NEMETSCHEK SCIA ENGINEER & ECtools

VERIFICATION DOCUMENT

FOR ACI 318-11 & ASCE/SEI 7-10

APRIL 2014

Preface

This report has been prepared by Penelis Consulting Engineers SA at the request of Nemetschek Scia in order to serve as a verification manual for the US version of Scia Engineer and ECtools.
The choice has been to verify the software against the well-known and generally accepted CSI Etabs. For the analysis Etabs 9.70 version has been used as its use is most wide spread. However for the design of concrete elements, the CSI Etabs 2013 ACI318/11 option was used, as the Etabs 9.70 version, includes a simplified ACI concrete design.
For the verification a 3 Storey Reinforced concrete building with one basement has been selected. This building includes many design cases (columns, T-Beams, I, C, L walls etc) and was deemed as a more appropriate reference that simple 1d or 2d examples.
Finally a simplified model of a complex actual building, which is seismically isolated with inverted pendulum isolators, which has been designed by Penelis Consulting Engineers, is briefly presented and compared with Etabs v9.70 and Scia Engineer. The building is the New Athens Opera House.

This report has been prepared by Penelis Consulting Engineers SA, and more specifically by the following engineers:

- Professor George Penelis
- Dr. Gregory Penelis
- Dr. Kostantinos Pashalidis
- Dr. Vassilis Papanikolaou
- Dr. Elias Paraskevopoulos
- Sotiria Stefanidou, MSc Eng

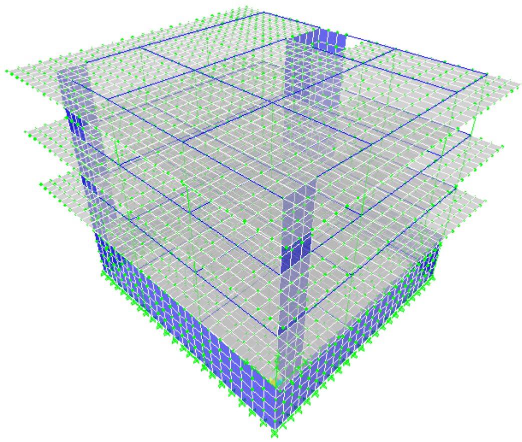
It should be notted that this document aims only to verify Scia Engineer using the respected in the US market CSI Etabs, and by no means does it contain any criticism on the latter.
The document and reference files (Etabs, S.EN., ECtools) may be be downloads from:
www.ectools.eu

Example 1: 3 Storey Building with one Basement

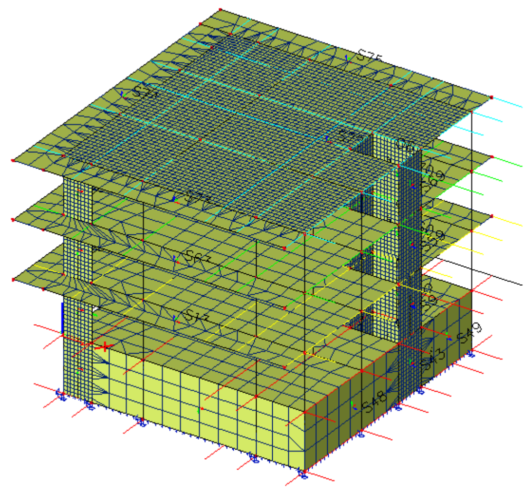
1. Geometry

The building is part of the ECtools example and is mentioned as Example 1. It is a very simple single storey dual system R/C building that includes shear walls, cores and Moment Resisting Frames (MRF).

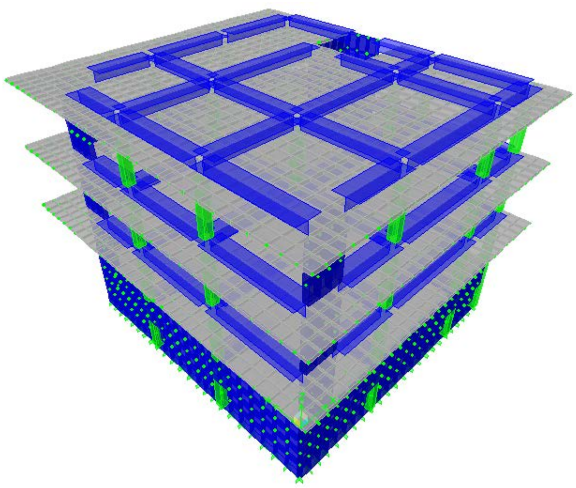
The geometry is shown in the plan drawings shown in the following two pages while the 3D modelling I shown in the following pictures



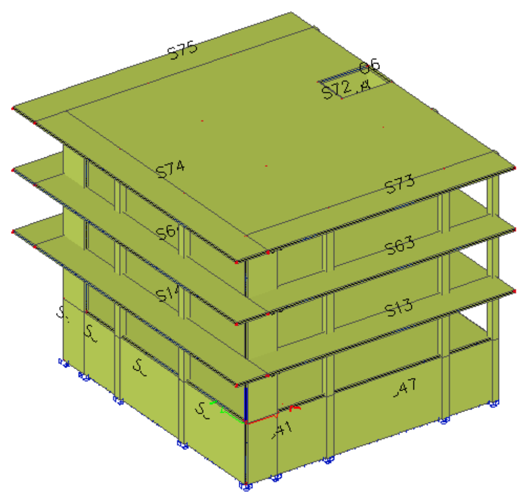
Etabs 3D Model



S.EN. 3D model

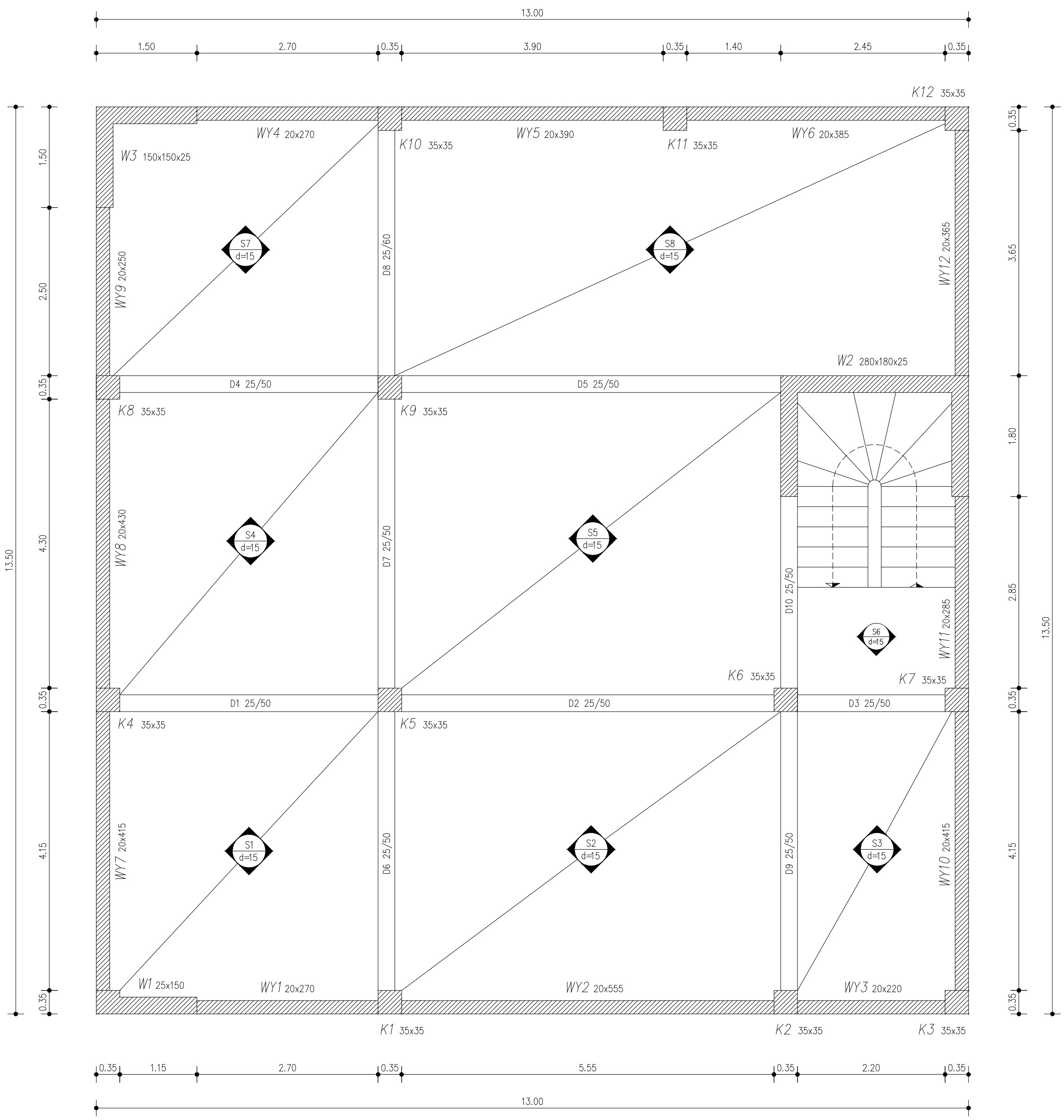


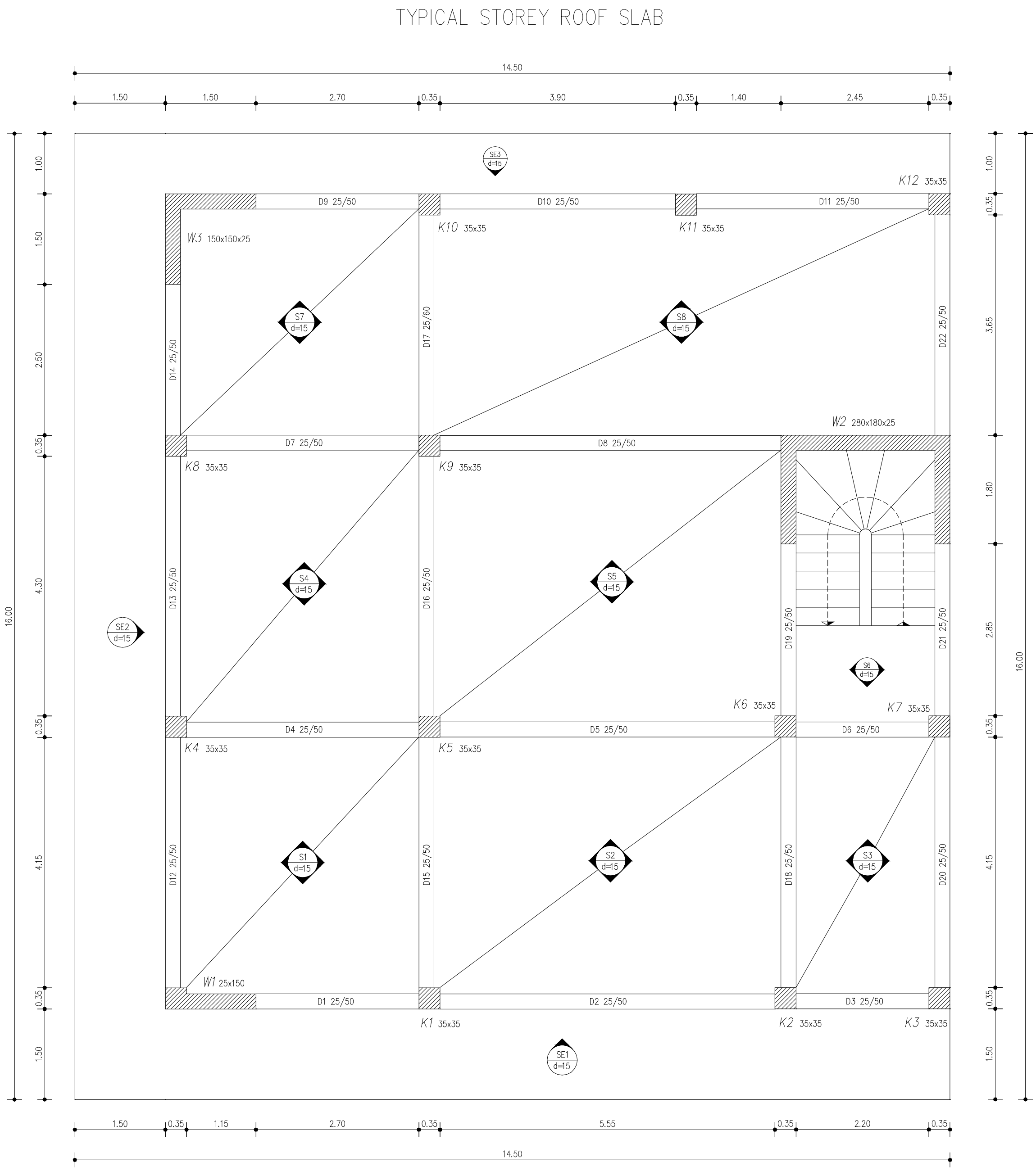
Etabs 3D extruded Model



S.EN. 3D Extruded model

BASEMENT ROOF SLAB



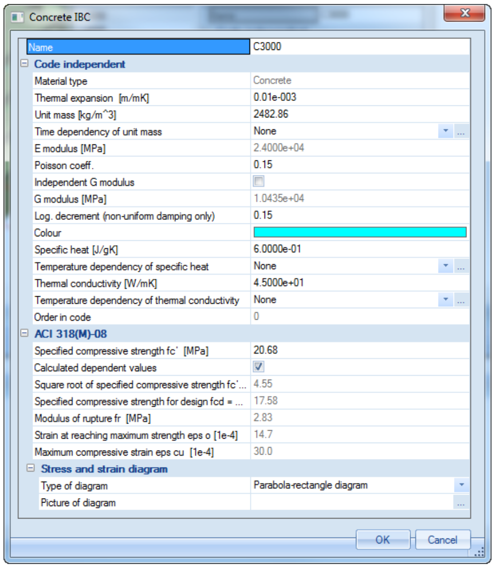

TYPICAL STOREY ROOF SLAB
14.50
1.50
1.50
2.70
0.35
3.90
0.35
1.40
2.45
0.35
SE3
d=15
K12 35x35
D9 25/50
D10 25/50
D11 25/50
W3 150x150x25
K10 35x35
K11 35x35
S7
d=15
S8
d=15
D14 25/50
D17 25/60
D22 25/50
W2 280x180x25
D7 25/50
D8 25/50
K8 35x35
K9 35x35
S4
d=15
S5
d=15
D13 25/50
D16 25/50
D19 25/50
D21 25/50
SE2
d=15
S6
d=15
K6 35x35
K7 35x35
D4 25/50
D5 25/50
D6 25/50
K4 35x35
K5 35x35
S1
d=15
S2
d=15
S3
d=15
D12 25/50
D15 25/50
D18 25/50
D20 25/50
W1 25x150
D1 25/50
D2 25/50
D3 25/50
K1 35x35
K2 35x35
K3 35x35
SE1
d=15
1.00
1.50
2.50
0.35
4.30
0.35
4.15
0.35
1.50
16.00
3.65
1.80
2.85
1.50
0.35
1.15
2.70
5.55
2.20

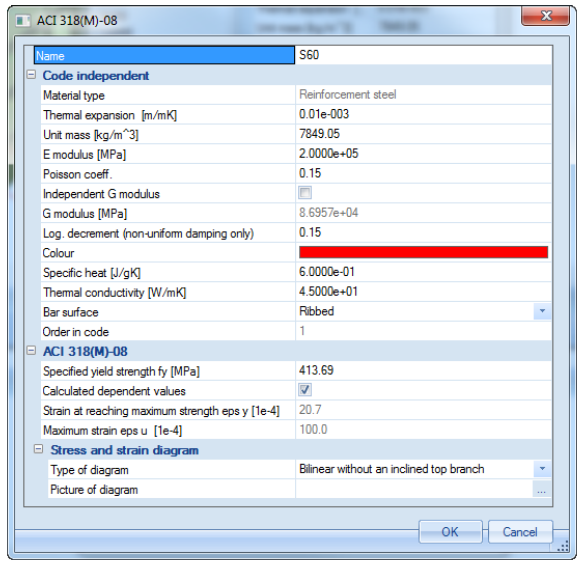
2. Materials

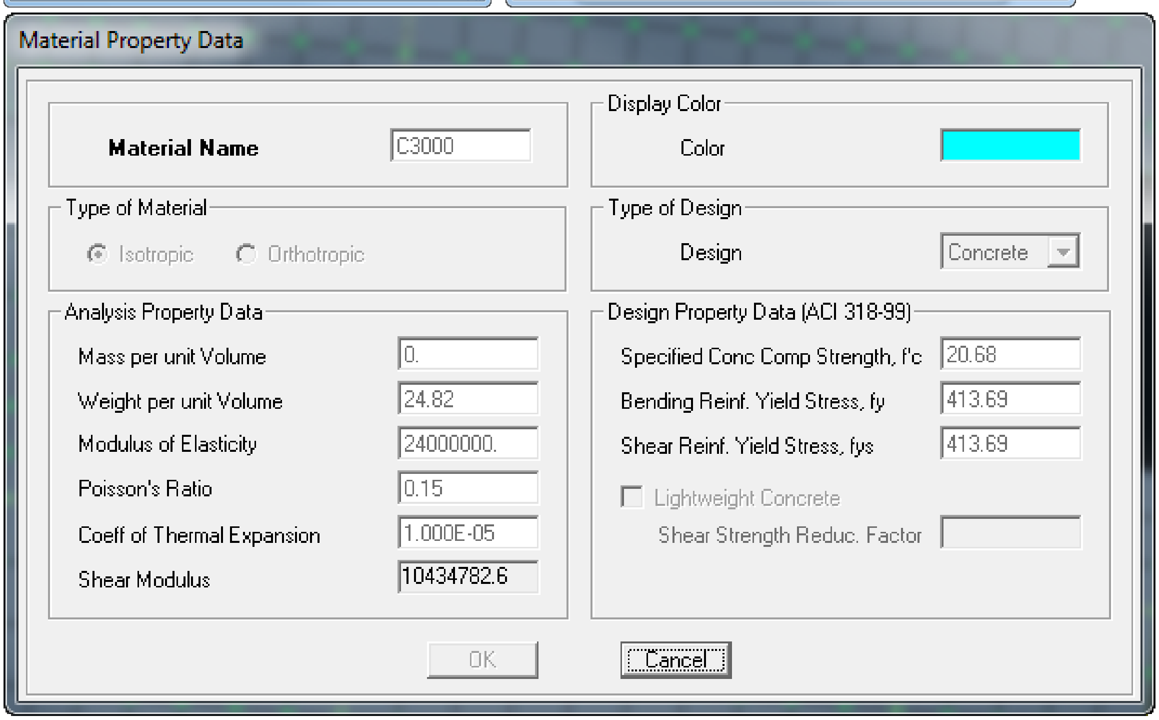
The materials used are:

- Concrete Grade C3000
- Reinforcing Steel S60

Below the material properties as included in S.EN. and Etabs are shown:







3. Loads

3.1. Gravity loads

The loads applied were for simplicity the following:
Self weight calculated automatically by the software
Additional dead weight : 1.5 kN/m^2
Live load: 5 kN/m^2 Balconies
2 kN/m^2 inner slabs and roof.

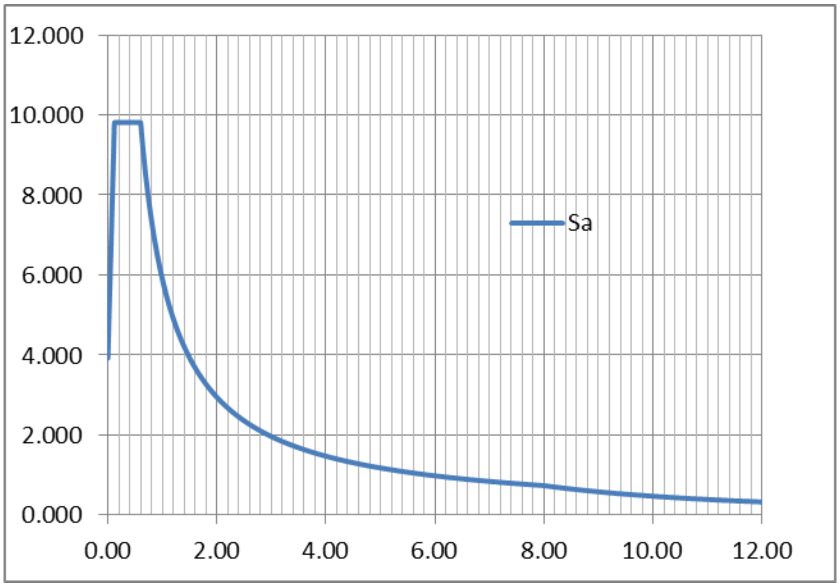
The global force balance for the total of dead weight (self + G), live loads (L) and the mass combination G+0.3Q is shown in the following table for the Etabs and S.EN. models. The comparison shows differences less than 2%.

ETABS Global Reactions		S.EN. Global Reactions		Diff
		GSW	5705.9	
DEAD	6865.53	DEAD	1296.75	2.00%
LIVE	2307.94	LIVE	2335.24	1.18%
G+0.3Q	7557.912	G+0.3Q	7703.222	1.92%

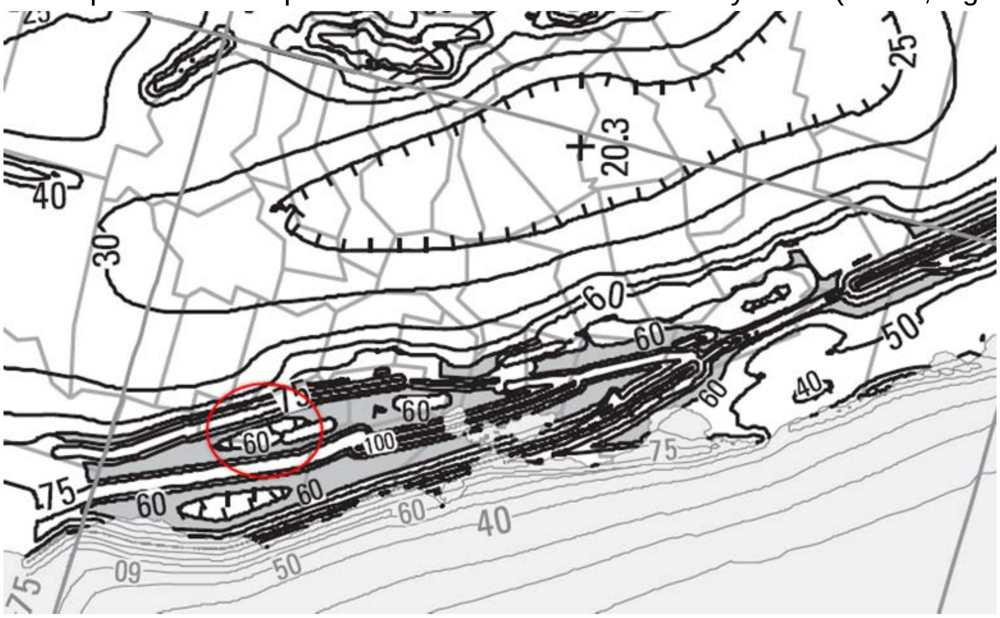
3.2. Seismic loads

The following spectra has been derived from ASCE SEI 7-10 using the following parameters:

S_S	1.5	g
S_1	0.6	g
Site Class	D	
F_a	1.00	
F_v	1.50	
S_{MS}	1.50	g
S_{M1}	0.90	g
S_{DS}	1.00	g
S_{D1}	0.60	g
T_0	0.12	s
T_S	0.6	g
T_L	8	s
mult.	9.81	m/s²



This spectra corresponds to the San Francisco bay area (Ch22, fig 22.2):



4. Mass

The mass of the building has been defined for the quasi permanent combination G+0.30 Q, and is being calculated automatically both by Etabs and S.EN. The mass is calculated by dividing the loads by g.

The table below includes the comparison which shows a difference less than 0.5%.

ETABS Assembled Masses (no lamping)		
Storey	MassX	MassY
STORY3	179.315	179.315

STORY2	187.428	187.428
STORY1	187.428	187.428
BASE1	194.791	194.791
BASE	16.826	16.826
Totals	765.789	765.789
S.EN. Assembled Masses (no lamping)		
Story	MassX	MassY
Totals	767.11	767.11
Difference	**0.17%**	**0.17%**

5. Dynamic response (Eigen Vector)

The following figures show the eigen periods as provided by Etabs and S.EN.

Mode	Period	UX	UY	UZ	SumUX	SumUY	SumUZ	RX
9	0.340101	13.5151	31.8862	0.0049	13.5151	31.8862	0.0049	37.4223
2	0.265973	30.0081	26.3199	0.0358	43.5232	58.2062	0.0407	30.9382
3	0.208837	18.7432	3.0913	0.0260	62.2664	61.2974	0.0667	3.6374
4	0.091281	3.4443	4.4705	0.1818	65.7107	65.7680	0.2485	0.4062
5	0.087025	0.0391	0.0363	0.6388	65.7498	65.8043	0.8872	0.4721
6	0.083920	0.1277	0.0562	19.1016	65.8775	65.8605	19.9889	2.3902
7	0.081808	0.0126	0.0033	0.0061	65.8902	65.8638	19.9950	0.1803
8	0.081603	0.0672	0.0011	0.9175	65.9573	65.8649	20.9124	0.0088
9	0.080963	0.0195	0.0020	0.2218	65.9768	65.8669	21.1343	0.1321

Eigen frequencies

N	f [Hz]	omega [1/s]	omega^2 [1/s^2]	T [s]
Mass combination : CM1				
1	2.99	18.77	352.37	0.33
2	3.63	22.82	520.70	0.28
3	4.79	30.08	904.55	0.21
4	10.89	68.44	4684.19	0.09
5	11.38	71.51	5113.58	0.09
6	11.49	72.18	5210.55	0.09
7	12.25	76.94	5920.08	0.08
8	12.36	77.67	6032.08	0.08
9	12.39	77.85	6061.38	0.08
10	12.92	81.18	6590.82	0.08
11	12.97	81.49	6639.99	0.08
12	13.26	83.30	6938.17	0.08
13	13.45	84.51	7142.66	0.07
14	13.48	84.69	7172.88	0.07
15	13.62	85.56	7321.31	0.07
16	13.80	86.68	7513.47	0.07
17	14.09	88.53	7837.66	0.07
18	14.14	88.81	7887.86	0.07
19	14.25	89.56	8020.27	0.07
20	14.32	90.00	8099.33	0.07

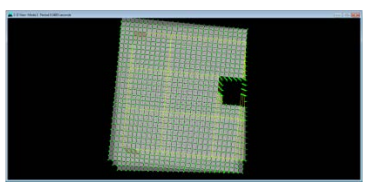
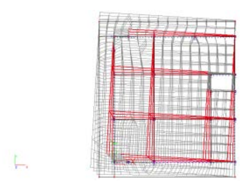
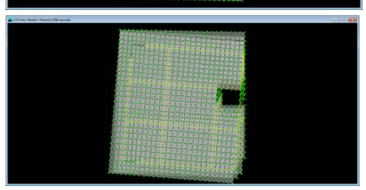
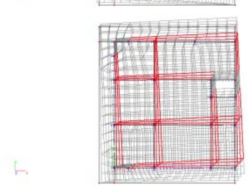
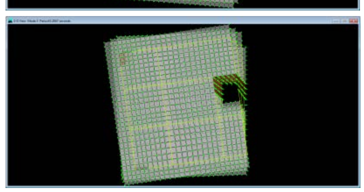
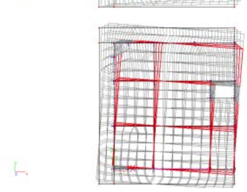
The table below compares the eigen periods as well as the participating mass ratios.

ETABS Eigen Frequency				S.EN. Eigen Frequency				Dif. T
Mode	Period	UX	UY	Mode	Period	Wxi	Wyi	
1	0.340	0.14	0.32	1	0.335	0.17	0.30	-1.59%
2	0.266	0.30	0.26	2	0.275	0.24	0.31	3.51%
3	0.209	0.19	0.03	3	0.209	0.22	0.01	0.03%
4	0.091	0.03	0.04	4	0.092	0.04	0.04	0.57%
5	0.087	0.00	0.00	5	0.088	0.00	0.00	1.01%
6	0.084	0.00	0.00	6	0.087	0.00	0.00	3.67%
7	0.082	0.00	0.00	7	0.082	0.00	0.00	-0.13%
8	0.082	0.00	0.00	8	0.081	0.00	0.00	-0.86%
9	0.081	0.00	0.00	9	0.081	0.00	0.00	-0.32%

It is clear that for the first 3 important modes the differences of S.EN. to Etabs are around 3%. Considering the several different modelling approaches used in the two software (i.e. lamped masses in Etabs Vs distributed masses in S.EN., T beams as sections in Etabs Vs T beams as Ribs under Shells in S.EN.) this coincidence is considered a match.

It is noted that for the insignificant modes (less than 4% active mass) the match is less accurate as one would expect between different software (hence the gray in the difference column).

The table below shows the eigen deformations for each of the first three modes of vibration, using Etabs and S.EN. (3D view from top –z)

Mode, T	Etabs	S.EN.
1, T=0.34/0.335		
2, T=0.266/0.275		
3, T=0.209/0.209		

6. Analysis results

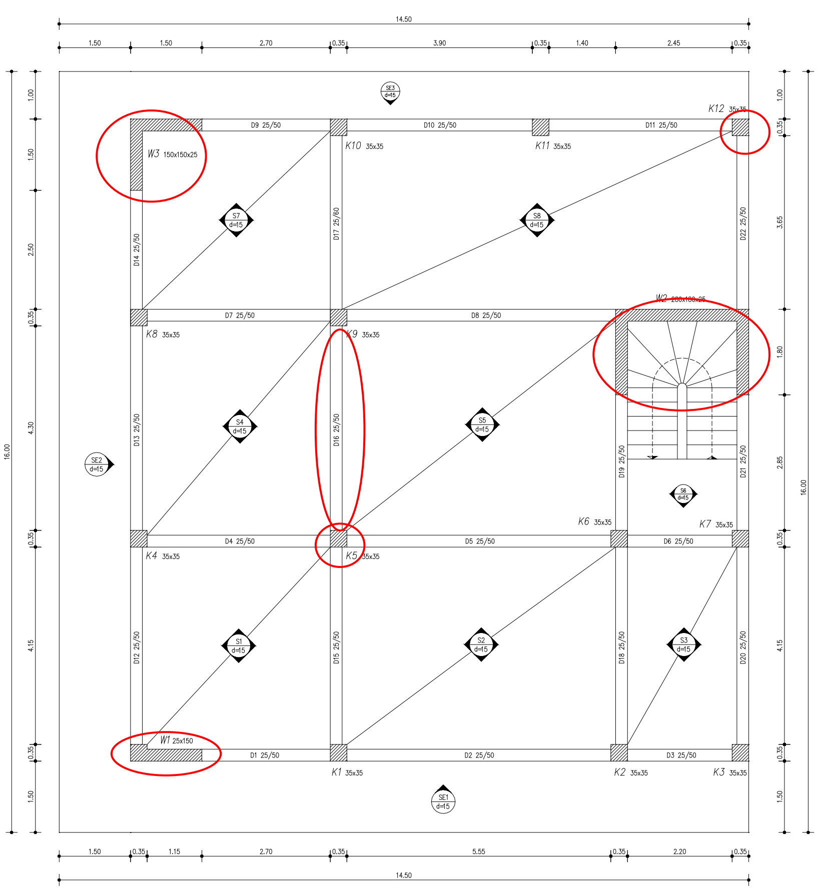
6.1. General

The following paragraphs compare internal forces on beams, columns and walls modeled in Etabs and S.EN. Considering the different modeling and F.E. approaches of the two software, the match is more than adequate.

As a reference the following elements have been selected:

- D16 beam of storey 3
- K12 column of storey 3
- K5 column at basement
- W1 wall at ground floor

TYPICAL STOREY ROOF SLAB



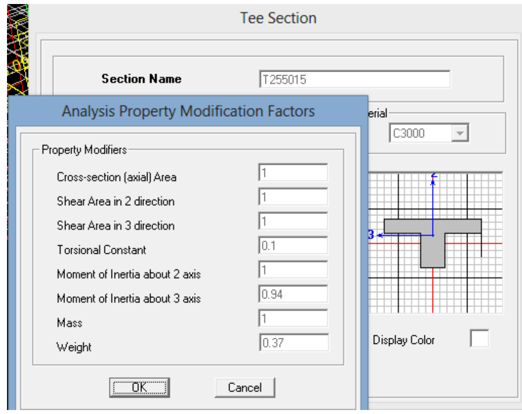
6.2. Beams

6.2.1. Beams modeling general

As it is known beams are modelled in S.EN. using a combined approach of 1D elements for the rib of a T-beam section and the slab F.E. for the flange. The resultant internal forces are a combination of the internal forces of the rib and the integrated stresses of the slab effective width.

The weight and stiffners modifiers for the Etabs model are calculated in the following table:

T25x50x15	Actual	Etabs slab
Lf	1	1
tf	0.15	0.15
h	0.5	
b	0.25	
A	0.2375	0.15
Weight Mod	**0.37**	**1**
A	0.2379	
J	4.628E-03	2.81E-04
Stiff Mod	**0.94**	**1**
J tot	4.632E-03	



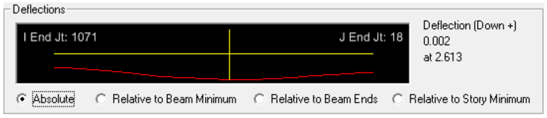
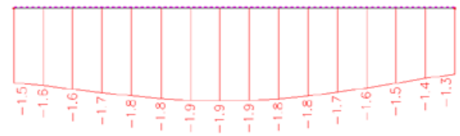
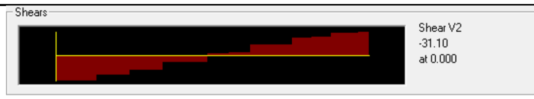
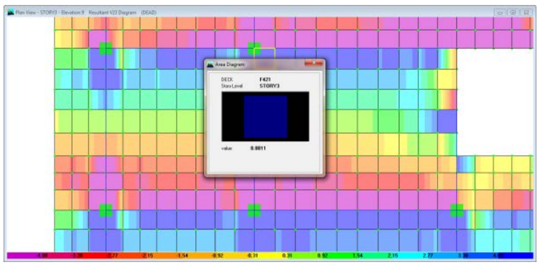
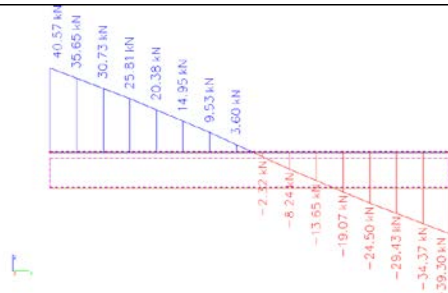
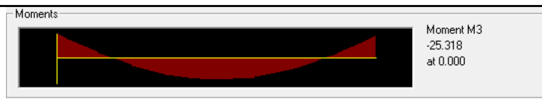
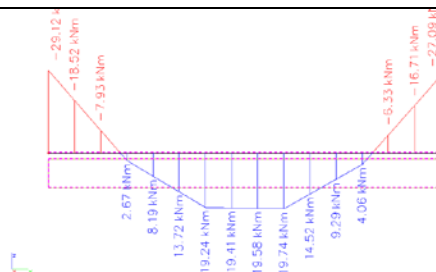
Etabs does not have the save functionality, so beams are modelled as T sections with a weight modification factor so that the self-weight of flange is not calculated twice (once from the T beam section and once for the slab F.E.).

Due to the fact that Etabs uses shell elements duplicated by the T-beam section, the correct moment and shear forces of the beam may only be calculated by adding to the beam forces the integrated sheel element corresponding forces. This is not very critical for the moment, while it is significant for the shear force.

In the following paragraphs this procedure has indeed been manually applied for the shear forces of the beams.

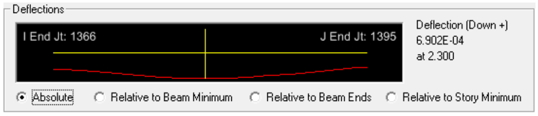
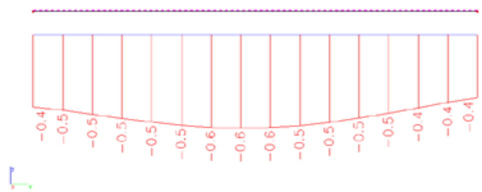
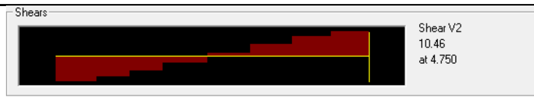
6.2.1.Beams Dead load (G)

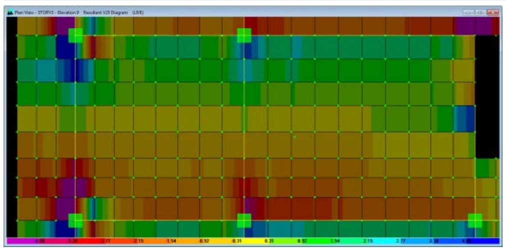
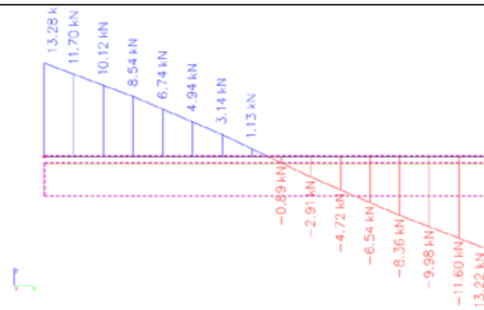
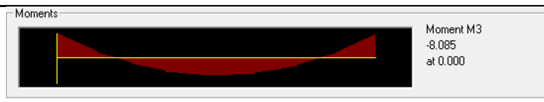
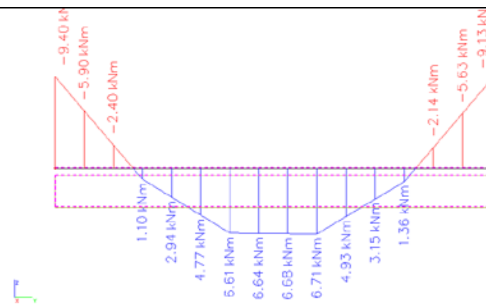
The following table compares the results of beam internal forces for the dead load case, which in both software includes the self weight (In S.EN. the Dead is a combination of G+GSW)

D16 S3	Etabs/ Dead	S.EN./ SW+Dead
Deflection		
Shear	  31.10+0.5x8+0.5x6 = 38.1 kN	
Moment		

6.2.1. Beams Live load (L)

The following table compares the results of beam internal forces for the liveload case.

<table>
<tr><th>D16
S3</th><th>Etabs/ Live</th><th>S.EN./ Live</th></tr>
<tr><td>Deflection</td><td>
</td><td>
</td></tr>
<tr><td>Shear</td><td>


10.46+ 0.5x3+0.5x2 =
10.46+2.50= 12.96 kN</td><td>
</td></tr>
<tr><td>Moment</td><td>
</td><td>
</td></tr>
</table>

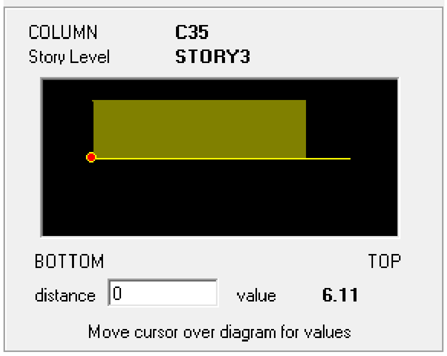
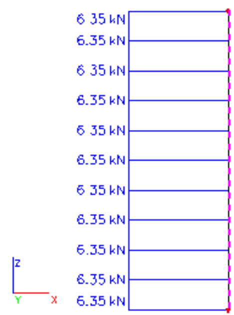
6.3. Columns

6.3.1.Column modeling in general

Columns are modelled in both software using 1D linear elements, therefore as the load transfer has been verified from the slabs and beams, the results are in agreement.

6.3.1.Dead load (G)

The following table compares the results of column internal forces for the dead load case.

<table>
<tr><th>K12
S3</th><th>Etabs/ Dead</th><th>S.EN./ Dead</th></tr>
<tr><td>Shear</td><td>
</td><td>
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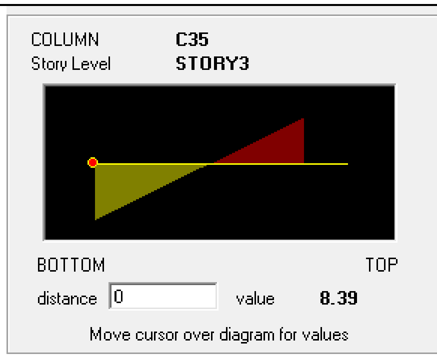
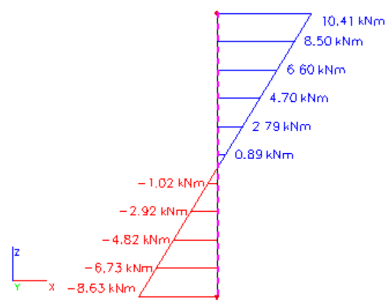
<tr><td>Moment</td><td>
</td><td>
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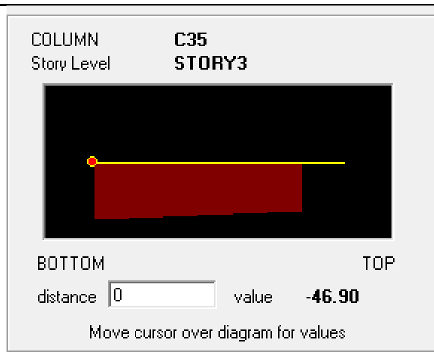
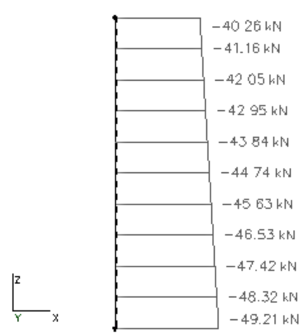
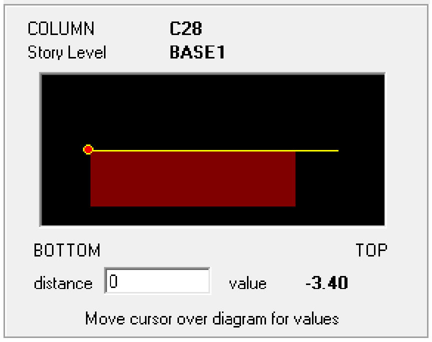
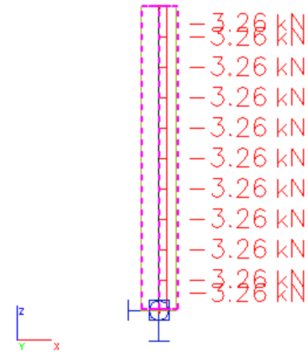
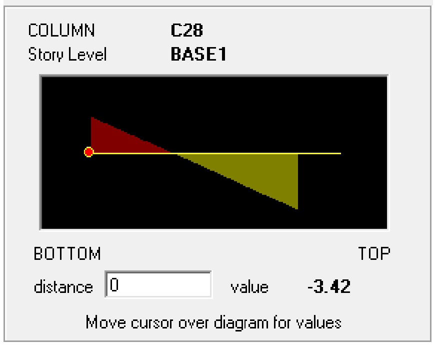
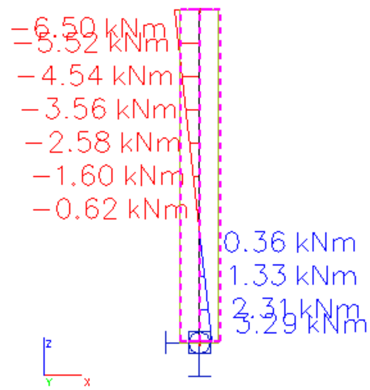
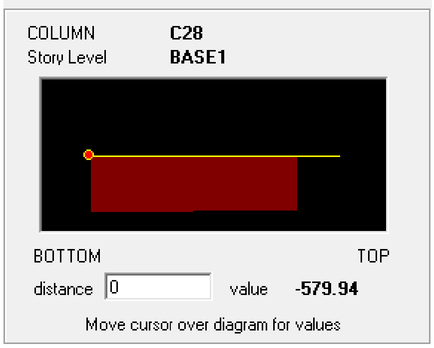
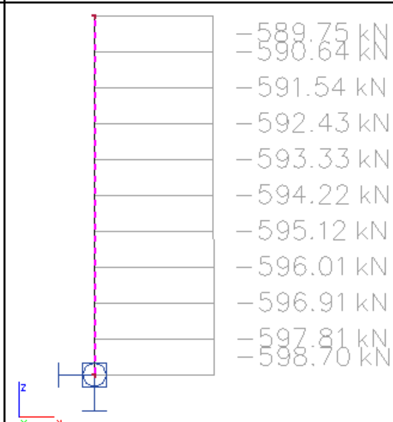
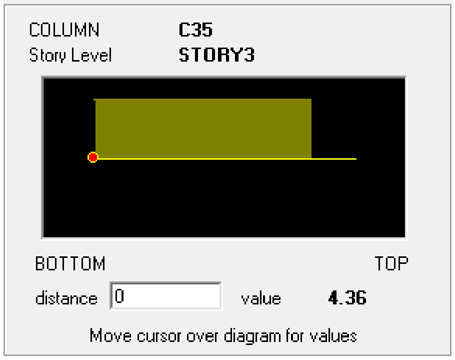
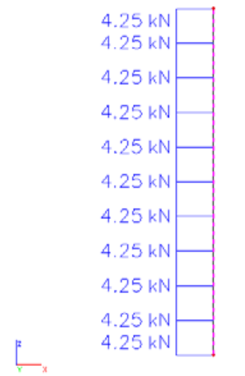
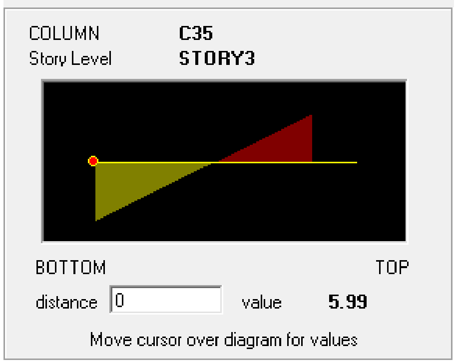
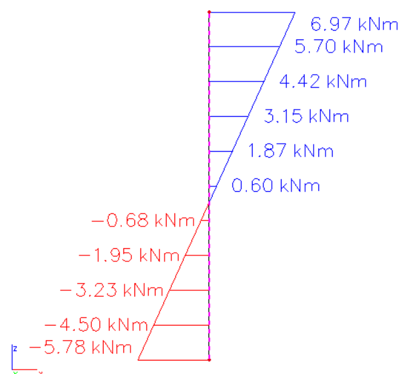
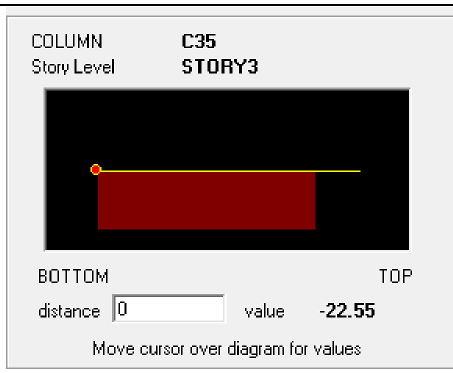
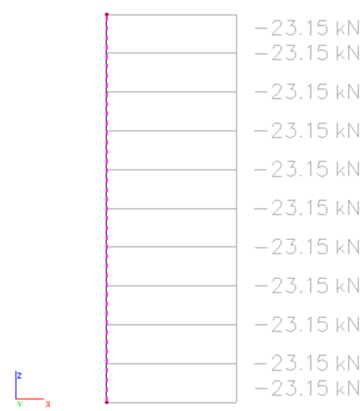
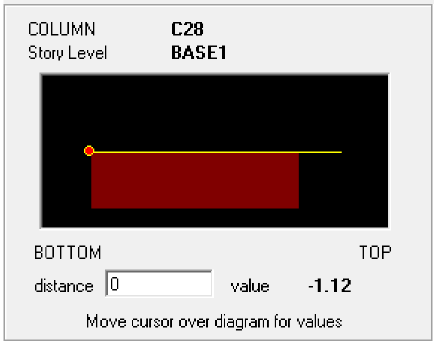
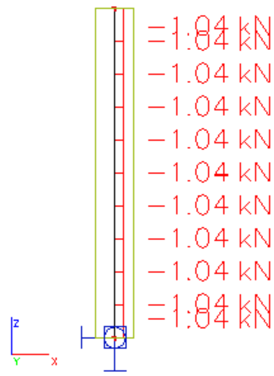
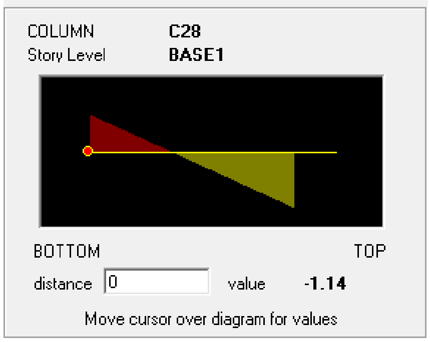
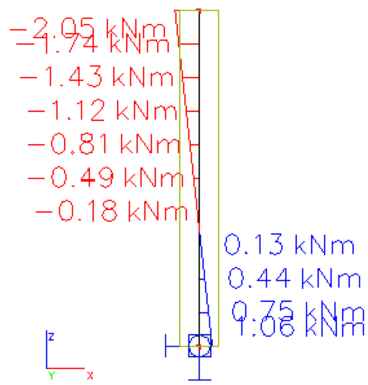
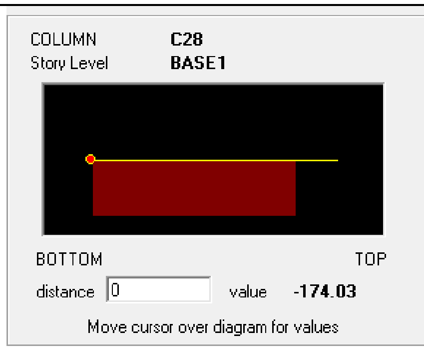
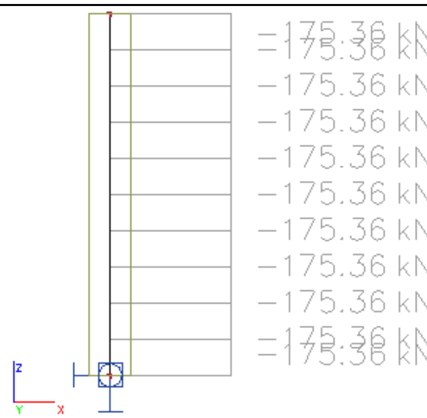
<tr><td>Axial</td><td>
</td><td>
</td></tr>
</table>

<table>
<tr><th>K5 U1</th><th>Etabs/ Dead</th><th>S.EN./ Dead</th></tr>
<tr><td>Shear</td><td>
</td><td>
</td></tr>
<tr><td>Moment</td><td>
</td><td>
</td></tr>
<tr><td>Axial</td><td>
</td><td>
</td></tr>
</table>

6.3.1.Live load (L)

The following table compares the results of column internal forces for the live load case.

K12 S3	Etabs/ Live	S.EN./ Live
Shear		
Moment		
Axial		

<table>
<tr><th>K5
U1</th><th>Etabs/ Live</th><th>S.EN./ Live</th></tr>
<tr><td>Shear</td><td>
</td><td>
</td></tr>
<tr><td>Moment</td><td>
</td><td>
</td></tr>
<tr><td>Axial</td><td>
</td><td>
</td></tr>
</table>

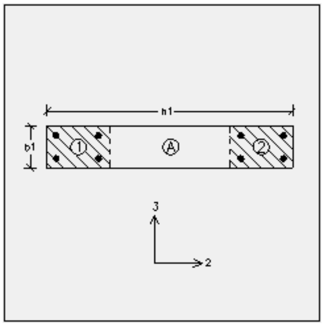
6.4. Walls

6.5. Walls modeling in general

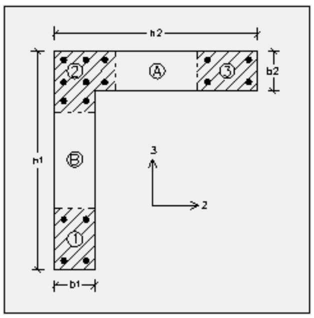
Walls are modelled in both software using Shell finite elements. The stresses from these F.E. are integrated to provide the internal forces of the wall. Etabs has this functionality using the Pier approach while S.EN. has it only for rectangular walls using the integration strips. All types of walls in S.EN. have their internal forces integrated from stresses using ECtools design tool.

As has been indicated three types of R/C walls shall be assessed:

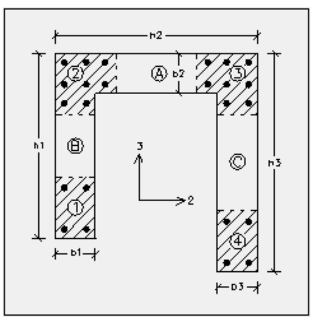
- The rectangular W1 which has a length of 1,50m and a thickness of 0.25m



- The L shaped W3 which has a two legs of 1,50m and a thickness of 0.25m

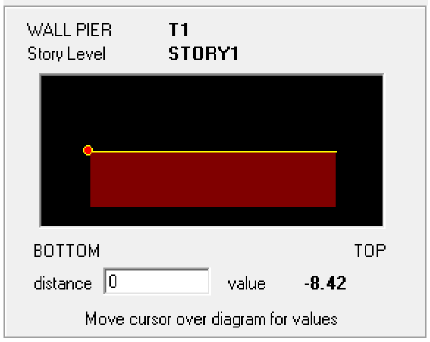
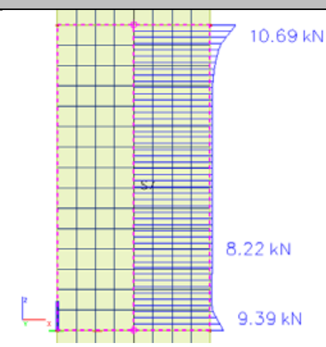
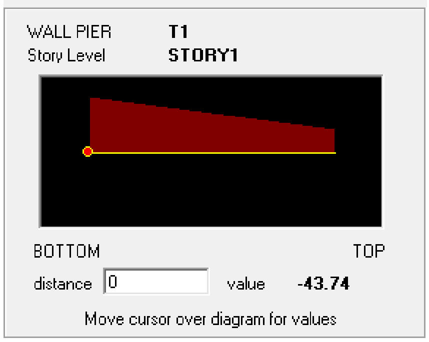
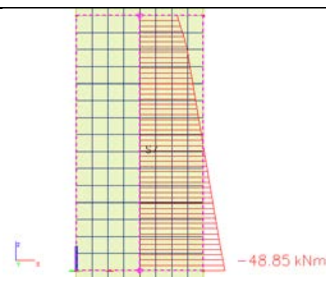
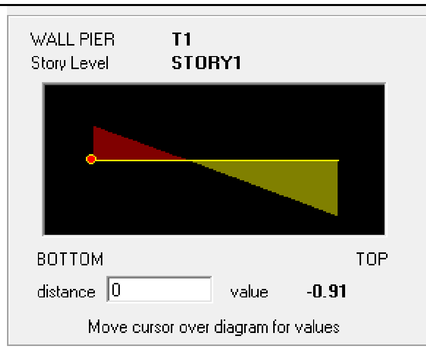
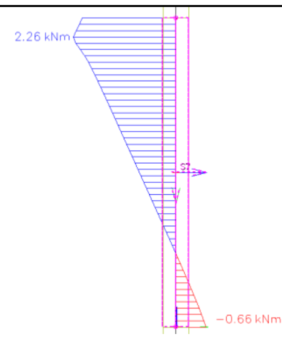


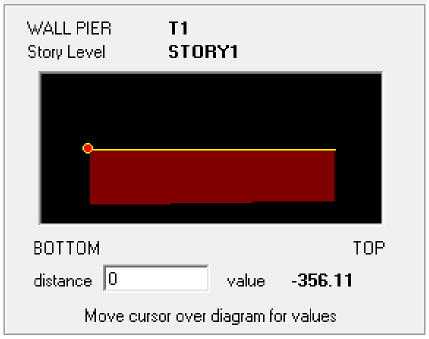
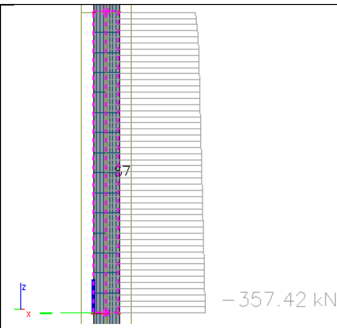
- The C shaped W2 core which has a two legs of 1,80m and a backbone of 2.80m with a thickness of 0.25m



6.5.1.Rectangular wall dead load (G)

Below the three approaches, Etabs/Pier, S.EN./integration strip and S.EN./ECtools, are verified for the rectangular wall W1 at ground floor. <u>It should be noted that only for a rectangular wall the comparison between Etabs and S.EN. is possible directly, as for all other shapes this is only available in S.EN. through ECtools which as shown here is a direct match to S.EN.</u>

W1 GF	Etabs/ Dead Automesh	S.EN./ Dead	S.EN./ECtools
Shear			7.63 KN
Moment (in plane M33)			-47.85kN
Mommen out of plane M22			0.70 kN

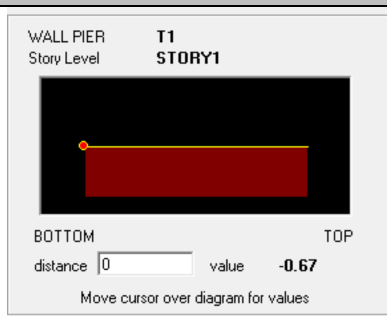
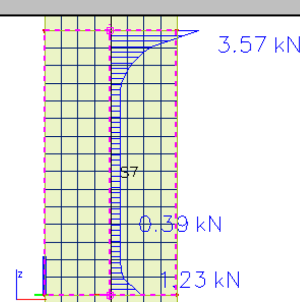
<table>
<tr><td>Axial</td><td>
</td><td>
</td><td>-356.08KN</td></tr>
</table>

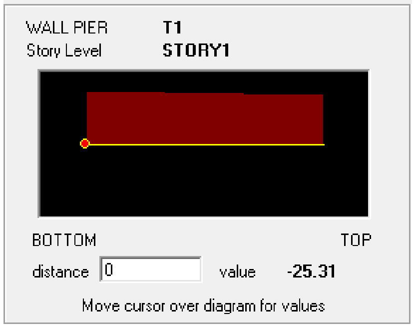
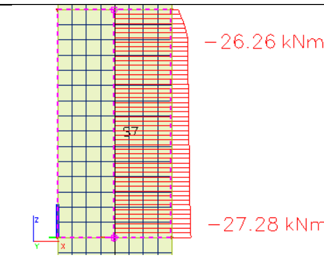
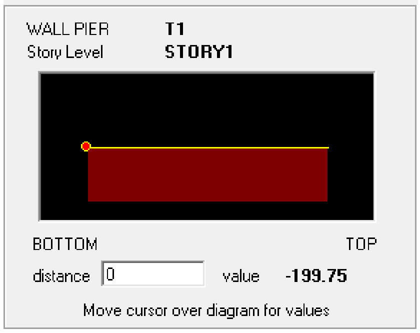
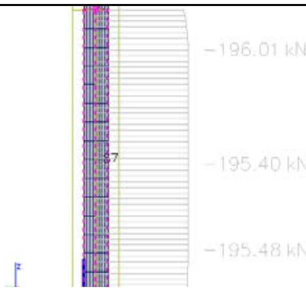
ECtools calculation is shown below (as exported by ECtools in Scia Translation.xls exported in the temporary S.EN. folder after ECtools is executed)

Pier Forces													
Story	ID	Combo	Nt	V2t	V3t	M2t	M3t	Nb	V2b	V3b	M2b	M3b
STORY1	P02	DEAD	-329.017	-5.518	0.983	-2.255	-25.177	-356.341	-5.518	0.983	0.696	-41.73

6.5.1. Rectangular wall live load (L)

Below the three approaches, Etabs/Pier, S.EN./integration strip and S.EN./ECtools, are verified for the rectangular wall W1 at ground floor. <u>It should be noted that only for a rectangular wall the comparison between Etabs and S.EN. is possible directly, as for all other shapes this is only available in S.EN. through ECtools which as shown here is a direct match to S.EN.</u>

<table>
<tr><th>W1 GF</th><th>Etabs/ Live</th><th>S.EN./ Live</th><th>S.EN./ECtools</th></tr>
<tr><td>Shear</td><td>
</td><td>
</td><td>0.28 KN</td></tr>
</table>

<table>
<tr>
<td>Moment (in plane M33)</td>
<td></td>
<td></td>
<td>-27.08 kN</td>
</tr>
<tr>
<td>Axial</td>
<td></td>
<td></td>
<td>-195.94KN</td>
</tr>
</table>

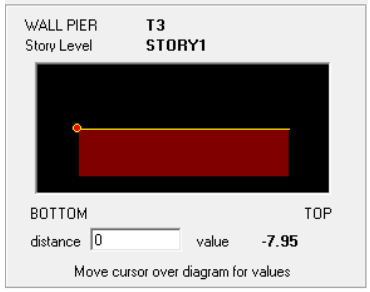
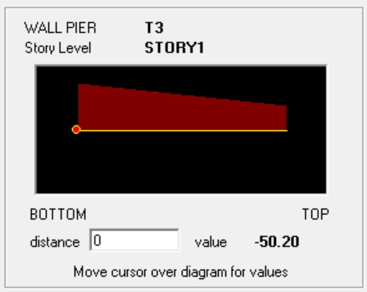
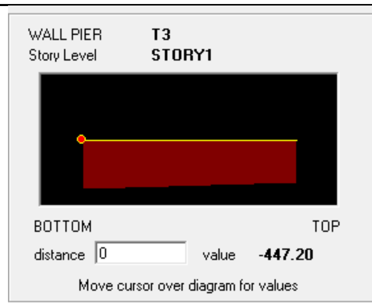
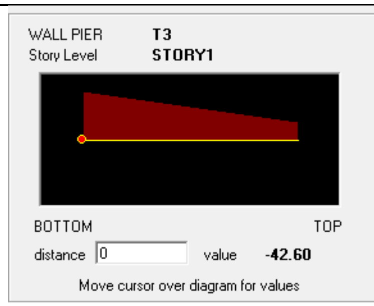
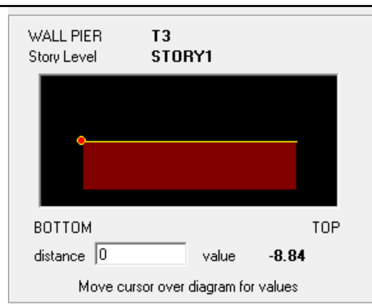
ECtools calculation is shown below (as exported by ECtools in Scia Translation.xls exported in the temporary S.EN. folder after ECtools is executed)

Pier Forces													
Story	ID	Combo	Nt	V2t	V3t	M2t	M3t	Nb	V2b	V3b	M2b	M3b
STORY1	P02	L	-194.186	-0.279	-0.293	2.394	-26.247	-195.938	-0.279	-0.293	1.516	-27.083

The differences observed between S.EN. and Etabs are attributed to the Etabs automesh option, which when deactivated, as will be shown in the subsequent cases where the effect is more significant, the results for walls between S.EN. and Etabs&ECtools match.

6.5.2.L shaped wall dead load case (G)

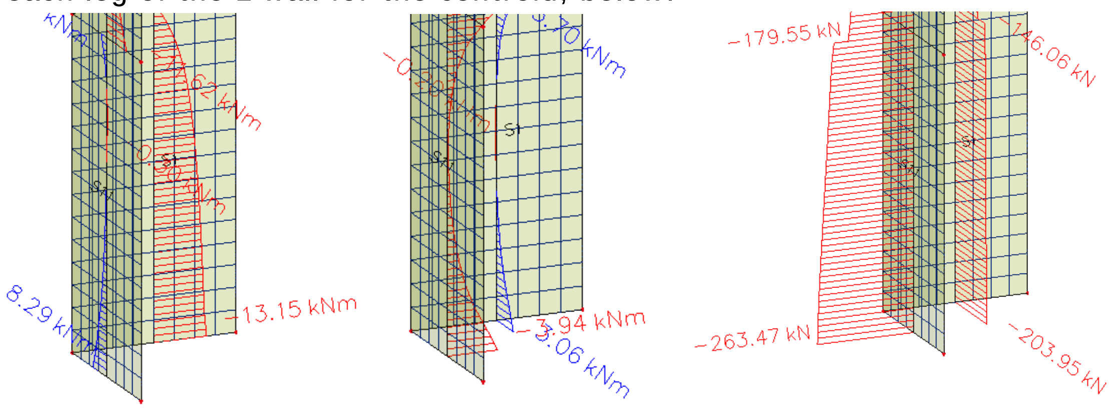
Below the two approaches, Etabs/Pier and S.EN./ECtools, are verified for the L Shaped wall W3 at ground floor.

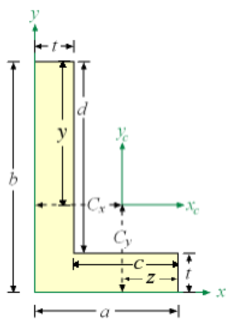
W3 GF	Etabs/ Dead Automesh option	S.EN./ECtools
Shear V22		-7.06
Moment (in plane M33)		-39.65
Axial		-467.7kN
Moment M22		-33.66
Shear V33		-4.95

ECtools calculation is shown below (as exported by ECtools in Scia Translation.xls exported in the temporary S.EN. folder after ECtools is executed)

Pier Forces												
Story	ID	Combo	Nt	V2t	V3t	M2t	M3t	Nb	V2b	V3b	M2b	M3b
STORY1	P01	DEAD	-413.714	-7.062	-4.945	-18.828	-18.469	-467.701	-7.062	-4.945	-33.664	-39.655

The axial and M33 moment are also calculated by using the integration strips of each leg of the L wall for the centroid, below.



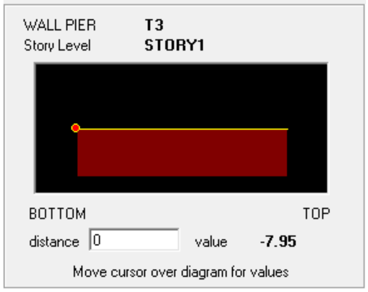
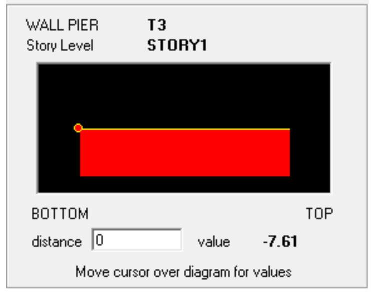
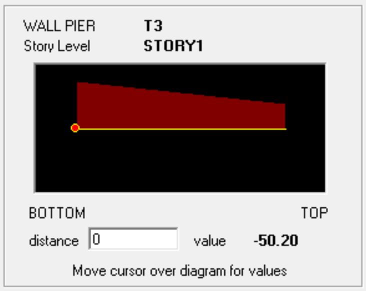
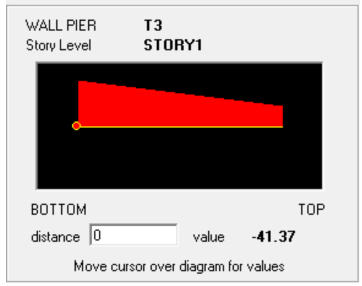
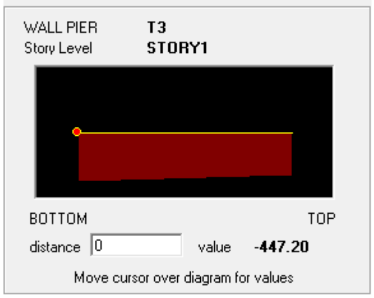
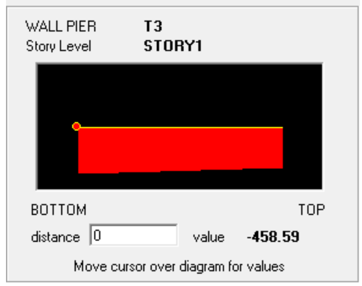


C_x	C_y
$\dfrac{t(2c+b)+c^2}{2(c+b)}$	$\dfrac{t(2d+a)+d^2}{2(d+a)}$

MoL1	-13.15
moL2	-3.94
Mo	-17.09
N1	-203.95
N2	-263.47
Cx	0.465909
dl(m)	0.340909
Mn	-20.2909
Mtot	-37.3809
Ntot	-467.42

This calculation, which is indirect shows a match between ECtools and S.EN., therefore the difference in the results of S.EN.&ECtools to Etabs are attributed to the analytical modeling itself.

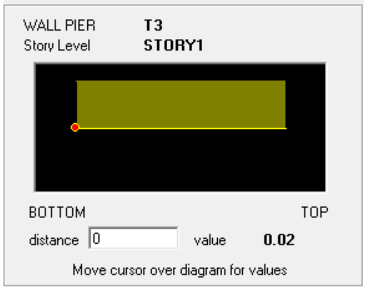
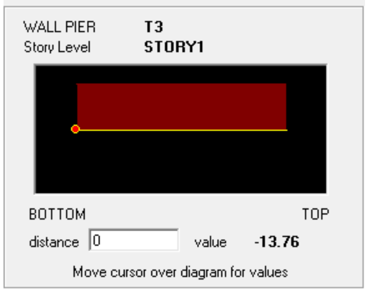
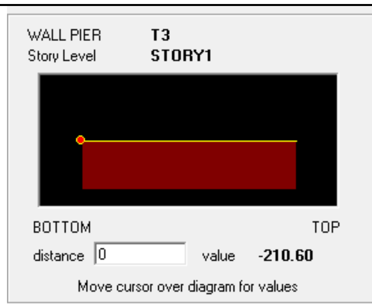
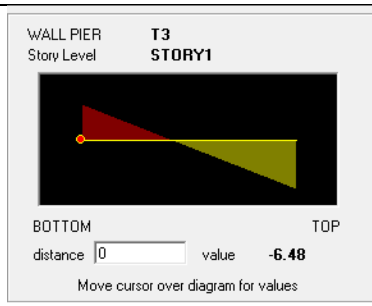
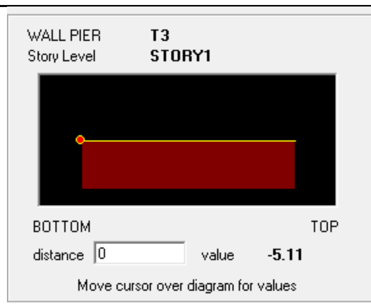
To further investigate the issue, the ETabs model is manually refined to a more dense mesh, thus rendering the automesh option useless. Below these results for the basic internal forces M33, N, V33 are shown:

W3 GF	Etabs/ Dead Automesh option	Etabs/ Dead Refined No Automesh	S.EN./EC tools
Shear V22			-7.06
Moment (in plane M33)			-39.65
Axial			-467.7kN

From the above it is clear that the automesh option in Etabs produces erroneous results in the case of R/C cores, and should be avoided. When this parameter is eliminated the differences between Etabs and S.EN. & ECtools are less than 10%.

6.5.1.L shaped wall live load case (L)

Below the two approaches, Etabs/Pier and S.EN./ECtools, are verified for the L Shaped wall W3 at ground floor.

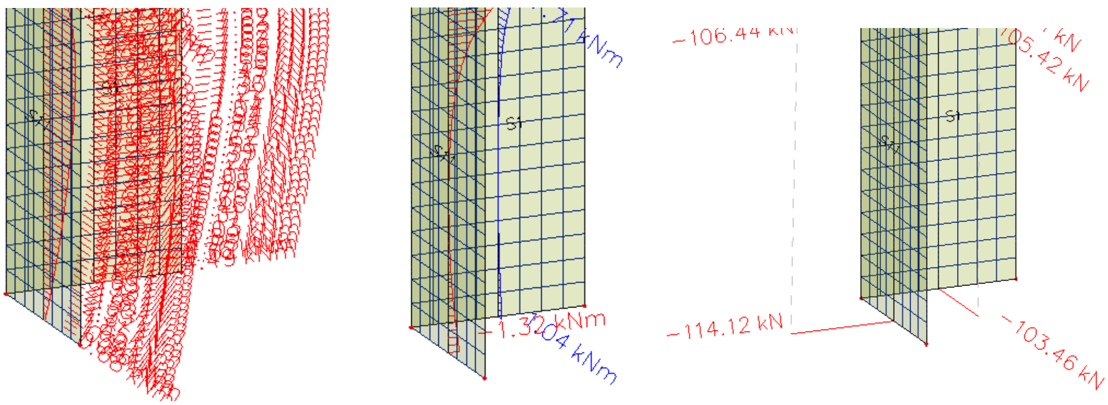
W3 GF	Etabs/ Live Automesh option	S.EN./ECtools
Shear V22		0.21
Moment (in plane M33)		-9.86
Axial		-218.58
Moment M22		-4.41
Shear V33		-0.185

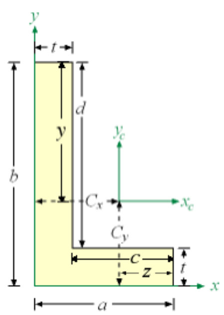
ECtools calculation is shown below (as exported by ECtools in Scia Translation.xls exported in the temporary S.EN. folder after ECtools is executed)

Pier Forces

Story	ID	Combo	Nt	V2t	V3t	M2t	M3t	Nb	V2b	V3b	M2b	M3b
STORY1	P01	L	-215.694	0.214	-0.185	-3.853	-10.501	-218.579	0.214	-0.185	-4.408	-9.858

The axial and M33 moment are also calculated by using the integration strips of each leg of the L wall for the centroid, below.



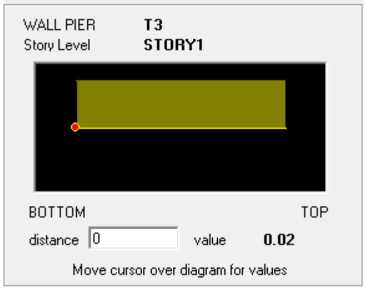


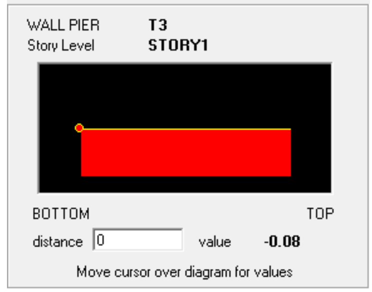
C_x	C_y
$\frac{t(2c+b)+c^2}{2(c+b)}$	$\frac{t(2d+a)+d^2}{2(d+a)}$

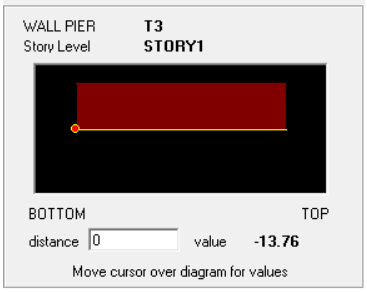
MoL1	-4.36
moL2	-1.32
Mo	-5.68
N1	-103.46
N2	-114.12
Cx	0.465909
dl(m)	0.340909
Mn	-3.63409
Mtot	-9.31409
Ntot	-217.58

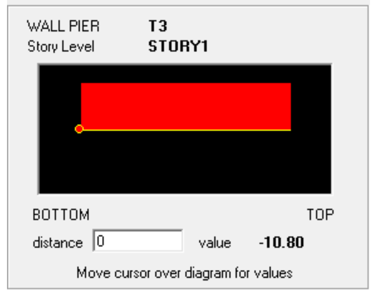
This calculation, which is indirect, shows a match between ECtools and S.EN., therefore the difference in the results of S.EN.&ECtools to Etabs are attributed to the analytical modeling itself.

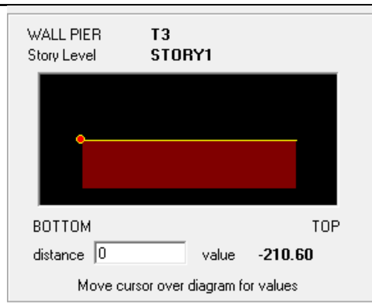
As in the case for the Dead loadcase, to further investigate the issue, the Etabs model is manually refined to a more dense mesh, thus rendering the automesh option useless. Below these results for the basic internal forces M33, N, V33 are shown:

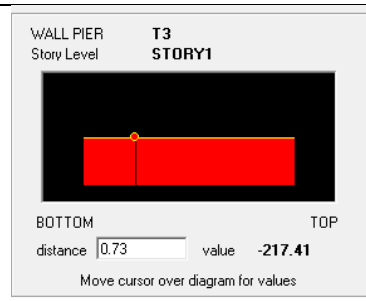
<table>
<tr><th>W3 GF</th><th>Etabs/ Live Automesh option</th><th>Etabs/ Live Refined No Automesh</th><th>S.EN./ECtools</th></tr>
<tr><td>Shear V22</td><td>


</td><td>


</td><td>0.21</td></tr>
<tr><td>Moment (in plane M33)</td><td>


</td><td>


</td><td>-9.858</td></tr>
<tr><td>Axial</td><td>


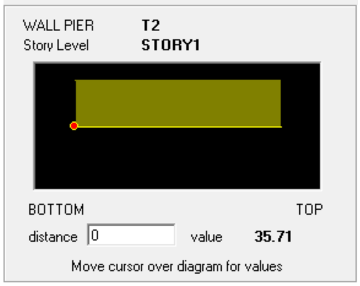
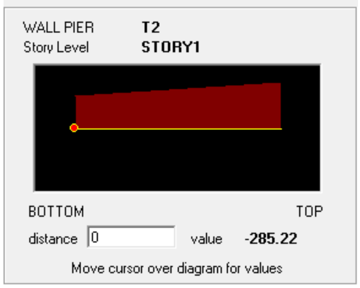
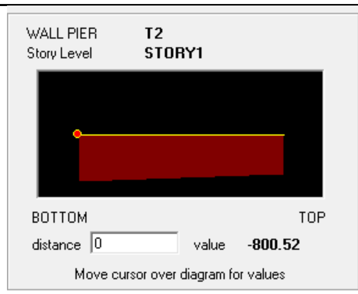
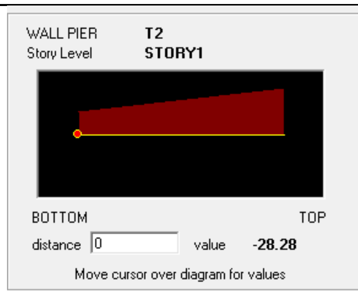
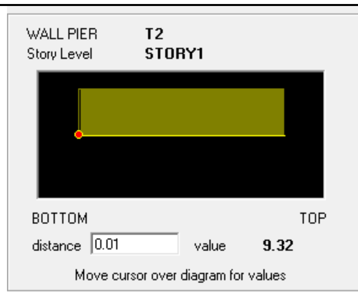
</td><td>


</td><td>-218.58</td></tr>
</table>

From the above it is clear that the automesh option in Etabs produces erroneous results in the case of R/C cores, and should be avoided. When this parameter is eliminated, the differences between Etabs and S.EN. & ECtools are less than 10%.

6.5.2.C shaped wall dead load case (G)

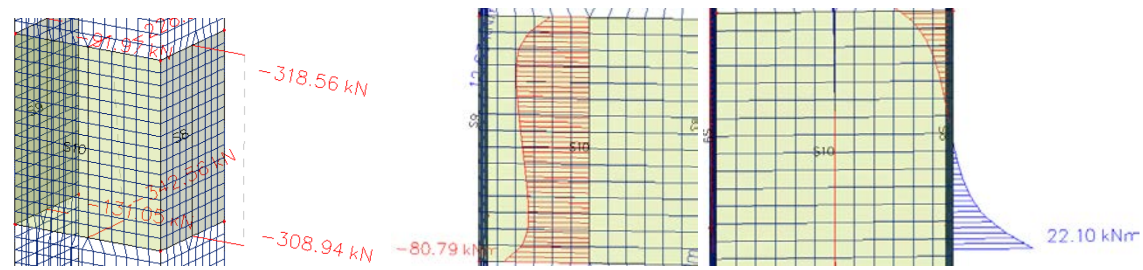
Below the two approaches, Etabs/Pier and S.EN./ECtools, are verified for the C Shaped wall W2 at ground floor.

W2 GF	Etabs/ Dead Automesh option	S.EN./ECtools
Shear V22		28.106
Moment (in plane M33)		-278.62
Axial		-775.25
Moment M22		0.477
Shear V33		17.981

ECtools calculation is shown below (as exported by ECtools in Scia Translation.xls exported in the temporary S.EN. folder after ECtools is executed)

Pier Forces												
Story	ID	Combo	Nt	V2t	V3t	M2t	M3t	Nb	V2b	V3b	M2b	M3b
STORY1	P03	DEAD	-653.885	28.106	17.981	-53.466	-362.939	-775.256	28.106	17.981	0.477	-278.621

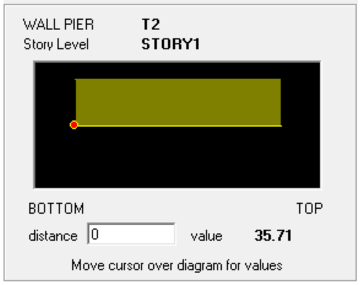
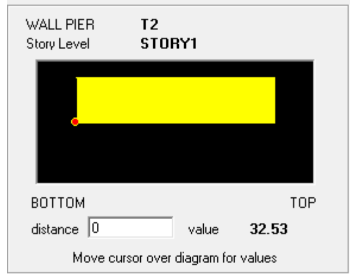
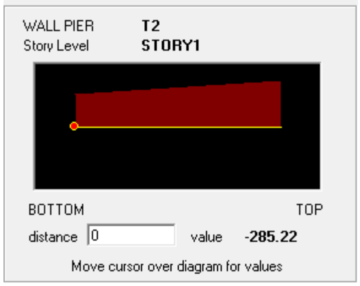
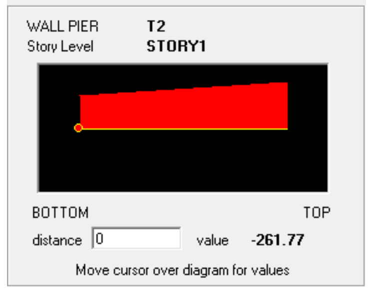
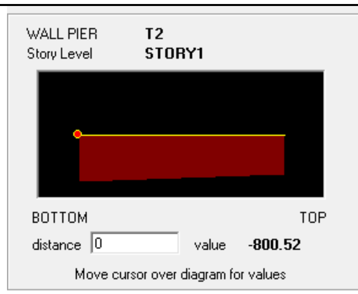
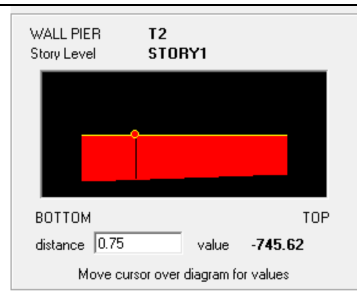
The axial and M33 moment are also calculated by using the integration strips of each leg of the L wall for the centroid, below.



S.EN	Dead
MoL1	-0.03
moL2	-80.79
moL3	22.1
Mo	-58.72
N1	131.05
N2	342.56
N3	308.94
Cy	1.4125
dl 1,3(m)	1.2875
Mn	-229.033
Mtot	-287.753
Ntot	782.55

This calculation, which is indirect, shows a match between ECtools and S.EN., therefore the difference in the results of S.EN.&ECtools to Etabs are attributed to the analytical modeling itself.

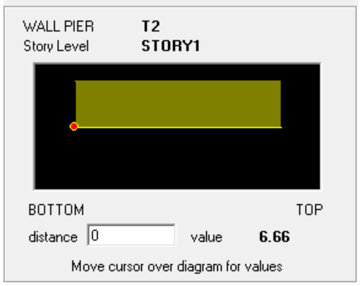
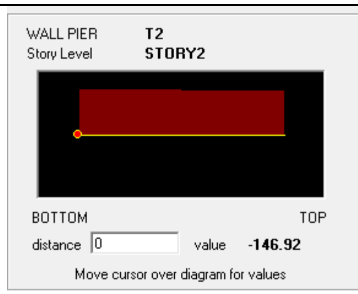
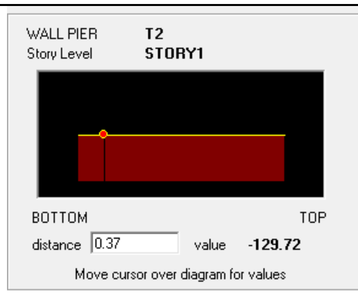
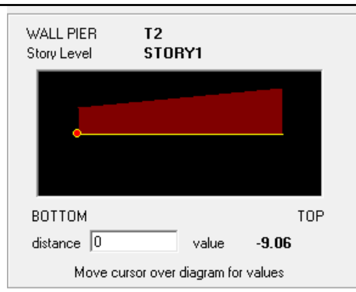
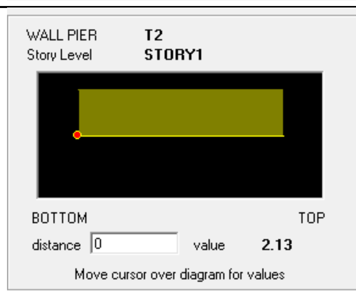
As in the case for the L shaped wall, to further investigate the issue, the Etabs model is manually refined to a more dense mesh, thus rendering the automesh option useless. Below these results for the basic internal forces M33, N, V33 are shown:

W2 GF	Etabs/ Dead Automesh option	Etabs/ Dead Refined No Automesh	S.EN./ECtools
Shear V22			28.106
Moment (in plane M33)			-278.62
Axial			-775.25

From the above it is clear that the automesh option in Etabs produces erroneous results in the case of R/C cores, and should be avoided. When this parameter is eliminated, the differences between Etabs and S.EN. & ECtools are less than 10%.

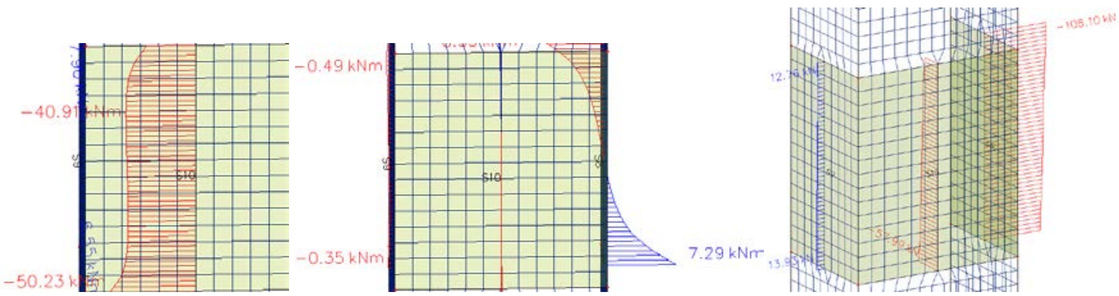
6.5.1.C shaped wall live load case (L)

Below the two approaches, Etabs/Pier and S.EN./ECtools, are verified for the C Shaped wall W2 at ground floor.

W2 GF	Etabs/ Live Automesh option	S.EN./ECtools
Shear V22		-4.28
Moment (in plane M33)		-184.06
Axial		-139.59
Moment M22		-7.94
Shear V33		6.717

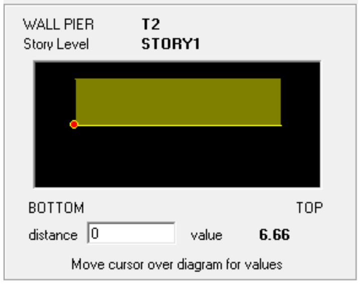
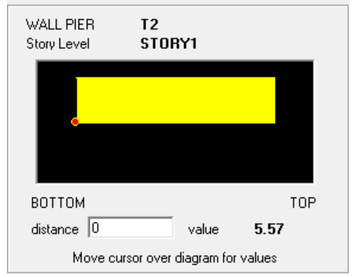
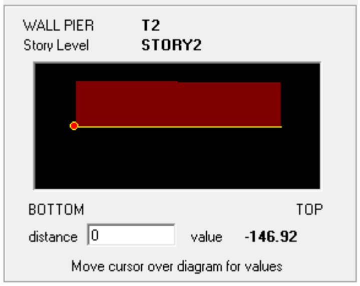
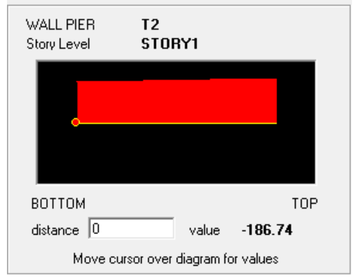
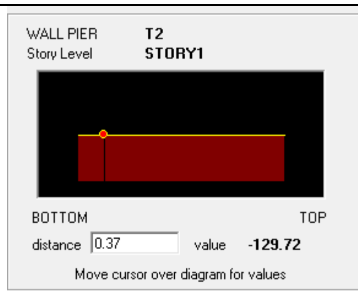
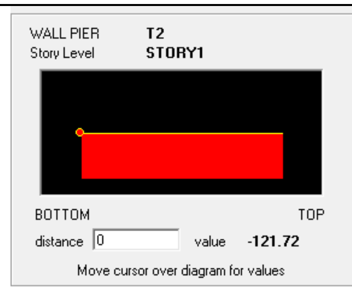
ECtools calculation is shown below (as exported by ECtools in Scia Translation.xls exported in the temporary S.EN. folder after ECtools is executed)

Pier Forces												
Story	ID	Combo	Nt	V2t	V3t	M2t	M3t	Nb	V2b	V3b	M2b	M3b
STORY1	P03	L	-135.075	4.28	6.717	-28.092	-196.906	-139.598	4.28	6.717	-7.939	-184.066



S.EN	Live
MoL1	7.29
moL2	-50.23
moL3	-0.35
Mo	-43.29
N1	97.39
N2	57.99
N3	-13.9
Cy	1.4125
dl 1,3(m)	1.2875
Mn	-143.286
Mtot	-186.576
Ntot	141.48

As in the case for the L shaped wall, to further investigate the issue, the Etabs model is manually refined to a more dense mesh, thus rendering the automesh option useless. Below these results for the basic internal forces M33, N, V33 are shown:

W2 GF	Etabs/ Live Automesh option	Etabs/ Dead Refined No Automesh	S.EN./ECtools
Shear V22			4.28
Moment (in plane M33)			-184.066
Axial			-139.59

From the above it is clear that the automesh option in Etabs produces erroneous results in the case of R/C cores, and should be avoided. When this parameter is eliminated, the differences between Etabs and S.EN. & ECtools are less than 10%.

6.6. Comments on the results of the analysis

The following conclusions have been derived for the comparison of the analysis results for Etabs and S.EN.&ECtools modelling:

- General static force balance is a direct match
- Global assembled masses are a direct match
- Dynamic characteristics (eigenvectors and eigen periods) have a match up to 3%
- Beams internal forces have significant differences of 20% between Etabs and S.EN. Despite the fact that the modelling in Etabs tried to compensate for the T beams modeling clash with the sheel elements of the slabs, the produced results by Etabs, both in bending and shear

behavior underestimate the actual forces as part of the Moment and shear is transferred to the shell elements of the slab that coincide with the flange of the T beams. This effect is more serious in shear than in moment behavior, and does not take place in S.EN. where the internal forces of T beams are calculated as an integration of the 1D rib internal forces with the effective flange of the slab shell elements. It has been proven, in the relevant paragraph that the S.EN. approach is the accurate solution.

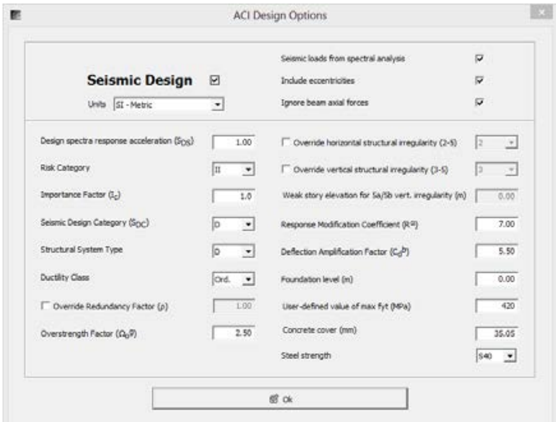
- Column internal forces are a direct match between the two software with less than 5% difference.
- Wall internal forces, either for rectangular walls or RC cores, although the modelling is different, produce results with less than 5% differences. It should be noted that again Etabs, when in automesh option, produces underestimated values for cores, a fact that has been demonstrated by comparing an automesh model to a manualy refined mesh model. S.EN. is not affected by the automatic meshing.

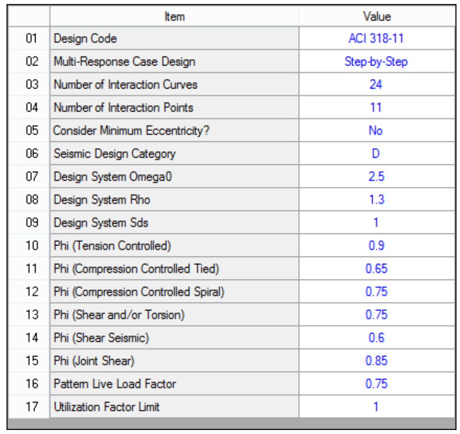
7. Design results

7.1. Beams Flexure ordinary frame

7.1.1. General results

Below the results for beam D16 at storey 3 are presented using the following design parameters for ECtools (left) and Etabs (right):





	Item	Value
01	Design Code	ACI 318-11
02	Multi-Response Case Design	Step-by-Step
03	Number of Interaction Curves	24
04	Number of Interaction Points	11
05	Consider Minimum Eccentricity?	No
06	Seismic Design Category	D
07	Design System Omega0	2.5
08	Design System Rho	1.3
09	Design System Sds	1
10	Phi (Tension Controlled)	0.9
11	Phi (Compression Controlled Tied)	0.65
12	Phi (Compression Controlled Spiral)	0.75
13	Phi (Shear and/or Torsion)	0.75
14	Phi (Shear Seismic)	0.6
15	Phi (Joint Shear)	0.85
16	Pattern Live Load Factor	0.75
17	Utilization Factor Limit	1

For both cases Ductility Class/ Framing type has been set to ordinary:

		Beam Left			**Beam Center**			**Beam Right**		
		Etabs	ECtools		Etabs	ECtools		Etabs	ECtools	
Top	Msd	-50.41	-55.65		0	0		-46.29	-53.46	
	Combo	Dcon26	1			2		Dcon26	3	
	As, cal	3	3.33	9.9%	0	0	0	2.75	3.19	13.8%
	As, min	3.88	7.86		0	2.58		3.67	7.86	
	As, req	3.88	7.86		0	2.58		3.67	7.86	
Bottom	Msd	0	0		39.097	37.87		0	0	
	Combo	Dcon26	2		Dcon26	4		Dcon26	2	
	As, cal	0	0	0.0%	2.27	2.2	-3.2%	0	0	0
	As, min	0	3.93		3.02	3.93		0	3.93	
	As, req	1.53	3.93		3.02	3.93		1.85	3.93	

<u>ECtools combinations</u>

Combo1 : 1.40·D+1.40·GSW+L+0.2·S-0.3·EX+0.9·ECCX-EY+3·ECCY

Combo 2: 0.70·D+0.70·GSW-0.3·EX-0.9·ECCX-EY-3·ECCY

Combo 3: 1.40·D+1.40·GSW+L+0.2·S-0.3·EX-0.9·ECCX-EY-3·ECCY

Combo 4: 1.40·D+1.40·GSW+L+0.2·S+0.3·EX-0.9·ECCX+EY-3·ECCY

<u>Etabs combinations</u>

Dcon26: 1.4D+L+0.2S±1.3EXY

With EXY: EX+0.3EY or EY+0.3EX

7.1.2. Calculated reinforcement

The following table shows the Etabs ACI318-11 design output for the beam D16 (envelope results):

Flexural Reinforcement for Major Axis Moment, M_{u3}

	End-I Rebar Area mm²	End-I Rebar %	Middle Rebar Area mm²	Middle Rebar %	End-J Rebar Area mm²	End-J Rebar %
Top (+2 Axis)	$\boxed{388}$	0.16	0	0	$\boxed{367}$	0.15
Bot (-2 Axis)	153	0.06	303	0.13	185	0.08

Flexural Design Moment, M_{u3}

	End-I Design M_u kN-m	End-I Station Loc mm	Middle Design M_u kN-m	Middle Station Loc mm	End-J Design M_u kN-m	End-J Station Loc mm
Top (+2 Axis)	$\boxed{-29.1863}$	175	0	3262.5	-46.2907	4575
Combo	DCon26		DCon32		DCon26	
Bot (-2 Axis)	$\boxed{19.8737}$	1125	39.0976	2625	$\boxed{23.9638}$	3575
Combo	DCon26		DCon26		DCon26	

The following table shows the ECtools design output.

```
Story STORY3 – Beam B178 – Drawing name : D16

Tee section - bw/h/bm/hf : 0.25/0.50/1.00/0.15, Materials C20.7/S414, Hoops S414, l = 4.75 m

Flexural reinf.          Amin     Amax     Acal     Areq     Asug
------------------------------------------------------------------
Left bottom              3.93     58.64    0.00     3.93     4#4
Left top                 7.86     18.00    3.33     7.86     4#5
Center bottom            3.93     58.64    2.20     3.93     4#4
Center top               2.58     18.00    0.00     2.58     2#5
Right bottom             3.93     58.64    0.00     3.93     4#4
Right top                7.86     18.00    3.19     7.86     4#5

Flexural actions           Nsd       Msd                    Combination
-------------------------------------------------------------------------------------
Left bottom               0.00    -27.01 0.70·D+0.70·GSW-0.39·EX-1.17·ECCX-1.3·EY-3.9·ECCY
Left top                  0.00    -55.65 1.40·D+1.40·GSW+L+0.2·S-0.39·EX+1.17·ECCX-1.3·EY+3.9·ECCY
Center bottom             0.00     37.87 1.40·D+1.40·GSW+L+0.2·S+0.39·EX-1.17·ECCX+1.3·EY-3.9·ECCY
Center top                0.00     13.48 0.70·D+0.70·GSW-0.39·EX-1.17·ECCX-1.3·EY-3.9·ECCY
Right bottom              0.00    -28.09 0.70·D+0.70·GSW-0.39·EX-1.17·ECCX-1.3·EY-3.9·ECCY
Right top                 0.00    -53.46 1.40·D+1.40·GSW+L+0.2·S-0.39·EX-1.17·ECCX-1.3·EY-3.9·ECCY

Shear reinf.             Hoops
```

From the Etabs output the following values seem out of place:

Top Left Moment = -29.18 kNm for DCon26 is not the correct value as is clear from the Etabs flexural detailed design that has the same Moment, for the same Combination as -50.41 kNm.

Bottom Left Moment = 19.87 kN, does not result from the design combination DCon26.

To confirm these observations, the results from the flexural design of Beam left are shown, from Etabs, as following:

Flexural Reinforcement for Moment, M_{u3}

	Required Rebar mm²	+Moment Rebar mm²	-Moment Rebar mm²	Minimum Rebar mm²
Top (+2 Axis)	388	0	300	388
Bottom (-2 Axis)	0	0	0	0

Design Moments, M_{u3}

Design +Moment kN-m	Design -Moment kN-m
0	-50.41

Bottom Right Moment = 23.96kN, does not result from the design combination DCon26.

To confirm these observations, the results from the flexural design of Beam Right are shown, from Etabs, as following:

Flexural Reinforcement for Moment, M_{u3}

	Required Rebar mm²	+Moment Rebar mm²	-Moment Rebar mm²	Minimum Rebar mm²
Top (+2 Axis)	367	0	275	367
Bottom (-2 Axis)	0	0	0	0

Design Moments, M_{u3}

Design +Moment kN-m	Design -Moment kN-m
0	-46.2907

Obviously in the comparison table of par 7.1.1, the correct values have been introduced.

7.1.3. Minimum reinforcement

The minimum calculated reinforcement for the T or rectangular beam as per ACI 318-11 is:

	$\frac{0.25\sqrt{f'_c}}{f_y} b_w d$	$1.4 b_w d / f_y$.	As, min
Rec	3.44	3.93	3.93
T beam	6.87	7.86	7.86

These values have been used by ECtools as minima in the appropriate cases that the beam behaves as T beam or rectangular beam, respectively. In these calculations the b_w for the T-beams has been determined as the minimum of b_{flange} or $2b_w$, as per ACI318M-11 §10.5.1-10.5.3 (in this case $2b_w$)

<u>Etabs uses the rectangular beam approach in all locations (based probably on the ACI commentary) or utilizes the (4/3)A_{cal} as a mimima.</u>
<u>ECtools introduces (4/3)A_{cal} only as a user option, as it is intended only for large beams.</u>

For reference the comparison table and ECtools output is repeated here with the 4/3As option activated:

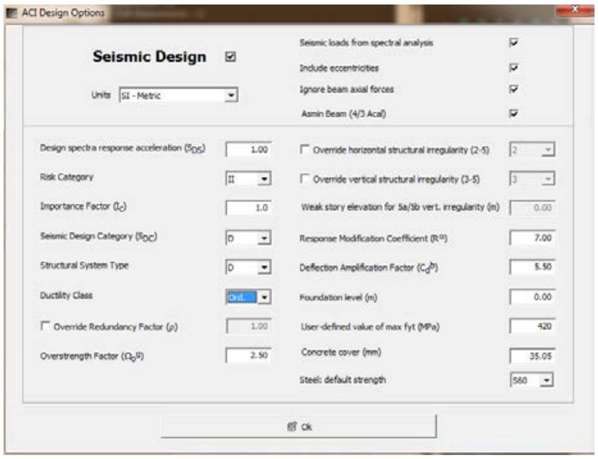
D16/S03 ord 3/4As		Beam Left			Beam Center			Beam Right		
		Etabs	ECtools		Etabs	ECtools		Etabs	ECtools	
Top	Msd	-50.41	-55.65		0	0		-46.29	-53.46	
	Combo	Dcon26	1			2		Dcon26	3	
	As, cal	3	3.33	9.9%	0	0	0	2.75	3.19	13.8%
	As, min	3.88	4.44		0	3.93		3.67	7.86	
	As, req	3.88	4.44		0	0		3.67	7.86	
Bottom	Msd	0	0		39.097	37.87		0	0	
	Combo	Dcon26	2		Dcon26	4		Dcon26	2	
	As, cal	0	0	0.0%	2.27	2.2	-3.2%	0	0	0
	As, min	0	3.93		3.02	3.93		0	3.93	
	As, req	1.53	0		3.02	2.94		1.85	0	

```
Story STORY3 - Beam B178 - Drawing name : D16

Tee section - bw/h/bm/hf : 0.25/0.50/1.00/0.15, Materials C20.7/S414, Hoops S414, l = 4.75 m

Flexural reinf.          Amin     Amax     Acal     Areq     Asug
-----------------------------------------------------------------
Left bottom              3.93    58.64     0.00     0.00      2#4
Left top                 7.86    18.00     3.33     4.44      4#4
Center bottom            3.93    58.64     2.20     2.94      3#4
Center top               2.58    18.00     0.00     0.00      2#4
Right bottom             3.93    58.64     0.00     0.00      2#4
Right top                7.86    18.00     3.19     4.26      4#4

Flexural actions            Nsd      Msd                  Combination
---------------------------------------------------------------------------------
Left bottom                0.00   -27.01 0.70·D+0.70·GSW-0.39·EX-1.17·ECCX-1.3·EY-3.9·ECCY
Left top                   0.00   -55.65 1.40·D+1.40·GSW+L+0.2·S-0.39·EX+1.17·ECCX-1.3·EY+3.9·ECCY
Center bottom              0.00    37.87 1.40·D+1.40·GSW+L+0.2·S+0.39·EX-1.17·ECCX+1.3·EY-3.9·ECCY
Center top                 0.00    13.48 0.70·D+0.70·GSW-0.39·EX-1.17·ECCX-1.3·EY-3.9·ECCY
Right bottom               0.00   -28.09 0.70·D+0.70·GSW-0.39·EX-1.17·ECCX-1.3·EY-3.9·ECCY
Right top                  0.00   -53.46 1.40·D+1.40·GSW+L+0.2·S-0.39·EX-1.17·ECCX-1.3·EY-3.9·ECCY

  4#4                         2#4                         4#4
  |----  #3/170(2) ----------|----------  #3/170(2) ----|
  2#4                         3#4                         2#4

Coupling beam: maximum bidiagonal reinforcement: 0.00 [cm²]
```

7.2. Beams Shear ordinary

7.2.1. General results

Below the results for beam D16 at storey 3 are presented using the following design parameters for ECtools (left) and Etabs (right):



	Item	Value
01	Design Code	ACI 318-11
02	Multi-Response Case Design	Step-by-Step
03	Number of Interaction Curves	24
04	Number of Interaction Points	11
05	Consider Minimum Eccentricity?	No
06	Seismic Design Category	D
07	Design System Omega0	2.5
08	Design System Rho	1.3
09	Design System Sds	1
10	Phi (Tension Controlled)	0.9
11	Phi (Compression Controlled Tied)	0.65
12	Phi (Compression Controlled Spiral)	0.75
13	Phi (Shear and/or Torsion)	0.75
14	Phi (Shear Seismic)	0.6
15	Phi (Joint Shear)	0.85
16	Pattern Live Load Factor	0.75
17	Utilization Factor Limit	1

For both cases Ductility Class/ Framing type has been set to ordinary:

D16/S03		**Beam Left**			**Beam Right**	
ord	Etabs	ECtools		Etabs	ECtools	
Vsd	61.98	88.57	30%	60.94	85.49	29%
Combo	Dcon26	Combo 1		Dcon29	Combo1	
Vc	65.87	67.43	2%	65.87	67.43	2%
As/S cal	2.08	1.47	29%	2.08	1.25	40%
Vwd	30.01	81.86		30.01	81.86	
As/S min		#3/250(2) 5.68			#3/250(2) 5.68	
Combo 1	/+3.9·ECCY					
Dcon26	r EY+0.3EX					
Dcon29	r EY+0.3EX					

7.2.2. Calculated reinforcement

The following table shows the Etabs ACI318-11 envelope design output for the beam D16 (envelope results):

Shear Reinforcement for Major Shear, V_{u2}

End-I Rebar A_v/s mm²/m	Middle Rebar A_v/s mm²/m	End-J Rebar A_v/s mm²/m
208.33	208.33	208.33

Design Shear Force for Major Shear, V_{u2}

End-I Design V_u kN	End-I Station Loc mm	Middle Design V_u kN	Middle Station Loc mm	End-J Design V_u kN	End-J Station Loc mm
36.8041	1125	33.725	3262.5	50.6187	4575
DCon26		DCon26		DCon29	

The output of ECtools shear design is shown in the following figure:

```
Shear reinf.                  Hoops
-------------------------------
Left end                  #3/250(2)
Right end                 #3/250(2)

Shear actions                   Vsd        Vconc        Vwd                              Combination
--------------------------------------------------------------------------------------------
Left end                      65.89        67.43      81.86            1.2·D+1.2·GSW+1.6·L+0.5·LR
Right end                     64.72        67.43      81.86            1.2·D+1.2·GSW+1.6·L+0.5·LR

Shear reinf.                  Hoops
-------------------------------
Left end                  #3/250(2)
Right end                 #3/250(2)

Shear actions               Vsd/Vcd        Vconc        Vwd                              Combination
--------------------------------------------------------------------------------------------
Left end                      88.57        67.43      81.86 1.40·D+1.40·GSW+L+0.2·S+0.39·EX+1.17·ECCX+1.3·EY+3.9·
Right end                     85.49        67.43      81.86 1.40·D+1.40·GSW+L+0.2·S-0.39·EX-1.17·ECCX-1.3·EY-3.9·
```

The Etabs shear force values pointed out in red in the summary table, do not correspond to the shear design as elaborated within Etabs, and the calculated shear reinforcement does not result from these values.

The design for combination Dcon 26 for the left of the beam is shown below:

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

Shear/Torsion Design for V_{u2} and T_u

Rbar A_{vs} mm²/m	Rbar A_t/S mm²/m	Rbar A_l mm²	Design V_{u2} kN	Design T_u kN-m	Design M_{u3} kN-m	Design P_u kN
208.33	0	0	61.9861	0.0306	-50.41	0

Design Forces

Design V_{u2} kN	Design M_{u3} kN-m
61.9861	-21.0218

Design Basis

Design V_{u2} kN	Conc.Area A_c cm²	Area A_g cm²	Tensn.Reinf A-st mm²	Strength f_{ys} MPa	Strength f_{cs} MPa	LtWt.Reduc Factor Unitless
61.9861	1163	1250	388	413.69	20.68	1

Shear Rebar Design

Stress v MPa	Conc.Capacity v_c MPa	Uppr.Limit v_{max} MPa	Conc.Capacity Φv_c MPa	Uppr.Limit Φv_{max} MPa	RebarArea A_v/s mm²/m	Shear ΦV_c kN	Shear ΦV_s kN	Shear ΦV_n kN
0.53	0.76	3.78	0.57	2.83	208.33	65.873	30.0699	95.9429

The design for combination Dcon 26 for the right of the beam is shown below:

Shear/Torsion Design for V_{u2} and T_u

Rbar A_{vs} mm²/m	Rbar A_t/S mm²/m	Rbar A_l mm²	Design V_{u2} kN	Design T_u kN-m	Design M_{u3} kN-m	Design P_u kN
208.33	0	0	60.9408	0.6067	-16.9885	0

Design Forces

Design V_{u2} kN	Design M_{u3} kN-m
60.9408	-46.2907

Design Basis

Design V_{u2} kN	Conc.Area A_c cm²	Area A_g cm²	Tensn.Reinf A-st mm²	Strength f_{ys} MPa	Strength f_{cs} MPa	LtWt.Reduc Factor Unitless
60.9408	1163	1250	367	413.69	20.68	1

Shear Rebar Design

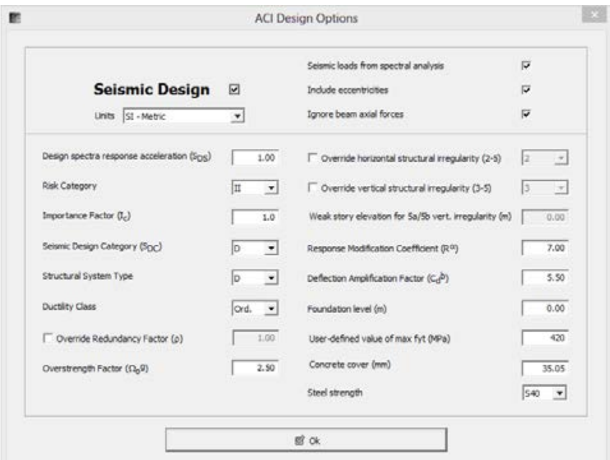
Stress v MPa	Conc.Capacity v_c MPa	Uppr.Limit v_{max} MPa	Conc.Capacity Φv_c MPa	Uppr.Limit Φv_{max} MPa	RebarArea A_v/s mm²/m	Shear ΦV_c kN	Shear ΦV_s kN	Shear ΦV_n kN
0.52	0.76	3.78	0.57	2.83	208.33	65.873	30.0699	95.9429

In both cases, in the comparison table, the correct Etabs values have been included.

7.3. Columns Flexure ordinary frame

7.3.1. General results

Below the results for beam K12 at storey 3 are presented using the following design parameters for ECtools (left) and Etabs (right):



	Item	Value
01	Design Code	ACI 318-11
02	Multi-Response Case Design	Step-by-Step
03	Number of Interaction Curves	24
04	Number of Interaction Points	11
05	Consider Minimum Eccentricity?	No
06	Seismic Design Category	D
07	Design System Omega0	2.5
08	Design System Rho	1.3
09	Design System Sds	1
10	Phi (Tension Controlled)	0.9
11	Phi (Compression Controlled Tied)	0.65
12	Phi (Compression Controlled Spiral)	0.75
13	Phi (Shear and/or Torsion)	0.75
14	Phi (Shear Seismic)	0.6
15	Phi (Joint Shear)	0.85
16	Pattern Live Load Factor	0.75
17	Utilization Factor Limit	1

For both cases Ductility Class/ Framing type has been set to ordinary.

K12/ S3	Bottom			Top		
	Etabs	S.EN. & ECtools	Dif% (max-max)	Etabs	S.EN. & ECtools	Dif% (max-max)
N	-18.45	-20.86		-13.13	-68.39	
M33	16.32	-0.54		-12.71	-33.88	
M22	24.86	25.36		-18.76	32.01	
Combo	Dcon32	COMBO1		Dcon32	COMBO 2	
As,min	12.25	12.25		12.25	12.25	
As,max		49			49	
As,cal	**4.9**	3.33	6.31%	3.76	**5.23**	6.31%
As,req	12.25	12.25	0%	12.25	12.25	0%
COMBO 1	0.70·D+0.70·GSW+1.3(0.3·EX+0.9·ECCX+EY+3·ECCY)					
Combo 2	1.40·D+1.40·GSW+L+0.2·S+1.3(EX-1.96·ECCX+0.3·EY-0.59·ECCY)					
Dcon32	0.7D+1.3EXY ; EXY: EX+0.3EY or EY+0.3EX					

7.3.2. Calculated reinforcement

The suggested reinforcement in both software is 12.25cm², which results from the minimum allowable reinforcement.

The results plotted by Etabs are shown in the following figure:

Longitudinal Reinforcement Design for P_u - M_{u2} - M_{u3} Interaction

Column End	Rebar Area mm²	Rebar %
Top	1225	1
Bottom	1225	1

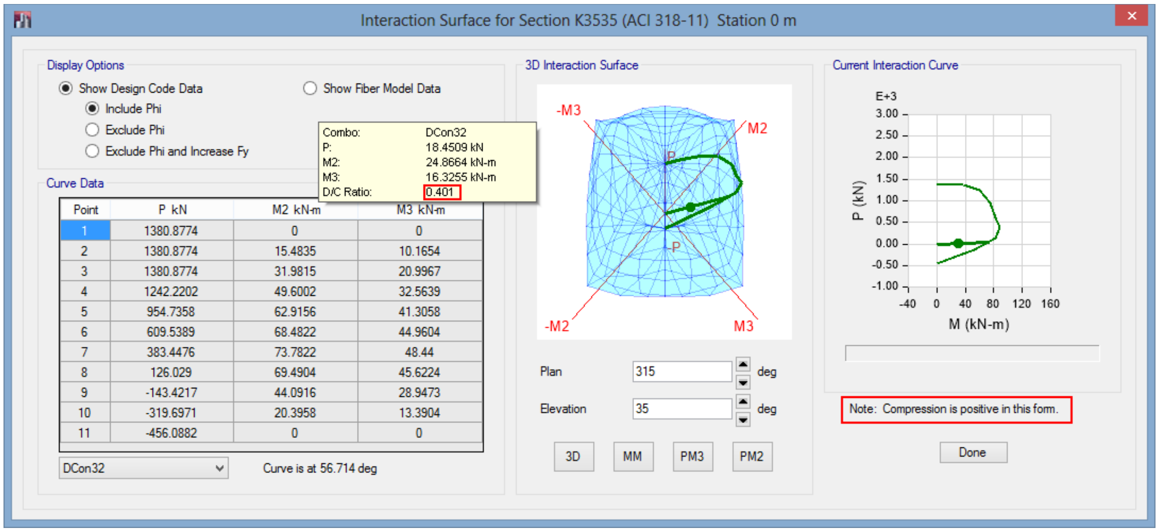
Design Axial Force & Biaxial Moment for P_u - M_{u2} - M_{u3} Interaction

Column End	Design P_u kN	Design M_{u2} kN-m	Design M_{u3} kN-m	Station Loc mm	Controlling Combo
	kN	kN-m	kN-m	mm	
Top	13.1301	-18.7656	-12.7166	2500	DCon32
Bottom	18.4509	24.8664	16.3255	0	DCon32

The results plotted by ECtools are shown in the following figure:

```
Story STORY3 - Column B161 - Drawing name : K12

Rectangular section - b/h : 0.35/0.35, Materials C20.7/S414, Hoops S414, l = 3 m

Flexural reinf.            Amin     Amax     Acal     Areq     Asug
-----------------------------------------------------------------
Top                       12.25    49.00     5.23    12.25     8#5
Bottom                    12.25    49.00     3.33    12.25     8#5

Flexural actions              Nsd        M2sd        M3sd                           Combination
-----------------------------------------------------------------------------------------------
Top                        -68.39      32.00      -33.88 1.40·D+1.40·GSW+L+0.2·S+1.3·EX-2.55·ECCX+0.39·EY-0.76·ECCY
Bottom                     -20.86      25.36       -0.54 0.70·D+0.70·GSW+0.39·EX+1.17·ECCX+1.3·EY+3.9·ECCY
```

<u>It should be noted that Etabs inverts the sign of the axial force for design purposes (+ means compression) as noted in the following graph:</u>



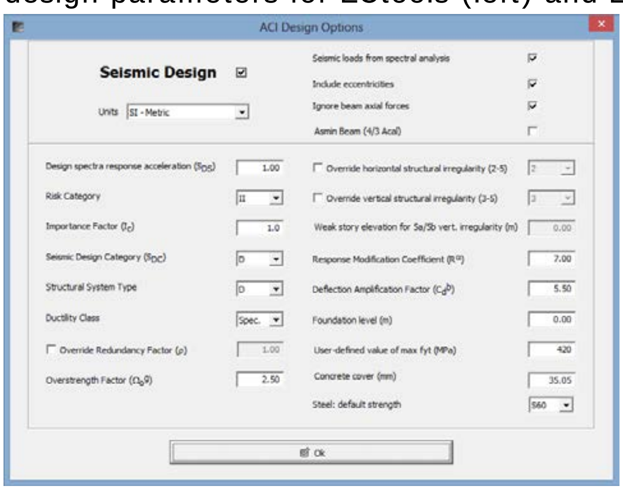
Point	P kN	M2 kN-m	M3 kN-m
1	1380.8774	0	0
2	1380.8774	15.4835	10.1654
3	1380.8774	31.9815	20.9967
4	1242.2202	49.6002	32.5639
5	954.7358	62.9156	41.3058
6	609.5389	68.4822	44.9604
7	383.4476	73.7822	48.44
8	126.029	69.4904	45.6224
9	-143.4217	44.0916	28.9473
10	-319.6971	20.3958	13.3904
11	-456.0882	0	0

From the same graph the utilization factor for the bottom of Dcon32 is 0.401, therefore the calculated As,cal = 4.9cm² (12.25x0.401) while for the top is 3.76cm² (12.25x0.307).

7.4. Columns Flexure special frame

7.4.1. General results

Below the results for beam K12 at storey 3 are presented using the following design parameters for ECtools (left) and Etabs (right):



	Item	Value
01	Design Code	ACI 318-11
02	Multi-Response Case Design	Step-by-Step
03	Number of Interaction Curves	24
04	Number of Interaction Points	11
05	Consider Minimum Eccentricity?	No
06	Seismic Design Category	D
07	Design System Omega0	2.5
08	Design System Rho	1.3
09	Design System Sds	1
10	Phi (Tension Controlled)	0.9
11	Phi (Compression Controlled Tied)	0.65
12	Phi (Compression Controlled Spiral)	0.75
13	Phi (Shear and/or Torsion)	0.75
14	Phi (Shear Seismic)	0.6
15	Phi (Joint Shear)	0.85
16	Pattern Live Load Factor	0.75
17	Utilization Factor Limit	1

For both cases Ductility Class/ Framing type has been set to special.

K12/ S3 Special	Bottom			Top		
	Etabs	S.EN. & ECtools	(max-max)	Etabs	S.EN. & ECtools	(max-max)
N	-18.45	-20.86		-13.13	-14.57	
M33	16.32	-0.54		-12.71	"-51.50/C"	
M22	24.86	25.36		-18.76	29.54	
Combo	Dcon32	COMBO3		Dcon32	COMBO 4	
As,min	12.25	12.25		12.25	12.25	
As,max		73.5			49	
As,cal	**4.9**	3.33	32.04%	3.76	**9.03**	58.36%
As,req	12.25	12.25	0%	12.25	12.25	0%
COMBO 3	0.70·D+0.70·GSW+0.39·EX+1.17·ECCX+1.3·EY+3.9·ECCY					
COMBO 4	0.70·D+0.70·GSW+0.39·EX-1.17·ECCX+1.3·EY-3.9·ECCY					
Dcon32	0.7D+1.3EXY ; EXY: EX+0.3EY or EY+0.3EX					

7.4.2. Calculated reinforcement and joint capacity rule

The suggested reinforcement, in both software, is 12.25cm², which results from the minimum allowable reinforcement.

The results plotted by Etabs are shown in the following figure:

Longitudinal Reinforcement Design for P_u - M_{u2} - M_{u3} Interaction

Column End	Rebar Area mm²	Rebar %
Top	1225	1
Bottom	1225	1

Design Axial Force & Biaxial Moment for P_u - M_{u2} - M_{u3} Interaction

Column End	Design P_u kN	Design M_{u2} kN-m	Design M_{u3} kN-m	Station Loc mm	Controlling Combo
	kN	kN-m	kN-m	mm	
Top	13.1301	-18.7656	-12.7166	2500	DCon32
Bottom	18.4509	24.8664	16.3255	0	DCon32

The results plotted by ECtools are shown in the following figure:

```
Story STORY3 - Column B161 - Drawing name : K12

Rectangular section - b/h : 0.35/0.35, Materials C20.7/S414, Hoops S414, l = 3 m

Flexural reinf.          Amin     Amax     Acal     Areq     Asug
-----------------------------------------------------------------
Top                      12.25    73.50    9.03     12.25    8#5
Bottom                   12.25    73.50    3.33     12.25    8#5

Flexural actions              Nsd        M2sd        M3sd                     Combination
-----------------------------------------------------------------------------------------
Top                          -14.57      29.54     -14.68 0.70·D+0.70·GSW+0.39·EX-1.17·ECCX+1.3·EY-3.9·ECCY
Bottom                       -20.86      25.36      -0.54 0.70·D+0.70·GSW+0.39·EX+1.17·ECCX+1.3·EY+3.9·ECCY
```

It should be noted that ECtools uses a “capacity” moment for the design of the Column resulting from the Moment Capacity of the adjacent beams. In that sense the Top M33 moment is 51.50kNm while the analysis is -14.68 kNm and it significantly differs from the moement used by Etabs which is the analysis one.

The above is based on the Etabs design methodology, which to fulfill the joint capacity rule, performs a check of the moment capacity of the beams and the columns, after "elastic design" has been finalized, as is shown in the following output:

Beam Capacities and Angles (Overstrength factor = 1.25, $\Phi_{(capacity)}$ = 1.0)

	Capacity +veM kN-m	Capacity -veM kN-m	Cos(Angle) Ratio	Sin(Angle) Ratio
Beam 1	19.1498	38.0215	-1	0
Beam 2	43.5494	49.6219	0	-1

Column Moment Capacities About the Axes of the Column Below (Over=1, Φ=1)

	AxialForce (Major)Pu kN	Capacity +veMmajor kN-m	Capacity -veMmajor kN-m	AxialForce (Minor)Pu kN	Capacity +veMminor kN-m	Capacity -veMminor kN-m
Column Below	-13.1301	75.7749	75.7749	-13.1301	75.7749	75.7749

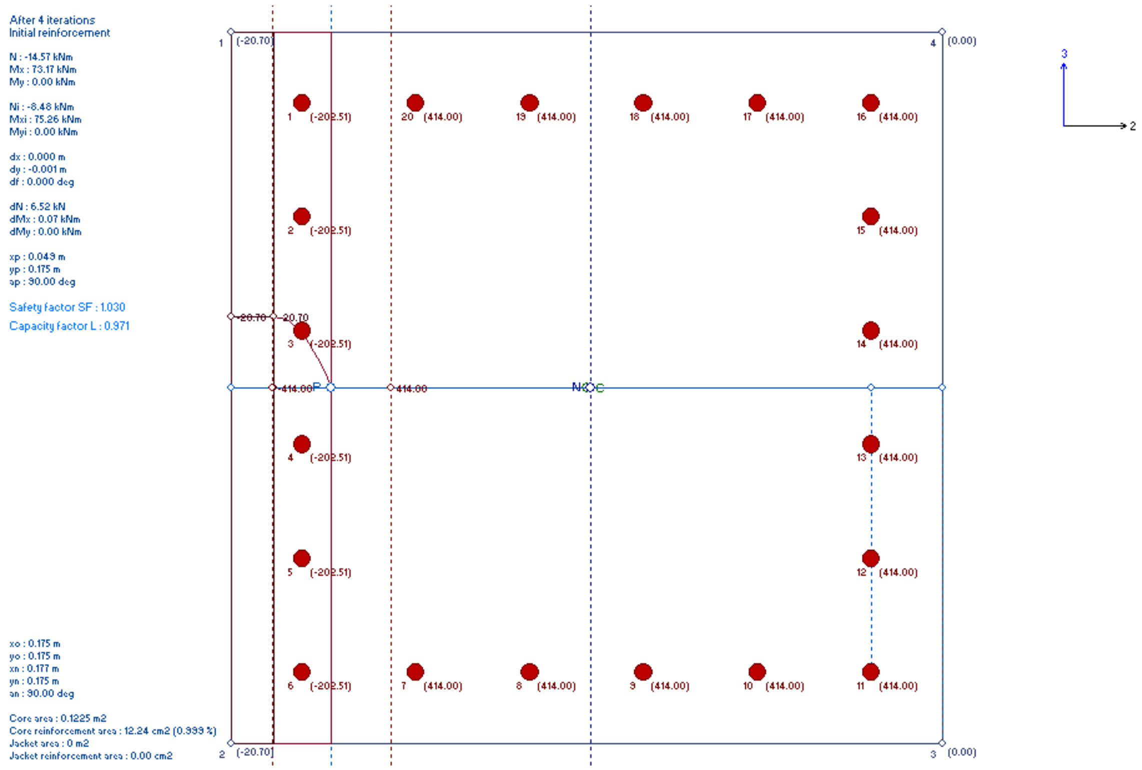
Sum of Beam and Column Capacities About the Axes of the Column Below

	SumBeamCap Major kN-m	SumColCap Major kN-m	SumBeamCap Minor kN-m	SumBeamCap Minor kN-m
Clockwise	30.5439	75.7749	39.898	75.7749
CounterClockwise	15.3276	75.7749	34.8766	75.7749

Beam-Column Flexural Capacity Ratios

	(6/5)B/C Major	(6/5)B/C Major	Col/Beam Minor	Col/Beam Minor
Clockwise	0.243	0.552	4.944	2.173
CounterClockwise	0.484	0.632	2.481	1.899

The value of the moment capacity 75.77 kNm of the column, used for the joint capacity rule application, corresponds to As,req=12.25cm². It is worth pointing out that also for ECtools, results the moment capacity value of this column is exactly the same as shown below:

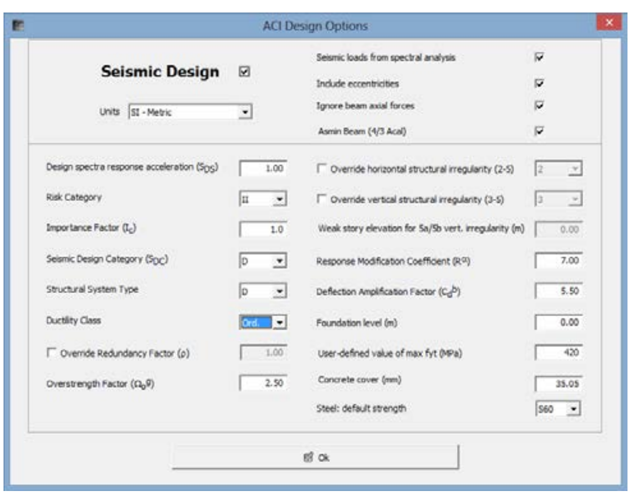


Therefore the joint capacity rule has been applied in both software, via a different path, resulting in the same values.

7.5. Columns Shear ordinary

7.5.1. General results

Below the results for column K12 at storey 3 are presented using the following design parameters for ECtools (left) and Etabs (right):



	Item	Value
01	Design Code	ACI 318-11
02	Multi-Response Case Design	Step-by-Step
03	Number of Interaction Curves	24
04	Number of Interaction Points	11
05	Consider Minimum Eccentricity?	No
06	Seismic Design Category	D
07	Design System Omega0	2.5
08	Design System Rho	1.3
09	Design System Sds	1
10	Phi (Tension Controlled)	0.9
11	Phi (Compression Controlled Tied)	0.65
12	Phi (Compression Controlled Spiral)	0.75
13	Phi (Shear and/or Torsion)	0.75
14	Phi (Shear Seismic)	0.6
15	Phi (Joint Shear)	0.85
16	Pattern Live Load Factor	0.75
17	Utilization Factor Limit	1

For both cases Ductility Class/ Framing type has been set to ordinary.

K12/ S3 Ordinary	Bottom			Top		
	Etabs	S.EN. & ECtools	Dif% (max-max)	Etabs	S.EN. & ECtools	Dif% (max-max)
Vmax	20.4	20.95	3%	20.4	20.95	3%
Combo	Dcon26	COMBO 5		Dcon26	COMBO 5	
Vc	64.12	66.3	3%	64.12	66.78	4%
As/s min	N/A	#3/170(2) 8.35		N/A	#3/170(2) 8.35	
As/s cal	**0**	**0**	**0.00%**	**0**	**0**	**0.00%**
Vwd	N/A	81.55		N/A	81.55	
As/s req	0	#3/170(2)		0	#3/170(2)	
COMBO 5	1.40·D+1.40·GSW+L+0.2·S+0.39·EX-1.17·ECCX+1.3·EY-3.9·ECCY					
Dcon26	1.40D+L+0.2·S+1.3·EXY ; EXY: EX+0.3EY or EY+0.3EX					

7.5.2. Shear reinforcement

The analytical calculation as is plotted from Etabs for the Top & Bottom of column.

Bottom of column detailed calculation is shown below:

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

	Rebar A_v/s mm²/m	Design V_u kN	Design P_u kN	Design M_u kN-m	ΦV_c kN	ΦV_s kN	ΦV_n kN
Major Shear(V2)	0	20.3966	106.3989	7.8614	64.1203	0	64.1203
Minor Shear(V3)	0	20.2208	106.3989	-28.9409	64.1203	0	64.1203

Design Forces

	Factored V_u kN	Factored P_u kN	Factored M_u kN-m
Major Shear(V2)	20.3966	75.3893	28.3898
Minor Shear(V3)	20.2208	75.3893	21.3781

Design Forces

	Factored V_u kN	Factored P_u kN	Factored M_u kN-m
Major Shear(V2)	20.3966	75.3893	28.3898
Minor Shear(V3)	20.2208	75.3893	21.3781

Top of column detailed calculation is shown below:

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

	Rebar A_v/s mm²/m	Design V_u kN	Design P_u kN	Design M_u kN-m	ΦV_c kN	ΦV_s kN	ΦV_n kN
Major Shear(V2)	0	20.3966	95.7573	-7.0641	63.7403	0	63.7403
Minor Shear(V3)	0	20.2208	95.7573	21.6113	63.7403	0	63.7403

Design Forces

	Factored V_u kN	Factored P_u kN	Factored M_u kN-m
Major Shear(V2)	20.3966	64.7478	-22.6019
Minor Shear(V3)	20.2208	64.7478	-16.1523

Design Forces

	Factored V_u kN	Factored P_u kN	Factored M_u kN-m
Major Shear(V2)	20.3966	64.7478	-22.6019
Minor Shear(V3)	20.2208	64.7478	-16.1523

The analytical calculation as is plotted from ECTools for the Top & Bottom of column, is shown below:

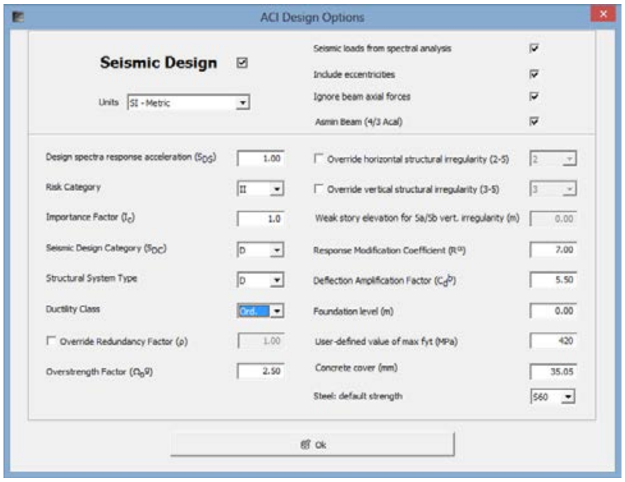
```
Shear friction check: pass

Shear reinf.                 Hoops
----------------------------------
Top                      #3/170(2)
Bottom                   #3/170(2)

Shear actions                  Vsd       Vconc       Vwd                  Combination
------------------------------------------------------------------------------------------
Top                          20.95       66.78      81.55 1.40·D+1.40·GSW+L+0.2·S+0.39·EX-1.17·ECCX+1.3·EY-3.9·
Bottom                       20.95       67.30      81.55 1.40·D+1.40·GSW+L+0.2·S+0.39·EX-1.17·ECCX+1.3·EY-3.9·
```

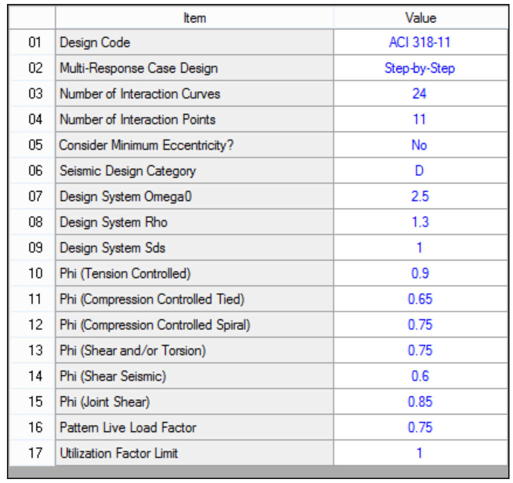
<u>In both software the capacity of the concrete is more than the required reinforcement. ECtools provides also the minimum required shear reinforcement, while Etabs does not (includes it in detailing options)</u>

7.6. Columns Shear Special

7.6.1. General results

Below the results for beam K12 at storey 3 are presented using the following design parameters for ECtools (left) and Etabs (right):





	Item	Value
01	Design Code	ACI 318-11
02	Multi-Response Case Design	Step-by-Step
03	Number of Interaction Curves	24
04	Number of Interaction Points	11
05	Consider Minimum Eccentricity?	No
06	Seismic Design Category	D
07	Design System Omega0	2.5
08	Design System Rho	1.3
09	Design System Sds	1
10	Phi (Tension Controlled)	0.9
11	Phi (Compression Controlled Tied)	0.65
12	Phi (Compression Controlled Spiral)	0.75
13	Phi (Shear and/or Torsion)	0.75
14	Phi (Shear Seismic)	0.6
15	Phi (Joint Shear)	0.85
16	Pattern Live Load Factor	0.75
17	Utilization Factor Limit	1

For both cases Ductility Class/ Framing type has been set to special.

K12/ S3 Special	Bottom			Top		
	Etabs	S.EN. & ECtools	Dif% (max-max)	Etabs	S.EN. & ECtools	Dif% (max-max)
Vmax	33	38.14	**13.48%**	33	38.14	**13.48%**
Combo	Dcon32	COMBO 5		Dcon32	COMBO 5	
Vc	0	0		0	p	
As/s min	N/A	#4/80(2) (32.25)		N/A	#4/80(2) (32.25)	
As/s cal	**3.5**	**4.96**	**29.50%**	**3.5**	**4.96**	**29.50%**
Vwd	36.4	247.76		36.4	247.76	
As/s req	3.5	#4/80(2) (32.25)		3.5	#4/80(2) (32.25)	
COMBO 5	1.40·D+1.40·GSW+L+0.2·S+0.39·EX-1.17·ECCX+1.3·EY-3.9·ECCY					
Dcon26	1.40D+L+0.2·S+1.3·EXY ; EXY: EX+0.3EY or EY+0.3EX					

7.6.2.Shear Capacity design

The analytical calculation as is plotted from Etabs for the Bottom of column, is shown below:

Capacity Shear (Part 1 of 2)

	Shear V_p kN	Long.Rebar $A_{s(Bot)}$ %	Long.Rebar $A_{s(Top)}$ %	Cap.Moment M_{posBot} kN-m
Major Shear(V2)	25.3477	1	1	97.757
Minor Shear(V3)	33.0813	1	1	97.757

Capacity Shear (Part 2 of 2)

Cap.Moment M_{negTop} kN-m	Cap.Moment M_{negBot} kN-m	Cap.Moment M_{posTop} kN-m
97.757	97.757	97.757
97.757	97.757	97.757

Design Basis

Shr Reduc Factor Unitless	Strength f_{ys} MPa	Strength f_{cs} MPa	Area A_g cm²
1	413.69	20.68	1225

Concrete Shear Capacity

	Design V_u kN	Conc.Area A_{cu} cm²	Tensn.Rein A_{st} mm²
Major Shear(V2)	25.3477	1065	613
Minor Shear(V3)	33.0813	1065	613

Shear Rebar Design

	Stress v MPa	Conc.Cpcty v_c MPa	Uppr.Limit v_{max} MPa	Φv_c MPa	Φv_{max} MPa	RebarArea A_v/s mm²/m
Major Shear(V2)	0.24	0.78	3.02	0.58	0	291.67
Minor Shear(V3)	0.31	0.78	3.02	0.58	2.27	350.41

The analytical calculation as is plotted from ECTools for the Top & Bottom of column, is shown below:

```
Shear friction check: pass

Shear reinf.               Hoops
--------------------------------
Top                      #4/80(2)
Bottom                   #4/80(2)

Shear actions             Vsd       Vconc      Vwd                          Combination
-----------------------------------------------------------------------------------------
Top                      38.14      0.00    247.76 1.40·D+1.40·GSW+L+0.2·S+0.39·EX-1.17·ECCX+1.3·EY-3.9
Bottom                   38.14      0.00    247.76 1.40·D+1.40·GSW+L+0.2·S+0.39·EX-1.17·ECCX+1.3·EY-3.9
```

The shear forces used in Etabs (pointed out in red) are calculated as the minimun of the Capacity Shear (V^c) due to the end moment capacity and the capacity of the beams (V^b), as following:

(a) V^c Capacity shear due to moments

$$V_e^c = \max\left\{V_{e1}^c, V_{e2}^c\right\} \qquad \text{(ACI 21.6.5.1, R21.6.5.1, Fig. R21.5.4)}$$

where,

$$V_{e1}^c = \frac{M_I^- + M_J^+}{L}, \qquad \text{(ACI 21.6.5.1, Fig. R21.5.4)}$$

$$V_{e2}^c = \frac{M_I^+ + M_J^-}{l}, \qquad \text{(ACI 21.6.5.1, Fig. R21.5.4)}$$

Which applied in this case results in a capacity shear of:
V^c= 2x97.75/3 = 65.1 KN instead of V^b= 33 kN.

(b) Capacity Shear due to capacity of framing beams, i.e.

$$V_{e1}^b = \frac{M_{r1}}{H},$$

$$V_{e2}^b = \frac{M_{r2}}{H},$$

M_{r1} = Sum of beam moment resistances with clockwise joint rotations,

M_{r2} = Sum of beam moment resistances with counter-clockwise joint rotations, and

H = Distance between the inflection points, which is equal to the mean height of the columns above and below the joint. If there is no column at the top of the joint, the distance is taken as one-half of the height of the column at the bottom of the joint.

Which applied in this case results in a capacity shear of:
V^b=33 kN

The resulting shear reinforcement 350 mm^2/m for Etabs corresponds to a shear force capacity of the rebars Vwd:

$$\frac{A_v}{s} = \frac{(V_u - \phi V_c)}{\phi f_{ys} d}, \qquad \text{(ACI 11.4.7.1, 11.4.7.2)}$$

Vwd=350x10^{-6}x0.75x414x10^{3}x0.335 = 36.4 kN, which corresponds to the shear force used ignoring the concrete contribution to the shear capacity.

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

	Rebar A_v /s mm²/m	Design V_u kN	Design P_u kN	Design M_u kN-m	ΦV_c kN	ΦV_s kN	ΦV_n kN
Major Shear(V2)	291.67	25.3477	49.4604	-4.2453	0	27.5354	27.5354
Minor Shear(V3)	350.41	33.0813	49.4604	-25.5172	0	33.0813	33.0813

Ignoring the concrete contribution to the shear capacity is a correct approach for Special MRF.

ECtools uses as capacity shear 38.14kN, which is calculated using the following equations, which essentially use the same approach as explained previously for Etabs:

$$\varphi V_n \geq min \begin{cases} V_u = \mathbf{1.25}\dfrac{M_{n,t} + M_{n,b}}{l_u}(see\ figure\ above) \\ V_u\ with\ E = {}^{\Omega_o}/_{\rho} \cdot E, \qquad e.g.\ V_u = 1.2D + \Omega_o E + (1.0L\ or\ 0.5L) + 0.2S \end{cases}$$

The concrete contribution ΦVc= 54kN is set to 0, and the calculated shear reinforcement is for 38kN, As/s = 496 mm^2/m.
The minimum shear reinforcement <u>3225mm^2/m</u> corresponds to Vwd= 247.76 kN which is much more than the required by the calculation.

<u>From the overview of this case, it is deemed that the capacity shear in Etabs, as calculated by the beam capacity shears, is underestimated as Etabs has underestimated the design shear forces for beams as has been proven in the analysis (ignoring the shear of the shell elements).</u>

7.7. Rectangular Wall Design ordinary ductility class

The design output from S.EN & ECtools for the rectangular wall W1 at story 1 (above basement) is shown in the following screen capture:

```
Story STORY1 - Wall P02 - Drawing name : W1

Section I (P02) - Materials C20.7/S414, Hoops S414

Base story : STORY1 - Total Height : 9.00 m
Bottom elev. : 0.00 m - Top elev. : 3.00 m

Flexural reinf.          | Col     Amin     Amax     Acal     Areq     Asug
Top                         1    11.13    44.50     0.73    11.13      6#5
Bottom                      1    11.13    44.50    11.00    11.13      6#5
Top                         2    11.13    44.50     0.73    11.13      6#5
Bottom                      2    11.13    44.50    11.00    11.13      6#5

Flexural actions            Nsd       M2sd       M3sd                              Combination
-----------------------------------------------------------------------------------------------
Top                     -223.02     -33.68     -30.98 0.70·D+0.70·GSW+0.39·EX+1.17·ECCX-1.3·EY+3.9·ECCY
Bottom                  -175.39      34.91     711.03 0.70·D+0.70·GSW+1.3·EX+2.55·ECCX-0.39·EY+0.76·ECCY

Shear reinf.           Col       Hoops
--------------------------------------
Top/Bottom               1      #3/250
Top/Bottom               2      #3/250

Adequate confined length in web A (required : 0.00 < 0.00)

Shear friction check passed for Web A.

Shear actions               Vsd                     Combination
-----------------------------------------------------------------
Vsd in dir. 2              7.88                     1.4·D+1.4·GSW
Vsd in dir. 3              1.36                     1.4·D+1.4·GSW

Web            Vsd      Vconc        Vwd     VRd,max Horizontal    Vertical
---------------------------------------------------------------
A             7.88     174.03     400.16     1132.88   2# 3@220   2# 3@220

Shear actions               Vsd                     Combination
-----------------------------------------------------------------
Vsd in dir. 2            258.28 1.40·D+1.40·GSW+L+0.2·S-1.3·EX-2.55·ECCX+0.39·EY-0.76·ECCY
Vsd in dir. 3             28.62 0.70·D+0.70·GSW+0.39·EX+1.17·ECCX-1.3·EY+3.9·ECCY

Web            Vsd      Vconc        Vwd     VRd,max Horizontal    Vertical
---------------------------------------------------------------
A           258.28     174.03     400.16      288.68   2# 3@220   2# 3@150
```

The design output from Etabs for the rectangular wall W1 at story 1 (above basement) is shown in the following screen capture:

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	0	0	1500	0	1500	250
Bottom	Leg 1	0	0	1500	0	1500	250

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	938	0.0025	0.0029	DWal32	302.9763	-19.5932	-88.5806	375000
Bottom	2449	0.0065	0.0029	DWal32	154.8526	-35.3405	-615.2632	375000

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	625	DWal29	354.5316	36.7899	184.7018	169.9211	402.6192
Bottom	Leg 1	625	DWal29	393.5676	527.1699	185.6632	128.8999	315.0584

Boundary Element Check

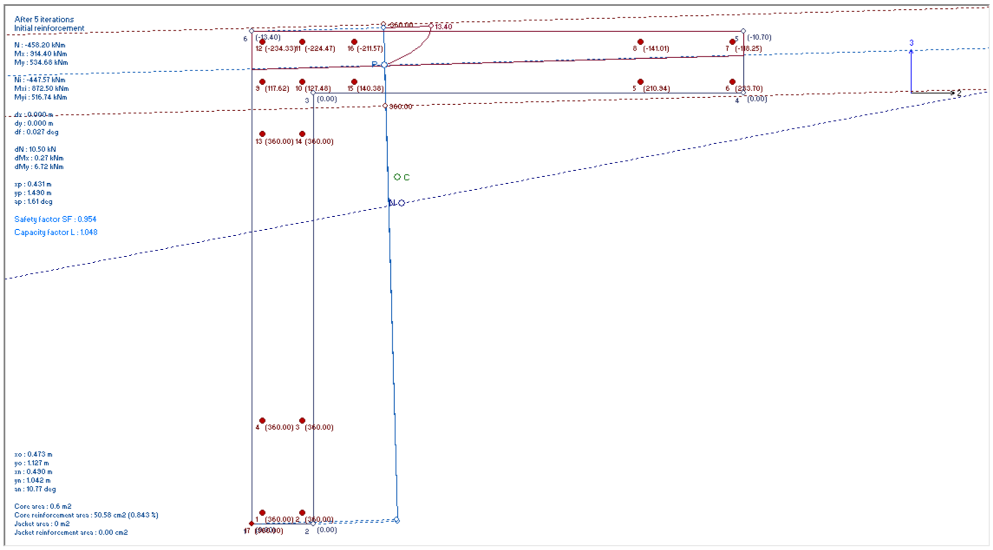
Station Location	ID	Edge Length (mm)	Governing Combo	P_u kN	M_u kN-m	Stress Comp MPa	Stress Limit MPa	C Depth mm	C Limit mm
Top–Left	Leg 1	0	DWal26	713.1184	-133.1026	3.32	4.14	272.8	357.1
Top–Right	Leg 1	0	DWal26	713.1184	9.5315	2	4.14	0	357.1
Bottom–Left	Leg 1	0	DWal26	752.2653	-670.4378	9.16	4.14	283.1	357.1
Botttom–Right	Leg 1	0	DWal26	752.2653	501.3597	7.35	4.14	283.1	357.1

ECtools calculates for the bottom of the wall (base of wall) flexural reinforcement of As, req= 11.13+11.13 = 22.26 cm^2 while Etabs calculates As,req= 24.49cm^2, i.e. a difference of 5%

ECtools calculates for the bottom of the wall (base of wall) shear reinforcement 2x3#/280 As/s=5.07 cm^2/m while Etabs calculates As/s = 6.25 cm^2/m, i.e. 18% difference.

7.8. L shaped Wall Design ordinary ductility class

The design output from S.EN & ECtools for the L shaped wall W3 at story 1 (above basement) is shown in the following screen captures:




```
Story STORY1 - Wall P01 - Drawing name : W3

Section L (P01) - Materials C20.7/S414, Hoops S414

Base story : STORY1 - Total Height : 9.00 m
Bottom elev. : 0.00 m - Top elev. : 3.00 m

Flexural reinf.          Col     Amin     Amax     Acal     Areq     Asug
Top                        1    11.13    44.50     2.31    11.13     6#6
Bottom                     1    11.13    44.50    13.78    13.78     6#6
Top                        2    16.00    64.00     3.32    16.00     6#7
Bottom                     2    16.00    64.00    19.82    19.82     6#7
Top                        3    11.13    44.50     2.31    11.13     6#6
Bottom                     3    11.13    44.50    13.78    13.78     6#6

Flexural actions            Nsd       M2sd      M3sd                          Combination
-------------------------------------------------------------------------------------------
Top                     -148.97    -249.85    -224.50 0.70·D+0.70·GSW+0.39·EX+1.17·ECCX-1.3·EY+3.9·ECCY
Bottom                  -458.20    1095.29     489.66 0.70·D+0.70·GSW-0.39·EX-1.17·ECCX+1.3·EY-3.9·ECCY

Shear reinf.          Col       Hoops
--------------------------------------
Top/Bottom              1      #3/250
Top/Bottom              2      #3/250
Top/Bottom              3      #3/250

Adequate confined length in web A (required : 0.00 < 0.00)
Adequate confined length in web B (required : 0.00 < 0.00)

Shear friction check passed for Web A.
Shear friction check passed for Web B.

Shear actions              Vsd                                 Combination
--------------------------------------------------------------------------
Vsd in dir. 2            10.33                               1.4·D+1.4·GSW
Vsd in dir. 3             7.02                               1.4·D+1.4·GSW

Web           Vsd      Vconc        Vwd    VRd,max Horizontal   Vertical
-------------------------------------------------------------------------
A           10.33     188.53     433.51    1227.29    2# 3@220   2# 3@220
B            7.02     188.53     433.51    1227.29    2# 3@220   2# 3@220

Shear actions              Vsd                                 Combination
--------------------------------------------------------------------------
Vsd in dir. 2             177.20 1.40·D+1.40·GSW+L+0.2·S-0.39·EX+1.17·ECCX-1.3·EY+3.9·ECCY
Vsd in dir. 3             296.78 1.40·D+1.40·GSW+L+0.2·S+0.39·EX+1.17·ECCX-1.3·EY+3.9·ECCY

Web           Vsd      Vconc        Vwd    VRd,max Horizontal   Vertical
-------------------------------------------------------------------------
A          177.20     188.53     433.51     341.64    2# 3@220   2# 3@220
B          296.78     188.53     433.51     341.64    2# 3@220   2# 3@220

.........................................................................................
```

The design output from Etabs for the L shaped wall W3 at story 1 (above basement) is shown in the following screen capture:

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	0	11800	0	13300	1500	250
Top	Leg 2	0	13300	1500	13300	1500	250
Bottom	Leg 1	0	11800	0	13300	1500	250
Bottom	Leg 2	0	13300	1500	13300	1500	250

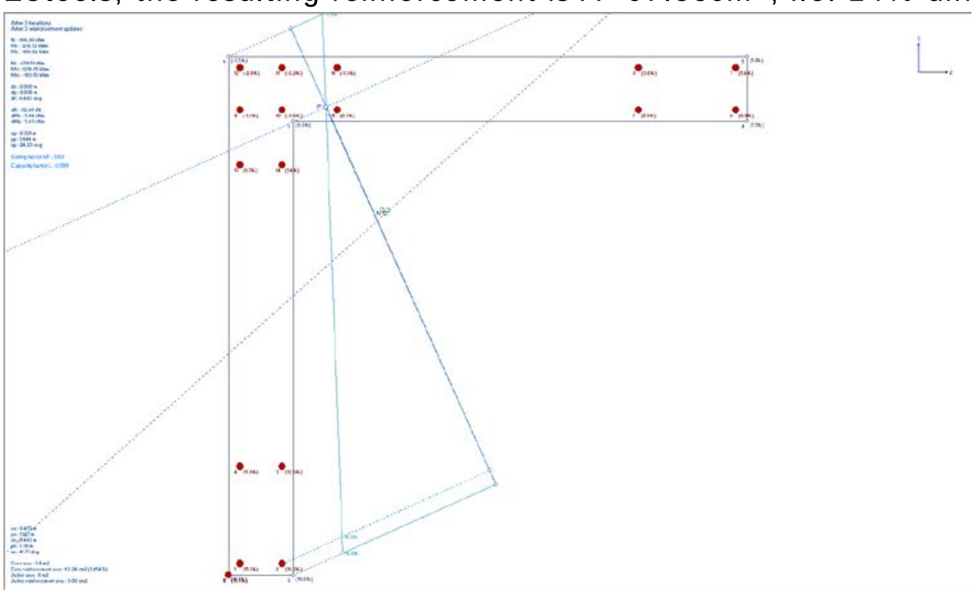
Flexural Design for P_u, M_{u2} and M_{u3}

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	2116	0.0028	0.0109	DWal32	147.0345	342.1035	-293.2779	750000
Bottom	7805	0.0104	0.0109	DWal32	186.0155	1090.5422	-701.9108	750000

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	625	DWal26	93.686	75.8405	259.8191	294.4227	527.1208
Top	Leg 2	625	DWal26	-66.7463	10.0877	173.0597	270.3579	503.0559
Bottom	Leg 1	625	DWal26	-233.106	513.3087	267.7524	151.5176	384.2157
Bottom	Leg 2	625	DWal29	-545.92	188.2162	144.9007	162.1656	394.8637

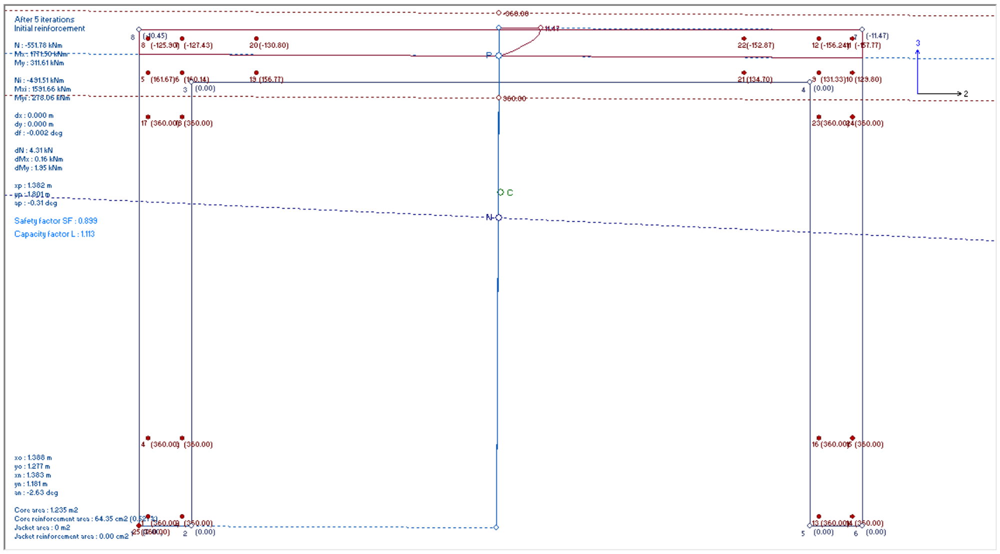
ECtools calculates for the bottom of the wall (base of wall) flexural reinforcement of As, req= 13.78+19.82+13.78= 47.38 cm^2 while Etabs calculates As,req= 78.7cm^2. If the N-M2-M3 of Etabs are used as input for ECtools, the resulting reinforcement is A=67.5ocm^2, i.e. 14% difference.



ECtools calculates for the bottom of the wall (base of wall) shear reinforcement per leg 2x3#/280 As/s=5.07 cm^2/m while Etabs calculates As/s = 6.25 cm^2/m, i.e. 18% difference per leg.

7.1. C shaped Wall Design ordinary ductility class

The design output from S.EN & ECtools for the C shaped wall W2 at story 1 (above basement) is shown in the following screen captures:



```
Story STORY1 - Wall P03 - Drawing name : W2

Section U (P03) - Materials C20.7/S414, Hoops S414

Base story : STORY1 - Total Height : 9.00 m
Bottom elev. : 0.00 m - Top elev. : 3.00 m

Flexural reinf.          Col    Amin    Amax    Acal    Areq    Asug
Top                       1    11.13   44.50    4.08   11.13    6#6
Bottom                    1    11.13   44.50   12.12   12.12    6#6
Top                       2    17.22   68.88    6.32   17.22    6#7
Bottom                    2    17.22   68.88   18.76   18.76    6#7
Top                       3    17.22   68.88    6.32   17.22    6#7
Bottom                    3    17.22   68.88   18.76   18.76    6#7
Top                       4    11.13   44.50    4.08   11.13    6#6
Bottom                    4    11.13   44.50   12.12   12.12    6#6

Flexural actions           Nsd       M2sd       M3sd                          Combination
------------------------------------------------------------------------------------------
Top                     -428.43    106.63   -1787.20 0.70·D+0.70·GSW-1.3·EX+2.55·ECCX+0.39·EY+0.76·ECCY
Bottom                  -551.78   2058.94     317.83 0.70·D+0.70·GSW+0.39·EX+1.17·ECCX+1.3·EY+3.9·ECCY

Shear reinf.          Col      Hoops
-----------------------------------
Top/Bottom             1     #3/250
Top/Bottom             2     #3/250
Top/Bottom             3     #3/250
Top/Bottom             4     #3/250

Adequate confined length in web A (required : 0.00 < 0.00)
Adequate confined length in web B (required : 0.00 < 0.00)
Adequate confined length in web C (required : 0.00 < 0.00)

Shear friction check passed for Web A.
Shear friction check passed for Web B.
Shear friction check passed for Web C.

Shear actions              Vsd                           Combination
---------------------------------------------------------------------
Vsd in dir. 2            41.63           1.2·D+1.2·GSW+1.6·L+0.5·LR
Vsd in dir. 3            32.42           1.2·D+1.2·GSW+1.6·L+0.5·LR

Web         Vsd      Vconc       Vwd    VRd,max Horizontal   Vertical
----------------------------------------------------------------
A         41.63     327.75    753.64    2133.59    2# 3@220   2# 3@220
B         16.21     223.33    513.54    1453.86    2# 3@220   2# 3@220
C         16.21     223.33    513.54    1453.86    2# 3@220   2# 3@220

Shear actions              Vsd                           Combination
---------------------------------------------------------------------
Vsd in dir. 2           502.99 1.40·D+1.40·GSW+L+0.2·S+1.3·EX-2.55·ECCX-0.39·EY-0.76·ECCY
Vsd in dir. 3           474.77 1.40·D+1.40·GSW+L+0.2·S+0.39·EX+1.17·ECCX+1.3·EY+3.9·ECCY

Web         Vsd      Vconc       Vwd    VRd,max Horizontal   Vertical
----------------------------------------------------------------
A        502.99     327.75    753.64     790.42    2# 3@220   2# 3@220
B        237.39     223.33    513.54     503.86    2# 3@220   2# 3@220
C        237.39     223.33    513.54     503.86    2# 3@220   2# 3@220
```

The design output from Etabs for the C shaped wall W2 at story 1 (above basement) is shown in the following screen capture:

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	12800	7475	12800	9275	1800	250
Top	Leg 2	10225	7475	10225	9275	1800	250
Top	Leg 3	10225	9275	12800	9275	2575	250
Bottom	Leg 1	12800	7475	12800	9275	1800	250
Bottom	Leg 2	10225	7475	10225	9275	1800	250
Bottom	Leg 3	10225	9275	12800	9275	2575	250

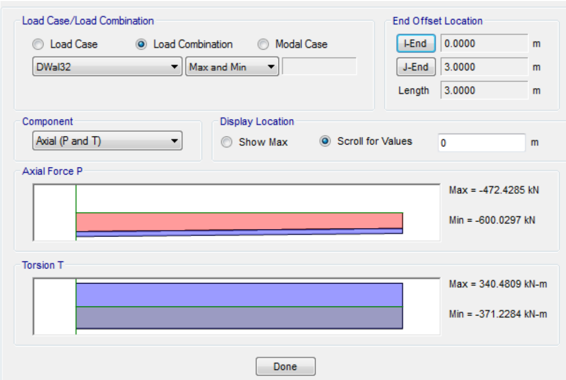
Flexural Design for P_u, M_{u2} and M_{u3}

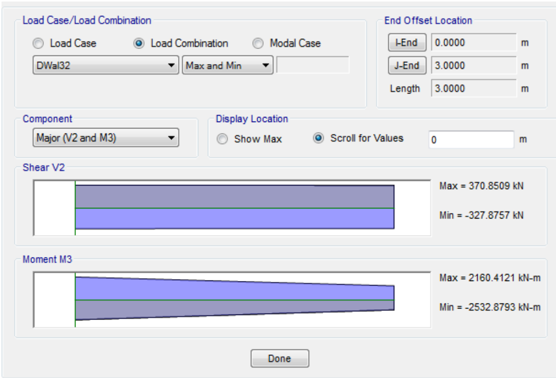
Station Location	Required Rebar Area (mm²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P_u kN	M_{u2} kN-m	M_{u3} kN-m	Pier A_g mm²
Top	3859	0.0025	0.0069	DWal32	519.5523	-717.8721	-1559.8943	1543750
Bottom	7815	0.0051	0.0069	DWal32	472.4285	1843.3362	-2532.8793	1543750

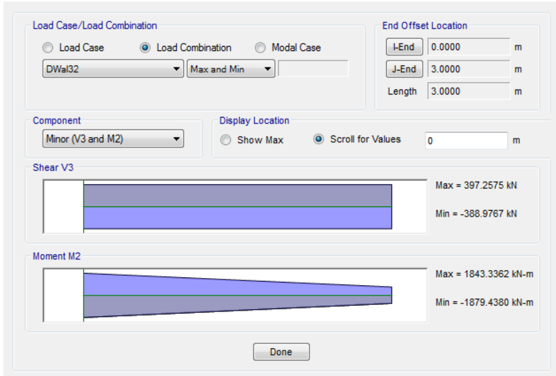
Shear Design

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	625	DWal26	-336.4461	106.7307	208.5755	228.7814	452.1716
Top	Leg 2	625	DWal26	52.0736	115.5843	201.2698	275.4038	498.7939
Top	Leg 3	625	DWal26	-159.2792	102.4188	338.6218	365.9277	685.4997
Bottom	Leg 1	625	DWal29	-563.8686	562.2376	201.2452	94.3901	317.7802
Bottom	Leg 2	625	DWal26	-288.6359	385.3445	176.2708	152.8475	376.2377
Bottom	Leg 3	625	DWal29	-532.5924	525.7283	306.3599	321.1301	640.7021

The forces used in the design, resulting from DWall32 combination, correspond to the forces from the analysis, which are shown for verification as screen captures below:







ECtools calculates for the bottom of the wall (base of wall) flexural reinforcement of As, req= 12.12+18.76+18.76+12.22 = 61.76 cm^2, while Etabs calculates As,req= 78.15cm^2, i.e a difference of 19%.

ECtools calculates for the bottom of the wall (base of wall) shear reinforcement per leg 2x3#/280 As/s=5.07 cm^2/m while Etabs calculates As/s = 6.25 cm^2/m, i.e. 18% difference per leg.

Example 2: Athens Opera House (SNFCC)

1. Introduction

The purpose of this report is to present the results of the design verification of the Opera House superstructure. The superstructure was modelled both in Etabs and Scia Engineer, by two teams working in parallel, so that human error or software bugs could be eliminated. This was decided due to the complexity and irregularity of the building.

As it can be easily seen from the numerical models, a large canopy on top of the Opera (100mx100m) has been accurately modelled both regarding geometry and loads, so that its effects are included in the opera static and dynamic response.

2. General Approach

An effort was made to reduce the number of factors that could produce discrepancies between the models. To that end:

- All loads, spectra, loading assumptions and load combinations were taken exactly the same

  Please refer to appendix “Codes, Loads and Materials” for a detailed analysis of the loads, the design combinations and the codes applied.

- Extra loads pertaining to the stage pit and the flytower were calculated from the relevant stage engineering technical descriptions.

- The comparison of foundation loads between was made using models without vertical springs (rigid foundation) since the addition of the deformability of the substructure would only increase the variability of the data.

Two separate numerical software were used to model the Opera House with the solar collector on top, ETABS v9.7.4 (CSI) and SCIA Engineer 2012 (Nemetschek). This double numerical modelling approach was deemed necessary given the complexity of the project, so that subsequent errors and discrepancies in the modelling of the geometry, in the application of loads etc. would be exposed and corrected.

The solar canopy was modelled both on top the main building.

3. Numerical Models

Two numerical models were created for the Opera House, one in SCIA and one in Etabs.

- In both software, the main structure was modelled with the solar collector on top.

- Columns were modelled using frame elements.

- T-beams were modelled using frame elements for the webs. These were assigned a vertical stiffness offset from the T section's flange, creating the actual beam stiffness. SCIA integrates the forces from the web and the flange automatically, producing the resulting T-section forces.

- Walls and spandrels were modelled using shell elements.

- Slabs were modelled using shell elements. Voided slabs were also modelled using shell elements with equivalent stiffnesses. Ribbed and waffle slabs were modelled using shell elements for the flanges and frames for the ribs. The rib frames were assigned a vertical stiffness offset in order to reproduce the actual slab section's stiffness.

- The solar collector's ribs were modelled via a stiffness modifier to the relevant flanges. The solar collector's beams were modelled using frame elements that were assigned a vertical stiffness offset.

- Surface loads were applied to slabs, line loads were applied to either existing beams or supplementary zero-weight and zero-stiffness linear elements connected to the slabs' mesh.

- The 172 isolators' horizontal stiffnesses were calculated using the following expression:

$$K_{eff}=\frac{\Delta F}{\Delta D}=\frac{F_{\max}-F_{\min}}{2\cdot D}=\frac{W}{R}+\frac{\mu\cdot W}{D}$$

where:

R = 2.7m,	isolator’s pendulum radius
D = 0.234m,	the design displacement for T=2.59s
μ = 0.054,	the friction coefficient (max value)
W	the vertical force for G+ψ_E·Q

The modal analysis of both models resulted in a period of T=2.59s - 2.60s for the three main eignemodes, as was expected.

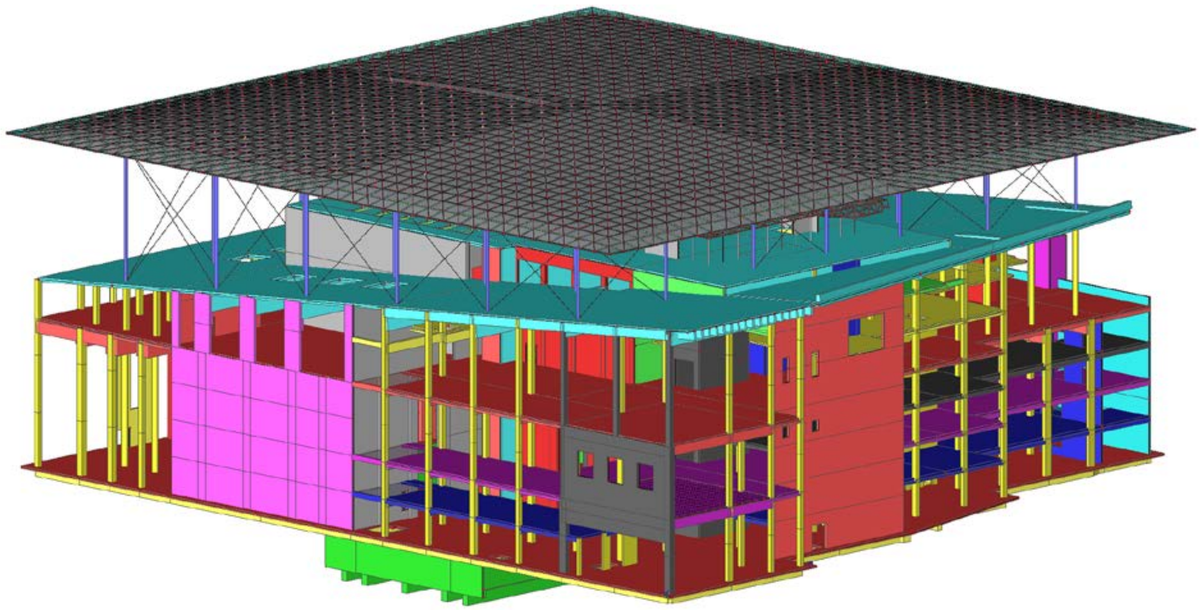
- The 172 isolators’ vertical stiffnesses were calculated from the undercroft numerical model iteratively:
    - vertical reactions of the fixed model were applied to the undercroft model
    - the resulting deflections at each isolator position were translated to vertical spring stiffnesses
    - these stiffnesses were assigned to the superstructure model and the analysis was repeated
    - the newly calculated reactions at the isolator positions were reapplied to the undercroft model and isolator deflections were recalculated
    - the process was repeated until the maximum change in stiffness between cycles stopped exceeding 5% for all isolators.

- The spring-damper column heads were modelled using link elements with a 10kN/mm axial stiffness. The connection of the column heads with the canopy was considered pinned.

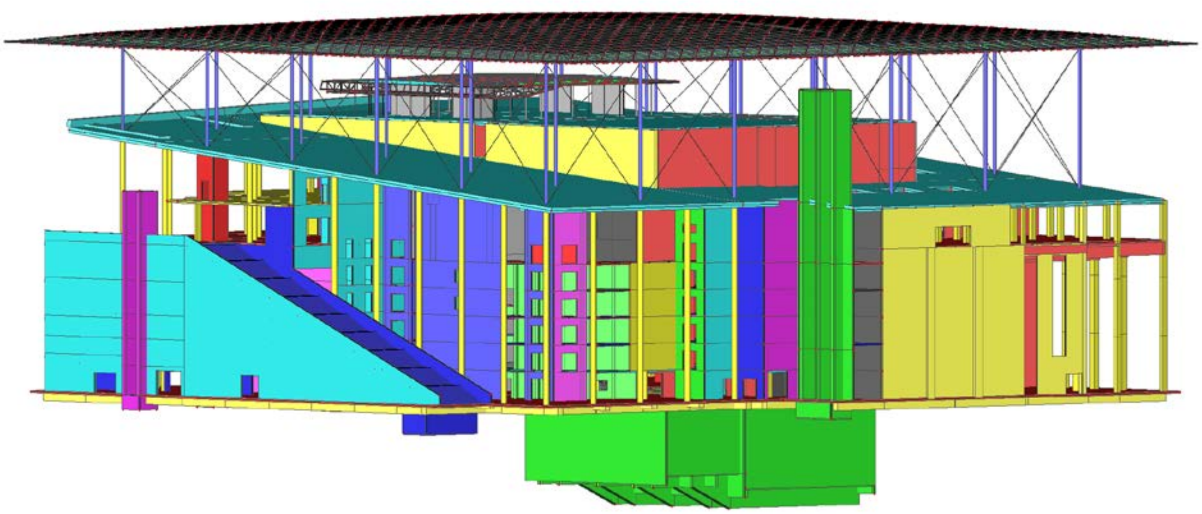
- The cables were modelled using single 45mm steel rods, with an axial stiffness modifier of 1.4, which represents the actual cross section of the pair of cables (same as in the ER analyses). The pretensioning force of 1MN was applied as a negative temperature change.

- The solver in SCIA, contrary to the one in ETABS, is multithreaded and allows for larger problems to be solved in a practical time frame. Thus, the SCIA model was modelled with a much finer mesh in order to avoid overestimation of the actual stiffness of plane elements. The

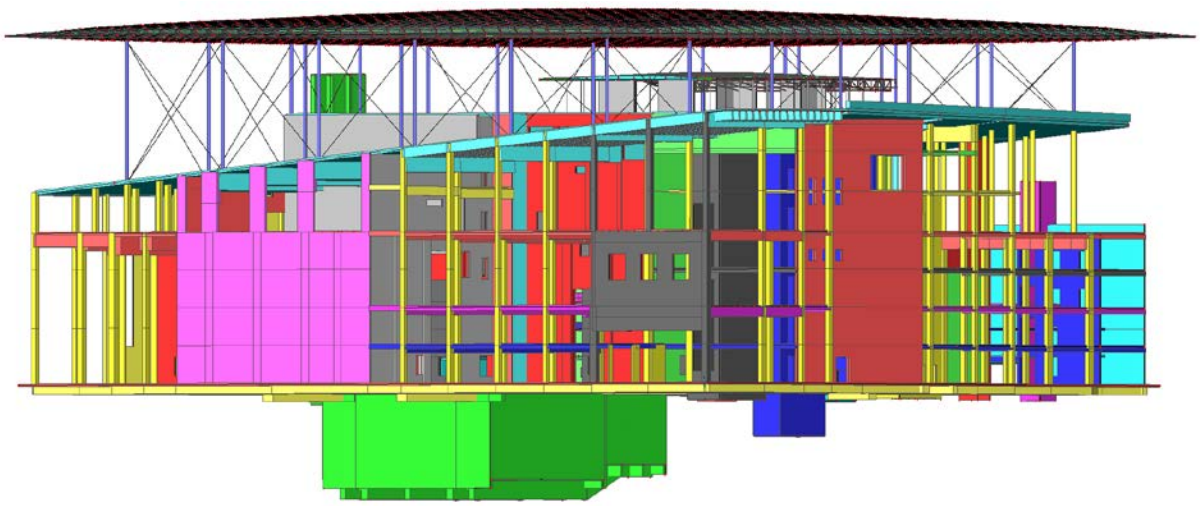
SCIA model has 75.000 shell elements, while the ETABS model has 28.000 shell elements. Even though this leads in general to more accurate results from the SCIA model, the two models are in good agreement due to a significant effort that was made to optimize the meshing of the walls in ETABS.



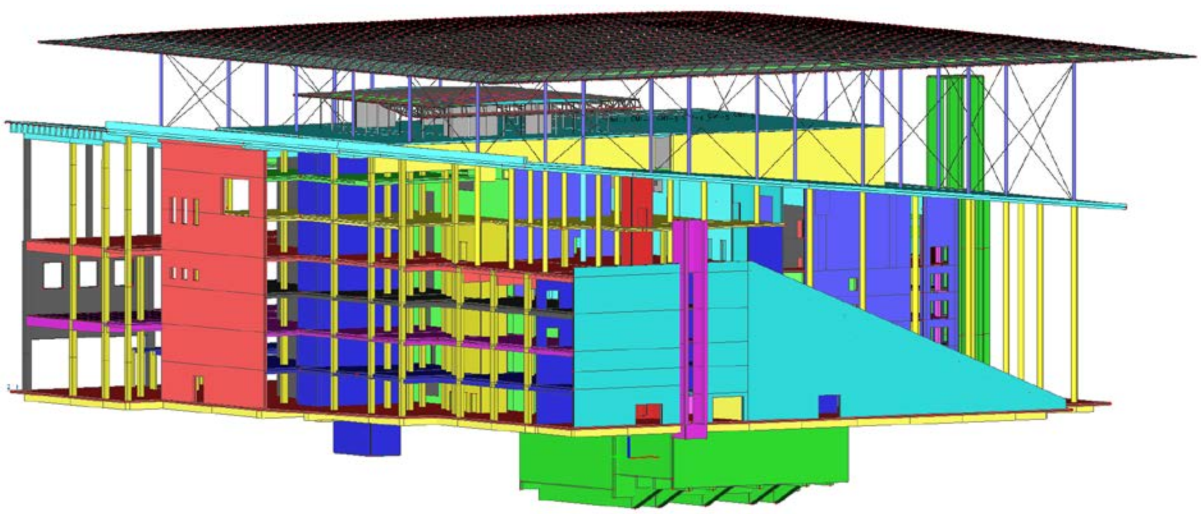
SCIA model, 1



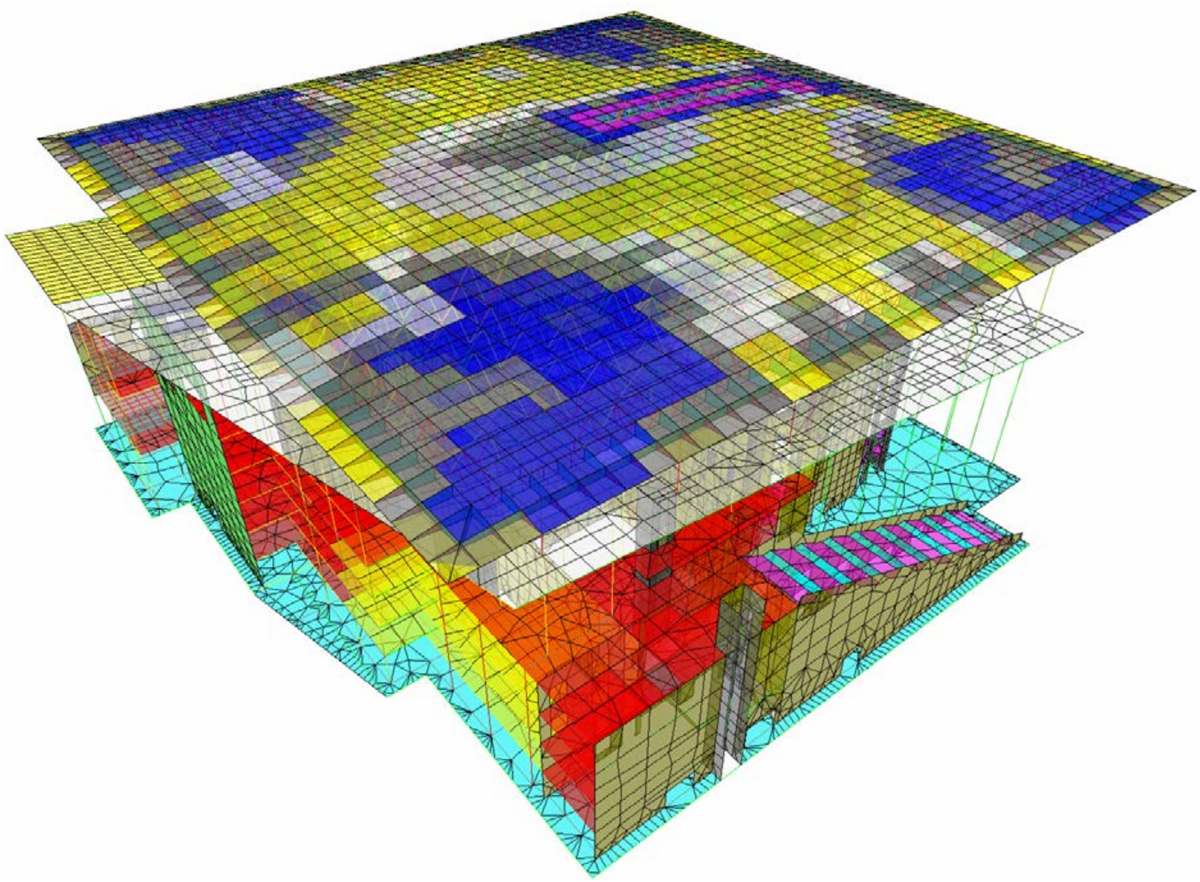
SCIA model, 2



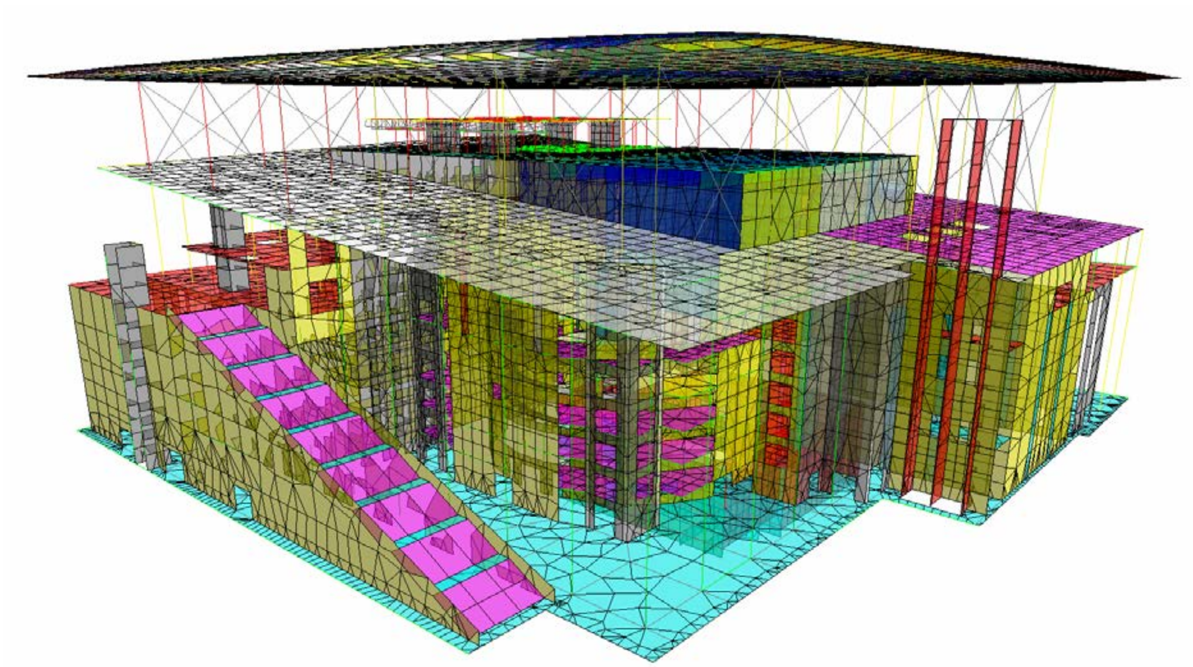
SCIA model, 3



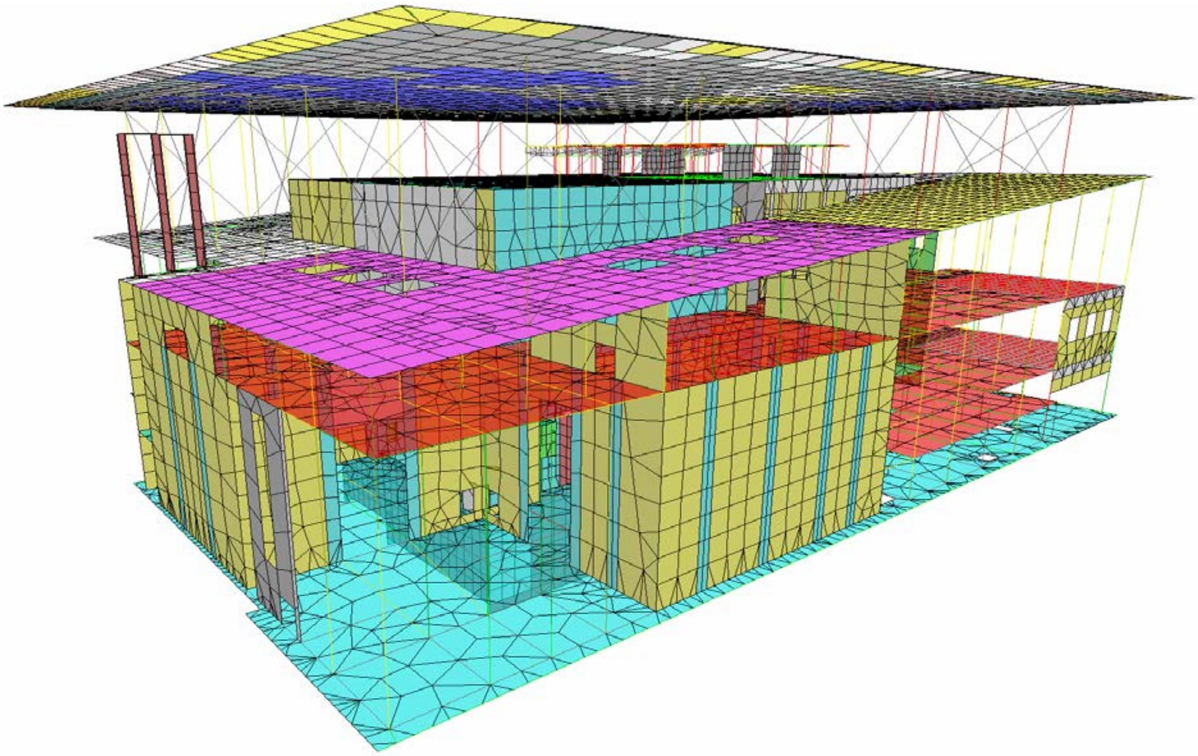
SCIA model, 4



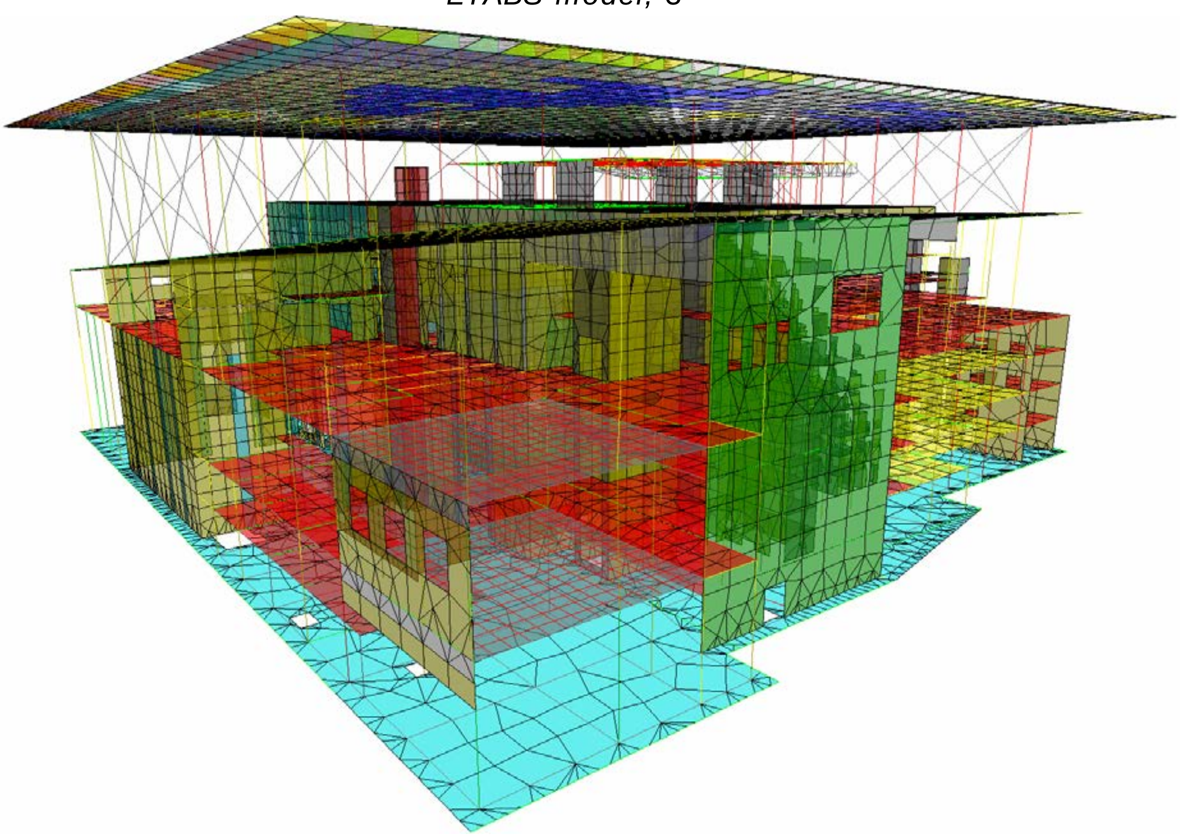
ETABS model, 1



ETABS model, 2



ETABS model, 3



ETABS model, 4

4. Global Model Verification – Gravity Loads

4.1. Summation of loads at base

The sum of forces for the combination **1.35G + 1.50Q** for the twomodels are presented in the following table:

ETABS	SCIA
1771693 kN	1772505 kN

The difference between models is less than 0.5 ‰ , rendering them equal in the total load application.

Since the total load has been calculated effectively the same, any individual differences that should arise will be the product of the load positioning and the modelling of the structure stiffnesses.

4.2. Comparison of reactions at individual isolator positions

The comparison of reactions for individual isolators was done between the JVIT models for three (3) cases:

1. One with fixed supports and with stiffnesses for walls and beams reduced by 50%
2. One with fixed supports and full stiffnesses
3. One with spring supports (calculated from undercroft ETABS model and SCIA superstructuremodel) and full stiffnesses

The results are presented in the following table:

Grid Position	ETABS ½K Fixed	SCIA ½K Fixed	Relative difference	ETABS FullK Fixed	SCIA FullK Fixed	Relative difference	ETABS FullK Springs	SCIA FullK Springs	Relative difference
CE/36	6467	6071	-6%	6541	6132	-6%	6913	6637	-4%
CH/36	6768	6473	-4%	6830	6578	-4%	7598	7464	-2%
DA-DB/36	6558	6631	1%	6418	6467	1%	5396	5236	-3%
DC-DD/36	4888	4923	1%	4918	4949	1%	4738	4709	-1%
DF/36	8409	8211	-2%	8655	8273	-4%	8236	8192	-1%
E/36	6492	6384	-2%	6513	6370	-2%	5471	5413	-1%
EC/36	5764	5829	1%	5748	5762	0%	4967	4977	0%
EF/36	4970	5074	2%	4905	4956	1%	4570	4600	1%

Grid Position	ETABS ½K Fixed	SCIA ½K Fixed	Relative difference	ETABS FullK Fixed	SCIA FullK Fixed	Relative difference	ETABS FullK Springs	SCIA FullK Springs	Relative difference
F/36	3982	4172	5%	3906	3995	2%	4108	4151	1%
FC/36	2512	2618	4%	2476	2476	0%	3283	3365	2%
FD-FE/36	1873	2061	10%	1839	1887	3%	2154	2186	1%
CE/40-41	9232	8734	-5%	10261	9543	-7%	8916	8686	-3%
CH-D/40-41	8103	8777	8%	9173	9862	8%	10494	10668	2%
DC-DD/40	9901	9838	-1%	10630	10261	-3%	12240	12150	-1%
DF/40	13556	14626	8%	13525	14823	10%	13512	13676	1%
E/40	10776	11191	4%	10567	10951	4%	13014	13207	1%
CE/41	7606	7905	4%	6703	7023	5%	5947	5986	1%
D/41	6166	6035	-2%	4718	4704	0%	5817	5855	1%
EC/41	6608	5447	-18%	6288	5283	-16%	4699	4470	-5%
EF/41	7750	7300	-6%	7814	7548	-3%	8014	7813	-3%
F/41	7119	6831	-4%	7131	7041	-1%	7471	7368	-1%
FC/41	7812	7569	-3%	7910	7825	-1%	7544	7558	0%
FF/41	5553	5492	-1%	5308	5331	0%	5115	5129	0%
G/41	6366	6705	5%	6427	6753	5%	5977	6261	5%
DC/43	11173	11466	3%	12446	12589	1%	11951	11767	-2%
E/43	17123	18025	5%	17699	18411	4%	17673	17668	0%
EC/43	858	1069	25%	866	1012	17%	2523	2343	-7%
DA/43-44	6409	7632	19%	6798	7729	14%	6928	7013	1%
CE/44	11155	11201	0%	11184	11351	1%	10826	10947	1%
D/44	9920	9169	-8%	8992	8379	-7%	9486	9244	-3%
DC/44	8908	7675	-14%	7600	6754	-11%	7458	7173	-4%
DF/44	11167	9724	-13%	10010	8935	-11%	9249	8802	-5%
FF/44	1257	1435	14%	1306	1423	9%	2125	2129	0%
G/44	5680	5736	1%	5616	5663	1%	5628	5661	1%
EE-EF/44-45	11569	11275	-3%	12102	11745	-3%	11498	11302	-2%
FA-FB/44-45	11010	11405	4%	11219	11789	5%	11095	11285	2%
EB/45	5363	5371	0%	5408	5353	-1%	4842	4759	-2%
ED/45	3700	3674	-1%	3877	3797	-2%	3787	3776	0%
FC/45	5100	4831	-5%	5319	4962	-7%	4967	4843	-3%
FE/45	7024	7108	1%	7208	7307	1%	7848	7980	2%
EG-EE/46	4243	4297	1%	4060	4021	-1%	4874	4986	2%
EH-F/46	4284	4000	-7%	4015	3737	-7%	4437	4455	0%
EE/47	5425	5893	9%	5667	5925	5%	6203	6408	3%
FA/47	5544	5702	3%	5711	5788	1%	5362	5470	2%

Grid Position	ETABS ½K Fixed	SCIA ½K Fixed	Relative difference	ETABS FullK Fixed	SCIA FullK Fixed	Relative difference	ETABS FullK Springs	SCIA FullK Springs	Relative difference
CE/47	9875	9841	0%	10200	10069	-1%	10335	10174	-2%
D/47	10114	8504	-16%	8775	7249	-17%	9296	8663	-7%
DA/47	12861	14263	11%	14082	15410	9%	14780	14977	1%
DE/47	2200	2266	3%	2116	2129	1%	2863	2741	-4%
E/47	20464	19768	-3%	21388	20739	-3%	17957	17822	-1%
EB/47	8274	7435	-10%	6690	6028	-10%	4686	4564	-3%
FE/47	6376	5990	-6%	6484	6294	-3%	6301	6298	0%
G/47	7006	7160	2%	6929	7064	2%	6944	7148	3%
FA-FB/47-48	1299	1370	5%	1197	1281	7%	1411	1470	4%
EE-EF/47-48	1322	1374	4%	1198	1270	6%	1515	1568	4%
EC/48-50	8164	8167	0%	7999	8004	0%	6810	6774	-1%
FD/48-50	7914	8376	6%	7637	8006	5%	7942	8265	4%
CC/50	11309	11007	-3%	11230	10915	-3%	10623	10407	-2%
CF/50	8141	8260	1%	6898	6980	1%	7568	7418	-2%
CG/50	7438	7129	-4%	8344	8102	-3%	9430	9344	-1%
DE/50-51	2872	2840	-1%	2915	2932	1%	3305	3232	-2%
DA/51	17024	17315	2%	18021	18153	1%	15873	15794	0%
E/51	18391	18080	-2%	19101	18705	-2%	18492	18355	-1%
FF/51	5238	4828	-8%	4407	4048	-8%	3998	4009	0%
G/51	5249	5742	9%	5117	5586	9%	4775	5106	7%
EB/51-52	9571	10445	9%	8554	9187	7%	7682	7665	0%
FE/51-52	8286	9624	16%	7797	8832	13%	7201	7348	2%
EE-EF/51-52	1300	1287	-1%	1360	1361	0%	1690	1611	-5%
FA-FB/51-52	1340	1317	-2%	1389	1367	-2%	1674	1616	-3%
CC/53	12122	11678	-4%	12014	11714	-2%	12139	11878	-2%
CG/53	7961	7633	-4%	8938	8646	-3%	6540	6328	-3%
EE-EF/53	1854	1597	-14%	1741	1535	-12%	2329	2113	-9%
FA-FB/53	1797	1575	-12%	1663	1506	-9%	2203	2019	-8%
FF/53	4374	3660	-16%	4692	3936	-16%	5743	5853	2%
FH/53	5137	5445	6%	5597	5661	1%	4460	4479	0%
EB/53-54	10459	10422	0%	10606	10528	-1%	15623	15862	2%
FE/53-54	8373	9177	10%	9116	9697	6%	12709	13094	3%
EC/54	9985	8848	-11%	9751	8792	-10%	12703	12321	-3%
FD/54	10552	9451	-10%	10146	9512	-6%	13080	12720	-3%
CF/54	8264	9033	9%	7473	8095	8%	7526	7636	1%
CH/54	6248	6324	1%	4760	4788	1%	4569	4581	0%

Grid Position	ETABS ½K Fixed	SCIA ½K Fixed	Relative difference	ETABS FullK Fixed	SCIA FullK Fixed	Relative difference	ETABS FullK Springs	SCIA FullK Springs	Relative difference
DA/54	17581	16566	-6%	18828	17623	-6%	14907	14459	-3%
G/54	4075	4872	20%	3583	4553	27%	3488	3901	12%
DE/54	2784	2574	-8%	2755	2601	-6%	3218	2992	-7%
E/55	19424	18475	-5%	19754	18814	-5%	18630	18352	-1%
FH/55	3255	3674	13%	3494	4004	15%	3297	3424	4%
G-GA/55	5479	6113	12%	5609	6092	9%	4157	4166	0%
CC/56	12599	12465	-1%	12098	11913	-2%	12267	11976	-2%
CF/56	8554	7753	-9%	7879	7105	-10%	8219	7711	-6%
CH/56	7190	6897	-4%	6347	6141	-3%	6124	5957	-3%
EB/56	11362	11369	0%	11369	11342	0%	14156	14592	3%
FE/56	9283	10288	11%	9282	10352	12%	11399	12016	5%
DA/56-57	14978	15238	2%	15602	15598	0%	14405	14123	-2%
DC-DD/57	9644	9057	-6%	8450	7690	-9%	8231	7749	-6%
DF-DG/57	10214	7453	-27%	8988	8912	-1%	8310	8314	0%
E/56-57	9315	10235	10%	9436	10066	7%	9007	9151	2%
FG/57	2086	2132	2%	2094	2142	2%	2457	2458	0%
FH/57	4544	4050	-11%	3419	3116	-9%	3140	3061	-3%
GA/57	8348	7253	-13%	8613	7561	-12%	7718	7647	-1%
GE/57	4720	4721	0%	4828	4756	-1%	3996	4040	1%
CA/60	12843	12967	1%	13046	12997	0%	13119	12894	-2%
CD/60	21773	20230	-7%	20503	18541	-10%	22052	20699	-6%
CF-CG/60	25120	24329	-3%	27238	26603	-2%	22968	21757	-5%
DA/60	17967	17850	-1%	19029	18893	-1%	20523	20115	-2%
DD/60	15668	16370	4%	16228	16523	2%	20763	20840	0%
DF/60	14082	14087	0%	14407	13957	-3%	18895	18947	0%
E/60	23079	23399	1%	23190	23441	1%	24139	24609	2%
EB/60	68581	75018	9%	70094	76127	9%	55121	56574	3%
FE/60	61883	63452	3%	63390	64485	2%	50574	50809	0%
FG/60	20528	14157	-31%	20458	14414	-30%	19709	18167	-8%
G-GA/60	16030	16890	5%	15676	16976	8%	18707	18838	1%
GE/60	10330	11427	11%	10282	11161	9%	11112	11564	4%
CD/63	9609	9060	-6%	7764	7315	-6%	8921	8280	-7%
CF-CG/63	11273	8295	-26%	12008	8999	-25%	9897	8922	-10%
CA/64-65	26233	26475	1%	27411	27784	1%	30854	30748	0%
DA/64-65	22756	21813	-4%	23212	22441	-3%	25938	25226	-3%
EA/64-65	2837	2562	-10%	2860	2560	-10%	3506	3233	-8%

Grid Position	ETABS ½K Fixed	SCIA ½K Fixed	Relative difference	ETABS FullK Fixed	SCIA FullK Fixed	Relative difference	ETABS FullK Springs	SCIA FullK Springs	Relative difference
FF/64-65	2592	2368	-9%	2543	2300	-10%	3265	2983	-9%
G-GA/64-65	3452	3399	-2%	3349	3313	-1%	3166	3147	-1%
DF-DG/64-65	3812	3769	-1%	3808	3792	0%	3665	3637	-1%
CG/64-65	17020	17304	2%	18191	18657	3%	14050	13637	-3%
CD/65	7194	5663	-21%	6492	5113	-21%	7239	6219	-14%
CD/68	9115	9196	1%	8490	8569	1%	8627	8546	-1%
CG/68-70	8864	8371	-6%	9705	9191	-5%	9471	9184	-3%
EB/68-70	27505	27611	0%	27529	27802	1%	31515	32303	3%
FE/68-70	24596	28551	16%	24522	28773	17%	26583	28859	9%
CA/70	17726	17237	-3%	18595	17976	-3%	15644	15133	-3%
BH/70	6341	5585	-12%	5824	5431	-7%	5724	5477	-4%
DA/70	18213	17205	-6%	18370	17063	-7%	20211	19498	-4%
DD/70	17338	17619	2%	17836	17858	0%	20169	20087	0%
DF/70	22152	21956	-1%	22870	22503	-2%	25025	24944	0%
ED/70	25890	27599	7%	26200	27957	7%	29461	30812	5%
FC/70	25292	25129	-1%	25538	25382	-1%	27702	28369	2%
FH/70	15129	15802	4%	15839	16365	3%	17356	18419	6%
GA-GB/70	14208	14364	1%	14804	14824	0%	16700	17075	2%
GE/70	15966	16209	2%	16151	16514	2%	14666	14928	2%
EH/71	2329	2180	-6%	2409	2233	-7%	2918	2730	-6%
BH/72	6971	7725	11%	6871	6694	-3%	5948	5851	-2%
CA/72	8973	9440	5%	8686	8819	2%	9102	9164	1%
CF/72	13968	14756	6%	13208	13861	5%	12948	13352	3%
EB/73	16884	16686	-1%	16982	16459	-3%	16820	16852	0%
FE/73	13184	14070	7%	13094	13863	6%	12283	12804	4%
GE/74	6117	6486	6%	6151	6421	4%	5893	6009	2%
ED/74-75	2328	2119	-9%	2140	1946	-9%	2184	2017	-8%
EH/74-75	3289	3141	-5%	3299	3182	-4%	3392	3216	-5%
FC/74-75	2253	2036	-10%	2118	1888	-11%	2436	2267	-7%
BH/75	13902	12379	-11%	13913	14077	1%	13807	13956	1%
CB-CC/75	2475	2418	-2%	2481	2435	-2%	3077	2987	-3%
CF/75	18622	19270	3%	18160	18637	3%	17959	18463	3%
D/75	10357	10307	0%	10082	10064	0%	9919	10123	2%
DC/75	14785	17404	18%	14124	16779	19%	14075	16016	14%
DF/75	9251	9132	-1%	9186	9217	0%	9037	9245	2%
E/75	10498	9384	-11%	9511	9434	-1%	9457	9346	-1%

Grid Position	ETABS ½K Fixed	SCIA ½K Fixed	Relative difference	ETABS FullK Fixed	SCIA FullK Fixed	Relative difference	ETABS FullK Springs	SCIA FullK Springs	Relative difference
EB/75	12092	12312	2%	12281	12257	0%	13965	13947	0%
FE/75	9907	10602	7%	9976	10612	6%	11429	11962	5%
FG/76	6171	6180	0%	5498	5402	-2%	5967	5988	0%
G/76	7405	7405	0%	6549	6735	3%	6997	7126	2%
GB/76	8216	8163	-1%	7361	7192	-2%	7476	7492	0%
GE/76	4441	4504	1%	4307	4394	2%	4277	4359	2%
BH/80	14786	15809	7%	15020	15446	3%	15264	15773	3%
CB-CC/80	1180	1150	-3%	1155	1113	-4%	1770	1713	-3%
CF/80	17288	18263	6%	17448	18484	6%	16225	17115	5%
D/80	10328	9895	-4%	10411	10153	-2%	10703	10500	-2%
DC/80	12246	11626	-5%	12409	11434	-8%	12427	11717	-6%
DF/80	10396	10142	-2%	10468	10276	-2%	10328	10102	-2%
E/80	8667	8193	-5%	8491	8026	-5%	8759	8324	-5%
EB/80	16051	15678	-2%	15877	15284	-4%	14354	14148	-1%
EH/80	23122	23815	3%	23466	24237	3%	18314	18542	1%
FE/80	15616	16461	5%	15308	15943	4%	12562	12846	2%
FG/80	3463	3696	7%	3527	3611	2%	3523	3628	3%
G/80	5004	4979	-1%	5425	5644	4%	5018	5123	2%
GB/80	5721	5767	1%	6098	6077	0%	5430	5508	1%
GE/80	4492	4805	7%	4505	4804	7%	4573	4802	5%
		Average	**0%**		**Average**	**-1%**		**Average**	**-1%**
		Stand. Dev.	**8%**		**Stand. Dev.**	**7%**		**Stand. Dev.**	**4%**
		Variance	**1.16**		**Variance**	**0.92**		**Variance**	**0.25**

The following observations are made from the above comparisons:

1. Even though the sum total for gravity loads is exactly the same for the two models, their distribution in the structure displays some variance.

2. The variance is 4x greater for the models supported on fixities than the variance observed for the models supported on springs.

The root cause for this behavior is the coarser mesh of the ETABS model compared to the SCIA, which results in an erroneously “stiffer” model. Combined with rigid supports, the error is compounded. Combined with elastic supports which are significantly less stiff than the elements they support, the error is mitigated.

8. Global Modelling Verification – Dynamic Analysis

The dynamic behavior of the superstructure is governed by the presence of the base isolators, their horizontal stiffnesses and their fundamental period. Their horizontal stiffness is directly proportional to the vertical force applied according to equation

$$K_{eff} = \frac{\Delta F}{\Delta D} = \frac{F_{\max} - F_{\min}}{2 \cdot D} = \frac{W}{R} + \frac{\mu \cdot W}{D}$$

In turn, the vertical force for each isolator is equal to the overlying mass times 9.81m/sec². Consequently:

- the center of stiffness of the group of isolators coincides with the center of mass of the structure
- the center of the polar mass moment of inertia of the superstructurearound the vertical axis coincides with the center of torsional stiffness of the group of isolators
- the ratios m/K_{isol} and J_m/J_{isol} are equal

The net effect is that the fundamental period for each degree of freedom (2 translational, 1 rotational, 3 total) is the same and equal to T = 2.59s. Furthermore the structure should exhibit no rotation under horizontal excitation along <u>any</u> direction.

These 3 eigenmodes were produced by both ETABS and SCIA JVIT models with periods between 2.56s and 2.59s. Combined they include 99.9% of the structure's mass for each degree of freedom.

The table below presents the results from ETABS.

Mode	Period	UX	UY	UZ	SumUX	SumUY	SumUZ	RZ	SumRZ
1	2.595	68.083	0.081	0.0	68.1	0.1	0.0	31.8	31.8
2	2.586	0.740	98.623	0.0	68.8	98.7	0.0	0.6	32.4
3	2.567	31.122	1.233	0.0	99.9	99.9	0.0	67.6	100.0
4	0.938	0.000	0.000	0.3	99.9	99.9	0.3	0.0	100.0
5	0.883	0.009	0.000	0.0	100.0	99.9	0.3	0.0	100.0
6	0.817	0.001	0.051	0.0	100.0	100.0	0.3	0.0	100.0
7	0.795	0.000	0.000	3.7	100.0	100.0	4.0	0.0	100.0
8	0.752	0.035	0.001	0.0	100.0	100.0	4.0	0.0	100.0
9	0.494	0.002	0.001	0.3	100.0	100.0	4.3	0.0	100.0
10	0.457	0.001	0.000	0.9	100.0	100.0	5.2	0.0	100.0
11	0.270	0.002	0.007	0.2	100.0	100.0	5.4	0.0	100.0
12	0.190	0.000	0.000	86.2	100.0	100.0	91.6	0.0	100.0

The table below presents the results from S.EN.

Mode	SCIA Model
#	T [sec]
1	2.636
2	2.611
3	2.571
4	1.008
5	0.883
6	0.873
7	0.843
8	0.794
9	0.660
10	0.629
11	0.615
12	0.594

The project design spectra were assigned on the two orthogonal directions X and Y. The result was a translational response along the direction of each excitation (X & Y) with virtually no rotation.

<u>Therefore the assignment of the horizontal springs was done correctly in both JVIT models.</u>

The calculated response spectrum displacements are:

Excitation Direction	AGORA		ROOF	
	UX	UY	UX	UY
X-X	154 mm	3 mm	158 mm	5 mm
Y-Y	0 mm	154 mm	0 mm	158 mm

Multiplied by q=1.50, they produce the elastic displacement, used for the base isolator design.

$$D = 154 \cdot 1.50 = 231\text{mm}$$

This value is in agreement with both calculations concerning the base isolation design. Therefore the numerical dynamic analysis is correct.

Conclusions

Two different approaches have been applied for the verification of Scia Engineer & ECtools using CSI Etabs as the reference software, for ACIA 318-11 reinforced concrete design.

- Approach of example 1, examines in depth all the modelling approaches and design options and results for a 3D R/C dual system with 3 storeys and one basement
- Approach of Athens Opera House, compares the two software on the application on one of the most demanding structural models and assessed global behavior analysis results.

From the detailed analysis examination of example 1, the following conclusions have been derived:

- Global force balance is identical for both software
- Global assembled masses are identical for both software
- The dynamic characteristics of the two models are identical with a deviation of less than 4%
- The modelling of beams in S.EN. (rib and integration flange approach) is more accurate than Etabs, as the latter ignores the moment and shear forces of the slab shell elements clashing with T-beam flanges. This difference is not considered significant in the design of a building.
- The modeling of columns in both software is a close match.
- The modelling of complex walls in S.EN. and Etabs are closer than 10%, when Etabs has a manual meshing of the finite elements of walls and slabs (in the automesh option, Etabs pier forces are not accurate)

From the design of R/C elements using S.EN. & ECtools or Etabs the following conclusions are derived:

- Beams design in Etabs does not take into account the minimum reinforcement requirements for T beams and uses as default the $4/3A_{cal}$ rule allowed by the ACI 318-11. S.EN. & ECtools uses the actual minima as defined in the main text of ACI318-11 and has the $4/3A_{cal}$ as a user option, as it is aimed only for large R/C beams (ACI commentary). The general design of beam, in both software, produces close match.

- Column design in both software produces identical results in flexure and shear, both regarding ordinary and special ductility class. Also the minima in both software are the same.
- The joint capacity rule, although applied using a different path in the two software, produces the same results and safety factor.
- Wall design for the ordinary case is comparable in both cases, both in flexure and shear

From the second example, the Athens Opera House, it is concluded that S.EN. can be used in very complex buildings and produce results directly comparable to CSI Etabs.

The general conclusion, derived from the development of this very elaborate report, is that an educated structural engineer, who is knowledgeable about any of the two software, may trust these without hesitation. It should however be noted, that both software are extremely advanced providing many user options, which are not to be used by newcomers or occasional users.