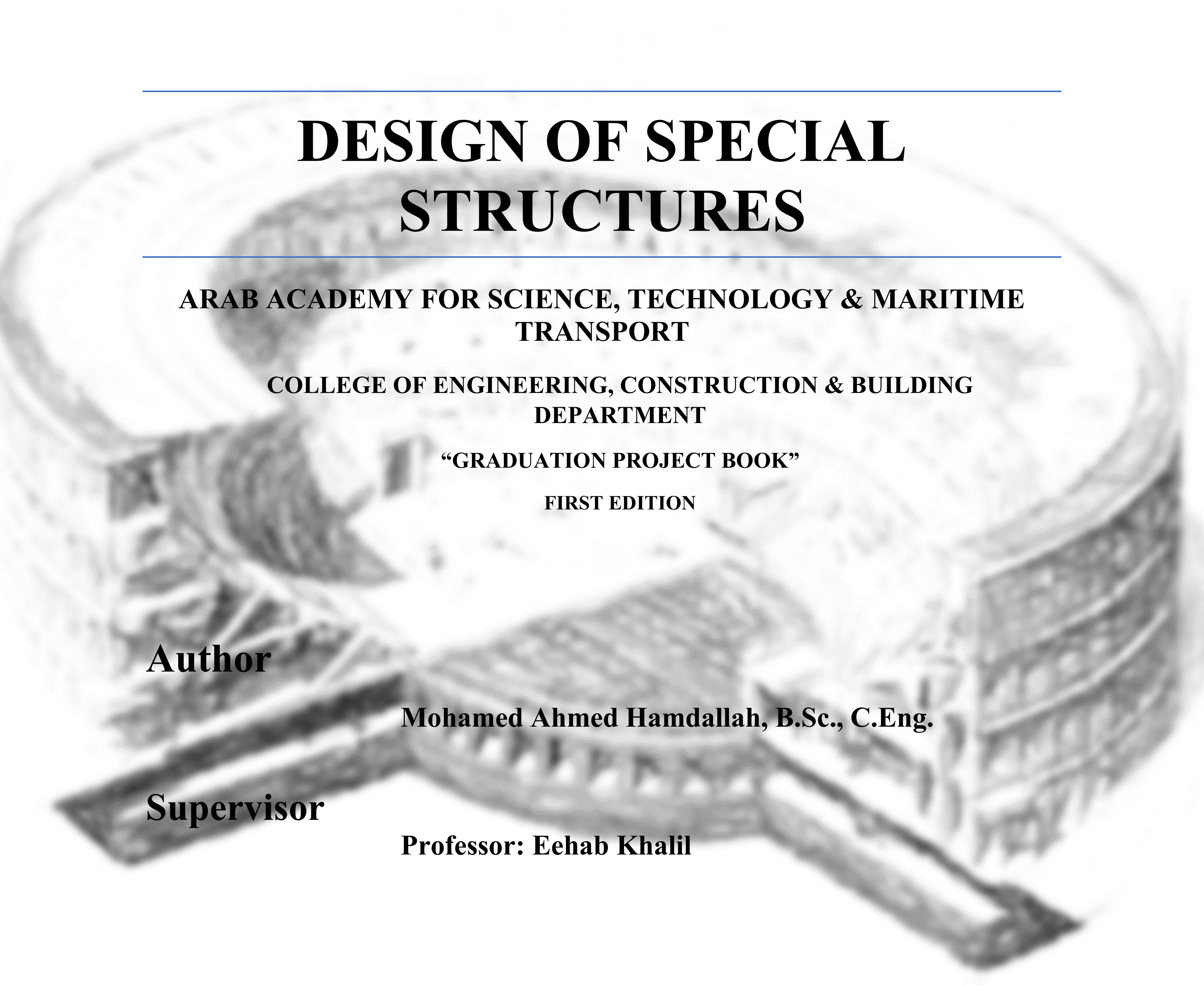

DESIGN OF SPECIAL STRUCTURES

ARAB ACADEMY FOR SCIENCE, TECHNOLOGY & MARITIME TRANSPORT

COLLEGE OF ENGINEERING, CONSTRUCTION & BUILDING DEPARTMENT

“GRADUATION PROJECT BOOK”

FIRST EDITION

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Supervisor

Professor: Eehab Khalil

JULY 24, 2020

AASTMT

South valley branch

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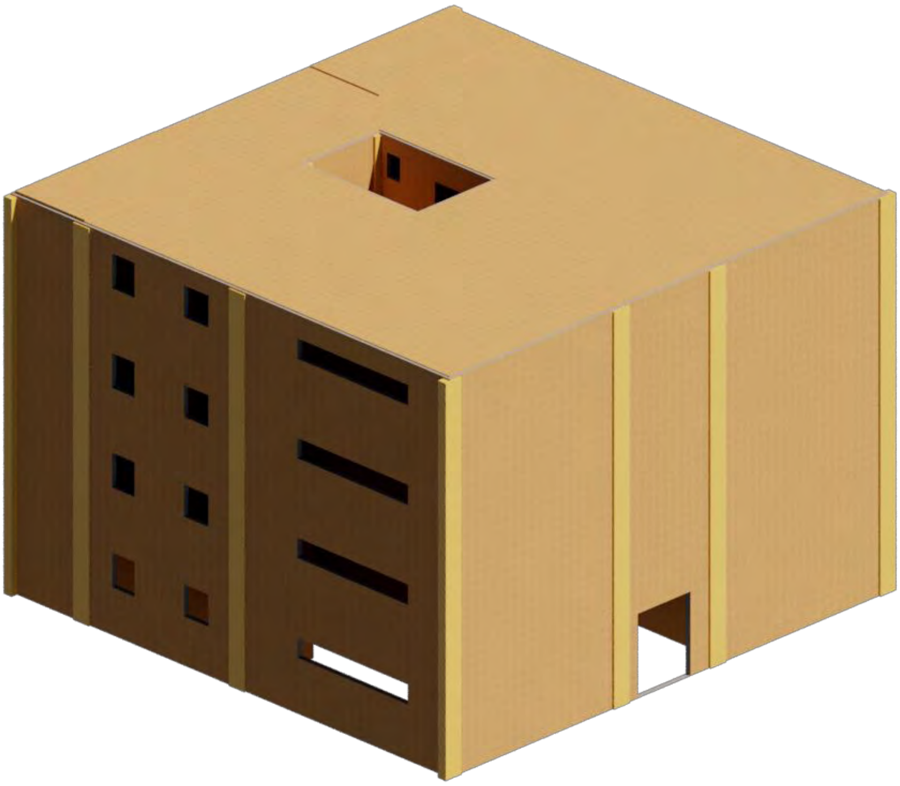
TECHNICAL DESIGN CALCULATION REPORT

DESIGN AND BEHAVIOR OF A MID-RISE CROSS LAMINATED TIMBER BUILDING

Part 1



Abstract

Cross laminated timber (CLT) is a new engineered wood material with a wide range of application as structural member in residential, commercial and educational buildings. CLT is developed in Austria and its production and application is increasing in Europe and around the world. One of the utilization fields of CLT is midrise residential and commercial buildings including single and multi-family residential buildings, educational institutions, and office buildings. Using Engineered wood products in construction field can contribute to solve climate change and global warming problems, reduce fresh water consumed in concrete buildings and build green society. This project proposes a modeling of CLT (Cross Laminated Timber) multi-stories building on DLUBAL RFEM software. Wall carrier structural system is proposed to resist gravity load by wall bearing and floor bending. Lateral loads are resisted by connector brackets with wood screws. The model contains LVL (Laminate Veneer Lumber) Paneled beams to enhance the performance of CLT floor in 7x7m hall. Due to the different deflection profiles. Frames experience “Racking” deflections , where the greatest inter-story drift is at the base of the structure, while walls experience a “Bending Deflection” deformation, with the greatest inter-story drift at the top of the structure. The combination of these two deformed shapes will compensate for each other’s shape, reducing lateral deflection along the whole height thus the GLULAM (Glued Laminated Timber) columns have been modeled.

I. Design codes and standards

1. **ANSI/AWC NDS-2018**

   CROSS-LAMINATED TIMBER – Chapter 4,10,11,12,13,15,16 and appendix

2. **ANSI/APA PRG 320-18**

   Manufacturing Standard

3. **IBC 2018**

   2018 Code Confirming Wood Design

4. **CLT HANDBOOK**

   US Edition

5. **TIMBER DESIGN AND CONSTRUCTION SOURCEBOOK**

   Large Halls and Roof Structures – Beam Grid

6. **ASCE/SEI 7-05**

   Load Combination

7. **ECP (201-2012)**

   Egyptian Code for Loading on Buildings

8. **ECP (204-2005)**

   Egyptian Code for Loading on Foundation

1. INTRODUCTION

The concept for structural design focuses on satisfying both the functional and the economic requirements of the building without jeopardizing its aesthetic and architectural features.

This Calculation Report presents the structural engineering aspect of the works due for the development construction work of GE BUILDING.

In this Report, a modeling of Cross Laminated Timber (CLT) multi-stories building on DLUBAL RFEM software is proposed. Egypt has 5510 feddan established Afforestation Areas Irrigated by Treated Sewage Water ;thus, using CLT panels in construct green cities in Egypt will be a solution of many problems like global warming, water consumed in concrete building and high building cost.

(The Role of Ministry of State for Environmental Affairs in implementing the National Program for safe use of treated sewage water for afforestation)

2. PROJECT FEATURES

the urbanization at 2018 equal 55.3 % and will be 60.4 % at 2030, 3 billion people (40% of the world) will need a new home at 2030. This translates into a demand for 96,000 new affordable and accessible housing units every day. One of three people today are actually live in slum that mean one billion people in the world live in slum. A hundred million people in the world are homeless. The scale of engineers challenge for society is to find a solution to house people but the challenge as we move to cities, cities were built in two materials that are steel and concrete. **(UN HABITAT, World Urbanization Prospects: The 2018 Revision)**

1. Steel and concrete are great materials but it consume very high energy in manufacturing process and emit green house gases. The embodied carbon emissions of building products and construction represent a significant portion global emissions: concrete, iron, and steel alone produce 9% of annual global GHG emissions; embodied carbon emissions from the building sector produce 11% of annual global GHG emissions. Every year, 6.13 billion square meters of buildings are constructed. The embodied carbon emissions of that construction is approximately 3729 million metric tons CO_2 per year **(ARCHETECTURE2030.ORG)**

- Wood is the only material that we can build with and grow with the power of sun.
- When the tree grows in the forests, it give us oxygen and store carbon dioxide, one cubic meter of wood can store one tone of carbon dioxide.
- Dead forests give carbon dioxide back to atmosphere into the ground and when the forests burn, it give carbon dioxide back to atmosphere.
- One cubic meter of concrete consumes almost 175 liter of fresh water while forests can grow with primary treatment of sanitary water.
- Wooden building is a fast building erection.
- light weight structure that will give us minimum foundation cost.
- minimize number of crews that mean minimize construction cost and conflicts.

the following link include energy consumption comparison between traditional structural system and wooden structural systems in Aswan province, Egypt. https://drive.google.com/file/d/1-Jb00F1nYV4FKd0ASX55xLXrbawRH2yt/view?usp=sharing

3. General description of the building

Location

Country: Egypt

City: Aswan

Description

Number of storeys: 4

Building length: 12 m

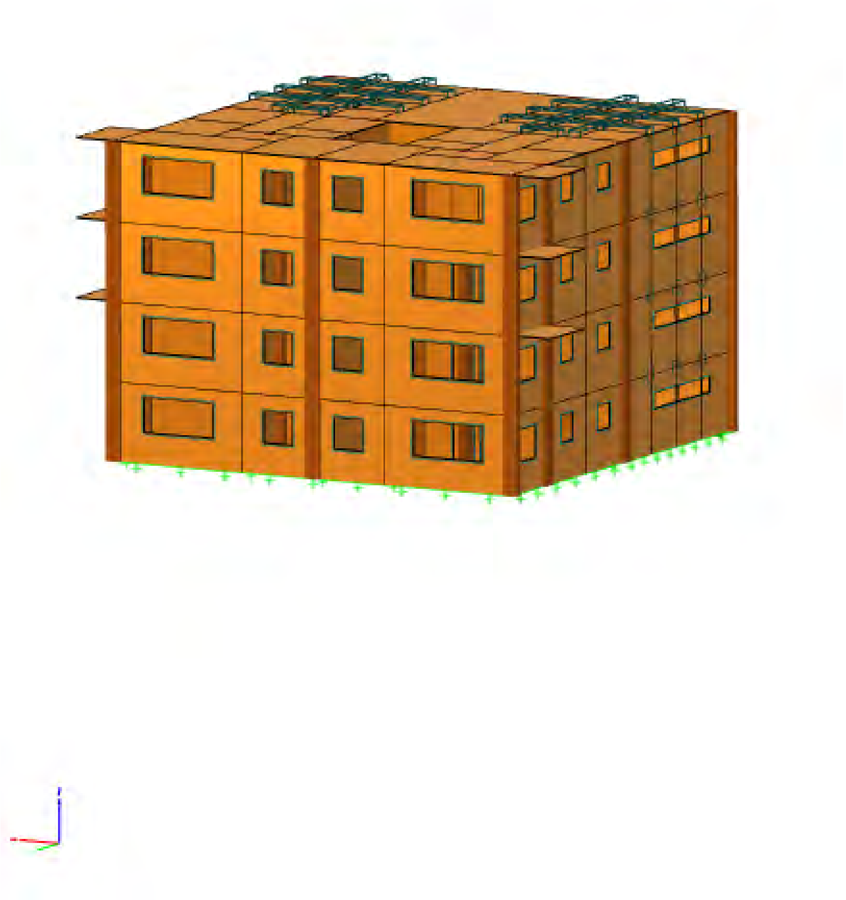
Building width: 16.7 m

Building height: 16.7 m

Wall carrier structural system is proposed to resist gravity load by wall bearing and floor bending. Lateral loads are resisted by connector brackets with wood screws.

Three-dimensional view South West

Three-dimensional view North West



Three-dimensional view South West

Three-dimensional view South East

4. Calculation Software Used

Calculation software features

The software used is *RFEM*, developed by DLUBAL COMPANY (Germany).

Technical specifications

Name:	RFEM
Version:	5.15.01
Producer:	DLUBAL
	www.dlubal.com

License registered is a student license.

5. OUTLINE SPECIFICATION AND MATERIAL PROPERITIES

5.1 Wooden materials

5.1.1 CLT walls and floors

Layer No.	Poisson's Ratio [-] ν_{xy}	Poisson's Ratio [-] ν_{yx}	Specific Weight γ [lbf/ft^3]	Coeff. of Th. Exp. α_T [1/°F]	Comment
1	0.000	0.000	29.00	2.8E-06	6 7/8 in, CLT Grade E1, ANSI/APA PRG 320 CLT - US
2	0.000	0.000	29.00	2.8E-06	6 7/8 in, CLT Grade E1, ANSI/APA PRG 320 CLT - US
3	0.000	0.000	29.00	2.8E-06	6 7/8 in, CLT Grade E1, ANSI/APA PRG 320 CLT - US
4	0.000	0.000	29.00	2.8E-06	6 7/8 in, CLT Grade E1, ANSI/APA PRG 320 CLT - US
5	0.000	0.000	29.00	2.8E-06	6 7/8 in, CLT Grade E1, ANSI/APA PRG 320 CLT - US

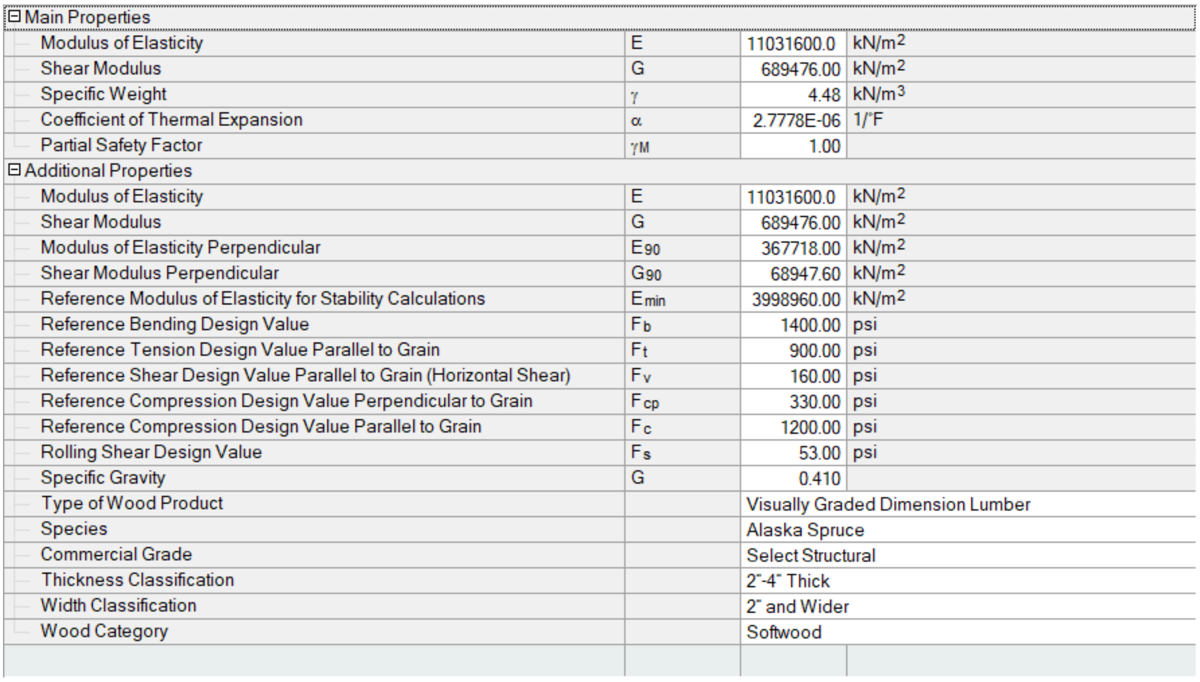
Layer	Material	Strengths for Bending / Tension / Compression [psi]					
No.	Description	$F_{b,0}$	$F_{b,90}$	$F_{t,0}$	$F_{t,90}$	$F_{c,0}$	$F_{c,90}$
1	1950f-1.7E Spruce-pine-fir MSR lumber	1950.0	1950.0	1375.0	15.0	1800.0	425.0
2	No. 3 Spruce-pine-fir lumber	500.0	500.0	250.0	15.0	650.0	425.0
3	1950f-1.7E Spruce-pine-fir MSR lumber	1950.0	1950.0	1375.0	15.0	1800.0	425.0
4	No. 3 Spruce-pine-fir lumber	500.0	500.0	250.0	15.0	650.0	425.0
5	1950f-1.7E Spruce-pine-fir MSR lumber	1950.0	1950.0	1375.0	15.0	1800.0	425.0

Layer	Material	Shear Strengths [psi]		
No.	Description	F_{xy}	F_v	F_s
1	1950f-1.7E Spruce-pine-fir MSR lumber	135.0	135.0	45.0
2	No. 3 Spruce-pine-fir lumber	135.0	135.0	45.0
3	1950f-1.7E Spruce-pine-fir MSR lumber	135.0	135.0	45.0
4	No. 3 Spruce-pine-fir lumber	135.0	135.0	45.0
5	1950f-1.7E Spruce-pine-fir MSR lumber	135.0	135.0	45.0

Layer	Material	Thickness	Orthotropic	Modulus of Elasticity [psi]	Column1	Shear Modulus [psi]	Column2	Column3
No.	Description	t [in]	Direction β [°]	E_x	E_y	G_{xz}	G_{yz}	G_{xy}
1	1950f-1.7E Spruce-pine-fir M	1.37	0.00	1699999.9	56667.0	106250.0	10625.0	106250.0
2	No. 3 Spruce-pine-fir lumber	1.37	90.00	1200000.0	40000.0	75000.0	7500.0	75000.0
3	1950f-1.7E Spruce-pine-fir M	1.37	0.00	1699999.9	56667.0	106250.0	10625.0	106250.0
4	No. 3 Spruce-pine-fir lumber	1.37	90.00	1200000.0	40000.0	75000.0	7500.0	75000.0
5	1950f-1.7E Spruce-pine-fir M	1.37	0.00	1699999.9	56667.0	106250.0	10625.0	106250.0

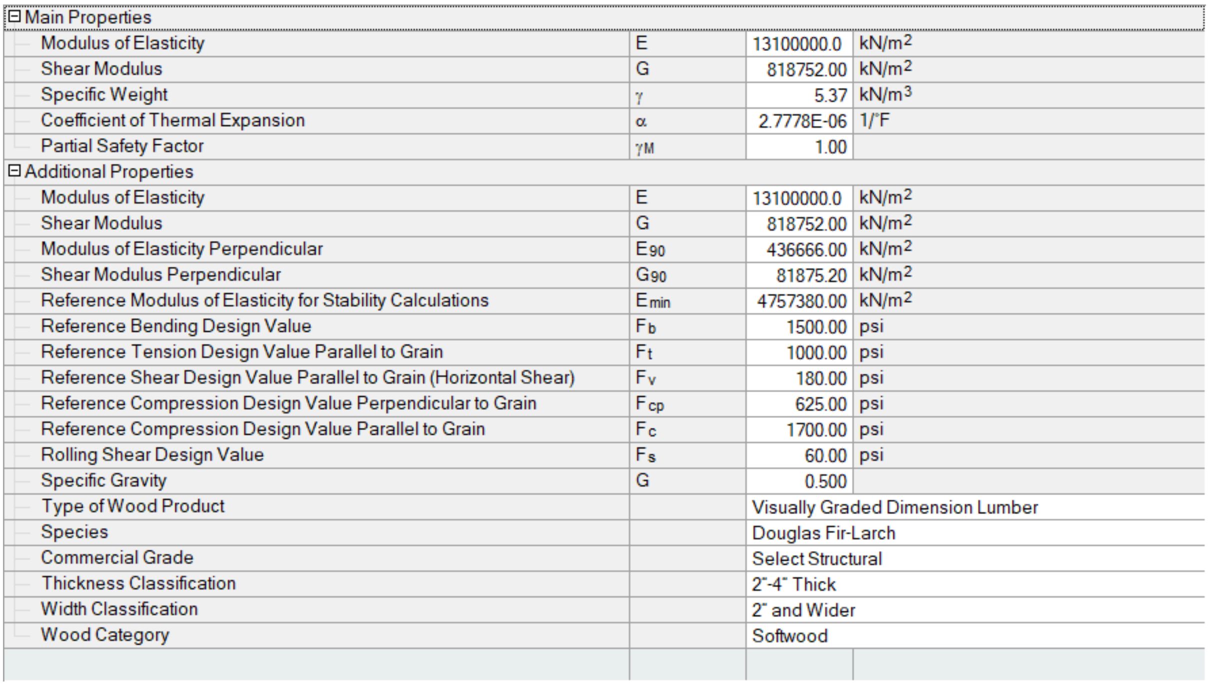
5.1.2 LVL Beams

Alaska Spruce, 2“-4“ Thick, 2“ and Wider, Select Structural | ANSI/AWC NDS-2015



⊟ Main Properties			
Modulus of Elasticity	E	11031600.0	kN/m^2
Shear Modulus	G	689476.00	kN/m^2
Specific Weight	γ	4.48	kN/m^3
Coefficient of Thermal Expansion	α	2.7778E-06	1/°F
Partial Safety Factor	γ_M	1.00	
⊟ Additional Properties			
Modulus of Elasticity	E	11031600.0	kN/m^2
Shear Modulus	G	689476.00	kN/m^2
Modulus of Elasticity Perpendicular	E_{90}	367718.00	kN/m^2
Shear Modulus Perpendicular	G_{90}	68947.60	kN/m^2
Reference Modulus of Elasticity for Stability Calculations	E_{min}	3998960.00	kN/m^2
Reference Bending Design Value	F_b	1400.00	psi
Reference Tension Design Value Parallel to Grain	F_t	900.00	psi
Reference Shear Design Value Parallel to Grain (Horizontal Shear)	F_v	160.00	psi
Reference Compression Design Value Perpendicular to Grain	F_{cp}	330.00	psi
Reference Compression Design Value Parallel to Grain	F_c	1200.00	psi
Rolling Shear Design Value	F_s	53.00	psi
Specific Gravity	G	0.410	
Type of Wood Product		Visually Graded Dimension Lumber	
Species		Alaska Spruce	
Commercial Grade		Select Structural	
Thickness Classification		2"-4" Thick	
Width Classification		2" and Wider	
Wood Category		Softwood	

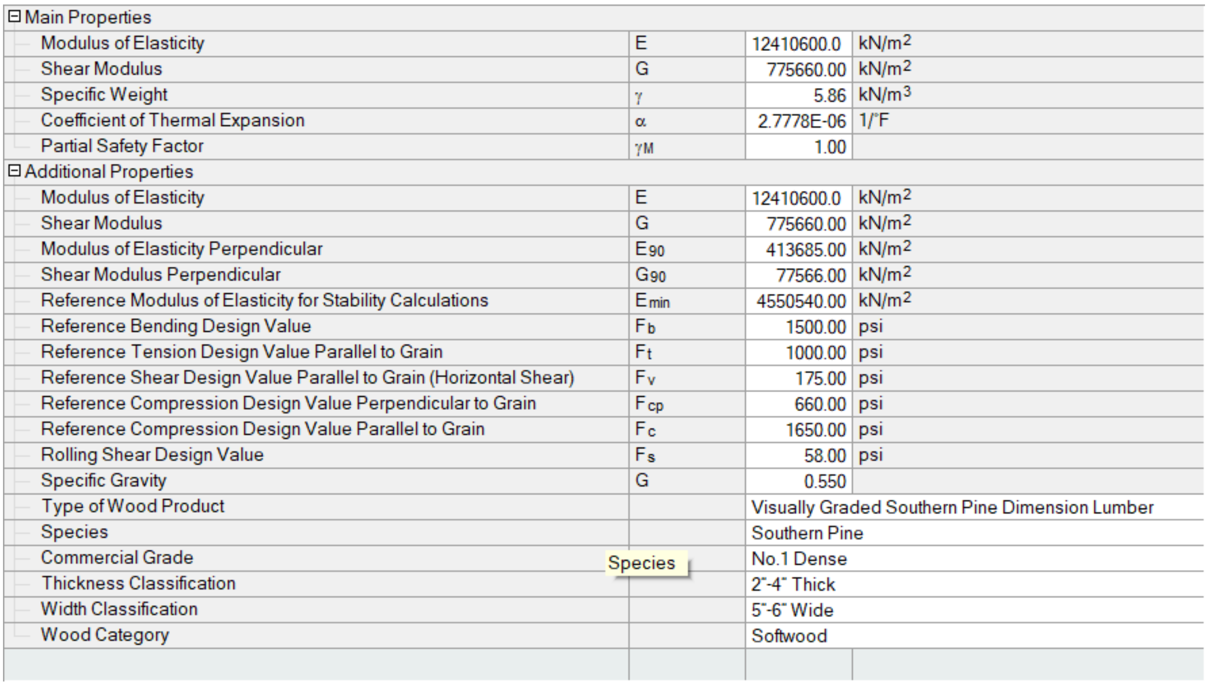
Douglas Fir-Larch, 2“-4“ Thick, 2“ and Wider, Select Structural | ANSI/AWC NDS-201



⊟ Main Properties			
Modulus of Elasticity	E	13100000.0	kN/m^2
Shear Modulus	G	818752.00	kN/m^2
Specific Weight	γ	5.37	kN/m^3
Coefficient of Thermal Expansion	α	2.7778E-06	1/°F
Partial Safety Factor	γ_M	1.00	
⊟ Additional Properties			
Modulus of Elasticity	E	13100000.0	kN/m^2
Shear Modulus	G	818752.00	kN/m^2
Modulus of Elasticity Perpendicular	E_{90}	436666.00	kN/m^2
Shear Modulus Perpendicular	G_{90}	81875.20	kN/m^2
Reference Modulus of Elasticity for Stability Calculations	E_{min}	4757380.00	kN/m^2
Reference Bending Design Value	F_b	1500.00	psi
Reference Tension Design Value Parallel to Grain	F_t	1000.00	psi
Reference Shear Design Value Parallel to Grain (Horizontal Shear)	F_v	180.00	psi
Reference Compression Design Value Perpendicular to Grain	F_{cp}	625.00	psi
Reference Compression Design Value Parallel to Grain	F_c	1700.00	psi
Rolling Shear Design Value	F_s	60.00	psi
Specific Gravity	G	0.500	
Type of Wood Product		Visually Graded Dimension Lumber	
Species		Douglas Fir-Larch	
Commercial Grade		Select Structural	
Thickness Classification		2"-4" Thick	
Width Classification		2" and Wider	
Wood Category		Softwood	

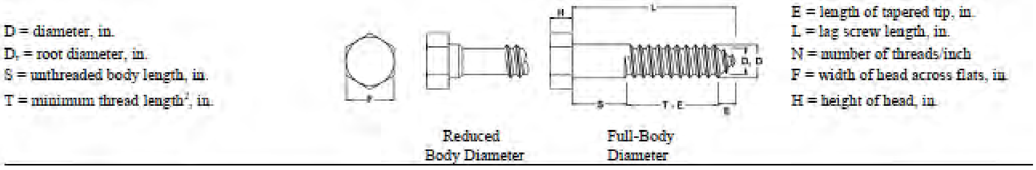
5.1.3 GLULAM Columns

Douglas Fir-Larch, 2“-4“ Thick, 2“ and Wider, Select Structural | ANSI/AWC NDS-2015



⊟ Main Properties			
Modulus of Elasticity	E	12410600.0	kN/m^2
Shear Modulus	G	775660.00	kN/m^2
Specific Weight	γ	5.86	kN/m^3
Coefficient of Thermal Expansion	α	2.7778E-06	1/°F
Partial Safety Factor	γ_M	1.00	
⊟ Additional Properties			
Modulus of Elasticity	E	12410600.0	kN/m^2
Shear Modulus	G	775660.00	kN/m^2
Modulus of Elasticity Perpendicular	E_{90}	413685.00	kN/m^2
Shear Modulus Perpendicular	G_{90}	77566.00	kN/m^2
Reference Modulus of Elasticity for Stability Calculations	E_{min}	4550540.00	kN/m^2
Reference Bending Design Value	F_b	1500.00	psi
Reference Tension Design Value Parallel to Grain	F_t	1000.00	psi
Reference Shear Design Value Parallel to Grain (Horizontal Shear)	F_v	175.00	psi
Reference Compression Design Value Perpendicular to Grain	F_{cp}	660.00	psi
Reference Compression Design Value Parallel to Grain	F_c	1650.00	psi
Rolling Shear Design Value	F_s	58.00	psi
Specific Gravity	G	0.550	
Type of Wood Product		Visually Graded Southern Pine Dimension Lumber	
Species		Southern Pine	
Commercial Grade		No.1 Dense	
Thickness Classification		2"-4" Thick	
Width Classification		5"-6" Wide	
Wood Category		Softwood	

5.2 Screws



D = diameter, in.
D_r = root diameter, in.
S = unthreaded body length, in.
T = minimum thread length[2], in.

E = length of tapered tip, in.
L = lag screw length, in.
N = number of threads/inch
F = width of head across flats, in.
H = height of head, in.

type	Diameter (D)	Root Diameter (Dr)	Length	Tapered Tip Length (E)	Thread Length (T)
0.5 in. Hexa Lag Screw	0.5 in	0.371 in	9 in	5/16 in	5 in
	0.5 in	0.371 in	12 in	5/16 in	6 in

6. Calculation method and numerical model

6.1 Model Description

6.1.1 Hypothesis adopted for the elements

The timber walls are constrained at the base by means of connection systems capable of transmitting both in-plane and out-of-plane actions.

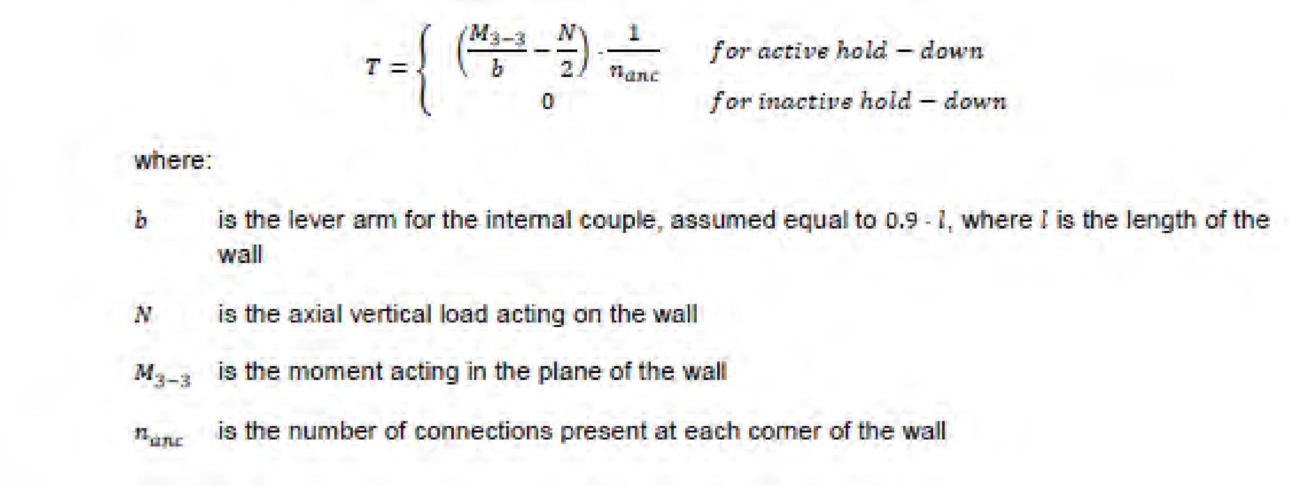
The floors are schematized simply supported by the walls or by the beams and the columns are modelled with hinged ends.

The horizontal elements are considered infinitely rigid in their plane and with three degrees of freedom: two translational and one rotational.

In the analysis, in presence of horizontal loads, some elements may be defined as “secondary”: this mean that their strength and stiffness are neglected in the calculation of the response of the building. In the model these elements are represented in columns.

6.1.2 Rigid body rocking – Forces on hold-down / tie-down

The hold-down or tie-down systems are used to prevent the rotation of the wall caused by the overturning moment of the horizontal force. The hold-down, placed on the in-tension edge of the wall, is loaded by a force equal to



$$T = \begin{cases} \left(\dfrac{M_{3-3}}{b} - \dfrac{N}{2}\right) \cdot \dfrac{1}{n_{anc}} & for\ active\ hold-down \\ 0 & for\ inactive\ hold-down \end{cases}$$

where:

b is the lever arm for the internal couple, assumed equal to $0.9 \cdot l$, where l is the length of the wall

N is the axial vertical load acting on the wall

M_{3-3} is the moment acting in the plane of the wall

n_{anc} is the number of connections present at each corner of the wall

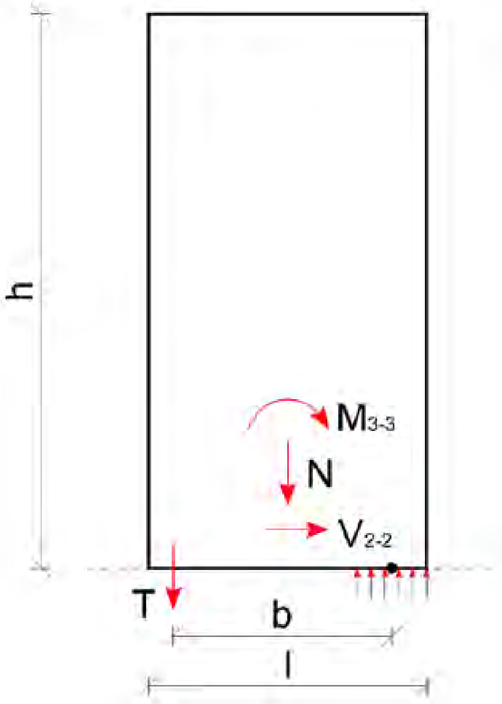


Figure 6.1: Calculation model for determining the tensile force acting on the hold-down

6.1.3 Wall horizontal stiffness

The wall stiffness can be estimated considering the contributions of all the components, as shown below

CLT walls

The overall stiffness of CLT walls is calculated taking into account the contribution of the following components:

- CLT panel (k_{XLAM})
- shear connections – angle brackets (k_a)
- hold-down or tie-down (k_h)

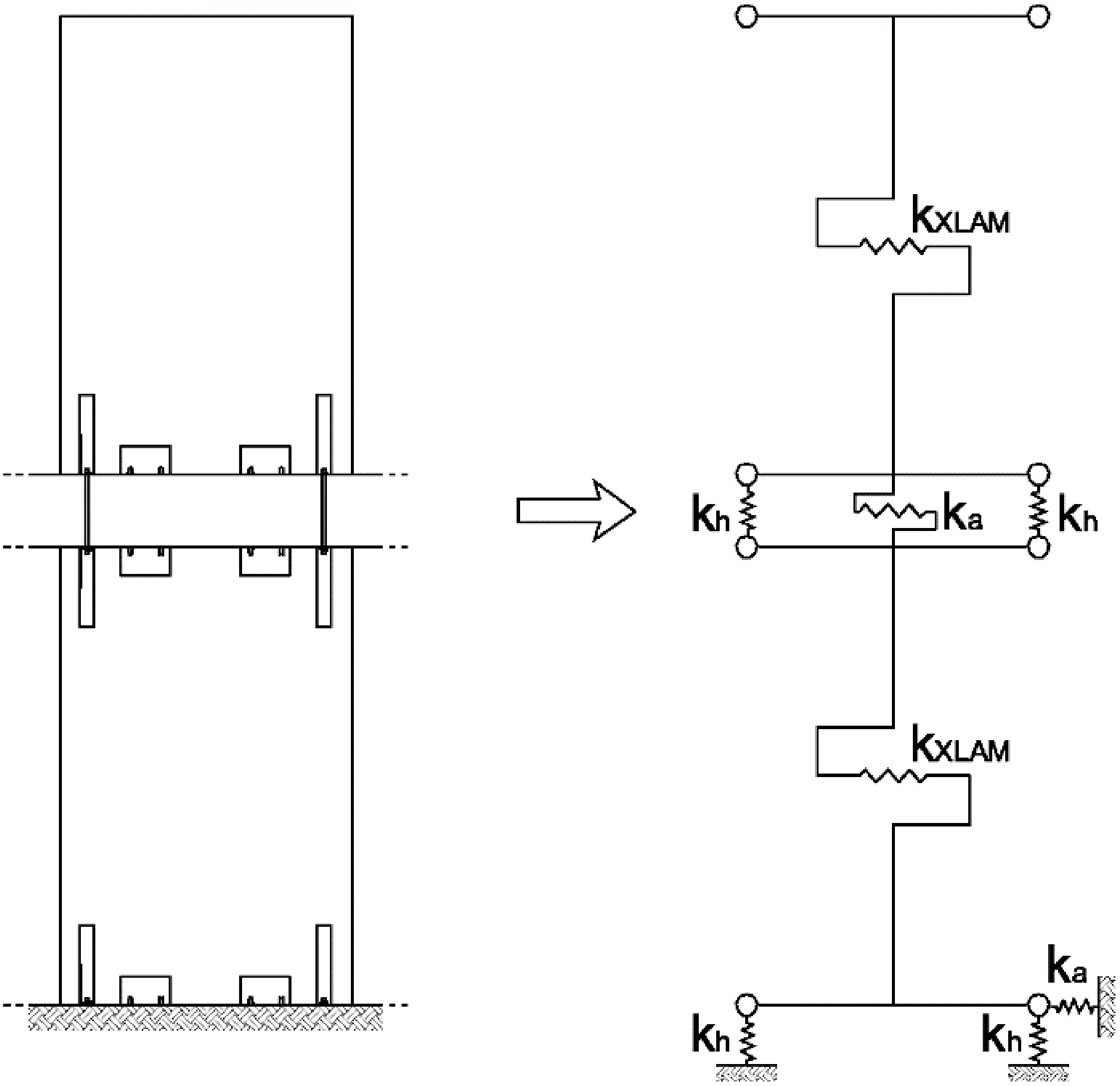


Figure 6.2: Mechanical model for determining the CLT walls overall stiffness

6.1.4 Types of structural elements and sign conventions

Linear elements

The linear elements are used to model beams and columns. They have a local reference system with respect to which stress/force components are shown. The sign convention adopted is shown in the figure below.

Force	Description	Unit of measure
N	Axial force	kN
My	Bending moment about local axis y	kN.m
Vz	Shear along local axis z	kN
Mz	Bending moment about local axis z	kN.m
Vy	Shear along local axis y	kN

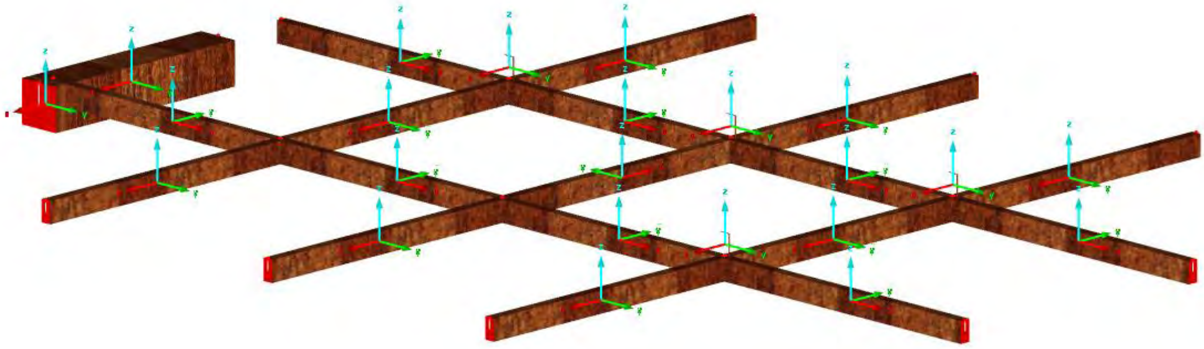


Figure 6.3: sign conventions for beams

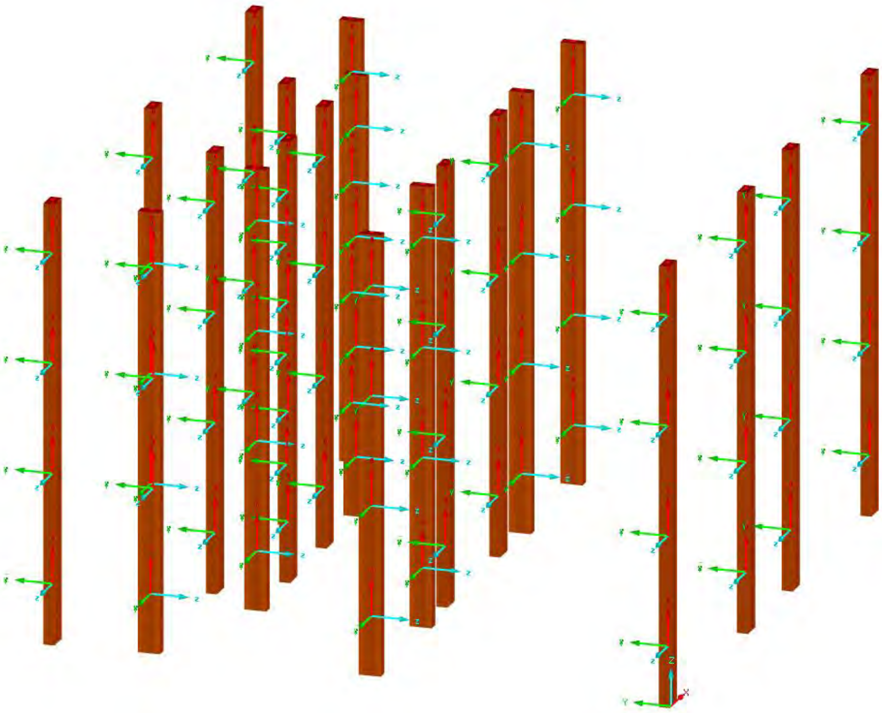


Figure 6.4: sign conventions for columns

Wall elements

The walls, regardless of type, have the following sign conventions.

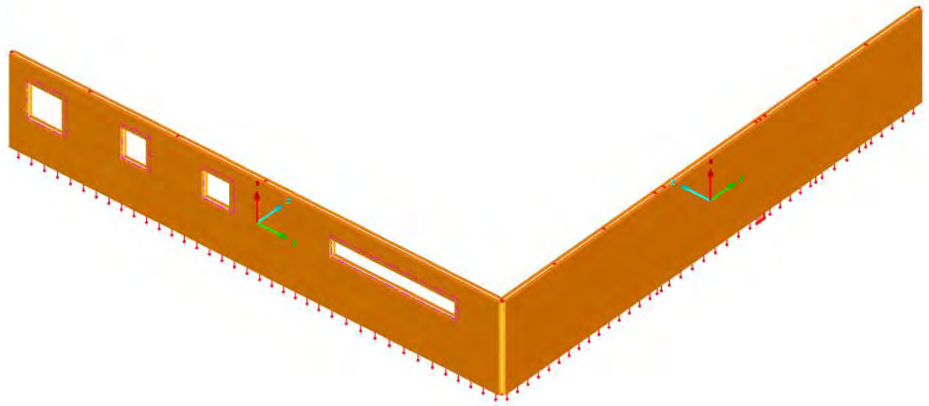
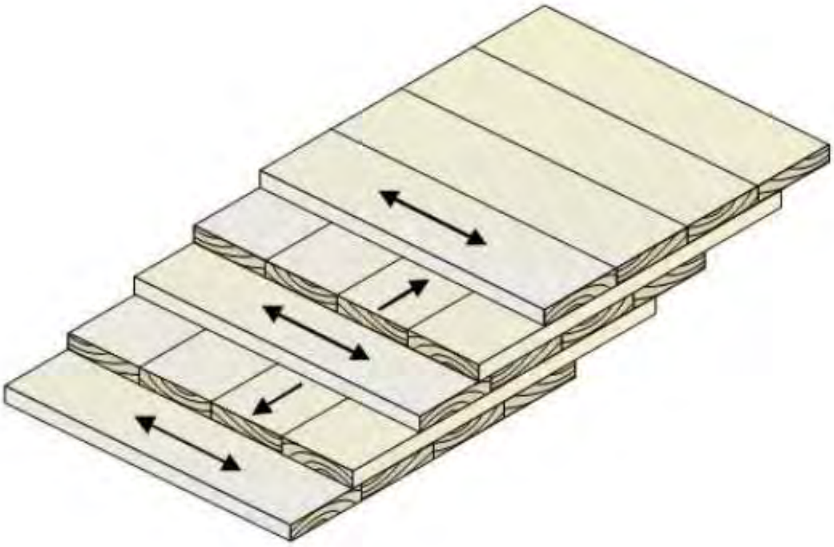


Figure 6.5: sign conventions for walls

	Stress	Description	Unit of measure
In-plane stresses	nx	Axial stress (per unit length)	kN/m
	mz	Bending moment about local axis z (per unit length)	kN.m/m
	vx	Shear along local axis x (per unit length)	kN/m
Out-of-plane stresses (plate)	mx	Bending moment about local axis x (per unit length)	kN.m/m
	vz	Shear along local axis z (per unit length)	kN/m

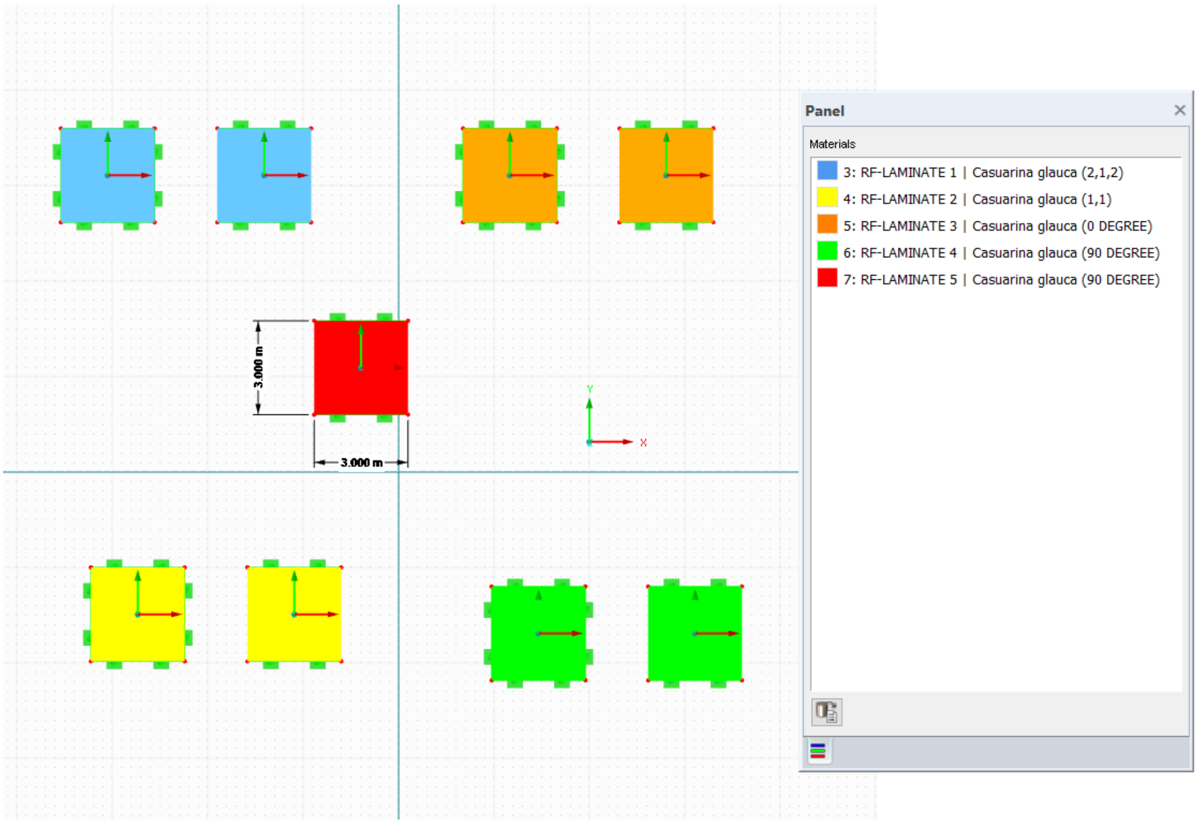
6.1.5 Orthotropic angle effect on CLT Panel.

- The innovation in massive wood appears in collect the wooden boards (laminations) and compress it together in transverse direction to create the first layer, after that, the layers has been collected together and compress it with structural adhesive, to create the section.



- There are many forms of layers composition, the most common is collect the layers in odd number. Each composition give different structural behavior for the section.

- The important inquery is, **what is the best form of layers compositions?,** to answer this question, a simulation of a nine slab panels with different layers compositions has been constructed and the results have been evaluated.



🟦 3: RF-LAMINATE 1 | Casuarina glauca (2,1,2)
🟨 4: RF-LAMINATE 2 | Casuarina glauca (1,1)
🟧 5: RF-LAMINATE 3 | Casuarina glauca (0 DEGREE)
🟩 6: RF-LAMINATE 4 | Casuarina glauca (90 DEGREE)
🟥 7: RF-LAMINATE 5 | Casuarina glauca (90 DEGREE)

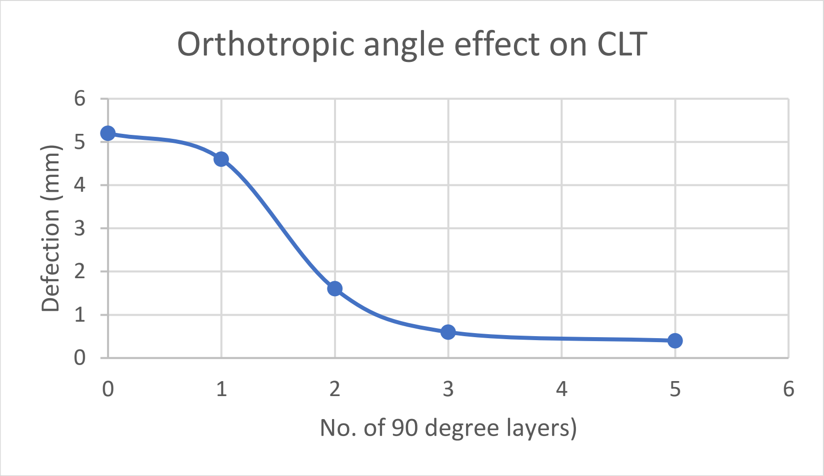
- The relationship between orthotropic direction and straining action in one way slabs and two way slabs shown in the following charts.

<u>Note that.</u>

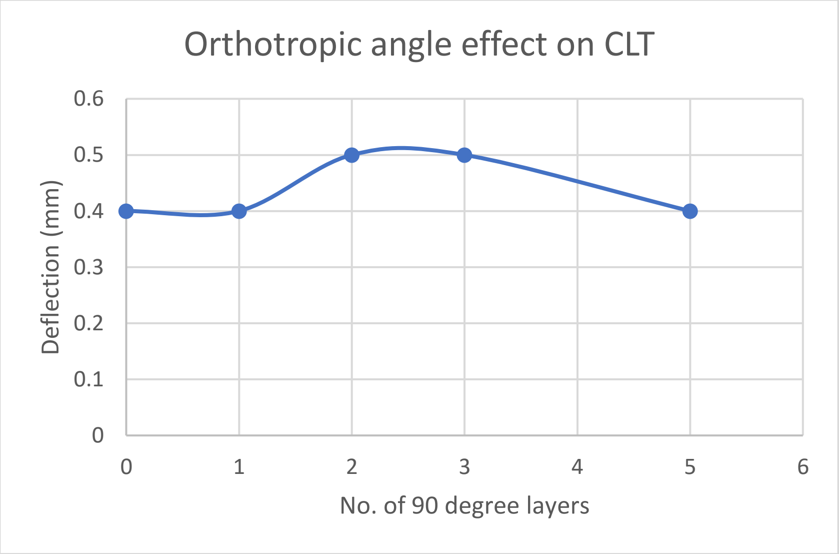
<u>Local axis x is considered as a strengthen axis and the orthotropic angle is measured about it.</u>

1- Orthotropic angle effect on deflection.

Deflection results on one way CLT slab	
Number of 90 degree layers	**Deflection value (mm)**
0	5.2
1	4.6
2	1.6
3	0.6
5	0.4

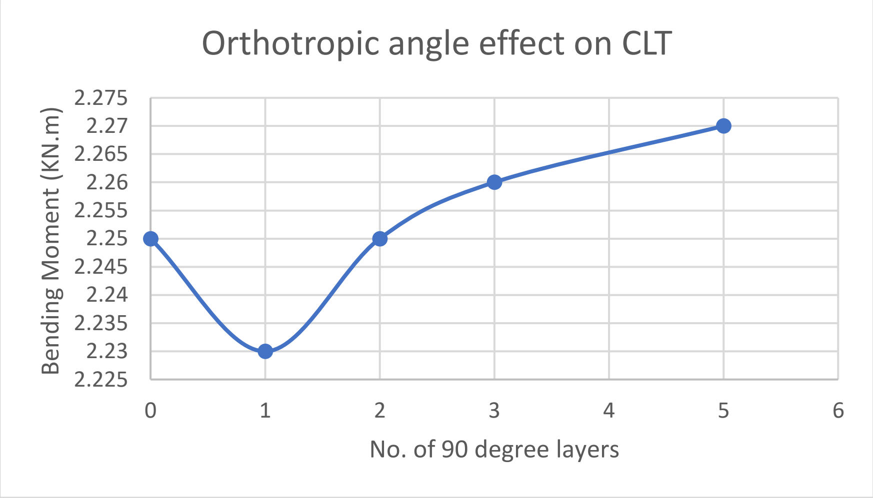


Deflection results on two way CLT slab	
Number of 90 degree layers	Deflection value (mm)
0	0.4
1	0.4
2	0.5
3	0.5
5	0.4

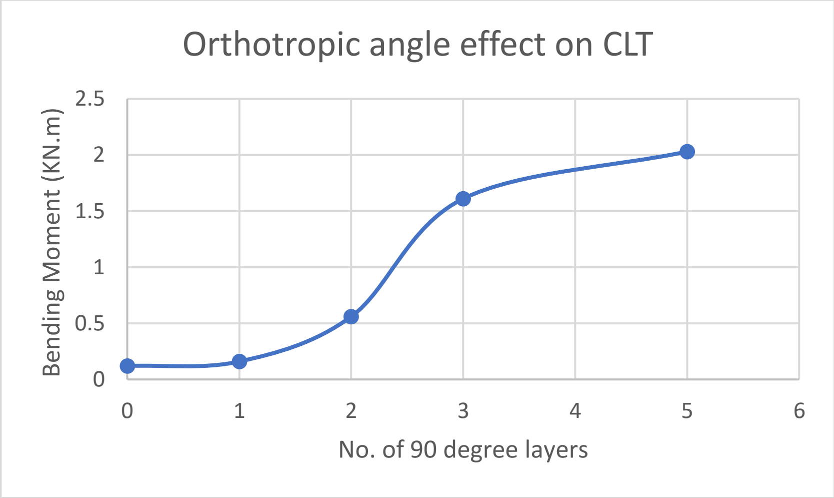


2- Orthotropic angle effect on Bending moment in y-direction.

M_y results on one way CLT slab	
Number of 90 degree layers	Bending moment value (KN.m)
0	2.25
1	2.23
2	2.25
3	2.26
5	2.27

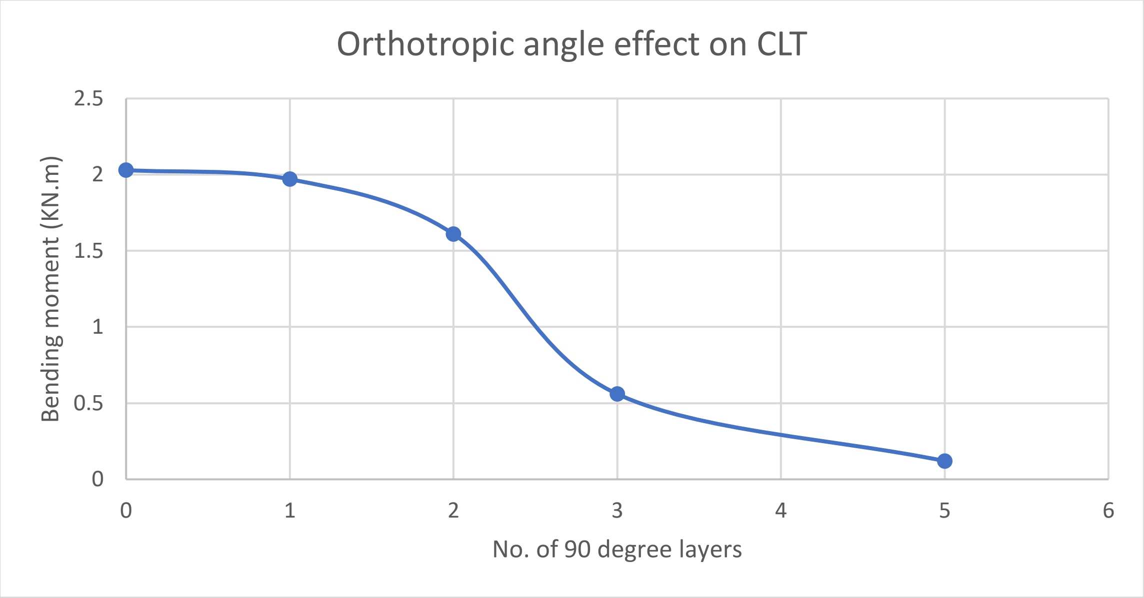


M_y results on two way CLT slab	
Number of 90 degree layers	Bending moment value (KN.m)
0	0.12
1	0.16
2	0.56
3	1.61
5	2.03



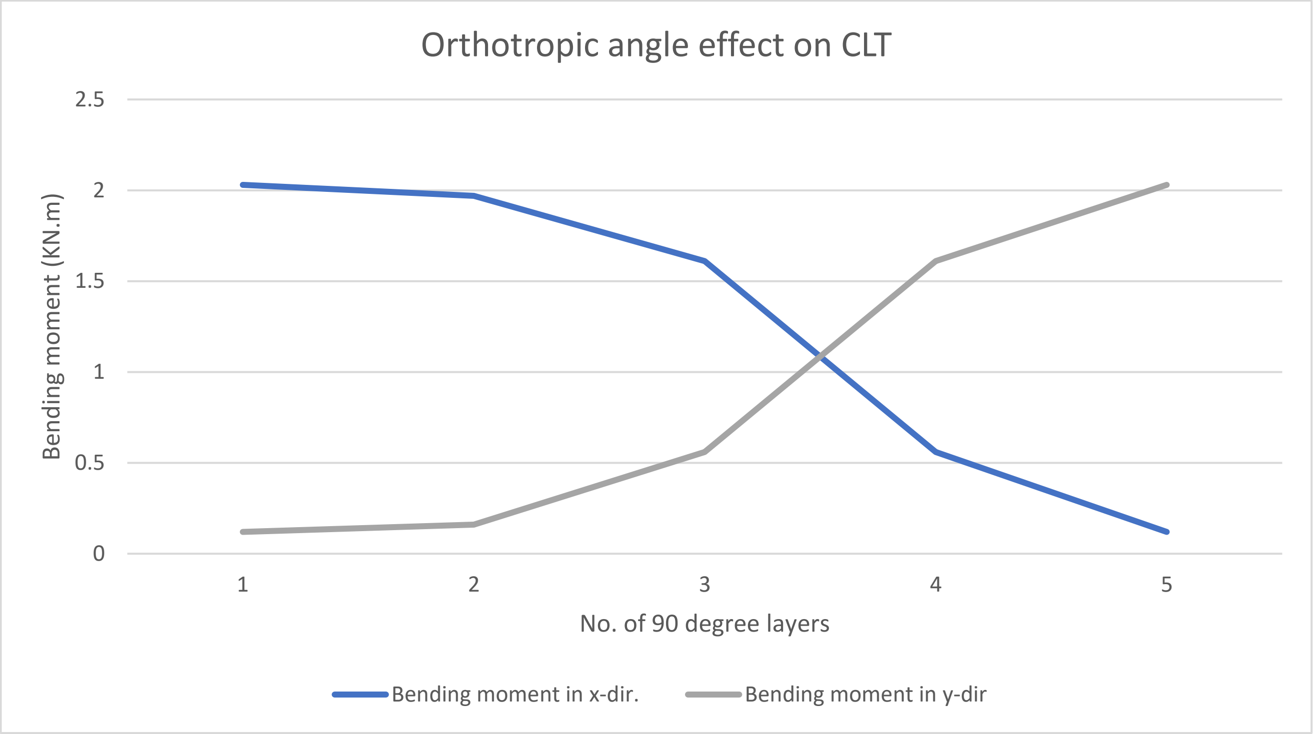
3- Orthotropic angle effect on Bending moment in x-direction.

M_x results on two way CLT slab	
Number of 90 degree layers	Bending moment value (KN.m)
0	2.03
1	1.97
2	1.61
3	0.56
5	0.12



Conclusion.

- Deflection is minimized when consider all layers in the same direction (support direction) with one way slabs.
- Deflection is minimized when consider all layers in the same direction with two way slabs.
- Bending moment is minimized when consider all layers in the same direction with one way slabs.
- Bending moment is minimized when consider odd section with two way slabs as show in the following figure.



Thus; it is recommended using layers compositions depending on the load direction on each span ratio in the building but that depending on manufacturing sections in each country. For moisture effect on the wood, thermal expansion and contraction, it is recommended, make opposite layers every two same direction layers.

7. Actions and design loads

7.1 STRUCTURAL LOADS

The following loads are considered in the design:

- Structural Dead Loads which include:

The own weight of the structural elements, slabs, columns, and walls.

Superimposed dead load from floorings.

- Live loads which cover the occupants, furniture, and mechanical equipment.

Wind loads on the external façade and roof.
Seismic loads according to ECP.

The basis for the considered design loads are summarized in the followings sections.

A. Dead Loads

- The weights of the structural materials are shown in the table below.

Description	Specific weight ɣ [kN/m^3]
Southern Pine, 2“-4“ Thick, 5“-6“ Wide, No.1 Dense | ANSI/AWC NDS-2015	5.86
Alaska Spruce, 2“-4“ Thick, 2“ and Wider, Select Structural | ANSI/AWC NDS-2015	4.48
Douglas Fir-Larch, 2“-4“ Thick, 2“ and Wider, Select Structural | ANSI/AWC NDS-2015	5.37
Cross Laminated Timber	4.65

Flooring shall be

➢ Typical floor	2.0	kN/m²
➢ Roof	4.0	kN/m²

B. Live Loads

Live loads for the different zone areas shall be calculated in accordance with (ECP 201-2012) as follows (uniformly distributed in kN/m²):

Living areas and bedrooms	2.0
Corridors	3.0
Toilets	3.0
Inaccessible roof	1.0

C. Wind Loads

The wind pressure shall be calculated in accordance with (ECP 201-2012)

Basic wind speed = 42 m/sec.

Wind pressure (or suction) distribution factor (Ce)

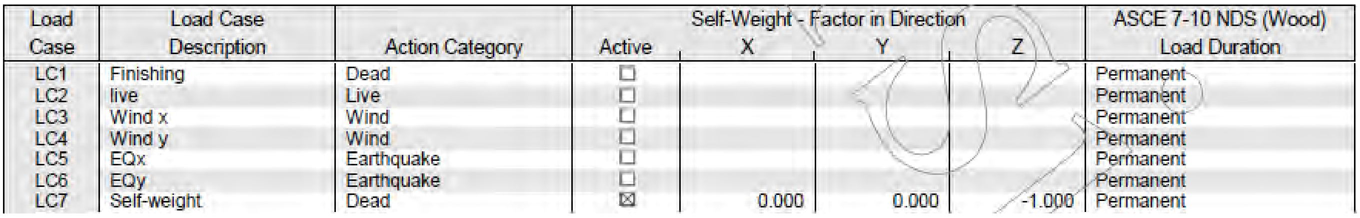
C_e = +0.8 for areas subjected to wind pressure

C_e = -0.5/-0.7 for areas subjected to suction wind

Exposure factor (according to height from ground level) (k = 1)

7.2 Load Cases and Load Combinations

7.2.1 Load Cases



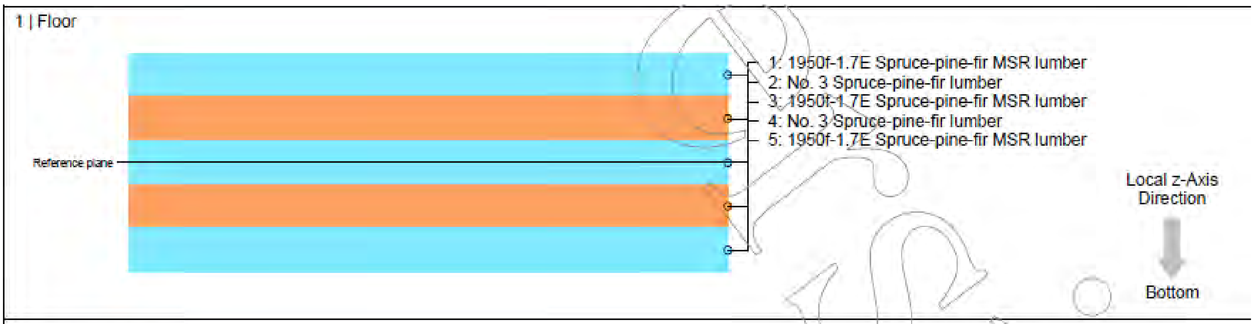
Load Case	Load Case Description	Action Category	Active	Self-Weight - Factor in Direction X	Y	Z	ASCE 7-10 NDS (Wood) Load Duration
LC1	Finishing	Dead	☐				Permanent
LC2	live	Live	☐				Permanent
LC3	Wind x	Wind	☐				Permanent
LC4	Wind y	Wind	☐				Permanent
LC5	EQx	Earthquake	☐				Permanent
LC6	EQy	Earthquake	☐				Permanent
LC7	Self-weight	Dead	☒	0.000	0.000	-1.000	Permanent

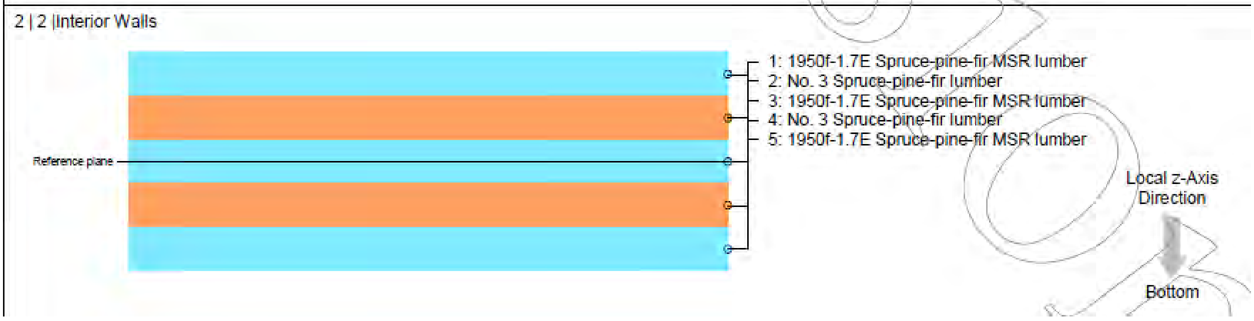
7.2.2 Load Combinations

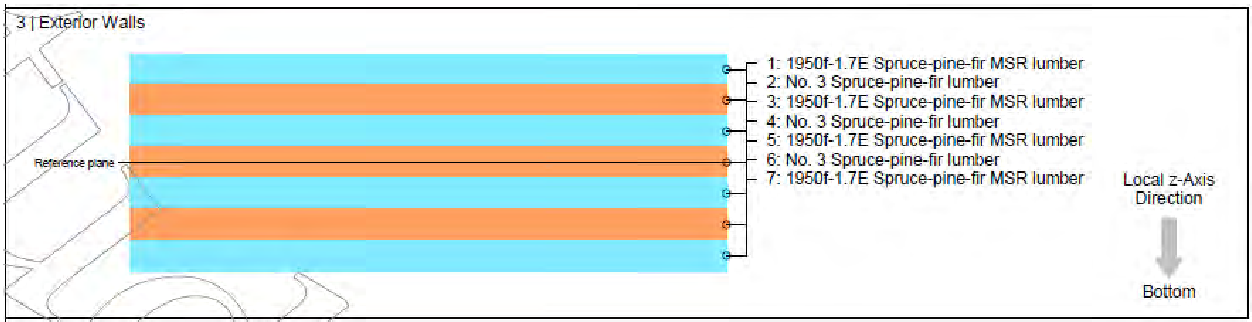
Load Combin.	DS	Load Combination Description	No.	Factor	Load Case	
CO1		1.4*LC1 + 1.4*LC7	1	1.40	LC1	Finishing
			2	1.40	LC7	Self-weight
CO2		1.2*LC1 + 1.6*LC2 + 1.2*LC7	1	1.20	LC1	Finishing
			2	1.60	LC2	live
			3	1.20	LC7	Self-weight
CO3		1.2*LC1 + 0.5*LC3 + 1.2*LC7	1	1.20	LC1	Finishing
			2	0.50	LC3	Wind x
			3	1.20	LC7	Self-weight
CO4		1.2*LC1 + 0.5*LC4 + 1.2*LC7	1	1.20	LC1	Finishing
			2	0.50	LC4	Wind y
			3	1.20	LC7	Self-weight
CO5		1.2*LC1 + LC2 + LC3 + 1.2*LC7	1	1.20	LC1	Finishing
			2	1.00	LC2	live
			3	1.00	LC3	Wind x
			4	1.20	LC7	Self-weight
CO6		1.2*LC1 + LC2 + LC4 + 1.2*LC7	1	1.20	LC1	Finishing
			2	1.00	LC2	live
			3	1.00	LC4	Wind y
			4	1.20	LC7	Self-weight
CO7		1.2*LC1 + LC3 + 1.2*LC7	1	1.20	LC1	Finishing
			2	1.00	LC3	Wind x
			3	1.20	LC7	Self-weight
CO8		1.2*LC1 + LC4 + 1.2*LC7	1	1.20	LC1	Finishing
			2	1.00	LC4	Wind y
			3	1.20	LC7	Self-weight
CO9		1.2*LC1 + LC5 + 1.2*LC7	1	1.20	LC1	Finishing
			2	1.00	LC5	EQx
			3	1.20	LC7	Self-weight
CO10		1.2*LC1 + LC2 + LC5 + 1.2*LC7	1	1.20	LC1	Finishing
			2	1.00	LC2	live
			3	1.00	LC5	EQx
			4	1.20	LC7	Self-weight
CO11		0.9*LC1 + LC3 + 0.9*LC7	1	0.90	LC1	Finishing
			2	1.00	LC3	Wind x
			3	0.90	LC7	Self-weight
CO12		0.9*LC1 + LC4 + 0.9*LC7	1	0.90	LC1	Finishing
			2	1.00	LC4	Wind y
			3	0.90	LC7	Self-weight
CO13		0.9*LC1 + LC5 + 0.9*LC7	1	0.90	LC1	Finishing
			2	1.00	LC5	EQx
			3	0.90	LC7	Self-weight
CO14		LC1 + LC7	1	1.00	LC1	Finishing
			2	1.00	LC7	Self-weight
CO15		LC1 + LC2 + LC7	1	1.00	LC1	Finishing
			2	1.00	LC2	live
			3	1.00	LC7	Self-weight
CO16		LC1 + 0.7*LC5 + LC7	1	1.00	LC1	Finishing
			2	0.70	LC5	EQx
			3	1.00	LC7	Self-weight
CO17		LC1 + 0.6*LC3 + LC7	1	1.00	LC1	Finishing
			2	0.60	LC3	Wind x
			3	1.00	LC7	Self-weight
CO18		LC1 + 0.6*LC4 + LC7	1	1.00	LC1	Finishing
			2	0.60	LC4	Wind y
			3	1.00	LC7	Self-weight
CO19		LC1 + 0.75*LC2 + 0.45*LC3 + LC7	1	1.00	LC1	Finishing
			2	0.75	LC2	live
			3	0.45	LC3	Wind x
			4	1.00	LC7	Self-weight
CO20		LC1 + 0.75*LC2 + 0.45*LC4 + LC7	1	1.00	LC1	Finishing
			2	0.75	LC2	live
			3	0.45	LC4	Wind y
			4	1.00	LC7	Self-weight
CO21		LC1 + 0.75*LC2 + 0.52*LC5 + LC7	1	1.00	LC1	Finishing
			2	0.75	LC2	live
			3	0.52	LC5	EQx
			4	1.00	LC7	Self-weight
CO22		0.6*LC1 + 0.6*LC3 + 0.6*LC7	1	0.60	LC1	Finishing
			2	0.60	LC3	Wind x
			3	0.60	LC7	Self-weight
CO23		0.6*LC1 + 0.6*LC4 + 0.6*LC7	1	0.60	LC1	Finishing
			2	0.60	LC4	Wind y
			3	0.60	LC7	Self-weight
CO24		0.6*LC1 + 0.7*LC5 + 0.6*LC7	1	0.60	LC1	Finishing
			2	0.70	LC5	EQx
			3	0.60	LC7	Self-weight

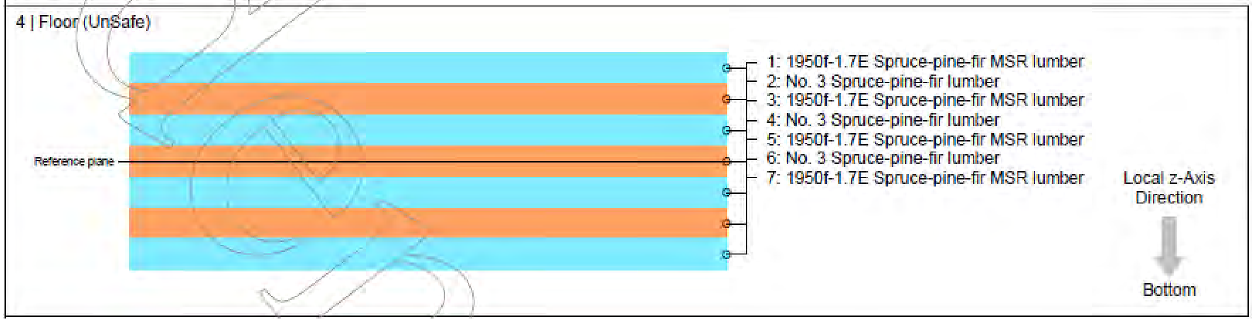
8. Sections of the structural elements

8.1 CLT walls and floors


1 | Floor


2 | 2 |Interior Walls


3 | Exterior Walls


4 | Floor (UnSafe)

h_b: CLT panel thickness

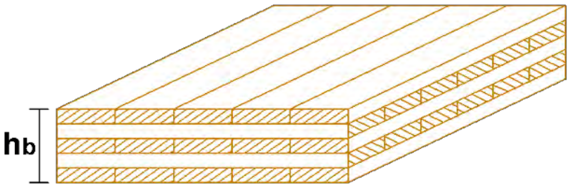


Figure 8.1: CLT geometric characteristics

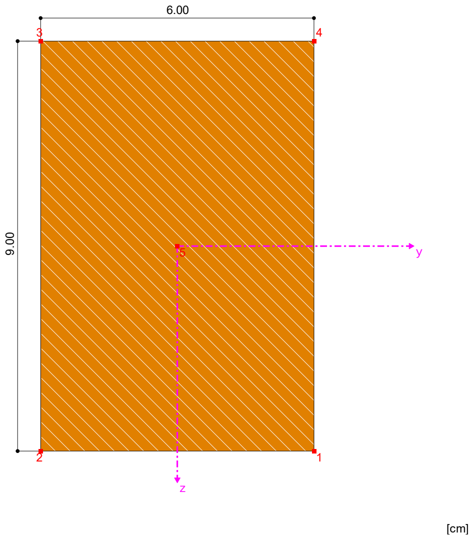
The following table sets out the details concerning the CLT floors.

Section name	Manufacturer	CLT panel Grade	Material	Number of layers	Thickness h_b[mm]	Layers Thickness (mm)	External layers orientation
CLT floor	ANSI/APA PRG-320-18	E_1	Spruce Pine	5	172	35	Parallel to the moment direction
CLT Unsafe floor	ANSI/APA PRG-320-18	E_1	Spruce Pine	7	244	35	Parallel to the moment direction
CLT Interior walls	ANSI/APA PRG-320-18	E_1	Spruce Pine	5	172	35	Parallel to the Loading direction
CLT exterior walls	ANSI/APA PRG-320-18	E_1	Spruce Pine	7	244	35	Parallel to the Loading direction

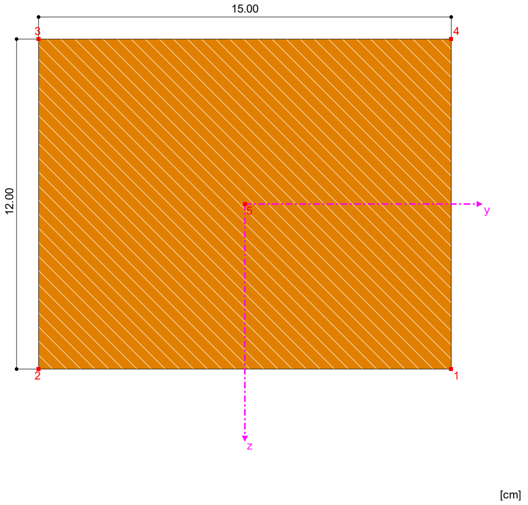
8.2 LVL Beams & Glulam Columns

8.2.1 LVL Beams

T-Rectangle 6/9

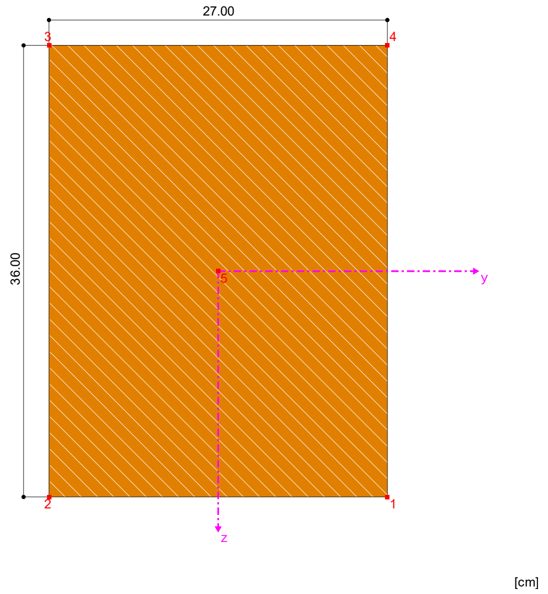


T-Rectangle 15/12



8.2.2 Glulam Columns

T-Rectangle 27/36



8.3 Connections

8.3.1 Hold Down

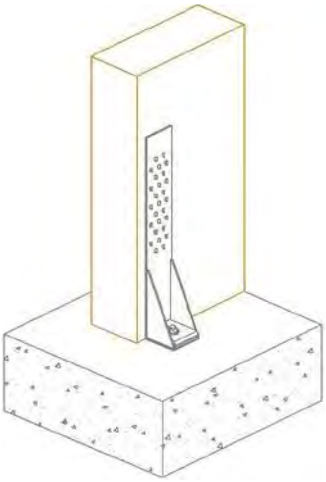
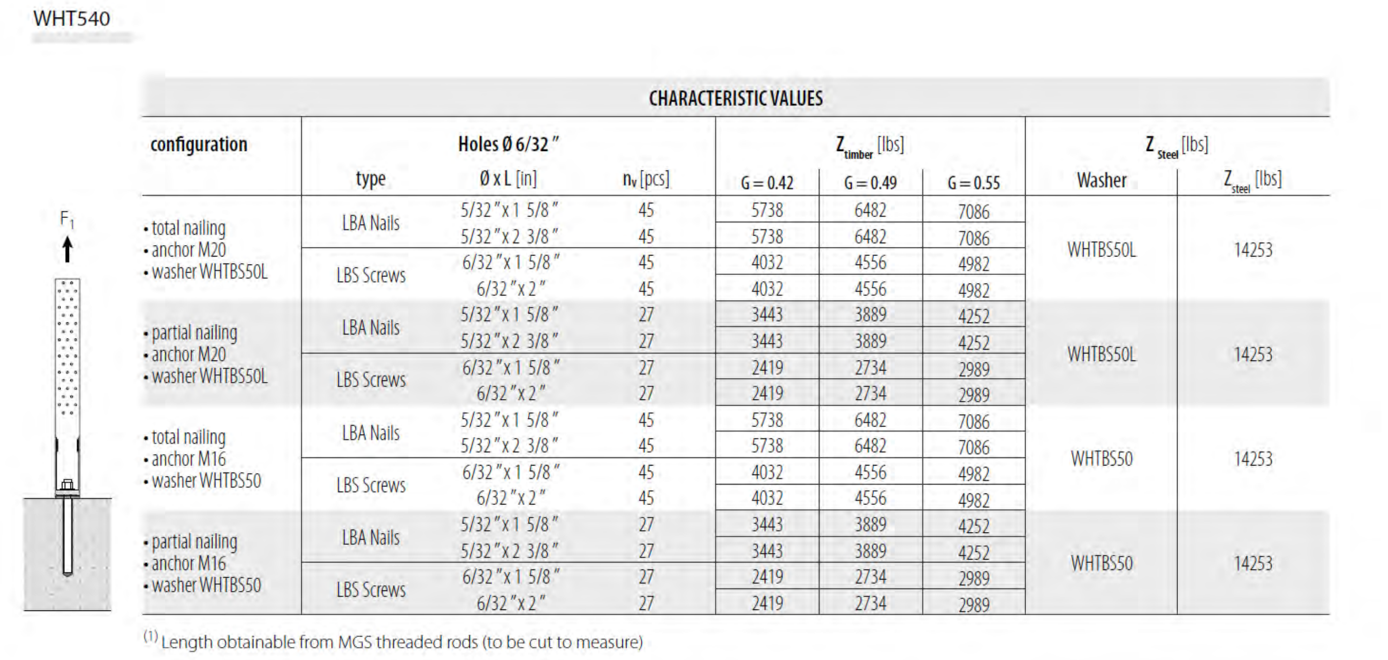


Figure 8.2: graphical representation of a hold-down in a base connection (timber wall – foundation connection)



WHT540

CHARACTERISTIC VALUES								
configuration	Holes Ø 6/32″			Z_{timber} [lbs]			Z_{Steel} [lbs]	
	type	Ø x L [in]	n_v [pcs]	G = 0.42	G = 0.49	G = 0.55	Washer	Z_{steel} [lbs]
• total nailing • anchor M20 • washer WHTBS50L	LBA Nails	5/32″x 1 5/8″	45	5738	6482	7086	WHTBS50L	14253
		5/32″x 2 3/8″	45	5738	6482	7086		
	LBS Screws	6/32″x 1 5/8″	45	4032	4556	4982		
		6/32″x 2″	45	4032	4556	4982		
• partial nailing • anchor M20 • washer WHTBS50L	LBA Nails	5/32″x 1 5/8″	27	3443	3889	4252	WHTBS50L	14253
		5/32″x 2 3/8″	27	3443	3889	4252		
	LBS Screws	6/32″x 1 5/8″	27	2419	2734	2989		
		6/32″x 2″	27	2419	2734	2989		
• total nailing • anchor M16 • washer WHTBS50	LBA Nails	5/32″x 1 5/8″	45	5738	6482	7086	WHTBS50	14253
		5/32″x 2 3/8″	45	5738	6482	7086		
	LBS Screws	6/32″x 1 5/8″	45	4032	4556	4982		
		6/32″x 2″	45	4032	4556	4982		
• partial nailing • anchor M16 • washer WHTBS50	LBA Nails	5/32″x 1 5/8″	27	3443	3889	4252	WHTBS50	14253
		5/32″x 2 3/8″	27	3443	3889	4252		
	LBS Screws	6/32″x 1 5/8″	27	2419	2734	2989		
		6/32″x 2″	27	2419	2734	2989		

(1) Length obtainable from MGS threaded rods (to be cut to measure)

8.3.2 Timber-reinforced concrete connection

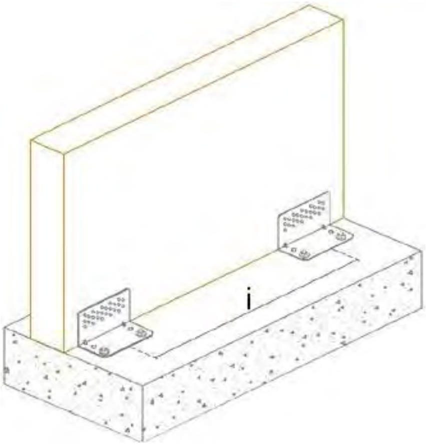
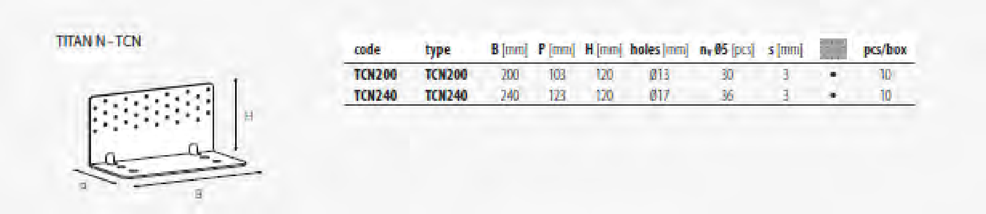


Figure 8.3: graphical representation of the shear connection with angle bracket



TITAN N - TCN

code	type	B [mm]	P [mm]	H [mm]	holes [mm]	n_v Ø5 [pcs]	s [mm]	▨	pcs/box
TCN200	TCN200	200	103	120	Ø13	30	3	•	10
TCN240	TCN240	240	123	120	Ø17	36	3	•	10

8.3.3 Double Hold Down

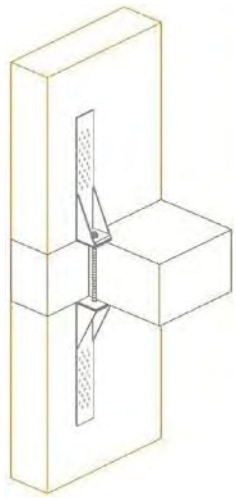


Figure 8.4: graphical representation of the hold-down connection at the upper floors

Connection name	Connection position	Manufacturer	Description	Fasteners number	Fastener typology	Bolt	Number of connections at each wall end
Upper level - 2 hold down - shear angle bracket	Upper level	Rotho Blass	WHT 540	42	Chiodi Anker 4,0 X	M16 5.8	1

8.3.4 Angle bracket - Timber to Timber connection

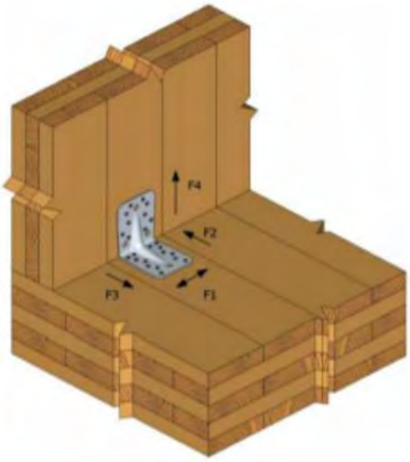
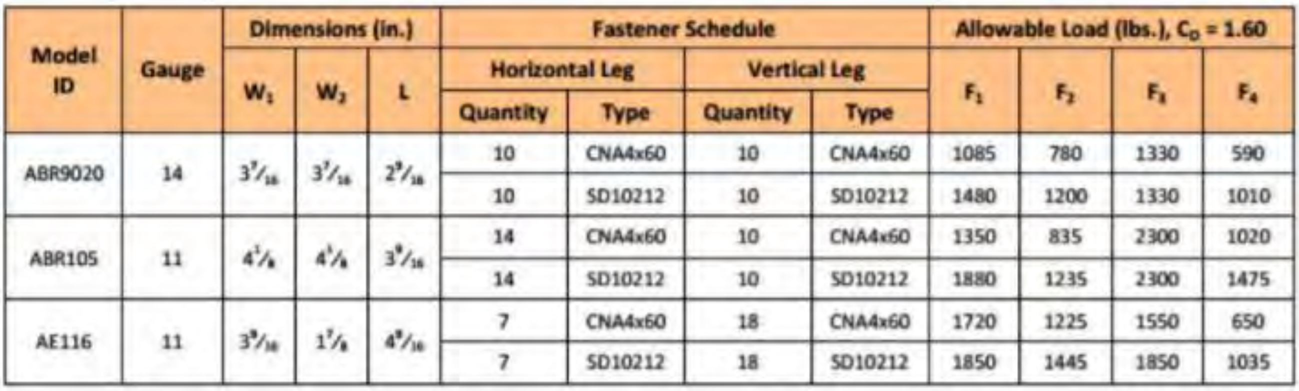


Figure 8.5: graphical representation of the timber to timber shear connection with angle brackets



Model ID	Gauge	Dimensions (in.)			Fastener Schedule				Allowable Load (lbs.), C_D = 1.60			
		W_1	W_2	L	Horizontal Leg		Vertical Leg		F_1	F_2	F_3	F_4
					Quantity	Type	Quantity	Type				
ABR9020	14	$3\frac{7}{16}$	$3\frac{7}{16}$	$2\frac{9}{16}$	10	CNA4x60	10	CNA4x60	1085	780	1330	590
					10	SD10212	10	SD10212	1480	1200	1330	1010
ABR105	11	$4\frac{1}{8}$	$4\frac{1}{8}$	$3\frac{9}{16}$	14	CNA4x60	10	CNA4x60	1350	835	2300	1020
					14	SD10212	10	SD10212	1880	1235	2300	1475
AE116	11	$3\frac{9}{16}$	$1\frac{7}{8}$	$4\frac{9}{16}$	7	CNA4x60	18	CNA4x60	1720	1225	1550	650
					7	SD10212	18	SD10212	1850	1445	1850	1035

9. STRUCTURAL SYSTEM

The following structural system is utilized to support the previously mentioned loads and satisfy the functional and architectural requirements of the building.

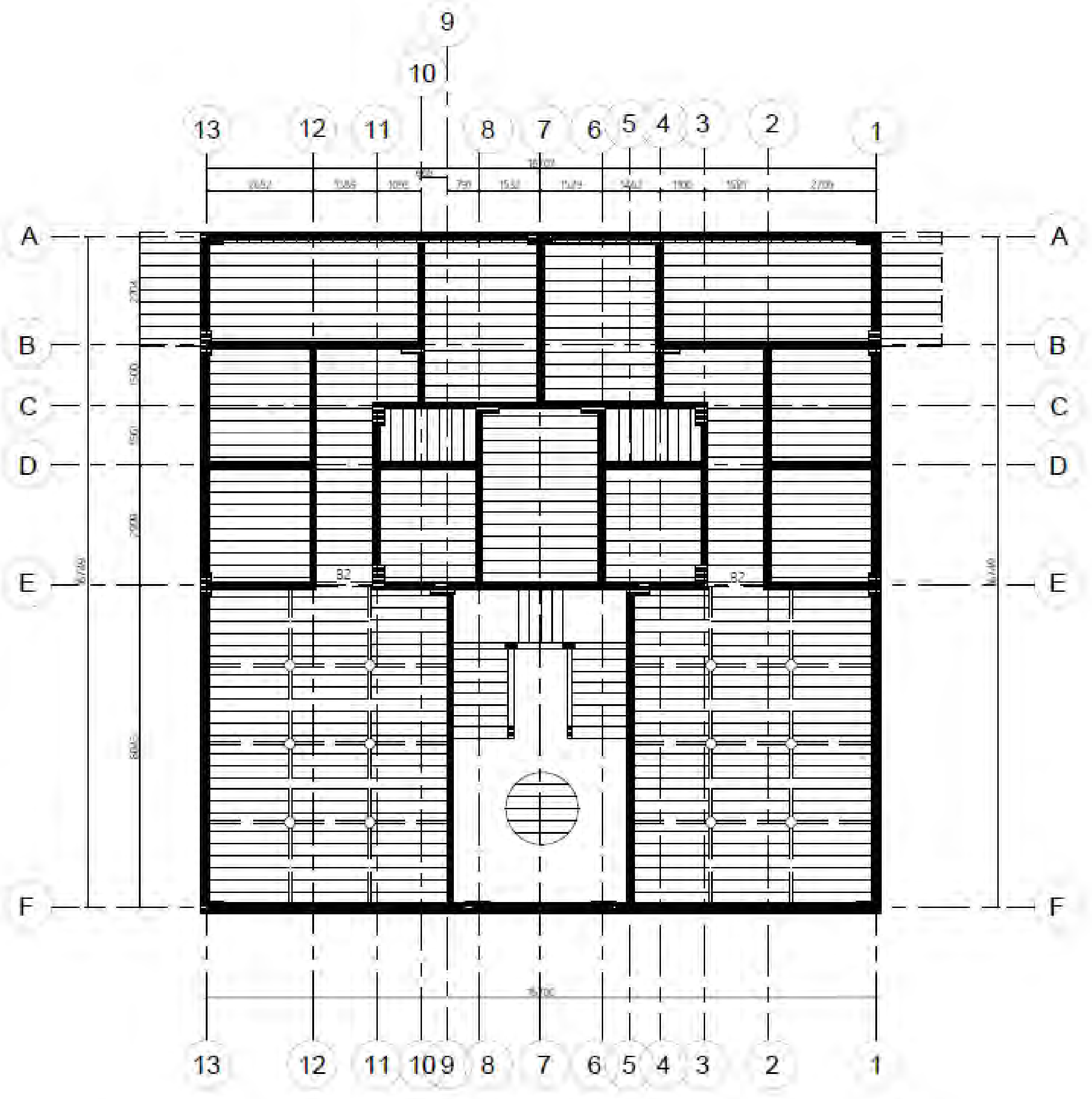


Figure 9.1: Floor Plan

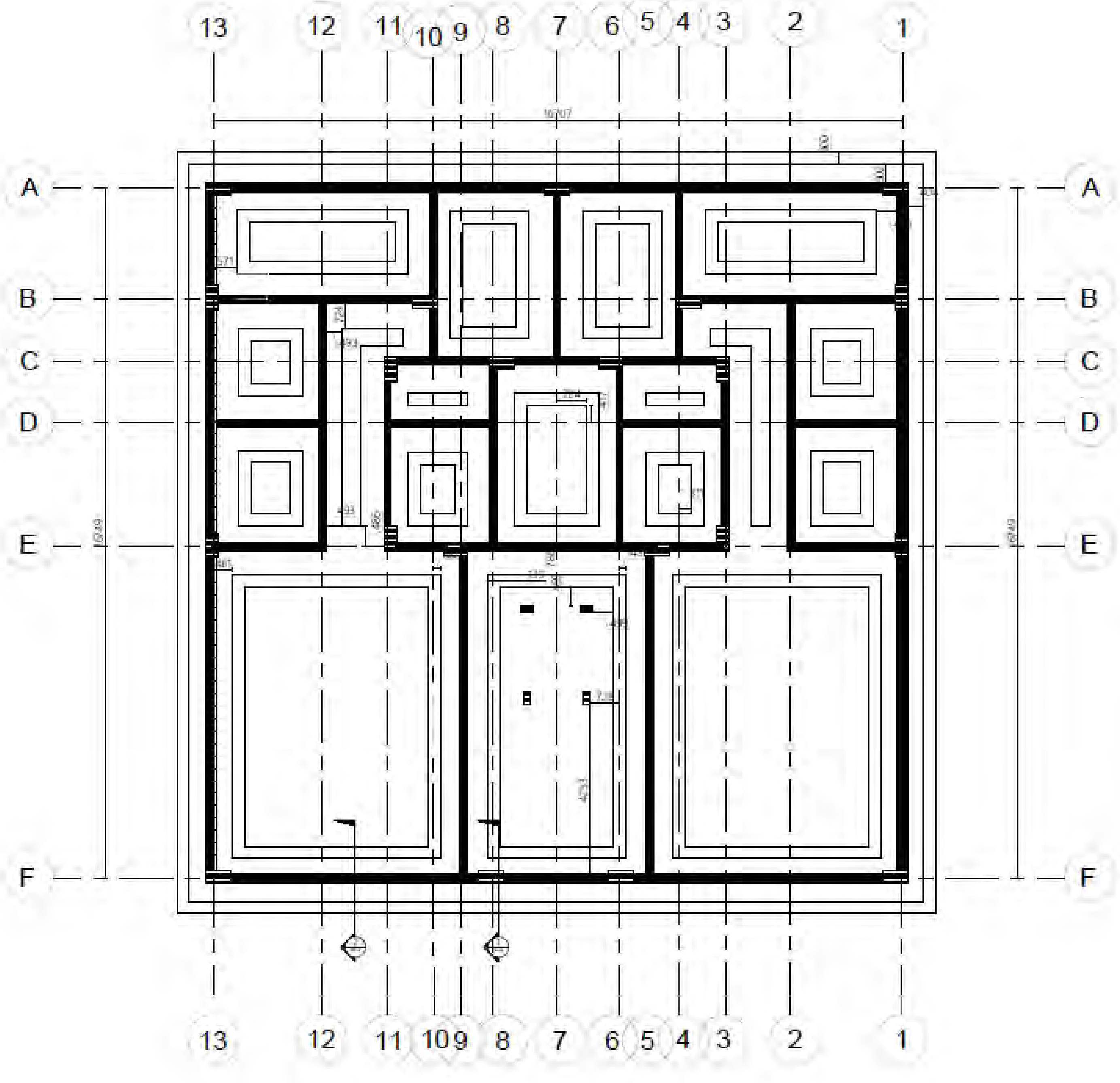


Figure 9.2: Strip footings foundations

10. STRUCTURAL ANALYSIS

10.1 ASSIGN OF LOADS

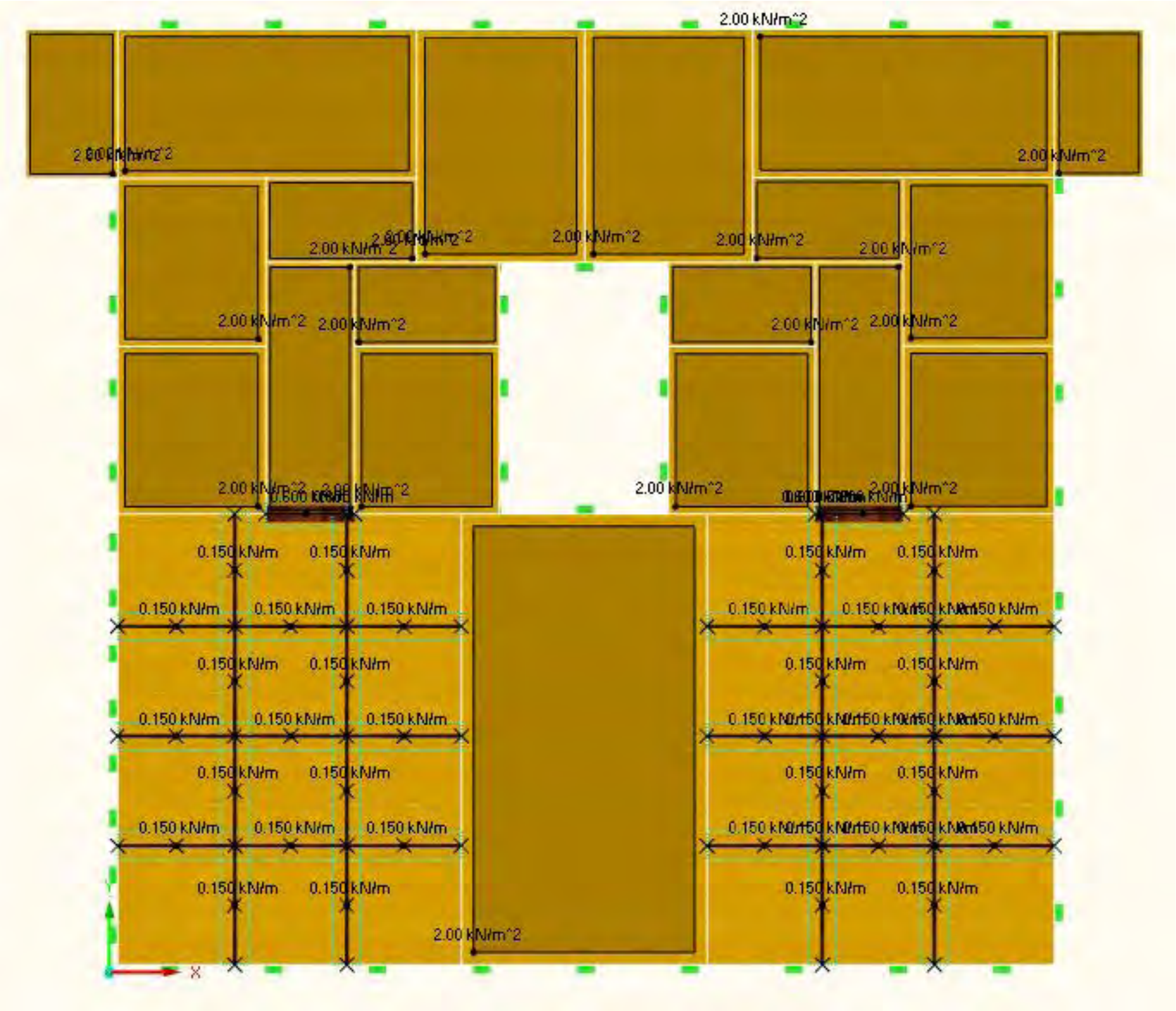


Figure 10.1.: Assign of Finishing and Live Loads on Typical Floor (KN-M Units)

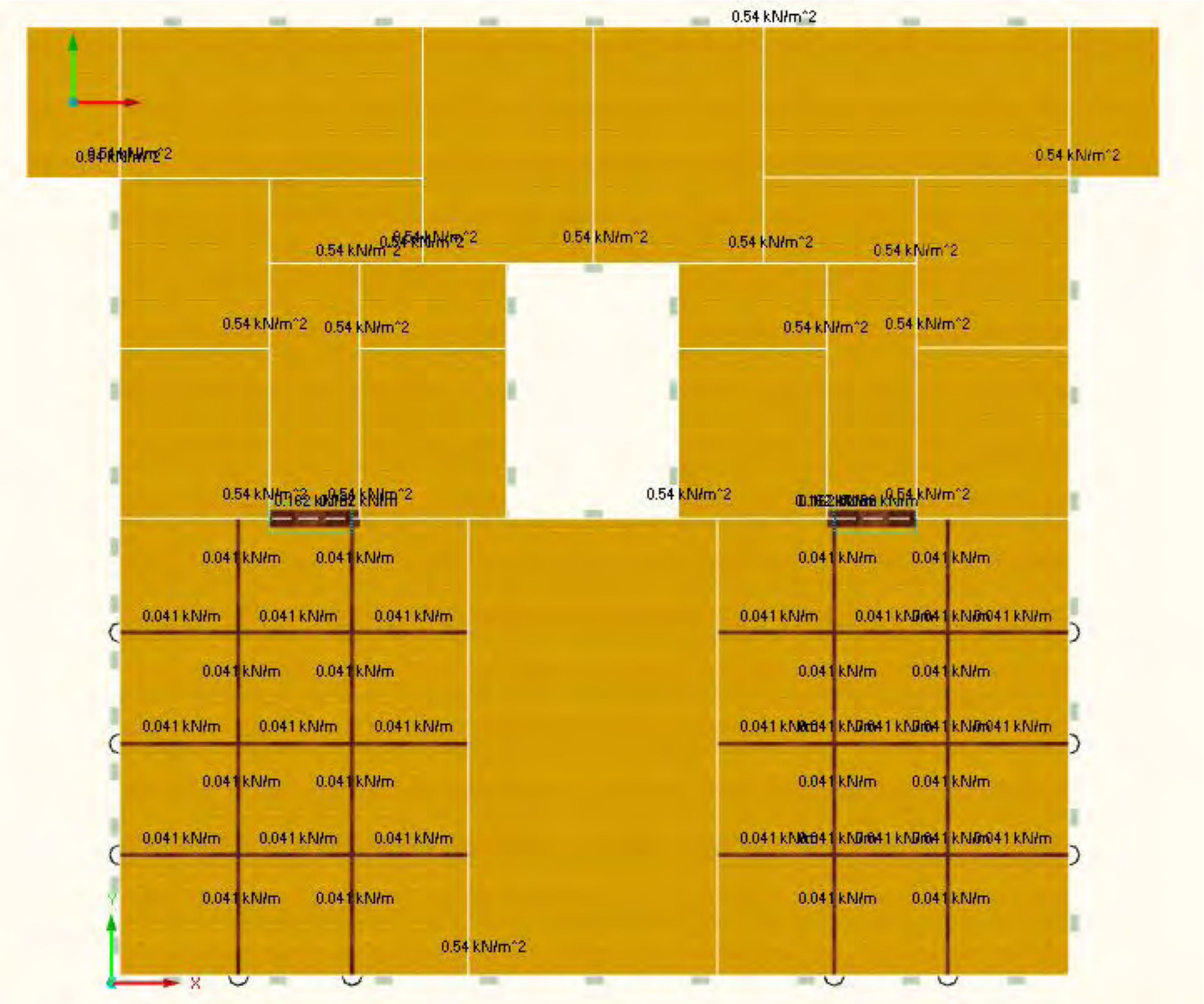


Figure 10.2.: Assign of Suction Wind Load on Typical Floor (KN-M Units)

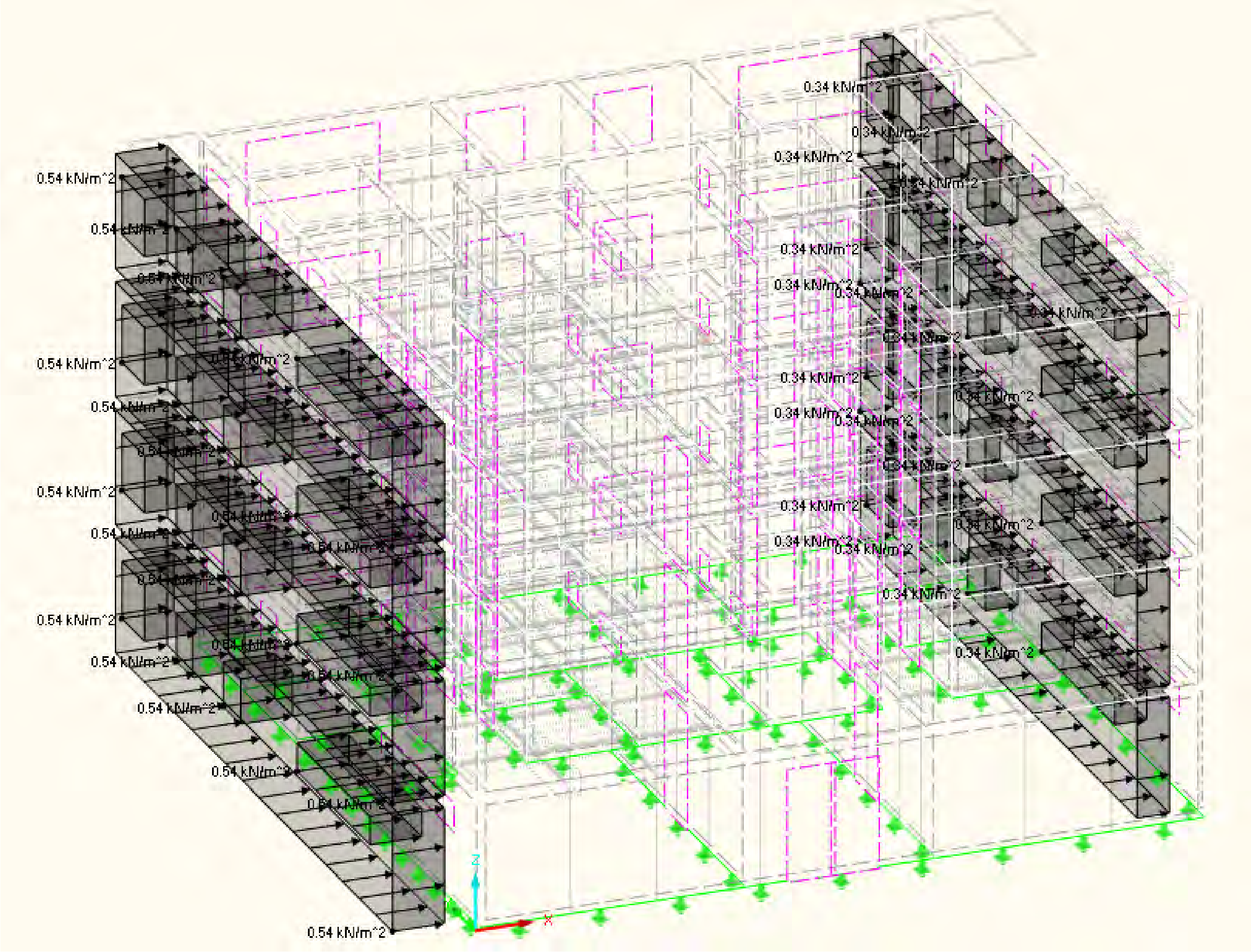


Figure 10.2.: Assign of Wind Load on Walls in x-direction (KN-M Units)

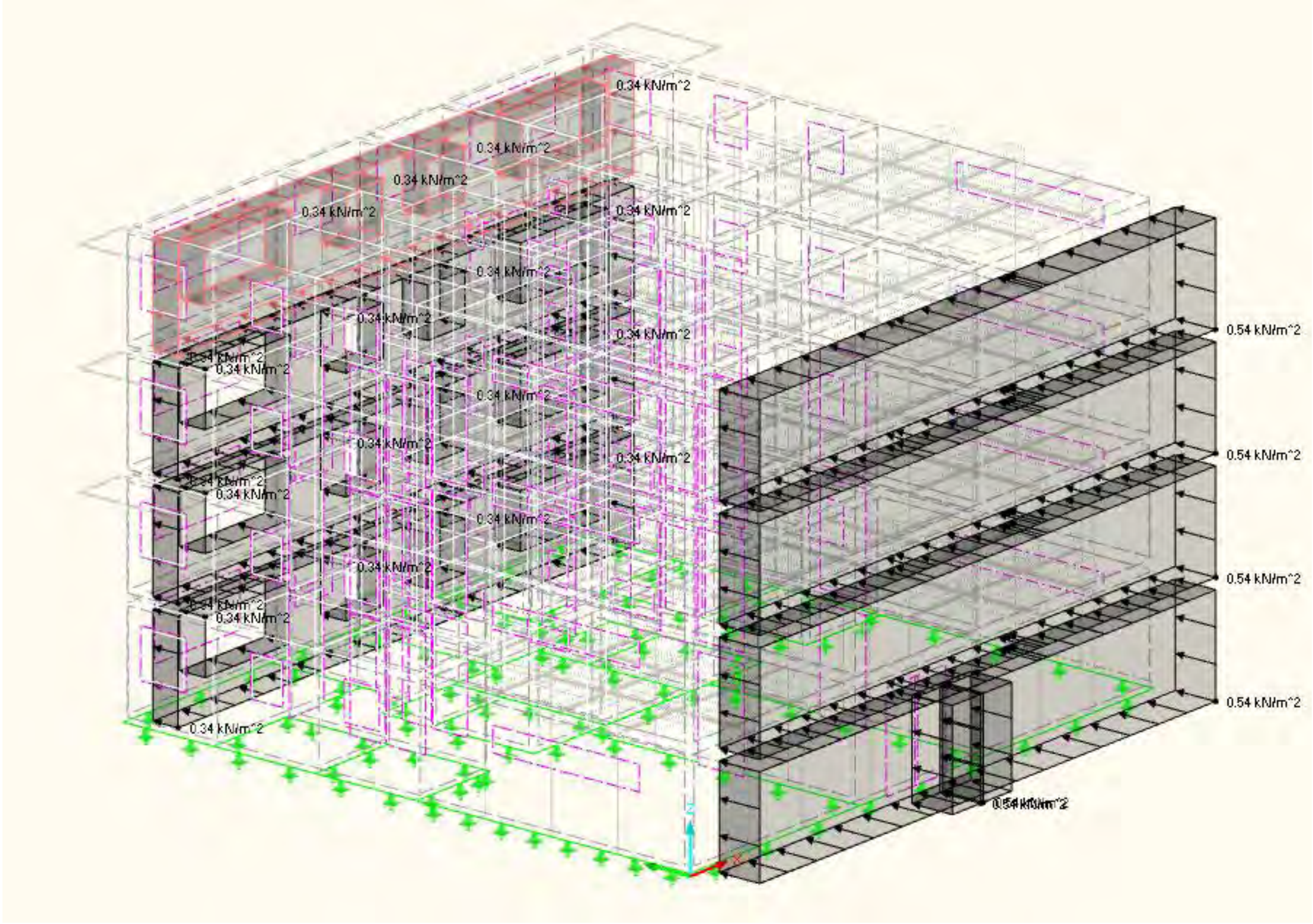
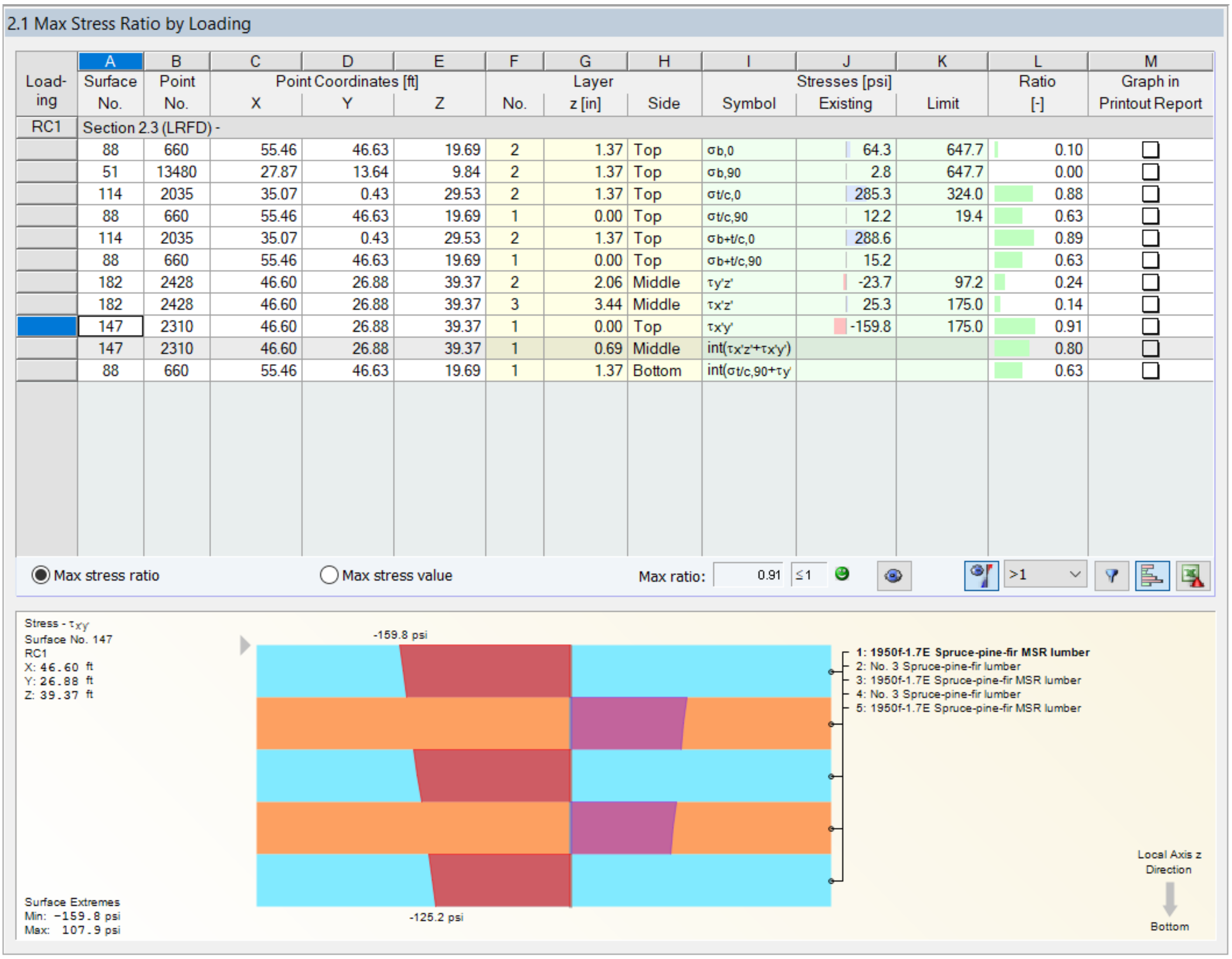


Figure 10.2.: Assign of Wind Load on Walls in y-direction (KN-M Units)

11. STRUCTURAL DESIGN

11.1 DESIGN OF CLT SLABS and WALLS

11.1.1 Max. Stress Ratio on CLT cross section



2.1 Max Stress Ratio by Loading

	A	B	C	D	E	F	G	H	I	J	K	L	M
Loading	Surface No.	Point No.	Point Coordinates [ft] X	Y	Z	Layer No.	z [in]	Side	Stresses [psi] Symbol	Existing	Limit	Ratio [-]	Graph in Printout Report
RC1	Section 2.3 (LRFD) -												
	88	660	55.46	46.63	19.69	2	1.37	Top	$\sigma_{b,0}$	64.3	647.7	0.10	☐
	51	13480	27.87	13.64	9.84	2	1.37	Top	$\sigma_{b,90}$	2.8	647.7	0.00	☐
	114	2035	35.07	0.43	29.53	2	1.37	Top	$\sigma_{t/c,0}$	285.3	324.0	0.88	☐
	88	660	55.46	46.63	19.69	1	0.00	Top	$\sigma_{t/c,90}$	12.2	19.4	0.63	☐
	114	2035	35.07	0.43	29.53	2	1.37	Top	$\sigma_{b+t/c,0}$	288.6		0.89	☐
	88	660	55.46	46.63	19.69	1	0.00	Top	$\sigma_{b+t/c,90}$	15.2		0.63	☐
	182	2428	46.60	26.88	39.37	2	2.06	Middle	$\tau_{y'z'}$	-23.7	97.2	0.24	☐
	182	2428	46.60	26.88	39.37	3	3.44	Middle	$\tau_{x'z'}$	25.3	175.0	0.14	☐
	147	2310	46.60	26.88	39.37	1	0.00	Top	$\tau_{x'y'}$	-159.8	175.0	0.91	☐
	147	2310	46.60	26.88	39.37	1	0.69	Middle	int($\tau_{x'z'}+\tau_{x'y'}$)			0.80	☐
	88	660	55.46	46.63	19.69	1	1.37	Bottom	int($\sigma_{t/c,90}+\tau_{y}$			0.63	☐

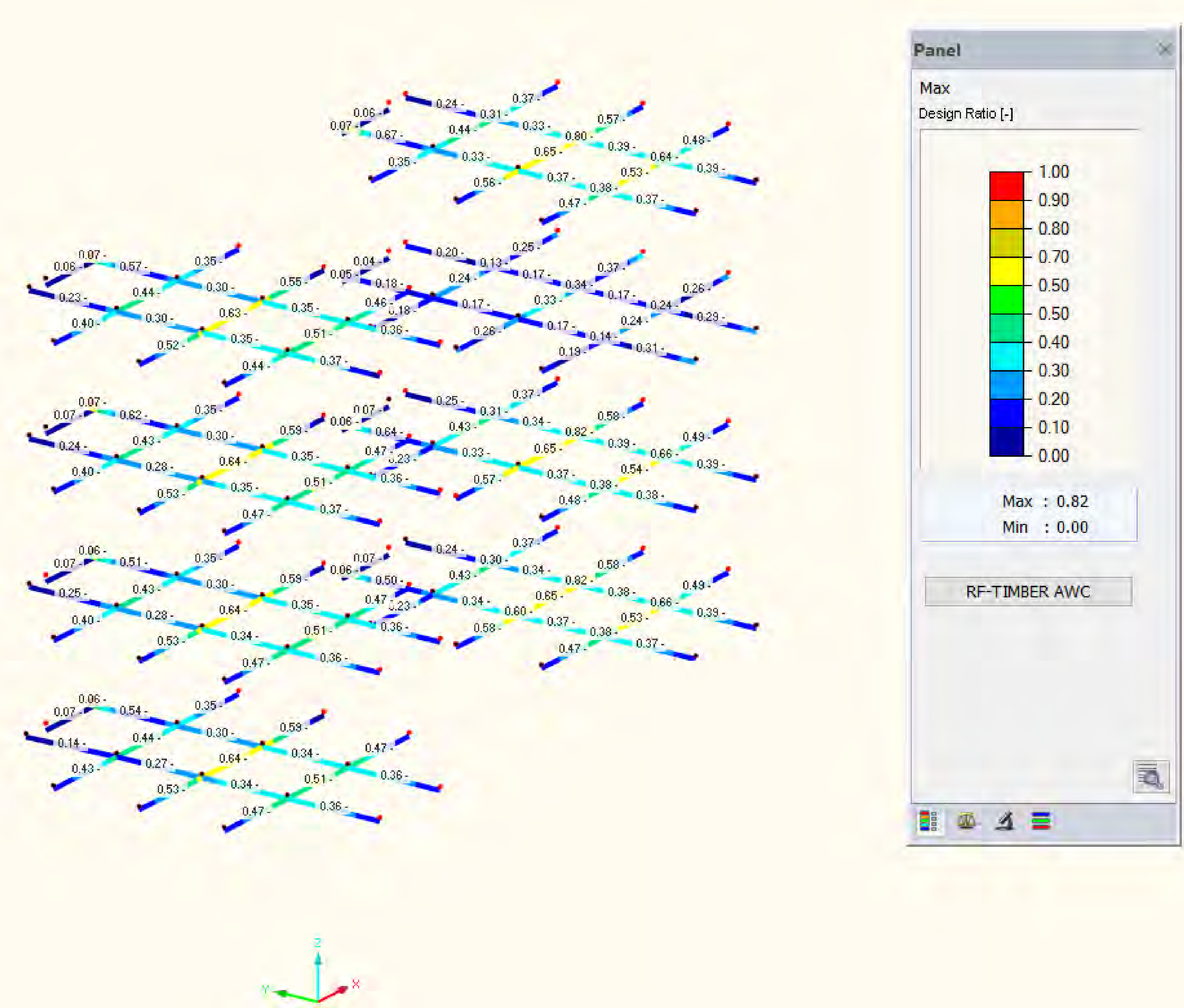
● Max stress ratio ○ Max stress value Max ratio: 0.91 ≤1

Stress - $\tau_{x'y'}$
Surface No. 147
RC1
X: 46.60 ft
Y: 26.88 ft
Z: 39.37 ft

1: 1950f-1.7E Spruce-pine-fir MSR lumber
2: No. 3 Spruce-pine-fir lumber
3: 1950f-1.7E Spruce-pine-fir MSR lumber
4: No. 3 Spruce-pine-fir lumber
5: 1950f-1.7E Spruce-pine-fir MSR lumber

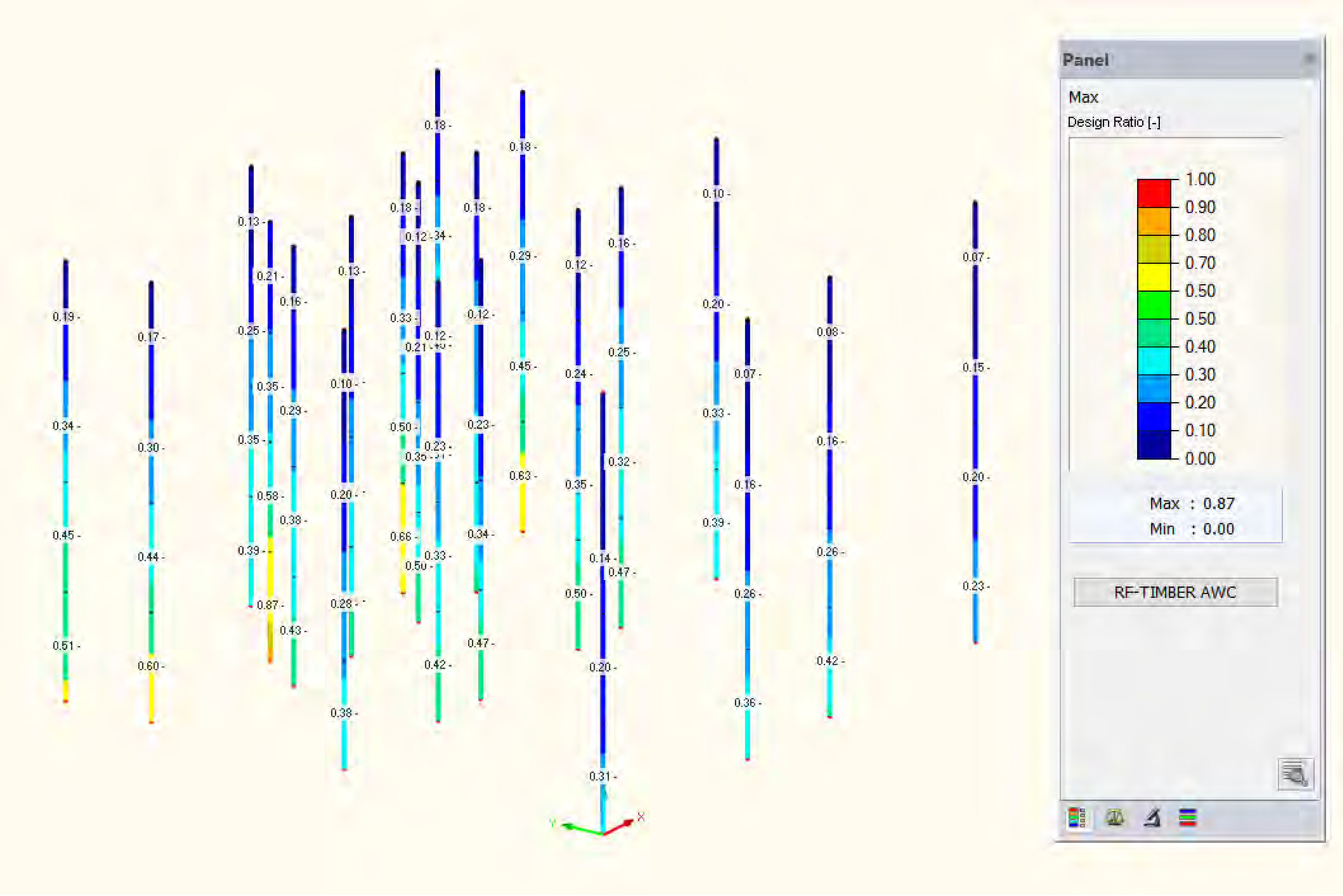
Surface Extremes
Min: -159.8 psi
Max: 107.9 psi

Max. Stress Ratio on LVL Beams



Max : 0.82
Min : 0.00

Max. Stress Ratio on Glulam Columns



Max : 0.87
Min : 0.00

11.1.2 Max. Stress Ratio on LVL & GLULAM cross sections

Load-ing	Description	Design		Design According to Formula
	Ultimate Limit State Design			
CO1	1.4*LC1 + 1.4*LC7	0.82	≤ 1	393) Stability - Biaxial bending with LTB and compression with buckling about both axes acc. to 3.9.2
CO2	1.2*LC1 + 1.6*LC2 + 1.2*LC7	0.87	≤ 1	303) Stability - Compression parallel to grain with buckling about both axes acc. to 3.6 and 3.7
CO3	1.2*LC1 + 0.5*LC3 + 1.2*LC7	0.52	≤ 1	303) Stability - Compression parallel to grain with buckling about both axes acc. to 3.6 and 3.7
CO4	1.2*LC1 + 0.5*LC4 + 1.2*LC7	0.51	≤ 1	393) Stability - Biaxial bending with LTB and compression with buckling about both axes acc. to 3.9.2
CO5	1.2*LC1 + LC2 + LC3 + 1.2*LC7	0.68	≤ 1	303) Stability - Compression parallel to grain with buckling about both axes acc. to 3.6 and 3.7
CO6	1.2*LC1 + LC2 + LC4 + 1.2*LC7	0.62	≤ 1	303) Stability - Compression parallel to grain with buckling about both axes acc. to 3.6 and 3.7
CO7	1.2*LC1 + LC3 + 1.2*LC7	0.49	≤ 1	303) Stability - Compression parallel to grain with buckling about both axes acc. to 3.6 and 3.7
CO8	1.2*LC1 + LC4 + 1.2*LC7	0.44	≤ 1	303) Stability - Compression parallel to grain with buckling about both axes acc. to 3.6 and 3.7
CO9	1.2*LC1 + LC5 + 1.2*LC7	0.52	≤ 1	303) Stability - Compression parallel to grain with buckling about both axes acc. to 3.6 and 3.7
CO10	1.2*LC1 + LC2 + LC5 + 1.2*LC7	0.70	≤ 1	303) Stability - Compression parallel to grain with buckling about both axes acc. to 3.6 and 3.7
CO11	0.9*LC1 + LC3 + 0.9*LC7	0.36	≤ 1	303) Stability - Compression parallel to grain with buckling about both axes acc. to 3.6 and 3.7
CO12	0.9*LC1 + LC4 + 0.9*LC7	0.31	≤ 1	303) Stability - Compression parallel to grain with buckling about both axes acc. to 3.6 and 3.7
CO13	0.9*LC1 + LC5 + 0.9*LC7	0.39	≤ 1	303) Stability - Compression parallel to grain with buckling about both axes acc. to 3.6 and 3.7
	Serviceability Limit State Design			
CO14	LC1 + LC7	0.09	≤ 1	401) Serviceability - Deflection in z/y-direction (Beam)
CO15	LC1 + LC2 + LC7	0.10	≤ 1	401) Serviceability - Deflection in z/y-direction (Beam)
CO16	LC1 + 0.7*LC5 + LC7	0.09	≤ 1	401) Serviceability - Deflection in z/y-direction (Beam)
CO17	LC1 + 0.6*LC3 + LC7	0.09	≤ 1	401) Serviceability - Deflection in z/y-direction (Beam)
CO18	LC1 + 0.6*LC4 + LC7	0.09	≤ 1	401) Serviceability - Deflection in z/y-direction (Beam)
CO19	LC1 + 0.75*LC2 + 0.45*LC3 + LC7	0.10	≤ 1	401) Serviceability - Deflection in z/y-direction (Beam)
CO20	LC1 + 0.75*LC2 + 0.45*LC4 + LC7	0.10	≤ 1	401) Serviceability - Deflection in z/y-direction (Beam)
CO21	LC1 + 0.75*LC2 + 0.52*LC5 + LC7	0.10	≤ 1	401) Serviceability - Deflection in z/y-direction (Beam)
CO22	0.6*LC1 + 0.6*LC3 + 0.6*LC7	0.05	≤ 1	401) Serviceability - Deflection in z/y-direction (Beam)
CO23	0.6*LC1 + 0.6*LC4 + 0.6*LC7	0.05	≤ 1	401) Serviceability - Deflection in z/y-direction (Beam)
CO24	0.6*LC1 + 0.7*LC5 + 0.6*LC7	0.05	≤ 1	401) Serviceability - Deflection in z/y-direction (Beam)

11.1.3 Stair Design

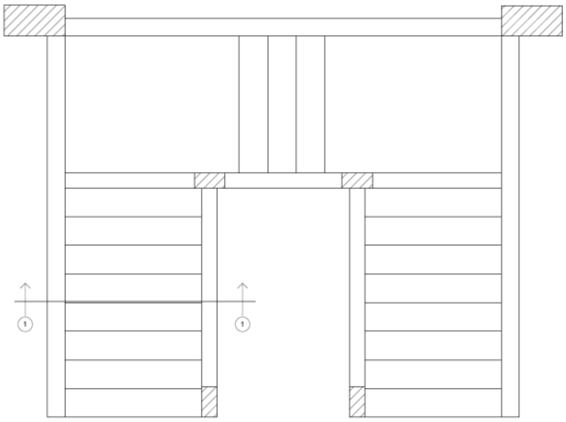
For a 5-layer, E1 panel:

h_i = Thickness of an individual layer = 1 3/8 in.
b = Design width = 12 in.

Major strength axis (parallel to grain)
$F_{b,0}$ = Bending strength = 1950 psi
E_0 = Modulus of elasticity = $1.7\text{x}10^6$ psi
$F_{t,0}$ = Tensile strength = 1375 psi
$F_{c,0}$ = Compression strength = 1800 psi
$F_{v,0}$ = Shear strength = 135 psi
$F_{s,0}$ = Rolling shear strength = 45 psi

Minor strength axis (perpendicular to grain)
$F_{b,90}$ = Bending strength = 500 psi
E_0 = Modulus of elasticity = $1.2\text{x}10^6$ psi
$F_{v,0}$ = Shear strength = 135 psi
$F_{s,0}$ = Rolling shear strength = 45 psi

Stair section (E1, 5-layers CLT)



L.L= 3 KN/m’, F.C=1 KN/m’

According to ASCE-7.15:-

W_u= 1.2(5x0.168+1)+1.6x3= 7 KN/m^2

M_u= $(7\text{x}1.4^2)/8$ = 1.72 KN.m

V = 4.9 KN

1. Check on flexural strength.

F_bS_{eff}’= 1x2.54x0.85x10400x0.6= 13472.16 Ib.ft= 18.7 KN.m (NDS- chapter 10 & PRG320-18)

$M_u < F_bS_{eff}$’ (Safe)

2. Check on shear strength.

$F_sIbQ_{eff}' = 1x1x2441 = 2441$ Ib= 11.1 KN. (NDS- chapter 10 & PRG320-18)

$V < F_sIbQ_{eff}'$ (Safe)

3. Check on vibrations.

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff}L^2}}$$

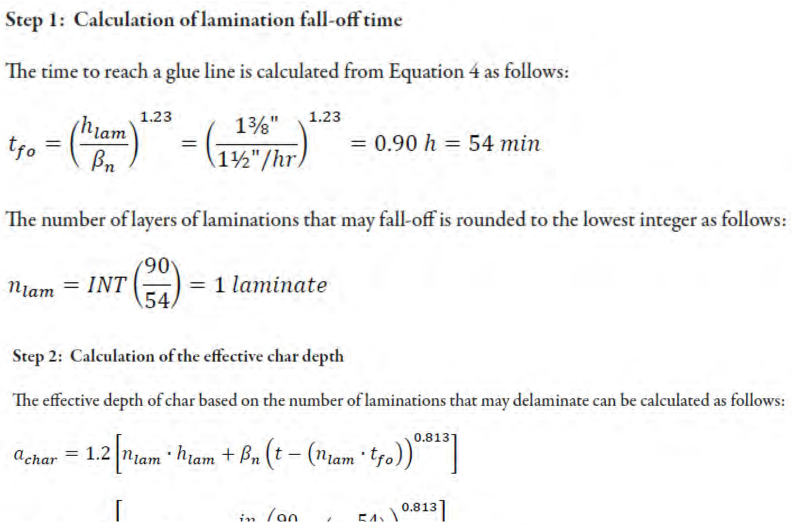
$$f = \frac{2.188}{2L^2}\sqrt{\frac{EI_{app}}{\rho A}} \geq 9.0\ Hz$$

$$F = \frac{2.188}{2x4.5^2} \text{ x } \sqrt{\frac{1.6x10^6}{1.0625x6.625x12}} = 11.83 \text{ HZ} > 9 \text{ HZ}$$

(Safe)

4. Check on fire resistance.

According to IBC, the required fire resistance is 90 min. (Type V construction).



Step 1: Calculation of lamination fall-off time

The time to reach a glue line is calculated from Equation 4 as follows:

$$t_{fo} = \left(\frac{h_{lam}}{\beta_n}\right)^{1.23} = \left(\frac{1⅜"}{1½"/hr}\right)^{1.23} = 0.90\ h = 54\ min$$

The number of layers of laminations that may fall-off is rounded to the lowest integer as follows:

$$n_{lam} = INT\left(\frac{90}{54}\right) = 1\ laminate$$

Step 2: Calculation of the effective char depth

The effective depth of char based on the number of laminations that may delaminate can be calculated as follows:

$$a_{char} = 1.2\left[n_{lam} \cdot h_{lam} + \beta_n \left(t - (n_{lam} \cdot t_{fo})\right)^{0.813}\right]$$

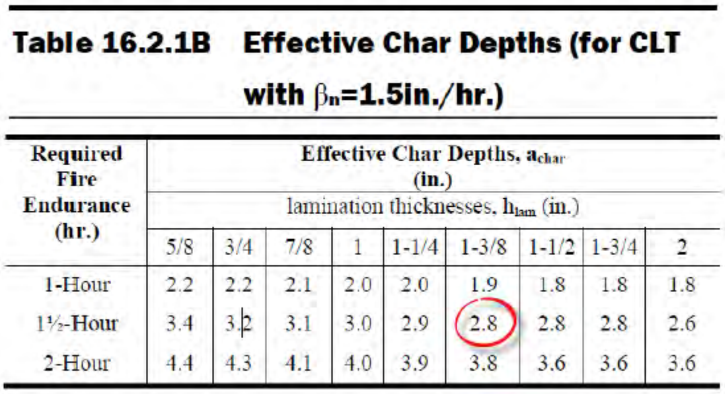


Table 16.2.1B Effective Char Depths (for CLT with β_n=1.5in./hr.)

Required Fire Endurance (hr.)	Effective Char Depths, a_{char} (in.)								
	lamination thicknesses, h_{lam} (in.)								
	5/8	3/4	7/8	1	1-1/4	1-3/8	1-1/2	1-3/4	2
1-Hour	2.2	2.2	2.1	2.0	2.0	1.9	1.8	1.8	1.8
1½-Hour	3.4	3.2	3.1	3.0	2.9	2.8	2.8	2.8	2.6
2-Hour	4.4	4.3	4.1	4.0	3.9	3.8	3.6	3.6	3.6

6 2018 Code Conforming Wood Design

2. Type of Construction

Chapter 6 of the IBC defines types of construction, with wood frame construction typically found in Type V, IV and III. Additionally, the IBC has specific applications that permit the use of wood in construction in Type I and II. These circumstances will be addressed in Sections 5 and 6 of this book.

Type V Construction

Type V construction permits the use of wood or other approved materials for structural elements, including structural frame members, bearing walls, floor and roof construction, as well as nonbearing elements such as exterior walls and interior partitions. Type V construction is further defined as Type VA (all interior and exterior load-bearing walls, floors, roofs and all structural members are designed or protected to provide a minimum 1-hour fire-resistance rating) and Type VB (no fire-resistance rating is required).

Figure 10: Type V Construction

Type IV Construction

Type IV construction (Heavy Timber, HT) has exterior walls made of noncombustible materials, fire-retardant-treated wood (FRTW), or cross-laminated timber (CLT) protected in accordance with Section 602.4.2. Interior building elements must be of solid or laminated wood without concealed spaces (for partitions, see below). Columns supporting roof and ceiling loads must be a minimum nominal dimension of 6 inches by 8 inches and 8 inches by 8 inches if supporting floor loads. Floor beams and girders must be a minimum nominal dimension of 6 inches by 10 inches, and roof beams and girders must be a minimum nominal dimension of 4 inches by 6 inches. Flooring must be a minimum nominal 3-inch thickness covered with 1-inch nominal dimension tongue-and-groove flooring or 4-inch-thick cross-laminated timber (CLT). Roof decking must be a minimum nominal 2-inch thickness, 1¹/₈-inch-thick wood structural panels, or 3-inch-thick CLT. Partitions must be 1-hour fire-resistance-rated construction or a minimum two layers of 1-inch nominal board or laminated construction 4 inches thick.

Figure 11: Type IV Construction

Type III Construction

Type III construction requires exterior walls to be noncombustible material or FRTW having a minimum 2-hour fire-resistance rating. All of the other building elements are permitted to be wood or other approved materials. Type IIIA construction needs to provide a minimum 1-hour fire-resistance rating for all building elements other than nonbearing walls, and Type IIIB construction does not require any fire-resistance rating other than the exterior load-bearing wall.

Figure 12: Type III Construction

American Wood Council International Code Council

$$\bar{y} = \frac{\sum_i \tilde{y}_i h_i}{\sum_i h_i} = \frac{\left(\frac{1.375}{2} \times 1.375\right) + (3.393 \times 1.285)}{1.375 + 1.285} = 1.994\, in.$$

$$\begin{aligned}
I_{eff} &= \sum_i \frac{b_i h_i^3}{12} + \sum_i b_i h_i d_i^2 \\
&= \left(\frac{12 \cdot (1.375)^3}{12}\right) + \left(\frac{12 \cdot (1.285)^3}{12}\right) \\
&\quad + \left(12 \cdot 1.375 \cdot \left(1.994 - \frac{1.375}{2}\right)^2\right) \\
&\quad + (12 \cdot 1.375 \cdot (3.393 - 1.994)^2) = 63.1\, \frac{in.^4}{ft.}
\end{aligned}$$

$S_{eff} = \frac{66.2}{4.035 - 2.019} = 32.9\ in^3/ft$

M = 83686 Ib.ft/ft

M_u= 1241.5 Ib.ft/ft

$M > M_u$ (Safe)

11.1.4 Connection Design

1- Half-Lapped Connection

7-ply grade v_3 CLT (seven times 1.375 in. plies= 9.625, specific gravity ‘G’= 0.5, 0.5 in. Lag Screw, root diameter ‘D_r’= 0.371 in., Lag Screw length= 9 in., Tip Length (E)= 0.3125 in.

$F_{e'0'}$= 11200 * 0.5 = 5600 psi

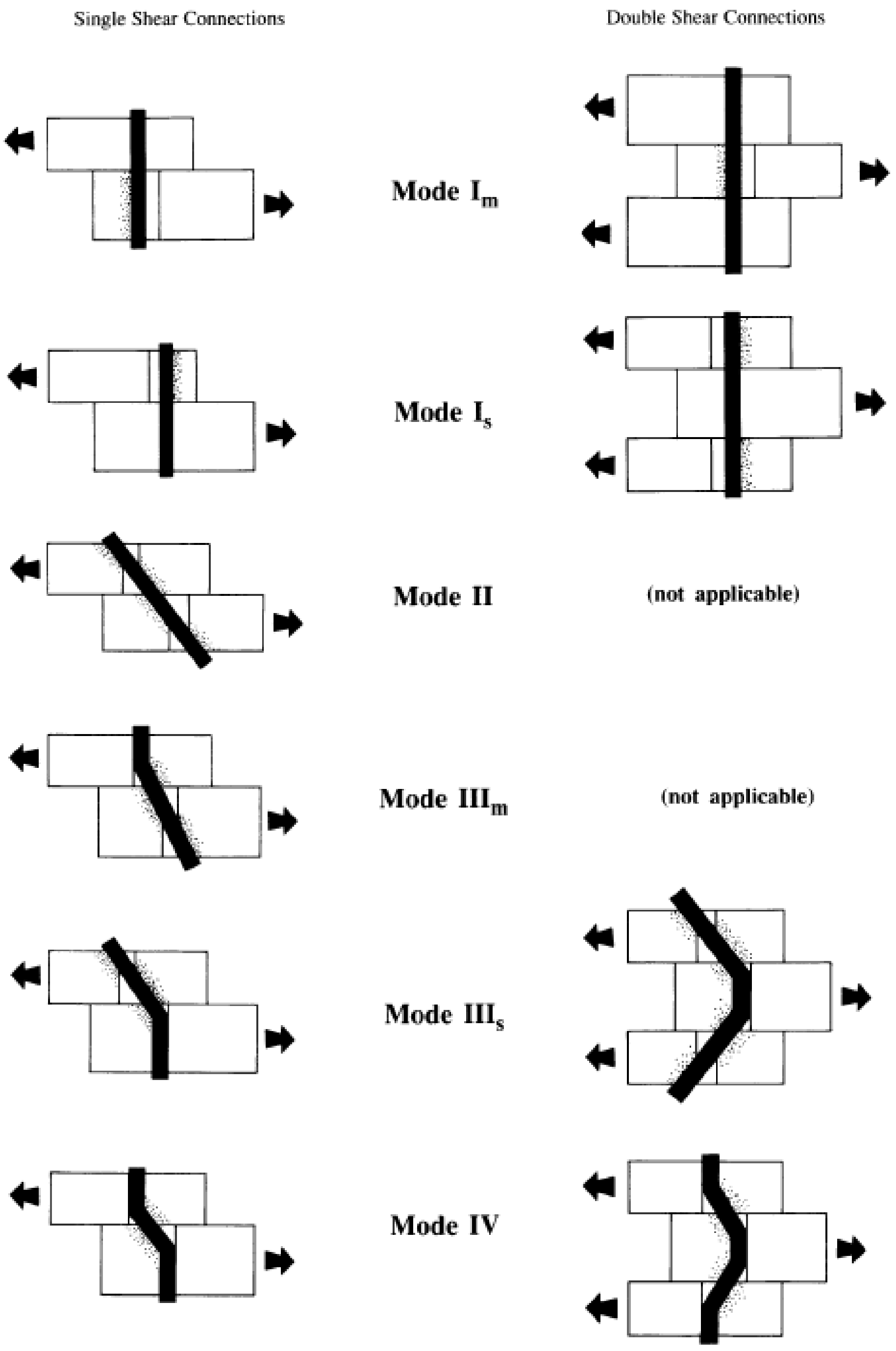
$F_{e'90'}$= 6100 * $0.5^{1.45}$ * $0.5^{-0.5}$ = 3157 psi

a. Bearing length of the lap joint:-
  - L_s= 3 * 1.375 + 0.5 * 1.375 = 4.81 in.
  - L_m= ((9 – 4.81) - $\frac{0.3125}{2}$) = 4.033 in.

b. Check on wood crashing in the side length:-
  - $P_{min.}$ = (9 – 4.81) – 0.3125 = 3.877 in. > 4*0.5= 2 in. = (Safe)

c. Adjusted bearing length for lateral calculations:-
  - $L_{s\text{-}adj.}$ = ((2 * 1.375 * $\frac{5600}{3157}$) + 1.375 + $\frac{1.375}{2}$) = 6.94 in.
  - $L_{m\text{-}adj.}$ = ($\frac{1.375}{2}$ + 1.375 + ((1.375 + 0.75 - $\frac{0.3125}{2}$) * $\frac{5600}{3157}$) + 1.375) = 6.92 in.

d. Calculation of ASD adjusted design values using NDS yield limit equations:-



- Mode $\mathrm{I}_{\mathbf{m}}$: $Z= \frac{0.37*6.92*3157}{5} = 1616.63$ Ib
- Mode $\mathrm{I}_{\mathbf{s}}$: $Z= \frac{0.37*6.94*3157}{5} = 1621.3$ Ib

- $K_1= \frac{\sqrt{1+2*1^2*(1+0.99+0.99^2)+0.99^2+1^2}-(1*1.99)}{1+1} = 0.412$
- $K_2= 1.036$
- $K_3= 1.036$

- Mode II: $Z= \frac{0.412*6.94*0.37*3157}{4.5} = 742$ Ib
- Mode $\mathrm{III}_{\mathbf{m}}$: $Z= \frac{1.036*0.37*6.92*3157}{(2+1)*4} = 697.9$ Ib
- Mode $\mathrm{III}_{\mathbf{s}}$: $Z= \frac{1.036*0.37*6.94*3157}{(2+1)*4} = 699.9$ Ib
- Mode IV: $Z=\frac{0.37^2}{4} * \sqrt{\frac{2*3157*45000}{3*2}} = 235.521$ Ib

Using the bearing length, mode IV still control and Z90= 235.52 Ib

$Z_{90\text{-adj}}$.= 1.6 * 235= 376 Ib

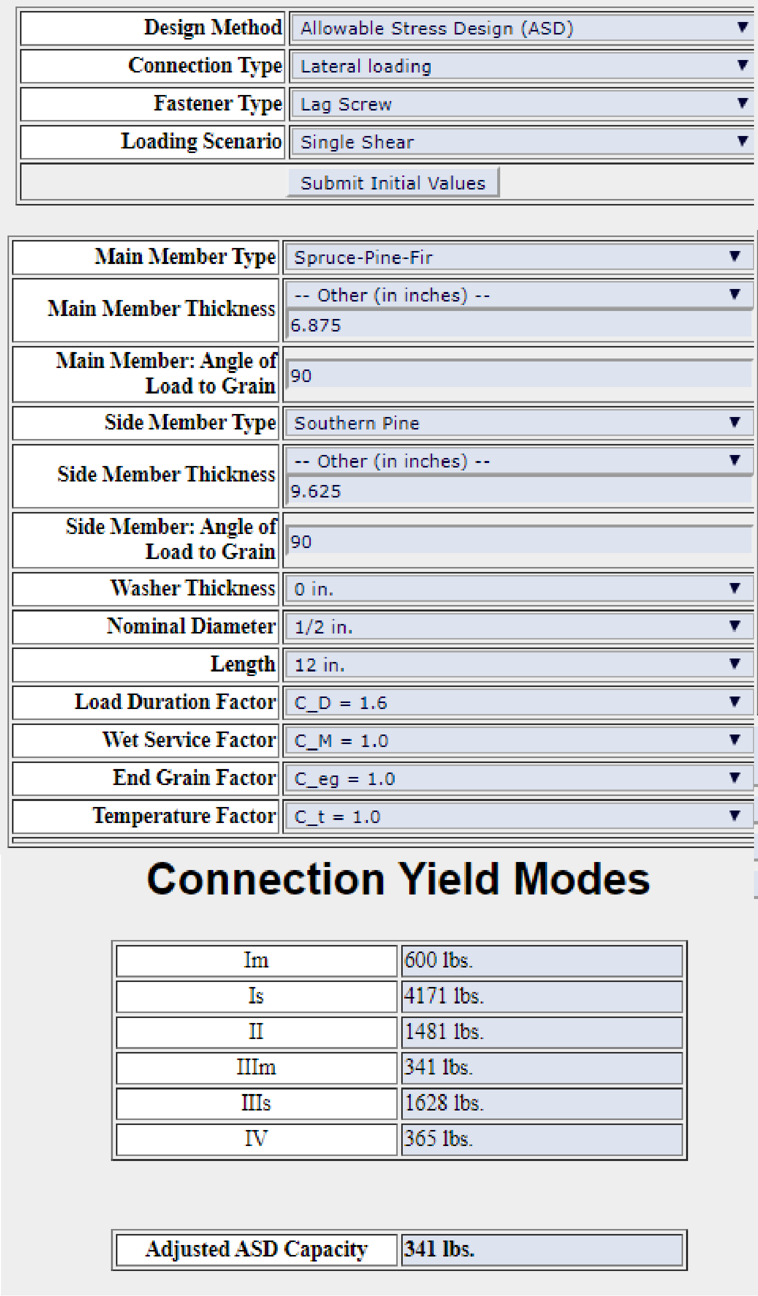
$V_{y\text{-max.=}}$ 2.186 KN/m$^{/}$= 481 Ib/m$^{/}$ (From Analysis)

Spacing= $\frac{376}{481}$= 0.78 m = 0.7 m

Edge distance= 4 * 0.5= 2 in.= 5 cm (According to NDS)

End distance= 0.45 m

2. Wall intersection connection.



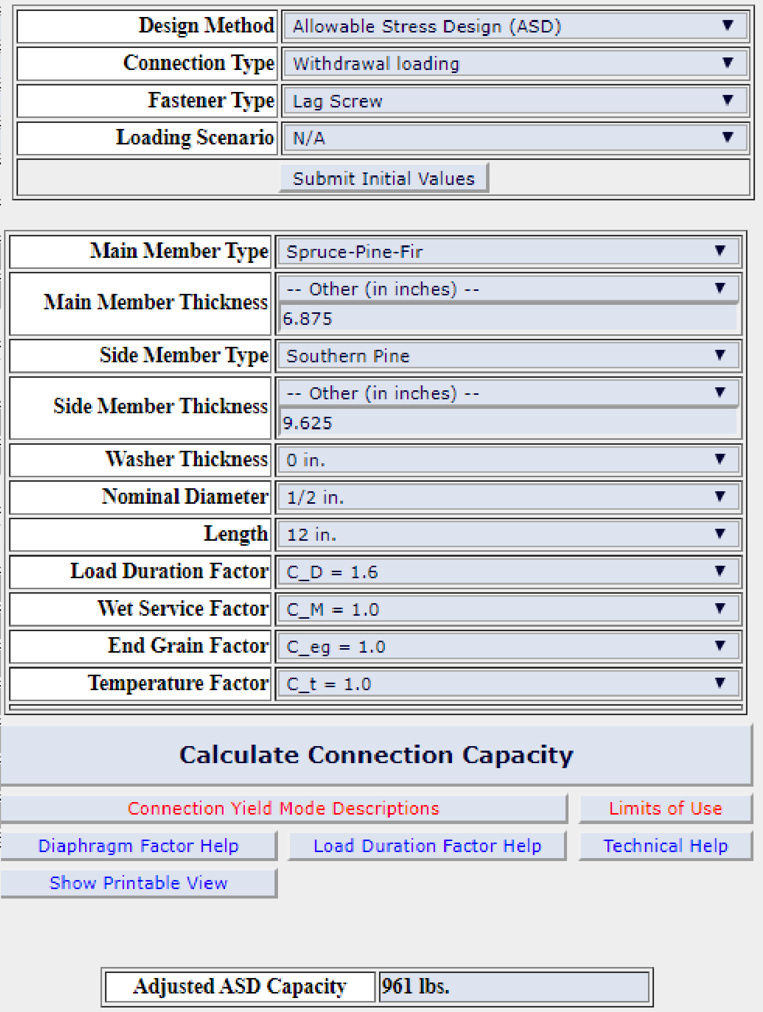
Design Method	Allowable Stress Design (ASD)
Connection Type	Lateral loading
Fastener Type	Lag Screw
Loading Scenario	Single Shear

Main Member Type	Spruce-Pine-Fir
Main Member Thickness	-- Other (in inches) -- 6.875
Main Member: Angle of Load to Grain	90
Side Member Type	Southern Pine
Side Member Thickness	-- Other (in inches) -- 9.625
Side Member: Angle of Load to Grain	90
Washer Thickness	0 in.
Nominal Diameter	1/2 in.
Length	12 in.
Load Duration Factor	C_D = 1.6
Wet Service Factor	C_M = 1.0
End Grain Factor	C_eg = 1.0
Temperature Factor	C_t = 1.0

Connection Yield Modes

Im	600 lbs.
Is	4171 lbs.
II	1481 lbs.
IIIm	341 lbs.
IIIs	1628 lbs.
IV	365 lbs.

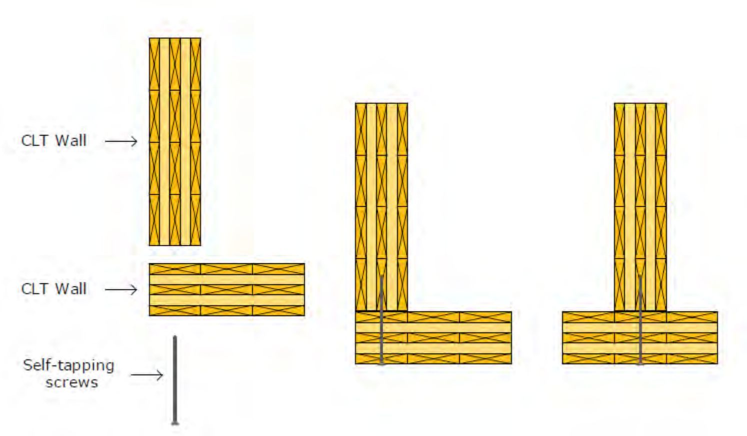
Adjusted ASD Capacity	341 lbs.



Design Method	Allowable Stress Design (ASD)
Connection Type	Withdrawal loading
Fastener Type	Lag Screw
Loading Scenario	N/A

Main Member Type	Spruce-Pine-Fir
Main Member Thickness	-- Other (in inches) -- 6.875
Side Member Type	Southern Pine
Side Member Thickness	-- Other (in inches) -- 9.625
Washer Thickness	0 in.
Nominal Diameter	1/2 in.
Length	12 in.
Load Duration Factor	C_D = 1.6
Wet Service Factor	C_M = 1.0
End Grain Factor	C_eg = 1.0
Temperature Factor	C_t = 1.0

Adjusted ASD Capacity	961 lbs.



- Withdrawal force= 6.985 ken/m'
- Withdrawal capacity= 4.3 kN/screw
- Number of screws$_{w}$= 7/4.3= 2 screw/m'
- Lateral force = 2.1 kN/m'
- Lateral capacity= 1.55 kN/screw
- Number of screws$_{L}$= 2.1/1.55= 2screw/m'

3. Floor to Wall connection.

Straining actions at connection From FE Analysis;-

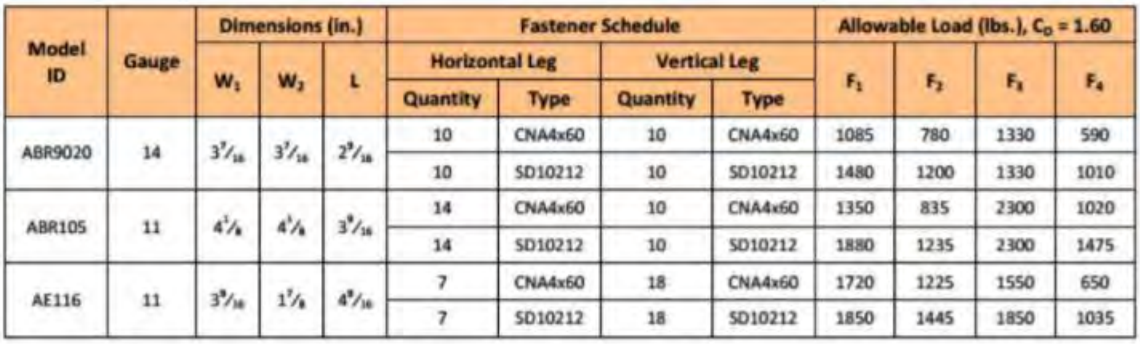
F1= 1.2 kN/m’

F2= 0.9 kN/m’

F3= 3.3 kN/m’

F4= 9.5 kN/m’

Connection capacity according to Simpson Strong Tie bracket design information:-

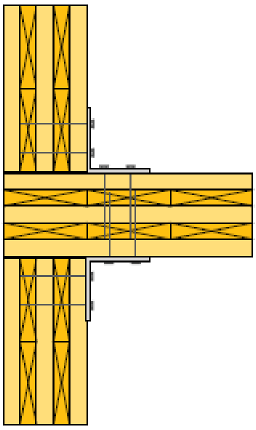


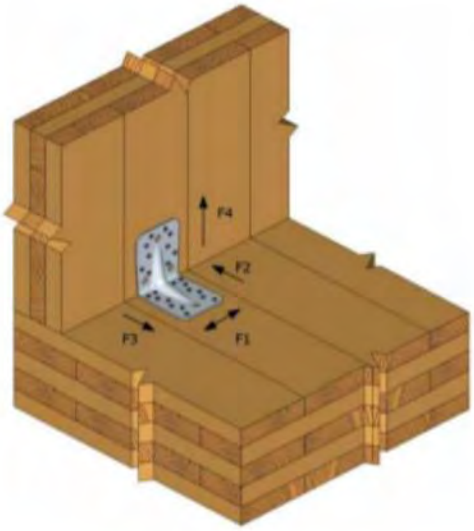
Model ID	Gauge	Dimensions (in.)			Fastener Schedule				Allowable Load (lbs.), C_D = 1.60			
					Horizontal Leg		Vertical Leg					
		W_1	W_2	L	Quantity	Type	Quantity	Type	F_1	F_2	F_3	F_4
ABR9020	14	$3\frac{7}{16}$	$3\frac{7}{16}$	$2\frac{9}{16}$	10	CNA4x60	10	CNA4x60	1085	780	1330	590
					10	SD10212	10	SD10212	1480	1200	1330	1010
ABR105	11	$4\frac{1}{8}$	$4\frac{1}{8}$	$3\frac{9}{16}$	14	CNA4x60	10	CNA4x60	1350	835	2300	1020
					14	SD10212	10	SD10212	1880	1235	2300	1475
AE116	11	$3\frac{9}{16}$	$1\frac{7}{8}$	$4\frac{9}{16}$	7	CNA4x60	18	CNA4x60	1720	1225	1550	650
					7	SD10212	18	SD10212	1850	1445	1850	1035

According to using ABR9020 bracket and Wall length is 4.5m:-

Bracket capacity= 2.68 kN

The Number of brackets= 9.5/2.86= 4 bracket





4- Wall to footing connection (shear resistance connection):-

- By using 6 brackets along the wall TITAN TCN240Angle Bracket along the wall.
- L_{wall}= 16.5m

Straining actions at connection From FE Analysis;-

- V_{wall}= 6.125 KN/m'
- $F_{bracket}$= (16.5/6)x6.125= 16.9 KN.

CONFIGURATION

• uncracked concrete

• fixing on concrete: VINYLPRO M12 x 130 (steel grade 5.8) anchors installed internally (IN)

• fixing on timber: LBS Ø5 x 50 screws

$$R_d = \min\begin{cases} \dfrac{R_{2/3,k\ timber} \cdot k_{mod}}{\gamma_m} \\ \dfrac{R_{2/3,k\ cls}}{\gamma_{cls}} \end{cases}$$

$R_{V\ 2/3,k\ cls}$ = 25 kN

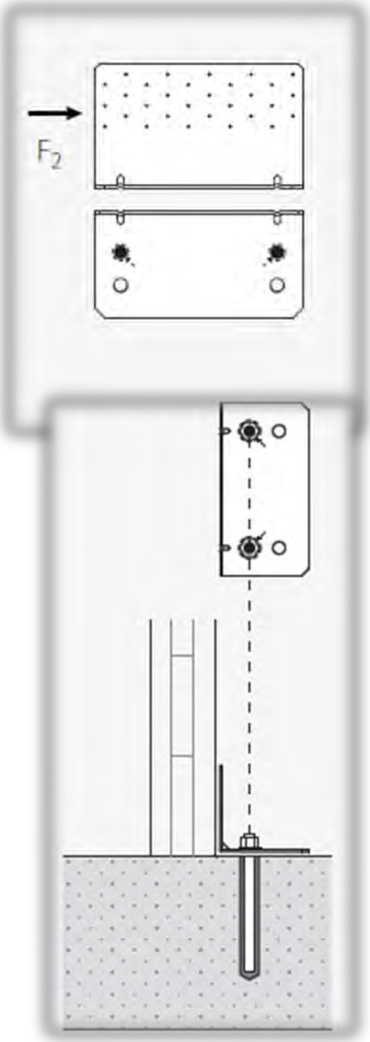
γ_{cls} = 1,25

$R_{V\ 2/3,k\ timber}$ = 26,5 kN

EN 1995:2008

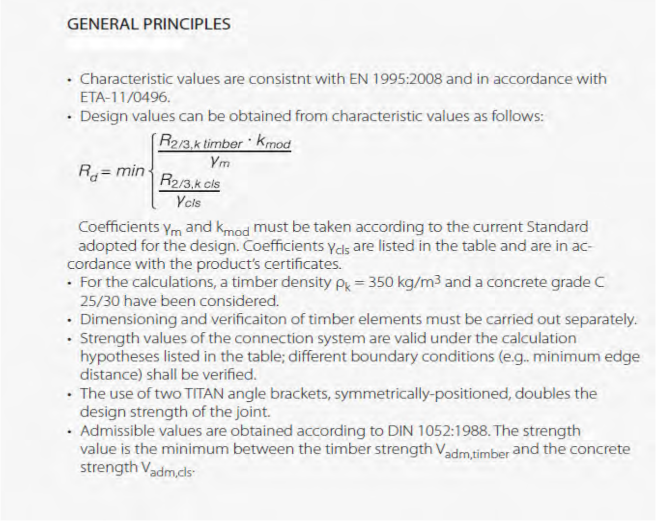
k_{mod} = 1,1

γ_m = 1,3



R_d = min { 20 ; 22.4 } = 20 Kn

$R_d \geq F_{bracket}$: 20 > 16,9 kN (Safe)



GENERAL PRINCIPLES

- Characteristic values are consistnt with EN 1995:2008 and in accordance with ETA-11/0496.
- Design values can be obtained from characteristic values as follows:

$$R_d = \min\begin{cases} \dfrac{R_{2/3,k\ timber} \cdot k_{mod}}{\gamma_m} \\ \dfrac{R_{2/3,k\ cls}}{\gamma_{cls}} \end{cases}$$

Coefficients γ_m and k_{mod} must be taken according to the current Standard adopted for the design. Coefficients γ_{cls} are listed in the table and are in accordance with the product's certificates.

- For the calculations, a timber density ρ_k = 350 kg/m³ and a concrete grade C 25/30 have been considered.
- Dimensioning and verificaiton of timber elements must be carried out separately.
- Strength values of the connection system are valid under the calculation hypotheses listed in the table; different boundary conditions (e.g.. minimum edge distance) shall be verified.
- The use of two TITAN angle brackets, symmetrically-positioned, doubles the design strength of the joint.
- Admissible values are obtained according to DIN 1052:1988. The strength value is the minimum between the timber strength $V_{adm,timber}$ and the concrete strength $V_{adm,cls}$.

5- Wall to footing connection (pull through resistance connection):-

- By using 2 WHT540 Angle brackets (total nailing anchor M20washer WHTBS50L- LBA Nails) at the 2 ends of the wall

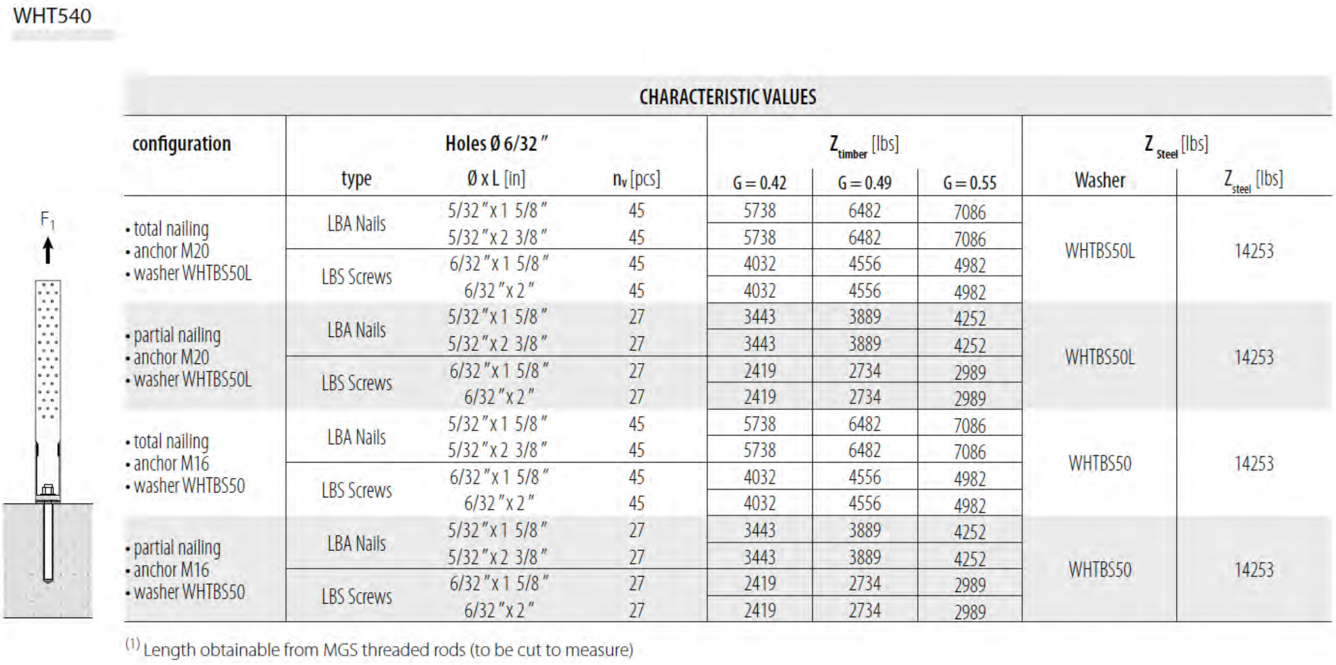
Straining actions at connection From FE Analysis;-

- T_{wall}= 26.73 KN=5880.6 Ib

pull through connection resistance:-

G= 0.55 Z_{timber}=7066 Ib Z_{steel}=14253 Ib

Twall < { Z_{timber}=7066 Ib ; Z_{steel}=14253 Ib } (Safe)



WHT540

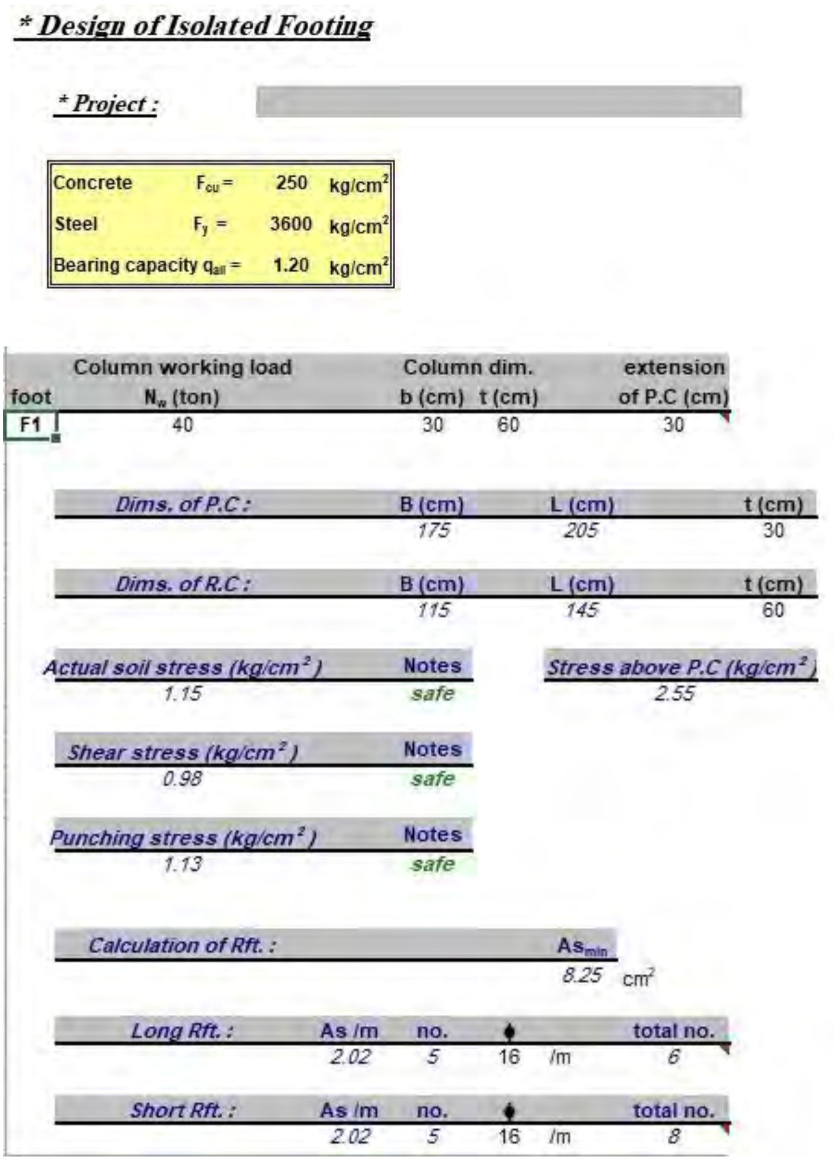
CHARACTERISTIC VALUES								
configuration	Holes Ø 6/32"			Z_{timber} [lbs]			Z_{Steel} [lbs]	
	type	Ø x L [in]	n_V [pcs]	G = 0.42	G = 0.49	G = 0.55	Washer	Z_{steel} [lbs]
• total nailing • anchor M20 • washer WHTBS50L	LBA Nails	5/32" x 1 5/8"	45	5738	6482	7086	WHTBS50L	14253
		5/32" x 2 3/8"	45	5738	6482	7086		
	LBS Screws	6/32" x 1 5/8"	45	4032	4556	4982		
		6/32" x 2"	45	4032	4556	4982		
• partial nailing • anchor M20 • washer WHTBS50L	LBA Nails	5/32" x 1 5/8"	27	3443	3889	4252	WHTBS50L	14253
		5/32" x 2 3/8"	27	3443	3889	4252		
	LBS Screws	6/32" x 1 5/8"	27	2419	2734	2989		
		6/32" x 2"	27	2419	2734	2989		
• total nailing • anchor M16 • washer WHTBS50	LBA Nails	5/32" x 1 5/8"	45	5738	6482	7086	WHTBS50	14253
		5/32" x 2 3/8"	45	5738	6482	7086		
	LBS Screws	6/32" x 1 5/8"	45	4032	4556	4982		
		6/32" x 2"	45	4032	4556	4982		
• partial nailing • anchor M16 • washer WHTBS50	LBA Nails	5/32" x 1 5/8"	27	3443	3889	4252	WHTBS50	14253
		5/32" x 2 3/8"	27	3443	3889	4252		
	LBS Screws	6/32" x 1 5/8"	27	2419	2734	2989		
		6/32" x 2"	27	2419	2734	2989		

(1) Length obtainable from MGS threaded rods (to be cut to measure)

11.1.5 Foundations Design

Isolated Footing Design

Isolated footing is considered under columns by rebars Intensification under columns in distance (L) equal 145 cm



**** Design of Isolated Footing***

**** Project :***

Concrete	F_{cu} =	250	kg/cm²
Steel	F_y =	3600	kg/cm²
Bearing capacity q_{all} =		1.20	kg/cm²

foot	Column working load N_w (ton)	Column dim. b (cm)	t (cm)	extension of P.C (cm)
F1	40	30	60	30

Dims. of P.C :	B (cm)	L (cm)	t (cm)
	175	*205*	30

Dims. of R.C :	B (cm)	L (cm)	t (cm)
	115	*145*	60

Actual soil stress (kg/cm²)	Notes	*Stress above P.C (kg/cm²)*
1.15	*safe*	*2.55*

Shear stress (kg/cm²)	Notes
0.98	*safe*

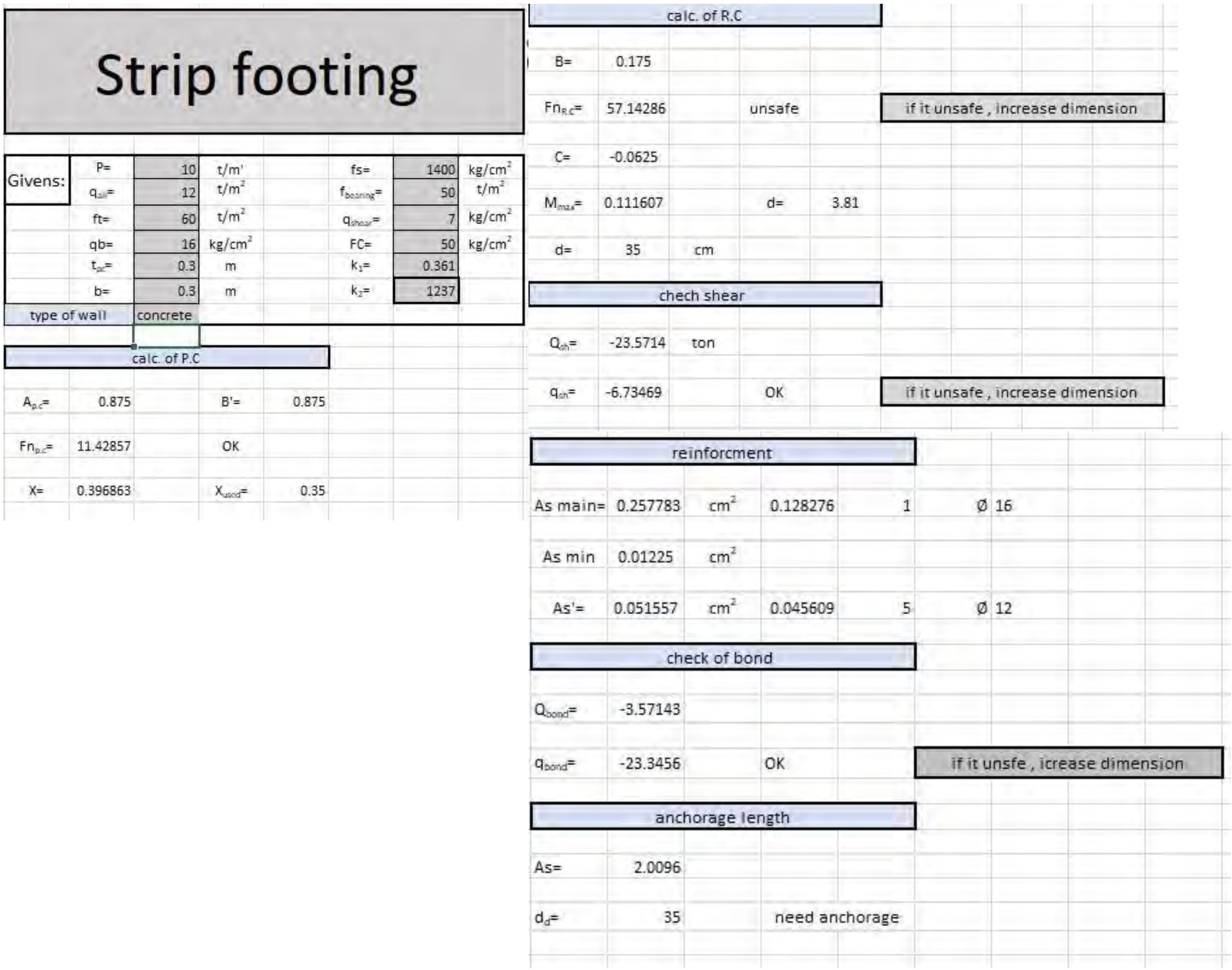
Punching stress (kg/cm²)	Notes
1.13	*safe*

Calculation of Rft. :	As_{min}	
	8.25	cm²

Long Rft. :	As /m	no.	ϕ		total no.
	2.02	*5*	16	/m	*6*

Short Rft. :	As /m	no.	ϕ		total no.
	2.02	*5*	16	/m	*8*

Strip footing Design



Strip footing

Givens:	P=	10	t/m'	fs=	1400	kg/cm^2
	q_{all}=	12	t/m^2	$f_{bearing}$=	50	t/m^2
	ft=	60	t/m^2	q_{shear}=	7	kg/cm^2
	qb=	16	kg/cm^2	FC=	50	kg/cm^2
	t_{pc}=	0.3	m	k_1=	0.361	
	b=	0.3	m	k_2=	1237	
type of wall	concrete					

calc. of P.C

$A_{p.c}$=	0.875	B'=	0.875
$Fn_{p.c}$=	11.42857	OK	
X=	0.396863	X_{used}=	0.35

calc. of R.C

B=	0.175			
$Fn_{R.C}$=	57.14286	unsafe		if it unsafe , increase dimension
C=	-0.0625			
M_{max}=	0.111607	d=	3.81	
d=	35	cm		

chech shear

Q_{sh}=	-23.5714	ton		
q_{sh}=	-6.73469		OK	if it unsafe , increase dimension

reinforcment

As main=	0.257783	cm^2	0.128276	1	Ø 16
As min	0.01225	cm^2			
As'=	0.051557	cm^2	0.045609	5	Ø 12

check of bond

Q_{bond}=	-3.57143		
q_{bond}=	-23.3456	OK	if it unsfe , icrease dimension

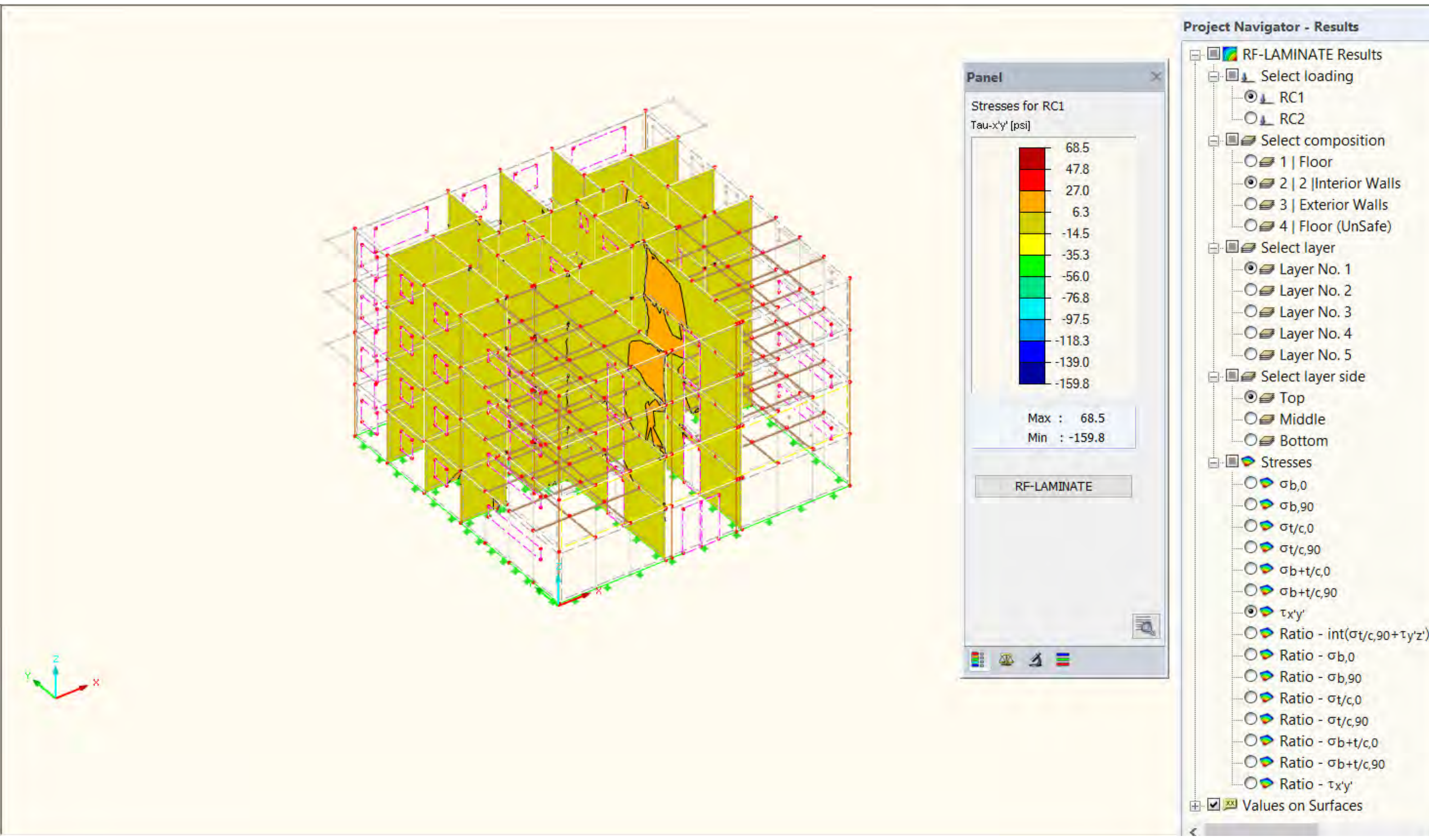
anchorage length

As=	2.0096	
d_d=	35	need anchorage

11.2 Structural analysis results

11.2.1 Walls

Max shear stresses on CLT Interior walls



Panel

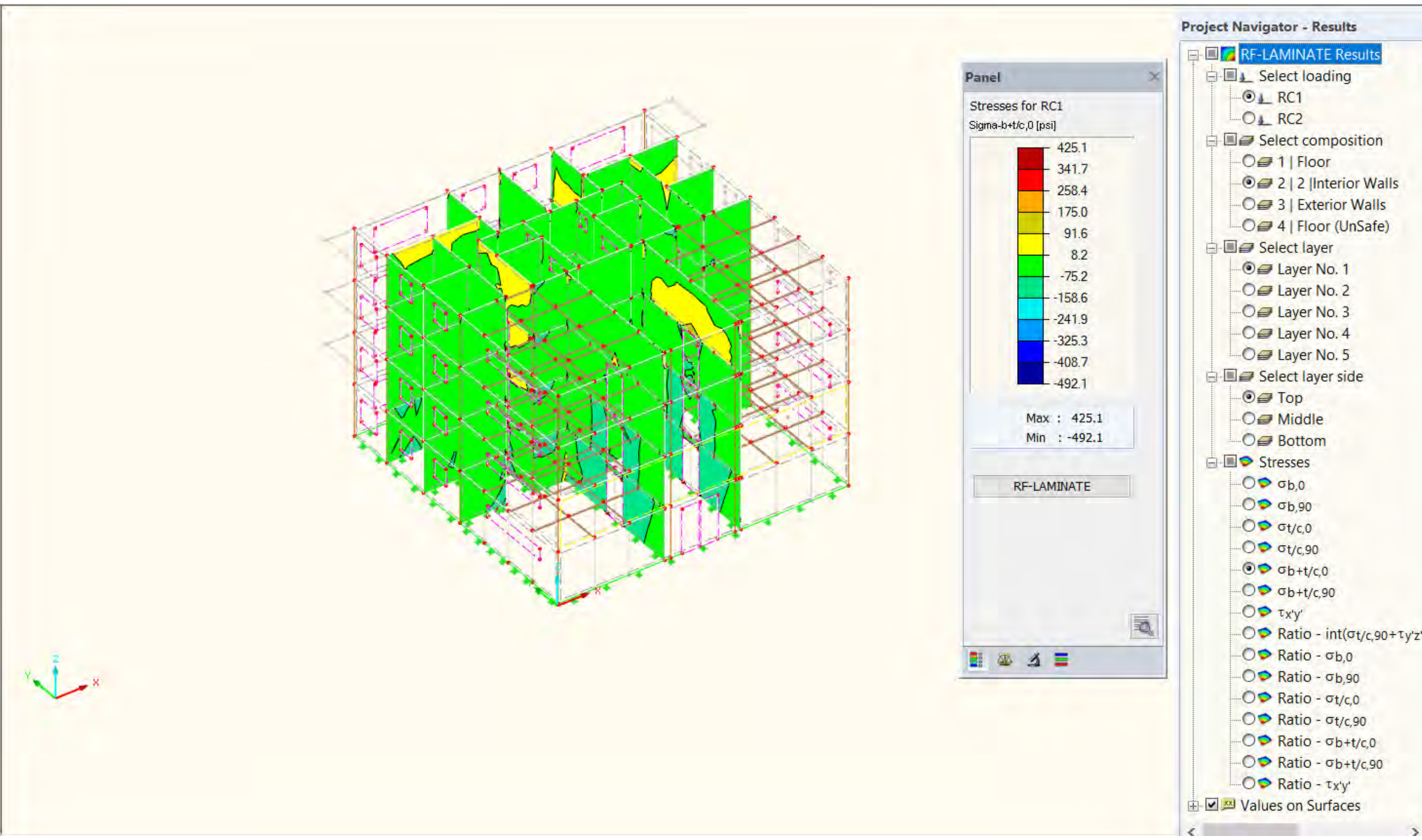
Stresses for RC1

Tau-x'y' [psi]

Max : 68.5

Min : -159.8

Combination between Max. bending stresses + Max. compression stresses on CLT Interior walls



Stresses for RC1

Sigma-b+t/c,0 [psi]

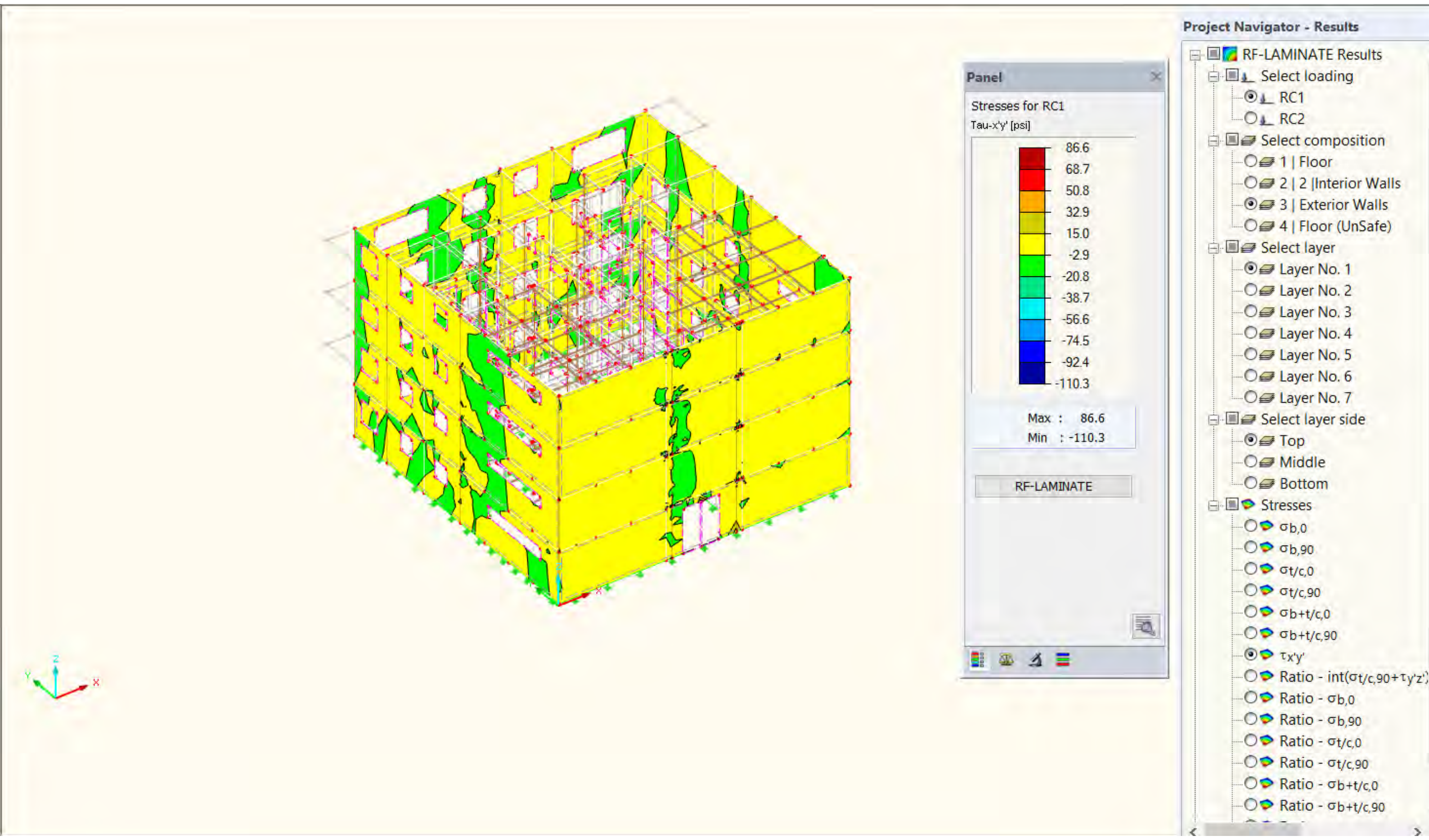
Legend
425.1
341.7
258.4
175.0
91.6
8.2
-75.2
-158.6
-241.9
-325.3
-408.7
-492.1

Max : 425.1
Min : -492.1

Project Navigator - Results

- RF-LAMINATE Results
  - Select loading
    - ● RC1
    - ○ RC2
  - Select composition
    - ○ 1 | Floor
    - ● 2 | 2 |Interior Walls
    - ○ 3 | Exterior Walls
    - ○ 4 | Floor (UnSafe)
  - Select layer
    - ● Layer No. 1
    - ○ Layer No. 2
    - ○ Layer No. 3
    - ○ Layer No. 4
    - ○ Layer No. 5
  - Select layer side
    - ● Top
    - ○ Middle
    - ○ Bottom
  - Stresses
    - ○ $\sigma_{b,0}$
    - ○ $\sigma_{b,90}$
    - ○ $\sigma_{t/c,0}$
    - ○ $\sigma_{t/c,90}$
    - ● $\sigma_{b+t/c,0}$
    - ○ $\sigma_{b+t/c,90}$
    - ○ $\tau_{x'y'}$
    - ○ Ratio - int($\sigma_{t/c,90}+\tau_{y'z'}$
    - ○ Ratio - $\sigma_{b,0}$
    - ○ Ratio - $\sigma_{b,90}$
    - ○ Ratio - $\sigma_{t/c,0}$
    - ○ Ratio - $\sigma_{t/c,90}$
    - ○ Ratio - $\sigma_{b+t/c,0}$
    - ○ Ratio - $\sigma_{b+t/c,90}$
    - ○ Ratio - $\tau_{x'y'}$
- ☑ Values on Surfaces

Max shear stresses on CLT Exterior walls



Panel

Stresses for RC1

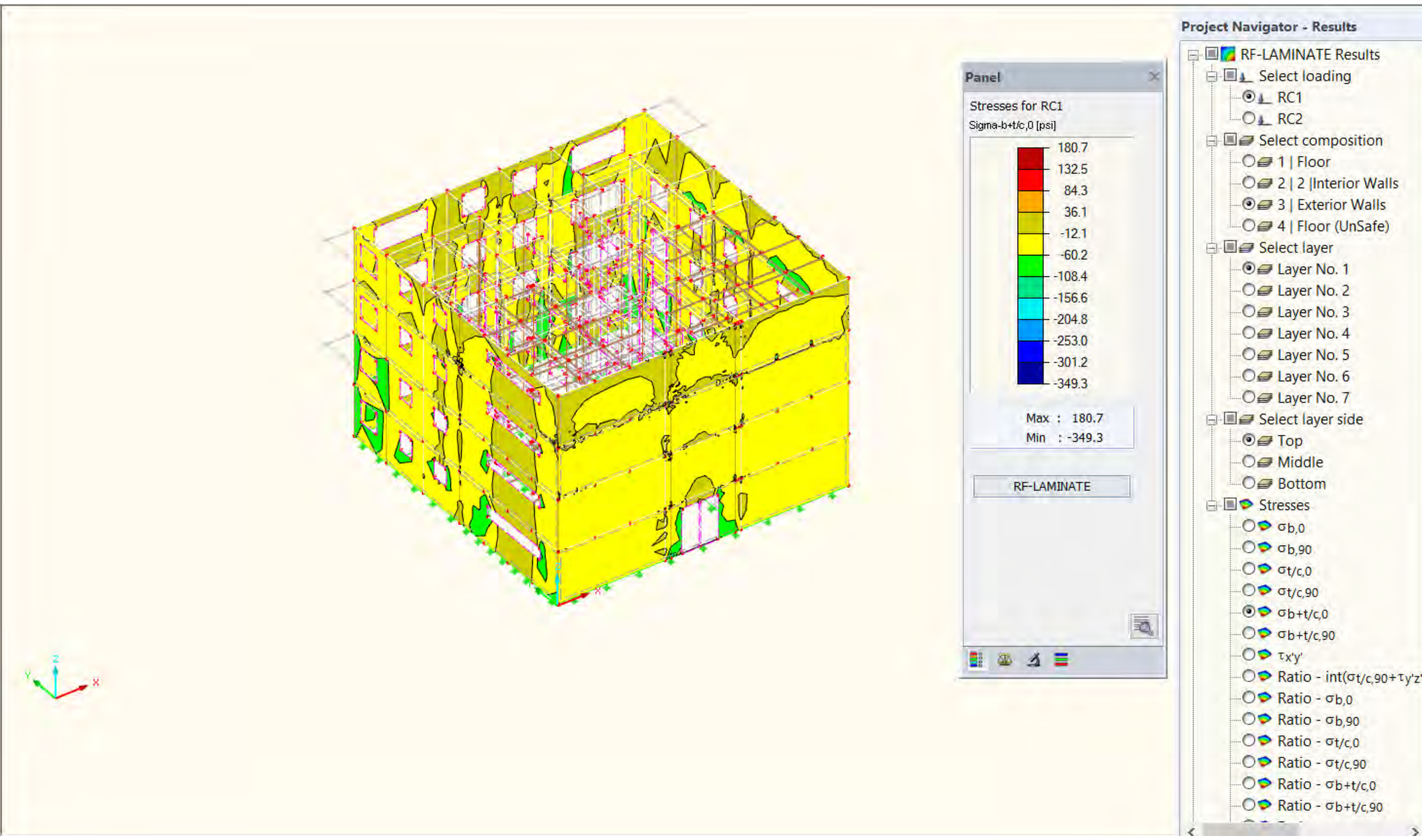
Tau-x'y' [psi]

🟥	86.6
🟥	68.7
🟧	50.8
🟨	32.9
🟨	15.0
🟩	-2.9
🟩	-20.8
🟦	-38.7
🟦	-56.6
🟦	-74.5
🟦	-92.4
	-110.3

Max : 86.6

Min : -110.3

Combination between Max. bending stresses + Max. compression stresses on CLT Interior walls



Panel

Stresses for RC1

Sigma-b+t/c,0 [psi]

180.7
132.5
84.3
36.1
-12.1
-60.2
-108.4
-156.6
-204.8
-253.0
-301.2
-349.3

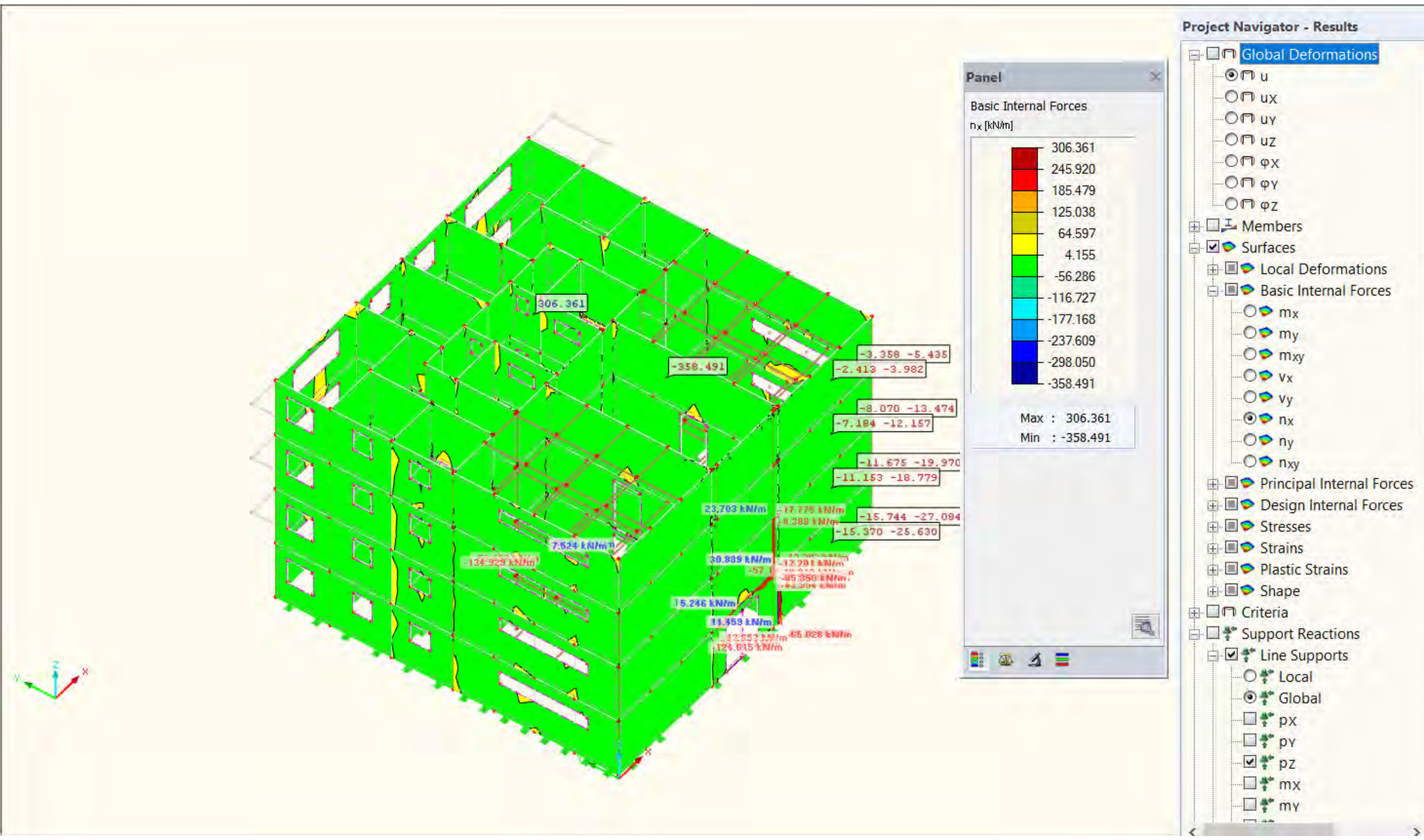
Max : 180.7
Min : -349.3

RF-LAMINATE

Project Navigator - Results

- RF-LAMINATE Results
  - Select loading
    - ● RC1
    - ○ RC2
  - Select composition
    - ○ 1 | Floor
    - ○ 2 | 2 |Interior Walls
    - ● 3 | Exterior Walls
    - ○ 4 | Floor (UnSafe)
  - Select layer
    - ● Layer No. 1
    - ○ Layer No. 2
    - ○ Layer No. 3
    - ○ Layer No. 4
    - ○ Layer No. 5
    - ○ Layer No. 6
    - ○ Layer No. 7
  - Select layer side
    - ● Top
    - ○ Middle
    - ○ Bottom
  - Stresses
    - ○ $\sigma_{b,0}$
    - ○ $\sigma_{b,90}$
    - ○ $\sigma_{t/c,0}$
    - ○ $\sigma_{t/c,90}$
    - ● $\sigma_{b+t/c,0}$
    - ○ $\sigma_{b+t/c,90}$
    - ○ $\tau_{x'y'}$
    - ○ Ratio - int($\sigma_{t/c,90}+\tau_{y'z'}$
    - ○ Ratio - $\sigma_{b,0}$
    - ○ Ratio - $\sigma_{b,90}$
    - ○ Ratio - $\sigma_{t/c,0}$
    - ○ Ratio - $\sigma_{t/c,90}$
    - ○ Ratio - $\sigma_{b+t/c,0}$
    - ○ Ratio - $\sigma_{b+t/c,90}$

Max. Compression/Tension forces on CLT walls (x-direction)



Panel

Basic Internal Forces

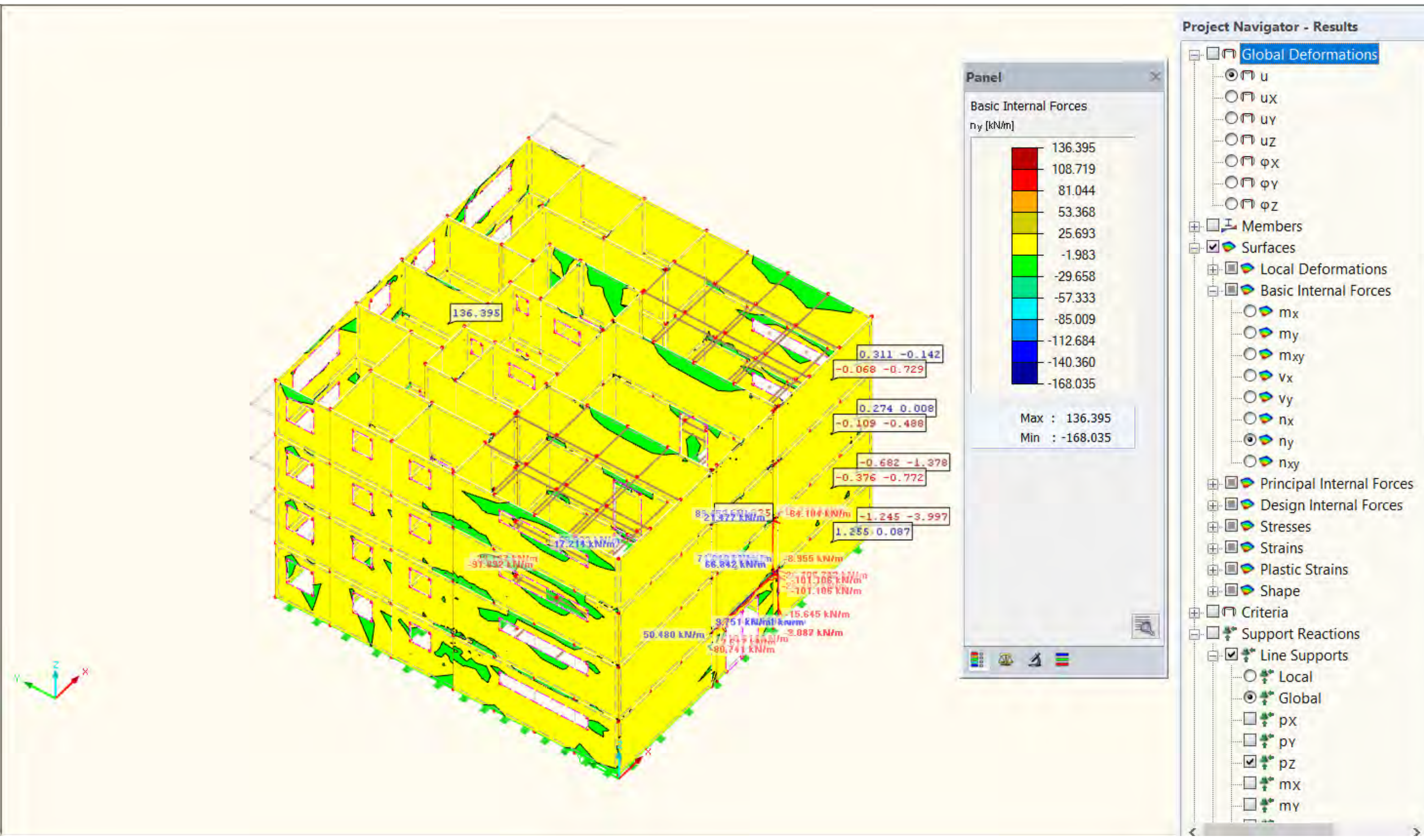
n_x [kN/m]

306.361
245.920
185.479
125.038
64.597
4.155
-56.286
-116.727
-177.168
-237.609
-298.050
-358.491

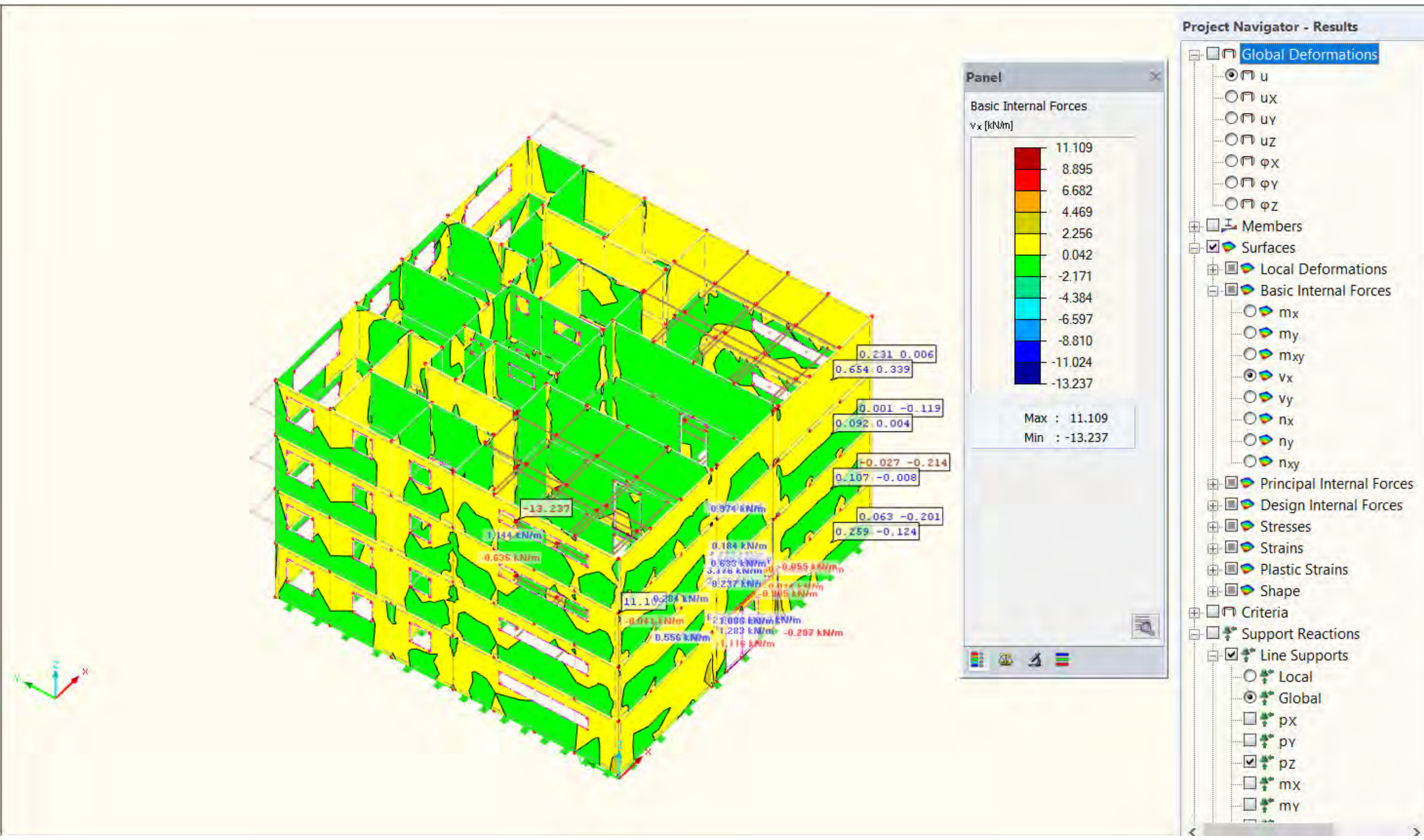
Max : 306.361

Min : -358.491

Max. Compression/Tension forces on CLT walls (y-direction)



Max. Shear force on CLT walls (x-direction)



Panel

Basic Internal Forces

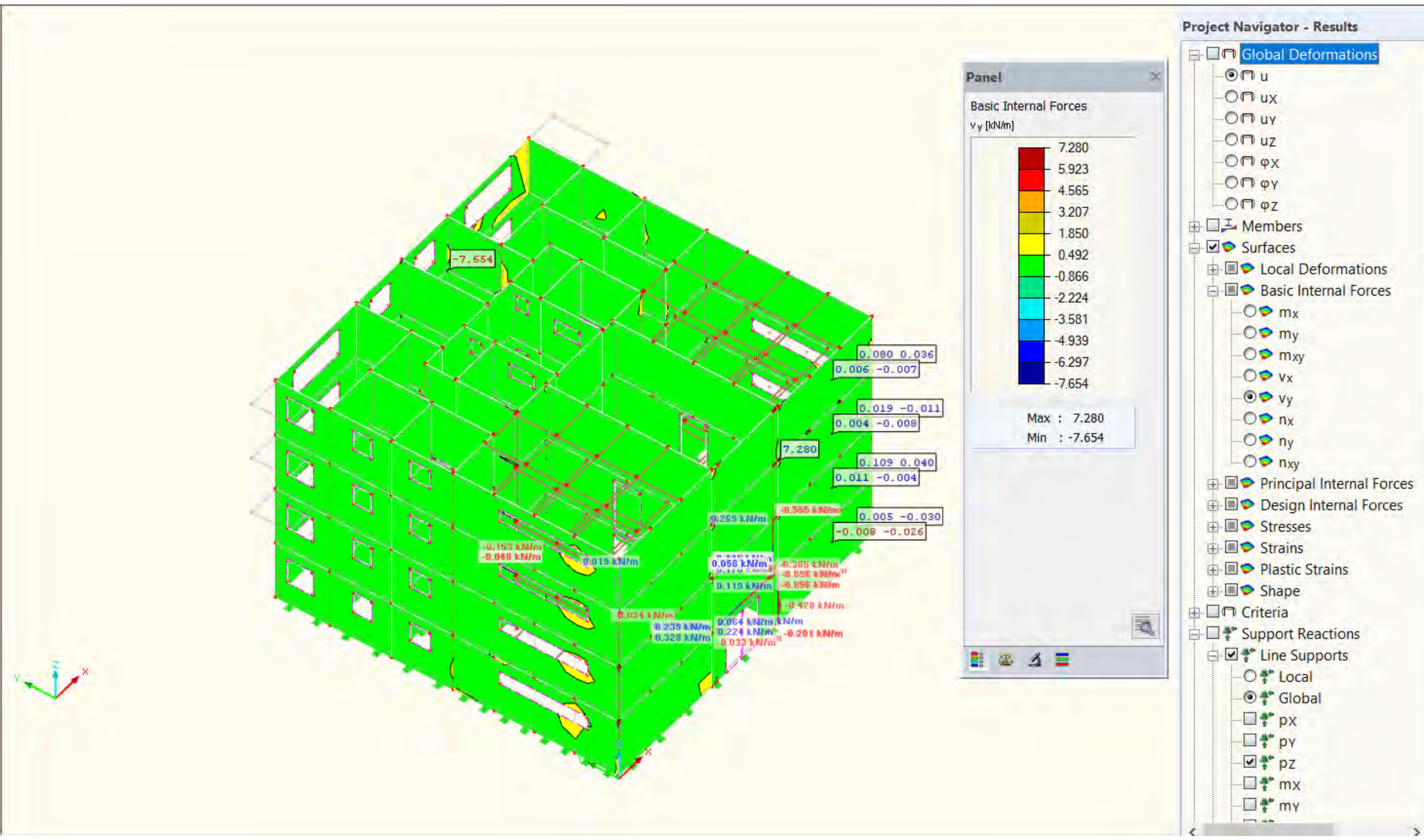
v_x [kN/m]

🟥	11.109
🟥	8.895
🟥	6.682
🟧	4.469
🟨	2.256
🟨	0.042
🟩	-2.171
🟩	-4.384
🟦	-6.597
🟦	-8.810
🟦	-11.024
🟦	-13.237

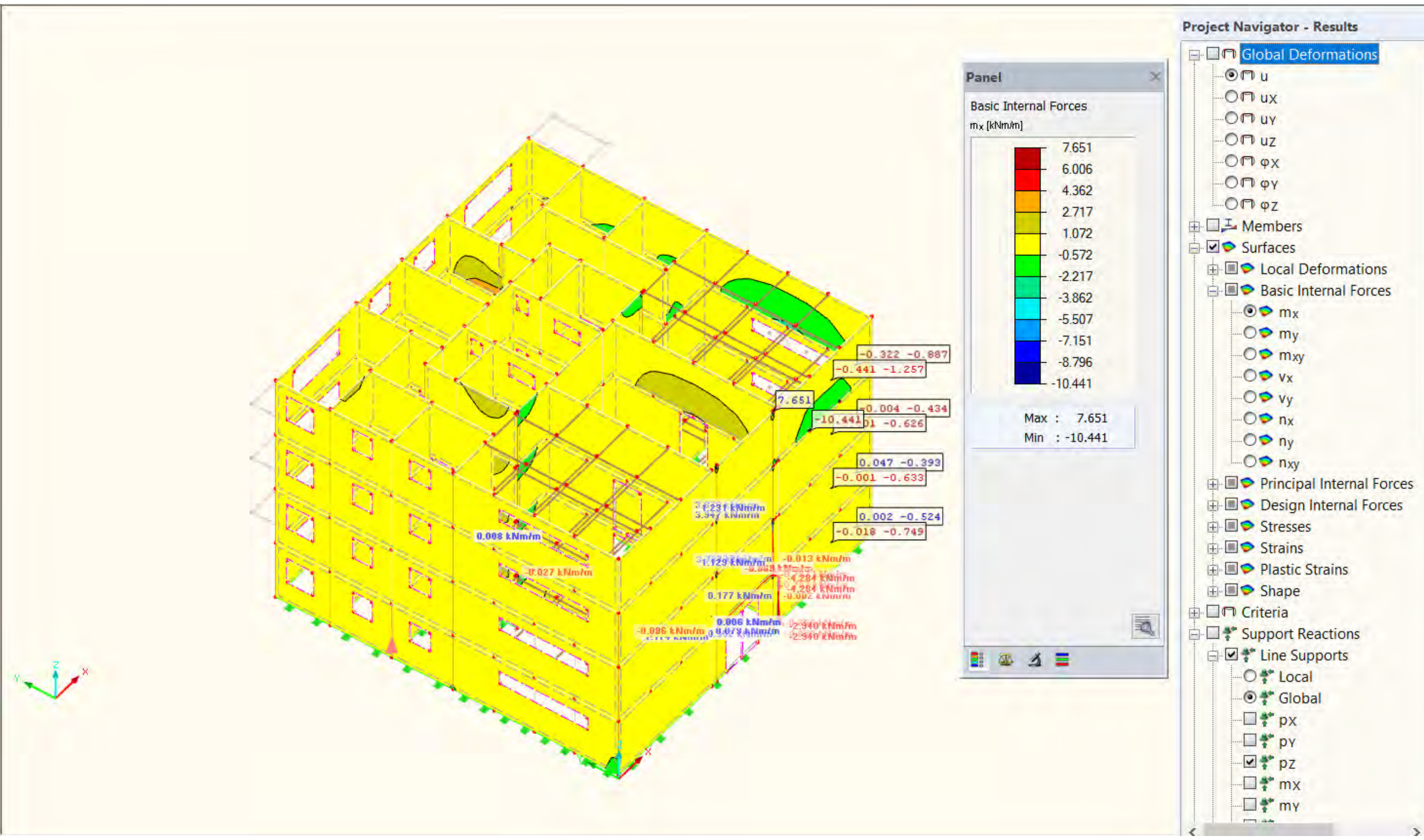
Max : 11.109

Min : -13.237

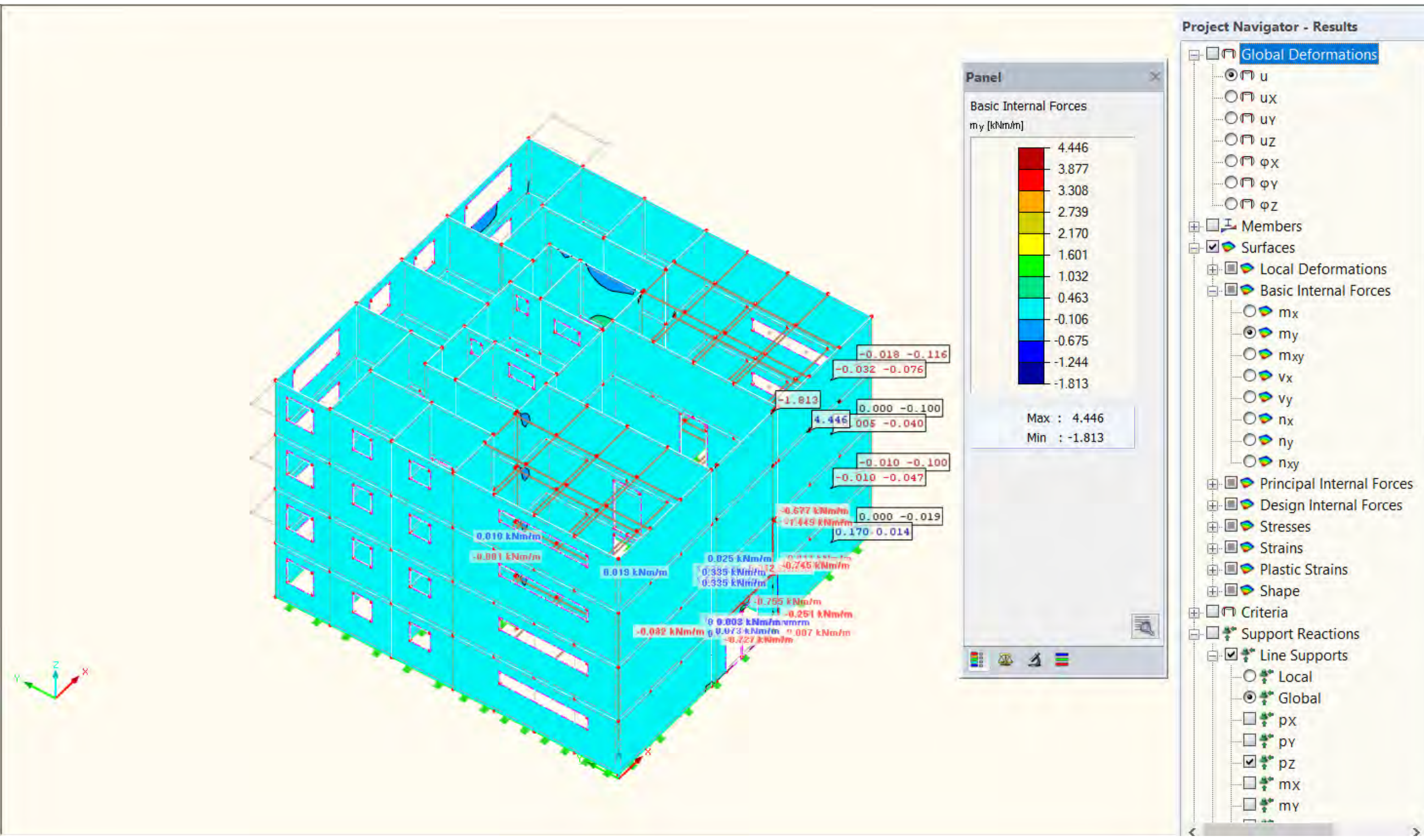
Max. Shear force on CLT walls (y-direction)



Max. Bending Moment on CLT walls (x-direction)

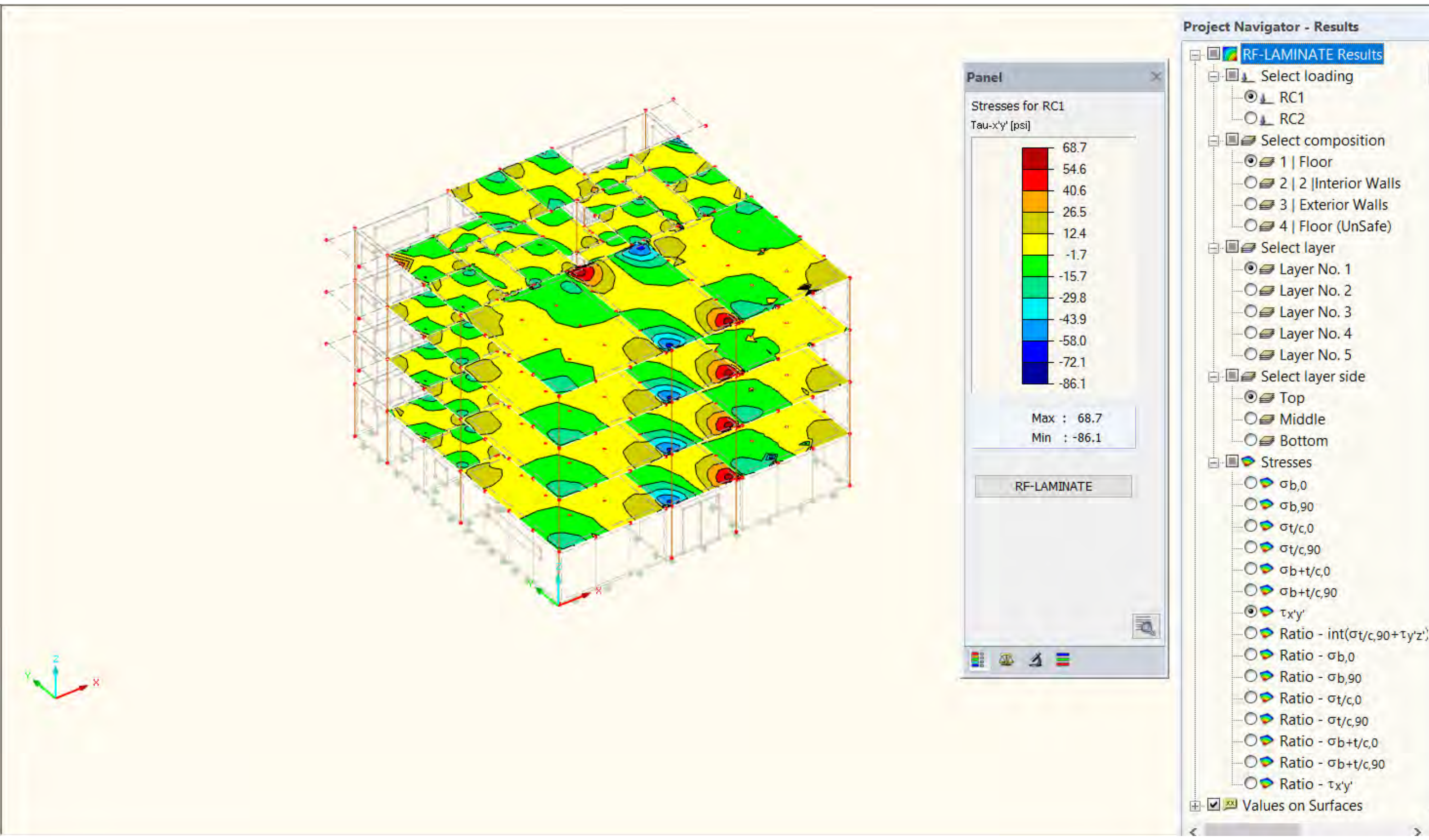


Max. Bending Moment on CLT walls (y-direction)

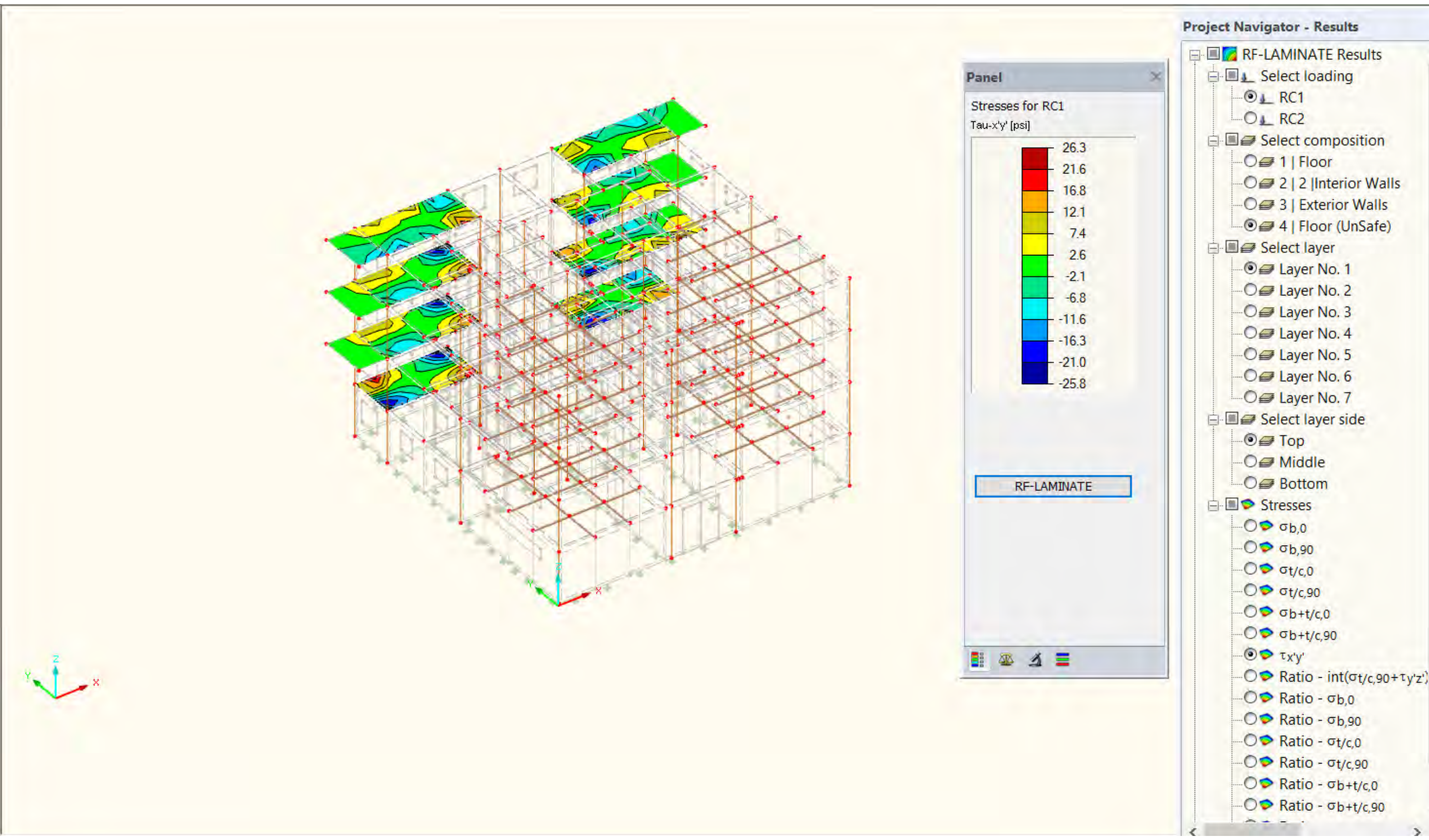


11.2.2 Floor

Max shear stresses on CLT Floors



Max shear stresses on CLT Floors



Panel

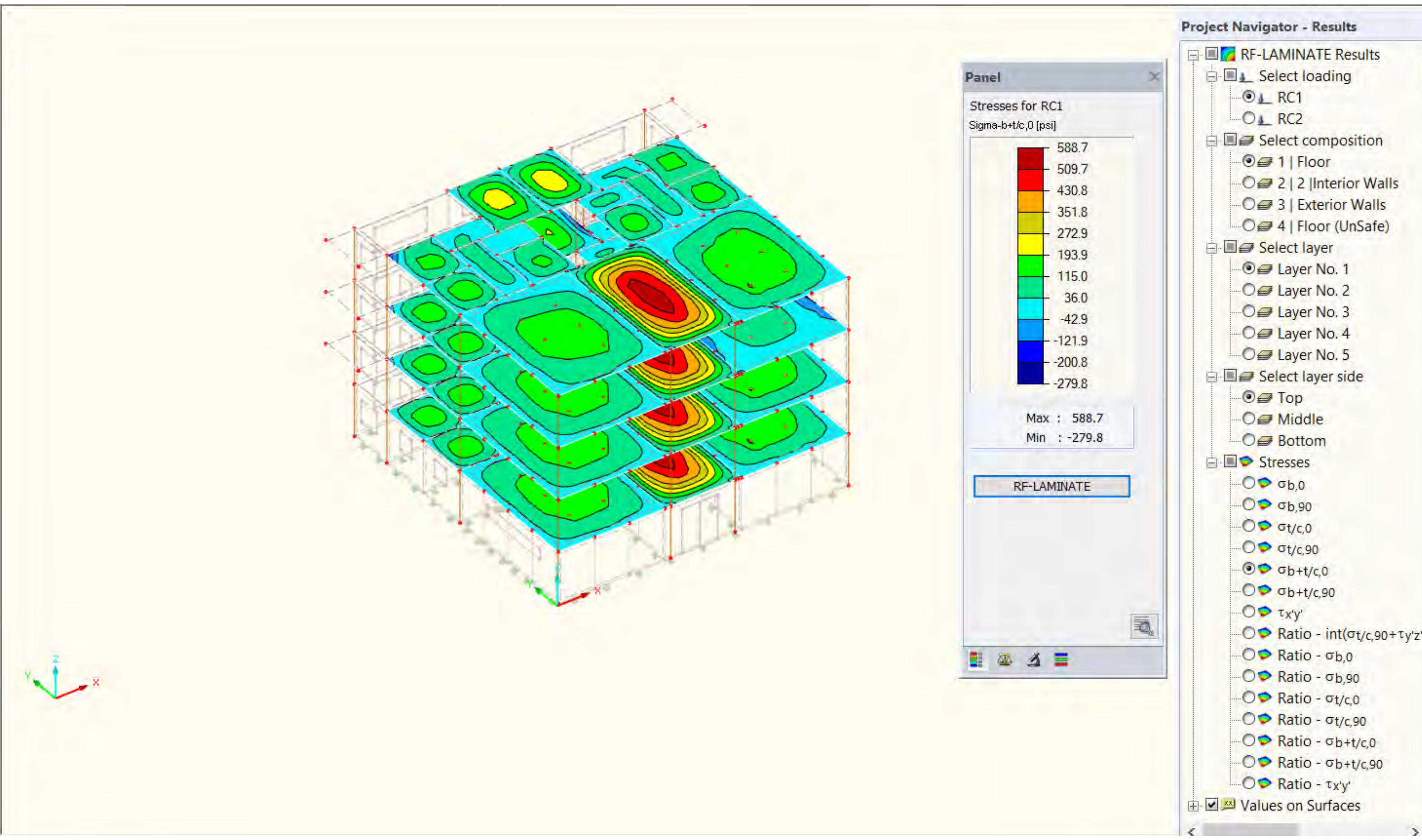
Stresses for RC1

Tau-x'y' [psi]

Project Navigator - Results

- RF-LAMINATE Results
  - Select loading
    - ● RC1
    - ○ RC2
  - Select composition
    - ○ 1 | Floor
    - ○ 2 | 2 |Interior Walls
    - ○ 3 | Exterior Walls
    - ● 4 | Floor (UnSafe)
  - Select layer
    - ● Layer No. 1
    - ○ Layer No. 2
    - ○ Layer No. 3
    - ○ Layer No. 4
    - ○ Layer No. 5
    - ○ Layer No. 6
    - ○ Layer No. 7
  - Select layer side
    - ● Top
    - ○ Middle
    - ○ Bottom
  - Stresses
    - ○ $\sigma_{b,0}$
    - ○ $\sigma_{b,90}$
    - ○ $\sigma_{t/c,0}$
    - ○ $\sigma_{t/c,90}$
    - ○ $\sigma_{b+t/c,0}$
    - ○ $\sigma_{b+t/c,90}$
    - ● $\tau_{x'y'}$
    - ○ Ratio - int($\sigma_{t/c,90}+\tau_{y'z'}$)
    - ○ Ratio - $\sigma_{b,0}$
    - ○ Ratio - $\sigma_{b,90}$
    - ○ Ratio - $\sigma_{t/c,0}$
    - ○ Ratio - $\sigma_{t/c,90}$
    - ○ Ratio - $\sigma_{b+t/c,0}$
    - ○ Ratio - $\sigma_{b+t/c,90}$

Combination between Max. bending stresses + Max. compression/tension stresses on CLT Floor



Panel

Stresses for RC1

Sigma-b+t/c,0 [psi]

🟥	588.7
🟥	509.7
🟧	430.8
🟨	351.8
🟨	272.9
🟩	193.9
🟩	115.0
🟦	36.0
🟦	-42.9
🟦	-121.9
🟦	-200.8
🟦	-279.8

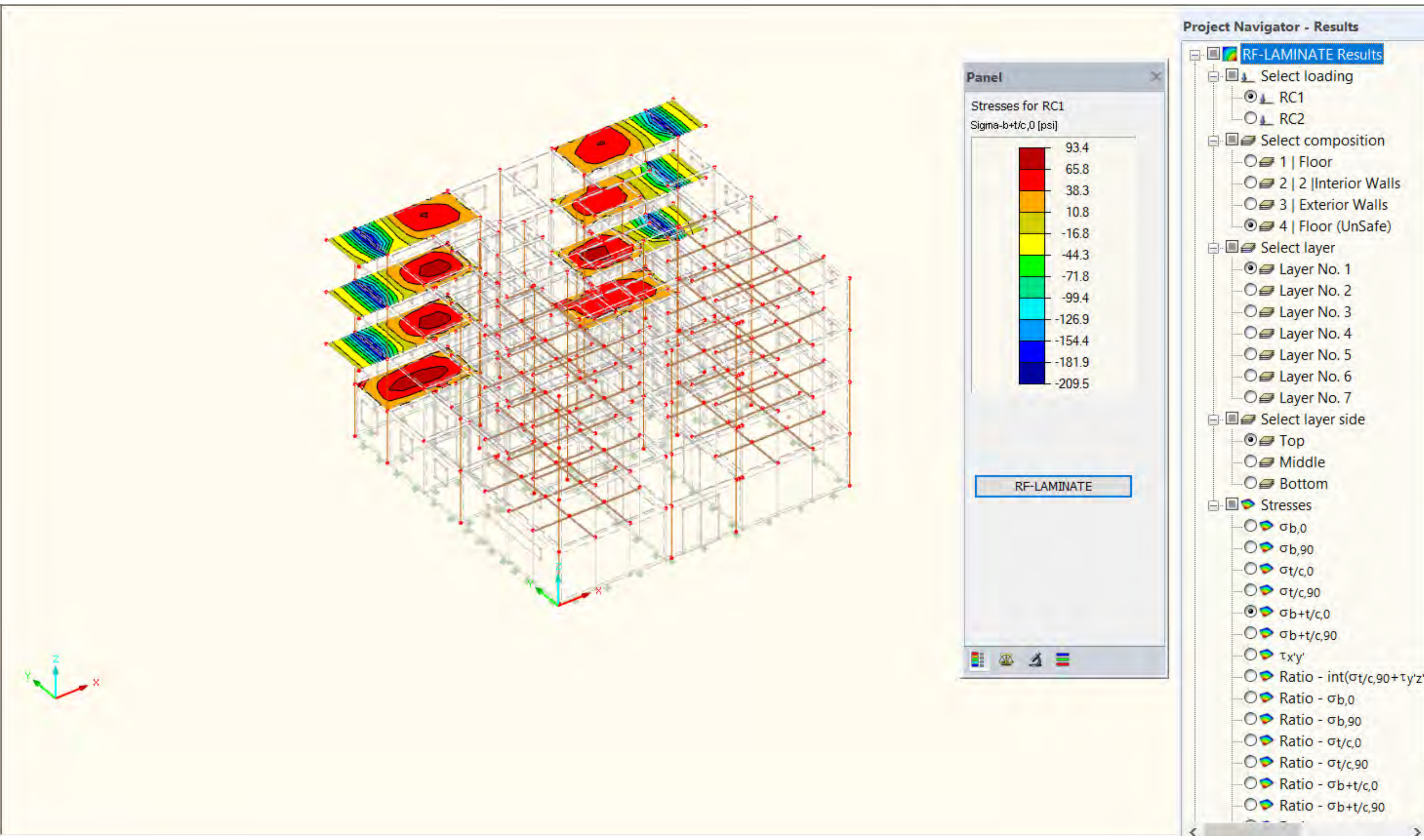
Max : 588.7

Min : -279.8

Project Navigator - Results

- RF-LAMINATE Results
  - Select loading
    - ● RC1
    - ○ RC2
  - Select composition
    - ● 1 | Floor
    - ○ 2 | 2 |Interior Walls
    - ○ 3 | Exterior Walls
    - ○ 4 | Floor (UnSafe)
  - Select layer
    - ● Layer No. 1
    - ○ Layer No. 2
    - ○ Layer No. 3
    - ○ Layer No. 4
    - ○ Layer No. 5
  - Select layer side
    - ● Top
    - ○ Middle
    - ○ Bottom
  - Stresses
    - ○ $\sigma_{b,0}$
    - ○ $\sigma_{b,90}$
    - ○ $\sigma_{t/c,0}$
    - ○ $\sigma_{t/c,90}$
    - ● $\sigma_{b+t/c,0}$
    - ○ $\sigma_{b+t/c,90}$
    - ○ $\tau_{x'y'}$
    - ○ Ratio - int($\sigma_{t/c,90}+\tau_{y'z'}$
    - ○ Ratio - $\sigma_{b,0}$
    - ○ Ratio - $\sigma_{b,90}$
    - ○ Ratio - $\sigma_{t/c,0}$
    - ○ Ratio - $\sigma_{t/c,90}$
    - ○ Ratio - $\sigma_{b+t/c,0}$
    - ○ Ratio - $\sigma_{b+t/c,90}$
    - ○ Ratio - $\tau_{x'y'}$
- ☑ Values on Surfaces

Combination between Max. bending stresses + Max. compression/tension stresses on CLT Floor



Panel

Stresses for RC1
Sigma-b+t/c,0 [psi]

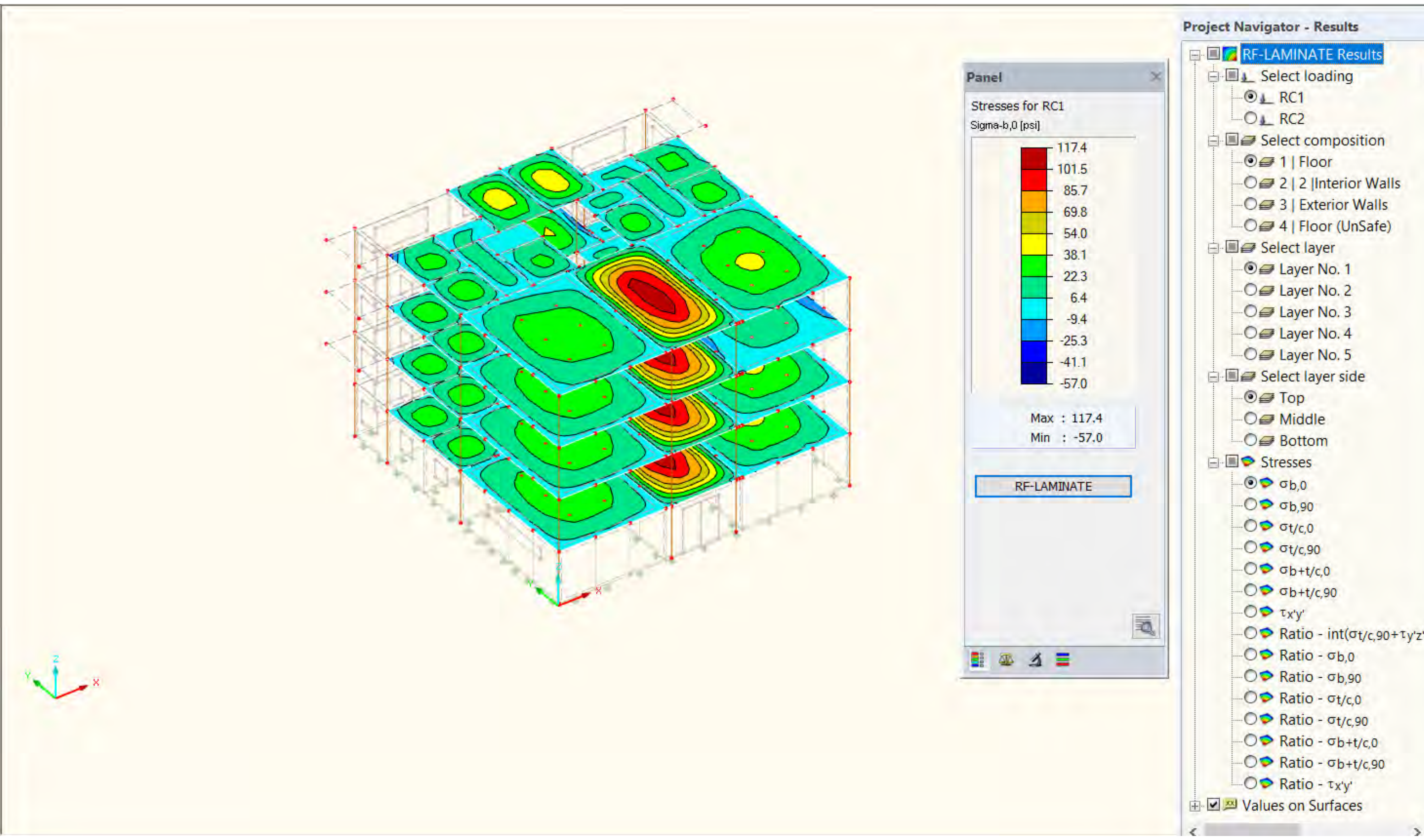
- 🟥 93.4
- 🟥 65.8
- 🟧 38.3
- 🟨 10.8
- 🟨 -16.8
- 🟩 -44.3
- 🟩 -71.8
- 🟦 -99.4
- 🟦 -126.9
- 🟦 -154.4
- 🟦 -181.9
- 🟦 -209.5

RF-LAMINATE

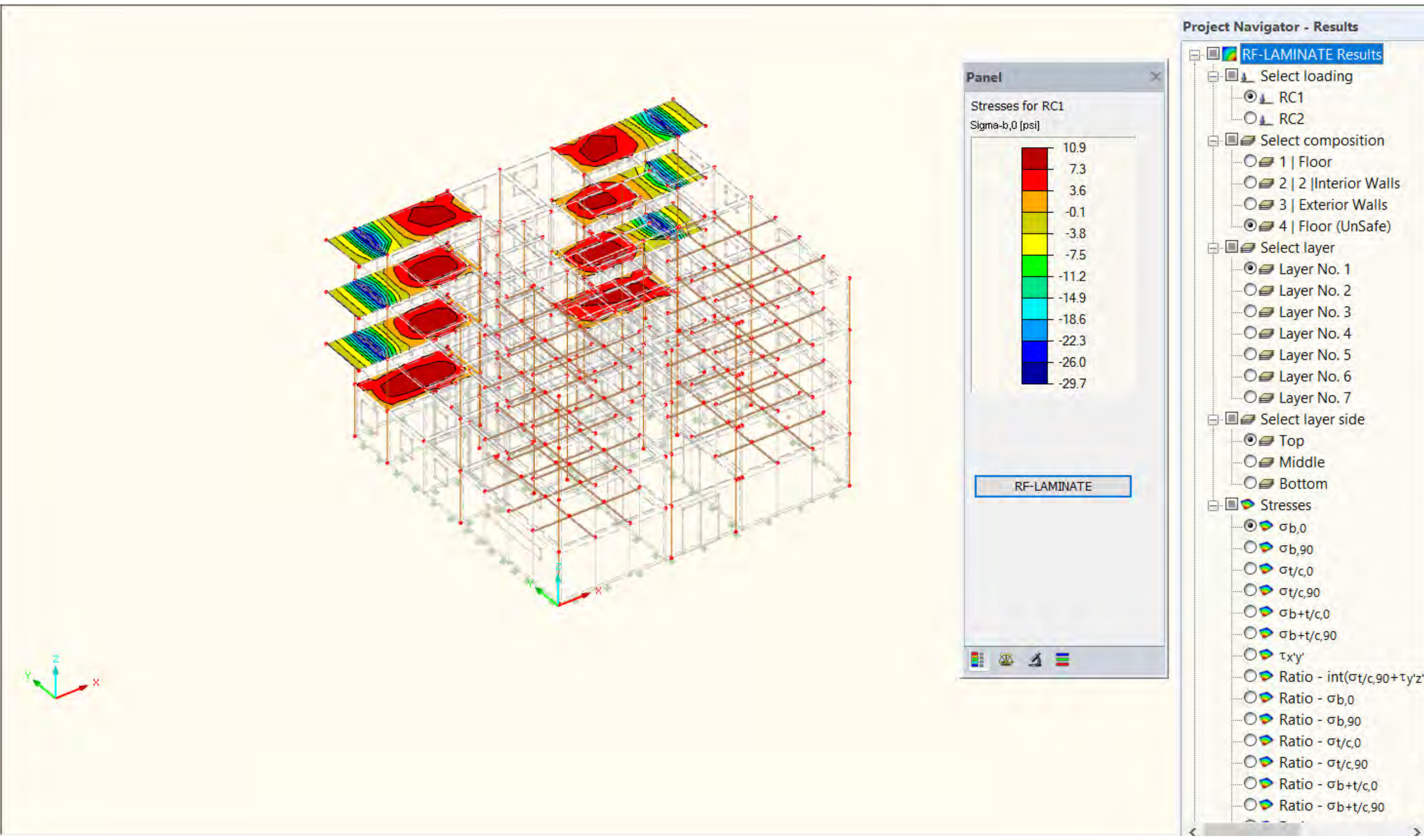
Project Navigator - Results

- RF-LAMINATE Results
  - Select loading
    - ● RC1
    - ○ RC2
  - Select composition
    - ○ 1 | Floor
    - ○ 2 | 2 |Interior Walls
    - ○ 3 | Exterior Walls
    - ● 4 | Floor (UnSafe)
  - Select layer
    - ● Layer No. 1
    - ○ Layer No. 2
    - ○ Layer No. 3
    - ○ Layer No. 4
    - ○ Layer No. 5
    - ○ Layer No. 6
    - ○ Layer No. 7
  - Select layer side
    - ● Top
    - ○ Middle
    - ○ Bottom
  - Stresses
    - ○ $\sigma_{b,0}$
    - ○ $\sigma_{b,90}$
    - ○ $\sigma_{t/c,0}$
    - ○ $\sigma_{t/c,90}$
    - ● $\sigma_{b+t/c,0}$
    - ○ $\sigma_{b+t/c,90}$
    - ○ $\tau_{x'y'}$
    - ○ Ratio - int($\sigma_{t/c,90}+\tau_{y'z'}$
    - ○ Ratio - $\sigma_{b,0}$
    - ○ Ratio - $\sigma_{b,90}$
    - ○ Ratio - $\sigma_{t/c,0}$
    - ○ Ratio - $\sigma_{t/c,90}$
    - ○ Ratio - $\sigma_{b+t/c,0}$
    - ○ Ratio - $\sigma_{b+t/c,90}$

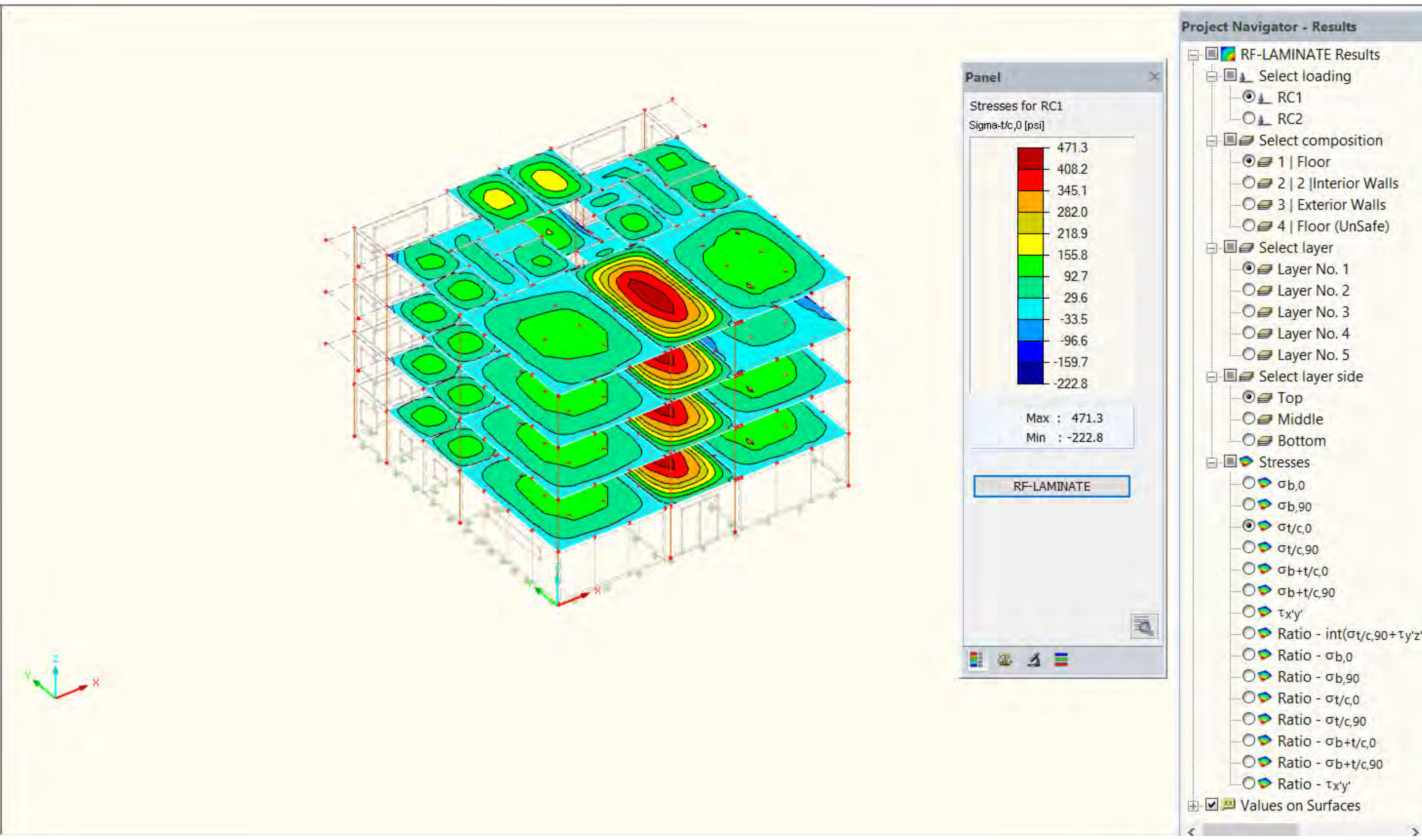
Max. bending stresses on CLT Floor



Max. bending stresses on CLT Floor



Max. compression/tension stresses on CLT Floor



Panel

Stresses for RC1
Sigma-t/c,0 [psi]

🟥	471.3
🟥	408.2
🟥	345.1
🟧	282.0
🟨	218.9
🟨	155.8
🟩	92.7
🟩	29.6
🟦	-33.5
🟦	-96.6
🟦	-159.7
🟦	-222.8

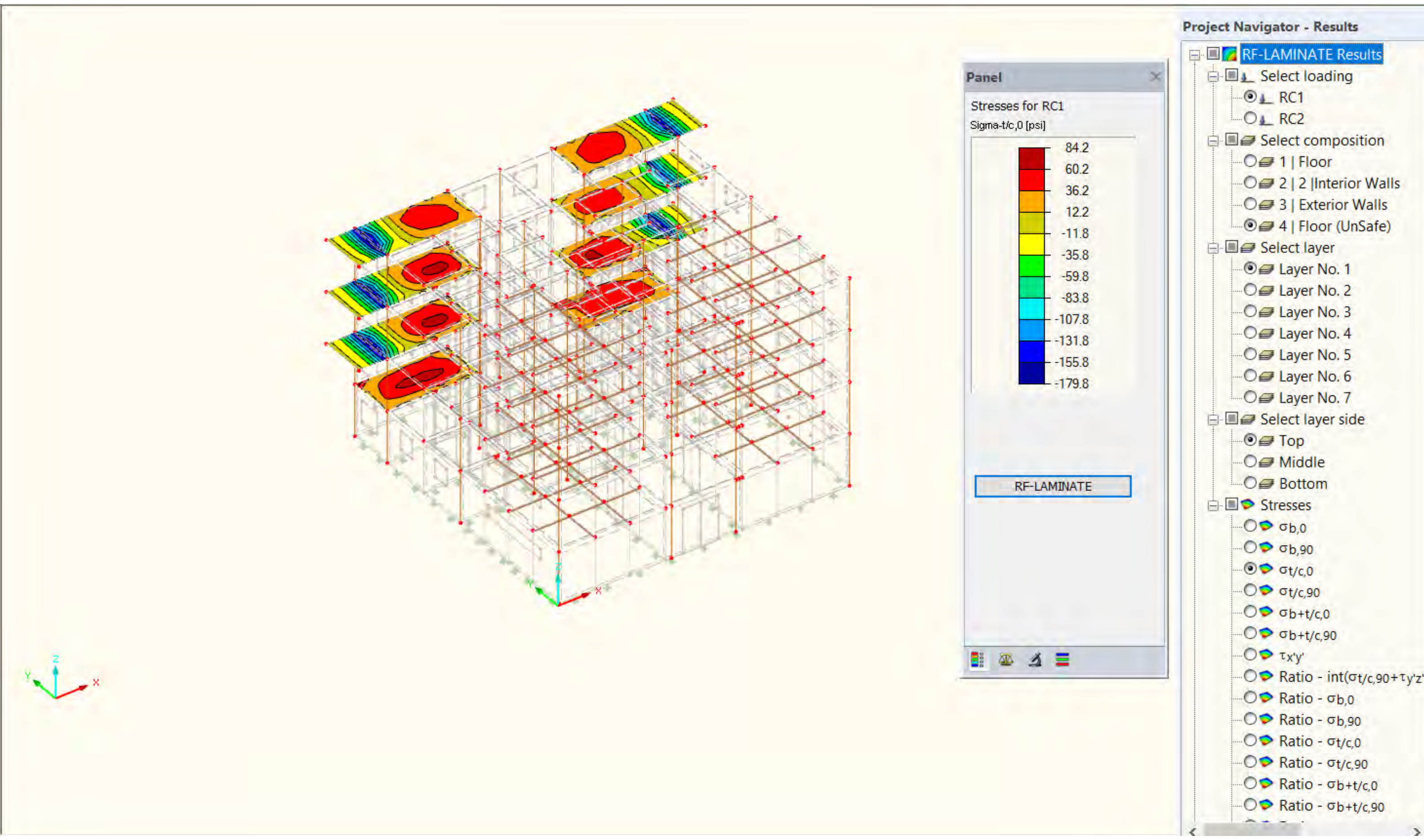
Max : 471.3
Min : -222.8

RF-LAMINATE

Project Navigator - Results

- RF-LAMINATE Results
  - Select loading
    - ● RC1
    - ○ RC2
  - Select composition
    - ● 1 | Floor
    - ○ 2 | 2 |Interior Walls
    - ○ 3 | Exterior Walls
    - ○ 4 | Floor (UnSafe)
  - Select layer
    - ● Layer No. 1
    - ○ Layer No. 2
    - ○ Layer No. 3
    - ○ Layer No. 4
    - ○ Layer No. 5
  - Select layer side
    - ● Top
    - ○ Middle
    - ○ Bottom
  - Stresses
    - ○ $\sigma_{b,0}$
    - ○ $\sigma_{b,90}$
    - ● $\sigma_{t/c,0}$
    - ○ $\sigma_{t/c,90}$
    - ○ $\sigma_{b+t/c,0}$
    - ○ $\sigma_{b+t/c,90}$
    - ○ $\tau_{x'y'}$
    - ○ Ratio - int($\sigma_{t/c,90}+\tau_{y'z'}$
    - ○ Ratio - $\sigma_{b,0}$
    - ○ Ratio - $\sigma_{b,90}$
    - ○ Ratio - $\sigma_{t/c,0}$
    - ○ Ratio - $\sigma_{t/c,90}$
    - ○ Ratio - $\sigma_{b+t/c,0}$
    - ○ Ratio - $\sigma_{b+t/c,90}$
    - ○ Ratio - $\tau_{x'y'}$
- ☑ Values on Surfaces

Max. compression/tension stresses on CLT Floor

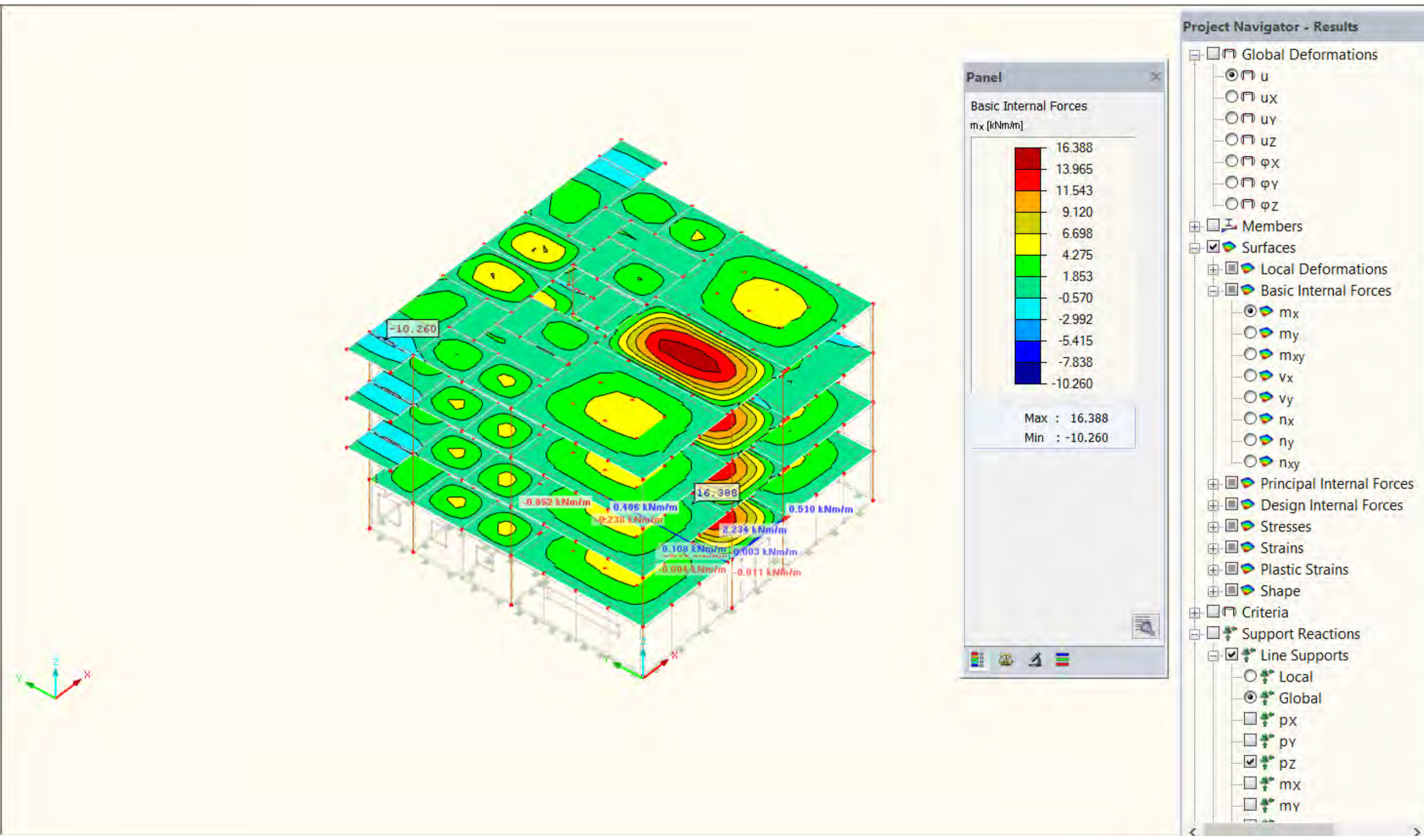


Stresses for RC1
Sigma-t/c,0 [psi]

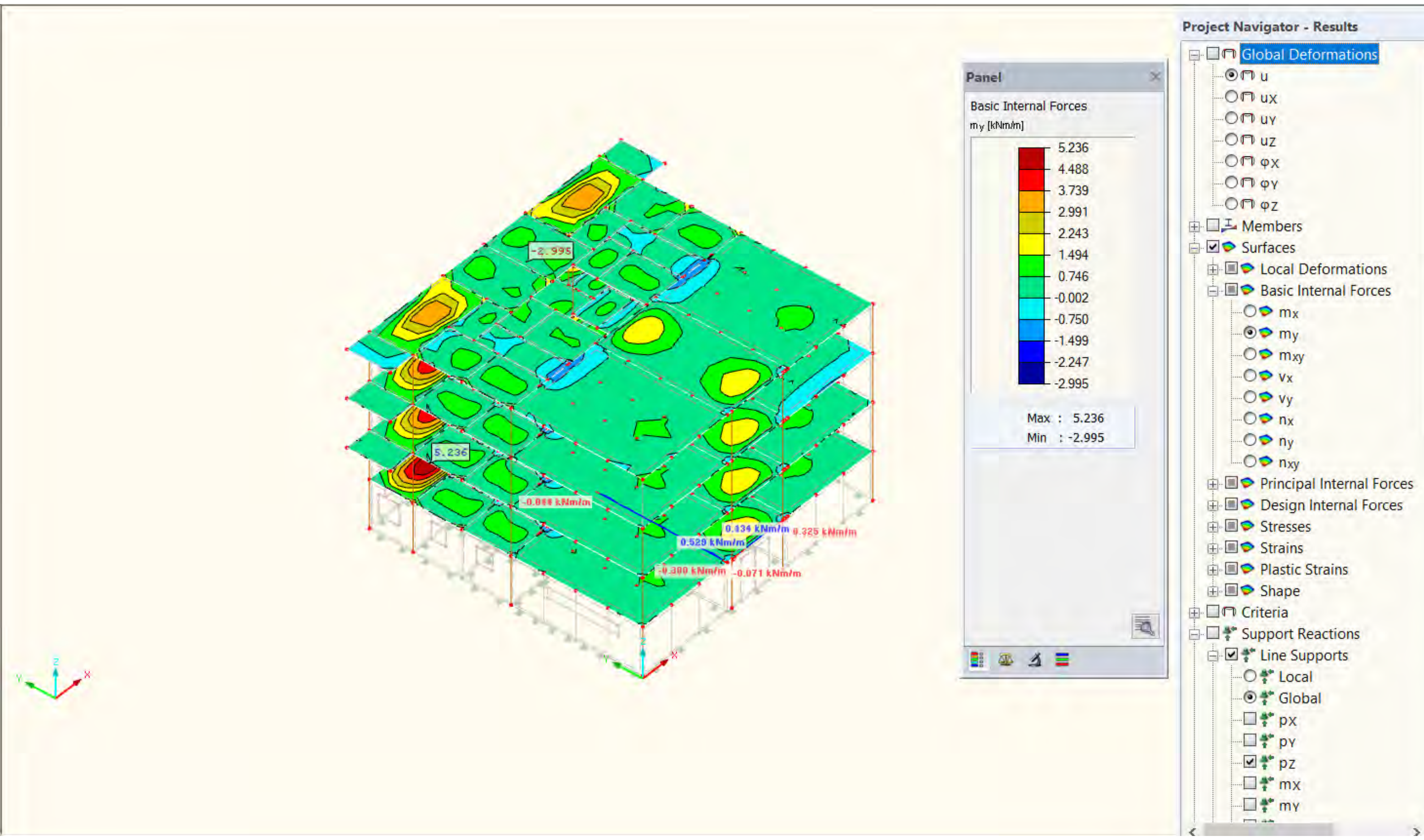
Project Navigator - Results

- RF-LAMINATE Results
  - Select loading
    - ● RC1
    - ○ RC2
  - Select composition
    - ○ 1 | Floor
    - ○ 2 | 2 |Interior Walls
    - ○ 3 | Exterior Walls
    - ● 4 | Floor (UnSafe)
  - Select layer
    - ● Layer No. 1
    - ○ Layer No. 2
    - ○ Layer No. 3
    - ○ Layer No. 4
    - ○ Layer No. 5
    - ○ Layer No. 6
    - ○ Layer No. 7
  - Select layer side
    - ● Top
    - ○ Middle
    - ○ Bottom
  - Stresses
    - ○ $\sigma_{b,0}$
    - ○ $\sigma_{b,90}$
    - ● $\sigma_{t/c,0}$
    - ○ $\sigma_{t/c,90}$
    - ○ $\sigma_{b+t/c,0}$
    - ○ $\sigma_{b+t/c,90}$
    - ○ $\tau_{x'y'}$
    - ○ Ratio - int($\sigma_{t/c,90}+\tau_{y'z'}$
    - ○ Ratio - $\sigma_{b,0}$
    - ○ Ratio - $\sigma_{b,90}$
    - ○ Ratio - $\sigma_{t/c,0}$
    - ○ Ratio - $\sigma_{t/c,90}$
    - ○ Ratio - $\sigma_{b+t/c,0}$
    - ○ Ratio - $\sigma_{b+t/c,90}$

Max. Bending Moment on CLT Floor (x-direction)

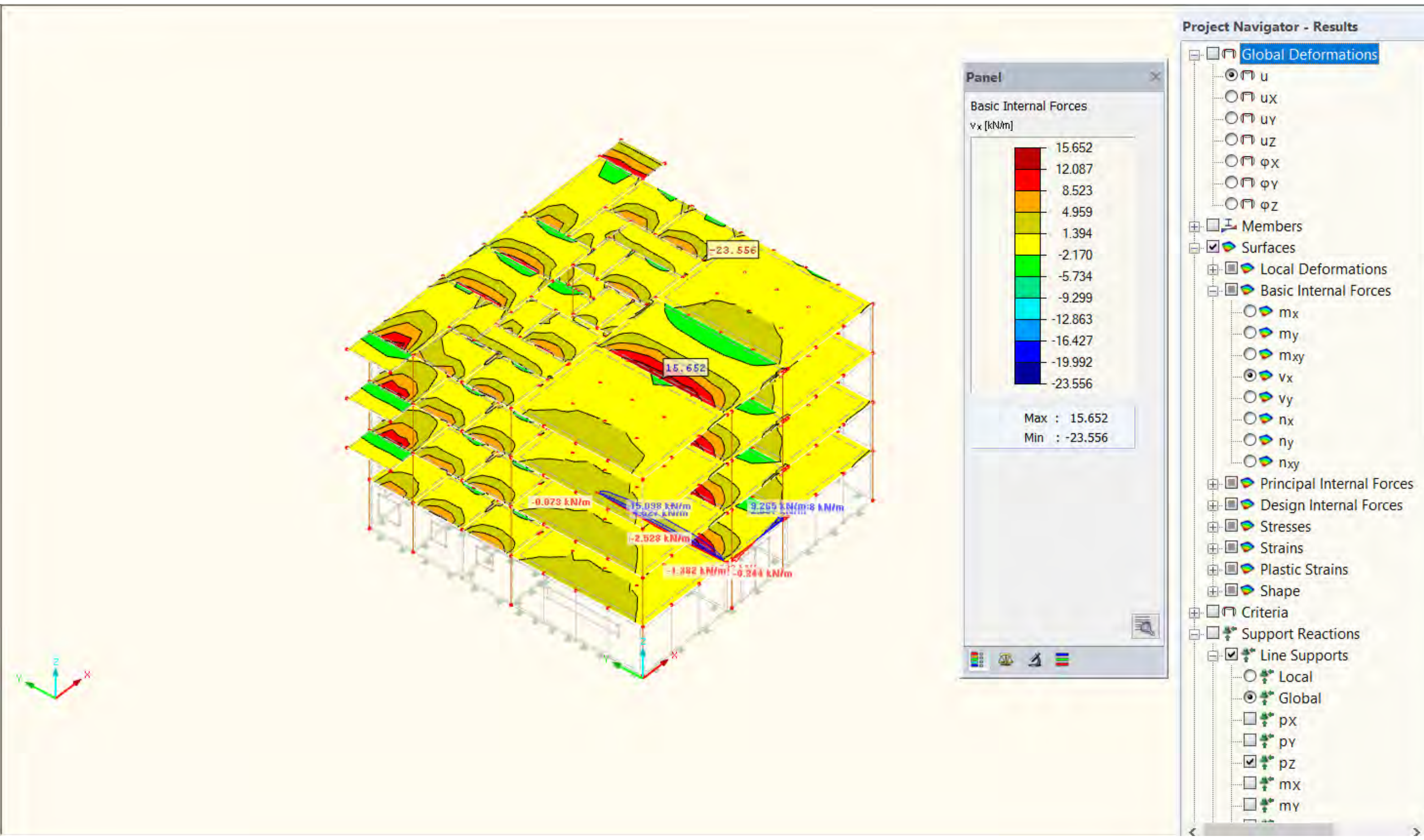


Max. Bending Moment on CLT Floor (y-direction)

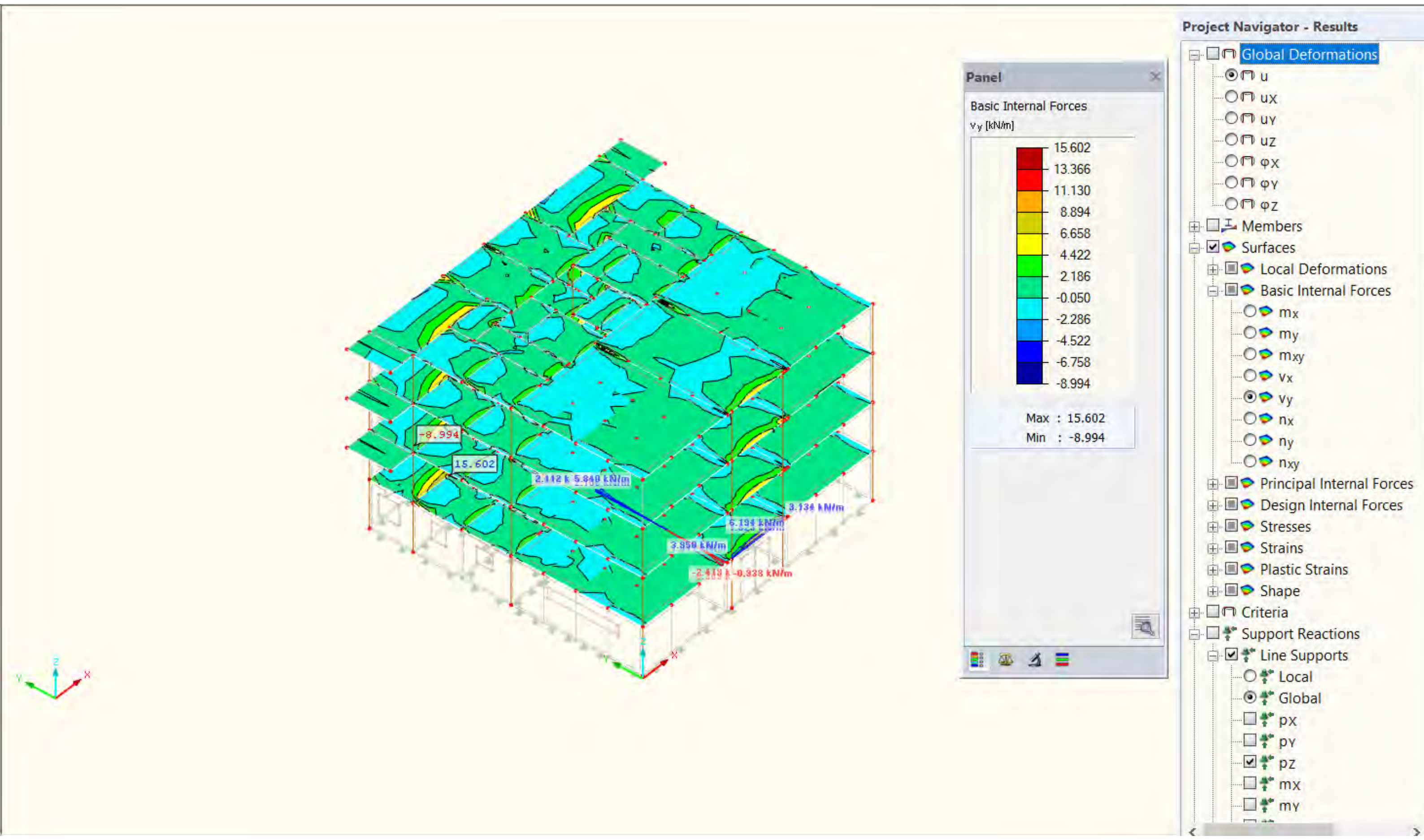


Max : 5.236
Min : -2.995

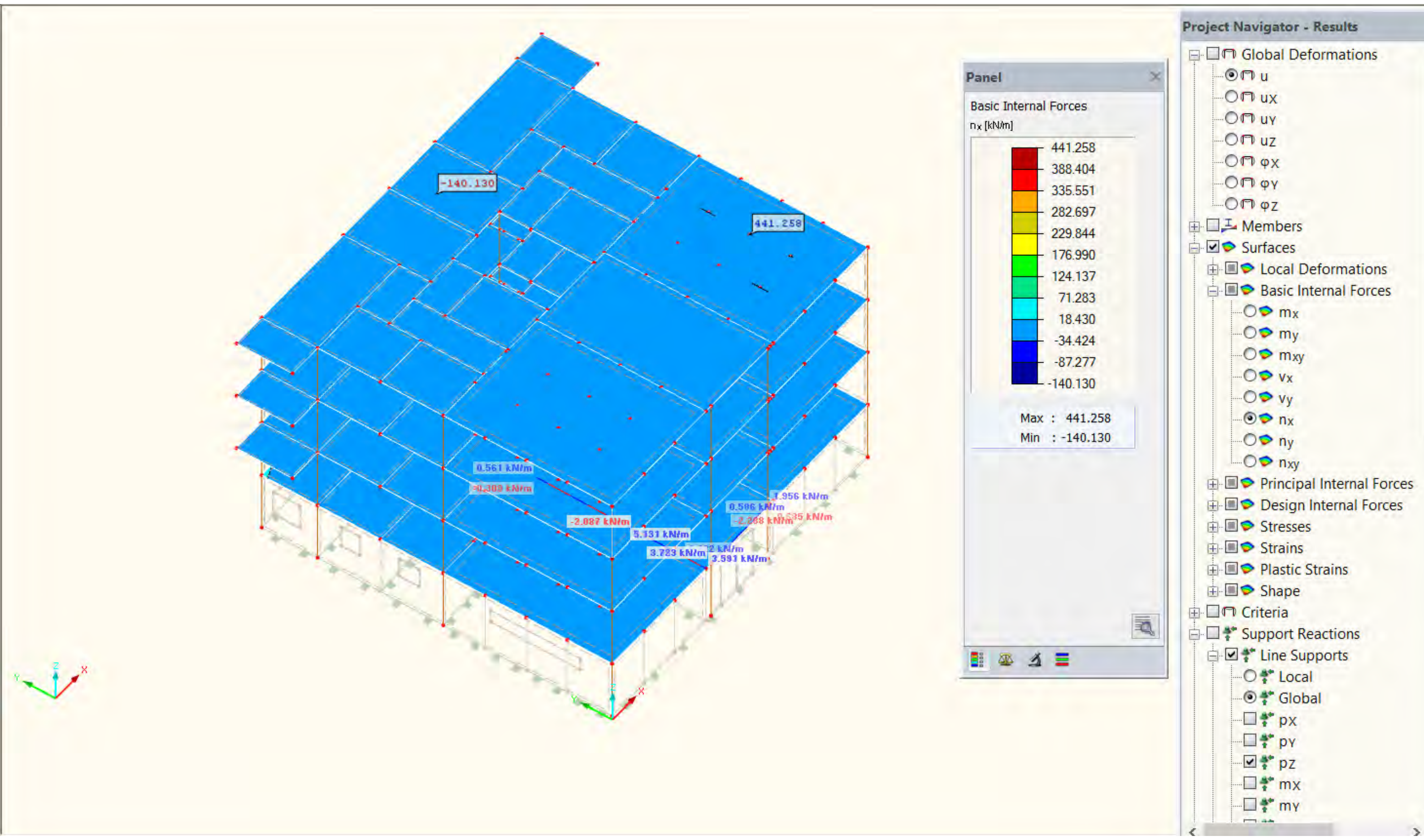
Max. Shear force on CLT Floor (x-direction)



Max. Shear force on CLT Floor (y-direction)

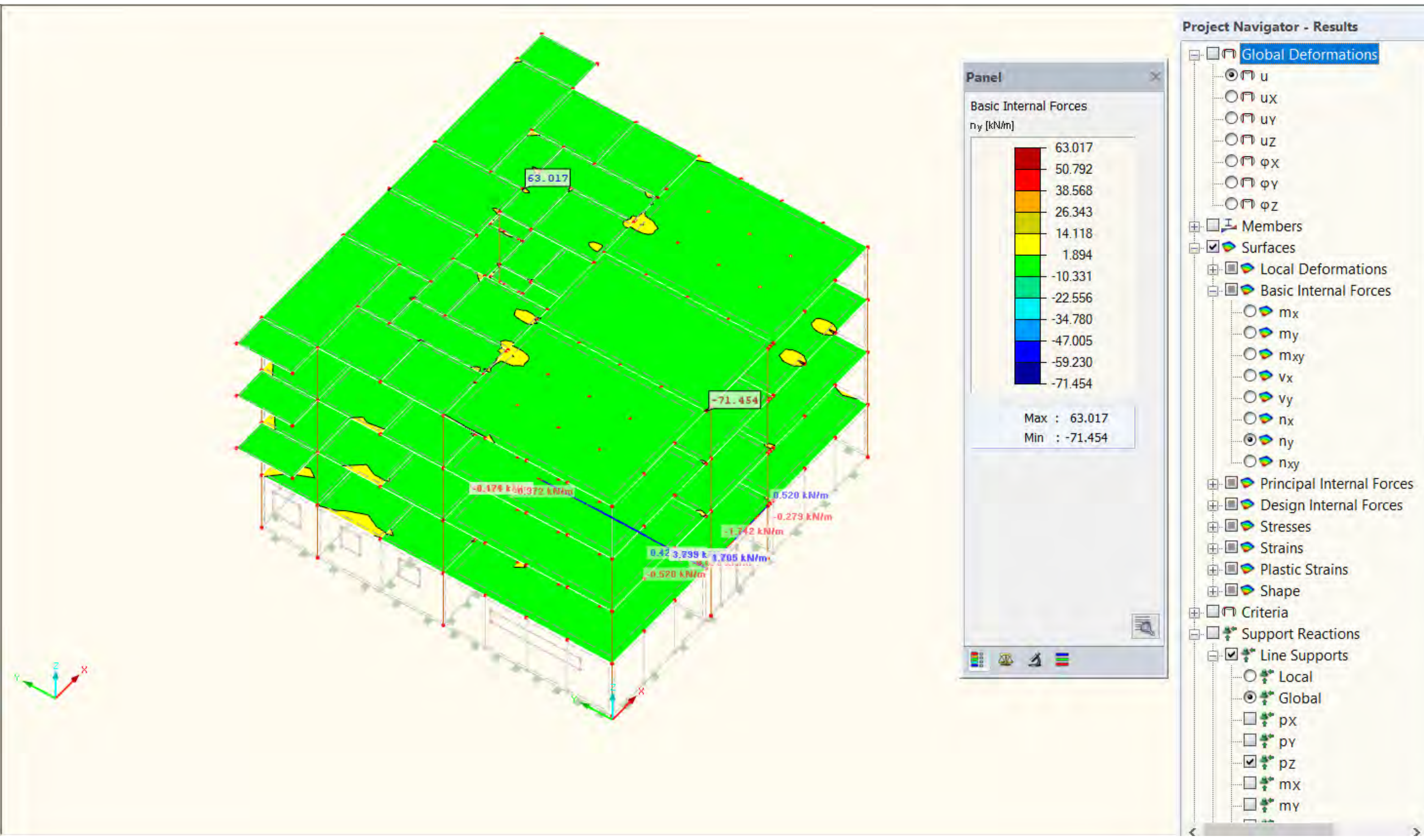


Max. Compression/Tension forces on CLT Floor (x-direction)



Max : 441.258
Min : -140.130

Max. Compression/Tension forces on CLT Floor (y-direction)



Panel

Basic Internal Forces

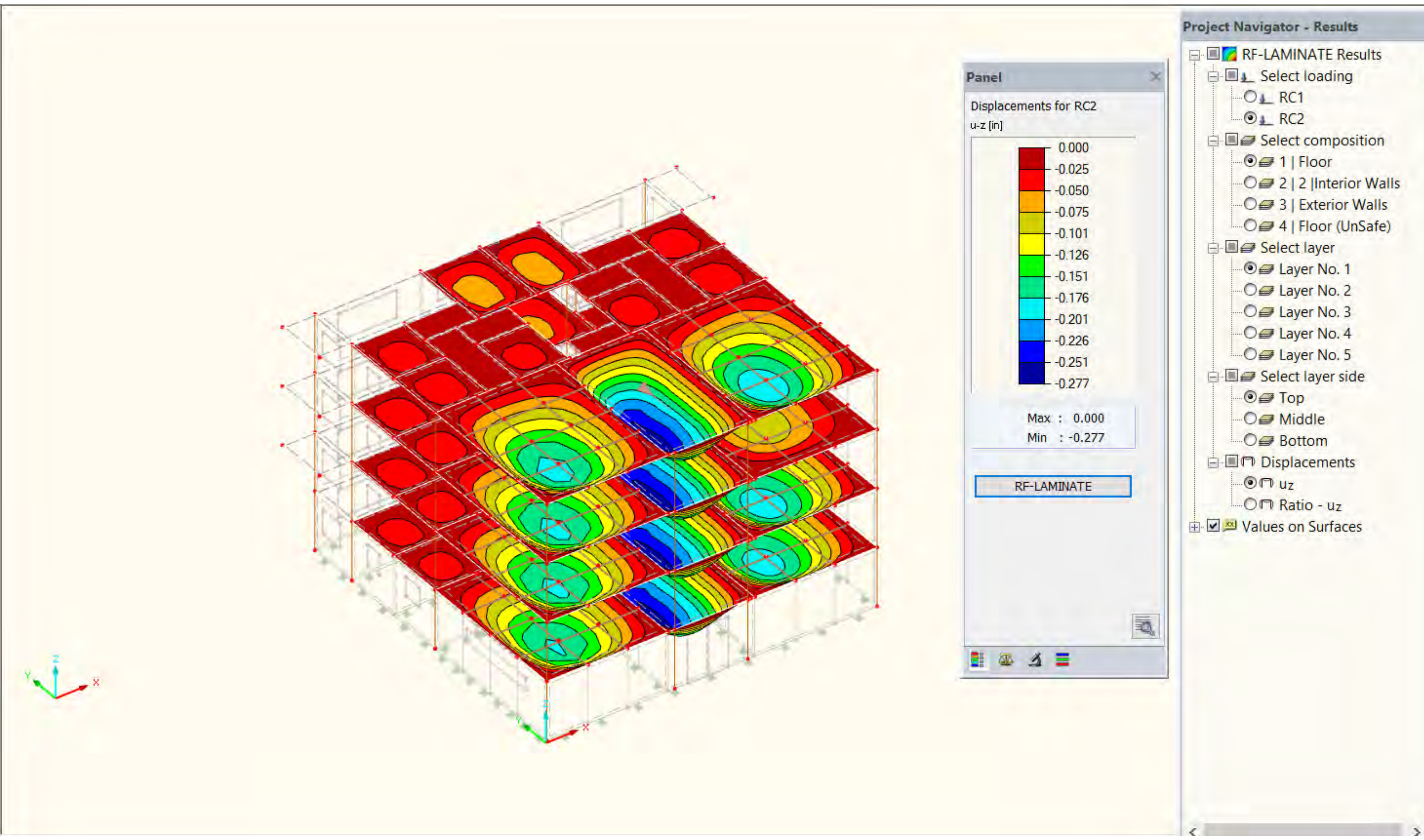
n_y [kN/m]

🟥	63.017
🟥	50.792
🟥	38.568
🟧	26.343
🟨	14.118
🟨	1.894
🟩	-10.331
🟩	-22.556
🟦	-34.780
🟦	-47.005
🟦	-59.230
🟦	-71.454

Max : 63.017

Min : -71.454

Max. Deflection on CLT Floor



Panel

Displacements for RC2

u-z [in]

0.000
-0.025
-0.050
-0.075
-0.101
-0.126
-0.151
-0.176
-0.201
-0.226
-0.251
-0.277

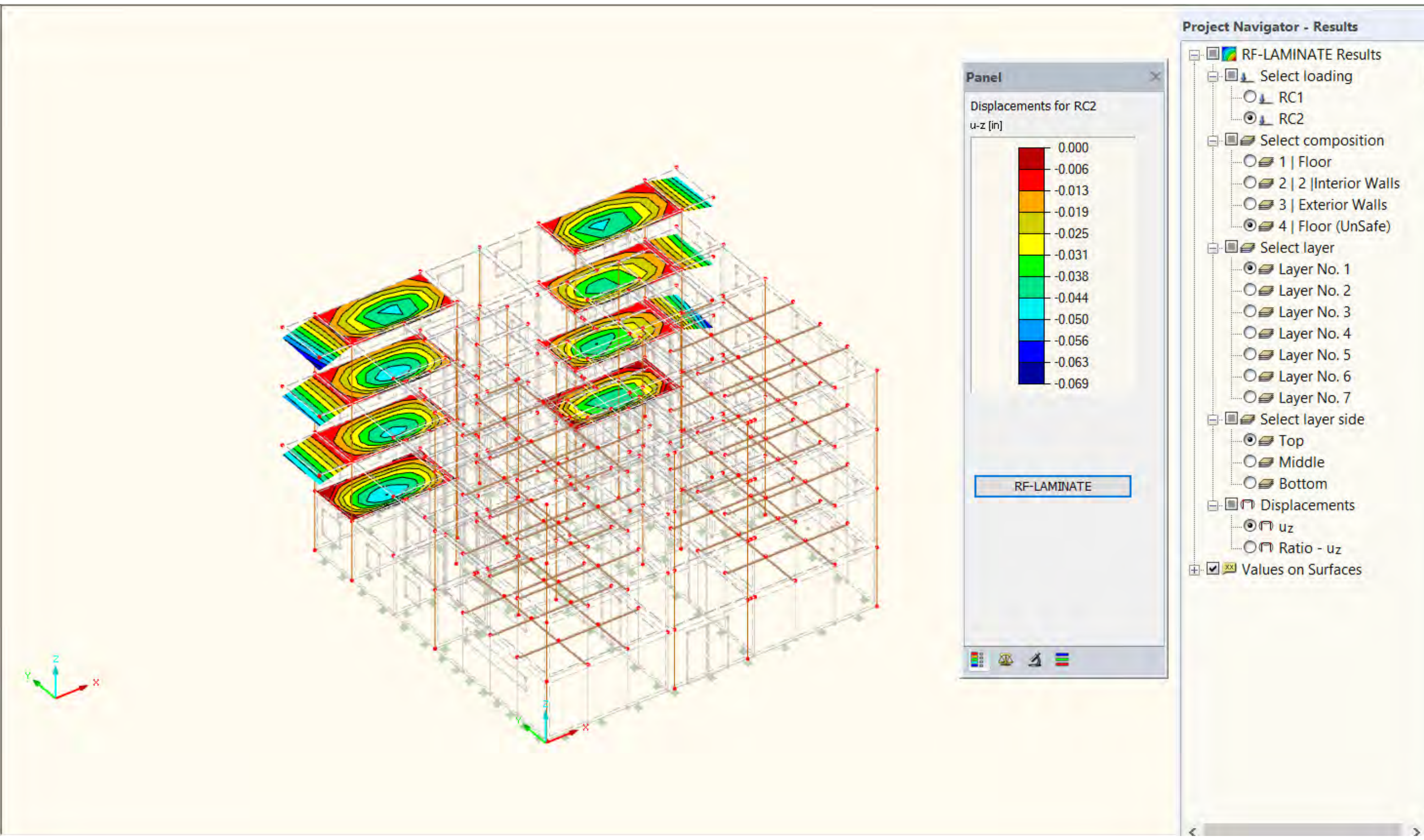
Max : 0.000

Min : -0.277

Project Navigator - Results

- RF-LAMINATE Results
  - Select loading
    - ○ RC1
    - ● RC2
  - Select composition
    - ● 1 | Floor
    - ○ 2 | 2 |Interior Walls
    - ○ 3 | Exterior Walls
    - ○ 4 | Floor (UnSafe)
  - Select layer
    - ● Layer No. 1
    - ○ Layer No. 2
    - ○ Layer No. 3
    - ○ Layer No. 4
    - ○ Layer No. 5
  - Select layer side
    - ● Top
    - ○ Middle
    - ○ Bottom
  - Displacements
    - ● uz
    - ○ Ratio - uz
- ☑ Values on Surfaces

Max. Deflection on CLT Floor



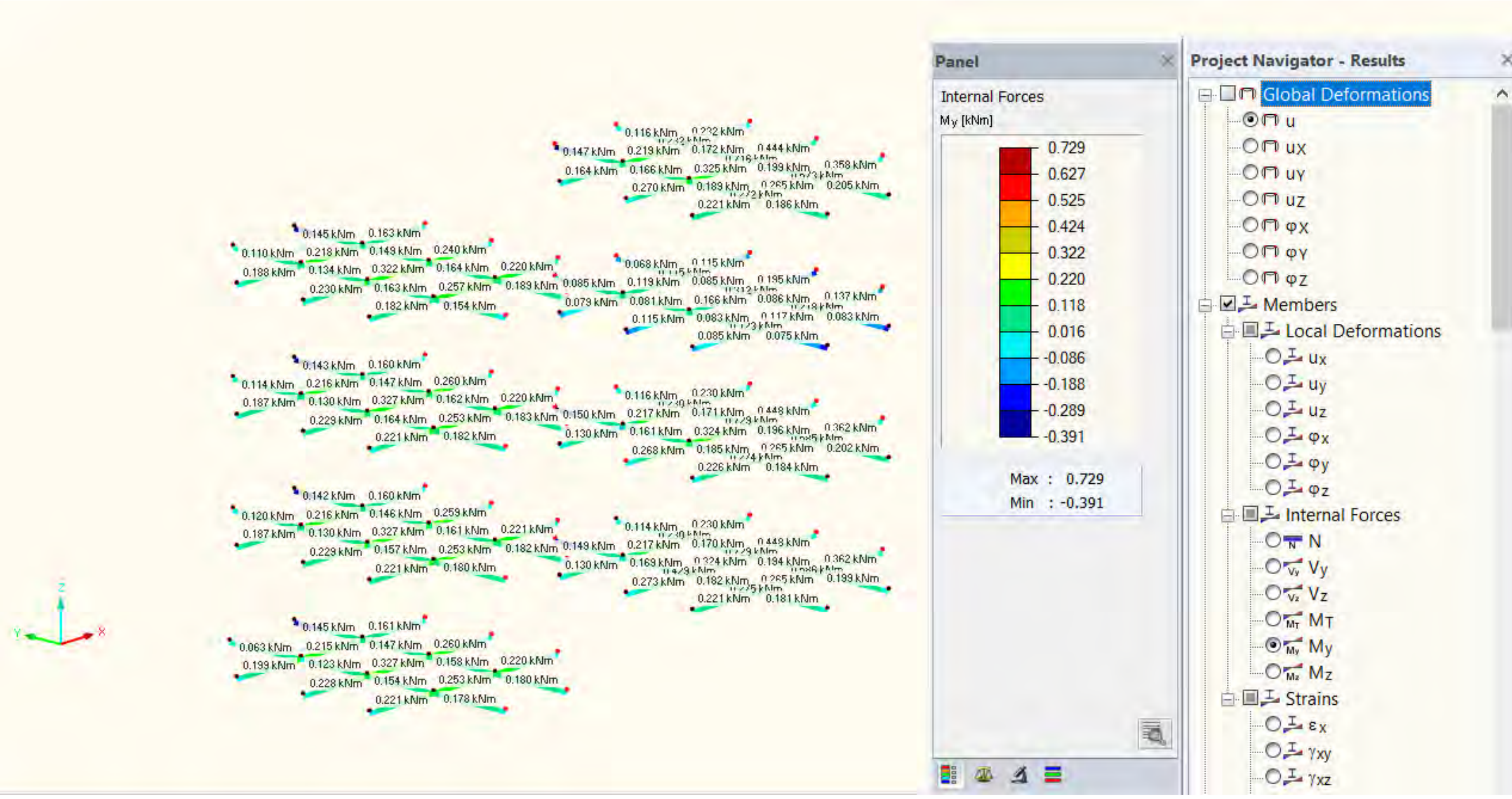
Displacements for RC2
u-z [in]

Project Navigator - Results

- RF-LAMINATE Results
  - Select loading
    - ○ RC1
    - ● RC2
  - Select composition
    - ○ 1 | Floor
    - ○ 2 | 2 |Interior Walls
    - ○ 3 | Exterior Walls
    - ● 4 | Floor (UnSafe)
  - Select layer
    - ● Layer No. 1
    - ○ Layer No. 2
    - ○ Layer No. 3
    - ○ Layer No. 4
    - ○ Layer No. 5
    - ○ Layer No. 6
    - ○ Layer No. 7
  - Select layer side
    - ● Top
    - ○ Middle
    - ○ Bottom
  - Displacements
    - ● uz
    - ○ Ratio - uz
- ☑ Values on Surfaces

11.2.3 LVL Beams

Max. Bending moment on LVL Grid Beams (y-direction)



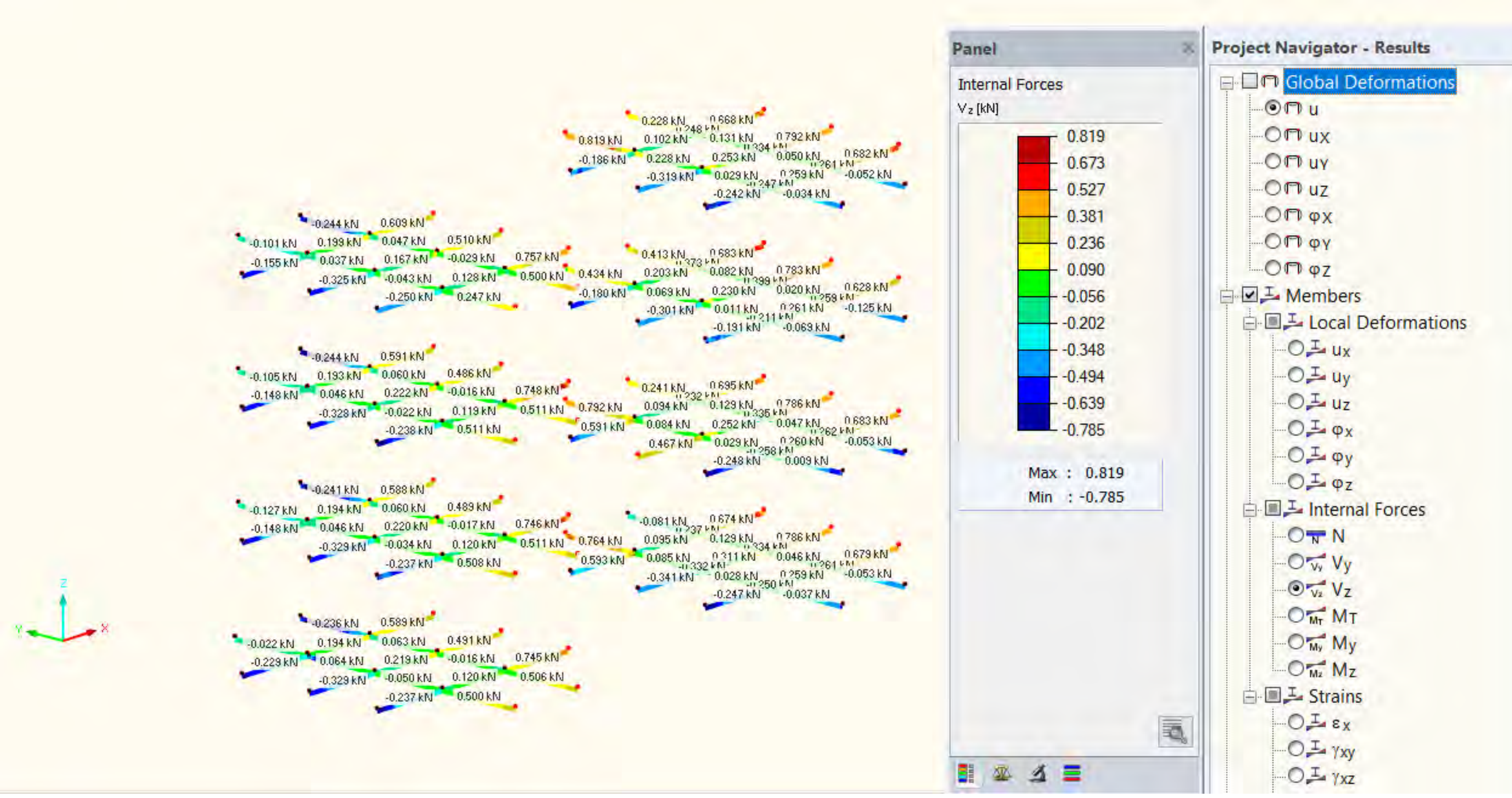
Internal Forces

My [kNm]

0.729
0.627
0.525
0.424
0.322
0.220
0.118
0.016
-0.086
-0.188
-0.289
-0.391

Max : 0.729
Min : -0.391

Max. Shear forces on LVL Grid Beams (z-direction)



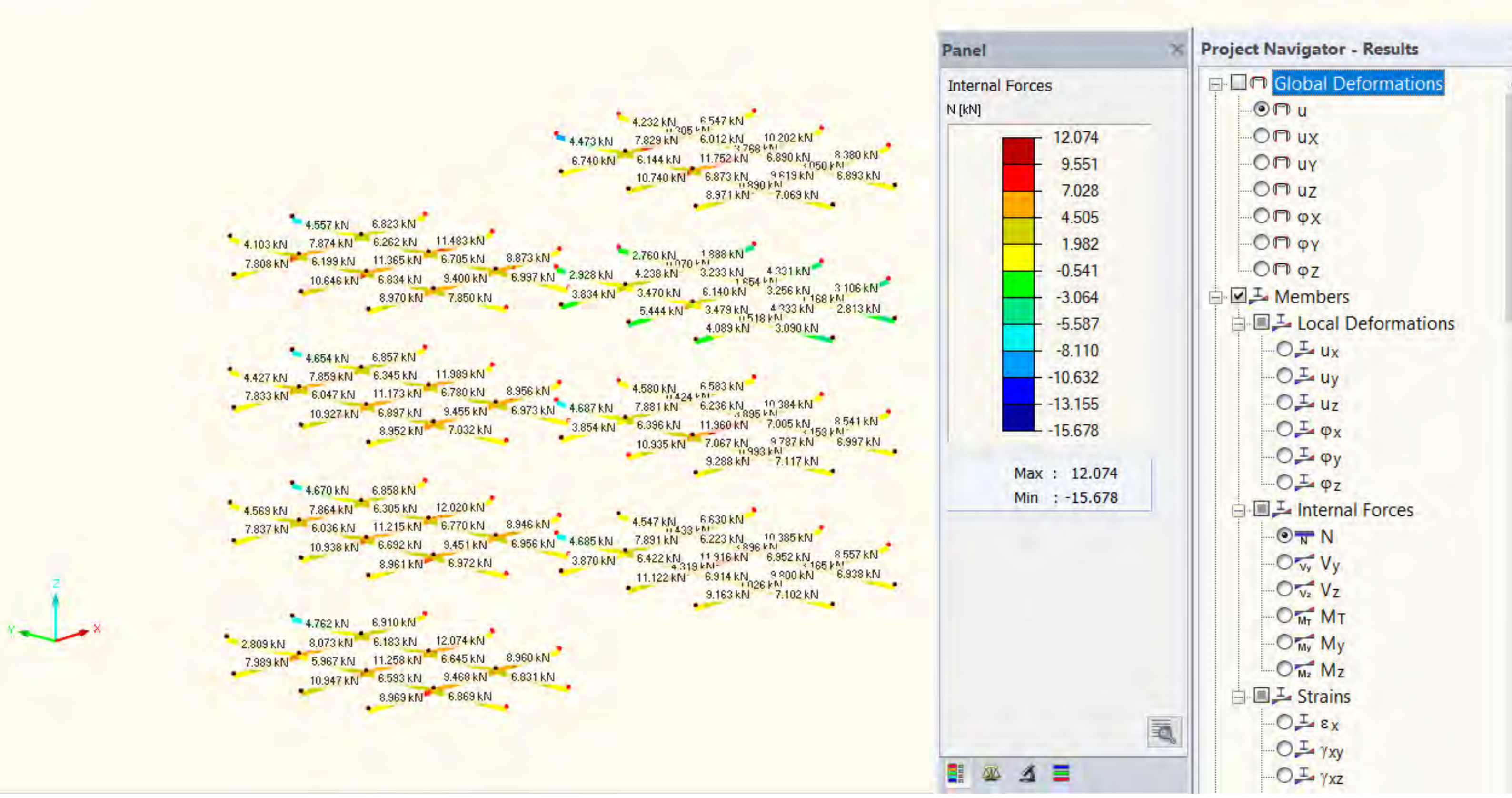
Internal Forces

V_z [kN]

0.819
0.673
0.527
0.381
0.236
0.090
-0.056
-0.202
-0.348
-0.494
-0.639
-0.785

Max : 0.819
Min : -0.785

Max. Normal forces on LVL Grid Beams



Panel

Internal Forces

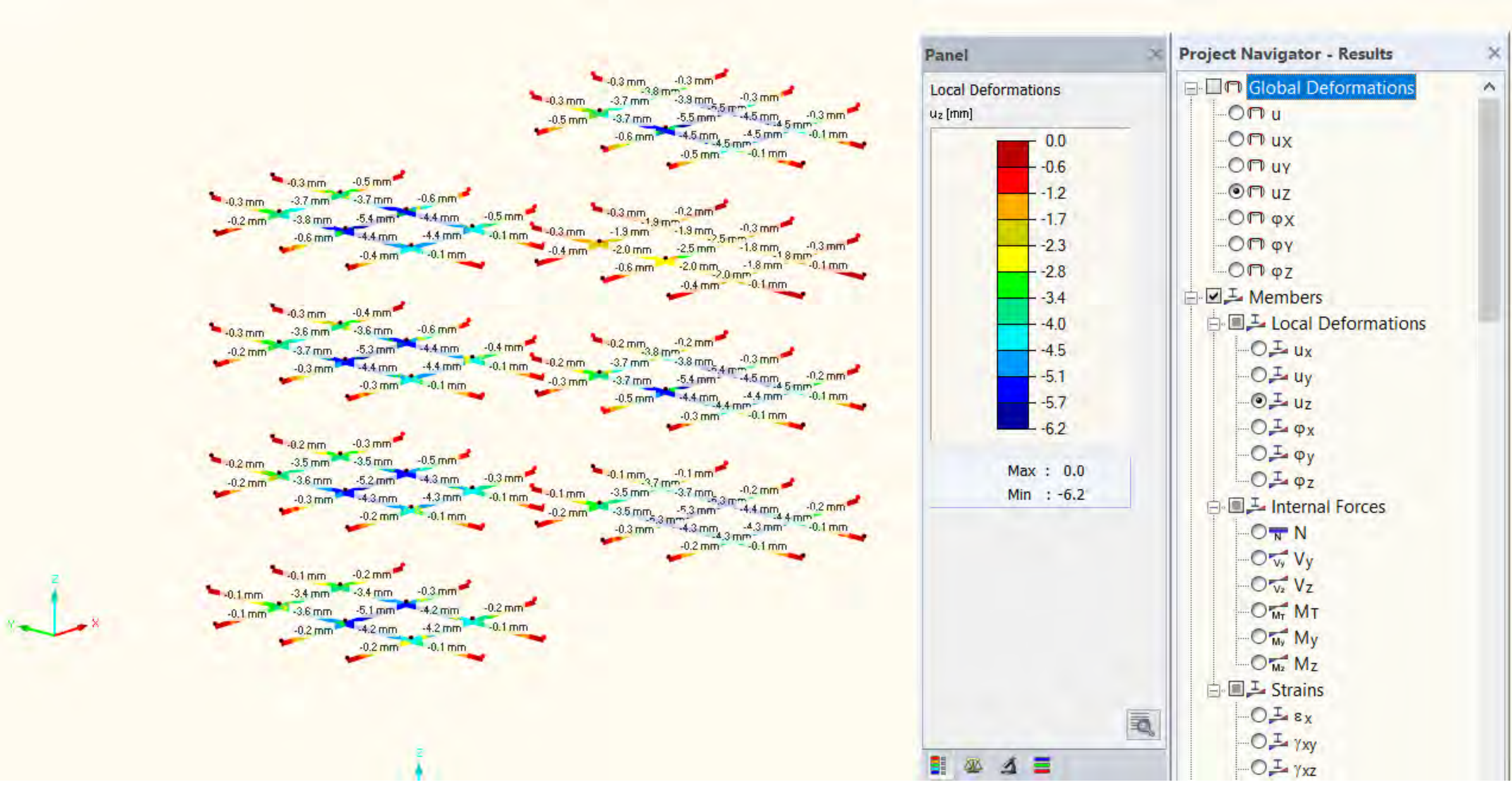
N [kN]

🟥 12.074
🟥 9.551
🟧 7.028
🟧 4.505
🟨 1.982
🟨 -0.541
🟩 -3.064
🟩 -5.587
🟦 -8.110
🟦 -10.632
🟦 -13.155
🟦 -15.678

Max : 12.074

Min : -15.678

Max. Deflection on LVL Grid Beams



Panel

Local Deformations

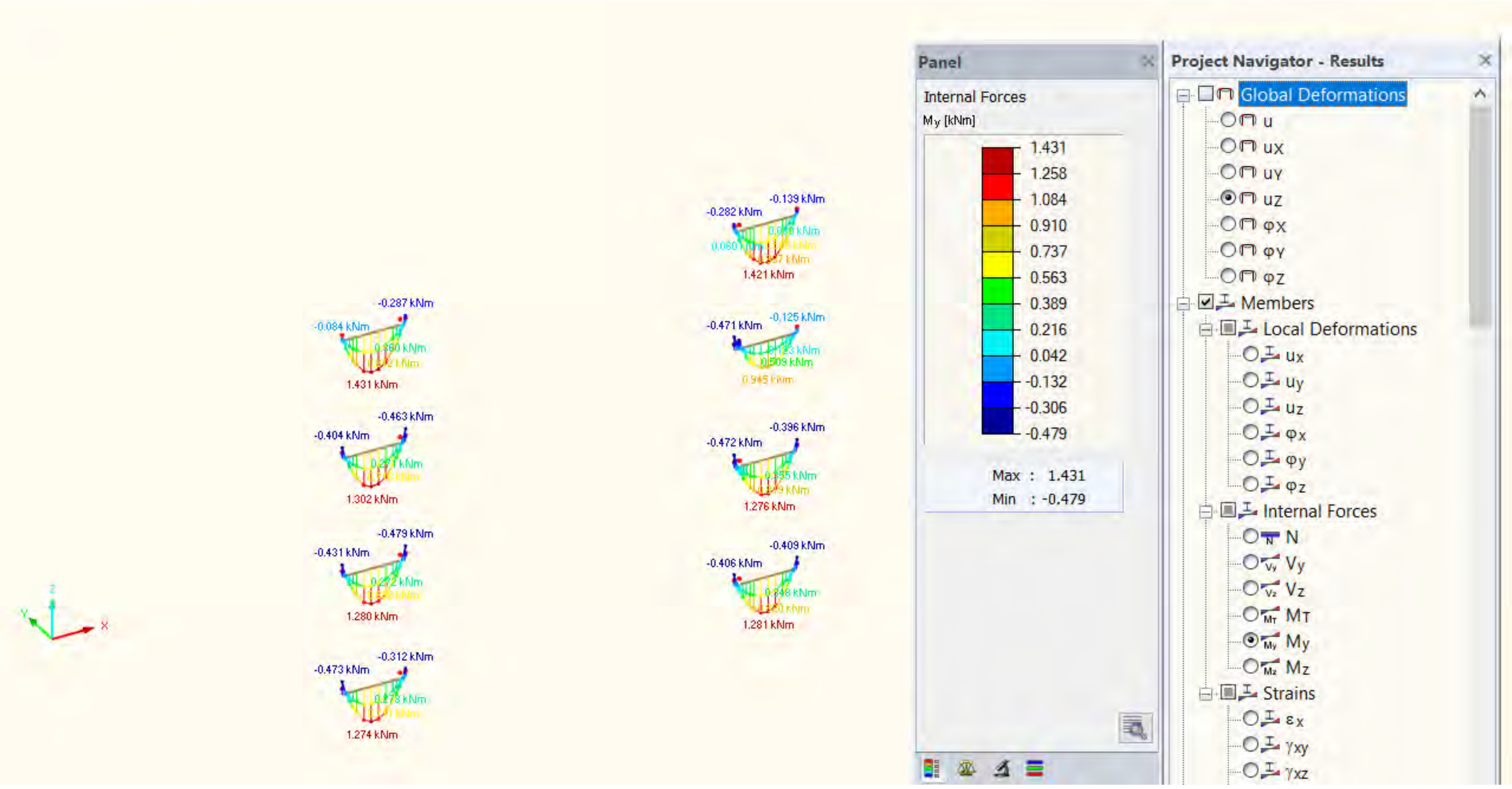
u_z [mm]

0.0
-0.6
-1.2
-1.7
-2.3
-2.8
-3.4
-4.0
-4.5
-5.1
-5.7
-6.2

Max : 0.0

Min : -6.2

Max. Bending Moment on LVL Beams (z-direction)



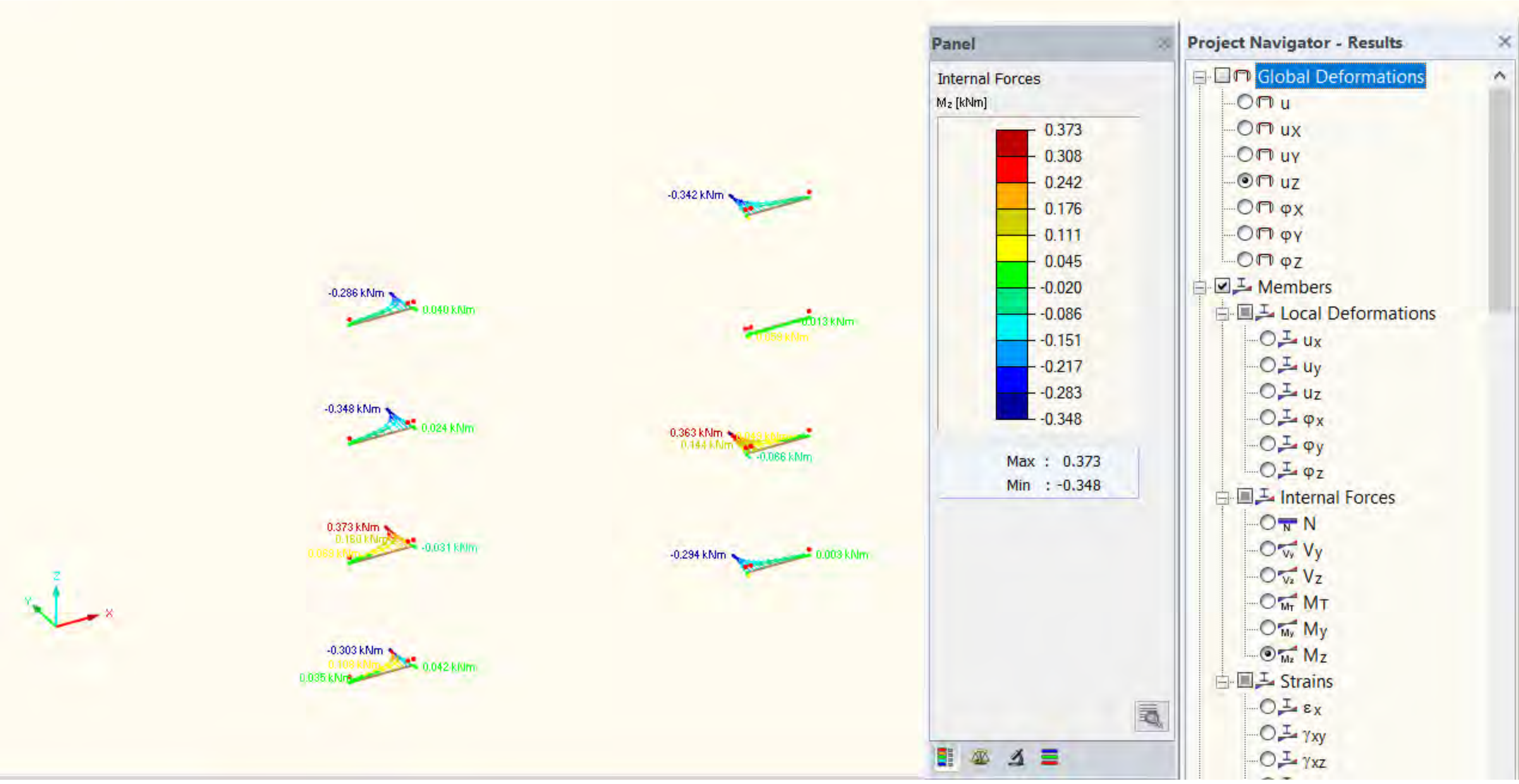
Internal Forces

My [kNm]

🟥	1.431
🟥	1.258
🟧	1.084
🟨	0.910
🟨	0.737
🟩	0.563
🟩	0.389
🟦	0.216
🟦	0.042
🟦	-0.132
🟦	-0.306
🟦	-0.479

Max : 1.431
Min : -0.479

Max. Bending Moment on LVL Beams (y-direction)



Panel

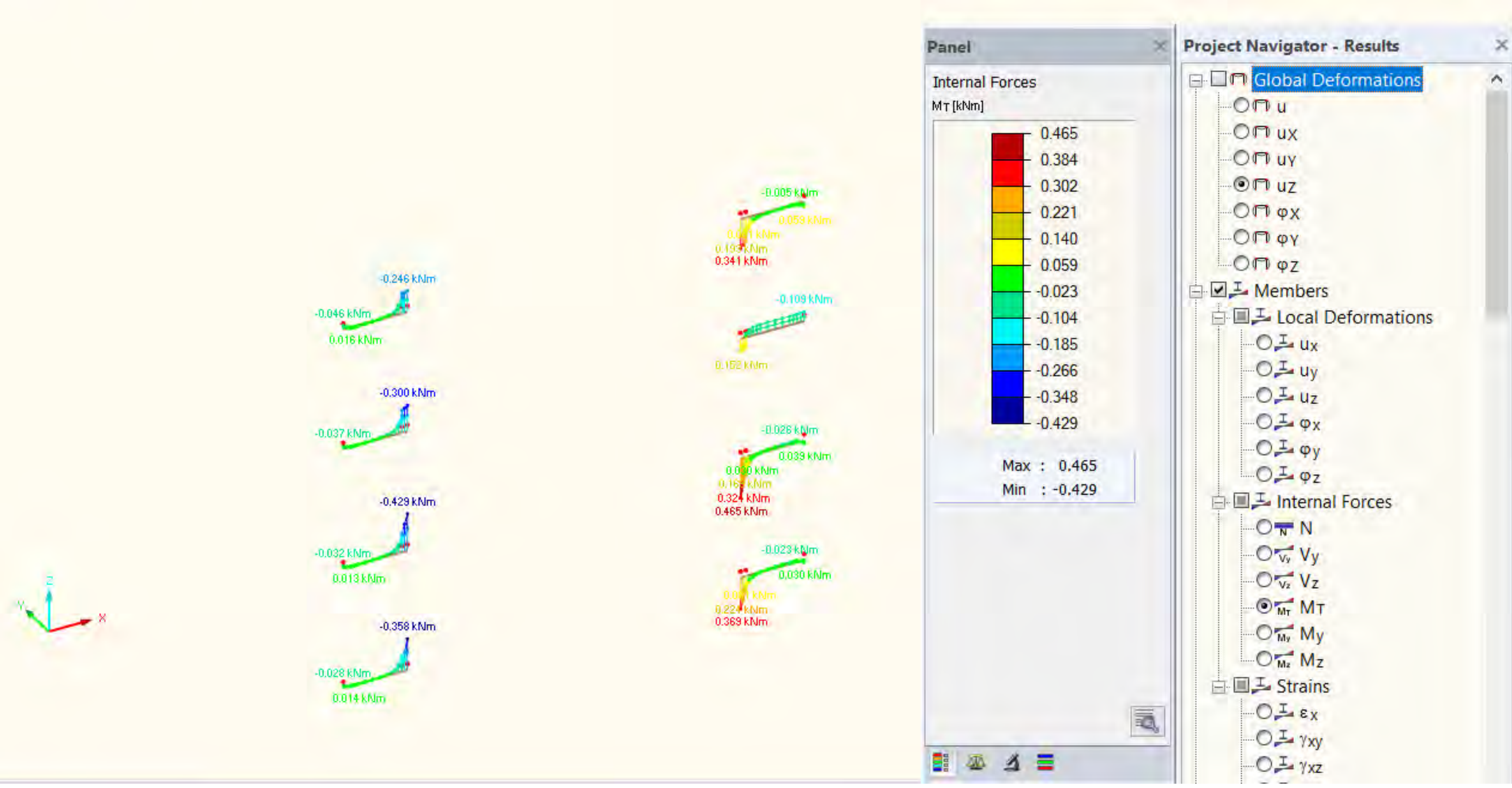
Internal Forces

M_z [kNm]

🟥	0.373
🟥	0.308
🟥	0.242
🟧	0.176
🟨	0.111
🟨	0.045
🟩	-0.020
🟩	-0.086
🟦	-0.151
🟦	-0.217
🟦	-0.283
🟦	-0.348

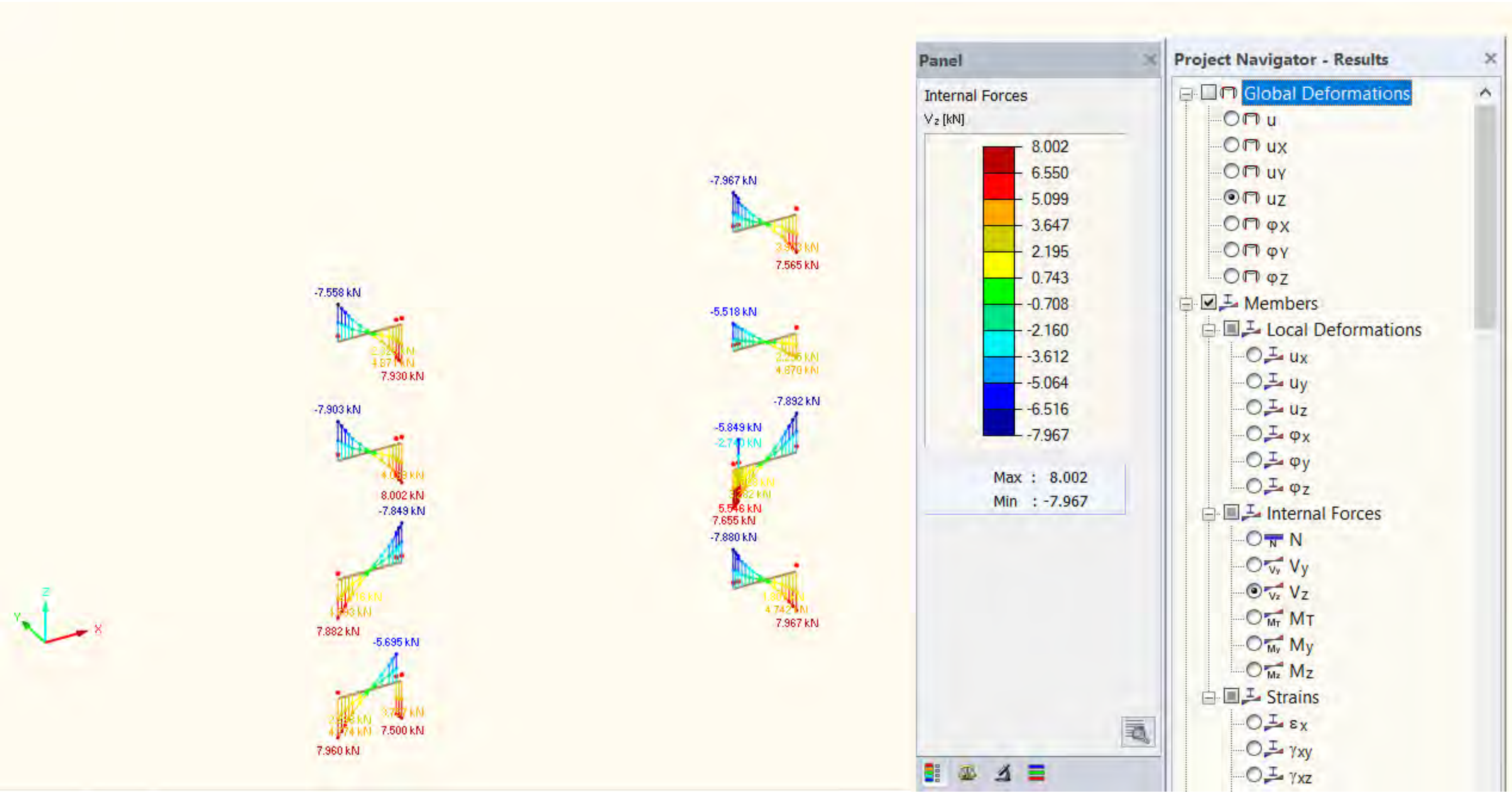
Max : 0.373
Min : -0.348

Max. Bending Moment on LVL Beams (x-direction)



Max : 0.465
Min : -0.429

Max. Shear forces on LVL Beams (z-direction)



Panel

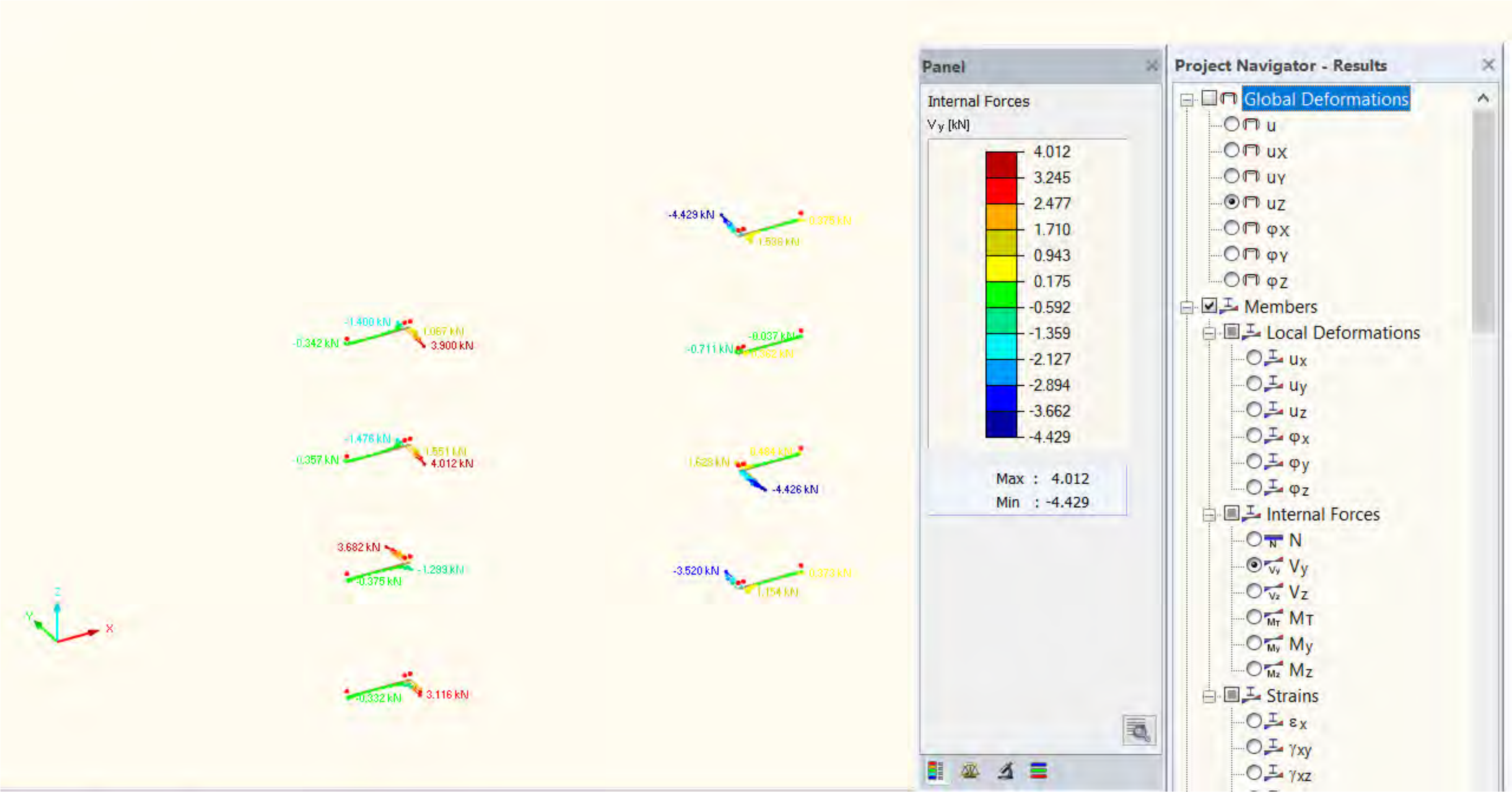
Internal Forces

Vz [kN]

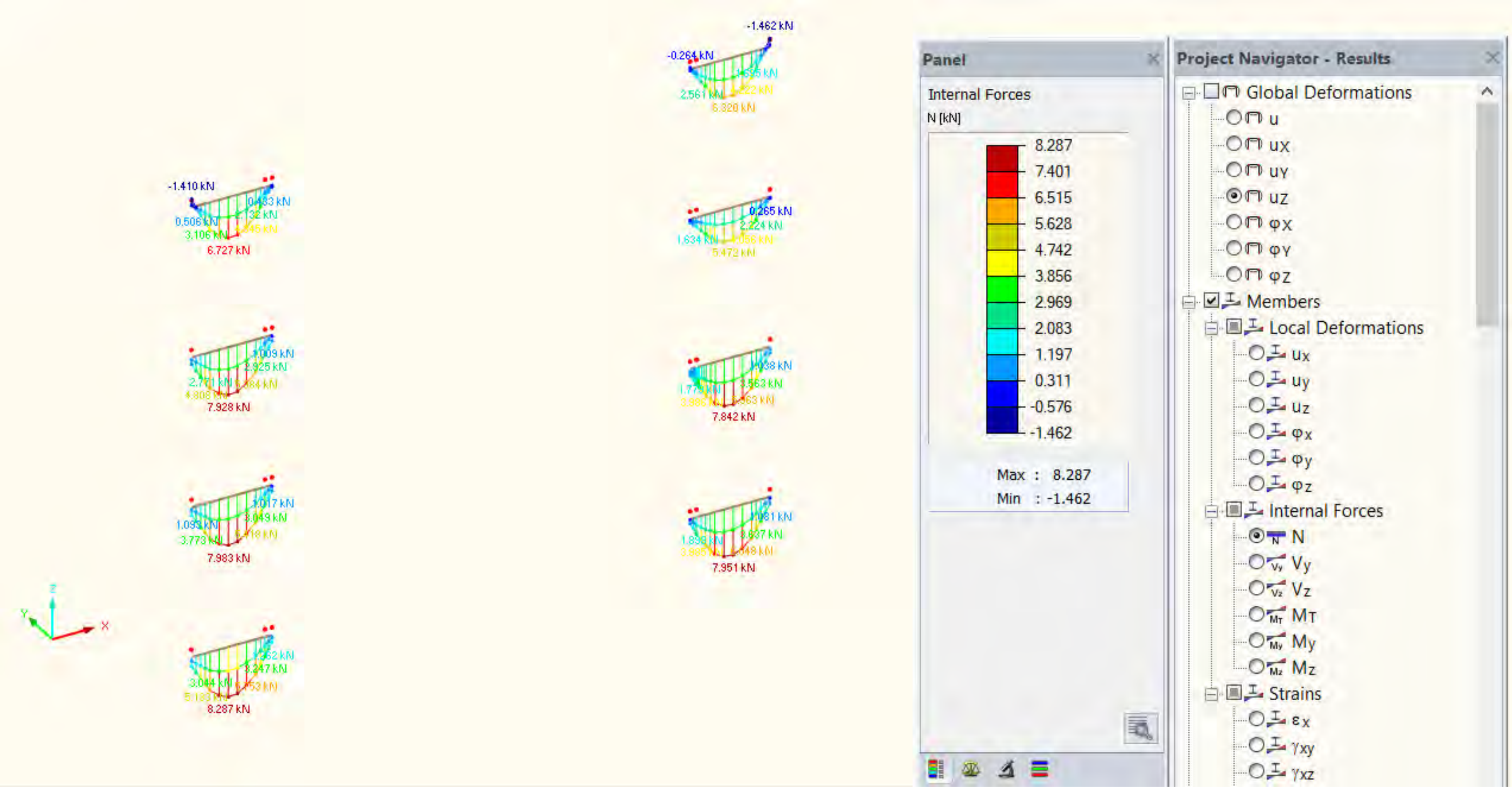
8.002
6.550
5.099
3.647
2.195
0.743
-0.708
-2.160
-3.612
-5.064
-6.516
-7.967

Max : 8.002
Min : -7.967

Max. Shear forces on LVL Beams (y-direction)

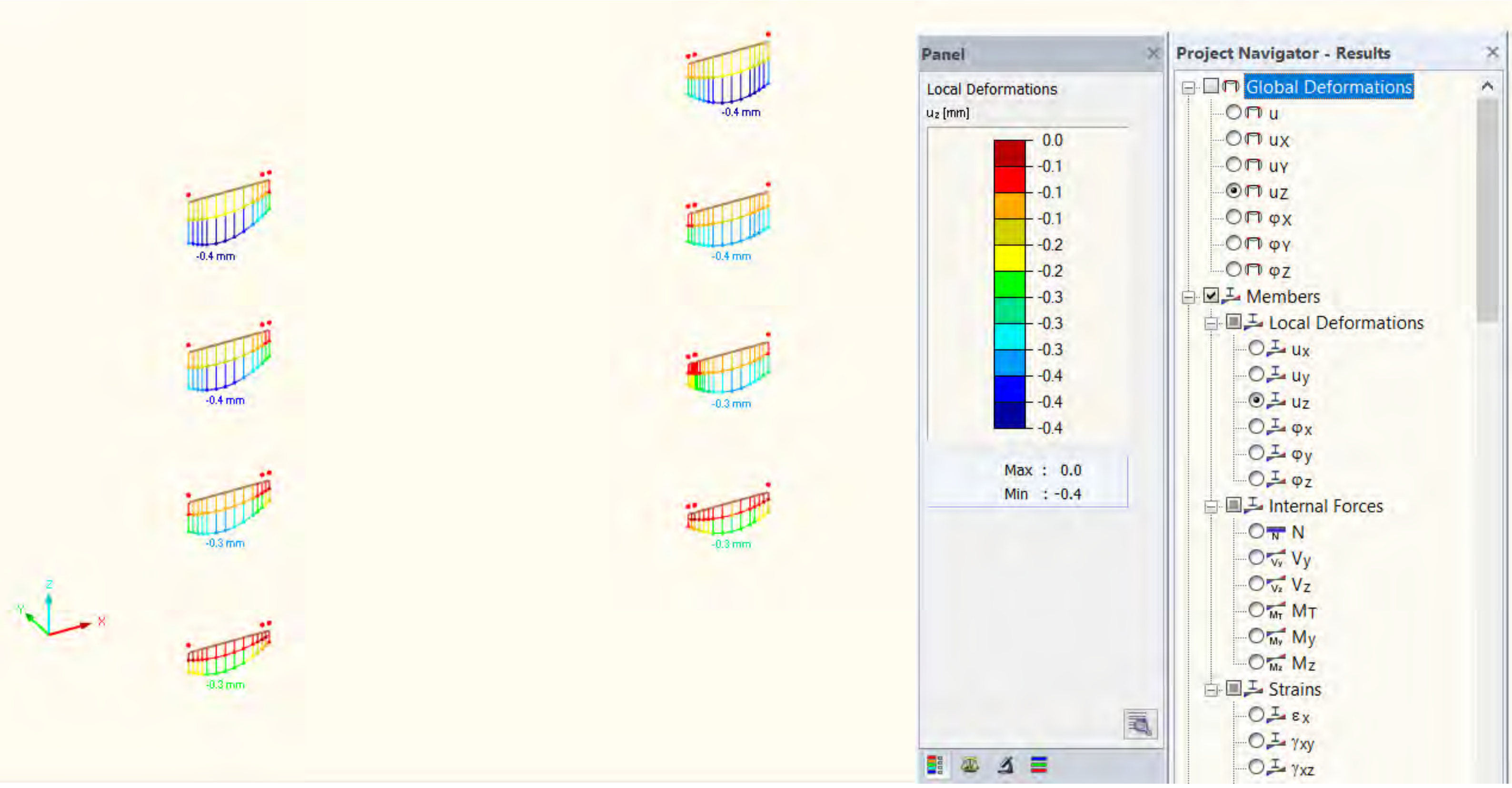


Max. Normal forces on LVL Beams



Max : 8.287
Min : -1.462

Max. Defection on LVL Beams (z-direction)



Panel

Local Deformations

uz [mm]

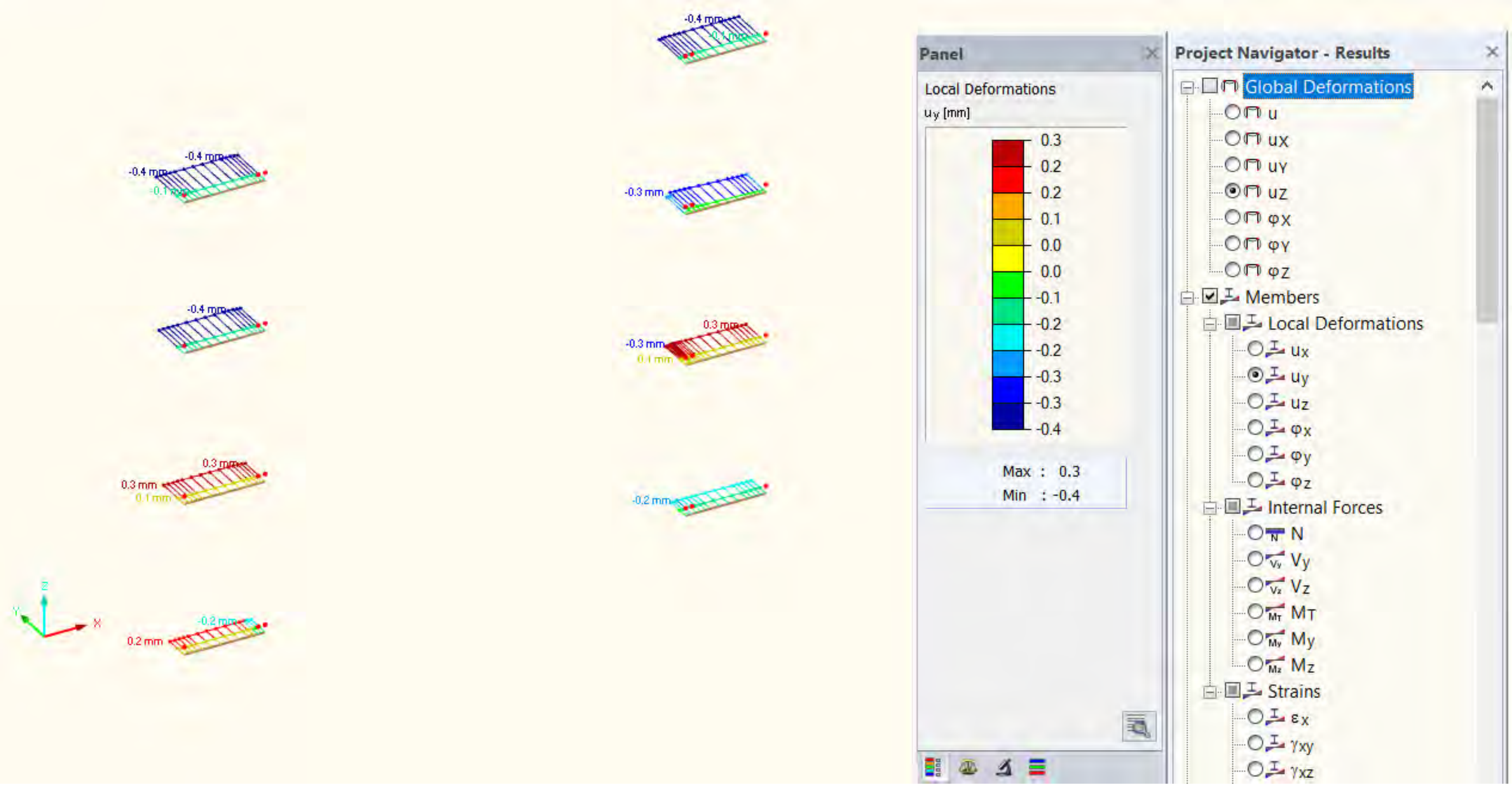
🟥	0.0
🟥	-0.1
🟧	-0.1
🟨	-0.1
🟨	-0.2
🟩	-0.2
🟩	-0.3
🟩	-0.3
🟦	-0.3
🟦	-0.4
🟦	-0.4
🟦	-0.4

Max : 0.0
Min : -0.4

Project Navigator - Results

- ☐ Global Deformations
  - ○ u
  - ○ uX
  - ○ uY
  - ● uZ
  - ○ φX
  - ○ φY
  - ○ φZ
- ☑ Members
  - Local Deformations
    - ○ ux
    - ○ uy
    - ● uz
    - ○ φx
    - ○ φy
    - ○ φz
  - Internal Forces
    - ○ N
    - ○ Vy
    - ○ Vz
    - ○ MT
    - ○ My
    - ○ Mz
  - Strains
    - ○ εx
    - ○ γxy
    - ○ γxz

Max. Defection on LVL Beams (y-direction)



Local Deformations

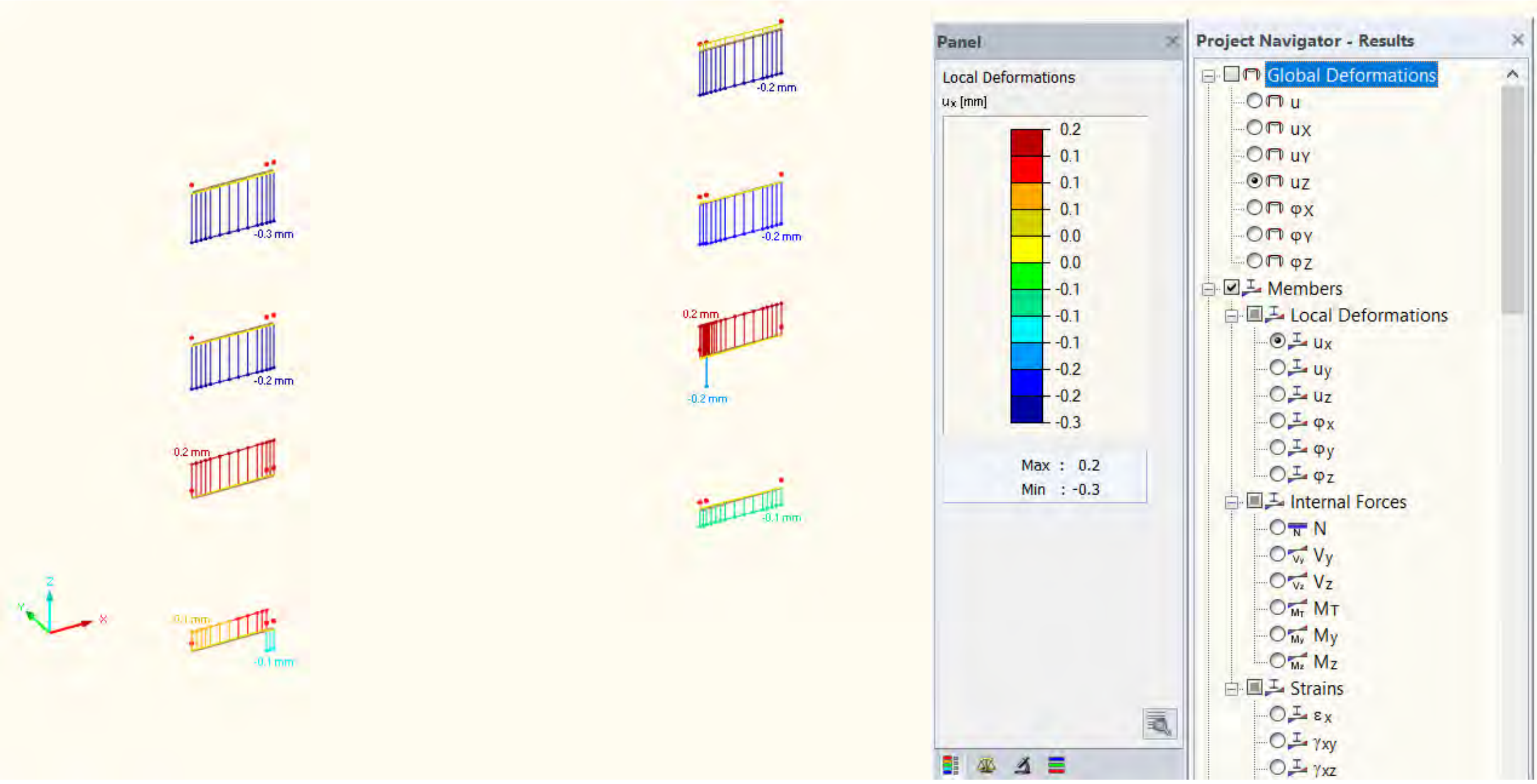
u_y [mm]

0.3
0.2
0.2
0.1
0.0
0.0
-0.1
-0.2
-0.2
-0.3
-0.3
-0.4

Max : 0.3

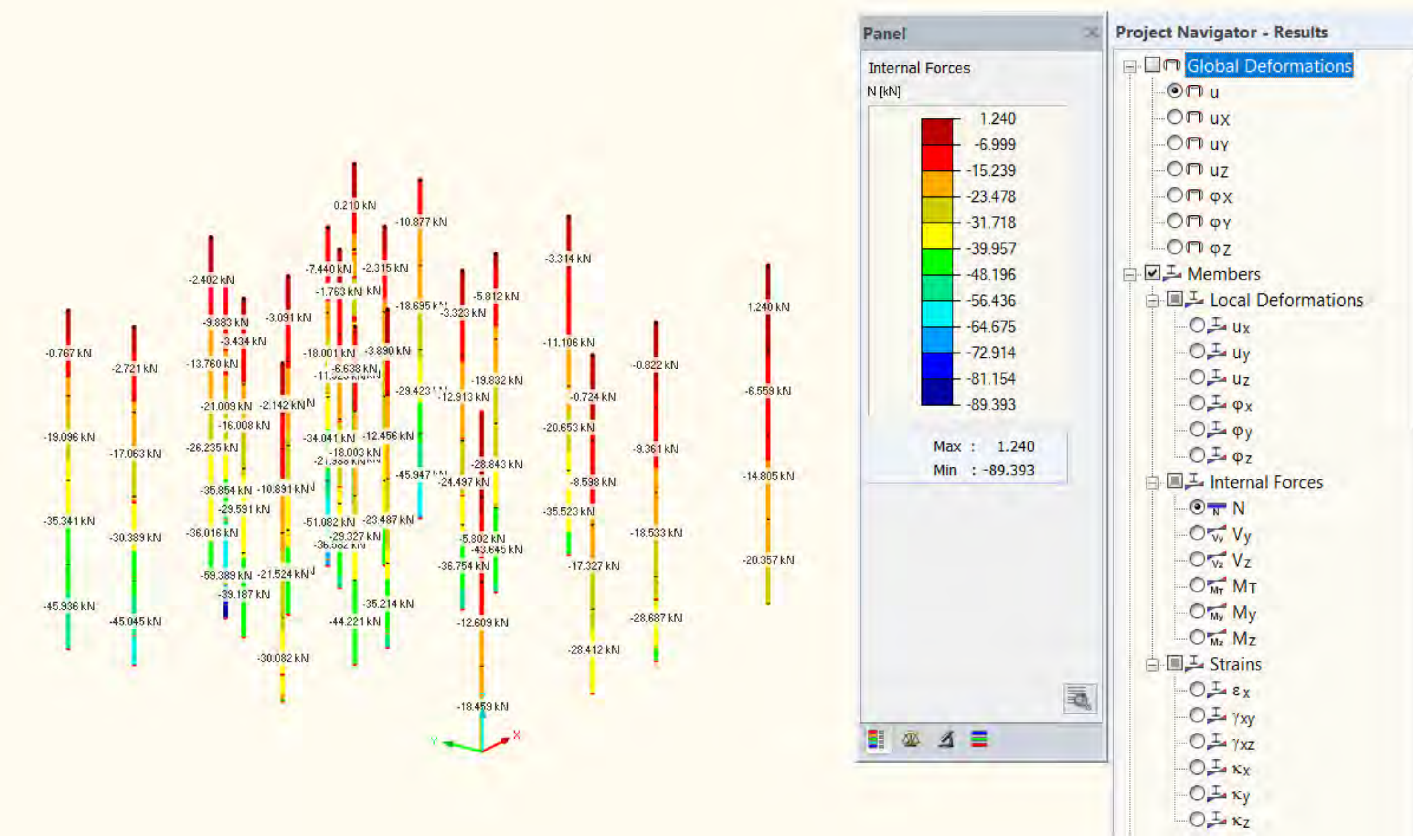
Min : -0.4

Max. Defection on LVL Beams (x-direction)



11.2.4 Glulam Columns

Max. Normal forces on GLULAM Columns



Panel

Internal Forces

N [kN]

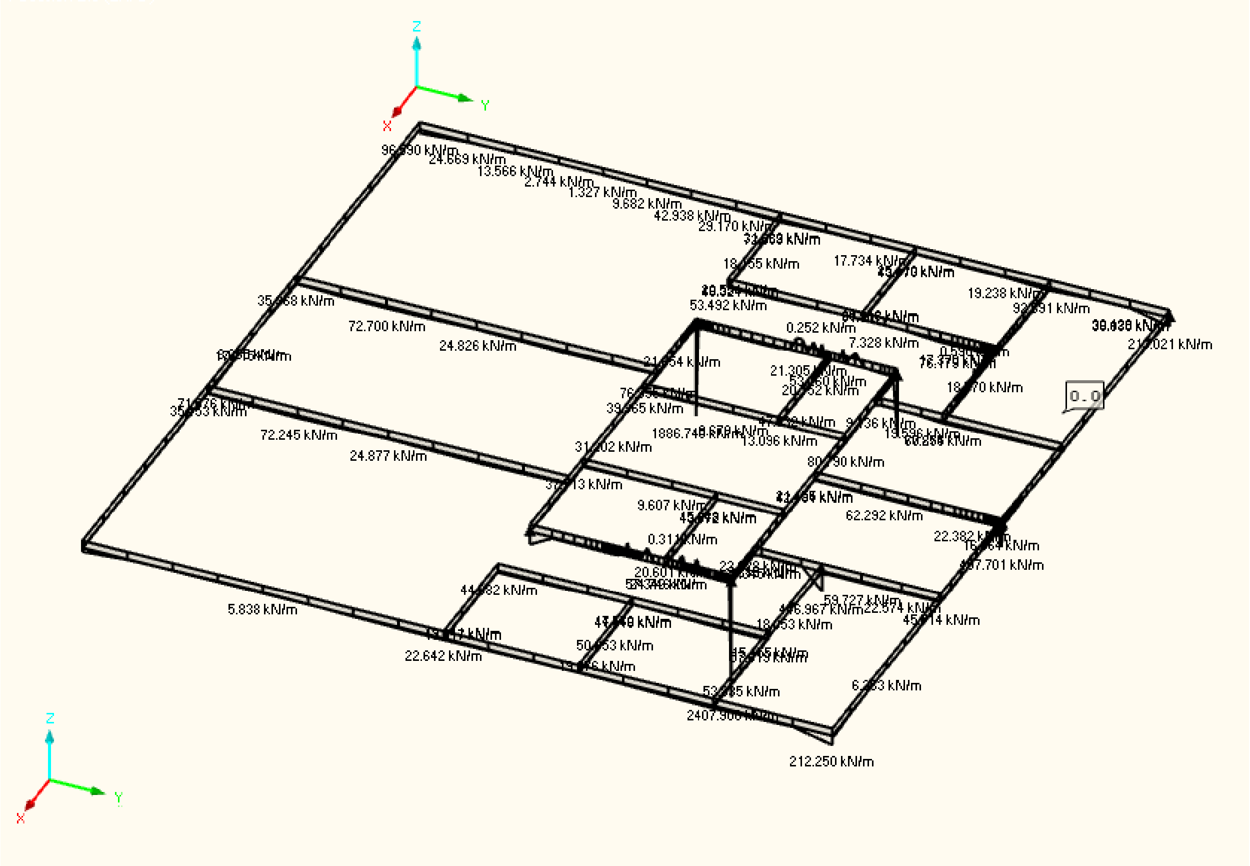
🟥 1.240
🟥 -6.999
🟧 -15.239
🟨 -23.478
🟨 -31.718
🟩 -39.957
🟩 -48.196
🟦 -56.436
🟦 -64.675
🟦 -72.914
🟦 -81.154
🟦 -89.393

Max : 1.240

Min : -89.393

11.2.5 Foundations

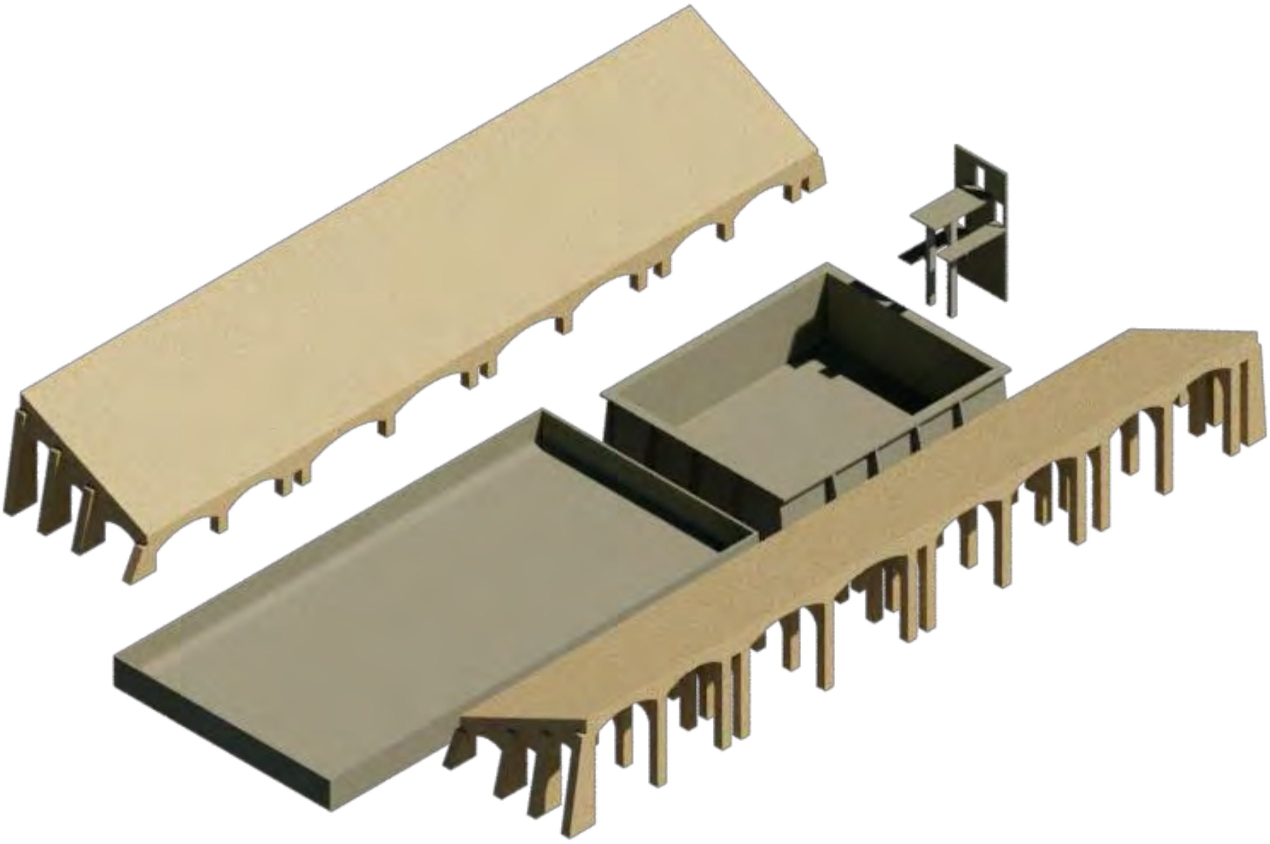
Line support Reactions



TECHNICAL DESIGN CALCULATION REPORT

Egyptian Aquatic Centre

Part 2

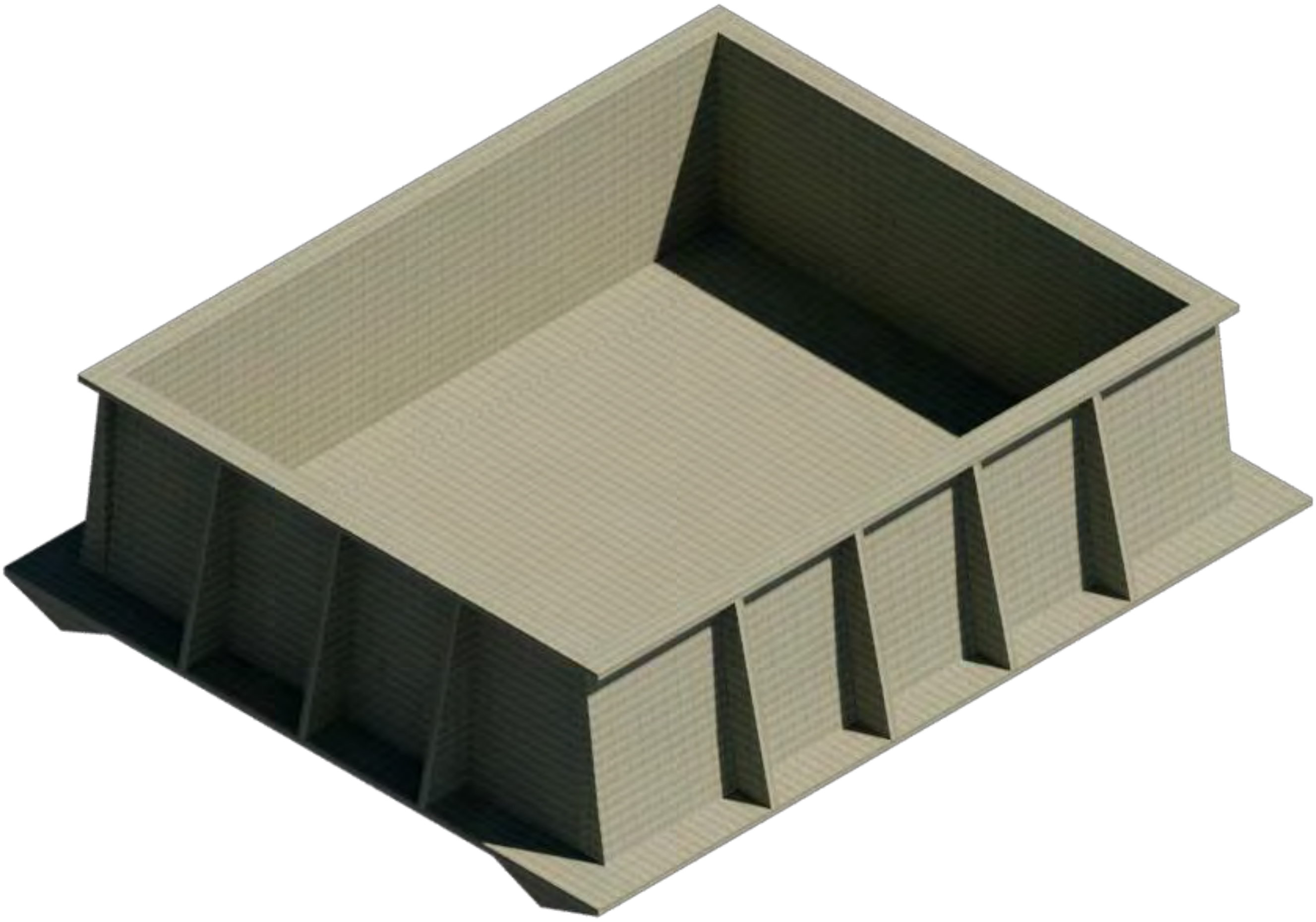


Abstract

Masonry structures have been used in ancient Egypt in many monuments like temples, pyramids and sphynx; thus, masonry structures expresses Egyptian civilization. Masonry structures expresses also the greatness of other civilization in Architecture like Romanian civilization with coliseum. One of important types of masonry is stone. Stone is a very strong material, durable, resistance to weather conditions, fire and insects proof. Stone considered also as eco-friendly material. Based on the concept of mixing between ancient Egyptian civilization and modern design solution and to achieve the sustainability, Egyptian Aquatic Centre is proposed in Aswan province. The centre contains two Olympic swimming pools, diving platform and masonry amphitheater.

Egyptian Aquatic Centre

Short Course Pool



I. Design codes and standards

1- **ECP (203–2018)**

Egyptian Code of Practice for Design and Construction of Concrete Structures.

2- **ECP (201-2010)**

Egyptian Code for Loading on Buildings.

3- **FINA FACILITIES RULES (2017-2021)**

International Swimming Federation.

1. INTRODUCTION

The concept for structural design focuses on satisfying both the functional and the economic requirements of the building without jeopardizing its aesthetic and architectural features. This Calculation Report presents the structural engineering aspect of the works due for the development construction work of Egypt Aquatic Centre.

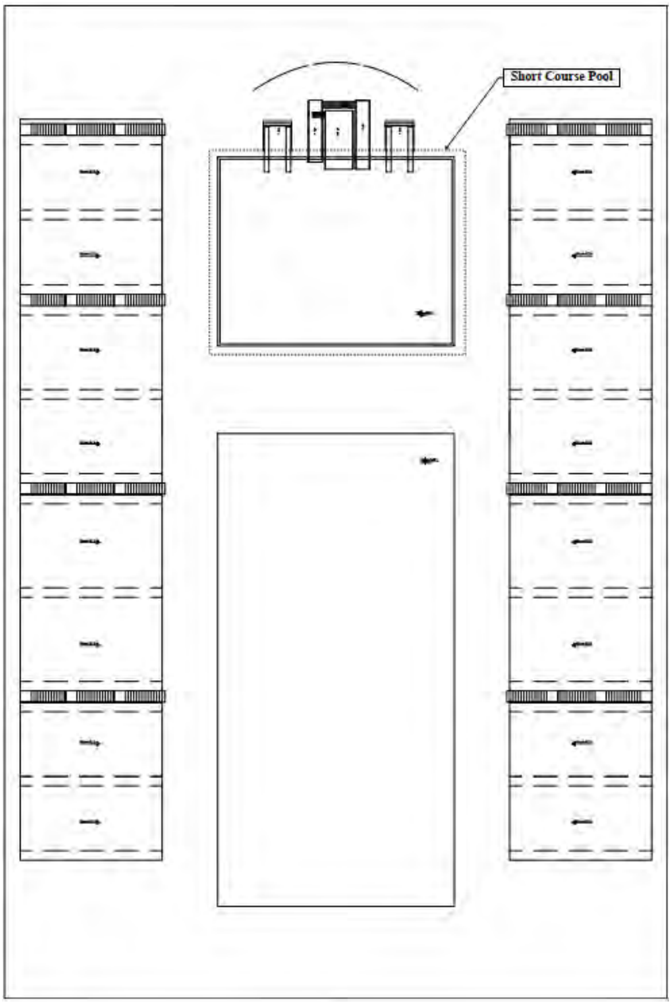


Figure 1-1 Short Course Pool location

2. Pool Drawings Details

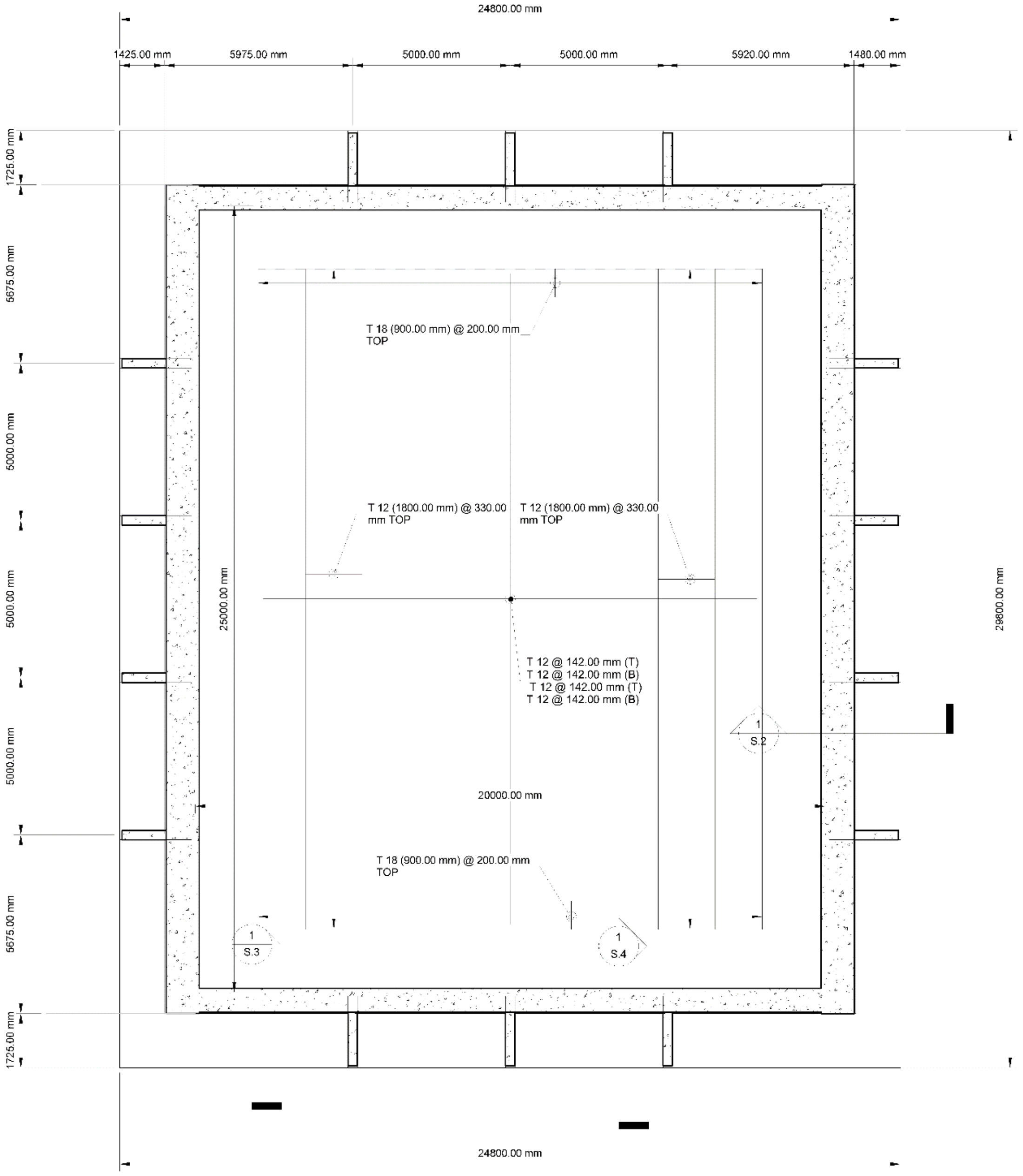


Figure 2-1 Pool Plan

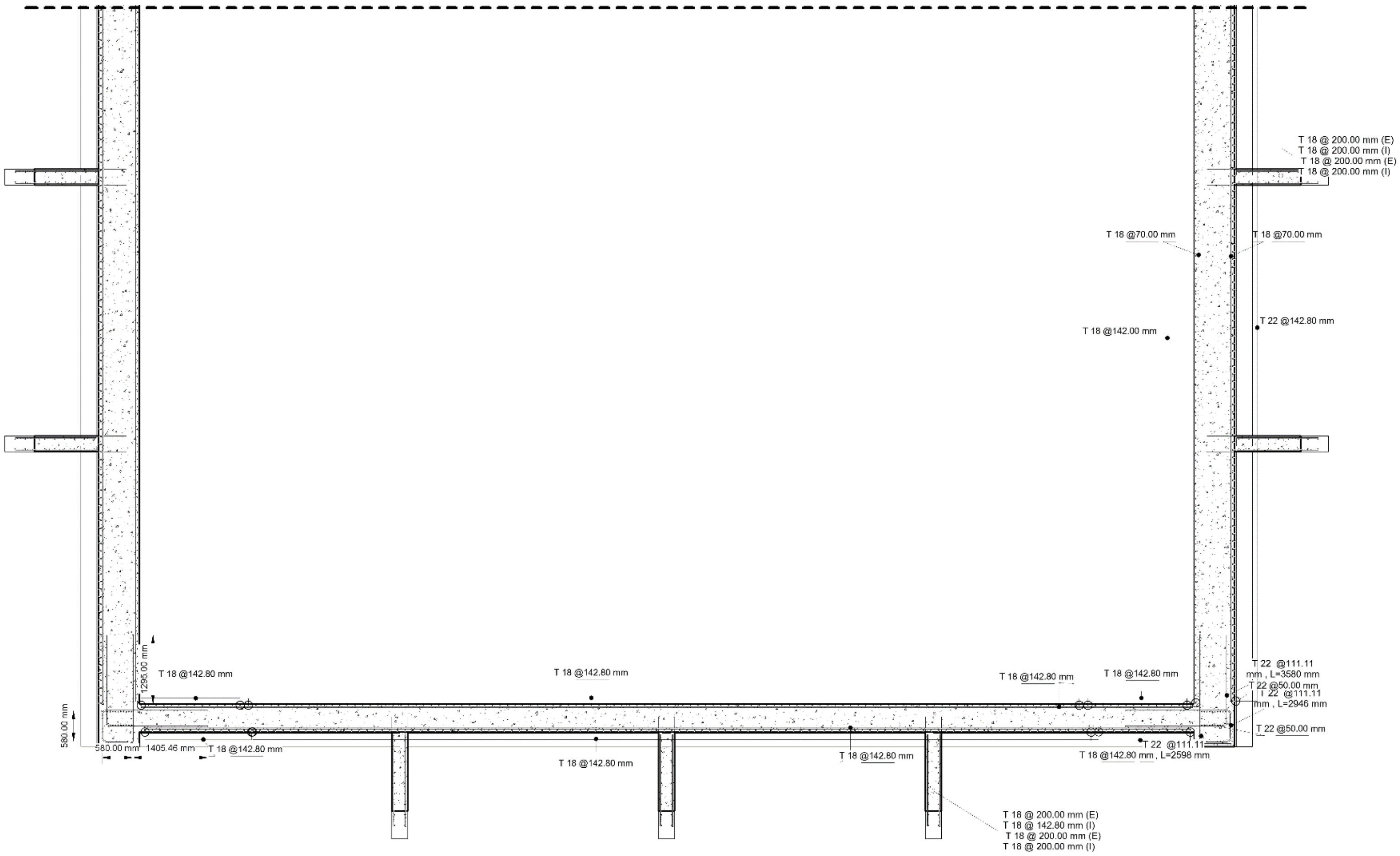


Figure 2-2 HZ Section

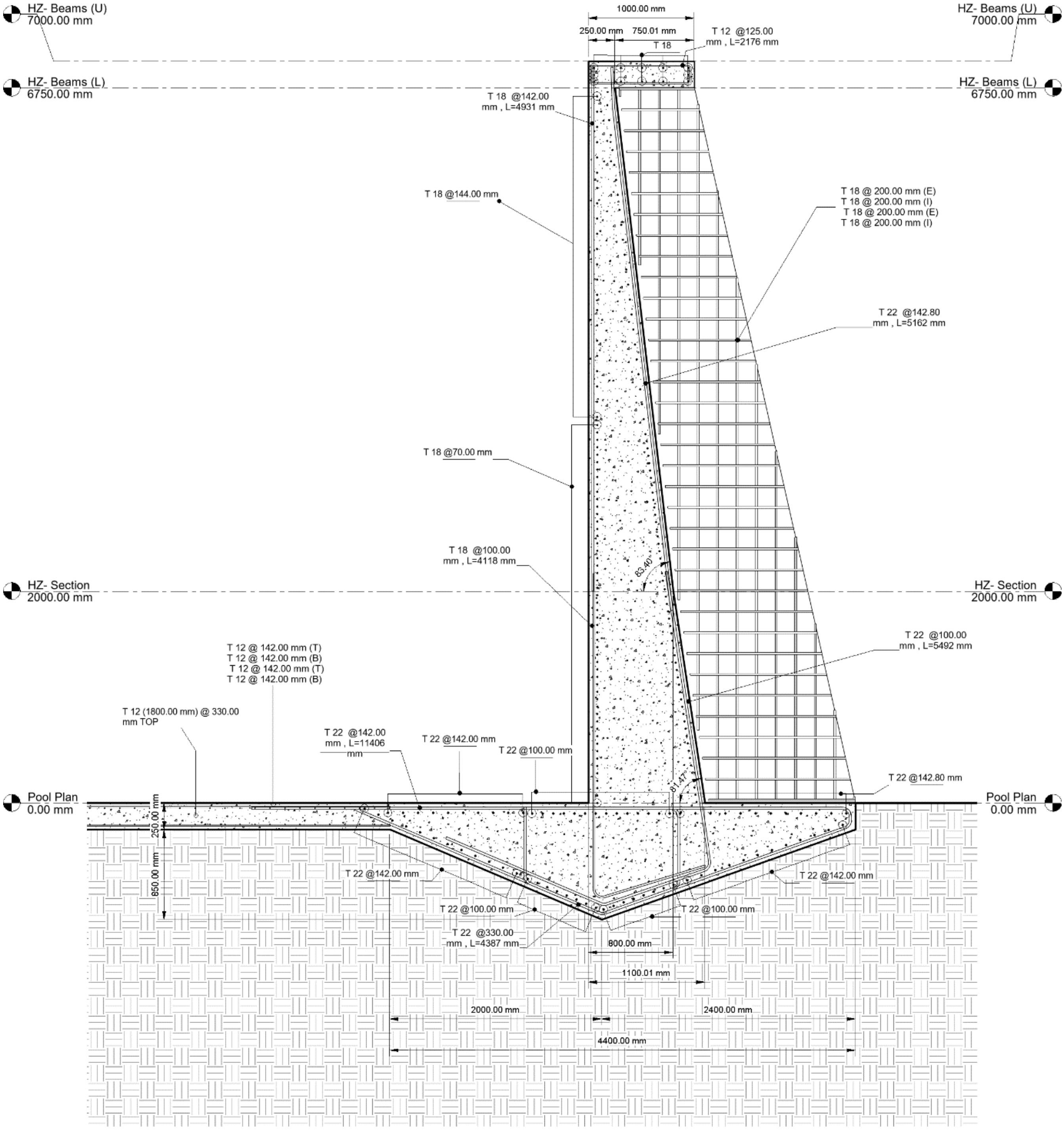


Figure 2-3 VL Section 1

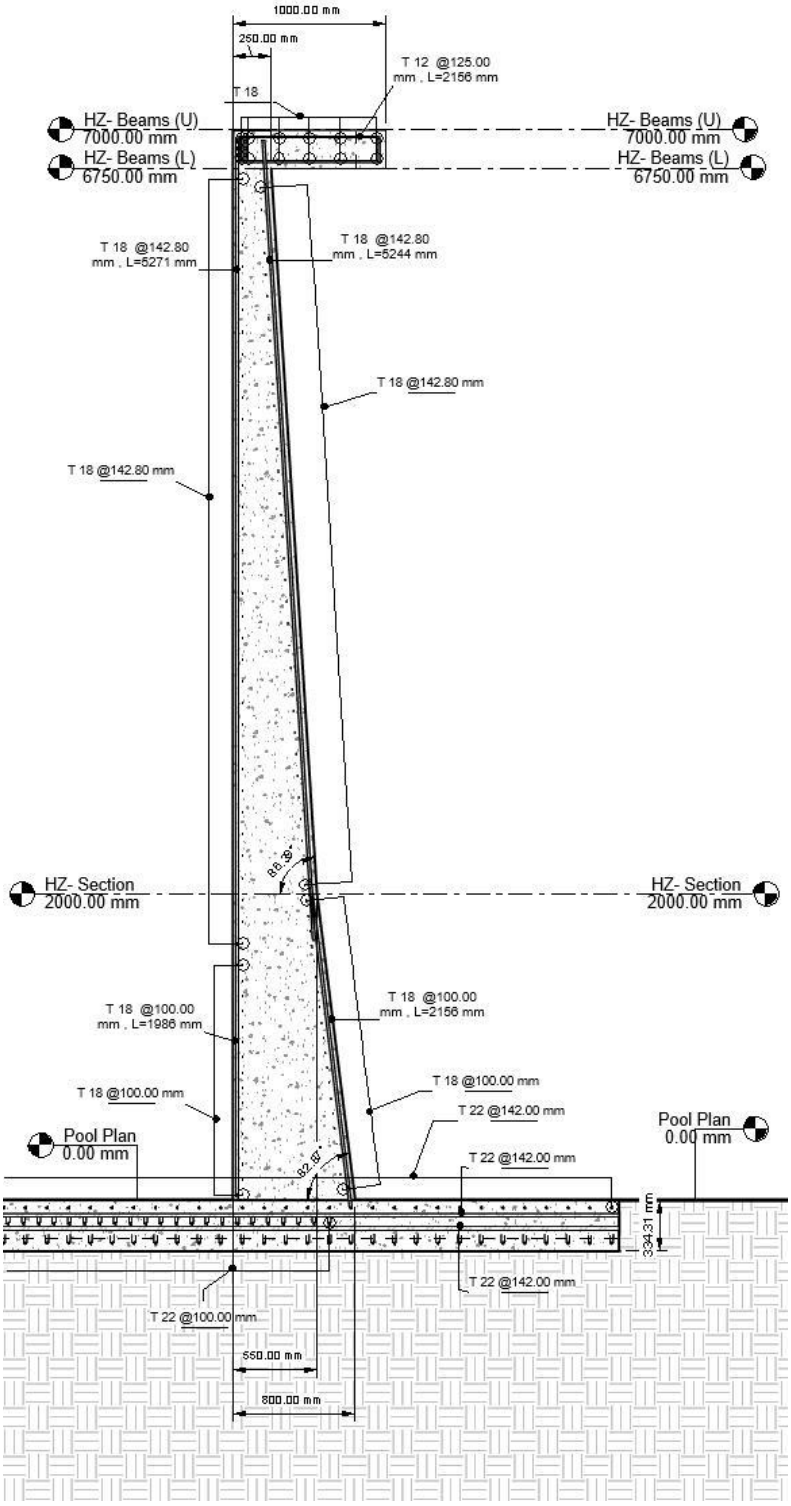


Figure 2- 4 VL Section 2

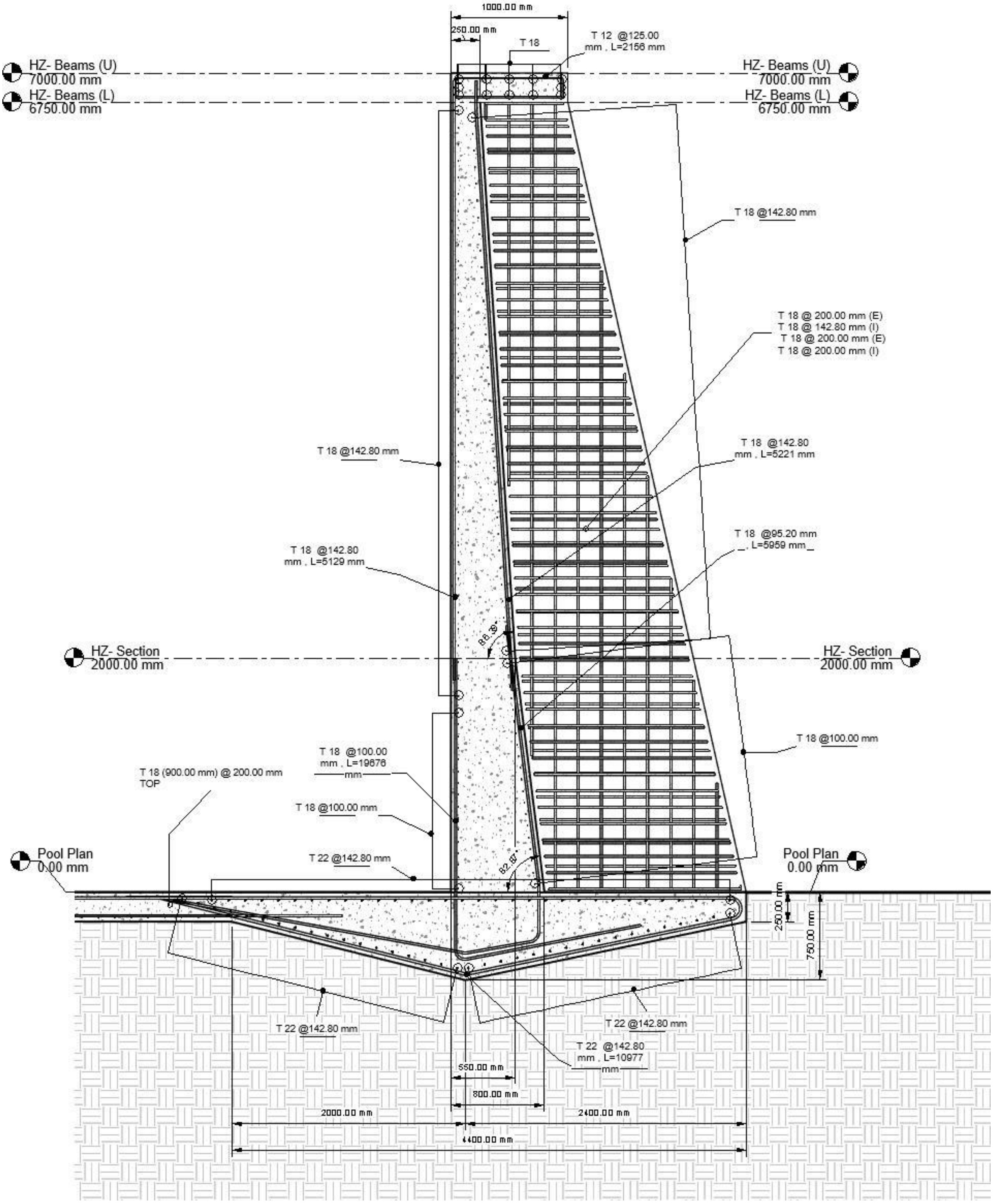


Figure 2- 5 VL Section 3

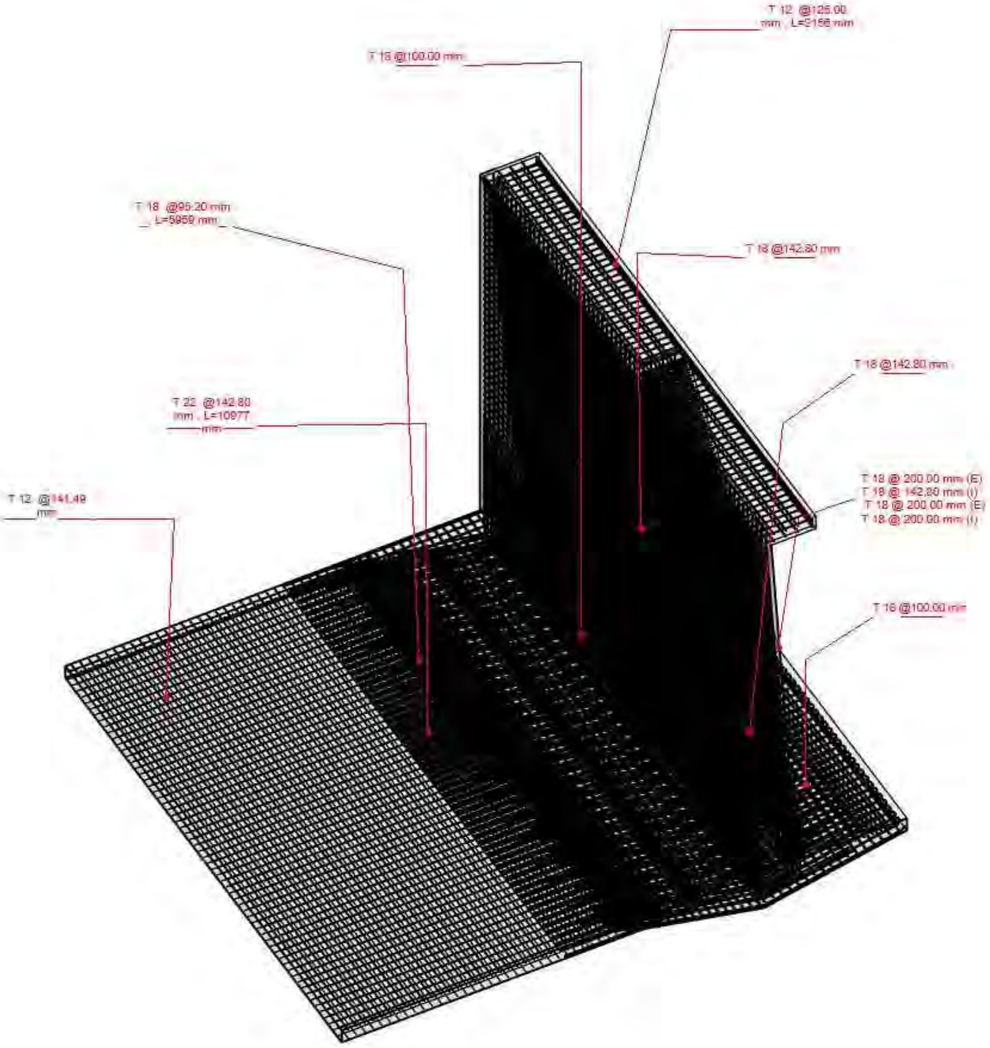


Figure 2- 6 3d wall to slab joint RFT

3. Calculation Software Used

Calculation software features

The software used is *RFEM*, developed by DLUBAL COMPANY (Germany).

Technical specifications

Name:	RFEM
Version:	5.22.03
Producer:	DLUBAL
	www.dlubal.com

License registered is a student license

4. OUTLINE SPECIFICATION AND MATERIAL PROPERITIES

REINFORCED CONCRETE

The grade of concrete will be according to the Egyptian Code of Practice (ECP). The grade of concrete is indicated in two numbers, the first one indicate the characteristic cube strength in (N/mm²) while the second one indicates the maximum nominal size of the aggregate in (mm) to be used;

Grade (20/20)	for plain concrete of foundations of thickness < 12 cm.
Grade (20/40)	for plain concrete of foundations of thickness >12 cm
Grade (30/20)	for all pool reinforced concrete elements.

Minimum thickness of blinding concrete is 100 mm.

Concrete cover is the concrete thickness to all steel reinforcement including links:

For all concrete (with protection) in contact with soil, cover shall be 70mm (or as will be recommended in the geotechnical report)

For all concrete elements above grade where concrete is protected from weathering, cover shall be 50mm for beams and 25 mm for slabs and walls.

- **SLUMP VALUES**

The following values are according to the Egyptian code of practice ECP 203-2018 section (2-3-1-2), Table (2-5).

Type of Structural Element	Type of Compaction	Slump-in mm (max.)
Massive concrete	Mechanical	25 - 50
- Concrete foundation. - Concrete sections with low reinforcement ratio (< 80 kg/m³)	Mechanical	50 - 75
Concrete sections with medium and high reinforcement ratio (80-150 kg/m³)	Mechanical/ Manual	75 – 125
Concrete sections with very high reinforcement ratio (> 150 kg/m³)	Light compaction	125 – 150**
Deep foundation	Light compaction	125 – 200**

** By using chemical additives.

REINFORCING STEEL

All reinforcing steel shall be complying with the Egyptian code of practice ECP203-2018, section 2-2-5-3, Table 2-4.

Reinforcing steel bars shall be uncoated high yield deformed bars of characteristic strength 360 N/mm².

Uncoated mild steel plain bars with characteristic strength 280 N/mm² may be used for links and binders.

Type	Grade	Yield Strength, f_y (N/mm²)
Normal mild steel	280/450	280
High grade steel	360/520	360
Cold formed welded mesh	450/520	450

Note: Bar size increment = 6, 8, 10, 12, 16, 18, 20, 22, 25, 28 and 32

5. Calculation method and numerical model

5.1 Model Description

5.1.1 Hypothesis adopted for the elements

- Based on the pool dimension to it's height ratio, the pool considered as rested on rigid foundation; thus we can assume uniform stress distribution under it.

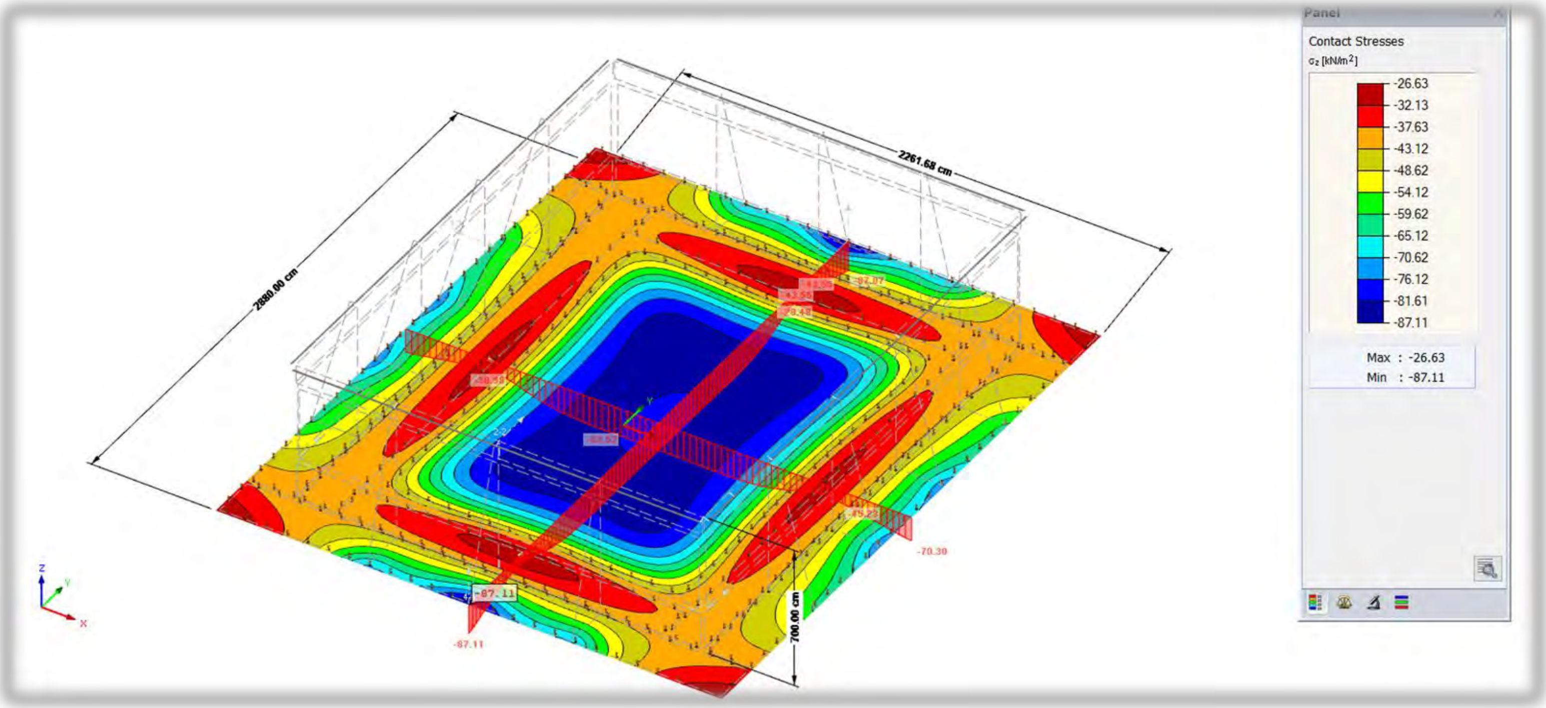


Figure 5.1 stress distribution under pool

- To achieve economy in design and logical concrete dimensions, the counterforts are modeled every 5 meter and a horizontal beam at the top of the pool. The horizontal beam converts the cantilever action of the wall into simply supported action, That can reduce the straining actions values in vertical direction of the wall. The counterforts lead a percentage of the wall loads to transfer in the horizontal direction depend on elastic analysis of plates (Grashoff's values).

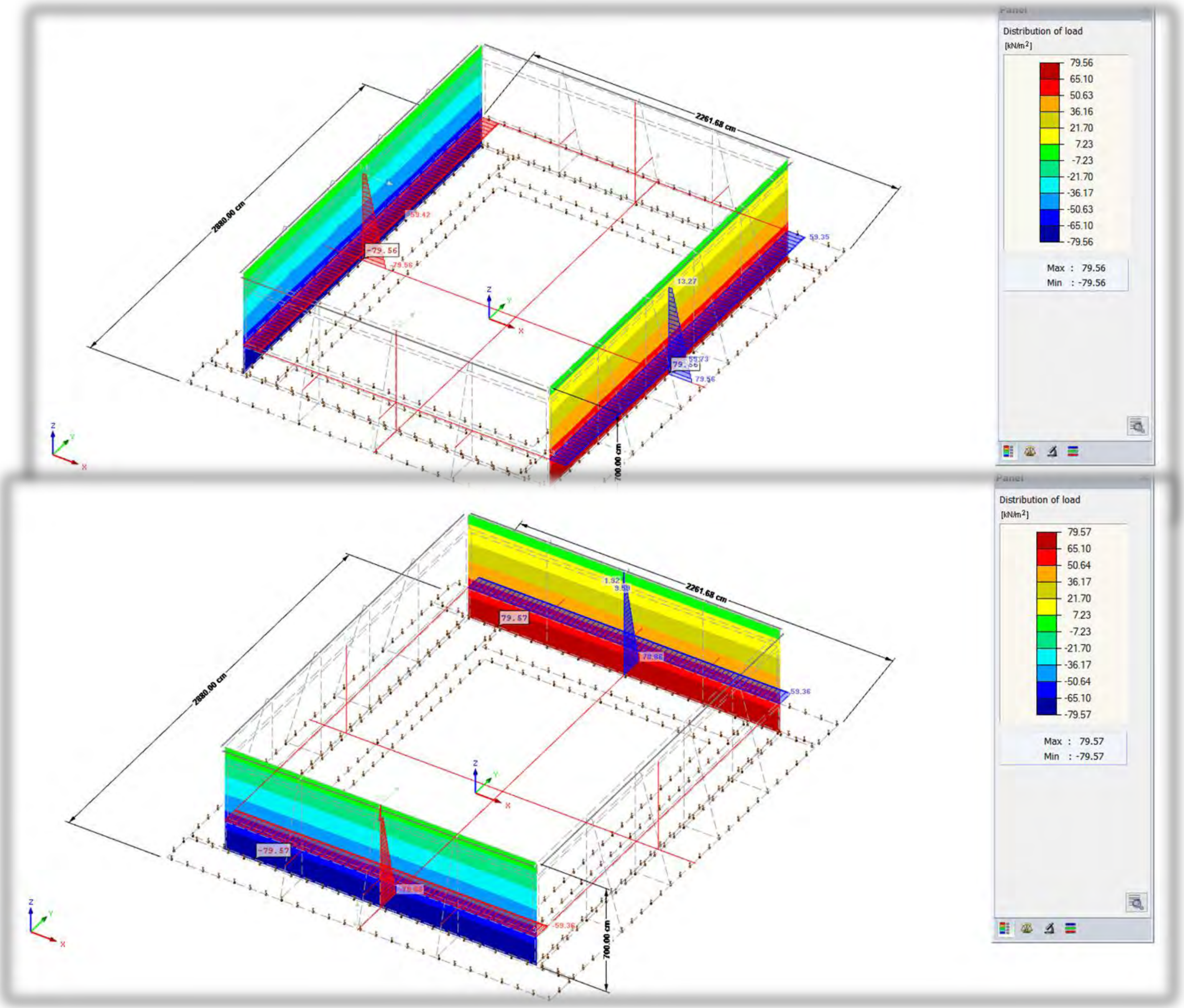


Figure 5.2 Loads distribution on the wall in x and y directions

- From economy point of view, the bilinear variable thickness in walls and raft are modeled as shown in fig. 5.3.

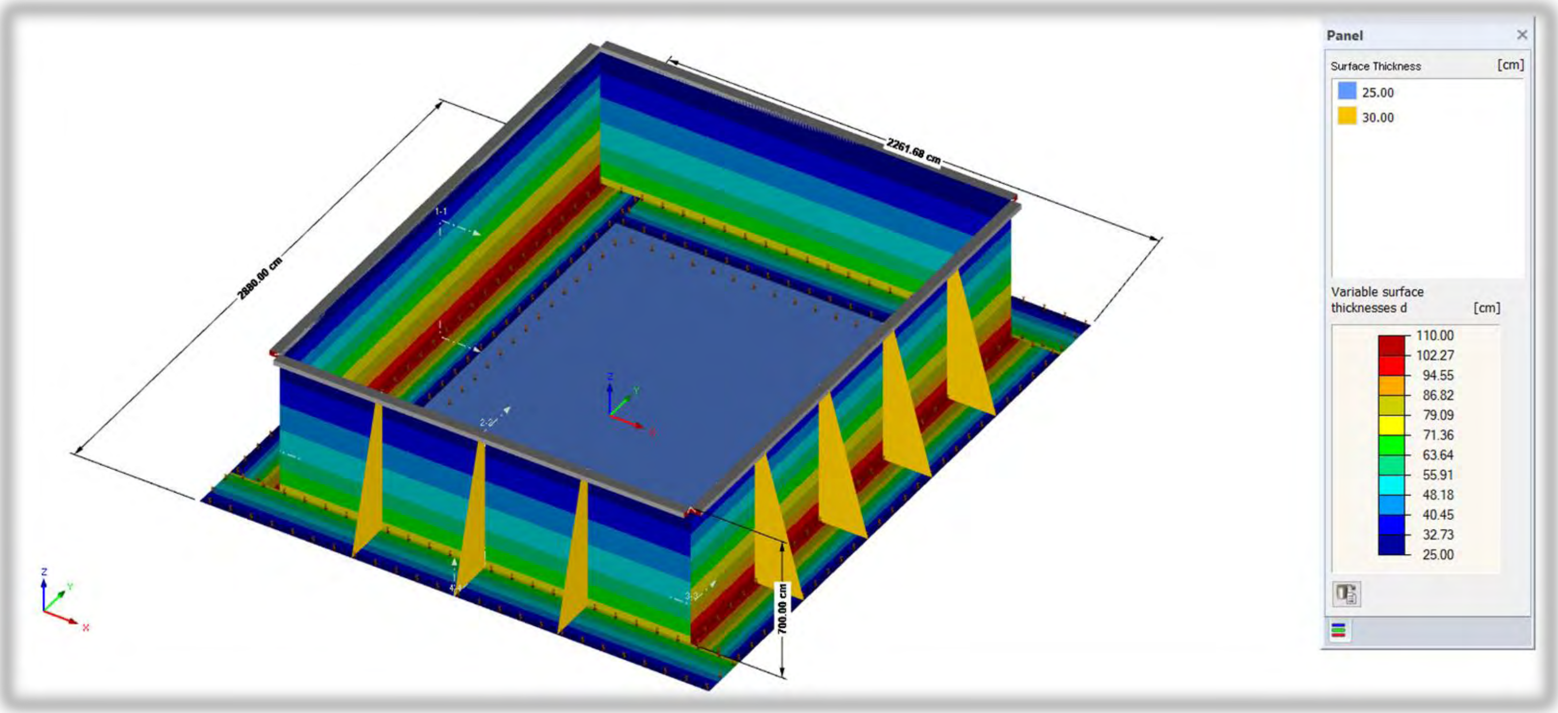
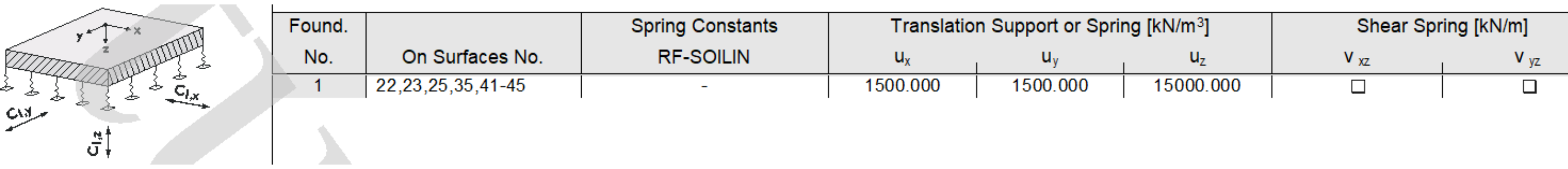


Figure 5.3 color scale of thicknesses in panels

- For simplicity, the soil is modeled as springs following this equation:
  Soil bearing capacity= 1cm * K_{spring}



Found. No.	On Surfaces No.	Spring Constants RF-SOILIN	Translation Support or Spring [kN/m³] u_x	u_y	u_z	Shear Spring [kN/m] v_{xz}	v_{yz}
1	22,23,25,35,41-45	-	1500.000	1500.000	15000.000	☐	☐

6. Actions and design loads

6.1 STRUCTURAL LOADS

The following loads are considered in the design:

Structural Dead Loads which include:

The own weight of the structural elements, beams, raft and walls.

Superimposed dead load from water and soil weights.

Live loads which cover the weight and movement of equipment and people on the sides of the pool (surcharge).

The basis for the considered design loads are summarized in the followings sections.

Dead Loads

Unit weight of concrete elements 25.0 kN/m³

Live Loads

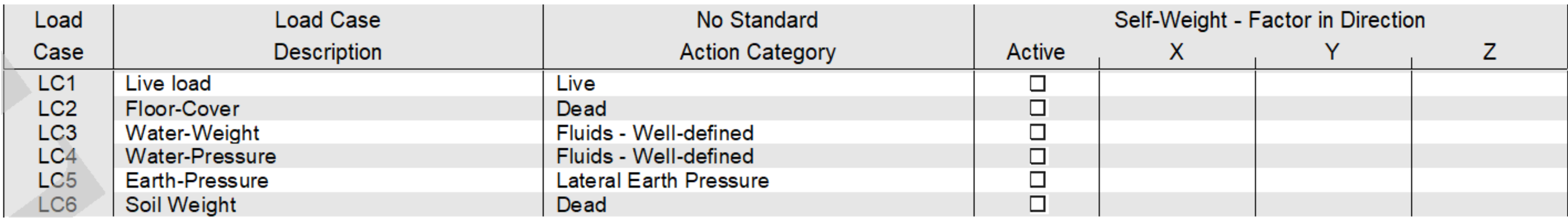
Live loads are considered equal to 30 kN/m^2 effect on the sides of the pool as a surcharge.

Earthquakes

from experience of prof. Eehab Khalil, the earthquake effect on pool is taken as increase by 15% of load combinations factors.

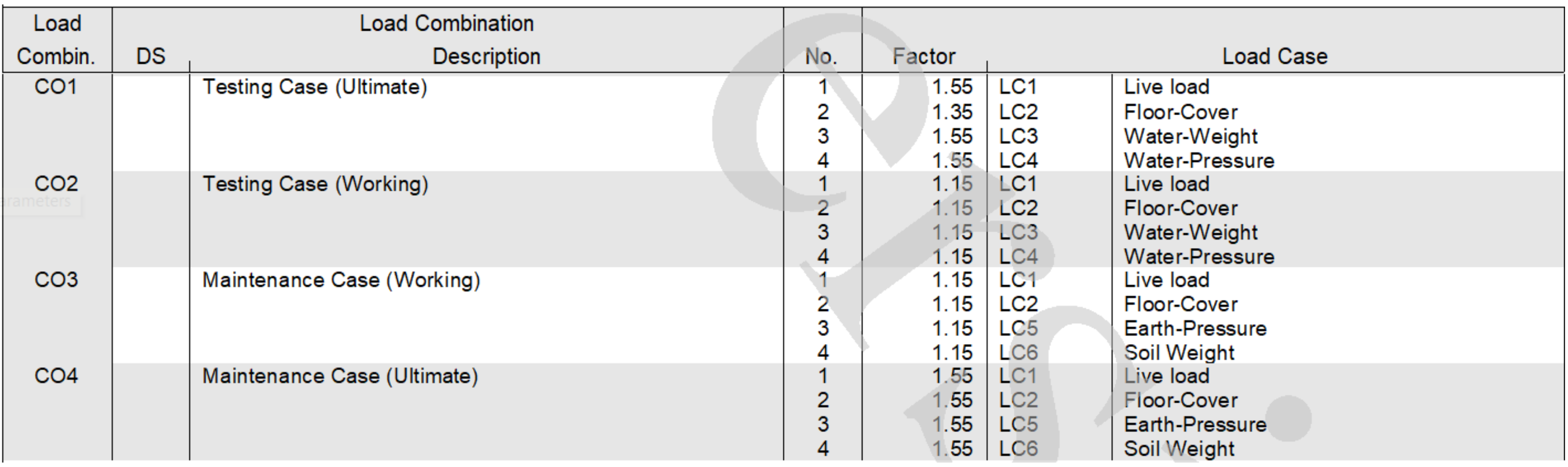
The following tables describe the load cases and load combinations on the pool:

Table 1, Load cases



Load Case	Load Case Description	No Standard Action Category	Active	Self-Weight - Factor in Direction X	Y	Z
LC1	Live load	Live	☐			
LC2	Floor-Cover	Dead	☐			
LC3	Water-Weight	Fluids - Well-defined	☐			
LC4	Water-Pressure	Fluids - Well-defined	☐			
LC5	Earth-Pressure	Lateral Earth Pressure	☐			
LC6	Soil Weight	Dead	☐			

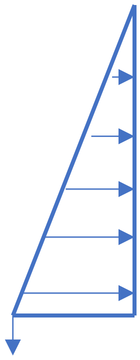
Table 2, Load combinations



Load Combin.	DS	Load Combination Description	No.	Factor	Load Case	
CO1		Testing Case (Ultimate)	1	1.55	LC1	Live load
			2	1.35	LC2	Floor-Cover
			3	1.55	LC3	Water-Weight
			4	1.55	LC4	Water-Pressure
CO2		Testing Case (Working)	1	1.15	LC1	Live load
			2	1.15	LC2	Floor-Cover
			3	1.15	LC3	Water-Weight
			4	1.15	LC4	Water-Pressure
CO3		Maintenance Case (Working)	1	1.15	LC1	Live load
			2	1.15	LC2	Floor-Cover
			3	1.15	LC5	Earth-Pressure
			4	1.15	LC6	Soil Weight
CO4		Maintenance Case (Ultimate)	1	1.55	LC1	Live load
			2	1.55	LC2	Floor-Cover
			3	1.55	LC5	Earth-Pressure
			4	1.55	LC6	Soil Weight

6.2 Calculations of water and earth pressure.

6.2.1 water pressure.

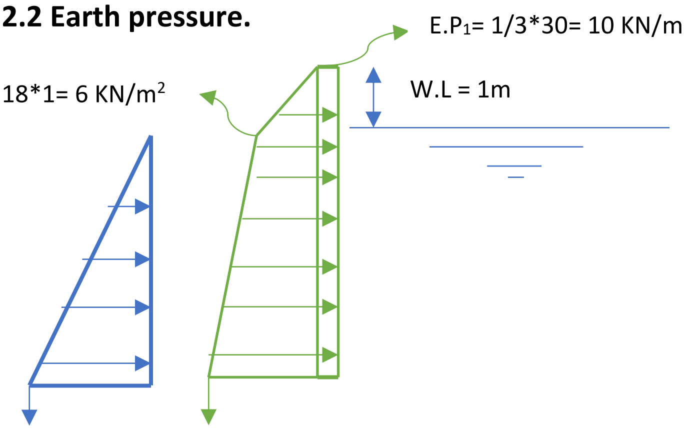


γ_{water}= 10 KN/m^3

Height= 7m

W.P= 10*7= 70 KN/m^2

6.2.2 Earth pressure.



E.P_1= 1/3*30= 10 KN/m^2

E.P_2= 1/3*18*1= 6 KN/m^2

W.L = 1m

γ_{water}= 10 KN/m^3

γ_{soil}= 18 KN/m^3

Ka= 1/3

Height= 7m

W.P= 10*6= 60 KN/m^2

E.P_3= 1/3*8*6+6= 22 KN/m^2

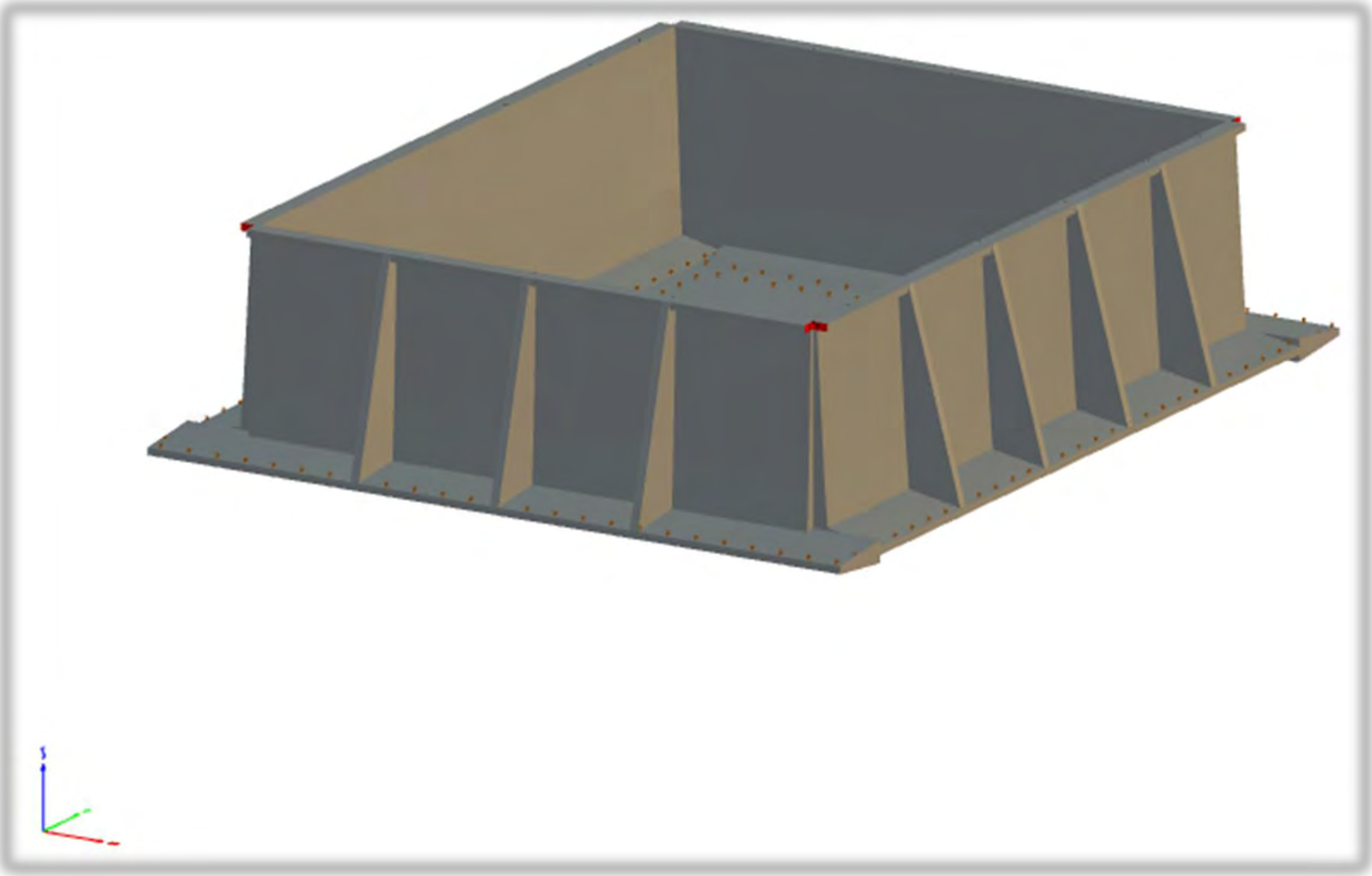
6.3 CRACKING

It will be calculated as stated in the “ECP 203-2018 - section 4-3-2” for the following maximum design crack width:

- 0.15 mm for water-side exposure.

7. STRUCTURAL ANALYSIS

7.1 3d-model



7.2 ASSIGN OF LOADS

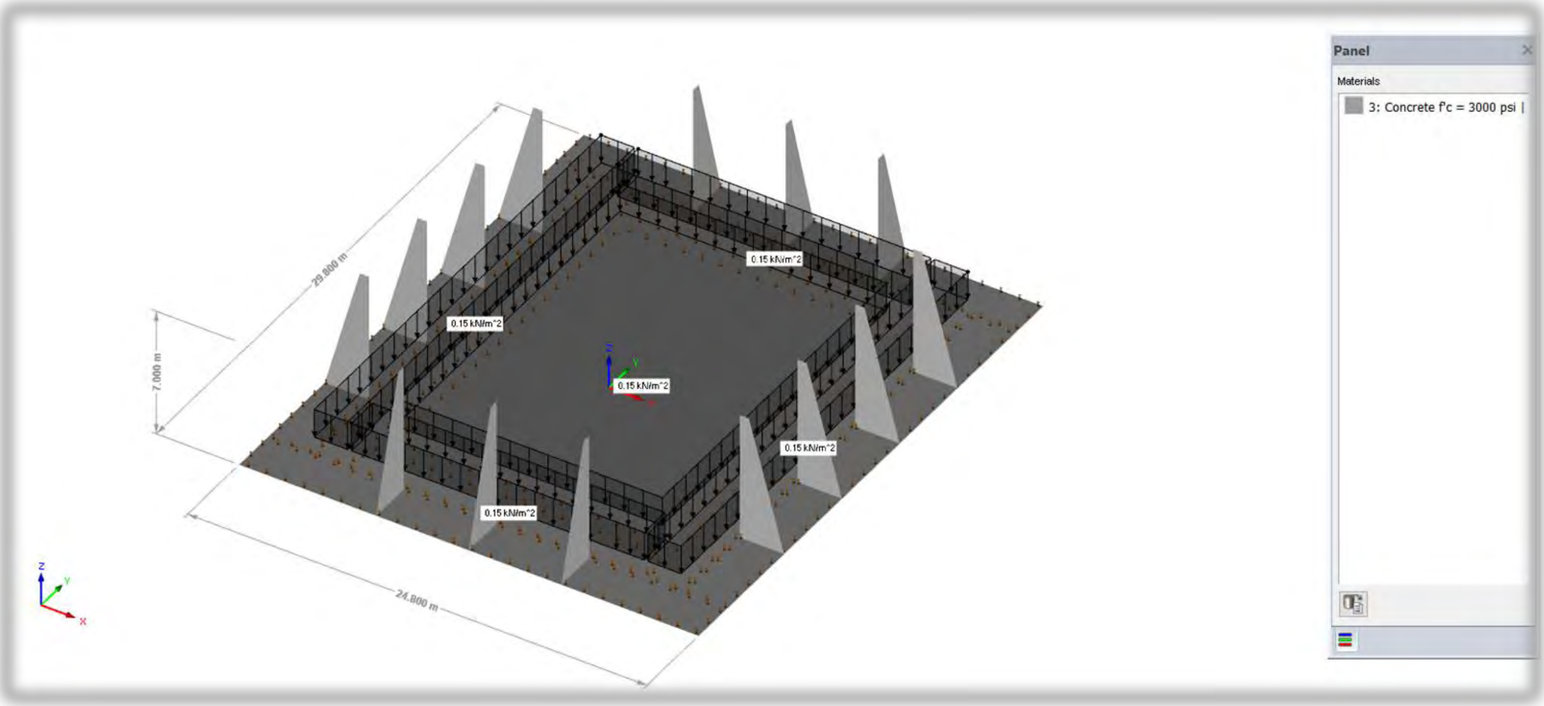


Figure 8.1.: Assign of Finishing Loads on the pool’s raft (KN-M^2 Units)

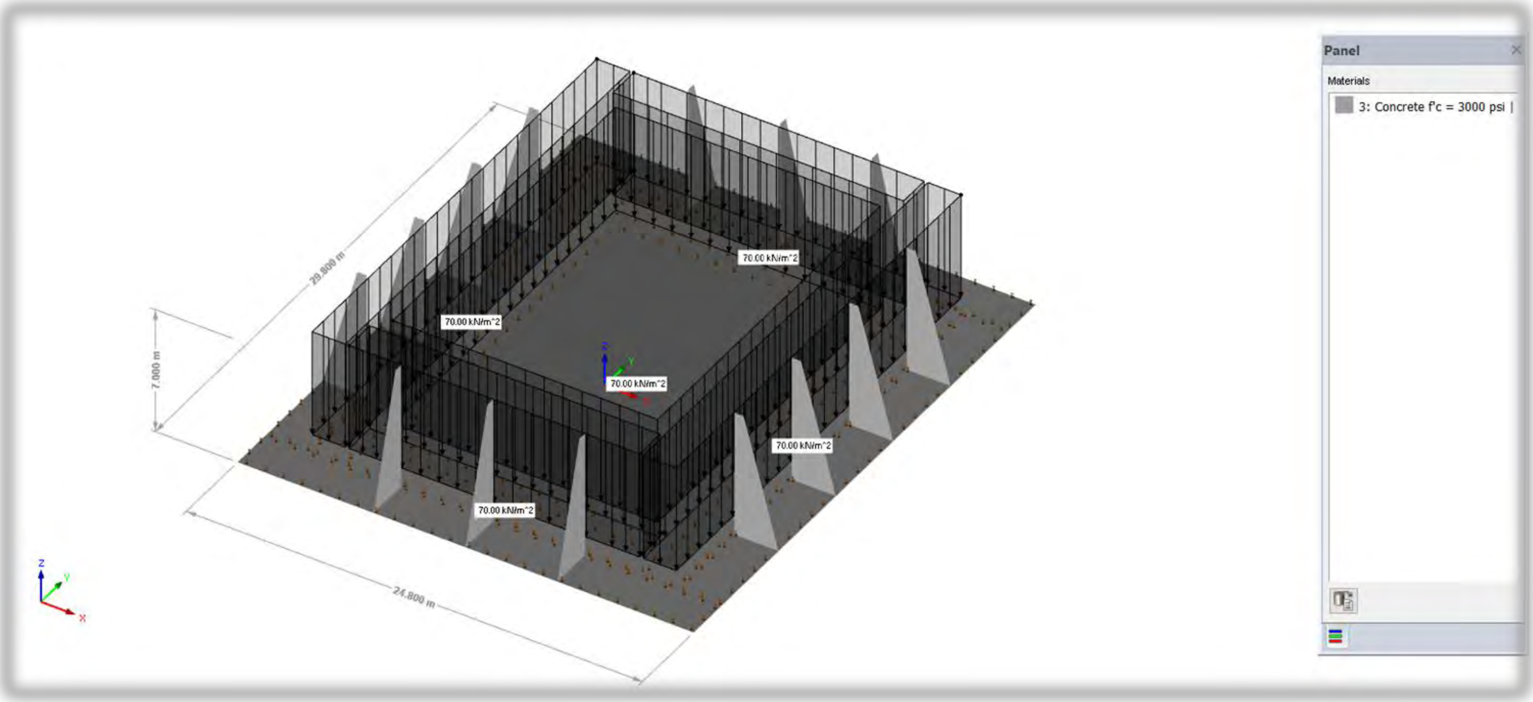


Figure 8.2.: Assign of Water weight on the pool’s raft (KN-M^2 Units)

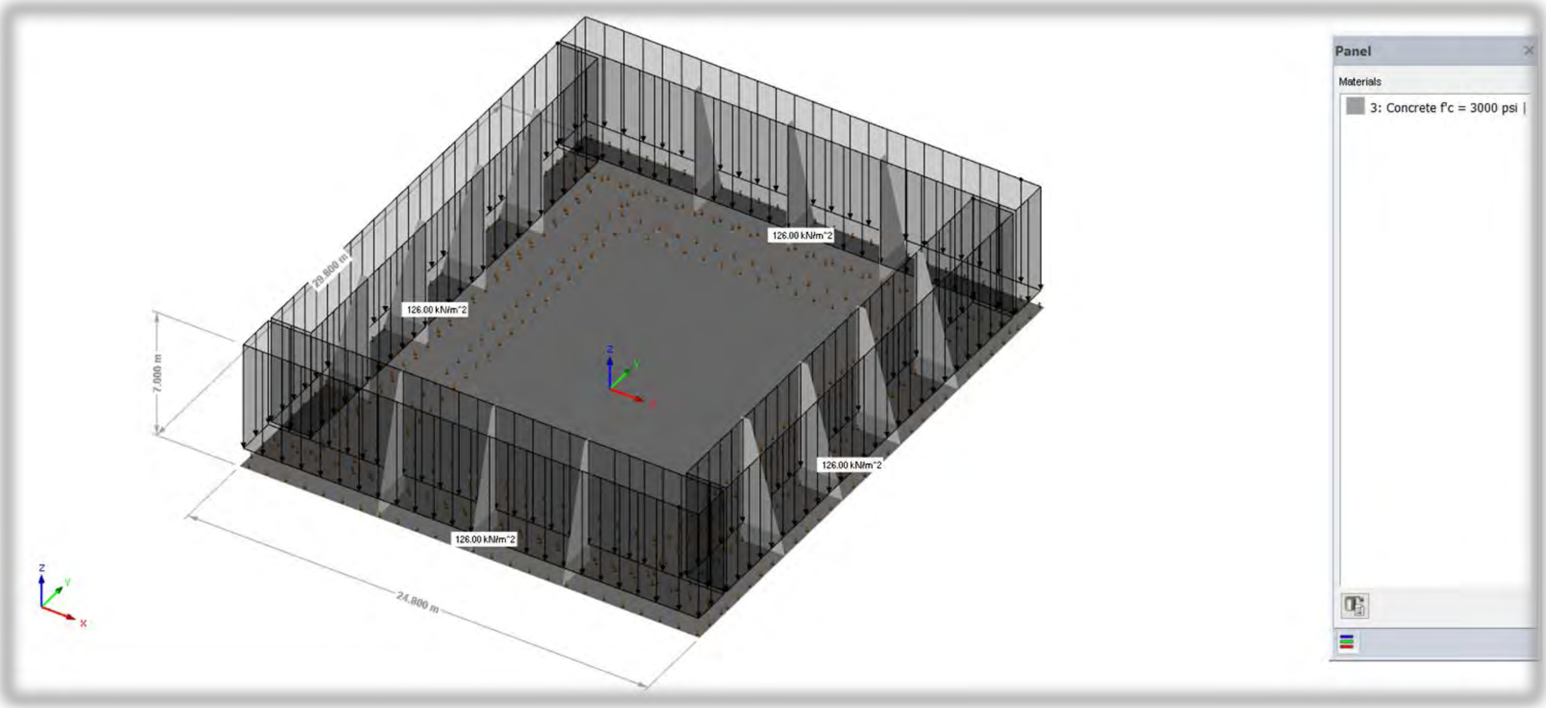


Figure 8.3.: Assign of Soil weight on the pool’s raft (KN-M^2 Units)

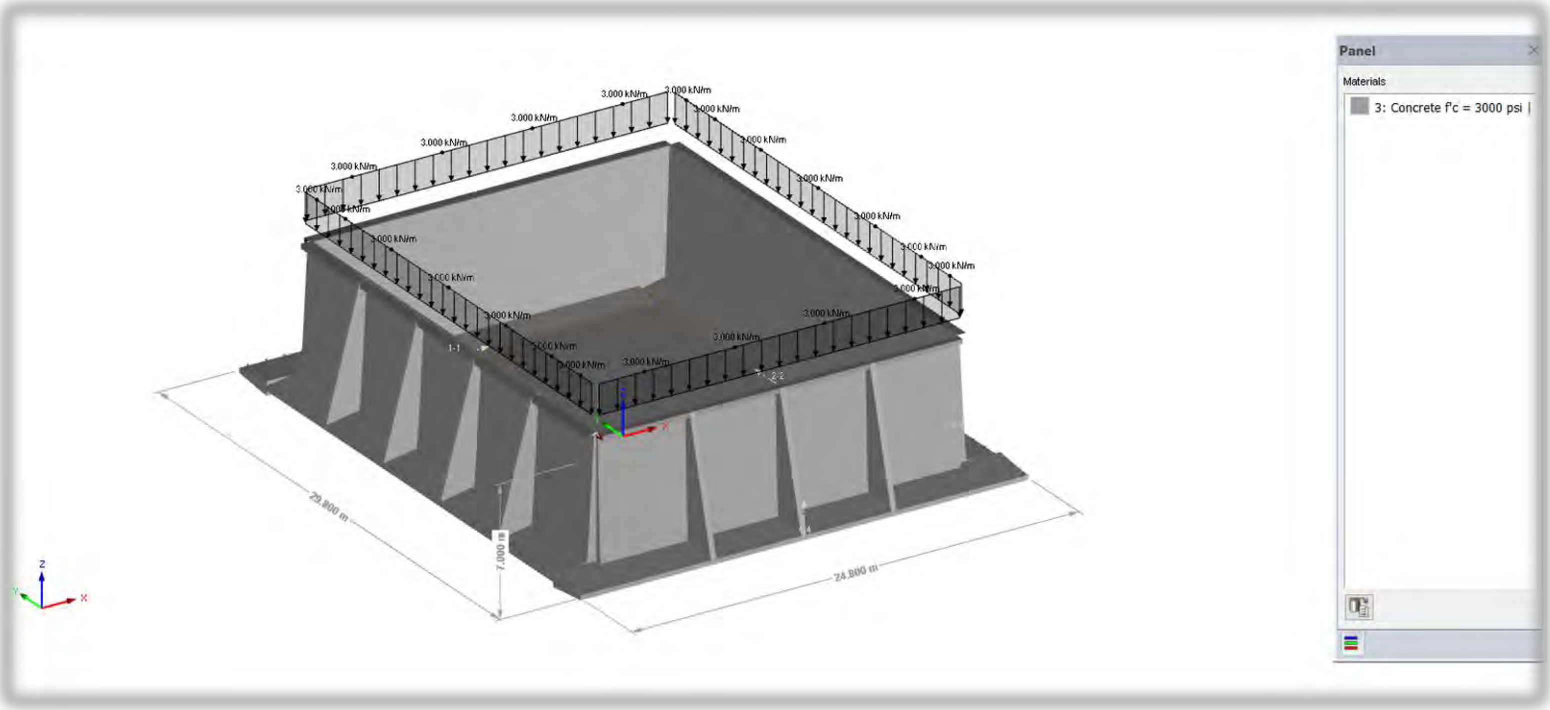


Figure 8.4.: Assign of Surcharge Load on the pool’s HZ-Beams (KN-M^2 Units)

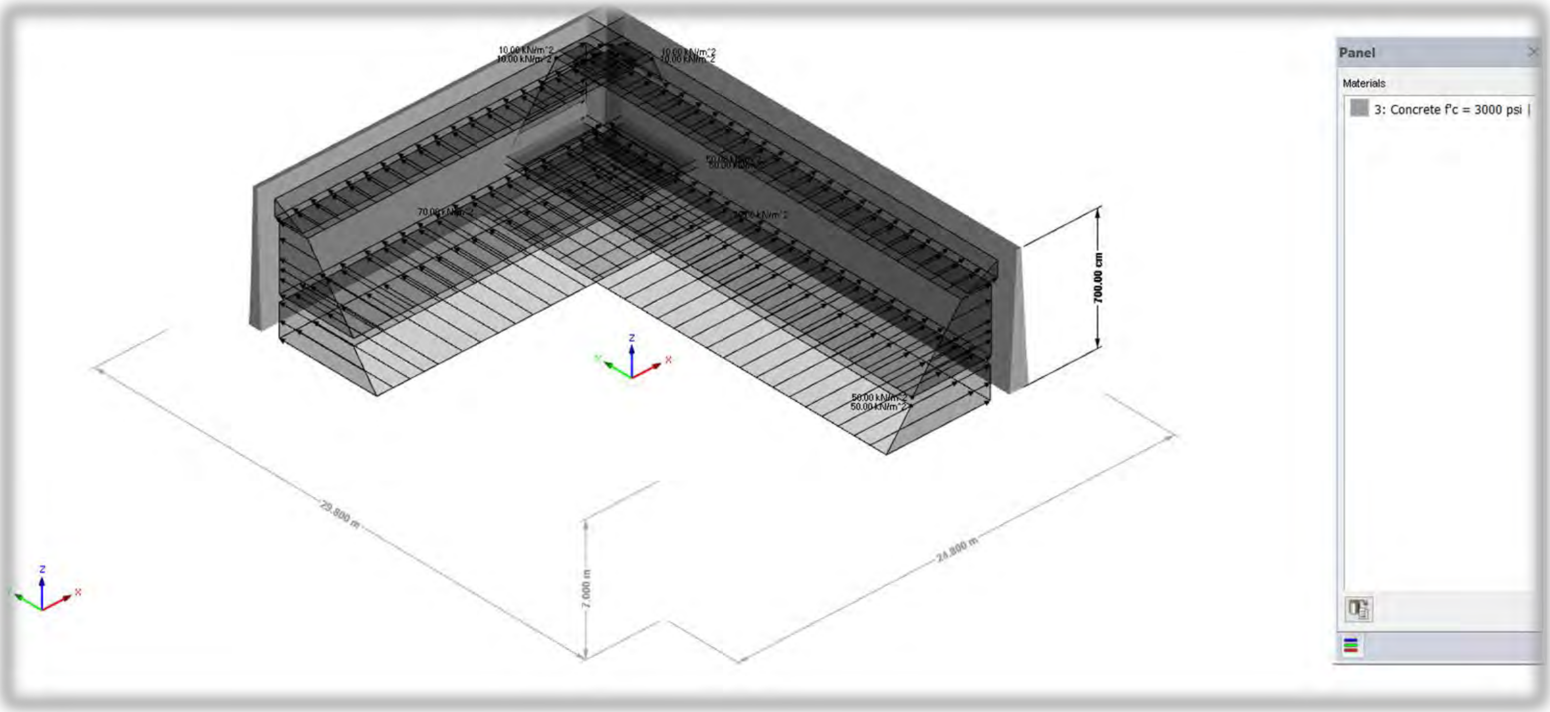


Figure 8.5.: Assign of Water on the pool's walls (KN-M^2 Units)

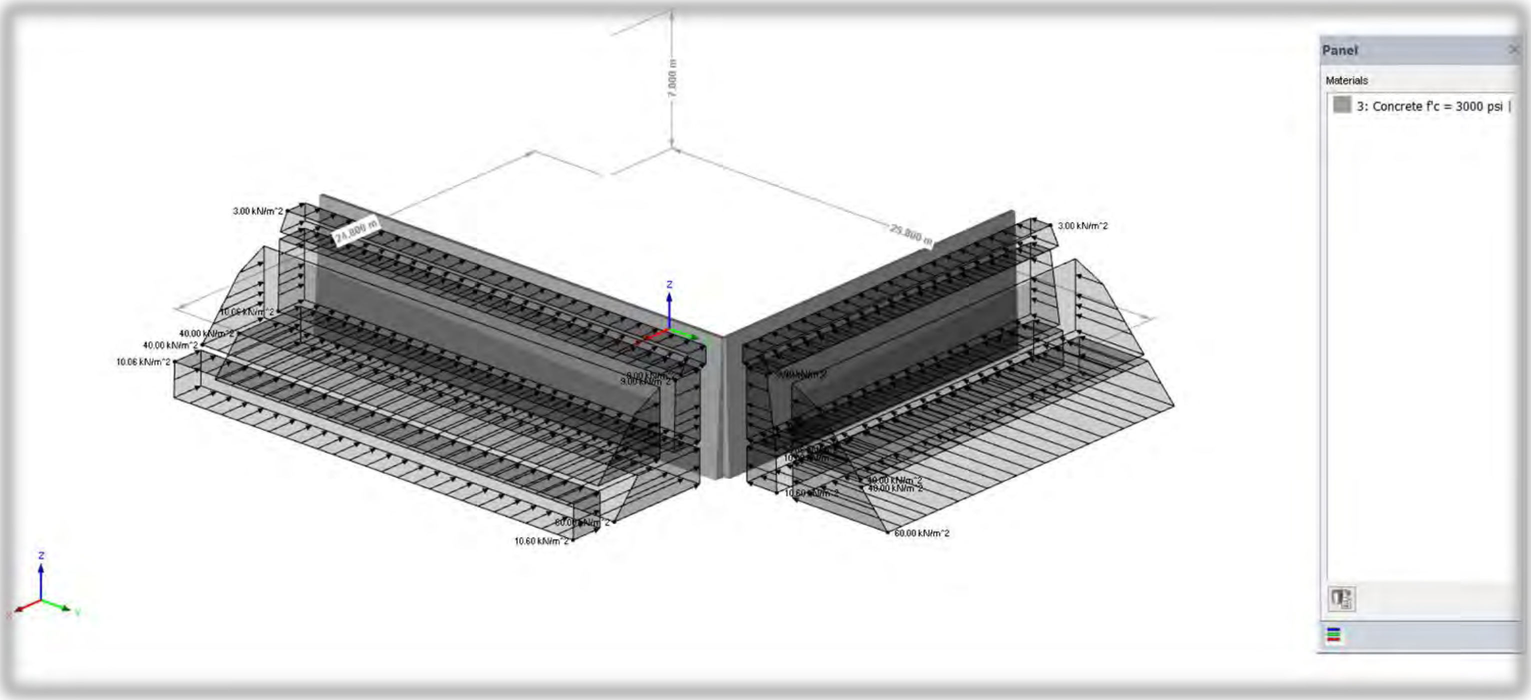


Figure 8.6.: Assign of Soil Pressure on the pool's walls (KN-M^2 Units)

8. STRUCTURAL DESIGN

8.1 Checks.

8.1.1 Bearing Capacity.

- From RFEM model, the maximum soil reaction equal 40664 KN when the tank is full (Testing case), as shown in Fig. 8.1.
- The maximum stress distributed under soil = $\frac{40664/10}{29.8*24.8}$ = 5.5 t/m2 < 15 t/m^2 (Safe)

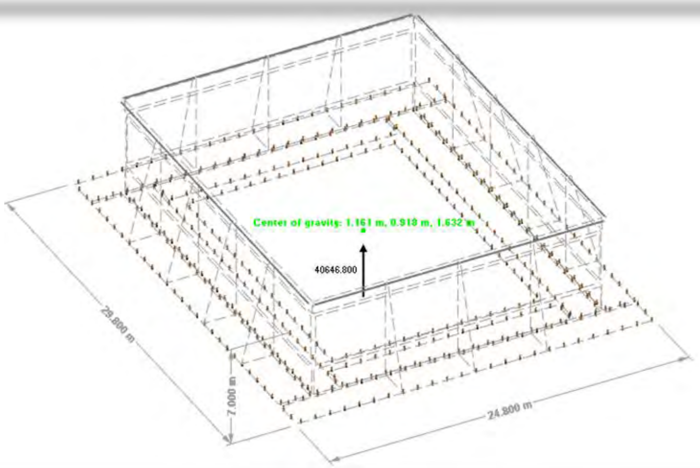


Figure 8.1: Soil Reaction under the pool, Testing case (KN)

8.1.2 Uplift.

- From RFEM model, the maximum soil reaction equal 35034 KN when the tank is empty (Maintenance case), as shown in Fig. 7.2.
- The maximum stress distributed under soil = $\frac{35034/10}{24.8*29.8}$ = 4.74 t/m^2 > 1.5 (Safe)

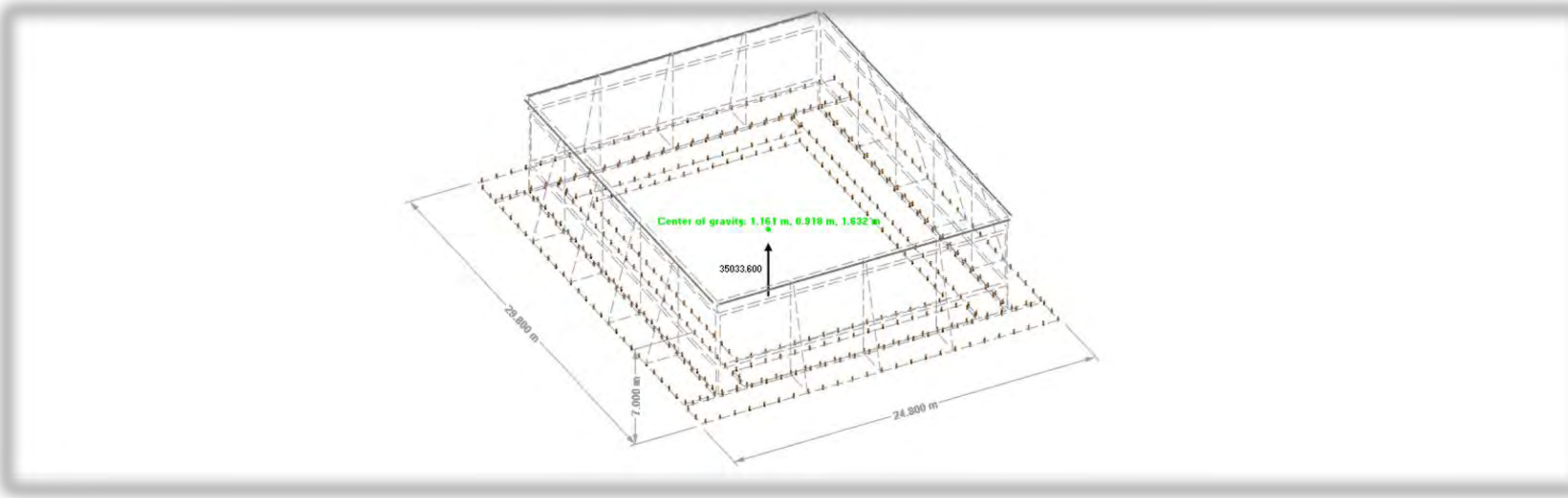


Figure 8.2: Soil Reaction under the pool, Maintenance case (KN)

8.1 Analysis.

8.1.1 Analysis results as contour range

- **Maintenance Case.**

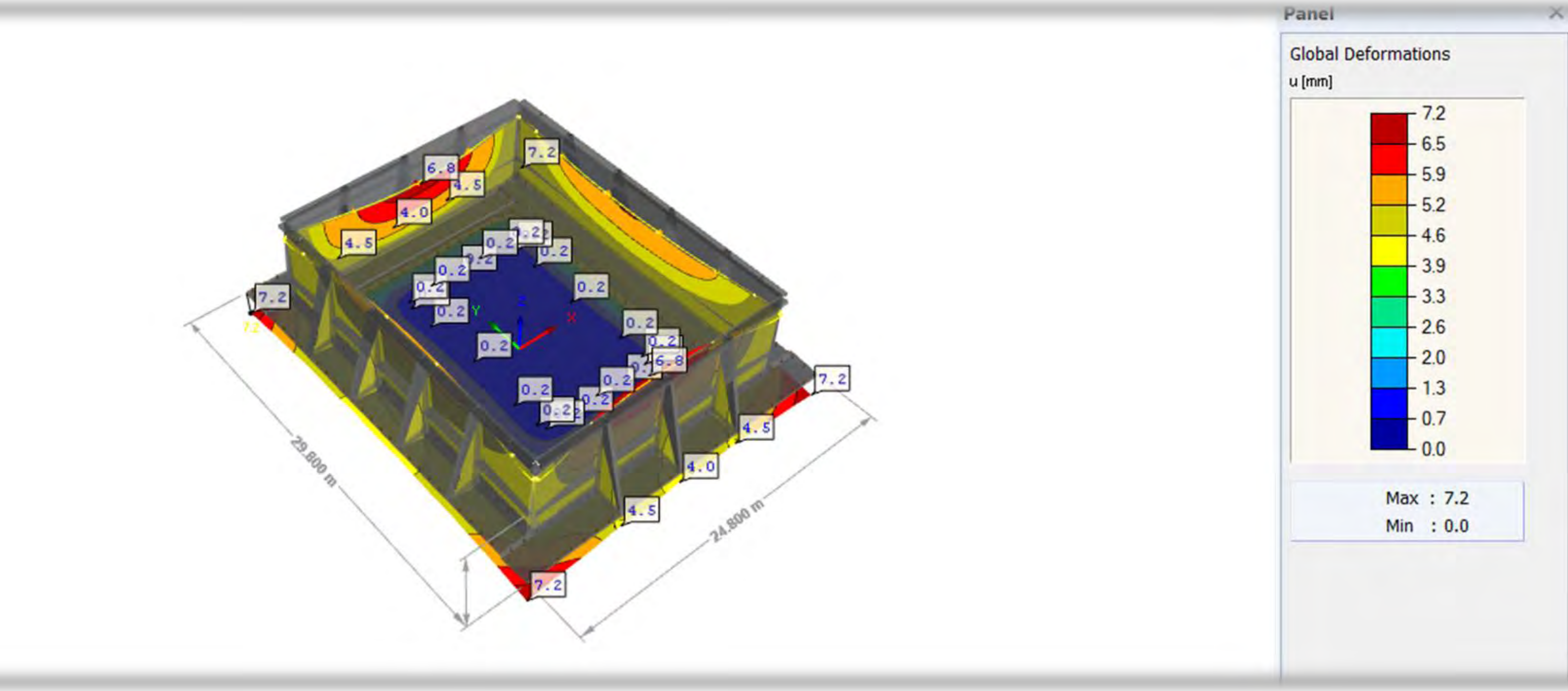


Figure 8.3: Deformations of pool’s panels (mm).

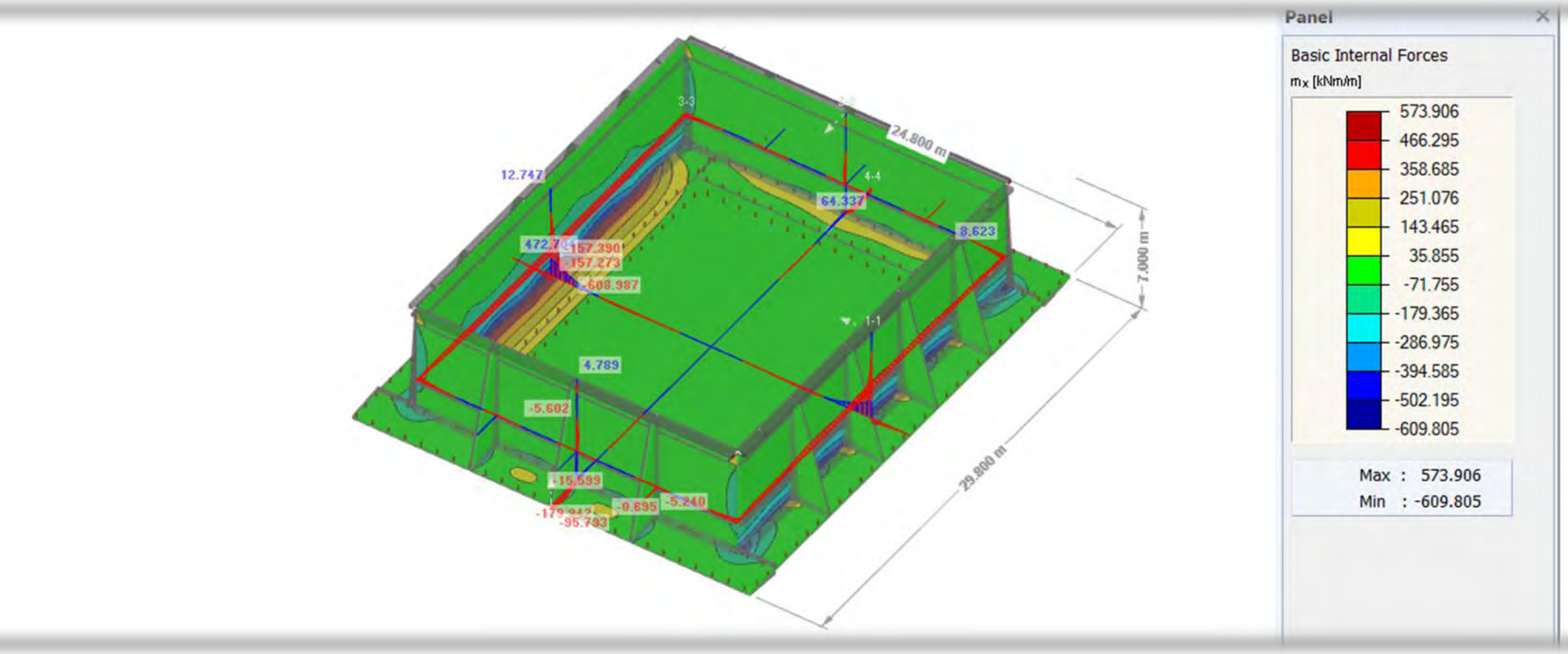


Figure 8.4: Bending Moment in x-direction (short direction) (KN-M).

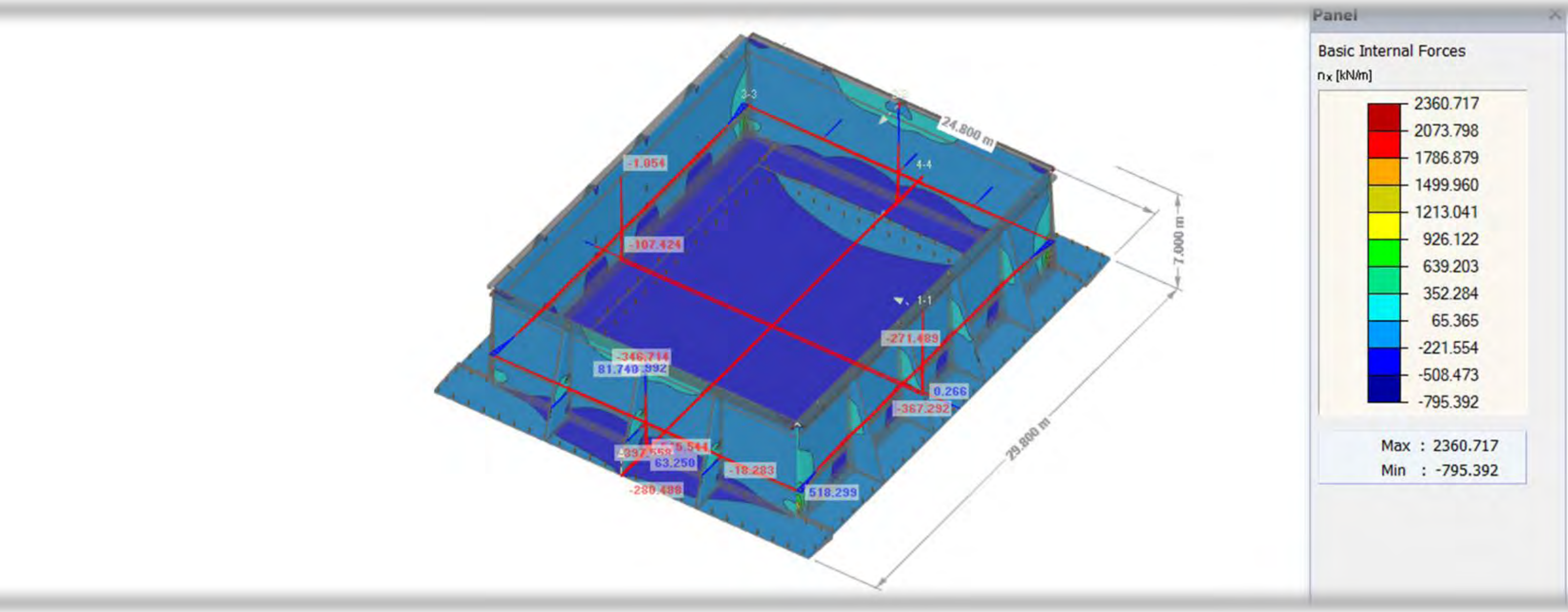


Figure 8.5: Normal Force in x-direction (short direction) (KN).

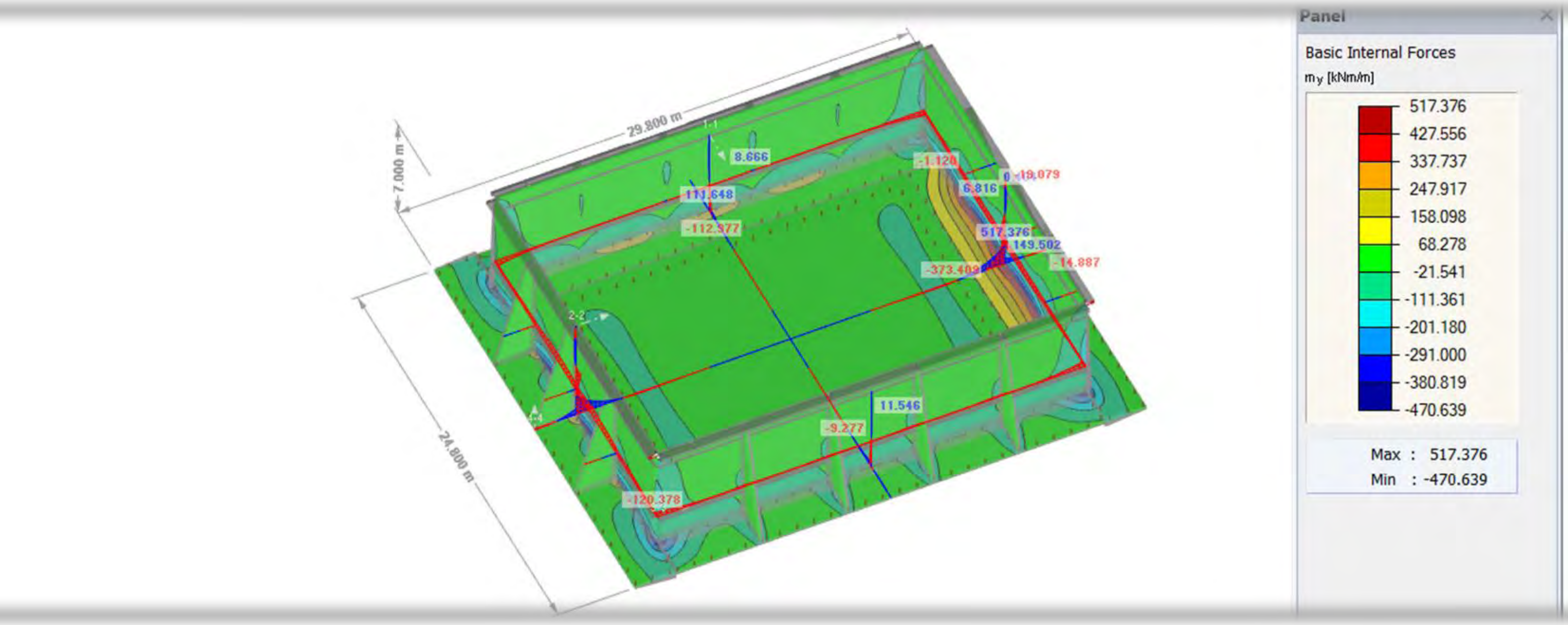


Figure 8.6: Bending moment in y-direction (long direction) (KN-M)

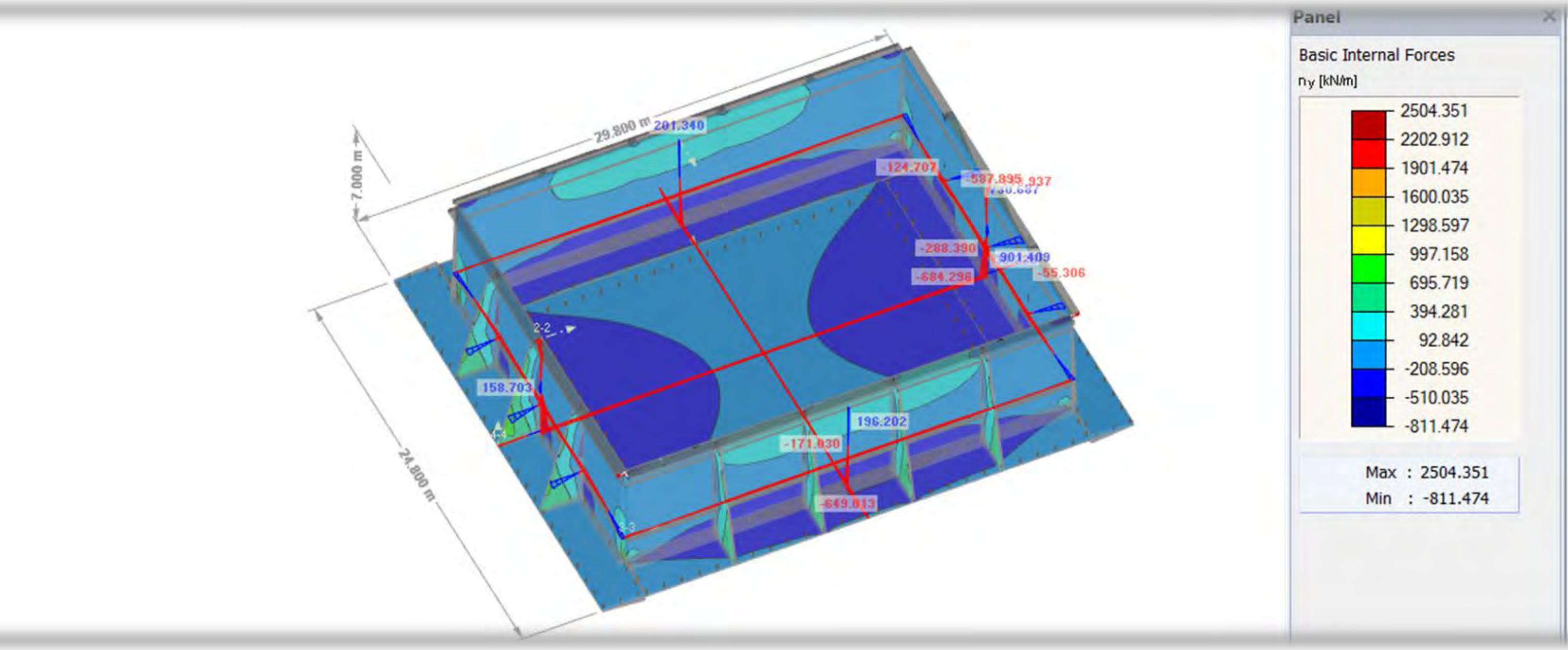


Figure 8.7: Normal Force in y-direction (long direction) (KN).

- **Working Case.**

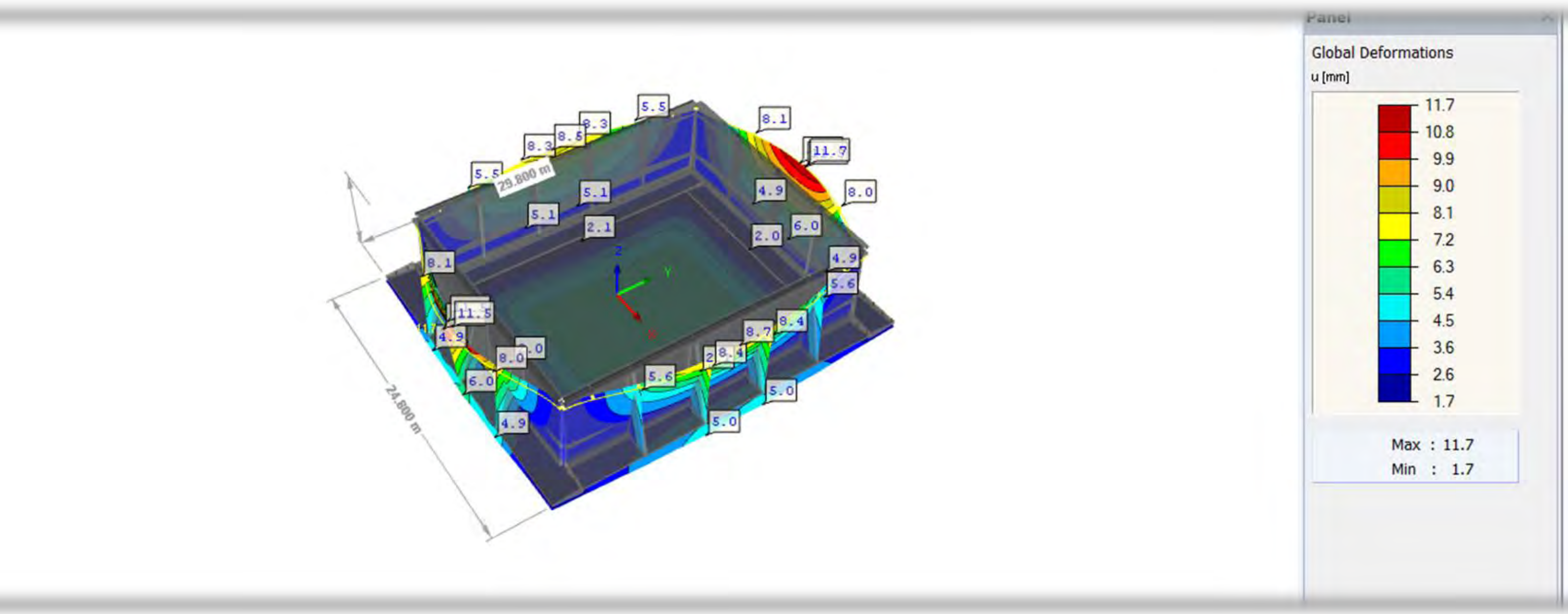


Figure 8.8: Deformations of pool's panels (mm).

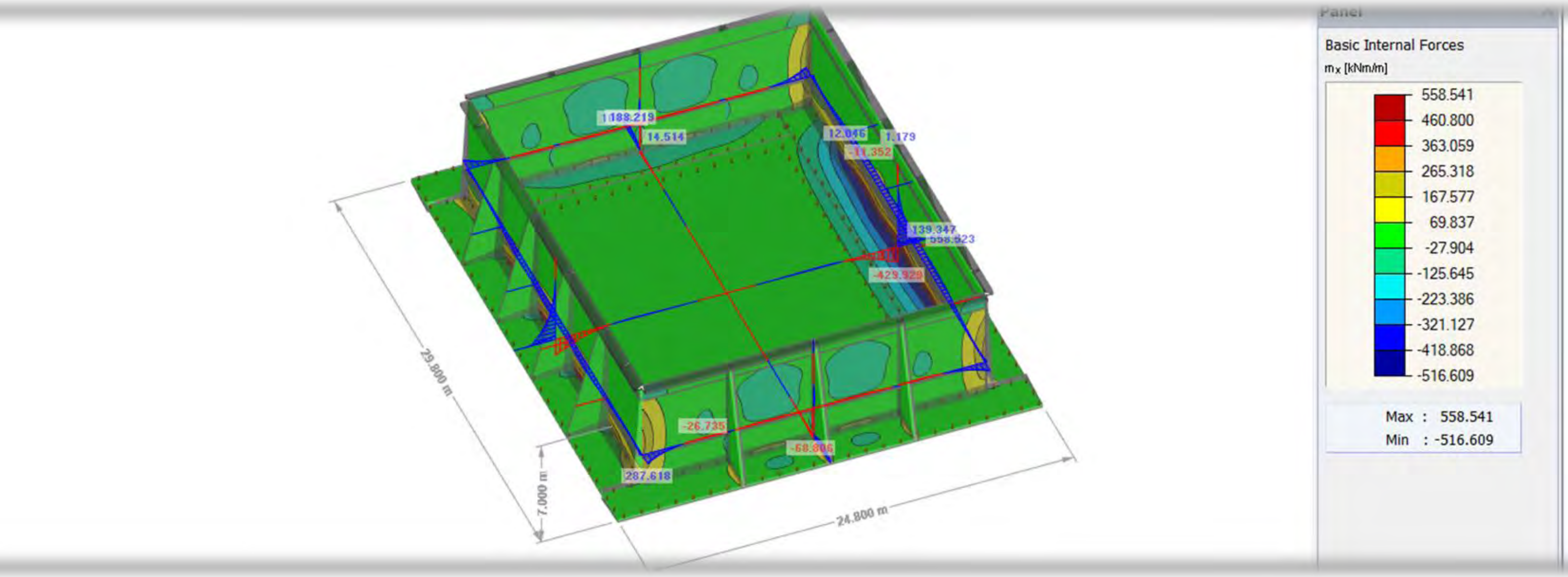


Figure 8.9: Bending Moment in x-direction (short direction) (KN-M).

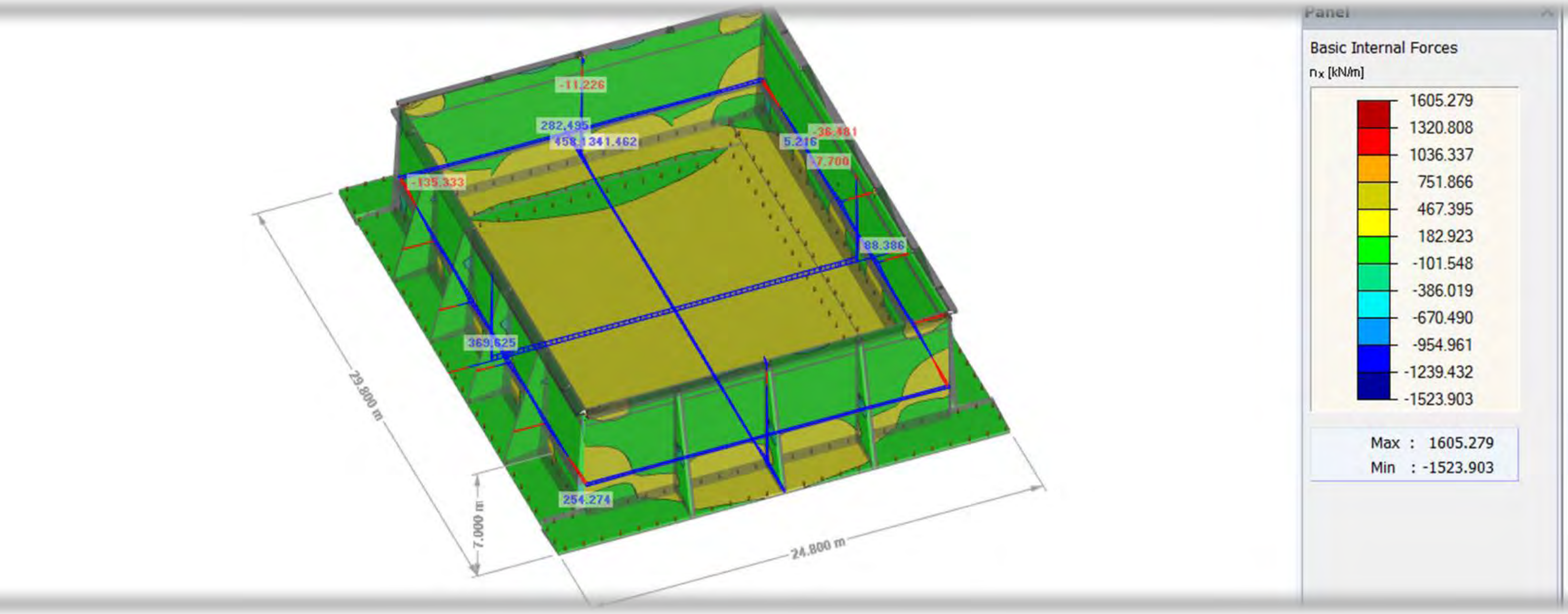


Figure 8.10: Normal Force in x-direction (short direction) (KN).

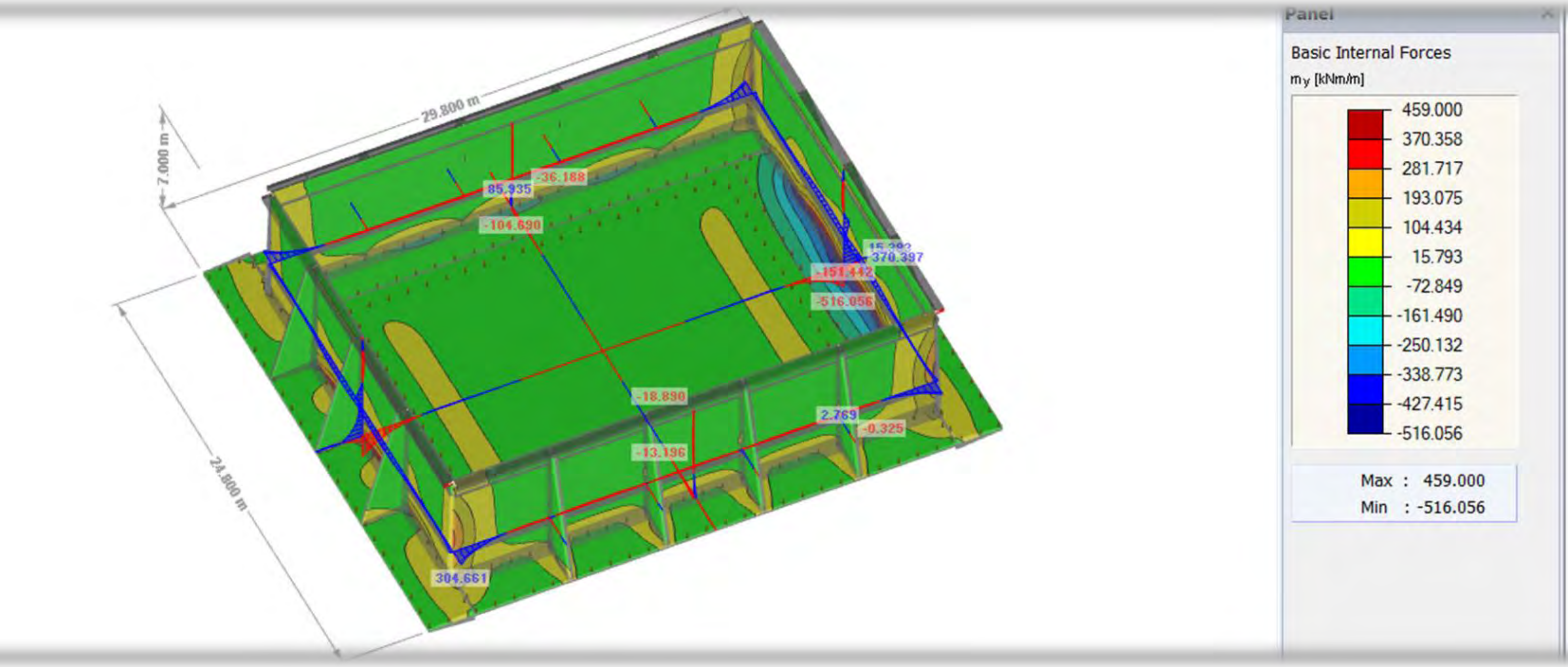


Figure 8.11: Bending moment in y-direction (long direction) (KN-M)

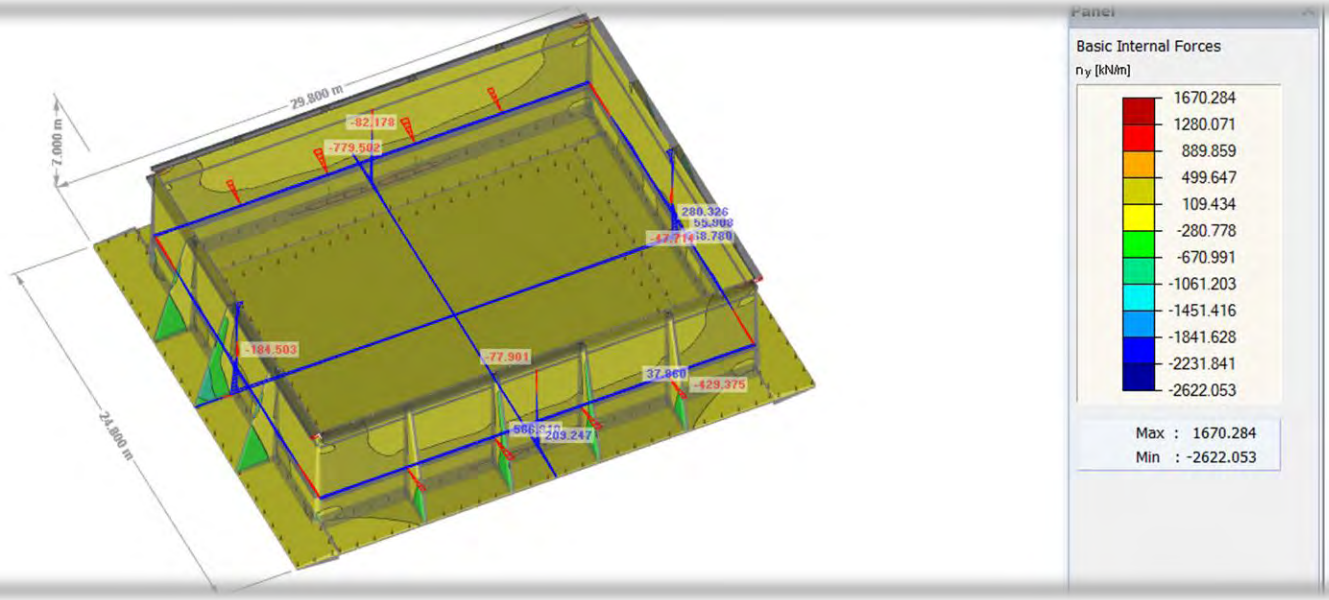


Figure 8.12: Normal Force in y-direction (long direction) (KN).

8.1.2 Analysis results as sections.

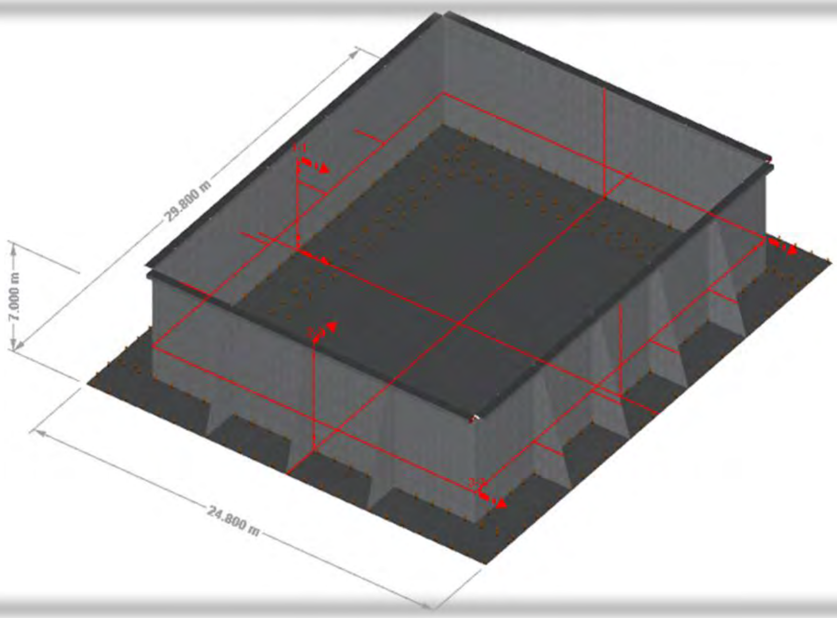


Figure 8.13: Sections through the pool.

➢ Maintenance Case.

- Section 1-1.

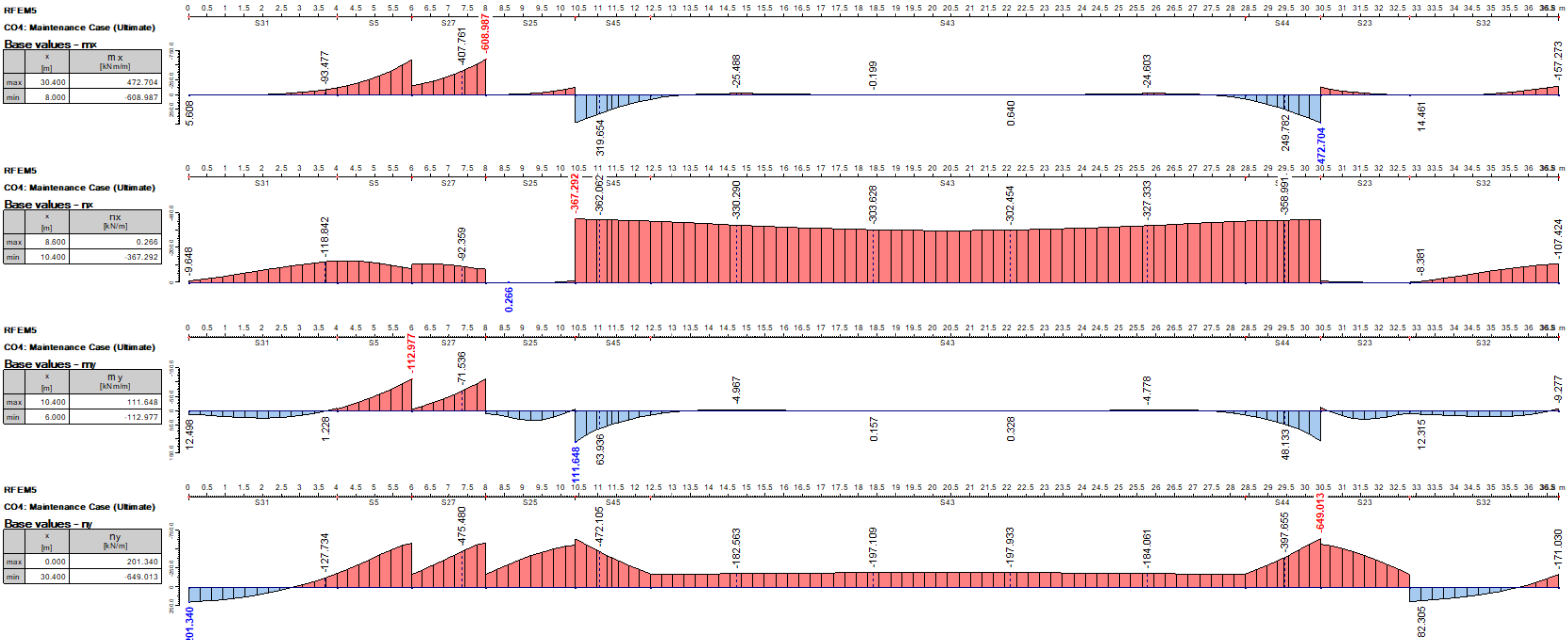


Figure 8.14: Bending moments and Normal forces through section 1-1 (Mx,Nx,My,Ny)..

- Section 2-2.

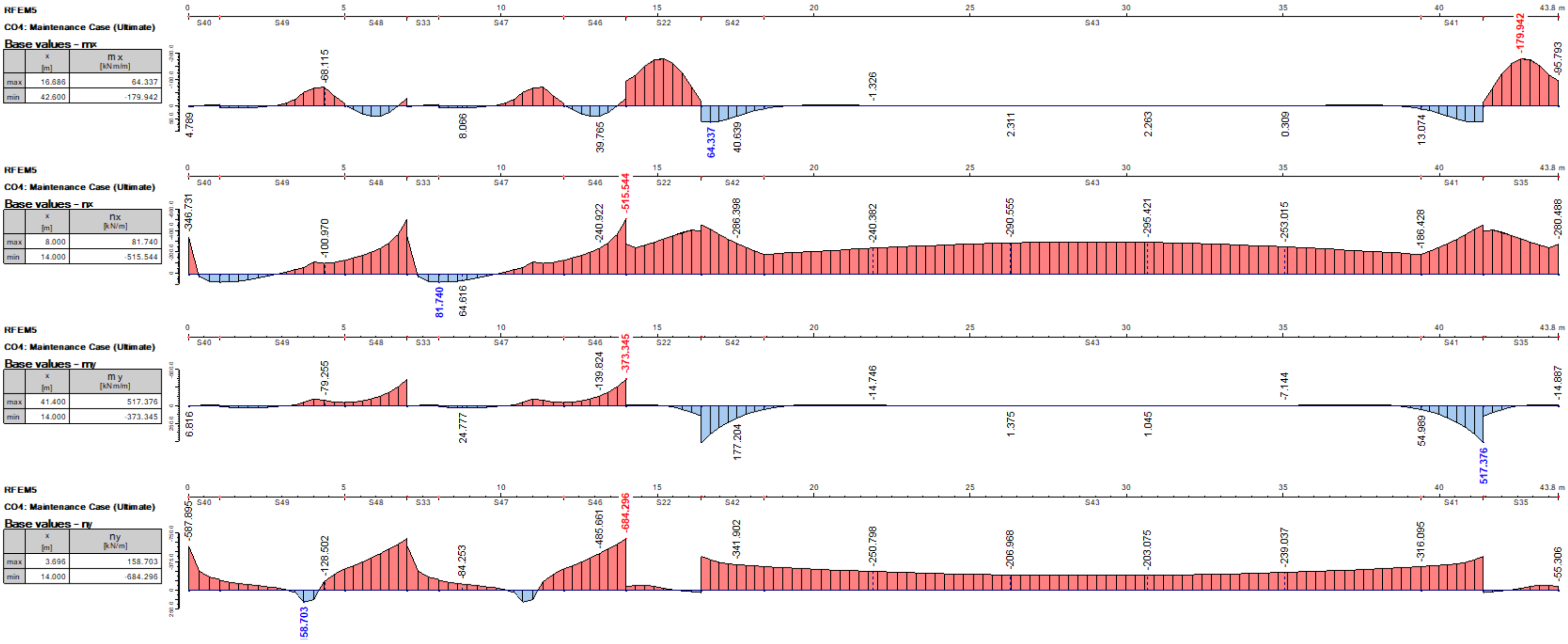


Figure 8.15: Bending moments and Normal forces through section 2-2 (Mx,Nx,My,Ny).

- Section 3-3.

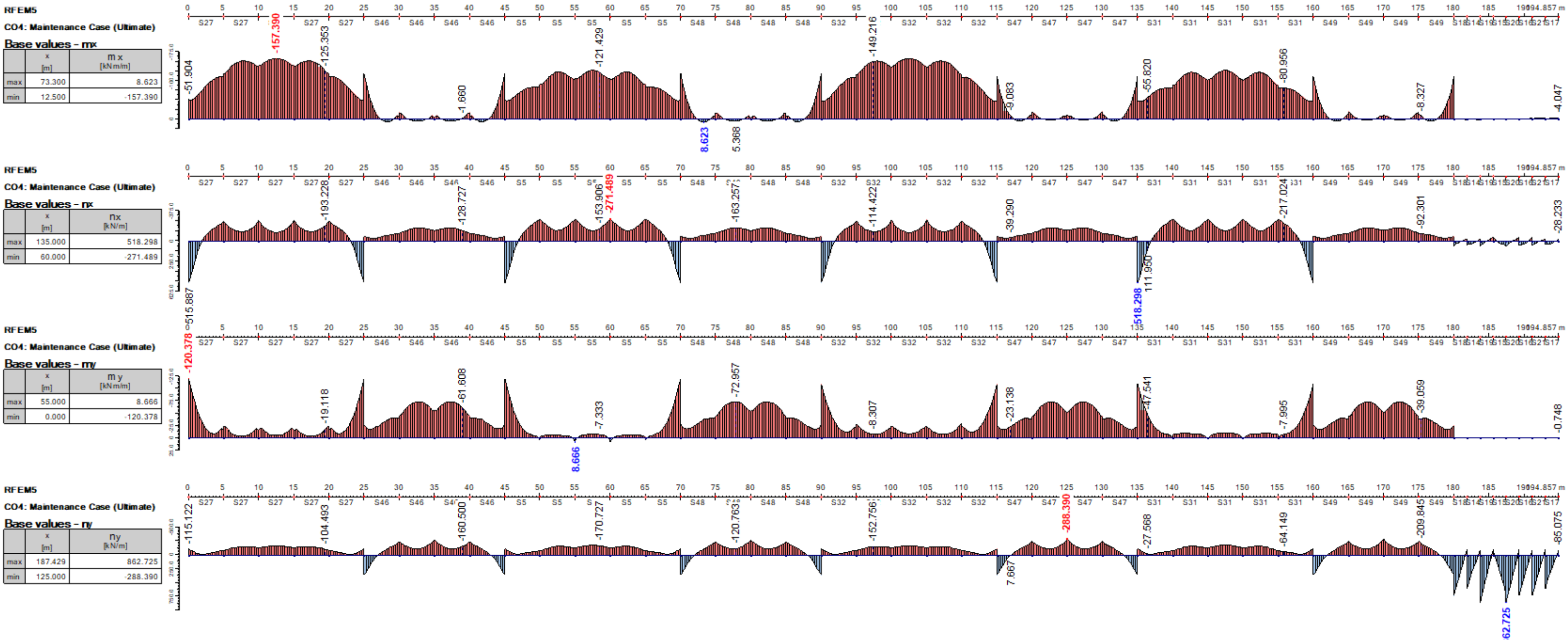


Figure 8.16: Bending moments and Normal forces through section 3-3 (Mx,Nx,My,Ny).

➢ **Working Case.**

- Section 1-1.

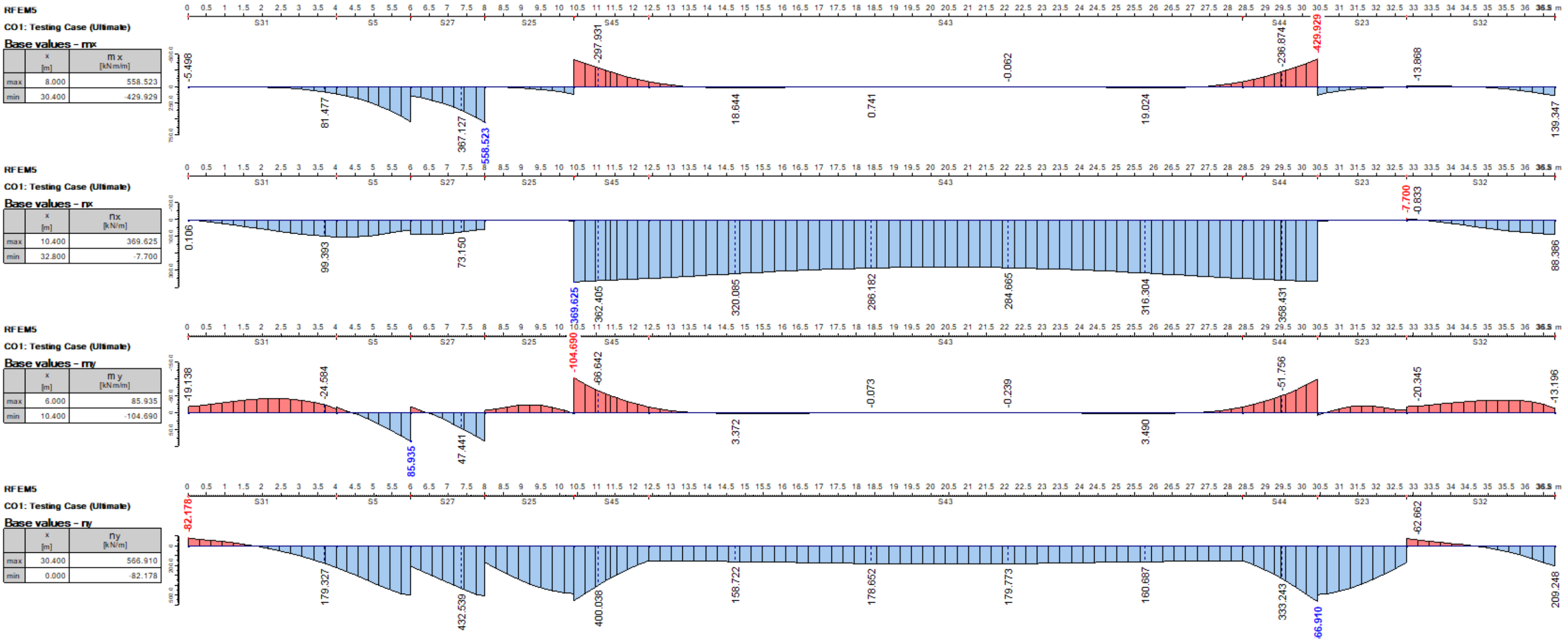
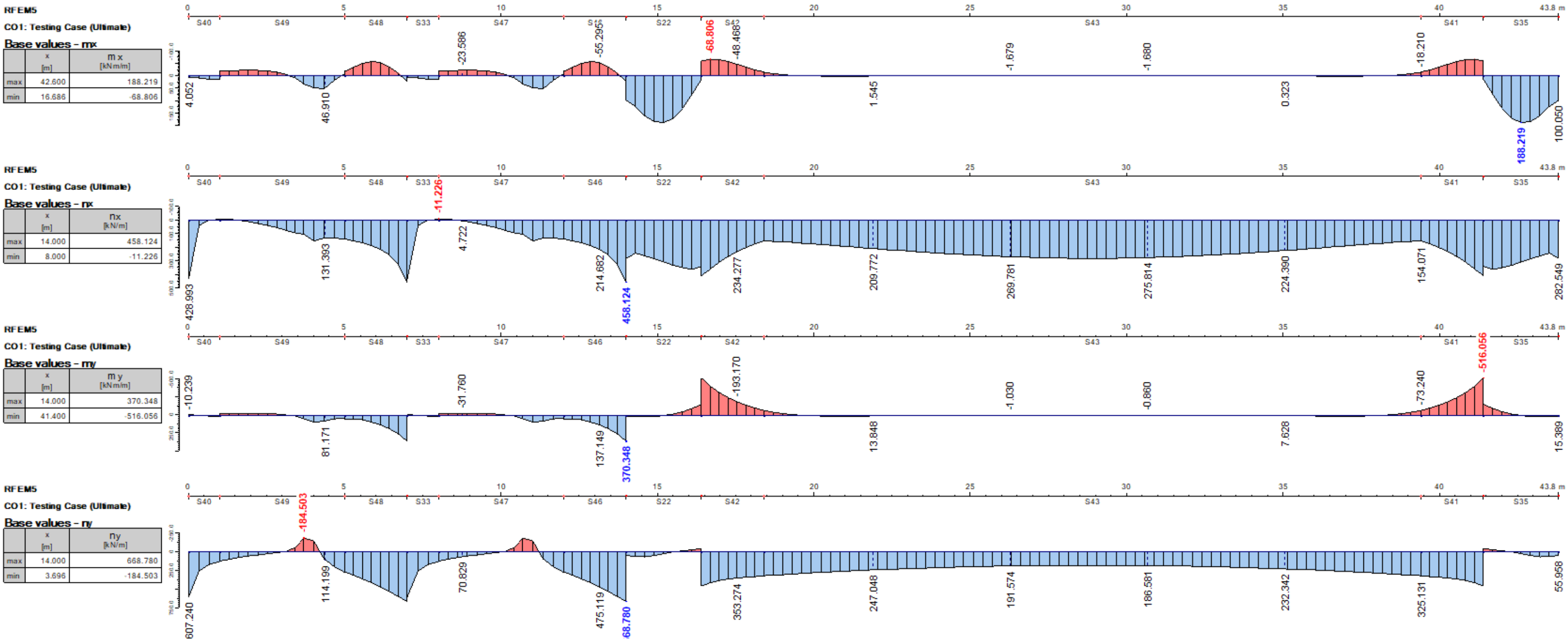


Figure 8.17: Bending moments and Normal forces through section 1-1 (Mx,Nx,My,Ny)..

- Section 2-2.



RFEM5

CO1: Testing Case (Ultimate)

Base values - mx

	x [m]	m x [kNm/m]
max	42.600	188.219
min	16.686	-68.806

RFEM5

CO1: Testing Case (Ultimate)

Base values - nx

	x [m]	nx [kN/m]
max	14.000	458.124
min	8.000	-11.226

RFEM5

CO1: Testing Case (Ultimate)

Base values - my

	x [m]	m y [kNm/m]
max	14.000	370.348
min	41.400	-516.056

RFEM5

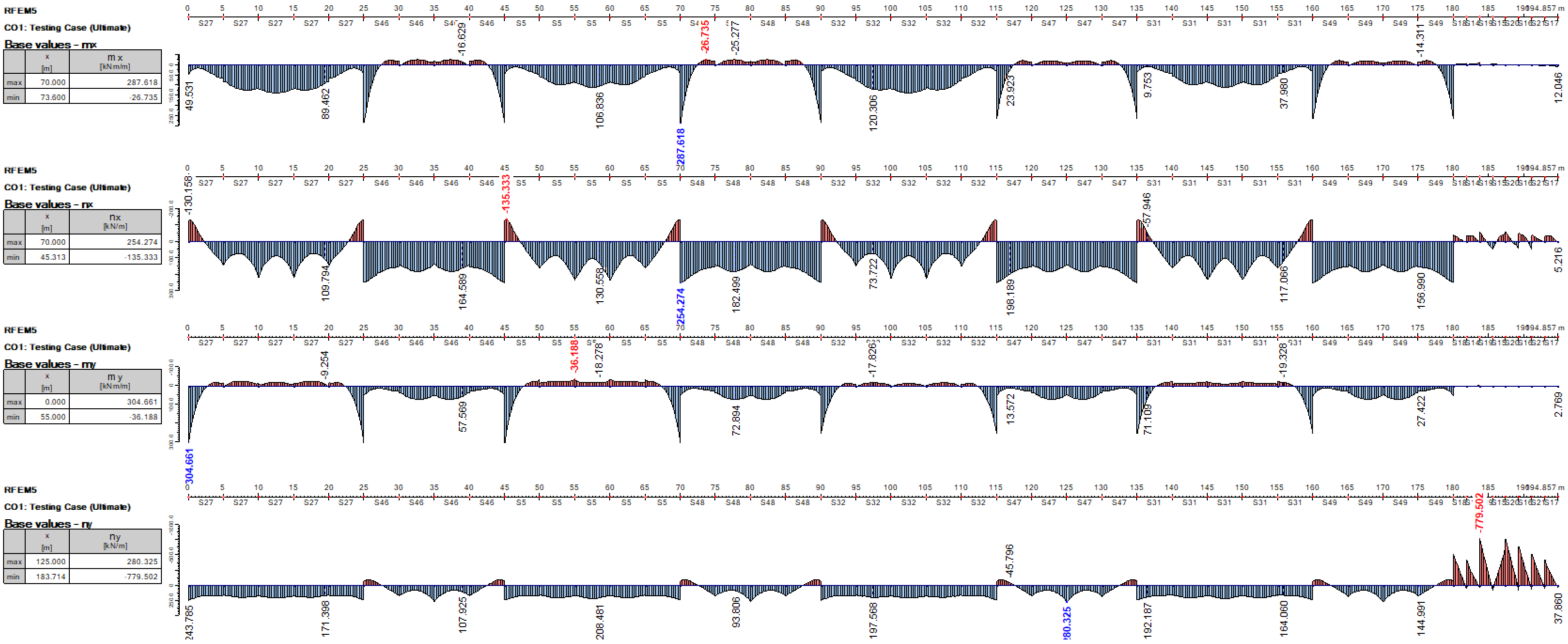
CO1: Testing Case (Ultimate)

Base values - ny

	x [m]	ny [kN/m]
max	14.000	668.780
min	3.696	-184.503

Figure 8.18: Bending moments and Normal forces through section 2-2 (Mx,Nx,My,Ny).

- Section 3-3.



RFEM5

CO1: Testing Case (Ultimate)

Base values - mx

	x [m]	m x [kNm/m]
max	70.000	287.618
min	73.600	-26.735

RFEM5

CO1: Testing Case (Ultimate)

Base values - nx

	x [m]	nx [kN/m]
max	70.000	254.274
min	45.313	-135.333

RFEM5

CO1: Testing Case (Ultimate)

Base values - my

	x [m]	m y [kNm/m]
max	0.000	304.661
min	55.000	-36.188

RFEM5

CO1: Testing Case (Ultimate)

Base values - ny

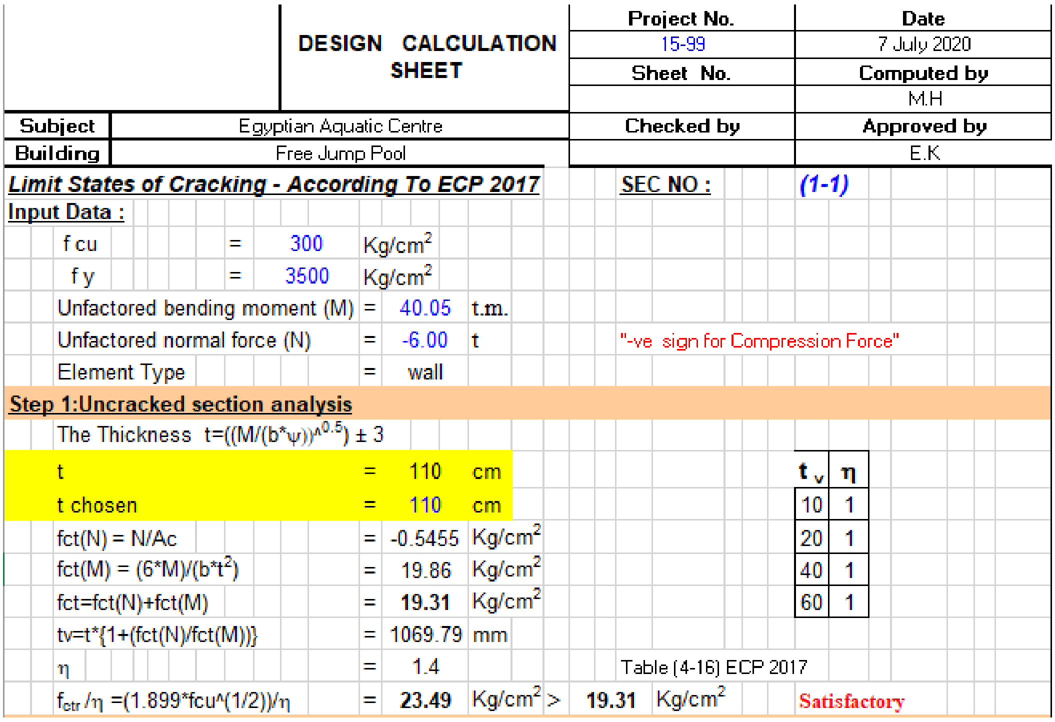
	x [m]	ny [kN/m]
max	125.000	280.325
min	183.714	-779.502

Figure 8.19: Bending moments and Normal forces through section 3-3 (Mx,Nx,My,Ny).

8.2 Design.

8.2.1 Design of Surfaces.

8.2.1.1 Concrete Dimensions.



	DESIGN CALCULATION SHEET	**Project No.**	**Date**
		15-99	7 July 2020
		Sheet No.	**Computed by**
			M.H
Subject	Egyptian Aquatic Centre	**Checked by**	**Approved by**
Building	Free Jump Pool		E.K

Limit States of Cracking - According To ECP 2017 **SEC NO :** ***(1-1)***

Input Data :

f cu = 300 Kg/cm^2

f y = 3500 Kg/cm^2

Unfactored bending moment (M) = 40.05 t.m.

Unfactored normal force (N) = -6.00 t "-ve sign for Compression Force"

Element Type = wall

Step 1:Uncracked section analysis

The Thickness t=((M/(b*ψ))^$^{0.5}$) ± 3

t = 110 cm

t chosen = 110 cm

fct(N) = N/Ac = -0.5455 Kg/cm^2

fct(M) = (6*M)/(b*t^2) = 19.86 Kg/cm^2

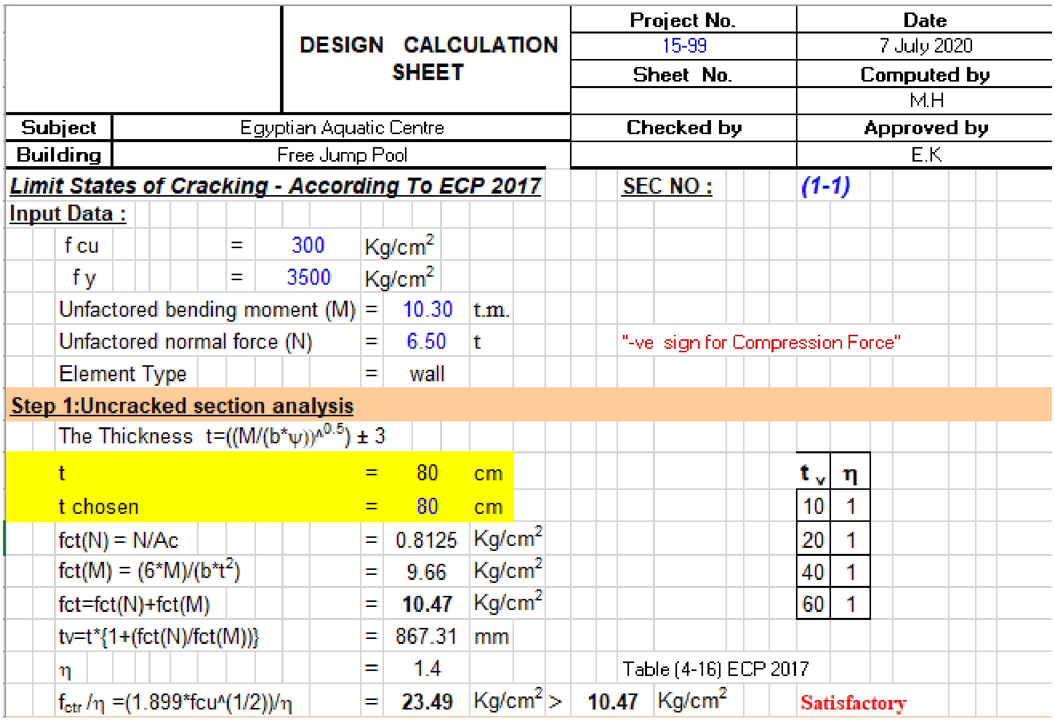
fct=fct(N)+fct(M) = **19.31** Kg/cm^2

tv=t*{1+(fct(N)/fct(M))} = 1069.79 mm

η = 1.4 Table (4-16) ECP 2017

f$_{ctr}$ /η =(1.899*fcu^(1/2))/η = **23.49** Kg/cm^2 > **19.31** Kg/cm^2 Satisfactory

t$_v$	**η**
10	1
20	1
40	1
60	1



	DESIGN CALCULATION SHEET	**Project No.**	**Date**
		15-99	7 July 2020
		Sheet No.	**Computed by**
			M.H
Subject	Egyptian Aquatic Centre	**Checked by**	**Approved by**
Building	Free Jump Pool		E.K

Limit States of Cracking - According To ECP 2017 **SEC NO :** ***(1-1)***

Input Data :

f cu = 300 Kg/cm^2

f y = 3500 Kg/cm^2

Unfactored bending moment (M) = 10.30 t.m.

Unfactored normal force (N) = 6.50 t "-ve sign for Compression Force"

Element Type = wall

Step 1:Uncracked section analysis

The Thickness t=((M/(b*ψ))^$^{0.5}$) ± 3

t = 80 cm

t chosen = 80 cm

fct(N) = N/Ac = 0.8125 Kg/cm^2

fct(M) = (6*M)/(b*t^2) = 9.66 Kg/cm^2

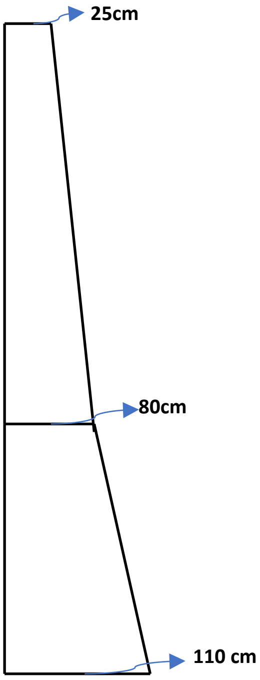
fct=fct(N)+fct(M) = **10.47** Kg/cm^2

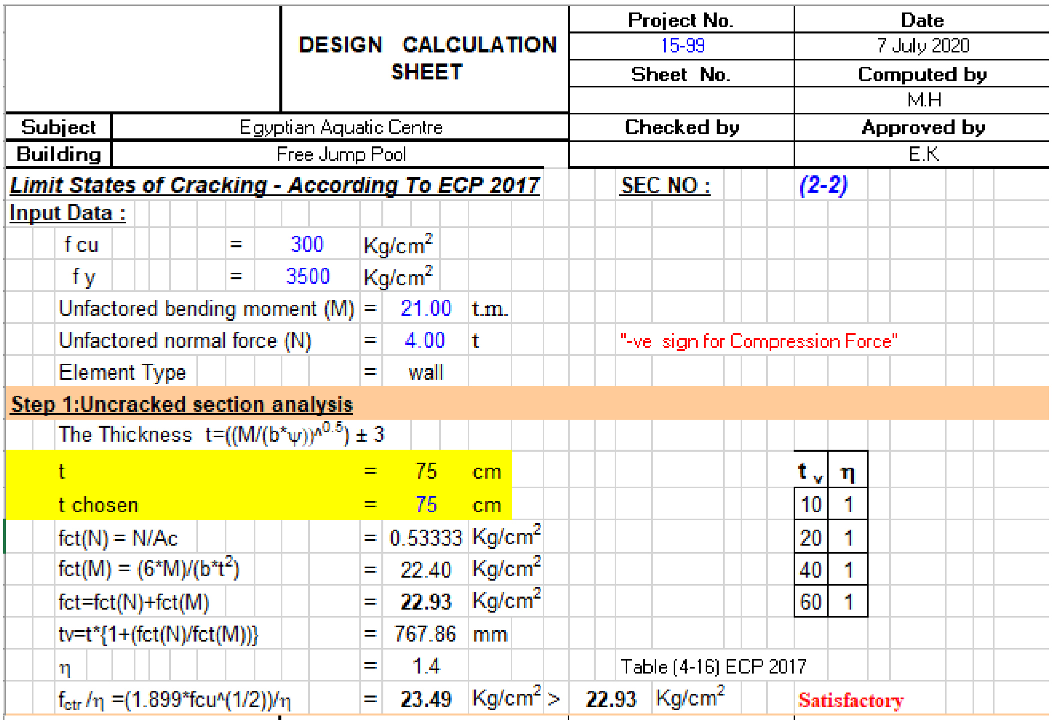
tv=t*{1+(fct(N)/fct(M))} = 867.31 mm

η = 1.4 Table (4-16) ECP 2017

f$_{ctr}$ /η =(1.899*fcu^(1/2))/η = **23.49** Kg/cm^2 > **10.47** Kg/cm^2 Satisfactory

t$_v$	**η**
10	1
20	1
40	1
60	1





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		15-99	7 July 2020
		Sheet No.	**Computed by**
			M.H
Subject	Egyptian Aquatic Centre	**Checked by**	**Approved by**
Building	Free Jump Pool		E.K

Limit States of Cracking - According To ECP 2017 — SEC NO : ***(2-2)***

Input Data :

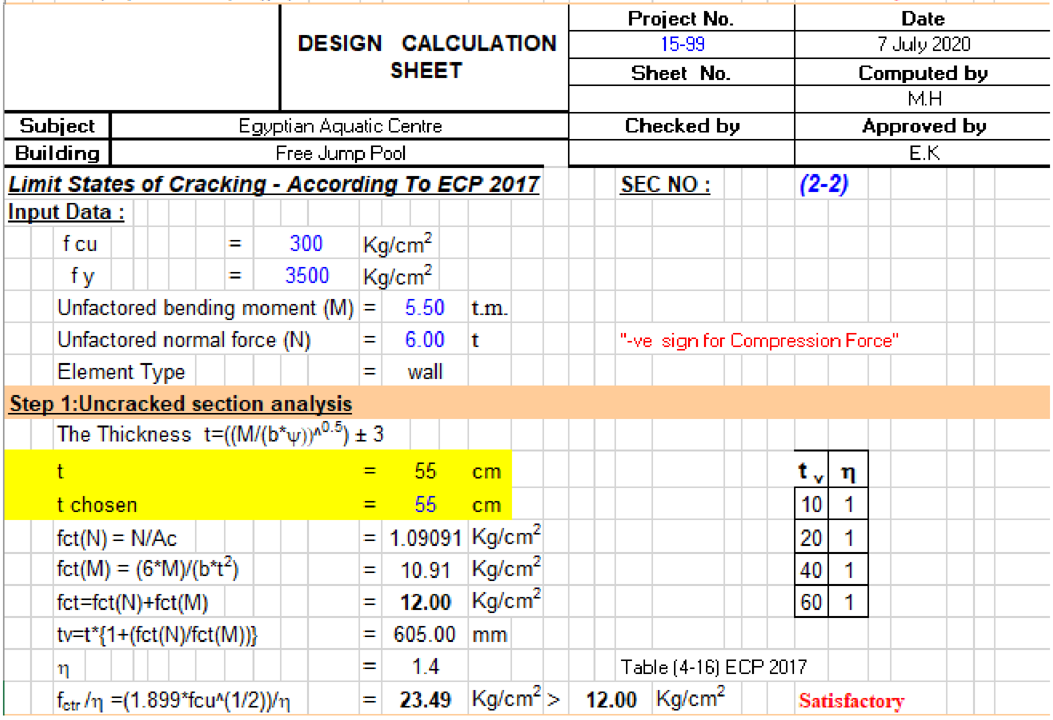
f cu	=	300	Kg/cm^2	
f y	=	3500	Kg/cm^2	
Unfactored bending moment (M)	=	21.00	t.m.	
Unfactored normal force (N)	=	4.00	t	"-ve sign for Compression Force"
Element Type	=	wall		

Step 1:Uncracked section analysis

The Thickness $t=((M/(b*\psi))^{0.5}) \pm 3$

t	=	75	cm			
t chosen	=	75	cm			
fct(N) = N/Ac	=	0.53333	Kg/cm^2			
fct(M) = (6*M)/(b*t^2)	=	22.40	Kg/cm^2			
fct=fct(N)+fct(M)	=	**22.93**	Kg/cm^2			
tv=t*{1+(fct(N)/fct(M))}	=	767.86	mm			
η	=	1.4		Table (4-16) ECP 2017		
f_{ctr}/η =(1.899*fcu^(1/2))/η	=	**23.49**	Kg/cm^2 >	**22.93**	Kg/cm^2	Satisfactory

t_v	η
10	1
20	1
40	1
60	1



	DESIGN CALCULATION SHEET	**Project No.**	**Date**
		15-99	7 July 2020
		Sheet No.	**Computed by**
			M.H
Subject	Egyptian Aquatic Centre	**Checked by**	**Approved by**
Building	Free Jump Pool		E.K

Limit States of Cracking - According To ECP 2017 — SEC NO : ***(2-2)***

Input Data :

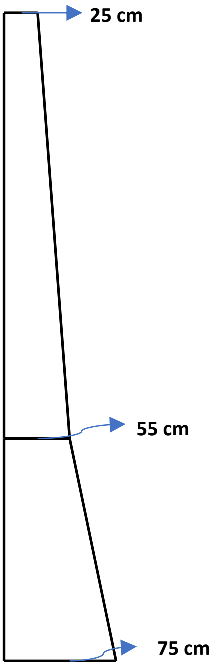
f cu	=	300	Kg/cm^2	
f y	=	3500	Kg/cm^2	
Unfactored bending moment (M)	=	5.50	t.m.	
Unfactored normal force (N)	=	6.00	t	"-ve sign for Compression Force"
Element Type	=	wall		

Step 1:Uncracked section analysis

The Thickness $t=((M/(b*\psi))^{0.5}) \pm 3$

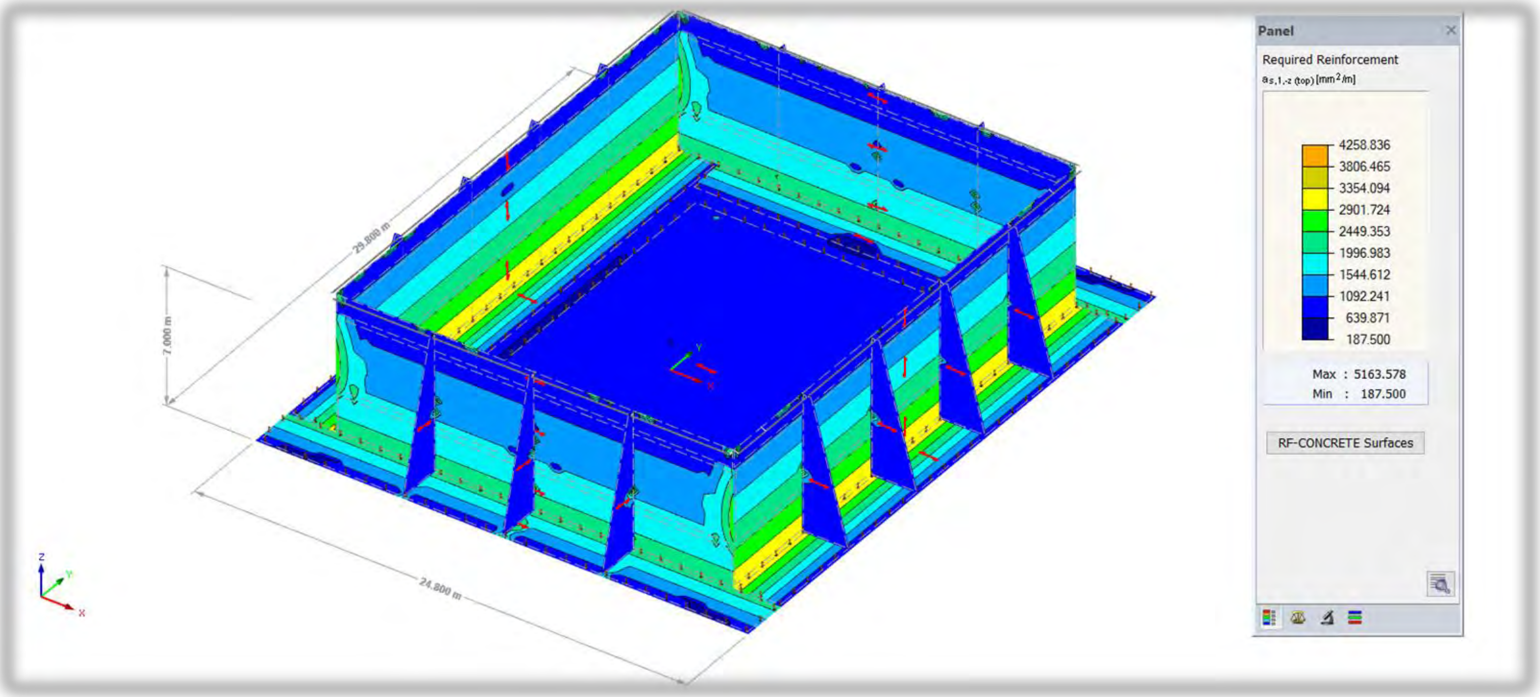
t	=	55	cm			
t chosen	=	55	cm			
fct(N) = N/Ac	=	1.09091	Kg/cm^2			
fct(M) = (6*M)/(b*t^2)	=	10.91	Kg/cm^2			
fct=fct(N)+fct(M)	=	**12.00**	Kg/cm^2			
tv=t*{1+(fct(N)/fct(M))}	=	605.00	mm			
η	=	1.4		Table (4-16) ECP 2017		
f_{ctr}/η =(1.899*fcu^(1/2))/η	=	**23.49**	Kg/cm^2 >	**12.00**	Kg/cm^2	Satisfactory

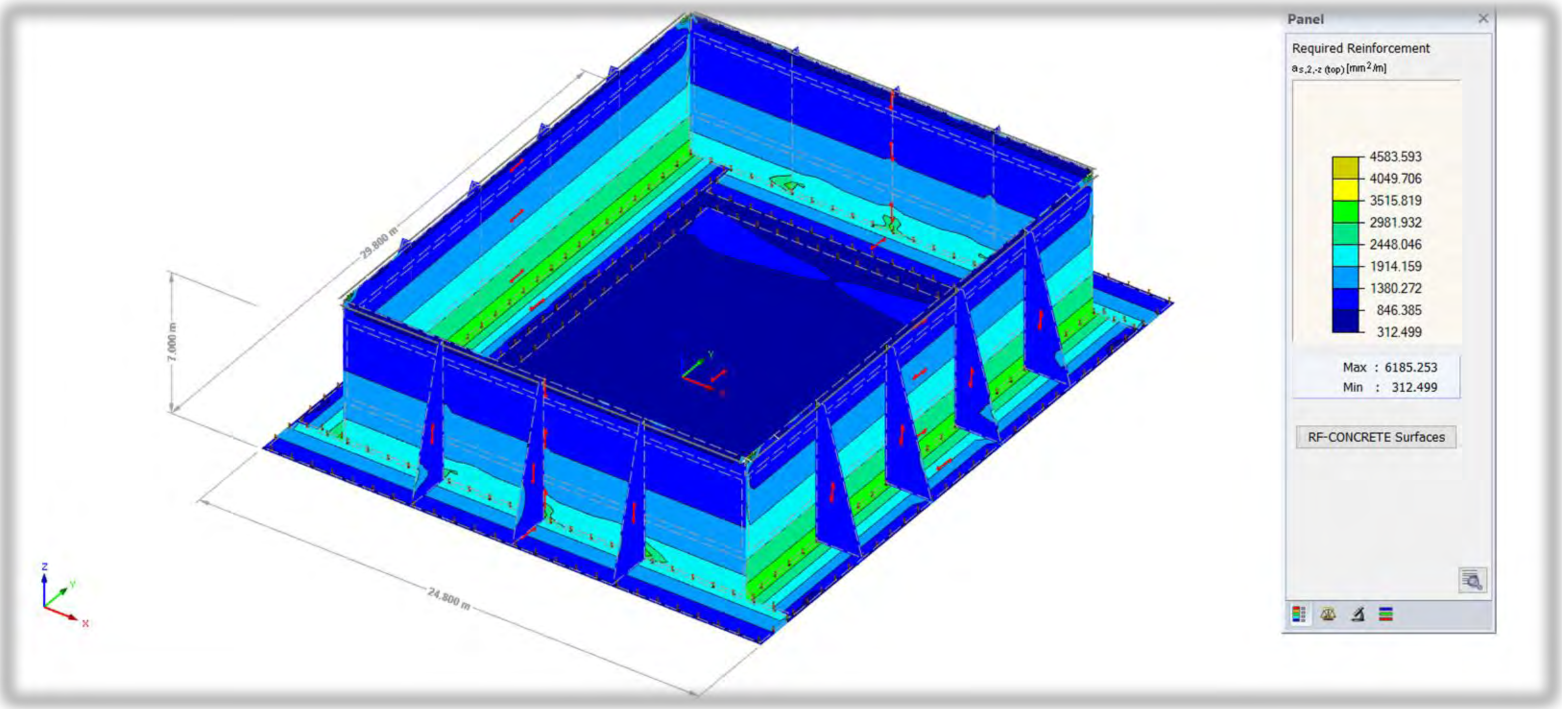
t_v	η
10	1
20	1
40	1
60	1



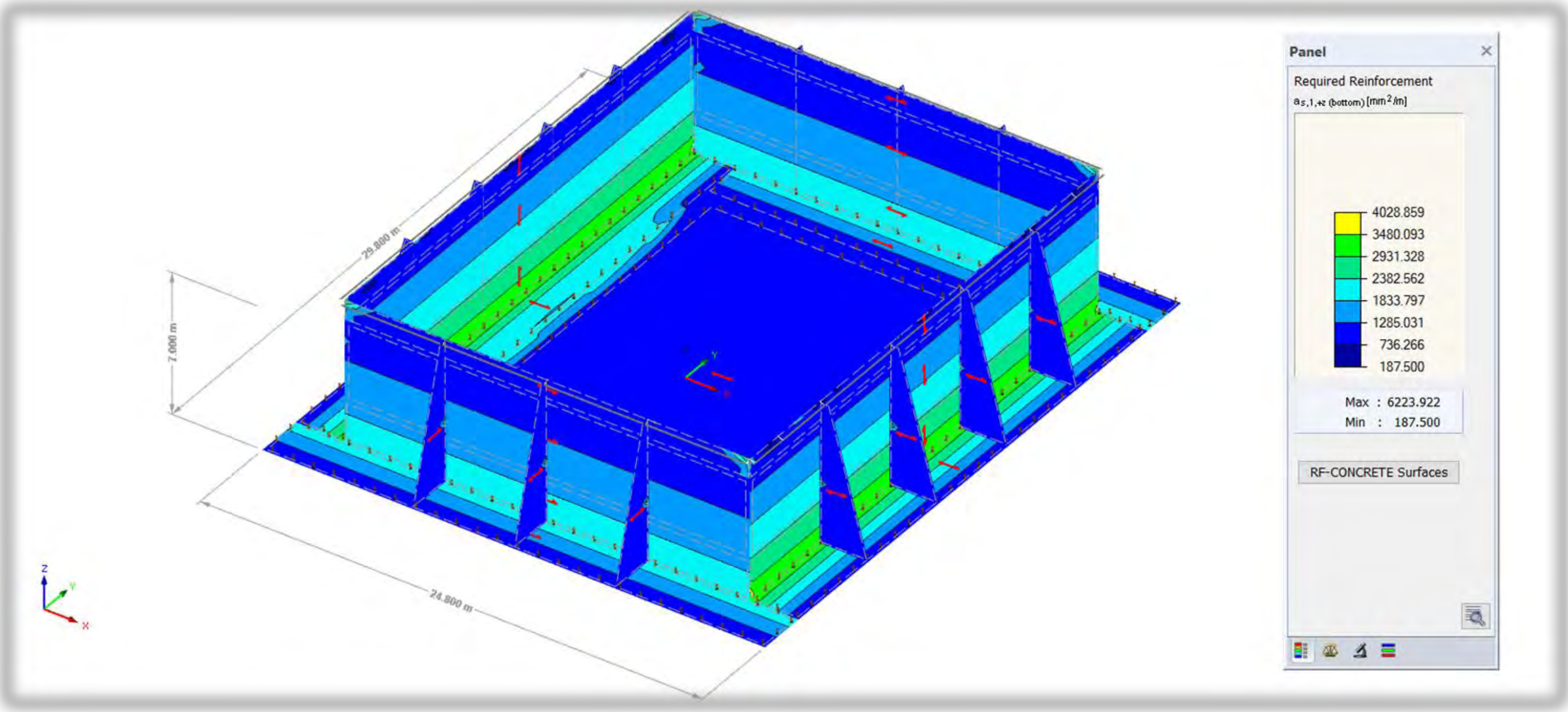
8.2.1.2 RFT Calculations.

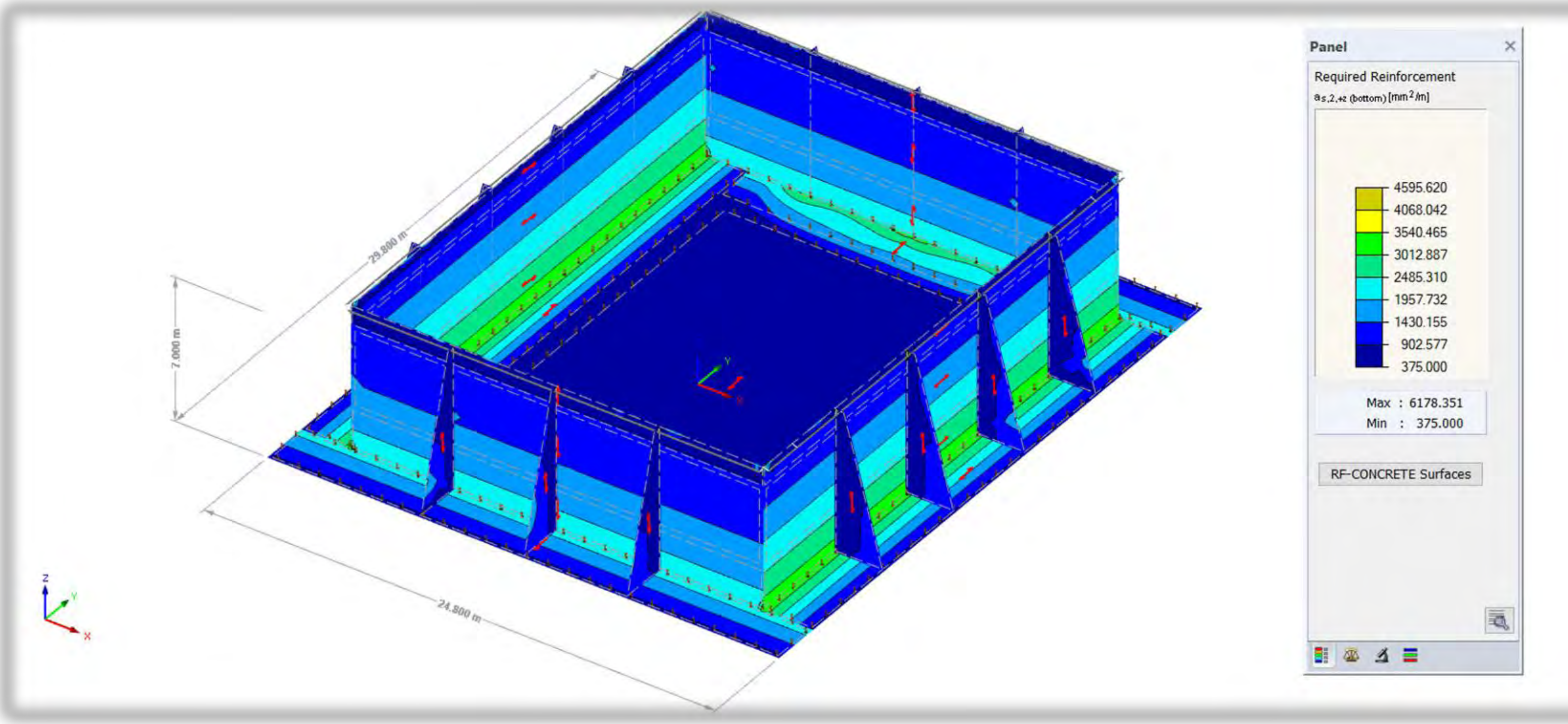
- Required Reinforcement areas as contour range.
  - External and Top Required Reinforcement.





- Internal and Bottom Required Reinforcement.





- Required Reinforcement areas as sections.

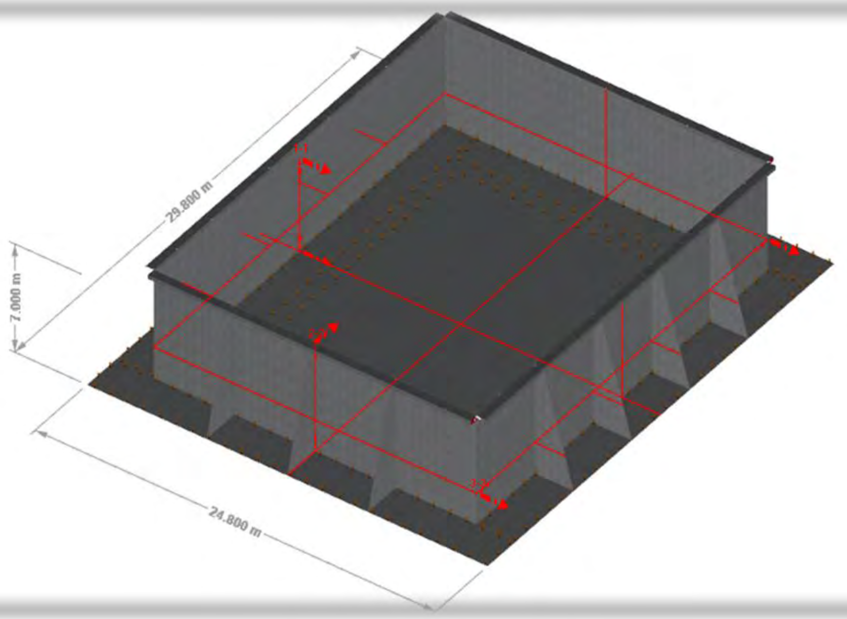
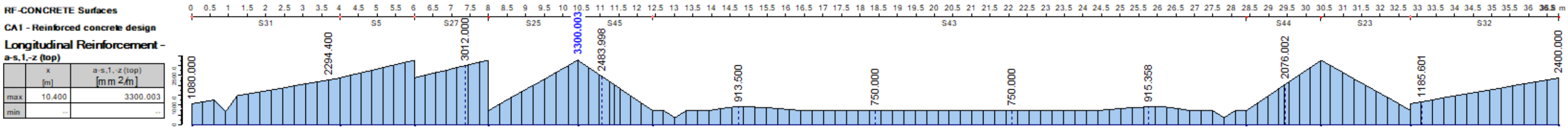


Figure 8.20: Sections through the pool.

- Section 1.

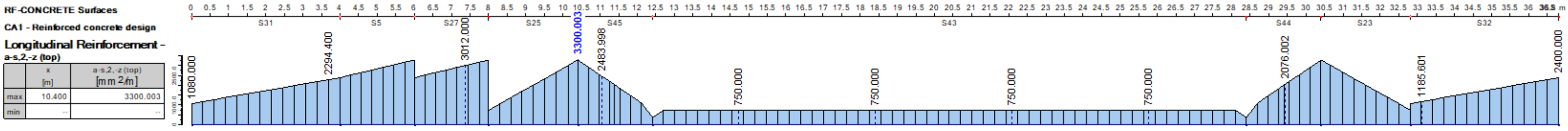


RF-CONCRETE Surfaces

CA1 - Reinforced concrete design

Longitudinal Reinforcement - a-s,1,-z (top)

	x [m]	a-s,1,-z (top) [mm 2/m]
max	10.400	3300.003
min	--	--

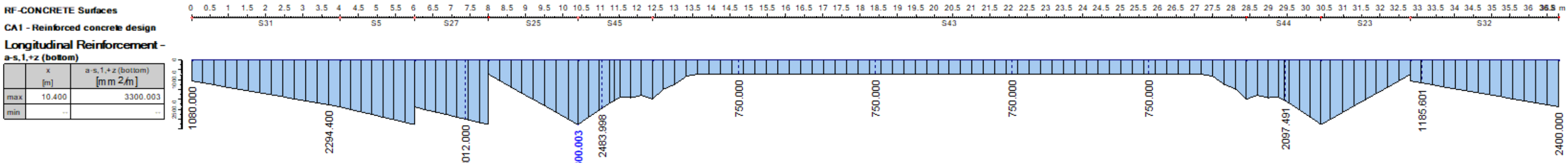


RF-CONCRETE Surfaces

CA1 - Reinforced concrete design

Longitudinal Reinforcement - a-s,2,-z (top)

	x [m]	a-s,2,-z (top) [mm 2/m]
max	10.400	3300.003
min	--	--

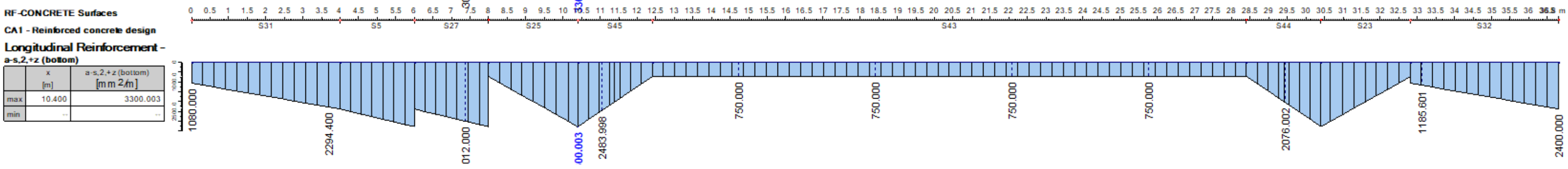


RF-CONCRETE Surfaces

CA1 - Reinforced concrete design

Longitudinal Reinforcement - a-s,1,+z (bottom)

	x [m]	a-s,1,+z (bottom) [mm 2/m]
max	10.400	3300.003
min	--	--



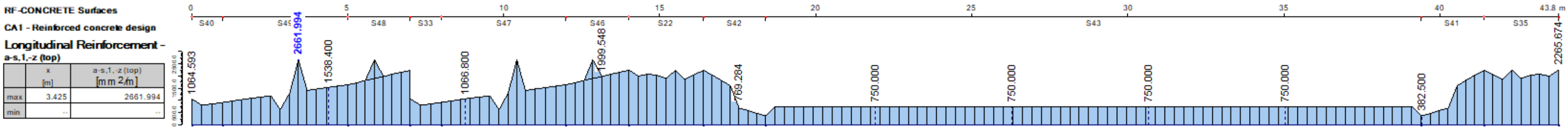
RF-CONCRETE Surfaces

CA1 - Reinforced concrete design

Longitudinal Reinforcement - a-s,2,+z (bottom)

	x [m]	a-s,2,+z (bottom) [mm 2/m]
max	10.400	3300.003
min	--	--

- Section 2.

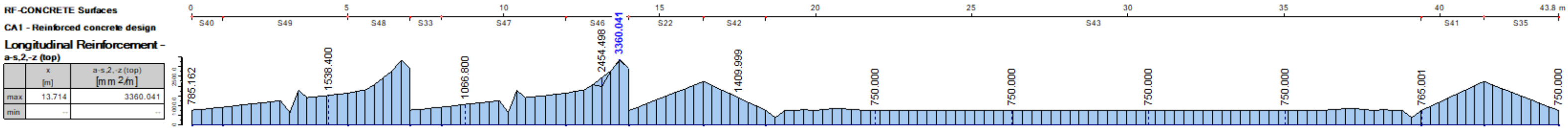


RF-CONCRETE Surfaces

CA1 - Reinforced concrete design

Longitudinal Reinforcement - a-s,1,-z (top)

	x [m]	a-s,1,-z (top) [mm 2/m]
max	3.425	2661.994
min	--	--

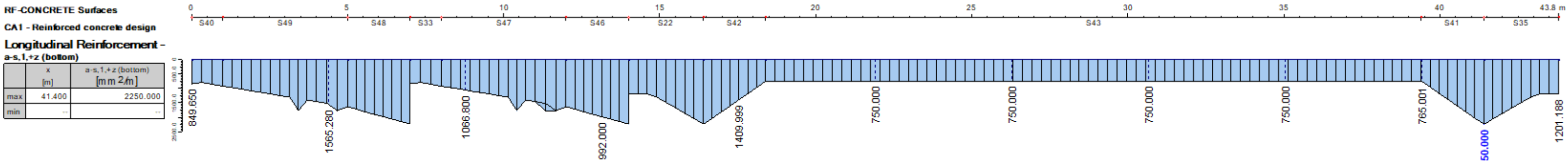


RF-CONCRETE Surfaces

CA1 - Reinforced concrete design

Longitudinal Reinforcement - a-s,2,-z (top)

	x [m]	a-s,2,-z (top) [mm 2/m]
max	13.714	3360.041
min	--	--

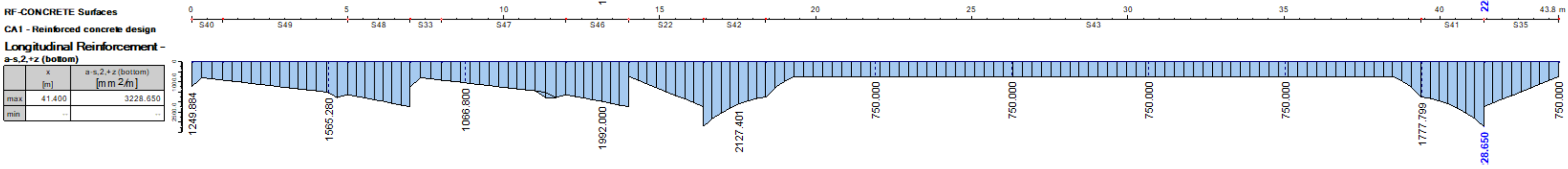


RF-CONCRETE Surfaces

CA1 - Reinforced concrete design

Longitudinal Reinforcement - a-s,1,+z (bottom)

	x [m]	a-s,1,+z (bottom) [mm 2/m]
max	41.400	2250.000
min	--	--



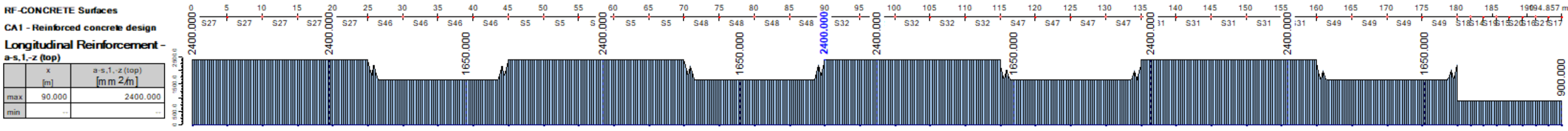
RF-CONCRETE Surfaces

CA1 - Reinforced concrete design

Longitudinal Reinforcement - a-s,2,+z (bottom)

	x [m]	a-s,2,+z (bottom) [mm 2/m]
max	41.400	3228.650
min	--	--

- Section 3.

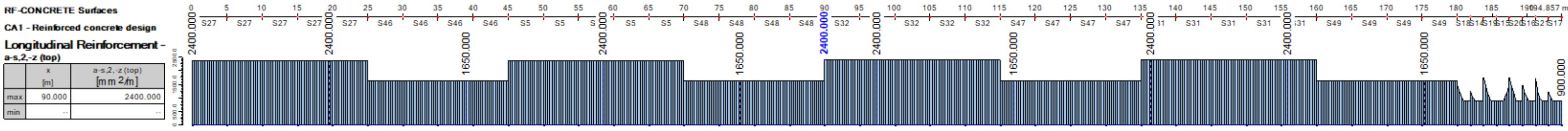


RF-CONCRETE Surfaces

CA1 - Reinforced concrete design

Longitudinal Reinforcement - a-s,1,-z (top)

	x [m]	a-s,1,-z (top) [mm 2/m]
max	90.000	2400.000
min	--	--

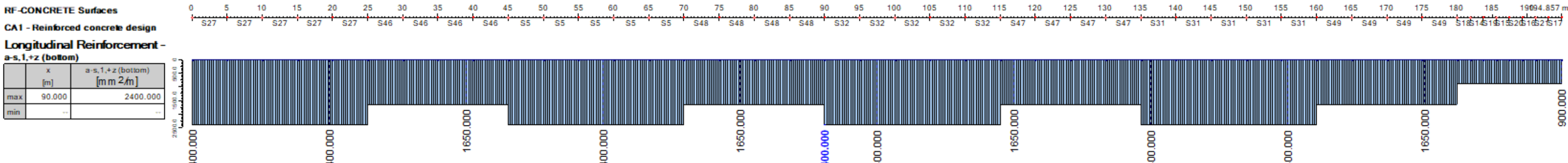


RF-CONCRETE Surfaces

CA1 - Reinforced concrete design

Longitudinal Reinforcement - a-s,2,-z (top)

	x [m]	a-s,2,-z (top) [mm 2/m]
max	90.000	2400.000
min	--	--

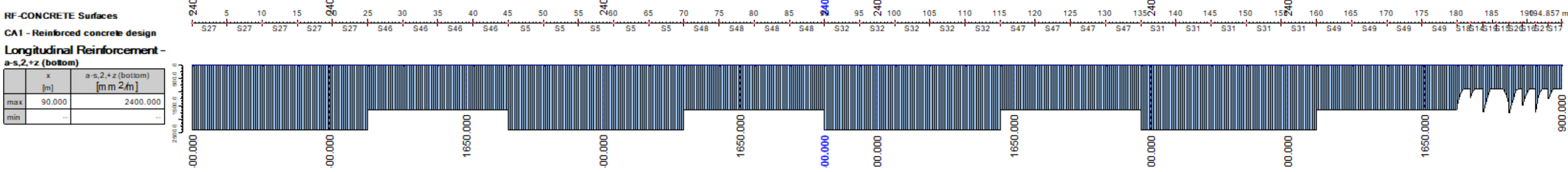


RF-CONCRETE Surfaces

CA1 - Reinforced concrete design

Longitudinal Reinforcement - a-s,1,+z (bottom)

	x [m]	a-s,1,+z (bottom) [mm 2/m]
max	90.000	2400.000
min	--	--



RF-CONCRETE Surfaces

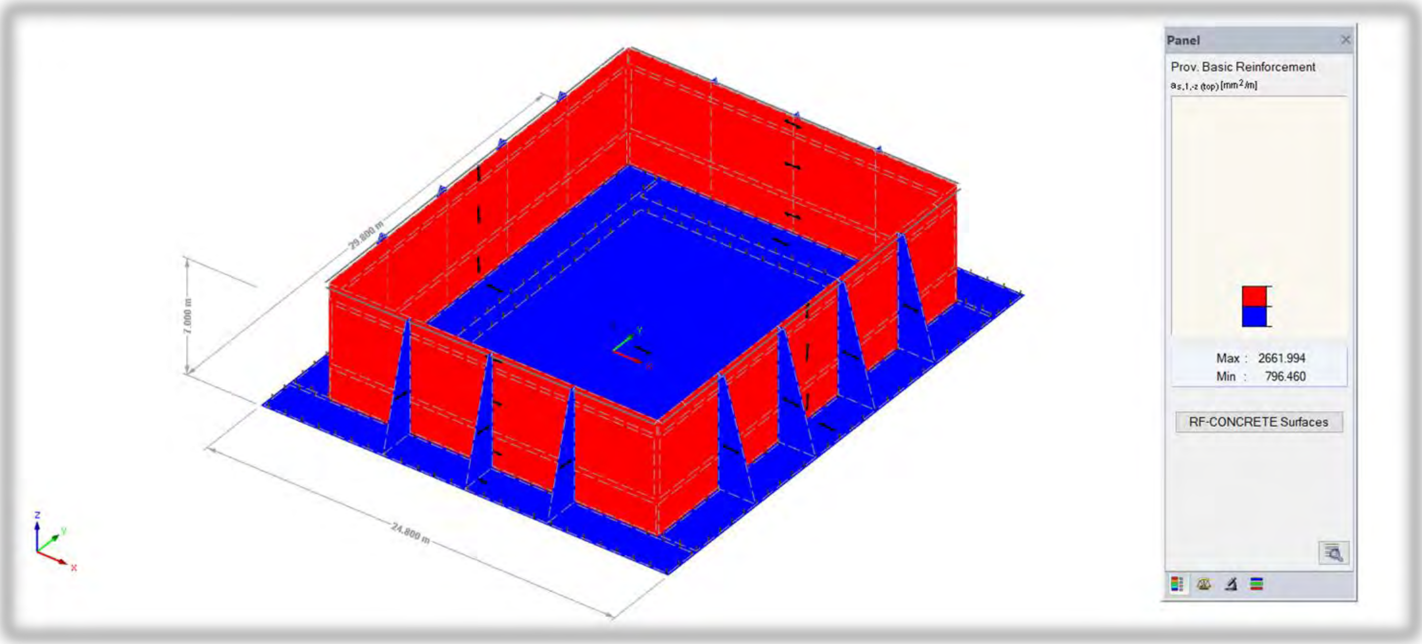
CA1 - Reinforced concrete design

Longitudinal Reinforcement - a-s,2,+z (bottom)

	x [m]	a-s,2,+z (bottom) [mm 2/m]
max	90.000	2400.000
min	--	--

➢ Additional Reinforcement areas.

- By assume using 7 T 22/m’ and 7 T 12/m’ as external and top reinforcement as shown in Fig. 8.21, the additional reinforcement will follow the values shown in Fig. 8.22.



Panel

Prov. Basic Reinforcement

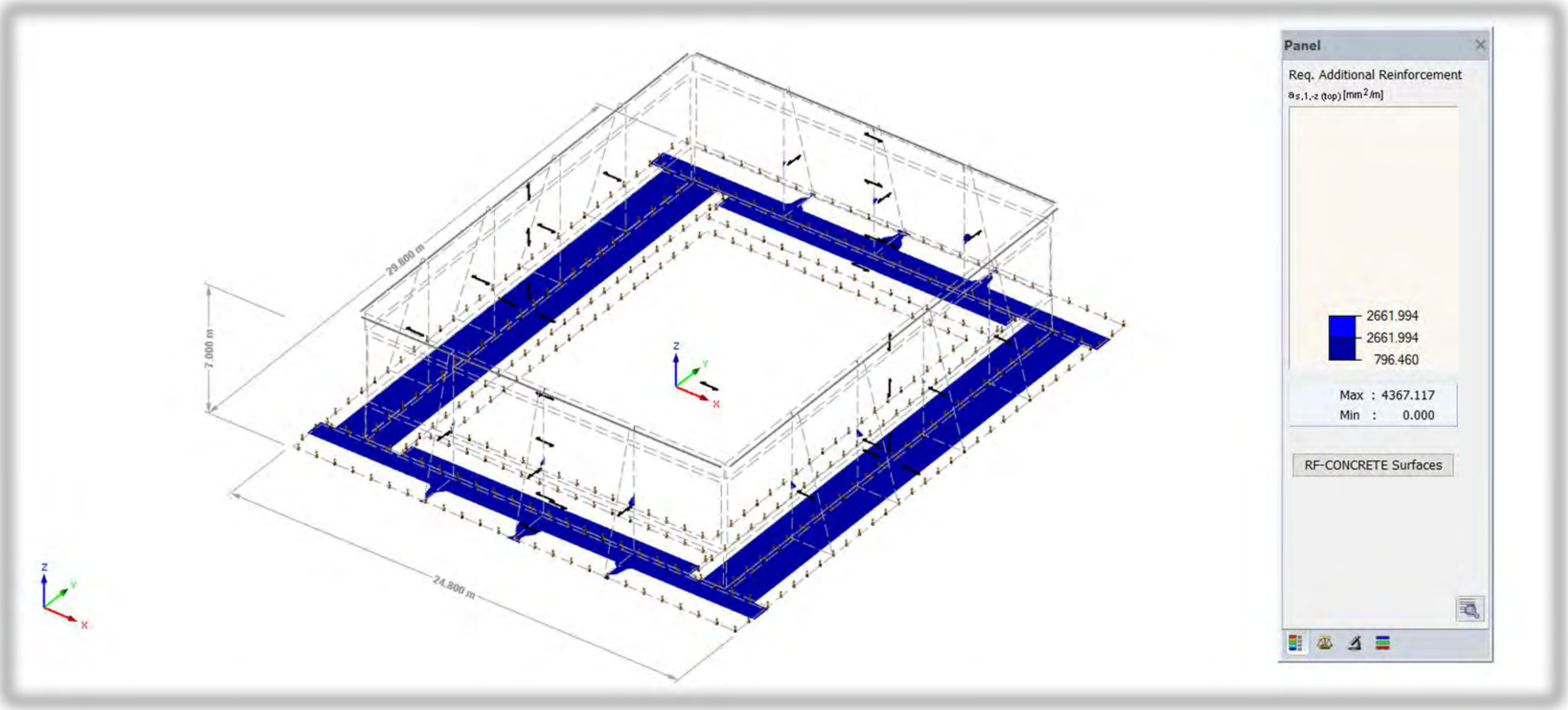
a s,1,-z (top) [mm 2/m]

Max : 2661.994

Min : 796.460

RF-CONCRETE Surfaces

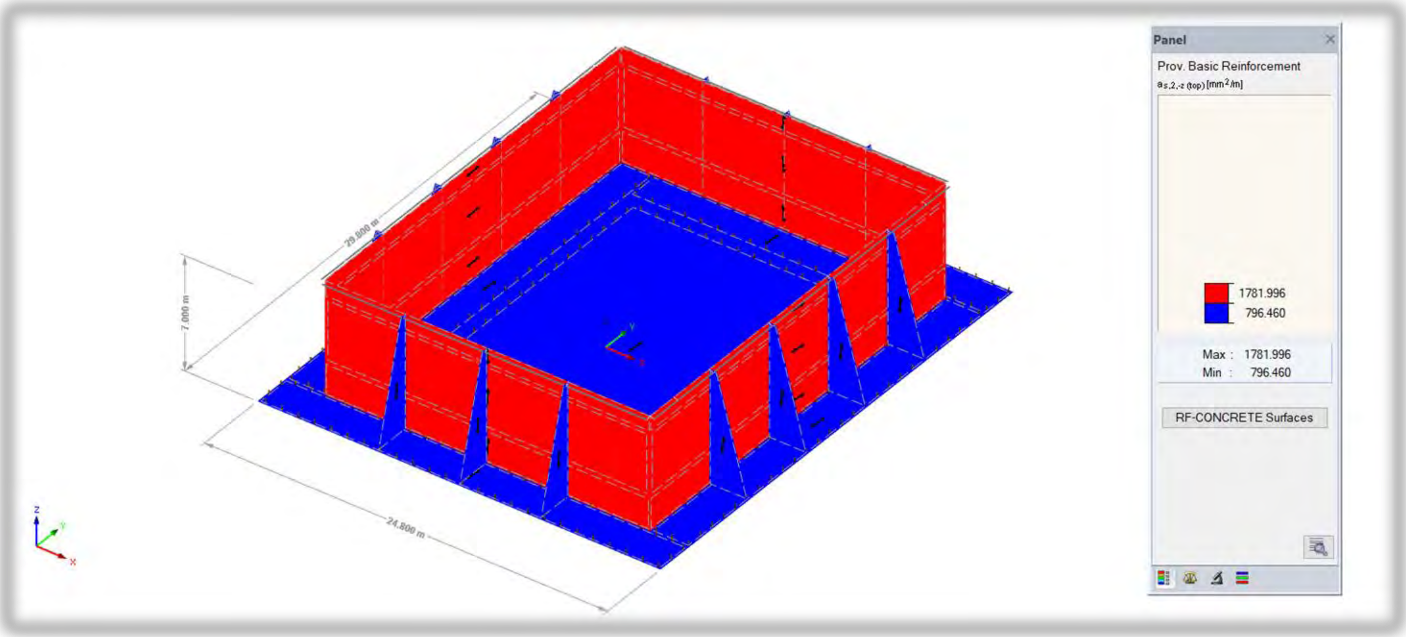
Figure 8.21: Provided Reinforcement Areas.



Max : 4367.117
Min : 0.000

Figure 8.22: Required Additional Reinforcement Areas (External & Top).

- By assume using 7 T 18/m’ and 7 T 12/m’ as external and top reinforcement as shown in Fig. 8.23, the additional reinforcement will follow the values shown in Fig. 8.24.



Max : 1781.996
Min : 796.460

Figure 8.24: Provided Reinforcement Areas.

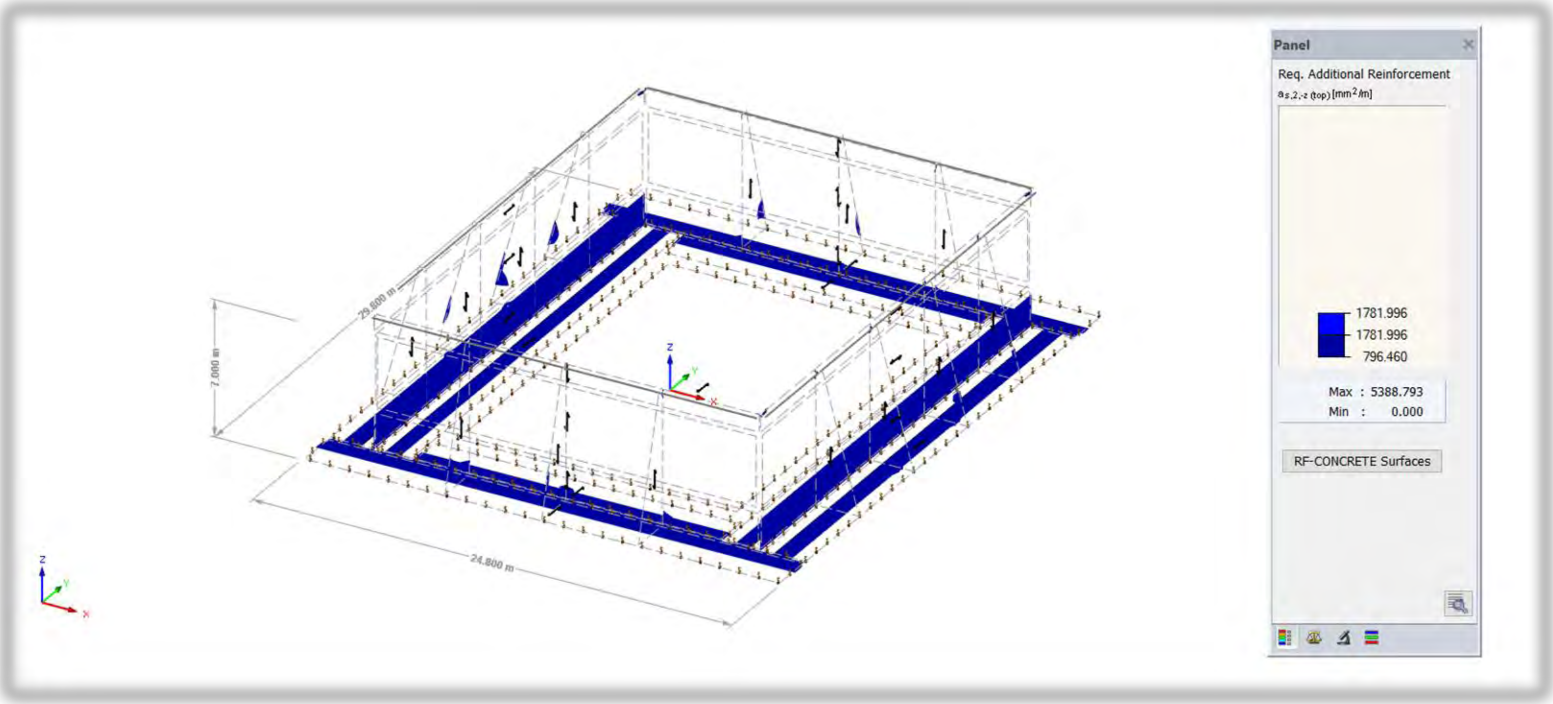


Figure 8.24: Required Additional Reinforcement Areas (External & Top).

- By assume using 7 T 18/m’ and 7 T 12/m’ as Internal and Bottom reinforcement as shown in Fig. 8.21, the additional reinforcement will follow the values shown in Fig. 8.25.

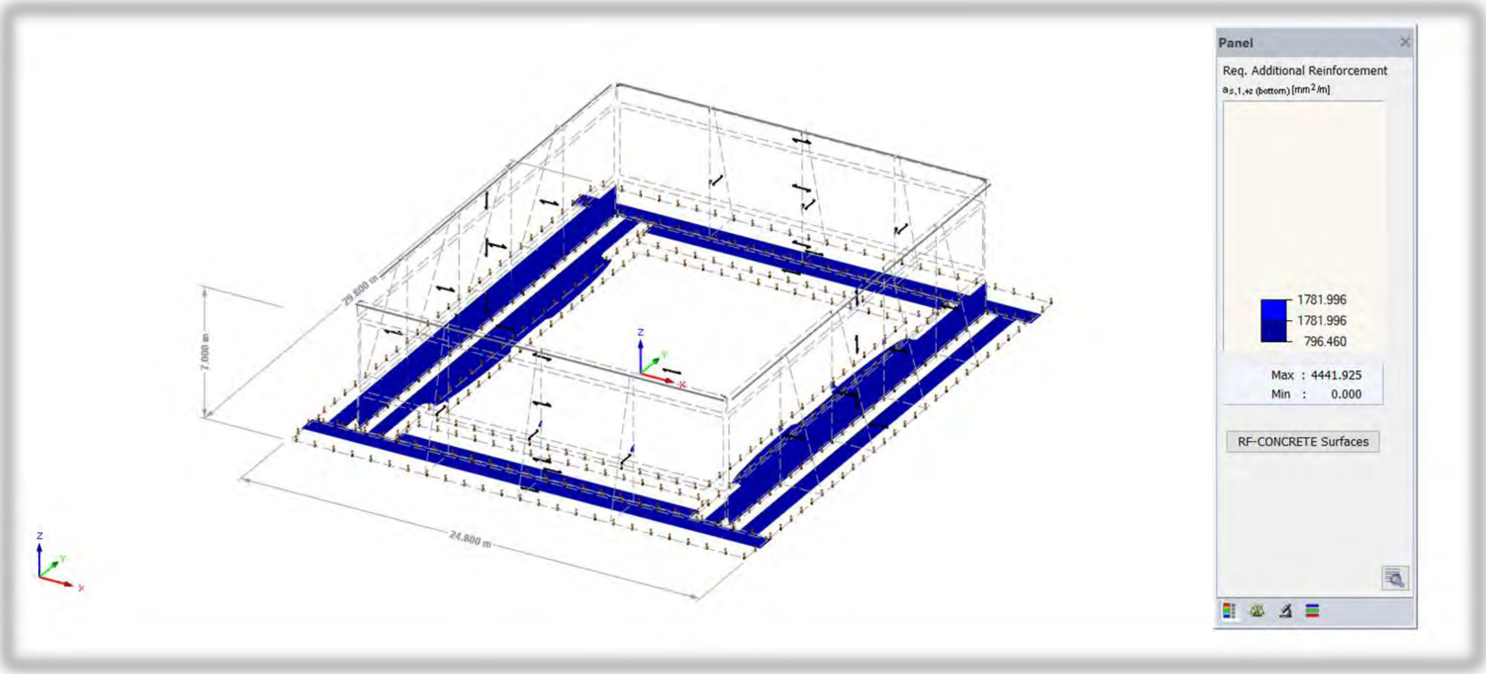


Figure 8.25: Required Additional Reinforcement Areas (Internal & Bottom).

- By assume using 7 T 18/m’ and 7 T 12/m’ as Internal and Bottom reinforcement as shown in Fig. 8.23, the additional reinforcement will follow the values shown in Fig. 8.26.

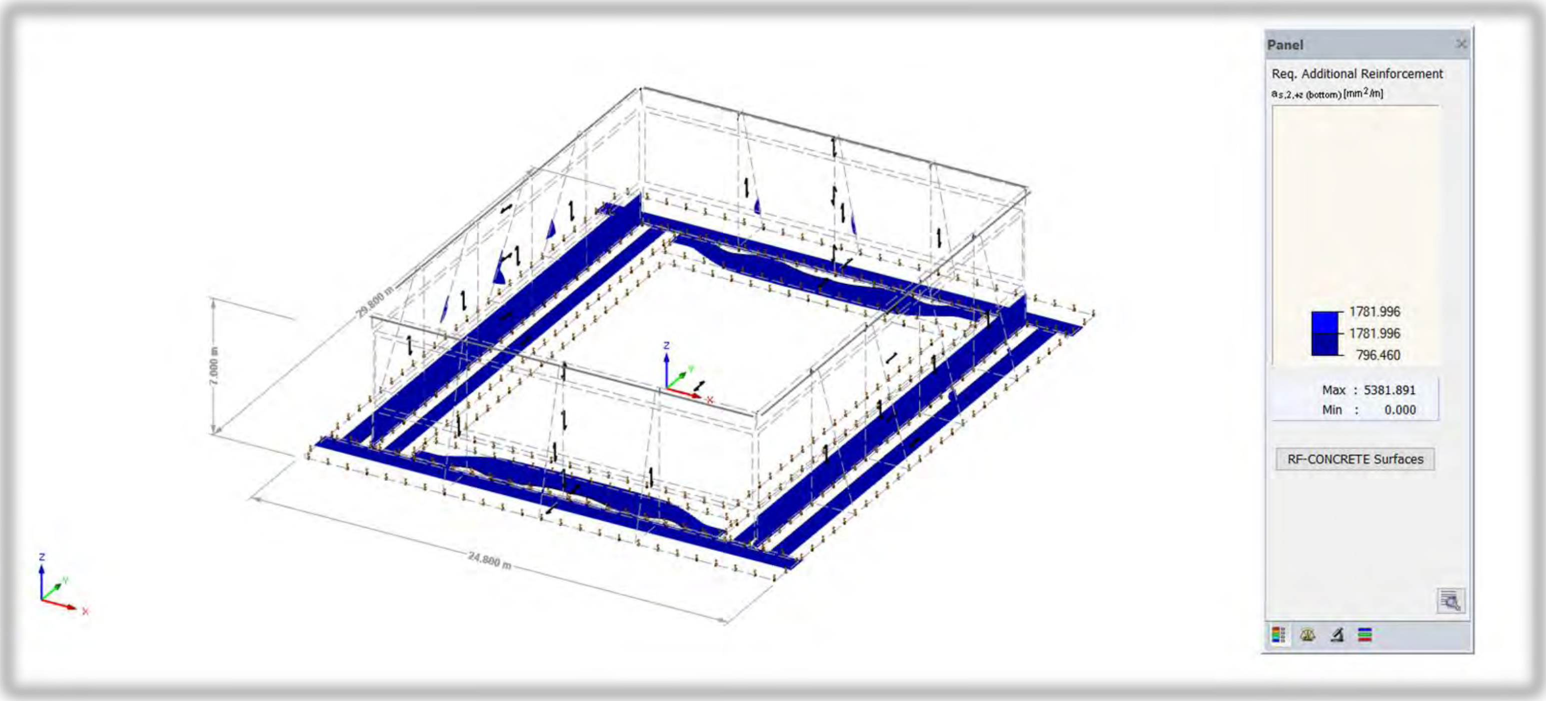
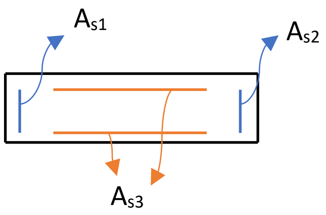


Figure 8.26 Required Additional Reinforcement Areas (Internal & Bottom).

Note That: For economic purposes, some of assumed reinforcement areas are reduced as shown in section 2.

8.2.2 Design of Horizontal Beams.

8.2.2.1 Design of As1.



- The Reinforcement A_{s1} should resist The horizontal bending moment (Mz) and the Normal force (N) generated from Testing case as shown in Fig. 8.27 and Fig. 8.28.

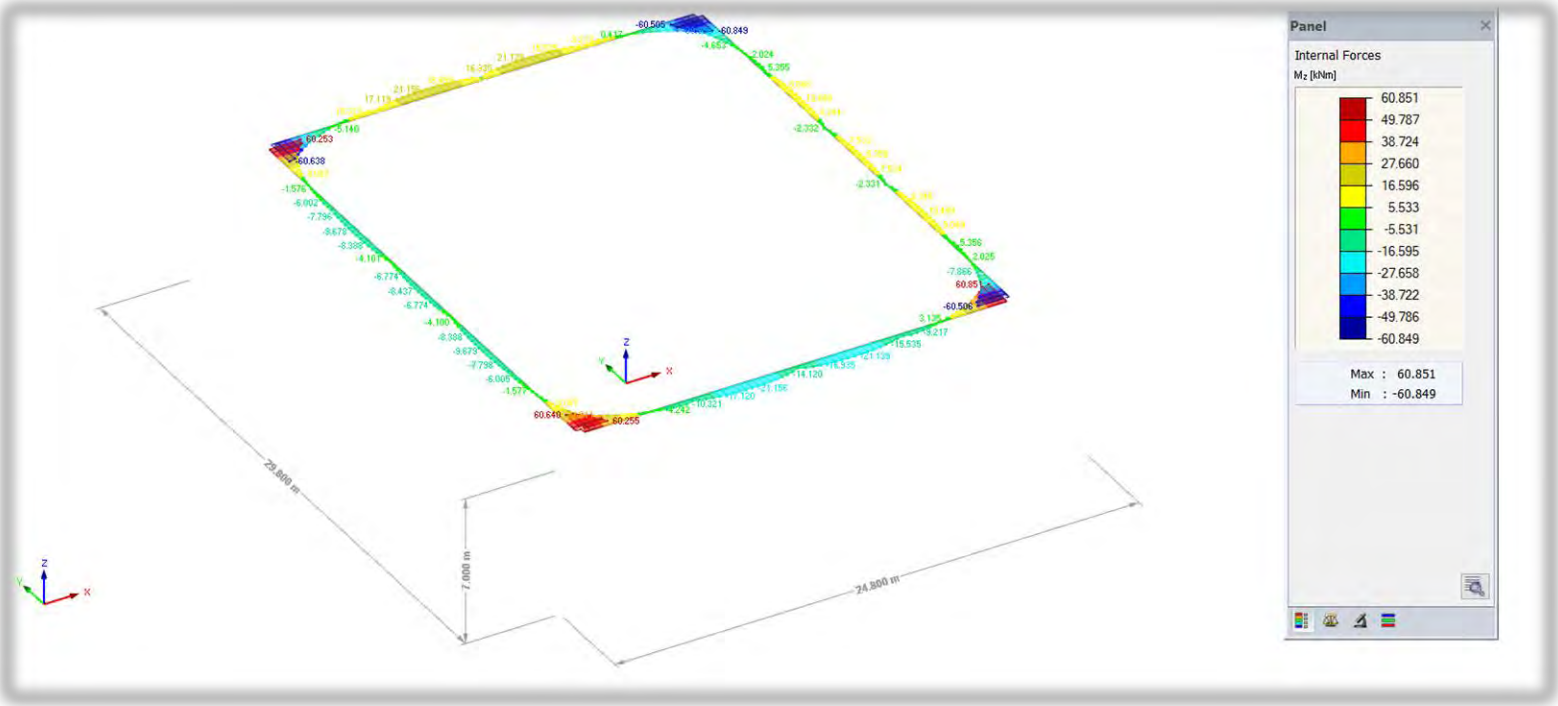


Figure 8.27: Bending Moment in z-direction.

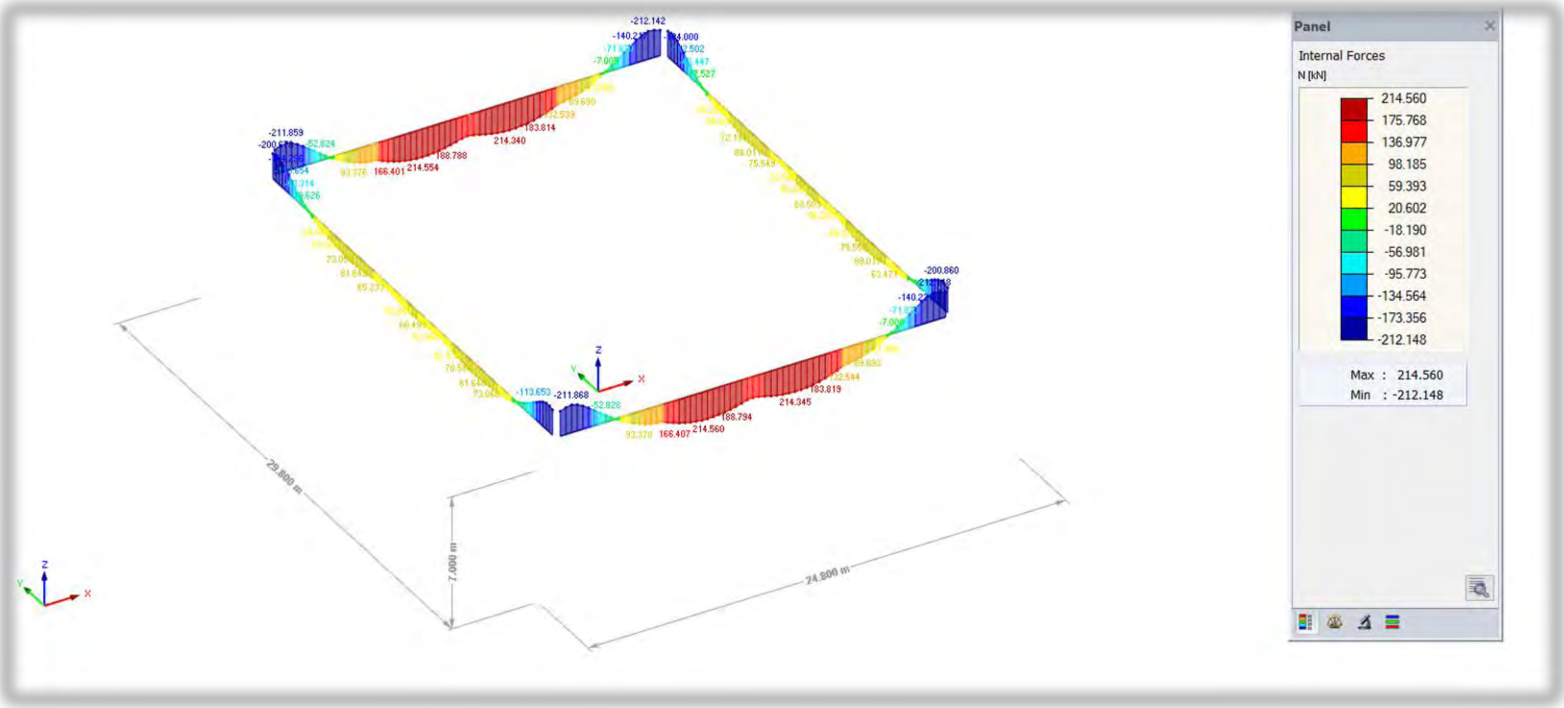
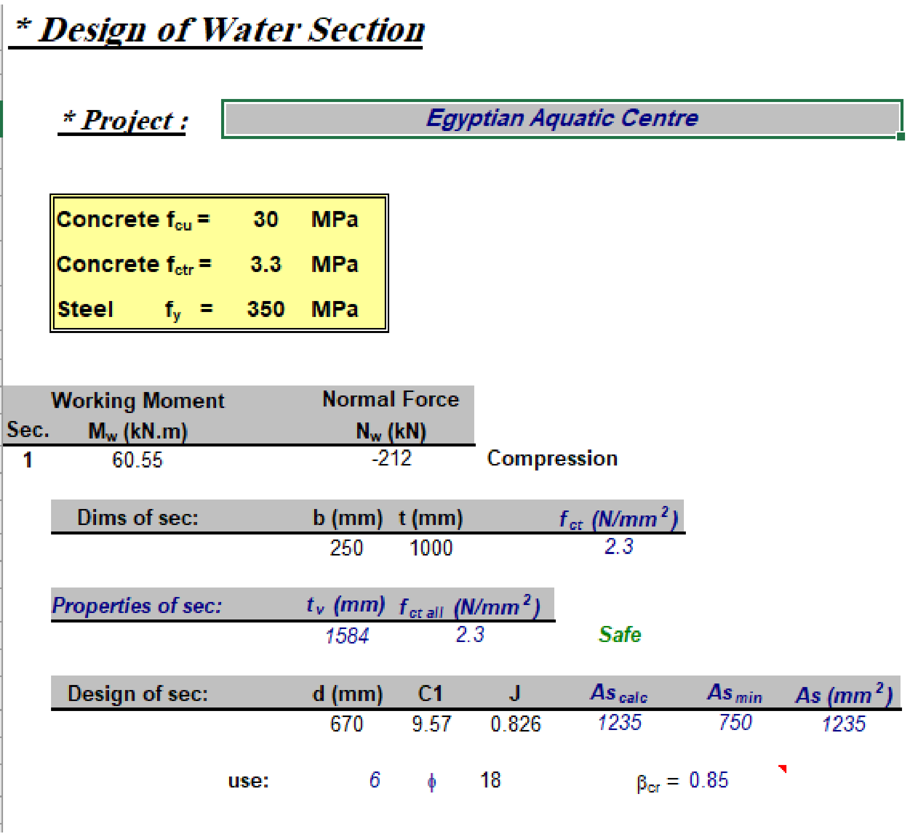


Figure 8.27: Normal Force.



* *Design of Water Section*

**** Project :*** ***Egyptian Aquatic Centre***

Concrete f_{cu} =	**30**	**MPa**
Concrete f_{ctr} =	**3.3**	**MPa**
Steel f_y =	**350**	**MPa**

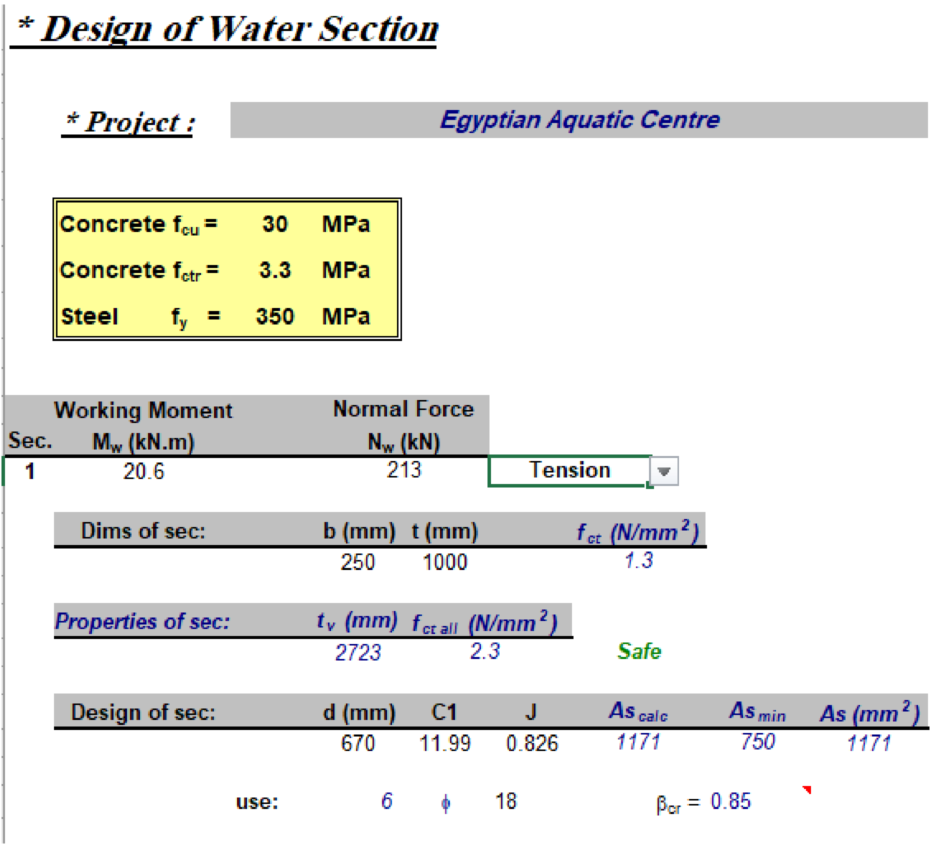
Sec.	Working Moment M_w (kN.m)	Normal Force N_w (kN)	
1	60.55	-212	**Compression**

Dims of sec:	b (mm)	t (mm)	f_{ct} (N/mm^2)
	250	1000	2.3

Properties of sec:	t_v (mm)	$f_{ct\ all}$ (N/mm^2)	
	1584	2.3	*Safe*

Design of sec:	d (mm)	C1	J	As_{calc}	As_{min}	As (mm^2)
	670	9.57	0.826	1235	750	1235

use: 6 ϕ 18 β_{cr} = 0.85



*** Design of Water Section**

*** Project :** Egyptian Aquatic Centre

Concrete f_{cu} = 30 MPa

Concrete f_{ctr} = 3.3 MPa

Steel f_y = 350 MPa

Sec.	Working Moment M_w (kN.m)	Normal Force N_w (kN)	
1	20.6	213	Tension

Dims of sec:	b (mm)	t (mm)	f_{ct} (N/mm^2)
	250	1000	1.3

Properties of sec:	t_v (mm)	$f_{ct\ all}$ (N/mm^2)	
	2723	2.3	Safe

Design of sec:	d (mm)	C1	J	As_{calc}	As_{min}	As (mm^2)
	670	11.99	0.826	1171	750	1171

use: 6 ϕ 18 β_{cr} = 0.85

8.2.2.2 Design of A_{s2}

- The Reinforcement A_{s1} should resist The horizontal bending moment (Mz) and the Normal force (N) generated from Maintenance case as shown in Fig. 8.29 and Fig. 8.30.

.

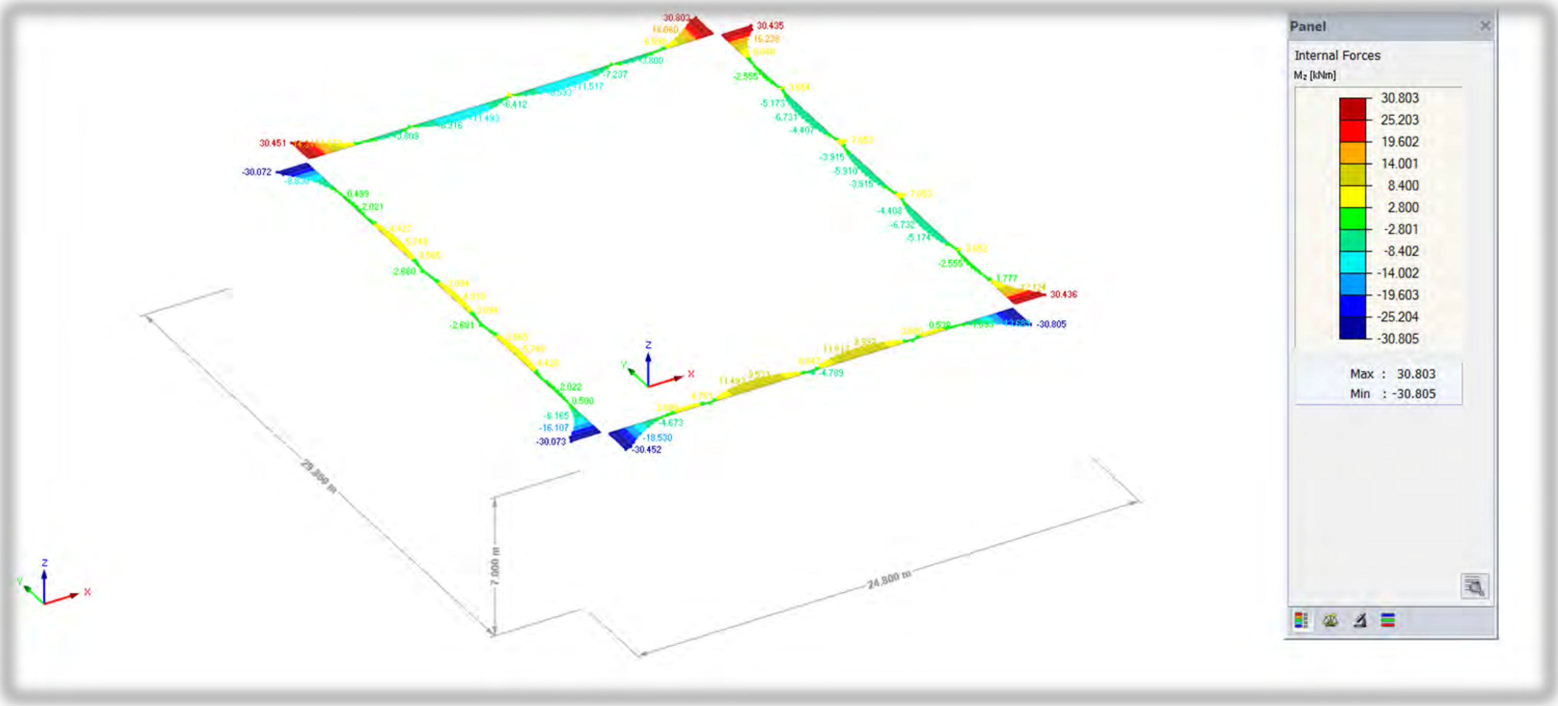


Figure 8.29: Bending Moment in z-direction (KN.m)

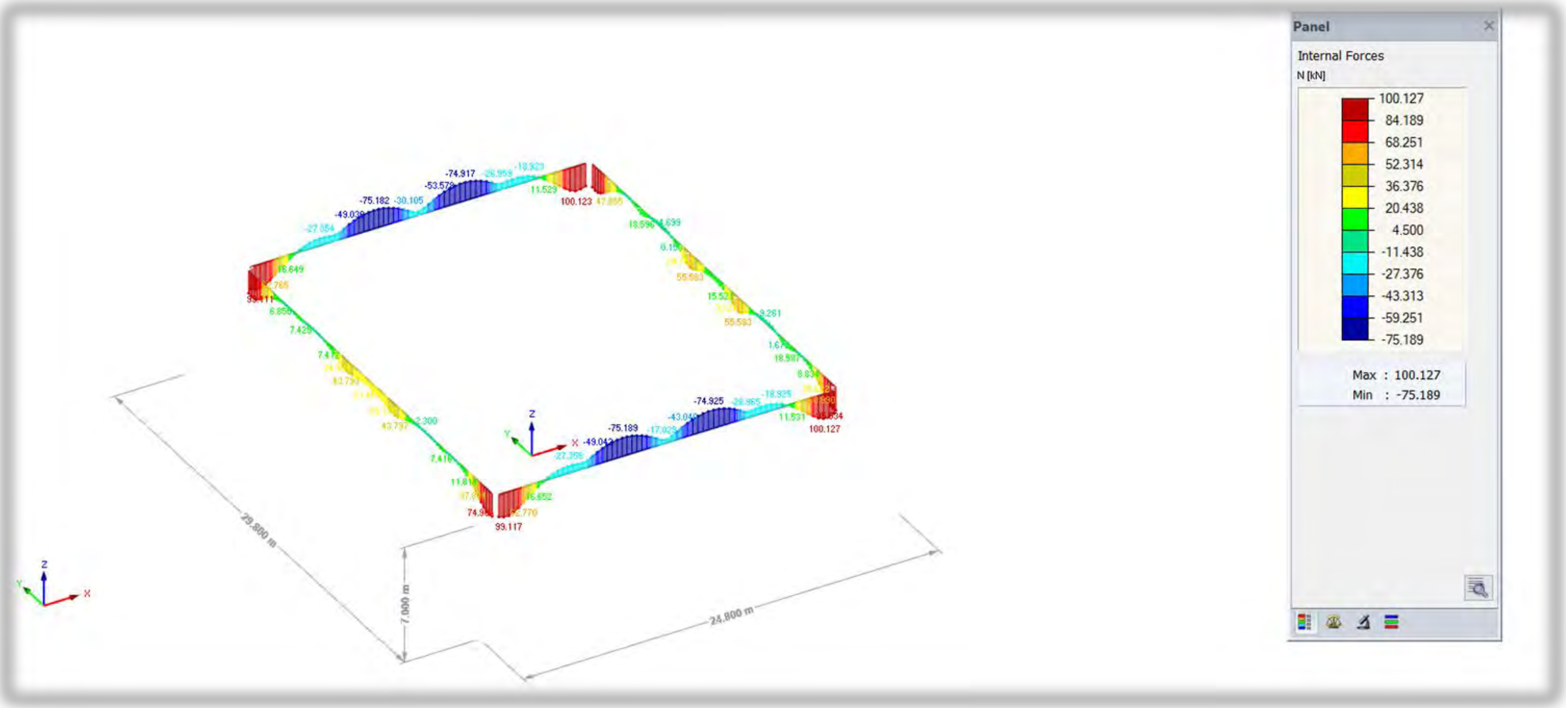
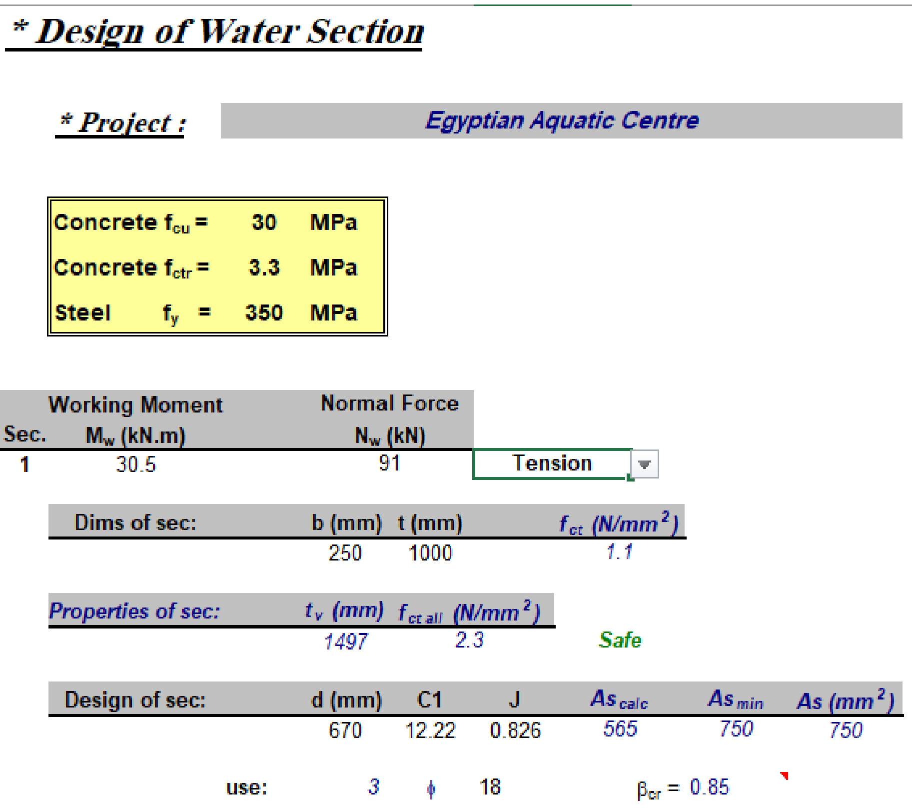


Figure 8.30: Normal Force (KN)



* Design of Water Section

*** Project :** Egyptian Aquatic Centre

Concrete f_{cu} =	30	MPa
Concrete f_{ctr} =	3.3	MPa
Steel f_y =	350	MPa

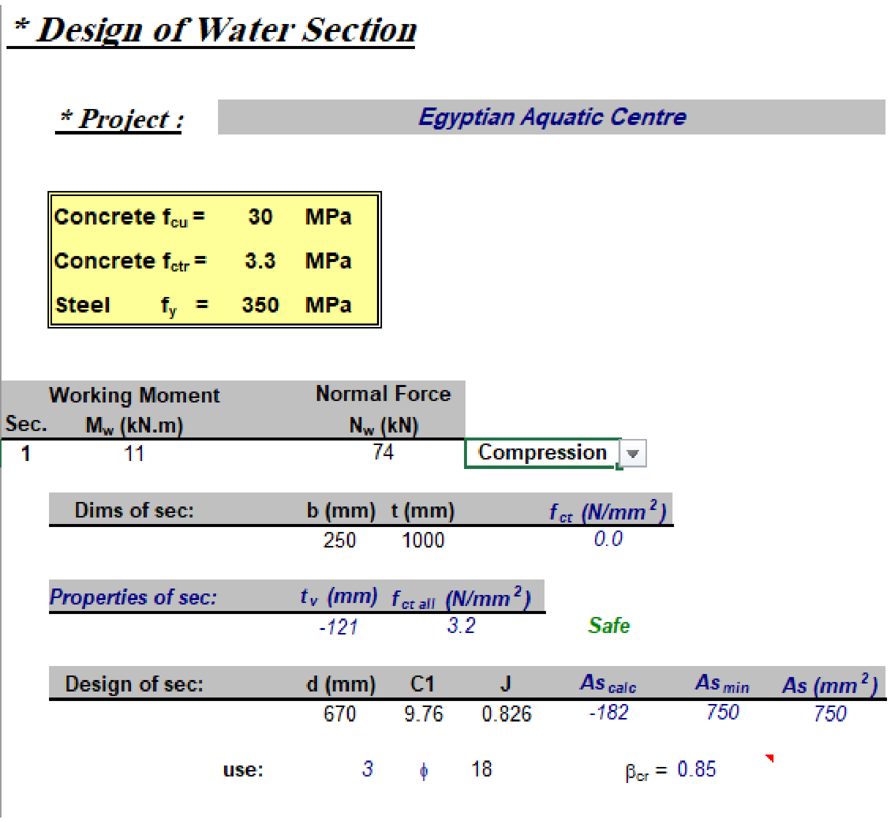
Sec.	Working Moment M_w (kN.m)	Normal Force N_w (kN)	
1	30.5	91	Tension

Dims of sec:	b (mm)	t (mm)	f_{ct} (N/mm^2)
	250	1000	1.1

Properties of sec:	t_v (mm)	$f_{ct\ all}$ (N/mm^2)	
	1497	2.3	Safe

Design of sec:	d (mm)	C1	J	As_{calc}	As_{min}	As (mm^2)
	670	12.22	0.826	565	750	750

use: 3 ϕ 18 β_{cr} = 0.85



* Design of Water Section

*** Project :** *Egyptian Aquatic Centre*

Concrete f_{cu} =	30	MPa
Concrete f_{ctr} =	3.3	MPa
Steel f_y =	350	MPa

Sec.	Working Moment M_w (kN.m)	Normal Force N_w (kN)	
1	11	74	Compression ▾

Dims of sec:	b (mm)	t (mm)	f_{ct} (N/mm^2)
	250	1000	0.0

Properties of sec:	t_v (mm)	$f_{ct\ all}$ (N/mm^2)	
	-121	3.2	Safe

Design of sec:	d (mm)	C1	J	As_{calc}	As_{min}	As (mm^2)
	670	9.76	0.826	-182	750	750

use: 3 ϕ 18 β_{cr} = 0.85

8.2.2.3 Design of A_{s3} & Stirrups.

- The Reinforcement A_{s3} and the stirrups should resist The Torsional moment (M_T) and the Shear forces (V_y & V_z) generated from Maintenance case as shown in Fig. 8.31, Fig. 8.32. and Fig. 8.33.

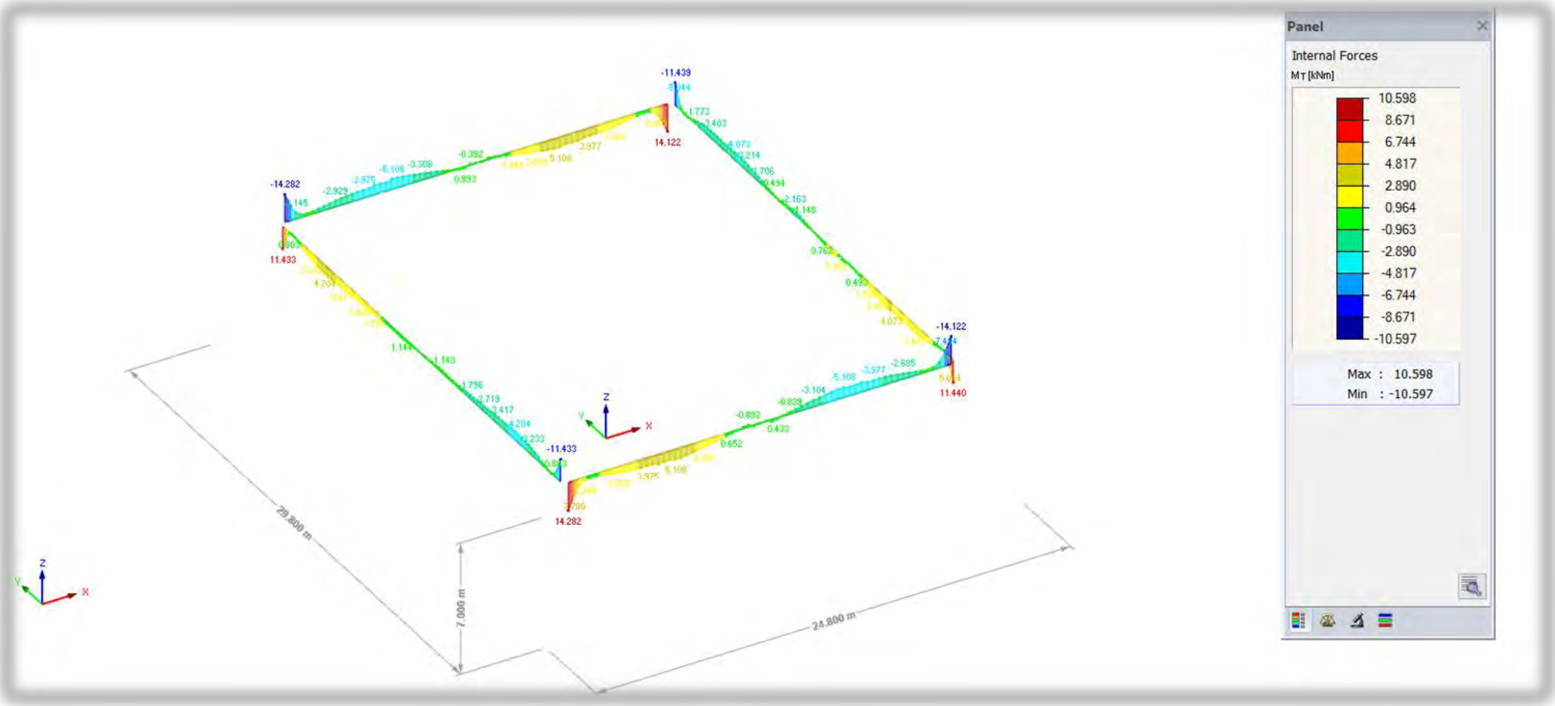


Figure 8.31: Torsional Moment (KN.m).

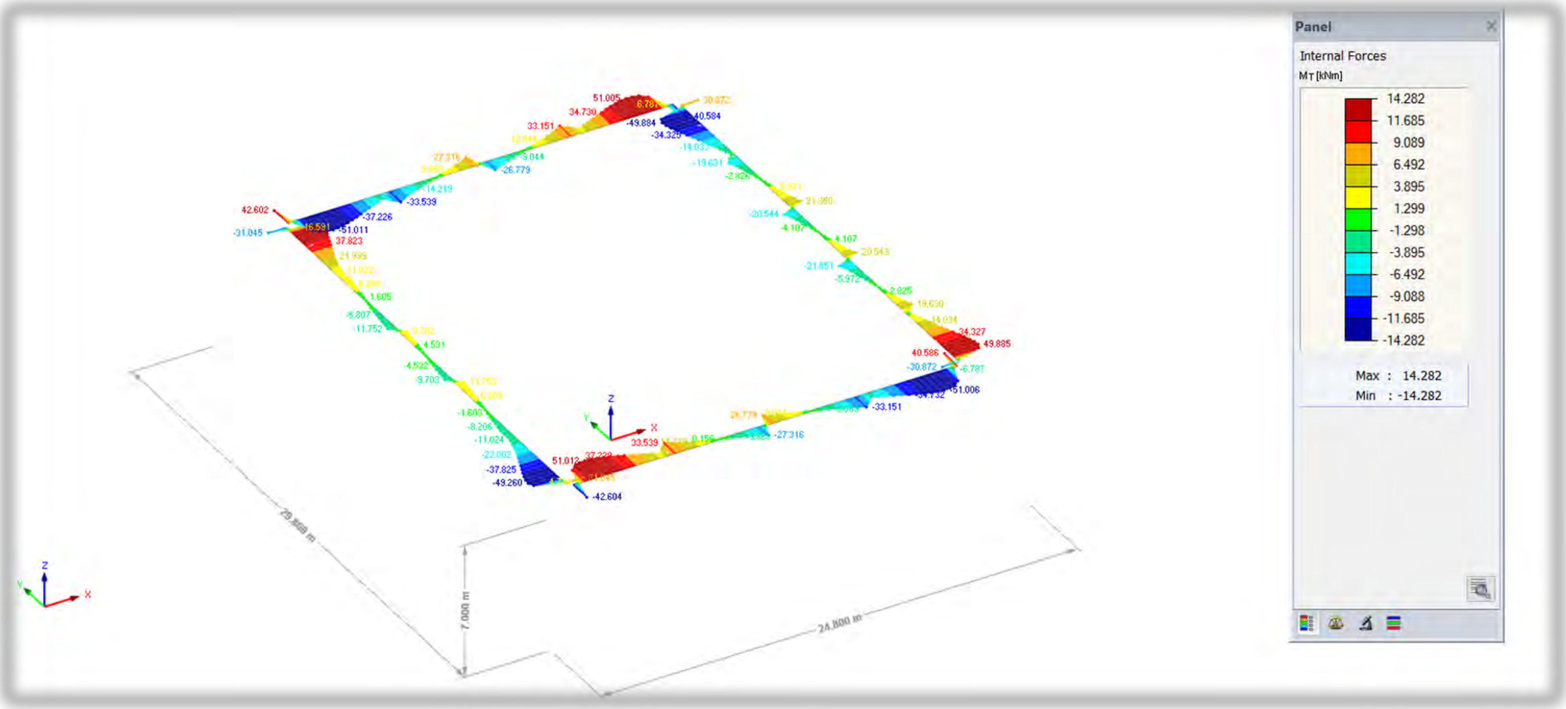


Figure 8.32: Shear Forces in y-direction (KN).

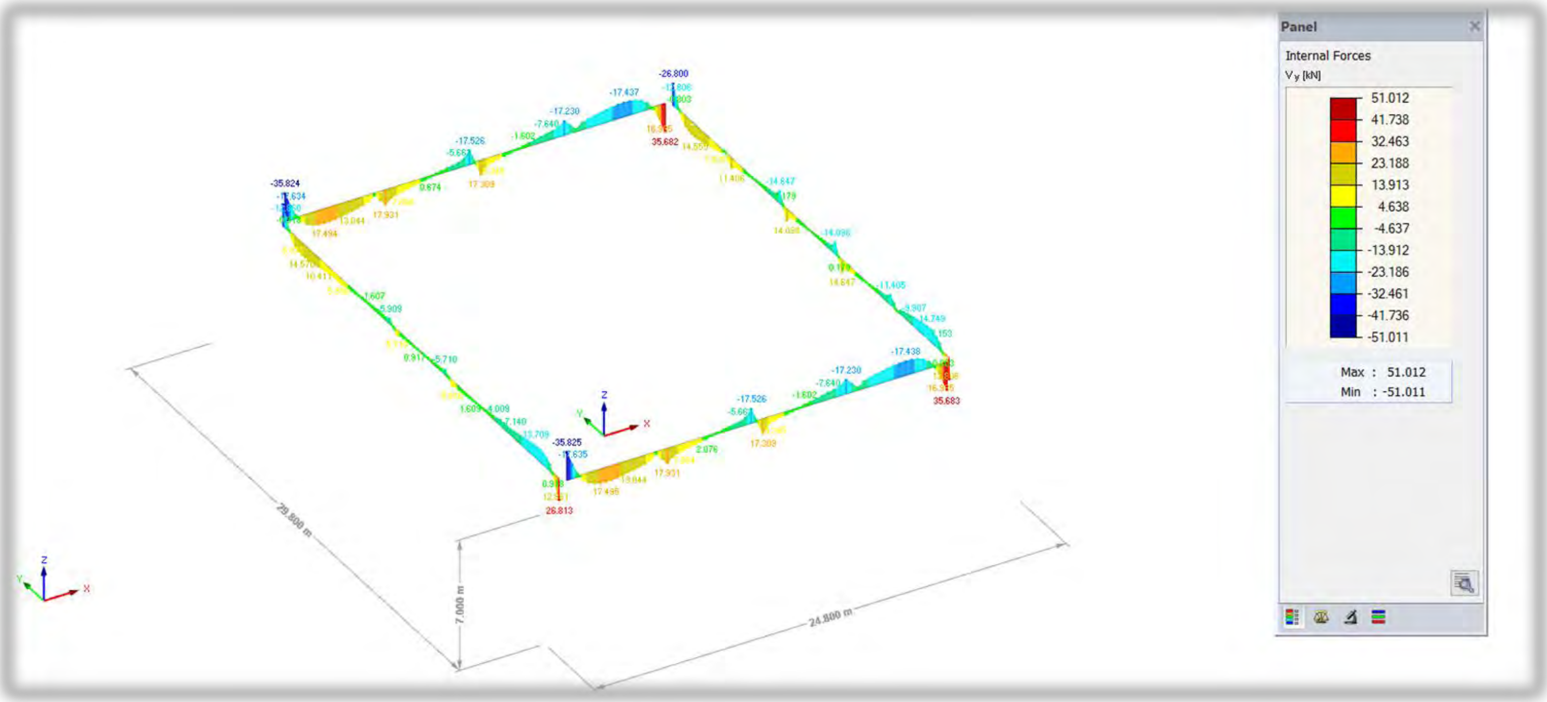
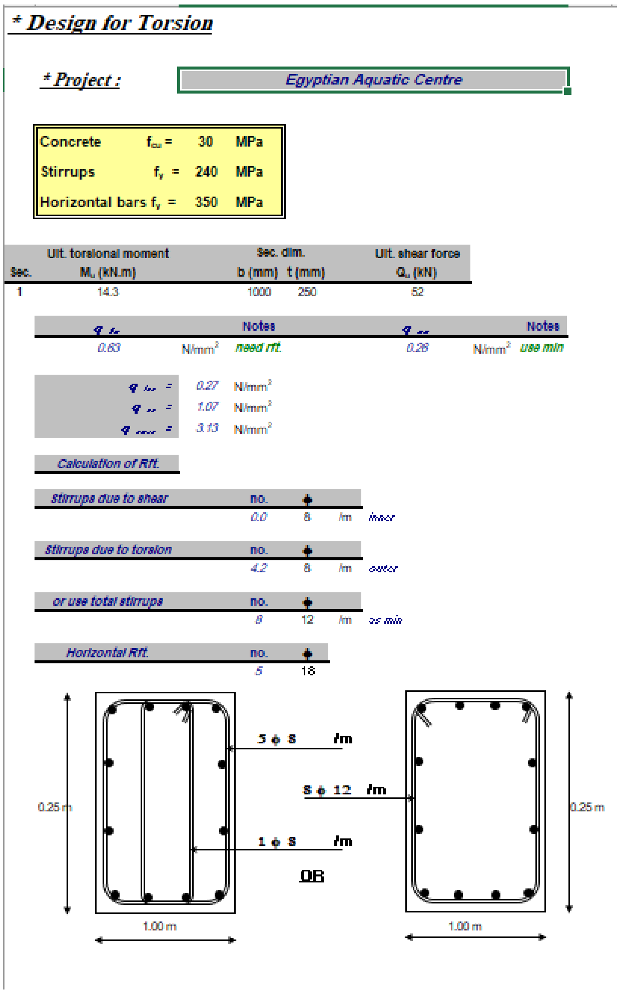


Figure 8.33: Shear Forces in z-direction (KN)



* Design for Torsion

*** Project :*** *Egyptian Aquatic Centre*

Concrete	f_{cu} =	30	MPa
Stirrups	f_y =	240	MPa
Horizontal bars	f_y =	350	MPa

Sec.	Ult. torsional moment M_u (kN.m)	Sec. dim. b (mm)	t (mm)	Ult. shear force Q_u (kN)
1	14.3	1000	250	52

q		Notes	q		Notes
0.63	N/mm^2	need rft.	0.26	N/mm^2	use min

q =	0.27	N/mm^2
q =	1.07	N/mm^2
q =	3.13	N/mm^2

Calculation of Rft.

Stirrups due to shear	no.	ϕ		
	0.0	8	/m	inner

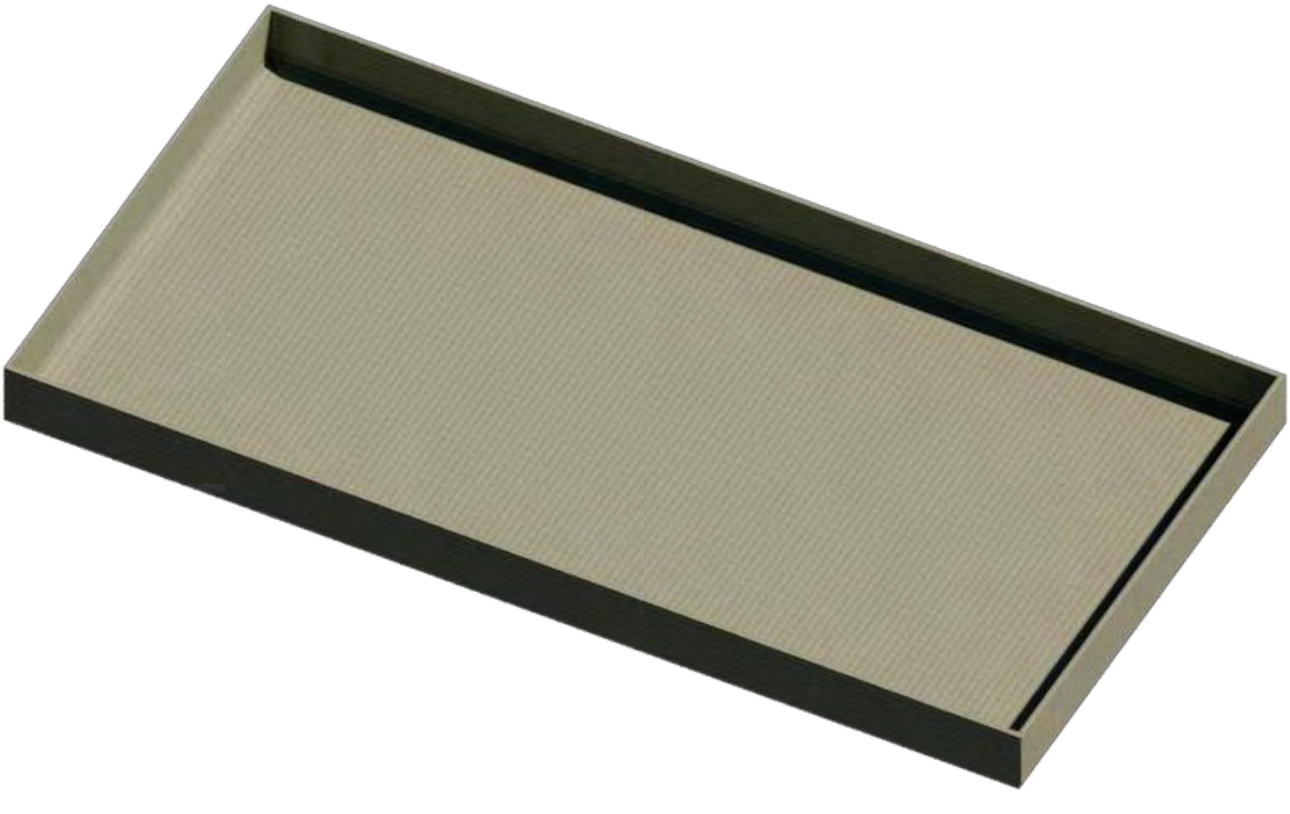
Stirrups due to torsion	no.	ϕ		
	4.2	8	/m	outer

or use total stirrups	no.	ϕ		
	8	12	/m	as min

Horizontal Rft.	no.	ϕ
	5	18

Egyptian Aquatic Centre

Long Course Pool



I. Design codes and standards

5- ECP (203–2018)

Egyptian Code of Practice for Design and Construction of Concrete Structures.

6- ECP (201-2010)

Egyptian Code for Loading on Buildings.

7- FINA FACILITIES RULES (2017-2021)

International Swimming Federation.

1. INTRODUCTION

The concept for structural design focuses on satisfying both the functional and the economic requirements of the building without jeopardizing its aesthetic and architectural features. This Calculation Report presents the structural engineering aspect of the works due for the development construction work of Egyptain Aquatic Centre.

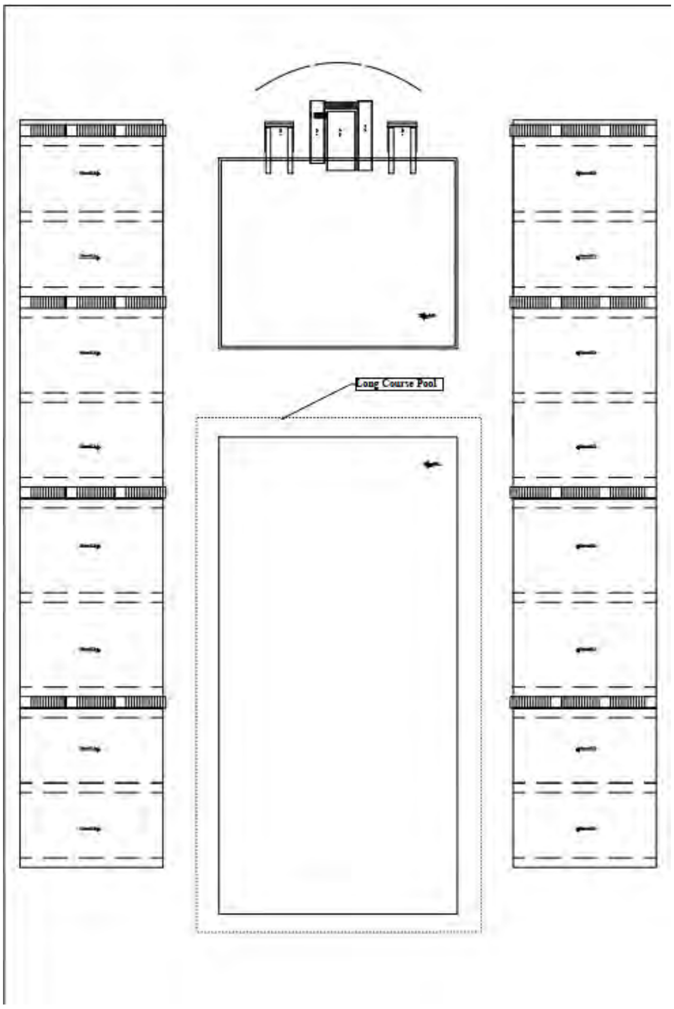


Figure 1-2 Long Course Pool location

2. Pool Drawings Details

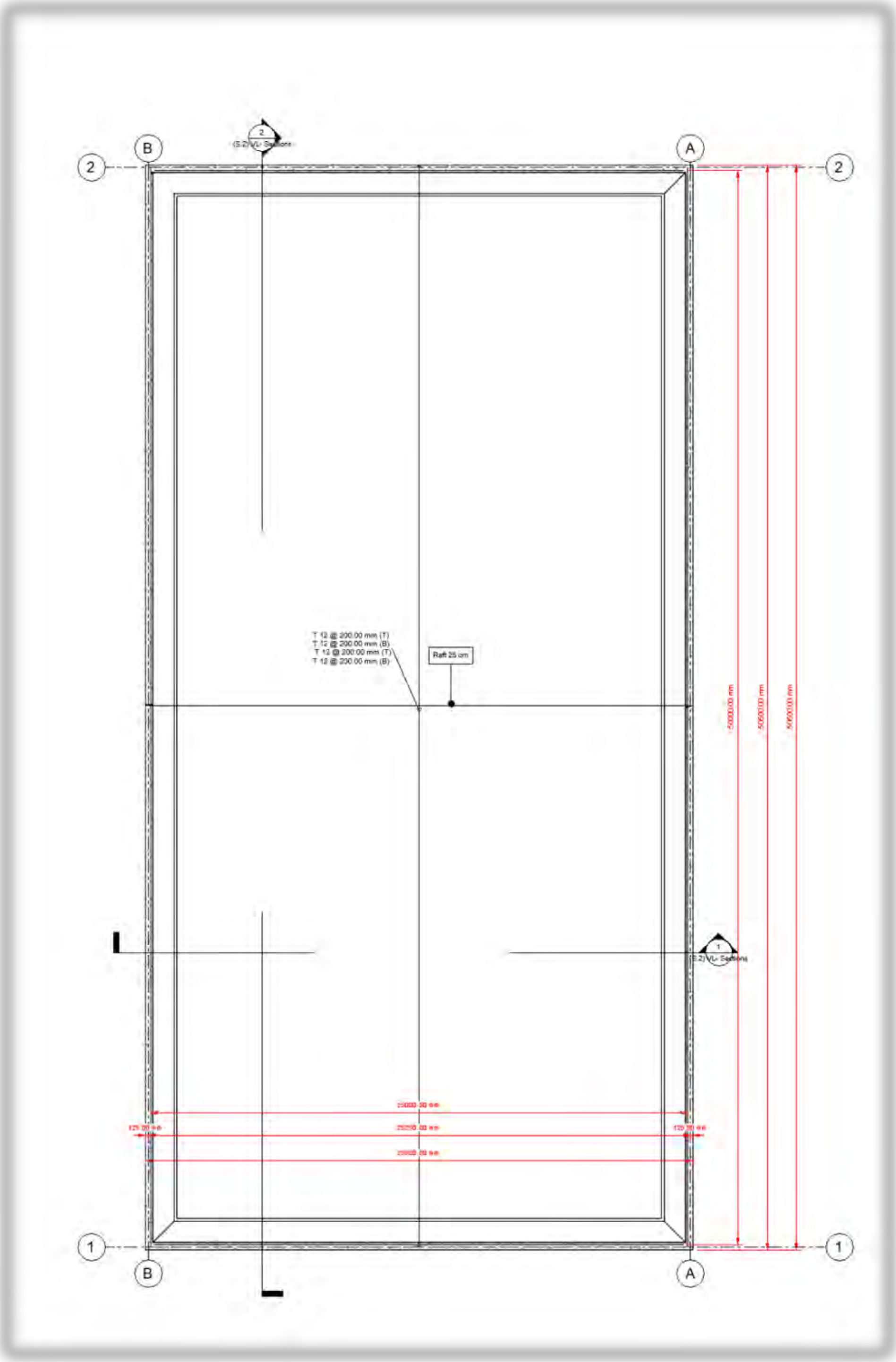


Figure 2-1 Pool Plan

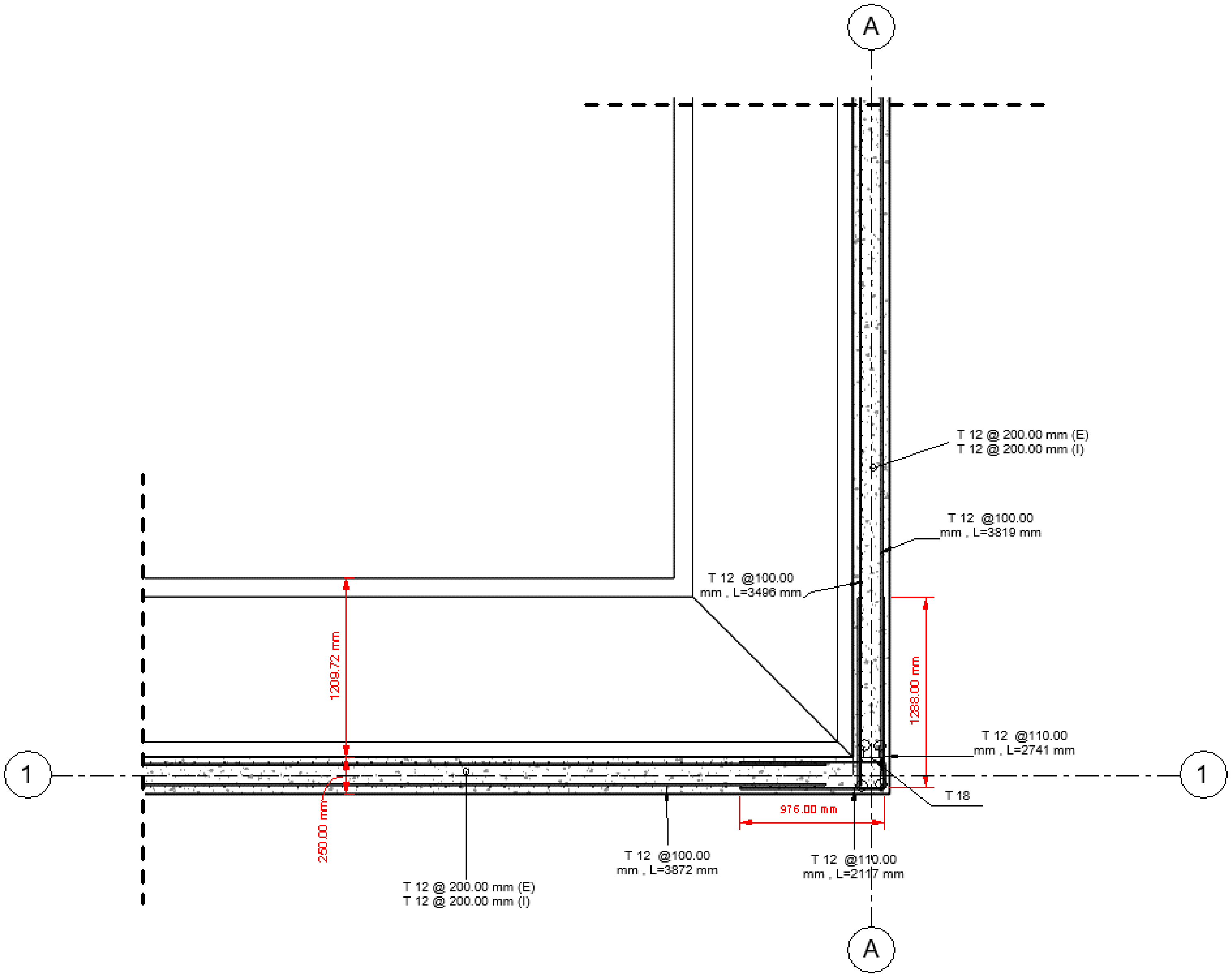


Figure 2-2 HZ Section (Detail)

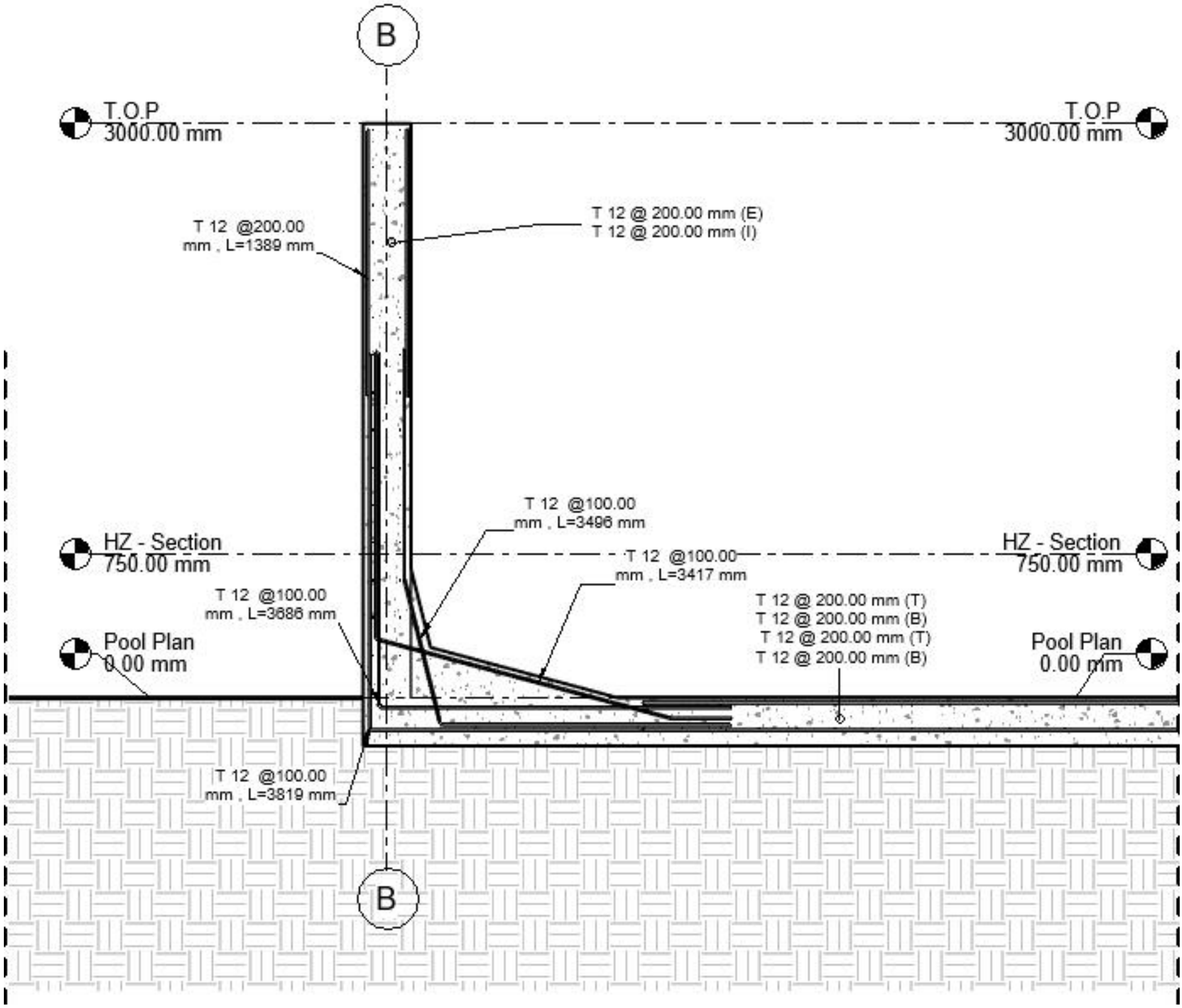


Figure 2-3 VL Sections (Detail)

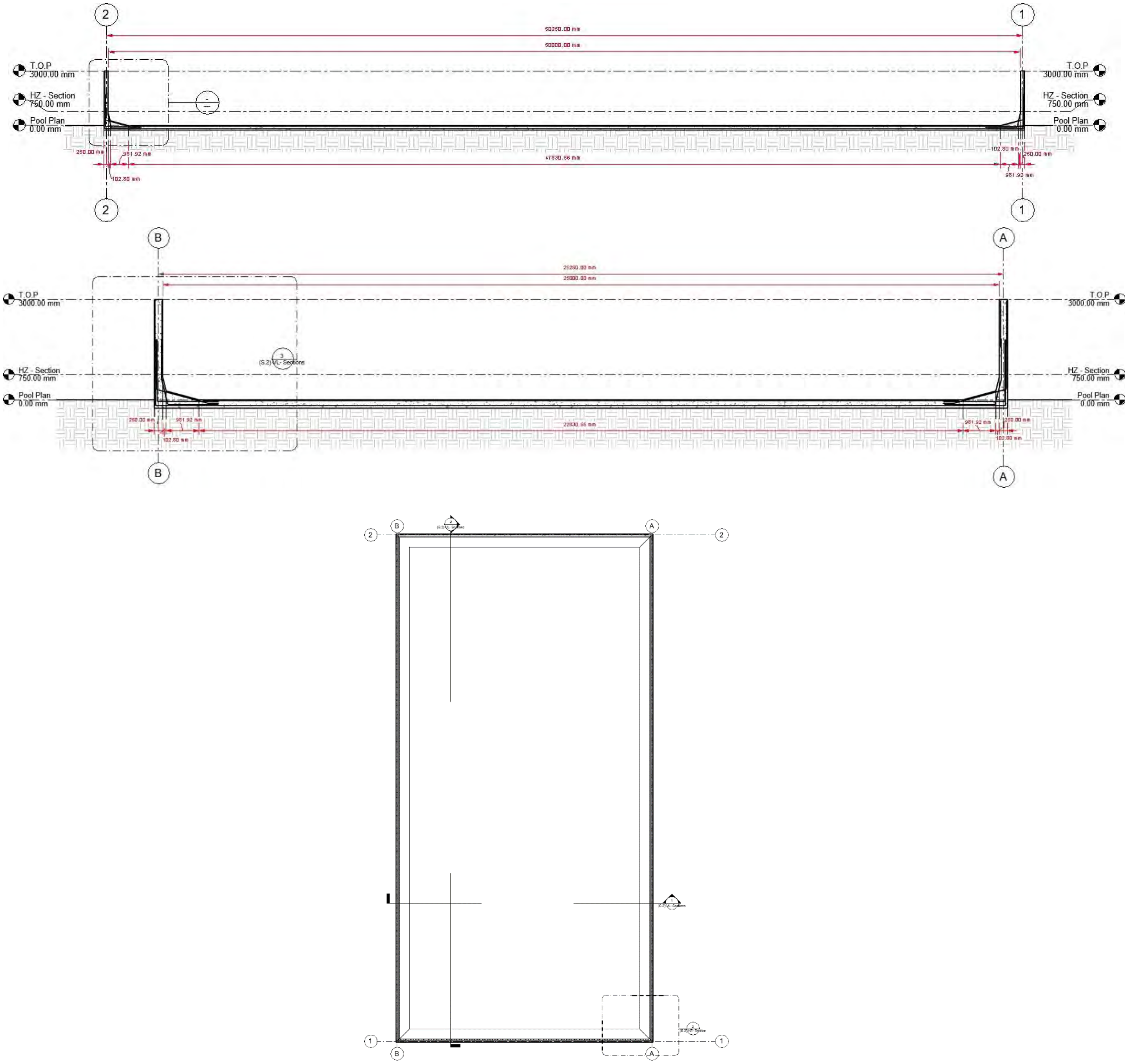


Figure 2- 4 Pool's Sections

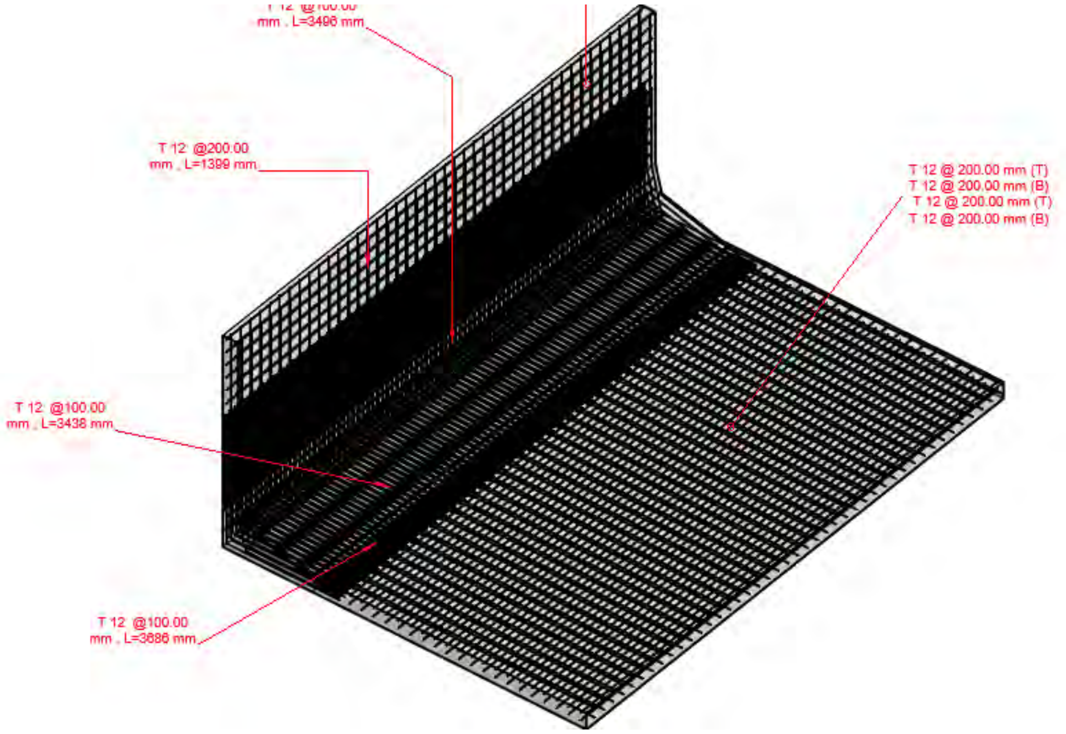


Figure 2- 5 3d wall to slab joint RFT

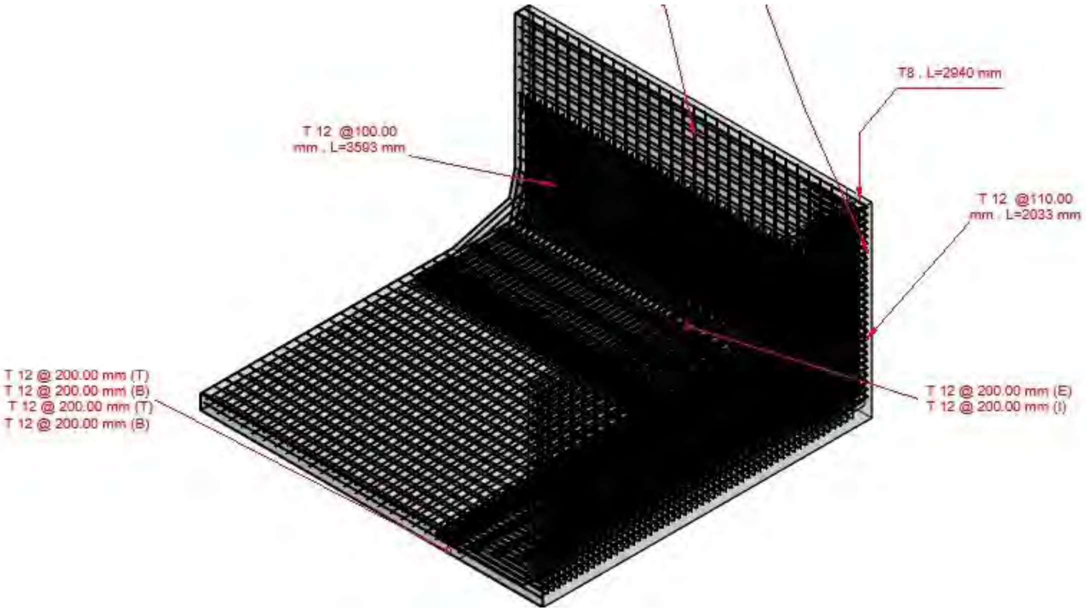


Figure 2- 6 3d wall to slab joint RFT

3. Calculation Software Used

Calculation software features

The software used is *SAP2000*, developed by Computer & Structures (United States).

Technical specifications

Name: Sap2000

Version: 14.2.2

Producer:

Computer&Structures

https://www.csiamerica.com/

4. OUTLINE SPECIFICATION AND MATERIAL PROPERITIES

REINFORCED CONCRETE.

The grade of concrete will be according to the Egyptian Code of Practice (ECP). The grade of concrete is indicated in two numbers, the first one indicate the characteristic cube strength in (N/mm²) while the second one indicates the maximum nominal size of the aggregate in (mm) to be used;

Grade (20/20)	for plain concrete of foundations of thickness < 12 cm.
Grade (20/40)	for plain concrete of foundations of thickness >12 cm
Grade (30/20)	for all pool reinforced concrete elements.

Minimum thickness of blinding concrete is 100 mm.

Concrete cover is the concrete thickness to all steel reinforcement including links:

For all concrete (with protection) in contact with soil, cover shall be 70mm (or as will be recommended in the geotechnical report)

For all concrete elements above grade where concrete is protected from weathering, cover shall be 50mm for beams and 25 mm for slabs and walls.

- **SLUMP VALUES.**

The following values are according to the Egyptian code of practice ECP 203-2018 section (2-3-1-2), Table (2-5).

Type of Structural Element	Type of Compaction	Slump-in mm (max.)
Massive concrete	Mechanical	25 - 50
- Concrete foundation. - Concrete sections with low reinforcement ratio (< 80 kg/m³)	Mechanical	50 - 75
Concrete sections with medium and high reinforcement ratio (80-150 kg/m³)	Mechanical/ Manual	75 – 125
Concrete sections with very high reinforcement ratio (> 150 kg/m³)	Light compaction	125 – 150**
Deep foundation	Light compaction	125 – 200**

** By using chemical additives.

REINFORCING STEEL

All reinforcing steel shall be complying with the Egyptian code of practice ECP203-2018, section 2-2-5-3, Table 2-4.

Reinforcing steel bars shall be uncoated high yield deformed bars of characteristic strength 360 N/mm².

Uncoated mild steel plain bars with characteristic strength 280 N/mm² may be used for links and binders.

Type	Grade	Yield Strength, f_y (N/mm²)
Normal mild steel	280/450	280
High grade steel	360/520	360
Cold formed welded mesh	450/520	450

Note: Bar size increment = 6, 8, 10, 12, 16, 18, 20, 22, 25, 28 and 32

5. Calculation method and numerical model

5.1 Model Description

5.1.1 Hypothesis adopted for the elements

- Based on the pool dimension to it’s height ratio, the pool considered as rested on elastic foundation; thus we can assume nonuniformity in stress distribution under it.

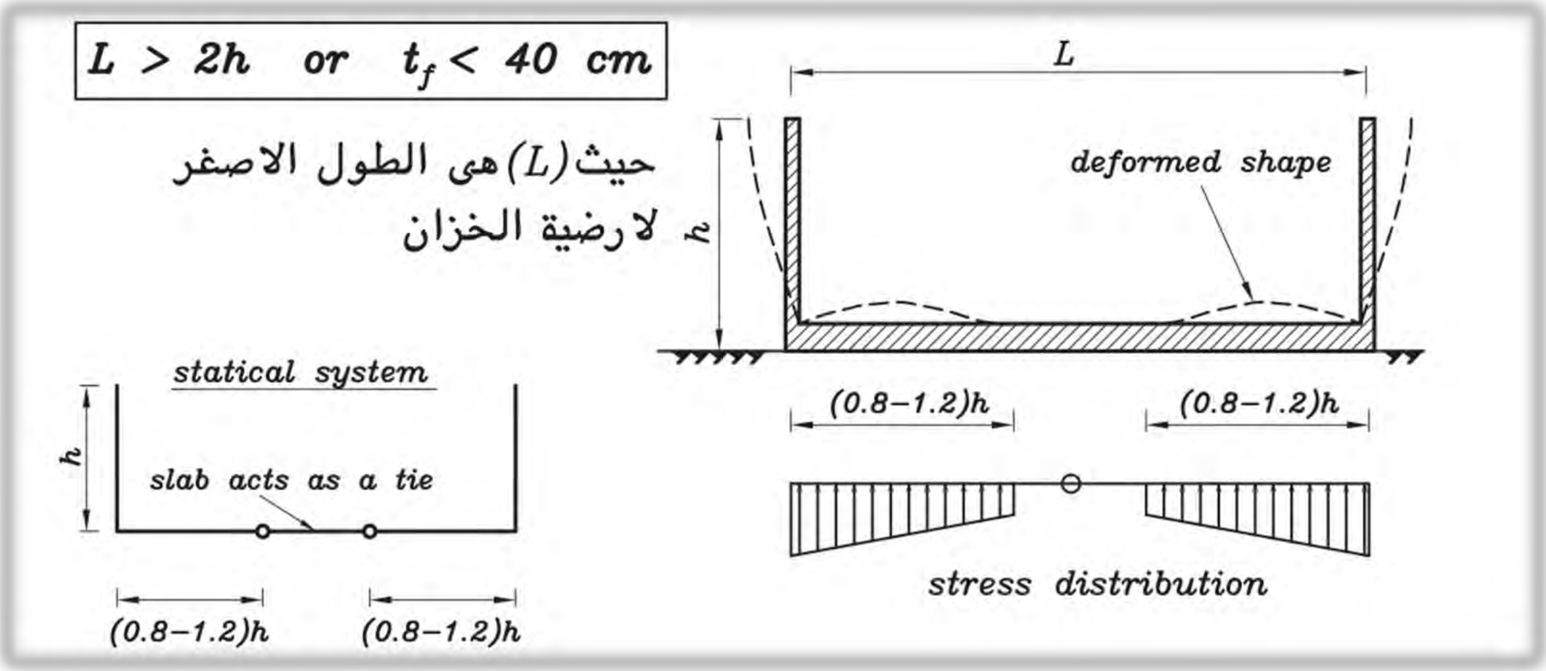
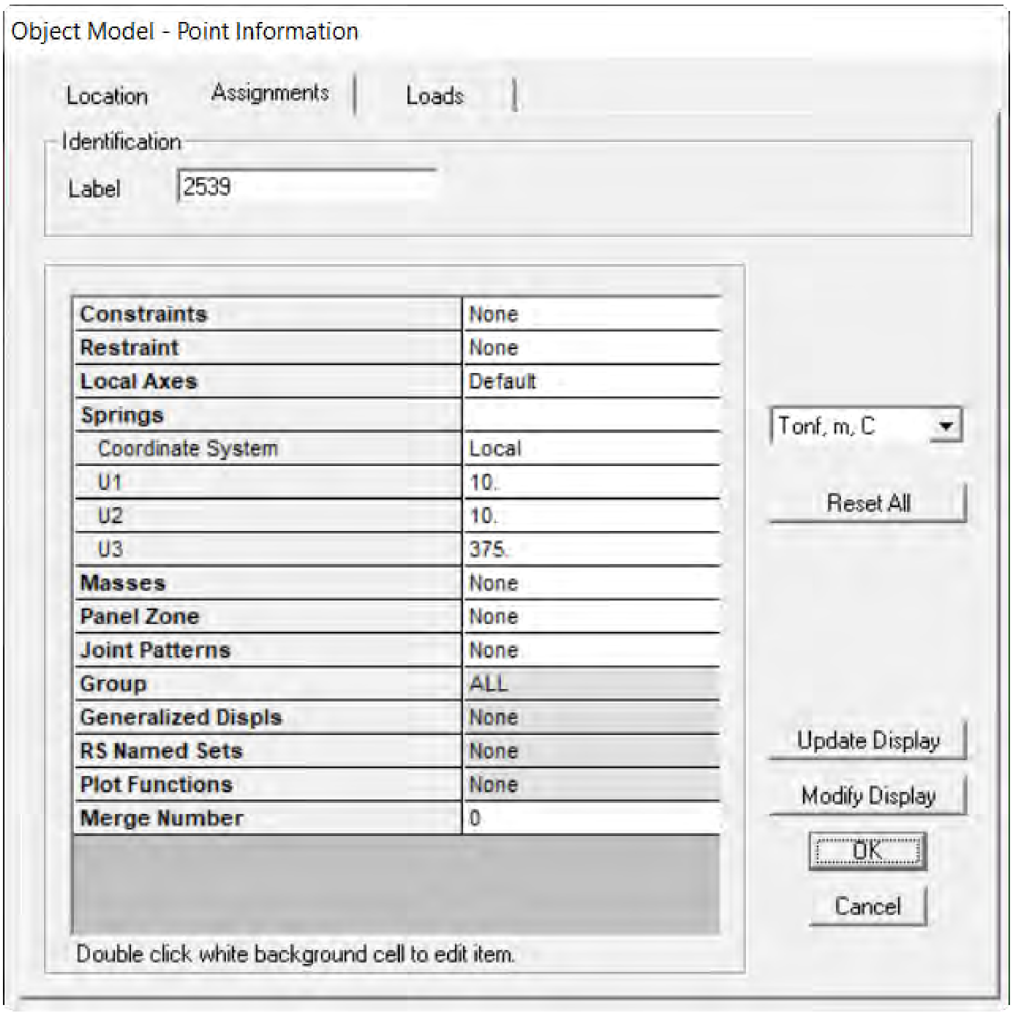


Figure 5.1 stress distribution under pool

- For simplicity, the soil is modeled as springs following this equation:
  Soil bearing capacity= 1cm * K$_{spring}$


Object Model - Point Information

Identification

Label 2539

Constraints	None
Restraint	None
Local Axes	Default
Springs	
Coordinate System	Local
U1	10.
U2	10.
U3	375.
Masses	None
Panel Zone	None
Joint Patterns	None
Group	ALL
Generalized Displs	None
RS Named Sets	None
Plot Functions	None
Merge Number	0

Tonf, m, C

Double click white background cell to edit item.

6. Actions and design loads

6.1 STRUCTURAL LOADS.

The following loads are considered in the design:

Structural Dead Loads which include:

The own weight of the structural elements, beams, raft and walls.

Superimposed dead load from water and soil weights.

Live loads which cover the weight and movement of equipment and people on the sides of the pool (surcharge).

The basis for the considered design loads are summarized in the followings sections.

A. Dead Loads

Unit weight of concrete elements	25.0	kN/m³

B. Live Loads

Live loads are considered equal to 30 kN/m^2 effect on the sides of the pool as a surcharge.

C. Earthquakes

from best Practices, the earthquake effect on pool is taken as increase by 15% of load combinations factors.

The following tables describe the load cases and load combinations on the pool:

Table 1: Load cases

Case	Type	Self. Wt. Mult.	Design Type
DEAD	LinStatic	1.000000	DEAD
MODAL	LinModal	0.000000	OTHER
L.L	LinStatic	0.000000	OTHER
F.C	LinStatic	0.000000	DEAD
W.P	LinStatic	0.000000	OTHER
E.P	LinStatic	0.000000	OTHER
Water weight	LinStatic	0.000000	OTHER
W.P (soil)	LinStatic	0.000000	OTHER

Table 2: Load combinations

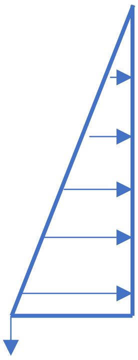
Table 2: Combination Definitions

Combo. Name	Load Case	Combo. Type	Auto Design	Case Type	CaseName	Scale Factor	Steel Design
Testing Case (working)	W.P	Linear Add	No	Linear Static	DEAD	1.150000	None
Testing Case (working)	water weight			Linear Static	F.C	1.150000	

Testing Case (working)	DEAD			Linear Static	W.P	1.150000	
Testing Case (working)	F.C			Linear Static	water weight	1.150000	
Mainteinance Case (working)	E.P	Linear Add	No	Linear Static	DEAD	1.150000	None
Mainteinance Case (working)	W.P (soil)			Linear Static	F.C	1.150000	
Mainteinance Case (working)	DEAD			Linear Static	E.P	1.150000	
Mainteinance Case (working)	F.C			Linear Static	W.P (soil)	1.150000	
Testing case (ULT.)	W.P	Linear Add	No	Linear Static	DEAD	1.550000	None
Testing case (ULT.)	water weight			Linear Static	F.C	1.550000	
Testing case (ULT.)	DEAD			Linear Static	W.P	1.550000	
Testing case (ULT.)	F.C			Linear Static	water weight	1.550000	
Meintenance Case (ULT.)	E.P	Linear Add	No	Linear Static	DEAD	1.750000	None
Meintenance Case (ULT.)	W.P (soil)			Linear Static	F.C	1.750000	
Meintenance Case (ULT.)	DEAD			Linear Static	E.P	1.750000	
Meintenance Case (ULT.)	F.C			Linear Static	W.P (soil)	1.750000	

6.2 Calculations of water and earth pressure.

6.2.1 water pressure.

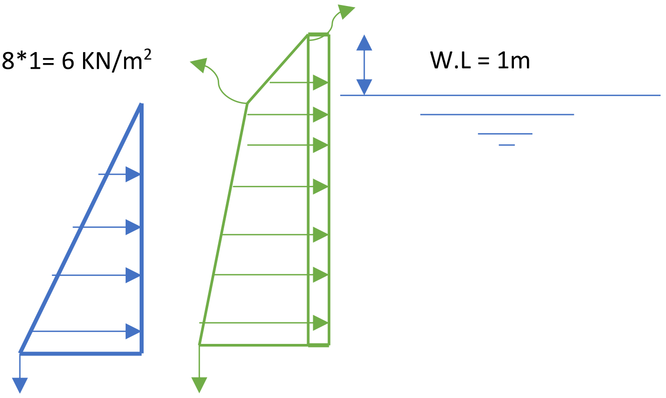


γ_{water}= 10 KN/m^3

Height= 3m

W.P= 10*3= 30 KN/m^2

6.2.2 Earth pressure.



E.P$_1$= 1/3*30= 10 KN/m^2

E.P$_2$= 1/3*18*1= 6 KN/m^2

W.P= 10*2= 20 KN/m^2

E.P$_3$= 1/3*8*2+6= 11.3 KN/m^2

γ_{water}= 10 KN/m^3

γ_{soil}= 18 KN/m^3

Ka= 1/3

Height= 3m

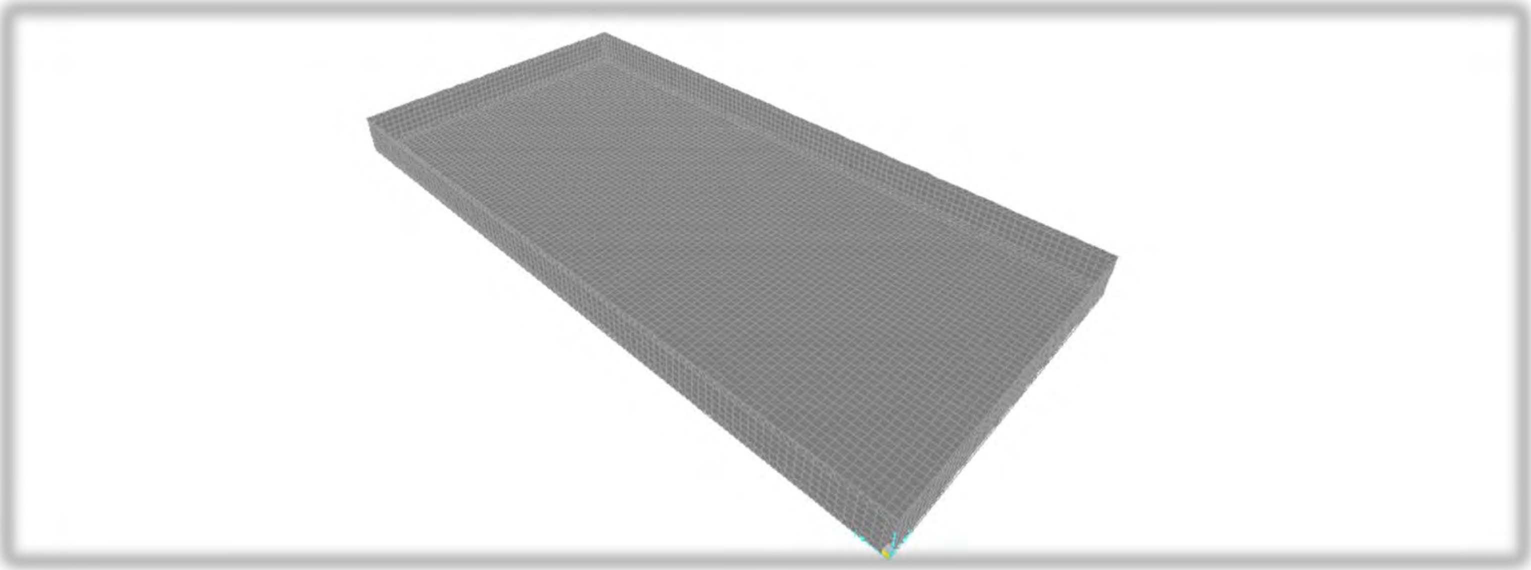
6.3 CRACKING

It will be calculated as stated in the “ECP 203-2018 - section 4-3-2” for the following maximum design crack width:

- 0.15 mm for water-side exposure.

7. STRUCTURAL ANALYSIS

7.1 3d-model



7.2 ASSIGN OF LOADS

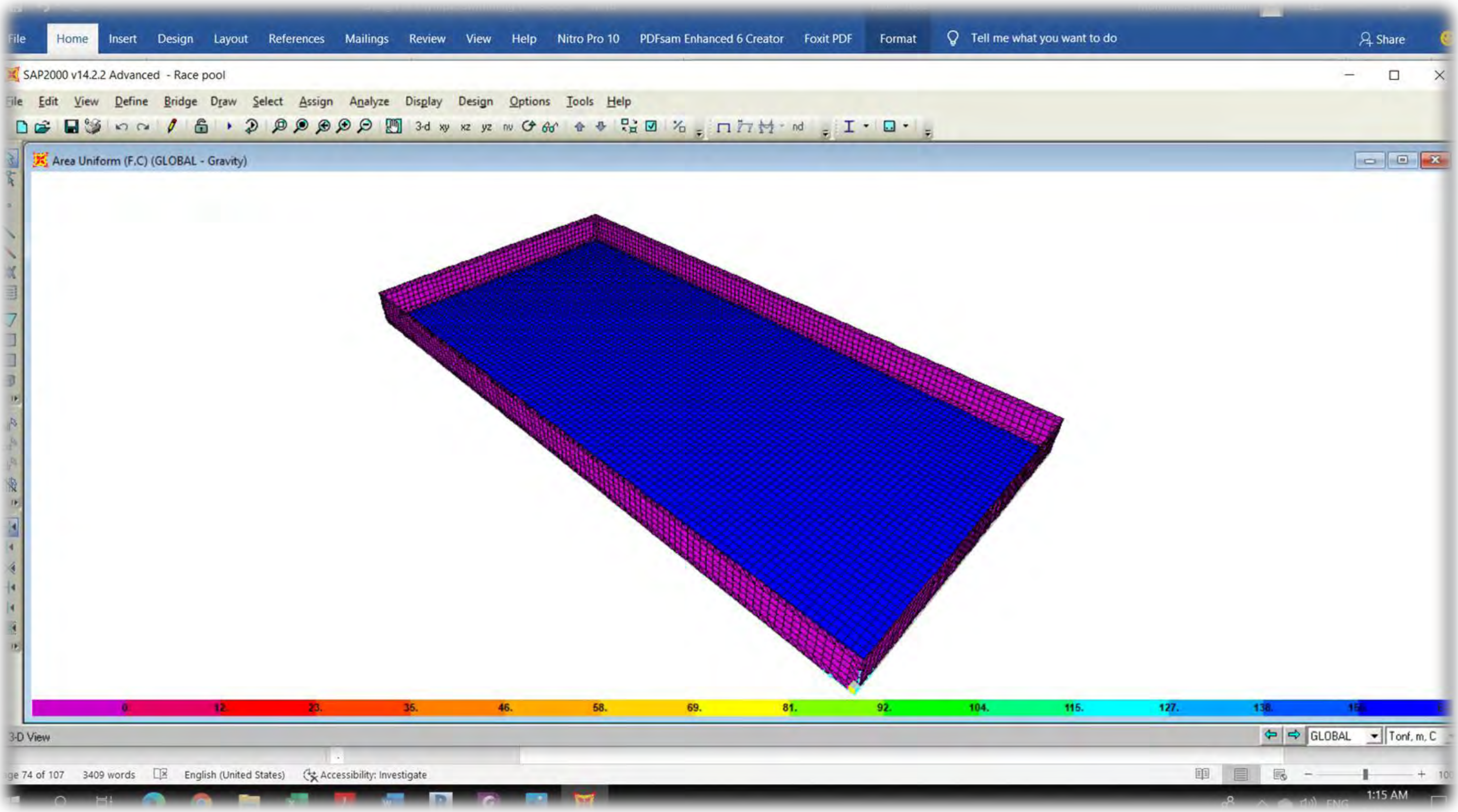


Figure 7.1.: Assign of Finishing Loads on the pool's raft (T-M^2 Units)

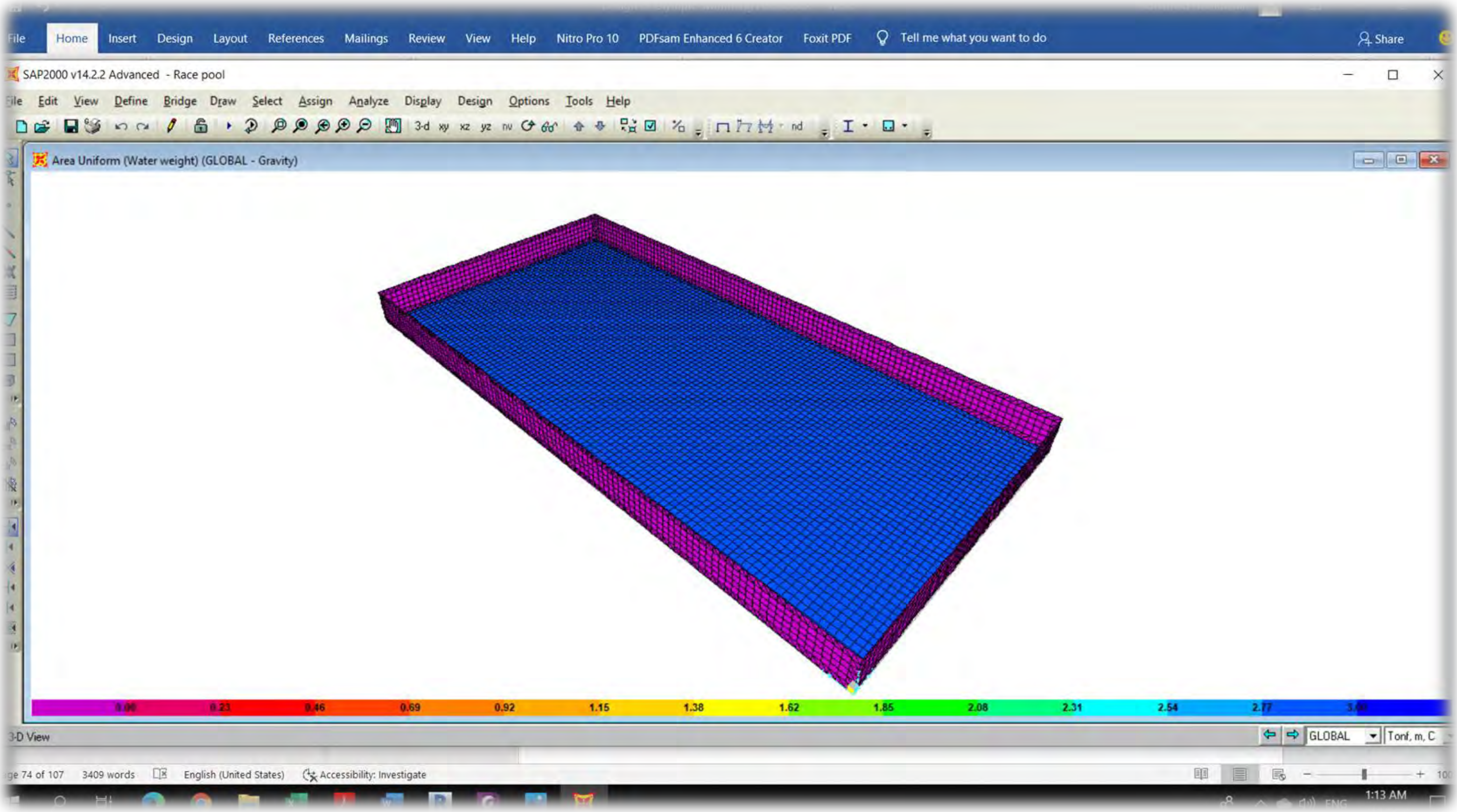


Figure 7.2.: Assign of Water weight on the pool’s raft (T-M^2 Units)

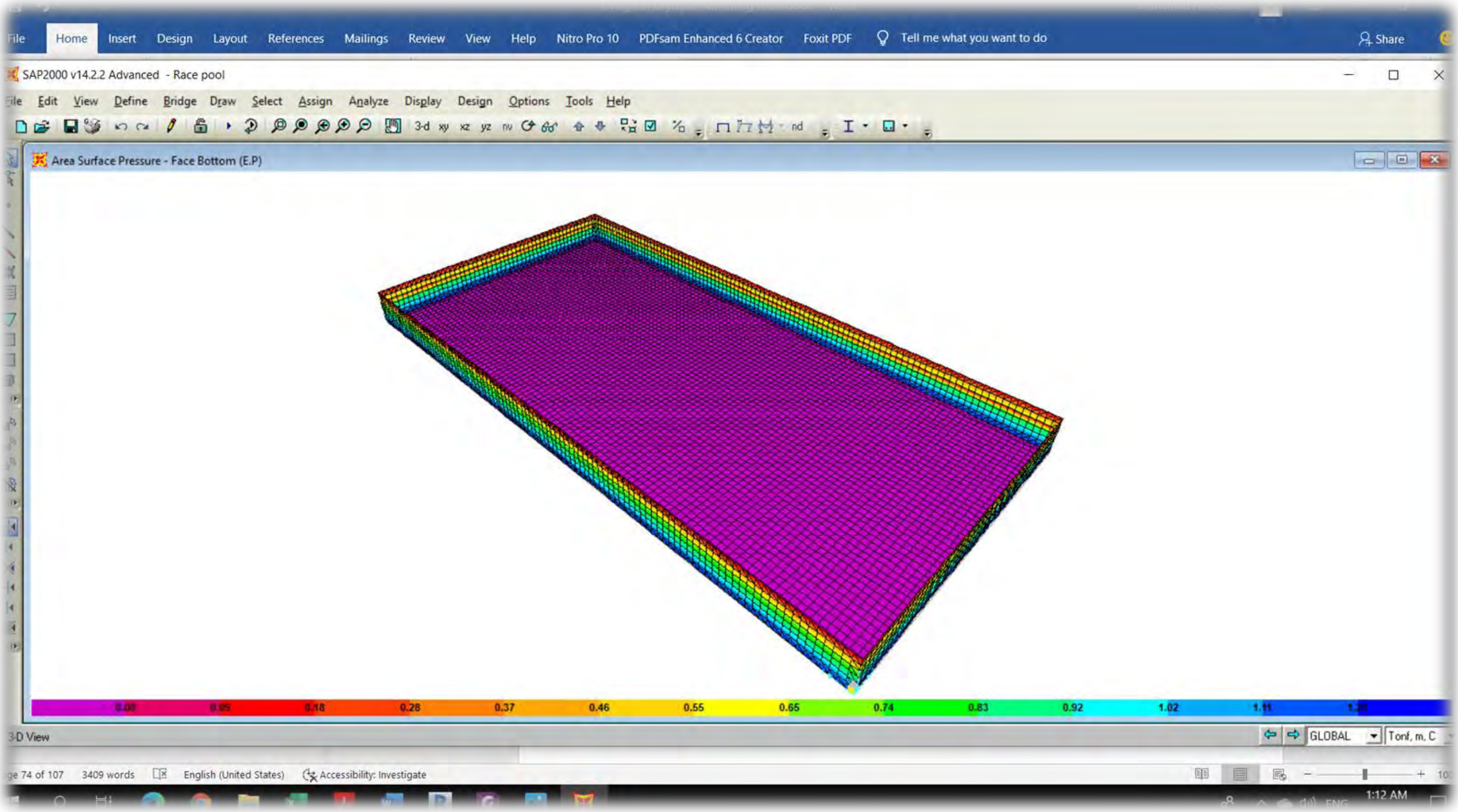
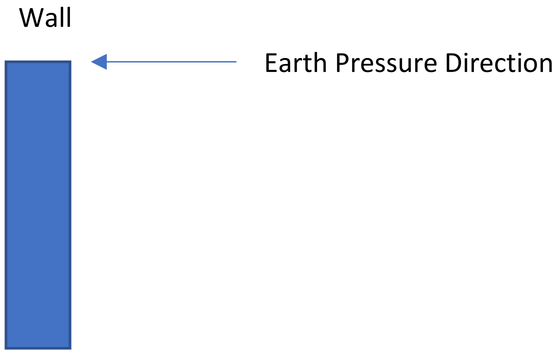


Figure 7.3.: Assign of Earth Pressure on the pool's Walls (T-M^2 Units)



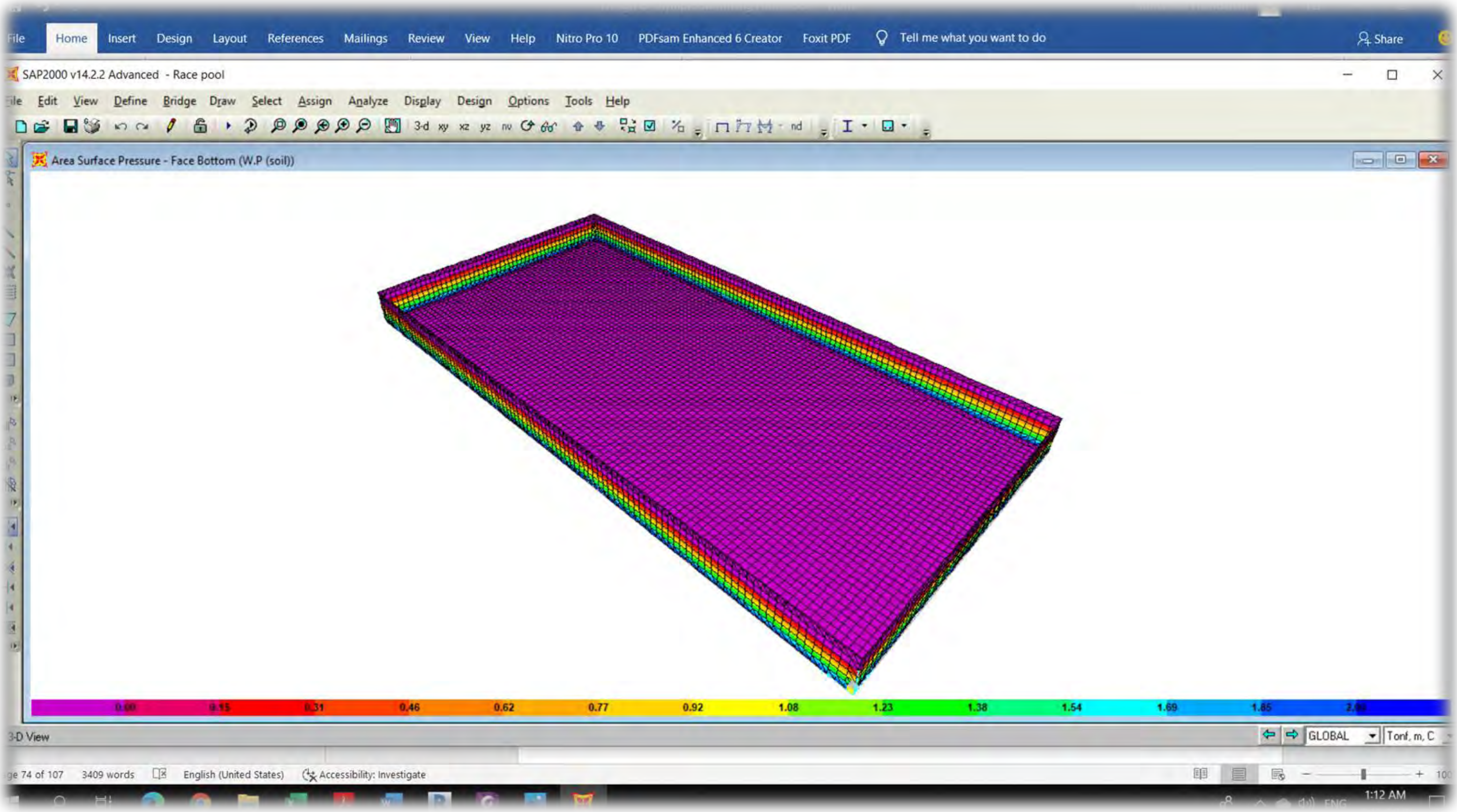
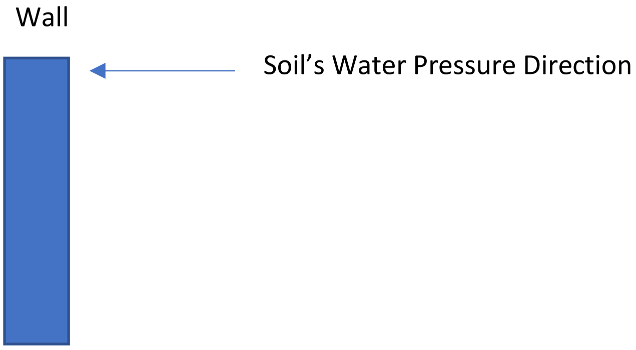


Figure 7.4.: Assign of Water pressure induced from soil on the pool's Walls (T-M^2 Units)



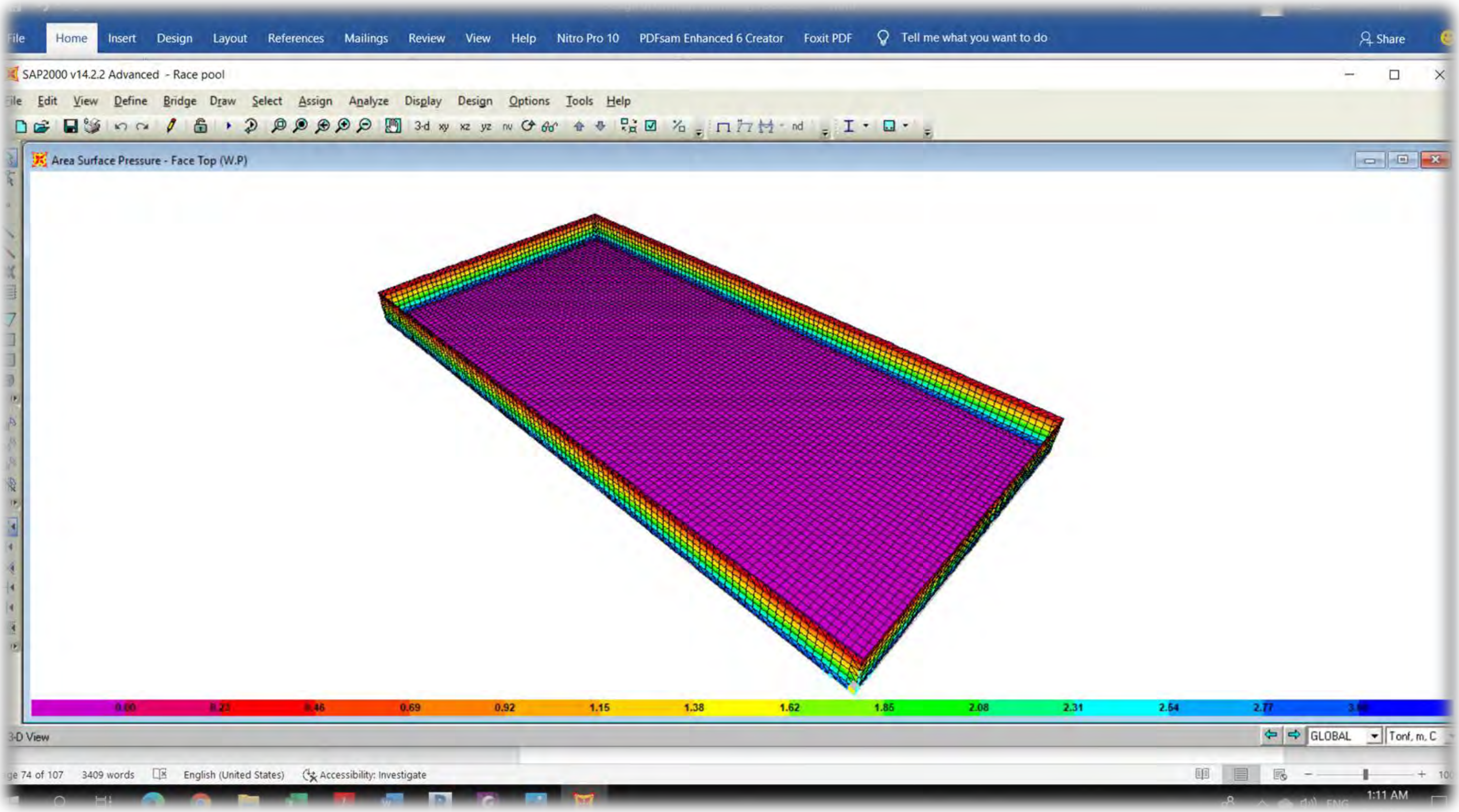
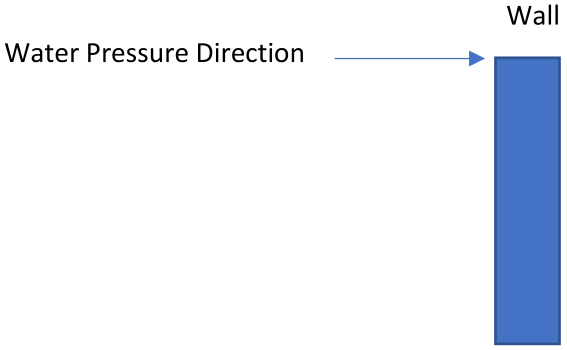


Figure 7.5.: Assign of Water Pressure on the pool’s walls (T-M^2 Units)

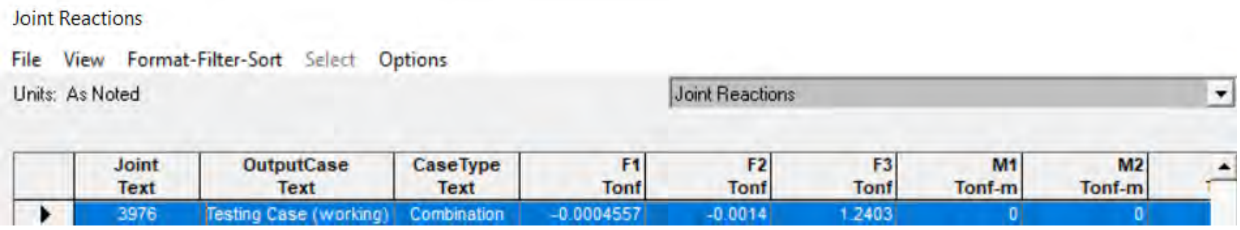


8. STRUCTURAL DESIGN.

8.1 Checks.

8.1.1 Bearing Capacity.

From SAP model, the maximum soil reaction equal 1.24 ton when the tank is full (Testing case)



Units: As Noted

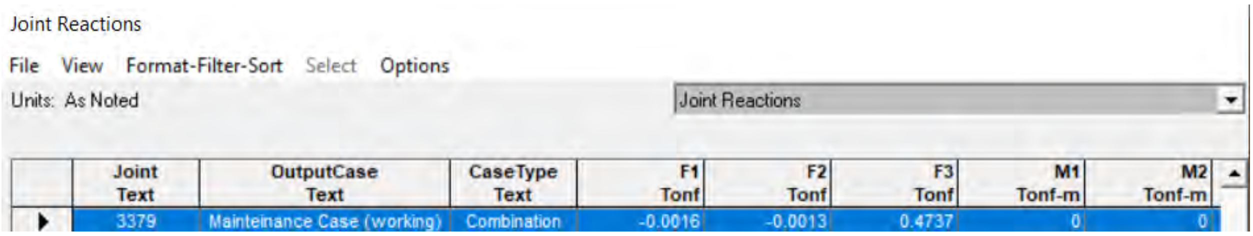
Joint Text	OutputCase Text	CaseType Text	F1 Tonf	F2 Tonf	F3 Tonf	M1 Tonf-m	M2 Tonf-m
3976	Testing Case (working)	Combination	-0.0004557	-0.0014	1.2403	0	0

The maximum stress distributed under soil = $\frac{1.24}{0.5*0.5}$ = 4.96 t/m2 < 15 t/m^2 (Safe)

8.1.2 Uplift.

From SAP model, the maximum soil reaction equal 0.47 ton when the tank is empty

(Maintenance case).



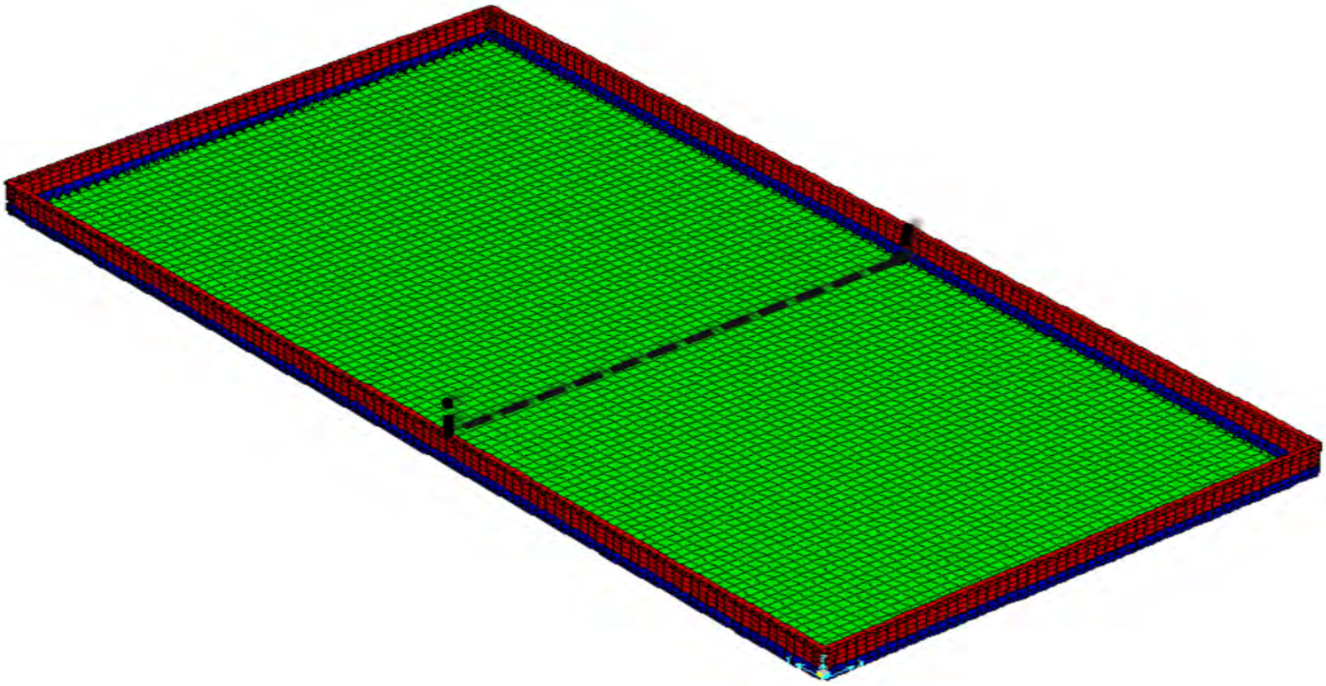
Units: As Noted

Joint Text	OutputCase Text	CaseType Text	F1 Tonf	F2 Tonf	F3 Tonf	M1 Tonf-m	M2 Tonf-m
3379	Mainteinance Case (working)	Combination	-0.0016	-0.0013	0.4737	0	0

The maximum stress distributed under soil = $\frac{0.47}{0.5*0.5}$ = 1.88 t/m^2 > 1.5 (Safe)

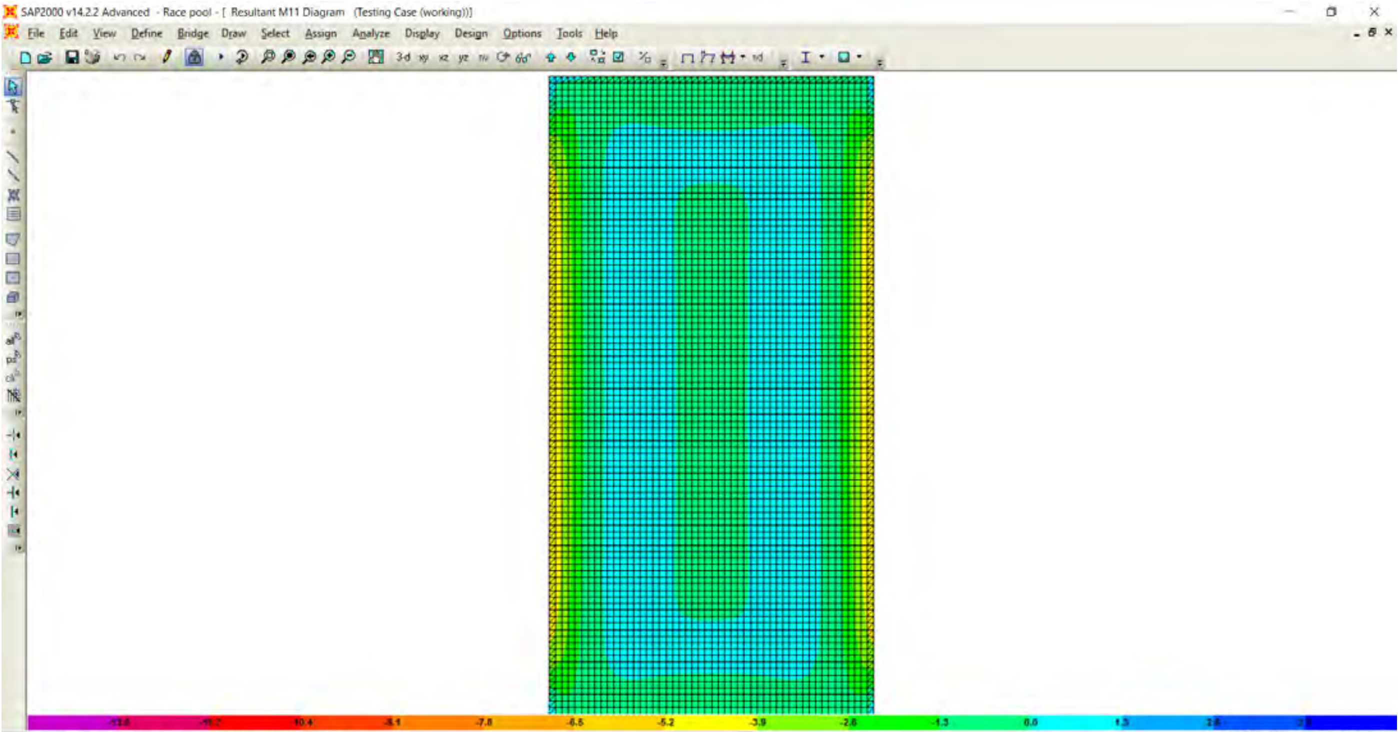
8.2 Analysis.

8.2.1 Short direction strip.



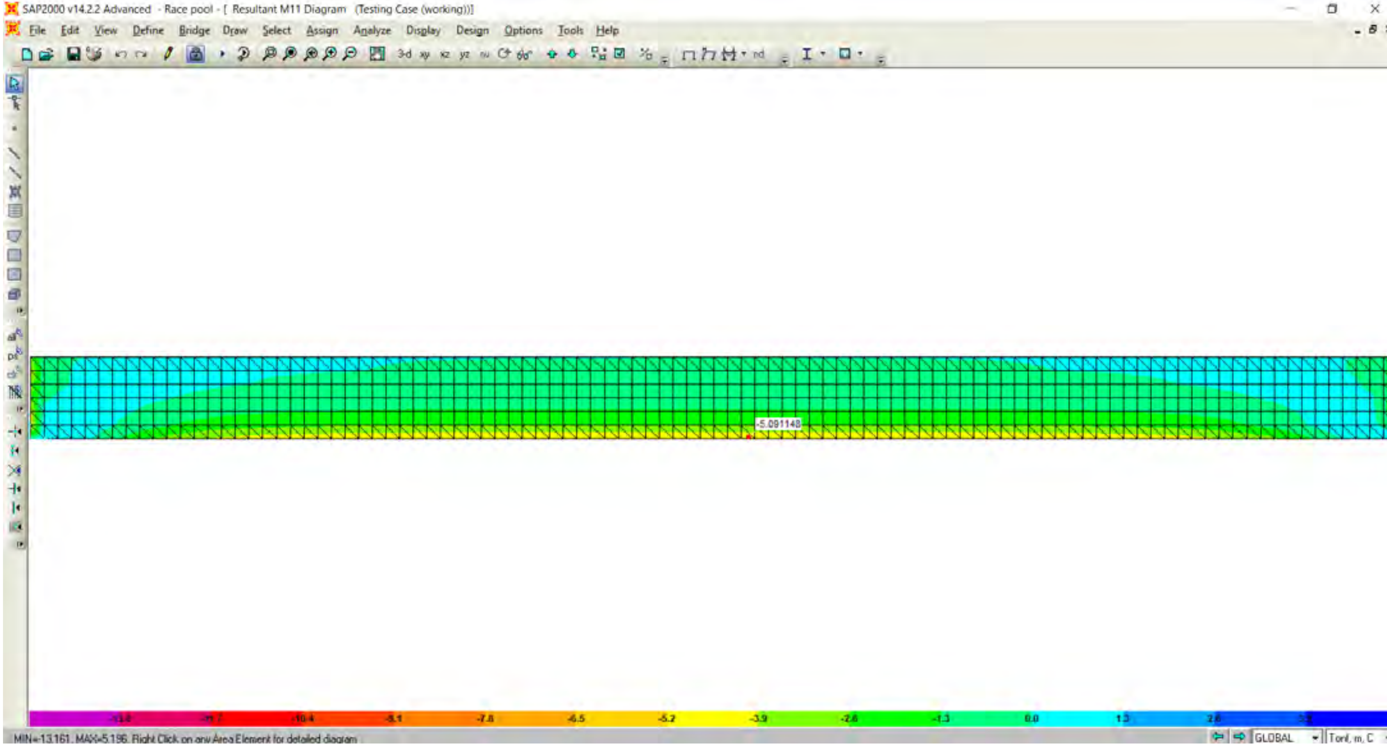
➢ Testing Case (Working straining actions).

Bending moment.



SAP2000 v14.2.2 Advanced - Race pool - [Resultant M11 Diagram (Testing Case (working))]

MIN=-13.161, MAX=5.196, Right Click on any Area Element for detailed diagram

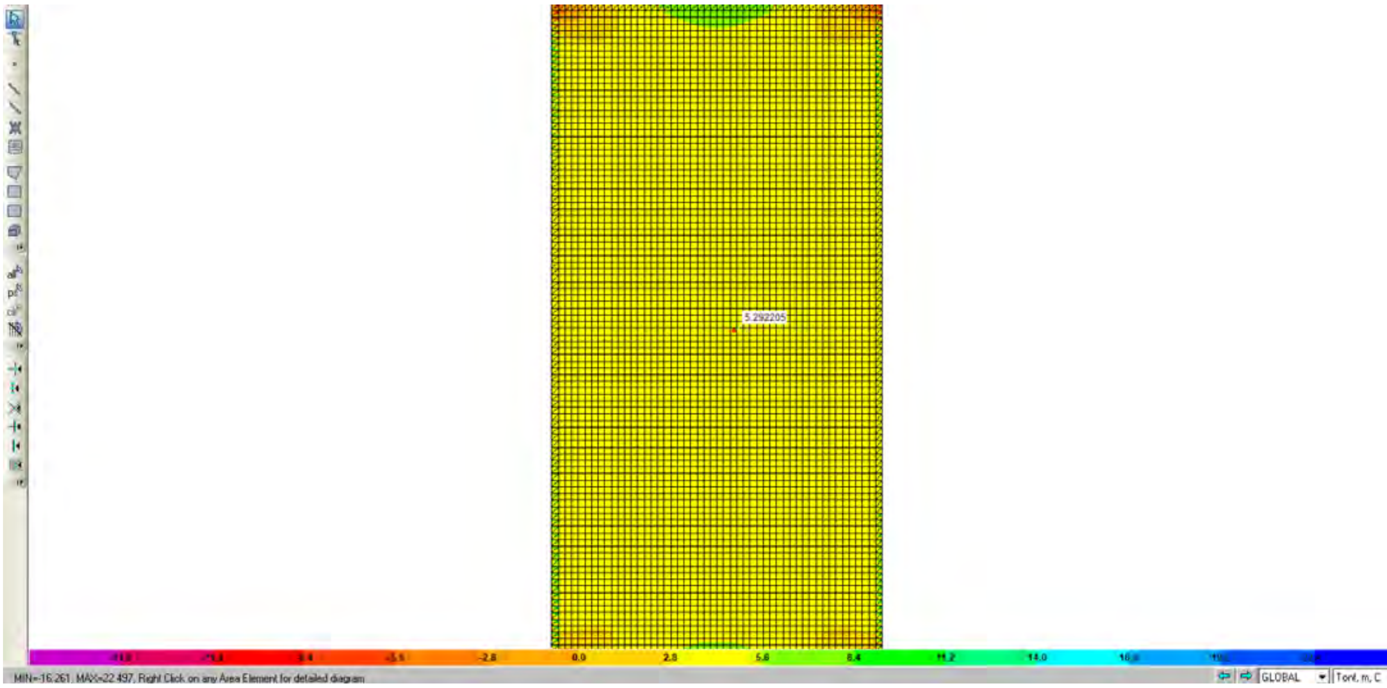


SAP2000 v14.2.2 Advanced - Race pool - [Resultant M11 Diagram (Testing Case (working))]

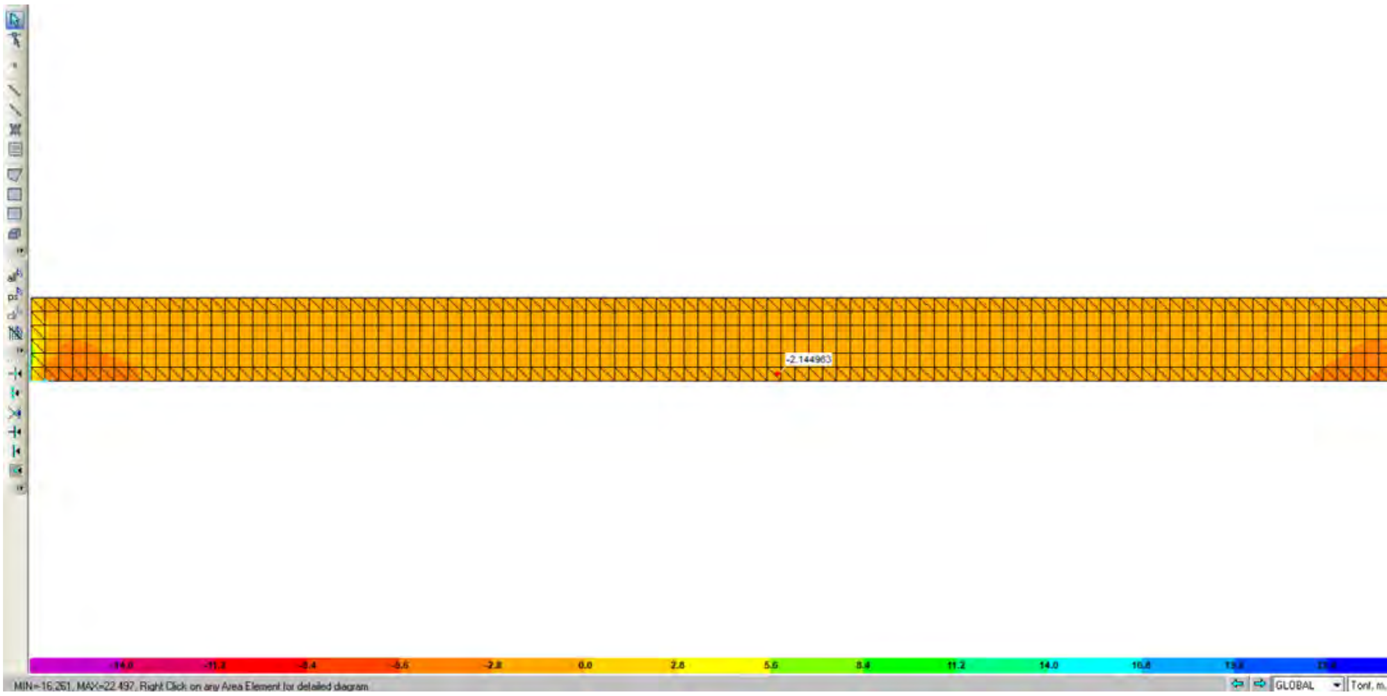
-5.091148

MIN=-13.161, MAX=5.196, Right Click on any Area Element for detailed diagram

Normal Forces.



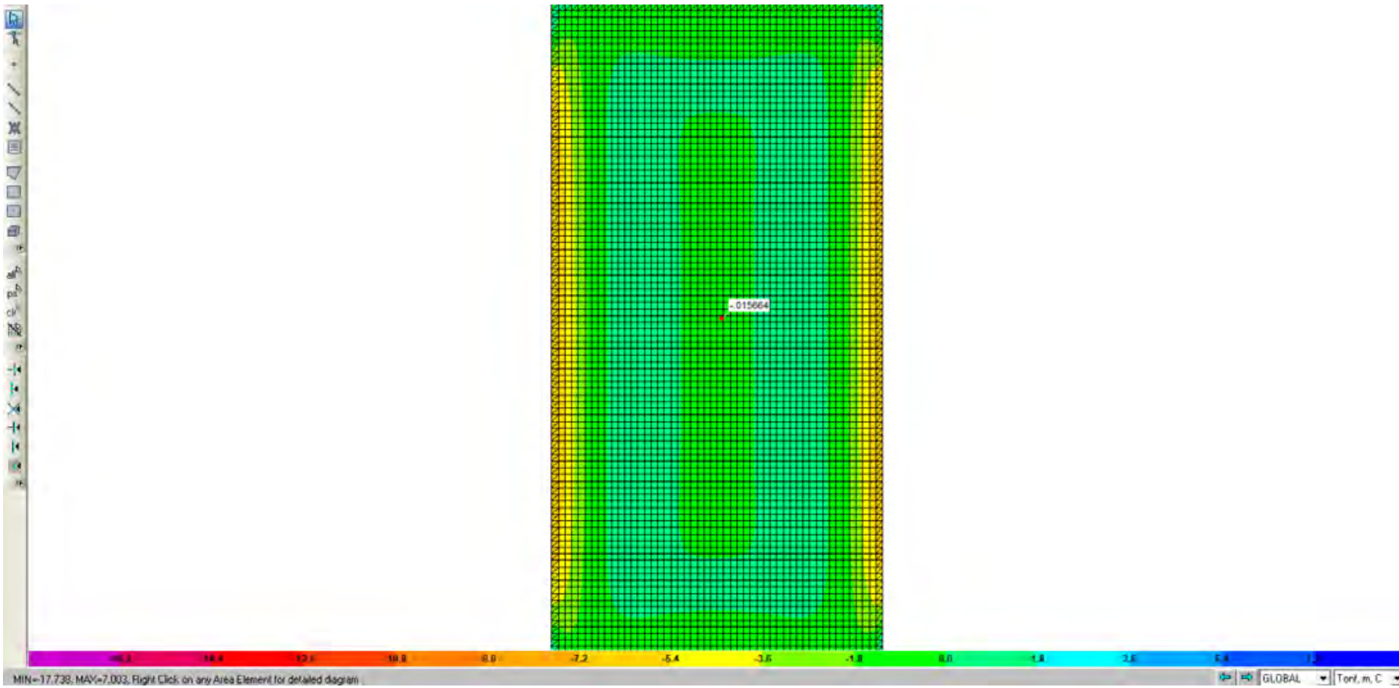
MIN=-16.261, MAX=22.497, Right Click on any Area Element for detailed diagram



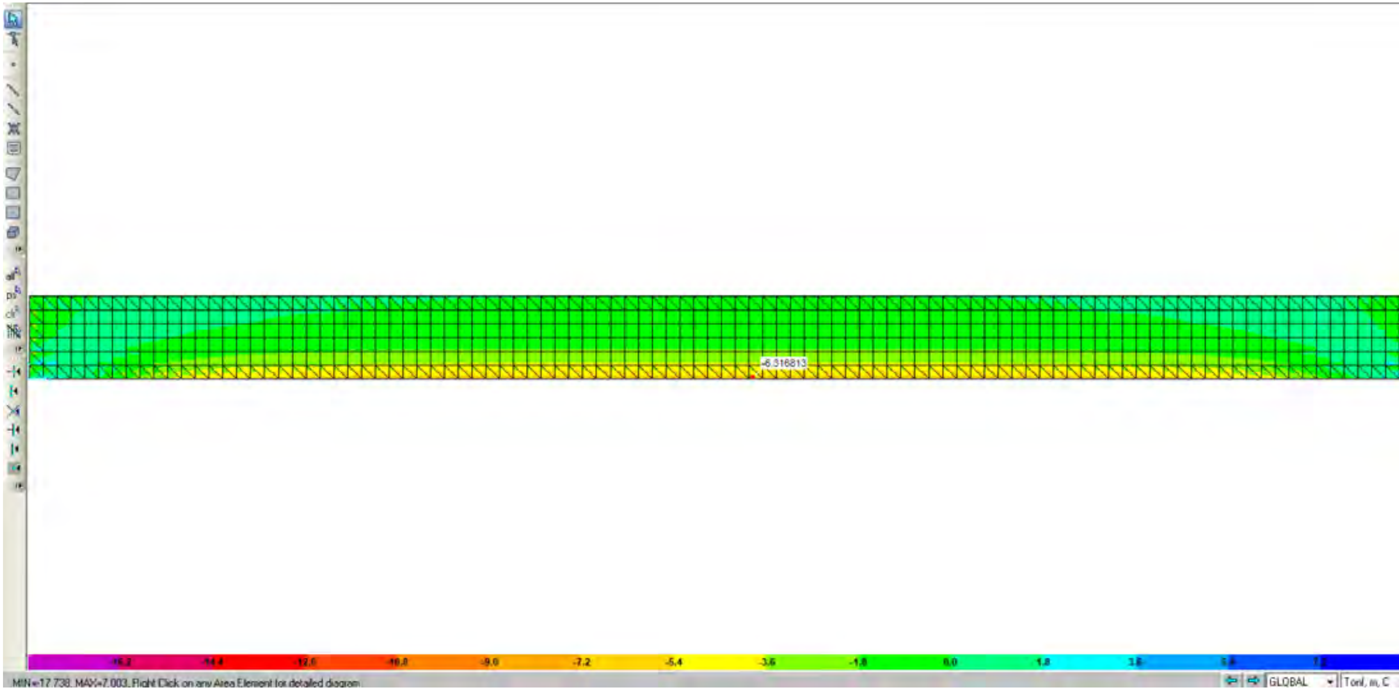
MIN=-16.261, MAX=22.497, Right Click on any Area Element for detailed diagram

- **Testing Case (Ultimate straining actions).**

Bending Moment.

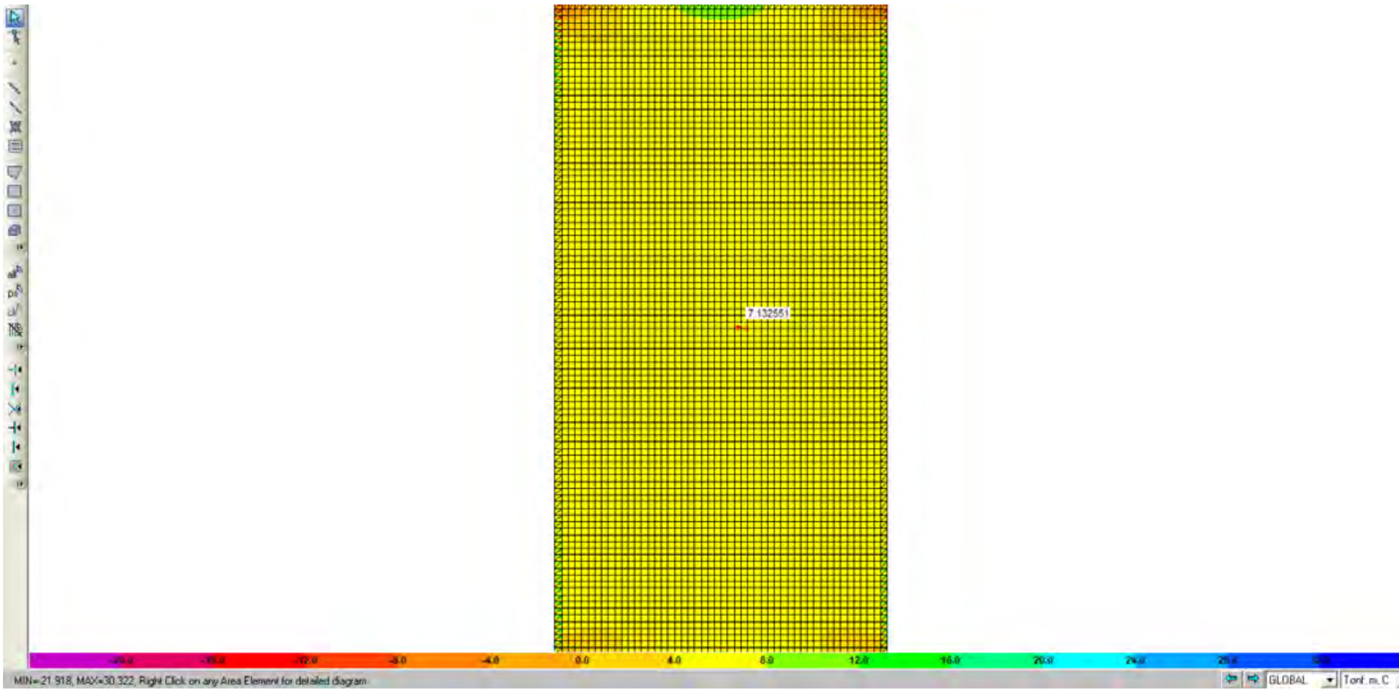


MIN=-17.738, MAX=7.003, Right Click on any Area Element for detailed diagram

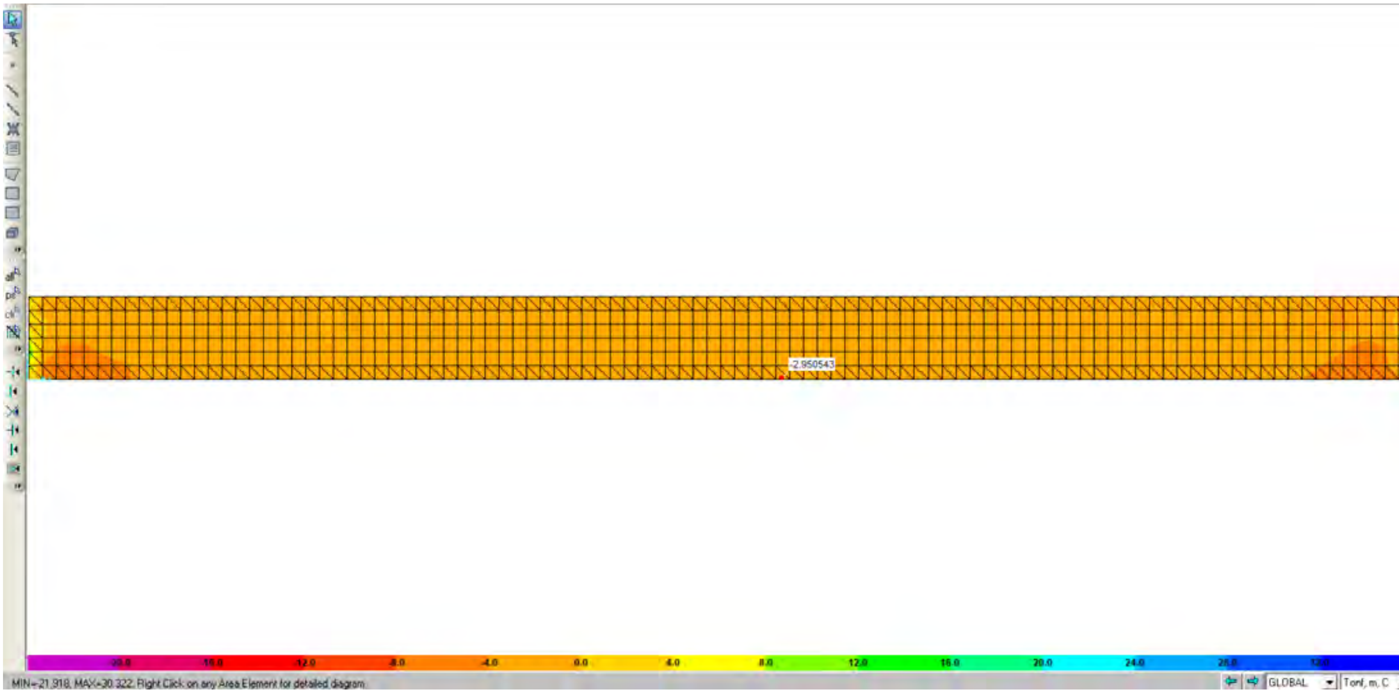


MIN=-17.738, MAX=7.003, Right Click on any Area Element for detailed diagram

Normal Forces.

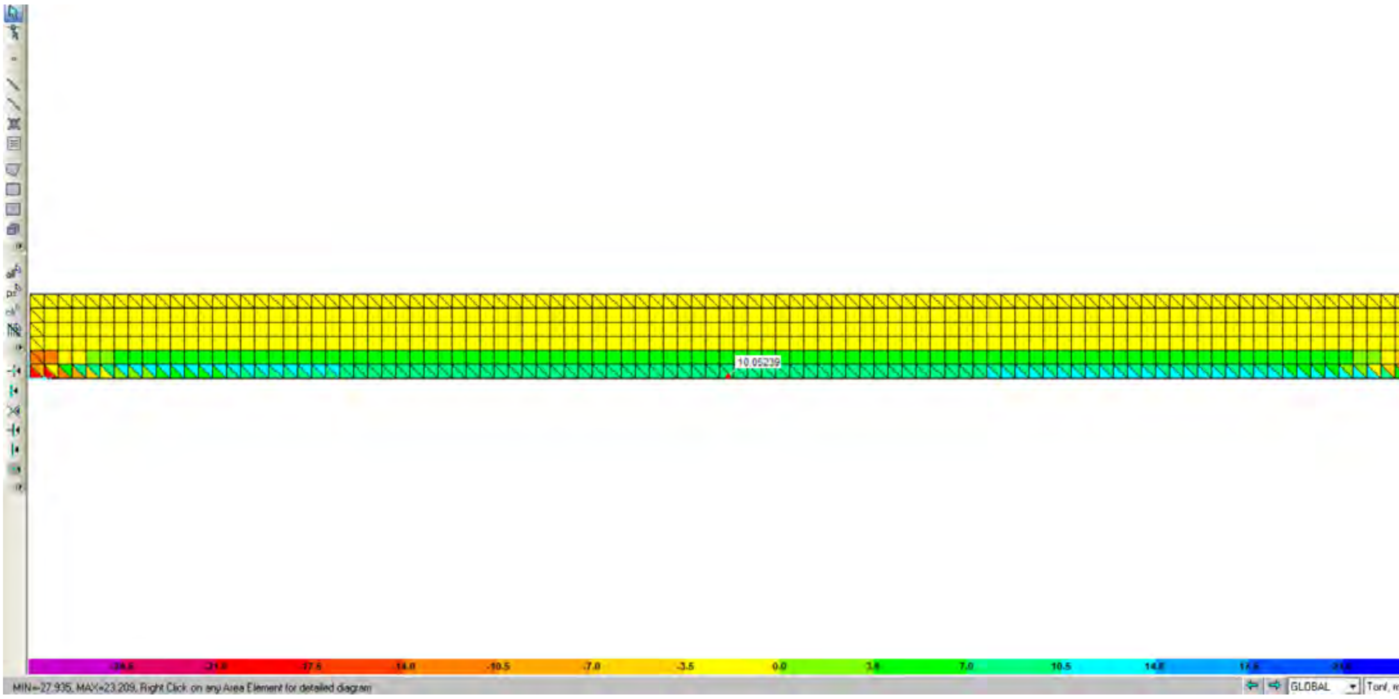


MIN=-21.918, MAX=30.322, Right Click on any Area Element for detailed diagram



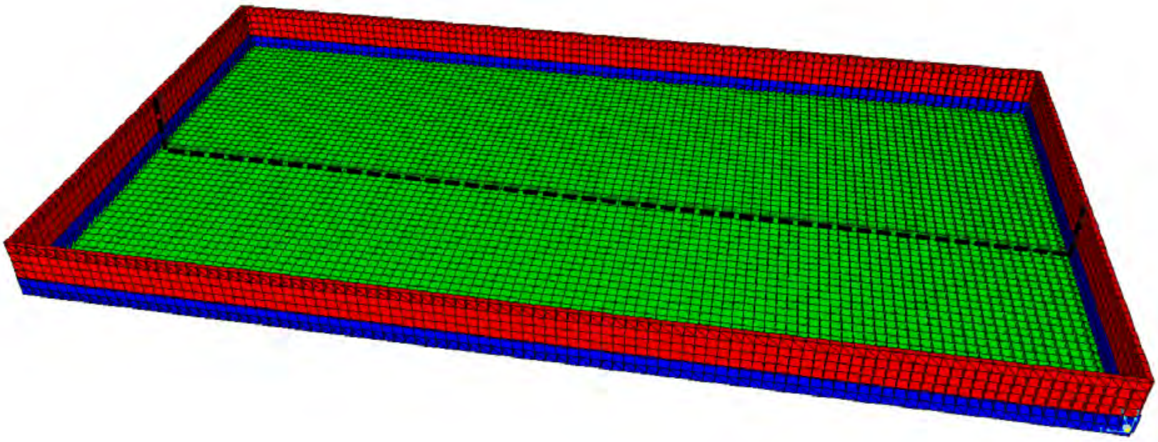
MIN=-21.918, MAX=30.322, Right Click on any Area Element for detailed diagram

Shear Forces.



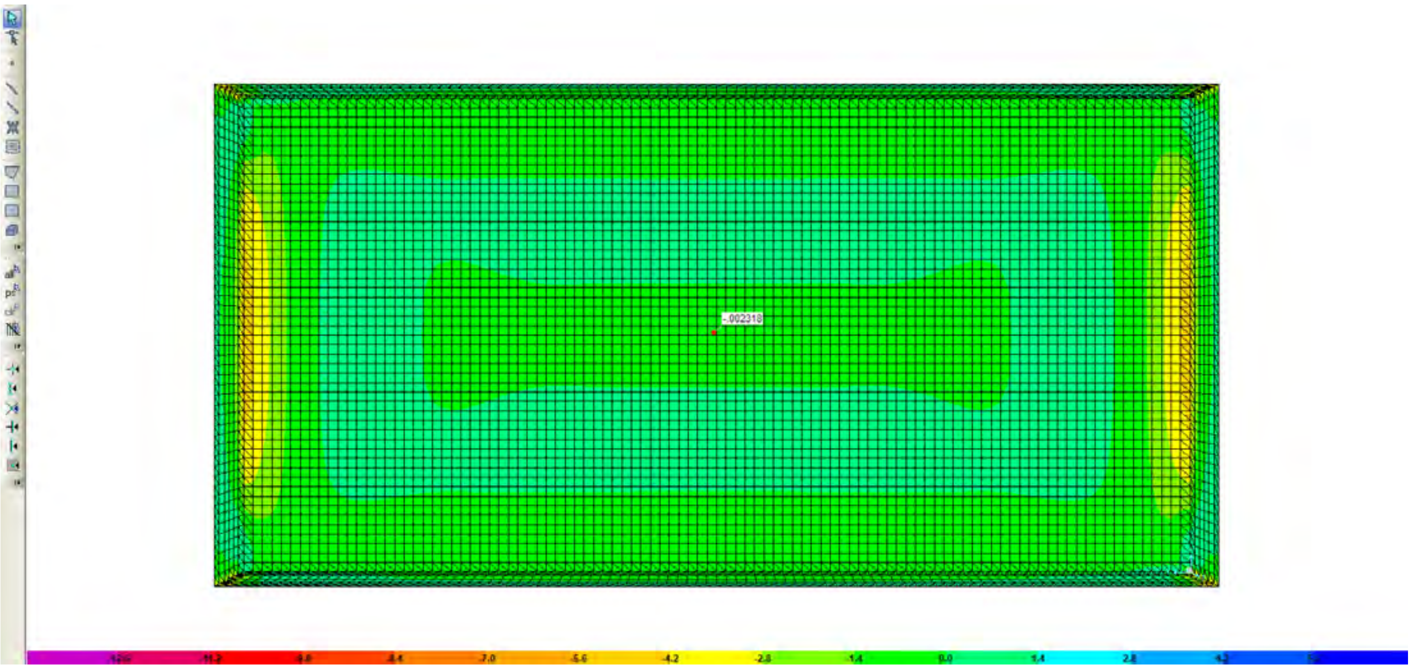
MIN=-27.935, MAX=23.209, Right Click on any Area Element for detailed diagram

8.2.2 Long direction strip.

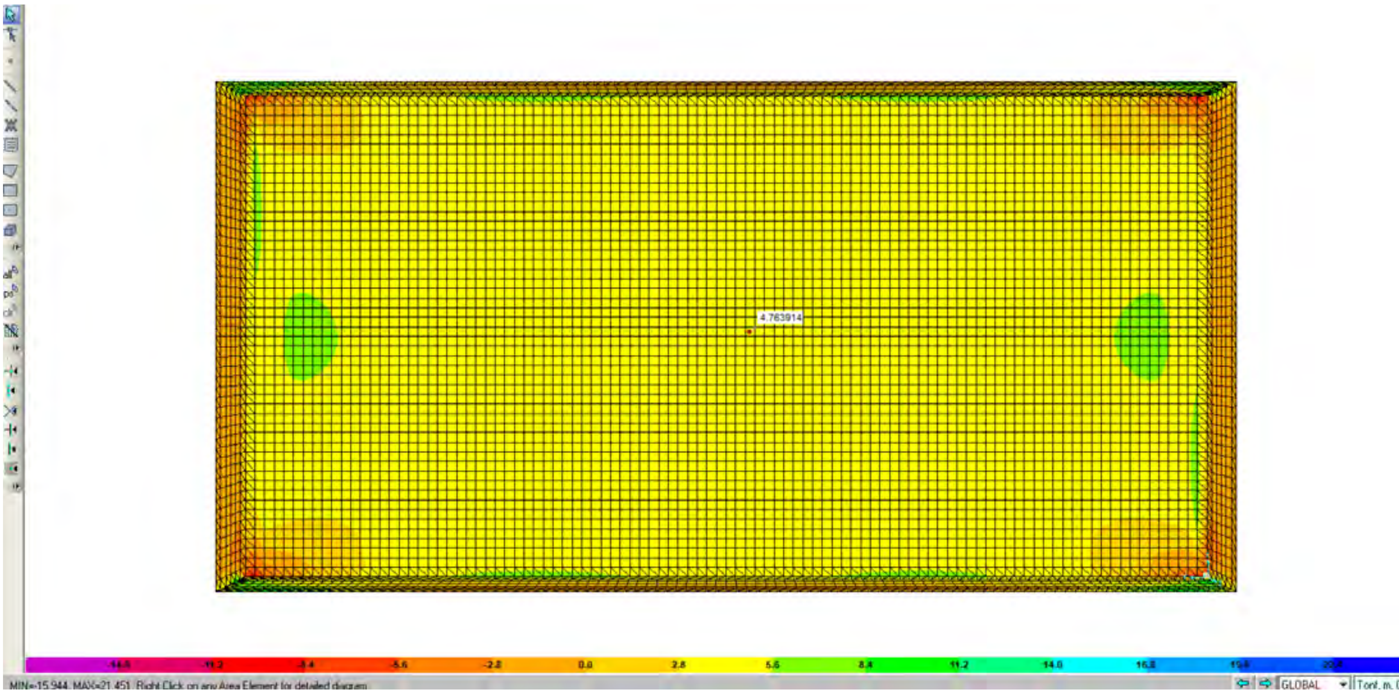


- **Testing Case (Working straining actions)**

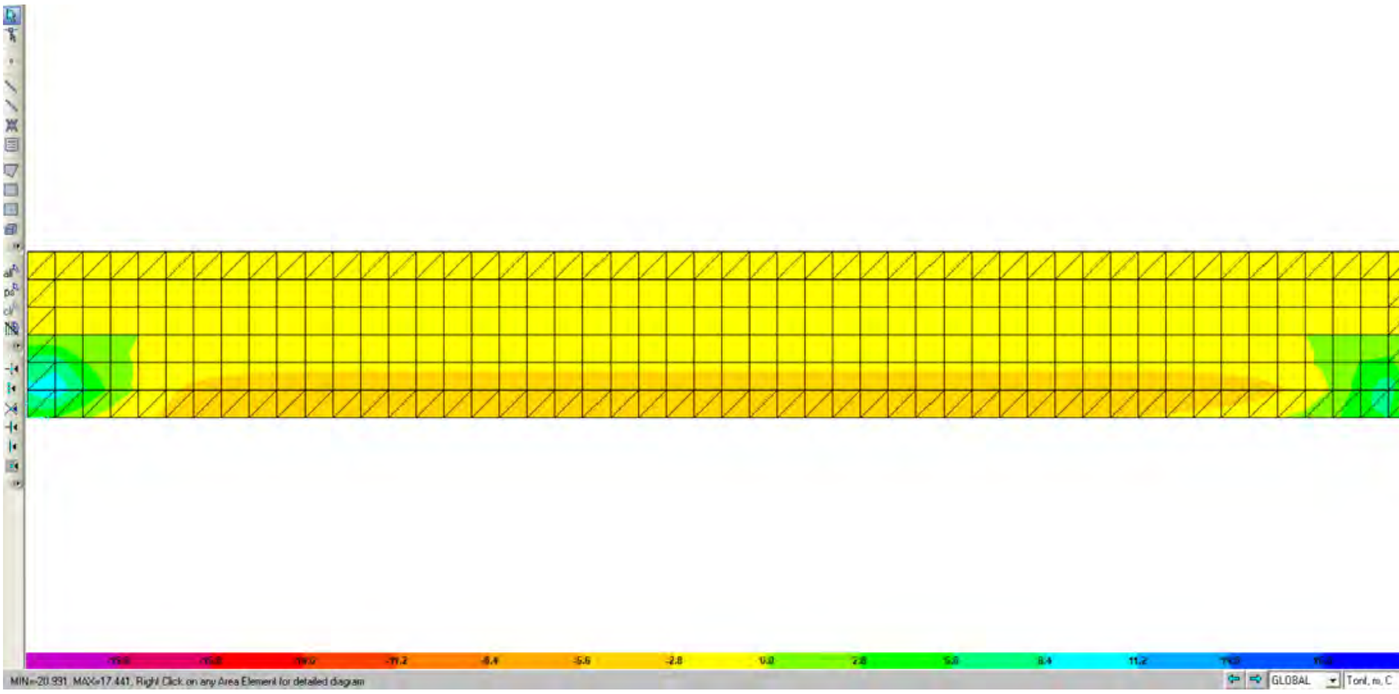
Bending Moment



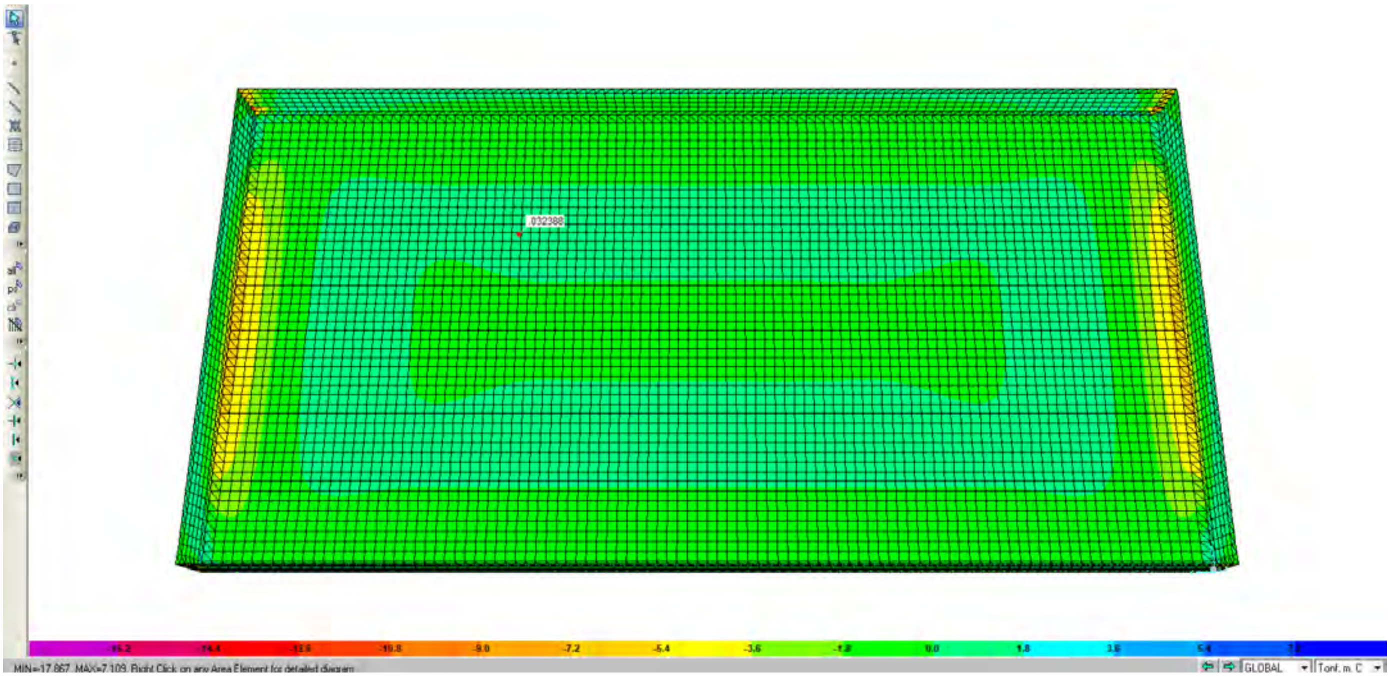
Normal Forces.



Shear Force.

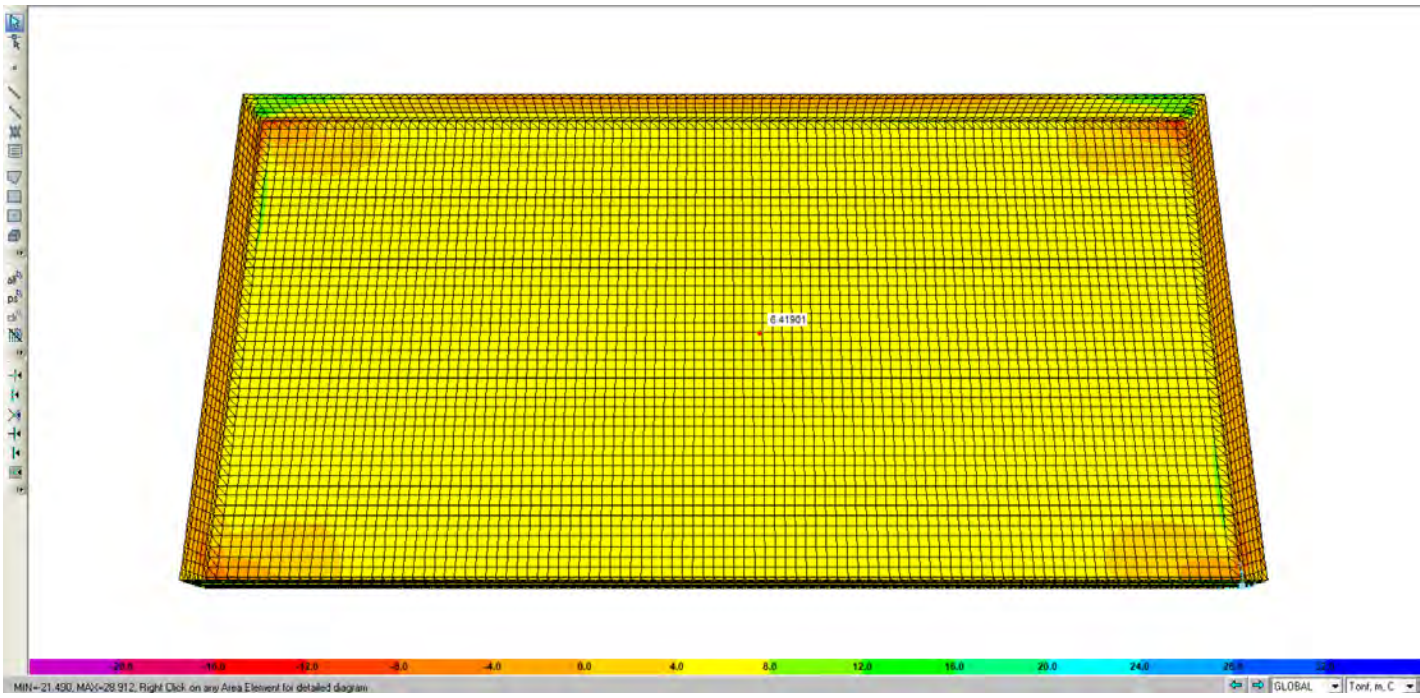


- **Testing Case (Ultimate straining actions).**
  Bending Moment



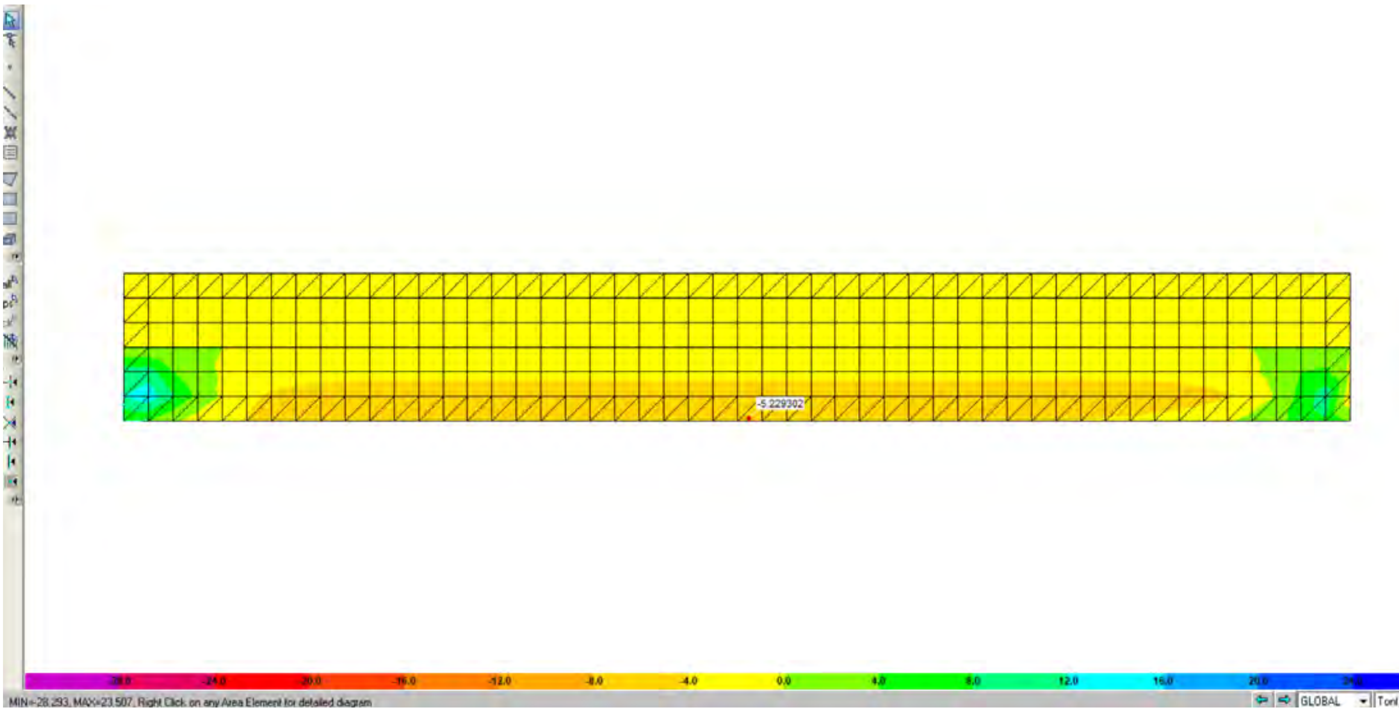
MIN=-17.867, MAX=7.109, Right Click on any Area Element for detailed diagram

Normal Force.



MIN=-21.490, MAX=28.912, Right Click on any Area Element for detailed diagram

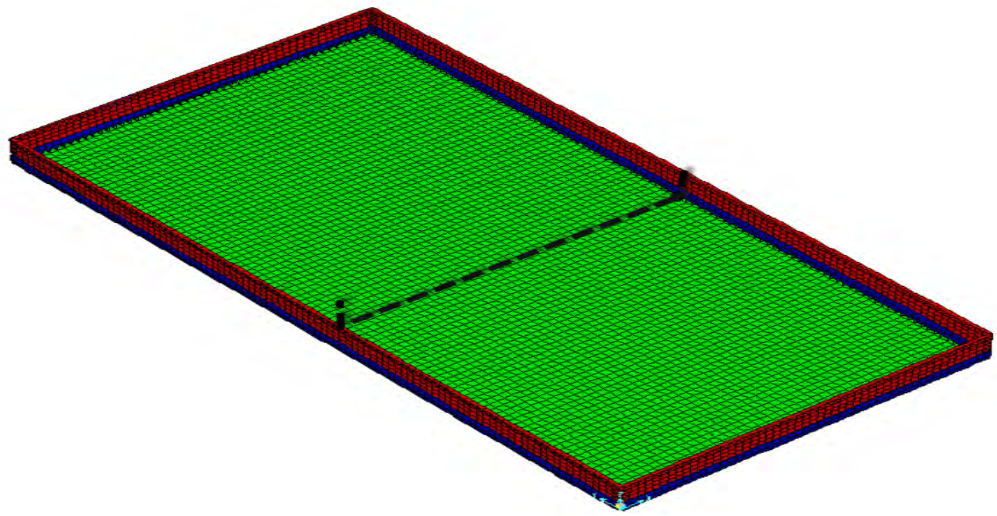
Shear Force.

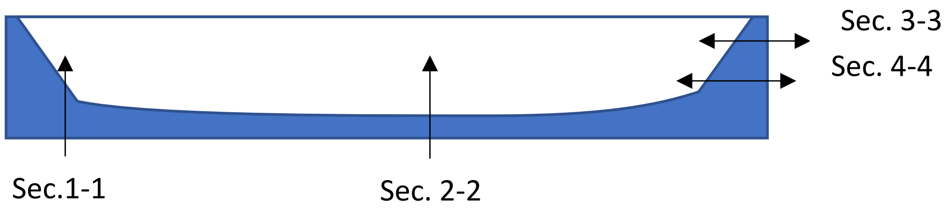


MIN=-28.293, MAX=23.507, Right Click on any Area Element for detailed diagram

8.3 Design.

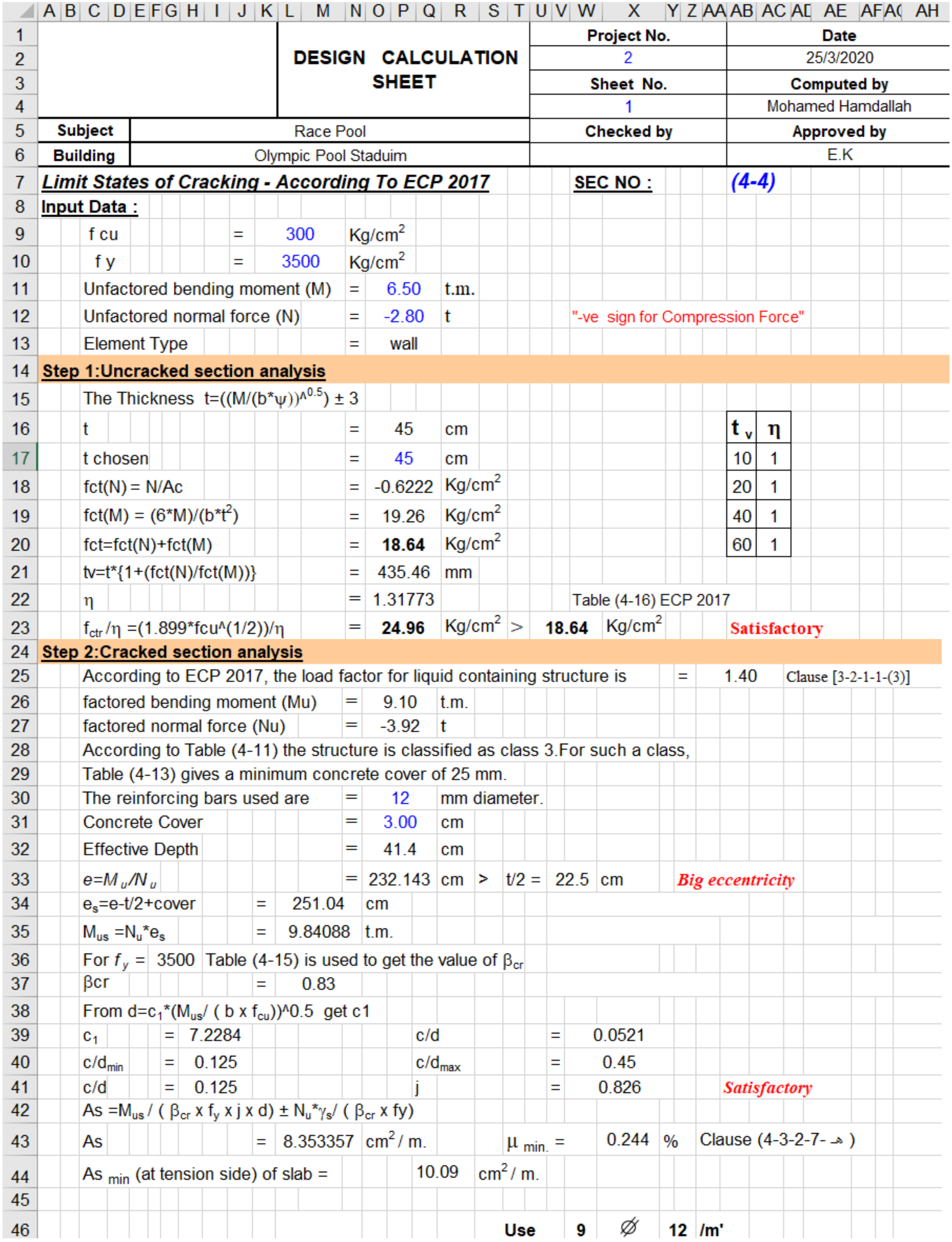
8.3.1 Short direction strip.





- **Testing Case.**
  - Wall Design.

Sec. 4-4



	DESIGN CALCULATION SHEET	**Project No.**	**Date**
		2	25/3/2020
		Sheet No.	**Computed by**
		1	Mohamed Hamdallah
Subject	Race Pool	**Checked by**	**Approved by**
Building	Olympic Pool Staduim		E.K

7 ***Limit States of Cracking - According To ECP 2017*** **SEC NO :** ***(4-4)***

8 **Input Data :**

9 f cu = 300 Kg/cm^2

10 f y = 3500 Kg/cm^2

11 Unfactored bending moment (M) = 6.50 t.m.

12 Unfactored normal force (N) = -2.80 t "-ve sign for Compression Force"

13 Element Type = wall

14 **Step 1:Uncracked section analysis**

15 The Thickness $t=((M/(b*\psi))^{\wedge 0.5}) \pm 3$

16 t = 45 cm

17 t chosen = 45 cm

18 fct(N) = N/Ac = -0.6222 Kg/cm^2

19 $fct(M) = (6*M)/(b*t^2)$ = 19.26 Kg/cm^2

20 fct=fct(N)+fct(M) = **18.64** Kg/cm^2

21 tv=t*{1+(fct(N)/fct(M))} = 435.46 mm

22 η = 1.31773 Table (4-16) ECP 2017

23 f_{ctr}/η =(1.899*fcu^(1/2))/η = **24.96** Kg/cm^2 > **18.64** Kg/cm^2 **Satisfactory**

t_v	η
10	1
20	1
40	1
60	1

24 **Step 2:Cracked section analysis**

25 According to ECP 2017, the load factor for liquid containing structure is = 1.40 Clause [3-2-1-1-(3)]

26 factored bending moment (Mu) = 9.10 t.m.

27 factored normal force (Nu) = -3.92 t

28 According to Table (4-11) the structure is classified as class 3.For such a class,

29 Table (4-13) gives a minimum concrete cover of 25 mm.

30 The reinforcing bars used are = 12 mm diameter.

31 Concrete Cover = 3.00 cm

32 Effective Depth = 41.4 cm

33 $e=M_u/N_u$ = 232.143 cm > t/2 = 22.5 cm ***Big eccentricity***

34 e_s=e-t/2+cover = 251.04 cm

35 $M_{us} = N_u * e_s$ = 9.84088 t.m.

36 For f_y = 3500 Table (4-15) is used to get the value of β_{cr}

37 βcr = 0.83

38 From $d=c_1*(M_{us}/ (b \times f_{cu}))^{\wedge}0.5$ get c1

39 c_1 = 7.2284 c/d = 0.0521

40 c/d_{min} = 0.125 c/d_{max} = 0.45

41 c/d = 0.125 j = 0.826 ***Satisfactory***

42 $As = M_{us} / (\beta_{cr} \times f_y \times j \times d) \pm N_u*\gamma_s/ (\beta_{cr} \times fy)$

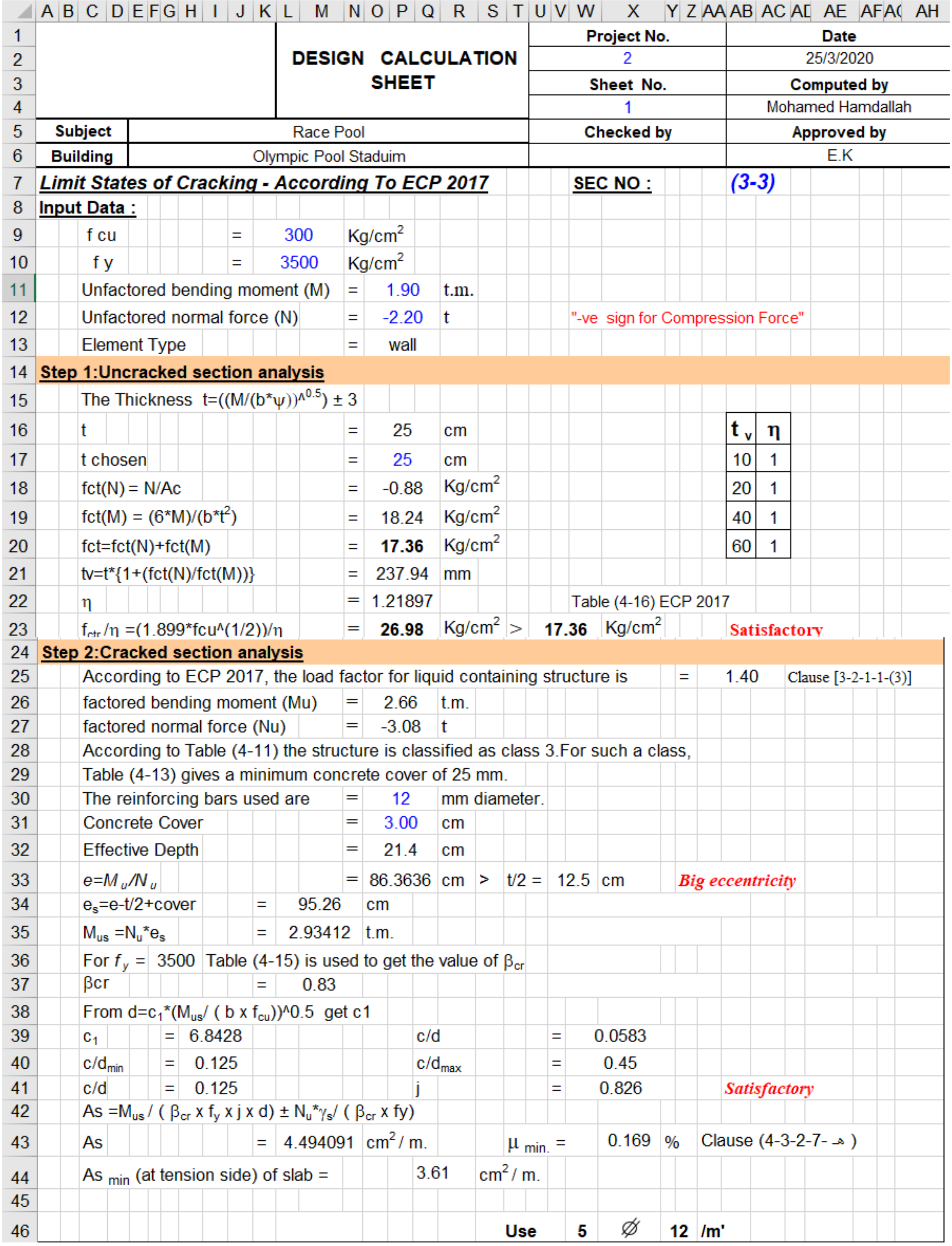
43 As = 8.353357 cm^2 / m. $\mu_{min.}$ = 0.244 % Clause (4-3-2-7- هـ)

44 As_{min} (at tension side) of slab = 10.09 cm^2 / m.

45

46 **Use 9 ∅ 12 /m'**

Sec. 3-3



		Project No.	Date
	DESIGN CALCULATION SHEET	2	25/3/2020
		Sheet No.	**Computed by**
		1	Mohamed Hamdallah
Subject	Race Pool	**Checked by**	**Approved by**
Building	Olympic Pool Staduim		E.K

Limit States of Cracking - According To ECP 2017 **SEC NO :** ***(3-3)***

Input Data :

f cu = 300 Kg/cm^2

f y = 3500 Kg/cm^2

Unfactored bending moment (M) = 1.90 t.m.

Unfactored normal force (N) = -2.20 t "-ve sign for Compression Force"

Element Type = wall

Step 1:Uncracked section analysis

The Thickness t=((M/(b*ψ))^$^{0.5}$) ± 3

t = 25 cm

t chosen = 25 cm

fct(N) = N/Ac = -0.88 Kg/cm^2

fct(M) = (6*M)/(b*t^2) = 18.24 Kg/cm^2

fct=fct(N)+fct(M) = **17.36** Kg/cm^2

tv=t*{1+(fct(N)/fct(M))} = 237.94 mm

η = 1.21897 Table (4-16) ECP 2017

f_{ctr}/η =(1.899*fcu^(1/2))/η = **26.98** Kg/cm^2 > **17.36** Kg/cm^2 **Satisfactory**

t_v	η
10	1
20	1
40	1
60	1

Step 2:Cracked section analysis

According to ECP 2017, the load factor for liquid containing structure is = 1.40 Clause [3-2-1-1-(3)]

factored bending moment (Mu) = 2.66 t.m.

factored normal force (Nu) = -3.08 t

According to Table (4-11) the structure is classified as class 3.For such a class, Table (4-13) gives a minimum concrete cover of 25 mm.

The reinforcing bars used are = 12 mm diameter.

Concrete Cover = 3.00 cm

Effective Depth = 21.4 cm

$e=M_u/N_u$ = 86.3636 cm > t/2 = 12.5 cm ***Big eccentricity***

e_s=e-t/2+cover = 95.26 cm

M_{us} =N_u*e_s = 2.93412 t.m.

For f_y = 3500 Table (4-15) is used to get the value of β_{cr}

βcr = 0.83

From d=c_1*(M_{us}/ (b x f_{cu}))^0.5 get c1

c_1 = 6.8428 c/d = 0.0583

c/d_{min} = 0.125 c/d_{max} = 0.45

c/d = 0.125 j = 0.826 ***Satisfactory***

As =M_{us} / (β_{cr} x f_y x j x d) ± N_u*γ_s/ (β_{cr} x fy)

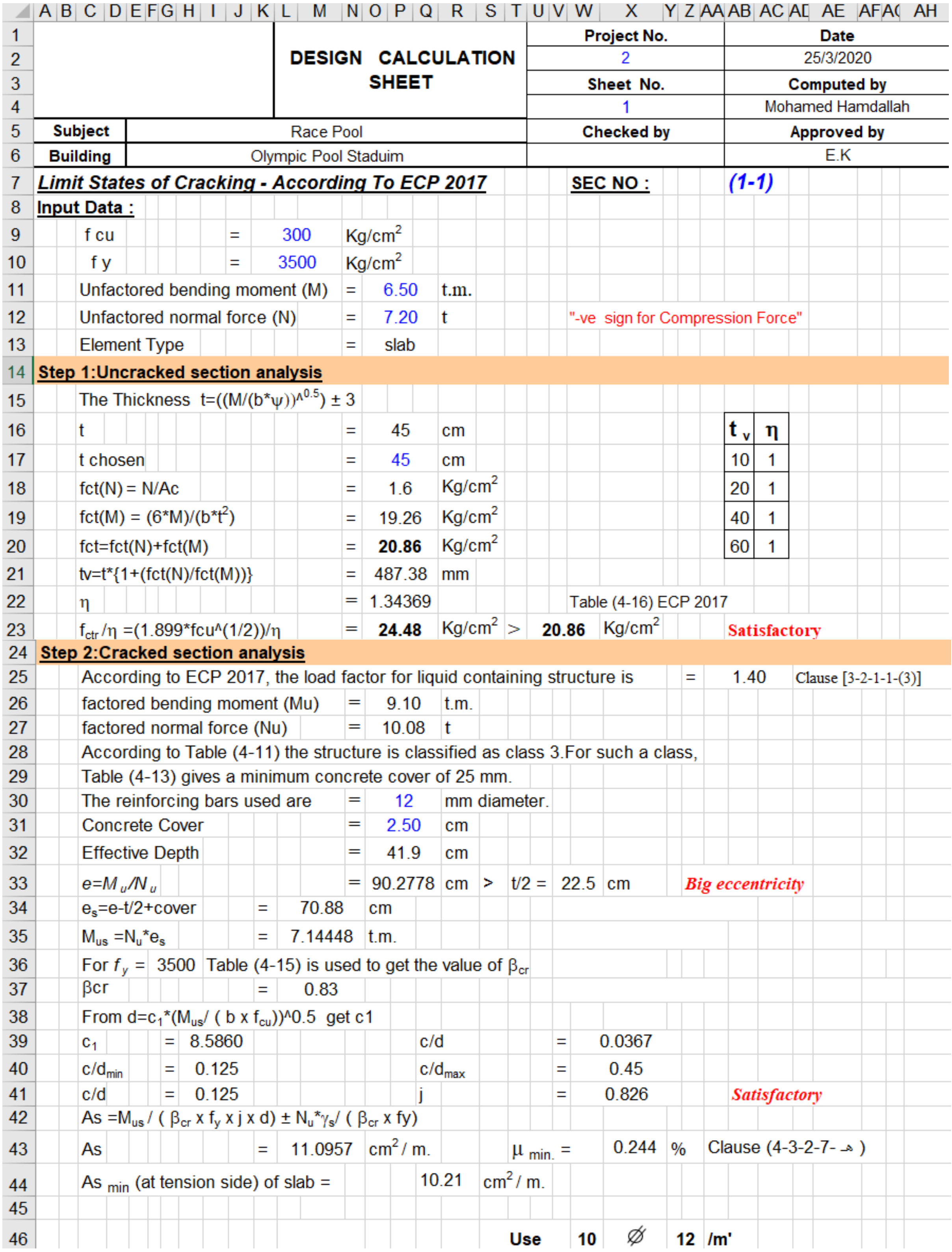
As = 4.494091 cm^2 / m. $\mu_{min.}$ = 0.169 % Clause (4-3-2-7- هـ)

As $_{min}$ (at tension side) of slab = 3.61 cm^2 / m.

Use 5 Ø 12 /m'

- Floor Design.

Sec. 1-1



	DESIGN CALCULATION SHEET	**Project No.**	**Date**
		2	25/3/2020
		Sheet No.	**Computed by**
		1	Mohamed Hamdallah
Subject	Race Pool	**Checked by**	**Approved by**
Building	Olympic Pool Staduim		E.K

7 ***Limit States of Cracking - According To ECP 2017*** **SEC NO :** ***(1-1)***

8 **Input Data :**

9 f cu = 300 Kg/cm^2

10 f y = 3500 Kg/cm^2

11 Unfactored bending moment (M) = 6.50 t.m.

12 Unfactored normal force (N) = 7.20 t "-ve sign for Compression Force"

13 Element Type = slab

14 **Step 1:Uncracked section analysis**

15 The Thickness $t=((M/(b*\psi))^{0.5}) \pm 3$

16 t = 45 cm

17 t chosen = 45 cm

18 fct(N) = N/Ac = 1.6 Kg/cm^2

19 fct(M) = $(6*M)/(b*t^2)$ = 19.26 Kg/cm^2

20 fct=fct(N)+fct(M) = **20.86** Kg/cm^2

21 tv=t*{1+(fct(N)/fct(M))} = 487.38 mm

22 η = 1.34369 Table (4-16) ECP 2017

23 f_{ctr}/η =(1.899*fcu^(1/2))/η = **24.48** Kg/cm^2 > **20.86** Kg/cm^2 **Satisfactory**

t_v	η
10	1
20	1
40	1
60	1

24 **Step 2:Cracked section analysis**

25 According to ECP 2017, the load factor for liquid containing structure is = 1.40 Clause [3-2-1-1-(3)]

26 factored bending moment (Mu) = 9.10 t.m.

27 factored normal force (Nu) = 10.08 t

28 According to Table (4-11) the structure is classified as class 3.For such a class,

29 Table (4-13) gives a minimum concrete cover of 25 mm.

30 The reinforcing bars used are = 12 mm diameter.

31 Concrete Cover = 2.50 cm

32 Effective Depth = 41.9 cm

33 $e=M_u/N_u$ = 90.2778 cm > t/2 = 22.5 cm ***Big eccentricity***

34 e_s=e-t/2+cover = 70.88 cm

35 $M_{us} = N_u * e_s$ = 7.14448 t.m.

36 For f_y = 3500 Table (4-15) is used to get the value of β_{cr}

37 βcr = 0.83

38 From $d=c_1*(M_{us}/ (b \times f_{cu}))$^0.5 get c1

39 c_1 = 8.5860 c/d = 0.0367

40 c/d_{min} = 0.125 c/d_{max} = 0.45

41 c/d = 0.125 j = 0.826 ***Satisfactory***

42 $As = M_{us} / (\beta_{cr} \times f_y \times j \times d) \pm N_u*\gamma_s/ (\beta_{cr} \times fy)$

43 As = 11.0957 cm^2 / m. $\mu_{min.}$ = 0.244 % Clause (4-3-2-7- هـ)

44 As_{min} (at tension side) of slab = 10.21 cm^2 / m.

45

46 **Use 10 Ø 12 /m'**

Sec. 2-2

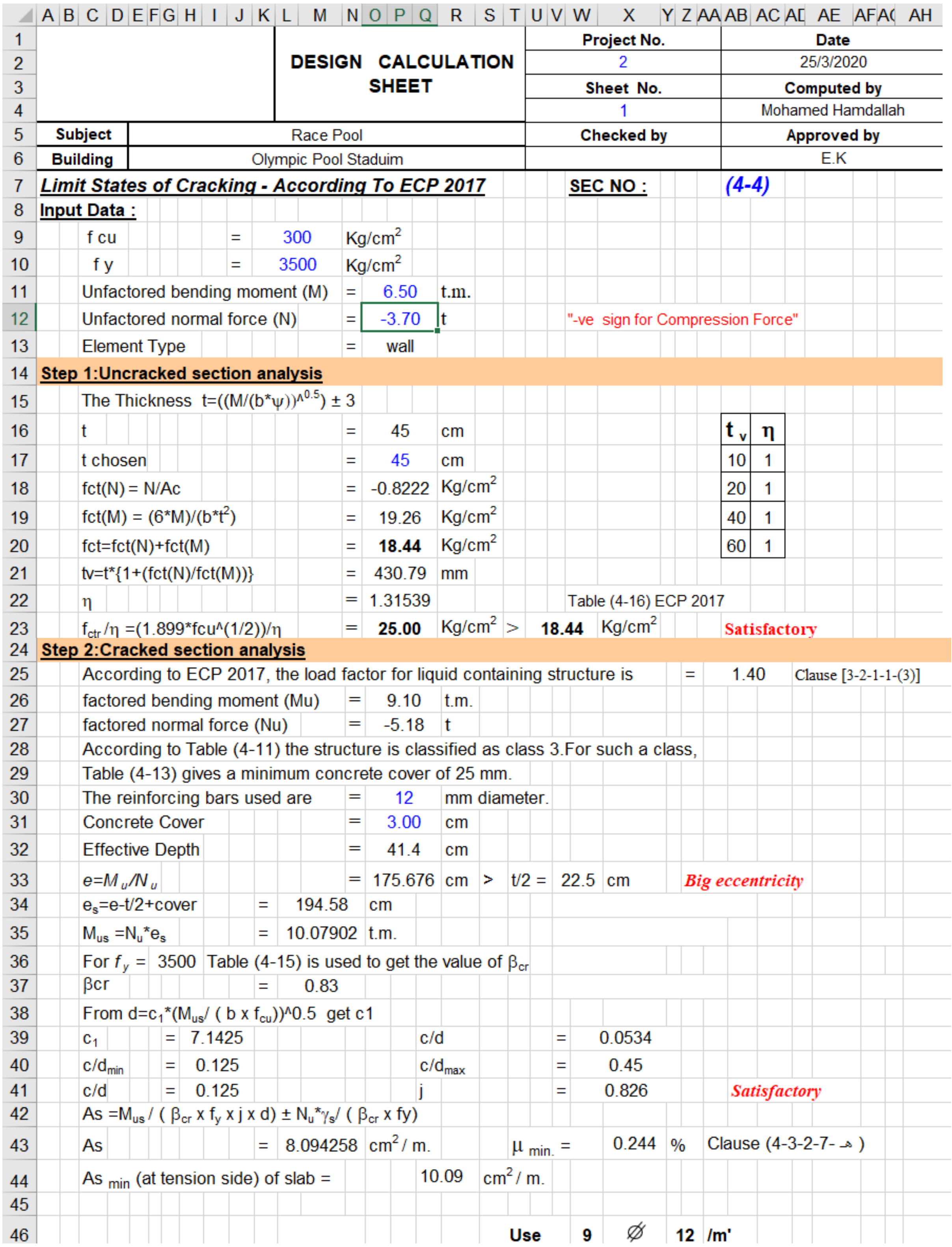
t_{mm} = 0.88 * 75 = 63.75mm …………… take t = 150 mm

$A_s = \frac{75*1000}{0.85*(\frac{360}{1.15})}$ = 282 mm^2 ……………….assume using T 12

Use 5 T 12 / m’

- **Maintenance Case**
    - Wall Design.

Sec. 4-4



	DESIGN CALCULATION SHEET	**Project No.**	**Date**
		2	25/3/2020
		Sheet No.	**Computed by**
		1	Mohamed Hamdallah
Subject	Race Pool	**Checked by**	**Approved by**
Building	Olympic Pool Staduim		E.K

Limit States of Cracking - According To ECP 2017 **SEC NO :** ***(4-4)***

Input Data :

f cu = 300 Kg/cm^2

f y = 3500 Kg/cm^2

Unfactored bending moment (M) = 6.50 t.m.

Unfactored normal force (N) = -3.70 t "-ve sign for Compression Force"

Element Type = wall

Step 1:Uncracked section analysis

The Thickness $t=((M/(b*\psi))^{0.5}) \pm 3$

t = 45 cm

t chosen = 45 cm

fct(N) = N/Ac = -0.8222 Kg/cm^2

fct(M) = (6*M)/(b*t^2) = 19.26 Kg/cm^2

fct=fct(N)+fct(M) = **18.44** Kg/cm^2

tv=t*{1+(fct(N)/fct(M))} = 430.79 mm

η = 1.31539 Table (4-16) ECP 2017

f_{ctr}/η =(1.899*fcu^(1/2))/η = **25.00** Kg/cm^2 > **18.44** Kg/cm^2 **Satisfactory**

t_v	η
10	1
20	1
40	1
60	1

Step 2:Cracked section analysis

According to ECP 2017, the load factor for liquid containing structure is = 1.40 Clause [3-2-1-1-(3)]

factored bending moment (Mu) = 9.10 t.m.

factored normal force (Nu) = -5.18 t

According to Table (4-11) the structure is classified as class 3.For such a class,

Table (4-13) gives a minimum concrete cover of 25 mm.

The reinforcing bars used are = 12 mm diameter.

Concrete Cover = 3.00 cm

Effective Depth = 41.4 cm

$e=M_u/N_u$ = 175.676 cm > t/2 = 22.5 cm ***Big eccentricity***

e_s=e-t/2+cover = 194.58 cm

$M_{us}=N_u*e_s$ = 10.07902 t.m.

For f_y = 3500 Table (4-15) is used to get the value of β_{cr}

βcr = 0.83

From $d=c_1*(M_{us}/(b \times f_{cu}))$^0.5 get c1

c_1 = 7.1425 c/d = 0.0534

c/d_{min} = 0.125 c/d_{max} = 0.45

c/d = 0.125 j = 0.826 ***Satisfactory***

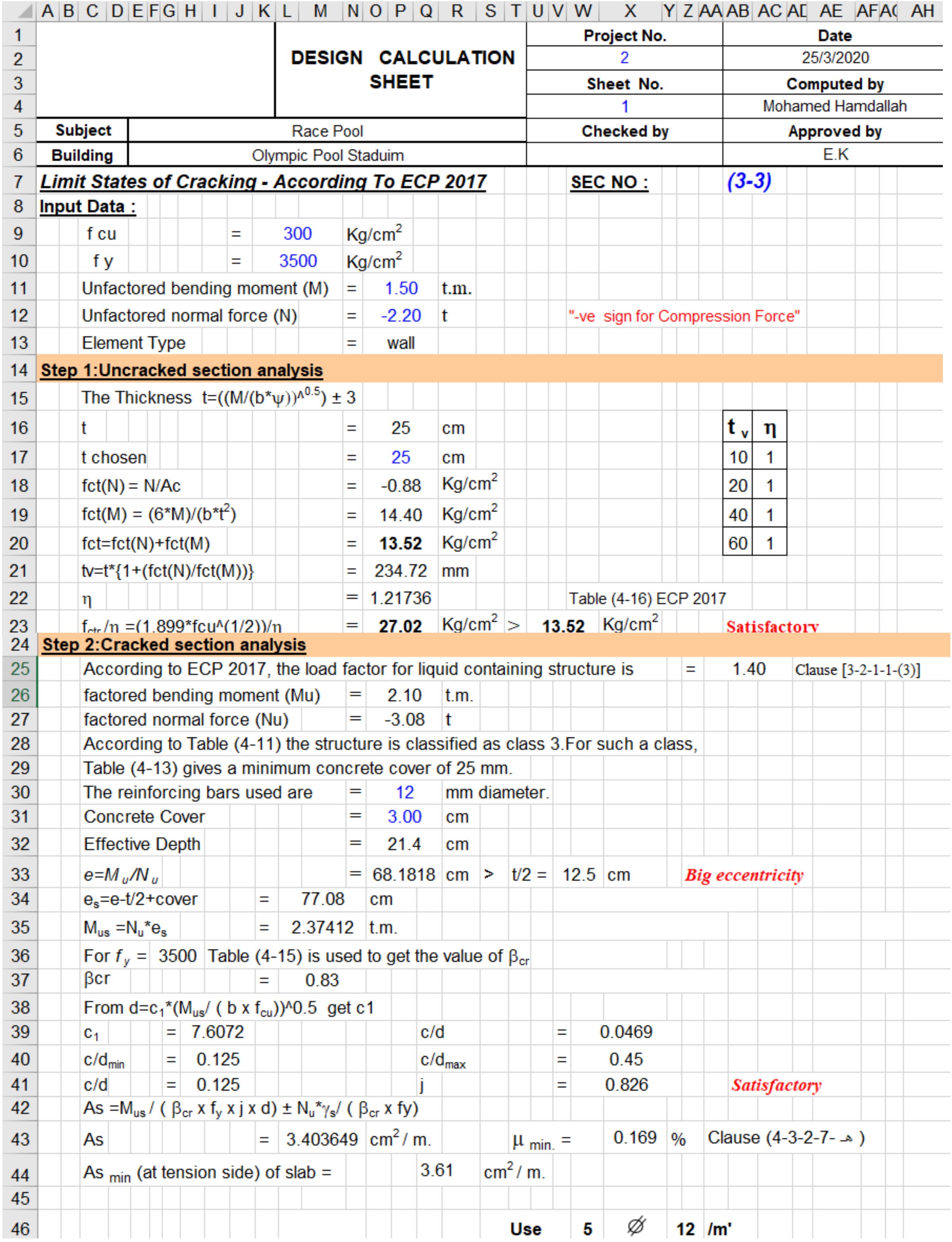
$As = M_{us}/(\beta_{cr} \times f_y \times j \times d) \pm N_u*\gamma_s/(\beta_{cr} \times fy)$

As = 8.094258 cm^2 / m. $\mu_{min.}$ = 0.244 % Clause (4-3-2-7- هـ)

As_{min} (at tension side) of slab = 10.09 cm^2 / m.

Use 9 Ø 12 /m'

Sec. 3-3



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	Sheet No.	**Computed by**	
	1	Mohamed Hamdallah	
Subject	Race Pool	**Checked by**	**Approved by**
Building	Olympic Pool Staduim		E.K

Limit States of Cracking - According To ECP 2017 **SEC NO :** ***(3-3)***

Input Data :

f cu = 300 Kg/cm^2

f y = 3500 Kg/cm^2

Unfactored bending moment (M) = 1.50 t.m.

Unfactored normal force (N) = -2.20 t "-ve sign for Compression Force"

Element Type = wall

Step 1:Uncracked section analysis

The Thickness $t=((M/(b*\psi))^{0.5}) \pm 3$

t = 25 cm

t chosen = 25 cm

fct(N) = N/Ac = -0.88 Kg/cm^2

fct(M) = $(6*M)/(b*t^2)$ = 14.40 Kg/cm^2

fct=fct(N)+fct(M) = **13.52** Kg/cm^2

tv=t*{1+(fct(N)/fct(M))} = 234.72 mm

η = 1.21736 Table (4-16) ECP 2017

f_{ctr}/η =(1.899*fcu^(1/2))/η = **27.02** Kg/cm^2 > **13.52** Kg/cm^2 **Satisfactory**

t_v	η
10	1
20	1
40	1
60	1

Step 2:Cracked section analysis

According to ECP 2017, the load factor for liquid containing structure is = 1.40 Clause [3-2-1-1-(3)]

factored bending moment (Mu) = 2.10 t.m.

factored normal force (Nu) = -3.08 t

According to Table (4-11) the structure is classified as class 3.For such a class,

Table (4-13) gives a minimum concrete cover of 25 mm.

The reinforcing bars used are = 12 mm diameter.

Concrete Cover = 3.00 cm

Effective Depth = 21.4 cm

$e=M_u/N_u$ = 68.1818 cm > t/2 = 12.5 cm ***Big eccentricity***

e_s=e-t/2+cover = 77.08 cm

$M_{us}=N_u*e_s$ = 2.37412 t.m.

For f_y = 3500 Table (4-15) is used to get the value of β_{cr}

βcr = 0.83

From $d=c_1*(M_{us}/(b \times f_{cu}))$^0.5 get c1

c_1 = 7.6072 c/d = 0.0469

c/d_{min} = 0.125 c/d_{max} = 0.45

c/d = 0.125 j = 0.826 ***Satisfactory***

$As = M_{us} / (\beta_{cr} \times f_y \times j \times d) \pm N_u*\gamma_s / (\beta_{cr} \times fy)$

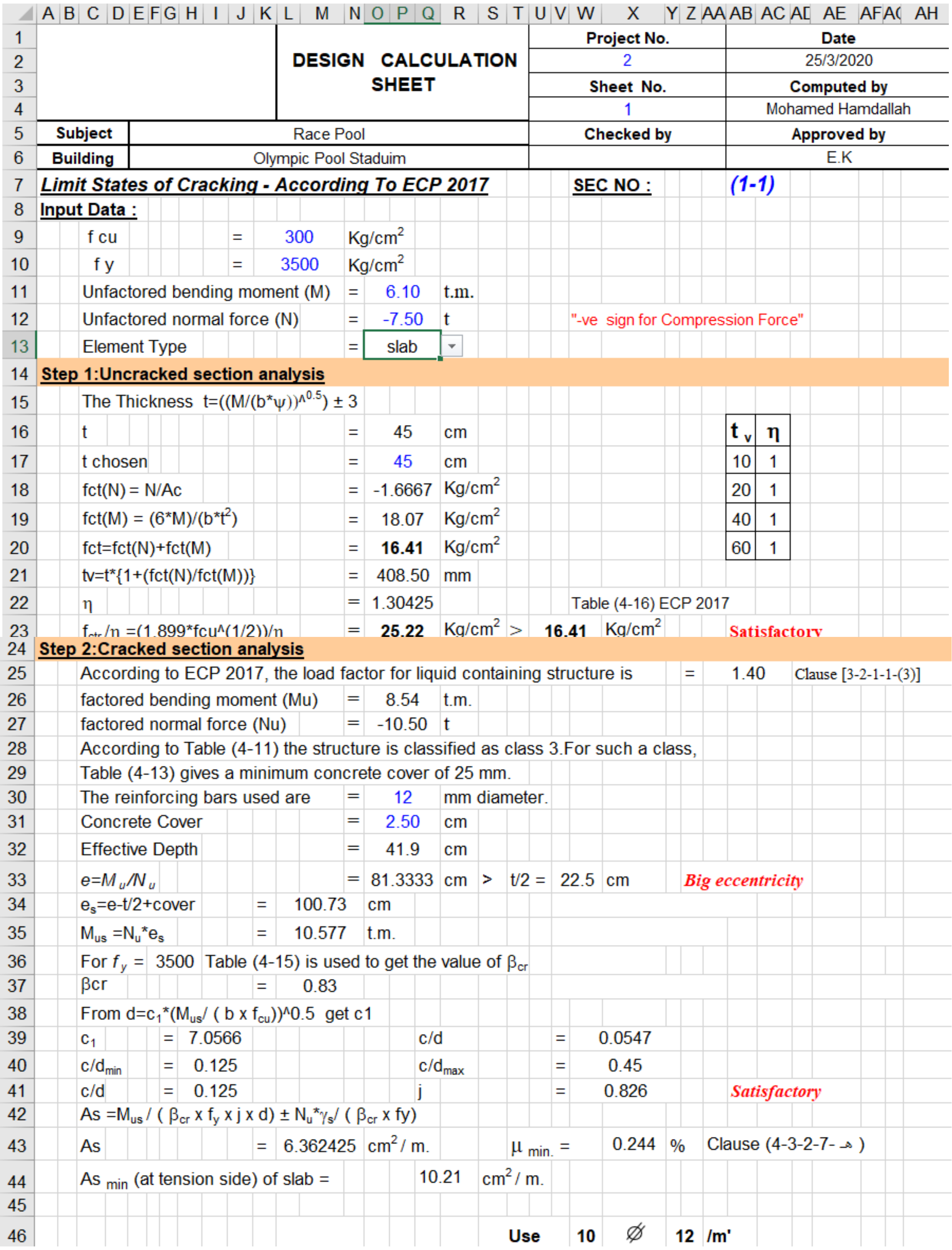
As = 3.403649 cm^2/ m. $\mu_{min.}$ = 0.169 % Clause (4-3-2-7- هـ)

As_{min} (at tension side) of slab = 3.61 cm^2/ m.

Use 5 Ø 12 /m'

- Floor Design.

Sec. 1-1



	DESIGN CALCULATION SHEET	**Project No.**	**Date**
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		1	Mohamed Hamdallah
Subject	Race Pool	**Checked by**	**Approved by**
Building	Olympic Pool Staduim		E.K

Limit States of Cracking - According To ECP 2017 **SEC NO :** ***(1-1)***

Input Data :

f cu = 300 Kg/cm^2

f y = 3500 Kg/cm^2

Unfactored bending moment (M) = 6.10 t.m.

Unfactored normal force (N) = -7.50 t "-ve sign for Compression Force"

Element Type = slab

Step 1:Uncracked section analysis

The Thickness t=((M/(b*ψ))^$^{0.5}$) ± 3

t = 45 cm

t chosen = 45 cm

fct(N) = N/Ac = -1.6667 Kg/cm^2

fct(M) = (6*M)/(b*t^2) = 18.07 Kg/cm^2

fct=fct(N)+fct(M) = **16.41** Kg/cm^2

tv=t*{1+(fct(N)/fct(M))} = 408.50 mm

η = 1.30425 Table (4-16) ECP 2017

f_{ctr}/η =(1.899*fcu^(1/2))/η = **25.22** Kg/cm^2 > **16.41** Kg/cm^2 **Satisfactory**

t $_v$	η
10	1
20	1
40	1
60	1

Step 2:Cracked section analysis

According to ECP 2017, the load factor for liquid containing structure is = 1.40 Clause [3-2-1-1-(3)]

factored bending moment (Mu) = 8.54 t.m.

factored normal force (Nu) = -10.50 t

According to Table (4-11) the structure is classified as class 3.For such a class,

Table (4-13) gives a minimum concrete cover of 25 mm.

The reinforcing bars used are = 12 mm diameter.

Concrete Cover = 2.50 cm

Effective Depth = 41.9 cm

$e=M_u/N_u$ = 81.3333 cm > t/2 = 22.5 cm ***Big eccentricity***

e_s=e-t/2+cover = 100.73 cm

M_{us} =N_u*e_s = 10.577 t.m.

For f_y = 3500 Table (4-15) is used to get the value of β_{cr}

βcr = 0.83

From d=c_1*(M_{us}/ (b x f_{cu}))^0.5 get c1

c_1 = 7.0566 c/d = 0.0547

c/d_{min} = 0.125 c/d_{max} = 0.45

c/d = 0.125 j = 0.826 ***Satisfactory***

As =M_{us} / (β_{cr} x f_y x j x d) ± N_u*γ_s/ (β_{cr} x fy)

As = 6.362425 cm^2 / m. µ min. = 0.244 % Clause (4-3-2-7- هـ)

As $_{min}$ (at tension side) of slab = 10.21 cm^2 / m.

Use 10 ∅ 12 /m'

Sec. 2-2

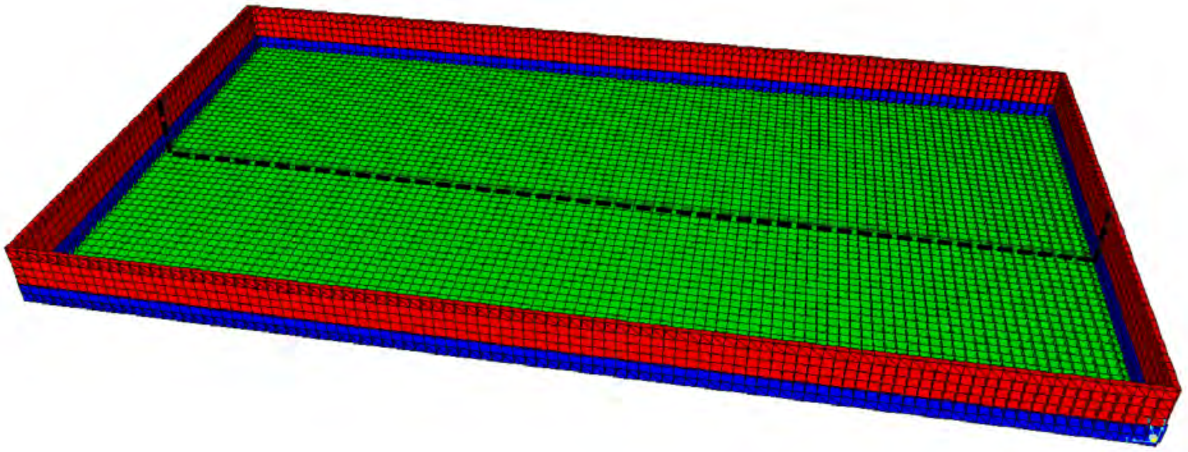
t_{mm} = 0.88 * 75 = 63.75mm …………… take t = 150 mm

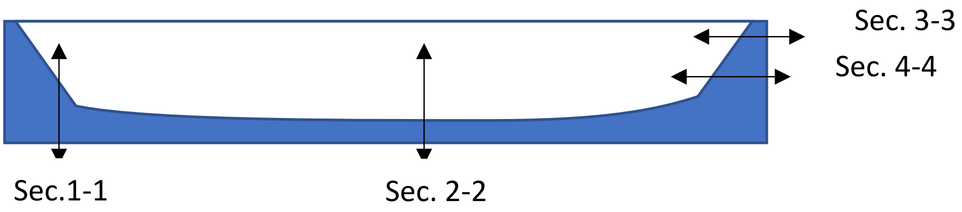
assume using 5 T 12 as section reinforcement.

Pu = 0.33 * 30 * (1000 * 150) + 0.67 * $\frac{0.85*360}{1.15}$ * 577 = 1586 KN > 75 KN (Safe)

Use 5 T 12 / m’

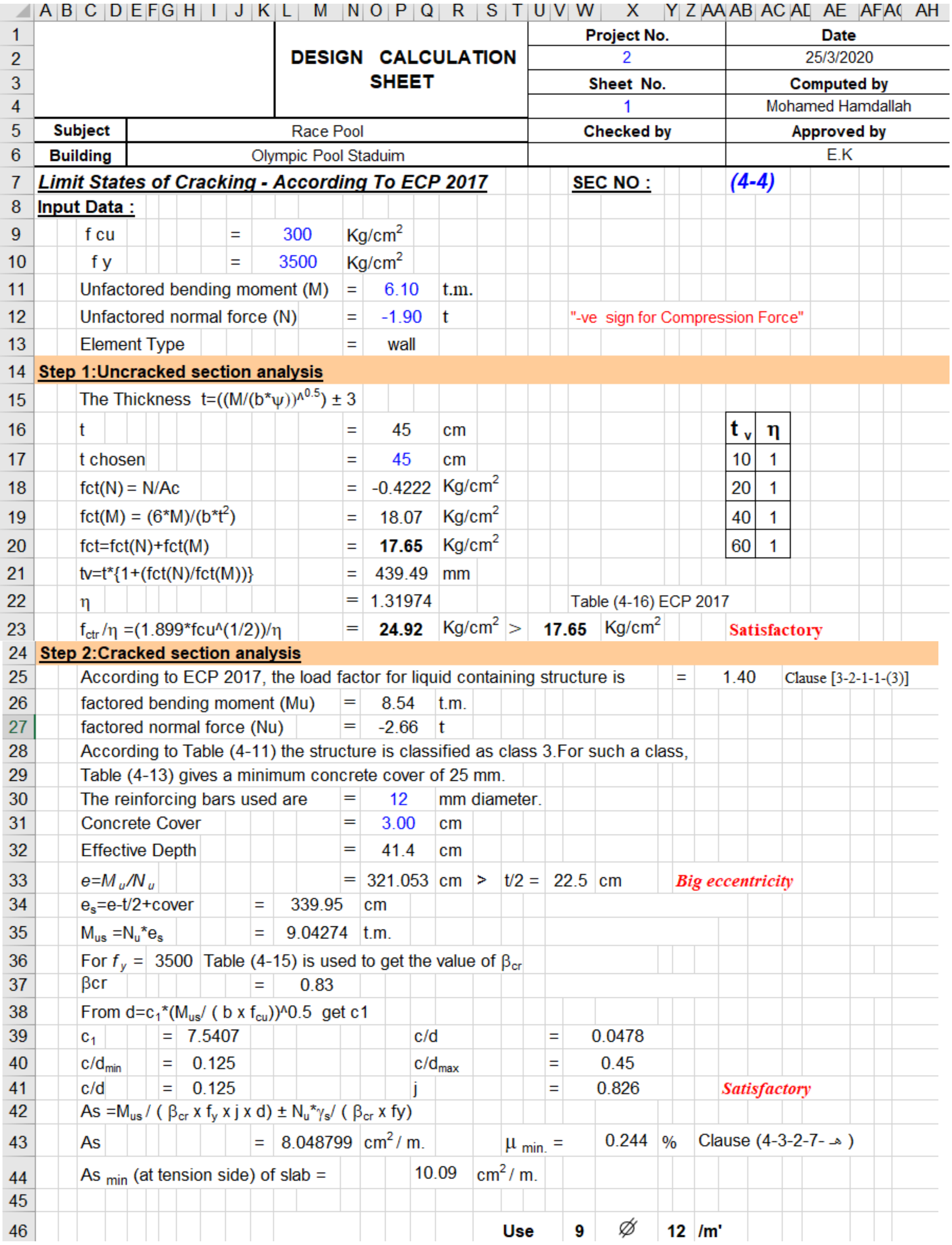
8.3.2 Long direction strip.





- **Testing Case**
  - Wall Design.

Sec. 4-4



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		1	Mohamed Hamdallah
Subject	Race Pool	**Checked by**	**Approved by**
Building	Olympic Pool Staduim		E.K

Limit States of Cracking - According To ECP 2017 **SEC NO :** *(4-4)*

Input Data :

f cu = 300 Kg/cm^2

f y = 3500 Kg/cm^2

Unfactored bending moment (M) = 6.10 t.m.

Unfactored normal force (N) = -1.90 t "-ve sign for Compression Force"

Element Type = wall

Step 1:Uncracked section analysis

The Thickness $t=((M/(b*\psi))^{0.5}) \pm 3$

t = 45 cm

t chosen = 45 cm

fct(N) = N/Ac = -0.4222 Kg/cm^2

fct(M) = $(6*M)/(b*t^2)$ = 18.07 Kg/cm^2

fct=fct(N)+fct(M) = **17.65** Kg/cm^2

tv=t*{1+(fct(N)/fct(M))} = 439.49 mm

η = 1.31974 Table (4-16) ECP 2017

f_{ctr}/η =(1.899*fcu^(1/2))/η = **24.92** Kg/cm^2 > **17.65** Kg/cm^2 **Satisfactory**

t_v	η
10	1
20	1
40	1
60	1

Step 2:Cracked section analysis

According to ECP 2017, the load factor for liquid containing structure is = 1.40 Clause [3-2-1-1-(3)]

factored bending moment (Mu) = 8.54 t.m.

factored normal force (Nu) = -2.66 t

According to Table (4-11) the structure is classified as class 3.For such a class,

Table (4-13) gives a minimum concrete cover of 25 mm.

The reinforcing bars used are = 12 mm diameter.

Concrete Cover = 3.00 cm

Effective Depth = 41.4 cm

$e=M_u/N_u$ = 321.053 cm > t/2 = 22.5 cm *Big eccentricity*

e_s=e-t/2+cover = 339.95 cm

$M_{us}=N_u*e_s$ = 9.04274 t.m.

For f_y = 3500 Table (4-15) is used to get the value of β_{cr}

βcr = 0.83

From $d=c_1*(M_{us}/(b \times f_{cu}))$^0.5 get c1

c_1 = 7.5407 c/d = 0.0478

c/d_{min} = 0.125 c/d_{max} = 0.45

c/d = 0.125 j = 0.826 *Satisfactory*

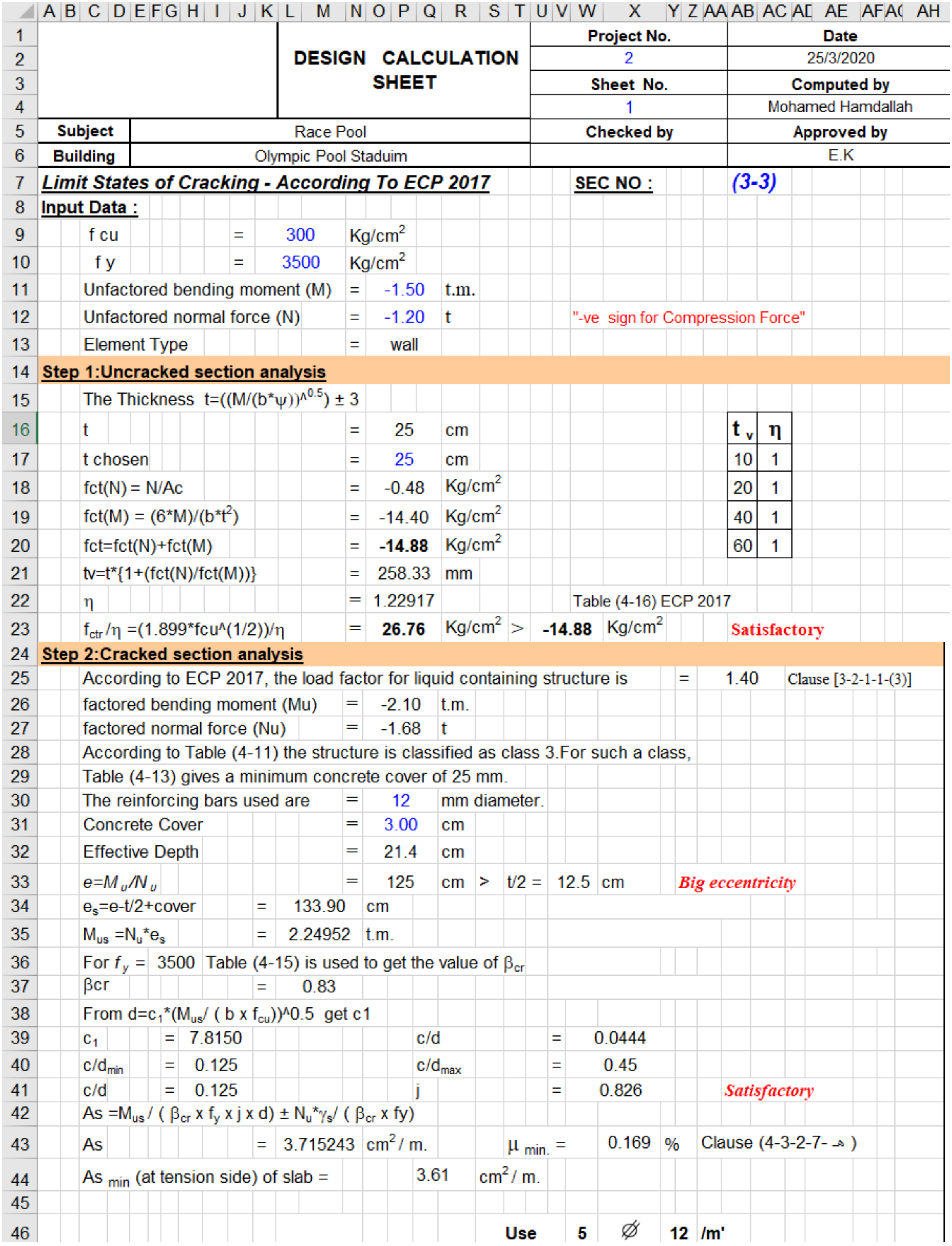
As $=M_{us}/(\beta_{cr} \times f_y \times j \times d) \pm N_u*\gamma_s/(\beta_{cr} \times fy)$

As = 8.048799 cm^2/ m. $\mu_{min.}$ = 0.244 % Clause (4-3-2-7- هـ)

As_{min} (at tension side) of slab = 10.09 cm^2/ m.

Use 9 ∅ 12 /m'

Sec. 3-3



	DESIGN CALCULATION SHEET	**Project No.**	**Date**
		2	25/3/2020
		Sheet No.	**Computed by**
		1	Mohamed Hamdallah
Subject	Race Pool	**Checked by**	**Approved by**
Building	Olympic Pool Staduim		E.K

Limit States of Cracking - According To ECP 2017 **SEC NO :** *(3-3)*

Input Data :

f cu = 300 Kg/cm^2

f y = 3500 Kg/cm^2

Unfactored bending moment (M) = -1.50 t.m.

Unfactored normal force (N) = -1.20 t "-ve sign for Compression Force"

Element Type = wall

Step 1:Uncracked section analysis

The Thickness $t=((M/(b*\psi))^{0.5}) \pm 3$

t = 25 cm

t chosen = 25 cm

fct(N) = N/Ac = -0.48 Kg/cm^2

$fct(M) = (6*M)/(b*t^2)$ = -14.40 Kg/cm^2

fct=fct(N)+fct(M) = **-14.88** Kg/cm^2

tv=t*{1+(fct(N)/fct(M))} = 258.33 mm

η = 1.22917 Table (4-16) ECP 2017

f_{ctr}/η =(1.899*fcu^(1/2))/η = **26.76** Kg/cm^2 > **-14.88** Kg/cm^2 Satisfactory

t_v	η
10	1
20	1
40	1
60	1

Step 2:Cracked section analysis

According to ECP 2017, the load factor for liquid containing structure is = 1.40 Clause [3-2-1-1-(3)]

factored bending moment (Mu) = -2.10 t.m.

factored normal force (Nu) = -1.68 t

According to Table (4-11) the structure is classified as class 3.For such a class,

Table (4-13) gives a minimum concrete cover of 25 mm.

The reinforcing bars used are = 12 mm diameter.

Concrete Cover = 3.00 cm

Effective Depth = 21.4 cm

$e=M_u/N_u$ = 125 cm > t/2 = 12.5 cm *Big eccentricity*

e_s=e-t/2+cover = 133.90 cm

$M_{us}=N_u*e_s$ = 2.24952 t.m.

For f_y = 3500 Table (4-15) is used to get the value of β_{cr}

βcr = 0.83

From $d=c_1*(M_{us}/(b \times f_{cu}))$^0.5 get c1

c_1 = 7.8150 c/d = 0.0444

c/d_{min} = 0.125 c/d_{max} = 0.45

c/d = 0.125 j = 0.826 *Satisfactory*

$As = M_{us}/(\beta_{cr} \times f_y \times j \times d) \pm N_u*\gamma_s/(\beta_{cr} \times fy)$

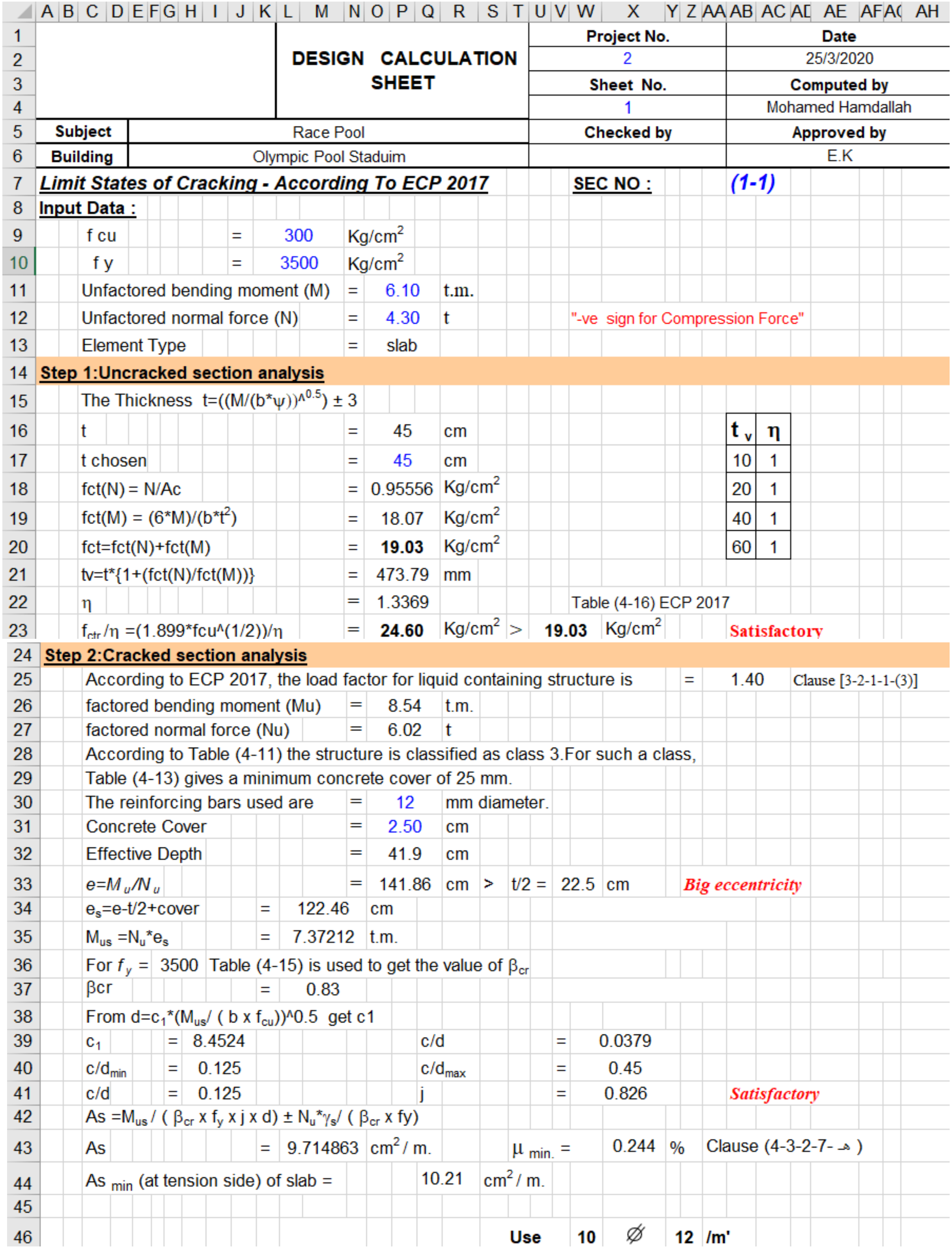
As = 3.715243 cm^2 / m. $\mu_{min.}$ = 0.169 % Clause (4-3-2-7- هـ)

As_{min} (at tension side) of slab = 3.61 cm^2 / m.

Use 5 ∅ 12 /m'

- Floor Design.

Sec. 1-1



DESIGN CALCULATION SHEET		**Project No.**	**Date**
		2	25/3/2020
		Sheet No.	**Computed by**
		1	Mohamed Hamdallah
Subject	Race Pool	**Checked by**	**Approved by**
Building	Olympic Pool Staduim		E.K

Limit States of Cracking - According To ECP 2017 **SEC NO :** *(1-1)*

Input Data :

f cu = 300 Kg/cm^2

f y = 3500 Kg/cm^2

Unfactored bending moment (M) = 6.10 t.m.

Unfactored normal force (N) = 4.30 t "-ve sign for Compression Force"

Element Type = slab

Step 1:Uncracked section analysis

The Thickness $t=((M/(b*\psi))^{\wedge 0.5}) \pm 3$

t = 45 cm

t chosen = 45 cm

fct(N) = N/Ac = 0.95556 Kg/cm^2

fct(M) = $(6*M)/(b*t^2)$ = 18.07 Kg/cm^2

fct=fct(N)+fct(M) = **19.03** Kg/cm^2

tv=t*{1+(fct(N)/fct(M))} = 473.79 mm

η = 1.3369 Table (4-16) ECP 2017

f_{ctr}/η =(1.899*fcu^(1/2))/η = **24.60** Kg/cm^2 > **19.03** Kg/cm^2 **Satisfactory**

t_v	η
10	1
20	1
40	1
60	1

Step 2:Cracked section analysis

According to ECP 2017, the load factor for liquid containing structure is = 1.40 Clause [3-2-1-1-(3)]

factored bending moment (Mu) = 8.54 t.m.

factored normal force (Nu) = 6.02 t

According to Table (4-11) the structure is classified as class 3.For such a class,

Table (4-13) gives a minimum concrete cover of 25 mm.

The reinforcing bars used are = 12 mm diameter.

Concrete Cover = 2.50 cm

Effective Depth = 41.9 cm

$e=M_u/N_u$ = 141.86 cm > t/2 = 22.5 cm *Big eccentricity*

e_s=e-t/2+cover = 122.46 cm

$M_{us}=N_u*e_s$ = 7.37212 t.m.

For f_y = 3500 Table (4-15) is used to get the value of β_{cr}

βcr = 0.83

From $d=c_1*(M_{us}/(b \times f_{cu}))$^0.5 get c1

c_1 = 8.4524 c/d = 0.0379

c/d_{min} = 0.125 c/d_{max} = 0.45

c/d = 0.125 j = 0.826 *Satisfactory*

$As = M_{us} / (\beta_{cr} \times f_y \times j \times d) \pm N_u*\gamma_s/(\beta_{cr} \times fy)$

As = 9.714863 cm^2 / m. $\mu_{min.}$ = 0.244 % Clause (4-3-2-7- هـ)

As $_{min}$ (at tension side) of slab = 10.21 cm^2 / m.

Use 10 Ø 12 /m'

Sec. 2-2

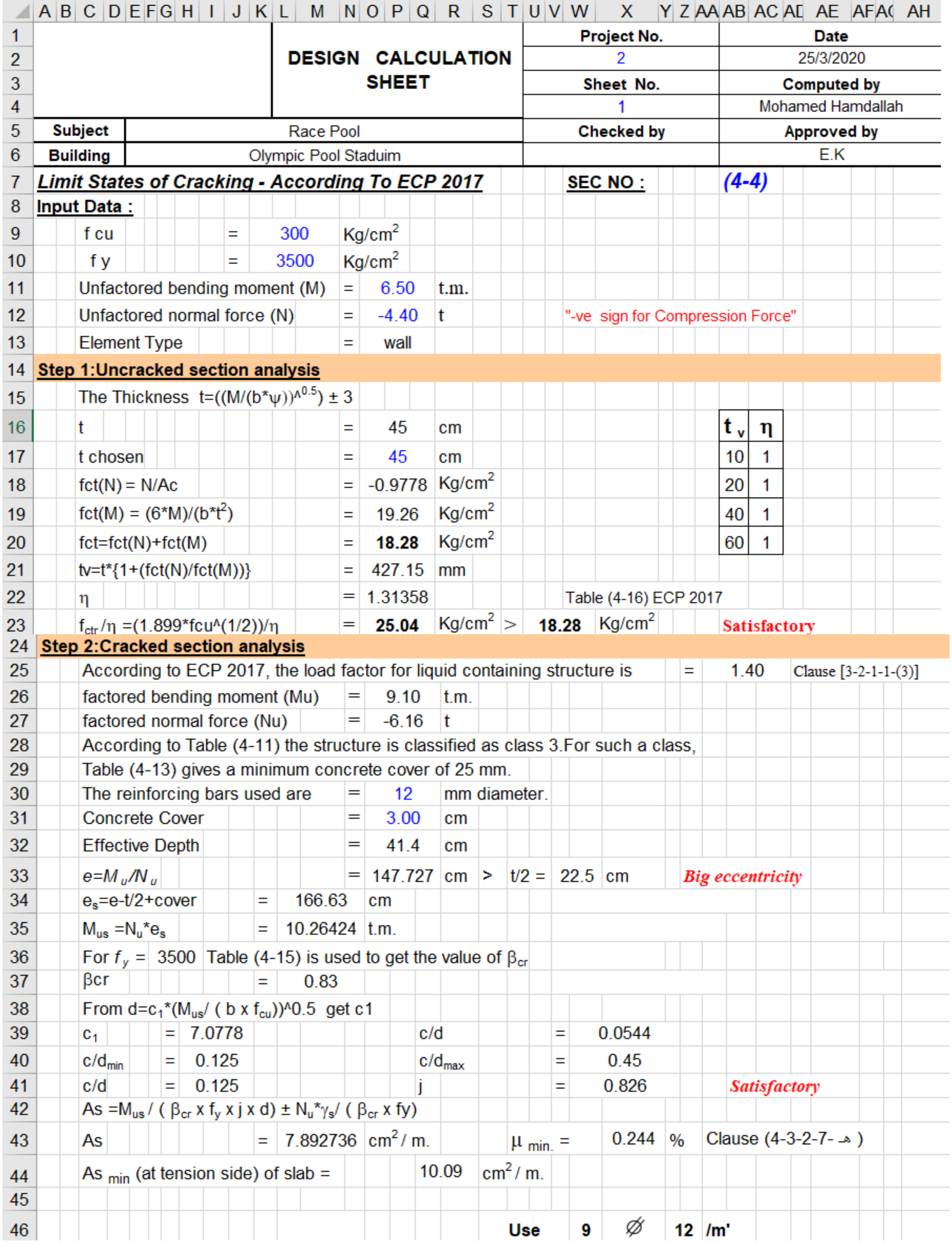
t_{mm} = 0.88 * 75 = 63.75mm …………… take t = 150 mm

$A_s = \frac{75*1000}{0.85*(\frac{360}{1.15})}$ = 282 mm^2 ……………….assume using T 12

Use 5 T 12 / m’

- **Maintenance Case**
  - Wall Design.

Sec. 4-4



	DESIGN CALCULATION SHEET	**Project No.**	**Date**
		2	25/3/2020
		Sheet No.	**Computed by**
		1	Mohamed Hamdallah
Subject	Race Pool	**Checked by**	**Approved by**
Building	Olympic Pool Staduim		E.K

7 ***Limit States of Cracking - According To ECP 2017*** **SEC NO :** ***(4-4)***

8 **Input Data :**

9 f cu = 300 Kg/cm^2

10 f y = 3500 Kg/cm^2

11 Unfactored bending moment (M) = 6.50 t.m.

12 Unfactored normal force (N) = -4.40 t "-ve sign for Compression Force"

13 Element Type = wall

14 **Step 1:Uncracked section analysis**

15 The Thickness t=((M/(b*ψ))^$^{0.5}$) ± 3

16 t = 45 cm

17 t chosen = 45 cm

18 fct(N) = N/Ac = -0.9778 Kg/cm^2

19 fct(M) = (6*M)/(b*t^2) = 19.26 Kg/cm^2

20 fct=fct(N)+fct(M) = **18.28** Kg/cm^2

t_v	**η**
10	1
20	1
40	1
60	1

21 tv=t*{1+(fct(N)/fct(M))} = 427.15 mm

22 η = 1.31358 Table (4-16) ECP 2017

23 f_{ctr}/η =(1.899*fcu^(1/2))/η = **25.04** Kg/cm^2 > **18.28** Kg/cm^2 **Satisfactory**

24 **Step 2:Cracked section analysis**

25 According to ECP 2017, the load factor for liquid containing structure is = 1.40 Clause [3-2-1-1-(3)]

26 factored bending moment (Mu) = 9.10 t.m.

27 factored normal force (Nu) = -6.16 t

28 According to Table (4-11) the structure is classified as class 3.For such a class,

29 Table (4-13) gives a minimum concrete cover of 25 mm.

30 The reinforcing bars used are = 12 mm diameter.

31 Concrete Cover = 3.00 cm

32 Effective Depth = 41.4 cm

33 $e=M_u/N_u$ = 147.727 cm > t/2 = 22.5 cm ***Big eccentricity***

34 e_s=e-t/2+cover = 166.63 cm

35 $M_{us}=N_u*e_s$ = 10.26424 t.m.

36 For f_y = 3500 Table (4-15) is used to get the value of β_{cr}

37 βcr = 0.83

38 From d=c_1*(M_{us}/ (b x f_{cu}))^0.5 get c1

39 c_1 = 7.0778 c/d = 0.0544

40 c/d_{min} = 0.125 c/d_{max} = 0.45

41 c/d = 0.125 j = 0.826 ***Satisfactory***

42 As =M_{us} / (β_{cr} x f_y x j x d) ± $N_u*\gamma_s$/ (β_{cr} x fy)

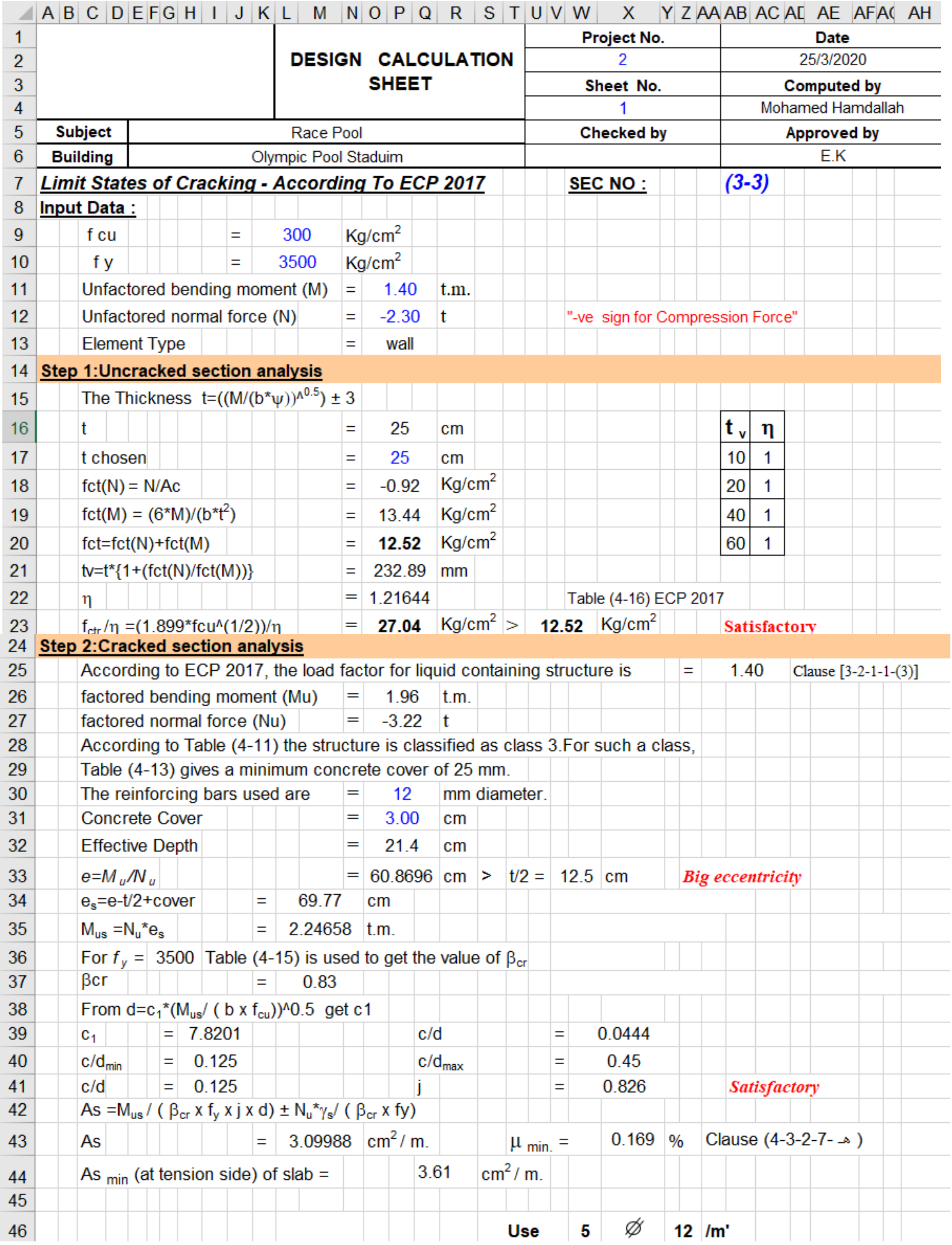
43 As = 7.892736 cm^2/ m. $\mu_{min.}$ = 0.244 % Clause (4-3-2-7- هـ)

44 As $_{min}$ (at tension side) of slab = 10.09 cm^2/ m.

45

46 **Use 9 ∅ 12 /m'**

Sec. 3-3



	DESIGN CALCULATION SHEET	**Project No.**	**Date**
		2	25/3/2020
		Sheet No.	**Computed by**
		1	Mohamed Hamdallah
Subject	Race Pool	**Checked by**	**Approved by**
Building	Olympic Pool Staduim		E.K

Limit States of Cracking - According To ECP 2017 **SEC NO :** *(3-3)*

Input Data :

f cu = 300 Kg/cm^2

f y = 3500 Kg/cm^2

Unfactored bending moment (M) = 1.40 t.m.

Unfactored normal force (N) = -2.30 t "-ve sign for Compression Force"

Element Type = wall

Step 1:Uncracked section analysis

The Thickness t=((M/(b*ψ))^$^{0.5}$) ± 3

t = 25 cm

t chosen = 25 cm

fct(N) = N/Ac = -0.92 Kg/cm^2

fct(M) = (6*M)/(b*t^2) = 13.44 Kg/cm^2

fct=fct(N)+fct(M) = **12.52** Kg/cm^2

tv=t*{1+(fct(N)/fct(M))} = 232.89 mm

η = 1.21644 Table (4-16) ECP 2017

f_{ctr}/η =(1.899*fcu^(1/2))/η = **27.04** Kg/cm^2 > **12.52** Kg/cm^2 **Satisfactory**

t_v	η
10	1
20	1
40	1
60	1

Step 2:Cracked section analysis

According to ECP 2017, the load factor for liquid containing structure is = 1.40 Clause [3-2-1-1-(3)]

factored bending moment (Mu) = 1.96 t.m.

factored normal force (Nu) = -3.22 t

According to Table (4-11) the structure is classified as class 3.For such a class,

Table (4-13) gives a minimum concrete cover of 25 mm.

The reinforcing bars used are = 12 mm diameter.

Concrete Cover = 3.00 cm

Effective Depth = 21.4 cm

$e=M_u/N_u$ = 60.8696 cm > t/2 = 12.5 cm ***Big eccentricity***

e_s=e-t/2+cover = 69.77 cm

$M_{us}=N_u*e_s$ = 2.24658 t.m.

For f_y = 3500 Table (4-15) is used to get the value of β_{cr}

βcr = 0.83

From d=c_1*(M_{us}/ (b x f_{cu}))^0.5 get c1

c_1 = 7.8201 c/d = 0.0444

c/d_{min} = 0.125 c/d_{max} = 0.45

c/d = 0.125 j = 0.826 ***Satisfactory***

As =M_{us} / (β_{cr} x f_y x j x d) ± N_u*γ_s/ (β_{cr} x fy)

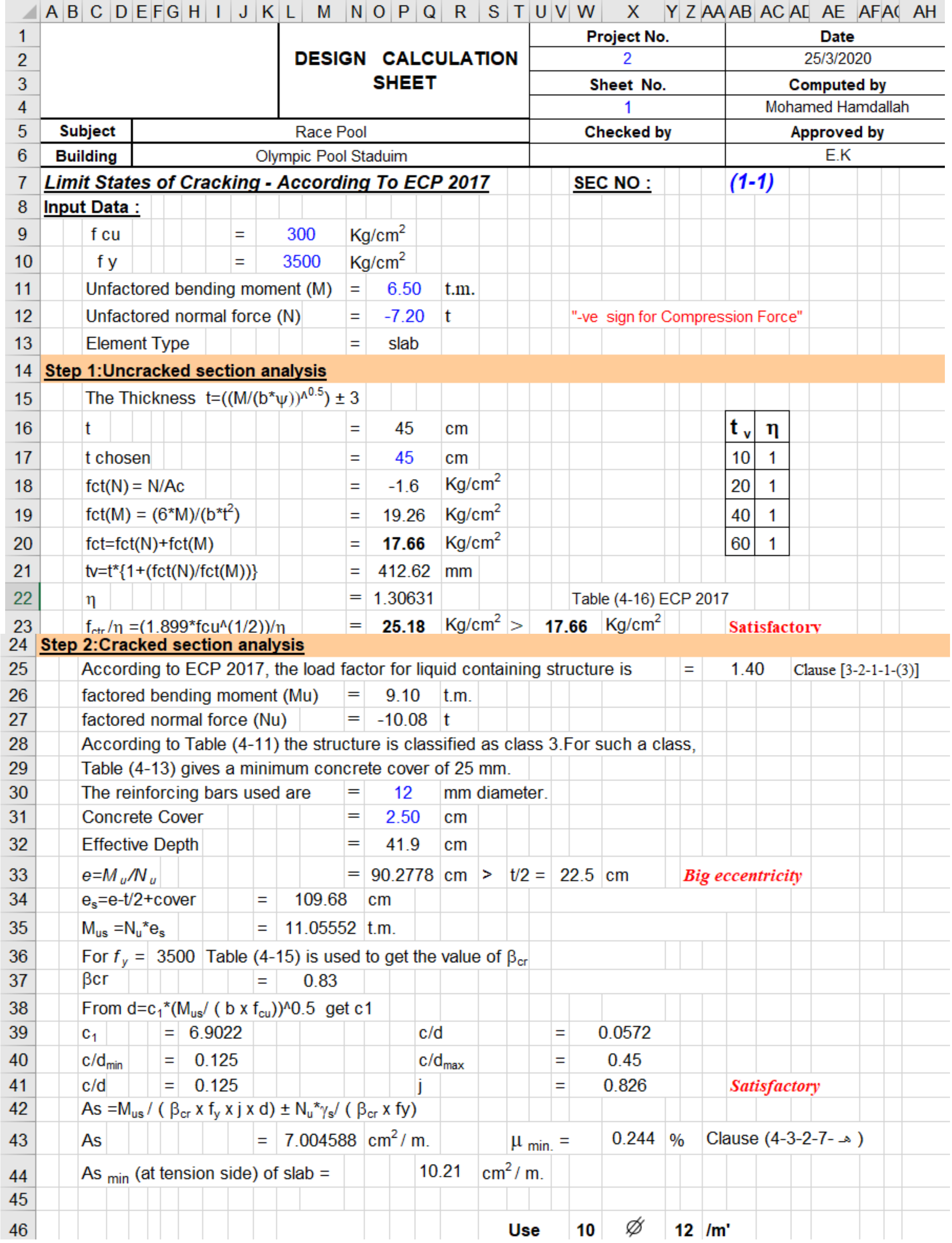
As = 3.09988 cm^2 / m. $\mu_{min.}$ = 0.169 % Clause (4-3-2-7- هـ)

As $_{min}$ (at tension side) of slab = 3.61 cm^2 / m.

Use 5 ∅ 12 /m'

- Floor Design.

Sec. 1-1



	DESIGN CALCULATION SHEET	Project No.	Date
		2	25/3/2020
		Sheet No.	Computed by
		1	Mohamed Hamdallah
Subject	Race Pool	Checked by	Approved by
Building	Olympic Pool Staduim		E.K

Limit States of Cracking - According To ECP 2017 **SEC NO :** ***(1-1)***

Input Data :

f cu = 300 Kg/cm^2

f y = 3500 Kg/cm^2

Unfactored bending moment (M) = 6.50 t.m.

Unfactored normal force (N) = -7.20 t "-ve sign for Compression Force"

Element Type = slab

Step 1:Uncracked section analysis

The Thickness $t=((M/(b*\psi))^{0.5}) \pm 3$

t_v	η
10	1
20	1
40	1
60	1

t = 45 cm

t chosen = 45 cm

fct(N) = N/Ac = -1.6 Kg/cm^2

fct(M) = $(6*M)/(b*t^2)$ = 19.26 Kg/cm^2

fct=fct(N)+fct(M) = **17.66** Kg/cm^2

tv=t*{1+(fct(N)/fct(M))} = 412.62 mm

η = 1.30631 Table (4-16) ECP 2017

f_{ctr}/η =(1.899*fcu^(1/2))/η = **25.18** Kg/cm^2 > **17.66** Kg/cm^2 **Satisfactory**

Step 2:Cracked section analysis

According to ECP 2017, the load factor for liquid containing structure is = 1.40 Clause [3-2-1-1-(3)]

factored bending moment (Mu) = 9.10 t.m.

factored normal force (Nu) = -10.08 t

According to Table (4-11) the structure is classified as class 3.For such a class,

Table (4-13) gives a minimum concrete cover of 25 mm.

The reinforcing bars used are = 12 mm diameter.

Concrete Cover = 2.50 cm

Effective Depth = 41.9 cm

$e=M_u/N_u$ = 90.2778 cm > t/2 = 22.5 cm ***Big eccentricity***

e_s=e-t/2+cover = 109.68 cm

$M_{us}=N_u*e_s$ = 11.05552 t.m.

For f_y = 3500 Table (4-15) is used to get the value of β_{cr}

βcr = 0.83

From $d=c_1*(M_{us}/(b \times f_{cu}))$^0.5 get c1

c_1 = 6.9022 c/d = 0.0572

c/d_{min} = 0.125 c/d_{max} = 0.45

c/d = 0.125 j = 0.826 ***Satisfactory***

$As = M_{us}/(\beta_{cr} \times f_y \times j \times d) \pm N_u*\gamma_s/(\beta_{cr} \times fy)$

As = 7.004588 cm^2 / m. $\mu_{min.}$ = 0.244 % Clause (4-3-2-7- هـ)

As_{min} (at tension side) of slab = 10.21 cm^2 / m.

Use 10 ∅ 12 /m'

Sec. 2-2

t_{mm} = 0.88 * 75 = 63.75mm …………… take t = 150 mm

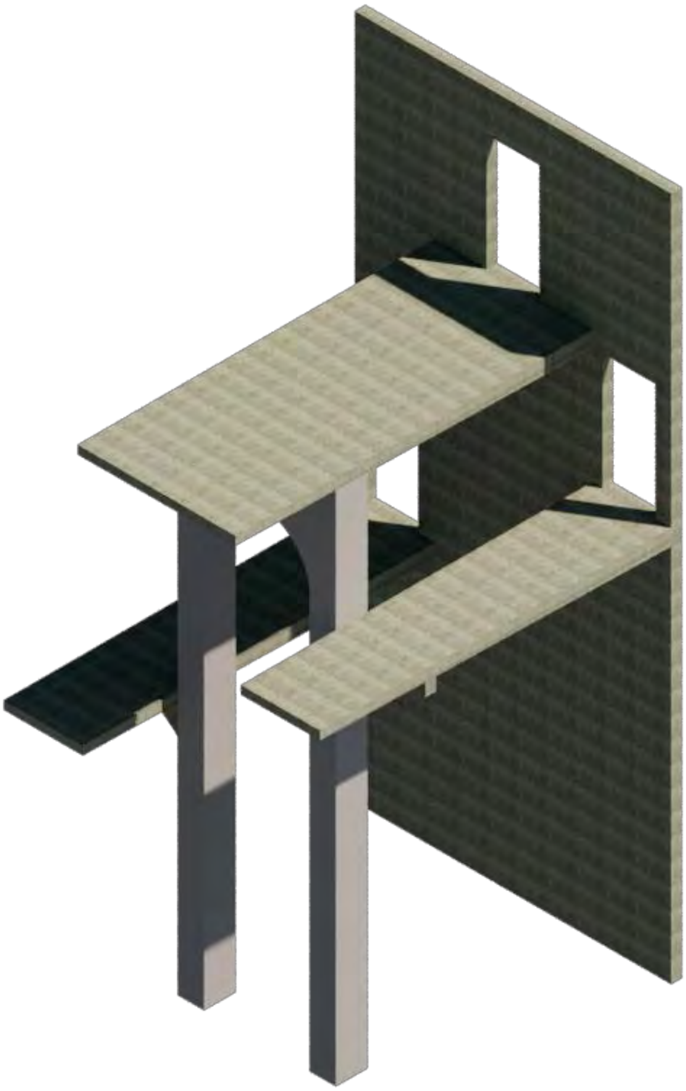
assume using 5 T 12 as section reinforcement.

Pu = 0.33 * 30 * (1000 * 150) + 0.67 * $\frac{0.85*360}{1.15}$ * 577 = 1586 KN > 75 KN (Safe)

Use 5 T 12 / m’

Egyptian Aquatic Centre

Diving Platforms



I. Design codes and standards

9- ECP (203–2018)

Egyptian Code of Practice for Design and Construction of Concrete Structures.

10- ECP (201-2010)

Egyptian Code for Loading on Buildings.

11- FINA FACILITIES RULES (2017-2021)

International Swimming Federation.

1. INTRODUCTION

The concept for structural design focuses on satisfying both the functional and the economic requirements of the building without jeopardizing its aesthetic and architectural features. This Calculation Report presents the structural engineering aspect of the works due for the development construction work of Egyptain Aquatic Centre.

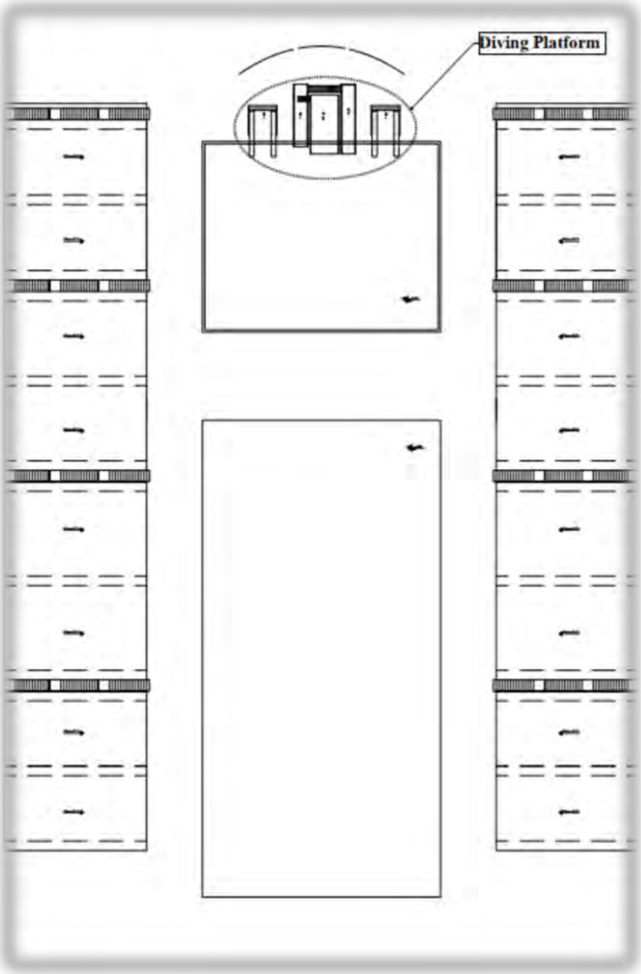


Figure 1-3 Diving Platform location

2. Diving Platform Drawings Details

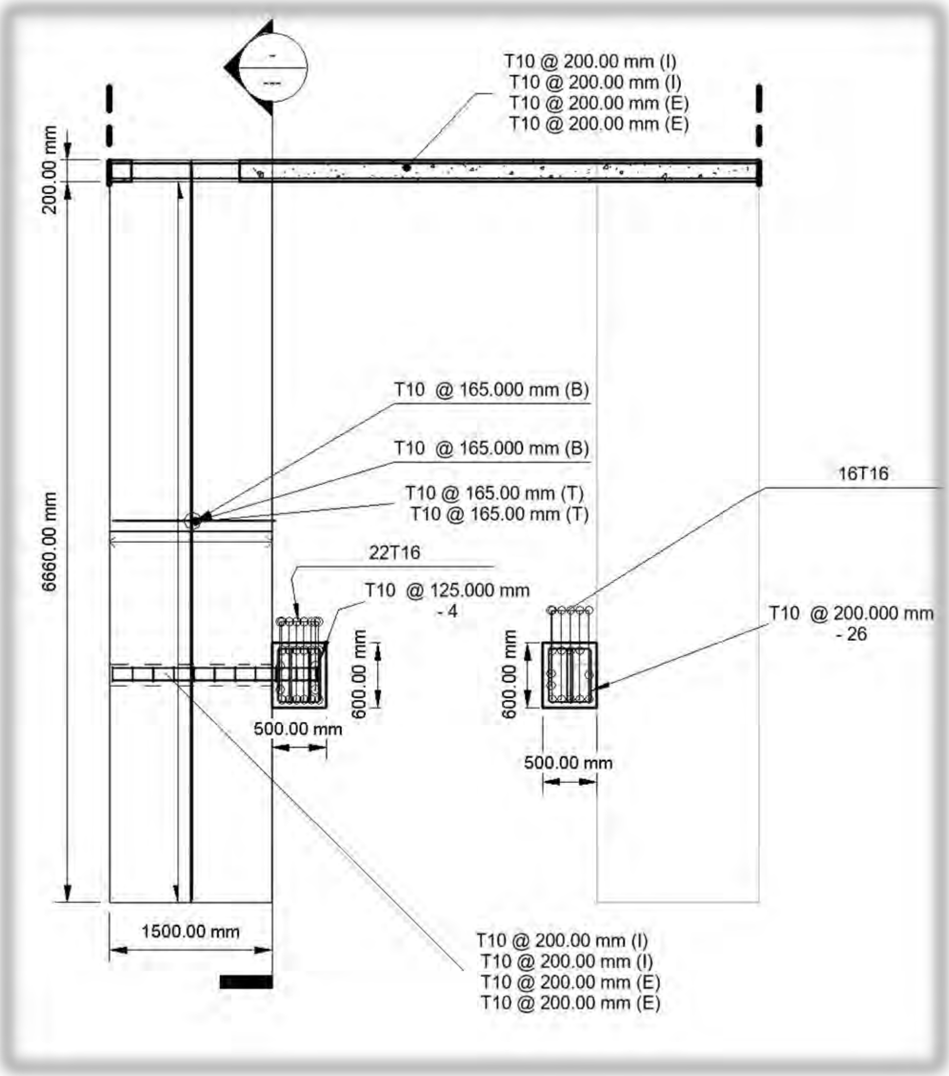


Figure 2-1: 1st Platform Plan

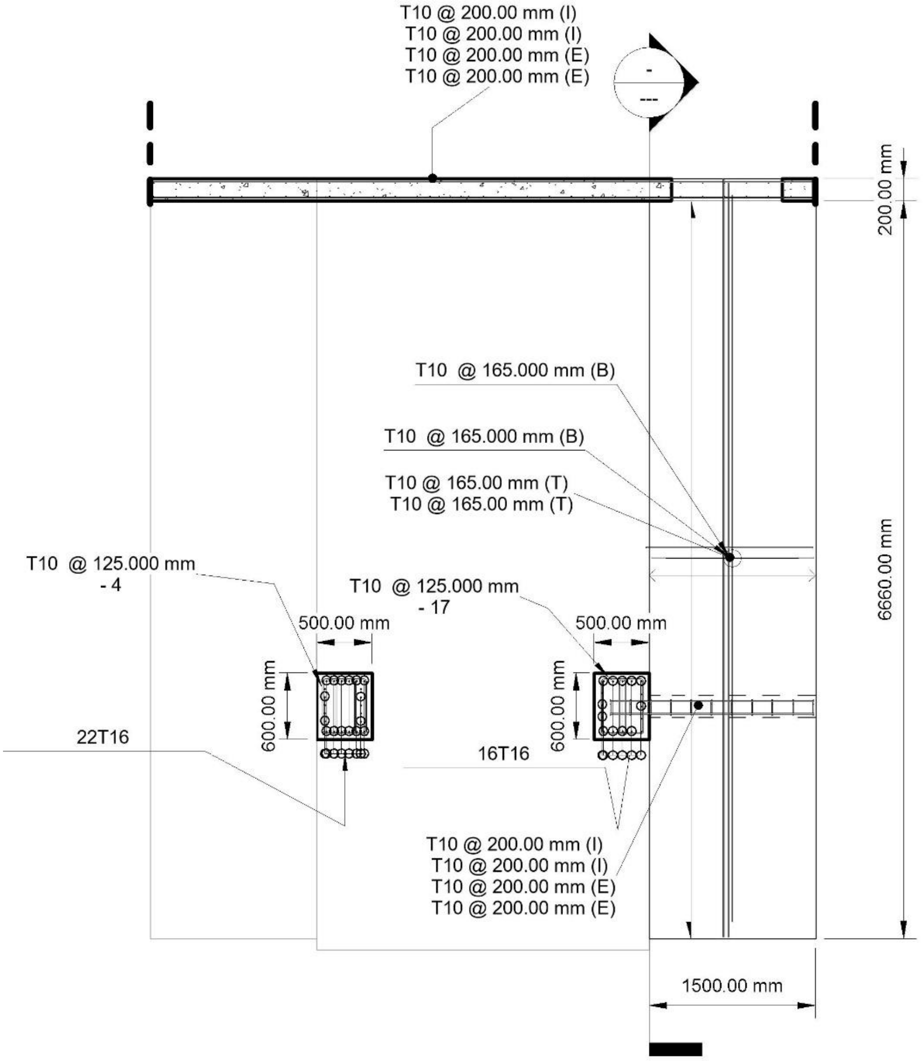


Figure 2-2: 2nd Platform Plan

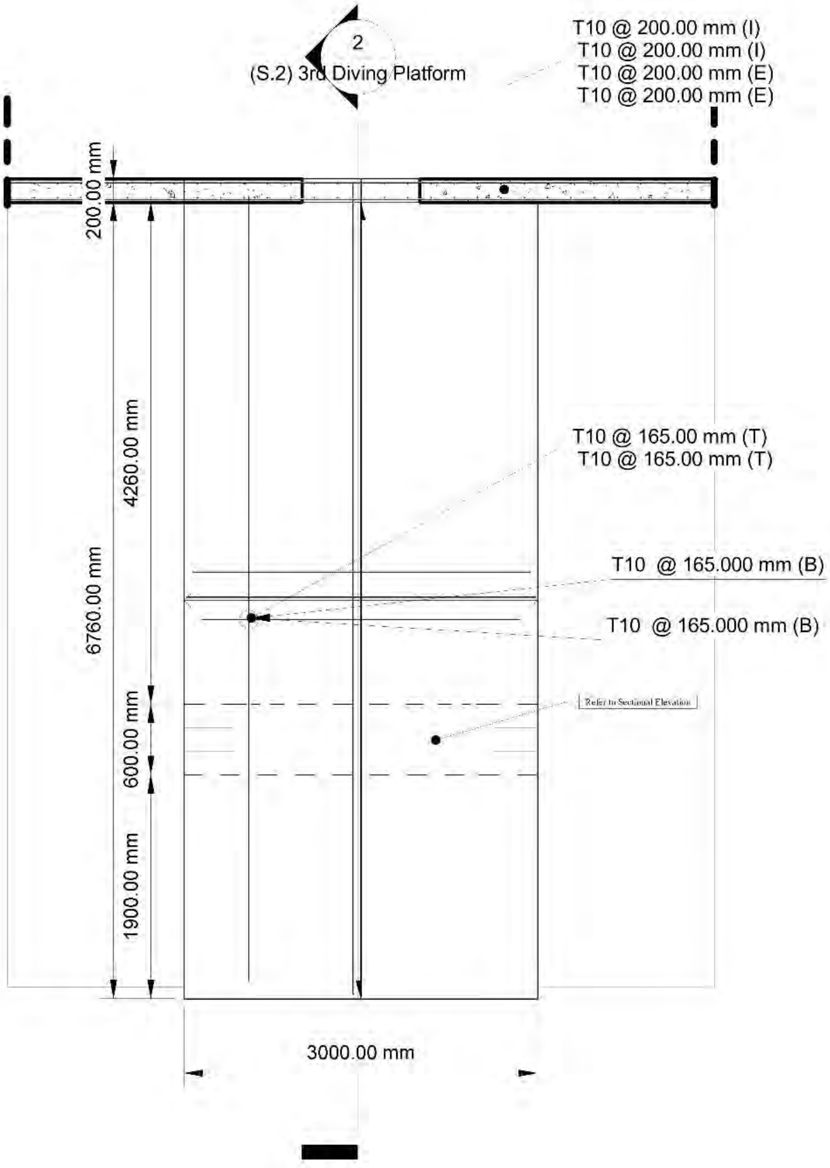


Figure 2-3: 3rd Platform Plan

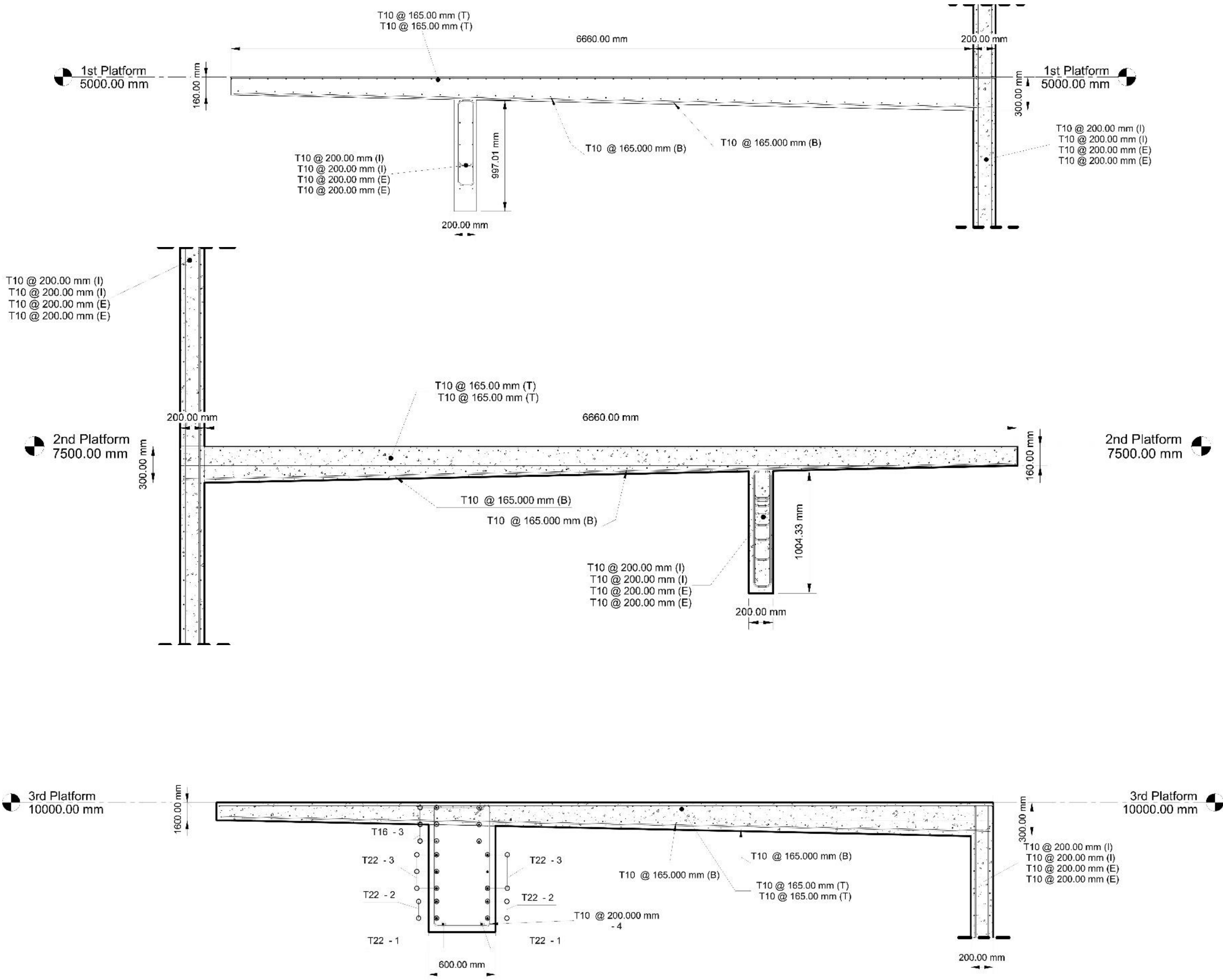


Figure 2-4: Platform's Sections.

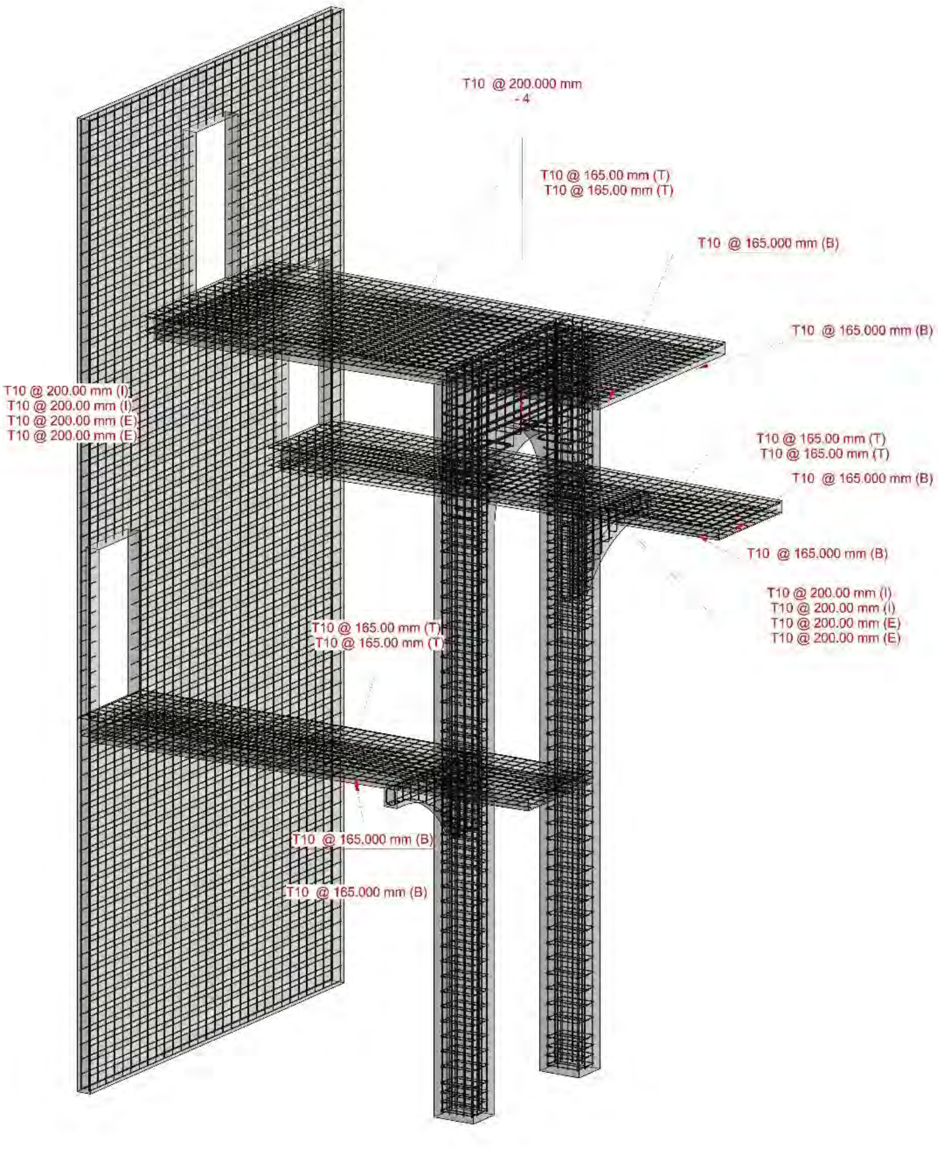


Figure 2-5: 3D-Reinforcement

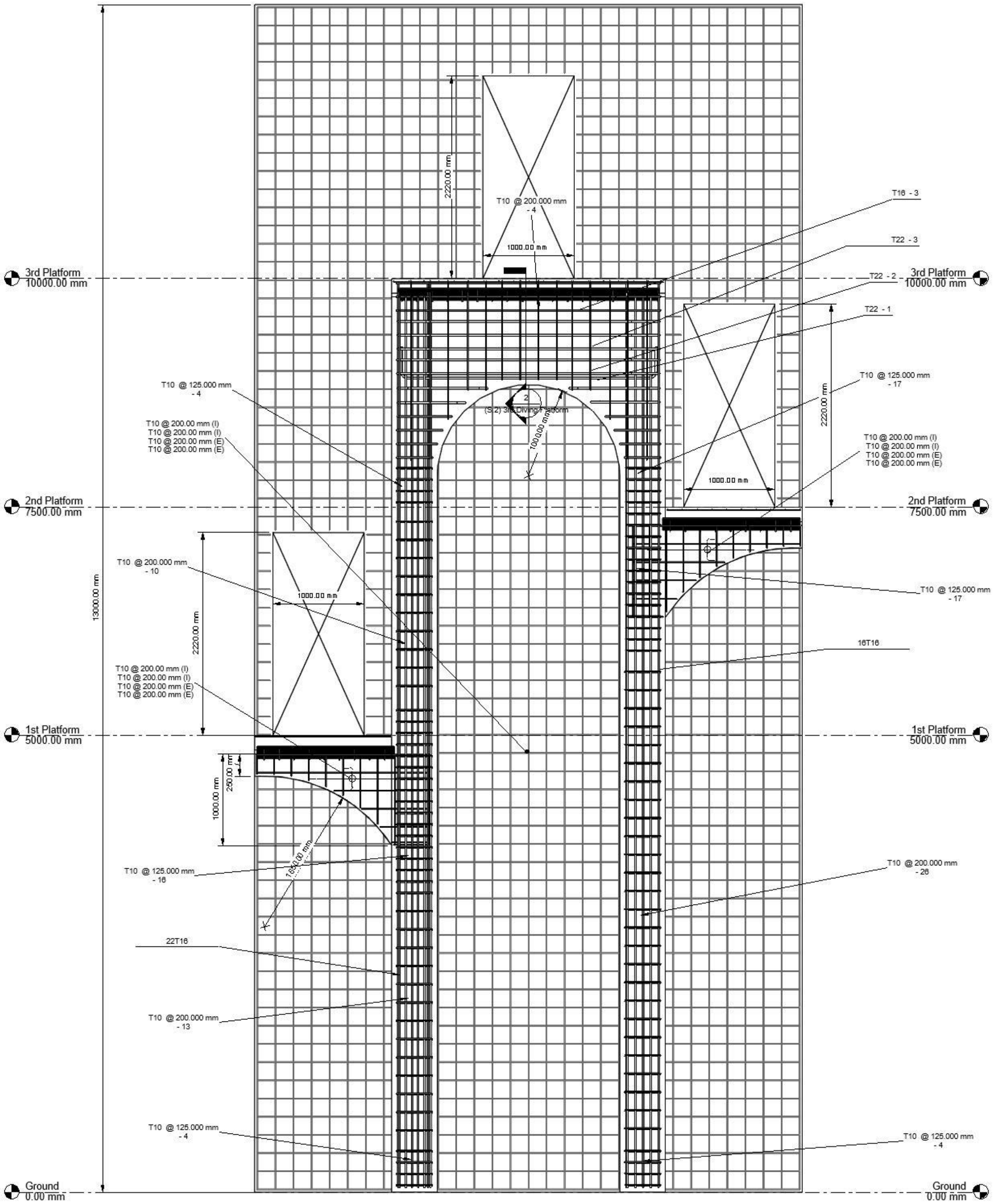


Figure 2-6: Sectional Elevation.

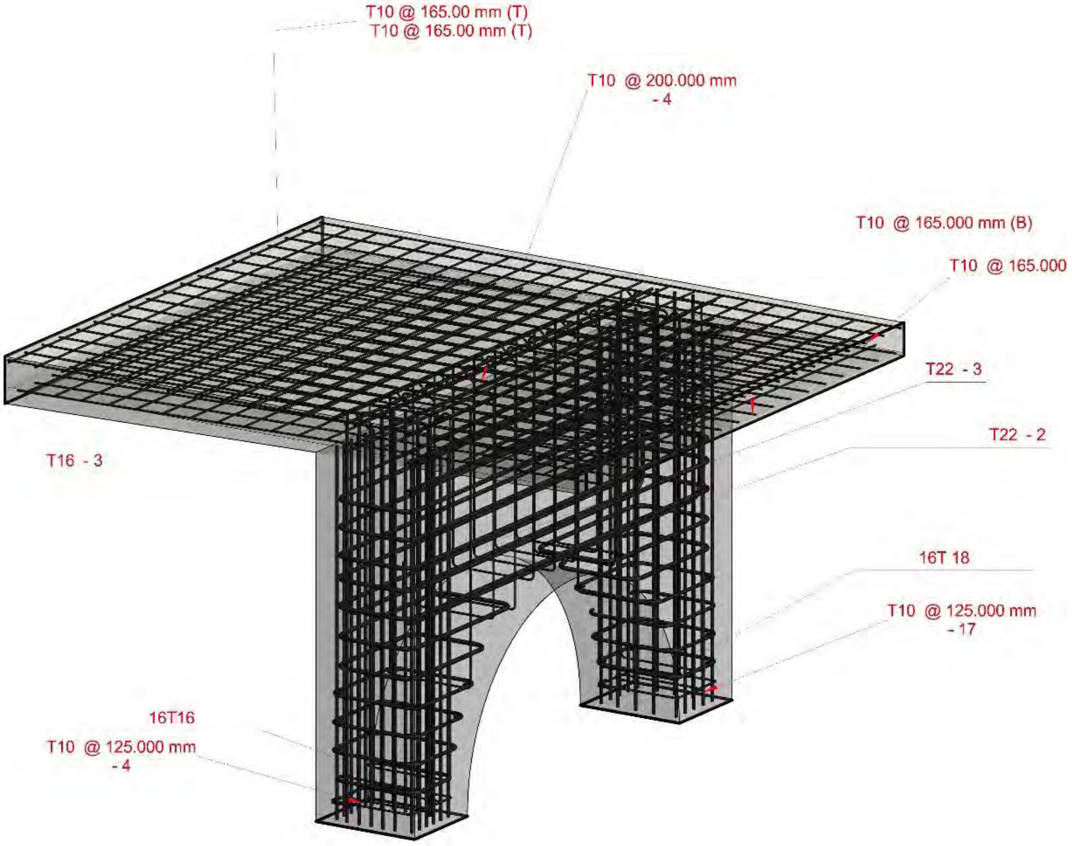


Figure 2-7: 3D-Arched Beam RFT

3. Calculation Software Used

Calculation software features

The software used is *SAP2000*, developed by Computer & Structures (United States).

Technical specifications

Name:	Sap2000
Version:	14.2.2
Producer:	Computer&Structures

https://www.csiamerica.com/

4. OUTLINE SPECIFICATION AND MATERIAL PROPERITIES.

REINFORCED CONCRETE.

The grade of concrete will be according to the Egyptian Code of Practice (ECP). The grade of concrete is indicated in two numbers, the first one indicate the characteristic cube strength in (N/mm²) while the second one indicates the maximum nominal size of the aggregate in (mm) to be used;

Grade (20/20)	for plain concrete of foundations of thickness < 12 cm.
Grade (20/40)	for plain concrete of foundations of thickness >12 cm
Grade (30/20)	for all pool reinforced concrete elements.

Minimum thickness of blinding concrete is 100 mm.

Concrete cover is the concrete thickness to all steel reinforcement including links:

For all concrete (with protection) in contact with soil, cover shall be 70mm (or as will be recommended in the geotechnical report)

For all concrete elements above grade where concrete is protected from weathering, cover shall be 50mm for beams and 25 mm for Colmuns, slabs and walls.

- **SLUMP VALUES.**

The following values are according to the Egyptian code of practice ECP 203-2018 section (2-3-1-2), Table (2-5).

Type of Structural Element	Type of Compaction	Slump-in mm (max.)
Massive concrete	Mechanical	25 - 50
- Concrete foundation. - Concrete sections with low reinforcement ratio (< 80 kg/m³)	Mechanical	50 - 75
Concrete sections with medium and high reinforcement ratio (80-150 kg/m³)	Mechanical/ Manual	75 – 125
Concrete sections with very high reinforcement ratio (> 150 kg/m³)	Light compaction	125 – 150**
Deep foundation	Light compaction	125 – 200**

** By using chemical additives.

REINFORCING STEEL

All reinforcing steel shall be complying with the Egyptian code of practice ECP203-2018, section 2-2-5-3, Table 2-4.

Reinforcing steel bars shall be uncoated high yield deformed bars of characteristic strength 360 N/mm².

Uncoated mild steel plain bars with characteristic strength 280 N/mm² may be used for links and binders.

Type	Grade	Yield Strength, f_y (N/mm²)
Normal mild steel	280/450	280
High grade steel	360/520	360
Cold formed welded mesh	450/520	450

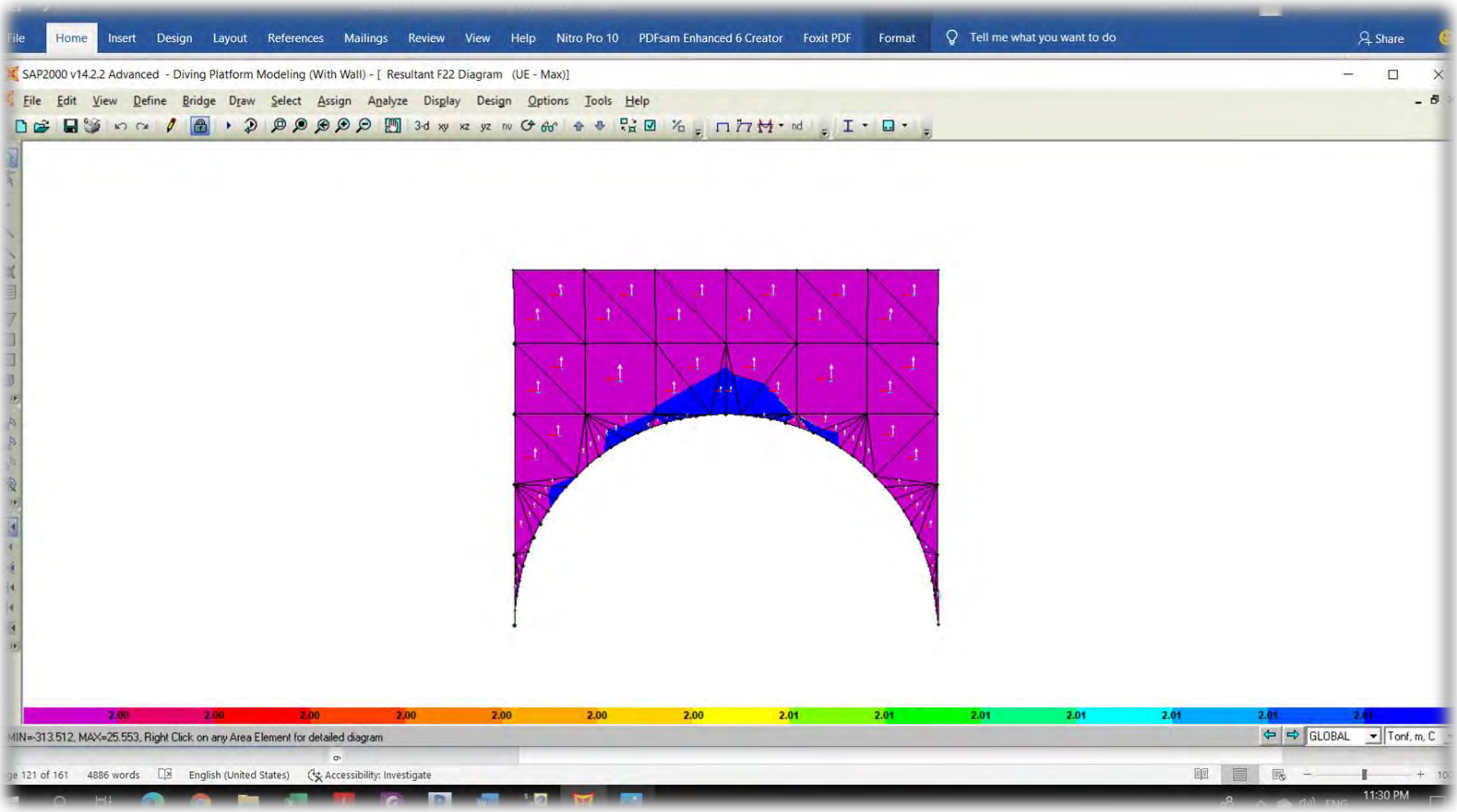
Note: Bar size increment = 6, 8, 10, 12, 16, 18, 20, 22, 25, 28 and 32

5. Calculation method and numerical model

5.1 Model Description

5.1.1 Hypothesis adopted for the elements

- The Diving Platform is modeled as spatial model, all element is defined as shell elements except the columns.
- The arch section of the beam, enhances it’s performance by reducing the tensioned area of the beam as shown in Fig. 5.1.



MIN=-313.512, MAX=25.553, Right Click on any Area Element for detailed diagram

Figure 5.1: Tension side of the Platform’s Beam.

- According to, depth to span ratio, the beam act as deep beam in the section near to the columns and act as shallow beam in the midspan's section; thus the reinforcement details follow the deep beams reinforcement requirements as shown in Fig. 5.2.

  .

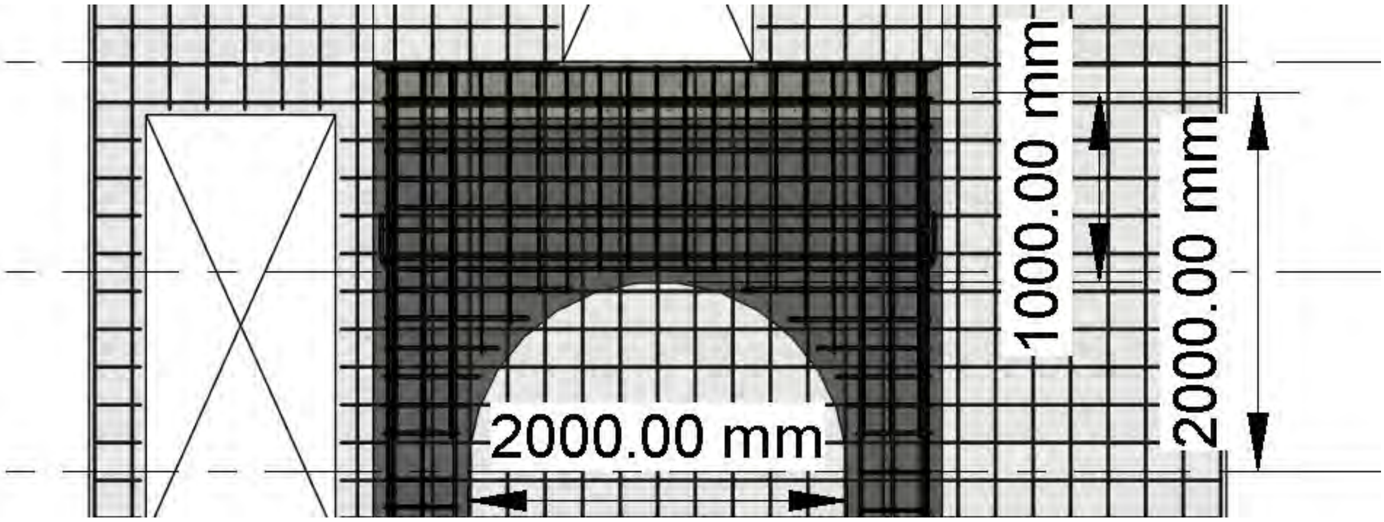


Figure 5.2: Arched Beam Detail.

6. Actions and design loads

6.1 STRUCTURAL LOADS.

The following loads are considered in the design:

- Structural Dead Loads which include:
    - The own weight of the structural elements, beams, Columns and Platforms.
    - Superimposed dead load from Finishing.
- Live loads which cover the spring boards and the movement of swimmers on the platform.
- Seismic loads according to ECP.

The basis for the considered design loads are summarized in the followings sections.

A. Dead Loads

Unit weight of concrete elements 25.0 kN/m³

B. Live Loads

Live loads are considered equal to 350 kg/m$^{/}$ for platforms and supporting structure according to FINA requirements.

C. Earthquakes

- Response modification factor (R = 5)
- Importance factor ($\gamma_i = 1.2$)
- The design acceleration ($a_g = 0.15g$)
- Design damping correction factor ($\eta = 1.0$)
- Zone 3
- Soil Type (C)
- Earthquake loads shall be comply with the (ECP 201-2010).

The following tables describe the load cases and load combinations on the pool:

Table 1: Load cases

Case	Type	Modal Case	Design Type	Auto Type
DEAD	LinStatic		DEAD	None
MODAL	LinModal		OTHER	None
L.L	LinStatic		OTHER	None
F.C	LinStatic		DEAD	None
Ex	LinStatic		QUAKE	None
Ey	LinStatic		QUAKE	None
Resp. x	LinRespSpec	MODAL	QUAKE	None
Resp. y	LinRespSpec	MODAL	QUAKE	None

Table 2: Load combinations

Table: Combination Definitions, Part 1 of 3

Combo .Name	Comb .Type	Auto Design	Case Type	Case Name	Scale Factor	Steel Design
UDLPRx	Linear Add	No	Linear Static	DEAD	1.120000	None
UDLPRx			Linear Static	L.L	0.500000	
UDLPRx			Response Spectrum	Resp. x	1.000000	
UDLPRx			Linear Static	F.C	1.120000	
UDLNRx	Linear Add	No	Linear Static	DEAD	1.120000	None
UDLNRx			Linear Static	L.L	0.500000	
UDLNRx			Response Spectrum	Resp. x	-1.000000	
UDLNRx			Linear Static	F.C	1.120000	
UDLPRy	Linear Add	No	Linear Static	DEAD	1.120000	None
UDLPRy			Linear Static	L.L	0.500000	
UDLPRy			Response Spectrum	Resp. y	1.000000	
UDLPRy			Linear Static	F.C	1.120000	
UDLNRy	Linear Add	No	Linear Static	DEAD	1.120000	None
UDLNRy			Linear Static	L.L	0.500000	
UDLNRy			Response Spectrum	Resp. y	-1.000000	
UDLNRy			Linear Static	F.C	1.120000	
UDLPRxPRy	Linear Add	No	Linear Static	DEAD	1.120000	None
UDLPRxPRy			Linear Static	L.L	0.500000	
UDLPRxPRy			Response Spectrum	Resp. x	1.000000	
UDLPRxPRy			Response Spectrum	Resp. y	0.300000	
UDLPRxPRy			Linear Static	F.C	1.120000	
UDLNRxNRy	Linear Add	No	Linear Static	DEAD	1.120000	None
UDLNRxNRy			Linear Static	L.L	0.500000	
UDLNRxNRy			Response Spectrum	Resp. x	-1.000000	
UDLNRxNRy			Response Spectrum	Resp. y	-0.300000	
UDLNRxNRy			Linear Static	F.C	1.120000	
UDLPRxNRy	Linear Add	No	Linear Static	DEAD	1.120000	None
UDLPRxNRy			Linear Static	L.L	0.500000	
UDLPRxNRy			Response Spectrum	Resp. x	1.000000	
UDLPRxNRy			Response Spectrum	Resp. y	-0.300000	
UDLPRxNRy			Linear Static	F.C	1.120000	
UDLNRxPRy	Linear Add	No	Linear Static	DEAD	1.120000	None
UDLNRxPRy			Linear Static	L.L	0.500000	
UDLNRxPRy			Response Spectrum	Resp. x	-1.000000	
UDLNRxPRy			Response Spectrum	Resp. y	0.300000	
UDLNRxPRy			Linear Static	F.C	1.120000	
UDLPRyPRx	Linear Add	No	Linear Static	DEAD	1.120000	None
UDLPRyPRx			Linear Static	L.L	0.500000	
UDLPRyPRx			Response Spectrum	Resp. x	0.300000	
UDLPRyPRx			Response Spectrum	Resp. y	1.000000	
UDLPRyPRx			Linear Static	F.C	1.120000	
UDLNRyNRx	Linear Add	No	Linear Static	DEAD	1.120000	None
UDLNRyNRx			Linear Static	L.L	0.500000	
UDLNRyNRx			Response Spectrum	Resp. x	-0.300000	
UDLNRyNRx			Response Spectrum	Resp. y	-1.000000	
UDLNRyNRx			Linear Static	F.C	1.120000	
UDLNRyPRx	Linear Add	No	Linear Static	DEAD	1.120000	None
UDLNRyPRx			Linear Static	L.L	0.500000	
UDLNRyPRx			Response Spectrum	Resp. x	0.300000	
UDLNRyPRx			Response Spectrum	Resp. y	-1.000000	
UDLNRyPRx			Linear Static	F.C	1.120000	

UDLPRyNRx	Linear Add	No	Linear Static	DEAD	1.120000	None
UDLPRyNRx			Linear Static	L.L	0.500000	
UDLPRyNRx			Response Spectrum	Resp. x	-0.300000	
UDLPRyNRx			Response Spectrum	Resp. y	1.000000	
UDLPRyNRx			Linear Static	F.C	1.120000	
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UDL			Linear Static	F.C	1.400000	
UDL			Linear Static	L.L	1.600000	
Ub	Linear Add	No	Linear Static	DEAD	1.120000	None
Ub			Linear Static	F.C	1.120000	
Ub			Linear Static	L.L	1.280000	
Ub			Linear Static	W-x	1.280000	
Ua	Linear Add	No	Linear Static	DEAD	1.120000	None
Ua			Linear Static	F.C	1.120000	
Ua			Linear Static	L.L	1.280000	
Ua			Linear Static	Wx	1.280000	
Uc	Linear Add	No	Linear Static	DEAD	1.120000	None
Uc			Linear Static	F.C	1.120000	
Uc			Linear Static	L.L	1.280000	
Uc			Linear Static	Wy	1.280000	
Ud	Linear Add	No	Linear Static	DEAD	1.120000	None
Ud			Linear Static	F.C	1.120000	
Ud			Linear Static	L.L	1.280000	
Ud			Linear Static	W-y	1.280000	
WDL	Linear Add	No	Linear Static	DEAD	1.000000	None
WDL			Linear Static	F.C	1.000000	
WDL			Linear Static	L.L	1.000000	
WDLPRx	Linear Add	No	Linear Static	DEAD	1.000000	None
WDLPRx			Linear Static	L.L	0.420000	
WDLPRx			Response Spectrum	Resp. x	0.720000	
WDLPRx			Linear Static	F.C	1.000000	
WDLNRx	Linear Add	No	Linear Static	DEAD	1.000000	None
WDLNRx			Linear Static	L.L	0.420000	
WDLNRx			Response Spectrum	Resp. x	-0.720000	
WDLNRx			Linear Static	F.C	1.000000	
WDLPRy	Linear Add	No	Linear Static	DEAD	1.000000	None
WDLPRy			Linear Static	L.L	0.420000	
WDLPRy			Response Spectrum	Resp. y	0.720000	
WDLPRy			Linear Static	F.C	1.000000	
WDLNRy	Linear Add	No	Linear Static	DEAD	1.000000	None
WDLNRy			Linear Static	L.L	0.420000	
WDLNRy			Response Spectrum	Resp. y	-0.720000	
WDLNRy			Linear Static	F.C	1.000000	
WDLPRxPRy	Linear Add	No	Linear Static	DEAD	1.000000	None
WDLPRxPRy			Linear Static	L.L	0.420000	
WDLPRxPRy			Response Spectrum	Resp. x	0.720000	
WDLPRxPRy			Response Spectrum	Resp. y	0.220000	
WDLPRxPRy			Linear Static	F.C	1.000000	
WDLNRxNRy	Linear Add	No	Linear Static	DEAD	1.000000	None
WDLNRxNRy			Linear Static	L.L	0.420000	
WDLNRxNRy			Response Spectrum	Resp. x	-0.720000	
WDLNRxNRy			Response Spectrum	Resp. y	-0.220000	
WDLNRxNRy			Linear Static	F.C	1.000000	
WDLPRxNRy	Linear Add	No	Linear Static	DEAD	1.000000	None
WDLPRxNRy			Linear Static	L.L	0.420000	
WDLPRxNRy			Response Spectrum	Resp. x	0.720000	

WDLPRxNRy			Response Spectrum	Resp. y	-0.220000	
WDLPRxNRy			Linear Static	F.C	1.000000	
WDLNRxPRy	Linear Add	No	Linear Static	DEAD	1.000000	None
WDLNRxPRy			Linear Static	L.L	0.420000	
WDLNRxPRy			Response Spectrum	Resp. x	-0.720000	
WDLNRxPRy			Response Spectrum	Resp. y	0.220000	
WDLNRxPRy			Linear Static	F.C	1.000000	
WDLPRyPRx	Linear Add	No	Linear Static	DEAD	1.000000	None
WDLPRyPRx			Linear Static	L.L	0.420000	
WDLPRyPRx			Response Spectrum	Resp. x	0.220000	
WDLPRyPRx			Response Spectrum	Resp. y	0.720000	
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WDLNRyNRx	Linear Add	No	Linear Static	DEAD	1.000000	None
WDLNRyNRx			Linear Static	L.L	0.420000	
WDLNRyNRx			Response Spectrum	Resp. x	-0.220000	
WDLNRyNRx			Response Spectrum	Resp. y	-0.720000	
WDLNRyNRx			Linear Static	F.C	1.000000	
WDLPRyNRx	Linear Add	No	Linear Static	DEAD	1.000000	None
WDLPRyNRx			Linear Static	L.L	0.420000	
WDLPRyNRx			Response Spectrum	Resp. x	-0.220000	
WDLPRyNRx			Response Spectrum	Resp. y	0.720000	
WDLPRyNRx			Linear Static	F.C	1.000000	
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WDLNRyPRx			Response Spectrum	Resp. x	0.220000	
WDLNRyPRx			Response Spectrum	Resp. y	-0.720000	
WDLNRyPRx			Linear Static	F.C	1.000000	
UE	Envelope	No	Response Combo	UDL	1.000000	None
UE			Response Combo	UDLNRx	1.000000	
UE			Response Combo	UDLNRxNRy	1.000000	
UE			Response Combo	UDLNRxPRy	1.000000	
UE			Response Combo	UDLNRy	1.000000	
UE			Response Combo	UDLNRyNRx	1.000000	
UE			Response Combo	UDLNRyPRx	1.000000	
UE			Response Combo	UDLPRx	1.000000	
UE			Response Combo	UDLPRxNRy	1.000000	
UE			Response Combo	UDLPRxPRy	1.000000	
UE			Response Combo	UDLPRy	1.000000	
UE			Response Combo	UDLPRyNRx	1.000000	
UE			Response Combo	UDLPRyPRx	1.000000	
WE	Envelope	No	Response Combo	WDL	1.000000	None
WE			Response Combo	WDLNRx	1.000000	
WE			Response Combo	WDLNRxNRy	1.000000	
WE			Response Combo	WDLNRxPRy	1.000000	
WE			Response Combo	WDLNRy	1.000000	
WE			Response Combo	WDLNRyNRx	1.000000	
WE			Response Combo	WDLNRyPRx	1.000000	
WE			Response Combo	WDLPRx	1.000000	
WE			Response Combo	WDLPRxNRy	1.000000	
WE			Response Combo	WDLPRxPRy	1.000000	
WE			Response Combo	WDLPRy	1.000000	
WE			Response Combo	WDLPRyNRx	1.000000	
WE			Response Combo	WDLPRyPRx	1.000000	

6.2 CRACKING

It will be calculated as stated in the “ECP 203-2018 - section 4-3-2” for the following maximum design crack width:

- 0.20 mm for concrete exposed to dry soil or air.

6.3 Deflection

- Total deflection for beams/slabs and cantilevers calculated taking all loads into consideration in addition to the effects of self-straining forces shall not exceed the following values:

a) For beams and slabs L/250
b) For cantilevers L/450

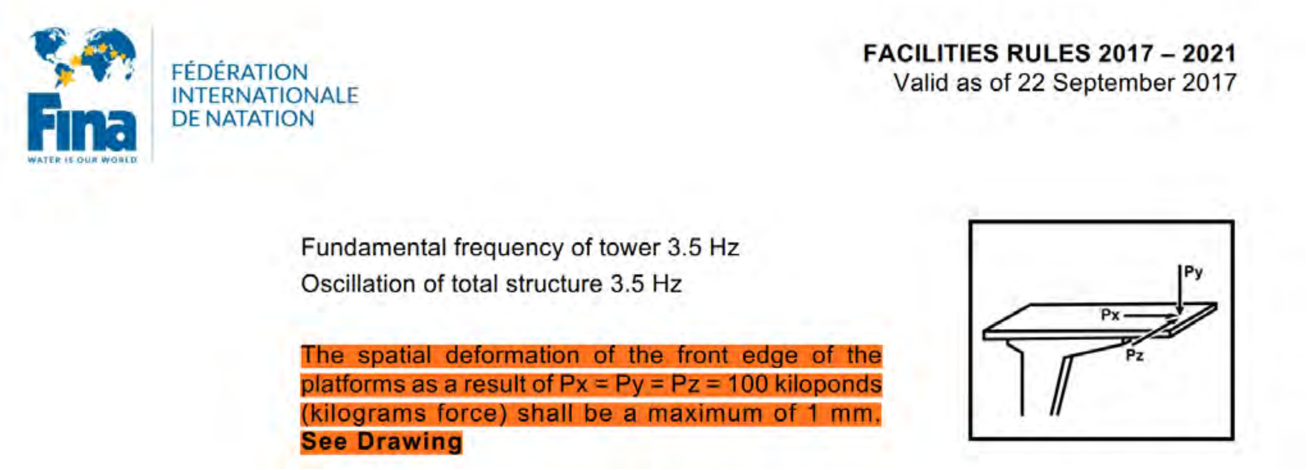
Immediate deflection due to live loads for beams and slabs supporting non-structural elements (which are not affected by deflection) shall not exceed L/360.

Additional total deflection (that occur after adding the floorings) for beams and slabs supporting non-structural elements (which are affected by deflections, such as spring boards) calculated taking all loads into consideration in addition to the effects of self-straining forces shall not exceed L/480.

Where L is distance between the inflection points for beams and slabs or the cantilever length.

L is calculated for the short span of the two way slabs, and for the long span of the flat slabs.

- The spatial deformation of the front edge of the platforms as a result of Px = Py = Pz = 100 kiloponds (kilograms force) shall be a maximum of 1 mm.


FÉDÉRATION INTERNATIONALE DE NATATION

FACILITIES RULES 2017 – 2021
Valid as of 22 September 2017

Fundamental frequency of tower 3.5 Hz
Oscillation of total structure 3.5 Hz

The spatial deformation of the front edge of the platforms as a result of Px = Py = Pz = 100 kiloponds (kilograms force) shall be a maximum of 1 mm. **See Drawing**

6.4 Fundamental Frequency.

According to FINA, Fundamental frequency of platforms 10.0 Hz

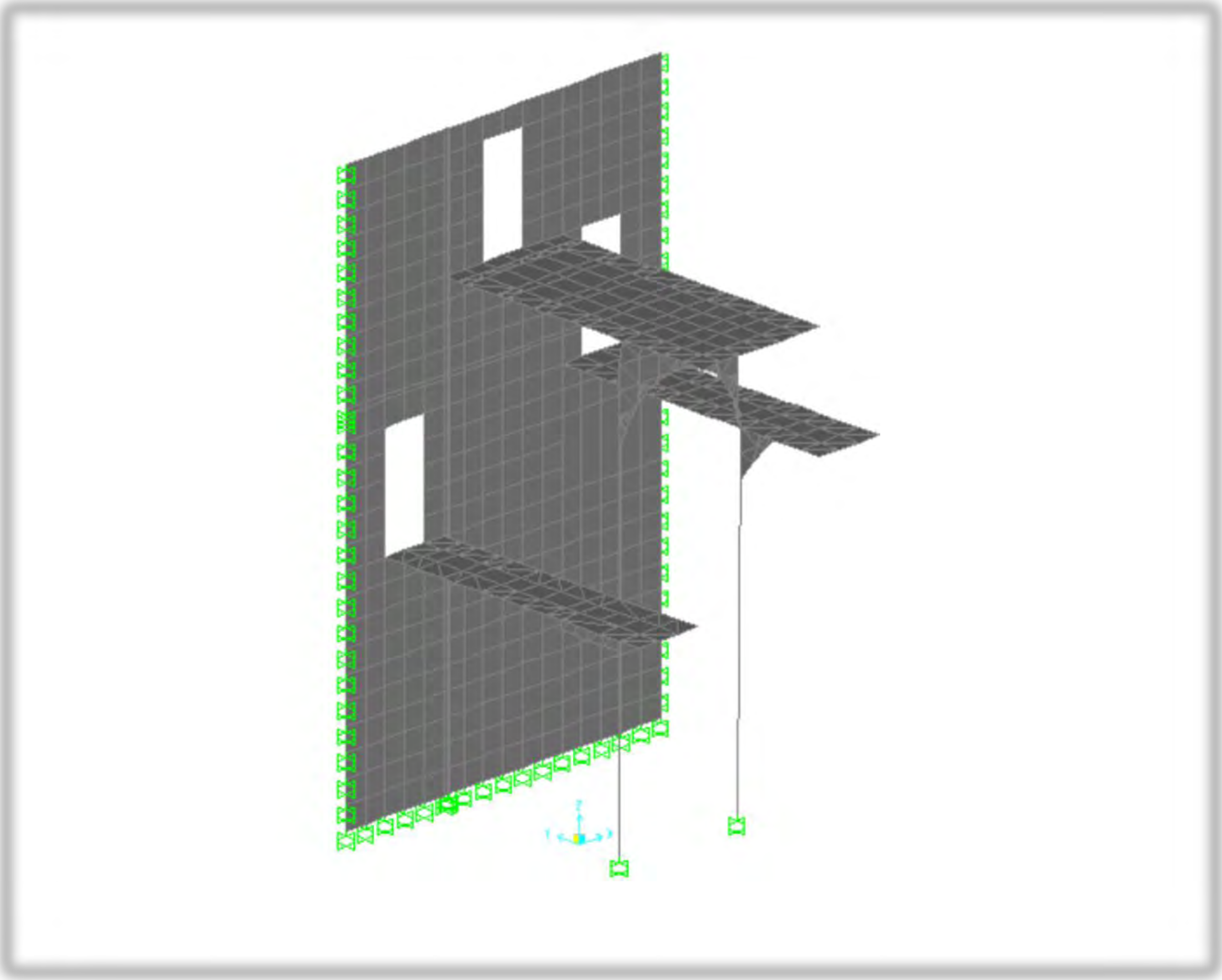
TOLERANCES:

PLATFORM	MINIMUM	MAXIMUM
10m	10 Hz	20 Hz
7.5m, 5m, 3m and 1m	10 Hz	30 Hz

Fundamental frequency of tower 3.5 Hz
Oscillation of total structure 3.5 Hz

7. STRUCTURAL ANALYSIS

7.1 3d-model



7.2 ASSIGN OF LOADS

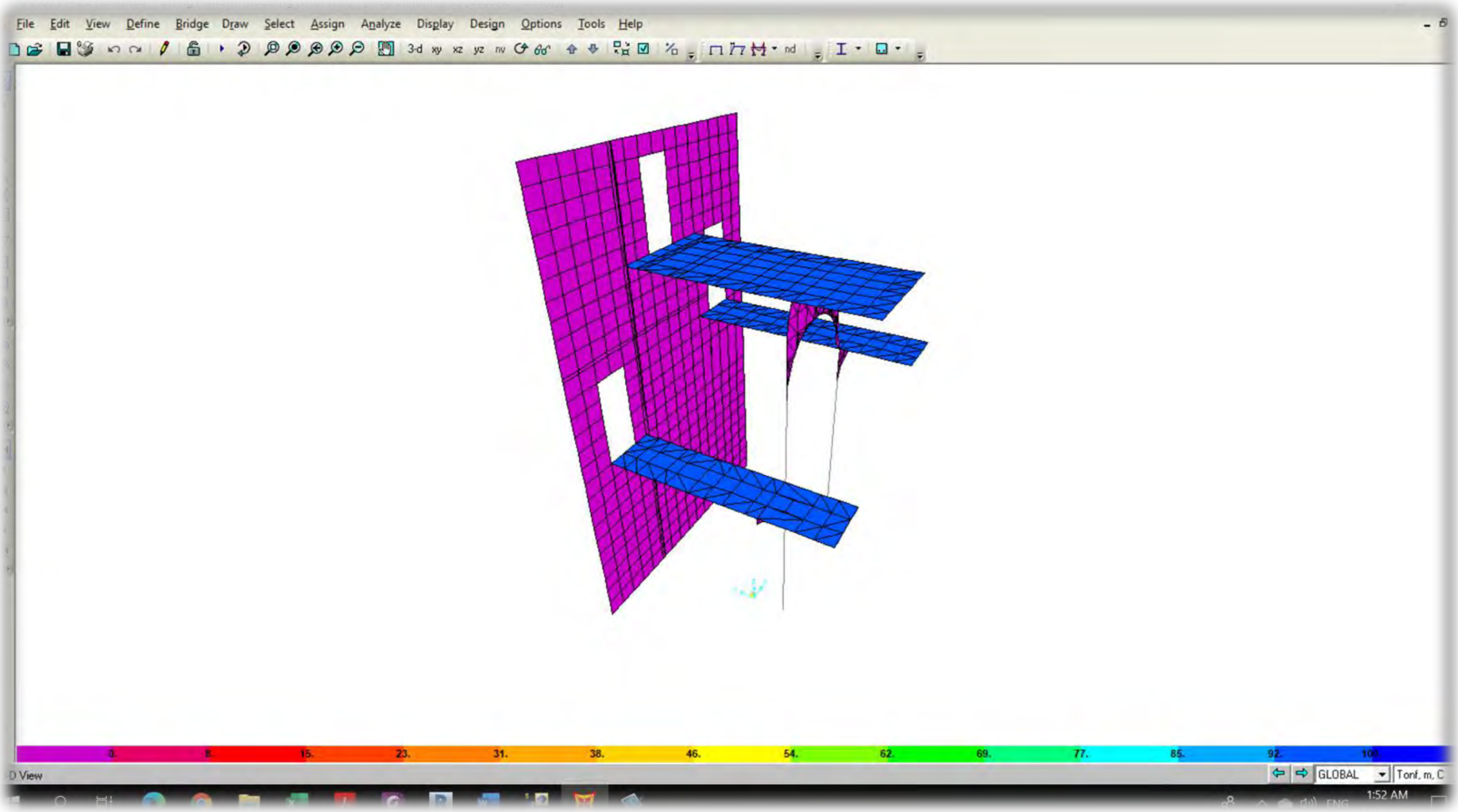


Figure 7.1.: Assign of Finishing Loads on the Platforms (T-M^2 Units)

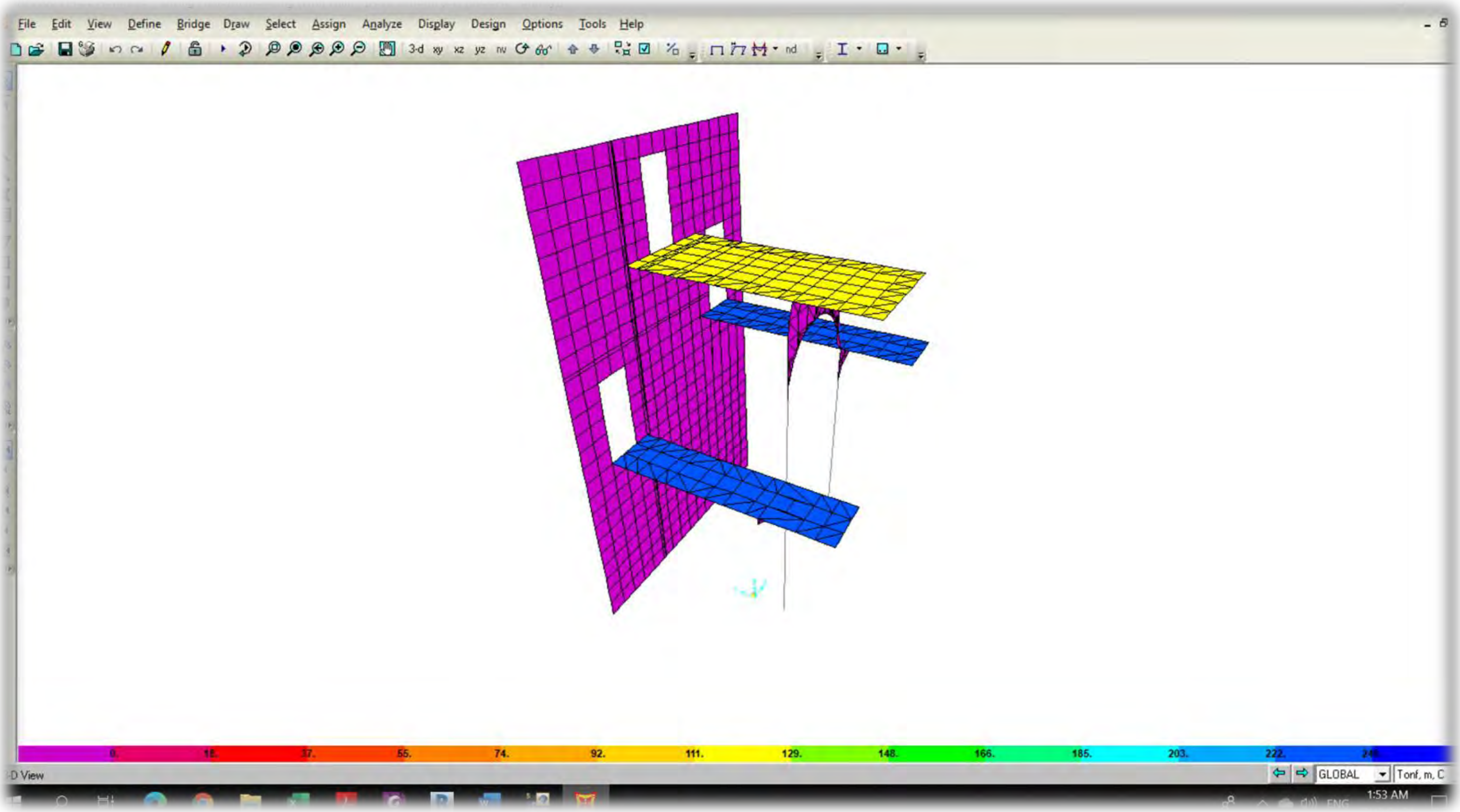


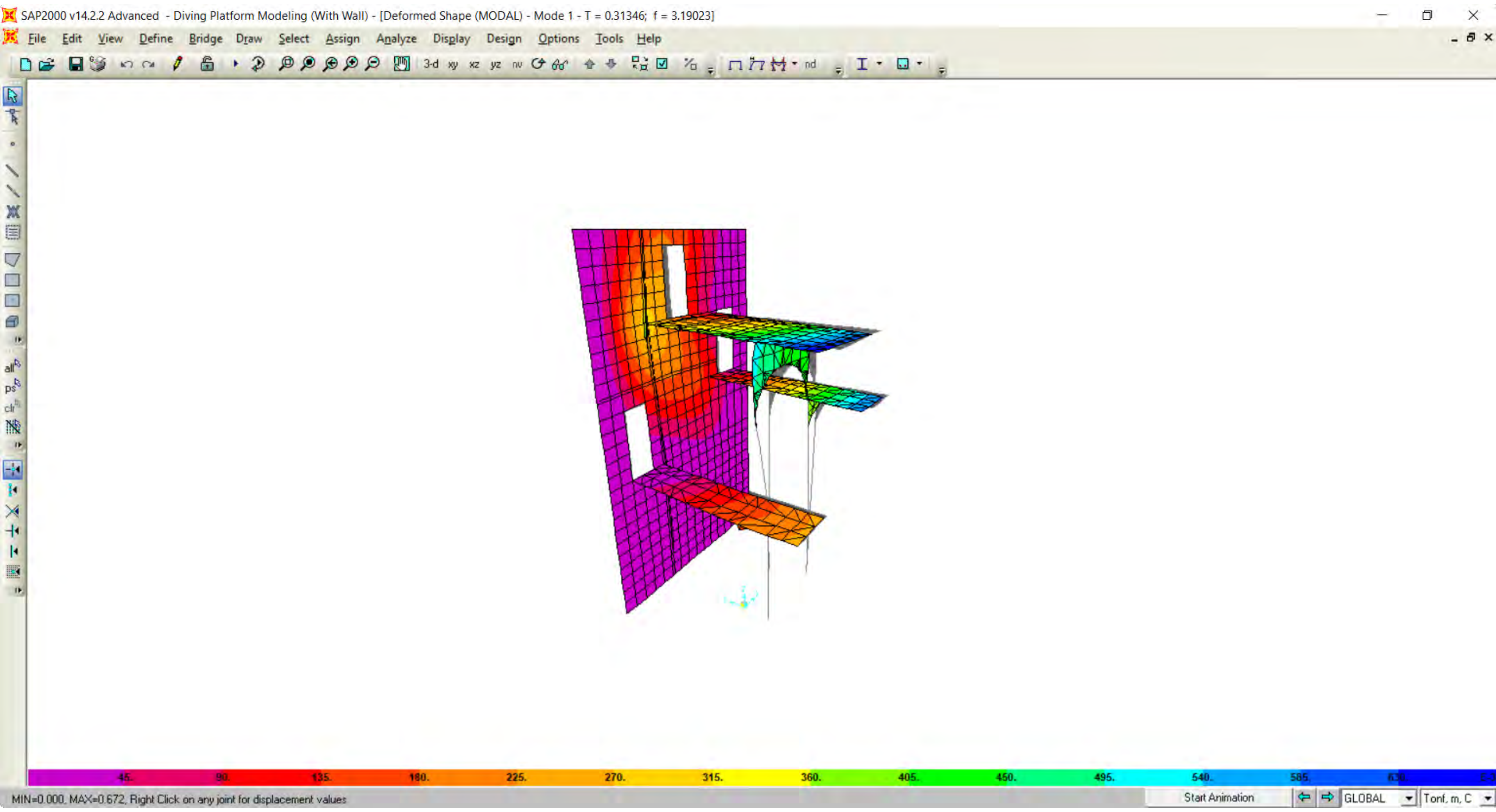
Figure 7.2.: Assign of Live loads on the Platforms (T-M^2 Units)

8. STRUCTURAL DESIGN.

8.1 Checks.

8.1.1 Frequency

From SAP model, the maximum Frequency of total structure = 3.19 HZ as shown in Fig. 8.1


SAP2000 v14.2.2 Advanced - Diving Platform Modeling (With Wall) - [Deformed Shape (MODAL) - Mode 1 - T = 0.31346; f = 3.19023]

MIN=0.000, MAX=0.672, Right Click on any joint for displacement values

Maximum Frequency = 3.19 HZ < 3.5 HZ (Acceptable)

8.1.2 Spatial Deformation.

From SAP model, the maximum deformation in the front edge of:

- 3rd platform = 1.8mm induced from 220 kg load; thus, for 100 kg load, it will equal 1.8/2.2= 0.82mm > 1mm (Acceptable)
- 1st and 2nd platforms= 1.6mm induced from 340 kg load; thus, for 100 kg load, it will equal 1.6/3.4= 0.47mm < 1mm (Acceptable)

8.1.3 Equivalent static loads.

According to clause 8.7.3.4 in ECP 201-2010 , the equivalent static load shouldn’t less than 80% of equivalent load calculated by using compound response spectrum method.

Table 3: Base Shear Reactions

Output Case	Case Type	Step Type	Global FX (Ton)	Global FY (Ton)
E_x	Lin. Static		-22.0827	3.164E-11
E_y	Lin. Static		-2.516E-11	-22.0827
Resp. x	Lin. Resp. Spec.	Max	22.0379	37.2215
Resp. y	Lin. Resp. Spec.	Max	12.3472	20.8542

8.1.4 Model Participation Mass Ratio.

According to clause 8.7.3.3.1, point 5.a in ECP 201-2010 , The considered Eigenvalues mode shapes in design should excite mass not less than 0.9 of total structure’s mass.

Table 4: Modal Participating Mass Ratios

Output Case	Step. Type	Step. Num.	Period (Sec)	UX	UY	UZ	Sum. UX	Sum. UY
MODAL	Mode	1.000000	0.313457	0.209093	0.119547	9.421E-06	0.209093	0.119547
MODAL	Mode	2.000000	0.234052	0.064936	0.343120	0.000290	0.274029	0.462667
MODAL	Mode	3.000000	0.230853	0.006611	0.002124	0.000084	0.280639	0.464791
MODAL	Mode	4.000000	0.193856	0.000462	3.707E-06	8.965E-06	0.281101	0.464794
MODAL	Mode	5.000000	0.187341	0.000058	9.715E-08	1.949E-07	0.281159	0.464794
MODAL	Mode	6.000000	0.185208	0.000013	4.052E-08	3.597E-08	0.281172	0.464794
MODAL	Mode	7.000000	0.184418	2.475E-06	1.544E-08	8.194E-09	0.281175	0.464794
MODAL	Mode	8.000000	0.177439	0.015936	0.060854	0.000045	0.297111	0.525648
MODAL	Mode	9.000000	0.130703	3.406E-06	0.003181	0.001976	0.297114	0.528829
MODAL	Mode	10.000000	0.122133	0.004049	0.225864	0.000530	0.301163	0.754692
MODAL	Mode	11.000000	0.120187	7.978E-06	0.014271	0.000030	0.301171	0.768963
MODAL	Mode	12.000000	0.117995	0.000695	0.007442	0.000191	0.301866	0.776406
MODAL	Mode	13.000000	0.116025	7.805E-06	0.000505	0.000015	0.301874	0.776911
MODAL	Mode	14.000000	0.115027	1.389E-08	0.000070	7.573E-06	0.301874	0.776981
MODAL	Mode	15.000000	0.114769	2.646E-07	0.000014	6.528E-07	0.301874	0.776995
MODAL	Mode	16.000000	0.113156	0.000480	0.000326	0.000083	0.302355	0.777321
MODAL	Mode	17.000000	0.108937	0.001857	0.009398	0.000376	0.304212	0.786719
MODAL	Mode	18.000000	0.102520	0.001114	0.004293	0.000022	0.305325	0.791012
MODAL	Mode	19.000000	0.101027	0.002156	0.003236	0.001141	0.307482	0.794248
MODAL	Mode	20.000000	0.092887	0.000742	0.008455	0.042936	0.308223	0.802703
MODAL	Mode	21.000000	0.089829	0.000816	0.000442	0.016445	0.309039	0.803145
MODAL	Mode	22.000000	0.085317	0.000585	0.000713	0.166644	0.309625	0.803859
MODAL	Mode	23.000000	0.077451	0.000012	0.001016	0.050161	0.309637	0.804874
MODAL	Mode	24.000000	0.074719	0.000511	0.000746	0.003557	0.310147	0.805620
MODAL	Mode	25.000000	0.070608	0.000178	0.001852	0.000344	0.310326	0.807472
MODAL	Mode	26.000000	0.066530	0.000816	0.010871	0.001701	0.311141	0.818343
MODAL	Mode	27.000000	0.060565	0.000055	0.005614	0.003322	0.311196	0.823957
MODAL	Mode	28.000000	0.055659	0.000224	0.000046	0.000090	0.311420	0.824003
MODAL	Mode	29.000000	0.053809	0.000400	0.000443	0.001237	0.311820	0.824445
MODAL	Mode	30.000000	0.052034	0.000126	0.000381	0.000694	0.311946	0.824827
MODAL	Mode	31.000000	0.050025	0.000868	0.009131	0.001337	0.312814	0.833958

MODAL	Mode	32.000000	0.048928	0.000327	0.001701	0.000152	0.313140	0.835659
MODAL	Mode	33.000000	0.046602	0.000985	0.000139	0.001797	0.314125	0.835798
MODAL	Mode	34.000000	0.045288	0.004698	0.000163	0.000316	0.318823	0.835961
MODAL	Mode	35.000000	0.040960	0.000030	0.002058	0.018340	0.318853	0.838019
MODAL	Mode	36.000000	0.040093	0.001530	0.017280	0.003272	0.320384	0.855299
MODAL	Mode	37.000000	0.037550	0.000111	0.026479	0.000060	0.320495	0.881779
MODAL	Mode	38.000000	0.037135	0.001401	0.000227	3.648E-07	0.321896	0.882006
MODAL	Mode	39.000000	0.036127	0.003696	0.006538	8.218E-07	0.325592	0.888543
MODAL	Mode	40.000000	0.035045	0.001617	0.002301	0.000176	0.327209	0.890844
MODAL	Mode	41.000000	0.034477	0.000100	0.000318	0.002295	0.327308	0.891163
MODAL	Mode	42.000000	0.033972	0.000124	0.000018	0.002296	0.327433	0.891180
MODAL	Mode	43.000000	0.033545	0.003712	0.000226	0.001503	0.331145	0.891406
MODAL	Mode	44.000000	0.032392	0.001432	0.001641	0.001540	0.332576	0.893047
MODAL	Mode	45.000000	0.031784	0.001108	0.000076	0.000785	0.333685	0.893124
MODAL	Mode	46.000000	0.031079	0.000012	0.000806	4.336E-06	0.333696	0.893929
MODAL	Mode	47.000000	0.030694	0.000138	0.001011	0.003334	0.333834	0.894940
MODAL	Mode	48.000000	0.030338	0.001311	0.000084	0.000410	0.335145	0.895024
MODAL	Mode	49.000000	0.029956	0.009726	0.000065	0.002178	0.344871	0.895089
MODAL	Mode	50.000000	0.029312	0.000746	0.001691	0.000947	0.345617	0.896780
MODAL	Mode	51.000000	0.028641	0.012178	0.000076	0.027823	0.357795	0.896856
MODAL	Mode	52.000000	0.028189	0.023713	0.000349	0.032400	0.381508	0.897205
MODAL	Mode	53.000000	0.027496	0.012505	0.001152	0.011996	0.394013	0.898357
MODAL	Mode	54.000000	0.027385	0.008744	0.001425	0.000107	0.402758	0.899782
MODAL	Mode	55.000000	0.026955	0.006122	0.000019	0.003727	0.408879	0.899802
MODAL	Mode	56.000000	0.026669	0.006180	0.001187	0.000239	0.415060	0.900989
MODAL	Mode	57.000000	0.026421	0.004929	1.455E-06	0.002927	0.419988	0.900990
MODAL	Mode	58.000000	0.025545	0.000604	0.001373	1.030E-06	0.420592	0.902364
MODAL	Mode	59.000000	0.024966	0.008190	0.003172	0.000072	0.428782	0.905536
MODAL	Mode	60.000000	0.023835	0.000432	5.359E-06	0.264635	0.429213	0.905542

8.1.5 Slenderness.

- **Y-Direction.**

  Unbraced Column

  He= 1.3*10= 13m

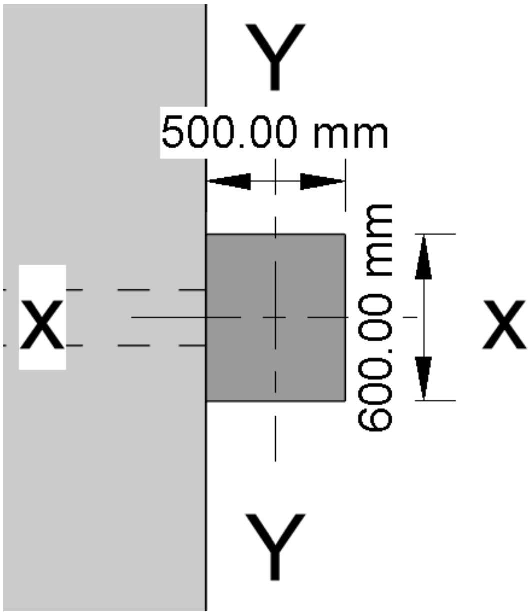
  λ= 13/0.6= 21 > 23 ………………… (Slender Column)

- **X-Direction.**

  Unbraced Column

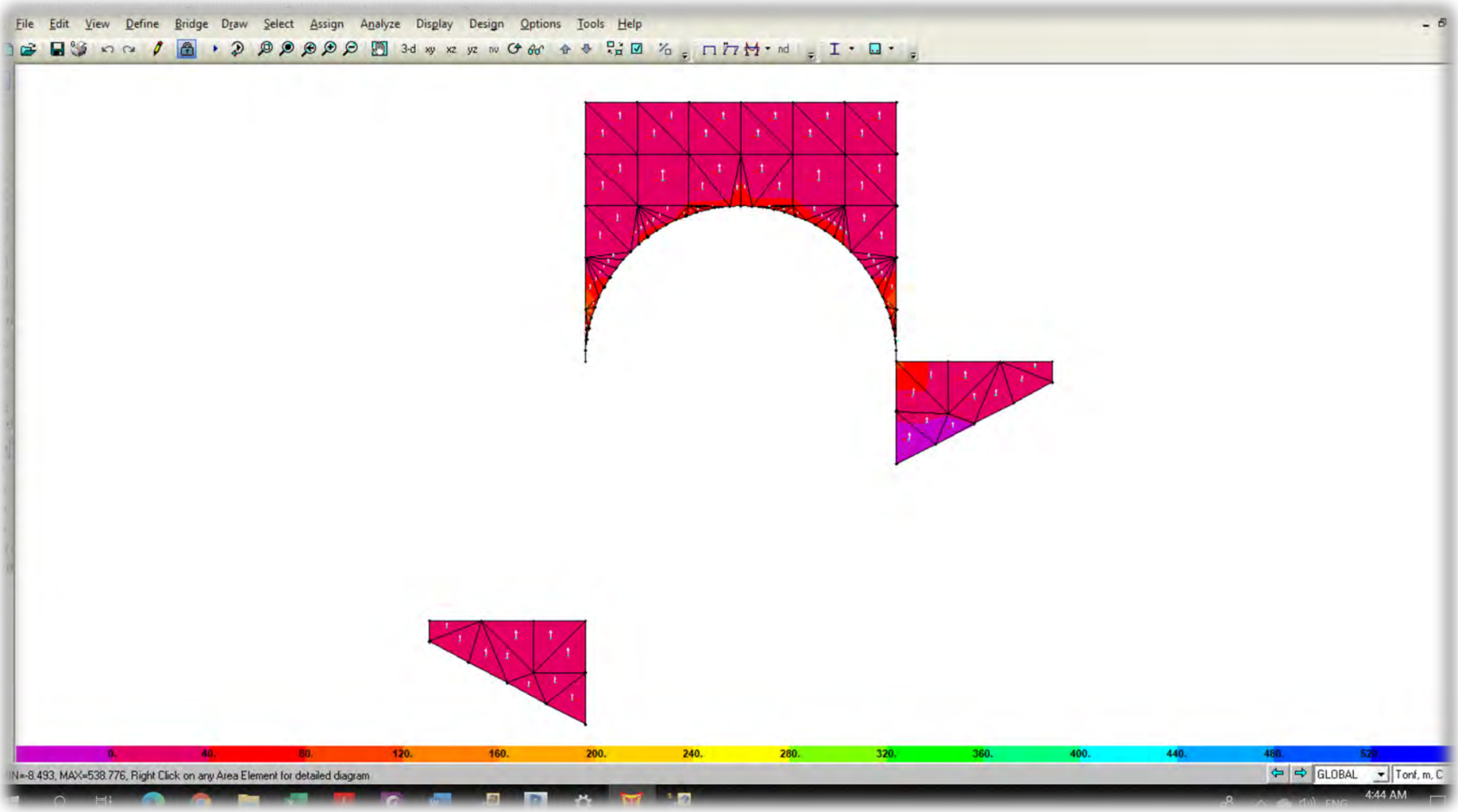
  He= 1.3*8= 10.4m

  λ= 10.4/0.5= 20.8 > 23 ……………. (Slender Column)



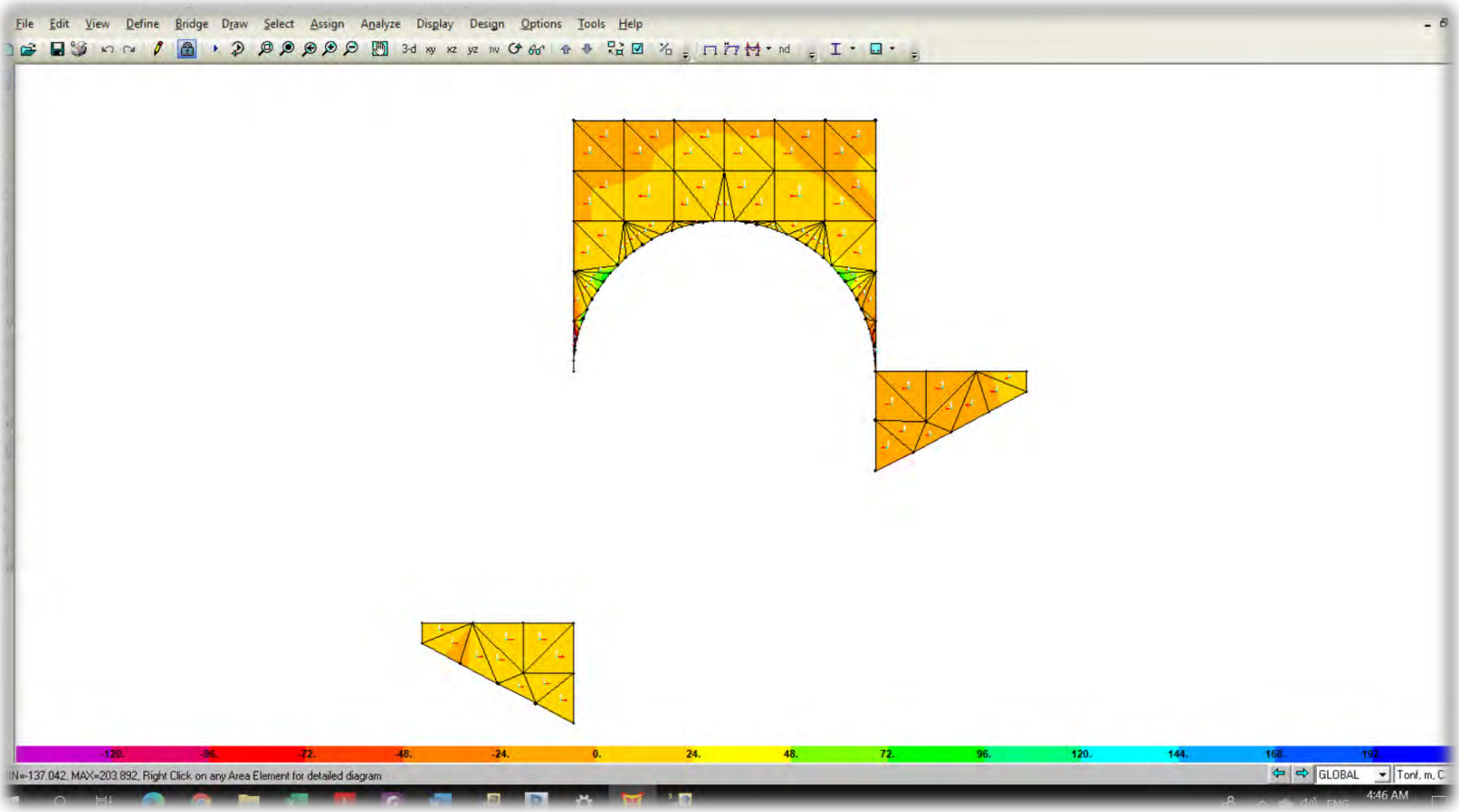
8.2 Analysis.

8.2.1 Beam and Cantilevers Analysis.



IN=-8.493, MAX=538.776, Right Click on any Area Element for detailed diagram

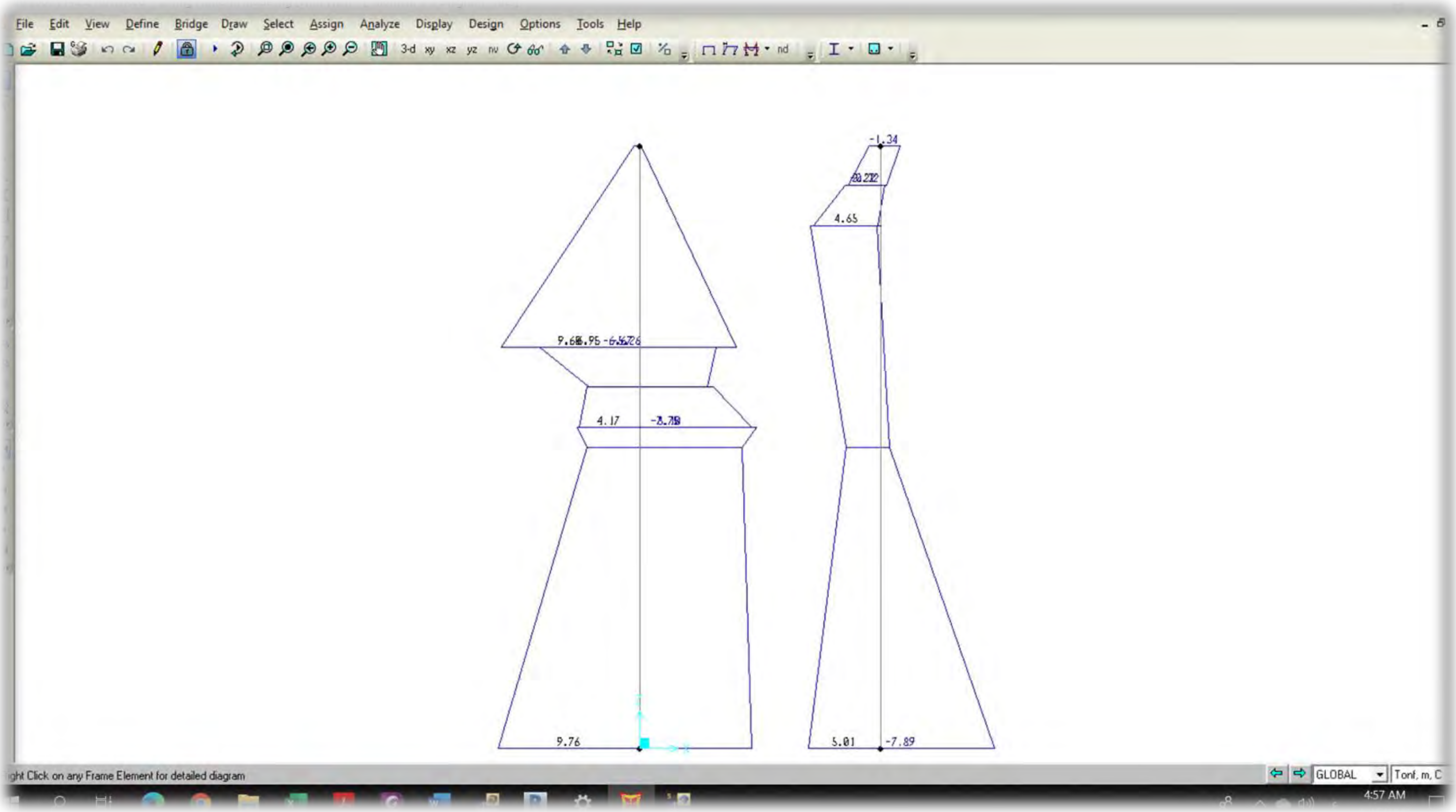
Figure 8.1: Internal Force in local axis 1 (red axis) direction (T-M’).



IN=-137.042, MAX=203.892, Right Click on any Area Element for detailed diagram

Figure 8.2: Internal Force in local axis 2 (white axis) direction (T-M’).

8.2.2 Columns Analysis.



ght Click on any Frame Element for detailed diagram

Figure 8.3: Bending Moment around Global axis Y (T.M').

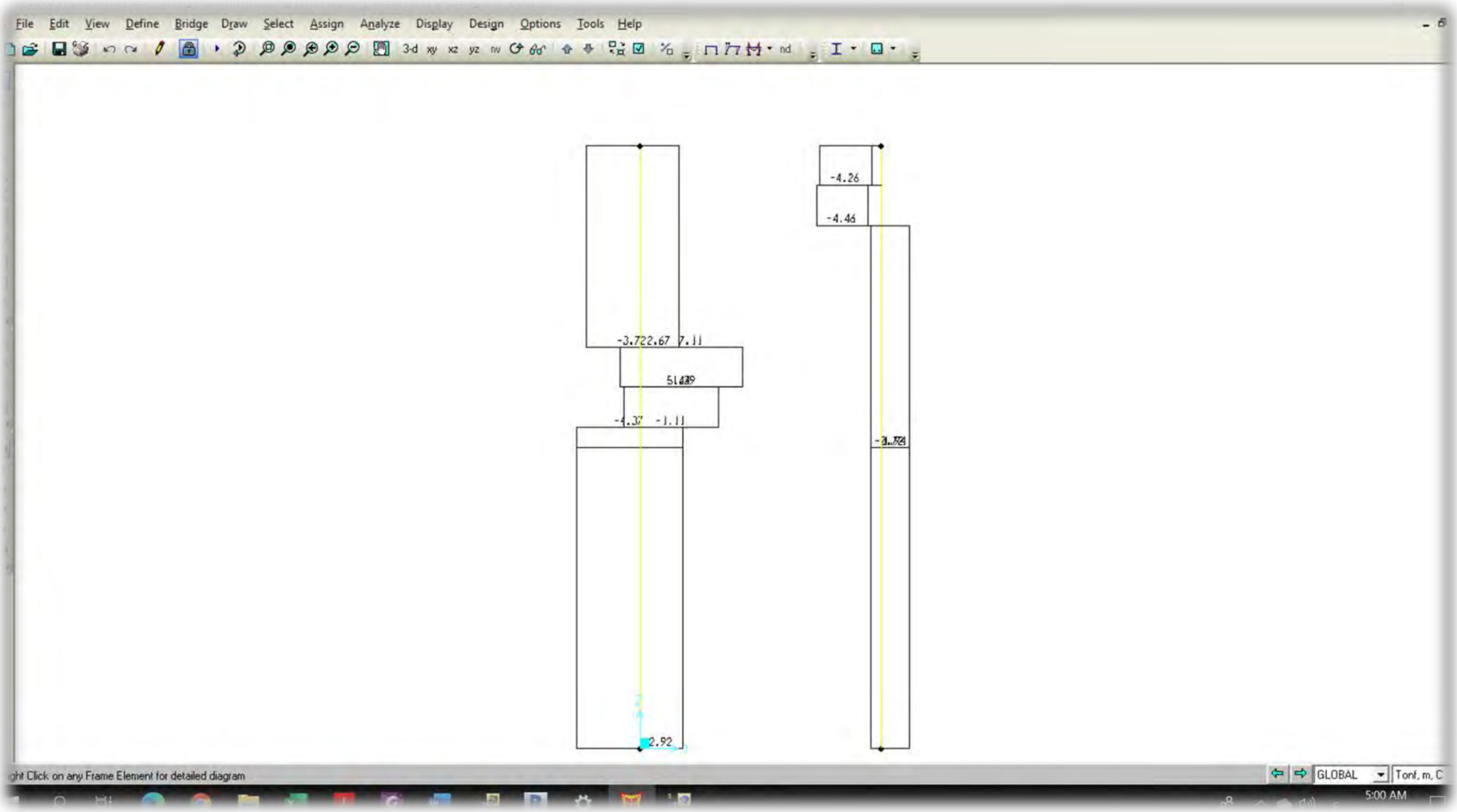
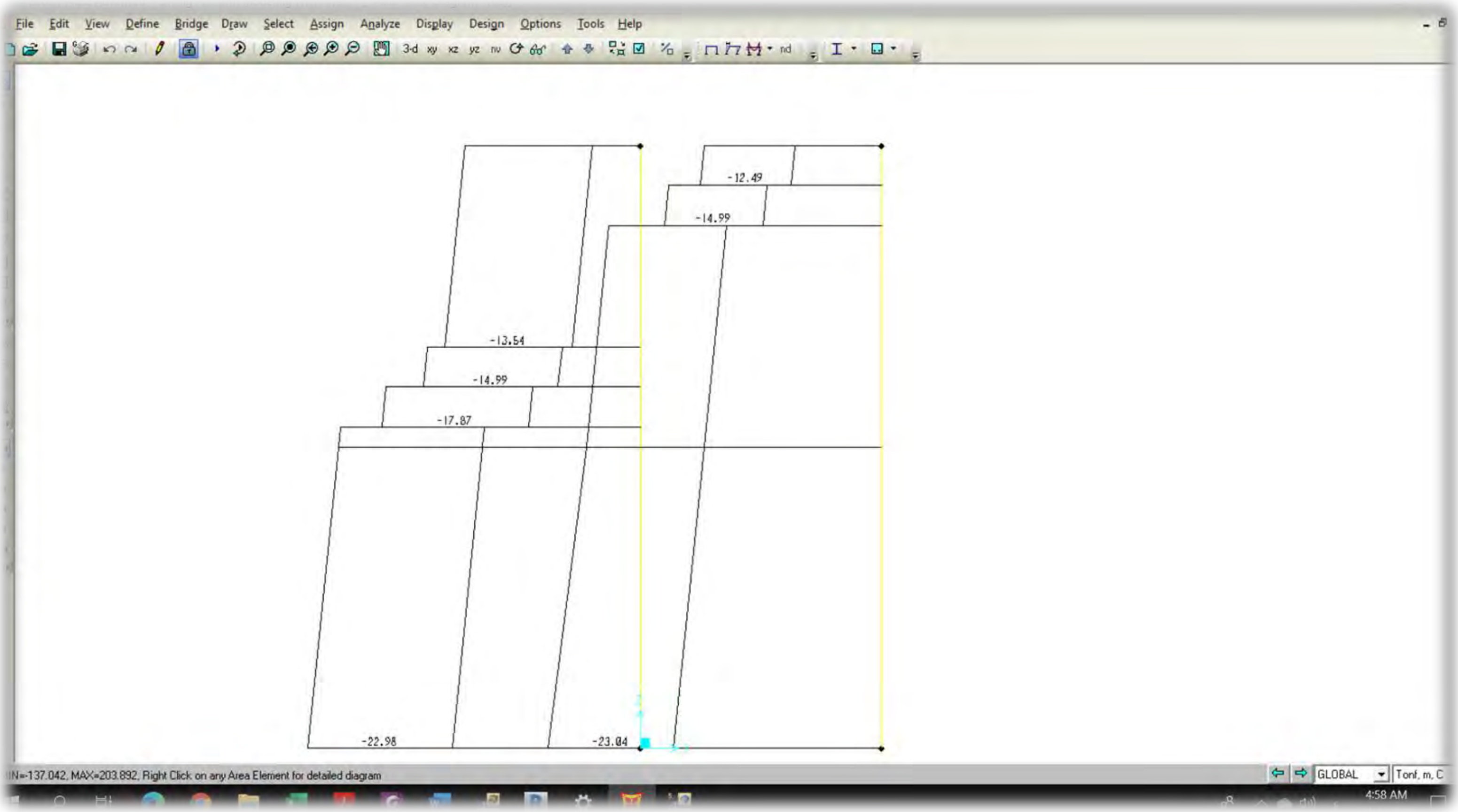


Figure 8.4: Shear Forces in Global axis X direction (T-M’).



IN=-137.042, MAX=203.892, Right Click on any Area Element for detailed diagram

Figure 8.5: Axial Forces in Global axis Z direction (T-M').

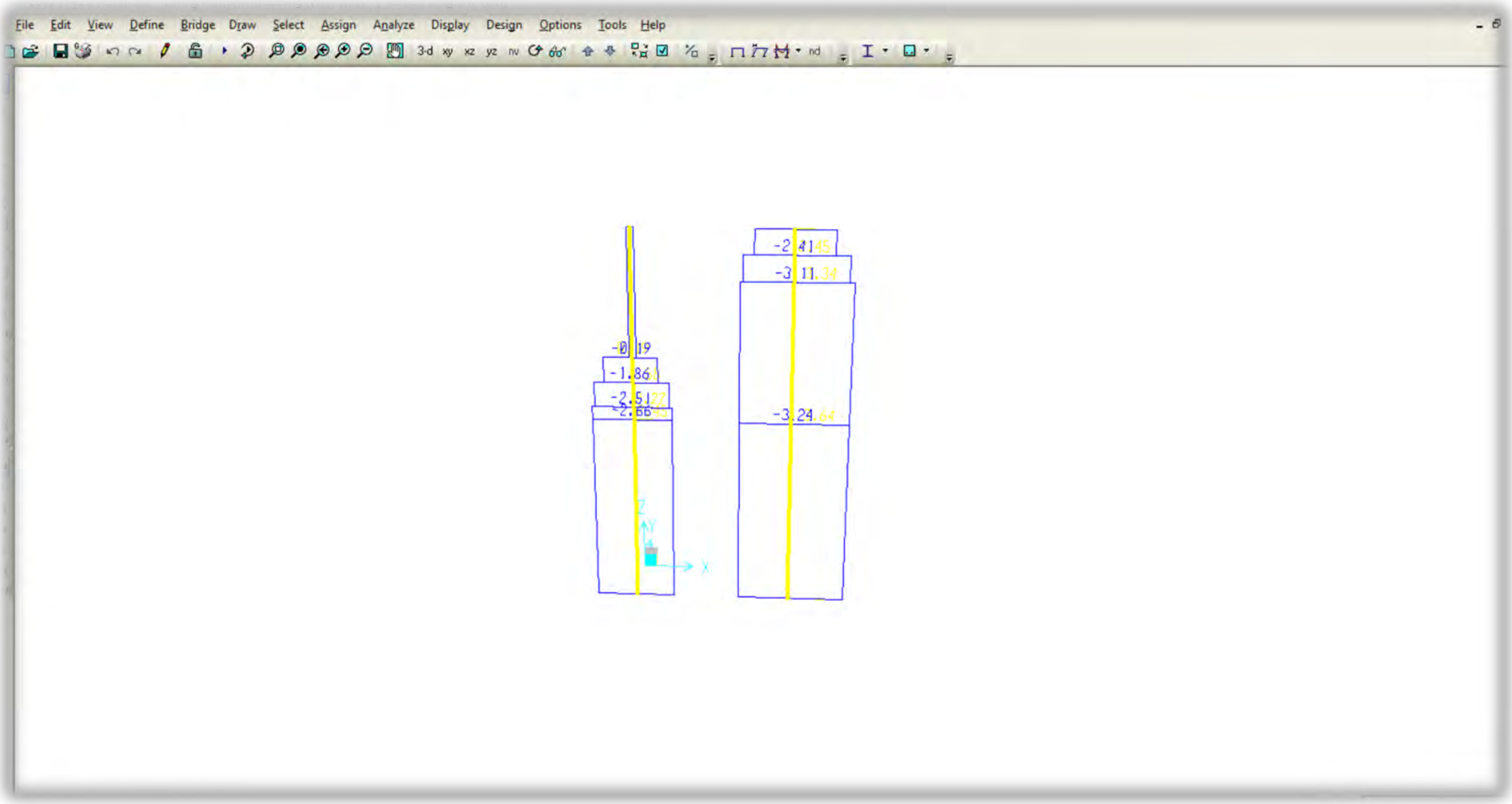
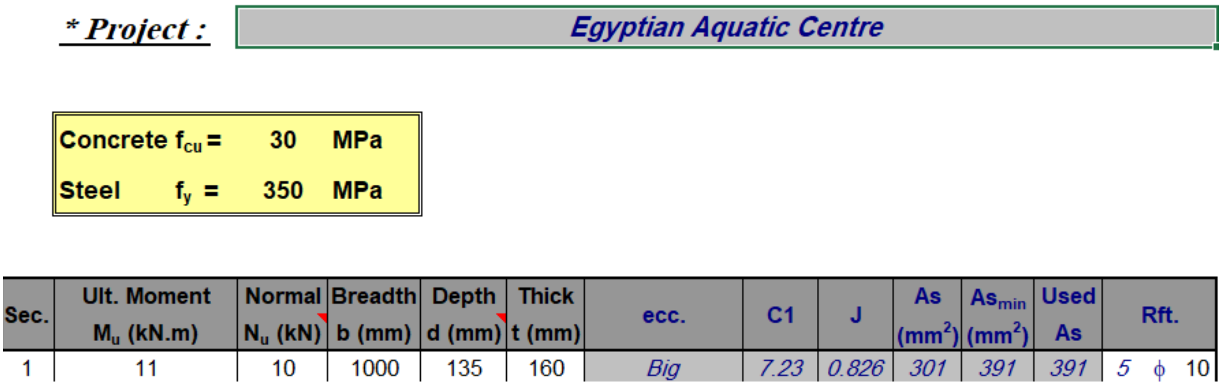


Figure 8.6: Torsion Forces in Global axis Z direction (T-M’).

8.3 Design.

8.3.1 Platforms & Wall

- By assume using 5 T 10/m’ mesh in the short and long directions, the covered bending moment induced from 5 T 10/m’ mesh is show in Fig. 8.6.
- No additional reinforcement required as show in Fig. 8.7 and Fig. 8.8.



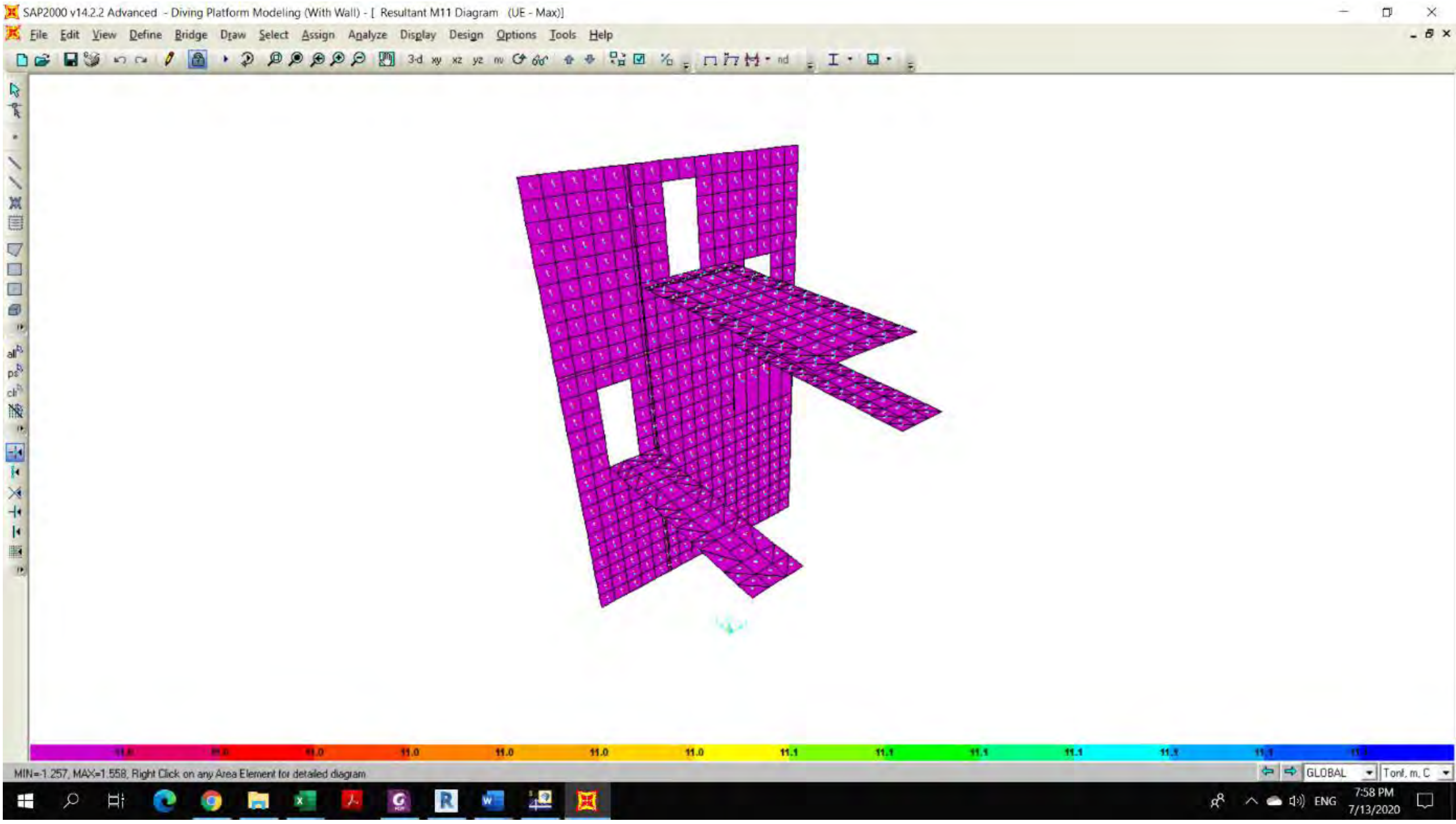
***** Project :** *Egyptian Aquatic Centre*

Concrete f_{cu} = 30 MPa

Steel f_y = 350 MPa

Sec.	Ult. Moment M_u (kN.m)	Normal N_u (kN)	Breadth b (mm)	Depth d (mm)	Thick t (mm)	ecc.	C1	J	As (mm^2)	As$_{min}$ (mm^2)	Used As	Rft.
1	11	10	1000	135	160	*Big*	*7.23*	*0.826*	*301*	*391*	*391*	*5* ϕ 10

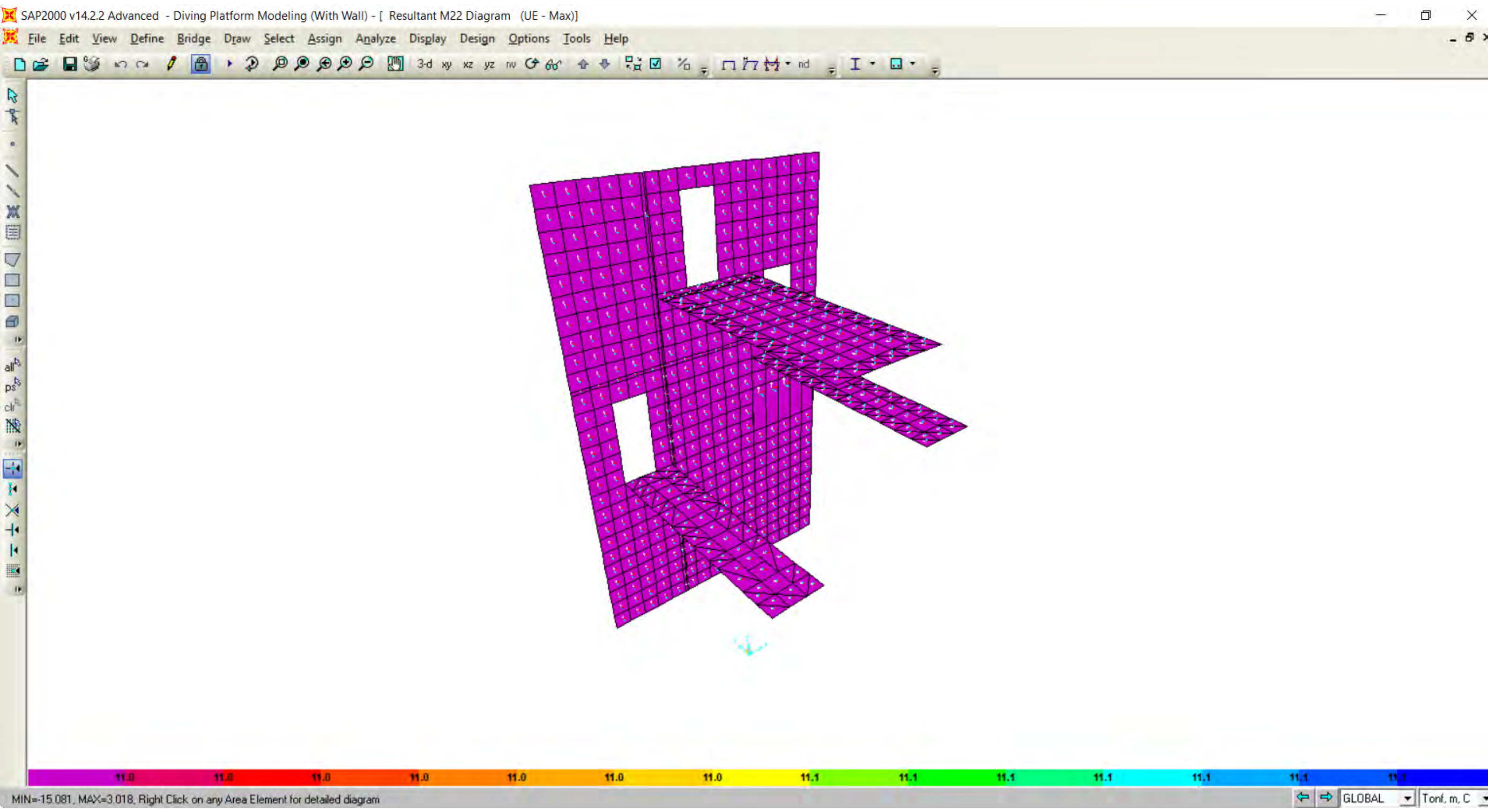
Figure 8.7: Covered Bending moment induced from 5 T 10/m’ mesh



SAP2000 v14.2.2 Advanced - Diving Platform Modeling (With Wall) - [Resultant M11 Diagram (UE - Max)]

MIN=-1.257, MAX=1.558, Right Click on any Area Element for detailed diagram

Figure 8.8: Additional Reinforcement required in local axes 1 direction (short direction)



SAP2000 v14.2.2 Advanced - Diving Platform Modeling (With Wall) - [Resultant M22 Diagram (UE - Max)]

MIN=-15.081, MAX=3.018, Right Click on any Area Element for detailed diagram

Figure 8.9: Additional Reinforcement required in local axes 2 direction (long direction)

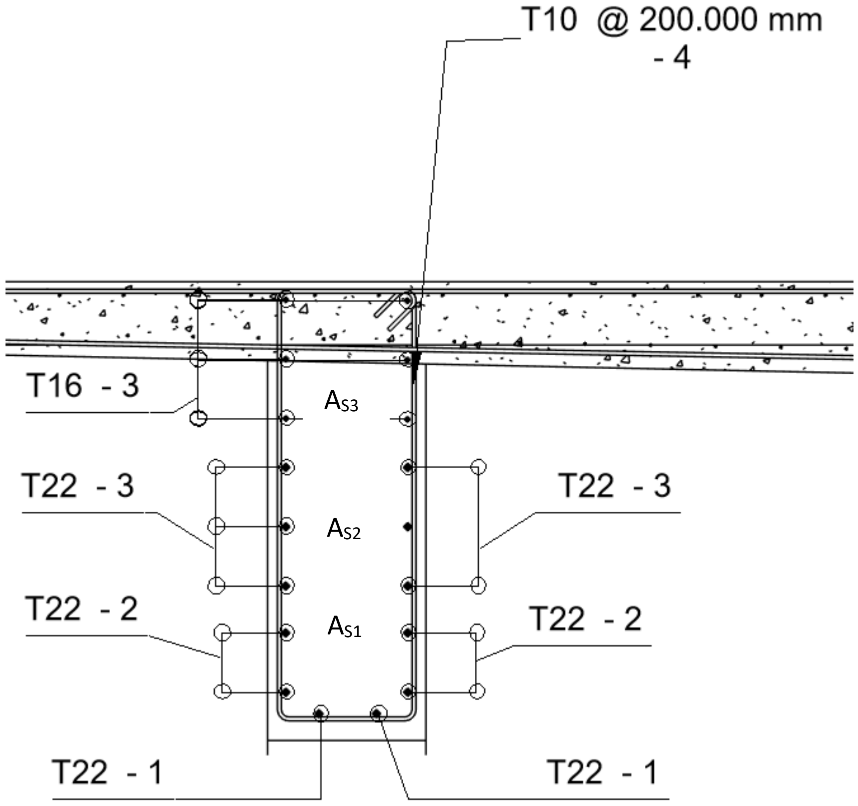
8.3.2 Beam Design.

8.3.2.1 Horizontal & Longitudinal RFT

- $A_{s1} = \frac{63*10000}{(\frac{360}{1.15})} = 2012.5 \text{ mm}^2$ ……………….assume using T 22 ……………………Use 6 T 22
- $A_{s2} = \frac{22*10000}{(\frac{360}{1.15})} = 703 \text{ mm}^2$ …………………….assume using T 22 ……………………Use 2 T 22
- $A_{s3} = \frac{12*10000}{(\frac{360}{1.15})} = 384 \text{ mm}^2$ ……………….……..assume using T 16 ……………..……Use 2 T 116

8.3.2.2 Stirrups.

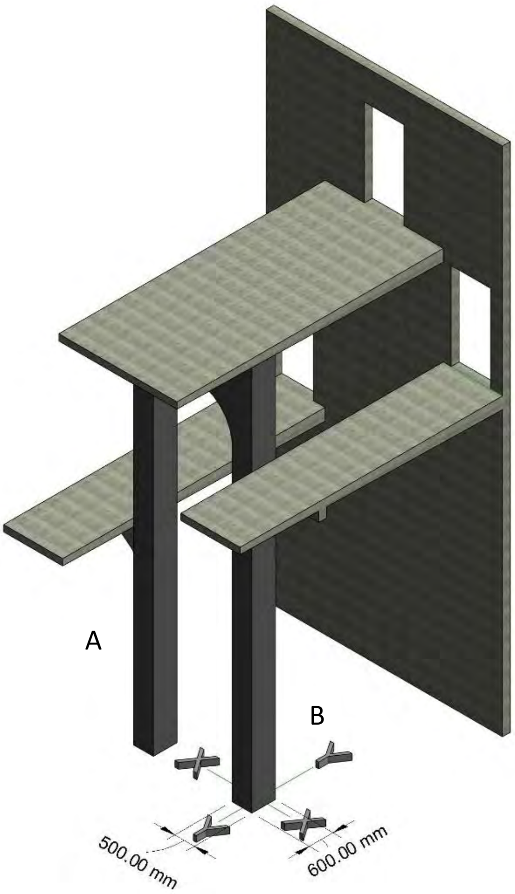
- $A_{st} = \frac{11*10000}{(\frac{360}{1.15})} = 352 \text{ mm}^2$ ……………………………………………………….Use stirrups 5 T 10/m'



8.3.3 Columns Design.

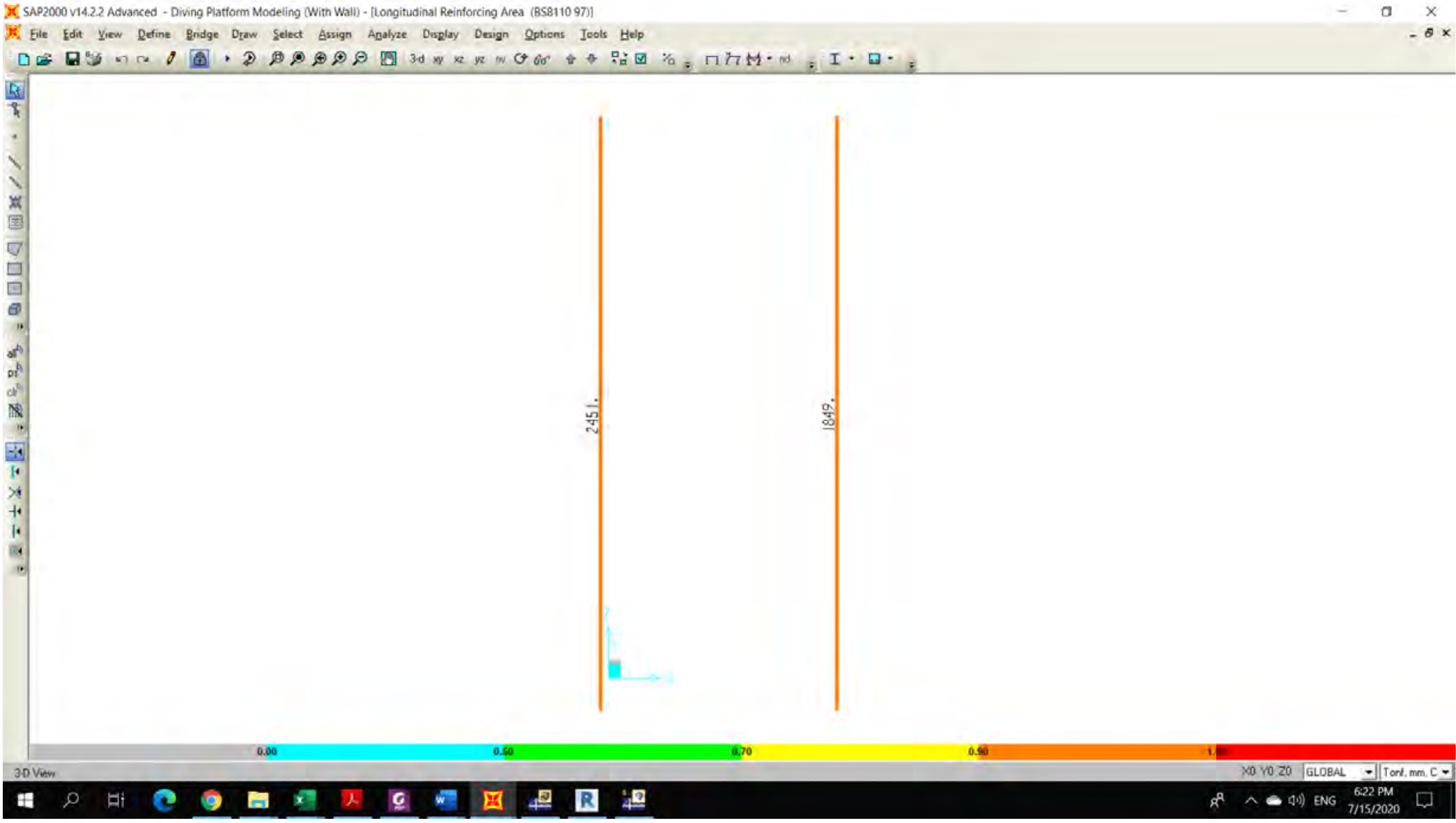
8.3.3.1 Longitudinal RFT

- б$_x$= (20.8^2 *0.5) / (2000)= 0.108 m
- б$_y$= (21^2 *0.6) / (2000)= 0.132 m
- M$_{add.x}$= 240*0.132= 31.68 KN.m
- M$_{add.y}$= 240*0.108=25.92 KN.m



- **Column A (Left Column).**
  - Mx= 149.8 KN.m
  - My= 56.5 KN.m
  - M$_{d\text{-}x}$= 149.8 + 31.68= 181.48 KN.m
  - M$_{d\text{-}y}$= 56.5 + 25.92= 82.42 KN.m
  - Pu= 240 KN
  - Use top and bottom steel ȧ=1
  - $\zeta = \frac{550-50}{600} = 0.83$
  - $R_b = \frac{pu}{Fcu*b*t} = \frac{240*1000}{30*500*600} = 0.027$
  - $\frac{Mdx}{Fcu*b*t^2} = \frac{181.48*1000000}{30*500*600^2} = 0.034$
  - $\frac{Mdy}{Fcu*b*t^2} = \frac{82.42*1000000}{30*600*500^2} = 0.0183$
  - From Interaction Diagram, ρ = 3
  - $\mu= 3 * 30*10^{-4} =9*10^{-3}$
  - As= $9*10^{-3} * 600*500 = 2700$ mm^2…………..Use 14 T 16
  - Torsion RFT. (see part 8.3.2.2) …… 7 T 16
  - Total vertical RFT. equal 22 T 16
- **Column B (Right Column).**
  - Mx= 100.5 KN.m
  - My= 37.5 KN.m
  - M$_{d\text{-}x}$= 100.5 + 31.68= 132.18 KN.m
  - M$_{d\text{-}y}$= 37.5 + 25.92= 63.42 KN.m
  - Pu= 240 KN
  - Use top and bottom steel ȧ=1
  - $\zeta = \frac{550-50}{600} = 0.83$
  - $R_b = \frac{pu}{Fcu*b*t} = \frac{240*1000}{30*500*600} = 0.027$
  - $\frac{Mdx}{Fcu*b*t^2} = \frac{132.18*1000000}{30*500*600^2} = 0.025$
  - $\frac{Mdy}{Fcu*t*b^2} = \frac{63.42*1000000}{30*600*500^2} = 0.014$
  - From Interaction Diagram, ρ = 2.1
  - $\mu= 2.1 * 30*10^{-4} =6.3*10^{-3}$
  - As= $6.3*10^{-3} * 600*500 = 1890$ mm^2………..Use 10 T 16
  - Torsion RFT. (see part 8.3.2.2) …… 6 T 16
  - Total vertical RFT. equal 16 T 16

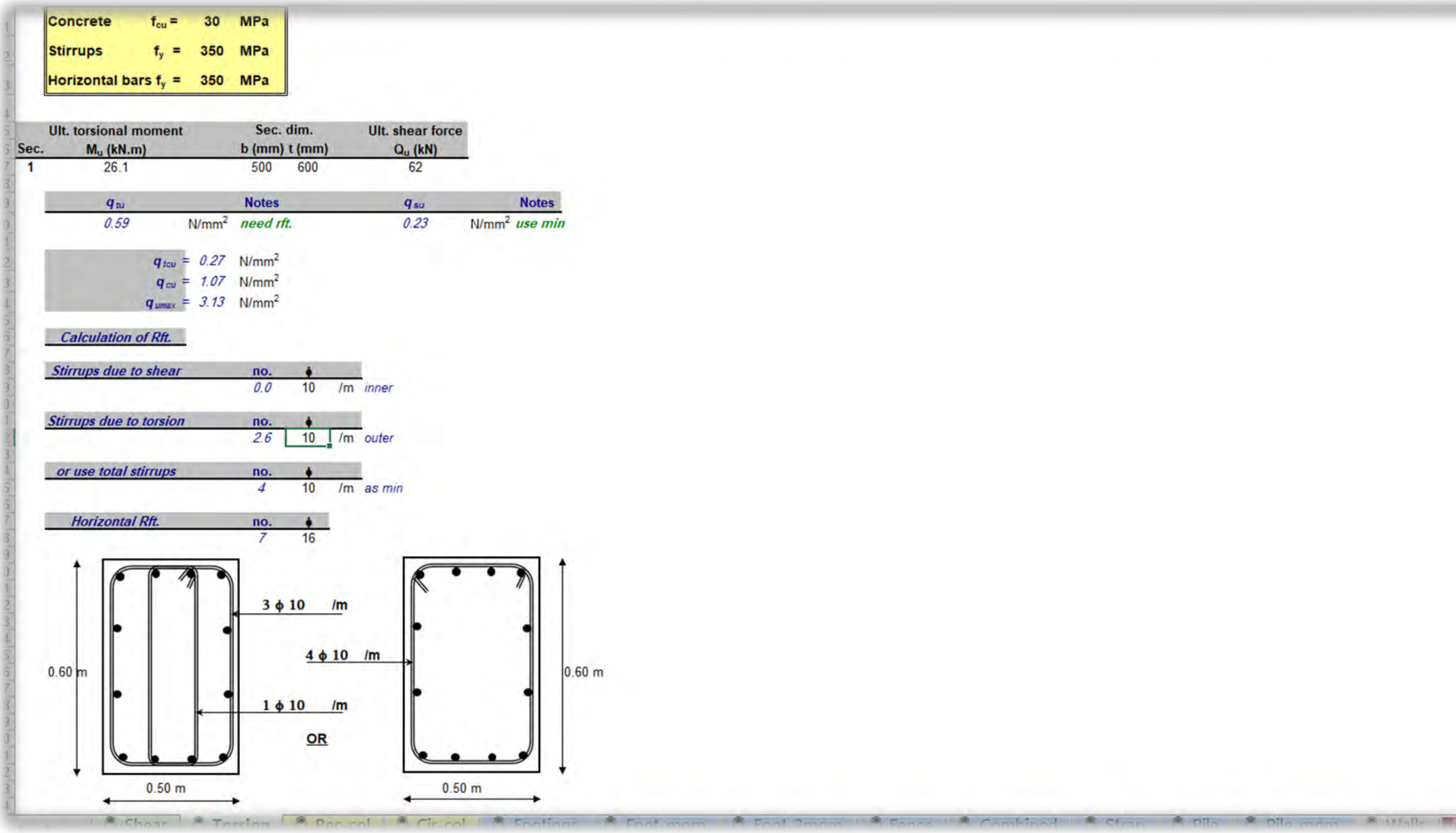
Results assessment by using Sap 2000 software (Design according to BS8110 97)



SAP2000 v14.2.2 Advanced - Diving Platform Modeling (With Wall) - [Longitudinal Reinforcing Area (BS8110 97)]

8.3.2.2 Stirrups.

- **Column A (Left Column).**



Concrete	f_{cu} =	30	MPa
Stirrups	f_y =	350	MPa
Horizontal bars	f_y =	350	MPa

Sec.	Ult. torsional moment M_u (kN.m)	Sec. dim. b (mm)	t (mm)	Ult. shear force Q_u (kN)
1	26.1	500	600	62

q_{tu}		Notes	q_{su}		Notes
0.59	N/mm²	need rft.	0.23	N/mm²	use min

q_{tcu} = 0.27 N/mm²
q_{cu} = 1.07 N/mm²
q_{umax} = 3.13 N/mm²

Calculation of Rft.

Stirrups due to shear	no.	ϕ		
	0.0	10	/m	inner

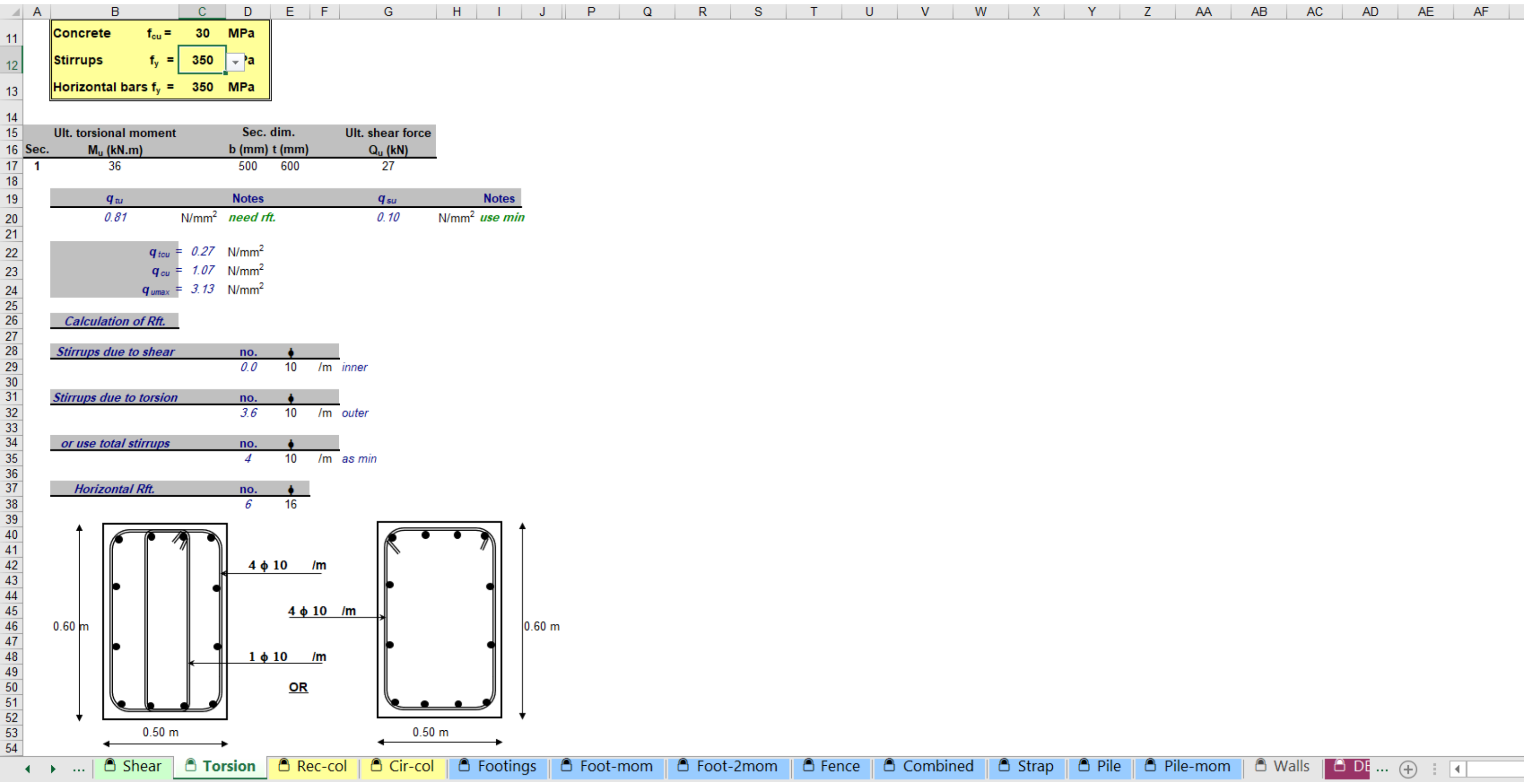
Stirrups due to torsion	no.	ϕ		
	2.6	10	/m	outer

or use total stirrups	no.	ϕ		
	4	10	/m	as min

Horizontal Rft.	no.	ϕ
	7	16

.

- **Column B (Right Column).**



Concrete f_{cu} = 30 MPa

Stirrups f_y = 350 ?a

Horizontal bars f_y = 350 MPa

Sec.	Ult. torsional moment M_u (kN.m)	Sec. dim. b (mm)	t (mm)	Ult. shear force Q_u (kN)
1	36	500	600	27

q_{tu}		Notes	q_{su}		Notes
0.81	N/mm^2	need rft.	0.10	N/mm^2	use min

q_{tcu} = 0.27 N/mm^2

q_{cu} = 1.07 N/mm^2

q_{umax} = 3.13 N/mm^2

Calculation of Rft.

Stirrups due to shear	no.	ɸ		
	0.0	10	/m	inner

Stirrups due to torsion	no.	ɸ		
	3.6	10	/m	outer

or use total stirrups	no.	ɸ		
	4	10	/m	as min

Horizontal Rft.	no.	ɸ
	6	16

8.3.4 Cantilevers

- Horizontal RFT.
  F11= 18 T/m'
  By using Interior and Exterior mesh, the F_{11} for eash mesh will equal 18/2= 9 T/m'

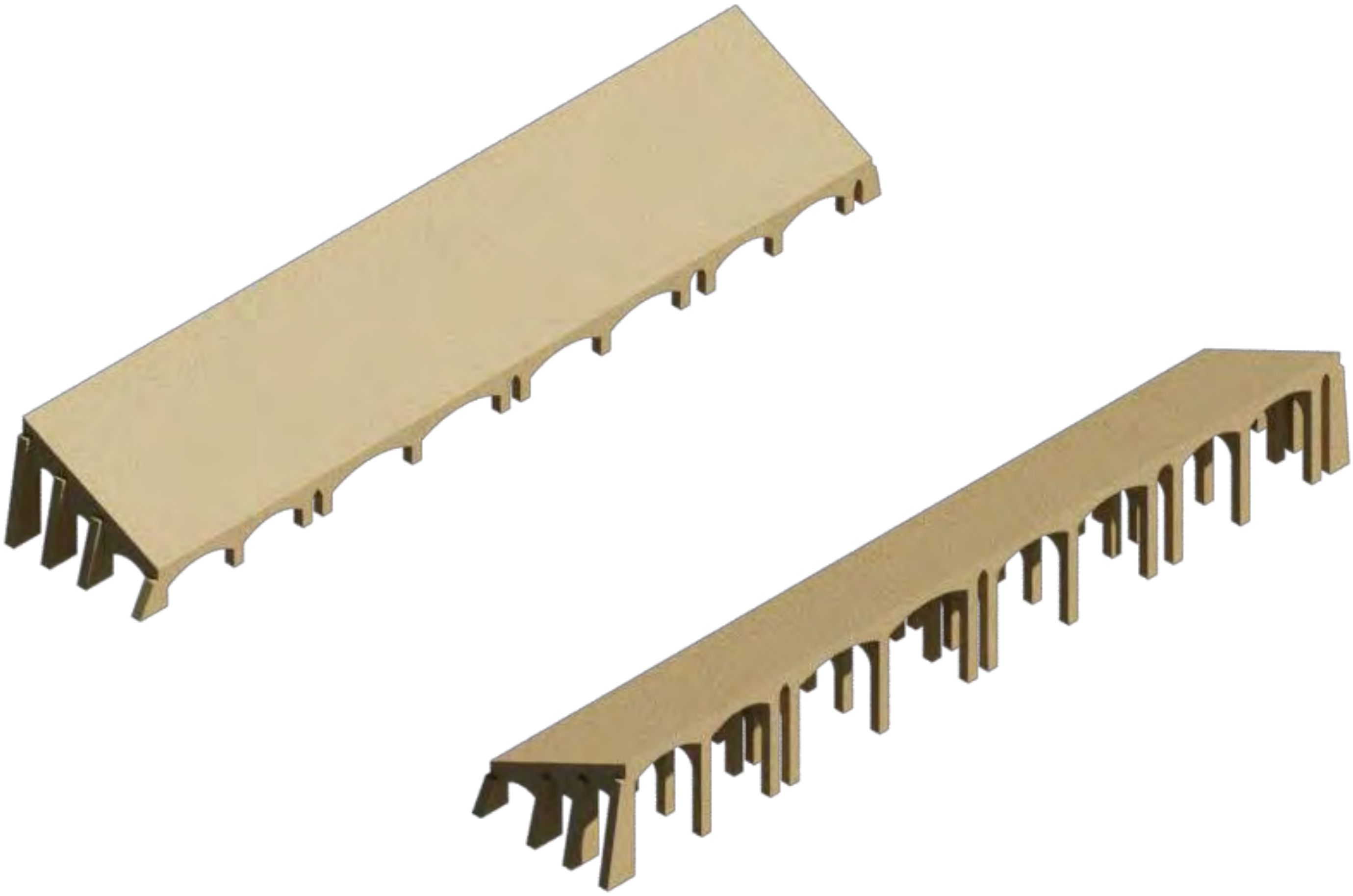
  As = $\frac{9*10000}{360/1.15}$ = 288 mm^2 Use …………………………………………………..Use 5 T 10 / m'

- Vertical RFT.
  F22= 6 T/m'
  By using Interior and Exterior mesh, the F_{22} for eash mesh will equal 6/2= 3 T/m'

  As = $\frac{3*10000}{360/1.15}$ = 96 mm^2 Use …………………………………………………..Use 5 T 10 / m'

Egyptian Aquatic Centre

Coliseum



I. Design codes and standards

1- **EN 1996-1-1**

Eurocode 6 - Design of masonry structures - Part 1-1: General rules for reinforced and unreinforced masonry structures.

2- **ECP (201-2010)**

Egyptian Code for Loading on Buildings.

3- **EN 1990:2002+A1**

Eurocode - Basis of structural design.

4- **Structural Designer's Manual**

Second Edition.

5- **DESIGN OF MASONRY STRUCTURES Book**

Third edition of Load Bearing Brickwork Design.

6- **Design of Masonry Structures According Eurocode 6**.

Prof. em. Dr.-Ing. Wieland Ramm Technical University of Kaiserslautern.

7- **Ain Shams Engineering Journal .**

Study of physical and mechanical properties for some of Eastern Desert dimension marble and granite utilized in building decoration.

1. INTRODUCTION

The concept for structural design focuses on satisfying both the functional and the economic requirements of the building without jeopardizing its aesthetic and architectural features. This Calculation Report presents the structural engineering aspect of the works due for the development construction work of Egyptian Aquatic Centre.

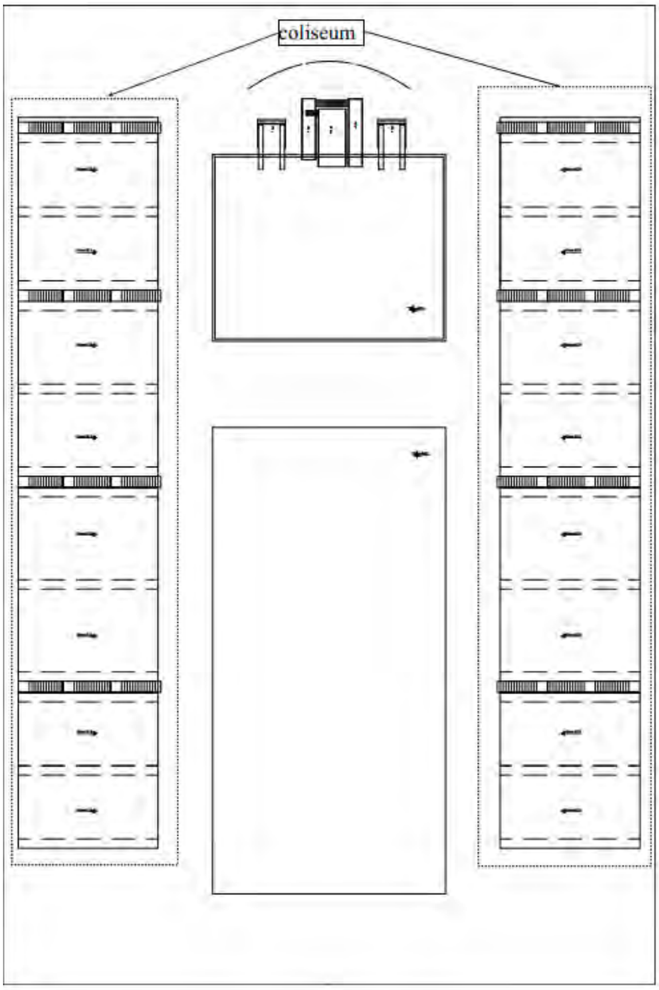
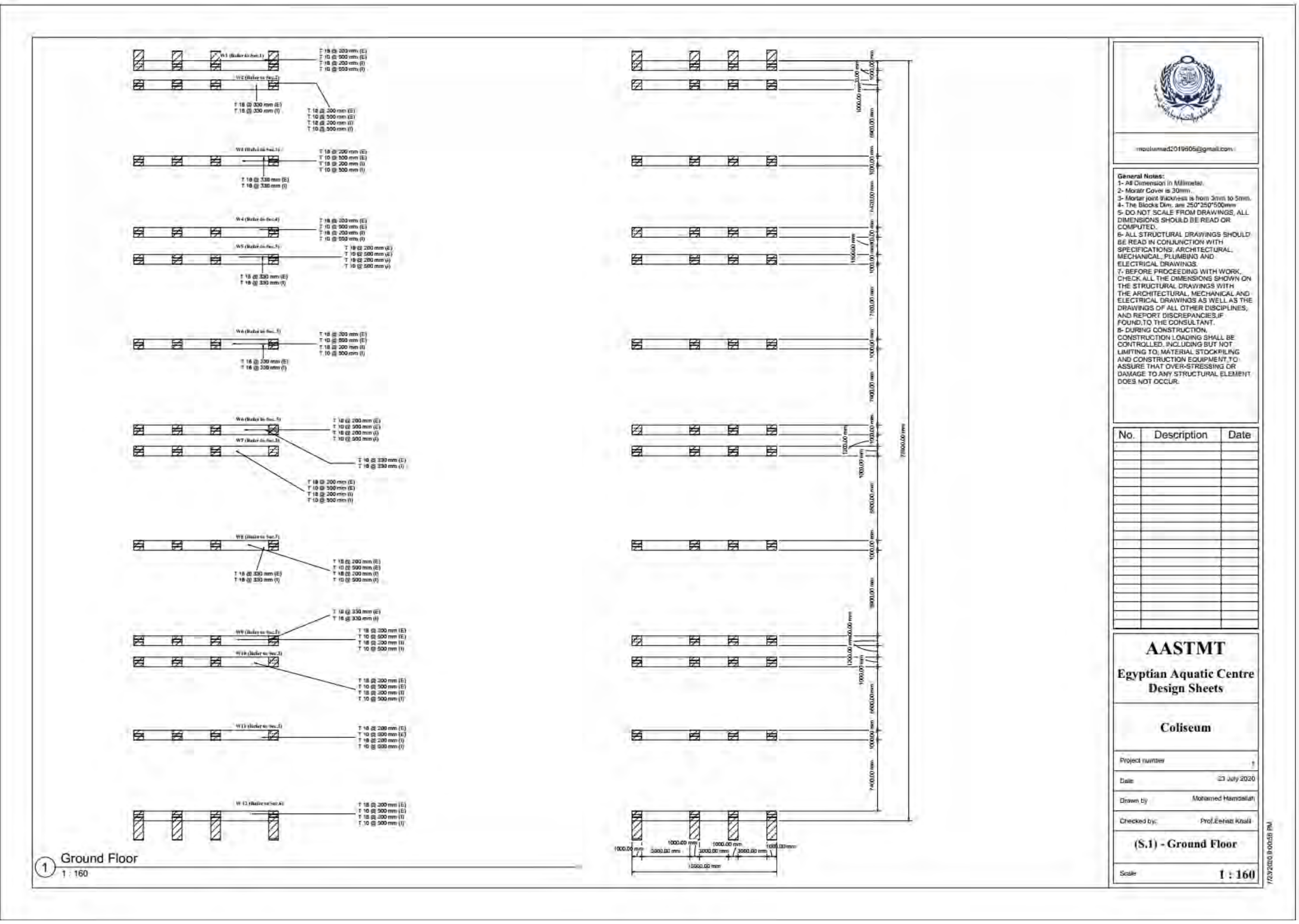


Figure 1-4 Coliseum location

2. Coliseum Drawings Details



General Notes:
1- All Dimension in Millimeter.
2- Moratr Cover is 30mm.
3- Mortar joint thickness is from 3mm to 5mm.
4- The Blocks Dim. are 250*250*500mm
5- DO NOT SCALE FROM DRAWINGS, ALL DIMENSIONS SHOULD BE READ OR COMPUTED.
6- ALL STRUCTURAL DRAWINGS SHOULD BE READ IN CONJUNCTION WITH SPECIFICATIONS, ARCHITECTURAL, MECHANICAL, PLUMBING AND ELECTRICAL DRAWINGS.
7- BEFORE PROCEEDING WITH WORK, CHECK ALL THE DIMENSIONS SHOWN ON THE STRUCTURAL DRAWINGS WITH THE ARCHITECTURAL, MECHANICAL AND ELECTRICAL DRAWINGS AS WELL AS THE DRAWINGS OF ALL OTHER DISCIPLINES, AND REPORT DISCREPANCIES,IF FOUND,TO THE CONSULTANT.
8- DURING CONSTRUCTION, CONSTRUCTION LOADING SHALL BE CONTROLLED, INCLUDING BUT NOT LIMITING TO; MATERIAL STOCKPILING AND CONSTRUCTION EQUIPMENT,TO ASSURE THAT OVER-STRESSING OR DAMAGE TO ANY STRUCTURAL ELEMENT DOES NOT OCCUR.

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Design Sheets

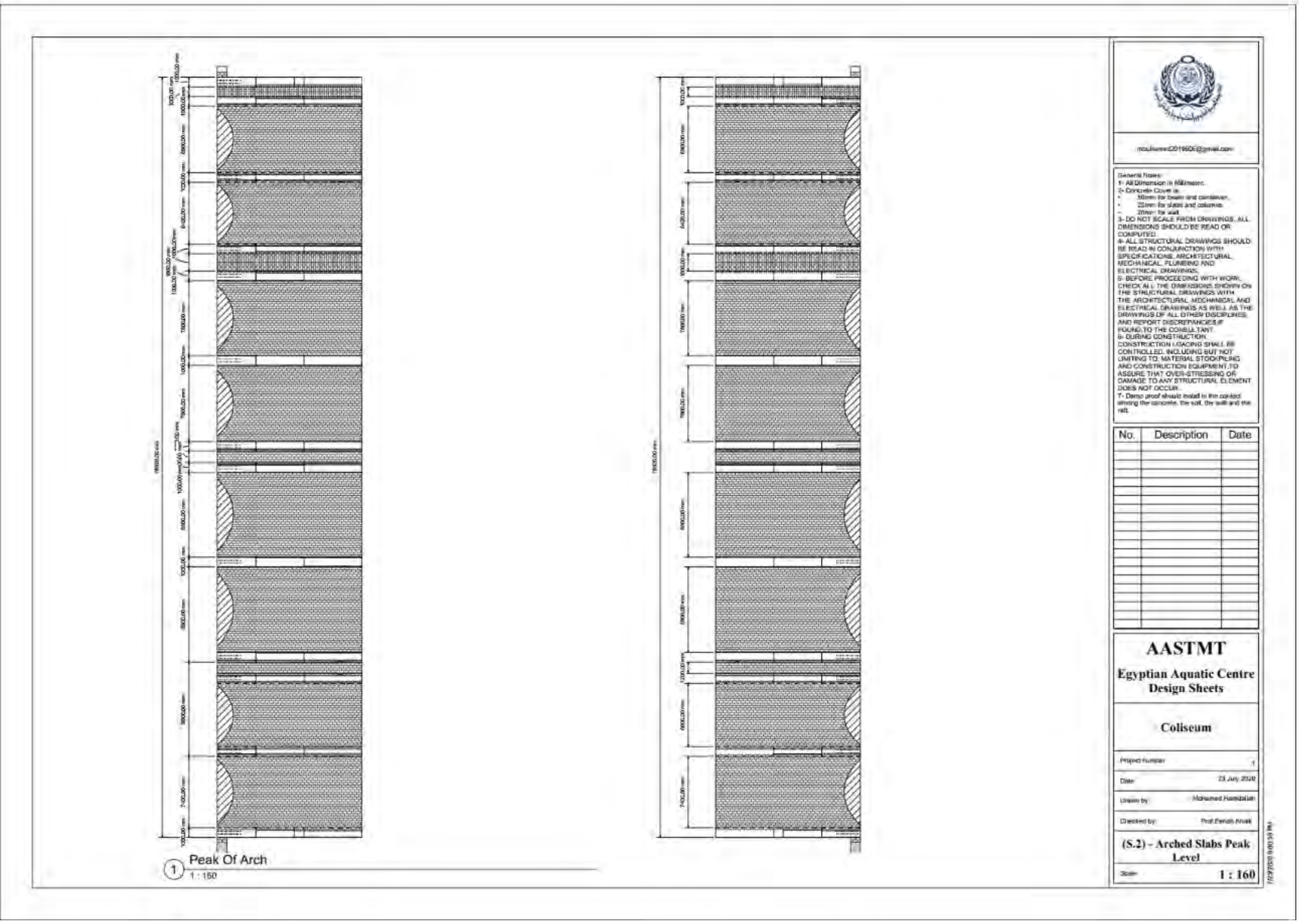
Coliseum

Project number 1
Date 23 July 2020
Drawn by Mohamed Hamdallah
Checked by: Prof.Eehab Khalil

(S.1) - Ground Floor

Scale **1 : 160**

Figure 2-: Ground Floor.



General Notes:
1- All Dimension in Millimeter.
2- Concrete Cover is:
- 50mm for beam and cantilever.
- 25mm for slabs and columns.
- 20mm for wall.
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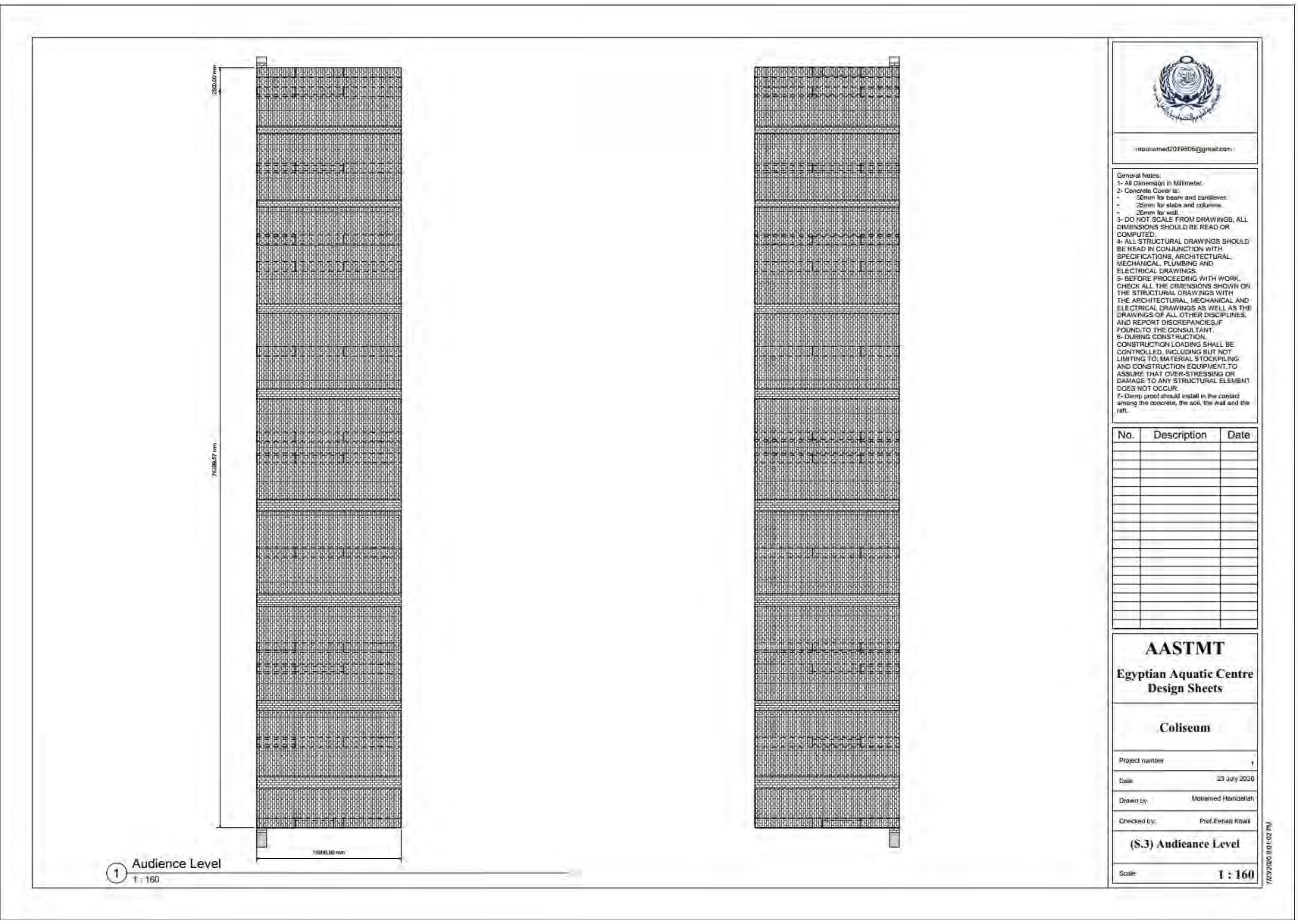
7- Damp proof should install in the contact among the concrete, the soil, the wall and the raft.

Figure 2-2: Arched Slabs Peak Level.



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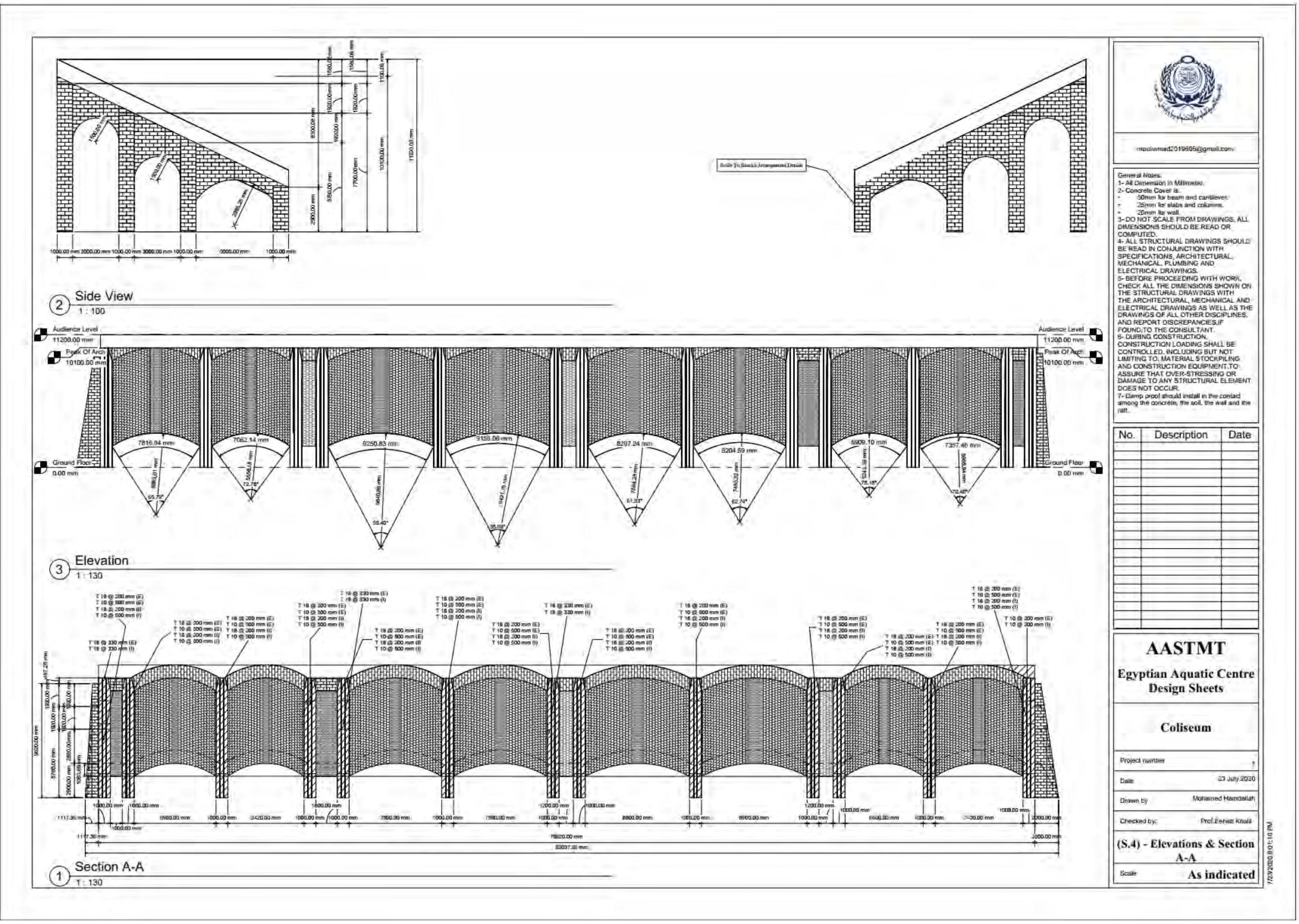
Coliseum

Project number 1
Date 23 July 2020
Drawn by: Mohamed Hamdallah
Checked by: Prof.Eehab Khalil

(S.3) Audieance Level

Scale **1 : 160**

Figure 2- 3: Audience Level.



General Notes:
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Coliseum

Project number: 1
Date: 23 July 2020
Drawn by: Mohamed Hamdallah
Checked by: Prof.Eehab Khalil
(S.4) - Elevations & Section A-A
Scale: As indicated
7/23/2020 8:01:10 PM

Figure 2- 4: Elevations & Section A-A.

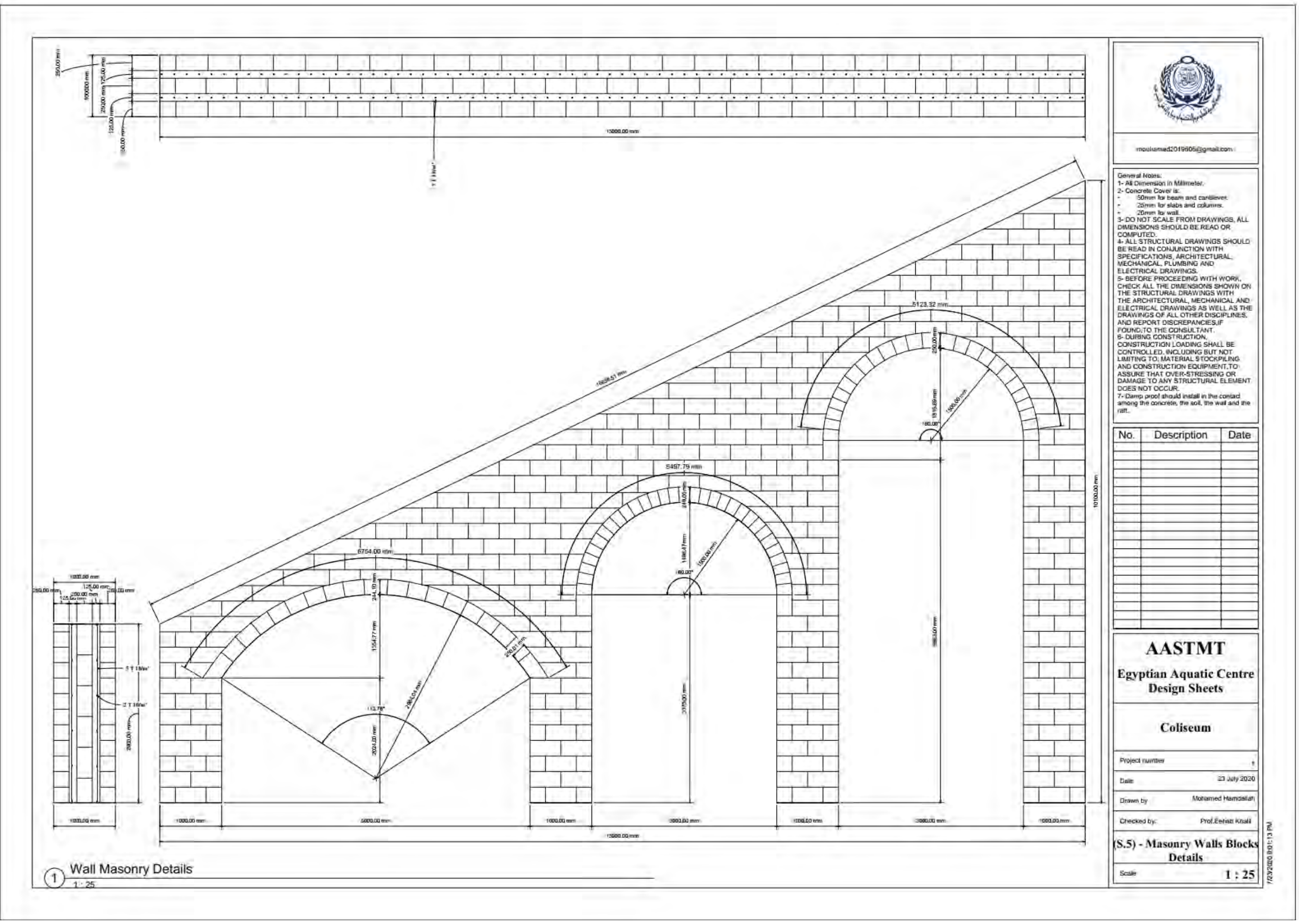
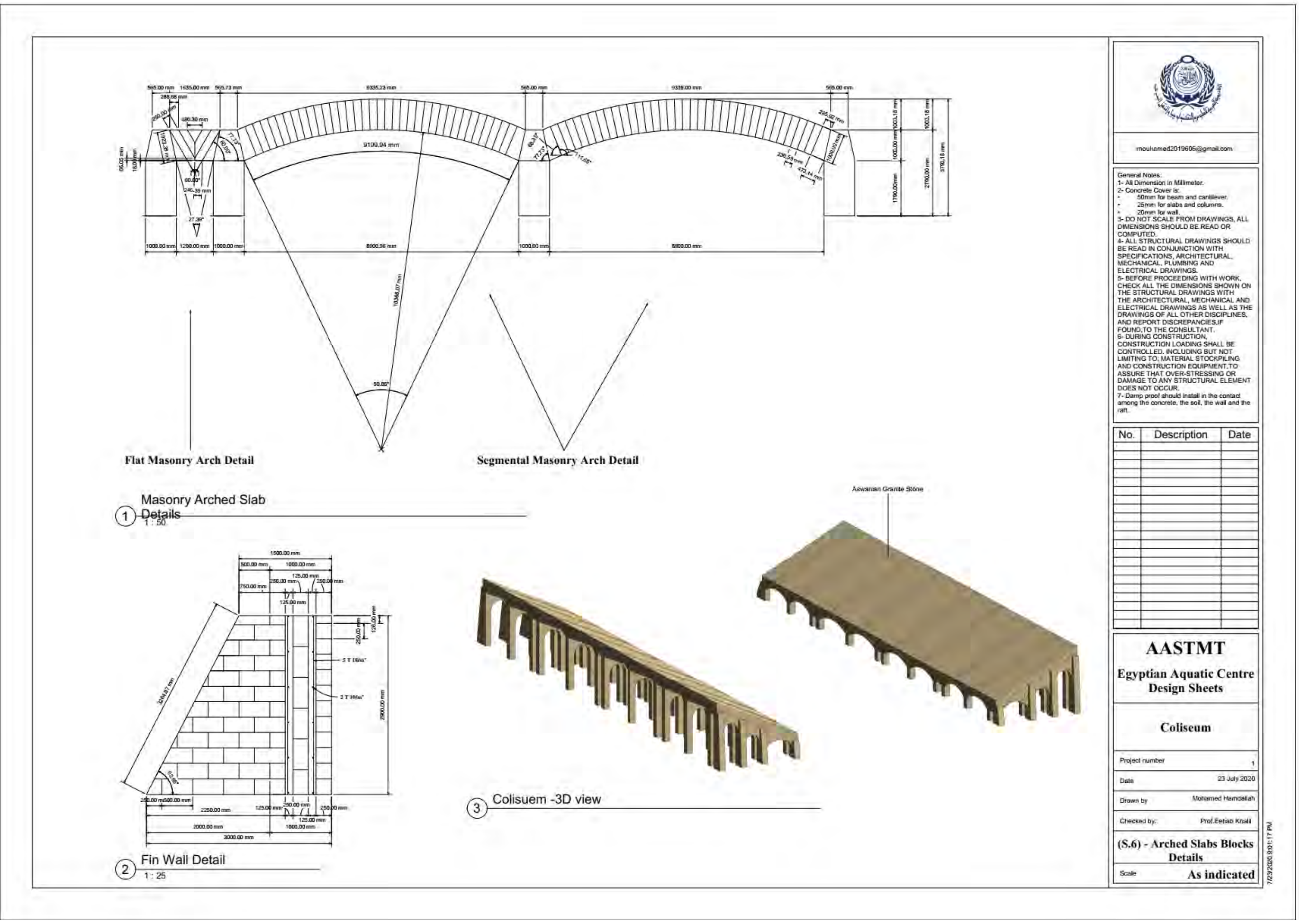


Figure 2- 5: Masonry Walls Blocks Details.



mouhamed2019606@gmail.com

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No.	Description	Date

AASTMT

Egyptian Aquatic Centre Design Sheets

Coliseum

Project number	1
Date	23 July 2020
Drawn by	Mohamed Hamdallah
Checked by:	Prof.Eehab Khalil

(S.6) - Arched Slabs Blocks Details

Scale	As indicated

Figure 2- 6: Arched Slabs Blocks Details.

3. Calculation Software Used

Calculation software features

The software used is *RFEM*, developed by DLUBAL COMPANY (Germany).

Technical specifications

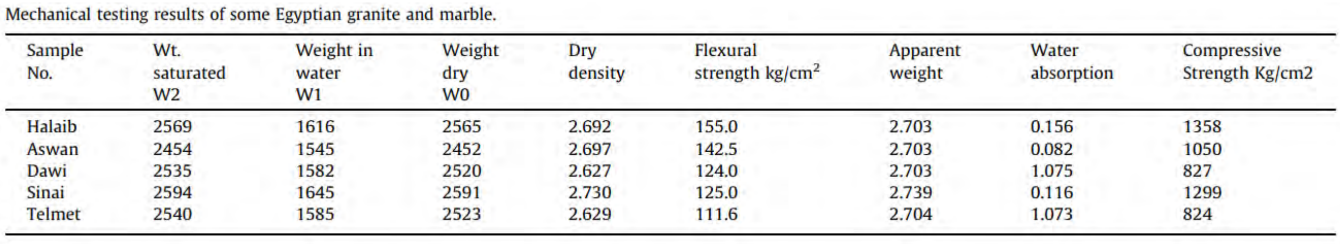
Name:	RFEM
Version:	5.22.03
Producer:	DLUBAL
	www.dlubal.com

License registered is a student license

4. OUTLINE SPECIFICATION AND MATERIAL PROPERITIES

Stone Masonry Blocks.

By using Red Aswanian Granit Blocks, the mechanical properties of Red Aswanian Granite shown in the following table (AinShams University Journal).



Mechanical testing results of some Egyptian granite and marble.

Sample No.	Wt. saturated W2	Weight in water W1	Weight dry W0	Dry density	Flexural strength kg/cm^2	Apparent weight	Water absorption	Compressive Strength Kg/cm2
Halaib	2569	1616	2565	2.692	155.0	2.703	0.156	1358
Aswan	2454	1545	2452	2.697	142.5	2.703	0.082	1050
Dawi	2535	1582	2520	2.627	124.0	2.703	1.075	827
Sinai	2594	1645	2591	2.730	125.0	2.739	0.116	1299
Telmet	2540	1585	2523	2.629	111.6	2.704	1.073	824

Mortar.

By using M12 mortar (the letter ‘M’ describes the compressive strength of the mortar), the mechanical properties of the mortar is:

- Compressive strength= 12 N/mm2.
- Mortar class= standard Mortar.
- Ratio g/t (width of the mortar bed to the thickness of bedded surface)= 1.
- Bed joint thickness around from 3mm to 5mm.
- Reinforced mortar cover= 30mm

Stone Masonry Combination.

According to EN 1996-1-1 Clause 3, the design characteristics strength of masonry combinations are:

- Compressive strength $_{Fk}$= 24.6 N/MM^2
- Flexural Tensile strength parallel to bed joint direction, Fxk1= 0.6 N/mm^2.
- Flexural Tensile strength perpendicular to bed joint direction, Fxk2= 1.2 N/mm^2.
- Shear strength $_{Fvk0}$= 0.6 N/mm^2.
- Final creep coeff.= 0.05
- Modules of Elasticity= 24600 N/mm^2
- Shear Modules= 10250 N/mm^2
- Poisson’s ratio= 0.2
- Partial Safety Factor (assume high quality in manufacturing and execution)= 2

Reinforcing Steel.

Reinforcing steel bars shall be uncoated high yield deformed bars of characteristic strength 360 N/mm².

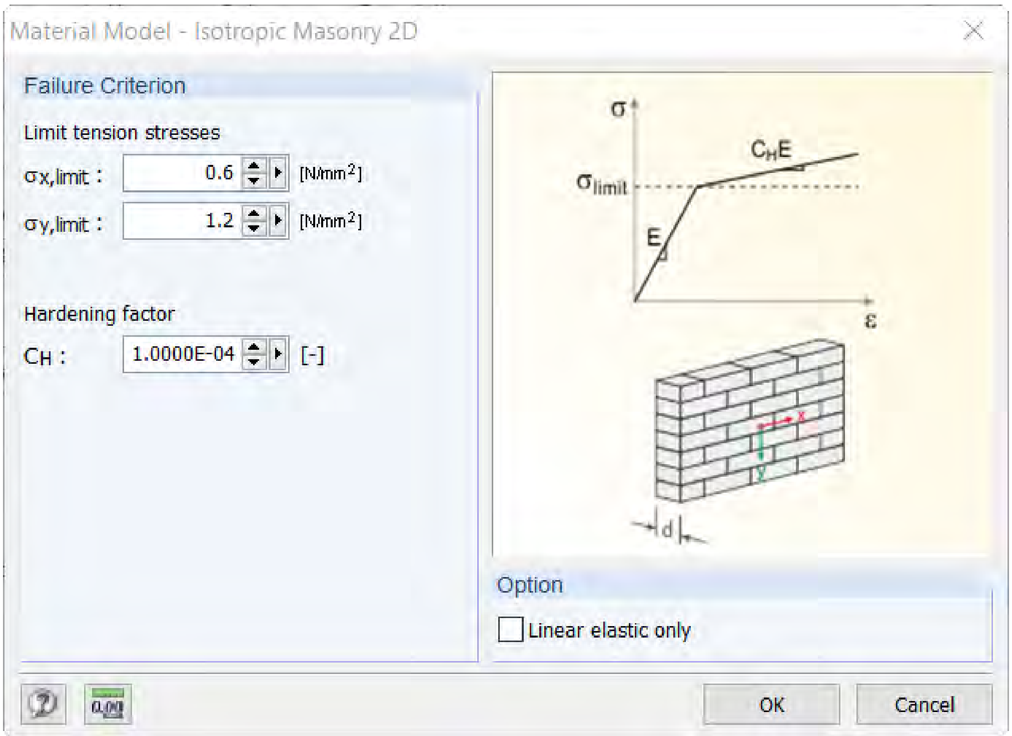
Note: Bar size increment = 6, 8, 10, 12, 16, 18, 20, 22, 25, 28 and 32

5. Calculation method and numerical model

5.1 Model Description

5.1.1 Hypothesis adopted for the elements

- Stine masonry is an orthotropic material, due to absence of testing data and for simplicity the material is modeled as nonlinear isotropic material as shown in the fig. 5.1.



Material Model - Isotropic Masonry 2D

Failure Criterion

Limit tension stresses

σx,limit : 0.6 [N/mm^2]

σy,limit : 1.2 [N/mm^2]

Hardening factor

CH : 1.0000E-04 [-]

Option

☐ Linear elastic only

Figure 5.1 Material model of stone masonry.

- After tensile strength limits, the plastic behavior of masonry is obsesses and fracturing cracks appear.
- The masonry after this limit can resist the load as a cracked section, depending on it’s stability moment of resistance.

6. Actions and design loads

6.1 STRUCTURAL LOADS.

The following loads are considered in the design:

- Structural Dead Loads which include:
    - The own weight of the structural elements, walls and arched slabs.
    - Superimposed dead load from Stone above arched slabs and Finishing.
- Live loads which cover all variable occupants above the coliseum.
- Seismic loads according to ECP.

The basis for the considered design loads are summarized in the followings sections.

D. Dead Loads

Unit weight of Masonry elements 27.0 kN/m³

E. Live Loads

Live loads are considered equal to 500 kg/m^2

F. Wind Loads

The wind pressure shall be calculated in accordance with (ECP 201-2012)

Basic wind speed = 36 m/sec.

Wind pressure (or suction) distribution factor (Ce)

C_e = +0.8 for areas subjected to wind pressure

C_e = -0.5/-0.7 for areas subjected to suction wind

Exposure factor (according to height from ground level) (k = 1)

G. Earthquakes

- Response modification factor (R = 2)
- Importance factor (γ_i = 1.2)
- The design acceleration (a_g = 0.15g)
- Design damping correction factor (η = 1.0)
- Zone 3
- Soil Type (C)
- Earthquake loads shall be comply with the (ECP 201-2010).

- **Earthquakes Input data and results.**

Activities	☒ Modal analysis (eigenvectors) ☒ Mass combinations ☐ Forced vibrations ☒ Response spectra ☐ Accelerograms ☐ Time diagrams ☒ Equivalent static force analysis	
Setting	Gravity acceleration	: 9.806 m/s^2

No.	Mass Case Description	Parameters	
MC1	Self - Weight	Mass Case Type	: Permanent
		Masses	☒ : From force components of Load Case LC5-Self-weight
MC2	Arch Load	Mass Case Type	: Permanent
		Masses	☒ : From force components of Load Case LC12-Arch Load
MC3	Imposed Load	Mass Case Type	: Imposed - category C (independent, p=0.5)
		Masses	☒ : From force components of Load Case LC6-Imposed load

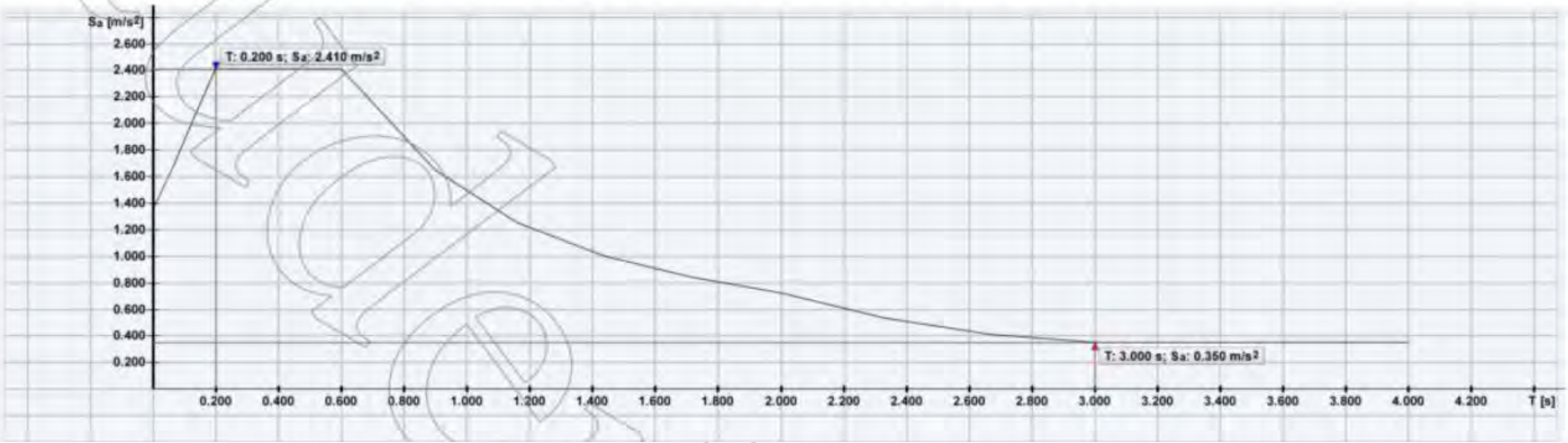
No.	Mass Combination Description	Parameters			
MCO1	DL + 0.5 LL	Mass Cases	:	1.00	MC1 - Self - Weight
				1.00	MC2 - Arch Load
				0.50	MC3 - Imposed Load
		Comment	:		

No.	Response Spectrum Description	No.	Time T[s]	Acceleration a [m/s^2]
RS1		1	0.000	1.350
		2	0.070	1.700
		3	0.133	2.050
		4	0.200	2.410
		5	0.330	2.410
		6	0.470	2.410
		7	0.600	2.410
		8	0.900	1.650
		9	1.160	1.250
		10	1.440	1.000
		11	1.720	0.840
		12	2.000	0.720
		13	2.330	0.530
		14	2.670	0.410
		15	3.000	0.350
		16	3.330	0.350

No.	Response Spectrum Description	No.	Time T[s]	Acceleration a [m/s²]
RS1		17	3.670	0.350
		18	4.000	0.350

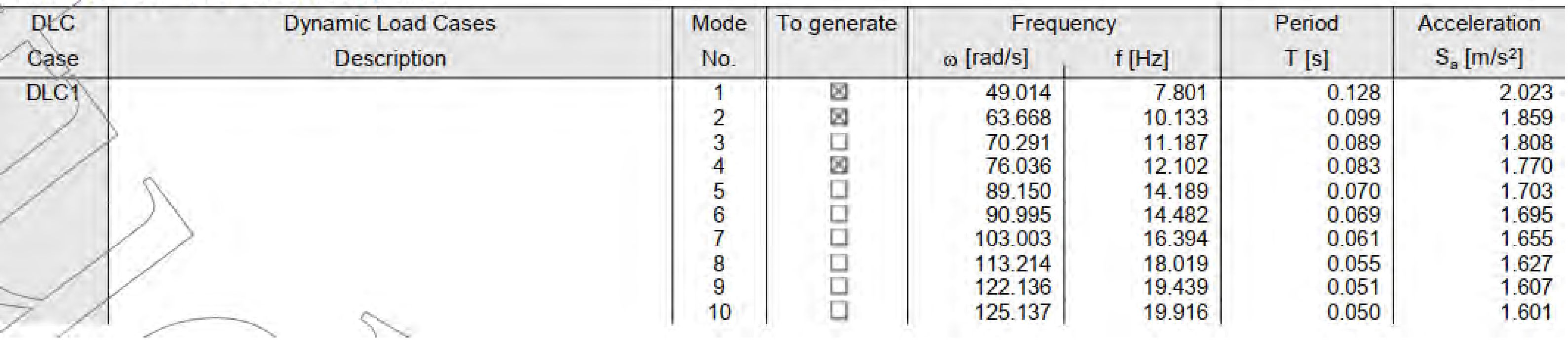
■ 1.5.4.1 RESPONSE SPECTRA - USER-DEFINED - GRAPH

RS1



DLC Case	Dynamic Load Cases Description	Parameters
DLC1		Assign response spectrum: Response Spectrum in Direction — Multiplication factor ☒ x: RS1 - — 1.000 ☒ y: RS1 - — 1.000 Rotate a_X a_Y about Z: — α = 0.00 [°] Settings: ☐ Consider accidental torsional actions: To generate: ☒ Load cases with $E_{X,i}$ / $E_{Z,i}$ from all modal shapes Number of first generated load case: 1 ☒ Result Combinations (modal combination) Number of first generated result combination: 1 ☒ Combination of directional components with ☐ SRSS ☒ 100 / 30 % ☐ 100 / 40 % Combination Rules: Modal response combination rule: ☐ SRSS ☒ CQC Options ☒ Use equivalent linear combination ☒ Signed results using dominant mode shape Y: Automatic Z: Automatic X: Automatic

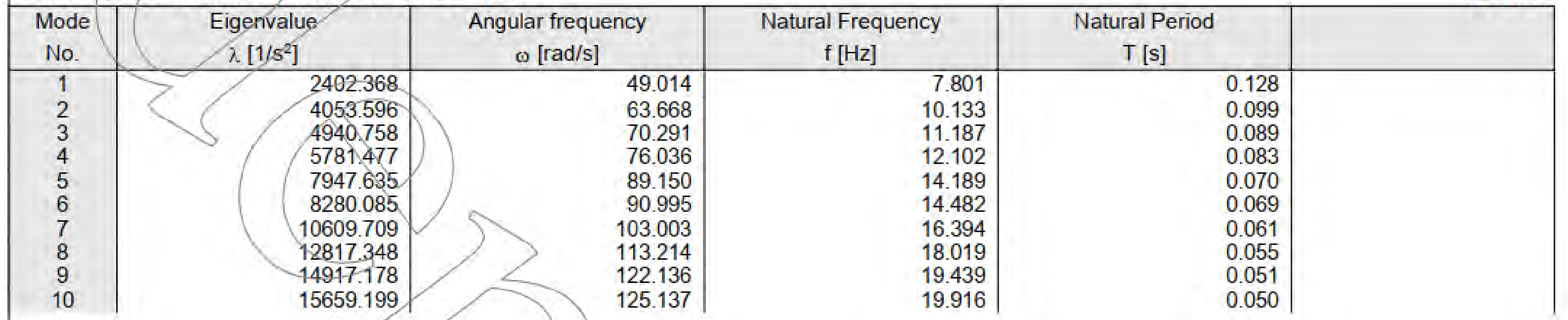
SHAPES TO GENERATE



DLC Case	Dynamic Load Cases Description	Mode No.	To generate	Frequency ω [rad/s]	Frequency f [Hz]	Period T [s]	Acceleration S_a [m/s²]
DLC1		1	☒	49.014	7.801	0.128	2.023
		2	☒	63.668	10.133	0.099	1.859
		3	☐	70.291	11.187	0.089	1.808
		4	☒	76.036	12.102	0.083	1.770
		5	☐	89.150	14.189	0.070	1.703
		6	☐	90.995	14.482	0.069	1.695
		7	☐	103.003	16.394	0.061	1.655
		8	☐	113.214	18.019	0.055	1.627
		9	☐	122.136	19.439	0.051	1.607
		10	☐	125.137	19.916	0.050	1.601

5.1 NATURAL FREQUENCIES

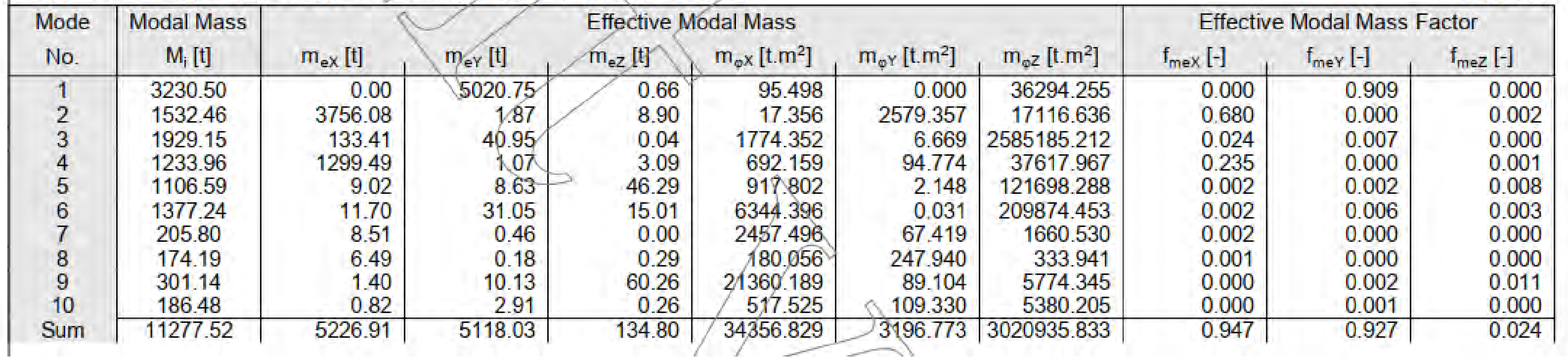
NVC1



Mode No.	Eigenvalue λ [1/s²]	Angular frequency ω [rad/s]	Natural Frequency f [Hz]	Natural Period T [s]
1	2402.368	49.014	7.801	0.128
2	4053.596	63.668	10.133	0.099
3	4940.758	70.291	11.187	0.089
4	5781.477	76.036	12.102	0.083
5	7947.635	89.150	14.189	0.070
6	8280.085	90.995	14.482	0.069
7	10609.709	103.003	16.394	0.061
8	12817.348	113.214	18.019	0.055
9	14917.178	122.136	19.439	0.051
10	15659.199	125.137	19.916	0.050

5.7 EFFECTIVE MODAL MASS FACTORS

NVC1

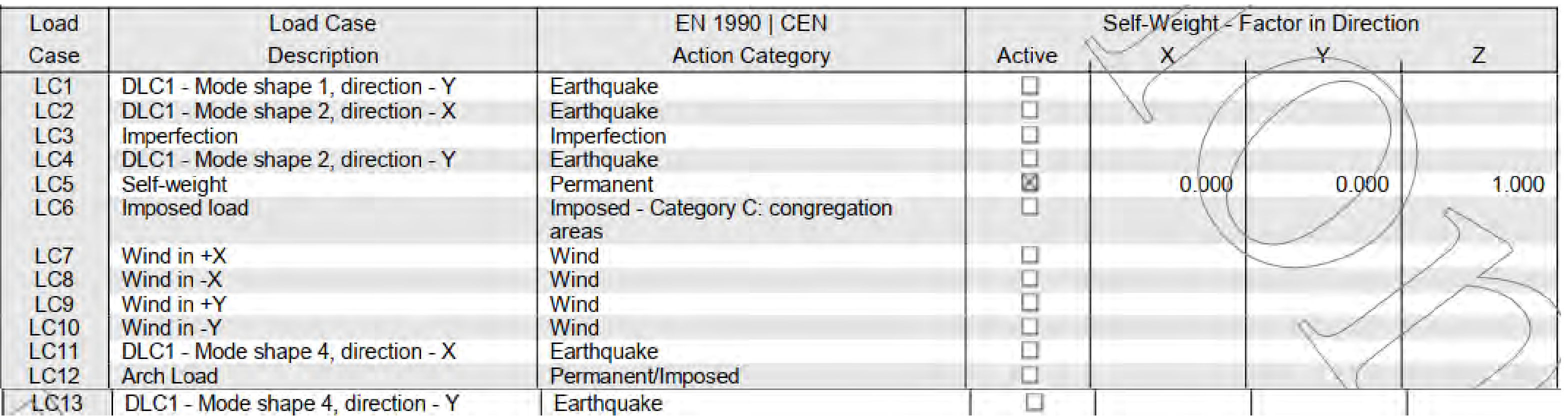


Mode No.	Modal Mass M_i [t]	Effective Modal Mass m_{eX} [t]	m_{eY} [t]	m_{eZ} [t]	$m_{\varphi X}$ [t.m²]	$m_{\varphi Y}$ [t.m²]	$m_{\varphi Z}$ [t.m²]	Effective Modal Mass Factor f_{meX} [-]	f_{meY} [-]	f_{meZ} [-]
1	3230.50	0.00	5020.75	0.66	95.498	0.000	36294.255	0.000	0.909	0.000
2	1532.46	3756.08	1.87	8.90	17.356	2579.357	17116.636	0.680	0.000	0.002
3	1929.15	133.41	40.95	0.04	1774.352	6.669	2585185.212	0.024	0.007	0.000
4	1233.96	1299.49	1.07	3.09	692.159	94.774	37617.967	0.235	0.000	0.001
5	1106.59	9.02	8.63	46.29	917.802	2.148	121698.288	0.002	0.002	0.008
6	1377.24	11.70	31.05	15.01	6344.396	0.031	209874.453	0.002	0.006	0.003
7	205.80	8.51	0.46	0.00	2457.496	67.419	1660.530	0.002	0.000	0.000
8	174.19	6.49	0.18	0.29	180.056	247.940	333.941	0.001	0.000	0.000
9	301.14	1.40	10.13	60.26	21360.189	89.104	5774.345	0.000	0.002	0.011
10	186.48	0.82	2.91	0.26	517.525	109.330	5380.205	0.000	0.001	0.000
Sum	11277.52	5226.91	5118.03	134.80	34356.829	3196.773	3020935.833	0.947	0.927	0.024

6.2 Load Cases and Load combinations.

The following tables describe the load cases and load combinations on the Coliseum:

Table 1: Load cases



Load	Load Case	EN 1990 | CEN		Self-Weight - Factor in Direction		
Case	Description	Action Category	Active	X	Y	Z
LC1	DLC1 - Mode shape 1, direction - Y	Earthquake	☐			
LC2	DLC1 - Mode shape 2, direction - X	Earthquake	☐			
LC3	Imperfection	Imperfection	☐			
LC4	DLC1 - Mode shape 2, direction - Y	Earthquake	☐			
LC5	Self-weight	Permanent	☒	0.000	0.000	1.000
LC6	Imposed load	Imposed - Category C: congregation areas	☐			
LC7	Wind in +X	Wind	☐			
LC8	Wind in -X	Wind	☐			
LC9	Wind in +Y	Wind	☐			
LC10	Wind in -Y	Wind	☐			
LC11	DLC1 - Mode shape 4, direction - X	Earthquake	☐			
LC12	Arch Load	Permanent/Imposed	☐			
LC13	DLC1 - Mode shape 4, direction - Y	Earthquake	☐			

Load	Load Case		
Case	Description	Angular Frequency [rad/s]	Lehr's damping [-]
LC1	DLC1 - Mode shape 1, direction - Y	49.01	0.070
LC2	DLC1 - Mode shape 2, direction - X	63.67	0.070
LC4	DLC1 - Mode shape 2, direction - Y	63.67	0.070
LC11	DLC1 - Mode shape 4, direction - X	76.04	0.070
LC13	DLC1 - Mode shape 4, direction - Y	76.04	0.070

Table 2: Load combinations

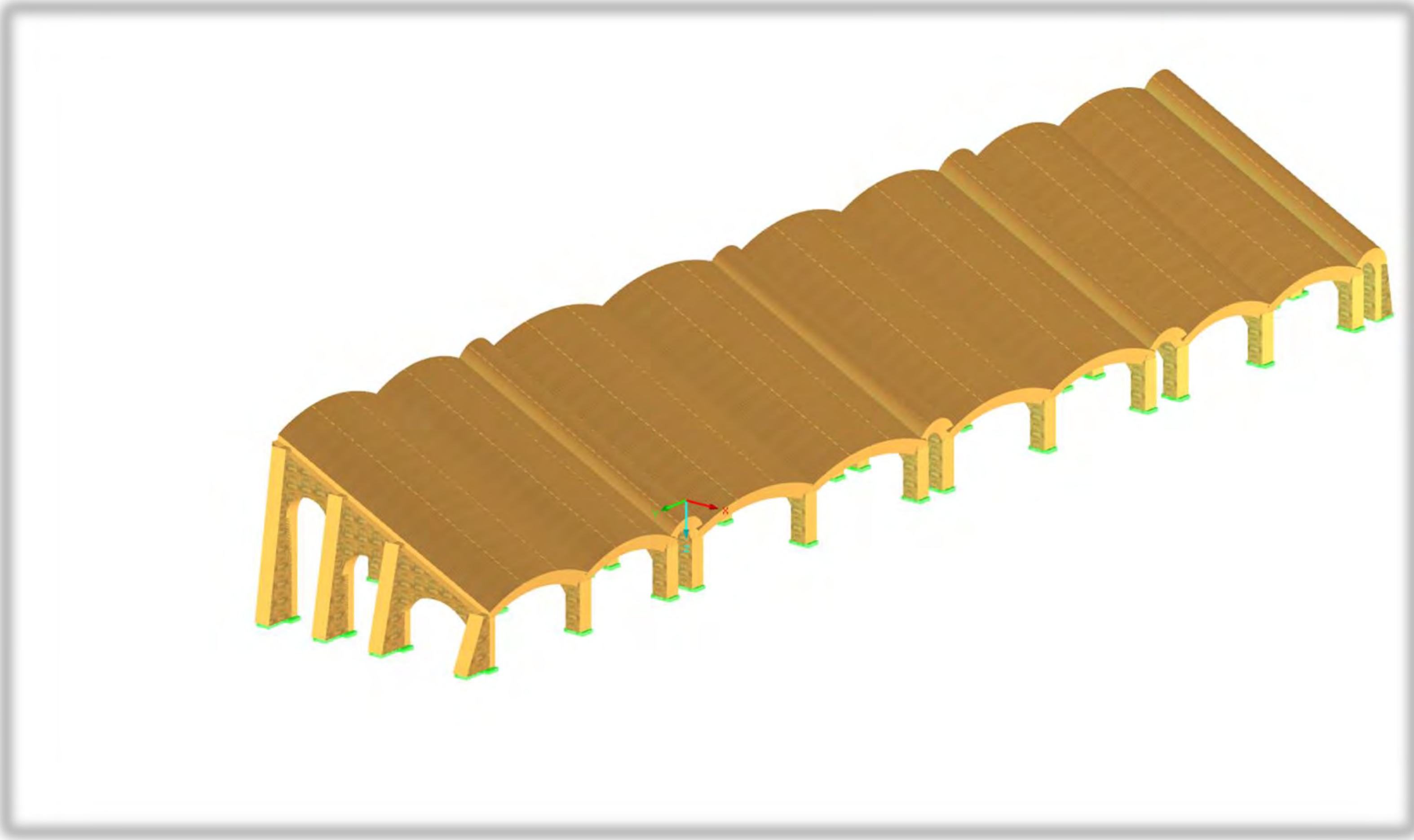
Load Combin	DS	Load Combination Description	No.	Factor	Load Case	
CO1	STR	1.35G + Imp	1	1.35	LC5	Self-weight
			2	1.00	LC3	Imperfection
CO2	STR	1.35G + 1.35Gq + Imp	1	1.35	LC5	Self-weight
			2	1.35	LC12	Arch Load
			3	1.00	LC3	Imperfection
CO3	STR	1.35G + 1.5QiC + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	1.00	LC3	Imperfection
CO4	STR	1.35G + 1.5QiC + 1.35Gq + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	1.35	LC12	Arch Load
			4	1.00	LC3	Imperfection
CO5	STR	G + Imp	1	1.00	LC5	Self-weight
			2	1.00	LC3	Imperfection
CO6	STR	G + 1.35Gq + Imp	1	1.00	LC5	Self-weight
			2	1.35	LC12	Arch Load
			3	1.00	LC3	Imperfection
CO7	STR	G + 1.5QiC + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	1.00	LC3	Imperfection
CO8	STR	G + 1.5QiC + 1.35Gq + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	1.35	LC12	Arch Load
			4	1.00	LC3	Imperfection
CO9	STR	1.35G + Imp	1	1.35	LC5	Self-weight
			2	1.00	LC3	Imperfection
CO10	STR	1.35G + 1.35Gq + Imp	1	1.35	LC5	Self-weight
			2	1.35	LC12	Arch Load
			3	1.00	LC3	Imperfection
CO11	STR	1.35G + 1.5QiC + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	1.00	LC3	Imperfection
CO12	STR	1.35G + 1.5QiC + 0.9Qw1 + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	0.90	LC7	Wind in +X
			4	1.00	LC3	Imperfection
CO13	STR	1.35G + 1.5QiC + 0.9Qw2 + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	0.90	LC8	Wind in -X
			4	1.00	LC3	Imperfection
CO14	STR	1.35G + 1.5QiC + 0.9Qw3 + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	0.90	LC9	Wind in +Y
			4	1.00	LC3	Imperfection
CO15	STR	1.35G + 1.5QiC + 0.9Qw4 + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	0.90	LC10	Wind in -Y
			4	1.00	LC3	Imperfection
CO16	STR	1.35G + 1.5QiC + 0.9Qw1 + 1.35Gq + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	0.90	LC7	Wind in +X
			4	1.35	LC12	Arch Load
			5	1.00	LC3	Imperfection
CO17	STR	1.35G + 1.5QiC + 0.9Qw2 + 1.35Gq + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	0.90	LC8	Wind in -X
			4	1.35	LC12	Arch Load
			5	1.00	LC3	Imperfection
CO18	STR	1.35G + 1.5QiC + 0.9Qw3 + 1.35Gq + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	0.90	LC9	Wind in +Y
			4	1.35	LC12	Arch Load
			5	1.00	LC3	Imperfection
CO19	STR	1.35G + 1.5QiC + 0.9Qw4 + 1.35Gq + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	0.90	LC10	Wind in -Y
			4	1.35	LC12	Arch Load
			5	1.00	LC3	Imperfection
CO20	STR	1.35G + 1.5QiC + 1.35Gq + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	1.35	LC12	Arch Load
			4	1.00	LC3	Imperfection
CO21	STR	1.35G + 1.5Qw1 + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC7	Wind in +X
			3	1.00	LC3	Imperfection

Load Combin.	DS	Load Combination Description	No.	Factor	Load Case	
CO22	STR	1.35G + 1.5Qw2 + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC8	Wind in -X
			3	1.00	LC3	Imperfection
CO23	STR	1.35G + 1.5Qw3 + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC9	Wind in +Y
			3	1.00	LC3	Imperfection
CO24	STR	1.35G + 1.5Qw4 + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC10	Wind in -Y
			3	1.00	LC3	Imperfection
CO25	STR	1.35G + 1.05QIC + 1.5Qw1 + Imp	1	1.35	LC5	Self-weight
			2	1.05	LC6	Imposed load
			3	1.50	LC7	Wind in +X
			4	1.00	LC3	Imperfection
CO26	STR	1.35G + 1.05QIC + 1.5Qw2 + Imp	1	1.35	LC5	Self-weight
			2	1.05	LC6	Imposed load
			3	1.50	LC8	Wind in -X
			4	1.00	LC3	Imperfection
CO27	STR	1.35G + 1.05QIC + 1.5Qw3 + Imp	1	1.35	LC5	Self-weight
			2	1.05	LC6	Imposed load
			3	1.50	LC9	Wind in +Y
			4	1.00	LC3	Imperfection
CO28	STR	1.35G + 1.05QIC + 1.5Qw4 + Imp	1	1.35	LC5	Self-weight
			2	1.05	LC6	Imposed load
			3	1.50	LC10	Wind in -Y
			4	1.00	LC3	Imperfection
CO29	STR	1.35G + 1.05QIC + 1.5Qw1 + 1.35Gq + Imp	1	1.35	LC5	Self-weight
			2	1.05	LC6	Imposed load
			3	1.50	LC7	Wind in +X
			4	1.35	LC12	Arch Load
			5	1.00	LC3	Imperfection
CO30	STR	1.35G + 1.05QIC + 1.5Qw2 + 1.35Gq + Imp	1	1.35	LC5	Self-weight
			2	1.05	LC6	Imposed load
			3	1.50	LC8	Wind in -X
			4	1.35	LC12	Arch Load
			5	1.00	LC3	Imperfection
CO31	STR	1.35G + 1.05QIC + 1.5Qw3 + 1.35Gq + Imp	1	1.35	LC5	Self-weight
			2	1.05	LC6	Imposed load
			3	1.50	LC9	Wind in +Y
			4	1.35	LC12	Arch Load
			5	1.00	LC3	Imperfection
CO32	STR	1.35G + 1.05QIC + 1.5Qw4 + 1.35Gq + Imp	1	1.35	LC5	Self-weight
			2	1.05	LC6	Imposed load
			3	1.50	LC10	Wind in -Y
			4	1.35	LC12	Arch Load
			5	1.00	LC3	Imperfection
CO33	STR	1.35G + 1.5Qw1 + 1.35Gq + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC7	Wind in +X
			3	1.35	LC12	Arch Load
			4	1.00	LC3	Imperfection
CO34	STR	1.35G + 1.5Qw2 + 1.35Gq + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC8	Wind in -X
			3	1.35	LC12	Arch Load
			4	1.00	LC3	Imperfection
CO35	STR	1.35G + 1.5Qw3 + 1.35Gq + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC9	Wind in +Y
			3	1.35	LC12	Arch Load
			4	1.00	LC3	Imperfection
CO36	STR	1.35G + 1.5Qw4 + 1.35Gq + Imp	1	1.35	LC5	Self-weight
			2	1.50	LC10	Wind in -Y
			3	1.35	LC12	Arch Load
			4	1.00	LC3	Imperfection
CO37	STR	G + Imp	1	1.00	LC5	Self-weight
			2	1.00	LC3	Imperfection
CO38	STR	G + 1.35Gq + Imp	1	1.00	LC5	Self-weight
			2	1.35	LC12	Arch Load
			3	1.00	LC3	Imperfection
CO39	STR	G + 1.5QIC + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	1.00	LC3	Imperfection
CO40	STR	G + 1.5QIC + 0.9Qw1 + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	0.90	LC7	Wind in +X
			4	1.00	LC3	Imperfection
CO41	STR	G + 1.5QIC + 0.9Qw2 + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	0.90	LC8	Wind in -X
			4	1.00	LC3	Imperfection
CO42	STR	G + 1.5QIC + 0.9Qw3 + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	0.90	LC9	Wind in +Y
			4	1.00	LC3	Imperfection
CO43	STR	G + 1.5QIC + 0.9Qw4 + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	0.90	LC10	Wind in -Y
			4	1.00	LC3	Imperfection
CO44	STR	G + 1.5QIC + 0.9Qw1 + 1.35Gq + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC6	Imposed load

Load Combin.	DS	Load Combination Description	No.	Factor	Load Case	
			3	0.90	LC7	Wind in +X
			4	1.35	LC12	Arch Load
			5	1.00	LC3	Imperfection
CO45	STR	G + 1.5QIC + 0.9Qw2 + 1.35Gq + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	0.90	LC8	Wind in -X
			4	1.35	LC12	Arch Load
			5	1.00	LC3	Imperfection
CO46	STR	G + 1.5QIC + 0.9Qw3 + 1.35Gq + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	0.90	LC9	Wind in +Y
			4	1.35	LC12	Arch Load
			5	1.00	LC3	Imperfection
CO47	STR	G + 1.5QIC + 0.9Qw4 + 1.35Gq + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	0.90	LC10	Wind in -Y
			4	1.35	LC12	Arch Load
			5	1.00	LC3	Imperfection
CO48	STR	G + 1.5QIC + 1.35Gq + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC6	Imposed load
			3	1.35	LC12	Arch Load
			4	1.00	LC3	Imperfection
CO49	STR	G + 1.5Qw1 + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC7	Wind in +X
			3	1.00	LC3	Imperfection
CO50	STR	G + 1.5Qw2 + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC8	Wind in -X
			3	1.00	LC3	Imperfection
CO51	STR	G + 1.5Qw3 + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC9	Wind in +Y
			3	1.00	LC3	Imperfection
CO52	STR	G + 1.5Qw4 + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC10	Wind in -Y
			3	1.00	LC3	Imperfection
CO53	STR	G + 1.05QIC + 1.5Qw1 + Imp	1	1.00	LC5	Self-weight
			2	1.05	LC6	Imposed load
			3	1.50	LC7	Wind in +X
			4	1.00	LC3	Imperfection
CO54	STR	G + 1.05QIC + 1.5Qw2 + Imp	1	1.00	LC5	Self-weight
			2	1.05	LC6	Imposed load
			3	1.50	LC8	Wind in -X
			4	1.00	LC3	Imperfection
CO55	STR	G + 1.05QIC + 1.5Qw3 + Imp	1	1.00	LC5	Self-weight
			2	1.05	LC6	Imposed load
			3	1.50	LC9	Wind in +Y
			4	1.00	LC3	Imperfection
CO56	STR	G + 1.05QIC + 1.5Qw4 + Imp	1	1.00	LC5	Self-weight
			2	1.05	LC6	Imposed load
			3	1.50	LC10	Wind in -Y
			4	1.00	LC3	Imperfection
CO57	STR	G + 1.05QIC + 1.5Qw1 + 1.35Gq + Imp	1	1.00	LC5	Self-weight
			2	1.05	LC6	Imposed load
			3	1.50	LC7	Wind in +X
			4	1.35	LC12	Arch Load
			5	1.00	LC3	Imperfection
CO58	STR	G + 1.05QIC + 1.5Qw2 + 1.35Gq + Imp	1	1.00	LC5	Self-weight
			2	1.05	LC6	Imposed load
			3	1.50	LC8	Wind in -X
			4	1.35	LC12	Arch Load
			5	1.00	LC3	Imperfection
CO59	STR	G + 1.05QIC + 1.5Qw3 + 1.35Gq + Imp	1	1.00	LC5	Self-weight
			2	1.05	LC6	Imposed load
			3	1.50	LC9	Wind in +Y
			4	1.35	LC12	Arch Load
			5	1.00	LC3	Imperfection
CO60	STR	G + 1.05QIC + 1.5Qw4 + 1.35Gq + Imp	1	1.00	LC5	Self-weight
			2	1.05	LC6	Imposed load
			3	1.50	LC10	Wind in -Y
			4	1.35	LC12	Arch Load
			5	1.00	LC3	Imperfection
CO61	STR	G + 1.5Qw1 + 1.35Gq + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC7	Wind in +X
			3	1.35	LC12	Arch Load
			4	1.00	LC3	Imperfection
CO62	STR	G + 1.5Qw2 + 1.35Gq + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC8	Wind in -X
			3	1.35	LC12	Arch Load
			4	1.00	LC3	Imperfection
CO63	STR	G + 1.5Qw3 + 1.35Gq + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC9	Wind in +Y
			3	1.35	LC12	Arch Load
			4	1.00	LC3	Imperfection
CO64	STR	G + 1.5Qw4 + 1.35Gq + Imp	1	1.00	LC5	Self-weight
			2	1.50	LC10	Wind in -Y
			3	1.35	LC12	Arch Load
			4	1.00	LC3	Imperfection

7. STRUCTURAL ANALYSIS

7.1 3d-model



7.2 ASSIGN OF LOADS

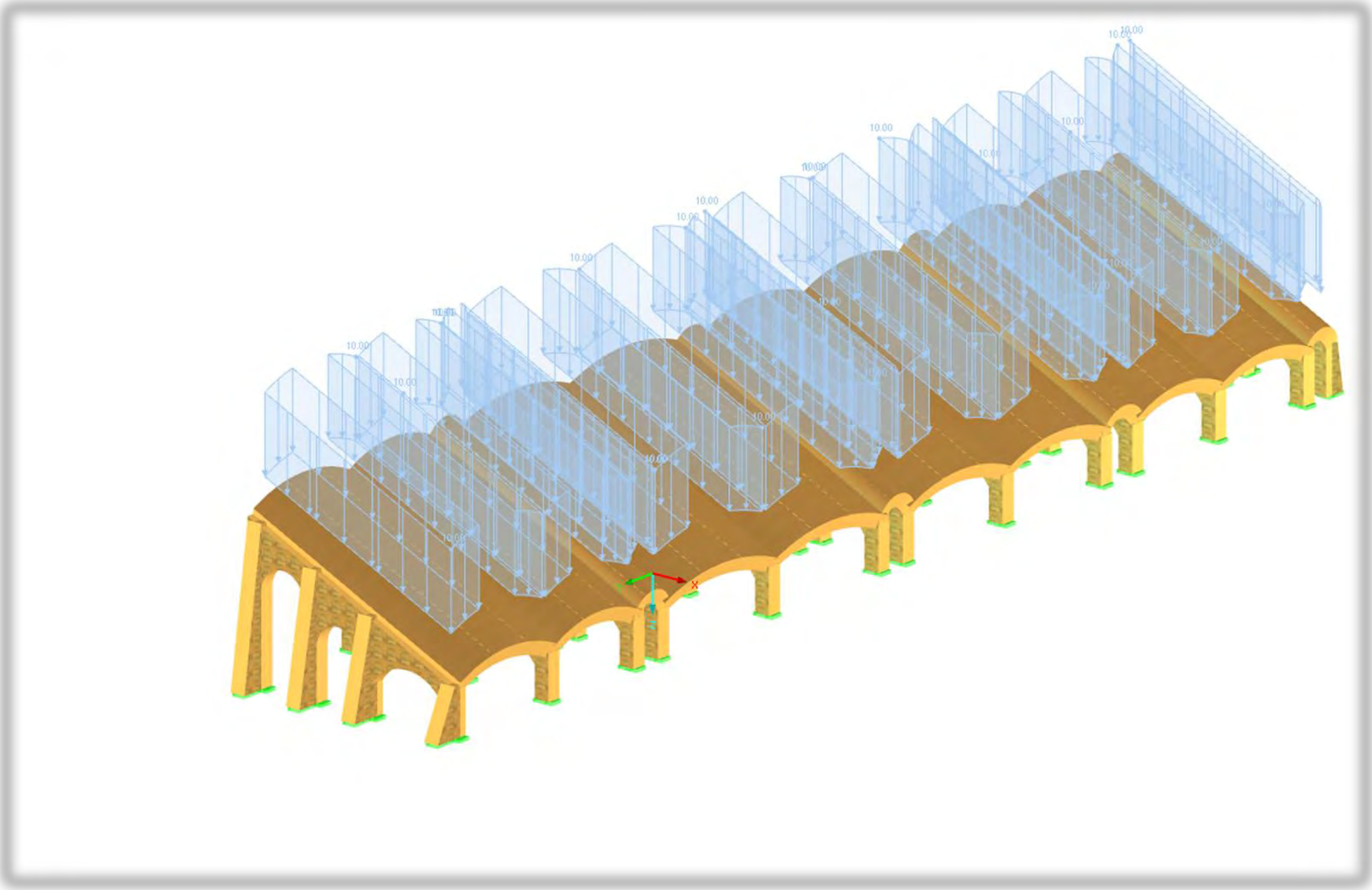


Figure 7.1.: Assign of stone masonry fill loads on the arched slab (KN-M^2 Units)

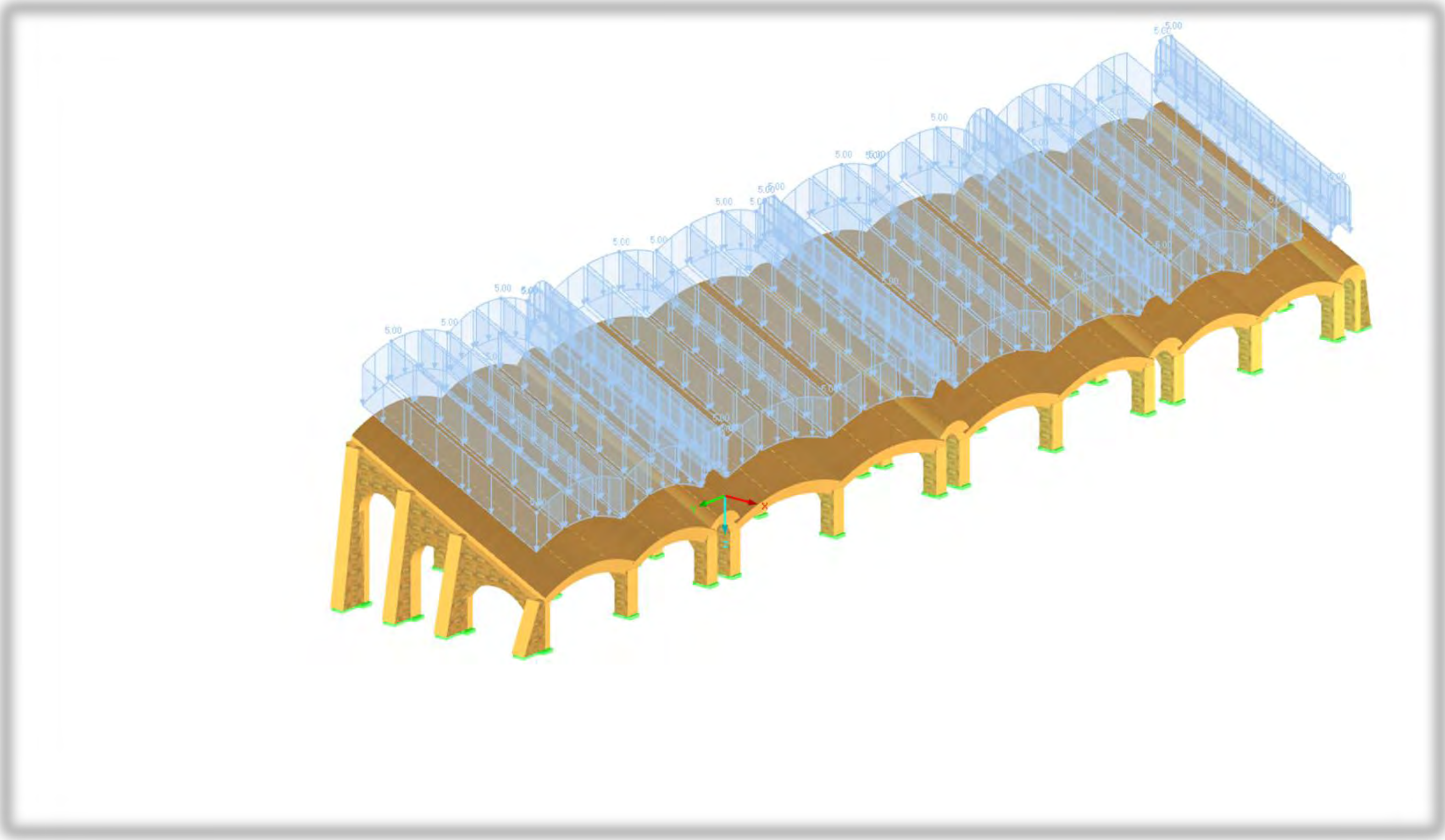


Figure 7.2.: Assign of Live loads on the arched slab (KN-M^2 Units)

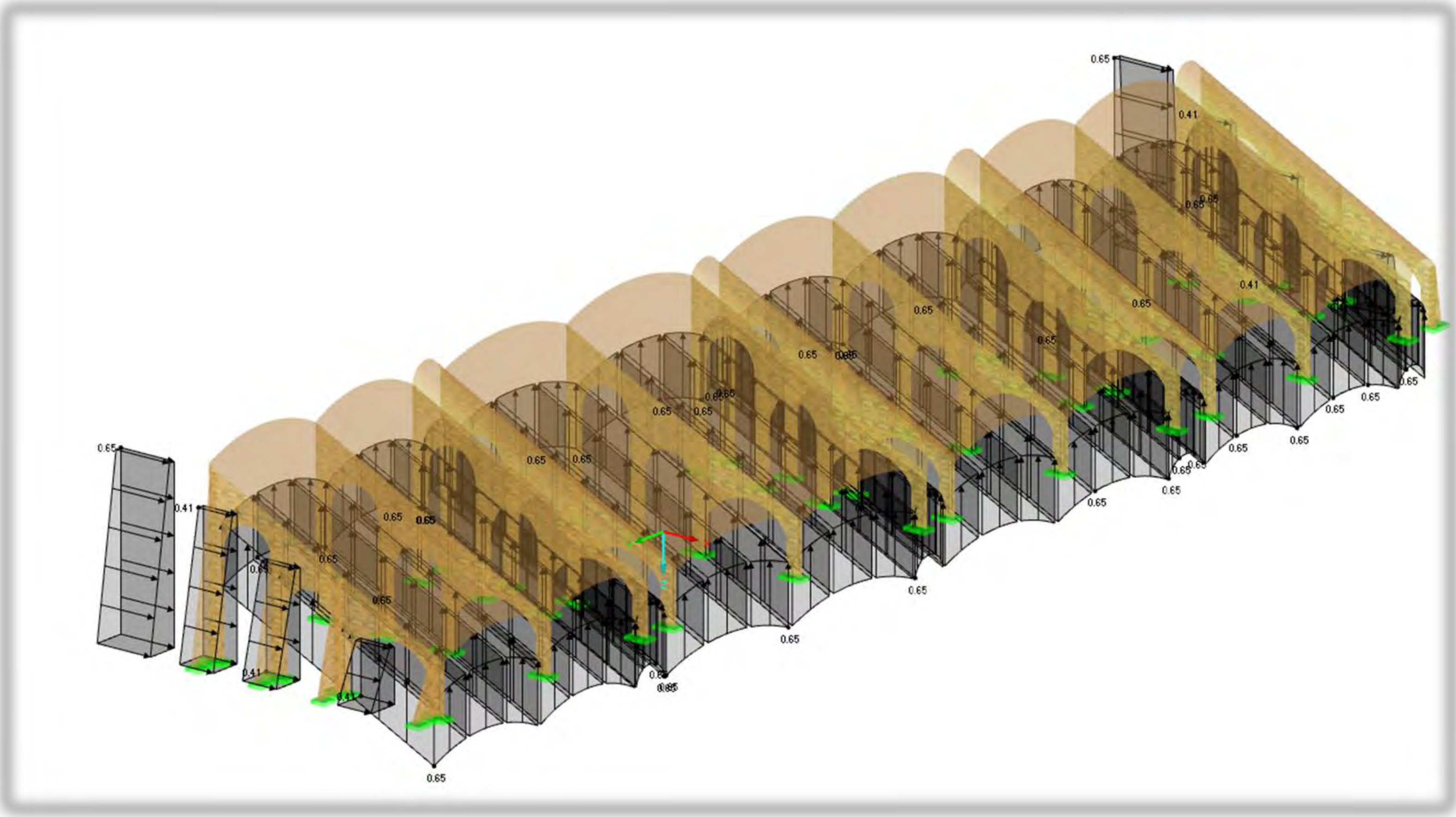


Figure 7.3.: Assign of Wind loads in positive x-direction (KN-M^2 Units)

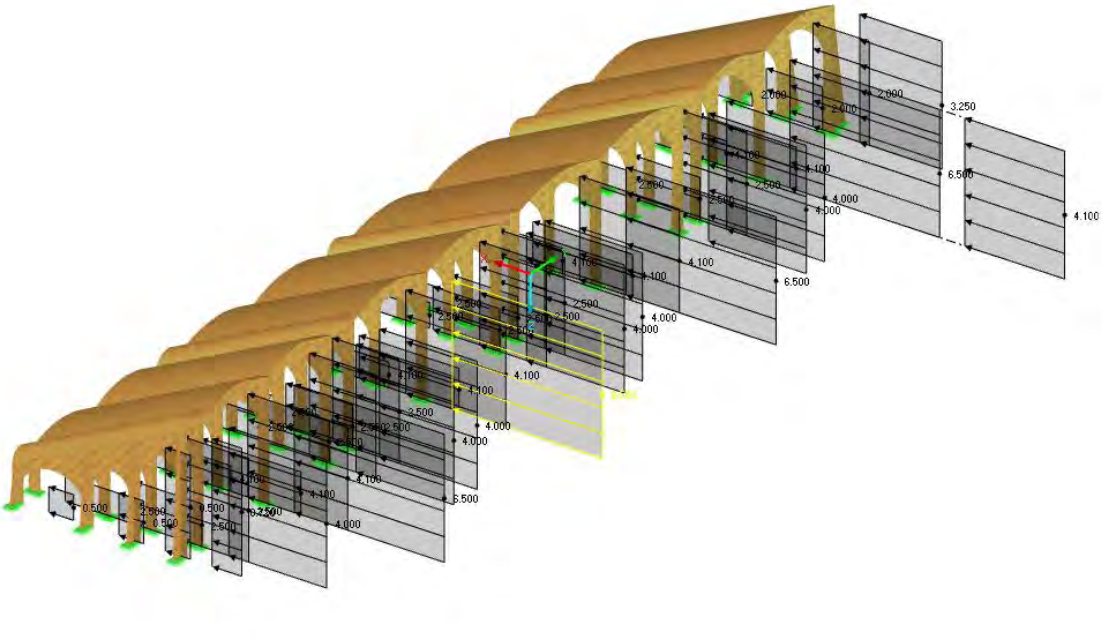


Figure 7.4.: Assign of Wind loads in positive x-direction (KN-M Units)

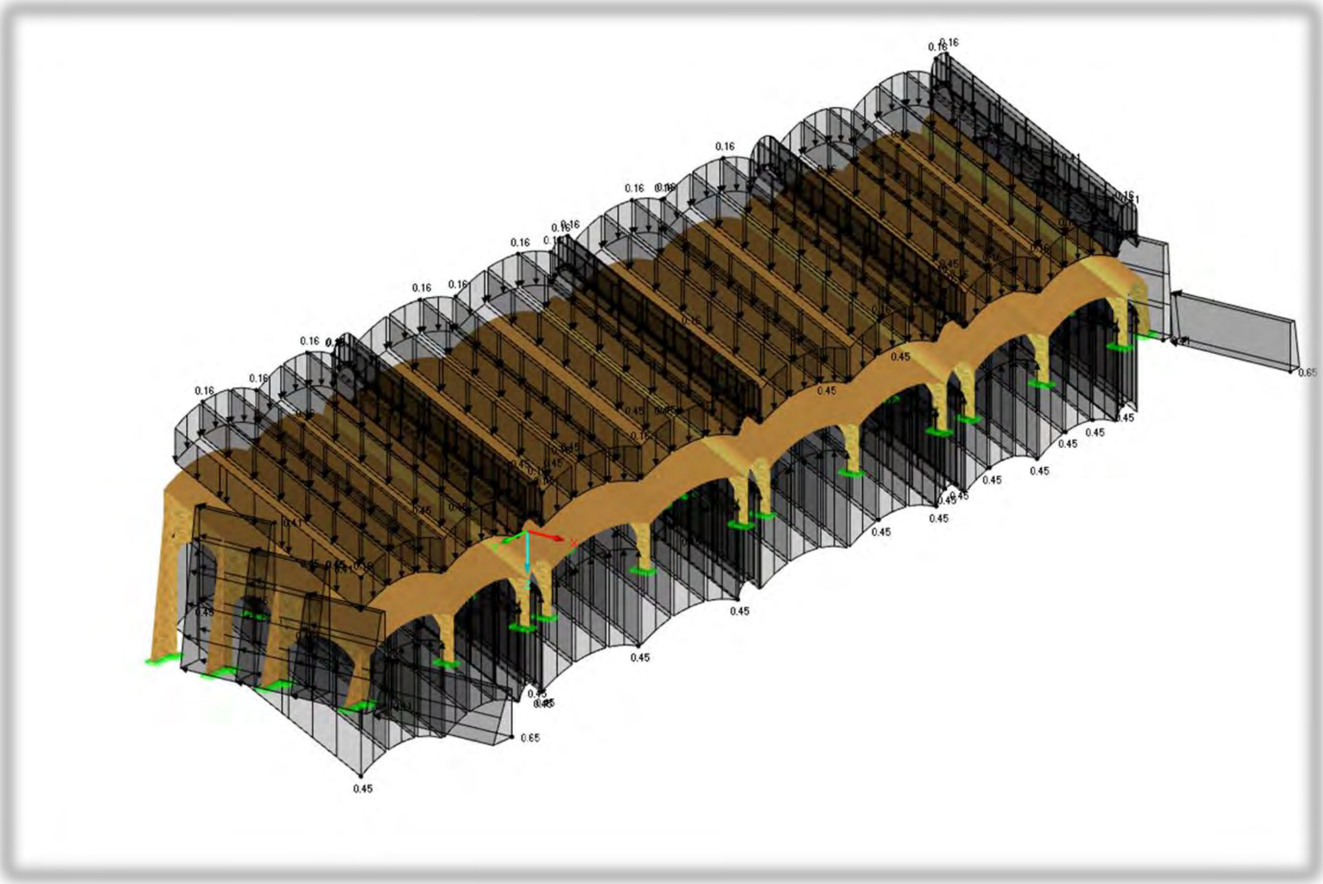


Figure 7.5.: Assign of Wind loads in negative x-direction (KN-M^2 Units)

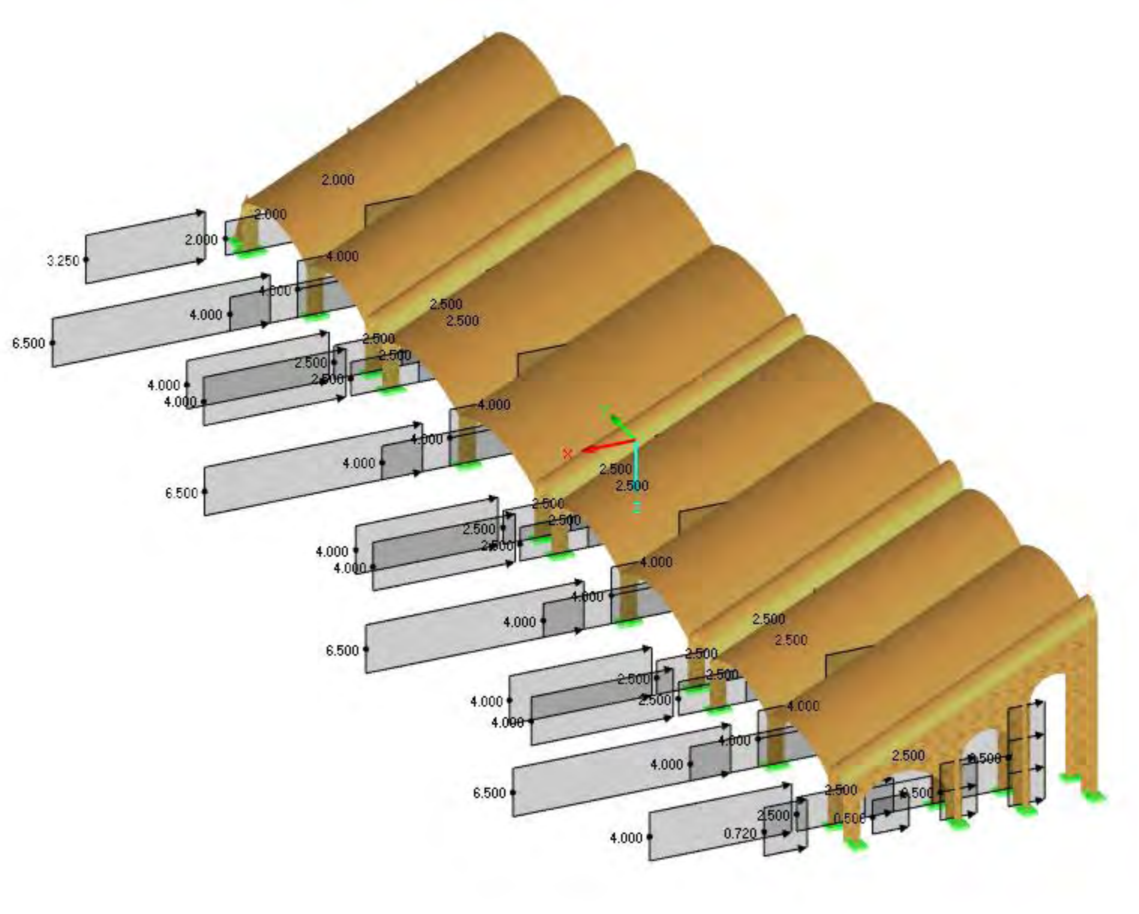


Figure 7.6.: Assign of Wind loads in negative x-direction (KN-M Units)

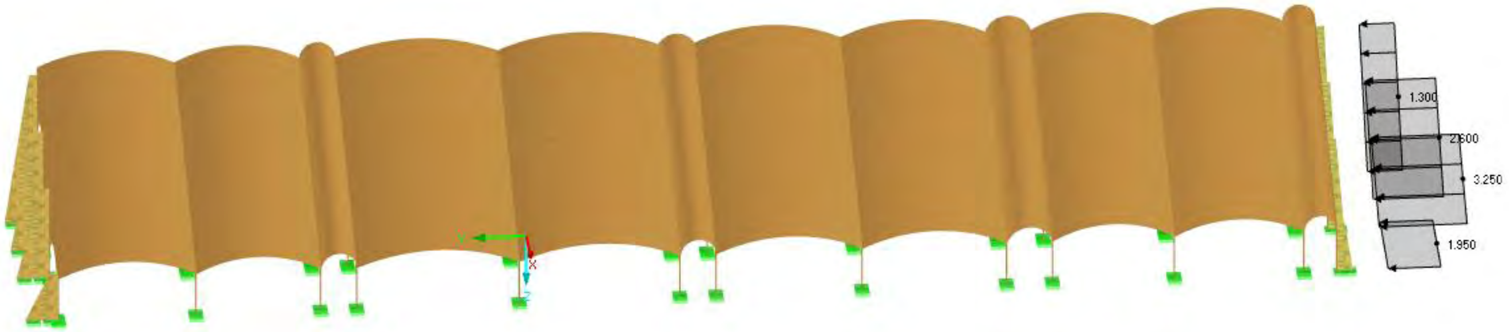


Figure 7.7.: Assign of Wind loads in positive y-direction (KN-M Units)

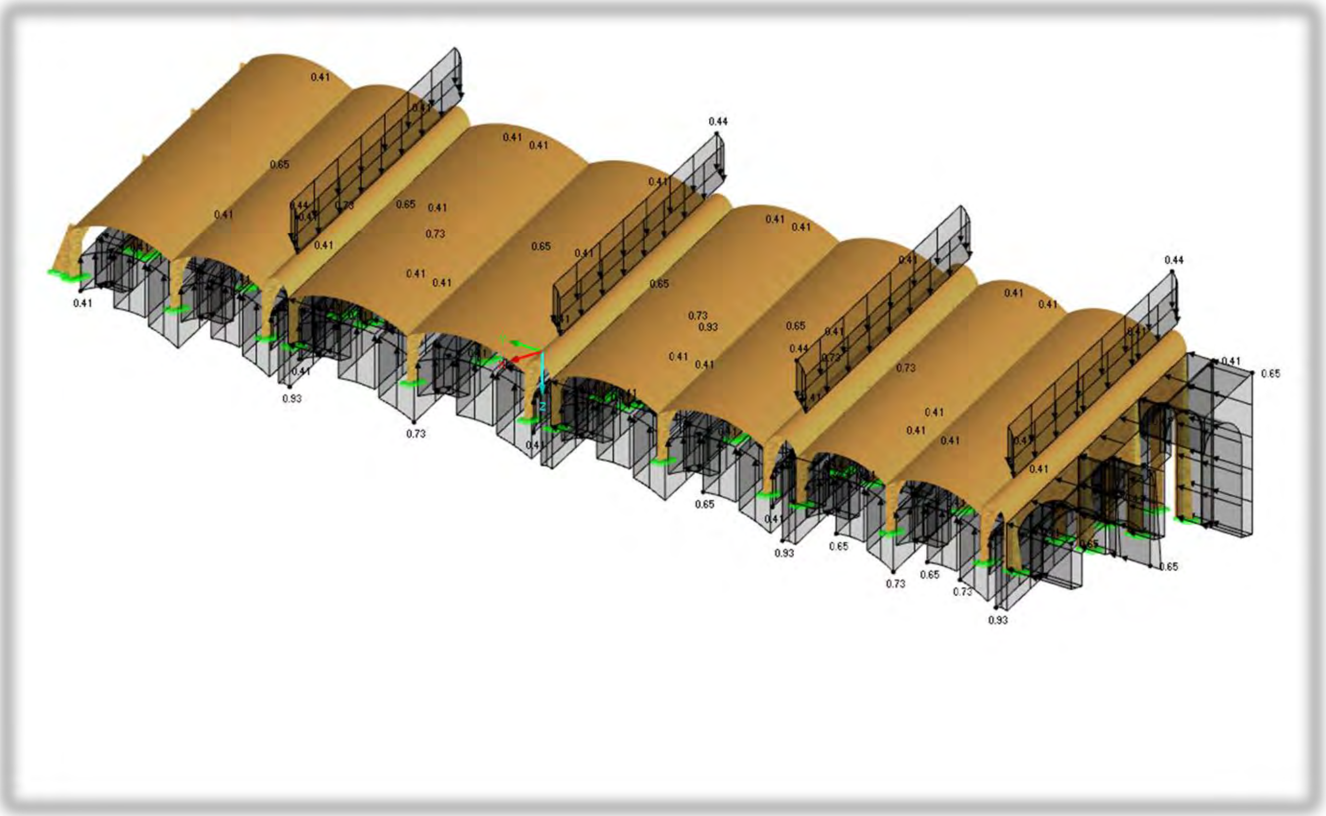


Figure 7.5.: Assign of Wind loads in positive y-direction (KN-M^2 Units)

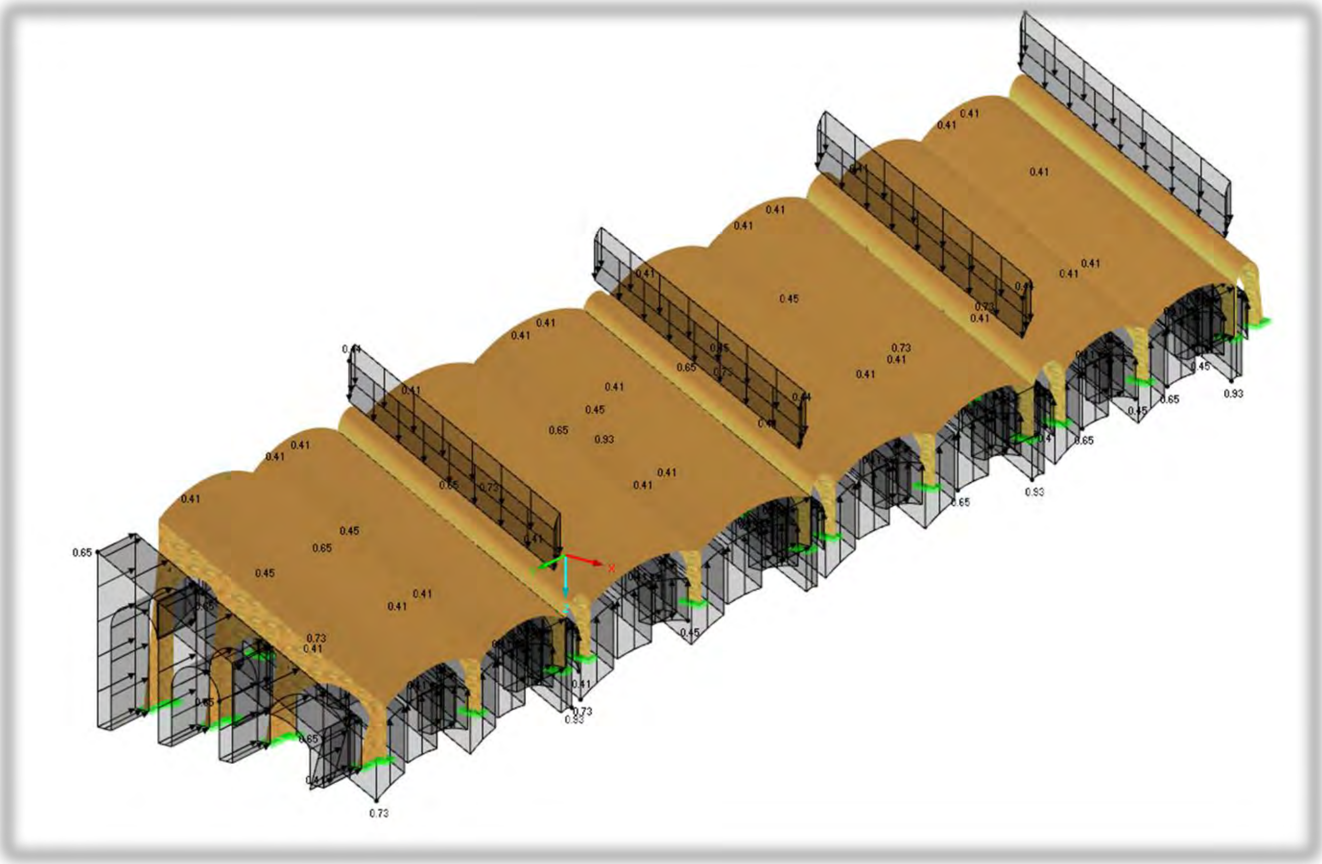


Figure 7.6.: Assign of Wind loads in negative y-direction (KN-M^2 Units)

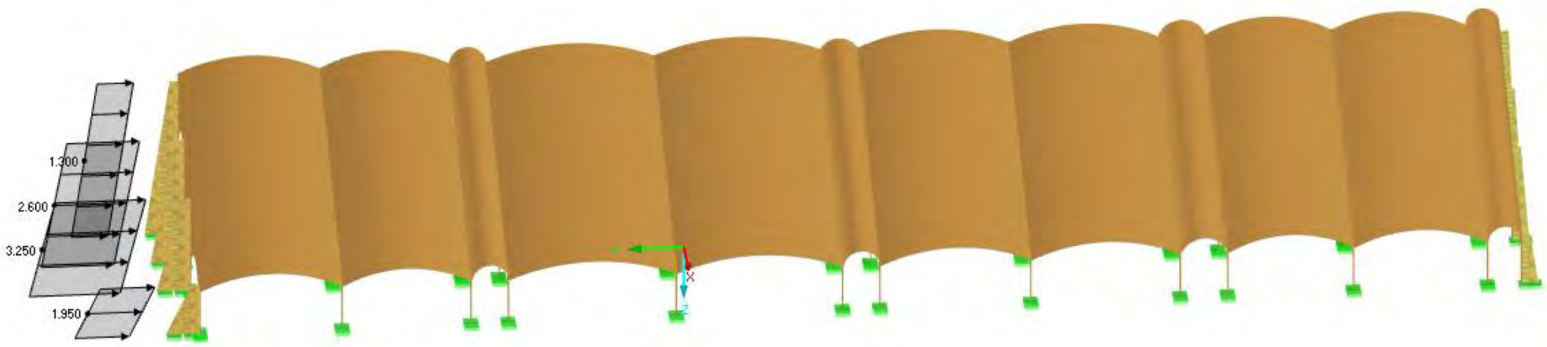
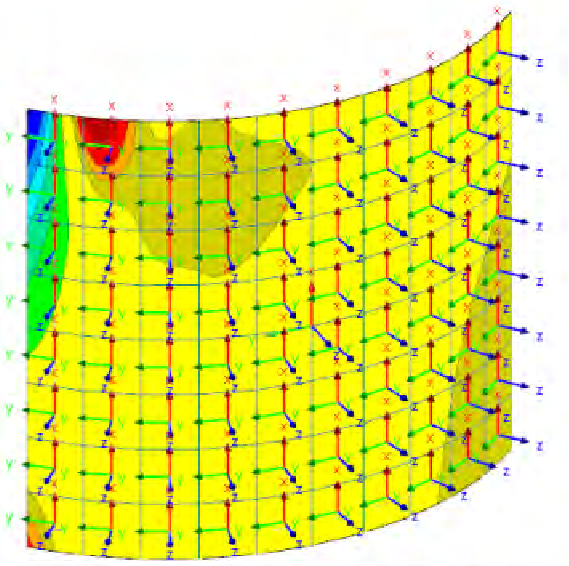


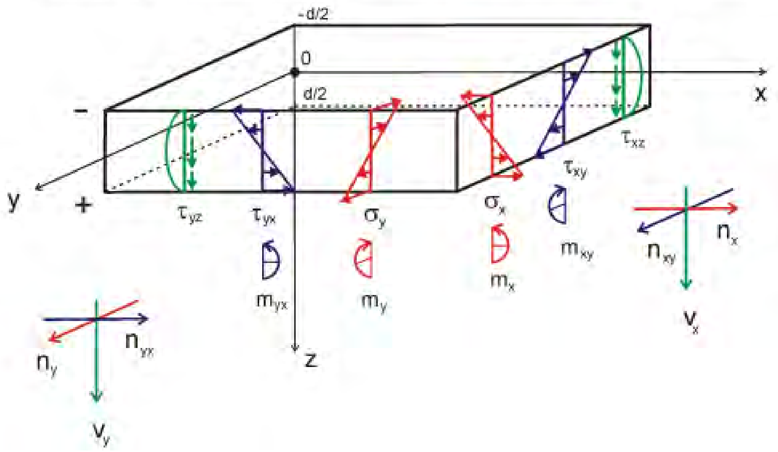
Figure 7.7: Assign of Wind loads in negative y-direction (KN-M Units)

8. STRUCTURAL DESIGN.

8.1 Local Axis Definitions.



The following figure illustrates the definition of basic internal forces in surfaces:



8.2 Walls Design.

8.2.1 Axial Loading Design.

(1) The design vertical load resistance of a single leaf wall per unit length, N_{Rd}, is given by:

$$N_{Rd} = \frac{\Phi_{i,m} \cdot t \cdot f_k}{\gamma_M}$$

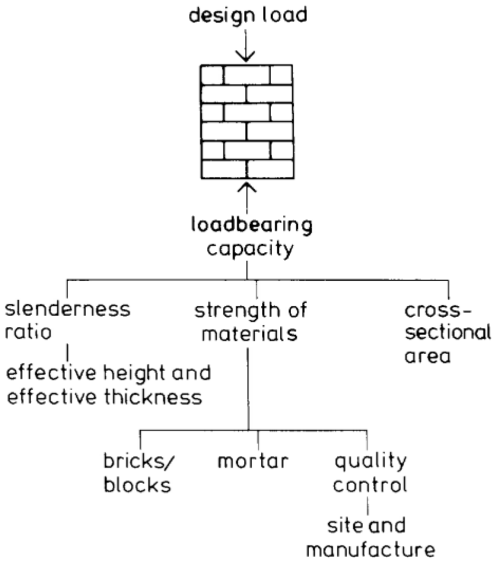
where:

$\Phi_{i,m}$ is the capacity reduction factor Φ_i or Φ_m, as appropriate, allowing for the effects of slenderness and eccentricity of loading;

f_k is the characteristic compressive strength of masonry;

γ_M is the partial safety factor for the material;

t is the thickness of the wall, taking into account the depth of recesses in joints greater than 5 mm.



Symbol: **Φ**

(I) At the top or bottom of the wall.

$$\Phi_i = 1 - 2\frac{e_i}{t}$$

where:

e_i is the eccentricity at the top or the bottom of the wall:

$$e_i = \frac{M_i}{N_i} + e_{hi} + e_a \geq 0{,}05\,t$$

M_i is the design bending moment at the top or the bottom of the wall resulting from the eccentricity of the floor load at the support,

N_i is the design vertical load at the top or bottom of the wall,

e_{hi} is the eccentricity at the top or bottom of the wall, if any, resulting from horizontal loads (for example, wind),

e_a is the accidental eccentricity

t is the thickness of the wall.

An accidental eccentricity, e_a,

- shall be assumed for the full height of the wall to allow for construction imperfections,
- may be assumed to be $h_{ef} / \boxed{450}$, where h_{ef} is the effective height of the wall.

- γ_M= 2
- t= 1 m
- f_k= 24649 KN/m^2
- The eccentricity is calculated in RFEM model and add to applied straining actions.
- Slenderness Ratio (S.R)= H/t= 10100/1000= 10.1 > 27 …………………..(**Safe**)
- ϕ_i= 1
- N_{RD}= $\frac{1*24649*1}{2}$= 12324.5 KN/m'

From RFEM model, the normal stresses bigger than N_{RD}, equal to zero as shown in Fig. 8.1.

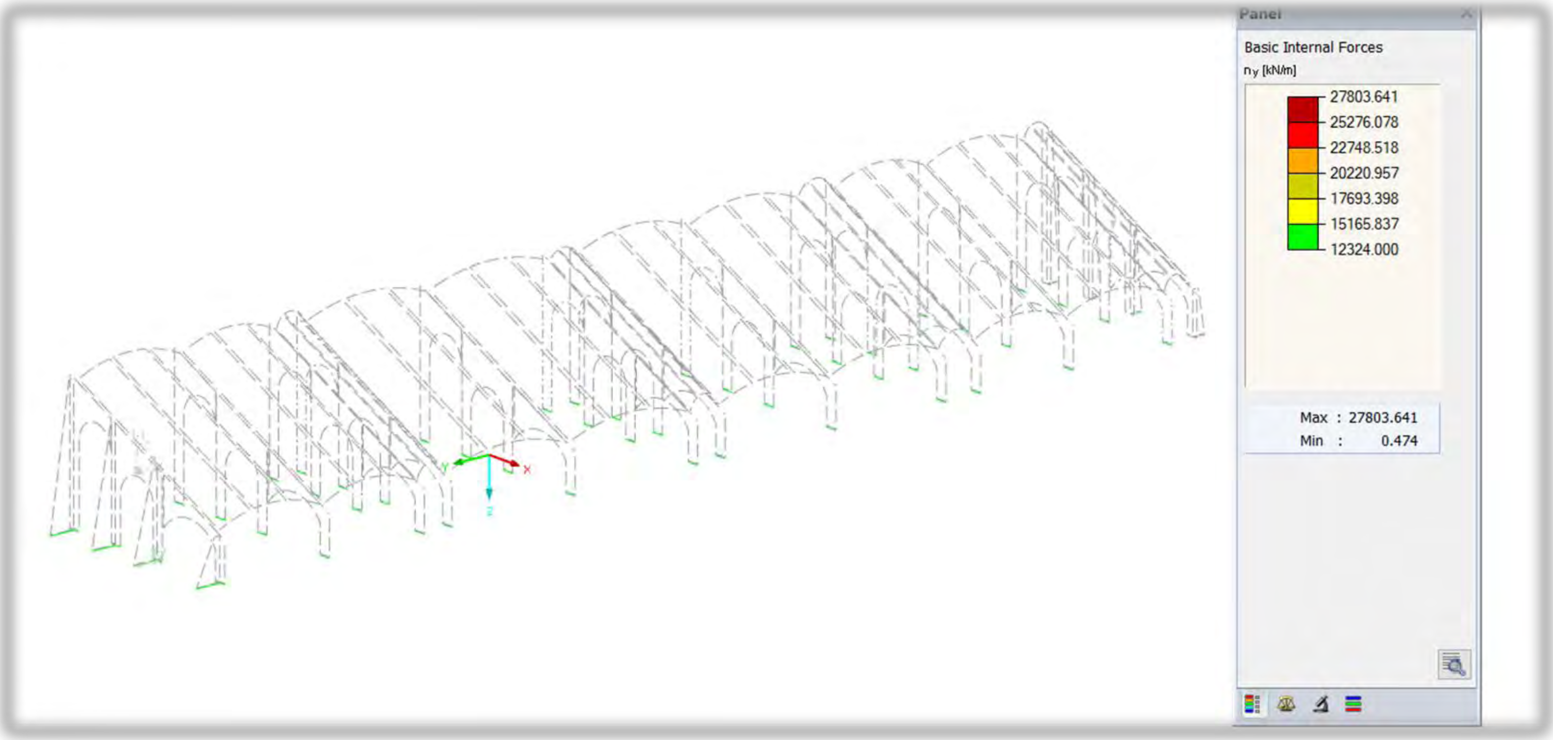
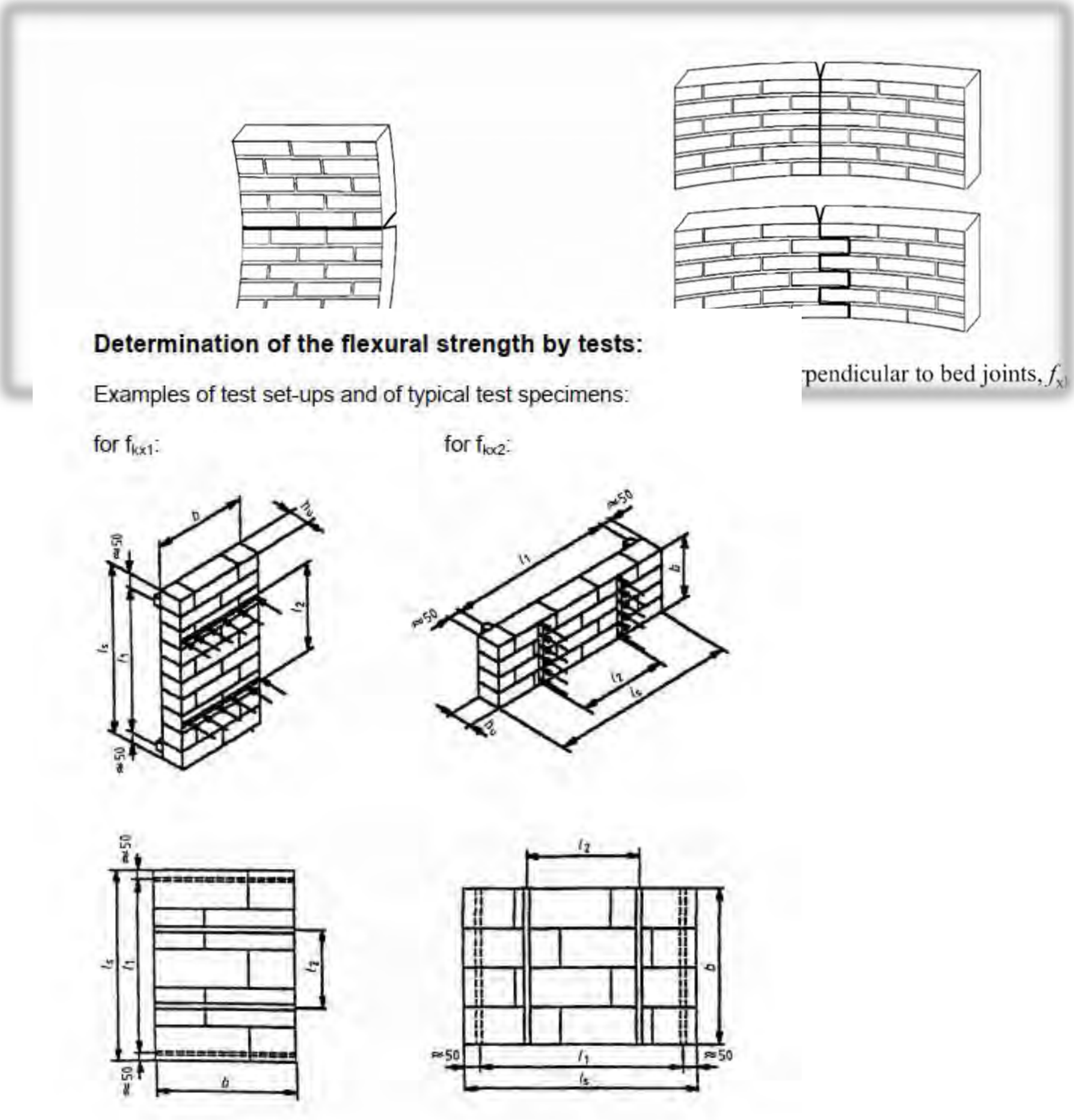


Figure 8.1: Check on Normal Stresses in local axis y direction (KN-m')

8.2.2 Bending Moment Design.

According to mortar characteristics and test flexural test results, the Tensile flexural strength of Natural stone masonry equal to:

- F_{xk1}= 0.6 N/mm^2
- F_{xk2}= 1.2 N/mm^2



pendicular to bed joints, f_x

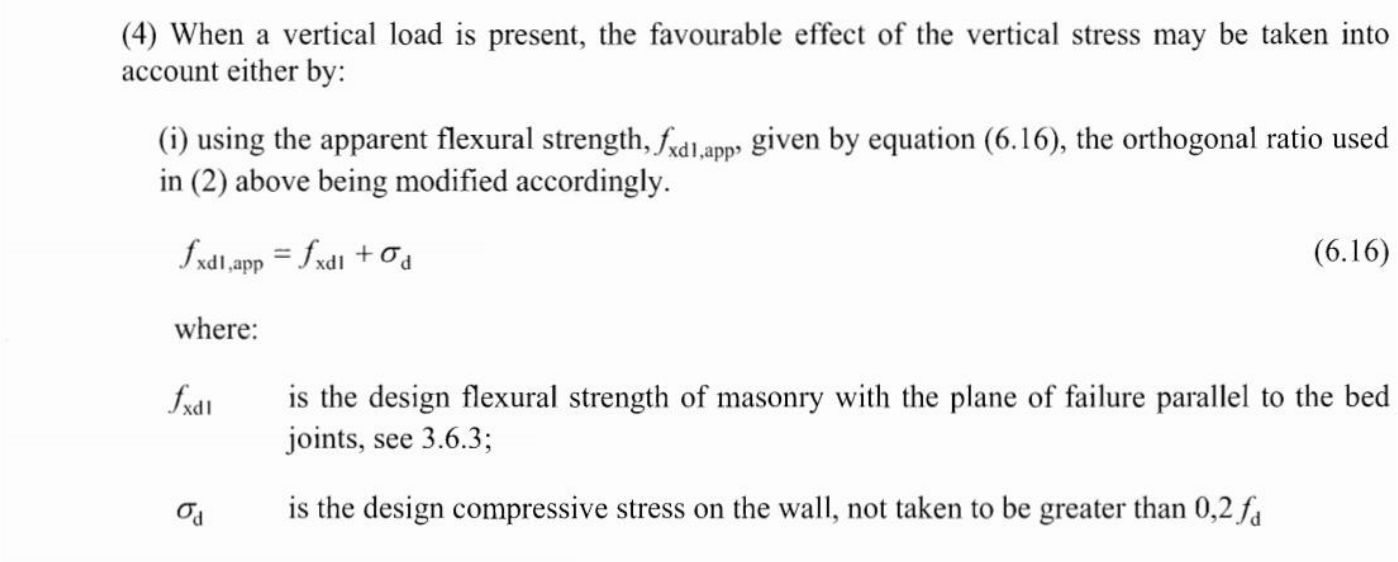
Determination of the flexural strength by tests:

Examples of test set-ups and of typical test specimens:

for f_{kx1}:

for f_{kx2}:

According to EN 1996-1-1, clause 6.3.1, the combined interaction between axial and bending moment will follow the following equations:



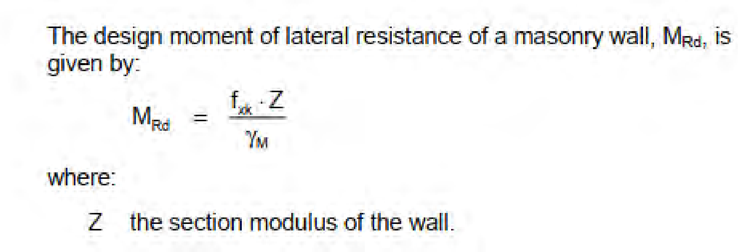
(4) When a vertical load is present, the favourable effect of the vertical stress may be taken into account either by:

(i) using the apparent flexural strength, $f_{xd1,app}$, given by equation (6.16), the orthogonal ratio used in (2) above being modified accordingly.

$$f_{xd1,app} = f_{xd1} + \sigma_d \tag{6.16}$$

where:

f_{xd1}	is the design flexural strength of masonry with the plane of failure parallel to the bed joints, see 3.6.3;
σ_d	is the design compressive stress on the wall, not taken to be greater than 0,2 f_d

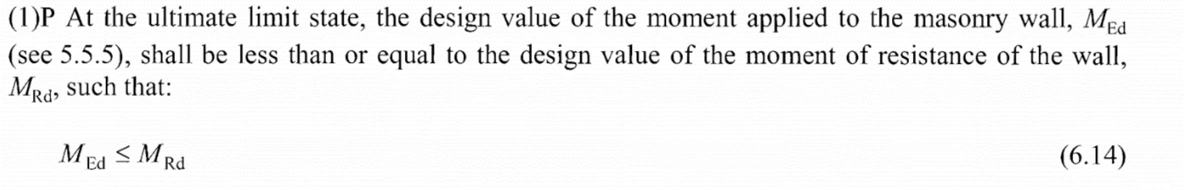


The design moment of lateral resistance of a masonry wall, M_{Rd}, is given by:

$$M_{Rd} = \frac{f_{xk} \cdot Z}{\gamma_M}$$

where:

Z the section modulus of the wall.



(1)P At the ultimate limit state, the design value of the moment applied to the masonry wall, M_{Ed} (see 5.5.5), shall be less than or equal to the design value of the moment of resistance of the wall, M_{Rd}, such that:

$$M_{Ed} \leq M_{Rd} \tag{6.14}$$

- **Plan of failure parallel to bed joints.**

  The average minimum compression force act on the wall is 26.5+40=66.5 KN/m^2 as show in Fig. 8.2.

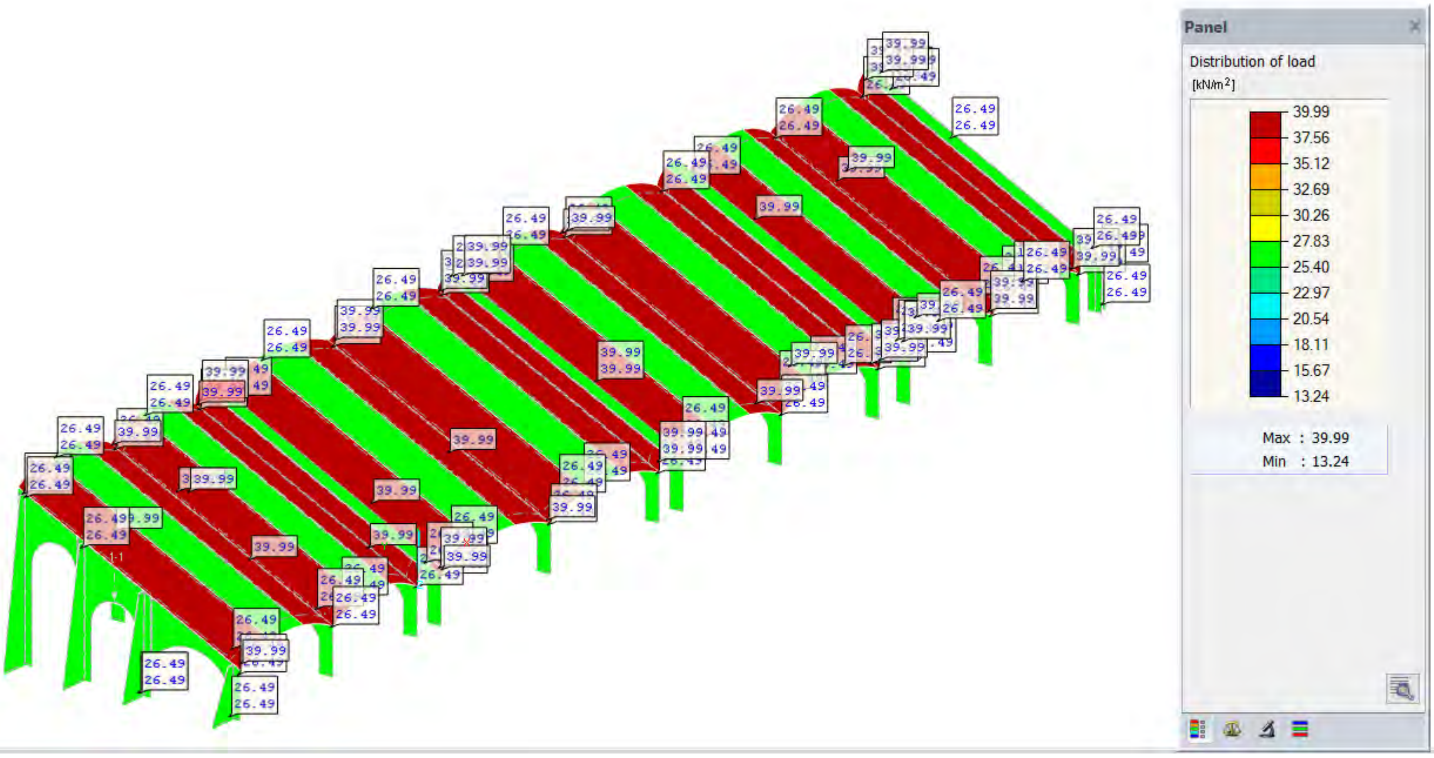


Figure 8.2: axial load induced from Favourable load combinations (KN-m^2)

- σ_d= 66.5 KN/m^2 = 0.0665N/mm^2
- $F_{cd1,app}$= 0.6 + 0.0665= 0.67 N/mm^2
- Z= bt^2/6 = (1*1)/6= 0.17 m^3
- M_{RD}= $\frac{666.5*0.17}{2}$ = 56.65 KN.m/m’

The applied bending moment (M_{ED}) < the resistance bending moment (M_{RD}) except some regions shown in Fig.8.3; thus, these regions will need reinforcement.

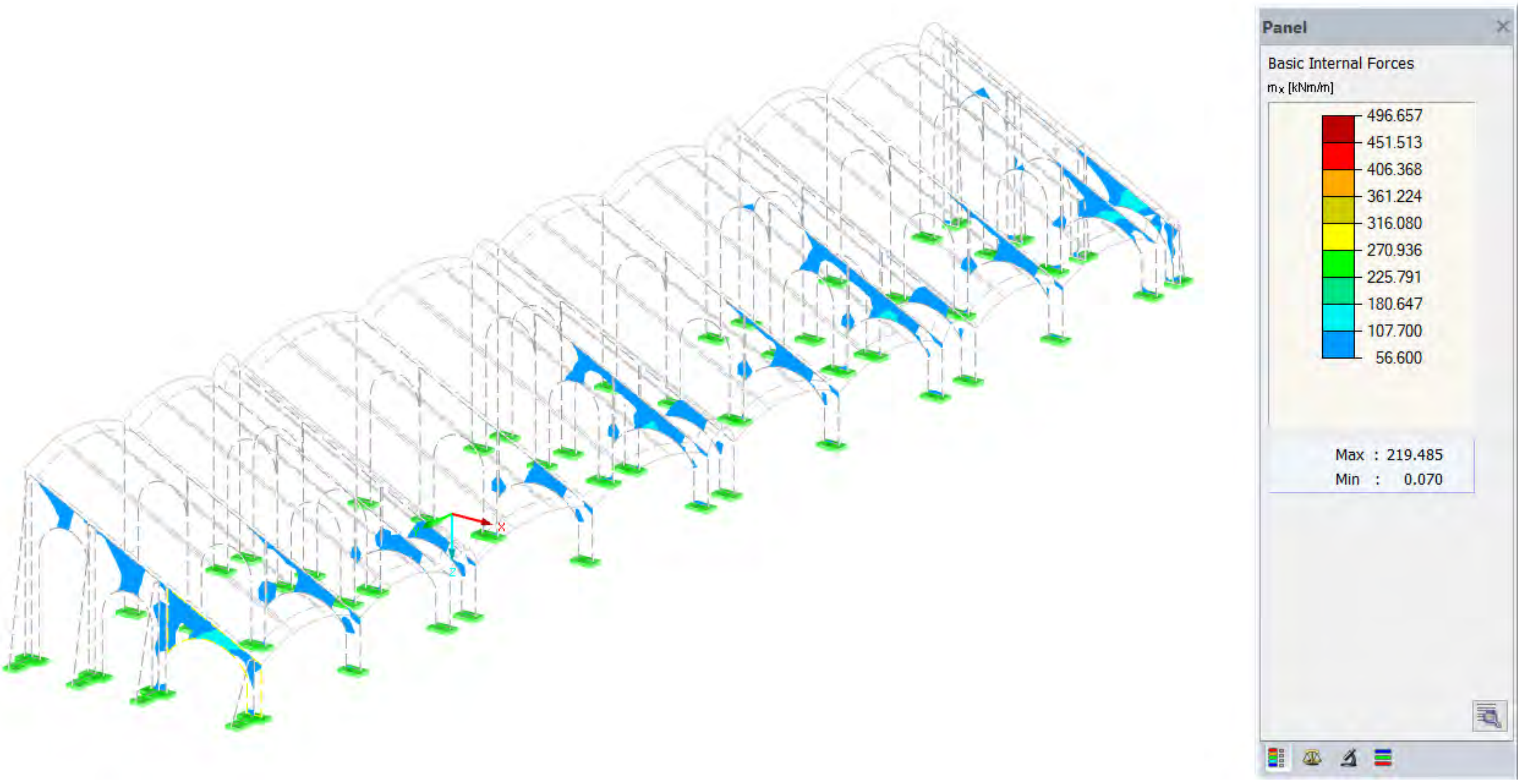


Figure 8.3: unsafe regions in bending moment in loxal axes x direction (KN.m-m′)

- **Plan of failure perpendicular to bed joints.**

  The average minimum compression force act on the wall is 26.5+40=66.5 KN/m^2 as show in Fig. 8.2.

  - σ_d= 66.5 KN/m^2 = 0.0665N/mm^2
  - $F_{cd1,app}$= 1.2 + 0.0665= 1.266 N/mm^2
  - Z= bt^2/6 = (1*1)/6= 0.17 m^3
  - M_{RD}= $\frac{1266.5*0.17}{2}$ = 107.7 KN.m/m’

  The applied bending moment (M_{ED}) > the resistance bending moment (M_{RD}) in the most regions of walls as shown in Fig.8.4 ; thus, walls will design as reinforced masonry walls in that direction.

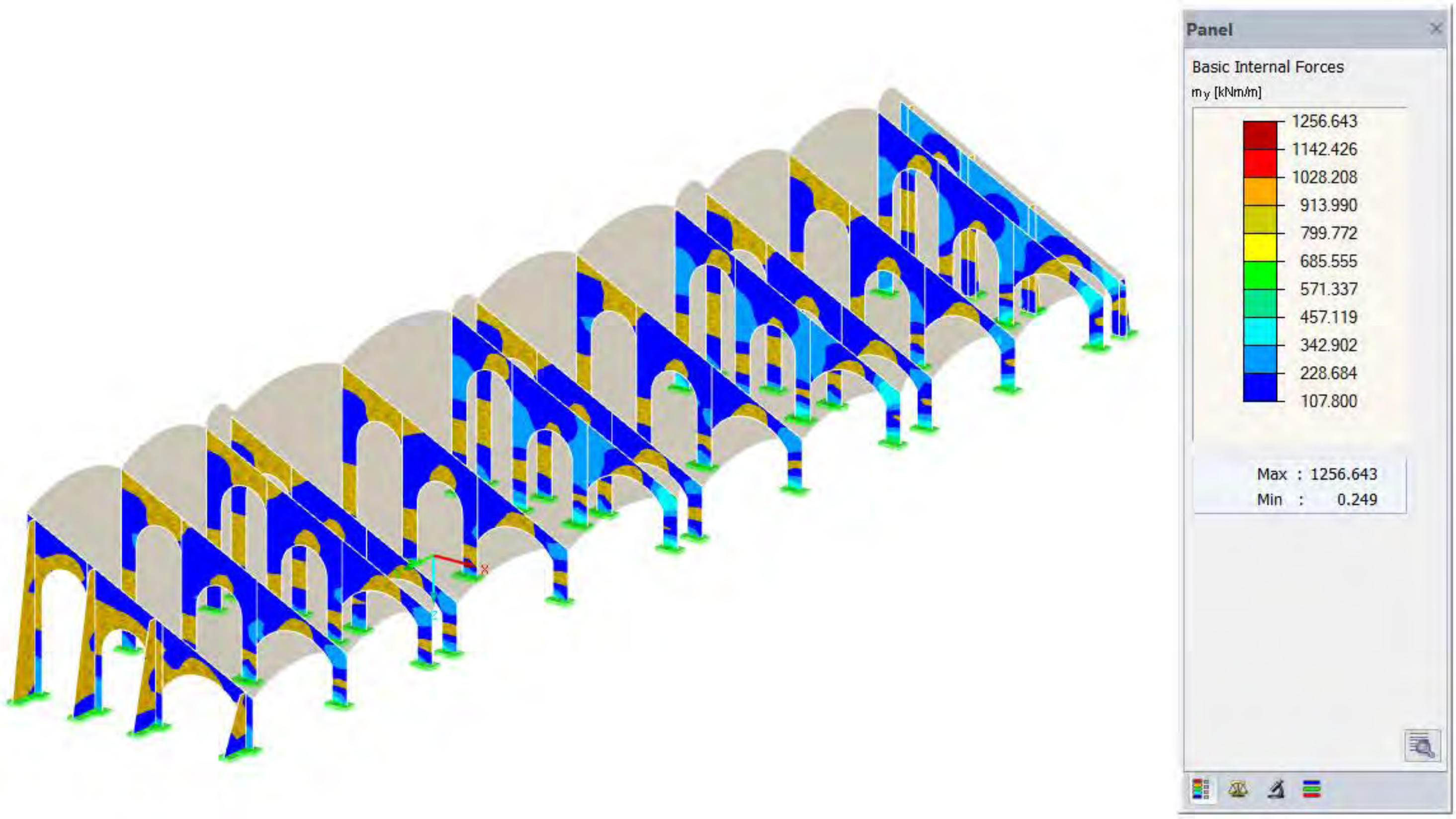
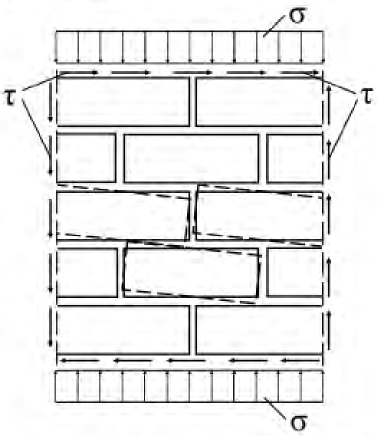


Figure 8.4: unsafe regions in bending moment in loxal axes y direction (KN.m-m$'$)

8.2.3 Shear Design.

Behaving of masonry under shear:

Element cut off a wall:

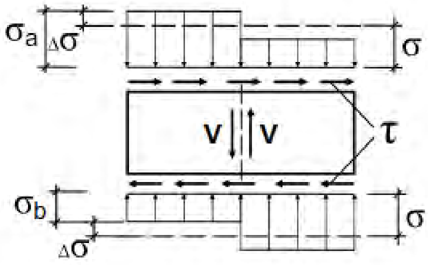


In the global system shear stresses τ act not only in horizontal but also in vertical direction (due to the equilibrium of moments at the element).

Locally in the perpend joints shear stresses cannot be transferred due to the following reasons:

- the surface of the unit heads are often very smooth,
- there are no normal stresses acting in the perpend joints, therefore there is no friction possible,
- the shrinkage of mortar reduces the possible adhesion,
- vertical joints often are not fully filled with mortar.

So if shear stresses only act in the bed joints, there must be a change in the distribution of the vertical normal stresses, as the equilibrium of a single stone shows. The stresses must become a stepped distribution due to the kinematics of deformations.



Three failure modes occur:

- small load σ: failure in the bed joint, due to τ under friction
- larger load σ: fracture of units, due to the principal tensile stress, deriving from σ and τ in the middle of units,
- very high load σ: failure of units, due to the pressure σ_a.

The characteristic shear strength f_{vk} of unreinforced masonry can be determined

- from the results of tests on masonry,
- by calculation in the following way:

For general purpose mortar and when all joints may be considered as filled, f_{vk} will not fall below the least of the values described below:

$$f_{vk} = f_{vko} + 0{,}4\,\sigma_d$$

or $= \boxed{0{,}065} \cdot f_b$, but not less than f_{vko}

or = the limiting value given in table 3.5

where:

f_{vko} is the shear strength, under zero compressive stress

σ_d is the design compressive stress perpendicular to the shear

for vertical loading and for shear loading:

$$V_{sd} \leq V_{Rd}$$

V_{sd}: design value of the applied shear load

V_{Rd}: design shear resistance

$$V_{Rd} = \frac{f_{vk}\ t\ l_c}{\gamma_M}$$

According to EN 1996-1-1, clause 3.6.2, the shear strength of stone masonry with M2 mortar will equal 0.1 N/mm2.

Assume when use mortar M12, the shear strength will equal 0.6 N/mm^2.

- F_{vk0}= 0.6 N/mm^2

The average minimum compression force act on the wall is 26.5+40=66.5 KN/m^2 as show in Fig. 8.2

- F_{vk}= 0.6 + 0.4*0.0665= 0.67 N/mm^2
- l_c= 1m
- $V_{RD} = \frac{670*1*1}{2}$= 335 KN/m’

the applied shear (V_{ED}) in bed joint direction is less than the resistance shear (V_{RD}) except some regions as shown in Fig.8.5; thus these regions will need reinforcement.

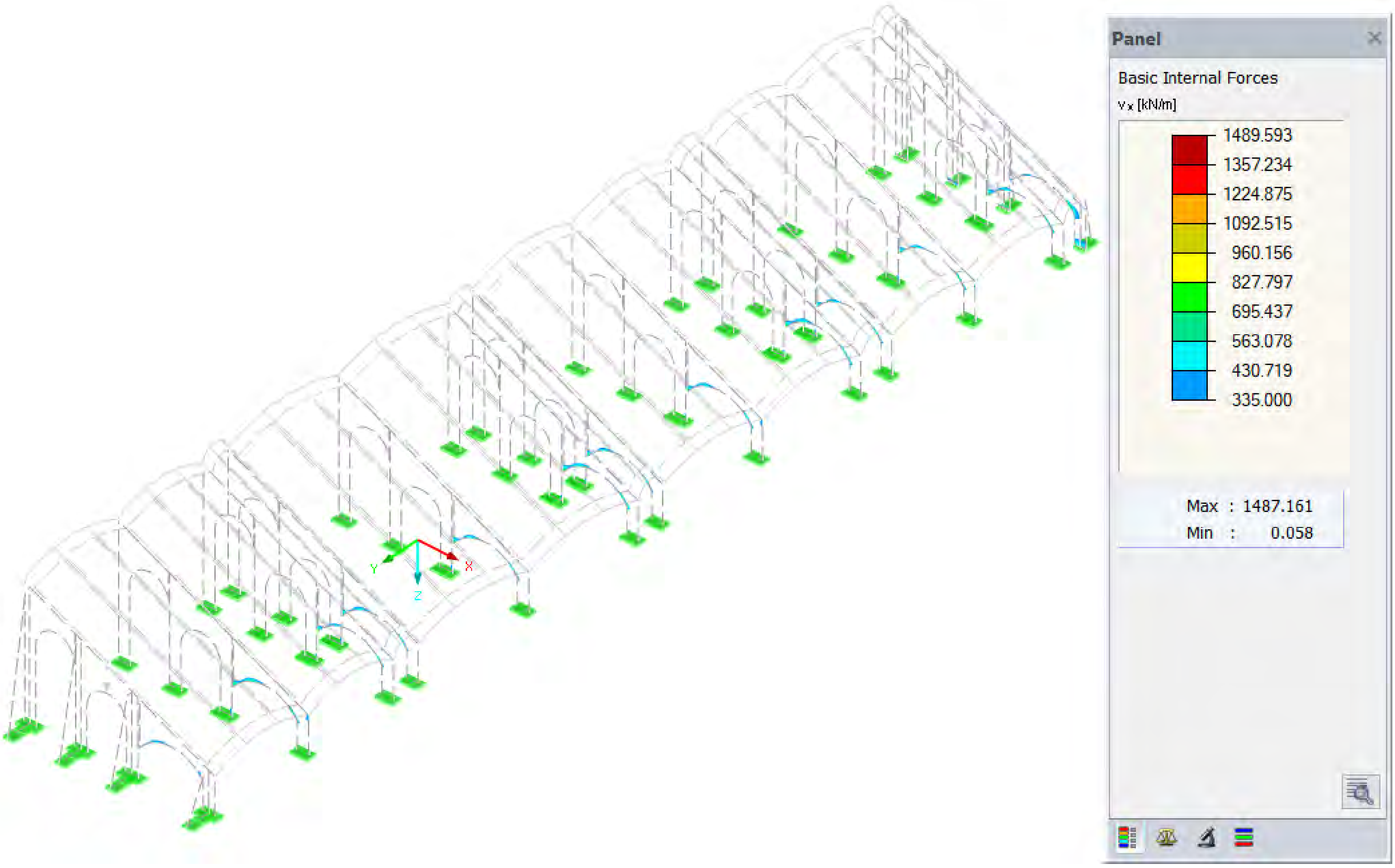
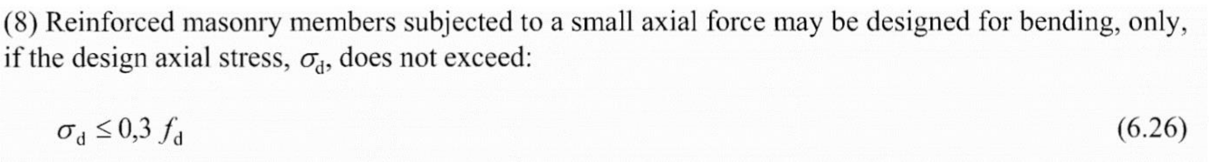


Figure 8.5: unsafe regions in Shear forces in loxal axes x direction (bed joints direction) (KN-m′)

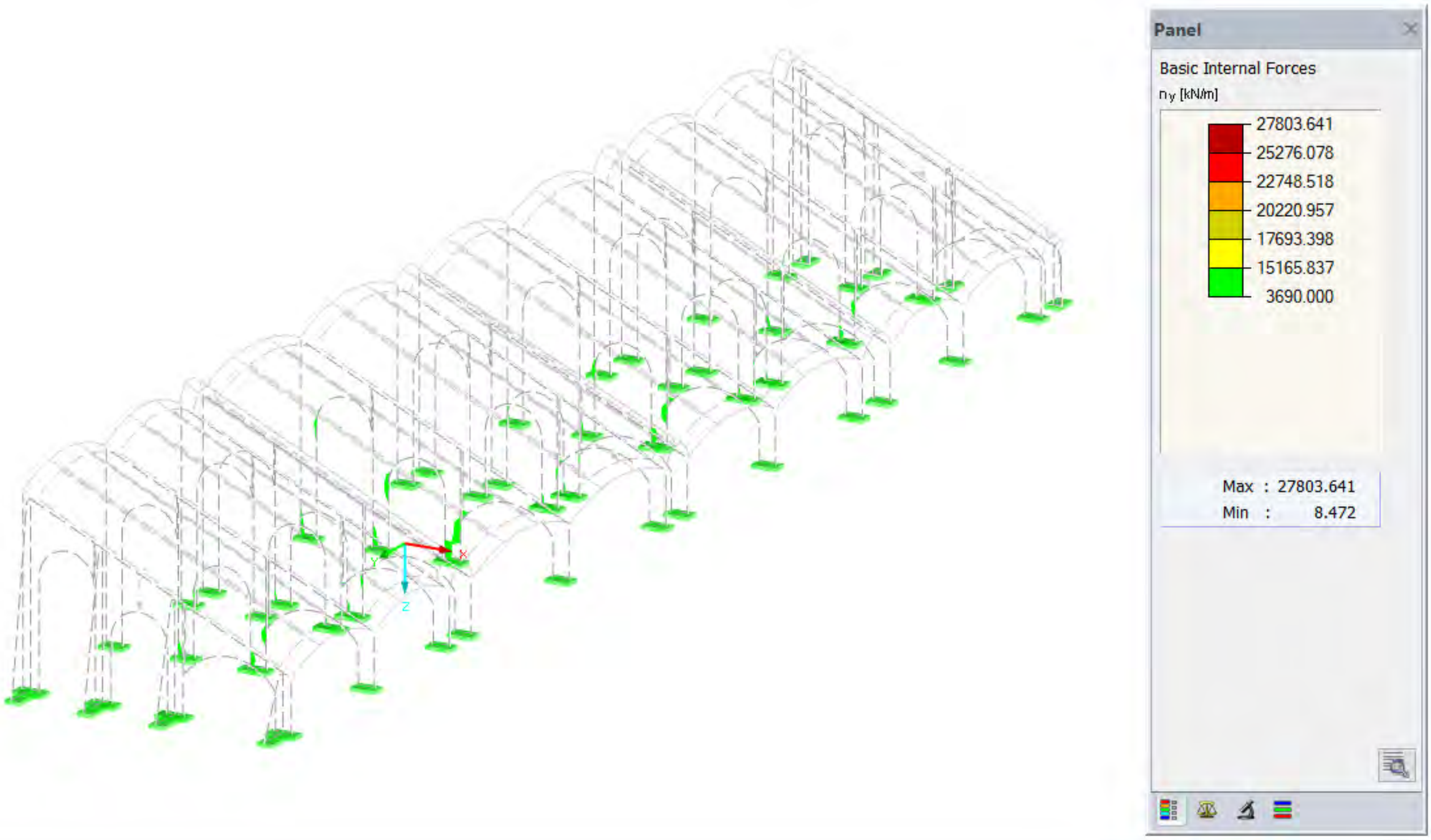
8.2.4 Reinforcement Calculations.

According to EN 1996-1-1, clause 6.6.2, eq. 6.26, the applied axial stress is less than 0.3*axil strength of masonry as show in fig. 8.6.



(8) Reinforced masonry members subjected to a small axial force may be designed for bending, only, if the design axial stress, σ_d, does not exceed:

$$\sigma_d \leq 0{,}3\, f_d \qquad (6.26)$$



Max : 27803.641
Min : 8.472

Figure 8.6: Axial forces values, bigger than thirty percent of axial strength of masonry (KN-m')

According the EC 1996-1-1, clause 6.6.2, the design of reinforced masonry subjected to bending moment follows the following equations.

$$M_d = A_s z f_y / \gamma_{ms}$$

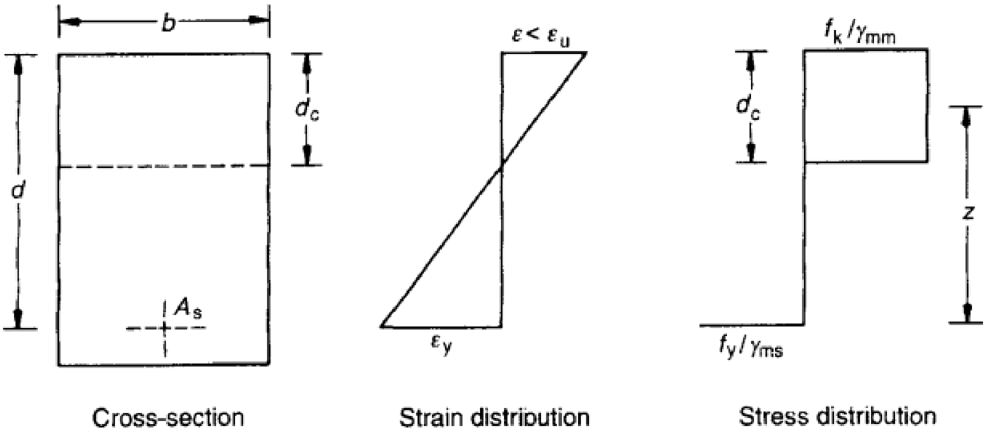
$$z = (d - d_c/2)$$

$$A_s f_y / \gamma_{ms} = b d_c f_k / \gamma_{mm}$$

so that

$$d_c/d = A_s f_y \gamma_{mm} / b d f_k \gamma_{ms}$$

$$z = d(1 - 0.5 A_s f_y \gamma_{mm} / b d f_k \gamma_{ms})$$



The methods of reinforcement shown in fig. 8.7.

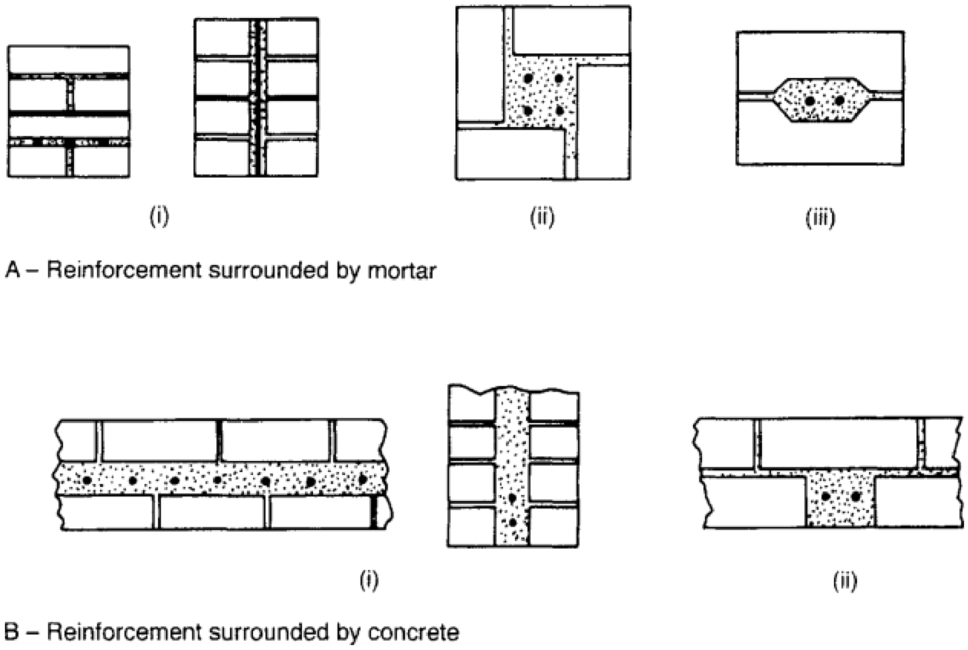


Figure 8.7: Methods of Masonry Reinforcement.

By using Type A reinforcement method as show in fig. 8.7 and fig. 8.8.

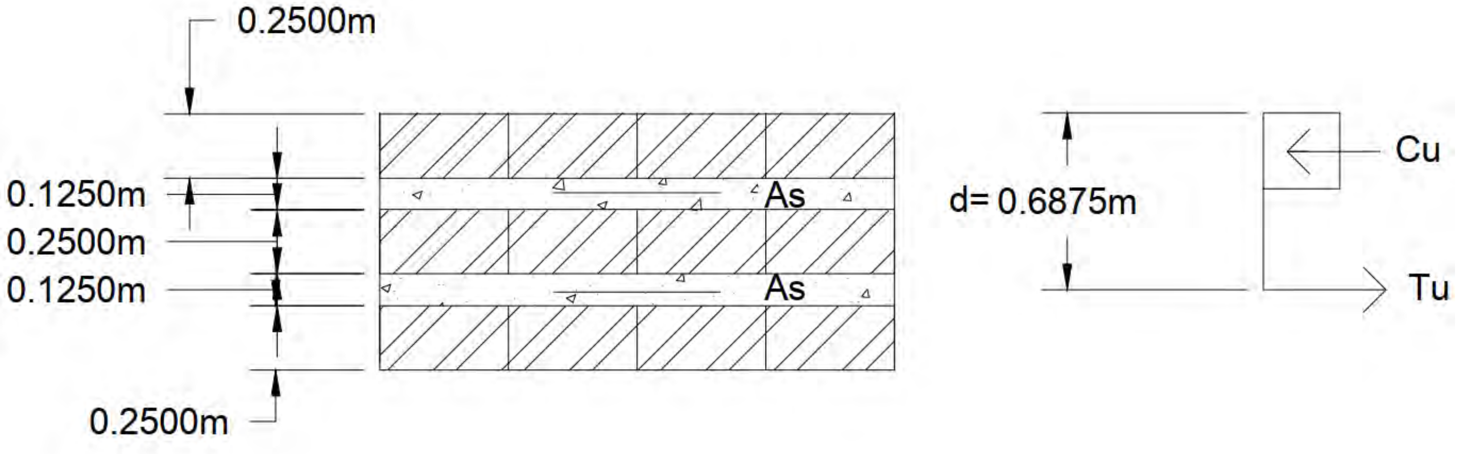


Figure 8.8: Section in masonry wall.

➢ Plan of failure perpendicular to bed joints.

- d= 687 mm
- M_d= 260 KN.m/m’
- F_y= 360 N/mm^2
- F_k= 24.6 N/mm^2
- γ_{ms}= 1.15
- γ_{mm}= 2
- Z= 687*(1 - 0.5*$\frac{360*As*2}{24.6*1000*687*1.15}$)
- Z= 686.987 As
- 260*10^6= As*(360/1.15)*686.987 As
- 260*10^6= 215056.8 As
- As= 260*10^6 / 215056.8 =1209 mm^2/m’……………………………….use 5 T 18/m’ as a vertical RFT

By using 5 T 18/m’ as a vertical RFT, the covered moment will equal:

- Z= 687*(1 - 0.5*$\frac{1272*2}{24.6*1000*687*1.15}$)= 686.98 mm
- M_d= 1272*(360/1.15)*686.98= 272 KN*m/m’

By using 3 T 18/m’ as additional RFT distributed in the regions shown in fig. 8.9, the covered moment will equal:

- $M_{d,\ additional}$= 763*(360/1.15)*686.98= 164 KN*m/m’
- $M_{d,total}$= 164 + 272= 436 KN.m/m’

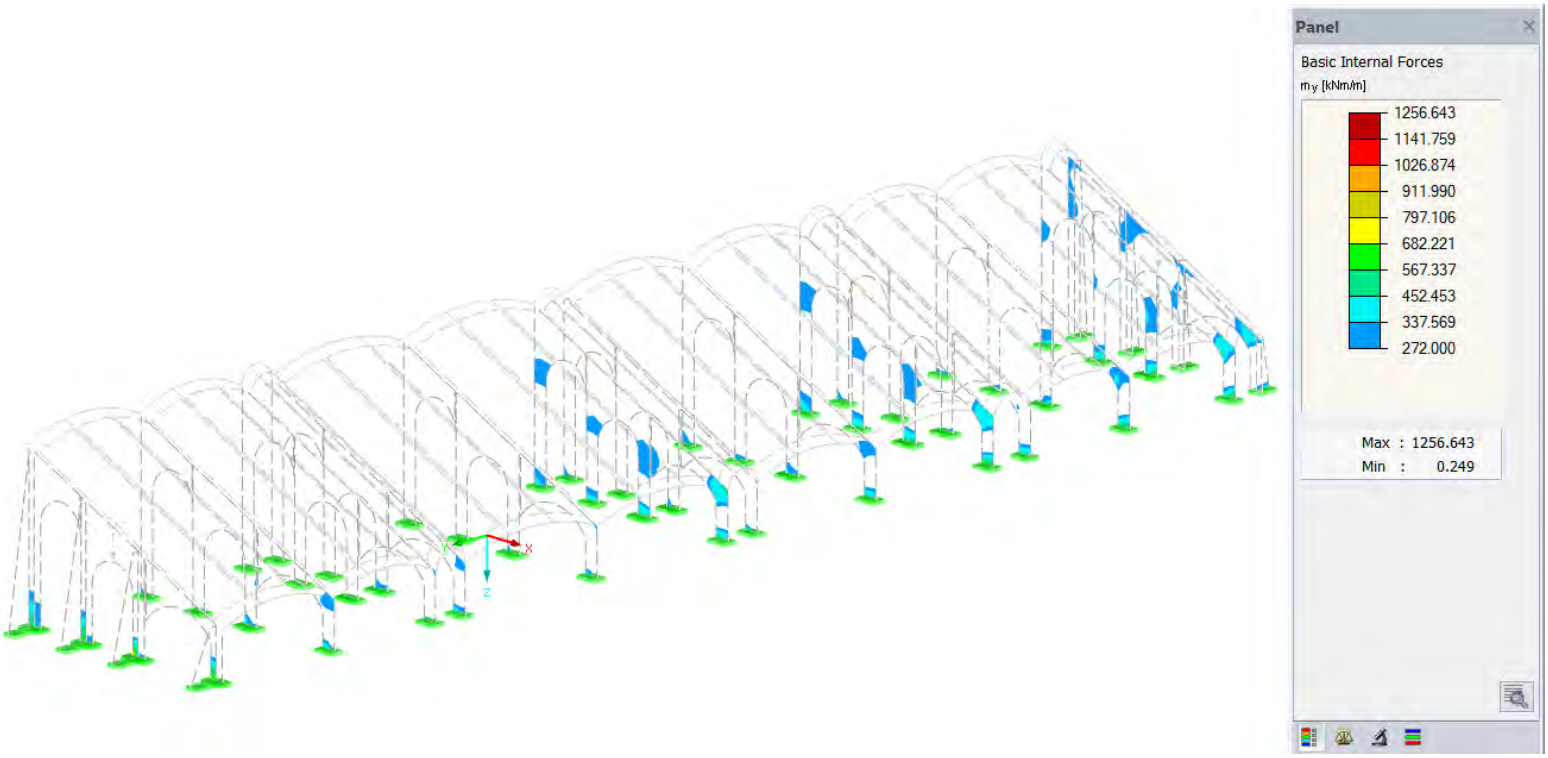


Figure 8.9: Covered bending moment in local axis y direction induced from 5 T 18/m’ rebars.

- **Plan of failure Parallel to bed joints.**

By using 2 T 10/m' as horizontal RFT distributed in the total area of the wall except

the regions shown in fig.8.3.

the function of using 2 T 10/m' rebars is to tie vertical RFT together for ease the construction.

The covered bending moment induced from 7 T 10/m' rebars in the unsafe regions shown in fig. 8.3 will equal

- $M_{d,\ additional}$= 549.7*(360/1.15)*686.98= 118.2 KN*m/m'

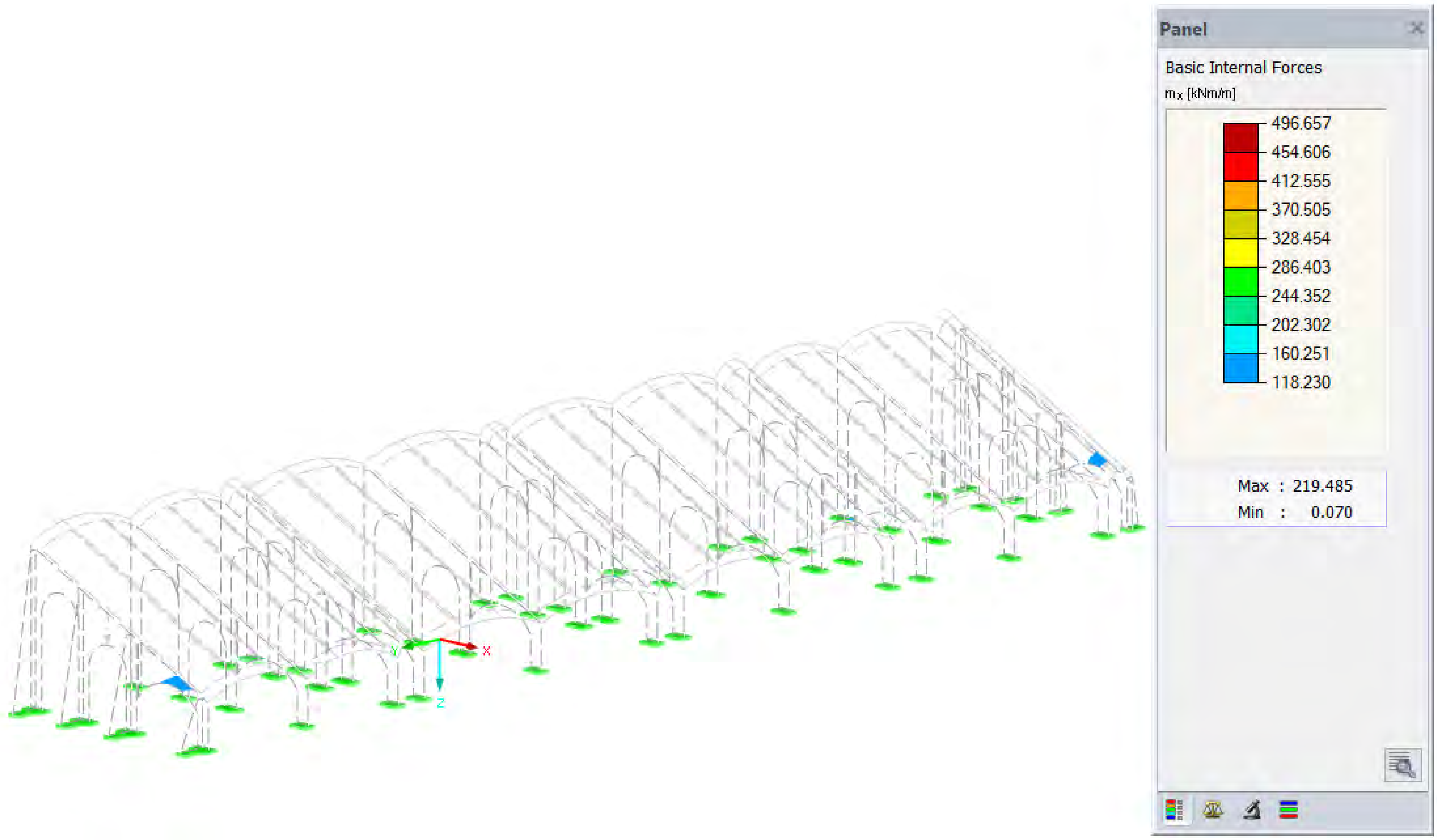


Figure 8.10: Covered bending moment in local axis x direction induced from 7 T 10/m' rebars.

There are small unsafe regions in bending moment in local x axis direction as shown in fig. 8.10. The author neglect these regions because of the following reasons:-

- These regions are small area regions.
- The unsafe bending moment in it, is low, equal to 47 KN.m/m'.
- EC 1996-1-1 assumes that, when design the reinforced masonry, the flexural tensile strength of masonry will be neglected and actually that doesn't occur because the masonry has flexural tensile strength as shown in part 8.3.2.

8.3 Arched Slab Design.

Most Masonry arches are considered to be fixed arches, there are no hinges. The downward loads on the arch creates lateral and comperession thrusts in the arch span (see fig. 8.11) which puch the masonry units against each other and compress them, and in turn the arch thrust against the abutments.

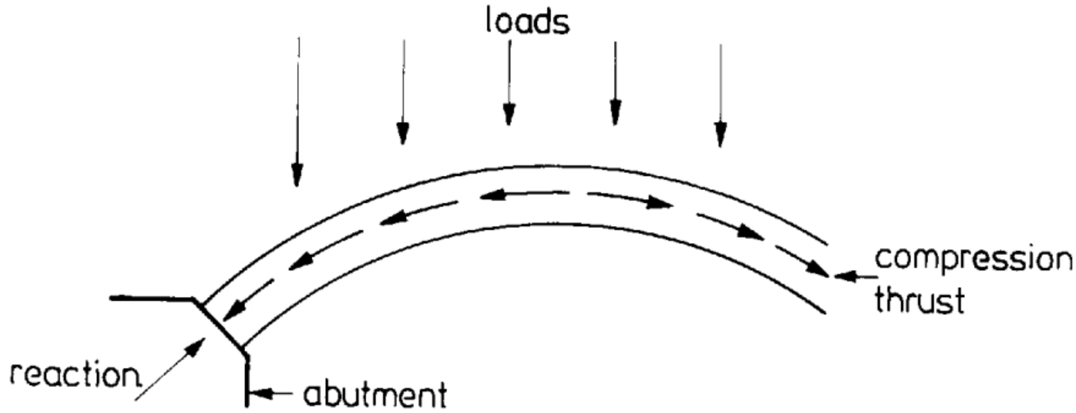


Figure: 8.11

If the line of thrust is on the center of the arch, the arch ring is under uniform compression stress (see fig. 8.12).

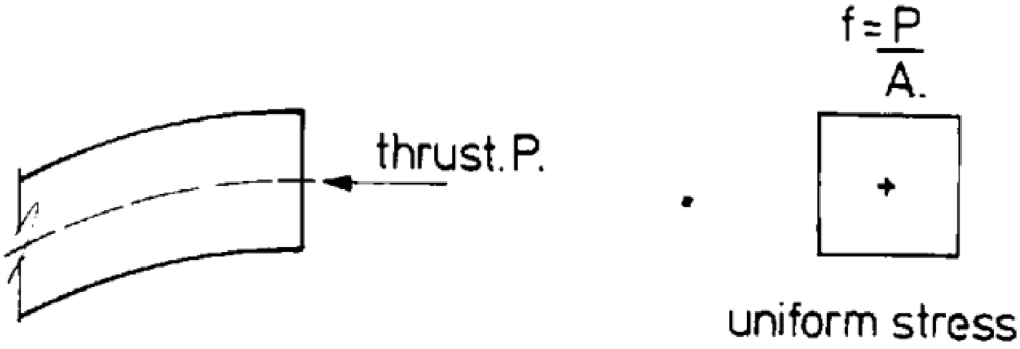


Figure: 8.12

The line of thrust doesn't always pass along the centerline of the arch, and the arch isn't then in uniform compression (see fig.8.13)

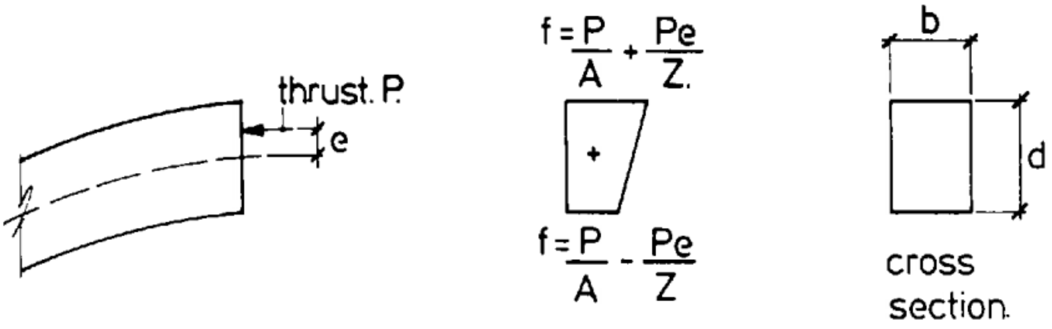


Figure: 8.13

In the fact, the line of thrust can lie outside the middle third of the arch thickness, tensile stress can develop and crack can occur. The line of thrust can move can move to the edge of the arch ring and a hinge will develop, but the arch necessary collapse.

8.3.1 Check on compression stresses.

- γ_M= 2
- t= 1 m
- Z= 0.17 m^3
- f_k= 24.649 N/mm^2
- Slenderness Ratio (S.R)= L/t= 9000/1000= 9 > 27 …………………..(**Safe**)
- ϕ_i= 1
- F_{RD}= $\frac{1*24.649}{2}$ = 12.3 N/mm^2

From RFEM model, the normal stresses bigger than F_{RD}, equal to zero as shown in Fig. 8.14.

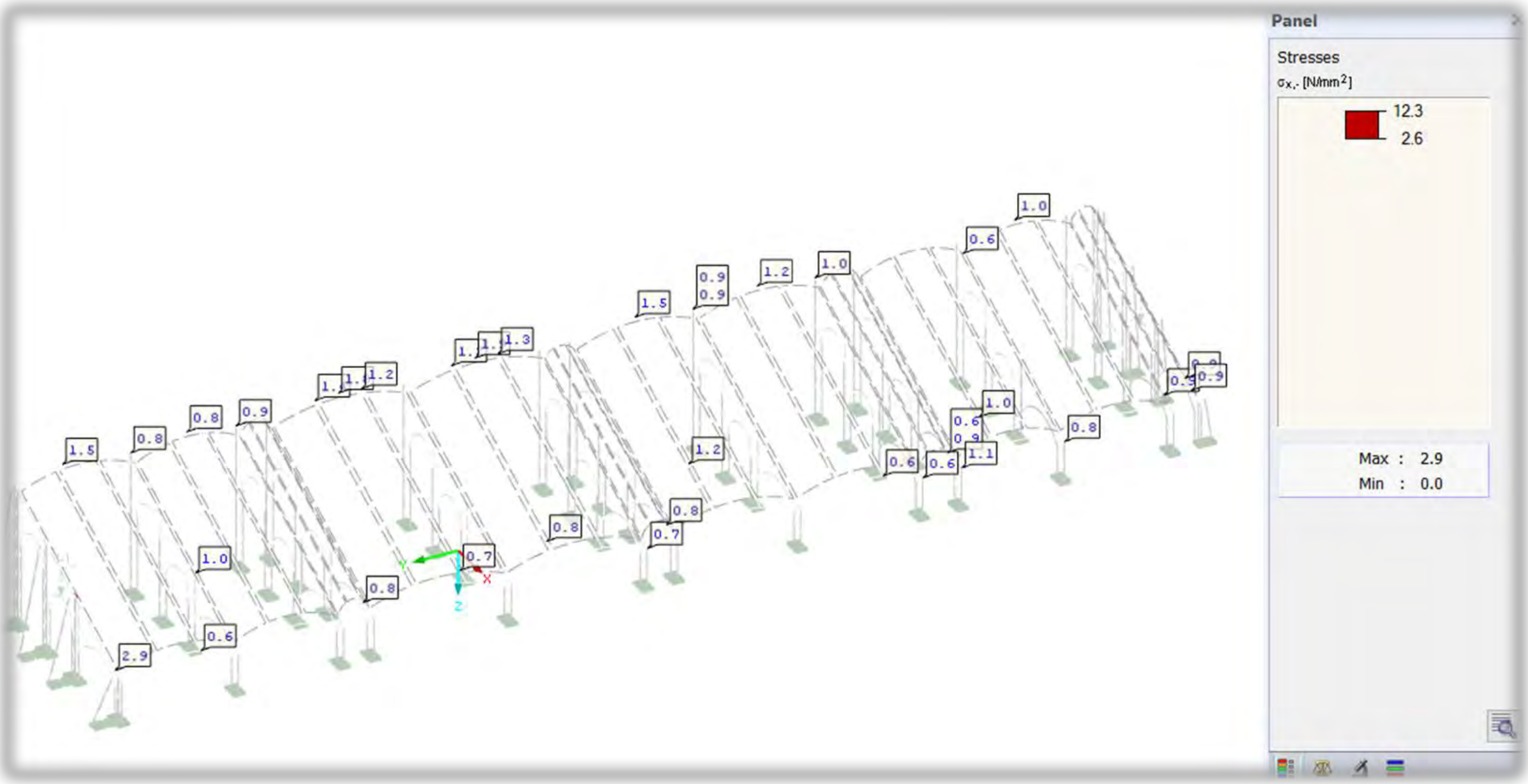
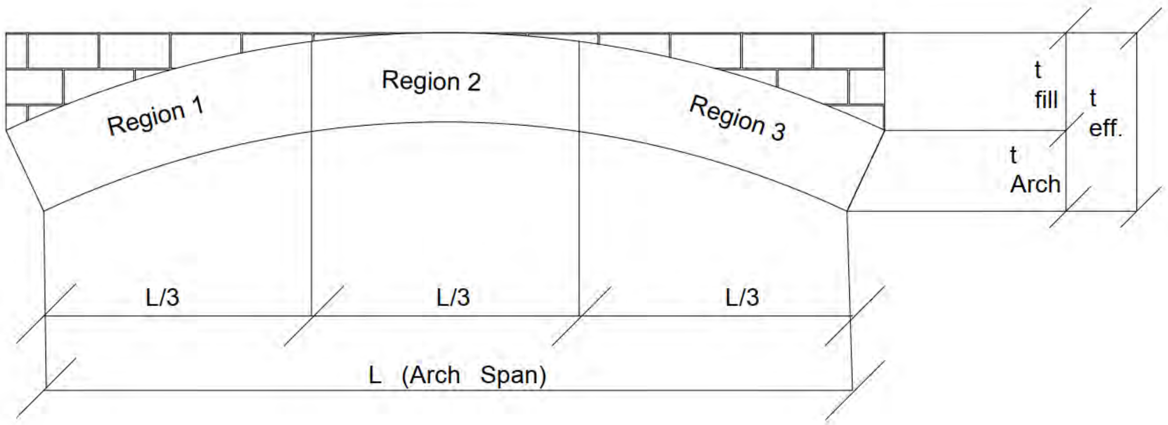


Figure 8.14: Check on Compression Stresses in arch thrust line direction (N-mm²)

8.3.2 Check on tension stresses.

The arched slab will divide into three regions as shown in the following figure, and the tension stresses will check in each region.



8.3.2.1 Region 1 & 3.

- γ_M= 2
- $t_{eff.}$= t_{arch} + t_{fill} = 1+1= 2m
- $t_{average,\ eff.}$= ($t_{eff.}$ + t_{arch}) / 2 = (2+1) / 2 = 1.5 m
- $Z_{average,eff.}$= 0.375 m^3
- $Z_{eff.=}$ 0.67 m^3

The minimum compression forces act on the arched slab are 40 KN/m^2 as show in Fig. 8.2.

➢ Plan of failure parallel to bed joints.

The Masonry tensile Strength.

- σ_{d1}= 40 KN/m^2 = 0.04N/mm^2
- $F_{cd,app1}$= 0.6 + 0.04= 0.64 N/mm^2
- $F_{Rd,app1}$= $\frac{0.64}{2}$ = 0.32 N/mm^2

The Maximum Applied Tensile Stresses.

- n_y= 100 KN/m'
- m_y= 225 KN.m/m'
- σ_y= $\frac{100*1000}{2000*1000} - \frac{225*1000000}{0.67*10^9}$= 0.285 (Tension) N/mm2 > 0.32………………… (**Safe**)

The Average Applied Tensile Stresses.

- n_y= 208 KN/m'
- m_y= 100 KN.m/m'
- σ_y= $\frac{208*1000}{1500*1000} - \frac{100*1000000}{0.375*10^9}$= 0.128 (Tension) N/mm2 > 0.32………………… (**Safe**)

➢ Plan of failure Perpendicular to bed joints (Thrust line direction).

The Masonry tensile Strength.

- σ_{d1}= 40 KN/m^2 = 0.04N/mm^2
- $F_{cd,app2}$= 1.2 + 0.04= 1.24 N/mm^2
- $F_{Rd,app2}$= $\frac{1.24}{2}$ = 0.62 N/mm^2

The Maximum Applied Tensile Stresses.

- n_x= 76 KN/m’
- m_x= 328 KN.m/m’
- σ_x= $\frac{76*1000}{2000*1000} - \frac{328*1000000}{0.67*10^9}$= 0.45 (Tension) N/mm2 > 0.62………………… (**Safe**)

The Average Applied Tensile Stresses.

- n_x= 60 KN/m’
- m_x= 127 KN.m/m’
- σ_x= $\frac{60*1000}{1500*1000} - \frac{127*1000000}{0.375*10^9}$= 0.29 (Tension) N/mm2 > 0.62………………… (**Safe**)

8.3.2.2 Region 2.

- γ_M= 2
- $t_=$ 1m
- $Z_{.=}$ 0.17 m^3

The minimum compression forces act on the arched slab are 26.5 KN/m^2 as show in Fig. 8.2.

➢ Plan of failure parallel to bed joints.

The Masonry Tensile Strength.

- σ_{d1}= 26.5 KN/m^2 = 0.026N/mm^2
- $F_{cd,app1}$= 0.6 + 0.026= 0.626 N/mm^2
- $F_{Rd,app1}$= $\frac{0.62}{2}$ = 0.31 N/mm^2

From RFEM model, the Regions shown in fig. 8.15 describe the applied tension stresses (σ_y) bigger than masonry tensile strength ($F_{Rd,app1}$).

The masonry sections in these regions, (shown in fig. 8.15) will crack and considered as a cracked sections.

It must check the cracked sections against stability, even not collapse as described in part 8.3.1 (check on compression stresses).

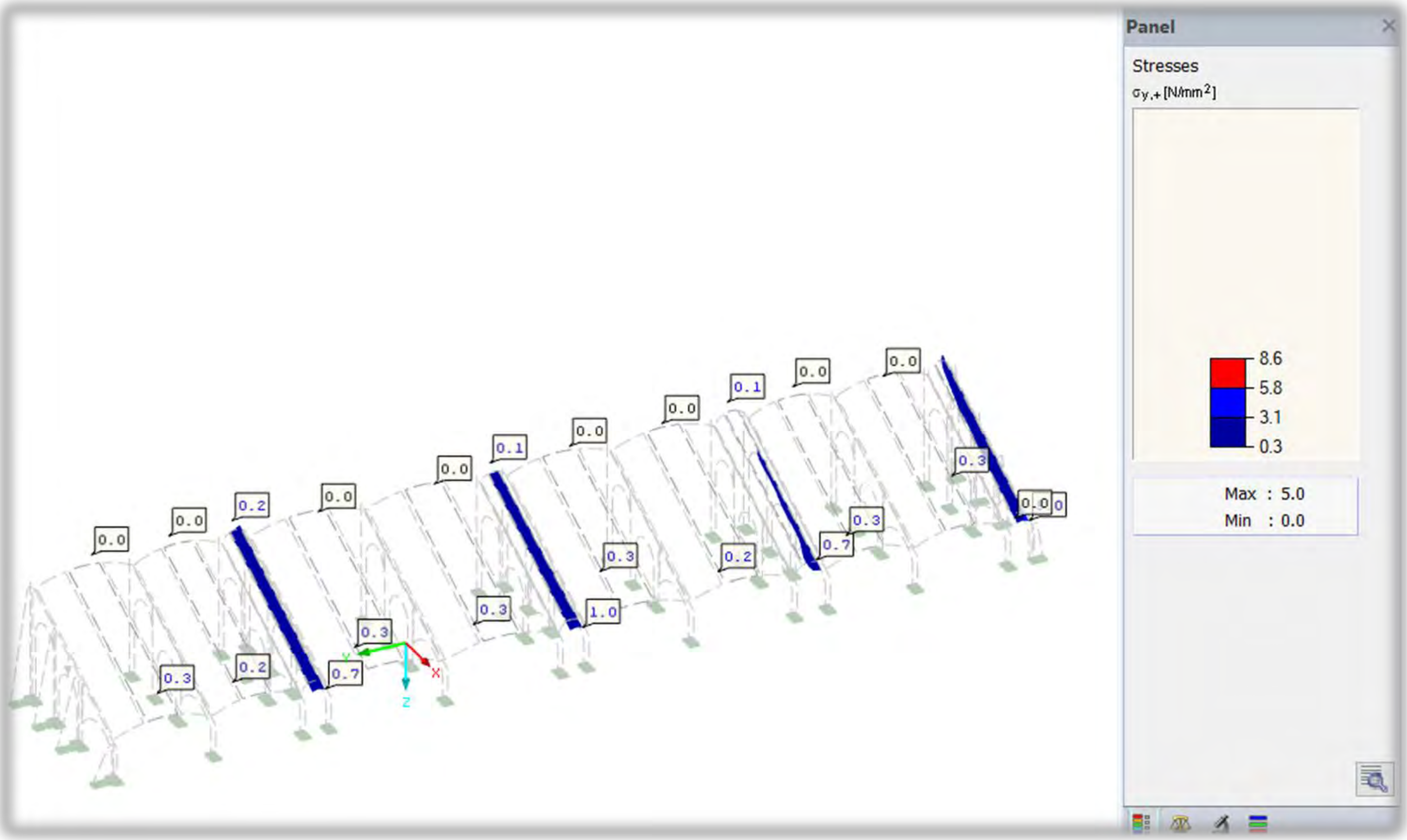


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Figure 8.15: Check on Tensile Stresses in local axis y direction (N-mm^2)

- Plan of failure Perpendicular to bed joints.

The Masonry Tensile Strength.

- σ_{d1}= 26.5 KN/m^2 = 0.026N/mm^2
- $F_{cd,app2}$= 1.2 + 0.026= 1.22 N/mm^2
- $F_{Rd,app2} = \frac{1.22}{2}$ = 0.61 N/mm^2

From RFEM model, the Regions shown in fig. 8.16 describe the applied tension stresses (σ_x) bigger than masonry tensile strength ($F_{Rd,app2}$).

The masonry sections in these regions, (shown in fig. 8.16) will crack and considered as a cracked sections.

It must check the cracked sections against stability, even not collapse as described in part 8.4.1 (check on compression stresses).

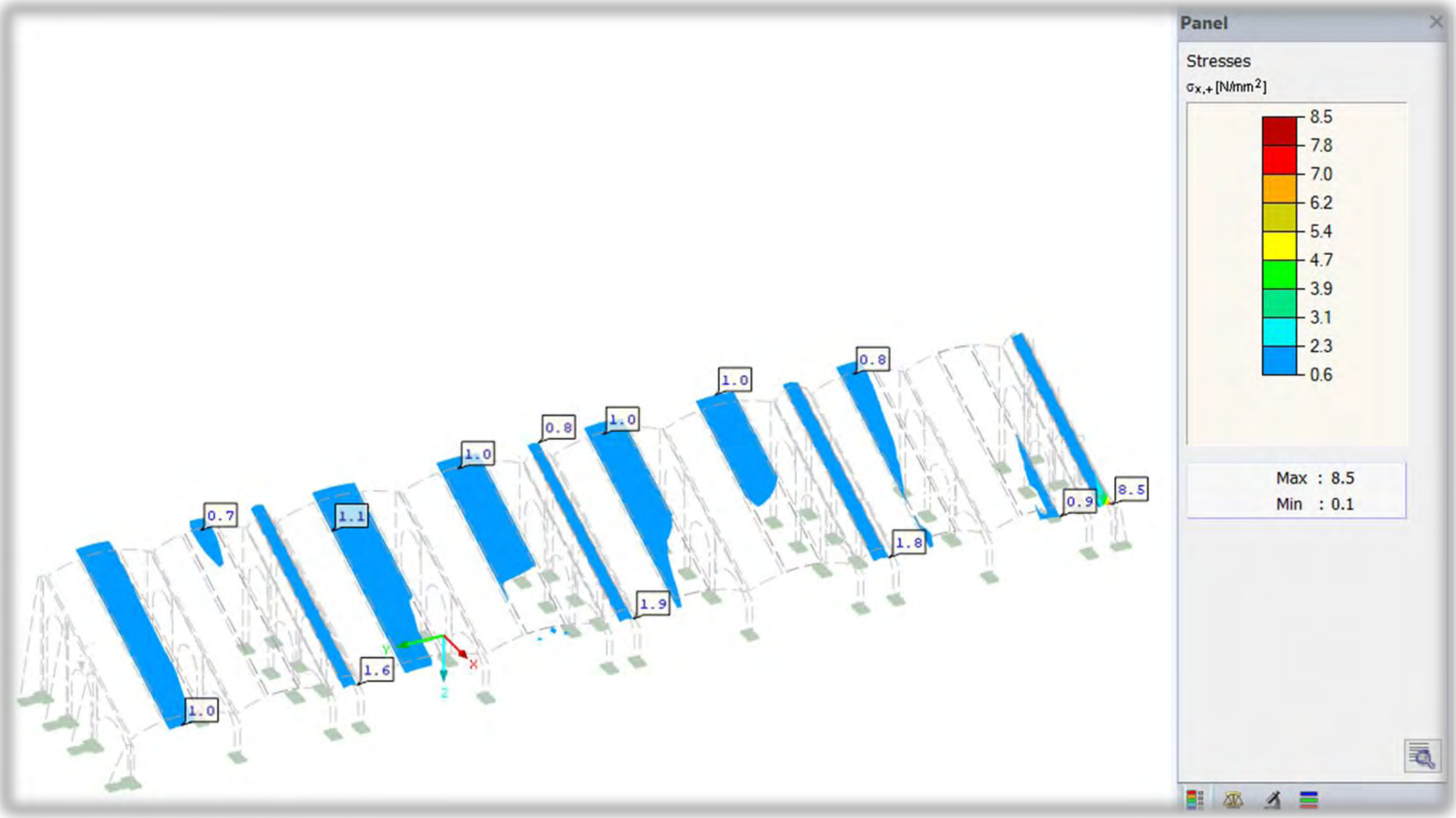

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Figure 8.16: Check on Tensile Stresses in local axis x direction (Thrust line direction) (N-mm^2)

8.3.3 Check on Shear stresses.

The minimum compression force act on the wall is 26.5 KN/m^2 as show in Fig. 8.2

- F_{vk}= 0.6 + 0.4*0.026= 0.61 N/mm^2

the applied shear stress in the direction of bed joints (τ_{yz}) is less than the shear stength (F_{vk}) as shown in fig. 8.17.

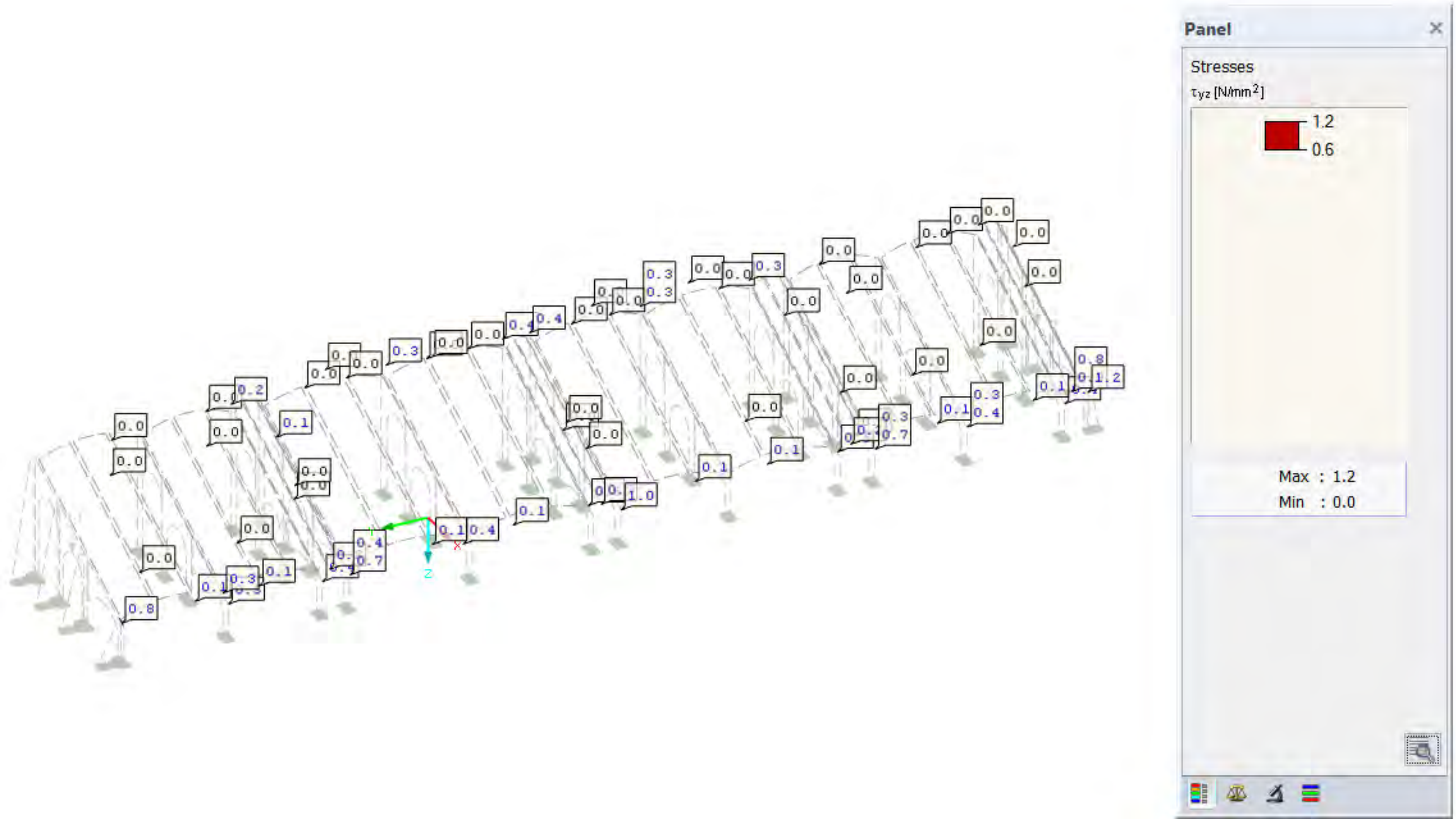

Figure 8.17: Check on Shear Stresses in local axis Y direction (bed joints direction) (N-mm^2)