UNIVERSITAT POLITÈCNICA DE VALÈNCIA

Dept. of Construction Engineering and Civil Engineering Projects

Rehabilitation of the structure of the underground car parking in the Plaza de la Reina in Valencia.

Master's Thesis

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Rehabilitation of the structure of the underground car parking in the Plaza de la Reina in Valencia

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DEPARTAMENTO DE INGENIERÍA DE LA CONSTRUCCIÓN Y PROYECTOS DE INGENIERÍA CIVIL

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Table of Contents

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Summary

The purpose of this study is to analyze the main structure of the car parking of the Plaza de la Reina, to provide the necessary data to assess the state of stability and safety of the structure as it was built.

With all the information of the different elements of the structure (slabs, columns, beams, and corbels), a SAP2000 model of the representative part of the structure has been made, to obtain the results of the loads affecting on the structure elements. Hereinafter, a comparison will be made between the obtained increased applicant forces and the limits resistance reduced values in each representative element of the structure.

Finally, for those elements that don't meet with the requirements and require to be strengthened or repaired, will be identified and a proposal plan will be made to improve ad ensure the structural stability, safety, and functionality for the public use of the car parking and the Plaza de la Reina.

Resumen

El objetivo de este estudio es analizar la estructura principal del aparcamiento de la Plaza de la Reina, para proporcionar los datos necesarios que permitan evaluar el estado de estabilidad y seguridad de la estructura tal y como fue construida.

Con toda la información de los diferentes elementos de la estructura (Losas, pilares, vigas y ménsulas), se ha realizado un modelo SAP2000 de partes representativas de la estructura, para obtener los resultados de las cargas que afectan a los elementos de la estructura. A continuación, se ha llevado a cabo una comparación entre los esfuerzos solicitantes mayorados obtenidos y los valores límites de la resistencia minorada en cada elemento representativo de la estructura.

Finalmente, se han identificado aquellos elementos que no cumplen con los requisitos y requieren ser reforzados o reparados, y se ha realizado un plan de propuestas para mejorar y asegurar la estabilidad estructural, la seguridad y la funcionalidad para el uso público del aparcamiento y la Plaza de la Reina.

1 Description of the structure

The old structure of the parking with its ramps was different than the modified situation. The modified structure of the parking is a result of some demolitions that has occurred to some structure elements, in addition to other modifications happened to the distributions of the ramps. The whole process resulted into improving the functionality and the circulation of the parking.

The old design had circular ramps differently located, (observe Figure 1), the current design had substituted them in a different location and with lineal ones. Additionally, many columns, including an entire row, has been demolished and removed, all those changes contributed into improving the area usage of the parking and the flow of traffic (observe, Figure 2).

Figure 3, Illustrates an overview of the exact location of the parking lot along with the surrounding streets, buildings, and monuments.

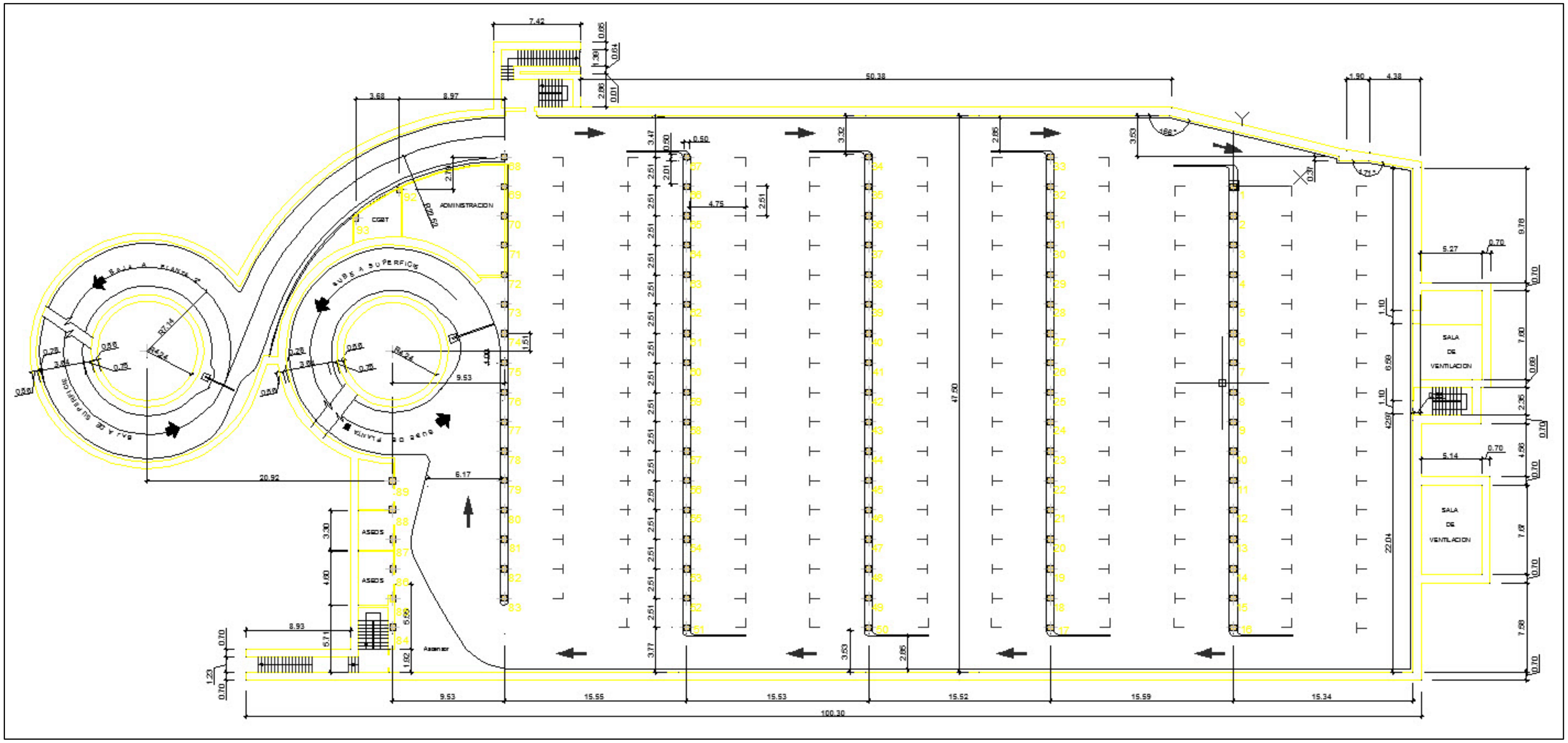


Figure 1. The structural plans of the old underground parking structure of the Plaza de la Reina.

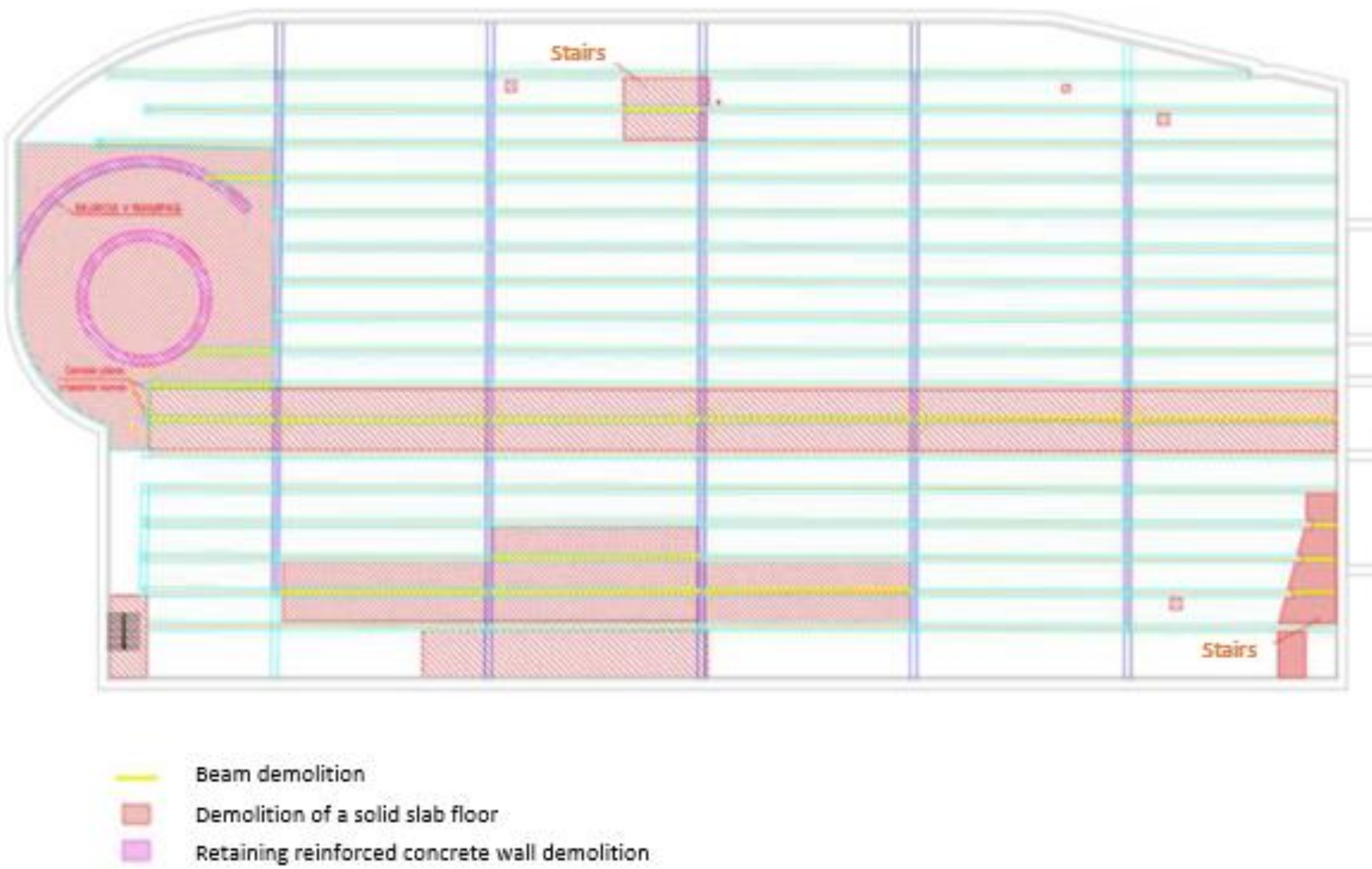


Figure 2. The structural plans of the new underground parking structure of the Plaza de la Reina.

Figure 3. general floor plan/underground parking

Source: (Auraval Ingenieros, DefinitivoPLZRYN_03_PLANOS_U_104_11.pdf, 2019) (SIGMA & Servicios de Ingeniería, 2016)

The car parking is constituted by two basement floors, each one of them, with long-span T-shaped beams, and supported on columns 2.5m apart. On the perimeter there is a 70cm wide mass concrete wall, where beams and slabs rest.

As for each of the floors, in addition to the T-beams, the existence of slabs can be observed, which cover the available space when the separations of pillars are greater than 2.5m. These areas of slabs are differentiated into zones.

In the first-floor **slab**, corresponding to the parking deck (1st floor ceiling), the beams have a total depth of 1.10 m with a slab thickness of 20 cm (observe Figure 4, Figure 5).

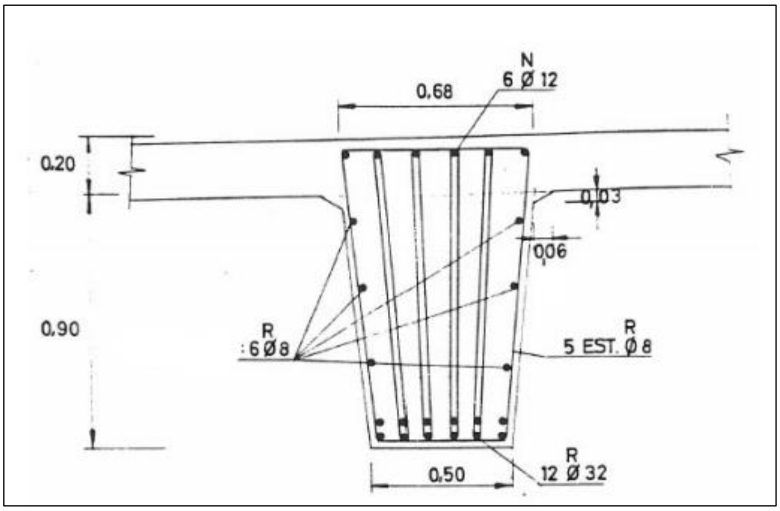


Figure 4. Section T-Beam V1.

Source: (SIGMA & Servicios de Ingeniería, 2016)

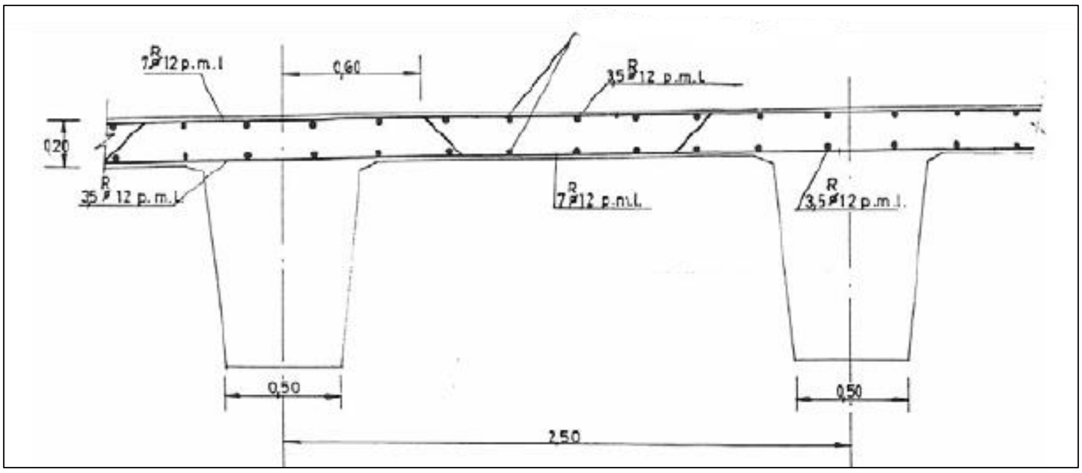


Figure 5.Slab Section F1- first floor ceiling

Source: (SIGMA & Servicios de Ingeniería, 2016)

In the second-floor **slab** (2nd floor ceiling), the beams have a total depth of 0.75 m with a slab thickness of 15 cm (observe Figure 6, Figure 7).

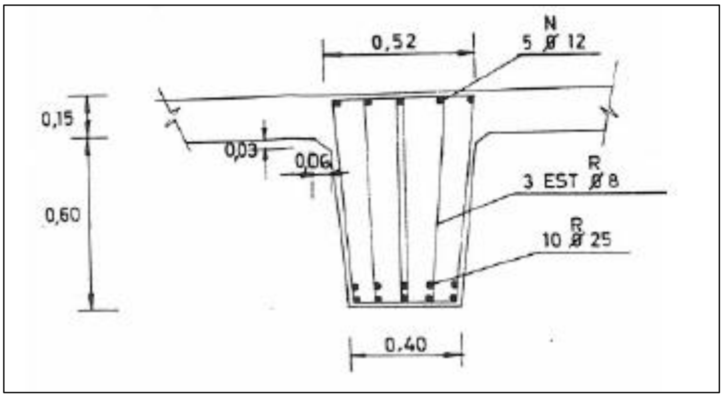


Figure 6. Section T-Beam V25.

Source: (SIGMA & Servicios de Ingeniería, 2016)

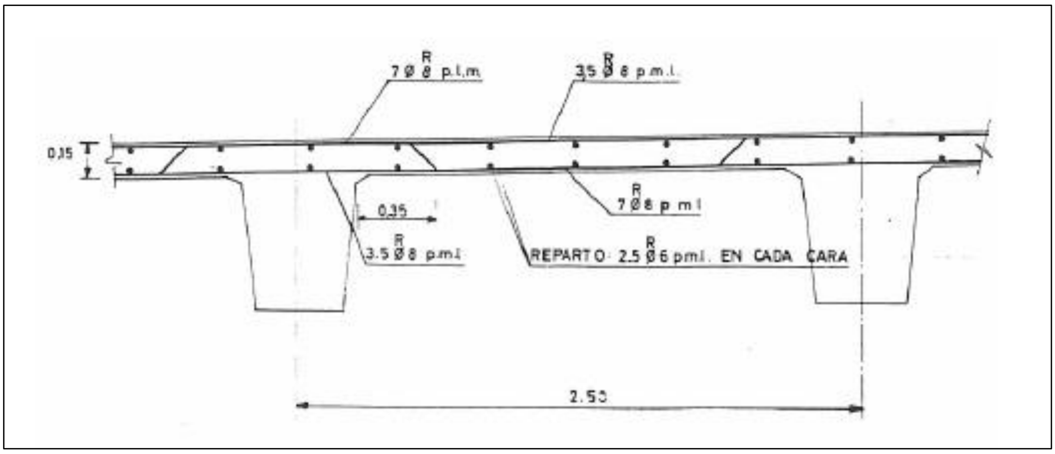


Figure 7. Slab Section F8- second floor ceiling

Source: (SIGMA & Servicios de Ingeniería, 2016)

It should be noted, that only in the last extreme rows (North and south), the beams that are located next to the retaining wall, their section change to V2 section on the first floor and V26 on the second floor. This change is justified due to the greater tributary area. Observe Figure 8, Figure 9.

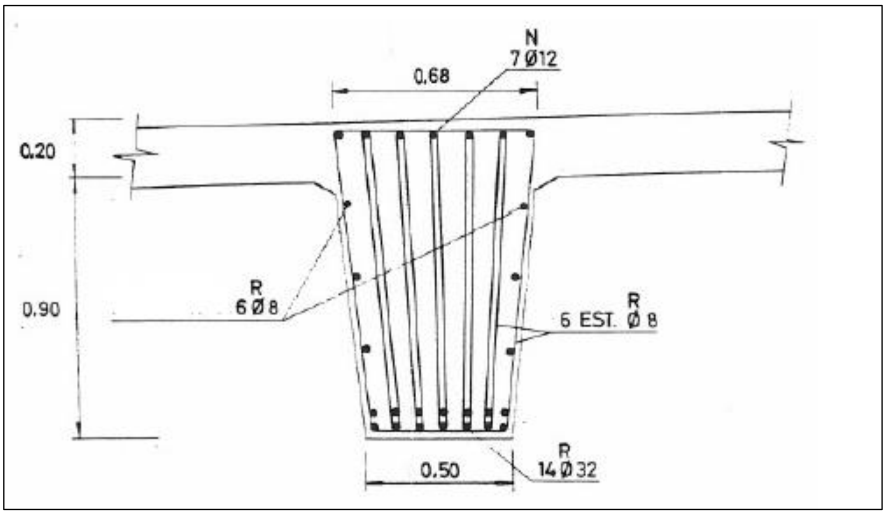


Figure 8. Beam V2 Section-First floor.

Source: (SIGMA & Servicios de Ingeniería, 2016)

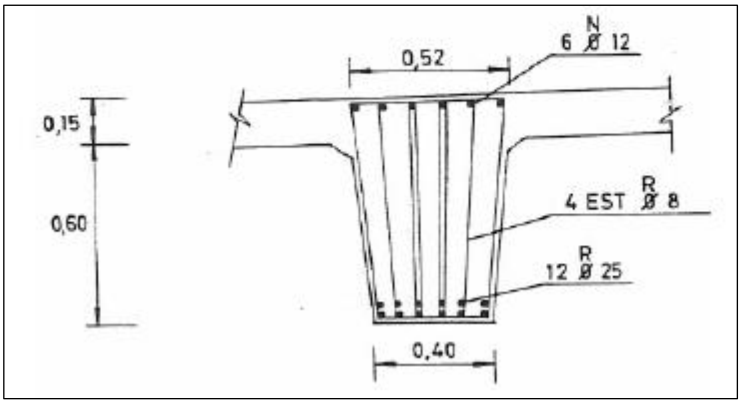


Figure 9. Beam V26 Section-Second floor.

Source: (SIGMA & Servicios de Ingeniería, 2016)

The same applies for the slabs, their section also change due to the greater area occupying. Observe Figure 10, Figure 11.

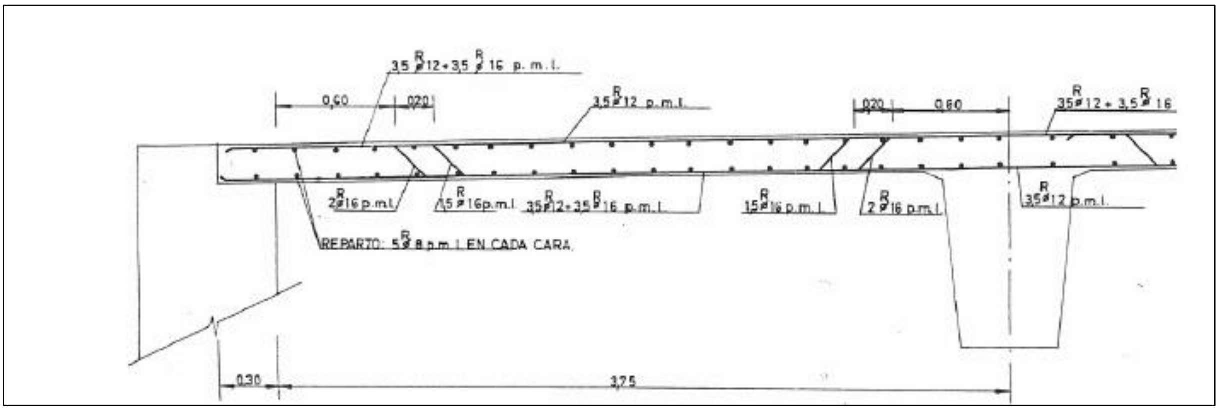


Figure 10. Extreme Slab first floor F2.

Source: (SIGMA & Servicios de Ingeniería, 2016)

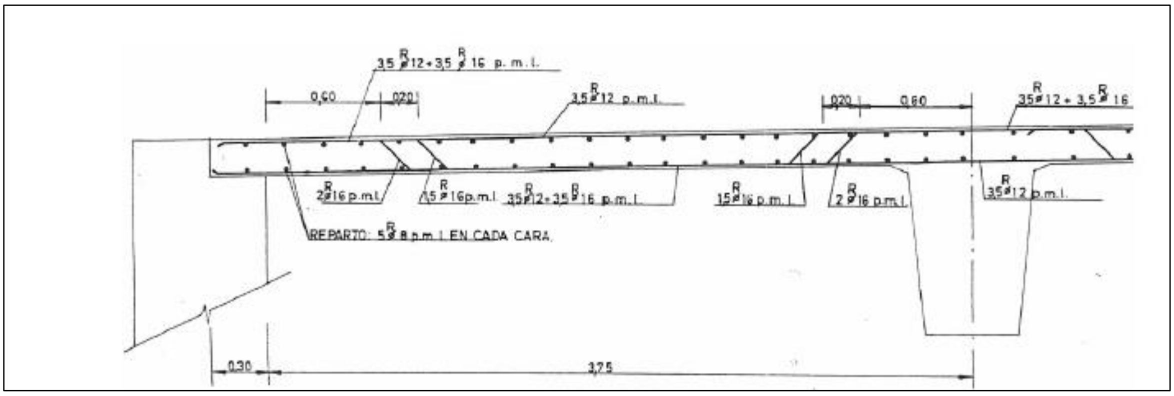


Figure 11. Extreme Slab second floor F9.

Source: (SIGMA & Servicios de Ingeniería, 2016)

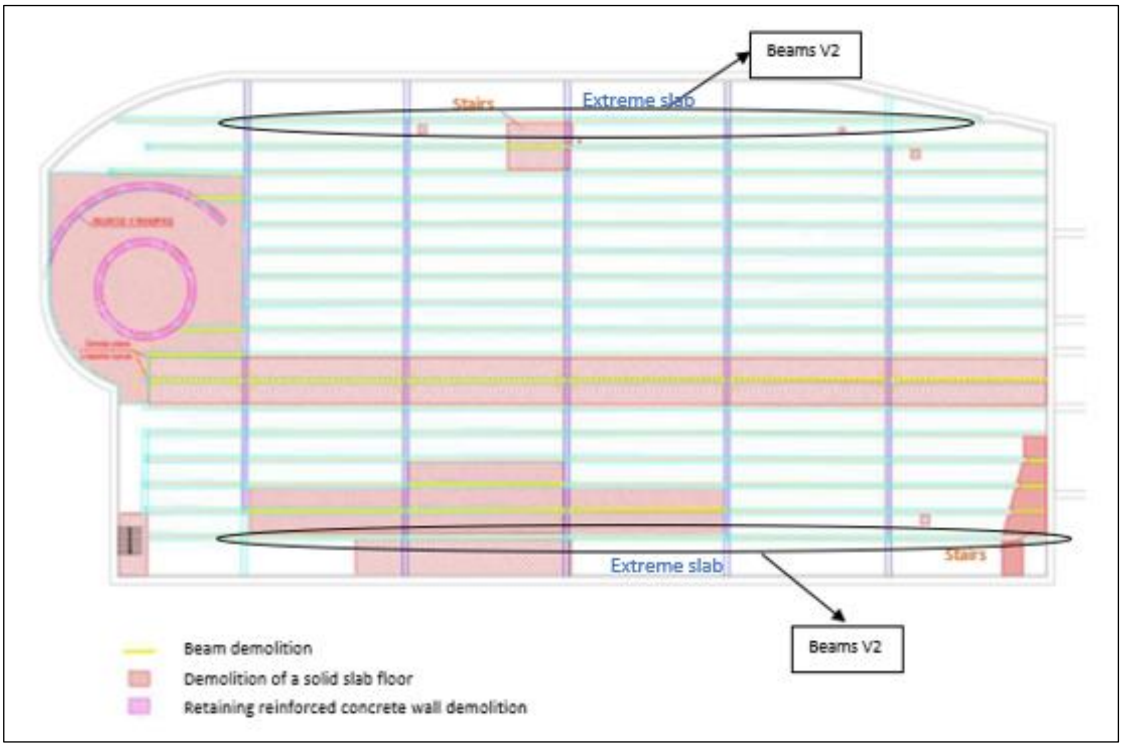


Figure 12. Beam V2 location, Source: (SIGMA & Servicios de Ingeniería, 2016)

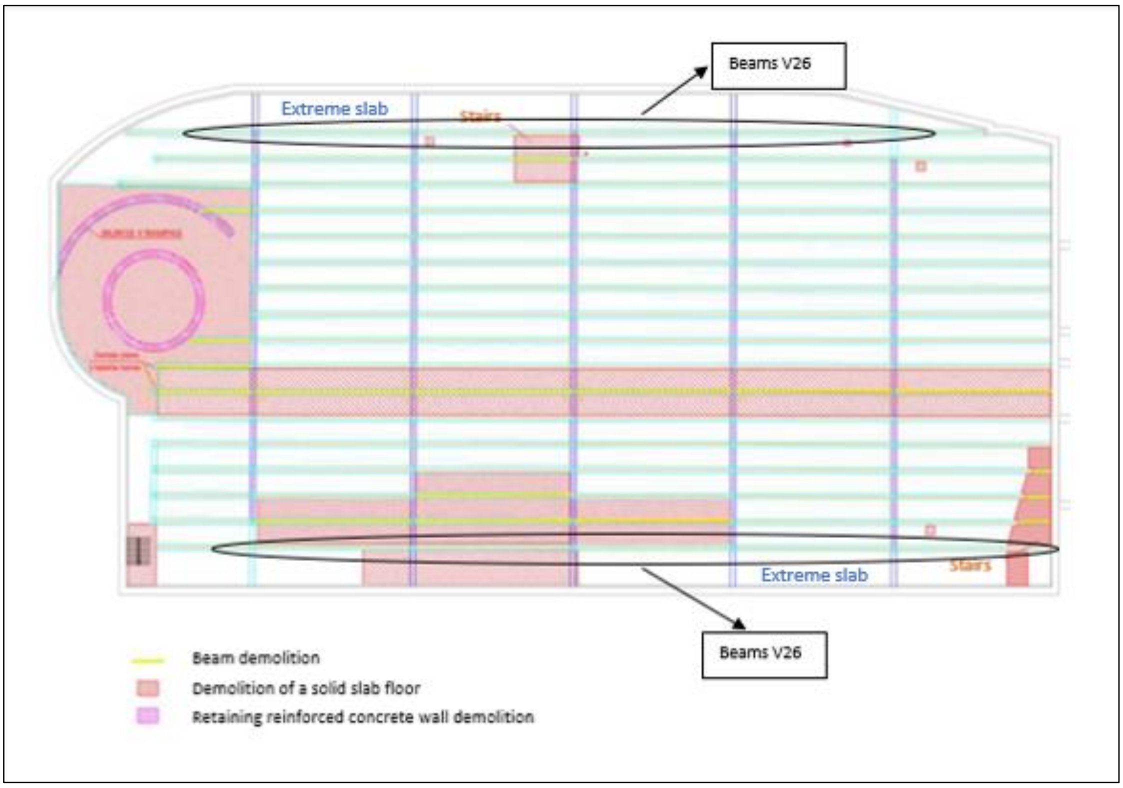


Figure 13. Beam V26 Location, source: (SIGMA & Servicios de Ingeniería, 2016).

As for the **columns**, they are supported on a Pile Cap with two-piles of 18m in length and 52cm in diameter. They have dimensions of 50x50cm2 in the upper basement (Figure 14) and 50x55cm2 in the lower basement Figure 15. These columns also have short corbels for the support of the beams (Figure 16, Figure 17).

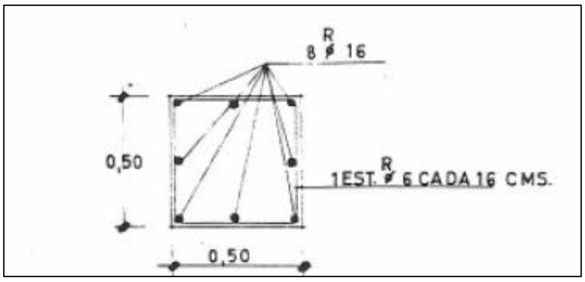


Figure 14. First floor Column Section. Source: (SIGMA & Servicios de Ingeniería, 2016)

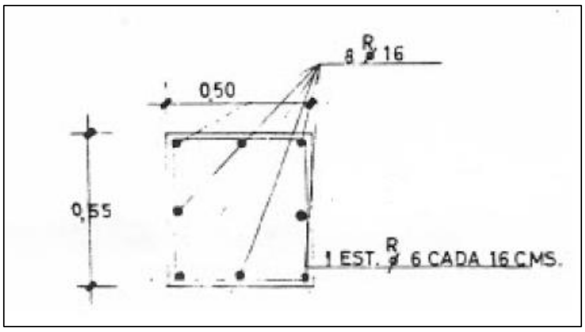


Figure 15. Second floor Column section. Source: (SIGMA & Servicios de Ingeniería, 2016)

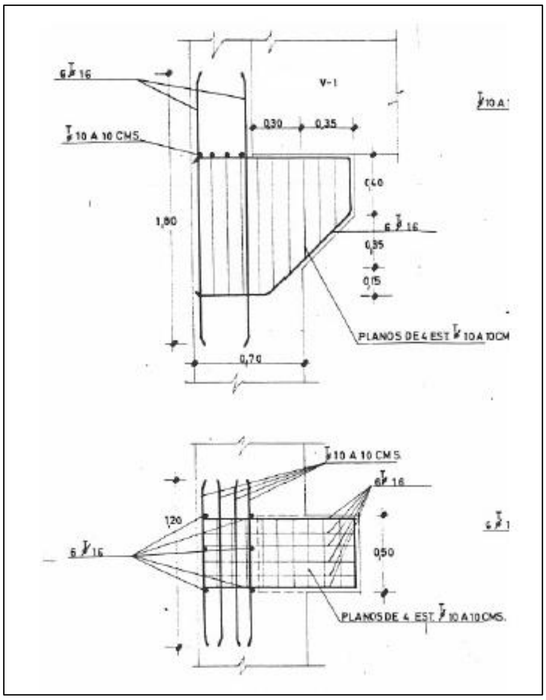


Figure 17. First Basement Corbel where beam V1 rests on. Source: (SIGMA & Servicios de Ingeniería, 2016)

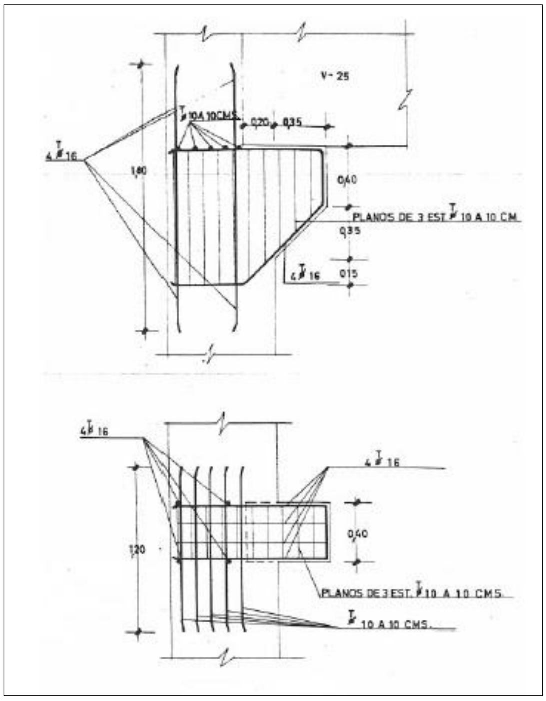


Figure 16. Second Basement Corbel where beam V25 rests on Source: (SIGMA & Servicios de Ingeniería, 2016)

As for the **Pavement**, there are different types of paving depending on the area:

Regarding the pavement from which the parking beneath the Plaza de la Reina is constructed of the following layers (observe Figure 18), from the most superficial counting downwards:

- travertine limestone 10 cm with tiles break.
- Mass concrete with a variable thickness.
- Reinforced concrete with fibers 15 cm.
- The existed slab

Therefore, the dead load of the upper slab has a higher value than the lower slab as a result of the significant loads of the pavement.

The whole construction of the pavement was built with drainage rainfall system in mind, having a slight declination, which directs the flow towards Calle la Paz. Additionally, the direction of the pavement with the main direction of the beams was made with an angel of 45°.

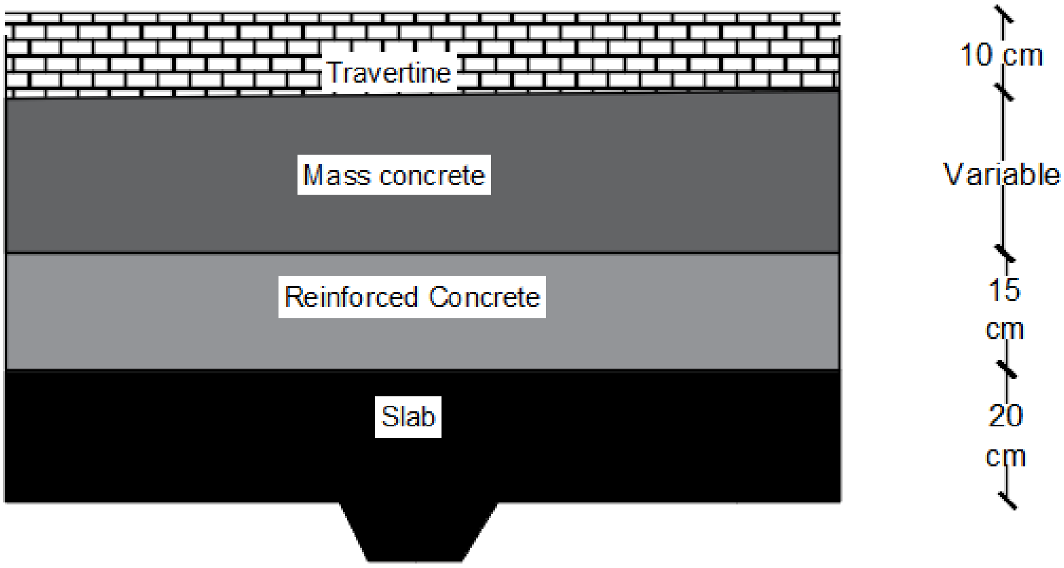


Figure 18. Pavement components.

2 Materials

- Concrete:

New reinforced concrete HA-30/B/20/(IIa-IIIa): used for the new elements due to the changes in the structure of the parking.

Existing concrete in horizontal elements: which was used for the construction of the original structure, its characteristic resistance fck: 17.7 MPa

Existing concrete in vertical elements: its characteristic resistance fck: 15.2 Mpa.

According to the information included in the original design documents:

- Water/cement ratio: 0.50.
- Cement content (kg/m^3): 300.

- Steel:

Passive steel embedded for new elements: Type B 500 S its characteristic strength

fyk: 500 MPa y Es: 200000 MPa

Passive steel for existing elements in the original structure: its characteristic strength

fyk: 450 MPa y Es: 200000 MPa

3 Actions to be considered

- **Permanent Actions:**

Self-Weight:

The self-weight of the different elements is automatically considered by the modelling and structural calculation program through the density of the reinforced concrete:

Self-weight of reinforced concrete **γ=25 kN/m³**

Dead loads:

They are due to the non-structural elements that gravitate on the structure, such as pavements, barriers, etc. It has been considered:

Self-weight of mass concrete: 23 KN/m^3

Self-weight of travertine limestone: 27 KN/m^3

- In lower slab:
  - **0.5 kN/m²**.
- In upper slab:
  - **10.0 kN/m²**.

1. **Variable actions:**

Live loads:

- Lower slab:
  - Category of use: E, traffic, and parking areas for light vehicles.
  - Uniform load of 2kN/m^2 and concentrated load of 20kN. It is considered that this last concentrated load can be replaced by a uniformly distributed one, leaving the sum of both in a value of **4kN/m²**.
- Upper slab:
  - Category of use: C3, areas without obstacles that prevent the free movement of people; or C5, agglomeration zones.

- Uniform load of 5kN/m^2.
- Overload reduction: For these usage categories, the uniform load can be reduced depending on the tributary area of each item. Considering that in general, the elements have tributary areas between 25m^2 and 50m^2, this reduction coefficient is 0.9 (Table 3.2 of the CTE-DB-SE-AE). Thus, the uniform load to be considered is of value **4.5kN/m^2**.

Wind:

wind actions are not considered, given the particularity of being an underground structure.

Snow:

According to Table 3.8 of the CTE-DB-SE-AE, the snow overload to be considered in the city of Valencia is **0.2kN/m^2**.

Thermal actions

The average annual reference temperature in the city of Valencia is 18ºC. The structure, from this reference temperature, will be subject to deformations and geometric changes due to variations in the temperature of the external environment.

According to the CTE-DB-AE (point 3.4), the global effects of thermal action can be obtained from the average temperature variation of the structural elements. The summer effects, expansion, and the winter effects, contraction, can be considered separately from the reference temperature when the element was built, being possible to take the annual average temperature of the site previously mentioned.

From this point on, the regulations establish two clearly differentiated situations:

- Elements exposed to the weather (without being exposed to solar radiation):

o Extreme minimum temperature: -10ºC.

o Extreme maximum temperature: 44ºC.

- Protected elements:

o A temperature of 20ºC can be taken, all year round.

- Elements of the envelope that are not directly exposed to the weather (Average value to the above):

o Minimum temperature: 5ºC.

o Maximum temperature: 32ºC.

Therefore, the different elements of the structure under study will be subjected to thermal variations according to the following criteria:

- Elements of the upper slab structure (beams and slabs):

o ΔT(-) = 5-18 = -13ºC.

o ΔT(+) = 32-18 = +14ºC.

- Rest of elements: they are considered, on the safety side for this action, equally to the previous ones because the structure is not protected (there are no false ceilings or other elements that isolate the structure from the ambient temperature) and its exposure to air temperature is high due to the large access areas to the parking lot that could always be open.

2. **Accidental actions:**

Earthquake:

No seismic actions are considered due to the particularity of being an underground structure. The earthquake actions in Valencia are very small. In the case of the structure under study, since it is a confined underground structure.

Firefighter's truck:

The action of firefighters is considered an accidental action because their presence on the Plaza de la Reina, in numerous vehicles, is mainly due to accidental actions on the structure under study or on adjacent structures. In addition, the CTE textually states that this action is included within the accidental actions, with the following particularities:

"In the transit areas of vehicles destined for fire protection services, an action of 20 kN/m2 arranged in a surface of 3 m wide by 8 m long, in any of the positions of a 5 m wide band, and the maneuvering areas, where the passage of this type of vehicles is foreseen and signposted, shall be considered."

The entire Plaza de la Reina is a space where this type of vehicle can circulate freely. Consequently, this accidental action extends over the entire upper surface. In addition, complying with the specified widths and bands, the overload of 20kN/m2, applied to certain surfaces, can be simplified, and reduced to a reduced overload applied uniformly distributed over the entire surface, since this situation is the most unfavorable for the various tests to be carried out on bending, shear, etc. on the different elements of the structure. The calculation of the reduced overload applied to the entire surface is as follows:

- It is considered that in the 15.5m length of an upper deck span, up to four 3m wide standard vehicles could gravitate.
- Beams and horizontal elements have a tributary width that is generally 2.5m, considerably less than the 8m length specified in the standard. Therefore, the entire width of each alignment would be affected by the full load and could not be distributed to adjoining alignments because they would also be fully loaded.
- Consequently, the load of 20kN/m2 would be applied to an area of 4x3x2.5 = 30m2. The total area of each alignment is 38.35m2.
- A load uniformly distributed over the entire area, equivalent to that specified, would be: 20x30/38.35 = 15.65kN/m2.

4 Representative calculation values and Load combinations

4.1 Representative Values

The representative values of the loads used for the verification of the limit states shall be:

- Permanent (G)

For permanent actions a single representative value will be considered, coinciding with the characteristic value Gk.

- Variable (Q)

Each of the variable actions may be considered with the following representative values:

Characteristic value Qk

This will be the value of the action when acting in isolation, as defined above.

Combination value ψ_0 Qk:

Shall be the value of the action when acting with some other variable action, to account for the small probability of the most unfavorable values of several independent actions acting simultaneously.

Frequent value ψ_1 Qk:

Shall be the value of the action that is exceeded during a period of short duration with respect to the useful life.

Almost permanent value ψ_2 Qk:

Shall be the value of the load that is exceeded during a large part of the useful life or the average value.

The values of the coefficients ψ are given in Table 4.2 of the CTE-DB-SE:

	ψ_0	ψ_1	ψ_2
Sobrecarga superficial de uso (Categorías según DB-SE-AE)			
• Zonas residenciales (Categoría A)	0,7	0,5	0,3
• Zonas administrativas(Categoría B)	0,7	0,5	0,3
• Zonas destinadas al público (Categoría C)	0,7	0,7	0,6
• Zonas comerciales (Categoría D)	0,7	0,7	0,6
• Zonas de tráfico y de aparcamiento de vehículos ligeros con un peso total inferior a 30 kN (Categoría E)	0,7	0,7	0,6
• Cubiertas transitables (Categoría F)		(1)	
• Cubiertas accesibles únicamente para mantenimiento (Categoría G)	0	0	0
Nieve			
• para altitudes > 1000 m	0,7	0,5	0,2
• para altitudes ≤ 1000 m	0,5	0,2	0
Viento	0,6	0,5	0
Temperatura	0,6	0,5	0
Acciones variables del terreno	0,7	0,7	0,7

(1) En las cubiertas transitables, se adoptarán los valores correspondientes al uso desde el que se accede.

Table 1. Simultaneity coefficient.

- Accidental (A)

For accidental actions, a single representative value coinciding with the characteristic value Ak will be considered.

4.2 Design Values

The design values of the different actions will be obtained by applying the corresponding partial safety factor to the representative values of the actions defined above.

The partial safety factors γ are shown in the following table for ultimate limit states (ELUs in CTE-DB-SE):

Tipo de verificación (1)	Tipo de acción	Situación persistente o transitoria	
		desfavorable	favorable
Resistencia	Permanente		
	Peso propio, peso del terreno	1,35	0,80
	Empuje del terreno	1,35	0,70
	Presión del agua	1,20	0,90
	Variable	1,50	0

Table 2. Partial safety coefficients for Ultimate Limit States.

The partial safety coefficients for serviceability limit states (ELS) are considered equal to unity according to point 4.3 of the CTE-DB-SE.

4.3 Load Combination

According to the CTE-DB-SE, the load hypotheses to be considered will be formed by combining the design values of the actions whose action may be simultaneous, according to the general criteria indicated below:

A) ULTIMATE LIMIT STATES

For the verification of the Ultimate Limit States, persistent, transitory, and accidental situations shall be considered.

A1) Persistent or transitory situations

The combinations of the different actions considered in these situations are carried out according to the following criterion:

$$\sum_{j\geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \cdot \Psi_{0,i} \cdot Q_{k,i}$$

where:

Gk,j = Representative value of each permanent stock.

Qk,1 = Representative value (characteristic value) of the dominant variable load.

ψ0,i Qk,i = Representative values (combination values) of the variable actions concomitant with the dominant variable action.

In general, as many hypotheses or combinations as necessary should be made, considering, in each of them, one of the variable actions as dominant and the rest as concomitant.

A2) Accidental situations - firefighters (other than earthquake)

The combinations of the different actions considered in these situations will be made according to the following criterion:

$$\sum_{j\geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_p \cdot P + A_d + \gamma_{Q,1} \cdot \Psi_{1,1} \cdot Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \cdot \Psi_{2,i} \cdot Q_{k,i}$$

where:

Gk,j = Representative value of each permanent action.

ψ1,1 Qk,1= Frequent representative value of the dominant variable action.

ψ2,i Qk,i = Near-permanent representative values of the variable actions concomitant with the dominant variable action and the accidental action.

Ad = Characteristic representative value of the accidental action.

For these combinations, the observations indicated in the approach for combinations A1) shall apply.

B) SERVICEABILITY LIMIT STATES

For the verifications related to the Serviceability Limit States, only the persistent and transitory situations for deflections will be considered, excluding the accidental ones and the horizontal displacements because it is a confined structure.

The combinations of the different actions considered in these situations will be carried out according to the following criteria:

B1) Characteristic Combination (unlikely or rare).

$$\sum_{j\geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i>1} \psi_{0,i} \cdot Q_{k,i}$$

The observations indicated in the approach to combinations A1) apply.

The load combinations for the parking:

1) Service: 1,00 (Dead Load + Self weight) + 1,00 Live Load
2) UDSTL1: 1,35 (Dead Load + Self weight) + 1,5 Live Load
3) UDSTL2.1: 1,35 (Dead Load + Self weight) + 1,5 Live Load + 1,5 Summer
4) UDSTL2.2: 1,35 (Dead Load + Self weight) + 1,5 Live Load + 1,5 Snow + 1,5 Winter
5) UDSTL2.3: 0,8 (Dead Load + Self weight) + 1,5 Winter
6) Accidental 1 TRUCK: 1,35 (Dead Load + Self weight) + 1,05 Live Load + 0,9 Summer + 1,0 Firefighter’s truck 1 Longitudinal
7) Accidental 3 Trucks: 1,35 (Dead Load + Self weight) + 1,05 Live Load + 0,9 Summer + 1,0 Firefighter’s trucks 3 Transversal

5 SAP2000 Model

For the introduction of the structure elements, materials, loads and load combinations, it has been done using SAP2000 program. In the following paragraph a specification for each step of the model will be explained.

The following figures (Figure 19, Figure 20) illustrate the final definition of the model.

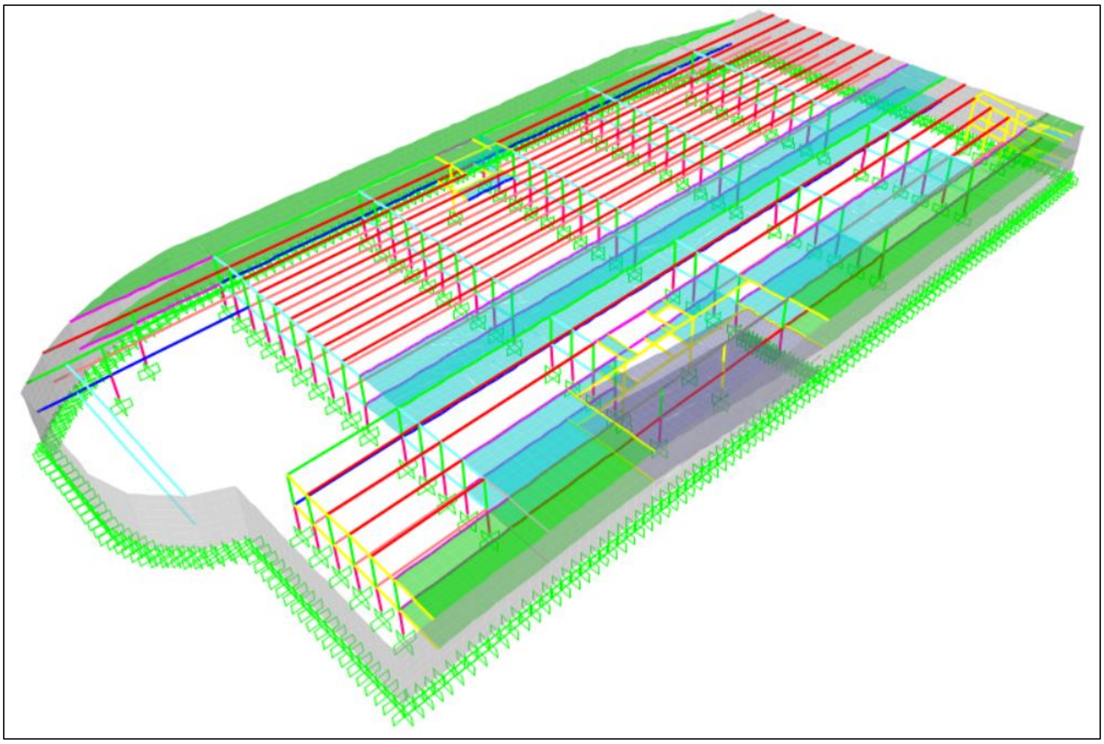


Figure 19. 3d view for the final model of the parking lot (1).

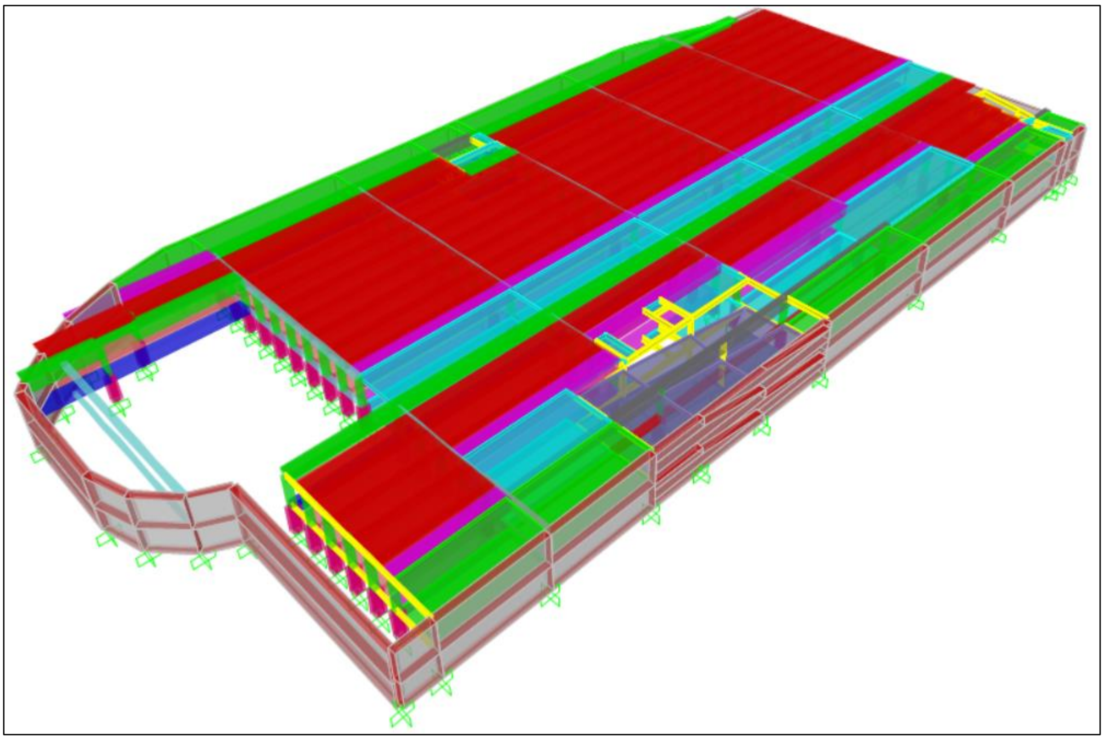


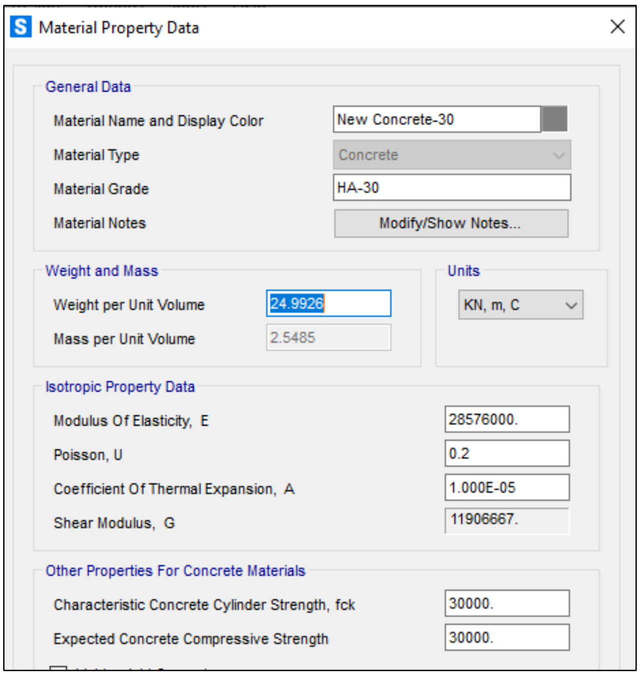
Figure 20. 3d view for the final model of the parking lot (2).

5.1 Materials

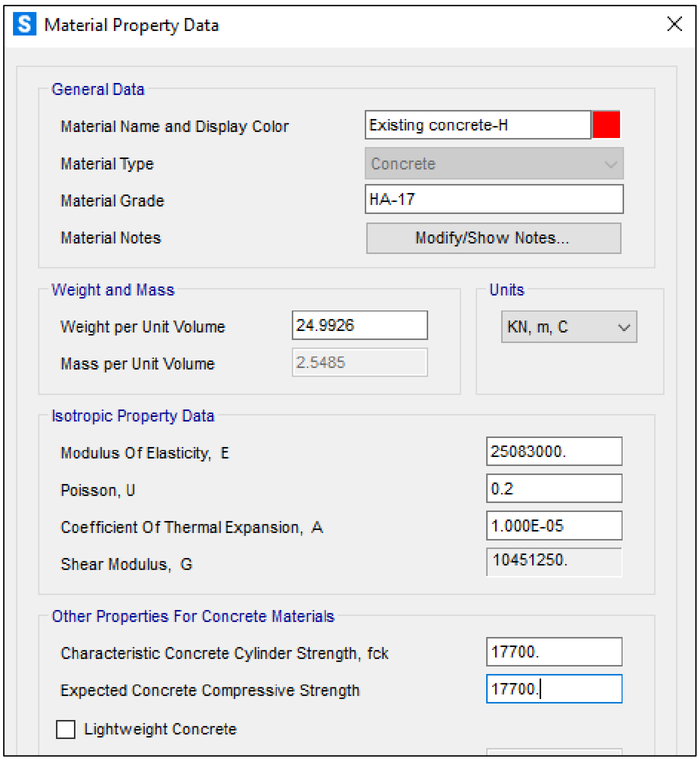
This section summarizes the characteristics of the constituent materials that have been used in the design of the elements of the structure to be evaluated. Both existing materials and new materials to be used are considered.

5.1.1 Concrete

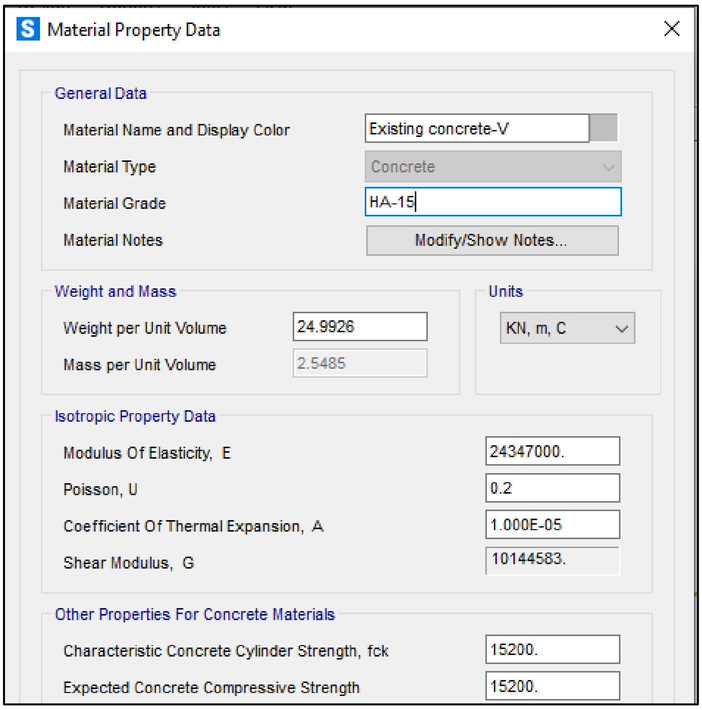
- New reinforced concrete HA-30, in new elements such as: New Ramps, new Beams, and new Columns.



- Existing concrete-H fck: 17.7 MPa, in horizontal elements such as: Existing Beams and Slabs.



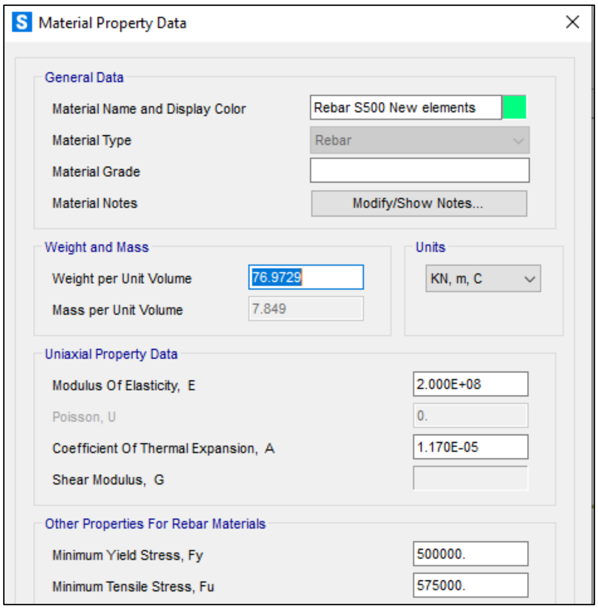
- Existing concrete-V fck: 15.2 MPa, in vertical elements such as: Columns and retaining walls.



5.1.2 Steel

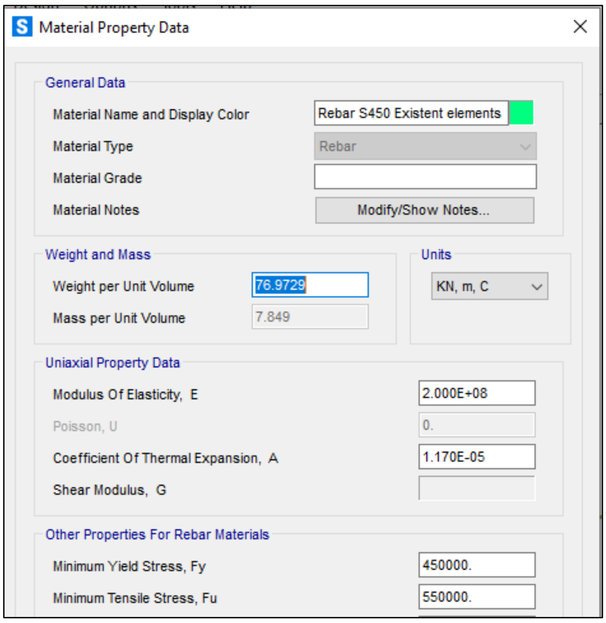
- Passive steel embedded for new elements: Type B 500 S its characteristic strength

fyk: 500 MPa y Es: 200000 MPa



- Passive steel for existing elements in the original structure: its characteristic strength

fyk: 450 MPa y Es: 200000 MPa



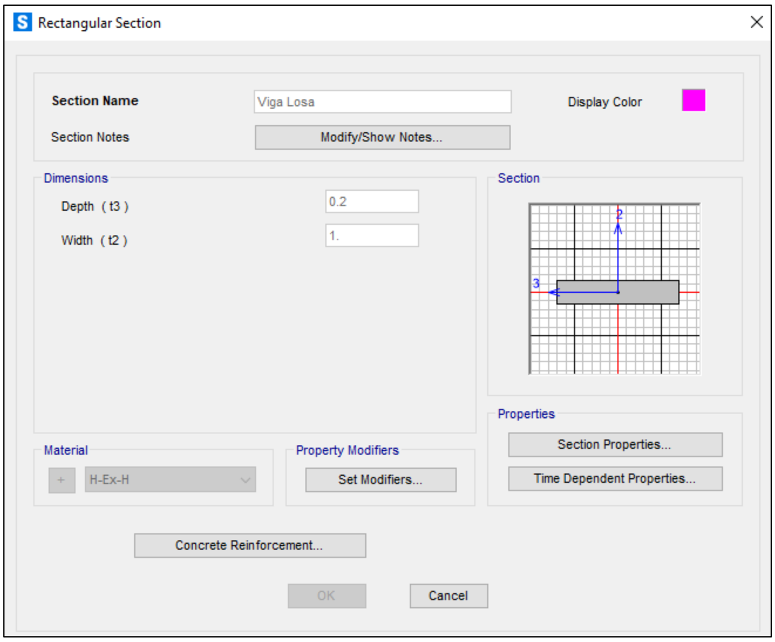
5.2 Estructural elements

5.2.1 Slabs

The slab was modeled using SAP2000 considering it as a multi-span beam. Each span measures 2,5 m.

The beam width of 1 meter and its depth is the same depth as the slab which is 0,2 meters (1 x 0,2) m.

All the slabs material are from existing concrete-H with Fck = 17,7 MPa and rebars S450 Existing.



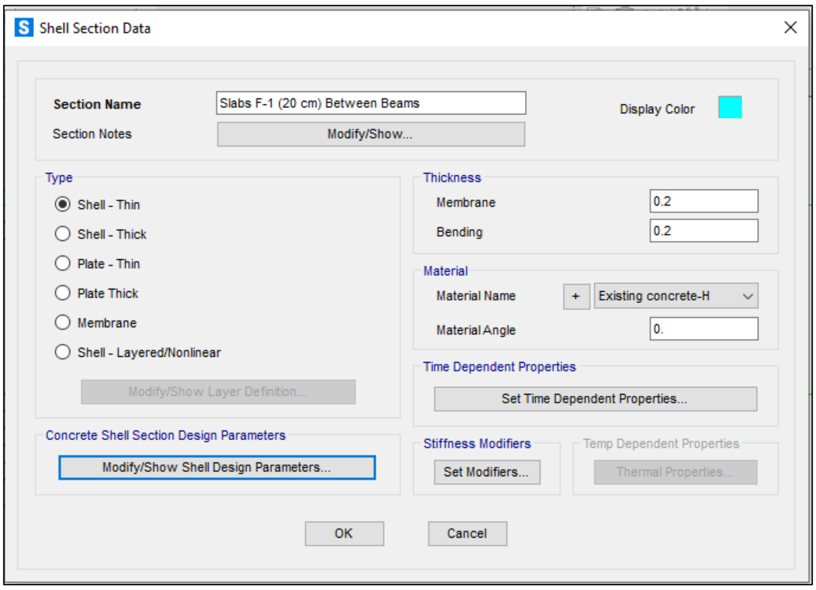


Figure 21. Slab F1 First floor ceiling.

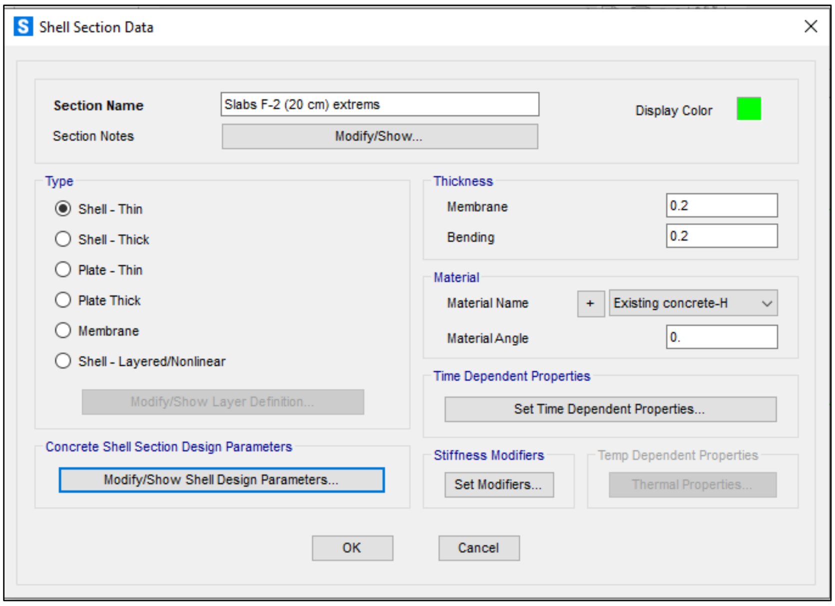


Figure 22. Extreme Slab F2 First floor ceiling.

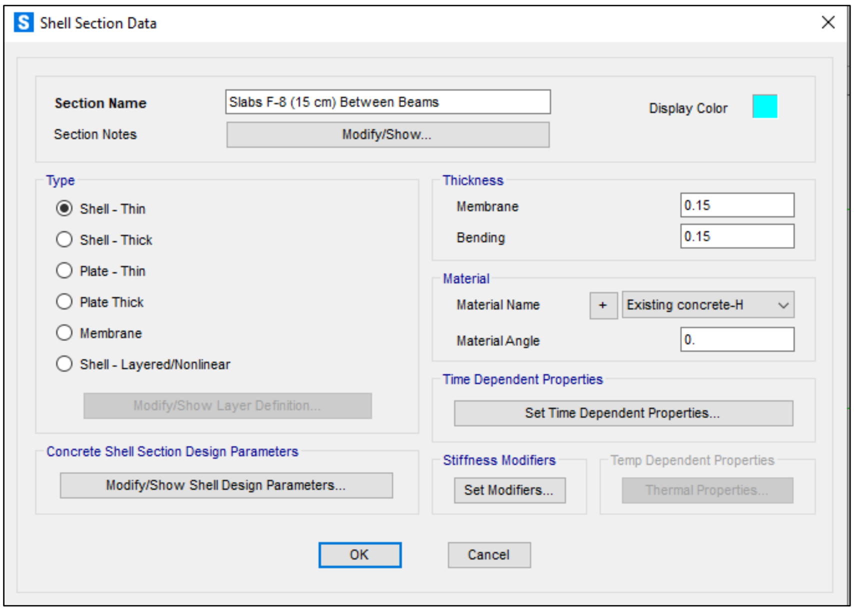


Figure 23. Slab F8 Second floor ceiling.

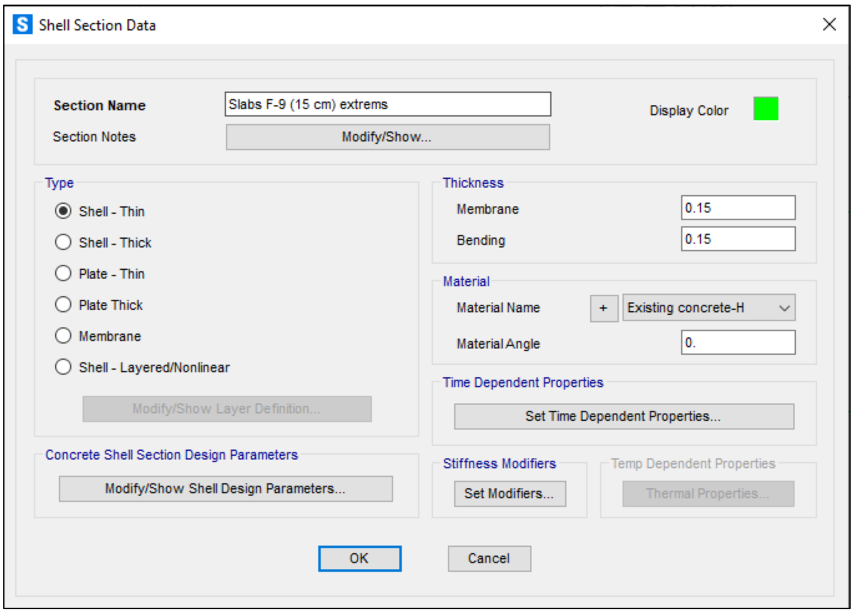


Figure 24. Extreme Slab F9 Second floor ceiling.

The ramps are also defined as shell elements; however, they have different materials:

New Concrete-30 with Rebars 500S for new elements.

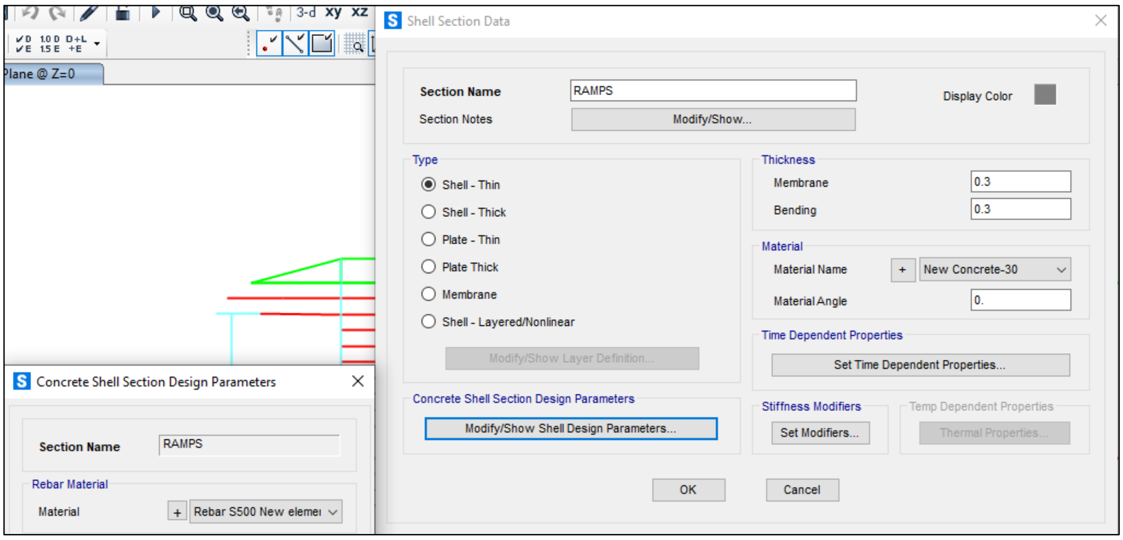


Figure 25. New ramps

The retaining wall is defined as shell elements; however, the materials that is consisted of are: existing Concrete-V, Fck = 15,2 MPa.

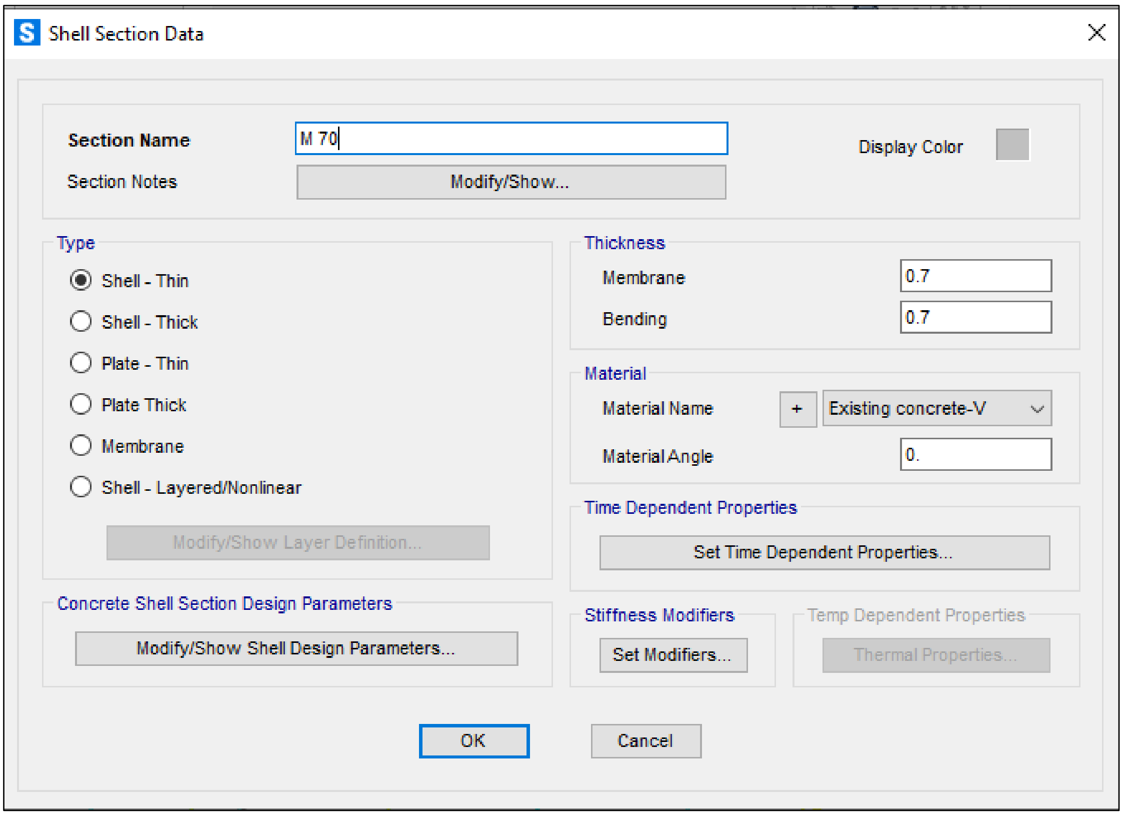


Figure 26. Retaining wall

5.2.2 Beams

Beams are divided into 4 different sections, in each floor there are two sections: V1 - V2 Beams in first floor, and V25 - V26 in second floor. All beams are from existing concrete-H and rebar S450 Existing.

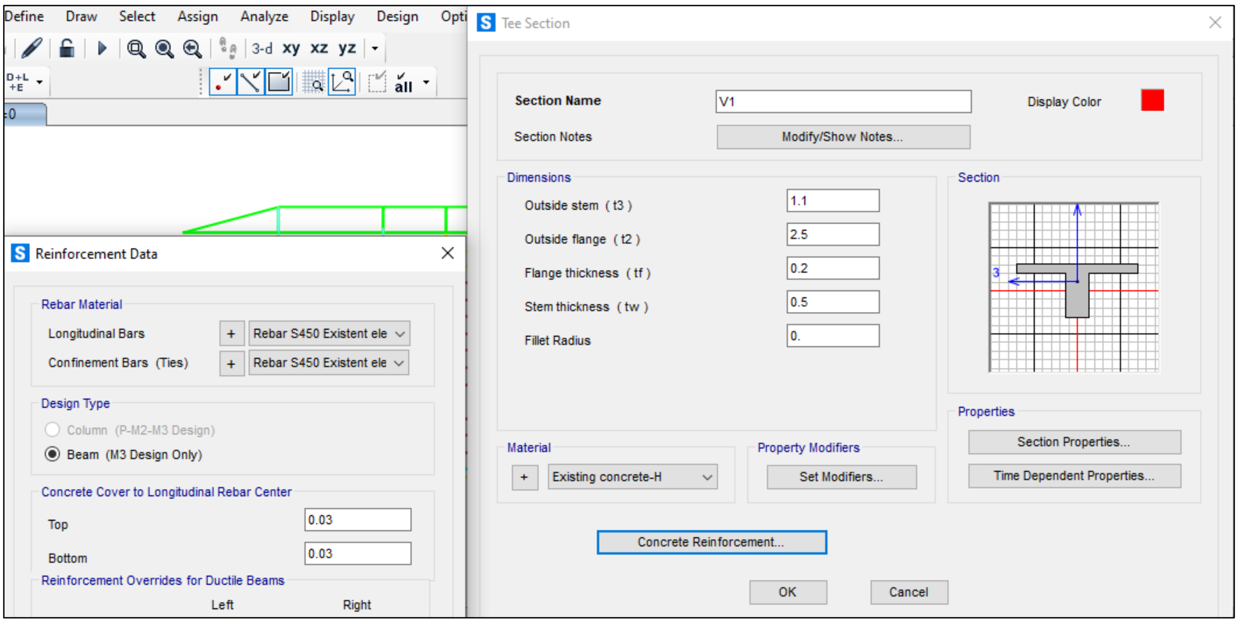


Figure 27. Beam V1 first floor.

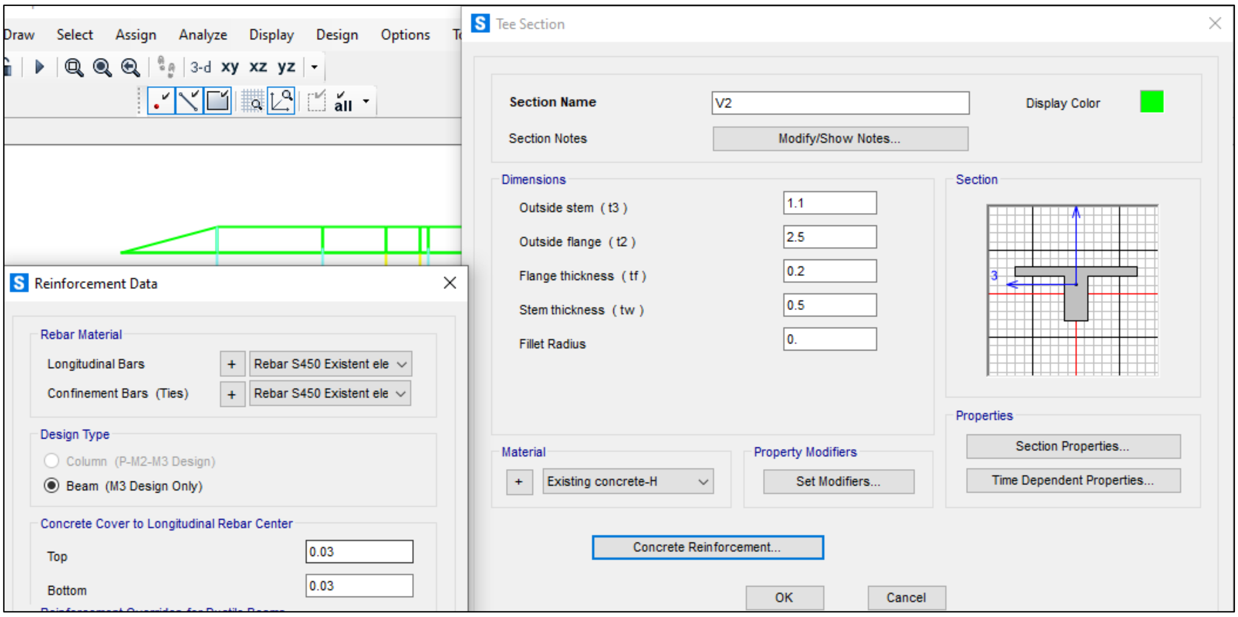


Figure 28. Extreme beam V2 first floor.

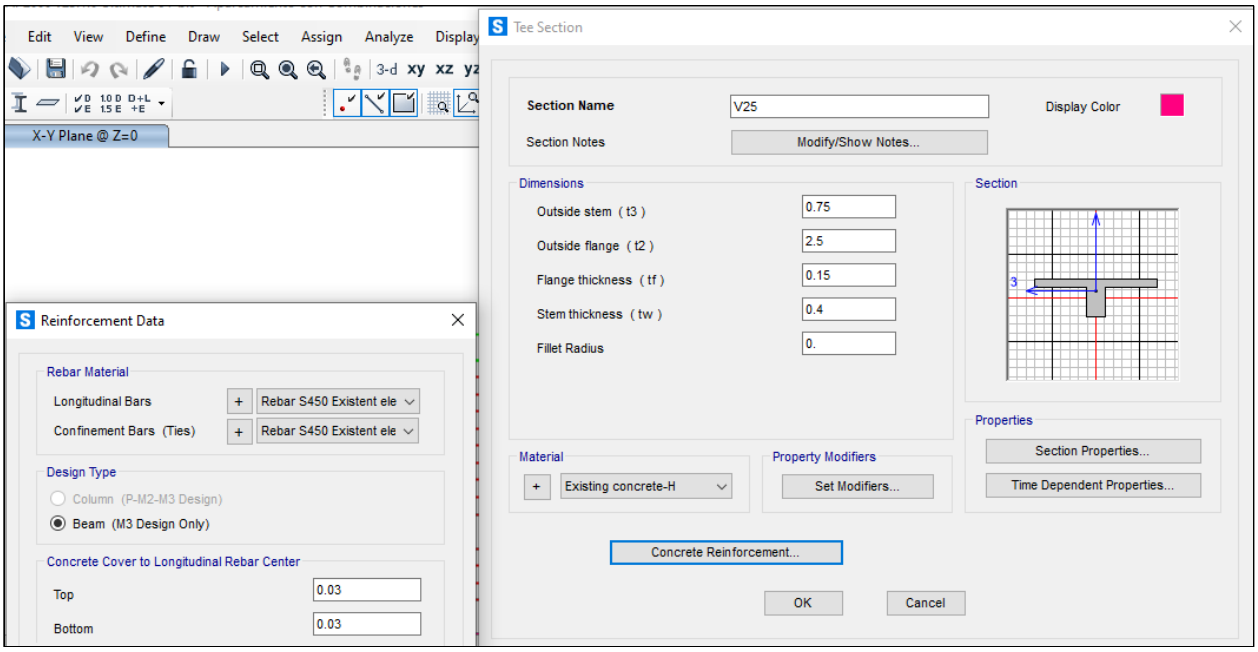


Figure 29. Beam V25 second floor.

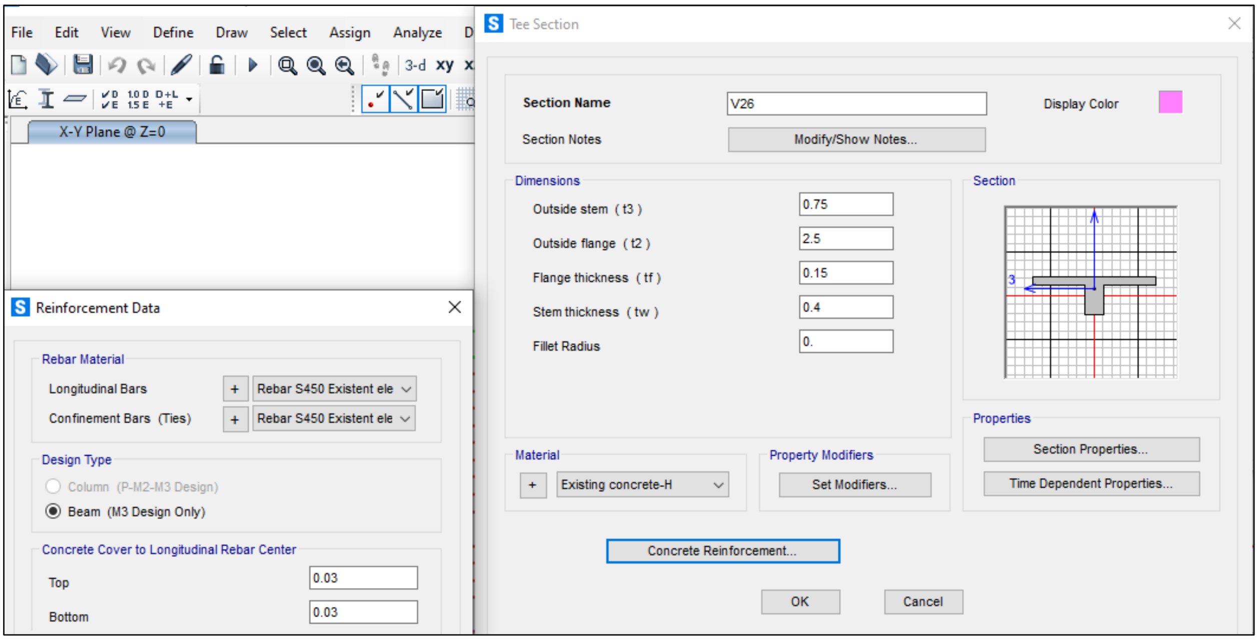


Figure 30. Extreme beam V26 second floor.

5.2.3 Columns

First floor basement: consists of square section Columns (50x50) cm with 8 bars of 16mm diameter.

Second floor basement: rectangular section 50x55 cm in with the same reinforcement.

All columns are from Existing columns-V Fck = 15,2 MPa, and rebars S450 existing.

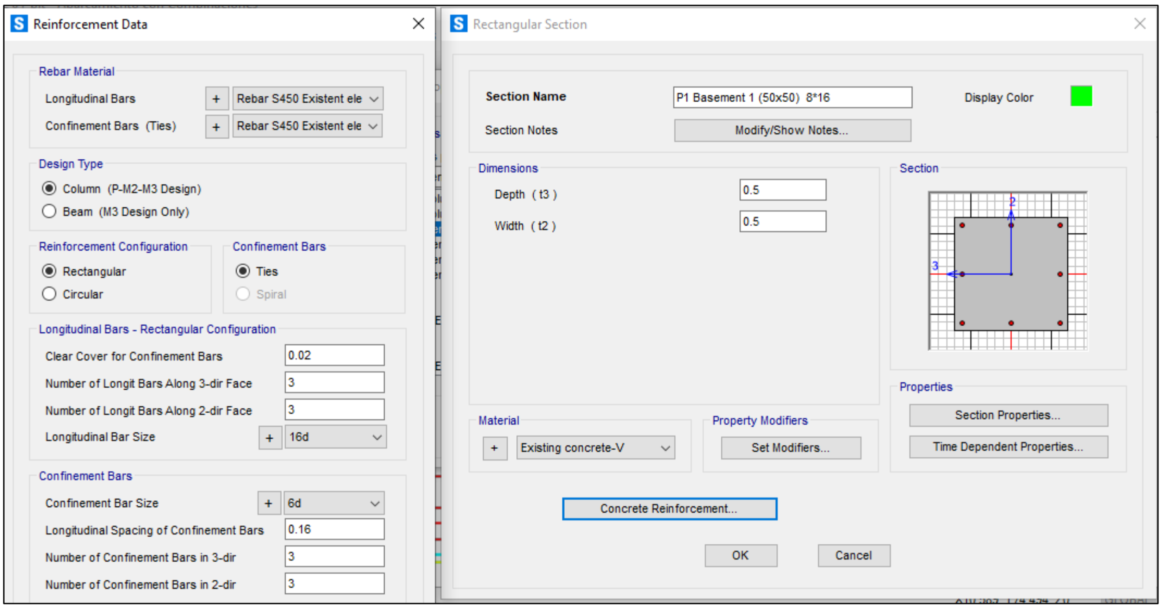


Figure 31. Columns section first floor.

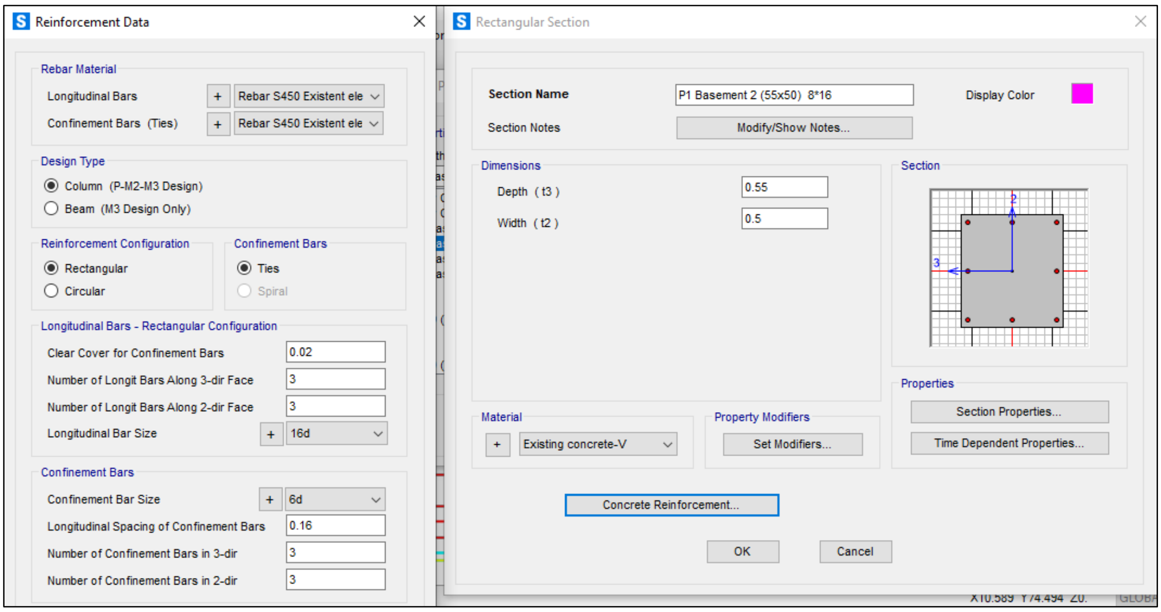
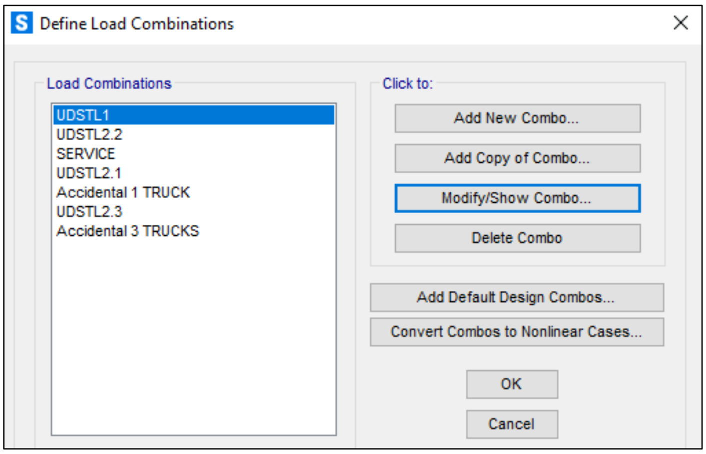


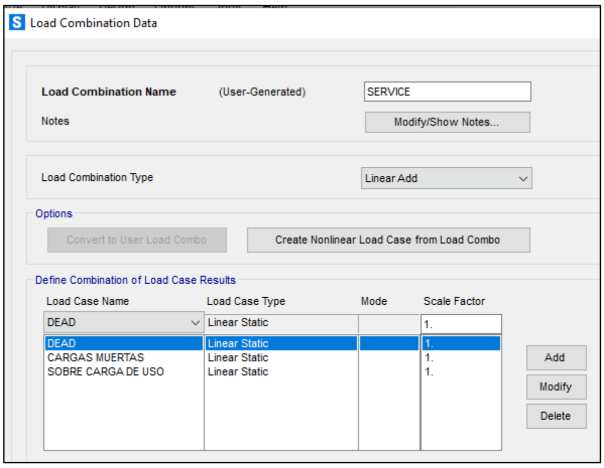
Figure 32. Column section second floor.

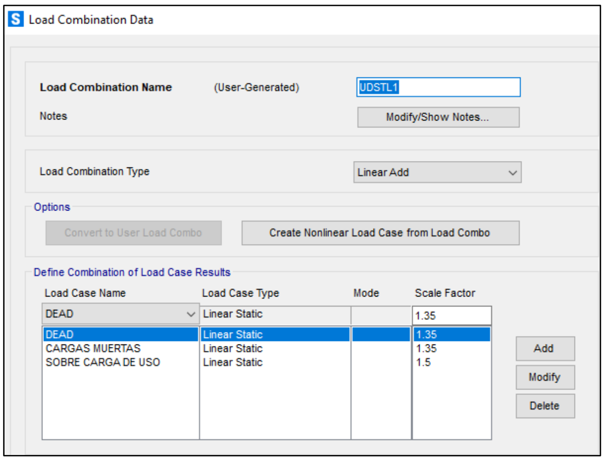
5.3 Load combinations

Load combinations were defined according to paragraph 4.3 Load Combination.



Hereafter, each load combination will be shown in detail:


Load Combination Data
Load Combination Name
(User-Generated)
SERVICE
Notes
Modify/Show Notes...
Load Combination Type
Linear Add
Options
Convert to User Load Combo
Create Nonlinear Load Case from Load Combo
Define Combination of Load Case Results
Load Case Name
Load Case Type
Mode
Scale Factor
DEAD
Linear Static
1.
DEAD
Linear Static
1.
CARGAS MUERTAS
Linear Static
1.
SOBRE CARGA DE USO
Linear Static
1.
Add
Modify
Delete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(User-Generated)
UDSTL1
Notes
Modify/Show Notes...
Load Combination Type
Linear Add
Options
Convert to User Load Combo
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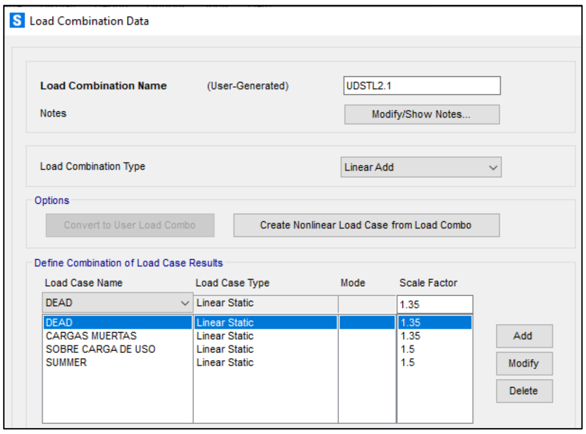
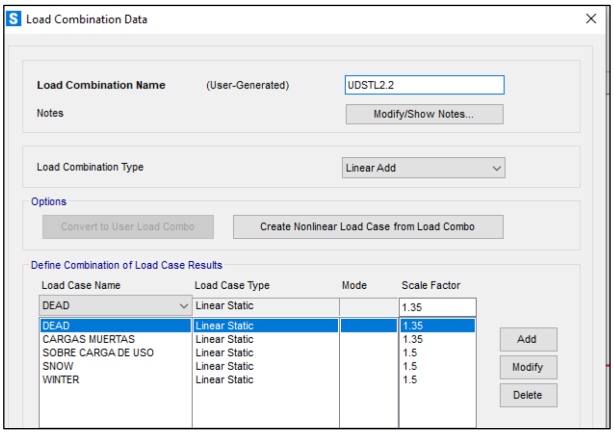
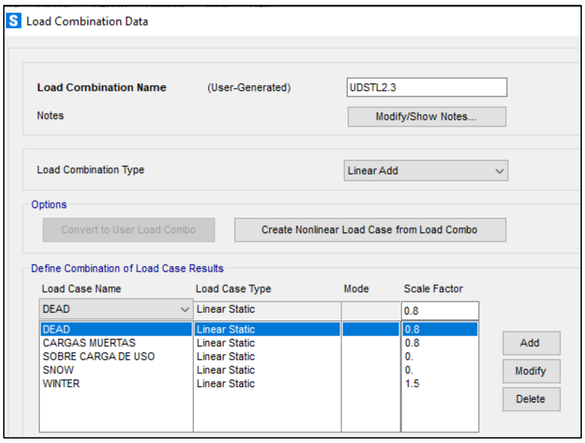
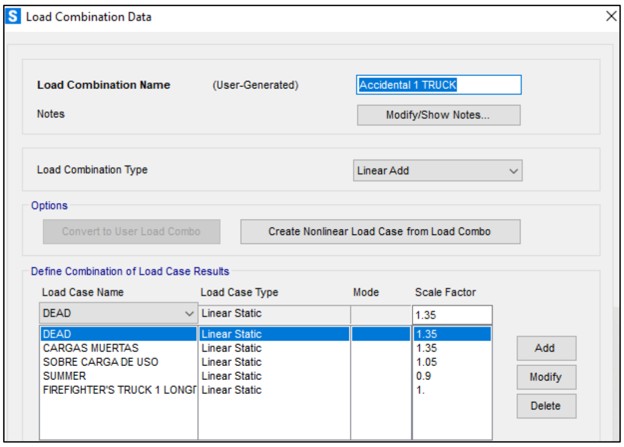
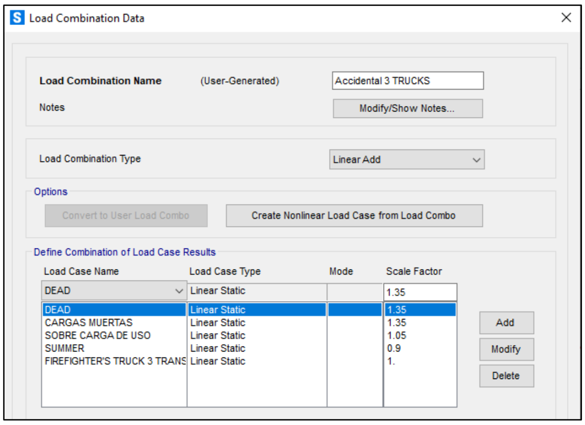
Define Combination of Load Case Results
Load Case Name
Load Case Type
Mode
Scale Factor
DEAD
Linear Static
1.35
DEAD
Linear Static
1.35
CARGAS MUERTAS
Linear Static
1.35
SOBRE CARGA DE USO
Linear Static
1.5
Add
Modify
Delete


Load Combination Data
Load Combination Name
(User-Generated)
UDSTL2.1
Notes
Modify/Show Notes...
Load Combination Type
Linear Add
Options
Convert to User Load Combo
Create Nonlinear Load Case from Load Combo
Define Combination of Load Case Results
Load Case Name
Load Case Type
Mode
Scale Factor
DEAD
Linear Static
1.35
DEAD
CARGAS MUERTAS
SOBRE CARGA DE USO
SUMMER
Linear Static
Linear Static
Linear Static
Linear Static
1.35
1.35
1.5
1.5
Add
Modify
Delete


Load Combination Data
Load Combination Name
(User-Generated)
UDSTL2.2
Notes
Modify/Show Notes...
Load Combination Type
Linear Add
Options
Convert to User Load Combo
Create Nonlinear Load Case from Load Combo
Define Combination of Load Case Results
Load Case Name
Load Case Type
Mode
Scale Factor
DEAD
Linear Static
1.35
DEAD
CARGAS MUERTAS
SOBRE CARGA DE USO
SNOW
WINTER
Linear Static
Linear Static
Linear Static
Linear Static
Linear Static
1.35
1.35
1.5
1.5
1.5
Add
Modify
Delete

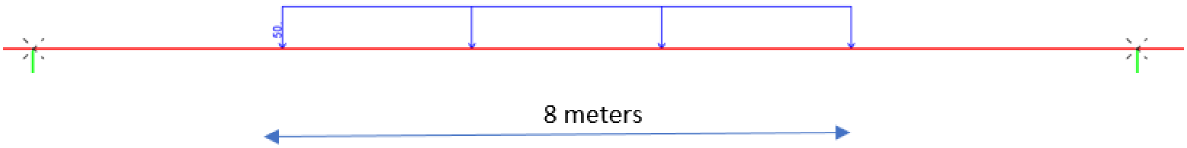




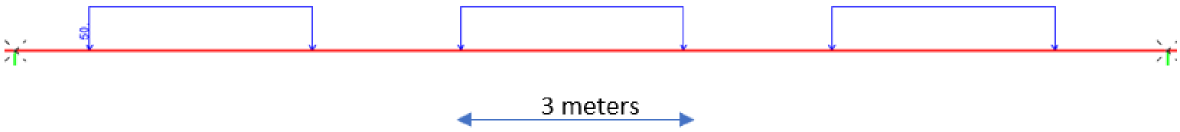


It is worth noting, that there are two possibilities for the positioning of the firefighter's truck in the case of analyzing the **beam** loads:

- One truck placed on the beam length, which is 15m, occupying 8 meters resting in the center of the beam:



- The second possibility, having three trucks placed on the transversal section of the Beam, each one occupies 3 meters of the beam length.

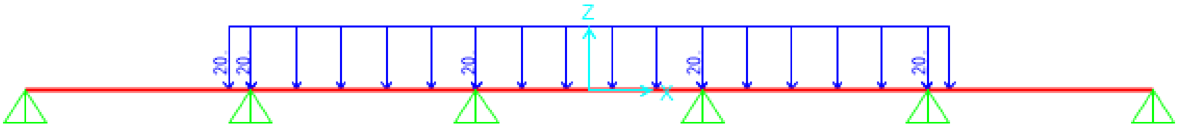


In both cases, the tributary area is 20 x 2,5 (Beam tributary area) = 50 KN.m.

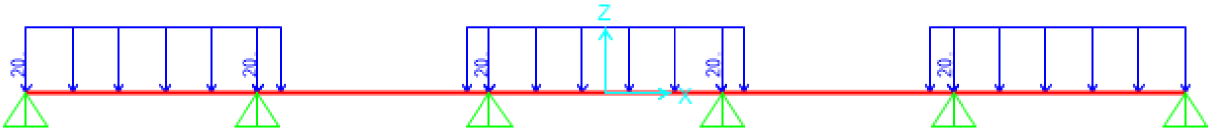
Therefore, we have two different accidental load combinations (Accidental 1 truck and Accidental 3 Trucks).

<u>It is worth noting,</u> that there are as well two possibilities for the positioning of the firefighter's truck in the case of analyzing the **slab** loads:

- One truck placed longitudinally, knowing that the separation is 2,5m:



- The second possibility, having three trucks placed transversely.



6 Results

6.1 Reactions

Table 3 Illustrates basement reactions due to each load combination.

TABLE: Base Reactions				
OutputCase	**CaseType**	**GlobalFX**	**GlobalFY**	**GlobalFZ**
Text	Text	KN	KN	KN
UDSTL1	Combination	-3.182E-09	1.391E-07	266049
UDSTL2.2	Combination	-1.133E-08	3.297E-07	267020
SERVICE	Combination	-2.302E-09	9.981E-08	192410
UDSTL2.1	Combination	2.877E-08	-0.000001252	266049
Accidental 1 TRUCK	Combination	1.621E-08	-7.085E-07	247960
UDSTL2.3	Combination	-9.588E-09	2.471E-07	120346
Accidental 3 TRUCKS	Combination	1.621E-08	-7.085E-07	248060

Table 3. Basement Reactions obtained from SAP2000.

6.2 First Basement

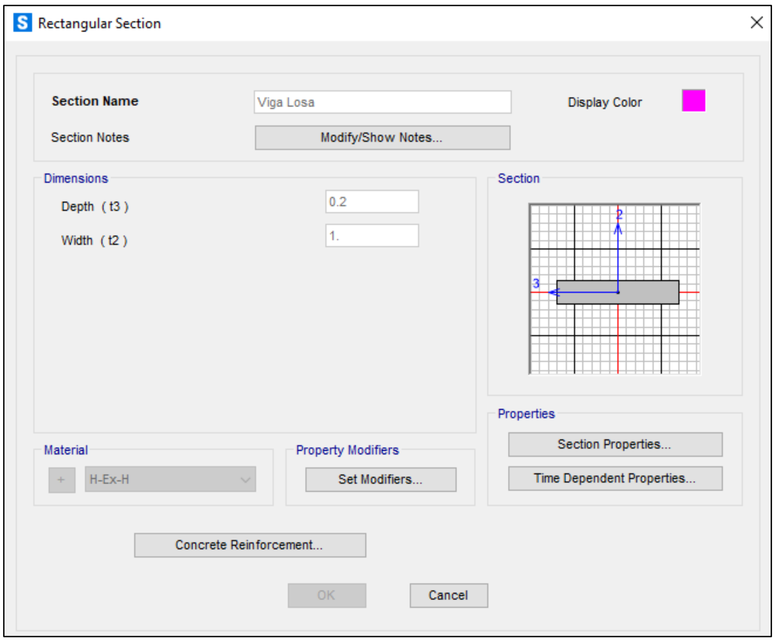
The analysis of the structure will be divided into 2 parts: first basement and second basement.

6.2.1 Slabs

6.2.1.1 Bending Moment Verification

The load combinations are:

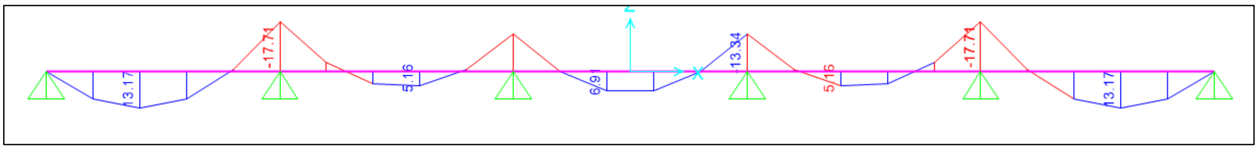
1) Service: 1,00 (Dead Load + Live Load) + 1,00 Live Load
2) UDSTL1: 1,35 (Dead Load + Live Load) + 1,5 Live Load
3) UDSTL2.1: 1,35 (Dead Load + Live Load) + 1,5 Live Load + 1,5 Summer
4) UDSTL2.2: 1,35 (Dead Load + Live Load) + 1,5 Live Load + 1,5 Snow + 1,5 Winter
5) UDSTL2.3: 0,8 (Dead Load + Live Load) + 1,5 Winter
6) Accidental 1 TRUCK: 1,35 (Dead Load + Live Load) + 1,05 Live Load + 0,9 Summer + 1,0 Firefighter's truck 1 Longitudinal
7) Accidental 3 Trucks: 1,35 (Dead Load + Live Load) + 1,05 Live Load + 0,9 Summer + 1,0 Firefighter's trucks 3 Transversal



All the loads are considered in the calculation of the slab including the dead loads, live loads, summer, winter, and firefighter's truck load. In addition, the same load combinations as well are considered for the calculation.

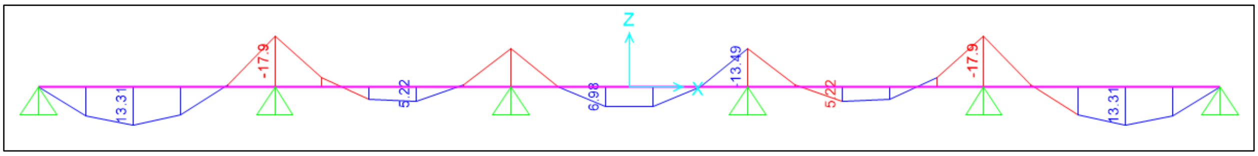
By introducing all the forces to SAP2000 it gives us the following results:

1) Moment diagram for USTDL 1



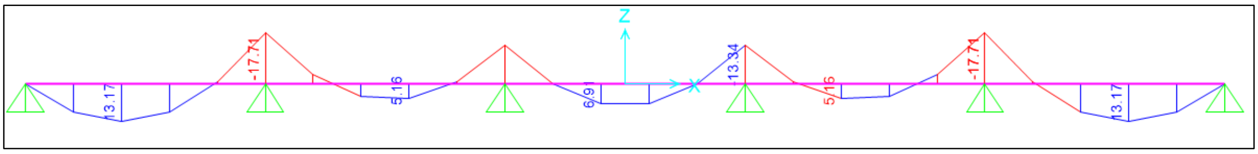
Md,max+ = 13,17 KN.m. Md,max- = -17,71 Kn.m.

2) Moment diagram for USTDL 2.1



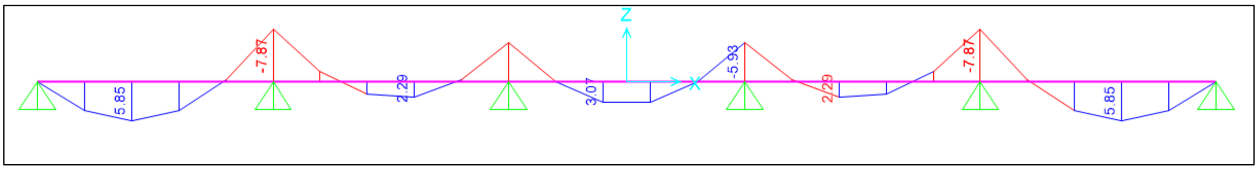
Md,max+ = 13,31 KN.m. Md,max- = -17,9 Kn.m.

3) Moment diagram for USTDL 2.2



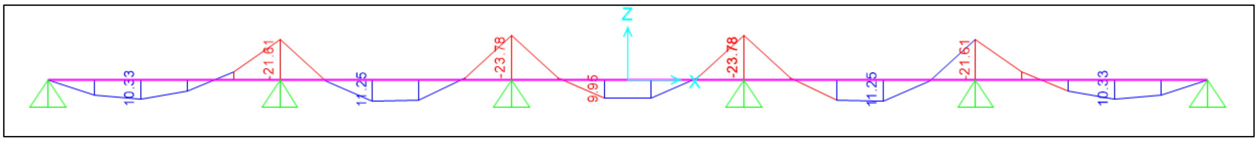
Md,max+ = 13,17 KN.m. Md,max- = -17,71 Kn.m.

4) Moment diagram for USTDL 2.3



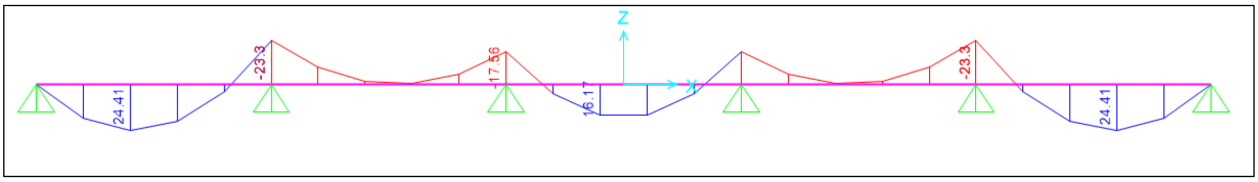
Md,max+ = 5,85 KN.m. Md,max- = -7,87 Kn.m.

5) Moment diagram for Accidental 1 truck



Md,max+ = 11,25 KN.m. Md,max- = -23,78 Kn.m.

6) Moment diagram for Accidental 3 trucks



Md,max+ = 24,41 KN.m.

Md,max- = -23,3 Kn.m.

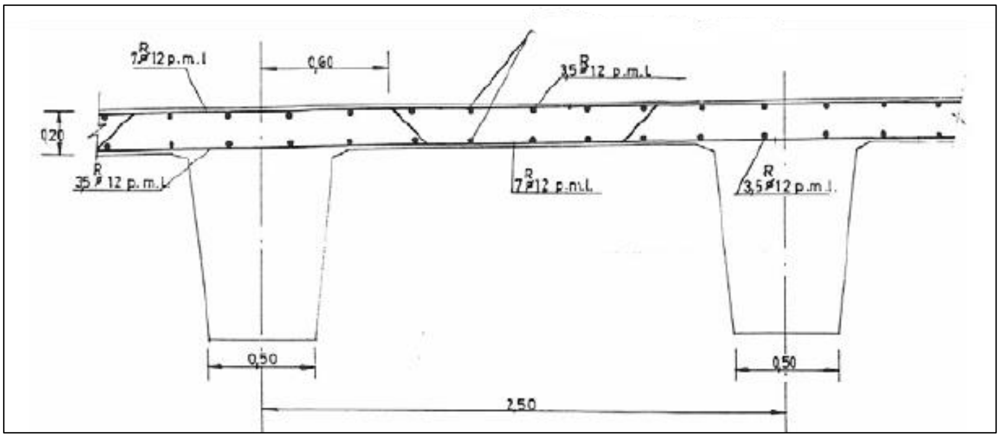


Figure 33. Slab Section

γc = 1,5 the partial safety factor for Concrete.

γs= 1,15 the partial safety factor for Steel.

As superior (mm^2) = 7 Φ 12 = 792 mm^2. b = 1 m.

h = 0,2 m.

r = 0.03 m (concrete cover to reinforcement)

d = h – r = 0,2 – 0,03 = 0,17 m.

Fck = 17,7 MPa >> Fcd = γc * Fck = 11,8 MPa.

Fyk = 450 Mpa >> Fyd = γs * Fd = 391,3 MPa.

As superior (mm^2) = 7 Φ 12 = 792 mm^2.

As a result of the calculation, the positive moment resistance is 47,41 KN.m.

As inferior (mm^2) = 3,5 Φ 12 = 396 mm^2.

The negative moment resistance

$* \; Mu = 23{,}7 \; KN.m$

Load Combination	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3	Accidental 1 Truck	Accidental 3 Trucks
Md,max+ (KN.m)	13,17	13,31	13,17	5,85	11,25	24,41
Reduced Resistance moment Mu+ (KN.m)	47,41	47,41	47,41	47,41	47,41	47,41
Safety factor	3,6	3,5	3,6	8,11	4,2	1,9

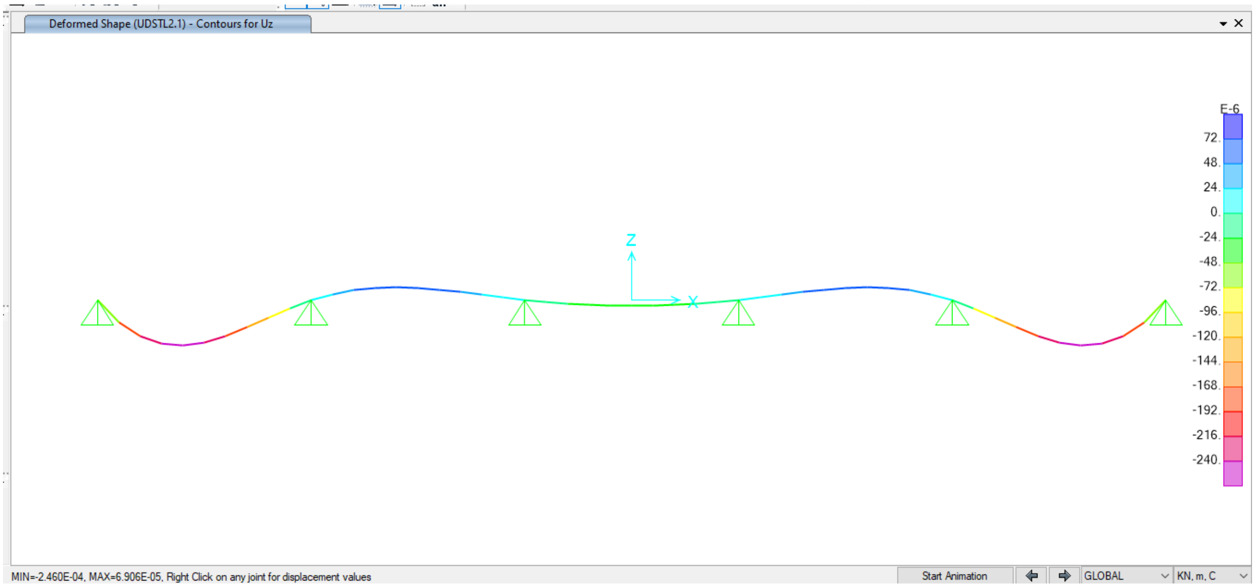
The reduced resistance moment is greater than all the positive bending moments, therefore the slab meets the requirements.

Load Combination	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3	Accidental 1 Truck	Accidental 3 Trucks
Md,max- (KN.m)	-17,71	-17,9	-17,71	-7,87	-23,87	-23,3
Reduced Resistance moment Mu- (KN.m)	23,70	23,70	23,70	23,70	23,70	23,70
Safety factor	1,33	1,32	1,33	3,00	0,99	1,01

The reduced resistance moment is greater than all the negative bending moments, therefore the slab meets the requirements.

6.2.1.2 Deflection

Load Combination	Service	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3
Deflections (mm)	0,22	0,24	0,24	0,24	0,1



The maximum deflection is of 0,24 mm in the first span of both sides.

According to CTE:

- The deformation should be inferior or equal ≤ L/300 = 2500/300 = 8,33 mm **OK**

6.2.2 Columns

In this report, we will try to analyze the most significant columns, which are loaded with the most severe loads, in order to obtain the most unfavorable cases and compare them with the own element resistant forces and with the allowable limits.

The columns of the first basement of the parking structure have a rectangular section (0,5 x 0,5) m.

6.2.2.1 Column 136

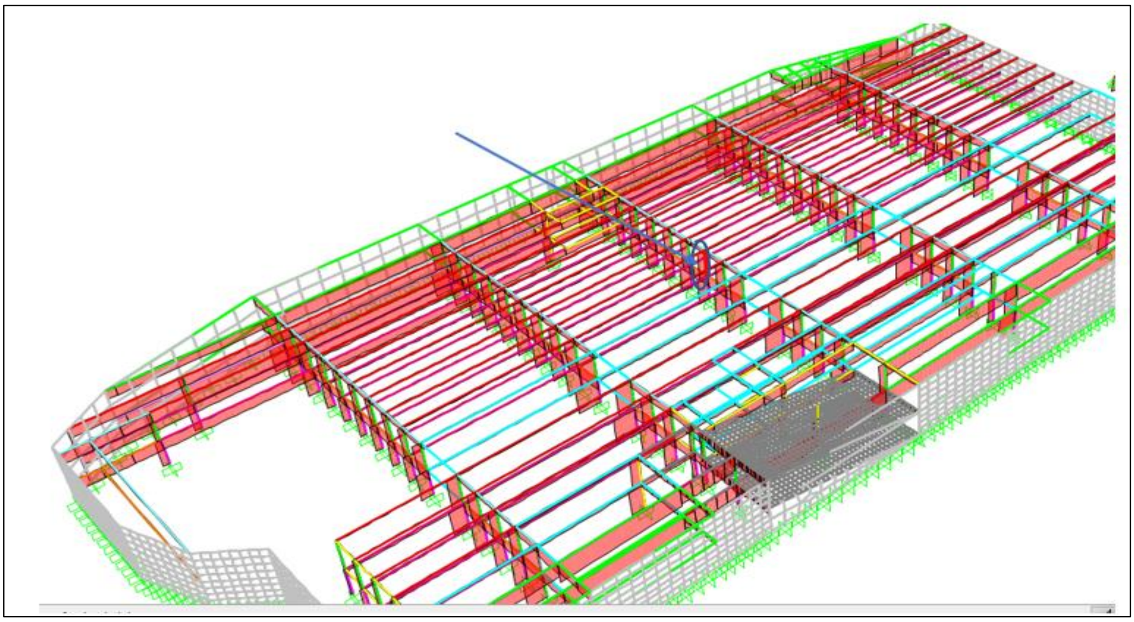


Figure 34. Column 136 Location.

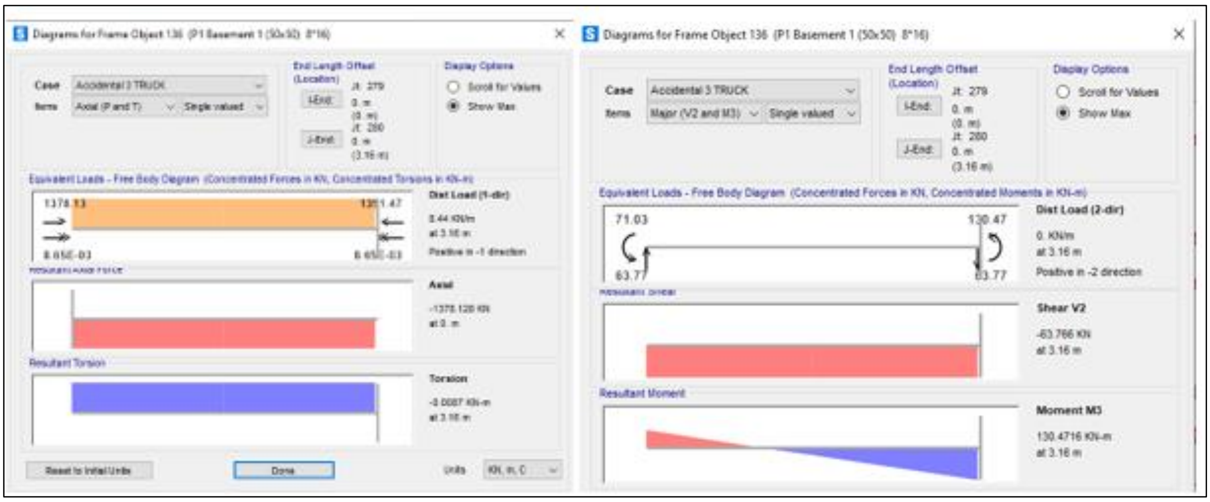


Figure 35. An example for the moment and axial diagram obtained from SAP2000

Figure 35 illustrates the column diagram forces for the combination 3 trucks.

	UDSTL1		UDSTL2.1		UDSTL2.2		UDSTL2.3		Accidental 1 Truck		Accidental 3 Trucks	
Point	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)
1) Mmax+	55	1290	73	1269	40	1302	7	620	142	1358	131	1372
2) Mmax-	45	1290	51	1269	39	1302	12	620	75	1358	71	1372

Table 4. Forces applied to the Column.

The column is subjected to an axial and moment. Therefore, the interaction diagram will be obtained from the program SAP2000.

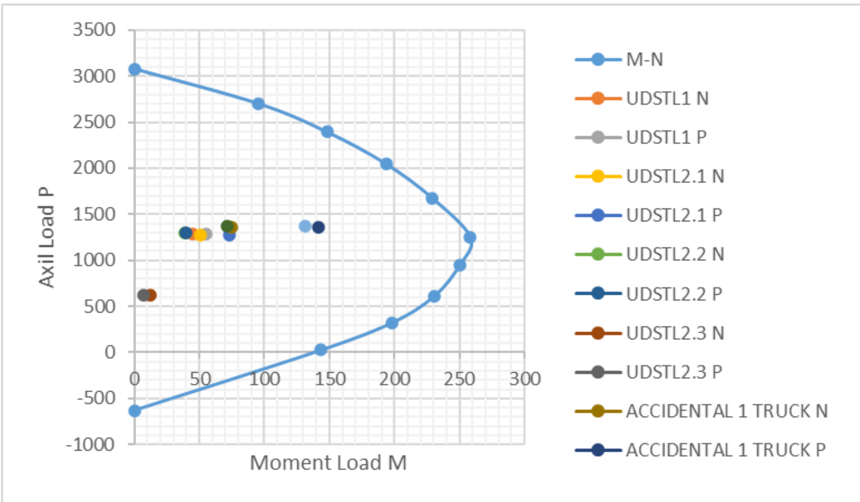


Figure 36. Interaction Diagram obtained from SAP2000 for all the combinations.

All points are inside the interaction diagram. Therefore, the column can resist the forces and loads applied to it.

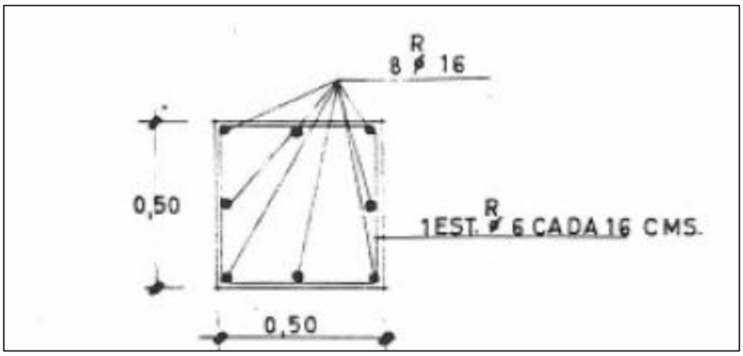


Figure 37. P1 Column Section.

The second-order effects can be neglected if the mechanical slenderness is less than the inferior slenderness limit. The inferior slenderness limit λinf can be approximated by the following expression:

$$\lambda_{\text{inf}} = 35\sqrt{\frac{C}{\nu}\left[1+\frac{0{,}24}{e_2/h}+3{,}4\left(\frac{e_1}{e_2}-1\right)^2\right]} \ngtr 100$$

Where:

- ν: Dimensionless axial
  $$\nu = \frac{N_d}{(A_c \cdot f_{cd})}$$
- e_2: First-order eccentricity at the end of the support with the greatest moment, considered positive.
- e_1: First-order eccentricity at the end of the support with the smallest moment, positive if it has the same sign as e2.
  In translational structures, e1/e2 will be taken equal to 1.0.
- h: Depth of the section in the considered bending plane.
- C: Coefficient that depends on the arrangement of reinforcement whose values are:
  0,24 for symmetric reinforcement on two opposite faces in the bending plane.
  0,20 for equal reinforcement on all four faces.
  0,16 for symmetric reinforcement on the lateral faces.
- L: Column length equal to 3,16 m
- γc = 1,5 the partial safety factor for Concrete.
- γs = 1,15 the partial safety factor for Steel.
- Fck = 15,2 MPa characteristic strength of concrete.

- Fyk = 450 MPa yield stress of steel.

$$Ic = \sqrt{\frac{Ic}{Ac}} = 0{,}1443m$$

$$\alpha = \frac{0{,}64+1{,}4(\Psi_A+\Psi_B)+3\Psi_A\Psi_B}{1{,}28+2(\Psi_A+\Psi_B)+3\Psi_A\Psi_B}$$

$$\psi = \frac{\sum soportes\frac{EI}{L}}{\sum vigas\frac{EI}{L}}$$

Pilar	Lower End					Top End					α	L_0 (m)
	I/L Pillar 1	I/L Pillar 2	I/L BEam 1	I/L Beam 2	ΨA	I/L Pillar 1	I/L Pillar 2	I/L BEam 1	I/L Beam 2	ΨB		
136	1648206.751	2530033.455	8.3E+06	8.3E+06	0.25	1648206.8	0	8.3E+06	8.3E+06	0.10	0.59	1.854273

$\lambda_{mec} = \frac{L0}{ic} = 12{,}847$

$$\lambda_{inf} = 35\sqrt{\frac{C}{\nu}\left[1+\frac{0{,}24}{e_2/h}+3{,}4\left(\frac{e_1}{e_2}-1\right)^2\right]} \ngtr 100$$

Where:

- v = $\frac{1364}{0{,}25*10{,}13/1000} = 0{,}54$. C = 0,2.
- h/20 = 500/20 = 25 mm. e_{min} = 20 mm

e = Md/Nd = 102,3 mm > max (h/20, e_{min}) = 25 mm OK

- λ_{inf} =56.40 > λ_{mec} is greater than the mechanical slenderness, no buckling check is necessary. And since the first basement column is longer than the second basement, the buckling check will no longer be done.

Another verification should be made to check the minimum reinforcement ratio for the column.

$As \geq \frac{4}{1000} Ac$

As = 8Φ16 = 1608 mm^2 > 1000 mm^2. OK

6.2.2.2 Column 118

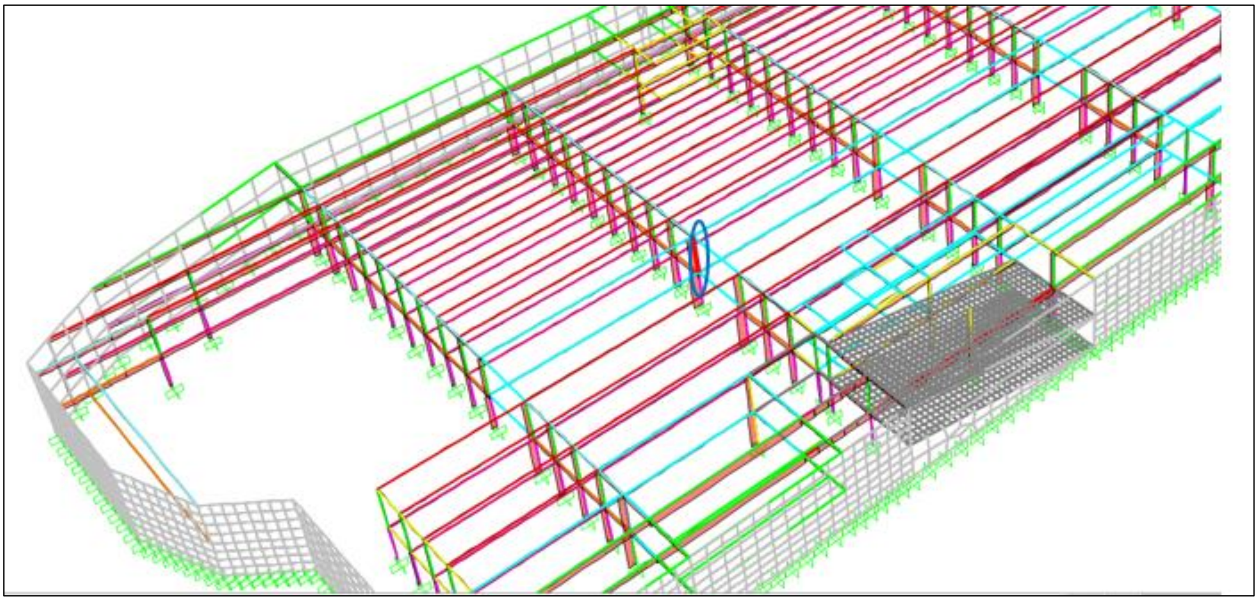


Figure 38. Column 118 Location.

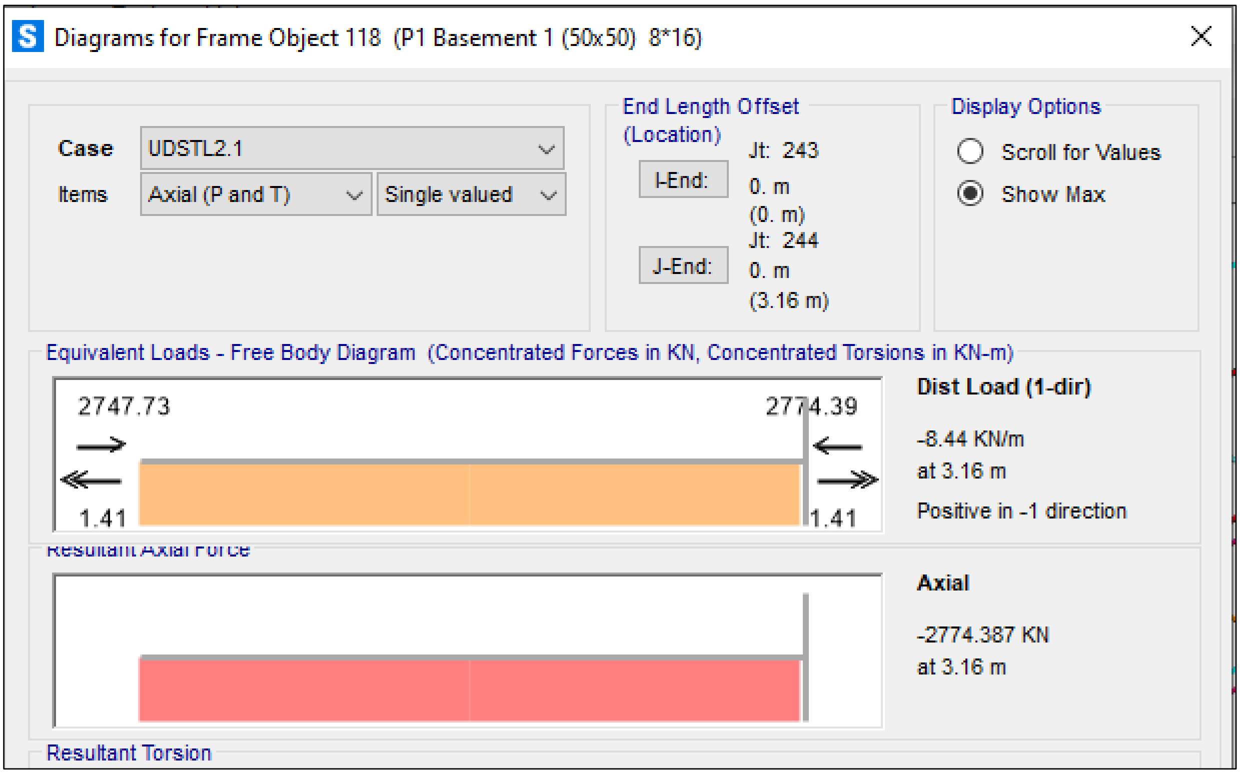


Figure 39. Axil forces obtained from SAP2000.

	UDSTL1		UDSTL2.1		UDSTL2.2		UDSTL2.3		Accidental 1 Truck		Accidental 3 Trucks	
Point	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)
1) Mmax+	38	2758	89	2775	14	2768	38	1251	71	2547	70	2551
2) Mmax-	75	2758	83	2775	66	2768	23	1251	93	2547	90	2551

Table 5. Forces applied to the Column

The column is subjected to an axial and moment. Therefore, the interaction diagram will be obtained from the program SAP2000.

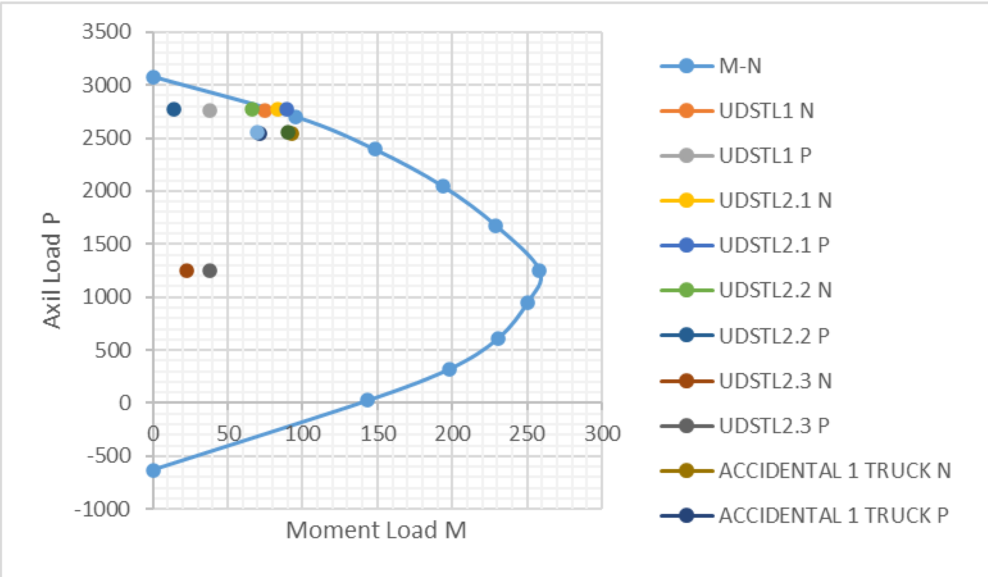


Figure 40. Interaction Diagram obtained from SAP2000.

All points are inside the interaction diagram expect for load combination UDSTL2.1. Therefore, the column doesn't meet the requirements and needs to be strengthened.

6.2.2.3 Column 172

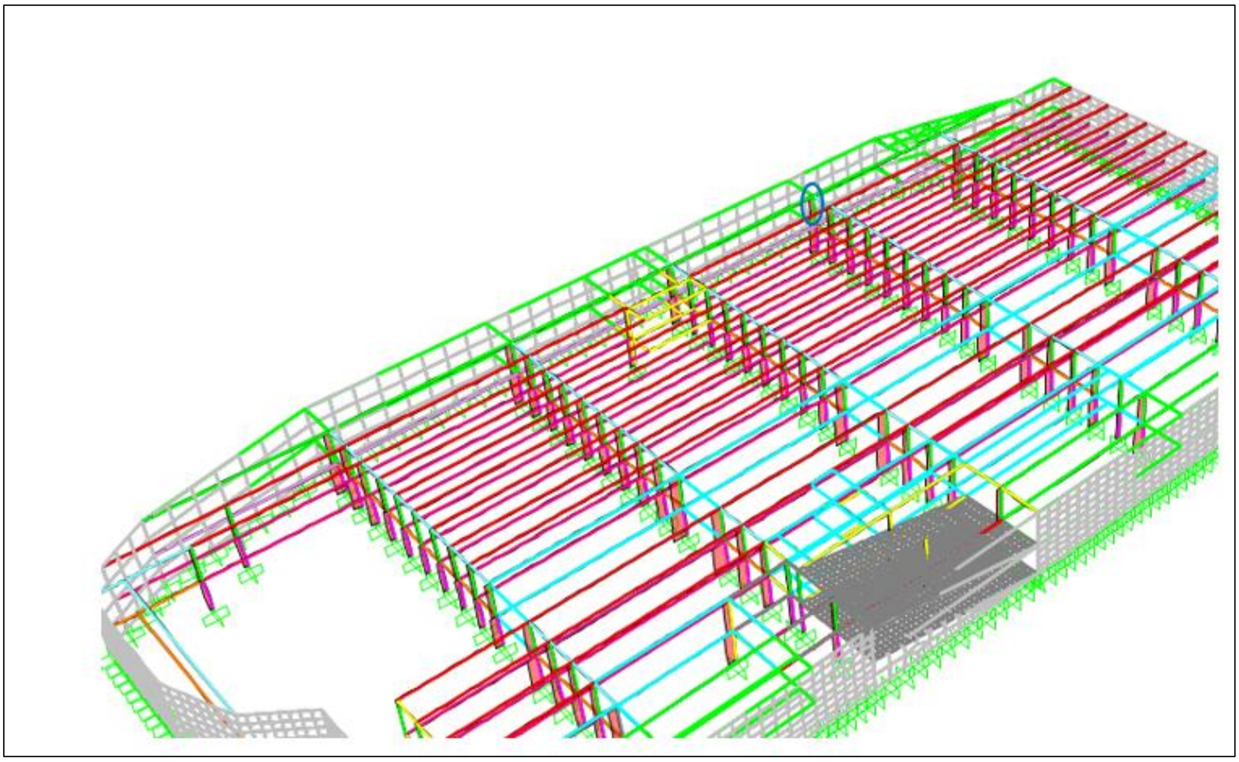


Figure 41. Column 172 Location.

	UDSTL1		UDSTL2.1		UDSTL2.2		UDSTL2.3		Accidental 1 Truck		Accidental 3 Trucks	
Point	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)
1) Mmax+	85	1987	98	1897	90	2008	40	930	157	1995	148	930
2) Mmax-	93	1987	99	1897	98	2008	40	930	115	1995	112	930

Table 6. Forces applied to the Column.

The column is subjected to an axial and moment. Therefore, the interaction diagram will be obtained from the program SAP2000.

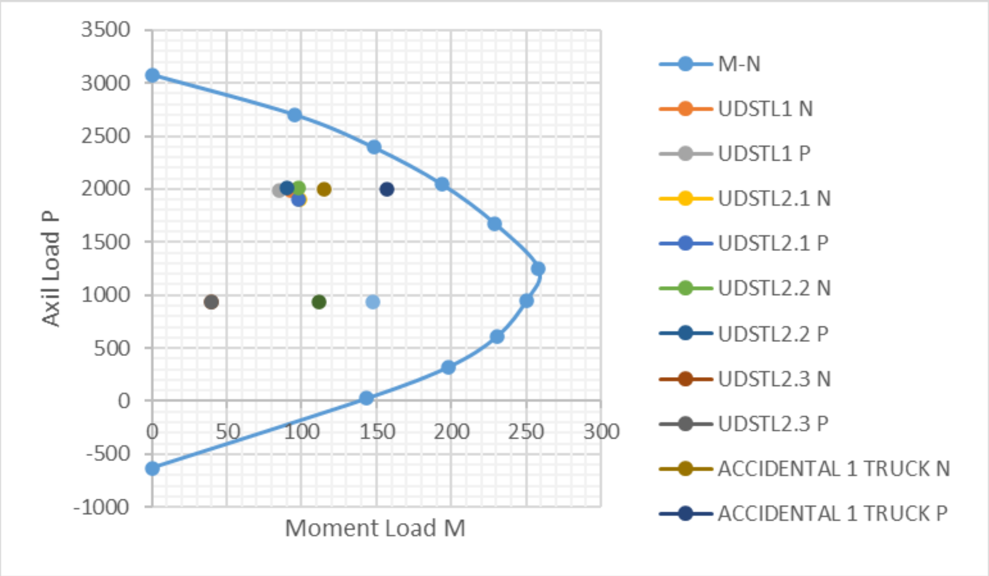


Figure 42. Interaction Diagram obtained from SAP2000.

All points are inside the interaction diagram. Therefore, the column can resist the forces and loads applied to it.

6.2.3 Beams

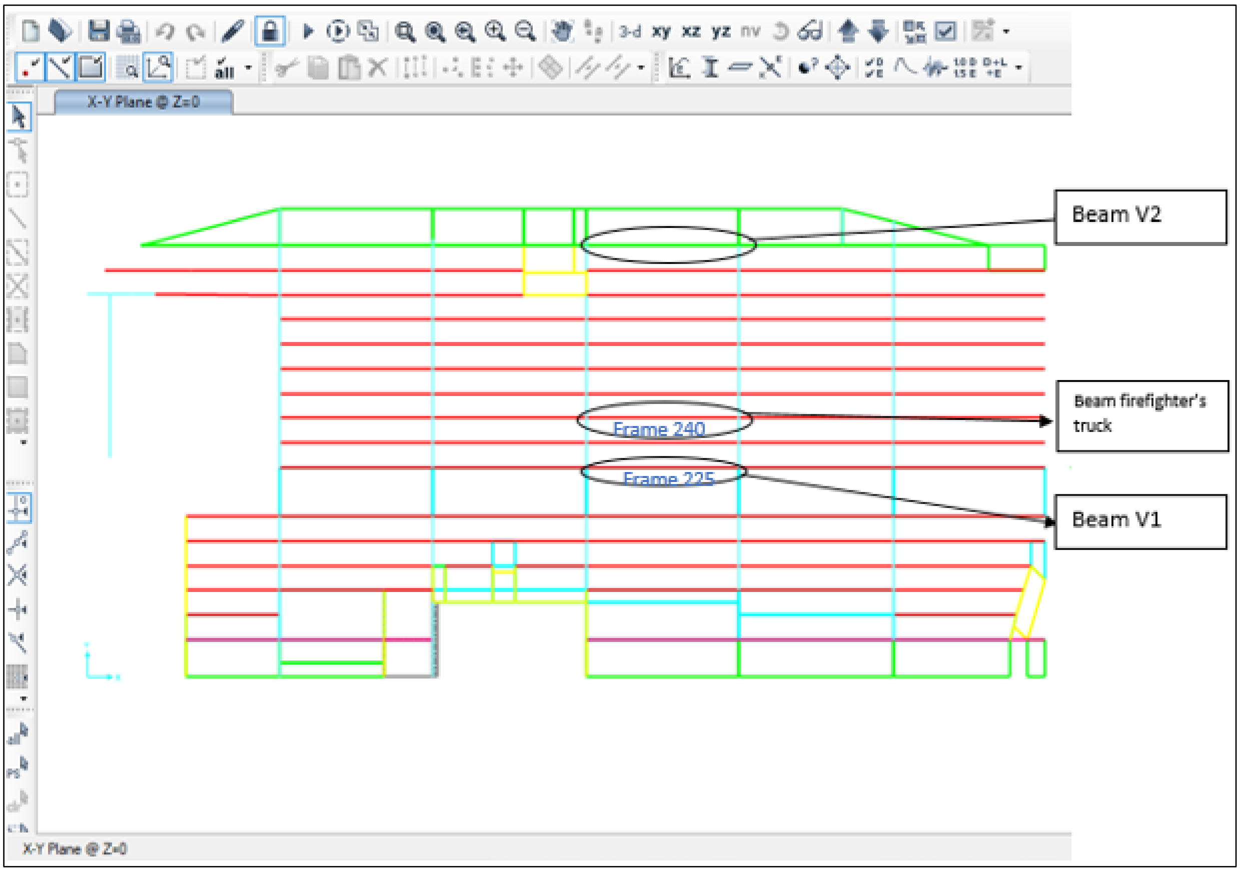


Figure 43. Plan view of the First Basement (X-Y)

In this report, we will analyze the most significant beams, to obtain the most unfavorable cases and compare them with the own element resistant forces and the allowable limits.

The parking structure has its beams of a T section

It is worth noting that the beams are classified into three groups that are more significantly important to analyze and verify:

- Beam 240 type V1: This beam has a tributary area of 2,5/2 + 2,5/2 = 2,5 m.
- Beam V1 – frame 225: has been chosen due to its greater tributary area of a 2,5/2 + 5/2 = 3,75 m
- Beam type V2: this beam has a tributary area of 2,5/2 + 3,75/2 = 3,125 m.
  This beam is located at the end of the beam's distribution, next to the north wall.

6.2.3.1 Beam 240

6.2.3.1.1 Bending Moment Verification

Figure 44 shows the Beam location and the moment diagram due to loading Condition 1 Truck Accidental.

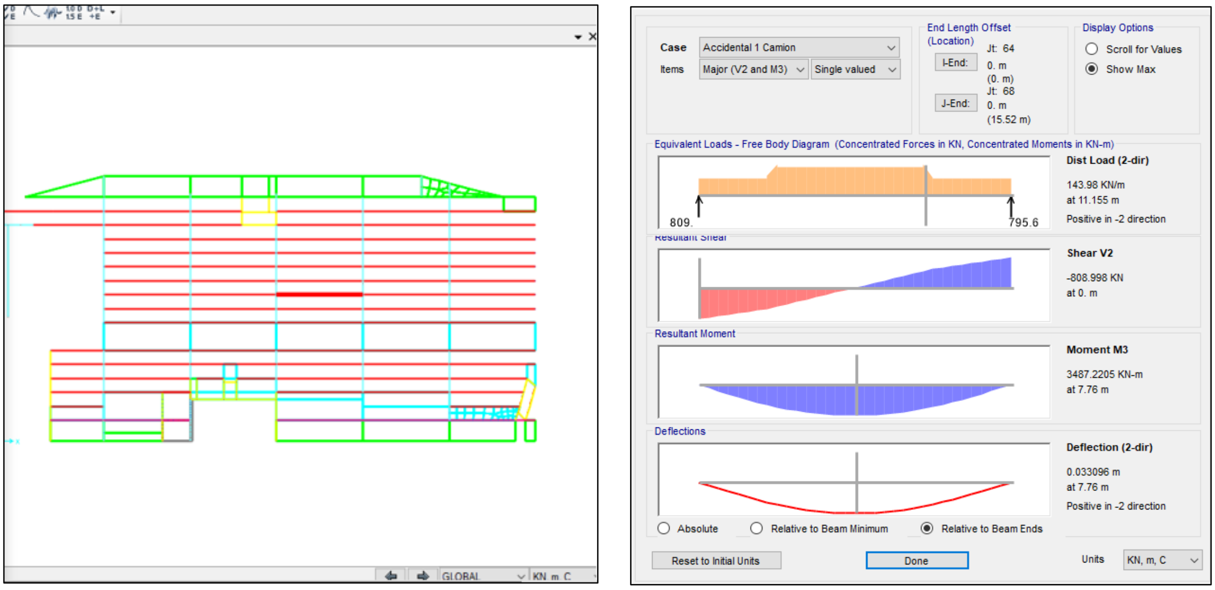
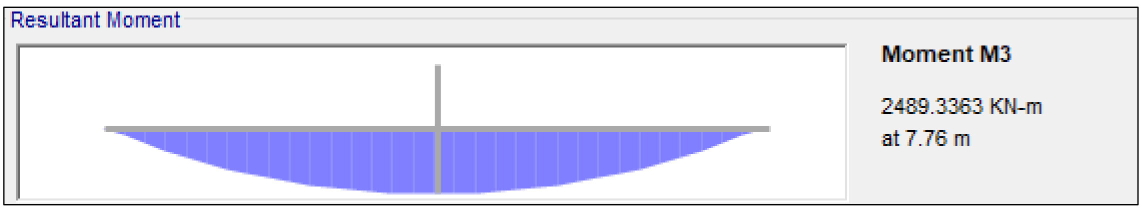


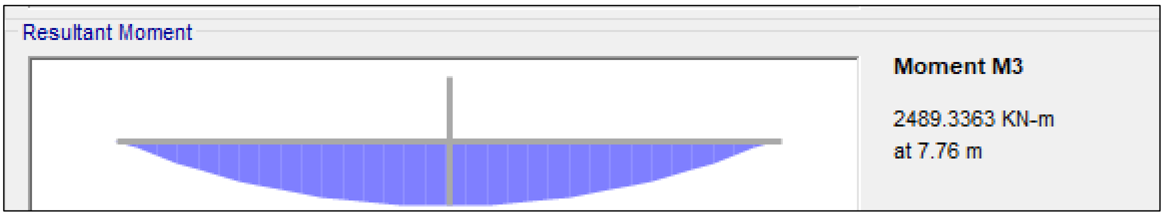
Figure 44 Beam 240 Location and bending moment

1) Moment diagram for USTDL 1:

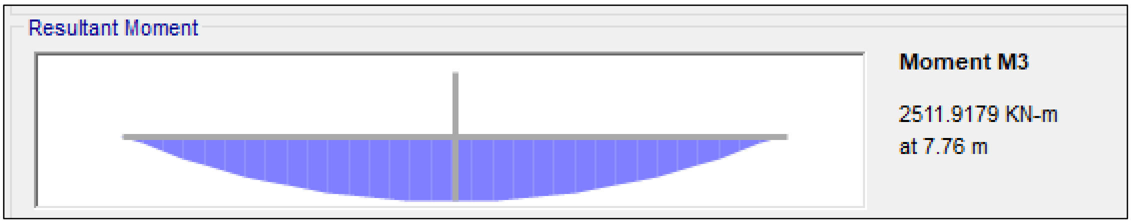


Md,max+ = 2490 KN.m.

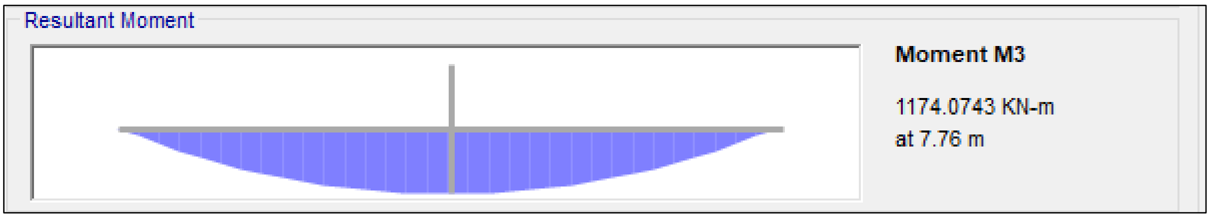
2) Moment diagram for USTDL 2.1



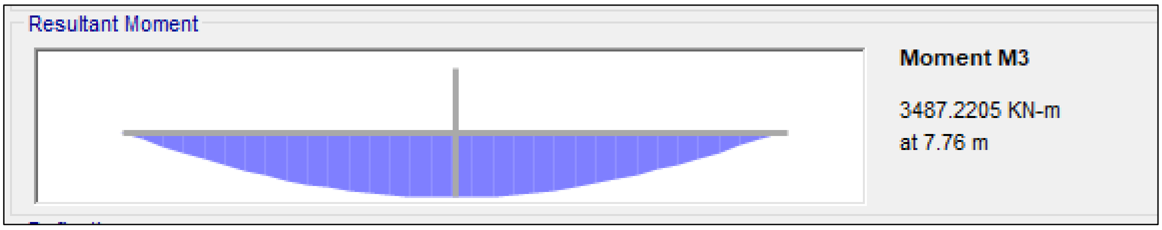
3) Moment diagram for USTDL 2.2



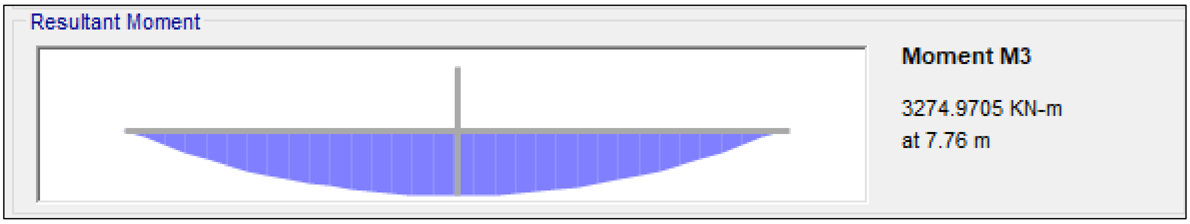
4) Moment diagram for USTDL 2.3



5) Moment diagram for Accidental 1 Truck



6) Moment diagram for Accidental 3 Truck



Md,max= 3488 KN.m

The worst combination that causes the maximum moment value for this beam is having one truck applying to the beam. Therefore, it governs the verification of the bending beam.

Where:

γc = 1,5 the partial safety factor for Concrete.

γs= 1,15 the partial safety factor for Steel.

a) There are two different ways that we may need to analyze our tea beams dependent on where the compression block lies in our section. In the first case, we can have our compression block entirely in our top flange. The depth of our neutral axis is C. The compression block has a depth of "a" which is equal to

$$a = 0{,}85 \times c$$

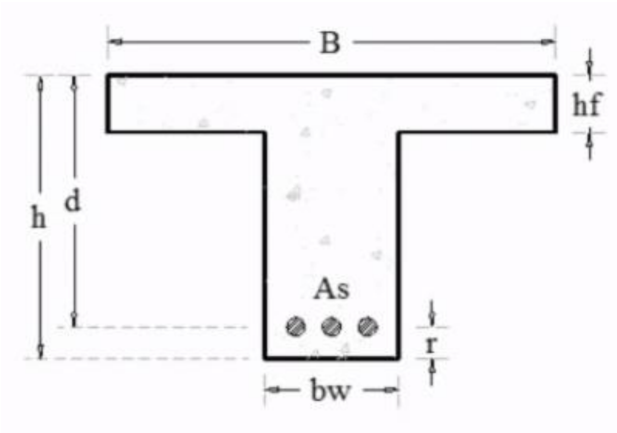
The tension force is:

$$T = As \times Fy$$

The compression force is:

$$C = 0{,}85 \times Fc \times a \times b$$

b) The second case is if our compression block extends into the web so in this case, we would have the neutral axis extending a little further down.



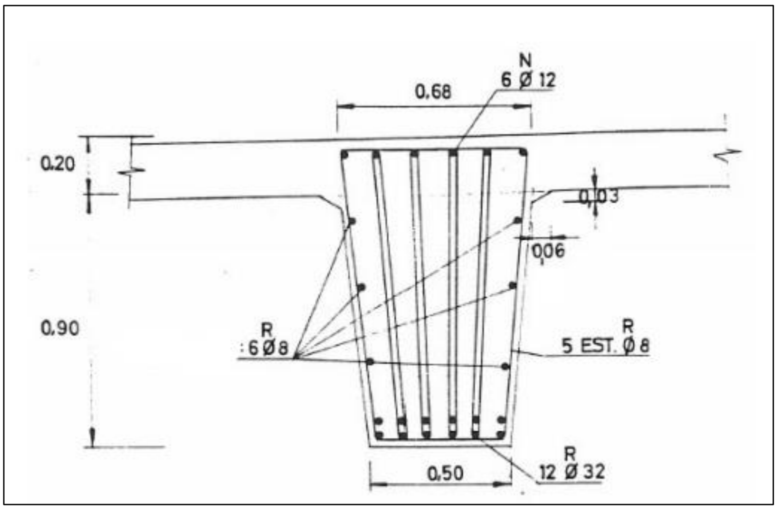


Figure 45. Section T-Beam V1.

As (mm^2) = 12 Φ 32 = 9646 mm^2.   bf = 2,5 m.

h = 1,1 m.   r = 0.03 m (concrete cover to reinforcement)

d = h – r = 1,1 – 0,03 = 1,07 m.   hf = 0,2 m.

Fck = 17,7 MPa >> Fcd = γc * Fck = 11,8 MPa.

Fyk = 450 Mpa >> Fyd = γs * Fd = 391,3 MPa.

T = 9646 * 450 = 4340651 N = 4341 KN.

$$C = 0{,}85 \times Fc \times a \times b$$

$$T = C$$

Through this equation, we can calculate “a”

$$a = \frac{4341}{0{,}85 * 17{,}7 * 1000 * 2{,}5} = 0{,}115\ m$$

a < hf = 0,2 m.

- The depth of the concrete block is at the flange. Therefore, we can consider our beam as if it was rectangular. we can see that this is just the same as if we had a rectangular section with a width of bf and depth of Steel equal to D.

In order to calculate the ultimate resistant moment, we apply the following equations:

$Mu = fcd.\ b.y.(d\text{-}y/2)$

$$Mu = 11{,}8 * 2500 * 115 * \left(1070 - \frac{115}{2}\right) = 3434906250\ N.mm = 3435\ KN.m$$

Load Combination	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3	Accidental 1 Truck	Accidental 3 Trucks
Md,max+ (KN.m)	2490	2490	2511	1147	3488	3275
Reduced Resistance moment Mu+ (KN.m)	3435	3435	3435	3435	3435	3435
Safety factor	1,39	1,39	1,37	3,00	0,98	1,05

Table 7. Beam 240 Safety Factor

The reduced resistance moment is greater than all the positive bending moments, therefore the beam meets the requirements.

6.2.3.1.2 Shear Verification

Load Combination	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3	Accidental 1 Truck	Accidental 3 Trucks
V_{Ed},max+ (KN)	643	643	650	304	811	836

Table 8. Shear forces according to each load combination.

The maximum shear force value applied on the Beam is of V_{Ed} = 836,2 KN due to the loading combination Accidental 3 Trucks.

Figure 46.Shear Forces according to SAP2000.

Definitions:

V_{Ed} - Applied shear force.

$V_{Rd,c:}$ Resistance of member without shear reinforcement.

$V_{Rd,s}$: Resistance of members governed by the yielding of shear reinforcement.

$V_{Rd,max}$: Resistance of member governed by the crushing of compression struts.

The formulas for shear concrete resistance with reinforcement are given by:

$$\boxed{V_{\mathrm{Rd,s}} = \frac{A_{\mathrm{sw}}}{s} z\, f_{ywd} \cot\theta}$$

$$\boxed{V_{\mathrm{Rd,max}} = \frac{\alpha_{cw}\, b_w\, z\, v_1\, f_{cd}}{\cot\theta + \tan\theta}}$$

Bw: is the minimum width.

A_{sw}: Area of the shear reinforcement.

$$\boxed{A_{sw} = 2.n.\frac{\pi.D^2}{4}}$$

f_{yd}: design yield strength = f_{yk}/1,15.

f_{cd}: design compressive strength f_{ck}/1,5.

α_{cw} = 1.0 Coefficient for stress in compression chord.

v1: strength reduction factor concrete cracked in shear v1 = 0.6(1-fck/250).

Θ: angle between the concrete compression strut and the beam axis= 45◦

Shear reinforcement diameter (D) (mm): 8 mm

Spacing between stirrups (s) (mm): 200

Number of stirrups by section (n): 5

While:

$A_{sw} = 2*5*\frac{\pi\ *8^2}{4}$ = 602,4 mm^2.

V1 = 0,6 (1-fck/250) = 0,6*(1-17,7/250) = 0,56

$$\boxed{V_{\mathrm{Rd,s}} = \frac{A_{\mathrm{sw}}}{s} z\, f_{ywd} \cot\theta}$$

$V_{Rd,s}$ = 502,4/200*0.9*1070*(450/1,15)*1 = 1135000 N = 1135 KN > V_{Ed} = 836,2 KN **OK.**

$$\boxed{V_{\mathrm{Rd,max}} = \frac{\alpha_{cw}\, b_w\, z\, v_1\, f_{cd}}{\cot\theta + \tan\theta}}$$

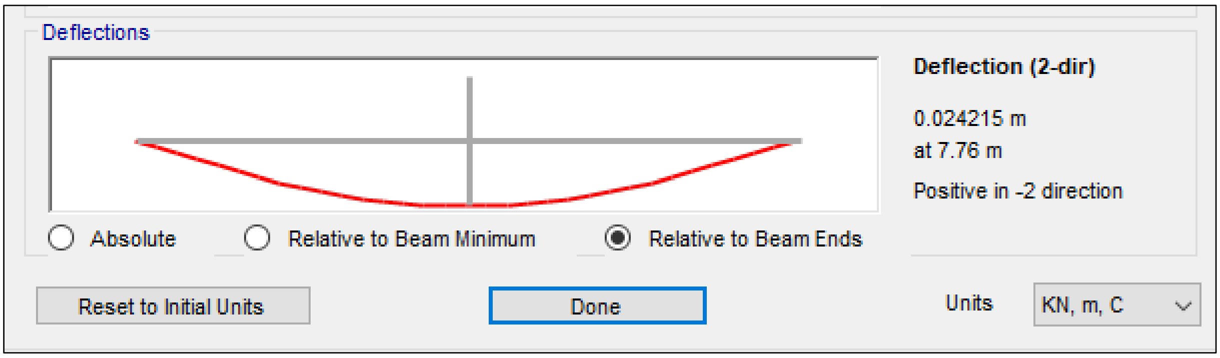
$V_{Rd,max}$ = (1*500*0,9*1070*0.56*17,7/1,5)/(1+1) = 1590876 N = 1590,87 KN > V_{Ed} = 836,2 KN **OK.**

The beam meets the requirements and can resist the shear forces applied.

- Safety factor: $\frac{VRd,s}{VEd} = \frac{1135}{836{,}2} = 1{,}358 > 0{,}9$ **OK.**

6.2.3.1.3 Deflection

Load Combination	Service	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3
Deflections (mm)	17,4	23.9	23,9	24,2	11,3



The worst combination is UDSTL2.2 which has a deflection of 24,2 mm.

According to CTE:

- The deformation should be inferior or equal 24,2 $\leq$ L/300 = 15500/300 = 51,6 mm **OK**

6.2.3.2 Beam Gap V1 Frame 225

6.2.3.2.1 Bending Moment Verification

Load Combination	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3	Accidental 1 Truck	Accidental 3 Trucks
Md,max+ (KN.m)	3252	3252	3274	1475	4173	3960
Reduced Resistance moment Mu+ (KN.m)	3435	3435	3435	3435	3435	3435
Safety factor	1,06	1,06	1,05	2,33	0,82	0.87

Table 9. Beam 225 Safety Factor

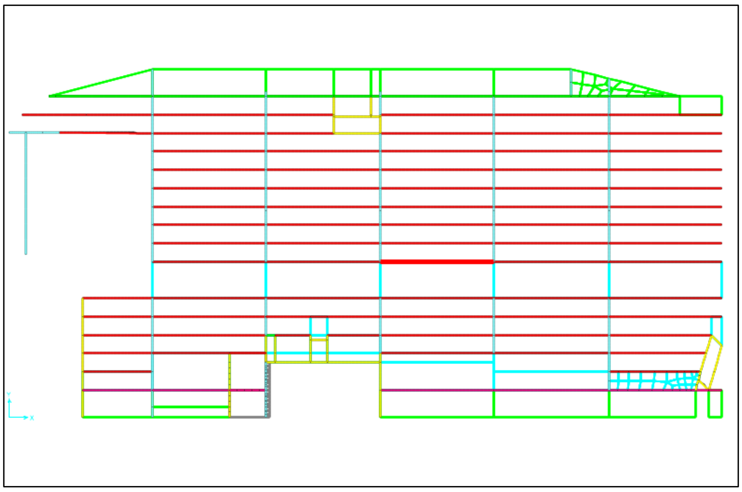


Figure 47. Beam 225 Location and bending moment

The worst combination that causes the maximum moment value for this beam is Accidental 1 Truck. Therefore, it governs the verification of the bending beam.

Figure 47 shows the Beam location and the moment diagram.

Where:

γc = 1,5 the partial safety factor for Concrete.

γs= 1,15 the partial safety factor for Steel.

Md,max= 3274KN.m

The depth of our neutral axis is C. The compression block has a depth of “a” which is equal to

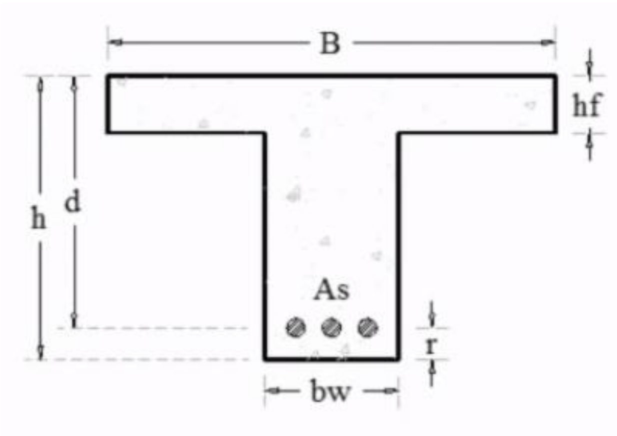
$$a = 0,85 \times c$$

The tension force is:

$$T = As \times Fy$$

The compression force is:

$$C = 0{,}85 \times Fc \times a \times b$$



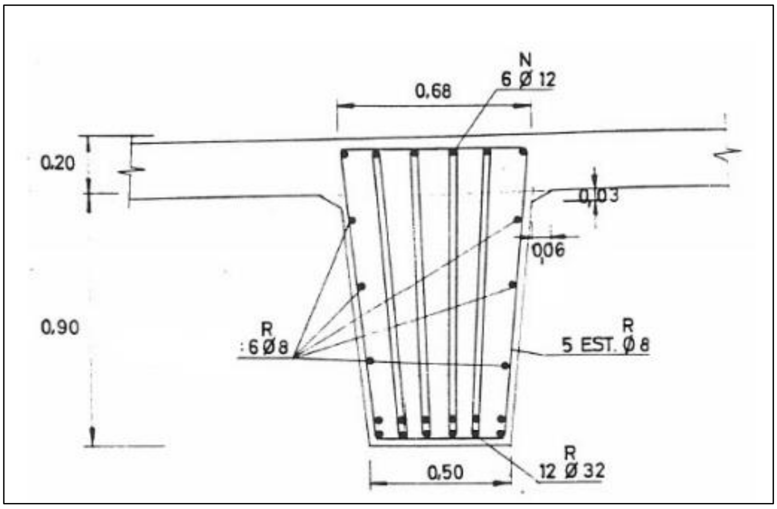


Figure 48. Section T-Beam V1.

As (mm^2) = 12 Φ 32 = 9646 mm^2. bf = 2,5 m.

h = 1,1 m. r = 0.03 m (concrete cover to reinforcement)

d = h – r = 1,1 – 0,03 = 1,07 m. hf = 0,2 m.

Fck = 17,7 MPa >> Fcd = γc * Fck = 11,8 MPa.

Fyk = 450 Mpa >> Fyd = γs * Fd = 391,3 MPa.

T = 9646 * 450 = 4340651 N = 4341 KN.

$$C = 0{,}85 \times Fc \times a \times b$$

T = C

Through this equation, we can calculate "a"

$$a = \frac{4341}{0{,}85 * 17{,}7 * 1000 * 2{,}5} = 0{,}115\ m$$

a < hf = 0,2 m.

- The depth of the concrete block is at the flange. Therefore, we can consider our beam as if it was rectangular. we can consider the section as if we had a rectangular section with a width of bf and depth of Steel equal to D.

In order to calculate the ultimate resistant moment, we apply the following equations:

$Mu = fcd.\ b.y.(d\text{-}y/2)$

$$Mu = 11{,}8 * 2500 * 115 * \left(1070 - \frac{115}{2}\right) = 3434906250\ N.mm = 3435\ KN.m$$

Safety Factor: $\frac{Mu}{Md} = \frac{3435}{4173} = 0{,}82 < 0{,}9$ This beam needs to be strengthened due to its low safety factor.

6.2.3.2.2 Shear Verification

Load Combination	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3	Accidental 1 Truck	Accidental 3 Trucks
V_{Ed},max+ (KN)	838	838	844	380	986	1012

Table 10. Shear forces according to each load combination.

The maximum shear force value applied on the Beam is of V_{Ed} = 1012 KN due to the loading combination Accidental 3 Trucks.

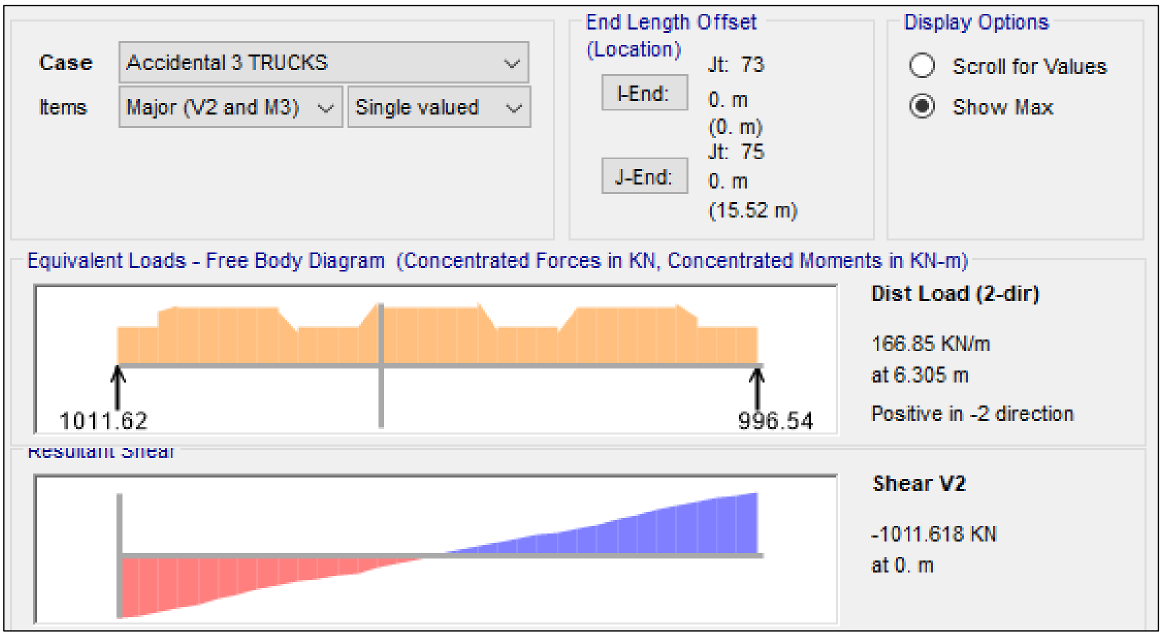


Figure 49. Shear Forces according to SAP2000.

The formulas for shear concrete resistance with reinforcement are given by:

$$\boxed{V_{Rd,s} = \frac{A_{sw}}{s} z\, f_{ywd} \cot\theta}$$

$$\boxed{V_{Rd,max} = \frac{\alpha_{cw}\, b_w\, z\, v_1\, f_{cd}}{\cot\theta + \tan\theta}}$$

Bw: is the minimum width.

A_{sw}: Area of the shear reinforcement.

$$\boxed{A_{sw} = 2.n.\frac{\pi.D^2}{4}}$$

f_{yd}: design yield strength = f_{yk}/1,15.

f_{cd}: design compressive strength f_{ck}/1,5.

α_{cw} = 1.0 Coefficient for stress in compression chord.

v1: strength reduction factor concrete cracked in shear v1 = 0.6(1-fck/250).

Θ: angle between the concrete compression strut and the beam axis= 45◦

Shear reinforcement diameter (D) (mm): 8 mm

Spacing between stirrups (s) (mm): 200

Number of stirrups by section (n): 5

While:

A_{sw} = 2*5*$\frac{\pi \ *8^2}{4}$ = 602,4 mm^2.

V1 = 0,6 (1-fck/250) = 0,6*(1-17,7/250) = 0,56

$$\boxed{V_{Rd,s} = \frac{A_{sw}}{s} z\, f_{ywd} \cot\theta}$$

$V_{Rd,s}$ = 502,4/200*0.9*1070*(450/1,15)*1 = 1135000 N = 1135 KN > V_{Ed} = 1012 KN **OK.**

$$\boxed{V_{Rd,max} = \frac{\alpha_{cw}\, b_w\, z\, v_1\, f_{cd}}{\cot\theta + \tan\theta}}$$

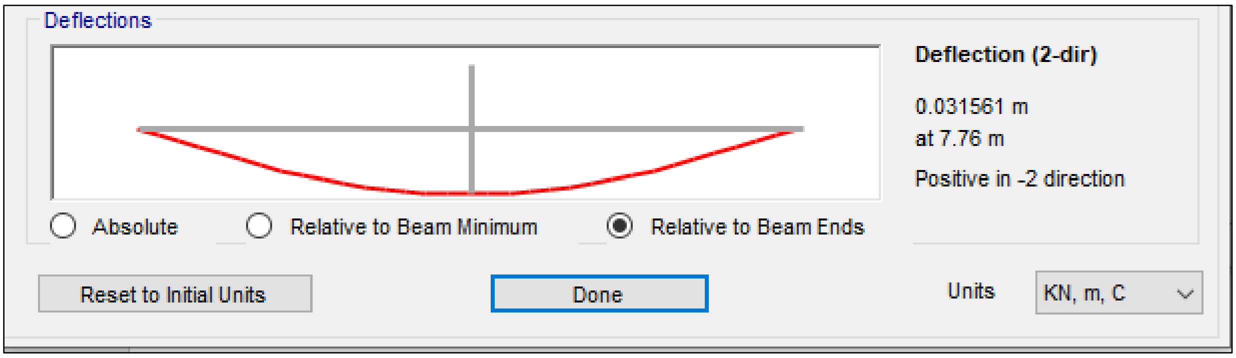
$V_{Rd,max}$ = (1*500*0,9*1070*0.56*17,7/1,5)/(1+1) = 1590876 N = 1590,8 KN > V_{Ed} = 1012 KN **OK.**

The beam meets the requirements and can resist the shear force applied.

- Safety factor: $\frac{VRd,s}{VEd} = \frac{1135,9}{1012} = 1,12 > 0,9$ **OK.**

6.2.3.2.3 Deflection

Load Combination	Service	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3
Deflections (mm)	22,6	31,3	31,3	31,6	14,2



The worst combination is UDSTL2.2 which has a deflection of 31,6 mm.

According to CTE:

- The deformation should be inferior or equal 31,6 $\leq$ L/300 = 15500/300 = 51,6 mm **OK**

6.2.3.3 Beam V2 Frame 275

6.2.3.3.1 Bending Moment Verification

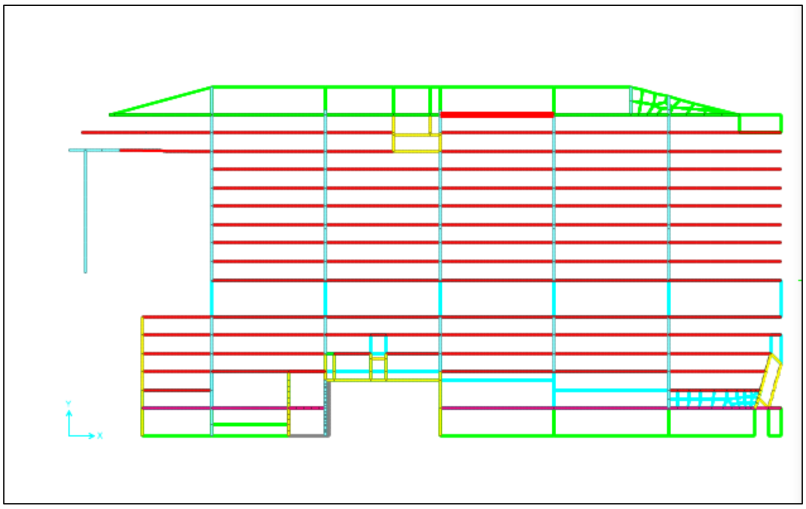


Figure 47 shows the Beam location.

Load Combination	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3	Accidental 1 Truck	Accidental 3 Trucks
Md,max+ (KN.m)	2870	2870	2910	1324	3834	3618
Reduced Resistance moment Mu+ (KN.m)	3992	3992	3992	3992	3992	3992
Safety factor	1,39	1,39	1,37	3,02	1,04	1,10

Table 11. Beam 275 Safety Factor.

The worst combination that causes the maximum moment value for this beam is Accidental 1 Truck. Therefore, it governs the verification of the bending beam.

Where:

γc = 1,5 the partial safety factor for Concrete.

γs= 1,15 the partial safety factor for Steel.

Md,max= 2870 KN.m.

The depth of our neutral axis is C. The compression block has a depth of “a” which is equal to

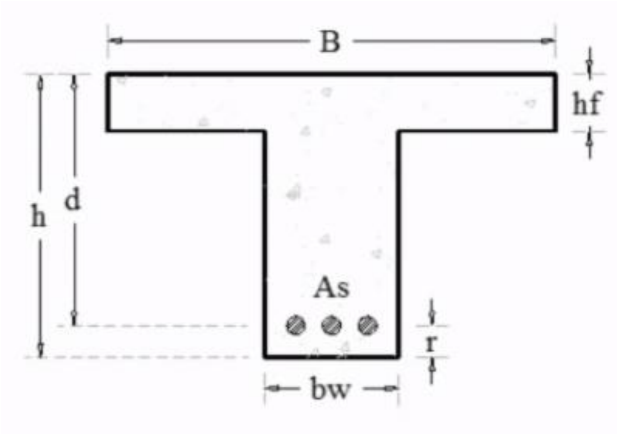
$$a = 0,85 \times c$$

The tension force is:

$$T = As \times Fy$$

The compression force is:

$$C = 0{,}85 \times Fc \times a \times b$$



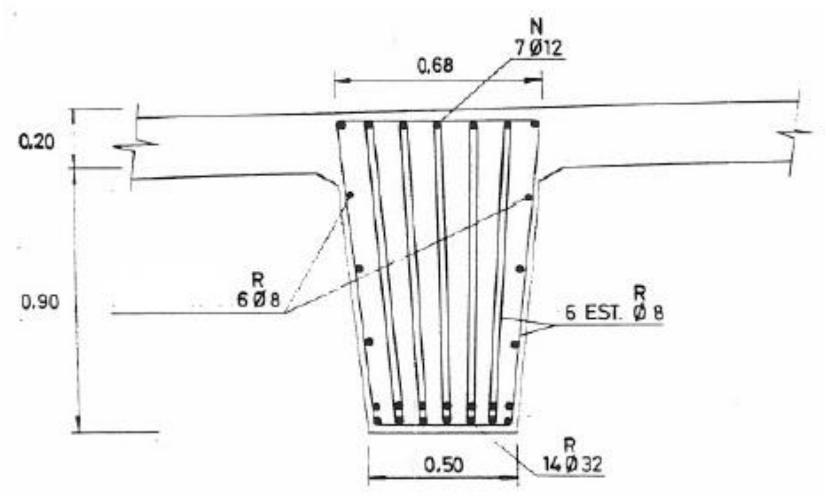


Figure 50. Section T-Beam V2.

As (mm^2) = 14 Φ 32 = 11260 mm^2. bf = 2,5 m.

h = 1,1 m. r = 0.03 m (concrete cover to reinforcement)

d = h – r = 1,1 – 0,03 = 1,07 m. hf = 0,2 m.

Fck = 17,7 MPa >> Fcd = γc * Fck = 11,8 MPa.

Fyk = 450 Mpa >> Fyd = γs * Fd = 391,3 MPa.

T = 11260 * 450 = 5067000 N = 5067 KN.

$$C = 0{,}85 \times Fc \times a \times b$$

T = C

Through this equation, we can calculate "a"

$$a = \frac{5067}{0{,}85 * 17{,}7 * 1000 * 2{,}5} = 0{,}135\ m$$

a < hf = 0,2 m.

- The depth of the concrete block is at the flange. Therefore, we can consider our beam as if it was rectangular. we can consider the section as if we had a rectangular section with a width of bf and depth of Steel equal to D.

In order to calculate the ultimate resistant moment, we apply the following equations:

$Mu = fcd.$ *b.y.(d-y/2)*

$$Mu = 11{,}8 * 2500 * 135 * \left(1070 - \frac{135}{2}\right) = 3992456250\ N.mm = 3992\ KN.m$$

Safety Factor: Ø.$\frac{Mu}{Md} = \frac{3992}{3834} = 1{,}14\ > 0{,}9$. This beam meets the requirements and resists the loads applied.

6.2.3.3.2 Shear Verification

Load Combination	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3	Accidental 1 Truck	Accidental 3 Trucks
V_{Ed},max+ (KN)	740	740	753	341	897	923

Table 12. Shear forces according to each load combination.

The maximum shear force value applied on the Beam is of V_{Ed} = 923 KN due to the loading Accidental combination with 3 trucks.

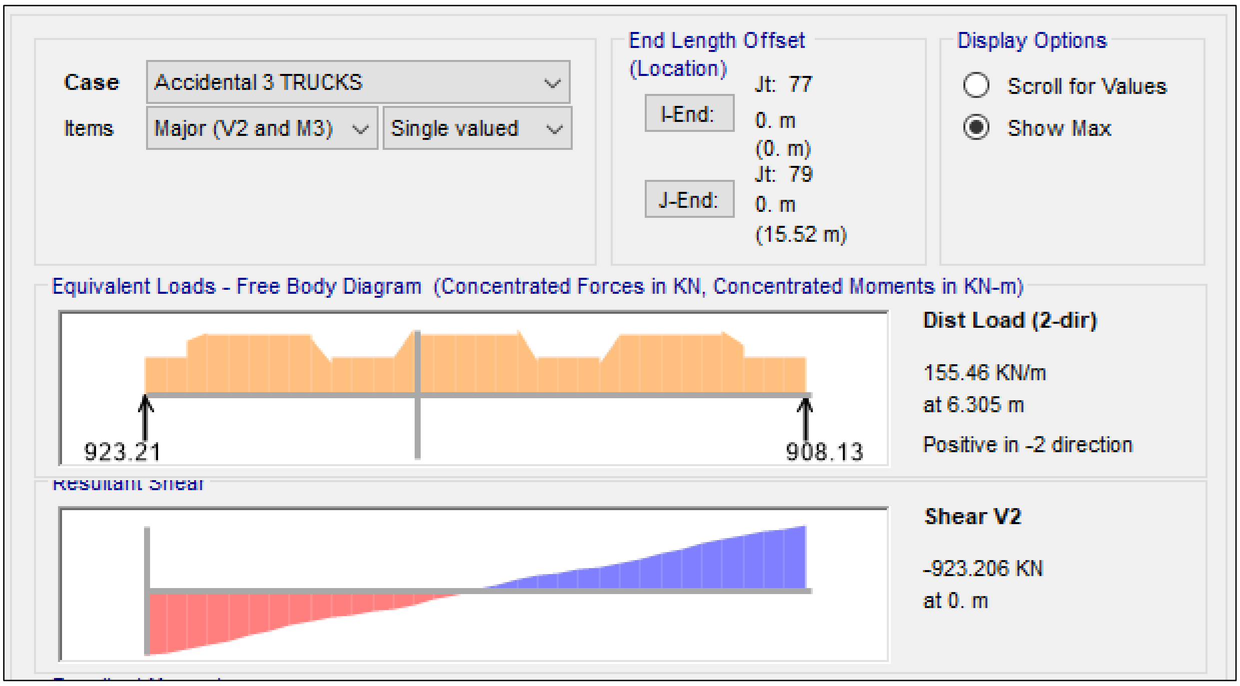


Figure 51. Shear Forces according to SAP2000.

The formulas for shear concrete resistance with reinforcement are given by:

$$\boxed{V_{Rd,s} = \frac{A_{sw}}{s} z \, f_{ywd} \cot\theta}$$

$$\boxed{V_{Rd,max} = \frac{\alpha_{cw} \, b_w \, z \, v_1 \, f_{cd}}{\cot\theta + \tan\theta}}$$

Bw: is the minimum width.

A_{sw}: Area of the shear reinforcement.

$$\boxed{A_{sw} = 2.n.\frac{\pi.D^2}{4}}$$

f_{yd}: design yield strength = f_{yk}/1,15.

f_{cd}: design compressive strength f_{ck}/1,5.

α_{cw} = 1.0 Coefficient for stress in compression chord.

v1: strength reduction factor concrete cracked in shear v1 = 0.6(1-fck/250).

Θ: angle between the concrete compression strut and the beam axis= 45◦

Shear reinforcement diameter (D) (mm): 8 mm

Spacing between stirrups (s) (mm): 200

Number of stirrups by section (n): 6

While:

A_{sw} = 2*6*$\frac{\pi \ *8^2}{4}$ = 602,88 mm^2.

V1 = 0,6 (1-fck/250) = 0,6*(1-17,7/250) = 0,56

$$\boxed{V_{Rd,s} = \frac{A_{sw}}{s} z\ f_{ywd} \cot\theta}$$

$V_{Rd,s}$ = 602,88/200*0.9*1070*(450/1,15)*1 = 1135904 N = 1135,9 KN > V_{Ed} = 923 KN **OK.**

$$\boxed{V_{Rd,max} = \frac{\alpha_{cw}\ b_w\ z\ v_1\ f_{cd}}{\cot\theta + \tan\theta}}$$

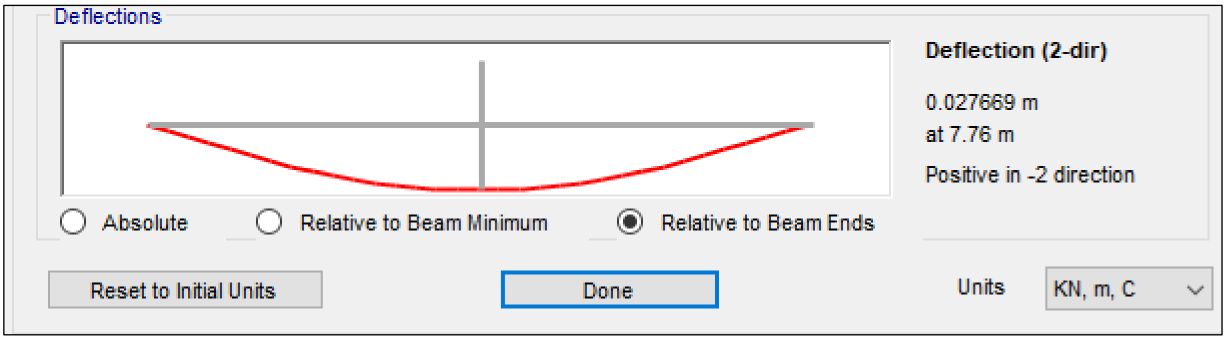
$V_{Rd,max}$ = (1*500*0,9*1070*0.56*17,7/1,5)/(1+1) = 1590876 N = 1590,8 KN > V_{Ed} = 923 KN **OK.**

The beam meets the requirements and can resist the shear force applied.

- Safety factor: $\frac{VRd,s}{VEd} = \frac{1135,9}{923} = 1,23 > 0,9$ **OK.**

6.2.3.3.3 Deflection

Load Combination	Service	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3
Deflections (mm)	20	27,7	27,7	28,6	12,7



The worst combination is UDSTL2.2 which has a deflection of 28,6 mm.

According to CTE:

- The deformation should be inferior or equal 28,6 $\leq$ L/300 = 15500/300 = 51,6 mm **OK**

6.2.4 Corbels

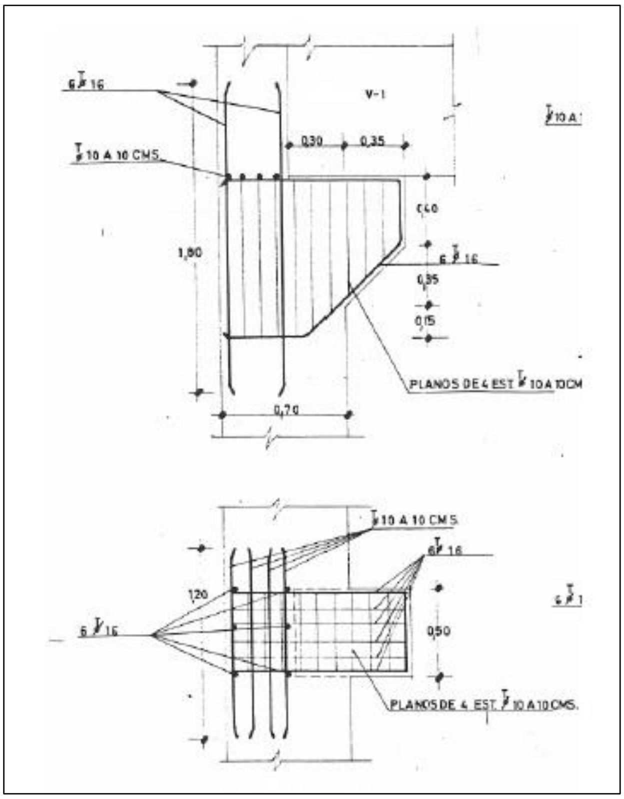
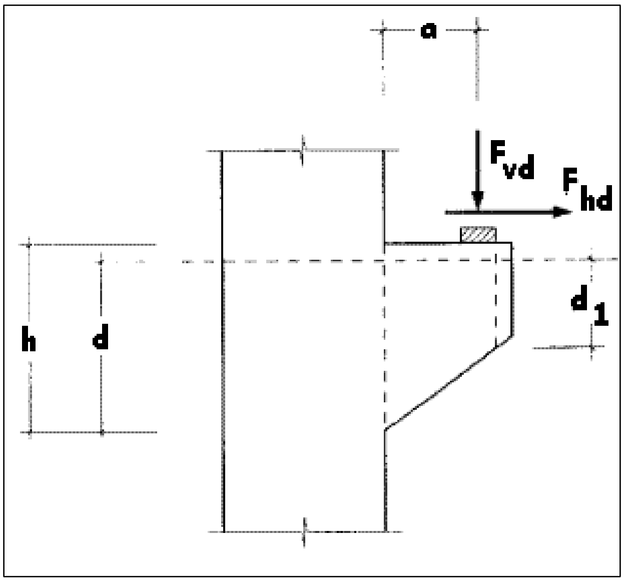


Figure 52. First Basement Corbel where beam V1 rests on.



6.2.4.1 Corbel verification for Beam V1 – Frame 240

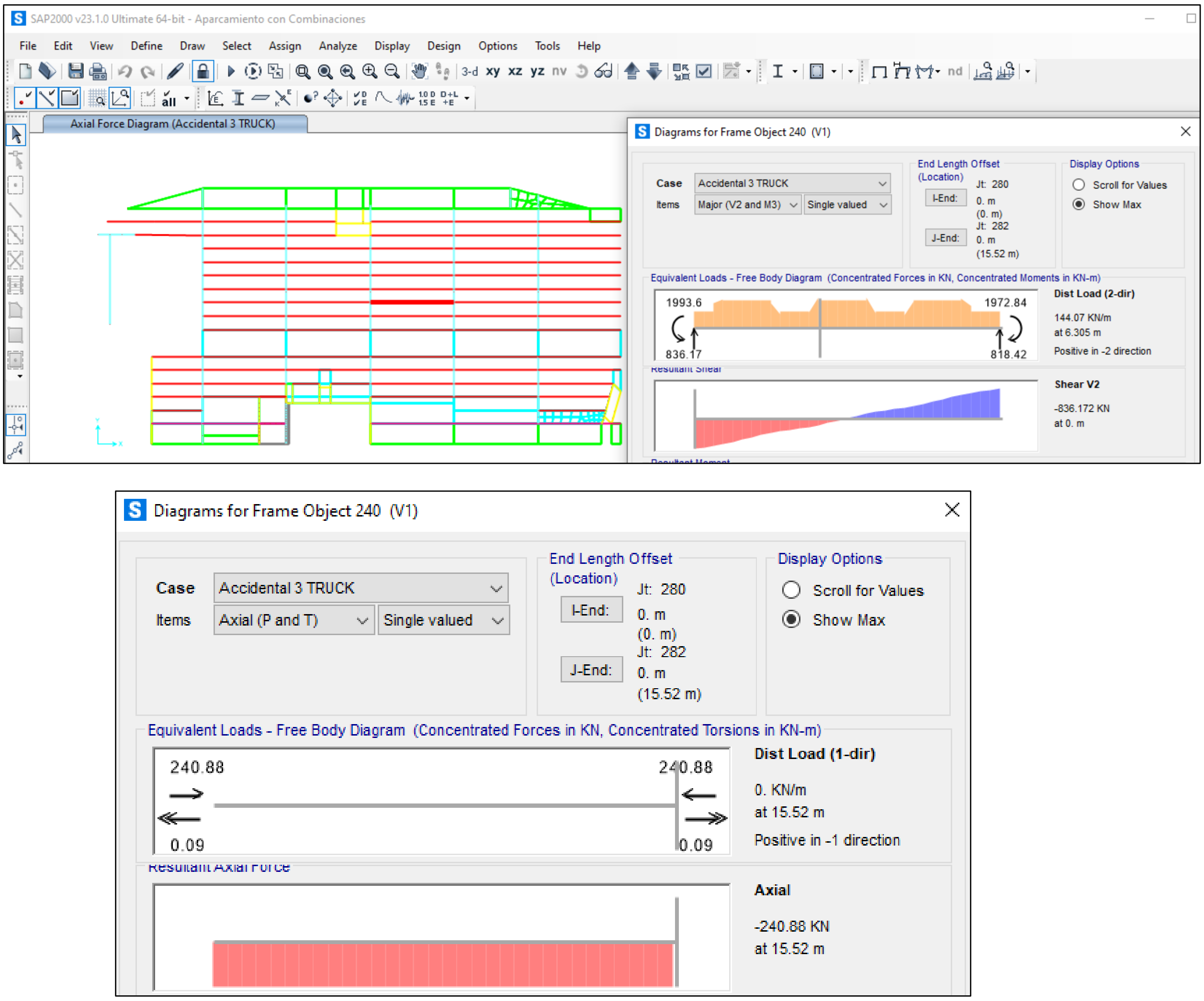


Figure 53. Shear and axial forces obtained from SAP2000.

The maximum shear and axial force values applied on the Corbel (Table 8. Shear forces according to each load combination.) are F_{Vd} = 836 KN, F_{hd} = 241/2 = 120,5 KN.

Corbels are considered short when “a” which is the distance between the point where the main vertical load applies and the section adjacent to the support, is less than or equal to the effective depth “d”.

The useful depth d1 measured at the outer edge of the area where the load is applied will be equal to or greater than 0.5d.

$$d \geqslant \frac{a}{0,85} \text{cotg}\theta$$

a) h = 0,40 + 0,35 + 0,15 = 0,90 m.

d = h – r = 0,90 – 0,03 = 0,87 m.

a = 0,30 m. $\text{cotg}\,\emptyset = 1,4.$

$d = 0,87 > \frac{0,3}{0,85} \times 1,4 = 0,494\,m$ **OK**

b) d1 = 0,75 m.

$0,5\,d = 0,5 \times 0,87 = 0,435$

d1 > 0,5 d **OK**

The corbel is considered as a short one.

<u>Tension ties calculation:</u>

- $T_{1d} = F_{vd}\text{tg}\,\theta + F_{hd}$ = 836 x 0,714 + 120,5 = 718,118 KN.

F_{yd} = 450/1,15 = 391,3 MPa < 400 MPa ok.

F_{yd} x A_s = 391,3 x 6 x 3,14 x $(16)^2$ /4= 471819 N = 471,82 KN.

$T_{1d} > A_s f_{yd}$. Fails

- $T_{2d} = 0,20F_{vd}$ = 0,20 x 836 = 167,40 KN.

$A_{se}f_{yd}$ = (2) x 4 x 3,14 x $(10)^2$/4 x 450/1,15 = 245739 N = 245,74 KN

$T_{2d} < A_{se}f_{yd}$. OK

The corbel does not meet the requirements and needs to be reinforced.

<u>Struts and Nodes:</u>

$$F_{vd}/(b\cdot c) \leq f_{1cd}$$

b, c: Plan dimensions of the support.

f_{1cd}: Compressive strength of concrete. $f_{1cd} = 0.70f_{cd}$ = 0,70 x 15,2/1,5 = 7,1 MPa.

$F_{vd}/(b\cdot c)$ = 836000/ (350 x 500) = 4,8 MPa.

$F_{vd}/(b\cdot c) \leq 7,1$ **OK**.

- According to structural code:

$$V_{Ed} \leq 0{,}5\ b_w\ d\ \nu\ f_{cd}$$

V_{Ed} = 836 KN < 0,5 x 500 x 870 x 0,56 x 15,2/1,5 = 1234240 N = 1234 KN **OK**.

6.2.4.2 Corbel verification for Beam V1 – Frame 225

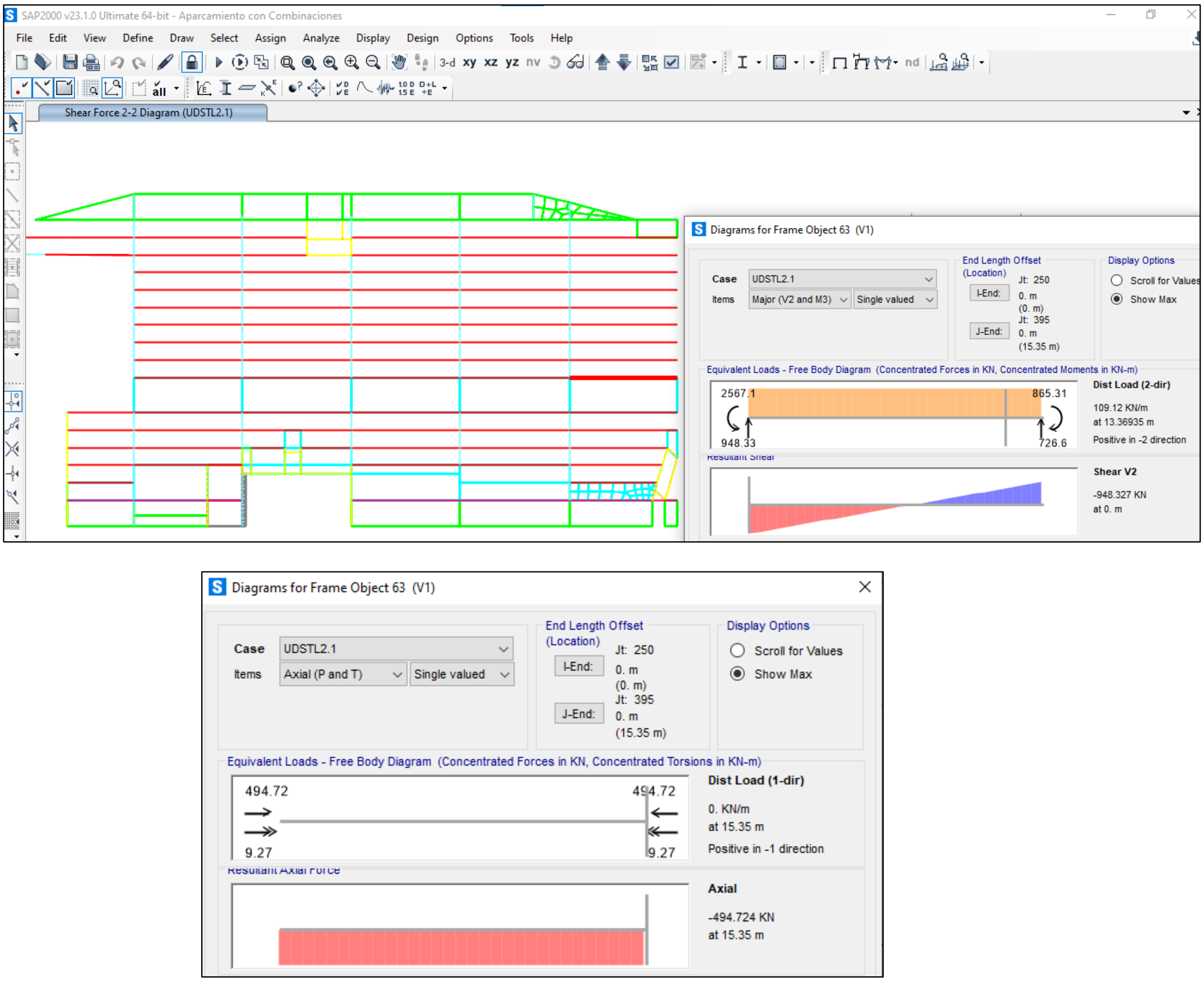


Figure 54. Shear and axial forces obtained from SAP2000.

The maximum shear and axial force value applied on the Corbel (Table 10. Shear forces according to each load combination.) are F_{Vd} = 1012 KN, F_{hd} = 495/2 = 247,5 KN, due to the loading combination Accidental 3 Trucks.

<u>Tension ties calculation:</u>

- $T_{1d} = F_{vd} \text{tg } \theta + F_{hd}$ = 1012 x 0,714 + 247,5 = 970 KN.

$F_{yd} \times A_s$ = 450/1,15 x 6 x 3,14 x $(16)^2$ /4= 471819 N = 471,82 KN.

$T_{1d} > A_s f_{yd}$. Fails

- T_{2d} = 0,20F_{vd} = 0,20 x 1012 = 202,4 KN.

$A_{se}f_{yd}$ = 2 x 4 x 3,14 x $(10)^2$/4 x 450/1,15 = 245739 N = 245,73KN.

$T_{2d} < A_{se}f_{yd}$. OK

The corbel does not meet the requirements and needs to be reinforced.

<u>Struts and Nodes:</u>

$$F_{vd}/(b\cdot c) \leq f_{1cd}$$

b, c: Plan dimensions of the support.

f_{1cd}: Compressive strength of concrete. f_{1cd} = 0.70f_{cd} = 0,70 x 15,2/1,5 = 7,1 MPa.

$F_{vd}/(b\cdot c)$ = 1012000/ (350 x 500) = 5,7 MPa.

$F_{vd}/(b\cdot c) \leq 7,1$ **OK**.

6.2.4.3 Corbel verification for Beam V1 – Frame 275

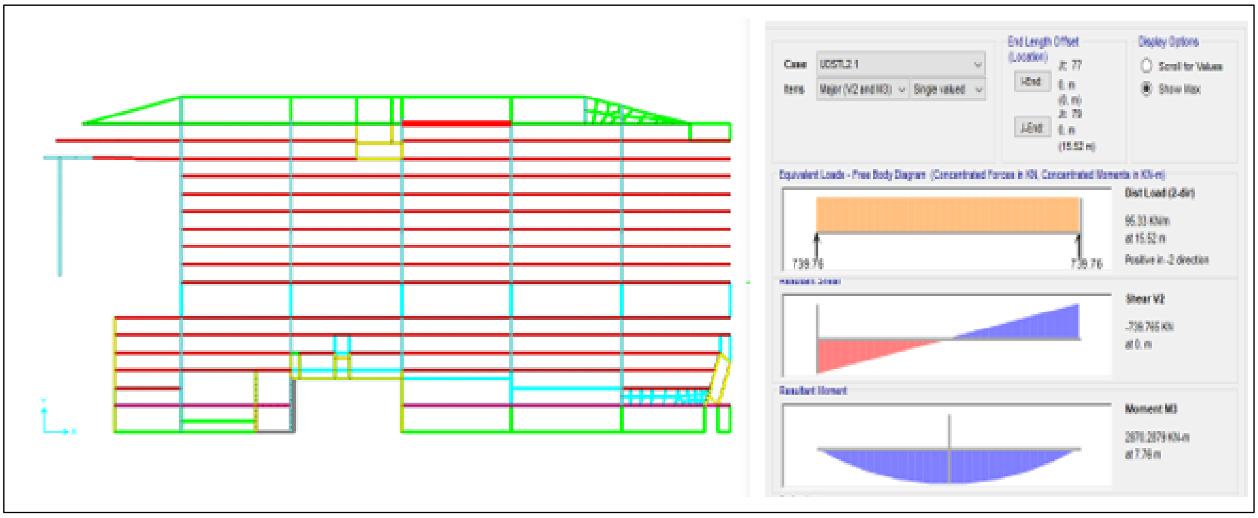


Figure 55. Shear and axial forces obtained from SAP2000.

The maximum shear and axial force value applied on the Corbel are F_{Vd} = 923 KN, F_{hd} = 423/2 = 211,5 KN, due to the loading combination Accidental 3 Trucks.

<u>Tension ties calculation:</u>

- $T_{1d} = F_{vd} \text{tg } \theta + F_{hd}$ = 923 x 0,714 + 211,5 = 870,52 KN.

F_{yd} x A_s = 450/1,15 x 6 x 3,14 x $(16)^2$ /4= 471819 N = 471,82 KN.

$T_{1d} > A_s f_{yd}$. Fails

- $T_{2d} = 0{,}20F_{vd}$ = 0,20 x 923 = 184,60 KN.

$A_{se}f_{yd}$ = 2 x 4 x 3,14 x $(10)^2$/4 x 450/1,15 = 245739 N = 245,73KN.

$T_{2d} < A_{se}f_{yd}$. **OK**

The corbel does not meet the requirements and needs to be reinforced.

<u>Struts and Nodes:</u>

$$F_{vd}/(b \cdot c) \leq f_{1cd}$$

b, c: Plan dimensions of the support.

f_{1cd}: Compressive strength of concrete. $f_{1cd} = 0.70 f_{cd}$ = 0,70 x 15,2/1,5 = 7,1 MPa.

$F_{vd}/(b \cdot c)$ = 923 000/ (350 x 500) = 5,3 MPa.

$F_{vd}/(b \cdot c) \leq 7{,}1$ **OK**.

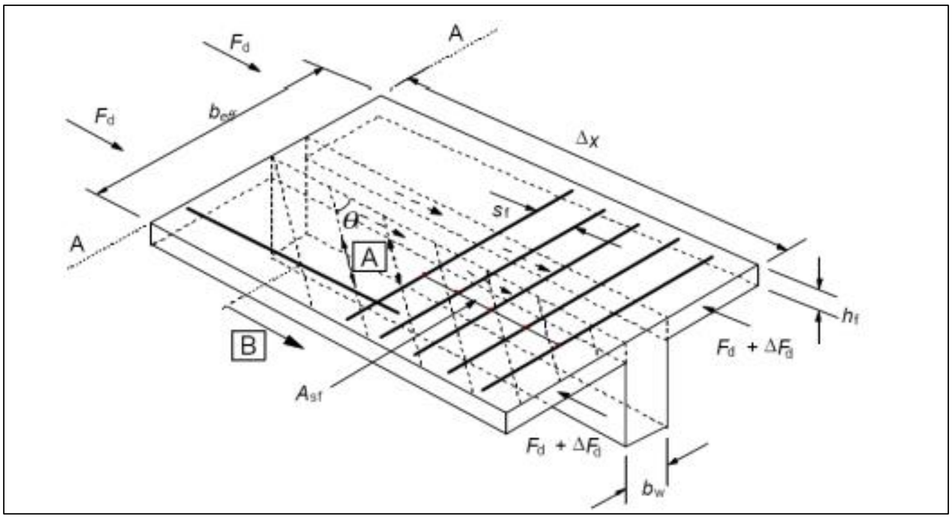
6.2.5 Shear stress

The connection between the wings and the web is subjected to a shearing stress.

- According to structural code Annex-19:

The shear stress, V_{Ed} occurring at the joint between the web and one side of the flange is given by the formula:

$$vEd = \Delta Fd/(hf \cdot \Delta x)$$



Where:

ΔF_d: Variation of the longitudinal force acting on the flange section in the distance Δx.

According to the structural technical code, the maximum value that can be accepted for Δx is half the distance between the null moment section and the maximum moment section.

6.2.5.1 Beam V1 – Frame 240

The maximum moment can be found in the middle of the beam, which mean at distance of the support 15,5/2 = 7,75 m.

Therefore Δx will be half of that distance = 7,75/2 = 3,875 m.

From Table 7 we can observe that the load combination Accidental 1 has the maximum moment value. Therefore, it is sufficient to only verify this case.

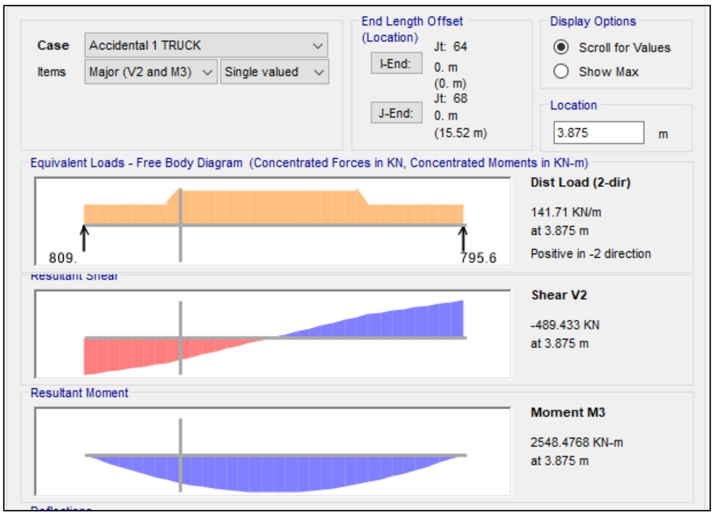


Figure 56. The moment at a distance 3,875 obtained from SAP2000

As we can observe in the Figure 56, the moment corresponding to the distance of 3,875 is 2549 KN.m

$F_d + \Delta F_d$ = 2549/d = 2549/1,07 = 2382 KN.

ΔF_d = 2382 – 0 = 2382 KN. h_f = 0,20 m.

V_{Ed} = 2382000/ (200 x 3875) = 3,07 MPa.

A_{sw} = 2*5*$\frac{\pi\ *8^2}{4}$ = 502,4 mm^2.

V1 = 0,6 (1-fck/250) = 0,6*(1-17,7/250) = 0,56

S = 200 mm. $\emptyset f\ = 45°$

V = 0,56

$$A_{sf} f_{yd}/s_f \geq v_{Ed} \cdot h_f/cot\theta_f$$

502,4 x (450/1,15)/200 = 983 N/mm > 3,07 x 200/1 = 614 N/mm **OK**.

To prevent failure of the wing compression struts, the following condition must be met.

condition:

$$v_{Ed} \leq v f_{cd} sen\theta_f cos\theta_f$$

3,07 < 0,56 x 17,7/1,5 x 0,707 x 0,707 =3,3 **OK**.

The Beam meets the requirements to resist the shear stress.

6.2.5.2 Beam V1 – Frame 225

The maximum moment can be found in the middle of the beam, which mean at distance of the support 15,5/2 = 7,75 m.

Therefore Δx will be half of that distance = 7,75/2 = 3,875 m.

From Table 9Table 7 we can observe that the load combination Accidental 1 has the maximum moment value. Therefore, it is sufficient to only verify this case.

The moment corresponding to the distance of 3,875 is 3062 KN.m

$F_d + \Delta F_d$ = 3062 /d = 3062 /1,07 = 2862 KN.

ΔF_d = 2862 – 0 = 2862 KN. h_f = 0,20 m.

V_{Ed} = 2862000/ (200 x 3875) = 3,69 MPa.

$A_{sw} = 2*5*\frac{\pi *8^2}{4}$ = 502,4 mm^2.

V1 = 0,6 (1-fck/250) = 0,6*(1-17,7/250) = 0,56

S = 200 mm. $\emptyset f = 45°$

V = 0,56

$$A_{sf}\, f_{yd}/s_f \geq v_{Ed} \cdot h_f/cot\theta_f$$

502,4 x (450/1,15)/200 = 983 N/mm > 3,69 x 200/1 = 739 N/mm **OK**.

To prevent failure of the wing compression struts, the following condition must be met.

condition:

$$v_{Ed} \leq v f_{cd} sen\theta_f cos\theta_f$$

3,69 > 0,56 x 17,7/1,5 x 0,707 x 0,707 =3,3 **Fails**.

The Beam does not meet the requirements and need to be strengthened.

6.2.5.3 Beam V2 – Frame 275

The maximum moment can be found in the middle of the beam, which mean at distance of the support 15,5/2 = 7,75 m.

Therefore Δx will be half of that distance = 7,75/2 = 3,875 m.

From Table 11 we can observe that the load combination Accidental 1 has the maximum moment value. Therefore, it is sufficient to only verify this case.

The moment corresponding to the distance of 3,875 is 2805 KN.m

$F_d + \Delta F_d$ = 2805 /d = 2805 /1,07 = 2622 KN.

ΔF_d = 2622 – 0 = 2622 KN. h_f = 0,20 m.

V_{Ed} = 2622000/ (200 x 3875) = 3,38 MPa.

$A_{sw} = 2*6*\frac{\pi\ *8^2}{4} = 604{,}2\ mm^2$.

V1 = 0,6 (1-fck/250) = 0,6*(1-17,7/250) = 0,56

S = 200 mm. $\emptyset f\ = 45°$

V = 0,56

$$A_{sf}\, f_{yd}/s_f \geq v_{Ed} \cdot h_f/cot\theta_f$$

604,2 x (450/1,15)/200 = 1182 N/mm > 3,38 x 200/1 = 676 N/mm **OK**.

To prevent failure of the wing compression struts, the following condition must be met.

condition:

$$v_{Ed} \leq v f_{cd} sen\theta_f cos\theta_f$$

3,38 > 0,56 x 17,7/1,5 x 0,707 x 0,707 =3,3 **Fails**.

The Beam does not meet the requirements and need to be strengthened.

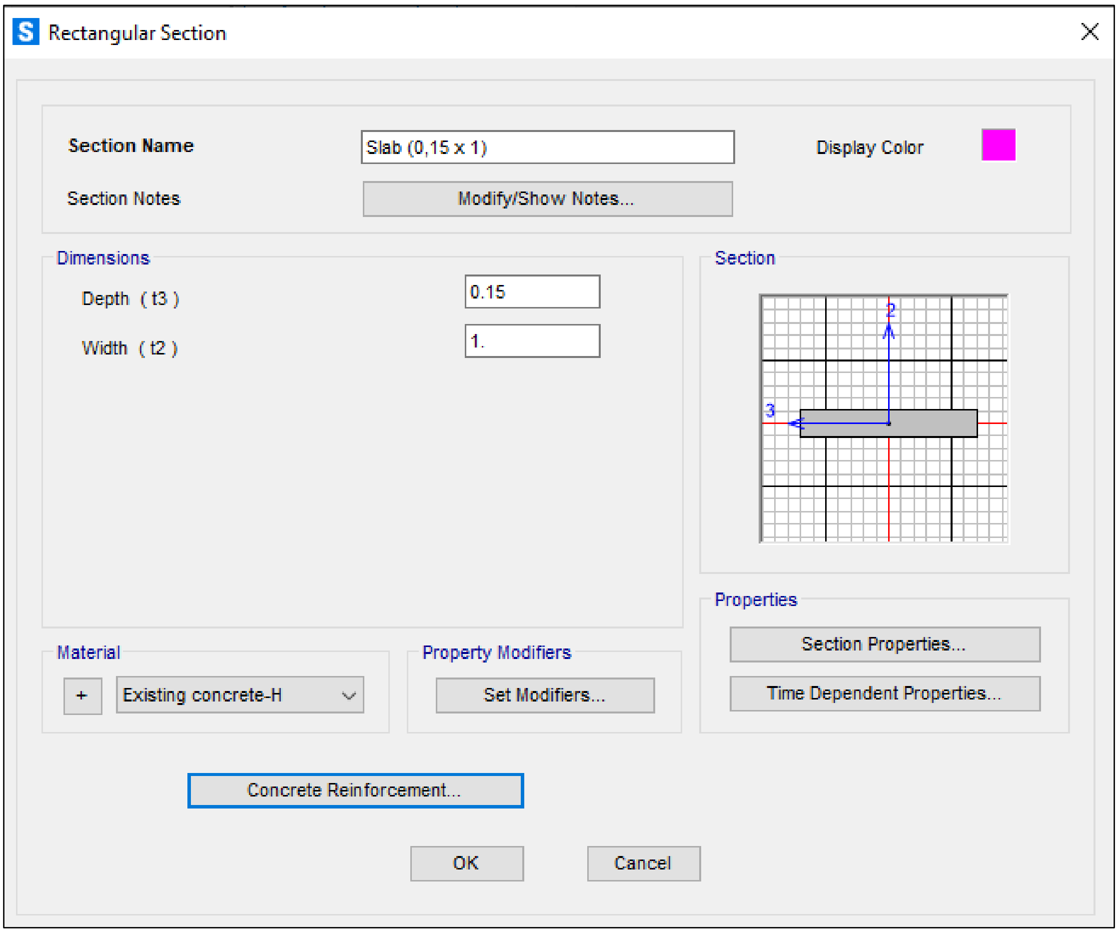
6.3 Second Basement

6.3.1 Slabs

6.3.1.1 Bending Moment Verification

The load combinations are:

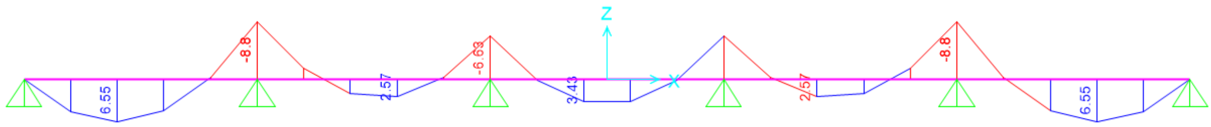
1. Service: 1,00 (Dead Load + Live Load) + 1,00 Live Load
2. UDSTL1: 1,35 (Dead Load + Live Load) + 1,5 Live Load
3. UDSTL2.1: 1,35 (Dead Load + Live Load) + 1,5 Live Load + 1,5 Summer
4. UDSTL2.2: 1,35 (Dead Load + Live Load) + 1,5 Live Load + 1,5 Snow + 1,5 Winter
5. UDSTL2.3: 0,8 (Dead Load + Live Load) + 1,5 Winter



All the loads are considered in the calculation of the slab including the dead loads, live loads, summer, winter.

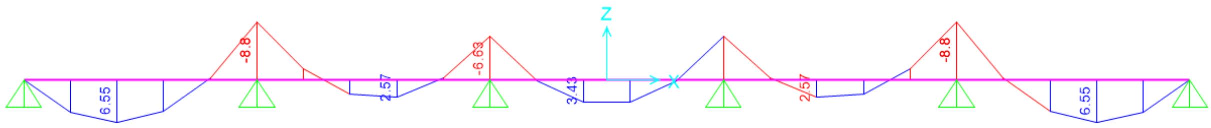
By introducing all the forces to SAP2000 it gives us the following results:

1. Moment diagram for USTDL 1



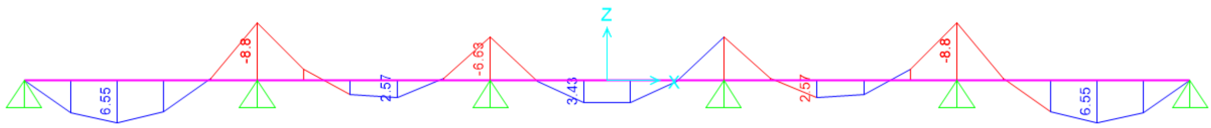
Md,max+ = 6,55 KN.m. Md,max- = -8,8 Kn.m.

2. Moment diagram for USTDL 2.1



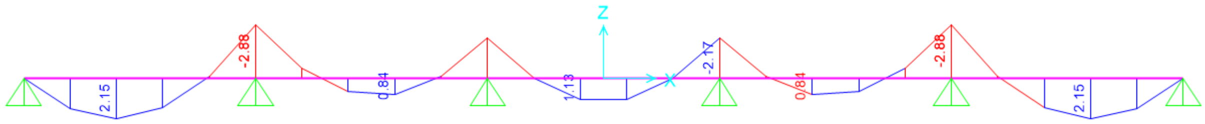
Md,max+ = 6,55 KN.m. Md,max- = -8,8 Kn.m.

3. Moment diagram for USTDL 2.2



Md,max+ = 6,55 KN.m. Md,max- = -8,8 Kn.m.

4. Moment diagram for USTDL 2.3



Md,max+ = 2,15 KN.m. Md,max- = -2,88 Kn.m.

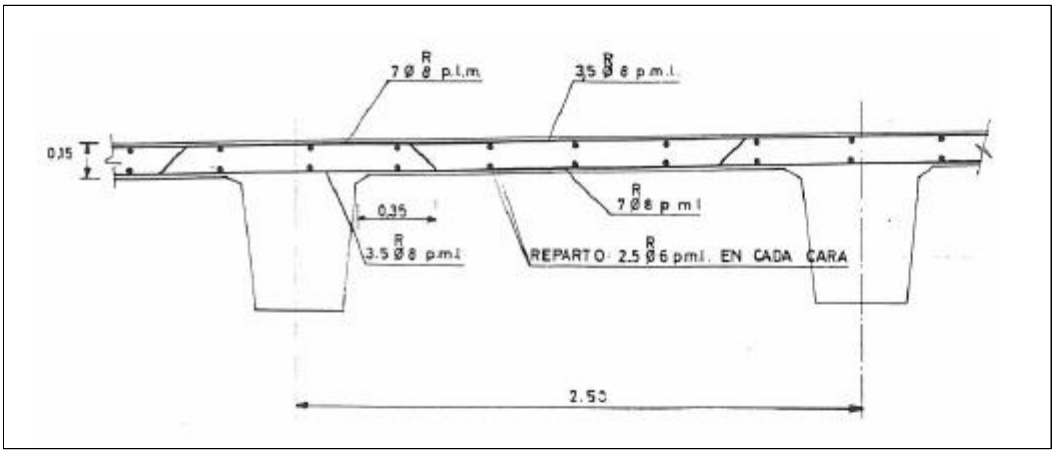


Figure 57. Slab Section.

γc = 1,5 the partial safety factor for Concrete.

γs= 1,15 the partial safety factor for Steel.

As superior (mm^2) = 7 Φ 8 = 352 mm^2.   b = 1 m.

h = 0,15 m.   r = 0.03 m (concrete cover to reinforcement)

d = h – r = 0,15 – 0,03 = 0,12 m.

Fck = 17,7 MPa >> Fcd = γc * Fck = 11,8 MPa.

Fyk = 450 Mpa >> Fyd = γs * Fd = 391,3 MPa.

As superior (mm^2) = 7 Φ 8 = 352 mm^2.

As a result of the calculation, the positive moment resistance is

$$* \ Mu = 16{,}1 \ KN.m$$

As inferior (mm^2) = 3,5 Φ 8 = 176 mm^2.

The negative moment resistance:

$$* \ Mu = 8{,}1 \ KN.m$$

Load Combination	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3
Md,max+ (KN.m)	6,55	6,55	6,55	2,88
Reduced Resistance	16,1	16,1	16,1	16,1

moment Mu+ (KN.m)				
Safety factor	2,45	2,45	2,45	7,49

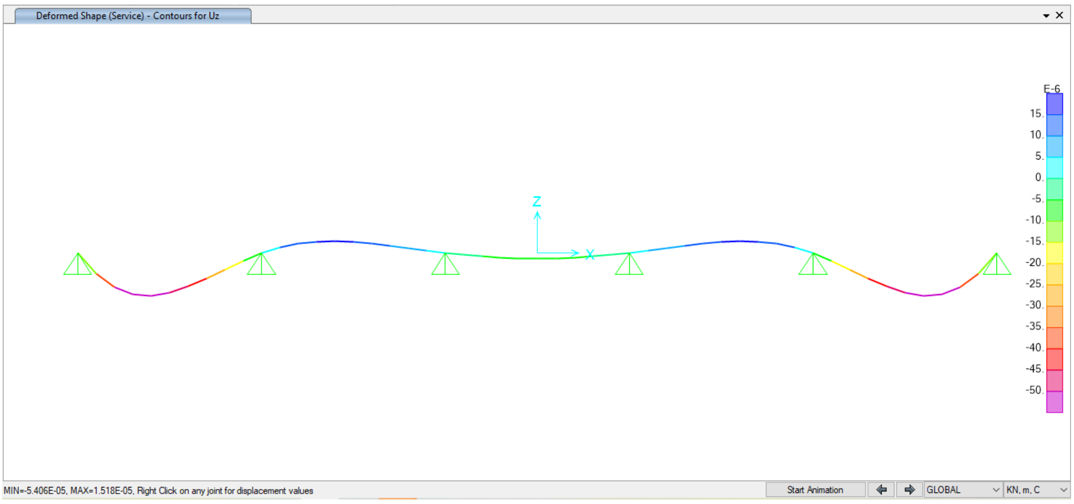
The reduced resistance moment is greater than all the positive bending moments, therefore the slab meets the requirements.

Load Combination	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3
Md,max- (KN.m)	-8,8	--8,8	-8,8	-2,88
Reduced Resistance moment Mu- (KN.m)	8,1	8,1	8,1	8,1
Safety factor	0,92	0,92	0,92	2,81

The reduced resistance moment is greater than all the negative bending moments, therefore the slab meets the requirements.

6.3.1.2 Deflection

Load Combination	Service	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3
Deflections (mm)	0,22	0,24	0,24	0,24	0,1



The maximum deflection is of 0,054 mm in the first span of both sides.

According to CTE:

- The deformation should be inferior or equal ≤ L/300 = 2500/300 = 8,33 mm **OK**

6.3.2 Columns

In this report, we will try to analyze the most significant columns, which are loaded with the most severe loads, in order to obtain the most unfavorable cases and compare them with the own element resistant forces and with the allowable limits.

The columns of the first basement of the parking structure have a rectangular section (0,5 x 0,55) m.

6.3.2.1 Column 398

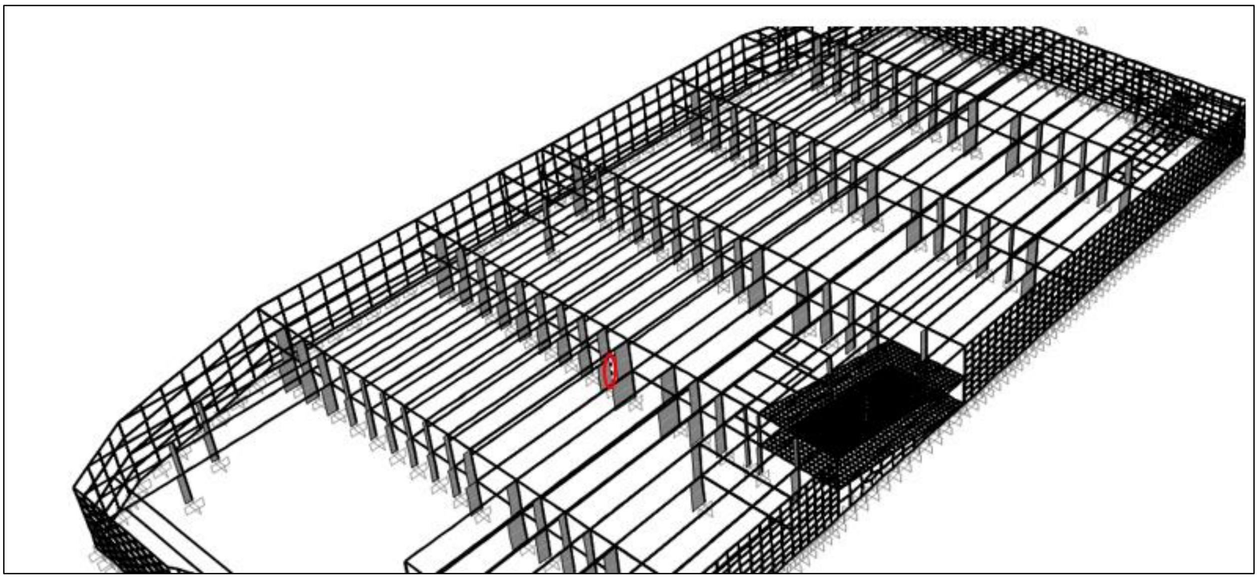


Figure 58. Column 398 Location.

	UDSTL1		UDSTL2.1		UDSTL2.2		UDSTL2.3	
Point	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)
1) Mmax+	13	2462	120	2400	162	2482	155	1088
2) Mmax-	8	2462	153	2400	142	2482	133	1088

Table 13. Forces applied to the Column.

The column is subjected to an axial and moment. Therefore, the interaction diagram will be obtained from the program SAP2000.

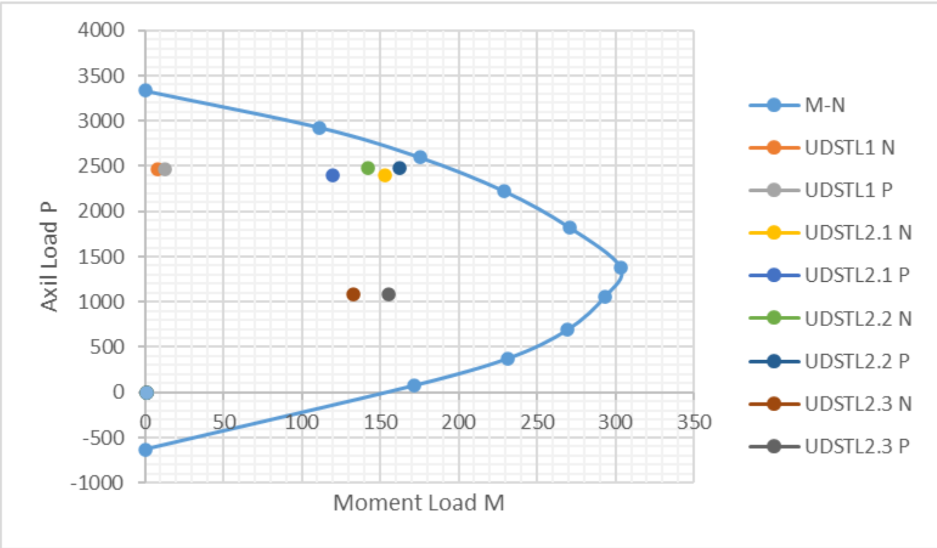


Figure 59. Interaction Diagram obtained from SAP2000 for all the combinations.

All points are inside the interaction diagram. Therefore, the column can resist the forces and loads applied to it.

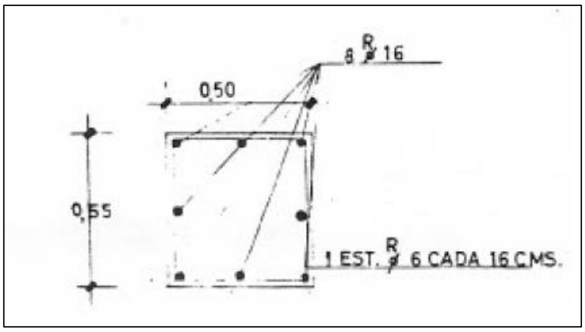


Figure 60. Second Basement Column Section.

A verification should be made to check the minimum reinforcement ratio for the column.

$$As \geq \frac{4}{1000} Ac$$

As = 8Φ16 = 1608 mm^2 > 1100 mm^2. OK

6.3.2.2 Column 278

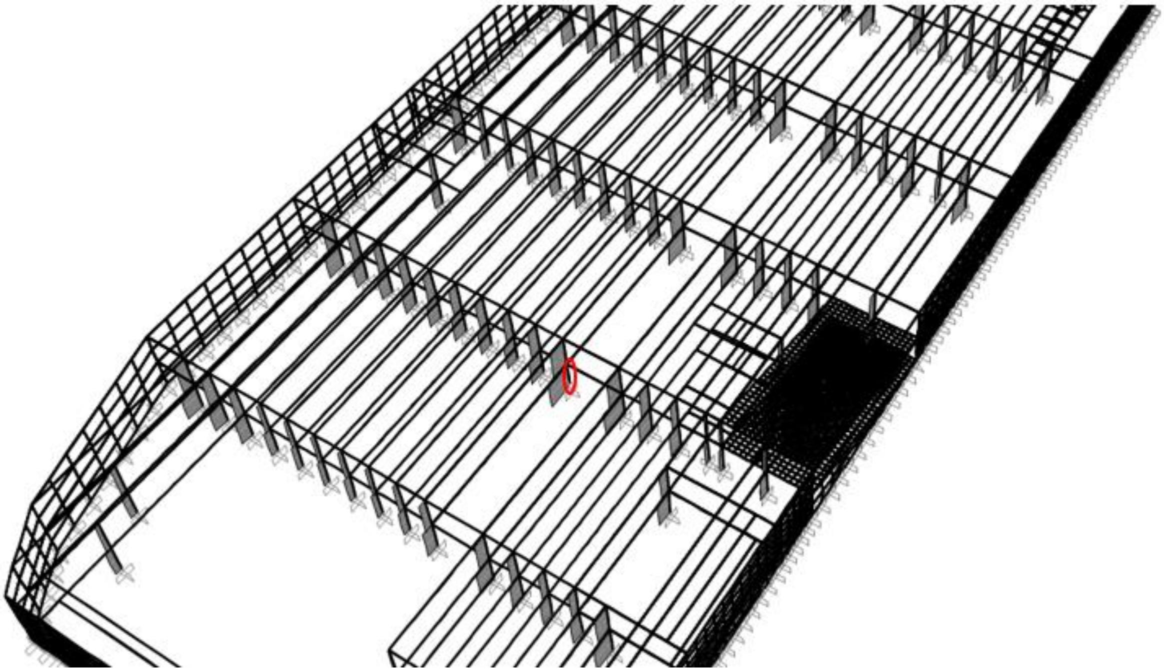


Figure 61. Column 278 Location.

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	UDSTL1		UDSTL2.1		UDSTL2.2		UDSTL2.3	
Point	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)
1) Mmax+	2	3906	430	3899	493	3991	491	1671
2) Mmax-	13	3906	515	3746	441	3991	427	1671

Table 14. Forces applied to the Column

The column is subjected to an axial and moment. Therefore, the interaction diagram will be obtained from the program SAP2000.

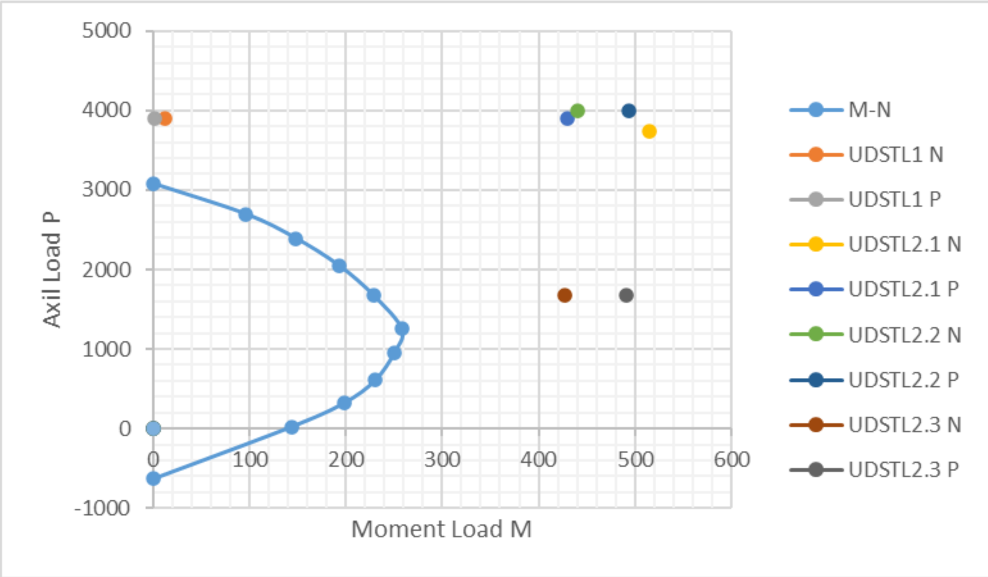

Figure 62. Interaction Diagram obtained from SAP2000.

All points are outside the interaction diagram. Therefore, the column doesn't meet the requirements and needs to be strengthened.

6.3.2.3 Column 440

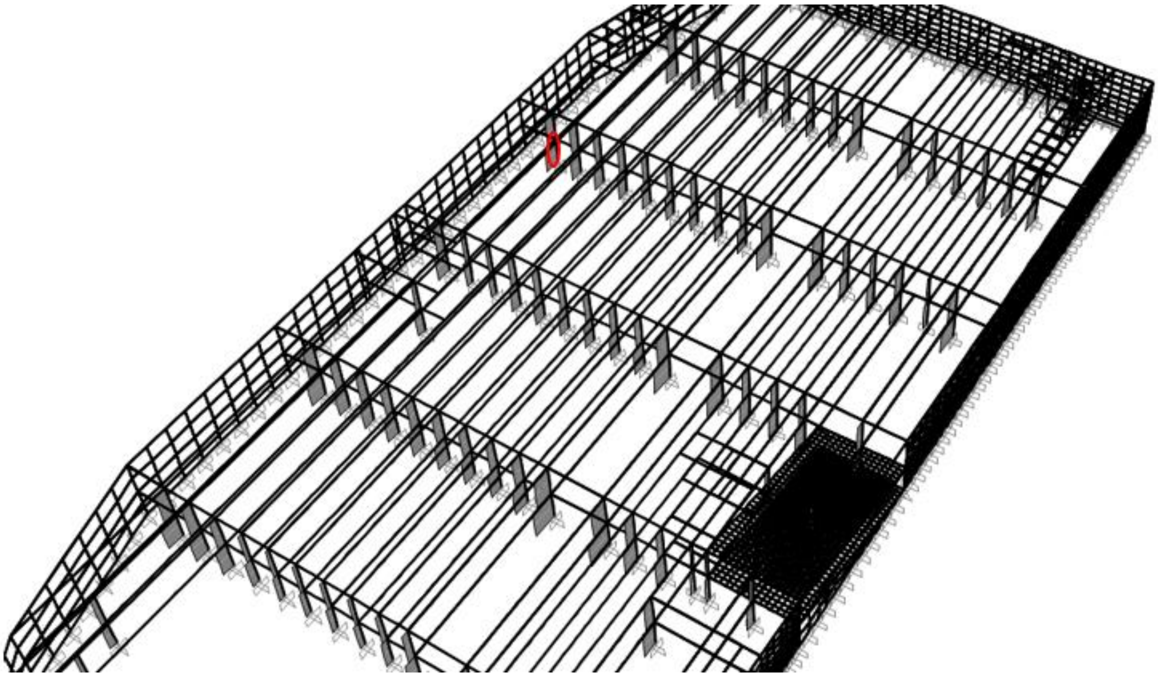

Figure 63. Column 440 Location.

	UDSTL1		UDSTL2.1		UDSTL2.2		UDSTL2.3	
Point	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)	Md(KN.m)	Nd (KN)
1) Mmax+	8	2878	38	2930	1	2907	2	1225
2) Mmax-	20	2878	51	2930	15	2907	4	1225

Table 15. Forces applied to the Column.

The column is subjected to an axial and moment. Therefore, the interaction diagram will be obtained from the program SAP2000.

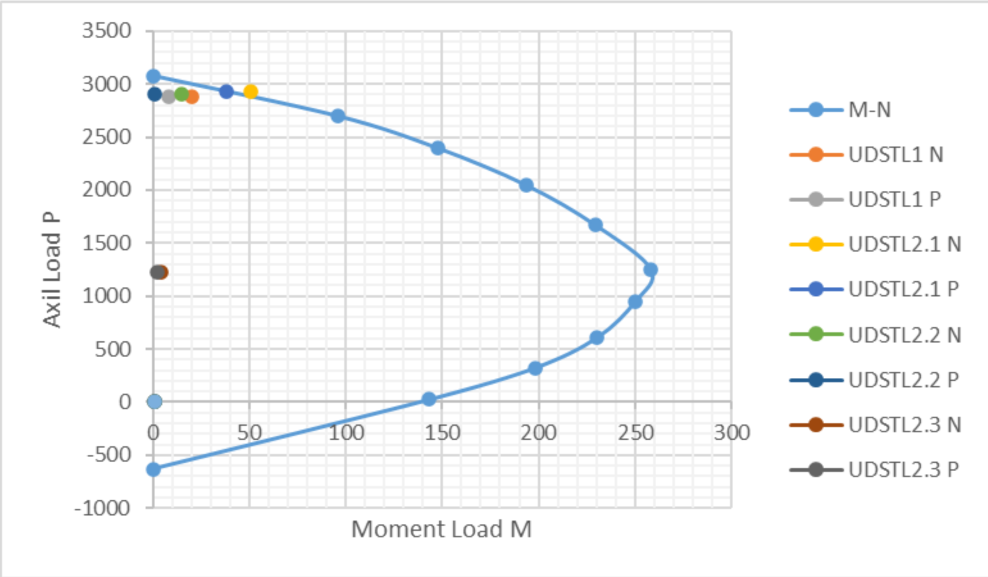

Figure 64. Interaction Diagram obtained from SAP2000.

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All points are inside the interaction diagram expect for load combination UDSTL2.1. Therefore, the column doesn't meet the requirements and needs to be strengthened.

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6.3.3 Beams

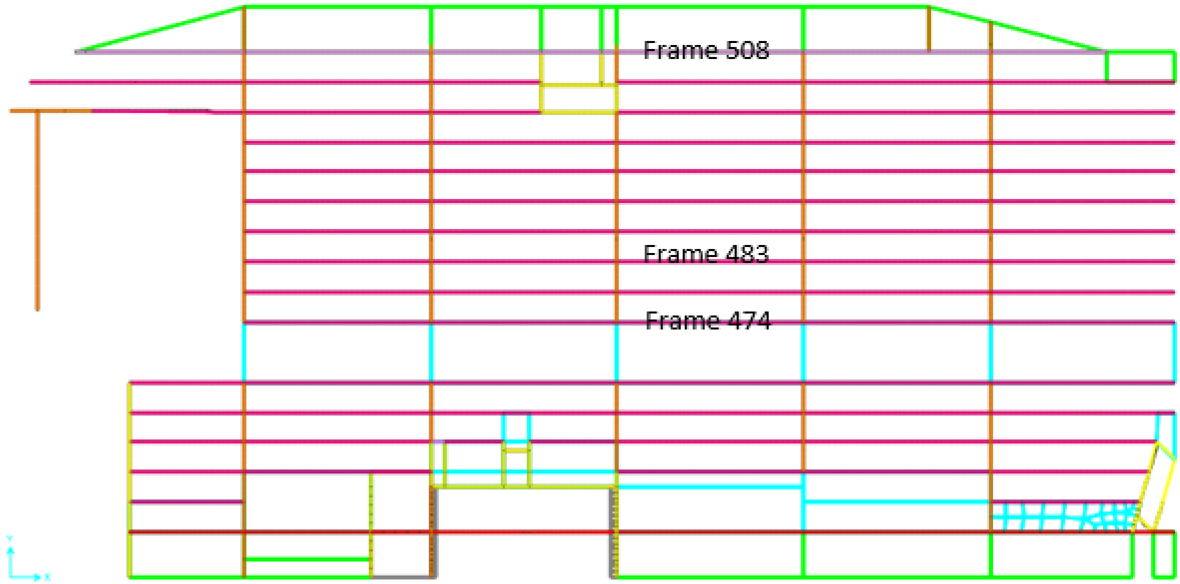


Figure 65. Plan view of the second Basement (X-Y)

It is worth noting that the beams are classified into three groups that are more significantly important to analyze and verify:

- Beam 483 type V25 this beam has a tributary area of 2,5/2 + 2,5/2 = 2,5 m.
- Beam 474 type V25 has been chosen due to its greater tributary area of a 2,5/2 + 5/2 = 3,75 m
- Beam 508 type V26: this beam has a tributary area of 2,5/2 + 3,75/2 = 3,125 m.
  This beam is located at the end of the beam's distribution, next to the north wall.

6.3.3.1 Beam 483

6.3.3.1.1 Bending Moment Verification

γc = 1,5 the partial safety factor for Concrete.

γs= 1,15 the partial safety factor for Steel.

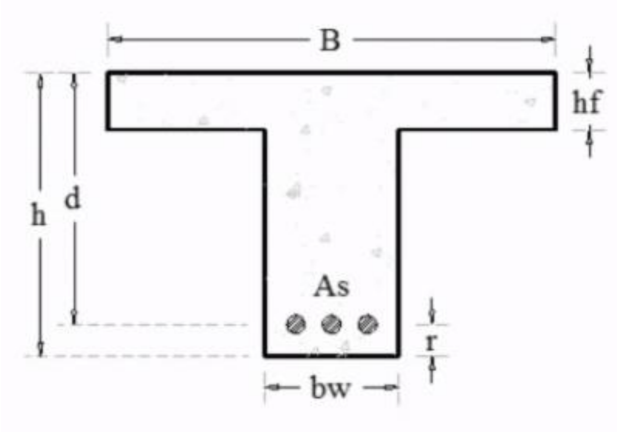


Figure 66 Beam 240 Location and bending moment

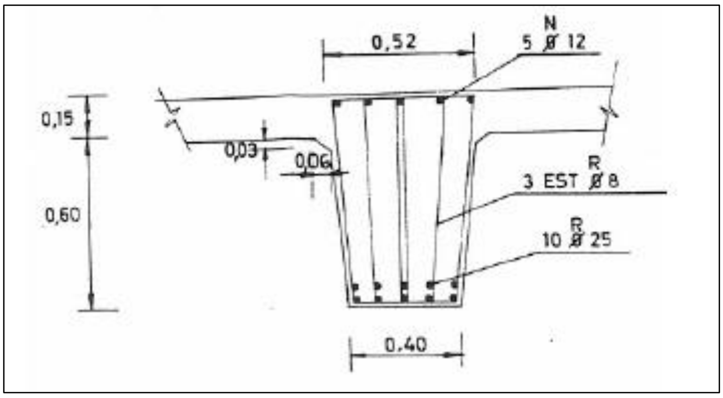


Figure 67. Section T-Beam V25.

As (mm^2) = 10 Φ 25 = 4906 mm^2. bf = 2,5 m.

h = 0,75 m. r = 0.03 m (concrete cover to reinforcement)

d = h – r = 0,75 – 0,03 = 0,72 m. hf = 0,15 m.

Fck = 17,7 MPa >> Fcd = γc * Fck = 11,8 MPa.

Fyk = 450 Mpa >> Fyd = γs * Fd = 391,3 MPa.

T = 4906 * 450 = 2208 KN.

$$C = 0{,}85 \times Fc \times a \times b$$

T = C

Through this equation, we can calculate "a"

$$a = \frac{2208}{0{,}85 * 17{,}7 * 1000 * 2{,}5} = 0{,}059\ m$$

a < hf = 0,2 m.

- The depth of the concrete block is at the flange. Therefore, we can consider our beam as if it was rectangular. we can see that this is just the same as if we had a rectangular section with a width of bf and depth of Steel equal to D.

In order to calculate the ultimate resistant moment, we apply the following equations:

$Mu = fcd.\ b.y.(d\text{-}y/2)$

$$Mu = 11{,}8 * 2500 * 59 * \left(720 - \frac{59}{2}\right) = 1202000\ N.mm = 1202\ KN.m$$

Load Combination	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3
Md,max+ (KN.m)	1127	1127	1146	400
Reduced Resistance moment Mu+ (KN.m)	1202	1202	1202	1202
Safety factor	1,06	1,06	1,05	3,00

Table 16. Beam 483 Safety Factor

The reduced resistance moment is greater than all the positive bending moments, therefore the beam meets the requirements.

6.3.3.1.2 Shear Verification

Load Combination	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3
V_{Ed},max+ (KN)	290	290	297	103

Table 17. Shear forces according to each load combination.

Definitions:

V_{Ed} - Applied shear force.

$V_{Rd,c:}$ Resistance of member without shear reinforcement.

$V_{Rd,s}$: Resistance of members governed by the yielding of shear reinforcement.

$V_{Rd,max}$: Resistance of member governed by the crushing of compression struts.

The formulas for shear concrete resistance with reinforcement are given by:

$$\boxed{V_{Rd,s} = \frac{A_{sw}}{s} z \, f_{ywd} \cot\theta}$$

$$\boxed{V_{Rd,max} = \frac{\alpha_{cw} \, b_w \, z \, \nu_1 \, f_{cd}}{\cot\theta + \tan\theta}}$$

Bw: is the minimum width.

A_{sw}: Area of the shear reinforcement.

$$\boxed{A_{sw} = 2.n.\frac{\pi.D^2}{4}}$$

f_{yd}: design yield strength = f_{yk}/1,15.

f_{cd}: design compressive strength f_{ck}/1,5.

α_{cw} = 1.0 Coefficient for stress in compression chord.

v1: strength reduction factor concrete cracked in shear v1 = 0.6(1-fck/250).

Θ: angle between the concrete compression strut and the beam axis= 45◦

Shear reinforcement diameter (D) (mm): 8 mm

Spacing between stirrups (s) (mm): 200

Number of stirrups by section (n): 3

While:

$A_{sw} = 2*3*\frac{\pi \ *8^2}{4} = 301{,}44 \text{ mm}^2$.

V1 = 0,6 (1-fck/250) = 0,6*(1-17,7/250) = 0,56

$$V_{Rd,s} = \frac{A_{sw}}{s} z \, f_{ywd} \cot\theta$$

$V_{Rd,s}$ = 382 KN > V_{Ed} = 297 KN **OK.**

$$V_{Rd,max} = \frac{\alpha_{cw} \, b_w \, z \, v_1 \, f_{cd}}{\cot\theta + \tan\theta}$$

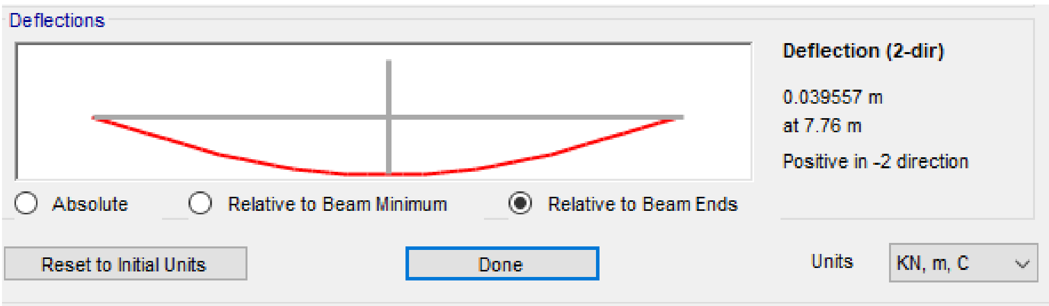
$V_{Rd,max}$ = 856,4 KN > V_{Ed} = 297 KN **OK.**

- Safety factor: $\frac{VRd,s}{VEd} = \frac{382}{297} = 1{,}286 > 0{,}9$ **OK.**

The beam meets the requirements and can resist the shear forces applied.

6.3.3.1.3 Deflection

Load Combination	Service	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3
Deflections (mm)	28	39,1	39,1	39,5	14



The worst combination is UDSTL2.2 which has a deflection of 39,5 mm.

According to CTE:

- The deformation should be inferior or equal 39,5 $\leq$ L/300 = 15500/300 = 51,6 mm **OK**

6.3.3.2 Beam 474

6.3.3.2.1 Bending Moment Verification

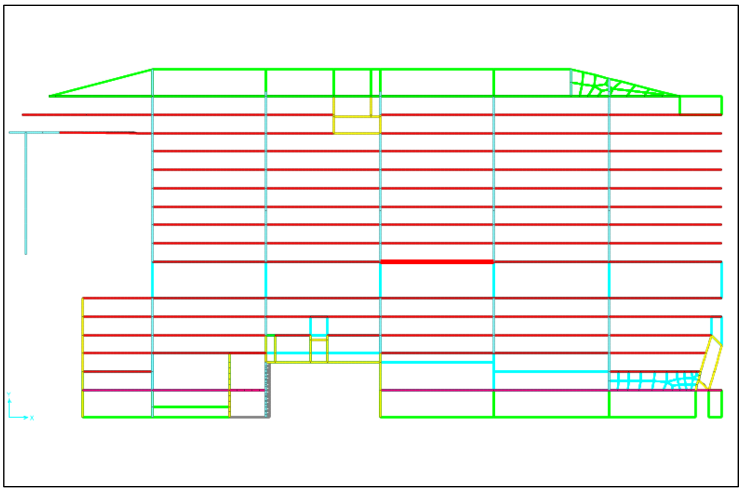


Figure 68. Beam 474 Location and bending moment

Load Combination	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3
Md,max+ (KN.m)	1378	1378	1410	415
Reduced Resistance moment Mu+ (KN.m)	1202	1202	1202	1202
Safety factor	0,87	0,87	0,85	2,89

Table 18. Beam 474 Safety Factor

The reduced resistance moment is inferior to the positive bending moments; therefore, the beam doesn't meet the requirements and need to be strengthened.

6.3.3.2.2 Shear Verification

Load Combination	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3
V_{Ed},max+ (KN)	355	355	359	107
VRd,s	382	382	382	382
Safety Factor	1,07	1,07	1,08	3,57

Table 19. Shear forces according to each load combination.

6.3.3.2.3 Deflection

Load Combination	Service	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3
Deflections (mm)	34	48,3	48,3	49,1	14,5

The worst combination is UDSTL2.2 which has a deflection of 49,1 mm.

According to CTE:

- The deformation should be inferior or equal 49,1 $\leq$ L/300 = 15500/300 = 51,6 mm **OK**

6.3.3.3 Beam 508

6.3.3.3.1 Bending Moment Verification

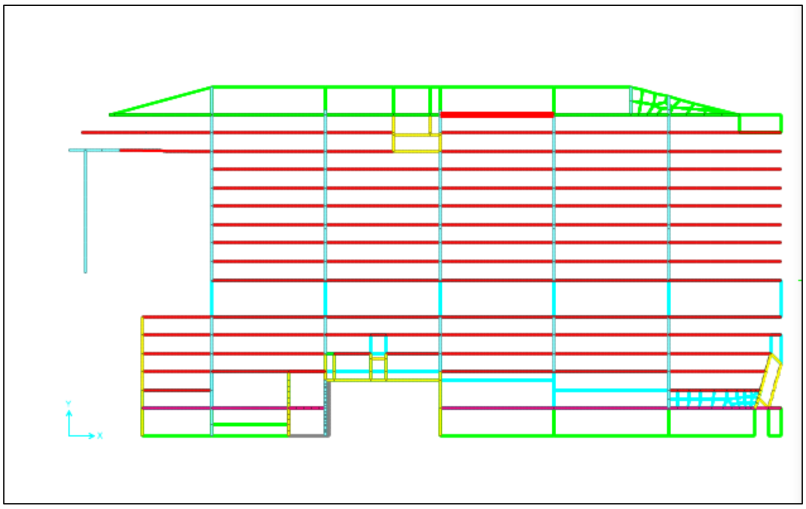


Figure 47 shows the Beam location.

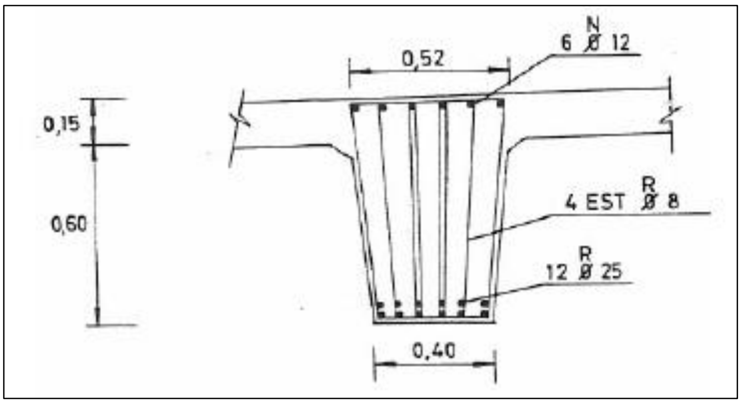


Figure 69. Section Beam 508 type V26.

γc = 1,5 the partial safety factor for Concrete.

γs= 1,15 the partial safety factor for Steel.

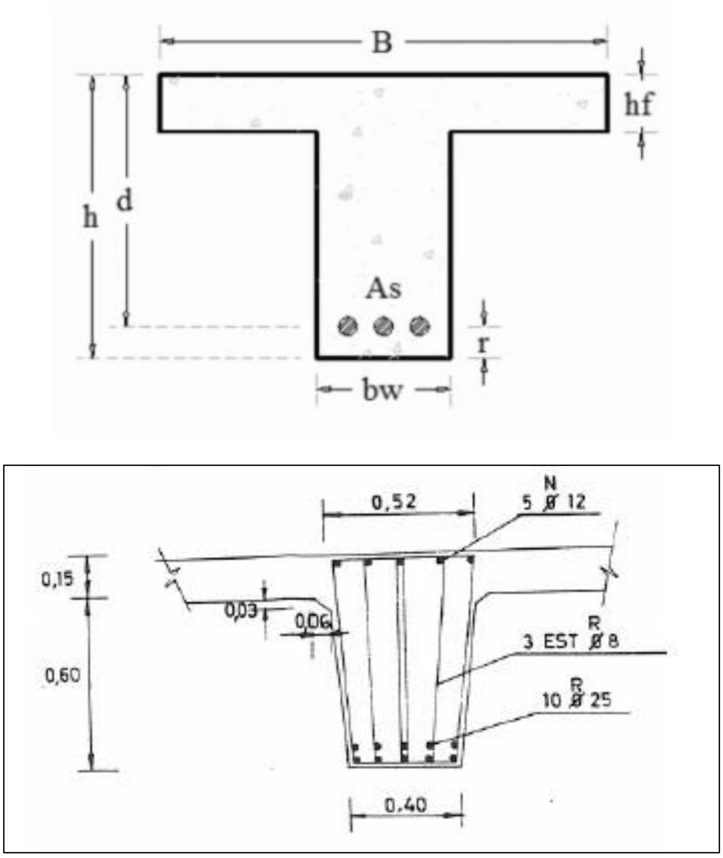


Figure 70. Section T-Beam V25.

As (mm^2) = 12 Φ 25 = 5888 mm^2. bf = 2,5 m.

h = 0,75 m. r = 0.03 m (concrete cover to reinforcement)

d = h – r = 0,75 – 0,03 = 0,72 m. hf = 0,15 m.

Fck = 17,7 MPa >> Fcd = γc * Fck = 11,8 MPa.

Fyk = 450 Mpa >> Fyd = γs * Fd = 391,3 MPa.

T = 5888 * 450 = 2649 KN.

$$C = 0{,}85 \times Fc \times a \times b$$

T = C

Through this equation, we can calculate “a”

$$a = \frac{2649}{0{,}85 * 17{,}7 * 1000 * 2{,}5} = 0{,}07\ m$$

a < hf = 0,2 m.

- The depth of the concrete block is at the flange. Therefore, we can consider our beam as if it was rectangular. we can see that this is just the same as if we had a rectangular section with a width of bf and depth of Steel equal to D.

In order to calculate the ultimate resistant moment, we apply the following equations:

$$Mu = fcd.\ b.y.(d\text{-}y/2)$$

$$Mu = 11{,}8 * 2500 * 70 * \left(720 - \frac{70}{2}\right) = 1415000\ N.mm = 1415\ KN.m$$

Load Combination	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3
Md,max+ (KN.m)	1252	1252	1261	407
Reduced Resistance moment Mu+ (KN.m)	1415	1415	1415	1415
Safety factor	1,13	1,13	1,12	3,47

Table 20. Beam 508 Safety Factor

The reduced resistance moment is greater than all the positive bending moments, therefore the beam meets the requirements.

6.3.3.3.2 Shear Verification

Load Combination	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3
V_{Ed},max+ (KN)	323	323	325	105
VRd,s	510	510	510	510
Safety Factor	1,58	1,58	1,57	4,85

Table 21. Shear forces according to each load combination.

$A_{sw} = 2*4*\frac{\pi\ *8^2}{4} = 401{,}92\ mm^2$.

V1 = 0,6 (1-fck/250) = 0,6*(1-17,7/250) = 0,56

$$\boxed{V_{Rd,s} = \frac{A_{sw}}{s} z\, f_{ywd} \cot\theta}$$

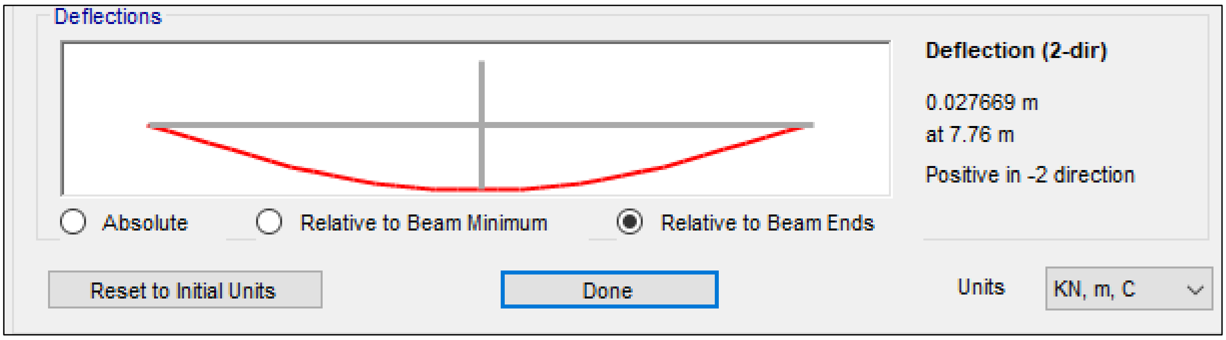
$V_{Rd,s}$ = 510 KN > V_{Ed} = 325 KN **OK.**

$$\boxed{V_{Rd,max} = \frac{\alpha_{cw}\, b_w\, z\, v_1\, f_{cd}}{\cot\theta + \tan\theta}}$$

$V_{Rd,max}$ = 856,4 KN > V_{Ed} = 325 KN **OK.**

6.3.3.3.3 Deflection

Load Combination	Service	USTDL1	USTDL2.1	USTDL2.2	USTDL2.3
Deflections (mm)	20	44	44	44,2	14,3



The worst combination is UDSTL2.2 which has a deflection of 44,2 mm.

According to CTE:

- The deformation should be inferior or equal 44,2 $\leq$ L/300 = 15500/300 = 51,6 mm **OK**

6.3.4 Corbels

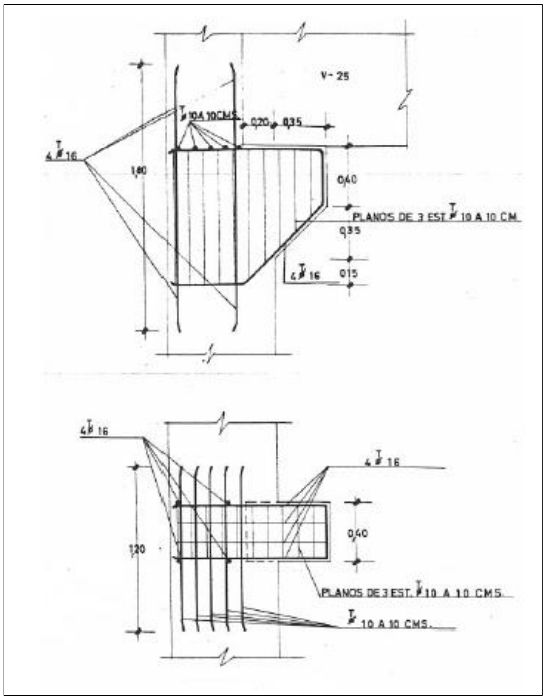
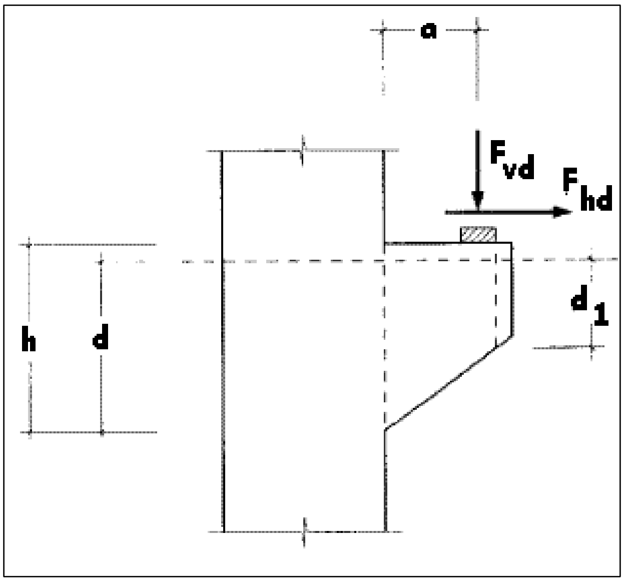


Figure 71. Section Corbel Second Basement.



6.3.4.1 Corbel verification for Beam 483

The maximum shear and axial force values applied on the Corbel (see Table 17) are F_{Vd} = 492 KN, F_{hd} = /2 = 246 KN.

Corbels are considered short when "a" which is the distance between the point where the main vertical load applies and the section adjacent to the support, is less than or equal to the effective depth "d".

The useful depth d1 measured at the outer edge of the area where the load is applied will be equal to or greater than 0.5d.

$$d \geqslant \frac{a}{0{,}85}\cot g\theta$$

a) h = 0,40 + 0,35 + 0,15 = 0,90 m.

d = h – r = 0,90 – 0,03 = 0,87 m.

a = 0,20 m. $\quad$ $\text{cotg } \emptyset = 1{,}4.$

$d = 0{,}87 > \frac{0{,}2}{0{,}85} \times 1{,}4 = 0{,}33\ m$ **OK**

b) d1 = 0,75 m.

$0{,}5\ d = 0{,}5 \times 0{,}87 = 0{,}435$

d1 > 0,5 d **OK**

The corbel is considered as a short one.

Tension ties calculation:

- $T_{1d} = F_{vd} \text{tg } \theta + F_{hd}$ = 297 x 0,714 + 246 = 458 KN.

F_{yd} = 450/1,15 = 391,3 MPa < 400 MPa ok.

F_{yd} x A_s = 391,3 x 4 x 3,14 x $(16)^2$ /4 = 314,54 KN.

$T_{1d} > A_s f_{yd}$. Fails

- $T_{2d} = 0{,}20 F_{vd}$ = 0,20 x 297 = 59,40 KN.

$A_{se} f_{yd}$ = (2) x 3 x 3,14 x $(10)^2$/4 x 450/1,15 = 184,3 KN

$T_{2d} < A_{se} f_{yd}$. OK

The corbel does not meet the requirements and needs to be strengthened.

Struts and Nodes:

$$F_{vd}/(b \cdot c) \leq f_{1cd}$$

b, c: Plan dimensions of the support.

f_{1cd}: Compressive strength of concrete. $f_{1cd} = 0.70 f_{cd}$ = 0,70 x 15,2/1,5 = 7,1 MPa.

$F_{vd}/(b \cdot c)$ = 297000/ (350 x 400) = 2,1 MPa.

$F_{vd}/(b \cdot c) \leq$ 7,1 **OK**.

- According to structural code:

$$V_{Ed} \leq 0{,}5\ b_w\ d\ \nu\ f_{cd}$$

V_{Ed} = 297 KN < 0,5 x 400 x 870 x 0,56 x 15,2/1,5 = 987 KN **OK**.

6.3.4.2 Corbel verification for Beam 474

The maximum shear and axial force values applied on the Corbel (seeTable 19) are F_{Vd} = 359 KN, F_{hd} = 946/2 = 473 KN.

Tension ties calculation:

- $T_{1d} = F_{vd} tg\ \theta + F_{hd}$ = 359 x 0,714 + 473 = 729,33 KN.

F_{yd} = 450/1,15 = 391,3 MPa < 400 MPa ok.

F_{yd} x A_s = 391,3 x 4 x 3,14 x $(16)^2$ /4 = 314,54 KN.

$T_{1d} > A_s f_{yd}$. Fails

- $T_{2d} = 0{,}20F_{vd}$ = 0,20 x 359 = 71,80 KN.

$A_{se}f_{yd}$ = (2) x 3 x 3,14 x $(10)^2$/4 x 450/1,15 = 184,3 KN

$T_{2d} < A_{se}f_{yd}$. OK

The corbel doesn't meet the requirements and needs to be strengthened

Struts and Nodes:

$$F_{vd}/(b\cdot c) \leq f_{1cd}$$

b, c: Plan dimensions of the support.

f_{1cd}: Compressive strength of concrete. $f_{1cd} = 0.70f_{cd}$ = 0,70 x 15,2/1,5 = 7,1 MPa.

$F_{vd}/(b\cdot c)$ = 359000/ (350 x 400) = 2,56 MPa.

$F_{vd}/(b\cdot c) \leq 7{,}1$ **OK**.

- According to structural code:

$$V_{Ed} \leq 0{,}5\ b_w\ d\ v\ f_{cd}$$

V_{Ed} = 359 KN < 0,5 x 400 x 870 x 0,56 x 15,2/1,5 = 987 KN **OK**.

6.3.4.3 Corbel verification for Beam 508

The maximum shear and axial force value applied on the Corbel are F_{Vd} = 325 KN, F_{hd} = 822/2 = 411KN, (see Table 21)

Tension ties calculation:

- $T_{1d} = F_{vd} \mathrm{tg}\, \theta + F_{hd}$ = 325 x 0,714 + 411 = 643,05 KN.

F_{yd} = 450/1,15 = 391,3 MPa < 400 MPa ok.

F_{yd} x A_s = 391,3 x 4 x 3,14 x $(16)^2$ /4 = 314,54 KN.

$T_{1d} > A_s f_{yd}$. Fails

- $T_{2d} = 0{,}20 F_{vd}$ = 0,20 x 325 = 65 KN.

$A_{se} f_{yd}$ = (2) x 3 x 3,14 x $(10)^2$/4 x 450/1,15 = 184,3 KN

$T_{2d} < A_{se} f_{yd}$. **OK**

The corbel doesn't meet the requirements and needs to be strengthened

Struts and Nodes:

$$F_{vd}/(b \cdot c) \leq f_{1cd}$$

b, c: Plan dimensions of the support.

f_{1cd}: Compressive strength of concrete. $f_{1cd} = 0.70 f_{cd}$ = 0,70 x 15,2/1,5 = 7,1 MPa.

$F_{vd}/(b \cdot c)$ = 325000/ (350 x 400) = 2,32 MPa.

$F_{vd}/(b \cdot c) \leq 7{,}1$ **OK**.

- According to structural code:

$$V_{Ed} \leq 0{,}5\, b_w\, d\, \nu\, f_{cd}$$

V_{Ed} = 325 KN < 0,5 x 400 x 870 x 0,56 x 15,2/1,5 = 987 KN **OK**.

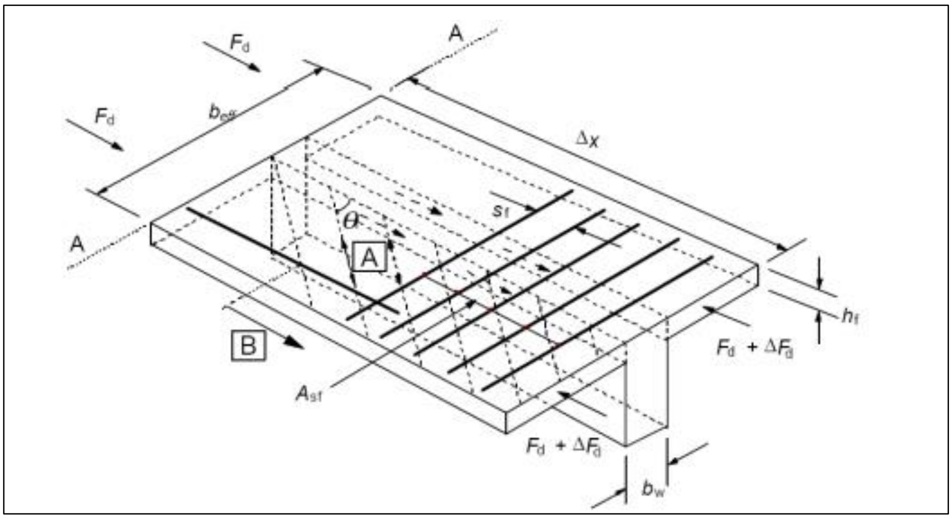
6.3.5 Shear stress

The connection between the wings and the web is subjected to a shearing stress.

- According to structural code Annex-19:

The shear stress, V_{Ed} , occurring at the joint between the web and one side of the flange is given by the formula:

$$vEd = \Delta Fd/(hf \cdot \Delta x)$$



Where:

ΔF_d: Variation of the longitudinal force acting on the flange section in the distance Δx.

According to the structural technical code, the maximum value that can be accepted for Δx is half the distance between the null moment section and the maximum moment section.

6.3.5.1 Beam 483

The maximum moment can be found in the middle of the beam, which mean at distance of the support 15,5/2 = 7,75 m.

Therefore Δx will be half of that distance = 7,75/2 = 3,875 m.

The load combination UDSTL2.2 has the maximum moment value (Table 16). Therefore, it is sufficient to only verify this case.

The moment corresponding to the distance of 3,875 is 845 KN.m

$F_d + \Delta F_d$ = 845/d = 845/1,07 = 790 KN.

ΔF_d = 790 – 0 = 790 KN. h_f = 0,15 m.

V_{Ed} = 790000/ (150 x 3875) = 1,36 MPa.

A_{sw} = 2*3*$\frac{\pi \ *8^2}{4}$ = 301,44 mm^2.

V1 = 0,6 (1-fck/250) = 0,6*(1-17,7/250) = 0,56

S = 200 mm. $\emptyset f\ = 45°$

V = 0,56

$$A_{sf} f_{yd}/s_f \geq v_{Ed} \cdot h_f/cot\theta_f$$

301,44 x (450/1,15)/200 = 590 N/mm > 1,36 x 150/1 = 204 N/mm **OK**.

To prevent failure of the wing compression struts, the following condition must be met.

condition:

$$v_{Ed} \leq v f_{cd} sen\theta_f cos\theta_f$$

1,36 < 0,56 x 17,7/1,5 x 0,707 x 0,707 =3,3 **OK**.

The Beam meets the requirements to resist the shear stress.

6.3.5.2 Beam 474

The maximum moment can be found in the middle of the beam, which mean at distance of the support 15,5/2 = 7,75 m.

Therefore Δx will be half of that distance = 7,75/2 = 3,875 m.

The load combination UDSTL2.2 has the maximum moment value (Table 18). Therefore, it is sufficient to only verify this case.

The moment corresponding to the distance of 3,875 is 1032 KN.m

$F_d + \Delta F_d$ = 1032/d = 1032/1,07 = 965 KN.

ΔF_d = 965 – 0 = 965 KN. h_f = 0,15 m.

V_{Ed} = 965000/ (150 x 3875) = 1,66 MPa.

$A_{sw} = 2*3*\frac{\pi\ *8^2}{4}$ = 301,44 mm^2.

V1 = 0,6 (1-fck/250) = 0,6*(1-17,7/250) = 0,56

S = 200 mm. $\emptyset f\ = 45°$

V = 0,56

$$A_{sf}\,f_{yd}/s_f \geq v_{Ed}\cdot h_f/cot\theta_f$$

301,44 x (450/1,15)/200 = 590 N/mm > 1,66 x 150/1 = 249 N/mm **OK**.

To prevent failure of the wing compression struts, the following condition must be met.

condition:

$$v_{Ed} \leq \nu f_{cd} sen\theta_f cos\theta_f$$

1,66 < 0,56 x 17,7/1,5 x 0,707 x 0,707 =3,3 **OK**.

The Beam meets the requirements to resist the shear stress.

6.3.5.3 Beam 508

The maximum moment can be found in the middle of the beam, which mean at distance of the support 15,5/2 = 7,75 m.

Therefore Δx will be half of that distance = 7,75/2 = 3,875 m.

The load combination UDSTL2.2 has the maximum moment value (Table 20). Therefore, it is sufficient to only verify this case.

The moment corresponding to the distance of 3,875 is 938 KN.m

$F_d + \Delta F_d$ = 938/d = 938/1,07 = 877 KN.

ΔF_d = 877 – 0 = 877 KN. h_f = 0,15 m.

V_{Ed} = 877000/ (150 x 3875) = 1,51 MPa.

$A_{sw} = 2*4*\frac{\pi\ *8^2}{4}$ = 401,92 mm^2.

V1 = 0,6 (1-fck/250) = 0,6*(1-17,7/250) = 0,56

S = 200 mm. $\emptyset f\ = 45°$

V = 0,56

$$A_{sf}\, f_{yd}/s_f \geq v_{Ed} \cdot h_f/cot\theta_f$$

401,92 x (450/1,15)/200 = 786 N/mm > 1,51 x 150/1 = 226 N/mm **OK**.

To prevent failure of the wing compression struts, the following condition must be met.

Condition:

$$v_{Ed} \leq v f_{cd} sen\theta_f cos\theta_f$$

1,51 < 0,56 x 17,7/1,5 x 0,707 x 0,707 =3,3 **OK**.

The Beam meets the requirements to resist the shear stress.

7 Strengthening proposal

As we can observe in Figure 72, the elements that are in a red color are the elements that didn’t pass the verifications. Additionally, none of the corbels in the first or the second basement comply with the requirements.

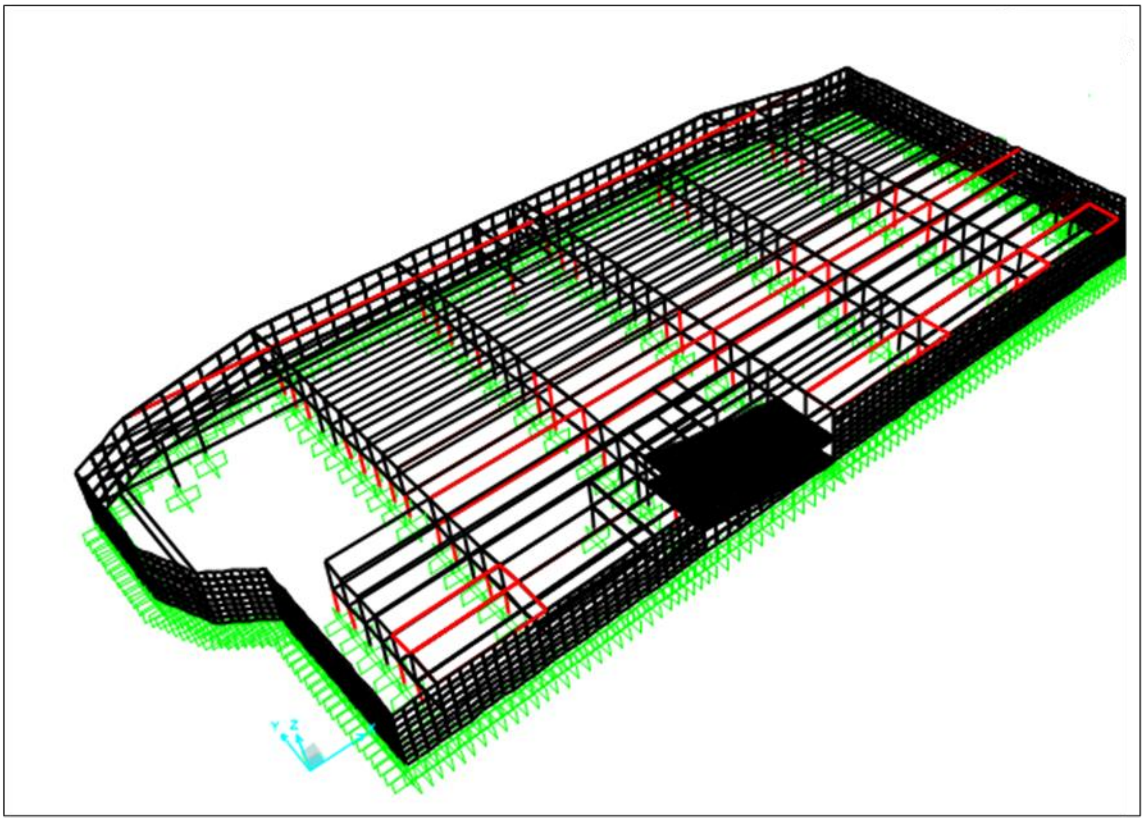


Figure 72. failed elements

7.1 Beams

There are several methods for strengthening reinforced concrete **beam** that could be followed:

1. **Increasing the beam cross section area and adding more reinforcement steel bars:**

In this method, the steps to be followed are:

Removing the concrete cover, cleaning the reinforcement steel bars, and coating them with a material that prevent the corrosion. 2. Making holes in the entire beam span and width under the slab at spacing of 15-25cm. 3. Those holes must be filled with cement mortar. Afterwards, steel connectors will be installed to fasten the new stirrups 5. Closing the added stirrups using steel wires. 6. Then the concrete should be coated with an appropriate epoxy material that guarantees the bond between the old and new concrete, just before pouring the concrete. 7. Pouring the concrete jacket.

2. **Adding steel plates to the beam without increasing the beam section**

This method is used when there is a need to strengthen the beam resistance against the applied moments. Steel plates are attached bellow the concrete beam for positive applied moments, allowing the beam to resist a greater moments. Meanwhile, the same plates are placed on top of the beam in order to resist the negative applied moments.

3. **Applying carbon fiber fabric**

The steps to be followed are:

Removing the concrete coating, then cleaning the concrete surface, applying adhesive onto the surface of the concrete, applying adhesive and cut the carbon fiber fabric, placing them onto the concrete surface.

Increasing the beam cross section will reduce the clearance of the parking lot, resulting in discomfort and perhaps reducing the clearance in a way that doesn't comply with the regulations. Therefore, **applying carbon fibers** to the inferior part of the beam will increase its resistance against the positive bending moment, maintaining the same parking clearance. Additionally, in comparison with the second method, the process of using the carbon fiber is more practical, faster, and easier in executing.

7.2 Columns and Corbels

There are several methods to improve the capacity of the reinforced concrete **columns** that could be followed:

1. **<u>Reinforced Concrete Jacketing:</u>**

By this method we increase the dimension of the column, this increase in dimension is achieved by adding new concrete to the existing concrete element. For this addition, steel reinforced is also placed to increase the load carrying capacity in order to resist the additional weight.

2. **<u>Steel jacketing</u>**

This method is used when require a large increase in the load bearing capacity but increasing the column cross sections area is not allowed.

The steps to be followed are: Removing the concrete cover and cleaning the reinforcement steel bars, coating them with an epoxy material that prevent corrosion. Installing the steel jacket, filling the space between the concrete column and the steel jacket.

3. **<u>FRP Confining or jacketing</u>**

This method consists in using a fiber reinforced polymer to gain more load bearing capacity and higher resistance to corrosion. FRP can be applied easily using sheets or laminates. Therefore, it is considered a fast and easy technic.

The reinforcement of the corbels was wrongly placed, as we can observe in the Figure 52. Instead of having the stirrups and ties horizontally placed, they were placed vertically, due to the lack of knowledge about how the method of stirrups and ties works in the period when the parking lot was built.

Applying the first method will help in taking advantage of the column strengthening plan to strengthen the corbel as well. This method will contribute to shortening the corbel resulting in having the resultant force applying on the corbel closer to the column.

Additionally, reinforced steel rebars for the corbels should be added.

8 Conclusions

1. Regarding to the **slabs**, all the structure slabs meet the requirements and no need to be strengthened.
2. Regarding to the **columns**, most of the first-floor columns have passed the verification expect the ones that are located next to the row where the columns were removed, while the second-floor columns, most of them didn't pass the verifications, due to the bigger loads applying on them, and needs a strengthening plan.
3. Regarding to the **beams**, most of the second-floor beams have passed the verification expect the ones that are located next to the row where the columns were removed, while for first floor beams, all the beams which have a tributary area bigger than 2,5 m didn't pass the verifications.
4. Regarding to the **corbels**, all the structure corbels didn't comply with the requirements and need to be strengthened in order to resist the loads applied.
5. Most of the elements that are located next to the row where the columns were removed have been subjected to a higher load, which the elements can't support.
6. Finally, with the situation of the existing elements, the strengthening plan should be done before putting the parking lot into service, to ensure the structural stability, safety, and functionality for the public use.

9 References

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