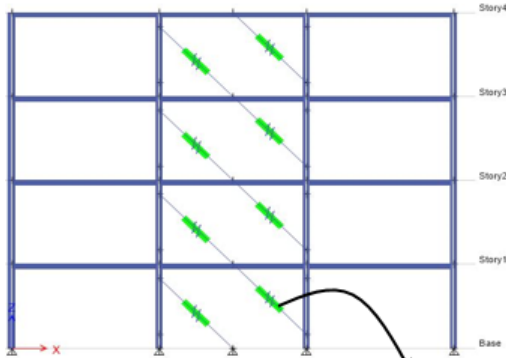
Expected shear capacity: $Q_{CE} = A_n \cdot v_{me-(zero \text{ gravity load})} = 5000(\text{mm}) \times 200(\text{mm}) \times 0.2(\text{Mpa}) = 200 \text{ kN}$

Should be obtained based on in-situ test: $v_{me-(zero \text{ gravity load})} = 0.2 \text{ Mpa}$ (assumed value)

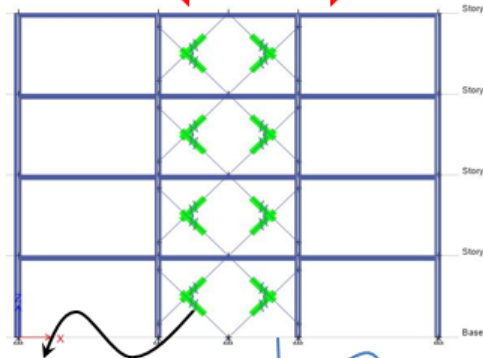


Example.1

for single-direction lateral loading



for positive/negative or cyclic lateral loading



Compression-only Multilinear plastic Link element

Length of each strut: $L=3.5\text{m}$

Area of each strut: $A=(1/2^{0.5})\times 0.2=0.14\text{ m}^2$

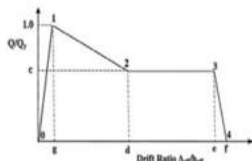
For each strut:

Initial axial stiffness= $EA/L=1400(\text{Mpa})\times 0.14/3.5=56\text{ kN/mm}$

Ultimate axial strength = $QCE/2/\cos(\theta)=200/2/\cos(45)=141\text{ kN}$

In this case, effective stiffness of link elements should be reduced by 50% as there are two struts. In nonlinear analysis only one of them would be active so the initial stiffness should not be reduced.

Example.1



$$V_{fr} = \sum (2M_{pc}/H_s)$$

Consider $C=0$
 $d=f$

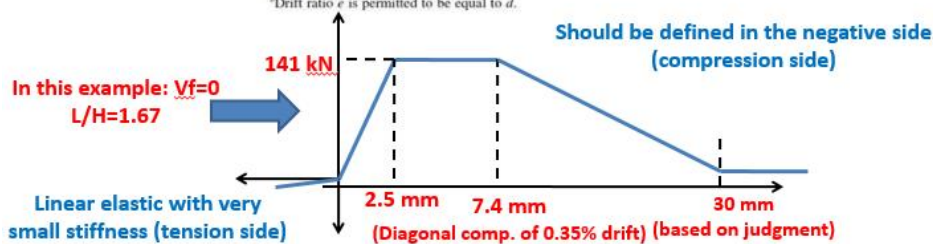
Both ASCE 41-13 and Code 360

Table 11-9. Nonlinear Procedure—Simplified Force-Deflection Relations for Masonry Infill Panels^a

$\beta = \frac{V_{fr}}{V_{sw}}$	$\frac{L_{inf}}{h_{sw}}$	Residual Strength Ratio c	d (%)	e^b (%)	Acceptance Criteria	
					LS (%)	CP (%)
$\beta < 0.7$	0.5	NA	0.5	NA	0.4	NA
	1.0	NA	0.4	NA	0.3	NA
	2.0	NA	0.3	NA	0.2	NA
$0.7 \leq \beta < 1.3$	0.5	NA	1.0	NA	0.8	NA
	1.0	NA	0.8	NA	0.6	NA
	2.0	NA	0.6	NA	0.4	NA
$\beta \geq 1.3$	0.5	NA	1.5	NA	1.1	NA
	1.0	NA	1.2	NA	0.9	NA
	2.0	NA	0.9	NA	0.7	NA

^aInterpolation shall be used between table values. In this table, NA means not available.

^bDrift ratio e is permitted to be equal to d .





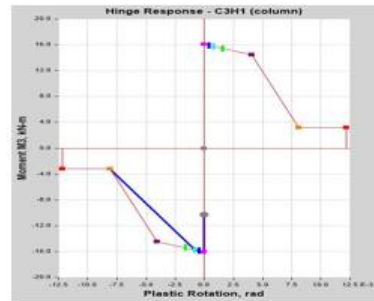
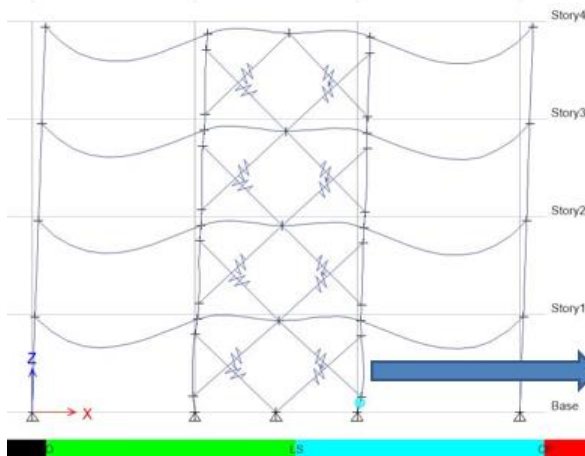
Example.1

Per Code 360

Seismic mass: $D+0.2L$

Gravity load: $1.1(D+0.25L)$

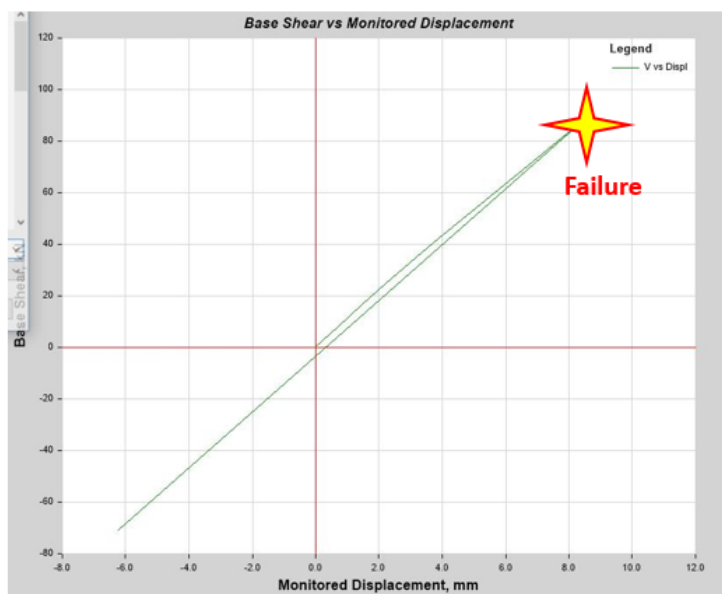
It turned out that the structure is too weak even
for gravity loads



Under gravity load, $P/PCL \approx 0.85$
At roof disp. Of only 8 mm, $P/PCL \approx 1$

Example.1

The column failed before infills and the pushover
curve is linear up to the failure

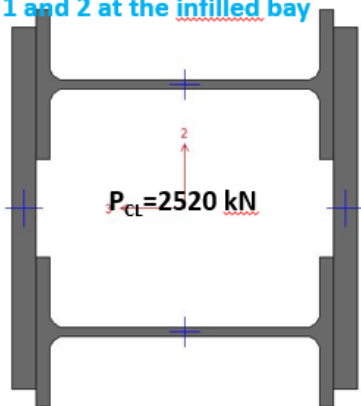




Example.1

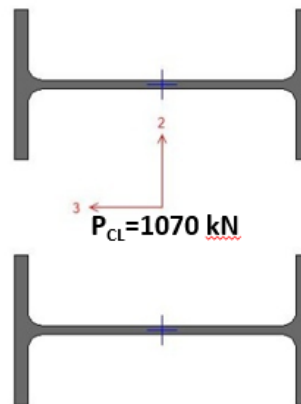
So it is decided to improve columns at the lower two stories but it is not intended to fully retrofit the frame (that is, columns are just moderately improved)

Column section for Stories 1 and 2 at the infilled bay



2 IPE 180 + 2PL220x15

Other columns

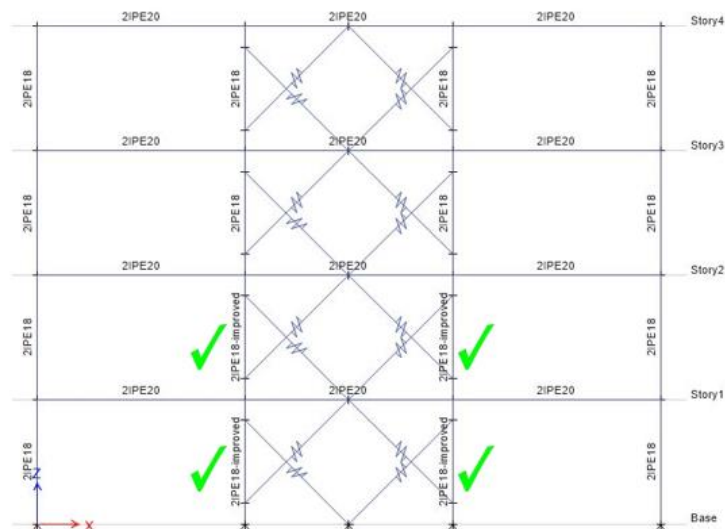


2 IPE 180

Example.1



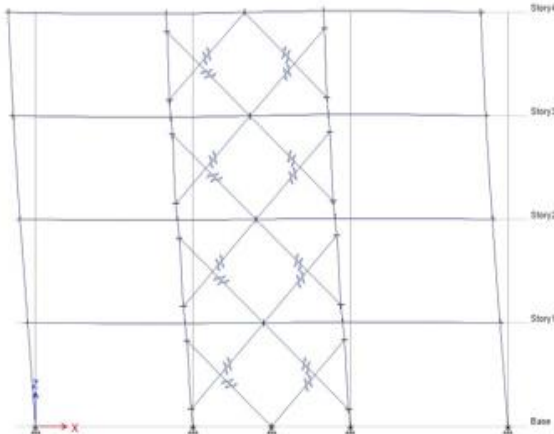
Improved columns



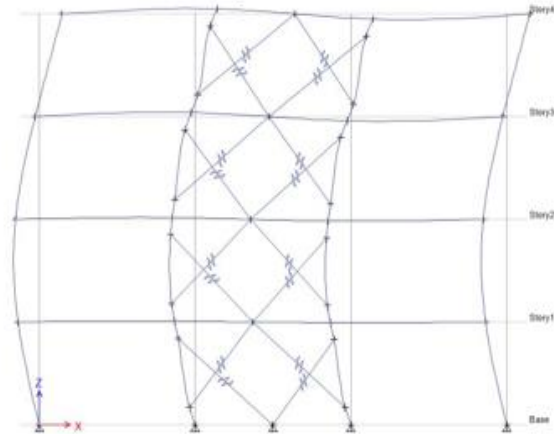


Example.1

1st mode period = 0.845 s



2nd mode period = 0.289 s



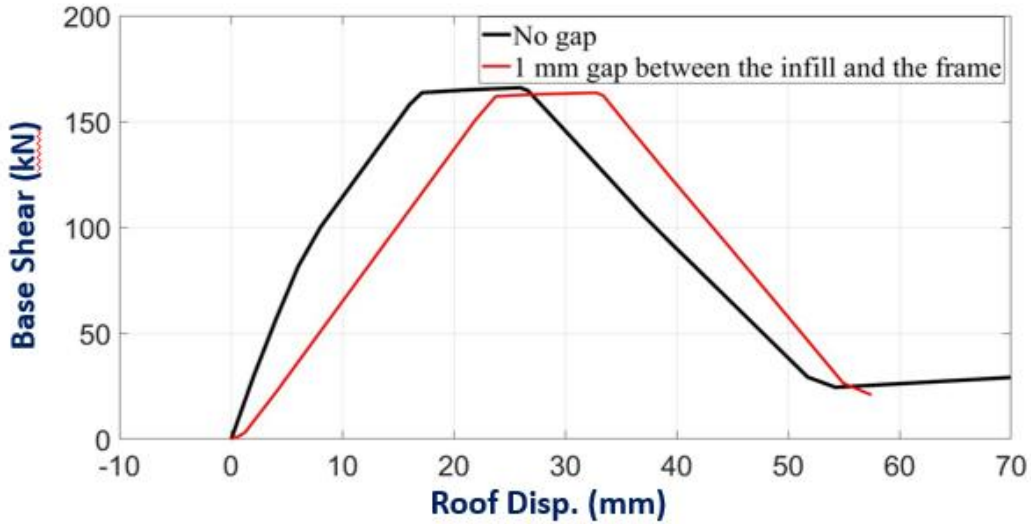
Example.1

We can define the gap between infill and the frame as well.
Considering a small gap (1 to 5mm) is recommended to avoid
unrealistic axial loads on the struts during gravity loading





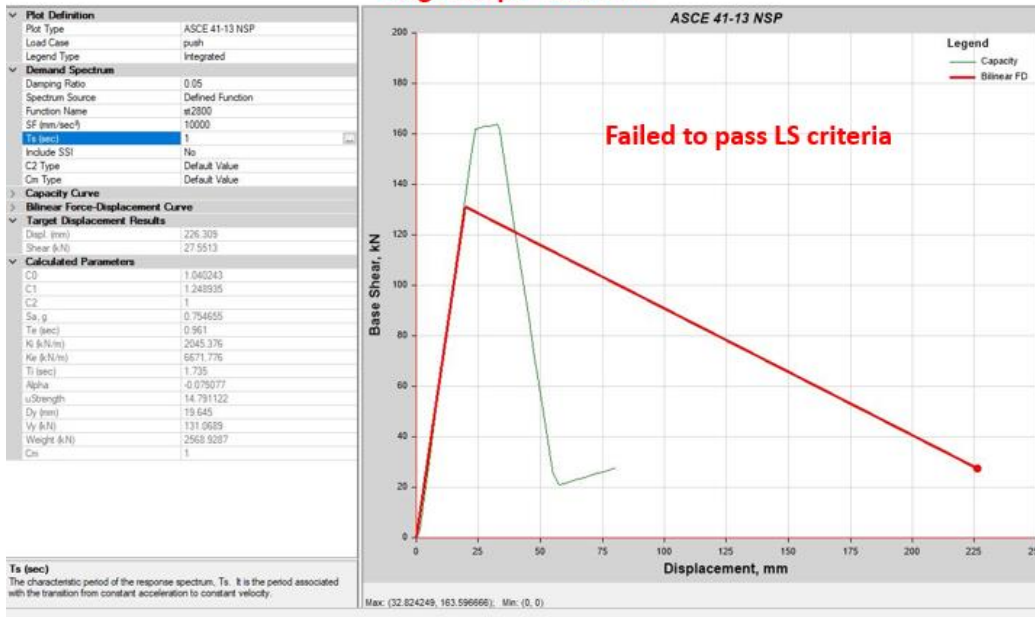
Example.1



Example.1

Level I Seismic hazard (return period of 475 years or 10%-50 years)

Target disp. = 226 mm

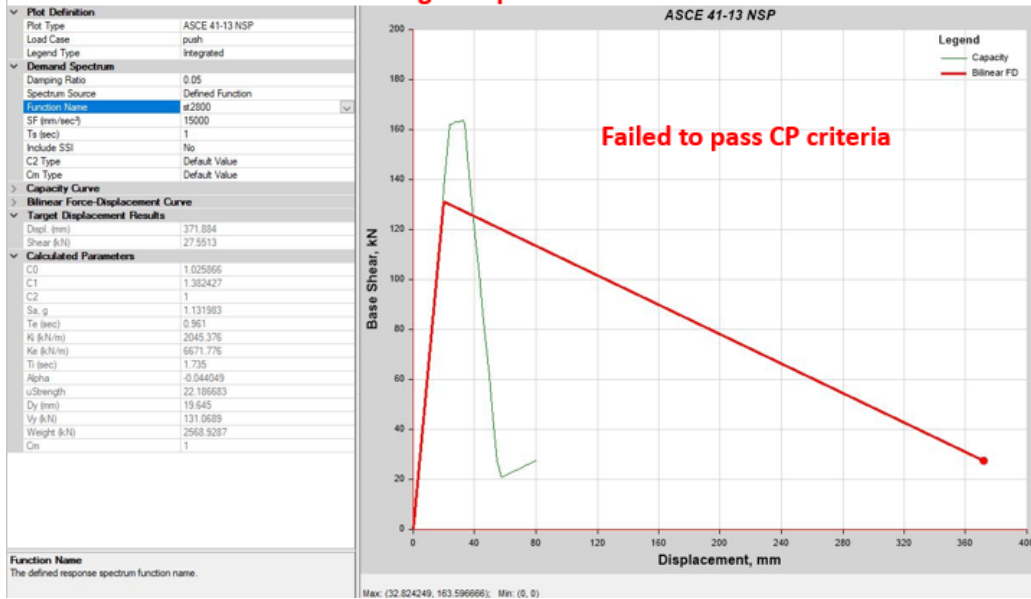




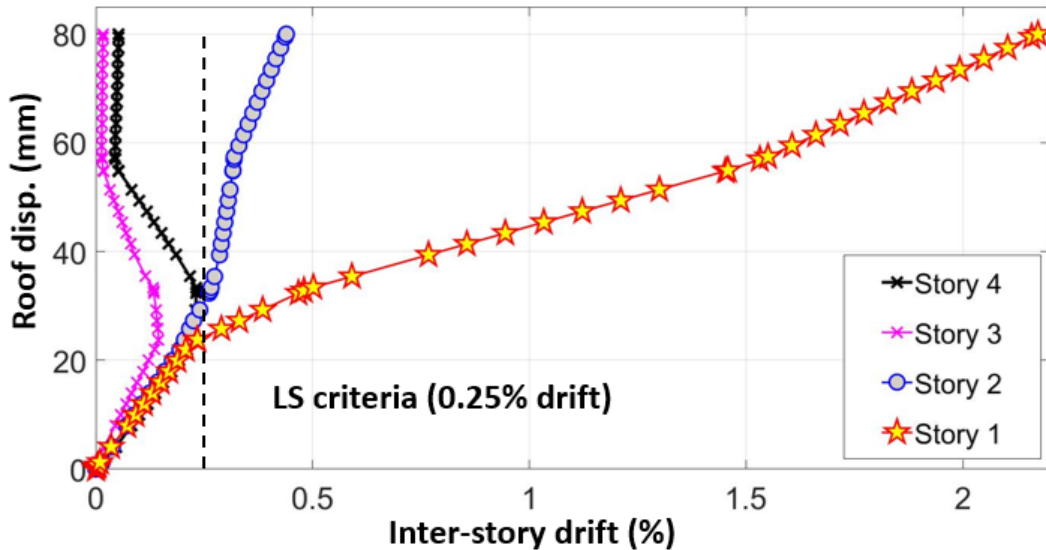
Example.1

Level II Seismic hazard (return period of 2500 years or 2%-50 years)

Target disp. = 371 mm



Example.1

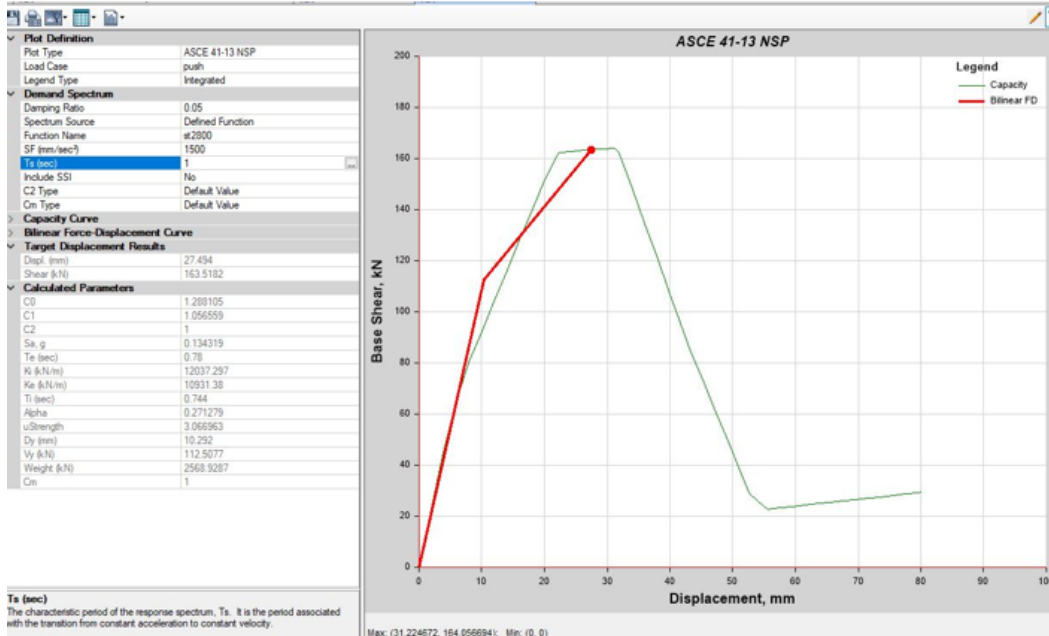


After roof disp. Of about 30 mm , significant deviation has occurred between drift of the first story and that of other stories indicating that a soft story mechanism would occur once the frame experience a roof displacement of more than about 30 mm

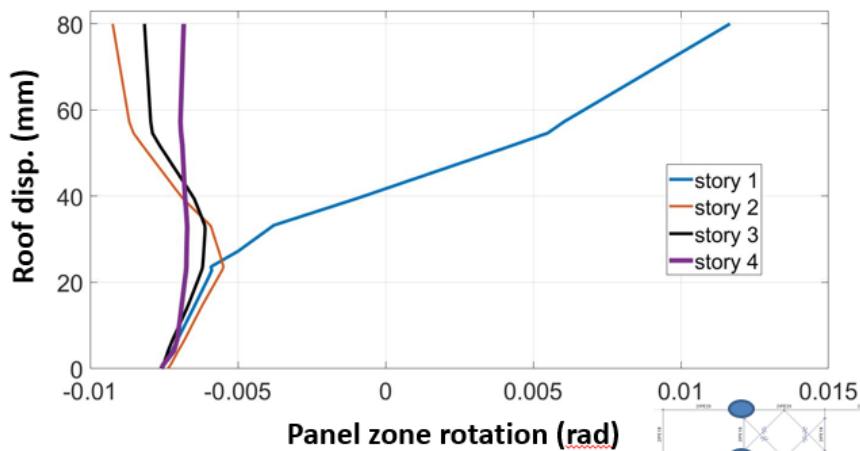


Example.1

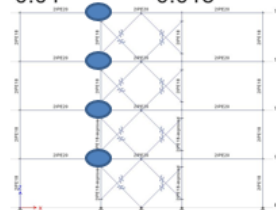
This frame can only sustain (in LS limit) an earthquake 6.5 times smaller than the design earthquake



Example.1



Panel zone rotation of this kind of Khorjini connection should be less than 0.01 rad for LS and 0.02 rad for CP criteria



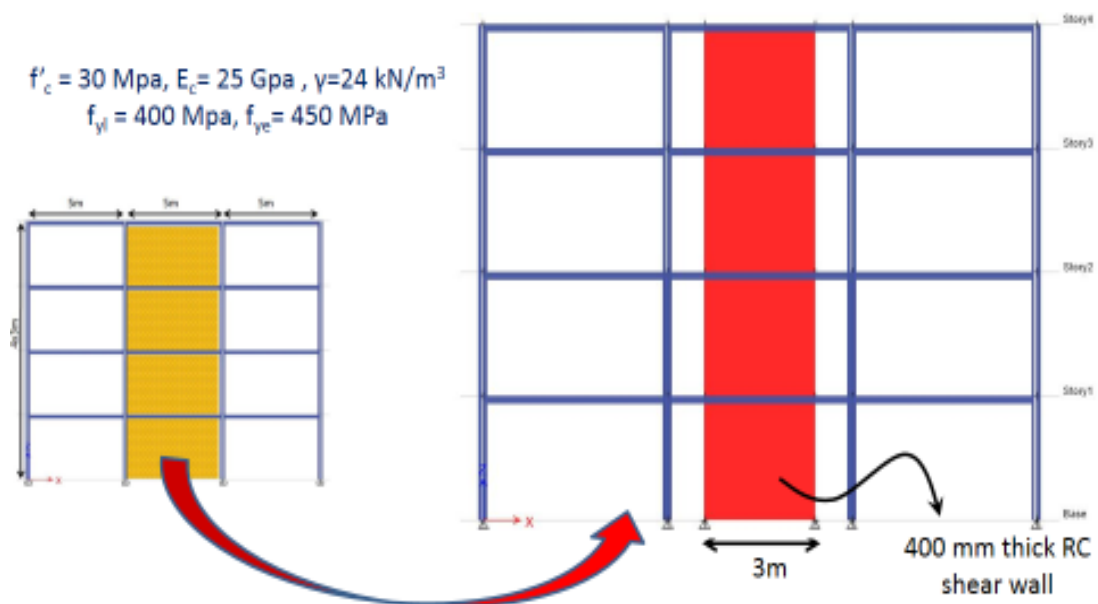
۲. قاب ساده خرجینی + دیوار برشی

مشابه حالت ۱ / بجای میانقاب از یک دیوار برشی لبه‌دار به ضخامت ۰/۴، طول ۳ متر و میلگرد طولی ۰/۵٪ استفاده شود.
خروجی‌ها:

۱. جابجایی هدف در خطر ۱ و ۲
۲. مدل دیوار برشی / مقاومت‌های برشی و خمشی، سختی، نمودار غیرخطی
۳. وضعیت لولای خمیری در دیوار برشی در خطر ۱ و ۲: لولای خمشی یا برشی؟
۴. دوران دیوار برشی در خطر ۱ و ۲
۵. نمودار رانش
۶. نمودار دوران اتصال برحسب جابجایی بام
۷. نمودار لولای خمشی در دیوار برشی برحسب جابجایی بام
۸. در صورتی که دیوار برشی بر روی دو شمع تکیه کرده باشد نیروهای فشاری و کششی در این شمع‌ها در حالت اوج

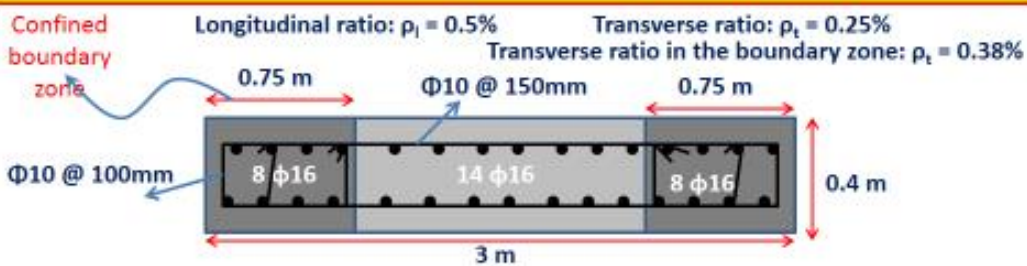
Example 2 Steel frame with Khorjini Connection and Shear Wall

Exactly similar to Example 1 (improved columns) but masonry infill is replaced by a RC shear wall

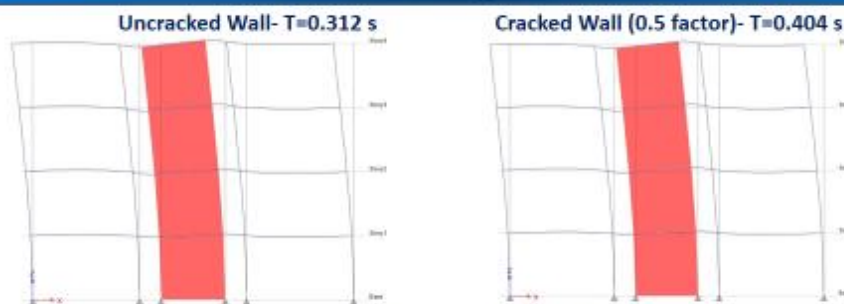




Example 2

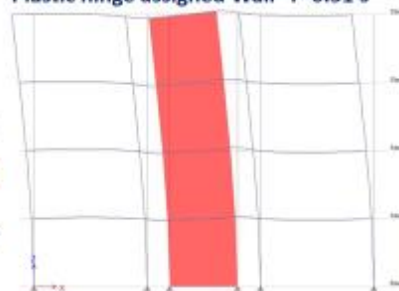


Example 2



Plastic hinge assigned Wall- T=0.31 s

Once you define a fiber plastic hinge for the wall, ETABS would use the defined fiber section and consider uncracked section, regardless of the modified stiffness you have previously defined



In this case, reinforcement details would affect stiffness and period of the structure



Example 2

Very important: Plastic hinges can be developed in any section along the beam length, especially in the case of unconventional structures like the one we are considering

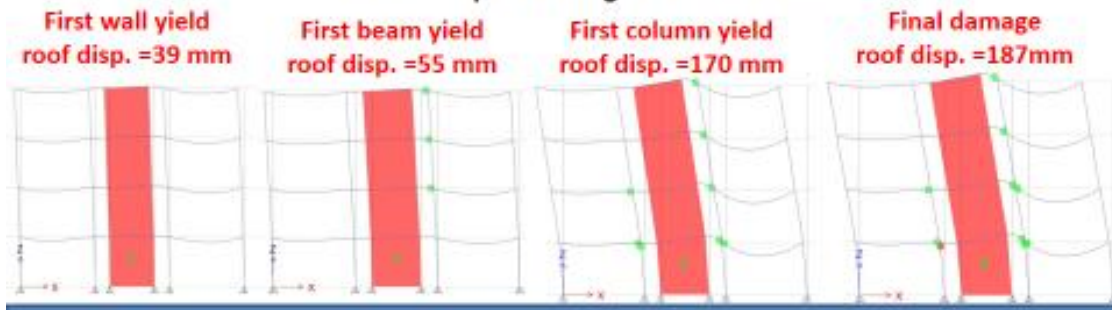
We consider two case:

Case 1 (wrong assumption): plastic hinges can be developed only at beam ends

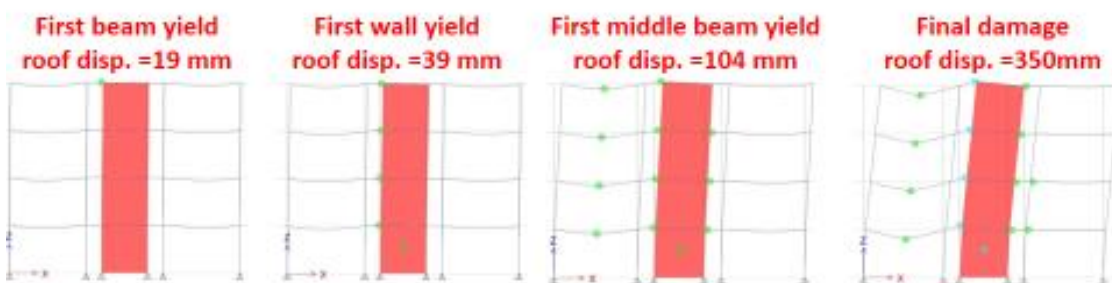
Case 2 (rational assumption): plastic hinges can be developed not only at the ends but also at the middle of the beam and just before wall-beam connection

Example 2

Case 1: 2 plastic hinge at beam ends

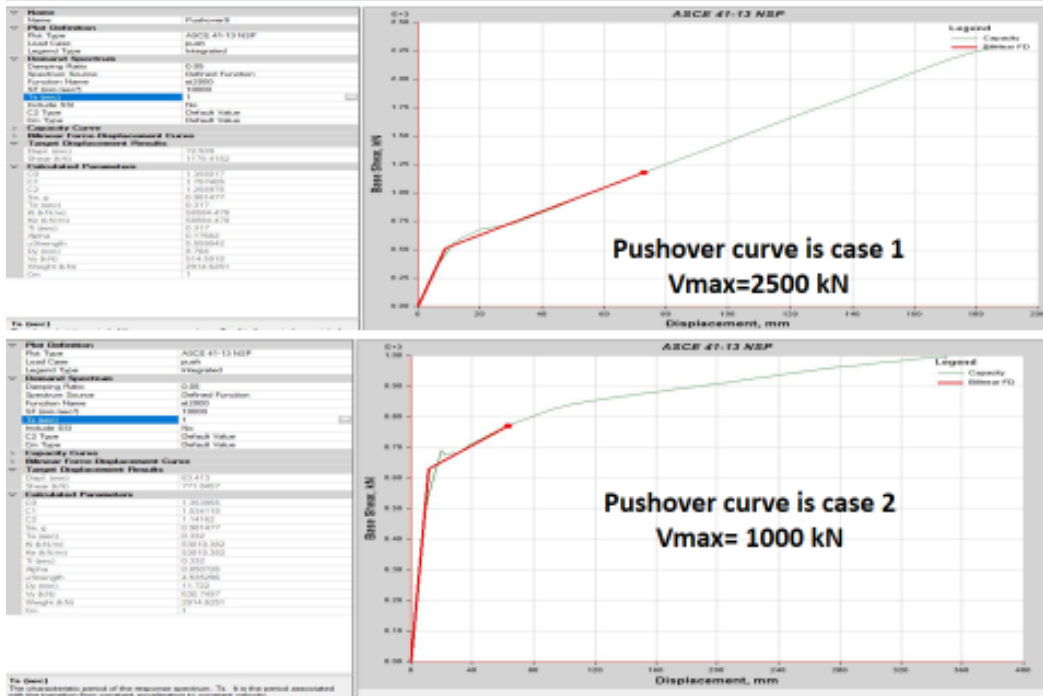


Case 2: 5 plastic hinges at each beam



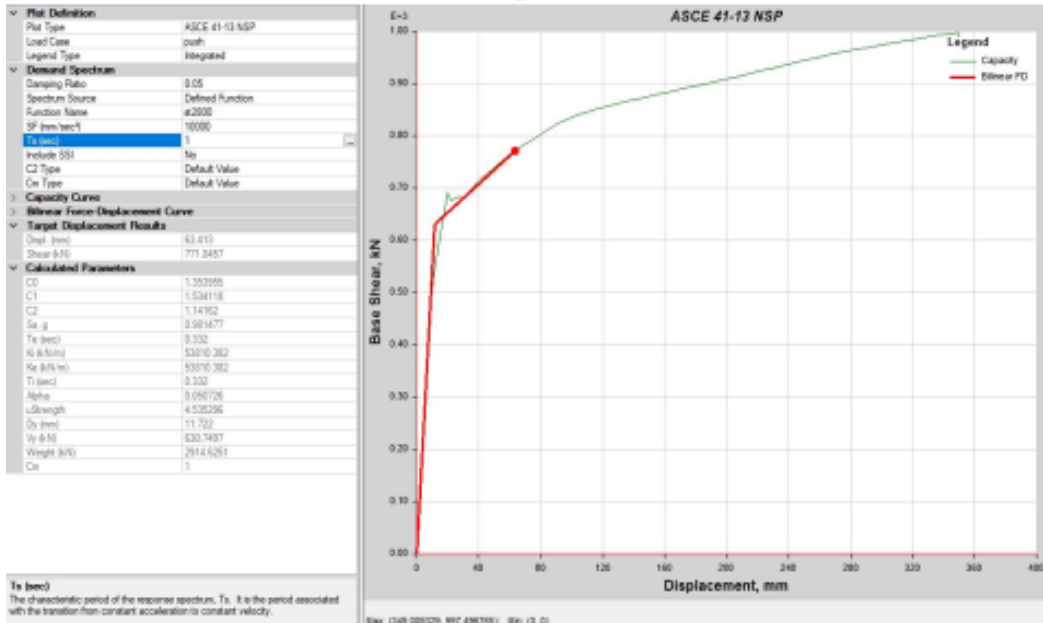


Example 2



Example 2

Under Seismic hazard I
roof. Disp.= 63mm





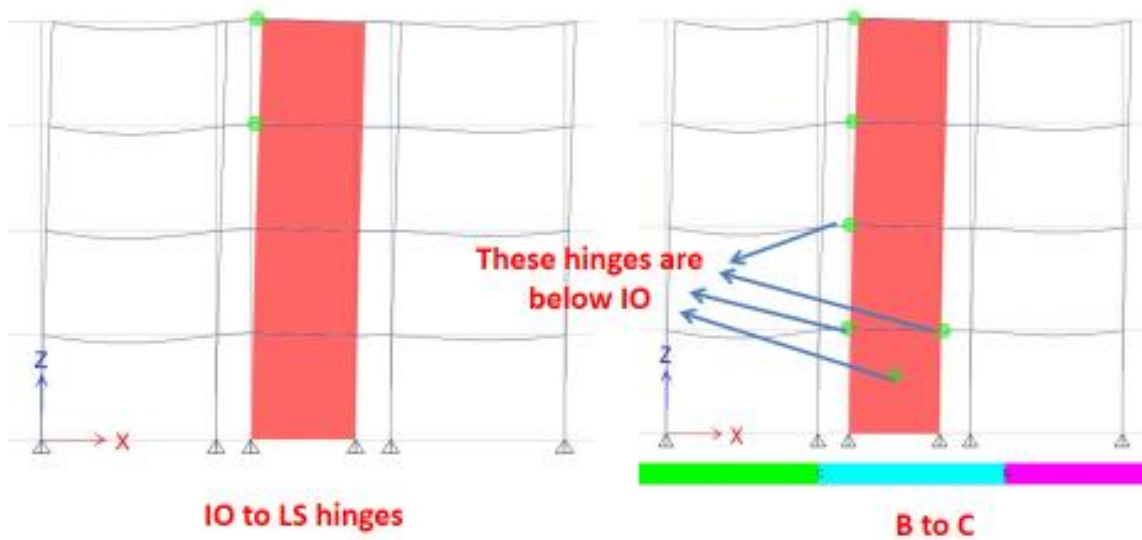
Example 2

So wrong plastic hinge placement can lead to significantly misleading results

Hereafter, let's consider only the true model (case 2 with distributed plasticity along the beams)

Example 2

**Under Seismic hazard I
roof. Disp.= 63mm**

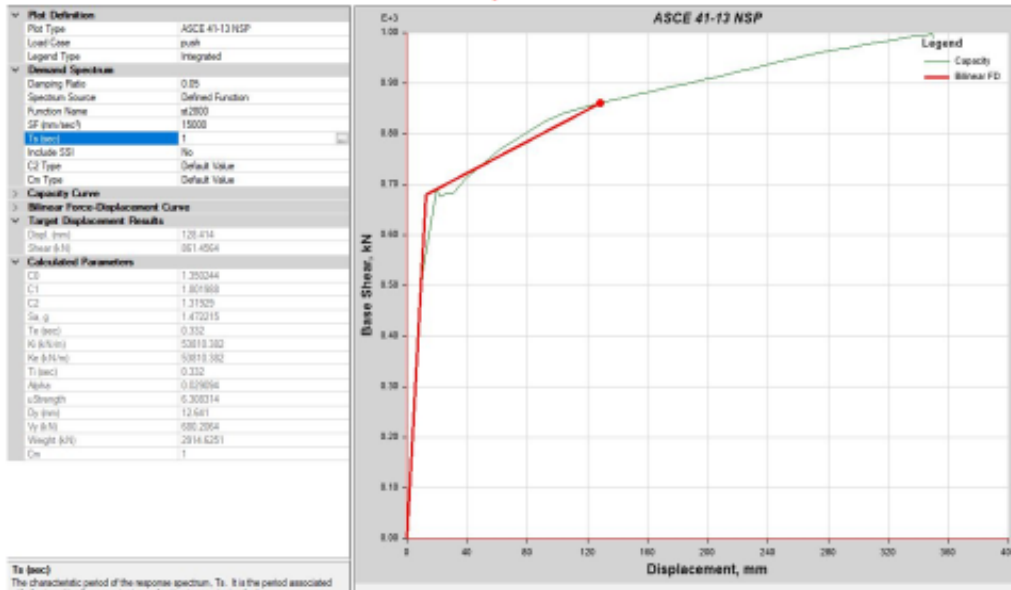


**The structure satisfied the LS criteria under
Seismic hazard level I**



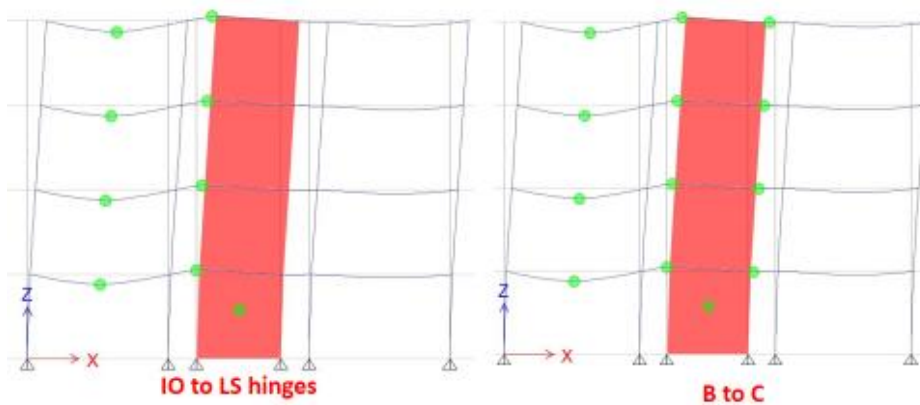
Example 2

Under Seismic hazard II
roof. Disp.= 128mm



Example 2

Under Seismic hazard II
roof. Disp.= 128mm

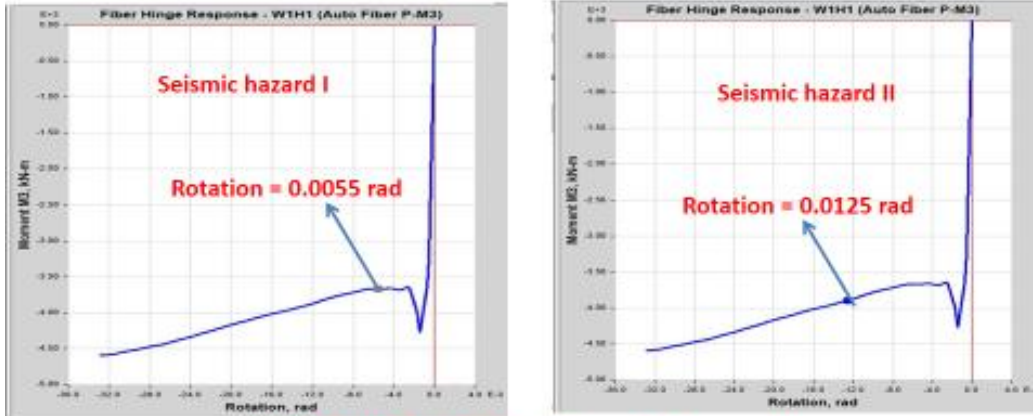


The structure satisfied not only the CP but also
the LS criteria under Seismic hazard level II



Example 2

Wall plastic hinge



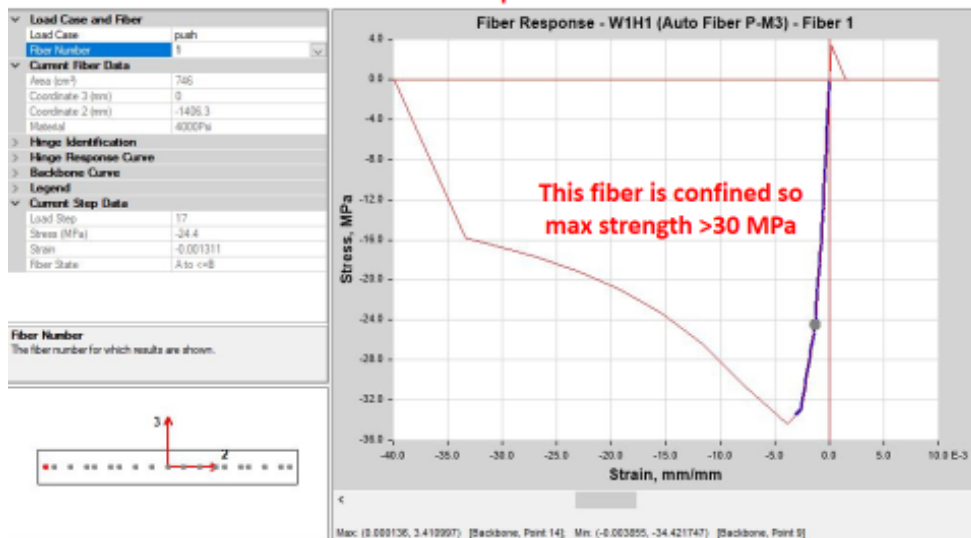
ASCE 41-13's Table 10-19 for flexural RC shear walls

Conditions	Plastic Hinge Rotation (radians)		Strength Ratio	Performance Level				
	a	b		ID	LS	CP		
i. Shear walls and wall segments $(A_c - A_c') f_c + P$ $\frac{V}{f_c A_c} \leq \frac{V}{f_c' \sqrt{A_c}}$	Confined Boundary ^b		0.015					
SD, I	54	Yes	0.010	0.020	0.75	0.005	0.015	0.020

Example 2

Seismic hazard II = Step 17

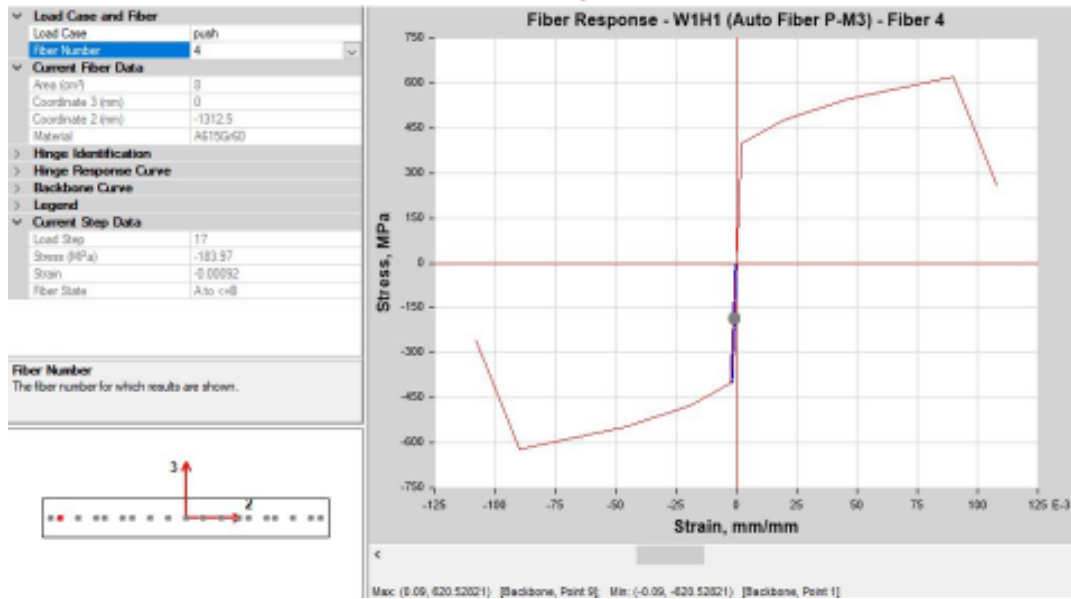
The most critical compressive concrete fiber



Example 2

Seismic hazard II = Step 17

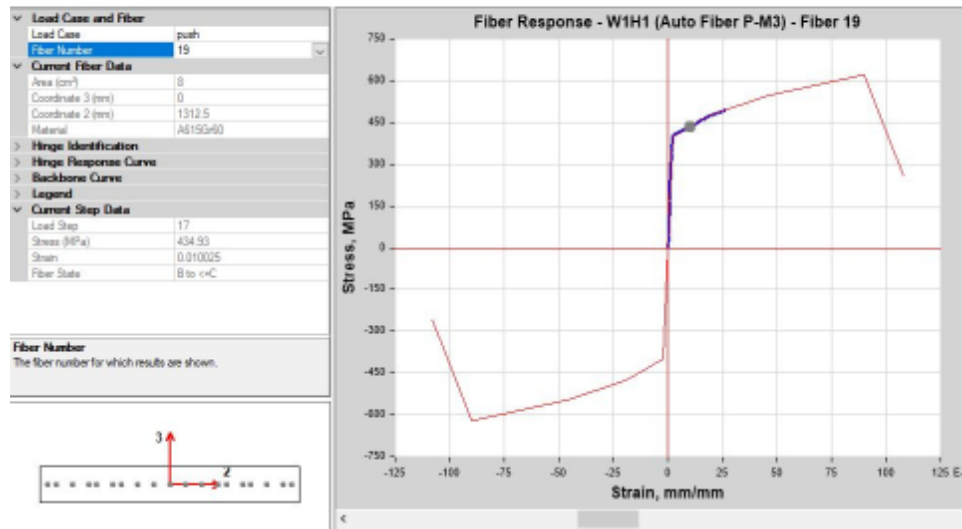
The most critical compressive rebar fiber



Example 2

Seismic hazard II = Step 17

The most critical tensile rebar fiber



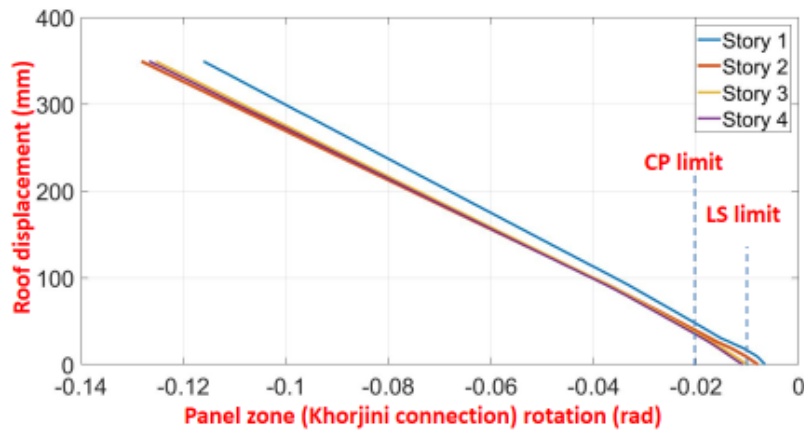


Example 2

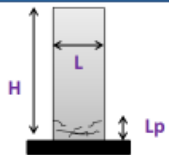
ETABS use strain based criteria to define IO, LS, and CP criteria
The results are not necessarily the same as ASCE 41's or Code 360's Tables which are based on plastic rotations. But in the case of well detailed flexural walls the results are rather in agreement.

Example 2

So Khorjini connections cannot sustain the imposed rotations. However, these connections are secondary component and their failure does not mean lateral instability. However, stability under gravity loads should be assessed.



Example 2



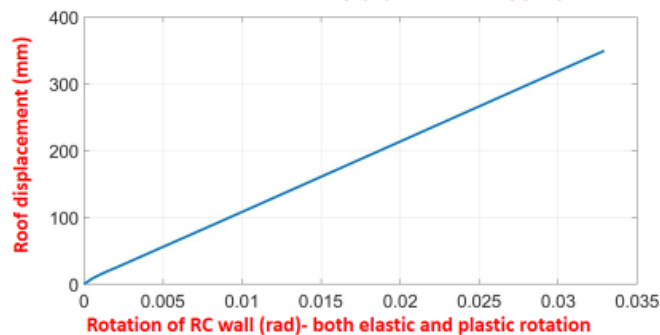
$$\text{Roof disp.} = \theta y H / 3 + \theta p (H - 0.5 L_p)$$

$$L_p = \min(0.5 L, \text{story height})$$

$$L_p = 1.5 \text{ m}$$

$$\theta_p = \left(\frac{M_p}{E_p I_p} \right) l_p \Rightarrow = 0.0025 \text{ rad}$$

$$\text{Roof disp. (m)} = 0.01 + 10.5 \theta_p \text{ (rad)}$$

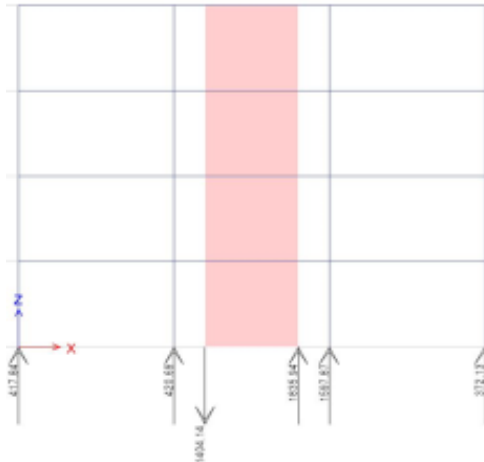




Example 2

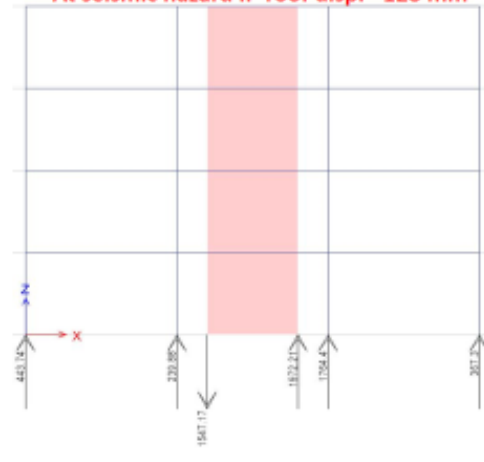
Axial demands on the piles at the two edges of the wall
SSI is not considered in this example. If SSI was considered, these demands would be different, probably smaller.

At seismic hazard I- roof disp. =63 mm



$P_t = 1404 \text{ kN}$ $P_c = 1836 \text{ kN}$

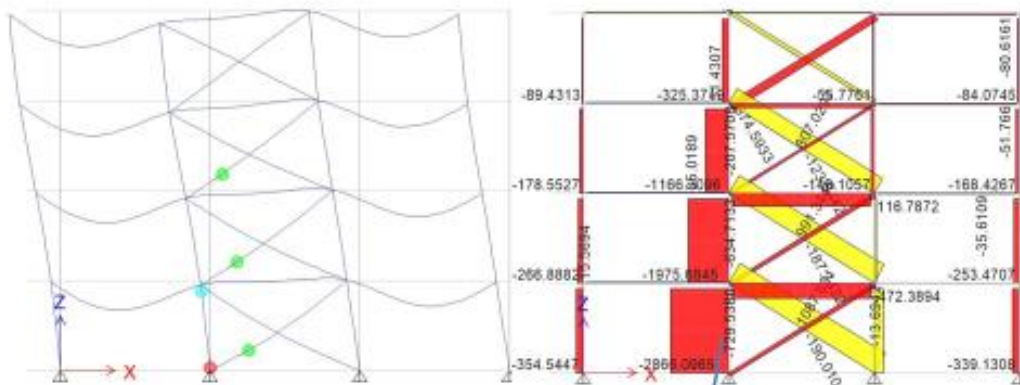
At seismic hazard II- roof disp. =128 mm



$P_t = 1547 \text{ kN}$ $P_c = 1972 \text{ kN}$

Example 3

The column failed after only roof disp. Of 40 mm



Axial load = 2866 kN
Axial lower-bound capacity = PCL=2520 kN

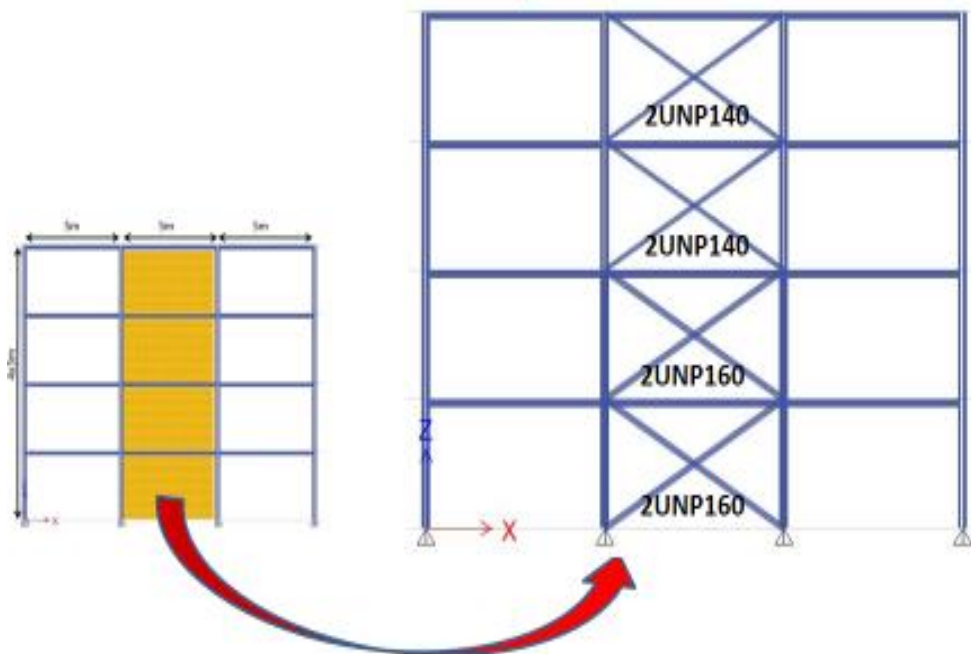
۳. قاب ساده خرچینی + مهاربند ضربدری

مشابه حالت ۱ / بجای میانقاب از مهاربند ضربدری / دابل ناودانی ۱۶ در دو طبقه اول و دابل ناودانی ۱۴ در طبقات ۳ و ۴. خروجی‌ها:

۱. جابجایی هدف در خطر ۱ و ۲
۲. مدل مهاربندهای کششی و فشاری / مقاومت‌های فشاری و کششی، نمودار غیرخطی
۳. وضعیت لولای خمیری در مهاربندهای فشاری و کششی در خطر ۱ و ۲
۴. تغییر شکل محوری مهاربندهای کششی و فشاری در خطر ۱ و ۲
۵. نمودار رانش
۶. نمودار دوران اتصال خرچینی برحسب جابجایی بام
۷. نمودار تغییر شکل محوری مهاربندهای کششی و فشاری برحسب جابجایی بام
۸. نمودار نیروی محوری ستون‌های مجاور مهاربند در طبقات اول و دوم

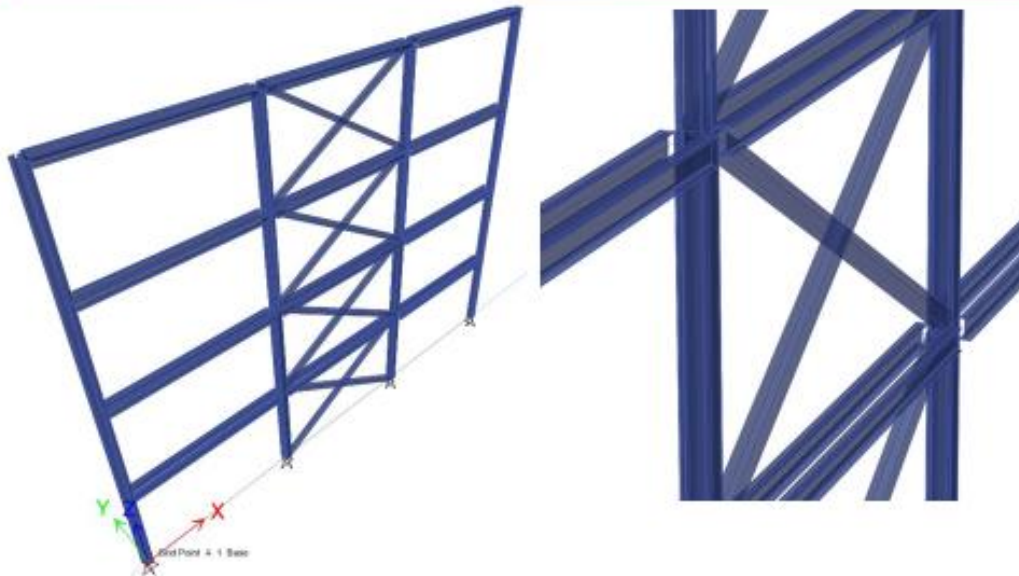
Example 3 Steel frame with Khorjini Connection and Cross Bracing

Exactly similar to Example 1 (improved columns) but masonry infill is replaced by a X-shape Braces





Example 3




Exactly similar to Example 1 (improved columns) but masonry infill is replaced by a X-shape Braces

Example 3

Table B.4.1 (AISC 360-05)

b/t	$1.12\sqrt{E/F_y}$	$1.40\sqrt{E/F_y}$	
	compact	noncompact	

Table D.1.1 (AISC 341-10)

b/t	$0.55\sqrt{E/F_y}^{(a)}$	$0.84\sqrt{E/F_y}^{(c)}$	
	Highly ductile member	Moderately ductile member	

^aSection compactness: Acceptance criteria applies to brace sections that are concrete-filled or seismically compact according to Table D1.1 of AISC 341 for highly ductile members. Where the brace section is noncompact according to Table B4.1 of AISC 360, the acceptance criteria shall be multiplied by 0.5. For intermediate compactness conditions, the acceptance criteria shall be multiplied by a value determined by linear interpolation between the seismically compact and the noncompact cases.

If $b/t <$ highly ductile compactness



Use the defined acceptance criteria in the ASCE 41 Table

If $b/t >$ compact



Multiply the above criteria by 0.5

For b/t in between above limits



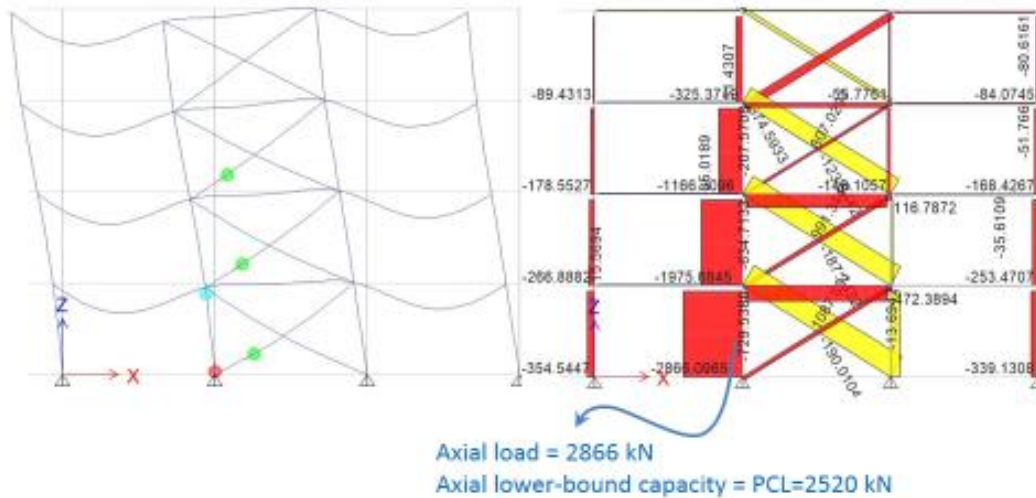
Use interpolation

For poorly detailed gusset plates, acceptance criteria should be multiplied by 0.8



Example 3

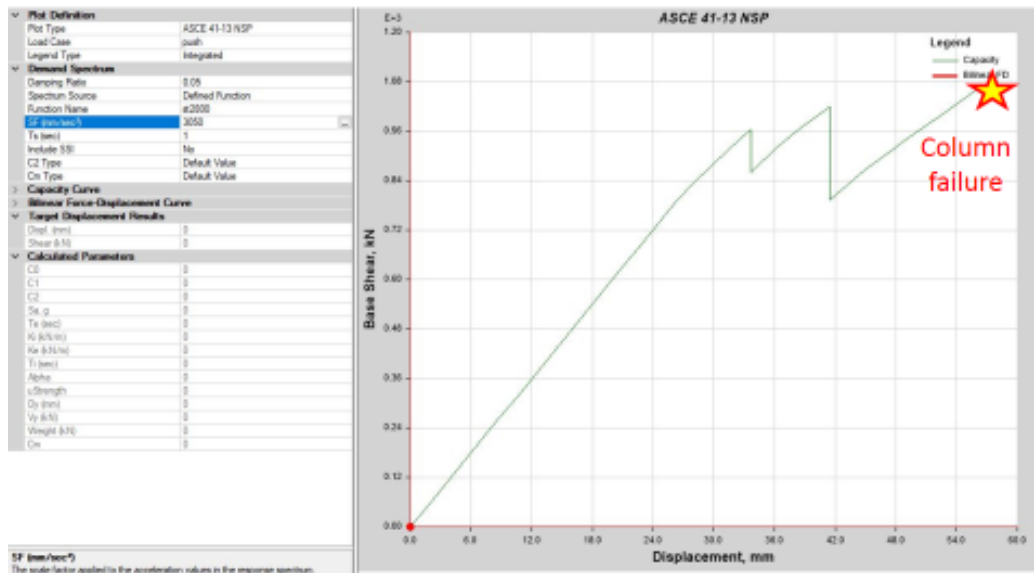
The column failed after only roof disp. Of 40 mm



Example 3

Pushover curve

This frame can sustain only 30% of level I seismic hazard. i.e. an earthquake with a return period of about 10 years





موسسه آموزشی و مهندسی ۸۰۸
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Example 3

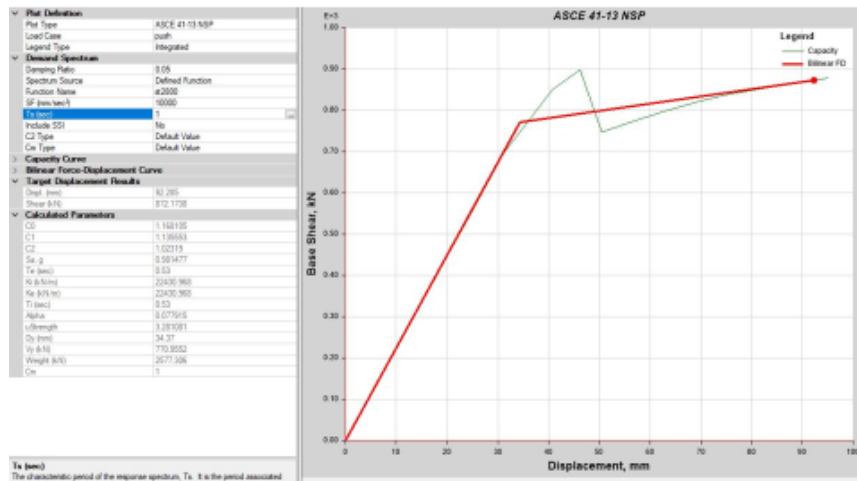
Let's consider weaker braces

2UNP100 for 1st story and 2UNP80 for other stories

Now, the columns can sustain additional axial demands and would fail after the braces

Example 3

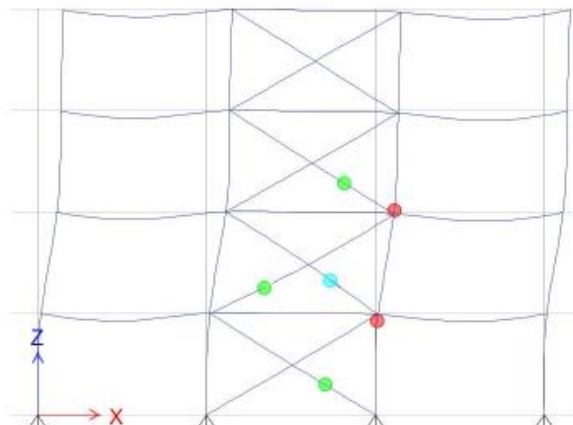
Seismic hazard I
Target disp. = 92 mm



Example 3

Seismic hazard I
Target disp. = 92 mm

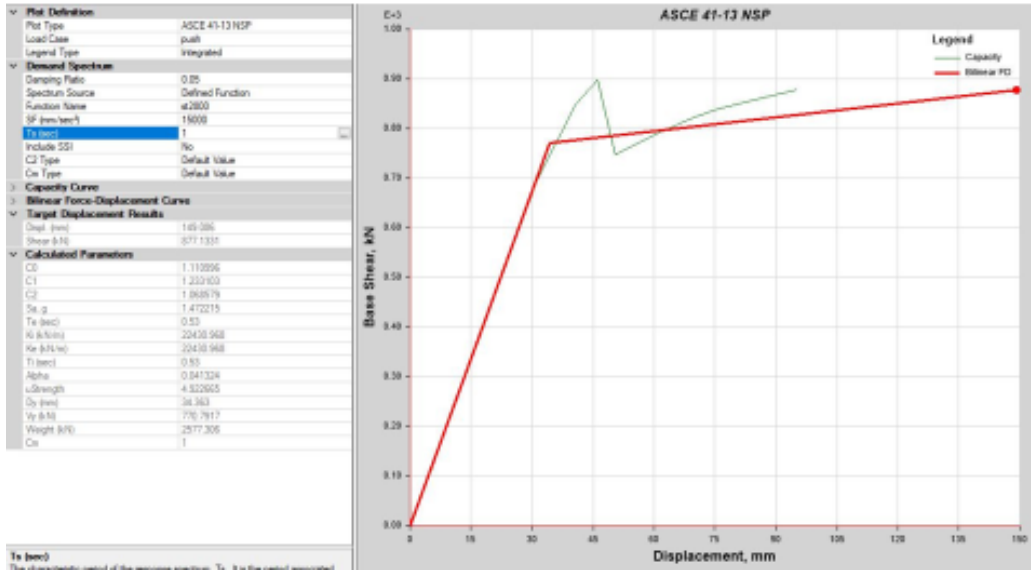
Some of column and braces cannot satisfy LS criteria





Example 3

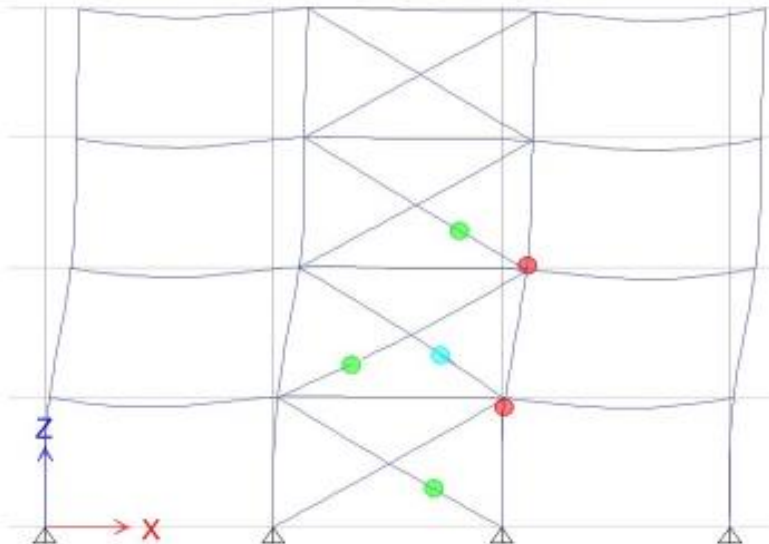
Seismic hazard II
Target disp. = 149 mm



Example 3

Seismic hazard II
Target disp. = 149 mm

The frame cannot satisfy CP criteria...it failed at roof disp. of 95 mm



Example 3

Brace plastic hinges are user-defined

1. Obtained tensile and compressive strength of the braces (from design capability of the program). Consider unbraced length modifications.
2. Increase obtained strength due to expected yield strength as well as unit strength reduction factor. For example, if the tensile strength from design is 500 kN. And expected to specified yield strength is 1.1, then the expected tensile strength would be $500 \times 1.1 / 0.9 = 611$ kN (0.9 is the reduction factor)
3. Obtain axial stiffness of the brace (EA/L)
4. Obtain tensile and compressive yield displacement
5. Input the strength and yield displacement in the Table of the plastic hinge definition

Example 3

Defined plastic hinge for 2UNP100

Point	Force/SF	Diap/SF
E	-0.3	-9
D	-0.3	-3
B	-1	-0.5
A	-1	0
A	0	0
B	1	0
D	1.25	9
D	0.8	10
E	0.8	11

Symmetric

Load Carrying Capacity Beyond Point E
 Drops To Zero
 Is Extrapolated

Scaling for Force and Disp

	Positive	Negative
<input type="checkbox"/> Use Yield Force	Force SF: 758	547 kN
<input type="checkbox"/> Use Yield Disp (Steel Objects Only)	Dep SF: 8.1	5.5 mm

Acceptance Criteria (Plastic Disp/SF)

	Positive	Negative
<input checked="" type="checkbox"/> Immediate Occupancy	0.5	-0.5
<input type="checkbox"/> Life Safety	8	-7
<input type="checkbox"/> Collapse Prevention	11	-9

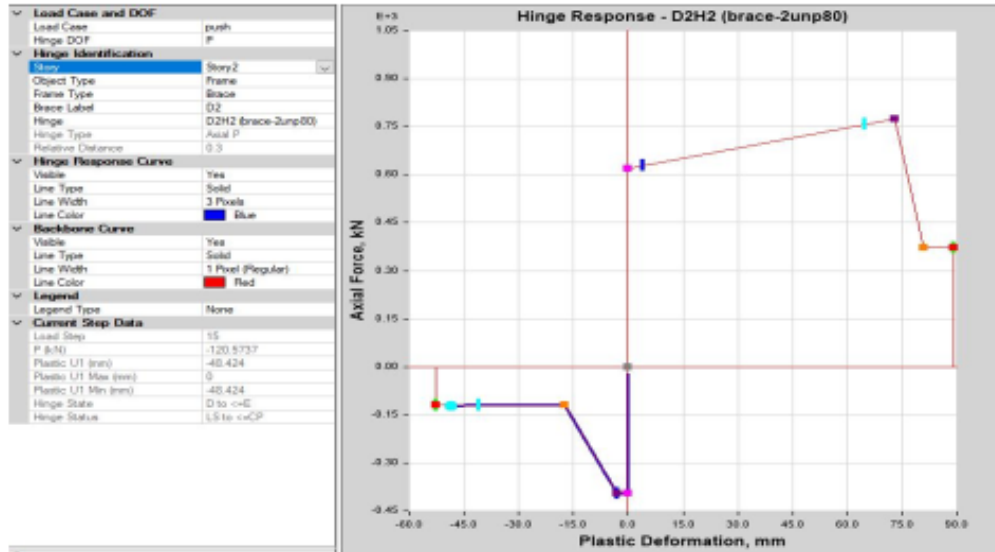
Show Acceptance Criteria on Plot

If you are using auto hinge, always check the generated hinges. Sometimes sign of the values on the table would not correctly generated.

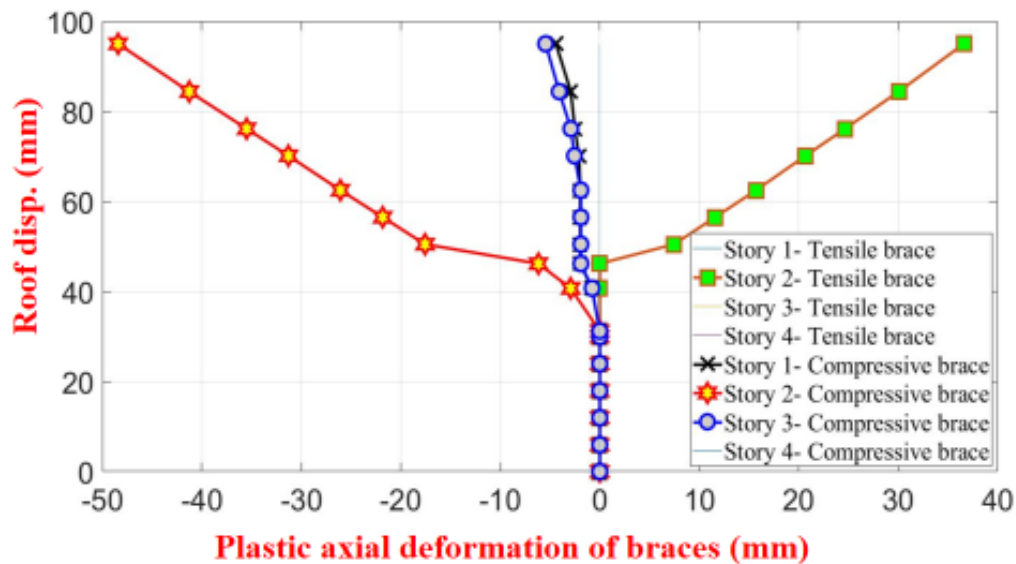


Example 3

Plastic hinge of the 2nd story brace (2UNP80)

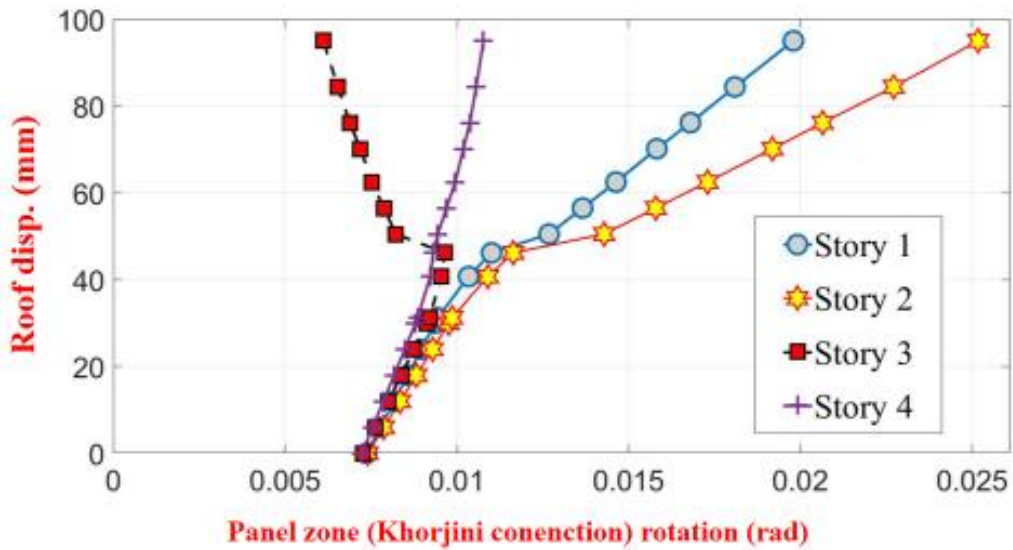


Example 3



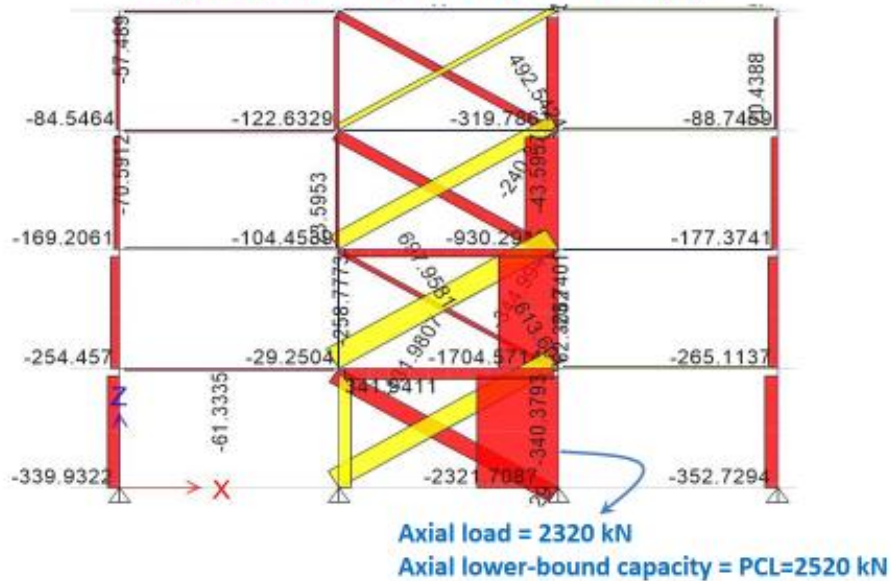


Example 3



Example 3

Seismic hazard I
Target disp. = 92 mm (Final step of the analysis)

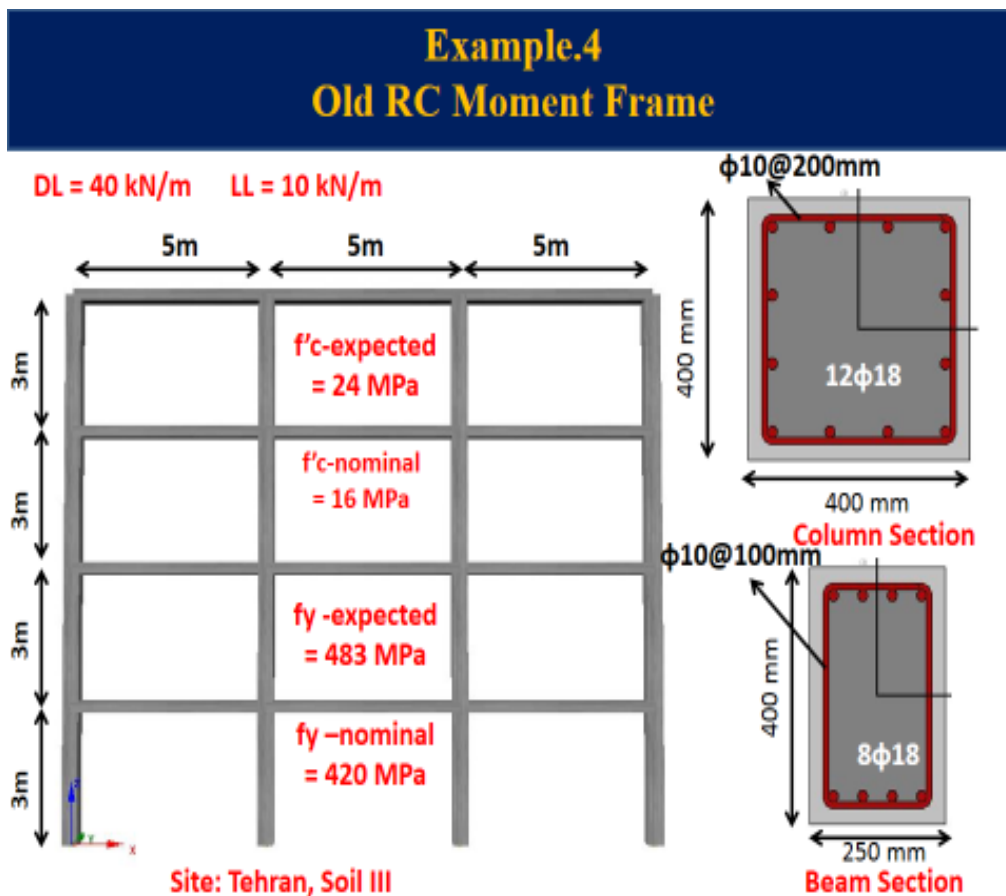




۴. قاب بتنی قدیمی (NC)

قاب ۴ طبقه سه دهانه / محل بنا در تهران، خاک نوع ۳، دهانه‌ها ۵ متر، ارتفاع همه طبقات ۳ متر، بار مرده و زنده طبقات به ترتیب برابر ۴ و ۱ تن بر متر/ ابعاد تیرها ۴۰۰×۲۵۰ م م و ستون‌ها ۴۰۰×۴۰۰ میلی‌متر. درصد میلگرد تیرها و ستون‌ها ۲٪ خروجی‌ها:

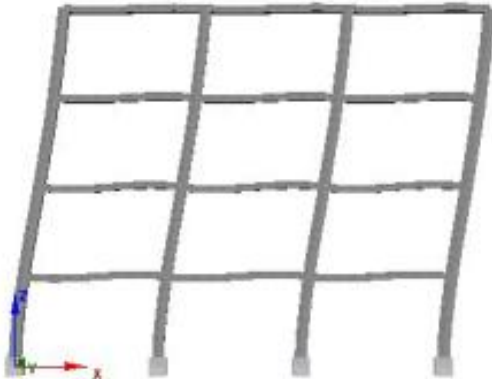
۱. جابجایی هدف در خطر ۱ و ۲
۲. مدل غیرخطی تیر/ مقاومت خمشی، نمودار غیرخطی
۳. مدل غیرخطی ستون/ مقاومت خمشی، نمودار غیرخطی
۴. وضعیت لولای خمیری بحرانی در تیرها در خطر ۱ و ۲
۵. وضعیت لولای خمیری بحرانی در ستون‌ها در خطر ۱ و ۲
۶. نمودار رانش
۷. نمودار دوران تیر برحسب جابجایی بام
۸. نمودار دوران تیر برحسب جابجایی بام



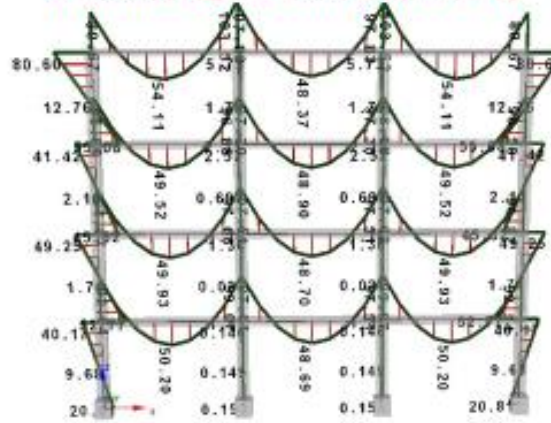


Example.4

1st mode period = 0.87 s



Moment diagram under 1.1(D+0.25L)



Example.4

Seismic hazard in ASCE 41-13 differs with that in Code 360. To comply with Code 360, BSE-2E should be defined to be equal to the Level I hazard (10%-50 years).

Materials Sections Element Classes Nodes Element Connectivity Constraints Restraints Applied Loads Loading Phases

Calculate Target Displacement (if checked, an eigenvalue analysis will run prior to the pushov

Code Employed in the Target Displacement Calculations

ASCE41-13 Control Node 22 Control Direction x

Performance Levels Seismic Action

Performance Levels Select one or more performance levels to be used in the

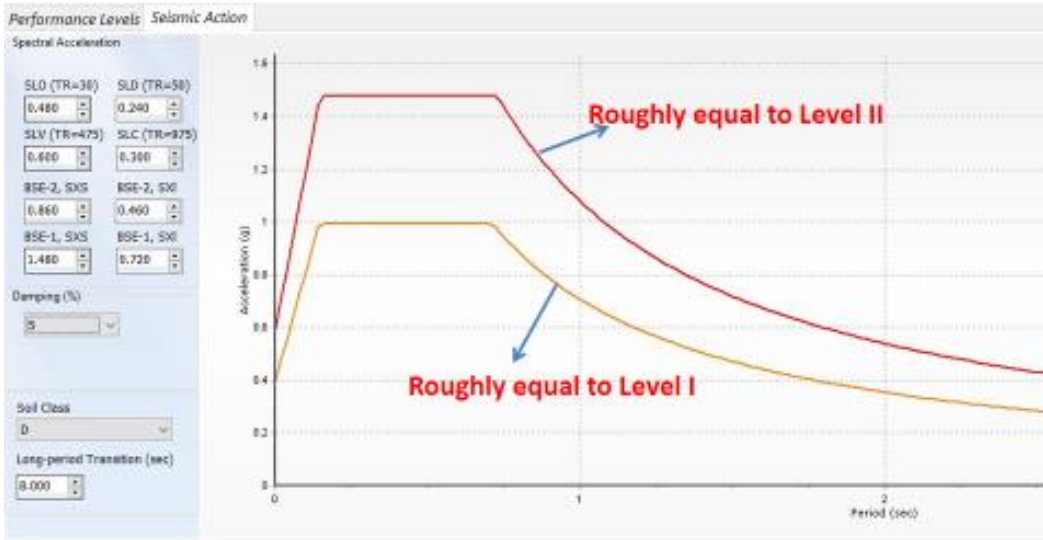
ASCE 41-13. Table C2-2: Rehabilitation Objectives

		Target Building Performance Levels			
		(1-A)	(1-B)	(3-C)	(5-D)
Earthquake Hazard Level	50%/50 years	<input type="checkbox"/> a	<input type="checkbox"/> b	<input type="checkbox"/> c	<input type="checkbox"/> d
	BSE-1E (20%/50	<input type="checkbox"/> e	<input type="checkbox"/> f	<input type="checkbox"/> g	<input type="checkbox"/> h
	BSE-2E (5%/50 years)	<input type="checkbox"/> i	<input type="checkbox"/> j	<input checked="" type="checkbox"/> k	<input type="checkbox"/> l
	BSE-2H (2%/50 years)	<input type="checkbox"/> m	<input type="checkbox"/> n	<input type="checkbox"/> o	<input checked="" type="checkbox"/> p



Example.4

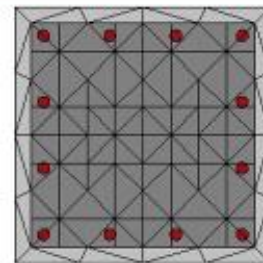
Note that these curves are from ASCE 41 which are similar to ASCE 7. target spectra in Code 360 is similar to that in Standard 2800. the target spectra of Standard 2800 and ASCE 7 differs a little, especially during longer periods



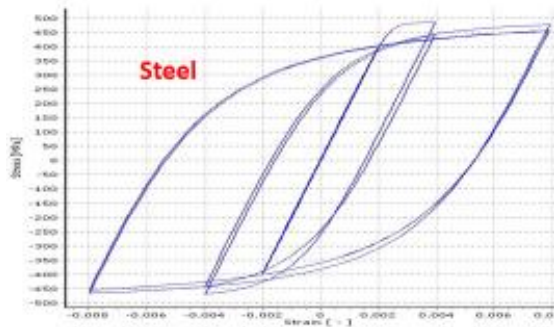
Example.4



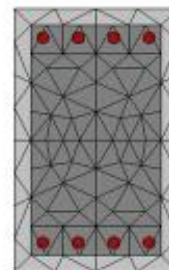
Concrete



150 fibers for column section



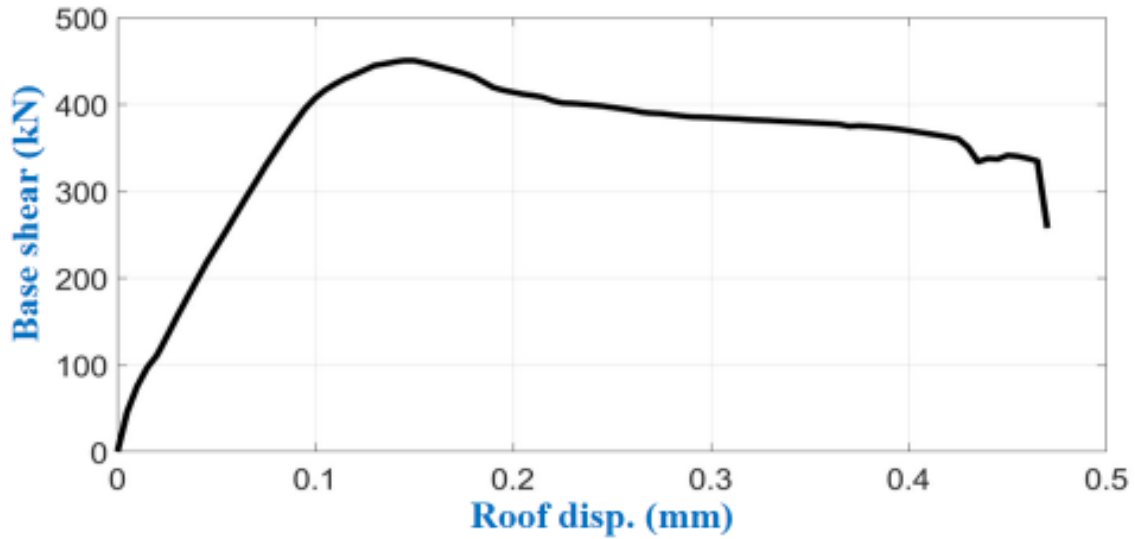
Steel



150 fibers for beam section



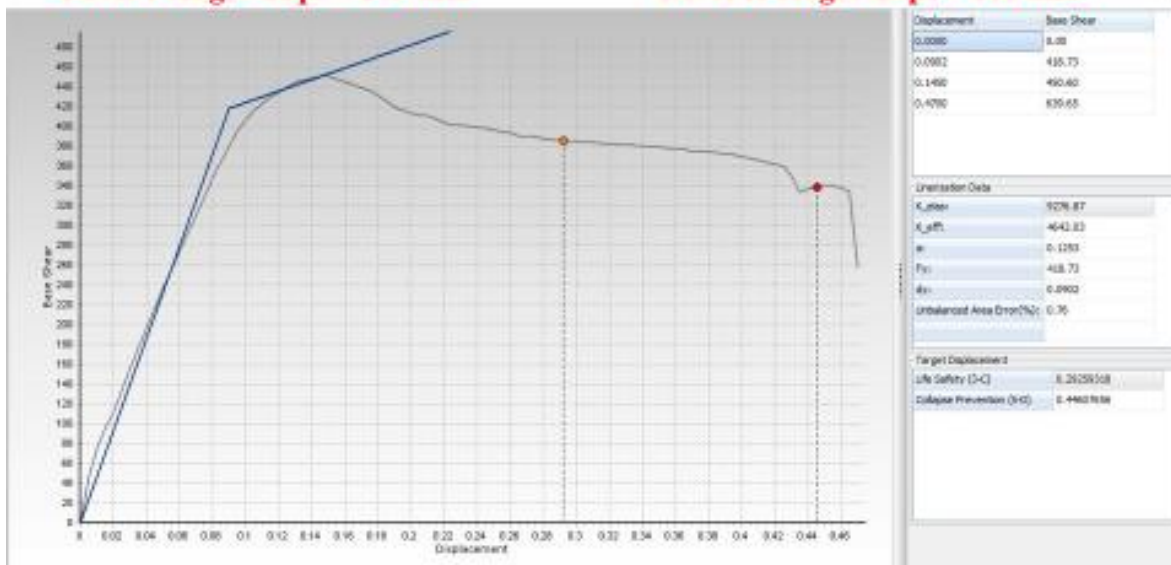
Example.4



Example.4

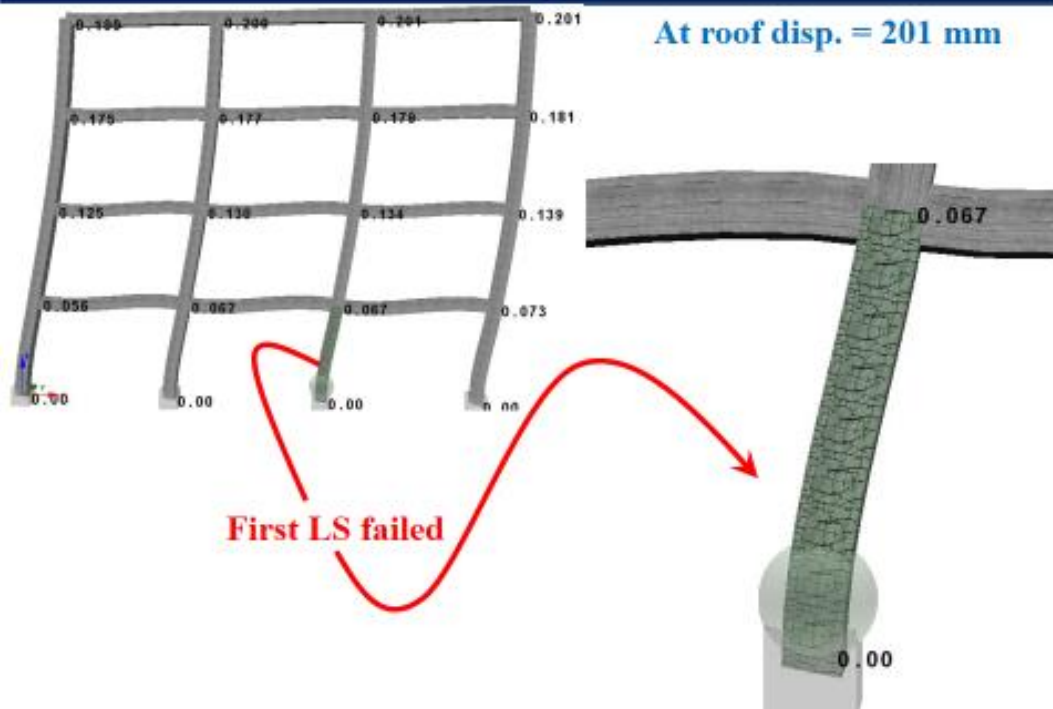
Level I target disp. = 293 mm

Level II target disp. = 446 mm



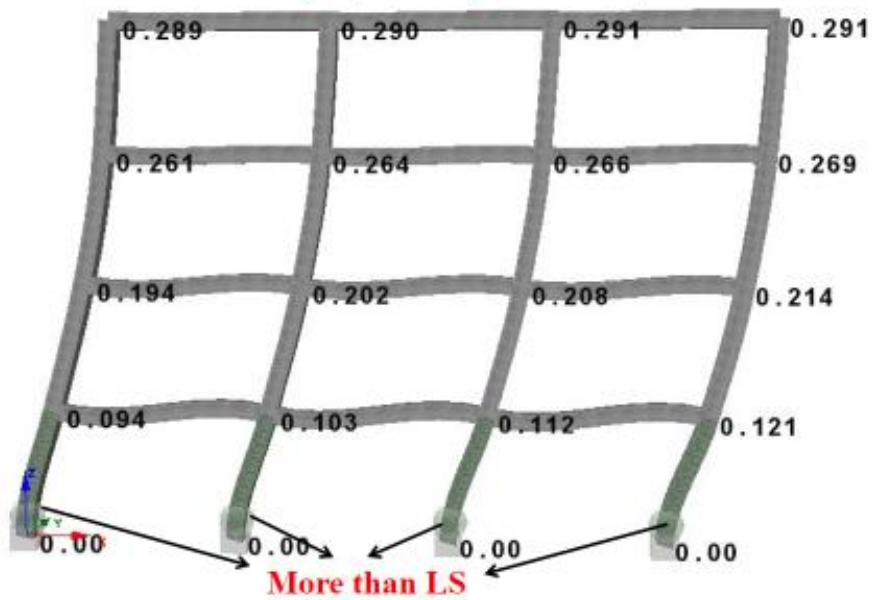


Example.4



Example.4

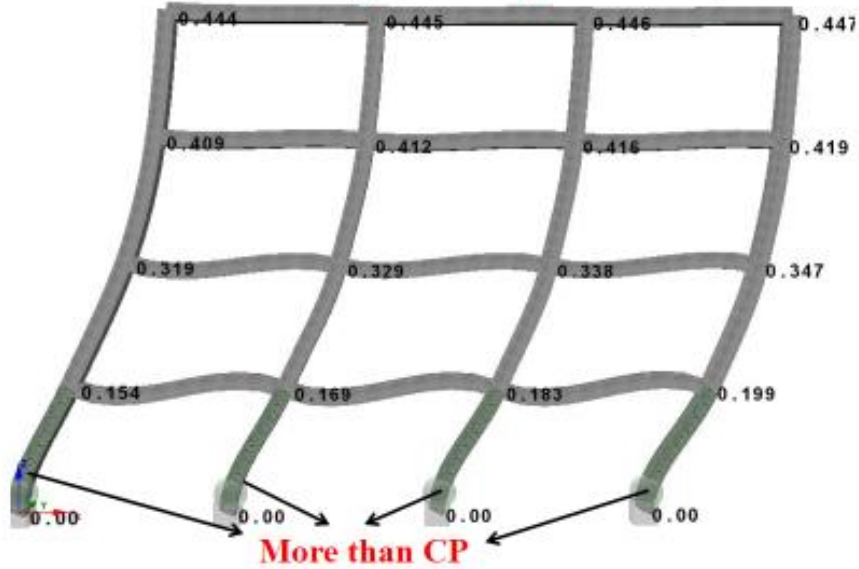
At level I target (roof disp. = 291 mm)



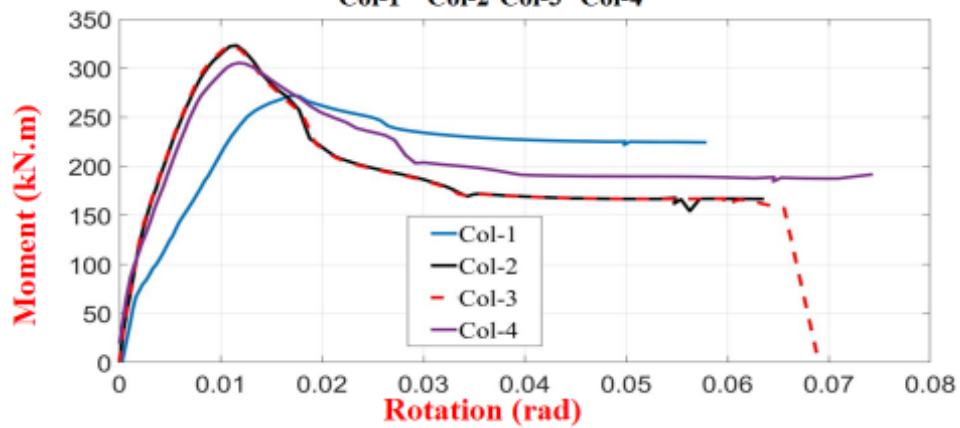
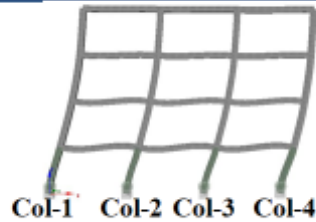


Example.4

At level II target (roof disp. = 446 mm)

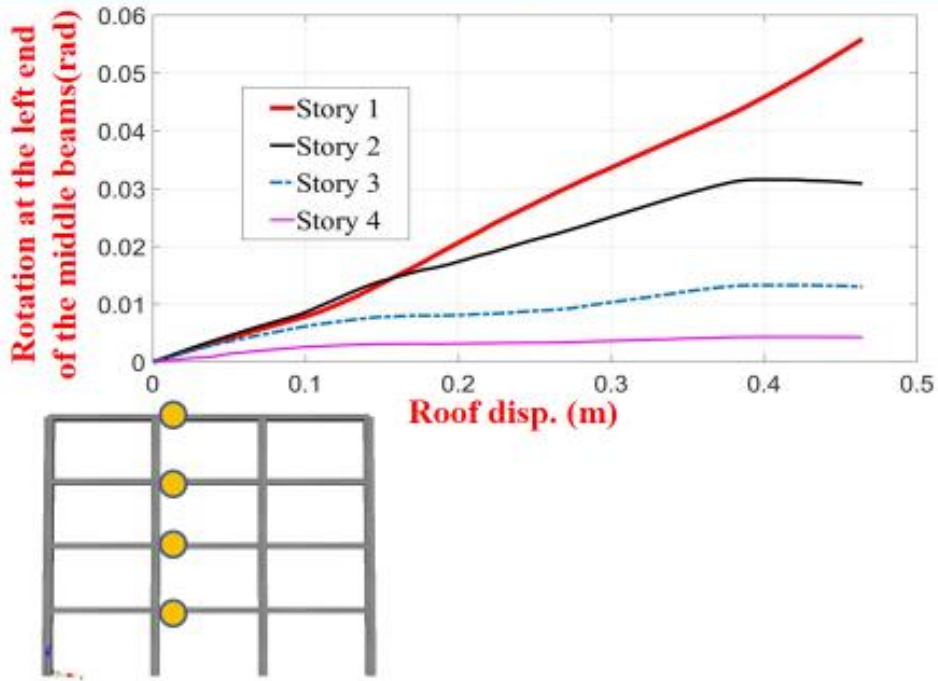


Example.4

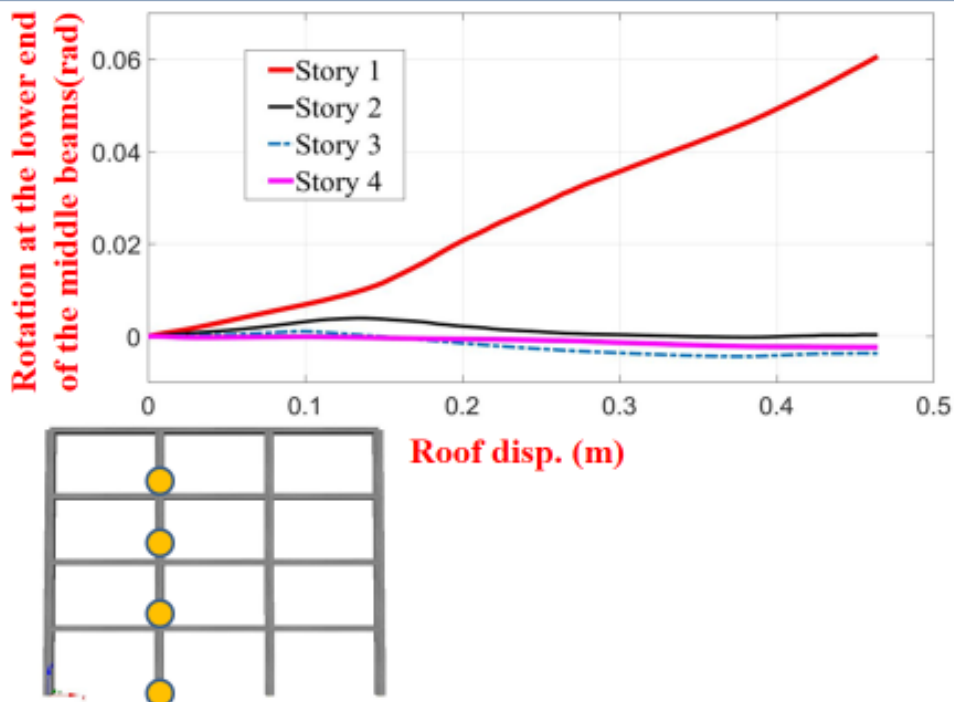




Example.4



Example.4





۵. قاب بتنی قدیمی (NC) / تقویت با پلیمر مسلح (اف آر پی)

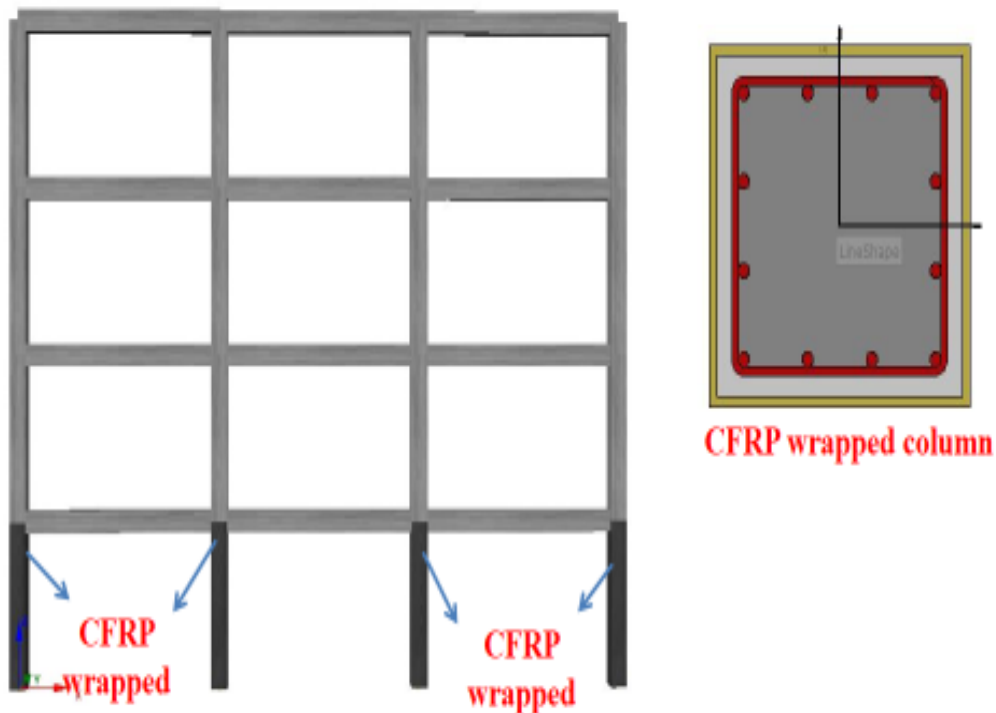
ستون‌های نمونه ۴ با پلیمر تقویت شوند

خروجی‌ها:

۱. جابجایی هدف در خطر ۱ و ۲
۲. مدل غیرخطی تیر/مقاومت خمشی، نمودار غیرخطی
۳. مدل غیرخطی ستون/مقاومت خمشی، نمودار غیرخطی
۴. وضعیت لولای خمیری بحرانی در تیرها و ستون‌ها در خطر ۱ و ۲
۵. نمودار رانش

Example.5 Old RC Moment Frame with FRP

Everything is Exactly similar to Example 4 except that the columns at the first story are wrapped with 3 layers of CFRP





Example.5

Properties of the wrapped CFRP

FRP Sheet

No FRP wrapping
 Select from a List
 User-defined Values

Company Name

TYPE

- KAPEI
- ISP
- SASP
- EXCEL
- ERACOLL
- QuakeWrap

Propose FRP system to SeismoSoft

Environmental conversion factor for different exposure conditions (NTC-08, NTC-18 only)

Internal Aggressive environment Factor na 0.95

External Custom

Number of layers 3 Radius of Rounding Corners R 4.0 cm

Cured Laminate or Dry Properties (design values)

Fiber Thickness (mm) 1.0000

Tensile Strength (MPa) 340.0

Tensile Modulus (MPa) 22000.0

Elongation (%) 0.90

Type

Carbon Fiber-Reinforced-Polymer, CFRP

Aramid Fiber-Reinforced-Polymer, APRP

Glass Fiber-Reinforced-Polymer, GFRP

Basalt Fiber-Reinforced-Polymer, BFRP

Steel Fiber-Reinforced-Polymer, SFRP

Number of Directions 1

Fiber Orientation 0°

Laminate Properties

Example.5

CFRP would improve confinement and shear capacity of the section

Shear Strength

Use Value to Carry out Shear Capacity Checks
Carry out shear capacity checks in elements that employ the current section (Performance Criteria Module)

Automatic Calculation of Shear Capacity

Include Concrete and Axial Force Contributions

Along axis 2-2	572.776
Along axis 3-3	572.776

NOTE: The shear strength values shown above are only indicative since they consider only the contribution of the transverse reinforcement and concrete. The actual shear strength of the member may vary during the analysis, depending on other factors such as axial force level and the displacement ductility demand. Further, no safety factors have been considered to derive the values above.

Elastic Section Properties

EA (kN)	3684000.000
EI2 (kNm ²)	49120.000
EI3 (kNm ²)	49120.000
GJ (kNm ²)	34590.334

Section Mass and Weight

Section Self Mass (tonne/m)	0.408
Section Self Weight (kN/m)	4.005

Confinement Factors

Automatic Calculation of Confinement Factors

Unconfined	1.528
Confined	1.528

Previous value = 1

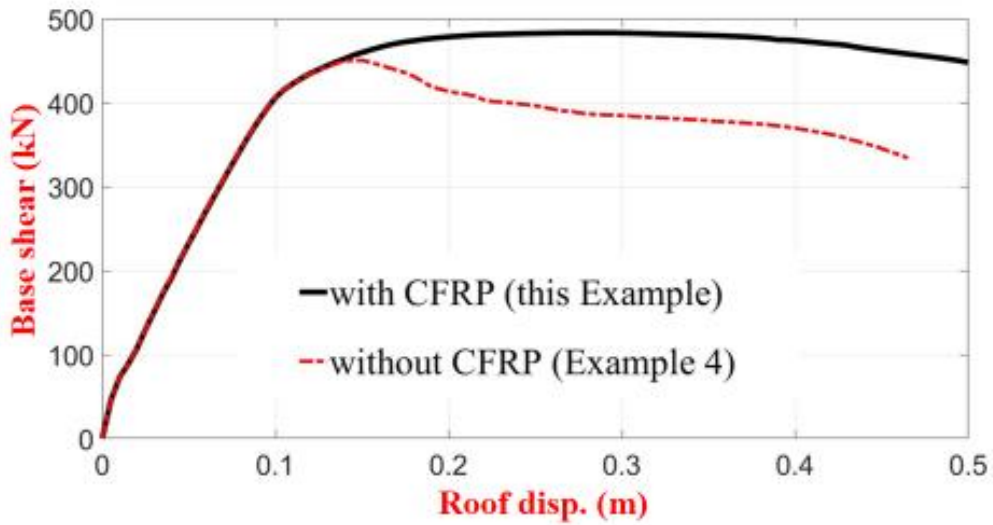
Previous value = 217 kN

Stirrups closed at 135

NOTE: The elastic section properties shown here are computed without considering the presence reinforcement bars, which instead are fully accounted for during the actual analyses.



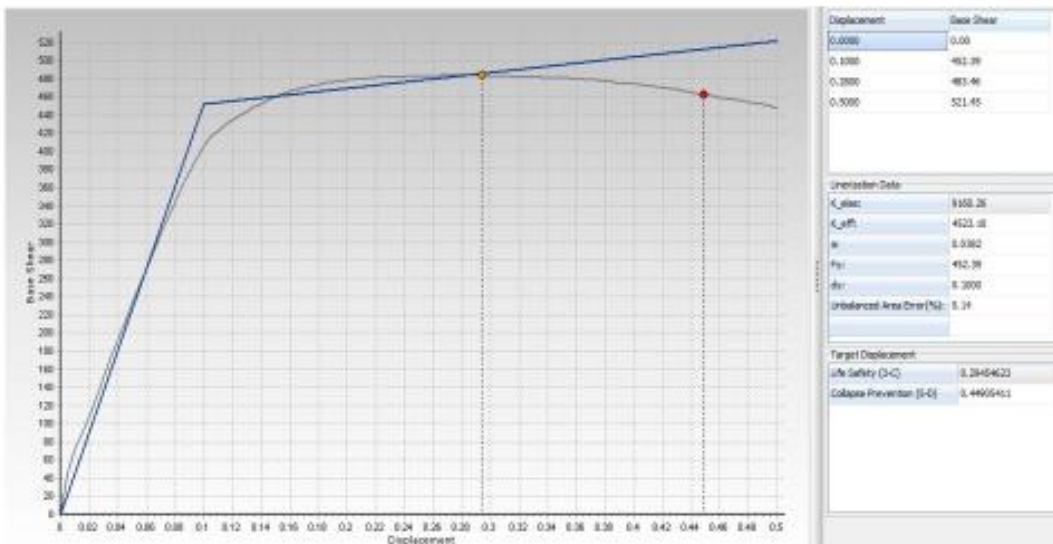
Example.5



Example.5

Level I target disp. = 295 mm

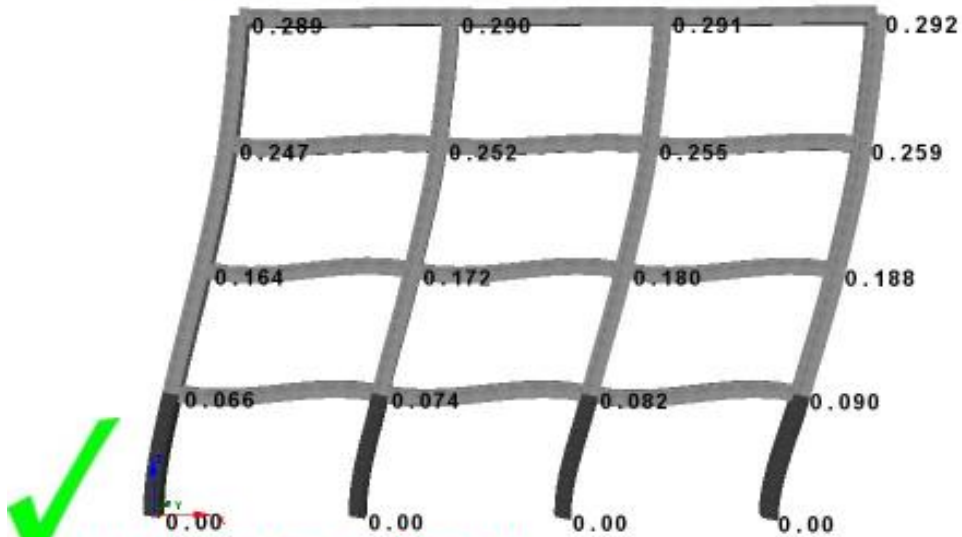
Level II target disp. = 449 mm





Example.5

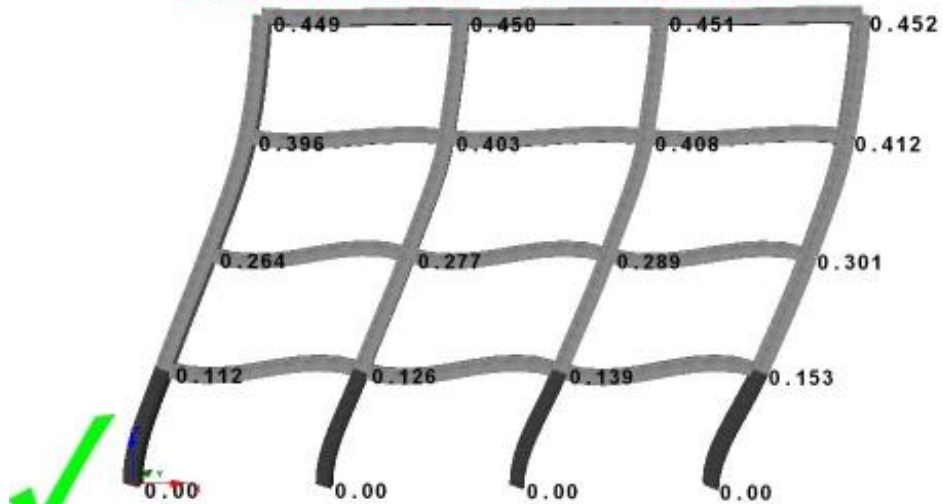
Under level I earthquake (roof disp.= 292 mm)



All elements are less than LS

Example.5

Under level II earthquake (roof disp.= 449 mm)



All elements are less than CP



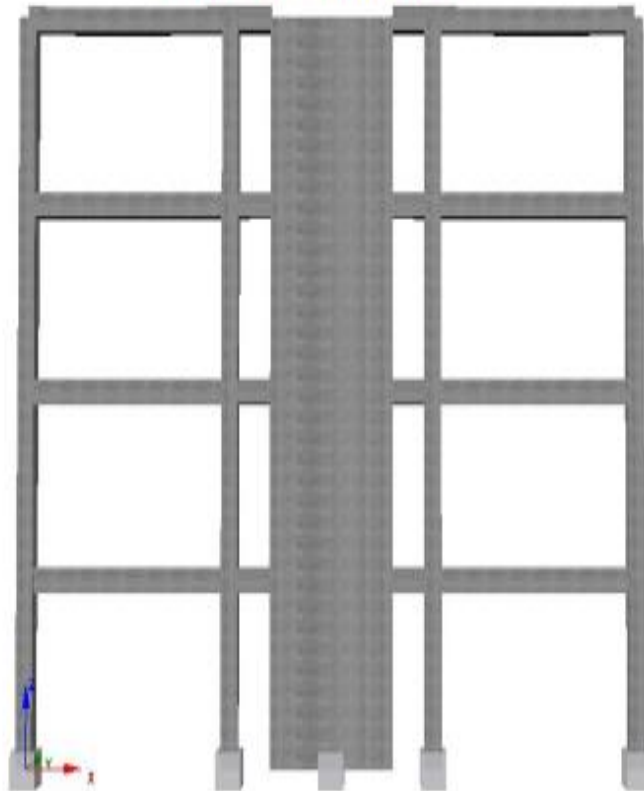
۶. قاب بتنی قدیمی (NC) / تقویت با دیوار برشی

نمونه شماره ۴ که با یک دیوار برشی مذکور در نمونه ۲ تقویت شده
خروجی‌ها:

۱. جابجایی هدف در خطر ۱ و ۲
۲. وضعیت لولای خمیری بحرانی در تیرها در خطر ۱ و ۲
۳. وضعیت لولای خمیری در تیرهای متصل به دیوار برشی در خطر ۱ و ۲
۴. وضعیت لولای خمیری بحرانی در ستون‌ها در خطر ۱ و ۲
۵. نمودار رانش
۶. جابجایی و گریز طبقات

Example.6 Old RC Moment Frame with additional RC wall

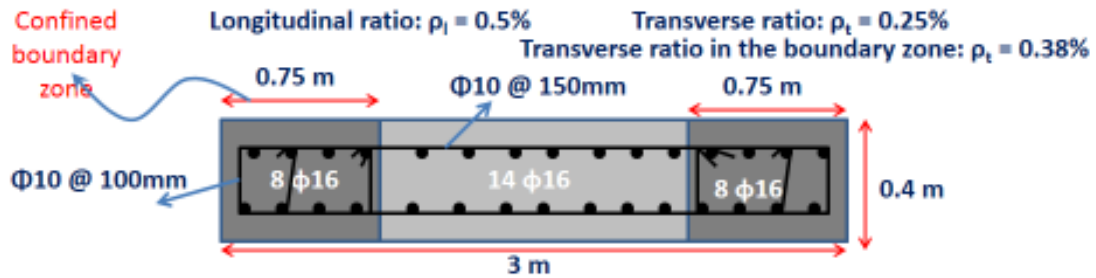
Everything is Exactly similar to Example 4 except that a 3m RC shear wall with thickness of 400 mm is added.



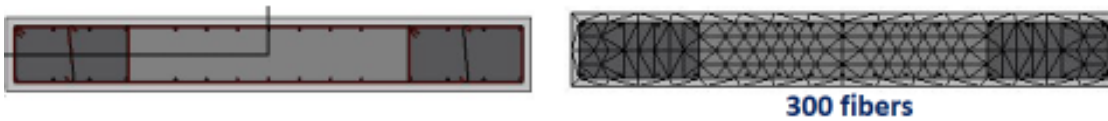


Example.6

The added RC wall is similar to that used in Example 2 for the steel frame with khorjini connections.

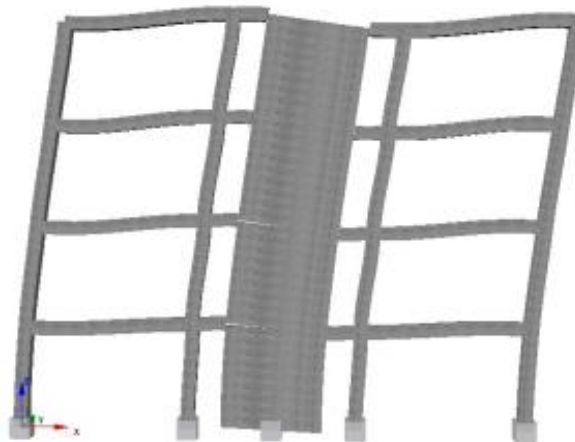


Modeled section in SStruct



Example.6

1st mode period = 0.208 s

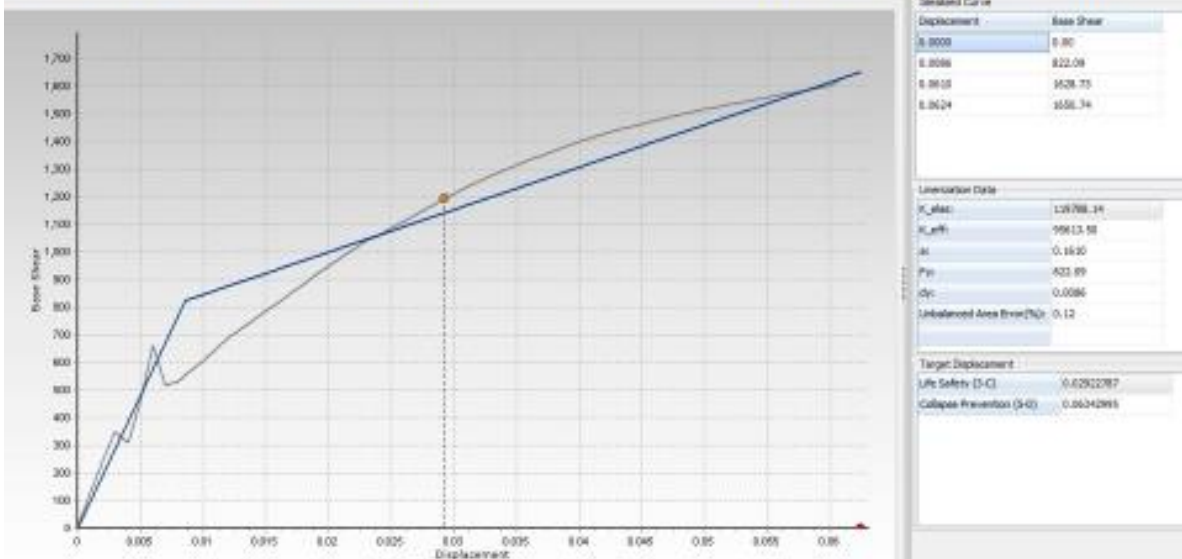




Example.6

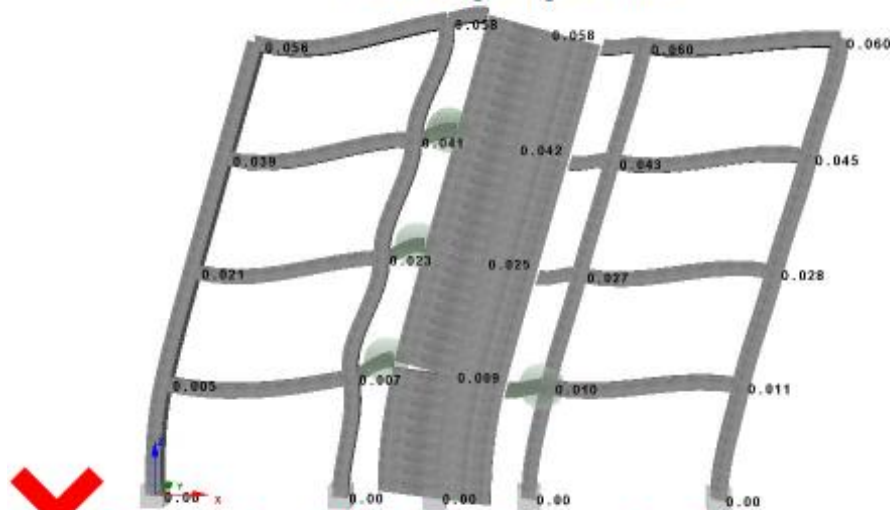
Level I target disp. = 29 mm

Level II target disp. = 62 mm



Example.6

Level II target disp. = 62 mm

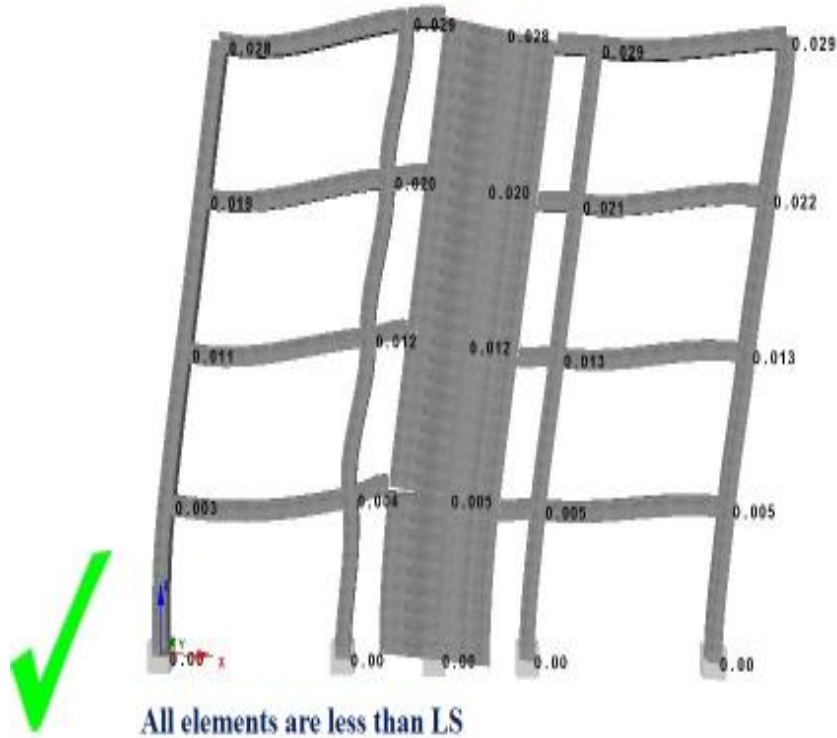


The short beams failed in shear (shear capacity check)



Example.6

Level I target disp. = 29 mm



Example.6

این مثال با دیوار 5 متری در دهانه وسط
تکرار شود



موسسه آموزش و مهندسی ۸۰۸
آموزشهای تخصصی عمران و معماری

لازم به ذکر است که بخش نرم‌افزاری این دوره در دو گروه به شرح زیر برگزار می‌شود:

مدت دوره	ساعت برگزاری	زمان برگزاری	بخش
۴۰ ساعت	۹ الی ۲۰	* گروه ۱: ۷، ۸، ۹ و ۱۰ تیر ۹۷ * گروه ۲: ۱۲، ۱۳، ۱۴ و ۱۵ تیر ۹۷	نرم‌افزاری

برای کسب اطلاعات بیشتر به لینک زیر مراجعه کنید:

<http://civil808.com/landing/pbdcourse>