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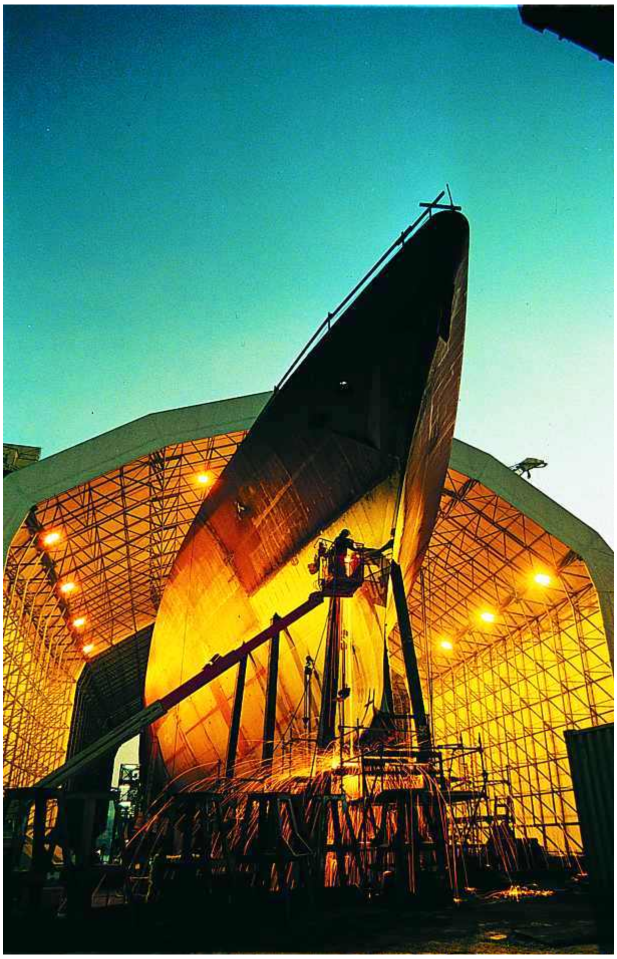

ΔΙΠΛΩΜΑΤΙΚΗ ΕΡΓΑΣΙΑ

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ΕΥΧΑΡΙΣΤΙΕΣ

Θα ήθελα να ευχαριστήσω την οικογένεια μου, τους φίλους μου και την Sonia Brana για την βοήθεια και την υποστήριξη που μου προσέφεραν.

INTRODUCTION

The subject of this diploma thesis is a shipyard building in Skaramangas Shipyards in Greece. The main use of this building is to build, repair and maintain ships. The project of this steel structure includes the description of the structural elements, the loads that are applied to them and the analysis-dimensioning with the program ETABS.

In the first chapter is presented the design and dimensioning of the structure, its structural elements (frames, secondary beams, purlins, girds, x-bracings) and the foundation.

In the second chapter are described the loads that are applied (self weight, wind, snow, earthquake) according to the Eurocode 1 and the EAK.

In the third chapter is presented the overhead crane-bridge and the crane loads applied on the structure. Also the dimensioning and analysis of the runway beam.

In the fourth chapter are presented all the loads combinations for the Ultimate Limit State and the Serviceability Limit State, according to Eurocode 1.

In the fifth chapter is presented the analysis and dimensioning of the structural elements with the program ETABS, according to the Eurocode 3. Also the results and the forces diagrams for the critical sections are presented.

In the sixth chapter are presented the front and side sliding gates, the loads that are applied to the gates and the checking of the main section of the gate.

CHAPTER 1: DESCRIPTION OF THE STRUCTURE

1.1 GENERAL

The shipyard buildings have big dimensions, in order to built, repair and preserve the ships. They are designed and constructed with big length and width to make easier the entrance of the ships. The requirements for big openings without columns make the choice of a steel structure necessary. In addition these buildings have overhead cranes for the transportation of ship pieces.

Specifically, our shipyard building is a steel structure with a rectangular ground plan and dimensions 36mx96m. The maximum height is 33,2. It's a frame structure, consisted of 17 frames every 6m parallel to the x dimension. Every frame is consisted of 30m columns and rafters that they create a duopitch roof with an angle of 11°.The columns are composite until the height of 24m, where the crane works. Secondary beams connect the frames.

For the elevation of ship pieces, the building has an overhead crane, which runs along a beam parallel to the y dimension. Its lifting capacity is 40tn and has an opening of 27m.More details for the overhead crane is given in the third chapter.

Between the frames, in the 1^o, 4^o, 8^o, 9^o, 13^o and 16^o panel of the side view and the duopitch roof, x- bracings have been put for the seismic and wind actions.

For the analysis of the building, we created a model with the software E TABS.

1.2 MAIN CARRIERS-DOUBLE SLOPE COLUMN FRAME

The main carriers are frames and can bear vertical and horizontal loads(wind, earthquake, crane), which are transmitted to the ground through the foundation. The bolted connections are moment connections. The main carriers and rafters create a duopitch roof with an 11° angle, so it can bear the vertical loads with axial and bending forces.

Because of the heavy overhead cranes and the big height, lattice columns are used, from the base(0m) until the crane beam(24m). For the outer beams of the lattice columns we use section HEB500, with their weak axis parallel to x dimension, with a 3m space between them and they are connected with the truss. For the girders of the truss we use HEA160, HEA180, HEA200, HEB300.Above the lattice column, continues a single column section HEB600, until the height of 30m. There the column is connected with bolts with the rafter section HEA700. The rafters are connected also with bolts in their top at the height of 33,2m (the maximum height of the building), creating the duopitch symmetric 11° roof.

The following figure is the design in the program ETABS.

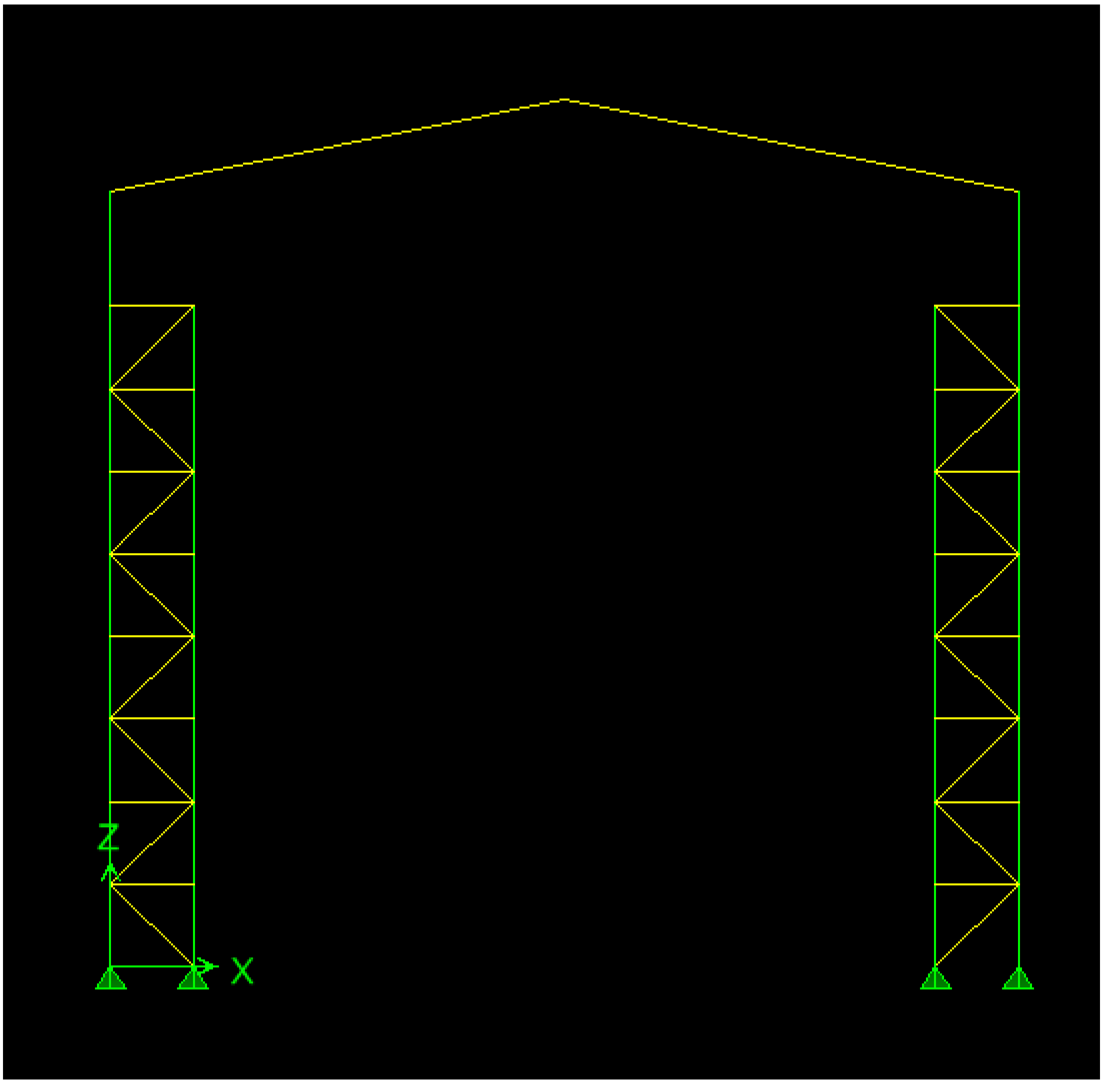


FIGURE 1.1-DOUBLE SLOPE COLUMN FRAME

1.3 SECONDARY BEAMS (ROOF BEAMS)

The roof beam is a horizontal structural component, which connects the heads of the columns and runs along the dimension y. The roof beams take the seismic and other horizontal forces from the horizontal bracings and transmit them to the vertical bracings (between the columns), so they finally end up to the foundation. We used sections HEB160, 6m long, which connect the frames at the height of 30m.

1.4 HORIZONTAL X-BRACING SYSTEM

The horizontal x-bracing system is placed in the level of the rafters in the bay between two frames. Is consisted of rafters, purlins and the x-bracings. The function of this system is to transmit the horizontal forces (wind, earthquake) to the vertical x-bracing system and finally to the foundation. The x-bracings are placed in every third purlin.

When an horizontal load is acting, only the tensile component of the bracing is considered active. The compressive component is ignored, if not we should use bigger sections to avoid buckling, which is uneconomic. L100.10 sections are used for the x-bracings, which are placed in the 1^o, 4^o, 8^o, 9^o, 13^o and 16^o panel.

The following figure shows the horizontal bracing system in ETABS:

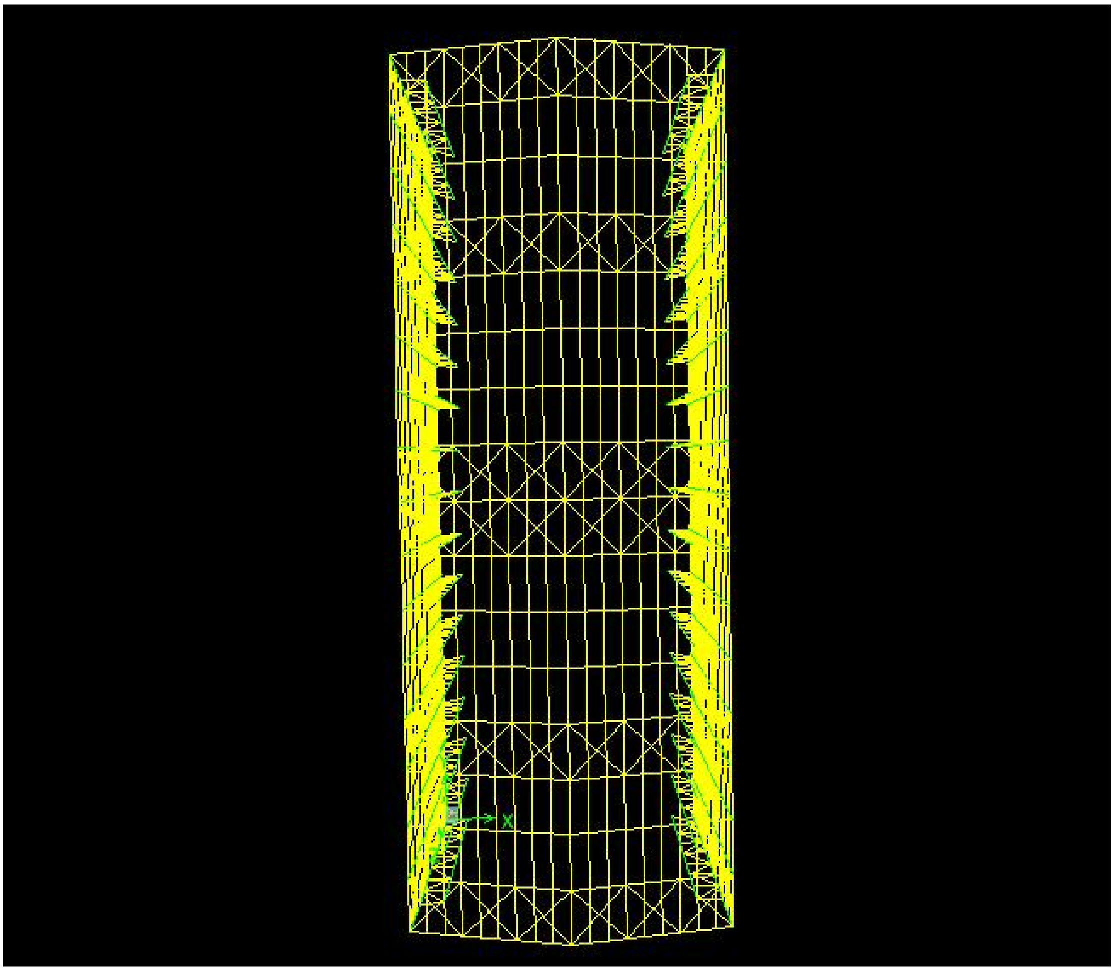


FIGURE 1.2-HORIZONTAL X-BRACING SYSTEM

1.5. VERTICAL X-BRACING SYSTEM

The vertical x-bracing system is placed between two columns and are used to resist horizontal forces (wind load, seismic action) and to transmit them to the foundations. The system is consisted of columns, girds, x-bracings which are placed in the same panels as the horizontal x-bracings (1^o, 4^o, 8^o, 9^o, 13^o and 16^o), every forth gird. UPN180 sections are used for the x-bracings.

The following figure shows the vertical bracing system in ETABS:

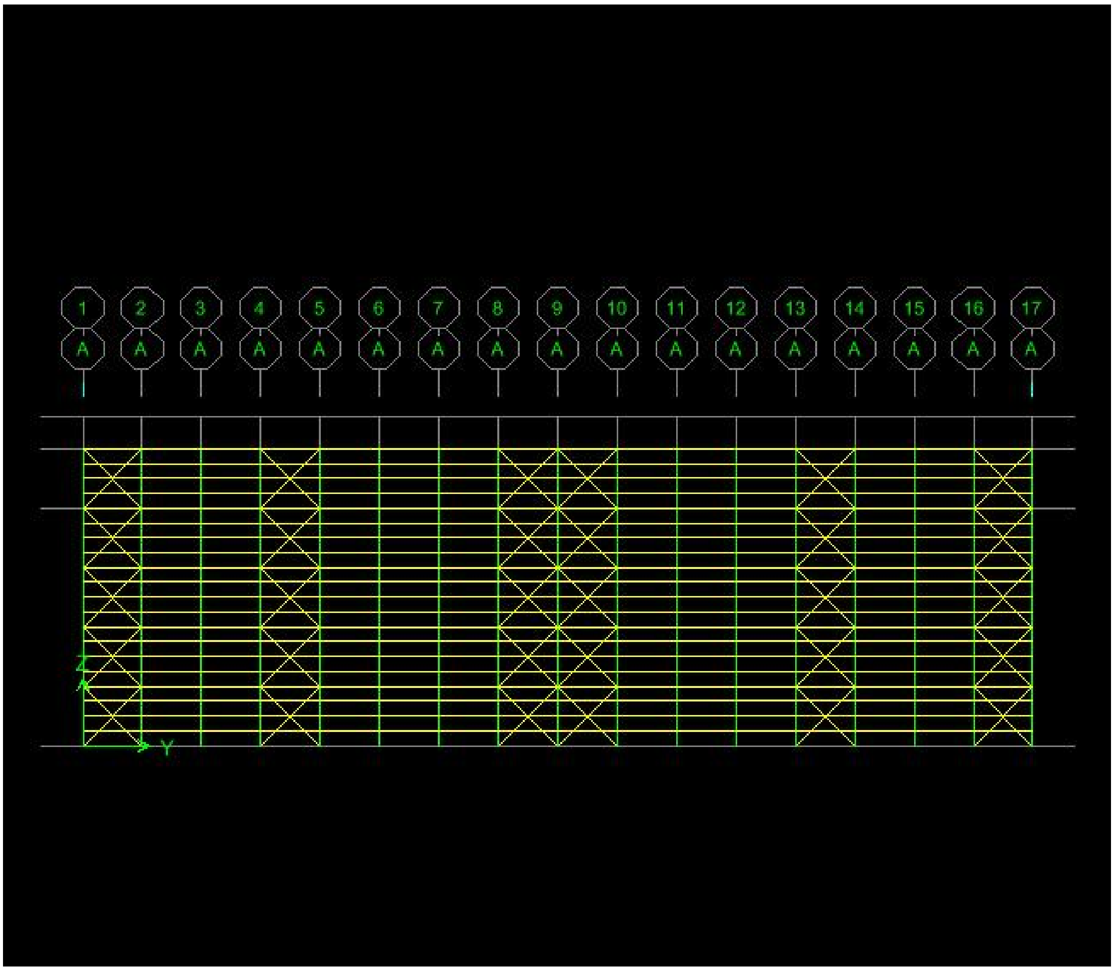


FIGURE 1.3-VERTICAL X-BRACING SYSTEM

1.6. PURLINS

Purlins are structural members in the roof, that connect the rafters and support the the loads of the roof, like the weight of the covering, the wind load and the wind load. The down flange of the purlin is connected to the upper flange of the rafter, so the strong axis is activated to support the loads. They are simple supported beams, 6m long, placed every 2,03m and the section chosen is IPE270. The middle purlins have the same loads , while the outer purlins have the half loads.

1.7. GIRDS

Girds are horizontal beams placed in the side view of the building every 1,5m and support the the wind loads. They are simple supported, 6m long, with their down flange connected to the upper part of the columns. In that way their strong axis is activated to support the wind loads(which is the main load), while their weak axis is activated to support their self-weight and the self-weight of the covering. They transfer the loads to the columns. Sections IPE240 have been chosen for the girds. The outer girds support half the load of the middle girds.

1.8. OVERHEAD CRANE AND CRANE BEAM

The overhead crane has a lifting capacity of 40 tones, runs along a horizontal beam and itself runs along two crane beams in the direction of the axis y, in 30m height. The overhead crane transfers horizontal and vertical loads to the crane beams. The crane beans are very important for the static model, first because they support the big loads of the crane and second because they are sensitive to fatigue due to extreme alternations of the loads. They are simple supported, 6m long, placed upon the lattice columns, with sections HEB700.A detailed analysis about the overhead crane is given to the third chapter. The following pictures show some examples:

PICTURE 1.1-EXAMPLE OF OVERHEAD CRANE

PICTURE 1.2-EXAMPLE OF OVERHEAD CRANE

1.9 SPACE REPRESENTATION OF THE STRUCTURE

The following figure shows the space representation, as it was designed in the program ETABS:

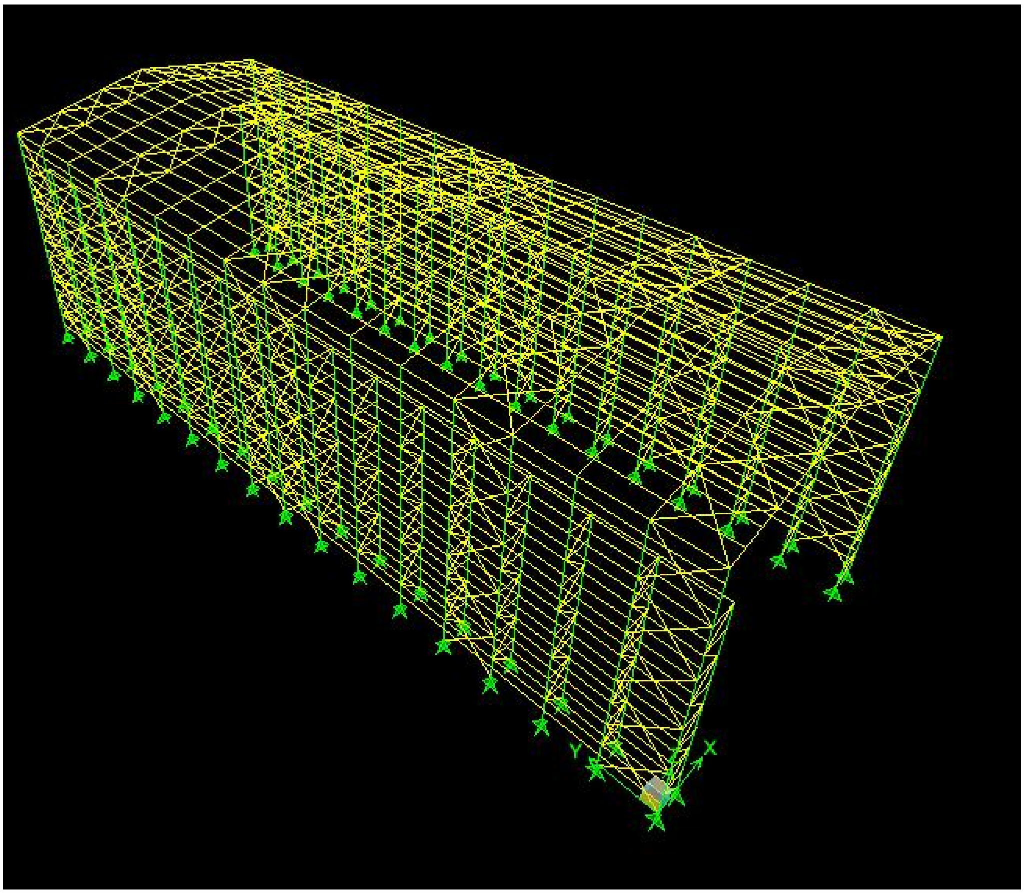


FIGURE 1.4 -SPACE REPRESENTATION

1.10 FOUNDATION

Due to the quality of the soil, the location of the structure near the sea and the big loads, a strong foundation is required. After a special geotechnical research, we used deep foundation with a concrete pile system, with 1m diameter. Every foundation is consisted of two concrete piles. Every pair of foundation is placed axial to the frame. At the lowest part of the column a base plate is welded, which is connected to the foundation with anchor bolts (heavy, threded bolts embedded in the foundation to secure the frame). The following figures show the ground plan of the foundation and details:

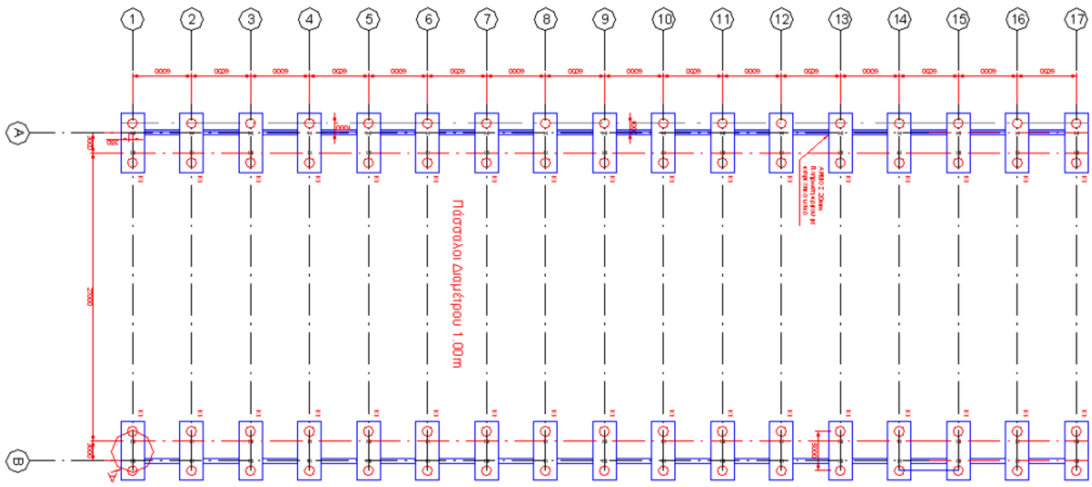


FIGURE 1.5-FOUNDATION GROUND PLAN

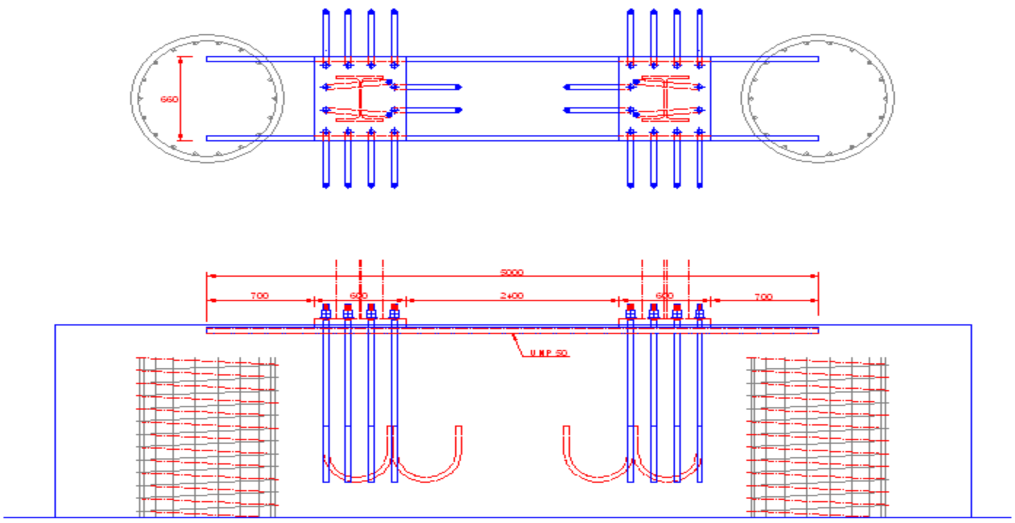


FIGURE 1.6 -CONNECTION OF THE BASE PLATE TO YHE FOUNDATION

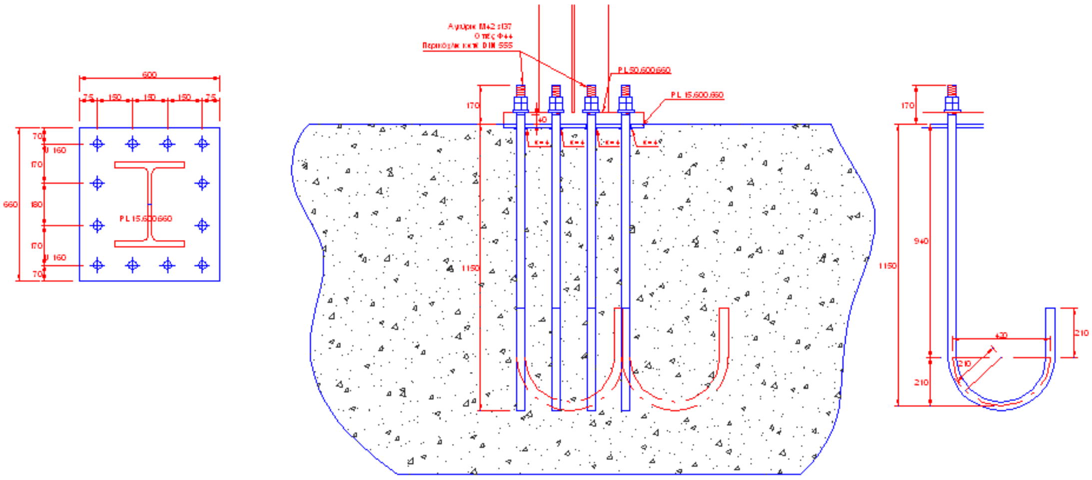


FIGURE 1.7 -DETAIL OF ANCHOR BOLTS

CHAPTER 2: ACTIONS ON STRUCTURE

2.1 GENERAL

The actions are classified in:

•Direct, like the self-weights of the elements, snow, wind etc.

•Indirect, like temperature and landslides

Depending on their variation through time are classified in:

a)Permanent actions

•self-weight of structural members

•self-weight of non structural elements

b)Variable actions

•wind

•snow

•crane loads

c)Accidental actions

•Earthquake

Next I will do a further description and calculation for each action according to the Eurocode 1 (EN 1991).

2.2 PERMENENT ACTIONS(G)

The self-weights of the structural members(double slope column frame, roof beam, crane beam, horizontal bracings, vertical bracings, purlins and girds) are automatically calculated by the designing program E TABS.

The additional permanent loads are self-weights of the non-structural elements:

Surfacing and coverings: 0,10 KN/m²

Electrical and mechanical equipment : 0,20 KN/m²

Total additional permanent loads: **0,30 KN/m²**

This load is considered distributed in the parallel frames of our construction, namely the main and the secondary beams.

2.3 VARIABLE ACTIONS(Q)

2.3.1 SNOW

Snow loads shall be classified as variable, fixed actions and should be classified as static action.

SNOW LOAD ON ROOF

The design shall recognize that snow can be deposited on a roof in many different patterns.

Properties of a roof causing different patterns can include:

- the shape of the roof,
- its thermal properties;
- the roughness of its surface;
- the amount of heat generated under the roof;
- the proximity of nearby buildings;
- the surrounding terrain
- the local meteorological climate, in particular its windiness, temperature variations, and likelihood of precipitation (either as rain or as snow).

Snow loads on roofs shall be determined as follows:

a) for the persistent / transient design situations

$s = \mu_i \cdot C_e \cdot C_t \cdot s_k$

b) for the accidental design situations where exceptional snow load is the accidental action

$s = \mu_i \cdot C_e \cdot C_t \cdot s_{Ad}$

where:

μ_i is the snow load shape coefficient

s_k is the characteristic value of snow load on the ground

s_{Ad} is the design value of exceptional snow load on the ground for a given location

C_e is the exposure coefficient and should usually be taken as $C_e = 1,0$

C_t is the thermal coefficient and should be used to account for the reduction of snow loads on roofs with high thermal transmittance (> 1 W/m2K), in particular for some glass covered roofs, because of melting caused by heat loss.

For all other cases:

$C_t = 1,0$

CALCULATION OF THE CHARACTERISTIC VALUE Sk

Greece is devided into three zones according to the snow

Zone 1 ($s_{k,0}$=0,40 kN/m^2)

Zone 2 ($s_{k,0}$=0,80 kN/m^2)

Zone 3 ($s_{k,0}$=1,7 kN/m^2)

$$s_k = s_{k,0}\left[1+(A/917)^2\right]$$

Where:

A : is the altitude,in our case A=0, because the structure is in the level of the sea

$s_{k,0}$=0,80 kN/m^2 because the structure is in the Zone 2

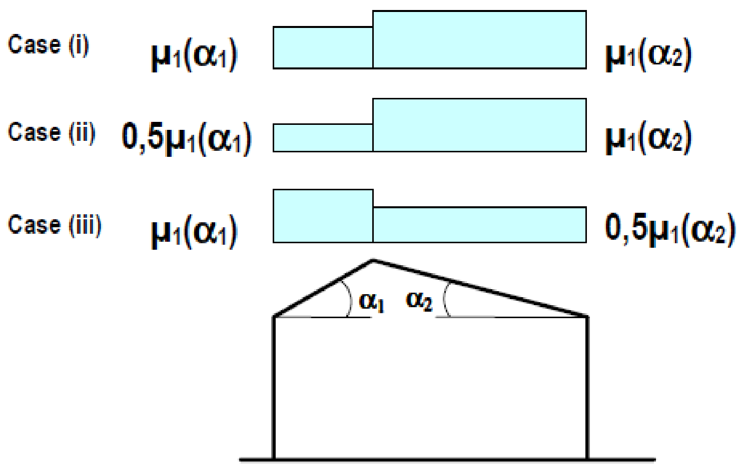
sk=0,80 kN/m^2

CALCULATION OF THE SHAPE COEFFICIEN μi

The values given in the following Table apply when the snow is not prevented from sliding off the roof. Where snow fences or other obstructions exist or where the lower edge of the roof is terminated with a parapet, then the snow load shape coefficient should not be reduced below 0,8.

Angle of pitch of roof a	$0° \leq \alpha \leq 30°$	$30° \leq \alpha \leq 60°$	$\alpha \geq 60°$
Coefficient μ_1	0,8	0,8(60-α)/30	0,0

Table 2.1



The angle of the roof is 11°, so μi = 0,8

According to all the previous the final snow load in the roof is:

$S = \mu_i \cdot C_e \cdot C_t \cdot S_k$ =0,8•1•1•0,8=0,64 kN/m^2

S*=0,64 *kN/m²

2.3.2 WIND LOADS

Wind actions should be classified as variable fixed actions. They fluctuate with time and act directly as pressures on the external surfaces of enclosed structures and, because of porosity of the external surface, also act indirectly on the internal surfaces. They may also act directly on the internal surface of open structures. Pressures act on areas of the surface resulting in forces normal to the surface of the structure or of individual cladding components. Additionally, when large areas of structures are swept by the wind, friction forces acting tangentially to the surface may be significant. Because the dynamic load is very small, we will disregard the dynamic vibration and will consider the wind as a static load.

a)WIND PRESSURE ON EXTERNAL SURFACES

The wind pressure acting on the external surfaces, w_e , should be obtained from the expression: $w_e = q_p \cdot (z_e) \cdot c_{pe}$

$q_p \cdot (z_e)$: is the peak velocity pressure

z_e : is the reference height for the external pressure

c_{pe} : is the pressure coefficient for the external pressure

b)WIND PRESSURE ON INTERNAL SURFACES

The wind pressure acting on the internal surfaces of a structure, *w*i , should be obtained from the expression: $w_i = q_p(z_i) \cdot c_{pi}$

$q_p(z_i)$: is the peak velocity pressure

z_i : is the reference height for the internal pressure

c_{pi} : is the pressure coefficient for the internal pressure

c)PEAK VELOCITY PRESSURE $q_p(z)$

The peak velocity pressure $q_p(z)$at height *z*, which includes mean and short-term velocity fluctuations, should be determined from the expression:

$$q_{p(z)} = [1 + 7 \cdot I_v(z)] \cdot \frac{1}{2} \cdot \rho \cdot v_m^2(z) = c_e(z) \cdot q_b$$

where:

ρ: is the air density, which depends on the altitude, temperature and barometric pressure to be expected in the region during wind storms. Usually ρ=1,25kg/m^3

*c*e(*z*): is the exposure factor

$$c_e(z) = \frac{q_p(z)}{q_b}$$

For flat terrain where $c_0(z)$ = 1,0, the exposure factor *c*e(*z*) is illustrated as a function of height above terrain and a function of terrain category.

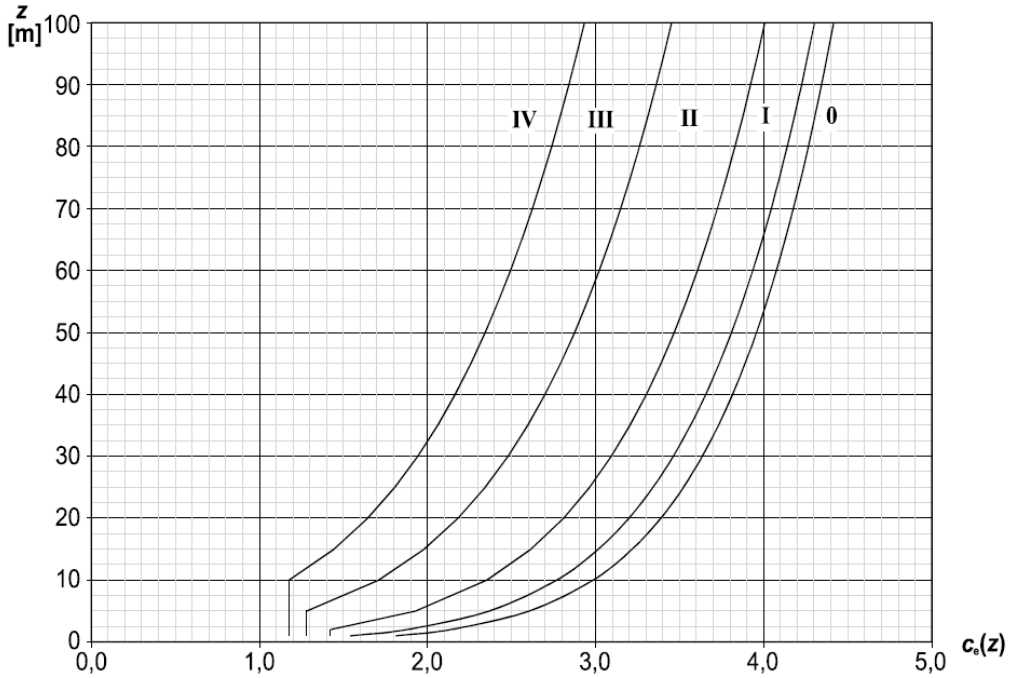


Figure 2.1- Illustrations of the exposure factor *c*e(*z*) for *c*O=1,0, *kI*=1,0

q_b:is the basic velocity pressure

$$q_b = \frac{1}{2} \cdot \rho \cdot v_b^2$$

The basic wind velocity is defined as a function of wind direction and time of year at 10 m above ground of terrain category II and shall be calculated from the expression:

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b,0}$$

where:

$v_{b,0}$:is the fundamental value of the basic wind velocity, is the characteristic 10 minutes mean wind velocity, irrespective of wind direction and time of year, at 10 m above ground level in open country terrain with low vegetation such as grass and isolated obstacles with separations of at least 20 obstacle heights. According to the national annex it's value is 33 m/s .

c_{dir} :is the directional factor, the recommended value is 1,0

c_{season} :is the season factor, the recommended value is 1,0.

The mean wind velocity $v_m(z)$at a height z above the terrain depends on the terrain roughness and orography and on the basic wind velocity and should be determined using the expression: $v_m(z) = c_r(z) \cdot c_0(z) \cdot v_b$

$c_r(z)$:is the roughness factor

$c_0(z)$:is the orography factor, taken as 1,0

The turbulence intensity $I_v(z)$ at height z is defined as the standard deviation of the turbulence divided by the mean wind velocity and should be determined using the expression:

$$I_v(z) = \frac{k_I}{c_0(z) \cdot \ln(z / z_{0)})} \quad , \text{ για } \quad z_{\min} \leq z \leq z_{\max}$$

$$I_v(z) = I_v(z_{\min}) \quad , \text{ για } \quad z \leq z_{\min}$$

Where:

k_I :is the turbulence factor. The value of kI to be used in a Country may be found in its National Annex.The recommended value is k_I = 1,0.

C)FINAL PRESSURE

The net pressure on a wall, roof or element is the difference between the pressures on the opposite surfaces taking due account of their signs. Pressure, directed towards the surface is taken as positive, and suction, directed away from the surface as negative. Examples are given in the Figure

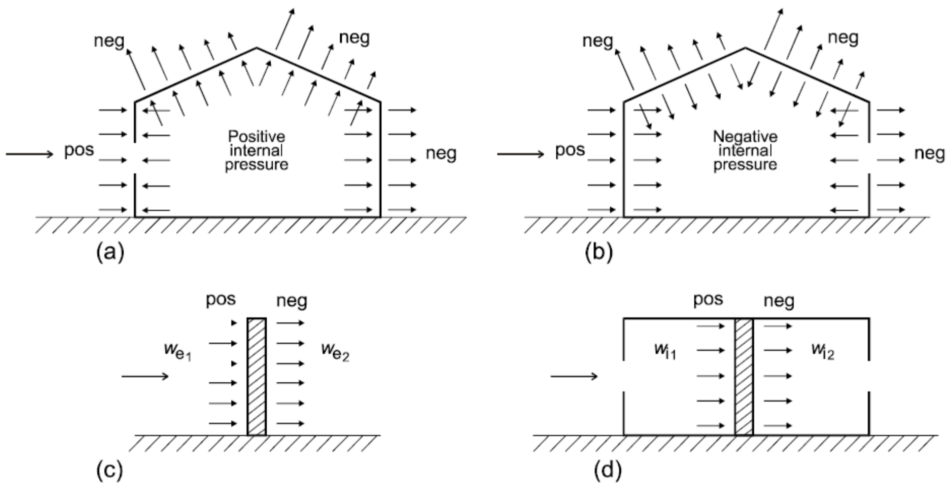


Figure 2.2 — Pressure on surfaces

WIND FORCES

A)TOTAL WIND FORCE (F_w)

The wind force *F*w acting on a structure or a structural component may be determined directly by using the expression:

$$F_w = c_s c_d \cdot c_f \cdot q_p(z_e) \cdot A_{ref}$$

or by vectorial summation over the individual structural elements by using the expression:

$$F_w = c_s c_d \cdot \sum_{elements} c_f \cdot q_p(z_e) \cdot A_{ref}$$

where:

$c_s c_d$	:is the structural factor
c_f	:is the force coefficient for the structure or structural element
$q_p(z_e)$	:is the peak velocity pressure at reference height z_e
A_{ref}	:is the reference area of the structure or structural element

B)FRICTION WIND FORCE

The frictional forces resulting from the friction of the wind parallel to the external surfaces, calculated using the expression:

$$F_{fr} = c_{fr} \cdot q_p(z_e) \cdot A_{fr}$$

Where:

c_{fr}	:is the friction coefficient
A_{fr}	:is the area of external surface parallel to the wind

The effects of wind friction on the surface can be disregarded when the total area of all surfaces parallel with (or at a small angle to) the wind is equal to or less than 4 times the total area of all external surfaces perpendicular to the wind (windward and leeward).

FACTORS

A)THE ROUGHNESS FACTOR $c_r(z)$

The roughness factor, $c_r(z)$, accounts for the variability of the mean wind velocity at the site of the structure due to:

– the height above ground level

– the ground roughness of the terrain upwind of the structure in the wind direction considered

The recommended procedure for the determination of the roughness factor at height *z* is given by expression (2.1) and is based on a logarithmic velocity profile

(2.1) $c_r(z) = k_r \cdot \ln(\frac{z}{z_0})$

for $z_{\min} \leq z \leq z_{\max} = 200m$

$c_r(z) = c_r(z_{\min}) = k_r \cdot \ln(\frac{z_{\min}}{z_0})$ για $z < z_{\min}$

where:

k_r : terrain factor depending on the roughness length z0 calculated using:

$k_r = 0{,}19 \cdot (\frac{z_0}{z_{0,II}})^{0{,}07}$

Where $Z_{0,II}$ =0,05 m for terrain category 2

z_0 : is the roughness length

z_{min} : is the minimum height defined in Table 2.1

z_{max} : is to be taken as 200 m

The terrain roughness to be used for a given wind direction depends on the ground roughness and the distance with uniform terrain roughness in an angular sector around the wind direction. Small areas (less than 10% of the area under consideration) with deviating roughness may be ignored.

	Terrain category	z_0 (m)	z_{min} (m)
0	Sea or coastal area exposed to the open sea	0,003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0,01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0,05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0,3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1,0	10

Table 2.2

B)OROGRAPHY FACTOR $c_t(z)$

The orography factor, $c_t(z)$accounts for the increase of mean wind speed over isolated hills and escarpments (not undulating and mountainous regions) according to slope Φ=H/Lu

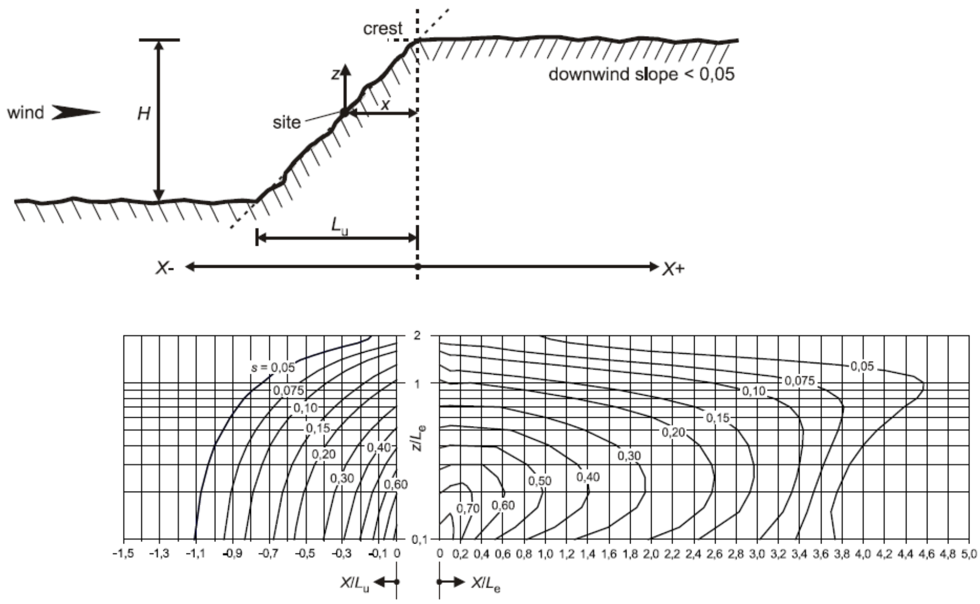


figure 2.3- Factor *s* for cliffs and escarpments

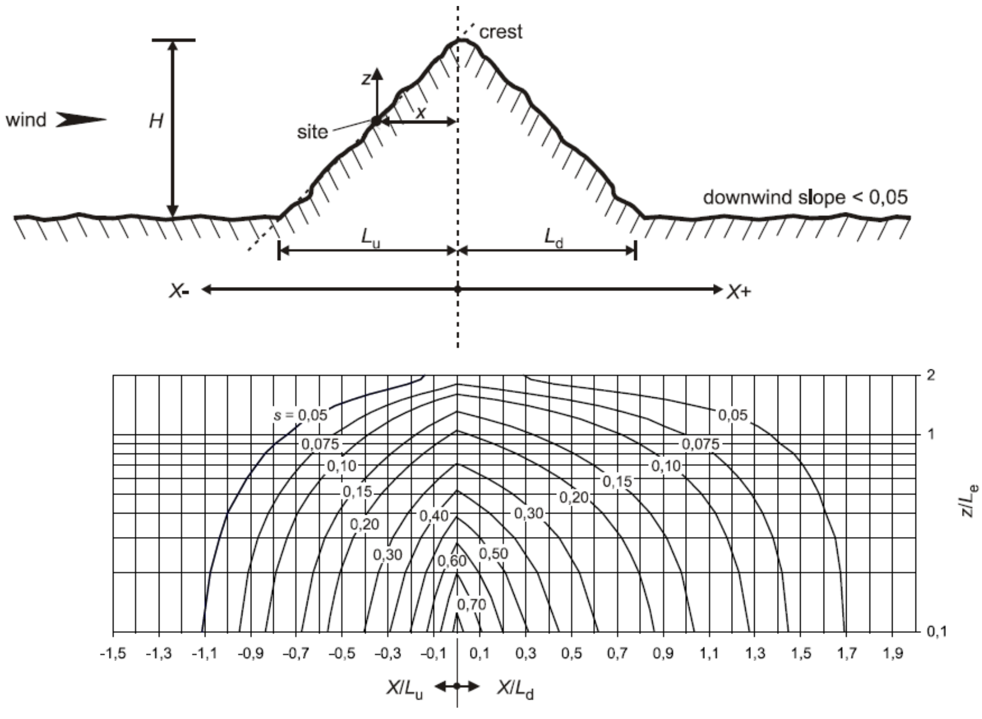


figure 2.4- Factor *s* for hills and ridges

In valleys,as the location where our structure is located, $c_t(z)$ may be set to 1,0 if no speed up due to funnelling effects is to be expected.So:

$C_t(z) = 1$

C)STRUCTURAL FACTOR CsCd

The structural factor CsCd should take into account the effect on wind actions from the nonsimultaneous occurrence of peak wind pressures on the surface together with the effect of the vibrations of the structure due to turbulence.
For framed buildings which have structural walls and which are less than 100 m high and whose height is less than 4 times the in-wind depth, the value of *c*s*c*d may be taken as 1. **CsCd=1**

D)AERODYNAMIC COEFFICIENTS

This section should be used to determine the appropriate aerodynamic coefficients for structures.Depending on the structure the appropriate aerodynamic coefficient will be:

– Internal and external pressure coefficients

– Net pressure coefficients

– Friction coefficients

– Force coefficients

They should be determined for: buildings, circular cylinders, canopy roofs, free standing walls, parapets, fences, signboards, structural elements with rectangular cross section, circular cylinders and spheres

EXTERNAL PRESSURE COEFFICIENT FOR BUILDINGS

The external pressure coefficients c_{pe} for buildings and parts of buildings depend on the size of the loaded area *A*, which is the area of the structure, that produces the wind action in the section to be calculated. The external pressure coefficients are given for loaded areas *A* of 1 m2 and 10 m2 in the tables for the appropriate building configurations as *c*pe,1, for local coefficients, and *c*pe,10, for overall coefficients, respectively.

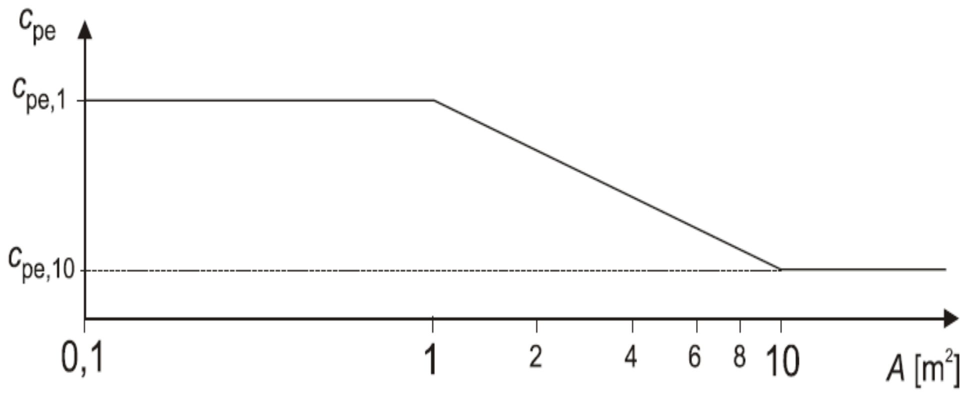


Figure 2.5

The figure is based on the following:

for $A \leq 1$ m2 $c_{pe} = c_{pe,1}$

for 1 m2 < A < 10 m2 $c_{pe} = c_{pe,1} - (c_{pe,1} - c_{pe,10}) \log_{10} A$

for $A \geq 10$ m2 $c_{pe} = c_{pe,10}$

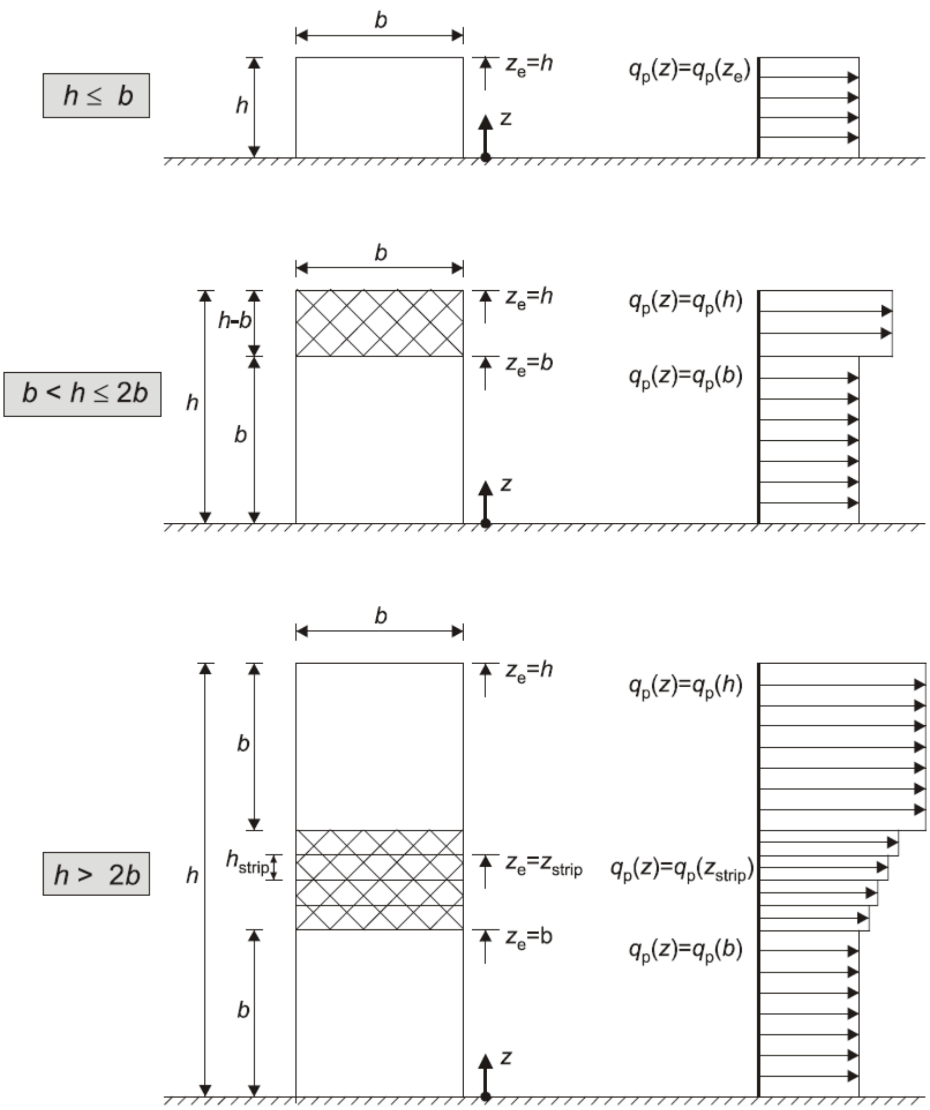
VERTICAL WALLS OF RECTANGULAR PLAN BUILDINGS

The reference heights, z_e, for walls of rectangular plan buildings depend on the aspect ratio h/b and are always the upper heights of the different parts of the walls. They are given in Figure for the following three cases:

-A building, whose height h is less than b should be considered to be one part.

-A building, whose height h is greater than b, but less than 2b, may be considered to be two parts, comprising: a lower part extending upwards from the ground by a height equal to b and an upper part consisting of the remainder.

-A building, whose height h is greater than 2b may be considered to be in multiple parts, comprising: a lower part extending upwards from the ground by a height equal to b; an upper part extending downwards from the top by a height equal to b and a middle region, between the upper and lower parts, which may be divided into horizontal strips with a height h_{strip} .



The external pressure coefficients $c$pe,10 and $c$pe,1 for zone A, B, C, D and E are defined in the Figure.

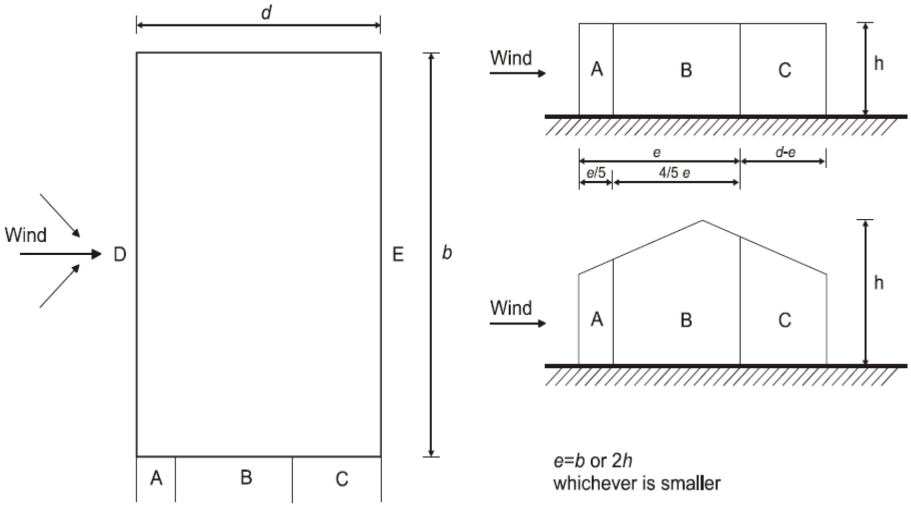


Figure 2.6- Key for vertical walls

The values of $c$pe,10 and $c$pe,1 for use in a Country may be given in its National Annex. The recommended values are given in Table , depending on the ratio h/d. For intermediate values of h/d, linear interpolation may be applied.

Table: Recommended values of external pressure coefficients for vertical walls of rectangular plan buildings

Zone	A		B		C		D		E	
h/d	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,7	
1	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,5	
≤ 0,25	-1,2	-1,4	-0,8	-1,1	-0,5		+0,7	+1,0	-0,3	

Table 2.3

In cases where the wind force on building structures is determined by application of the pressure coefficients $c$pe on windward and leeward side (zones D and E) of the building simultaneously, the lack of correlation of wind pressures between the windward and leeward side may be taken into account.

For buildings with $h/d \geq 5$ the resulting force is multiplied by 1. For buildings with $h/d \leq 1$, the resulting force is multiplied by 0,85. For intermediate values of h/d, linear interpolation should be applied.

DUOPITCH ROOF

The roof, including protruding parts, should be divided in zones as shown in Figure .The reference height ze should be taken as h. The pressure coefficients for each zone that should be used are given in Table .

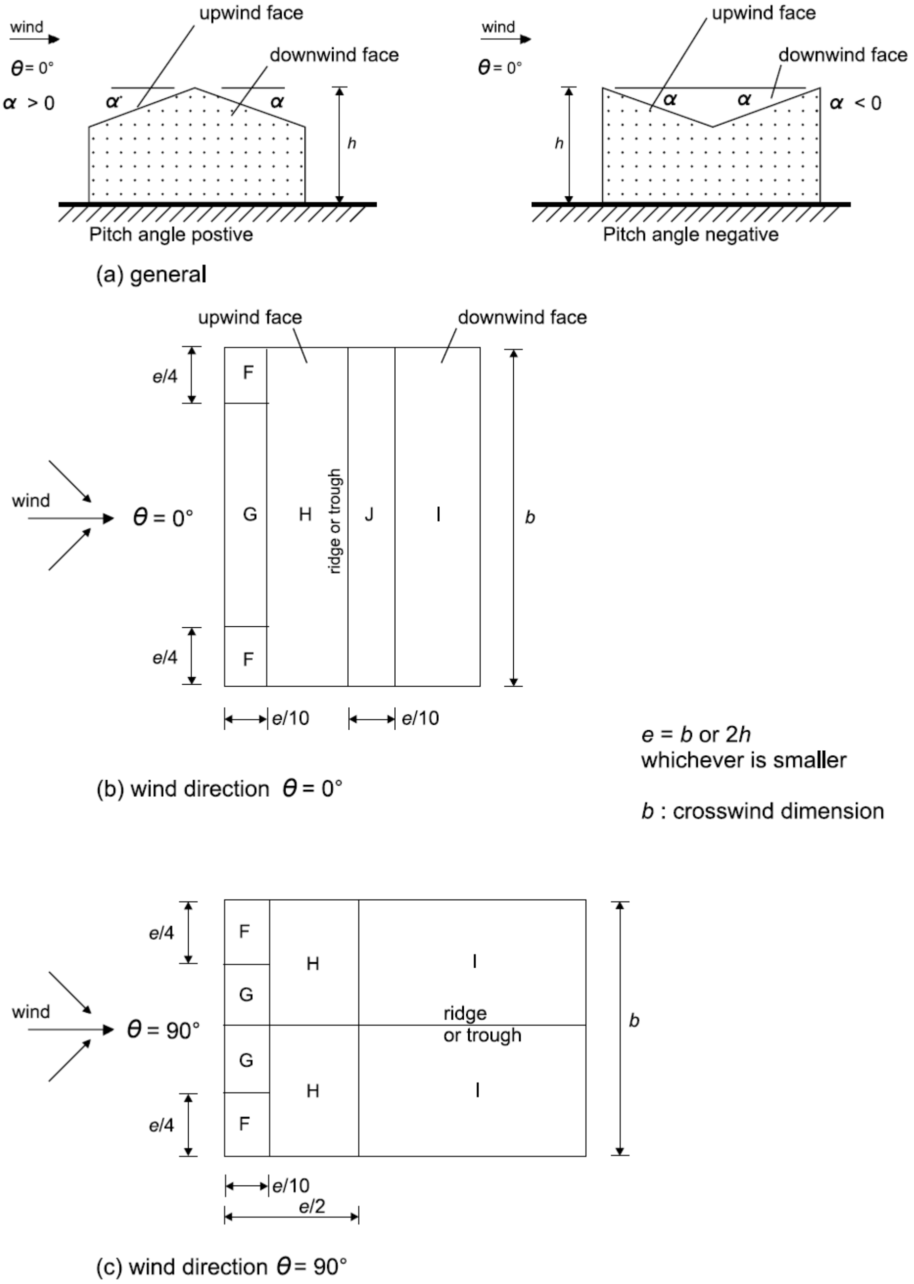


Figure 2.7 — Key for duopitch roofs

Pitch Angle α	Zone for wind direction $\Theta = 0°$									
	F		G		H		I		J	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
-45°	-0,6		-0,6		-0,8		-0,7		-1,0	-1,5
-30°	-1,1	-2,0	-0,8	-1,5	-0,8		-0,6		-0,8	-1,4
-15°	-2,5	-2,8	-1,3	-2,0	-0,9	-1,2	-0,5		-0,7	-1,2
-5°	-2,3	-2,5	-1,2	-2,0	-0,8	-1,2	+0,2		+0,2	
							-0,6		-0,6	
5°	-1,7	-2,5	-1,2	-2,0	-0,6	-1,2	-0,6		+0,2	
	+0,0		+0,0		+0,0				-0,6	
15°	-0,9	-2,0	-0,8	-1,5	-0,3		-0,4		-1,0	-1,5
	+0,2		+0,2		+0,2		+0,0		+0,0	+0,0
30°	-0,5	-1,5	-0,5	-1,5	-0,2		-0,4		-0,5	
	+0,7		+0,7		+0,4		+0,0		+0,0	
45°	-0,0		-0,0		-0,0		-0,2		-0,3	
	+0,7		+0,7		+0,6		+0,0		+0,0	
60°	+0,7		+0,7		+0,7		-0,2		-0,3	
75°	+0,8		+0,8		+0,8		-0,2		-0,3	

Table 2.4 — External pressure coefficients for duopitch roofs

Pitch angle α	Zone for wind direction $\Theta = 90°$							
	F		G		H		I	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
-45°	-1,4	-2,0	-1,2	-2,0	-1,0	-1,3	-0,9	-1,2
-30°	-1,5	-2,1	-1,2	-2,0	-1,0	-1,3	-0,9	-1,2
-15°	-1,9	-2,5	-1,2	-2,0	-0,8	-1,2	-0,8	-1,2
-5°	-1,8	-2,5	-1,2	-2,0	-0,7	-1,2	-0,6	-1,2
5°	-1,6	-2,2	-1,3	-2,0	-0,7	-1,2	-0,6	
15°	-1,3	-2,0	-1,3	-2,0	-0,6	-1,2	-0,5	
30°	-1,1	-1,5	-1,4	-2,0	-0,8	-1,2	-0,5	
45°	-1,1	-1,5	-1,4	-2,0	-0,9	-1,2	-0,5	
60°	-1,1	-1,5	-1,2	-2,0	-0,8	-1,0	-0,5	
75°	-1,1	-1,5	-1,2	-2,0	-0,8	-1,0	-0,5	

Table 2.5 — External pressure coefficients for duopitch roofs

At $\Theta = 0°$ the pressure changes rapidly between positive and negative values on the windward face around a pitch angle of $\alpha = -5°$ to $+45°$, so both positive and negative values are given**.** For those roofs, four cases should be considered where the largest or smallest values of all areas F, G and H are combined with the largest or smallest values in areas I and J. No mixing of positive and negative values is allowed on the same face**.**

Linear interpolation for intermediate pitch angles of the same sign may be used between values of the same sign. (Do not interpolate between α = +5° and α = -5°, but use the data for flat roofs).

INTERNAL PRESSURE

Internal and external pressures shall be considered to act at the same time. The worst combination of external and internal pressures shall be considered for every combination of possible openings and other leakage paths.

The internal pressure coefficient, c_{pi}, depends on the size and distribution of the openings in the building envelope. When in at least two sides of the buildings (facades or roof) the total area of openings in each side is more than 30 % of the area of that side, the actions on the structure should not be calculated from the rules given in this section.

The openings of a building include small openings such as: open windows, ventilators, chimneys, etc. as well as background permeability such as air leakage around doors, windows, services and through the building envelope. The background permeability is typically in the range 0.01% to 0.1% of the face area. Additional information may be given in a National Annex.

Where an external opening, such as a door or a window, would be dominant when open but is considered to be closed in the ultimate limit state, during severe windstorms, the condition with the door or window open should be considered as an accidental design situation in accordance with EN 1990.

Checking of the accidental design situation is important for tall internal walls (with high risk of hazard) when the wall has to carry the full external wind action because of openings in the building envelope.

A face of a building should be regarded as dominant when the area of openings at that face is at least twice the area of openings and leakages in the remaining faces of the building considered. This can also be applied to individual internal volumes within the building. When the area of the openings at the dominant face is twice the area of the openings in the remaining faces:

$\mathbf{c_{pi}=0,75c_{pe}}$

When the area of the openings at the dominant face is at least 3 times the area of the openings in the remaining faces,:

c_{pi}=0,90c_{pe}

where c_{pe} is the value for the external pressure coefficient at the openings in the dominant face. When these openings are located in zones with different values of external pressures an area weighted average value of c_{pe} should be used.

When the area of the openings at the dominant face is between 2 and 3 times the area of the openings in the remaining faces linear interpolation for calculating c_{pi} may be used.

For buildings without a dominant face, the internal pressure coefficient c_{pi} should be determined from Figure, and is a function of the ratio of the height and the depth of the building, h/d, and the opening ratio μ for each wind direction θ, which should be determined from expression:

$$\mu = \frac{\Sigma \text{area of openings where } cpe \text{ is negative or } - 0{,}0}{\Sigma \text{area of all openings}}$$

This applies to façades and roof of buildings with and without internal partitions.

Where it is not possible, or not considered justified, to estimate μ for a particular case then c_{pi} should be taken as the more onerous of +0,2 and -0,3.

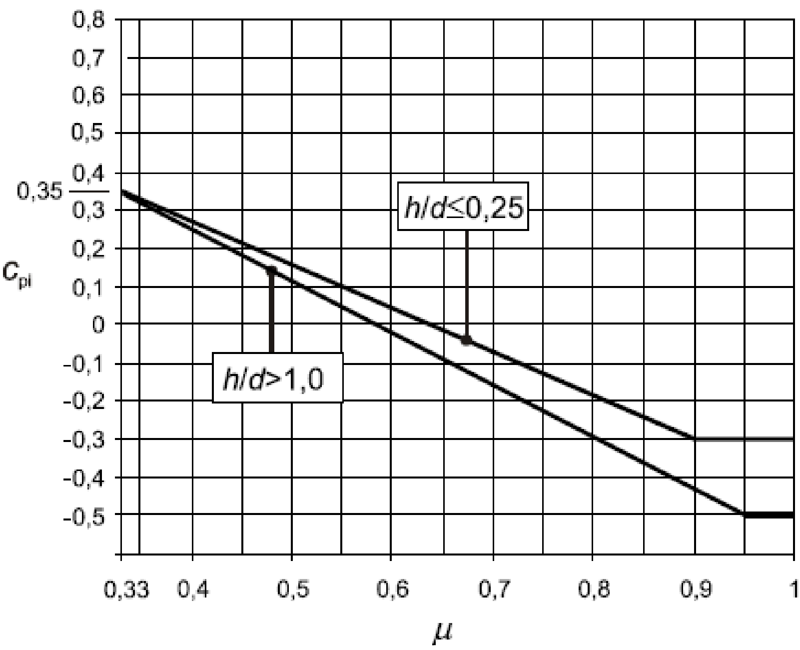


Figure 2.8— Internal pressure coefficients for uniformly distributed openings

The reference height z_i for the internal pressures should be equal to the reference height z_e for the external pressures (see 5.1) on the faces which contribute

by their openings to the creation of the internal pressure. If there are several openings the largest value of z_e should be used to determine z_i .
The internal pressure coefficient of open silos and chimneys should be based on expression: c_{pi}=-0,6. The internal pressure coefficient of vented tanks with small openings should be based on expression : c_{pi}=-0,4

CALCULATION OF WIND LOADS IN OUR STRUCTURE

The structure will be analyzed with open and closed doors.The analysis and calculation of the loads in both cases is considered with a wind direction of 0°, 90° and -90°.In every case I will calculate first the coefficients of external and internal pressure for the vertical surfaces and the roof and afterwards the final pressures.

PEAK VELOCITY PRESSURE $q_p(z)$

The peak velocity is the same in both cases and should be determined from the expression:

$$q_{p(z)} = [1 + 7 \cdot I_v(z)] \cdot \frac{1}{2} \cdot \rho \cdot v_m^2(z) = c_e(z) \cdot q_b$$

Where:

$$I_v(z) = \frac{k_I}{c_0(z) \cdot \ln(z/z_0)} \quad \text{για} \quad z_{\min} \le z \le z_{\max}$$

$$I_v(z) = I_v(z_{\min}) \quad \text{για} \quad z \le z_{\min}$$

Because z_{min}=2m ≤ z=33,2m ≤ z_{max} we use the first expression

<u>FACTORS</u>

$C_0(z)$ = 1,0 (is the orography factor)

K_I = 1,0

Z_{max} taken as 200m

Z = 33,2 (height of structure)

z_0 =0.05 (from table for terrain category II)

So:

$$I_v(z) = \frac{k_I}{c_0(z)\cdot\ln(z/z_0)} = \frac{1}{1\cdot\ln 33{,}2/0{,}05}$$

The mean wind velocity is:

$$v_m(z) = c_r(z)\cdot c_0(z)\cdot v_b$$

where: $c_r(z) = k_r\cdot\ln(\frac{z}{z_0})$ for $z_{\min} \le z \le z_{\max} = 200m$

$c_r(z) = c_r(z_{\min}) = k_r\cdot\ln(\frac{z_{\min}}{z_0})$ for $z < z_{\min}$

$$k_r = 0{,}19\cdot(\frac{z_0}{z_{0,II}})^{0{,}07}$$

Because z_{min}=2m $\le$ z=33,2m $\le z_{max}$. we use the first expression

$$k_r = 0{,}19\cdot(\frac{z_0}{z_{0,II}})^{0{,}07} = 0{,}19\cdot(\frac{0{,}05}{0{,}05})^{0{,}07} = 0{,}19$$

The roughness factor is:

$$C_r\cdot(z) = k_r\cdot\ln\left(\frac{z}{z_0}\right) = 0{,}19\cdot\ln\left(\frac{33{,}2}{0{,}05}\right) = 1{,}22$$

The basic wind velocity is:

C_{dir} = 1,0 (directional factor)

C_{season} = 1,0 (seasonal factor)

$V_{b,0}$ = 33m/s (according to the national annex).

So: $v_b = c_{dir}\cdot c_{season}\cdot v_{b,0} = 1{,}0\cdot 1{,}0\cdot 33 = 33m/\sec$

And the mean wind velocity will be:

$$v_m(z) = c_r(z)\cdot c_0(z)\cdot v_b = 1{,}22\cdot 1{,}0\cdot 33m/\sec = 40{,}26m/\sec$$

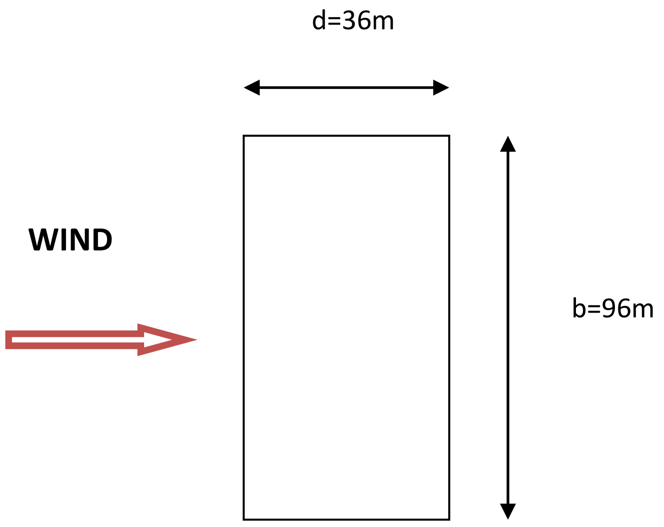
So the peak velocity pressure in height z is:

$$q_{p(z)} = [1+7\cdot I_v(z)]\cdot\frac{1}{2}\cdot\rho\cdot v_m^2(z) = [1+7\cdot 0{,}155]\cdot 0{,}5\cdot 0{,}00125\cdot 40{,}26^2 \Leftrightarrow q_{p(z)} = 2{,}11kN/m^2$$

$q_{p(z)}$* =2,11*kN/m^2

A)OPEN DOORS

●WIND DIRECTION θ=0°



The terrain category is II so the value of the basic wind velocity is $V_{b,0}$ = 33m/sec. The height of the building is 33,2.The dimension b is 96m.

Because h=33,2m≤ b=96m , z_e=h=33,2m

So: h=33,2m

b=96m

d=36m

z_e=h=33,2m

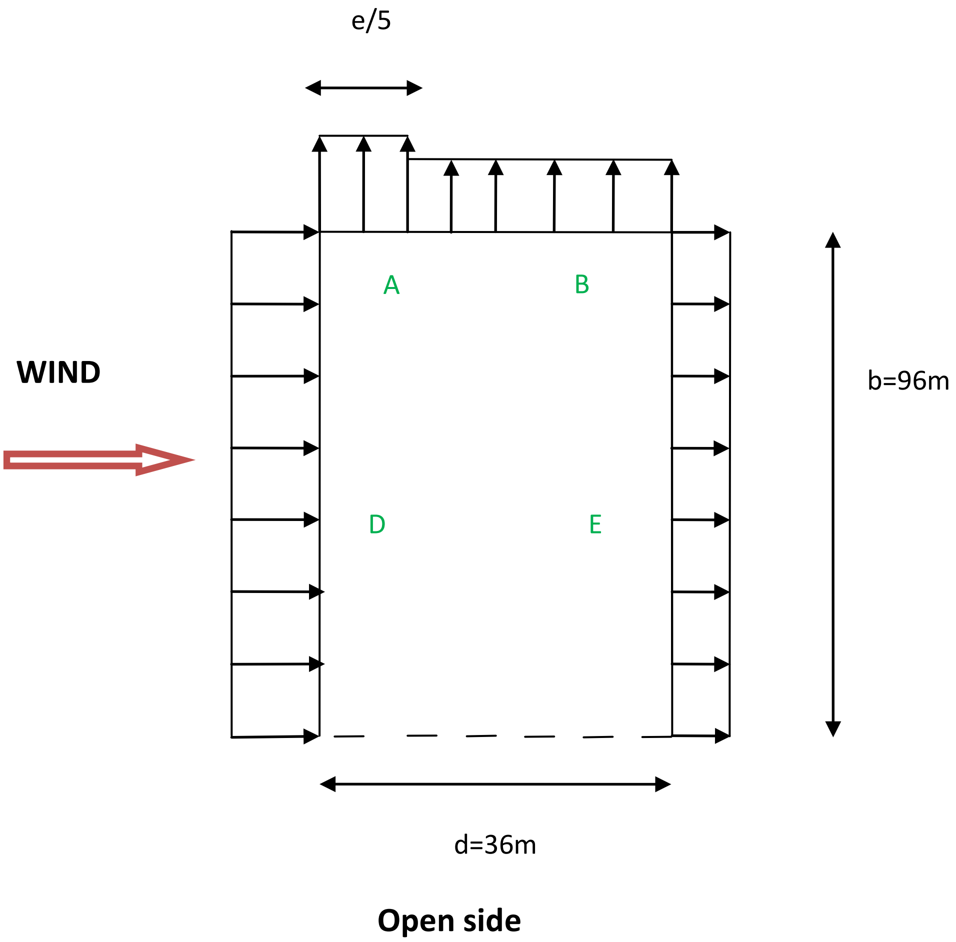
e=min(b,2h)=min(96, 66,4)=66,4m > d=36m

●external pressure coefficient

Using the table 2.3 and for h/d=33,2/36=0,93 we find the coefficients with linear interpolation:

zone	A	B	D	E
h/d	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
0,93	-1,2	-0,8	+0,8	-0,5

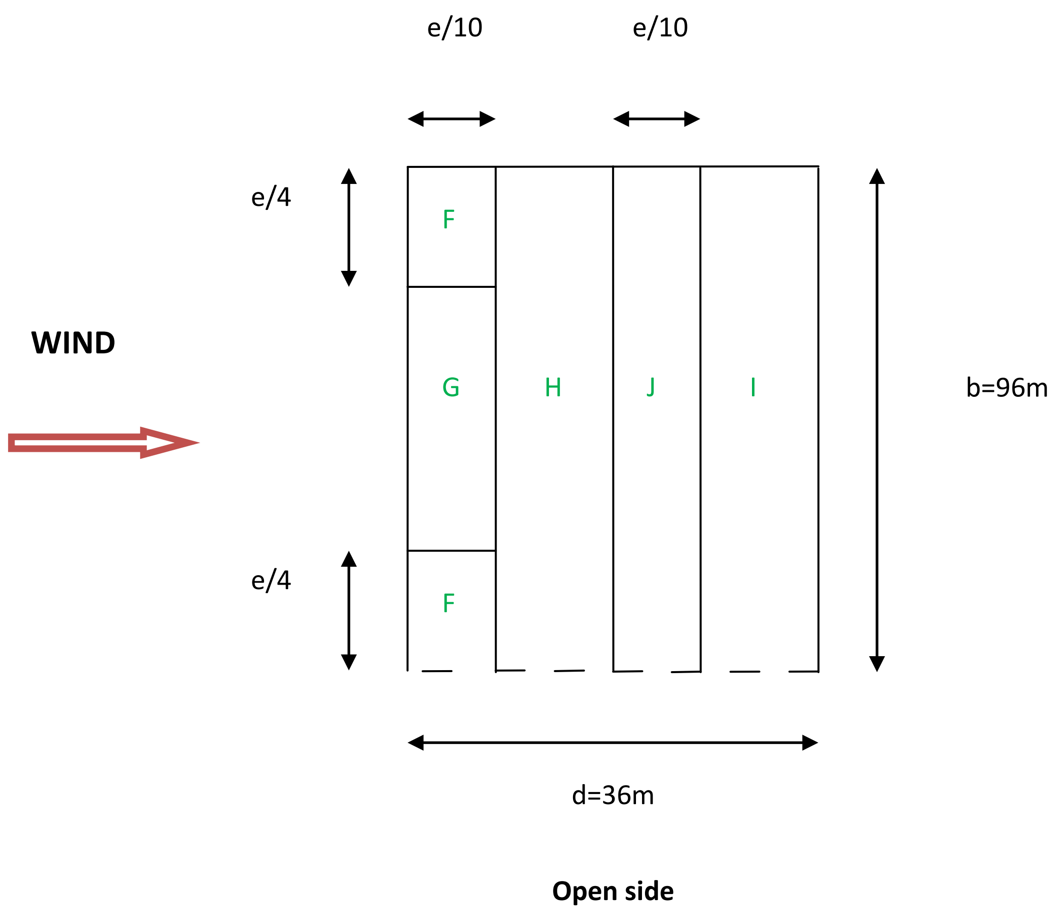
The following figure shows the external pressure coefficients of the vertical walls



Using the table 2.4 for wind direction θ=0° and pinch angle 11°,with linear interpolation we find the external pressure coefficients of the duopitch roof:

Pitch angle	ZONE				
	F	G	H	I	J
	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
Φ=11°	-1,18 +0,1	-0,94 +0,1	-0,405 +0,1	-0,47	-0,58

The following figure shows the external pressure coefficients for the duopitch roof:



●<u>internal pressure coefficient</u>

The internal pressure coefficient, *c*pi, depends on the size and distribution of the openings in the building.For the open side we use the expression: $C_{pi} = 0{,}75C_{pe}$

Where

*c*pe is the value for the external pressure coefficient at the openings in the dominant face. When these openings are located in zones with different values of external pressures an area weighted average value of *c*pe should be used. So:

Fot the vertical walls:

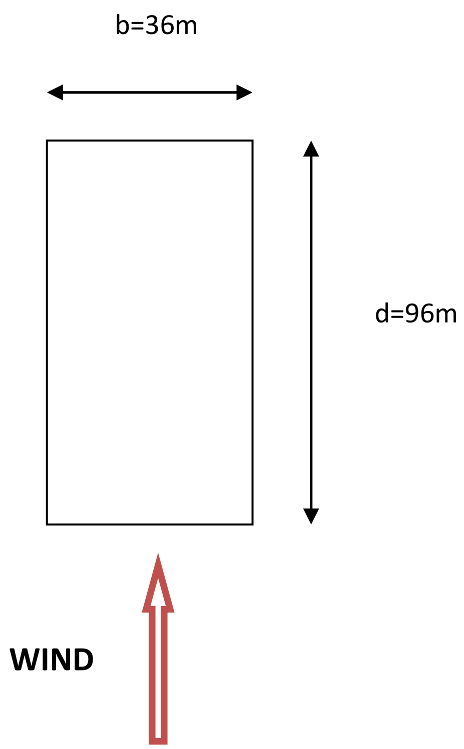
C_{pe}=0,38(-1,2)+0,62(-0,8)=0,952

For the duopitch roof:

C_{pe}=0,19(-1,18)+0,19(-0,58)+0,31(-0,405)+0,31(-0,47)= -0,61

C_{pe}=0,19(0,1)+0,31(0,1)+0,19(-0,5)+0,31(-0,5)= -0,2

●WIND DIRECTION θ=90°



In this case b=36m

Because h=33,2m ≤ b=36m , z_e=h=33,2m.

So: h=33,2m

b=36m

d=96m

z_e=h=33,2m

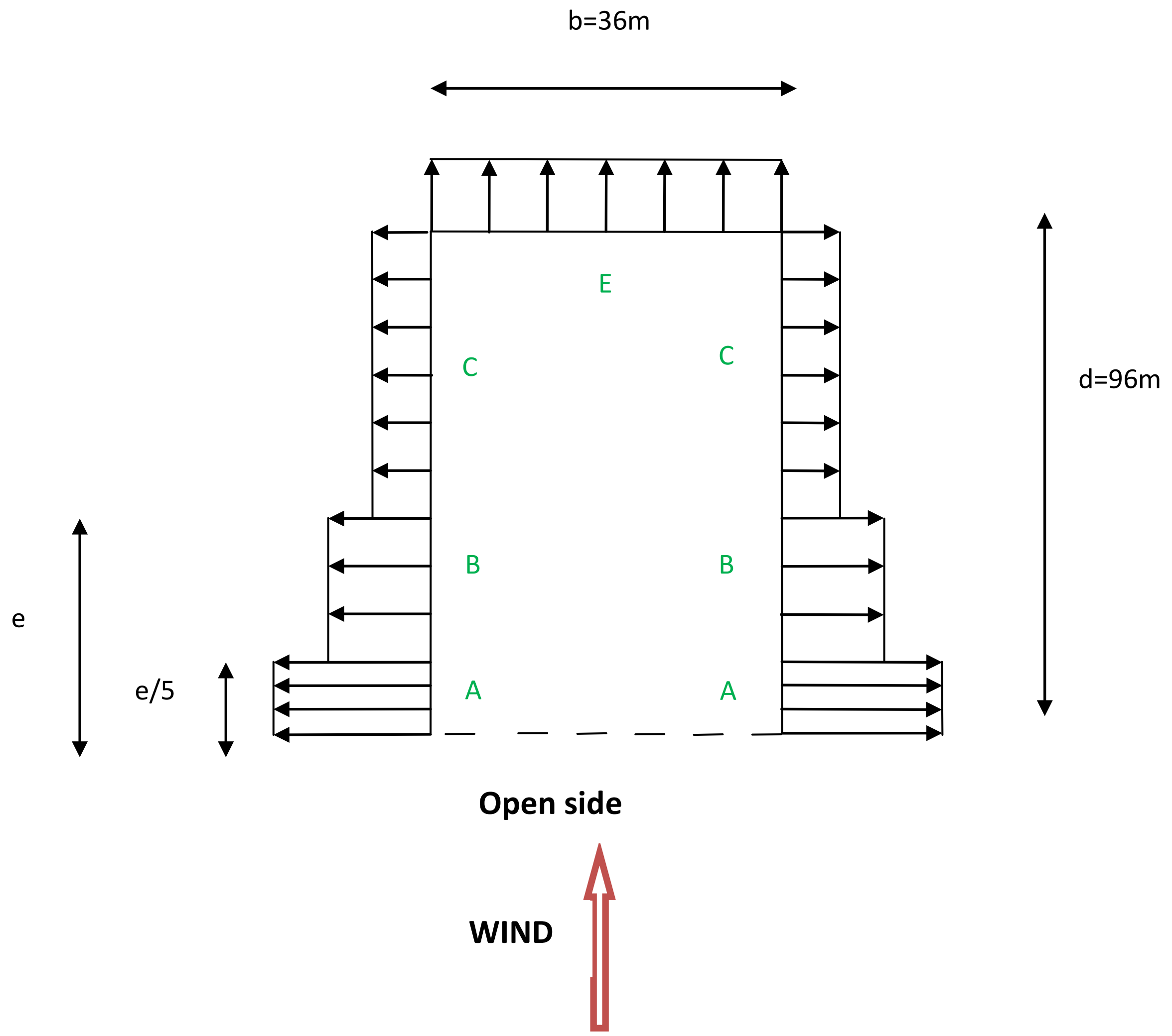
e = min(b,2h) = min(36 , 2·33,2) = 36m < d=96m

●<u>external pressure coefficient</u>

Using the table 2.3 and for h/d=33,2/96=0,34 we find the coefficients with linear interpolation:

zone	A	B	C	E
h/d	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
0,34	-1,2	-0,8	-0,5	-0,321

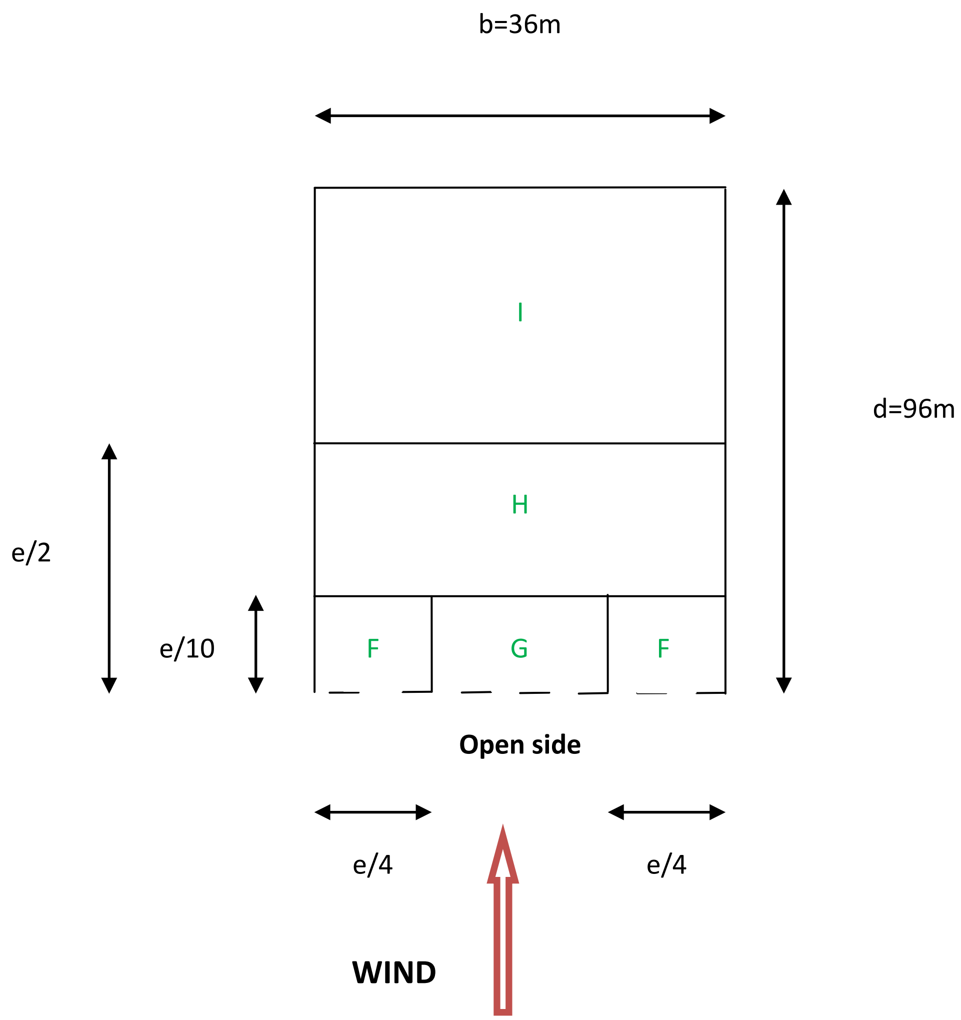
The following figure shows the coefficients of external pressure in vertical walls:



Using the table 2.4 for wind direction θ=90° and pinch angle 11°,with linear interpolation we find the external pressure coefficients of the duopitch roof:

Pitch angle	Zone			
	F	G	H	I
	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
φ=11°	-1,405	-1,3	-0,635	-0,535

The following figure shows the external pressure coefficients for the duopitch roof:



●<u>internal pressure coefficient</u>

The internal pressure coefficient, *c*pi, depends on the size and distribution of the openings in the building.For the open side we use the expression:

$C_{pi} = 0,75C_{pe}$

where *c*pe is the value for the external pressure coefficient at the openings in the dominant face. When these openings are located in zones with different values of external pressures an area weighted average value of *c*pe should be used. So:

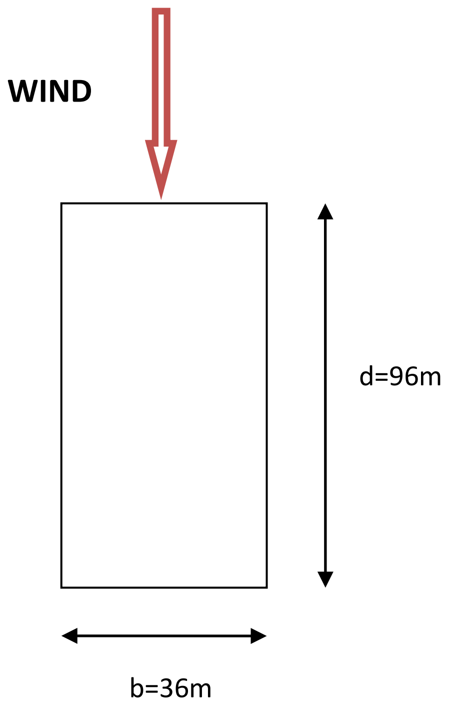
For the vertical walls:

C_{pe}=-0,321

For the duopitch roof:

C_{pe}=0,25·2·(-1.405)+0,5·(-1,3)= -1,353

●WIND DIRECTION θ=-90°



In this case b=36m

Because h=33,2m ≤ b=36m , z_e=h=33,2m.

So: h=33,2m

b=36m

d=96m

z_e=h=33,2m

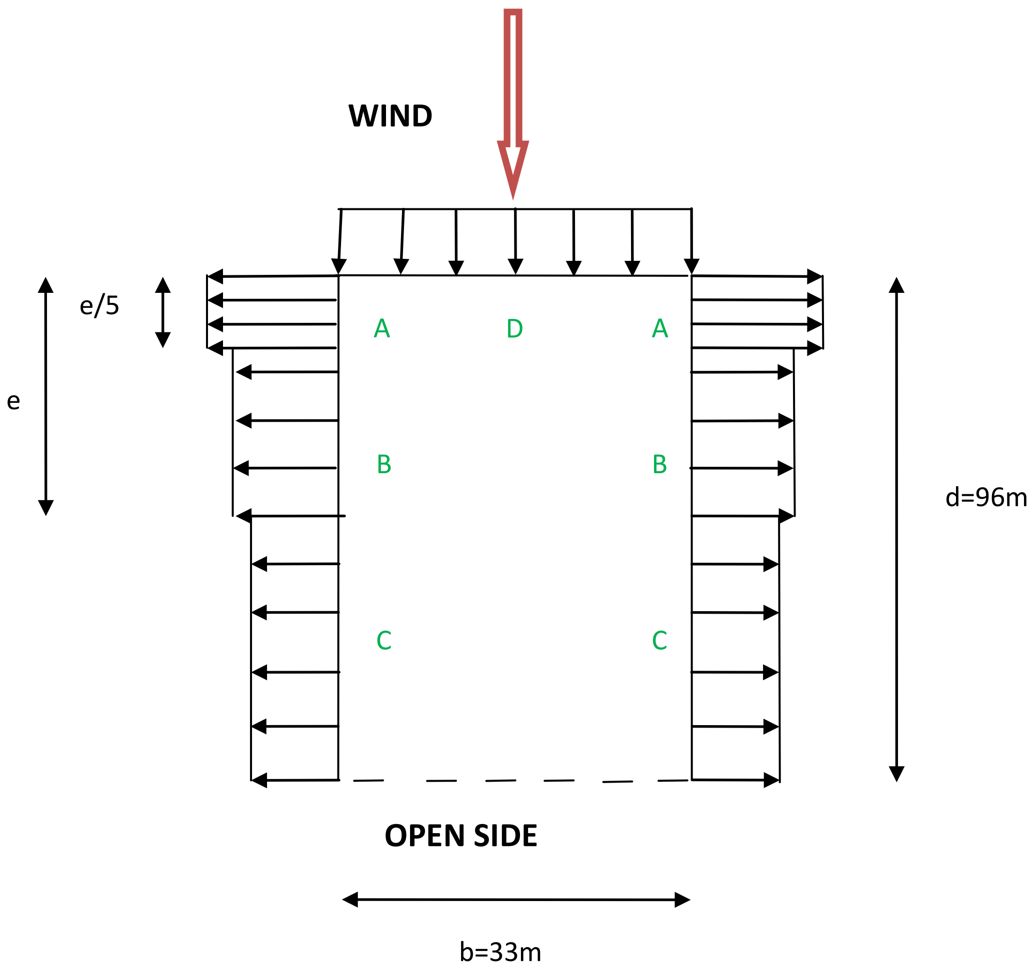
e = min(b,2h) = min(36 , 2·33,2) = 36m < d=96m

●external pressure coefficient

Using the table 2.3 and for h/d=33,2/96=0,34 we find the coefficients with linear interpolation:

Zone	**A**	**B**	**C**	**D**
h/d	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
0,34	-1,2	-0,8	-0,5	0,710

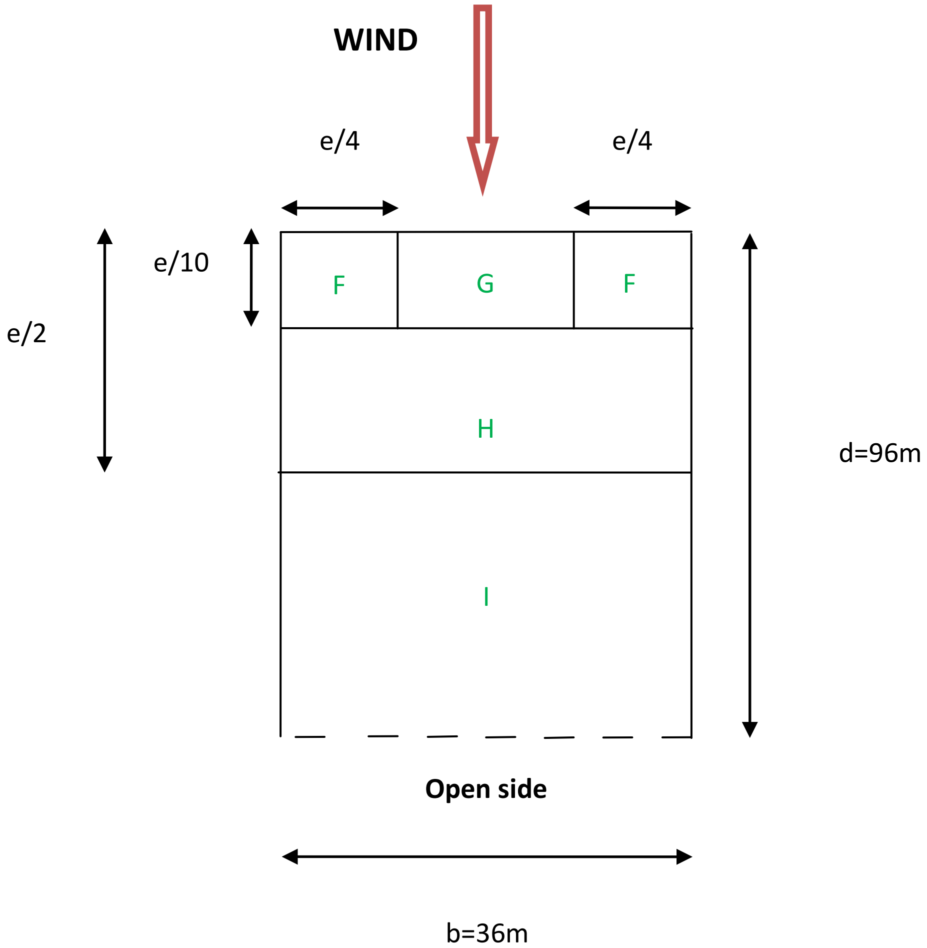
The following figure shows the coefficients of external pressure in vertical walls:



Using the table 2.4 for wind direction θ=-90° and pinch angle 11°,with linear interpolation we find the external pressure coefficients of the duopitch roof:

Pitch angle	Zone			
	F	G	H	I
	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
φ=11,5°	-1,405	-1,3	-0,635	-0,535

The following figure shows the external pressure coefficients for the duopitch roof:



● <u>internal pressure coefficient</u>

The internal pressure coefficient, *c*pi, depends on the size and distribution of the openings in the building. For the open side we use the expression:

$C_{pi} = 0{,}75C_{pe}$

where $c$pe is the value for the external pressure coefficient at the openings in the dominant face. When these openings are located in zones with different values of external pressures an area weighted average value of $c$pe should be used. So:

For the vertical walls:

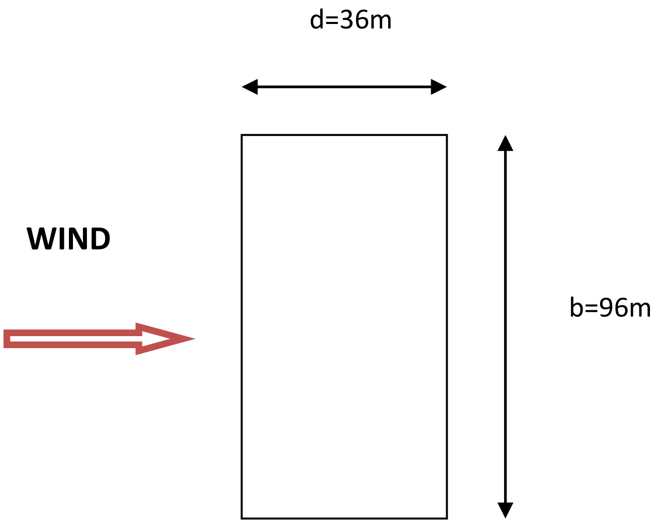
C_{pe}=0,710

For the duopitch roof:

C_{pe}= -0,535

B)CLOSED DOORS

●WIND DIRECTION θ=0°



The height of the building is 33,2.The dimension b is 96m.

Because h=33,2m≤ b=96m , z_e=h=33,2m

So: h=33,2m

b=96m

d=36m

z_e=h=33,2m

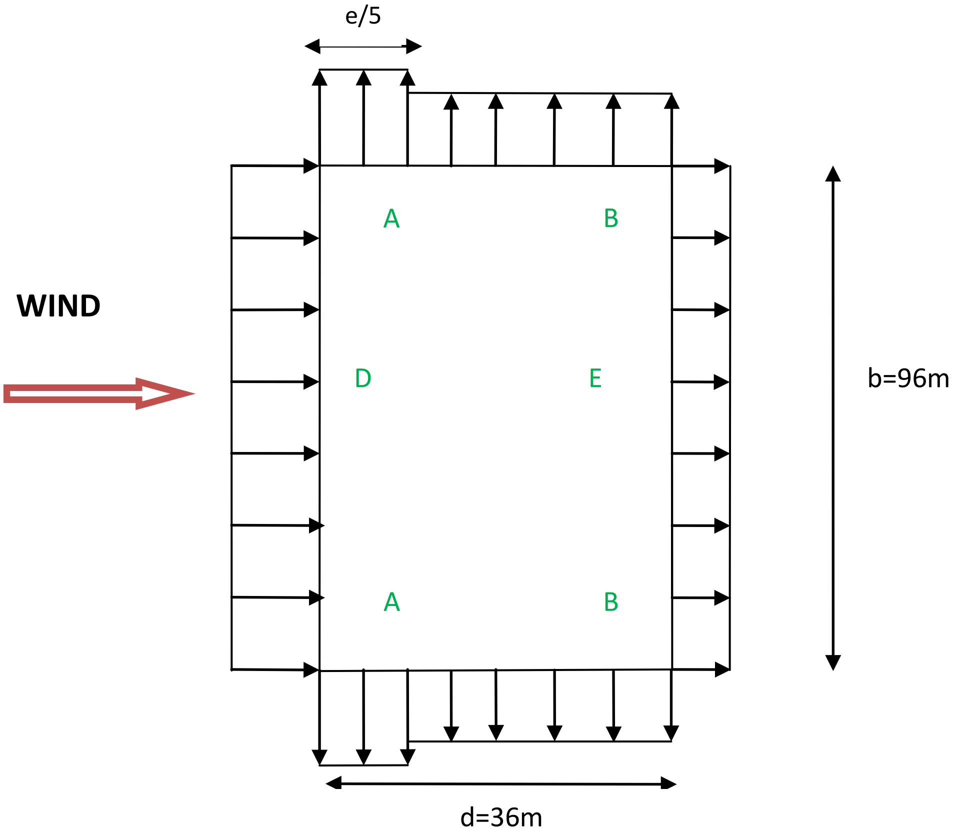
e=min(b,2h)=min(96, 66,4)=66,4m > d=36m

●<u>external pressure coefficient</u>

Using the table 2.3 and for h/d=33,2/36=0,93 we find the coefficients with linear interpolation:

zone	**A**	**B**	**D**	**E**
h/d	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
0,93	-1,2	-0,8	+0,8	-0,5

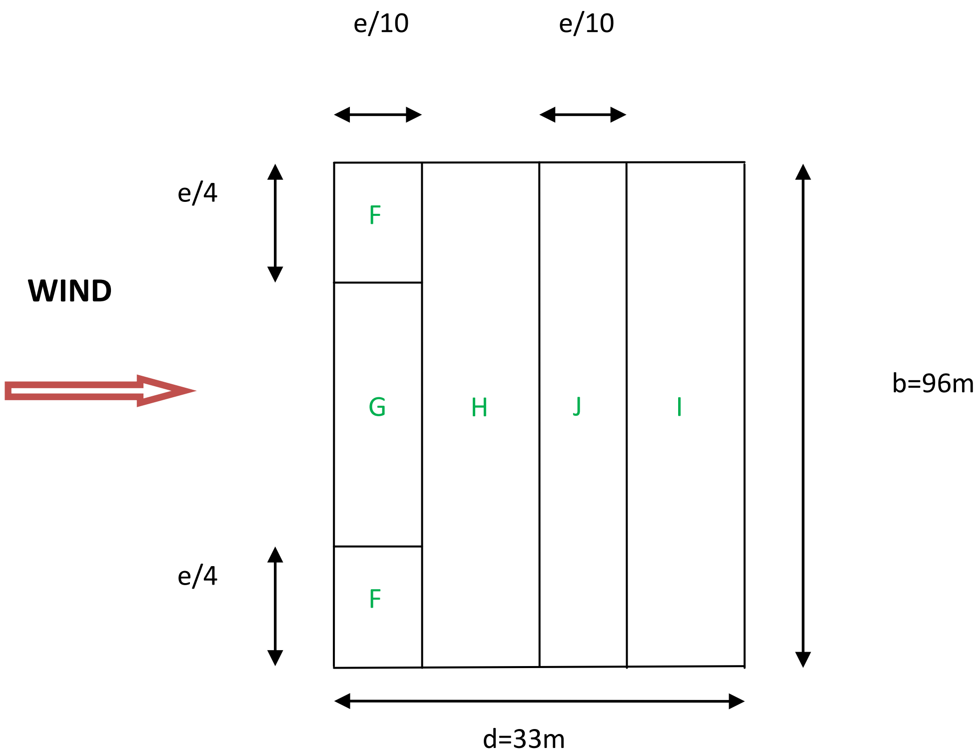
The following figure shows the external pressure coefficients of the vertical walls:



Using the table 2.4 for wind direction θ=0° and pinch angle 11°,with linear interpolation we find the external pressure coefficients of the duopitch roof:

Pitch angle	**ZONE**				
	F	**G**	**H**	**I**	**J**
	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
Φ=11°	-1,18 +0,1	-0,94 +0,1	-0,405 +0,1	-0,47	-0,58

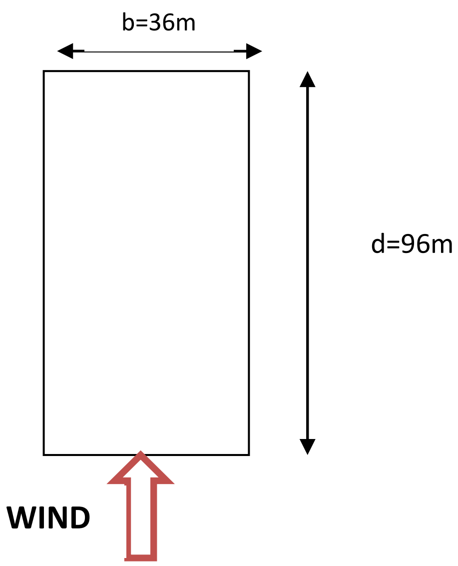
The following figure shows the external pressure coefficients for the duopitch roof:



●<u>internal pressure coefficient</u>

When the doors are closed the wind can’t enter the building so the interior pressure coefficient is equal to zero. : C_{pi}**=0**

●WIND DIRECTION θ=90°



In this case b=36m

Because h=33,2m ≤ b=36m , z_e=h=33,2m.

So: h=33,2m

b=36m

d=96m

z_e=h=33,2m

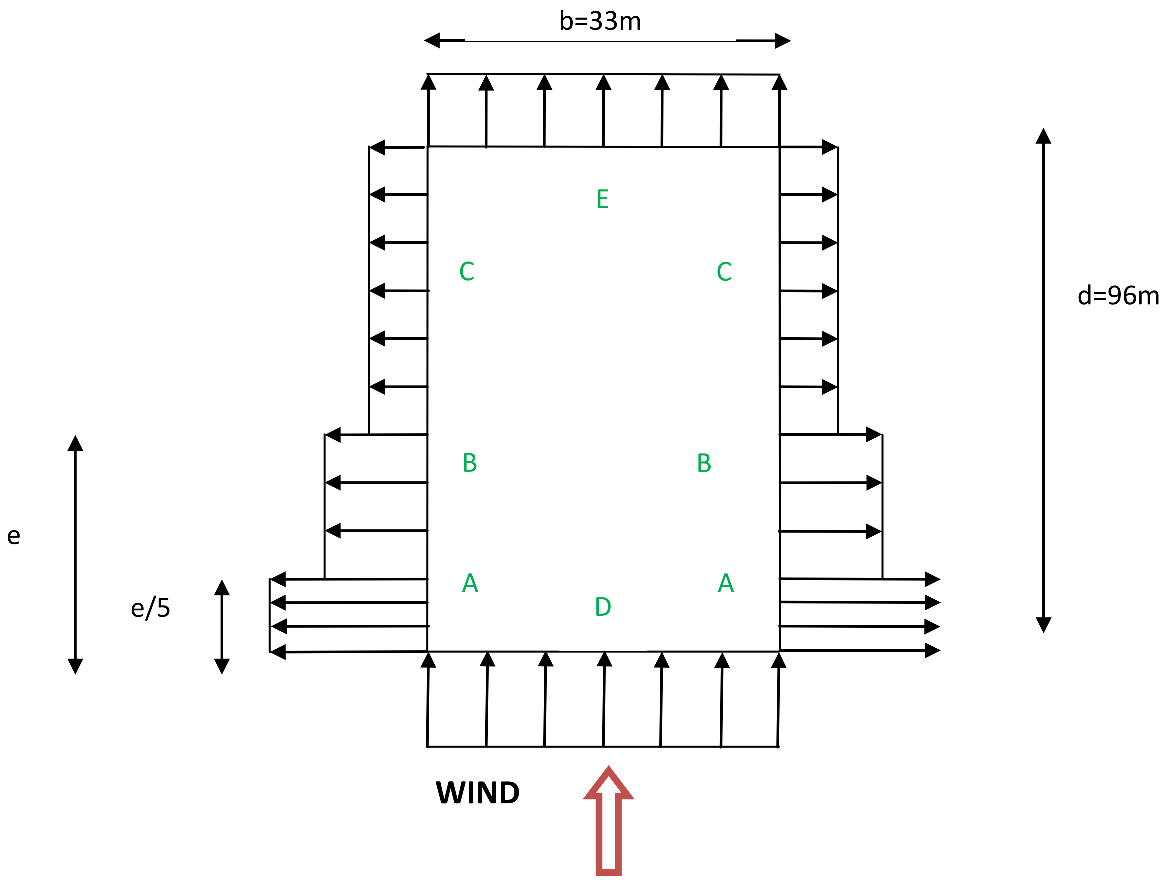
e = min(b,2h) = min(36 , 2·33,2) = 36m < d=96m

●external pressure coefficient

Using the table 2.3 and for h/d=33,2/96=0,34 we find the coefficients with linear interpolation:

ZONE	**A**	**B**	**C**	**D**	**E**
h/d	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
0,34	-1,2	-0,8	-0,5	+0,71	-0,321

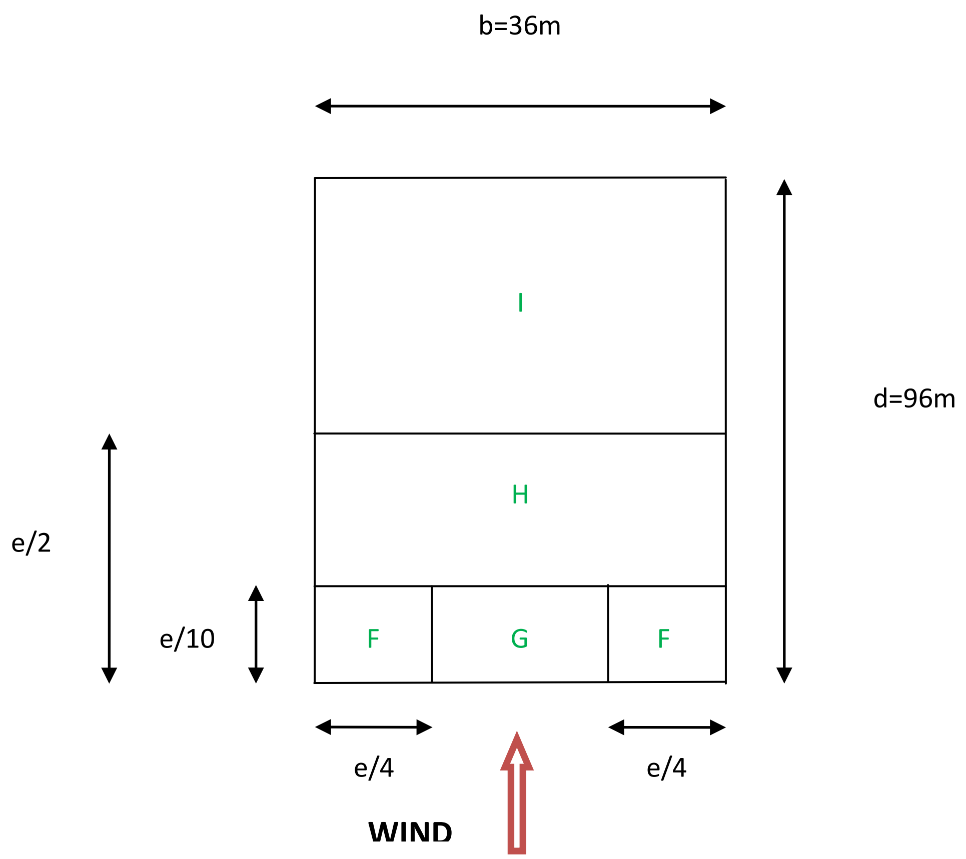
The following figure shows the external pressure coefficients of the vertical walls:



Using the table 2.4 for wind direction θ=90° and pinch angle 11°,with linear interpolation we find the external pressure coefficients of the duopitch roof:

Pitch angle	**Zone**			
	F	**G**	**H**	**I**
	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
φ=11°	-1,405	-1,3	-0,635	-0,535

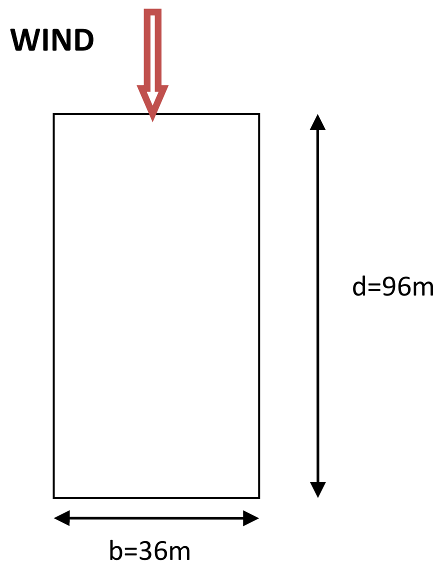
The following figure shows the external pressure coefficients for the duopitch roof:



●<u>internal pressure coefficient</u>

When the doors are closed the wind can't enter the building so the interior pressure coefficient is equal to zero. : C_{pi}=0

●WIND DIRECTION θ=-90°



In this case b=36m

Because h=33,2m ≤ b=36m , z_e=h=33,2m.

So: h=33,2m

b=36m

d=96m

z_e=h=33,2m

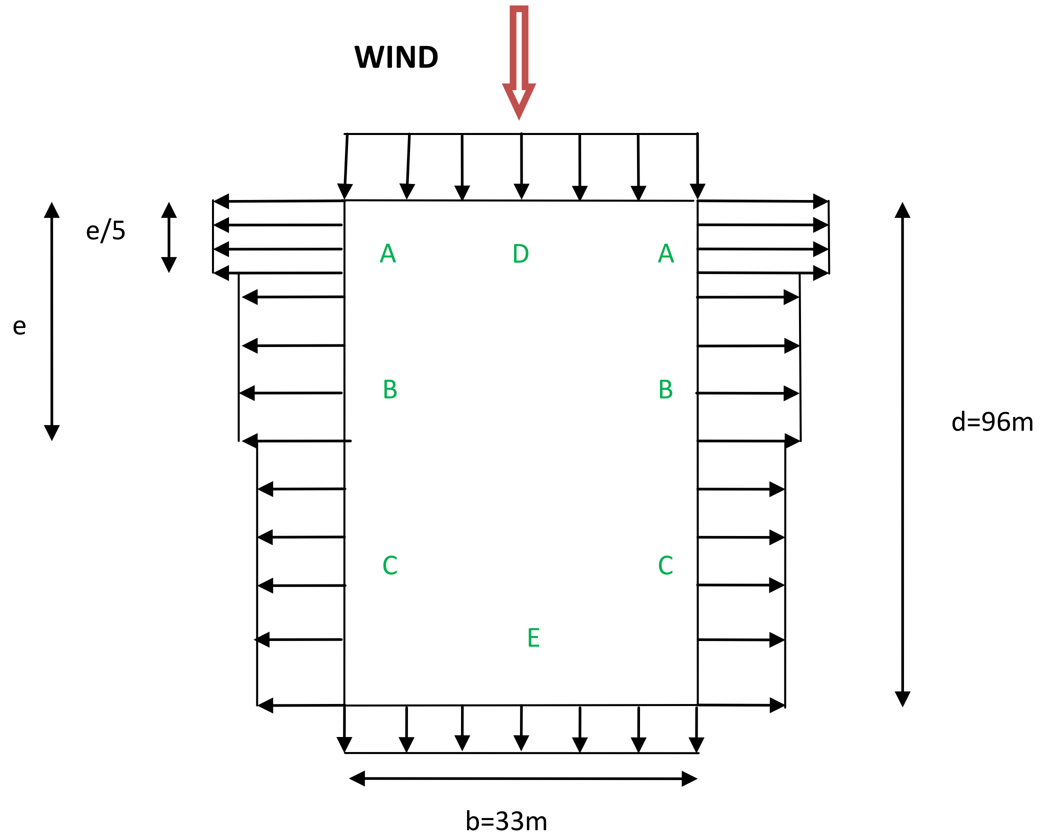
e = min(b,2h) = min(36 , 2·33,2) = 36m < d=96m

●<u>external pressure coefficient</u>

Using the table 2.3 and for h/d=33,2/96=0,34 we find the coefficients with linear interpolation:

Zone	**A**	**B**	**C**	**D**	**E**
h/d	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
0,34	-1,2	-0,8	-0,5	+0,71	-0,321

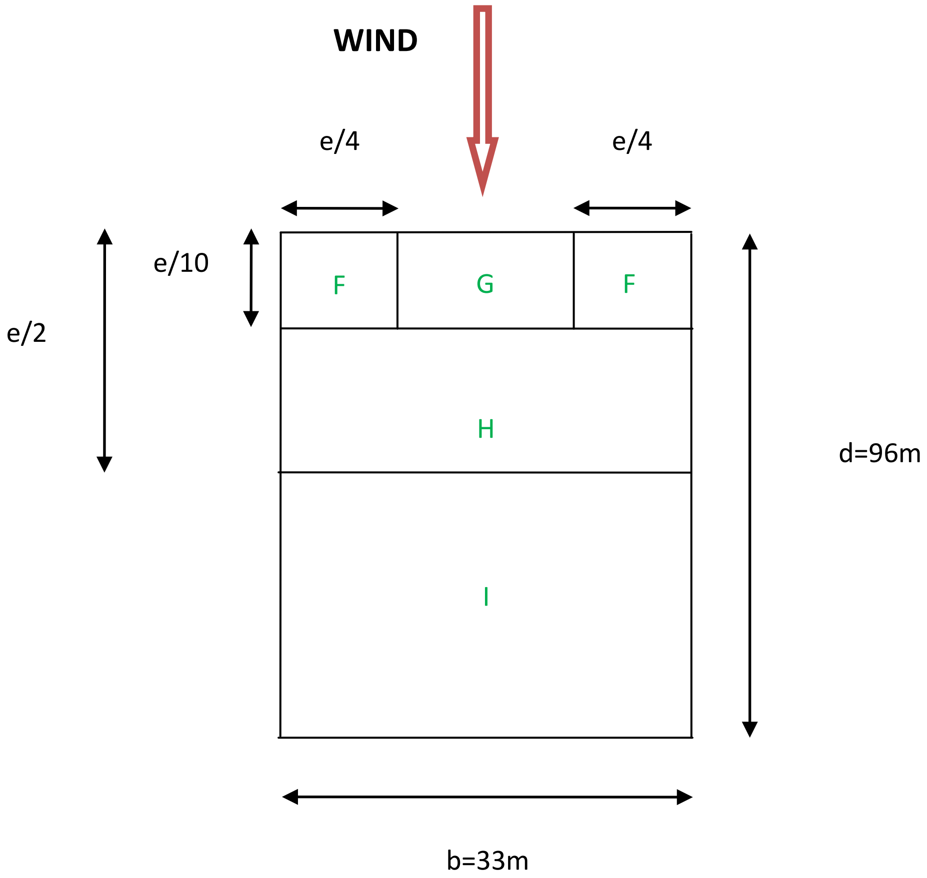
The following figure shows the external pressure coefficients of the vertical walls:



Using the table 2.4 for wind direction θ=90° and pinch angle 11°,with linear interpolation we find the external pressure coefficients of the duopitch roof:

Pitch angle	Zone			
	F	G	H	I
	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
φ=11°	-1,405	-1,3	-0,635	-0,535

The following figure shows the external pressure coefficients for the duopitch roof:



●<u>internal pressure coefficient</u>

When the doors are closed the wind can't enter the building so the interior pressure coefficient is equal to zero. : $\boldsymbol{C_{pi}=0}$

NET PRESSURE IN WALLS AND DUOPITCH

The net pressure is the difference between the pressures on the opposite surfaces taking due account of their signs. Pressure, directed towards the surface is taken as positive, and suction, directed away from the surface as negative.

The wind pressure acting on the external surfaces, w_e , should be obtained from the expression:

$$W_e = q_p(z_e)\cdot c_{pe} = 2{,}11\cdot c_{pe} \qquad (kN/m^2)$$

The wind pressure acting on the internal surfaces of a structure, *wi* , should be obtained from the expression:

$$W_i = q_p(z_i)\cdot c_{pi} = 2{,}11\cdot 0{,}75\cdot c_{pe} = 1{,}583\cdot c_{pe} \qquad (kN/m^2)$$

The final pressures are the algebraic sum of W_e and W_i for every surface in the walls and the roof.

$W = W_e + W_i \qquad (kN/m^2)$

According to the expressions above, the following tables show the final pressure in the walls and the roof for the two cases of open and closed doors, for different wind directions.

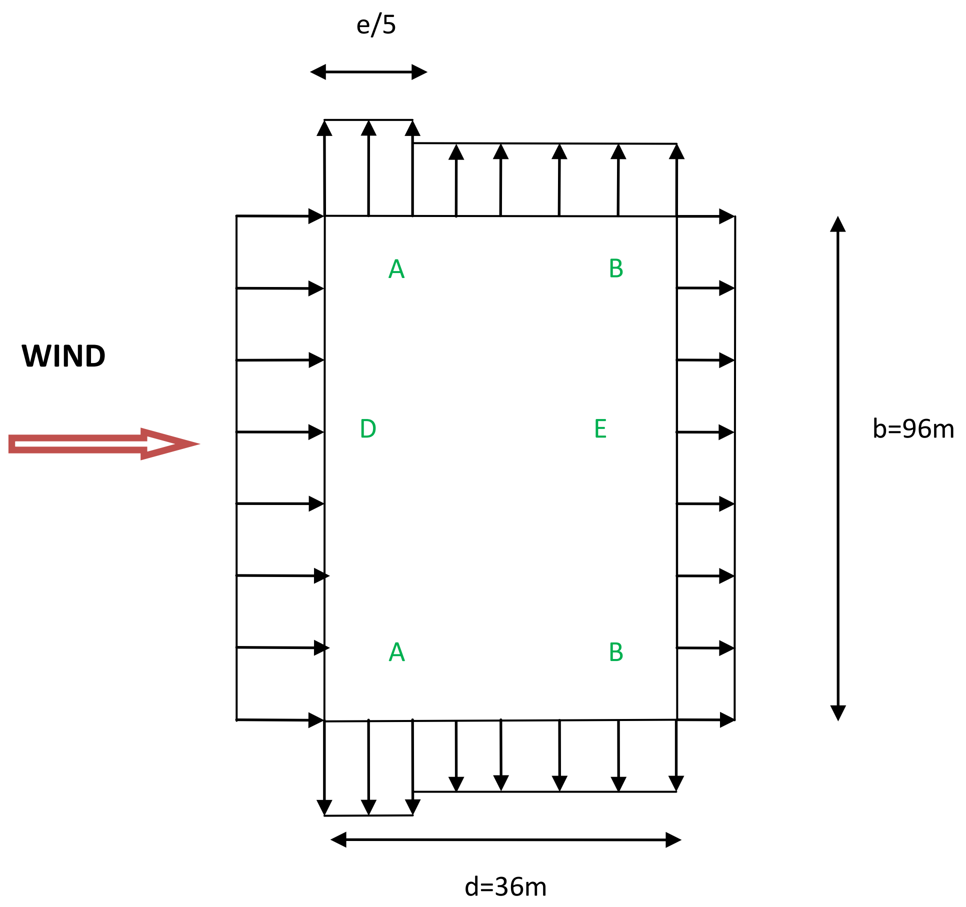
CLOSED DOORS

Zones / Wind directions	**A**	**B**	**C**	**D**	**E**	**F**	**G**	**H**	**I**	**J**
0°	-2,532	-1,688	-	1,688	-1,055	-2,49 +0,21	-1,983 +0,21	-0,854 +0,21	-0,992	-1,223
90°	-2,532	-1,688	-1,055	1,498	-0,677	-2,965	-2,743	-1,34	-1,129	-
-90°	-2,532	-1,688	-1,055	1,498	-0,677	-2,965	-2,743	-1,34	-1,129	-

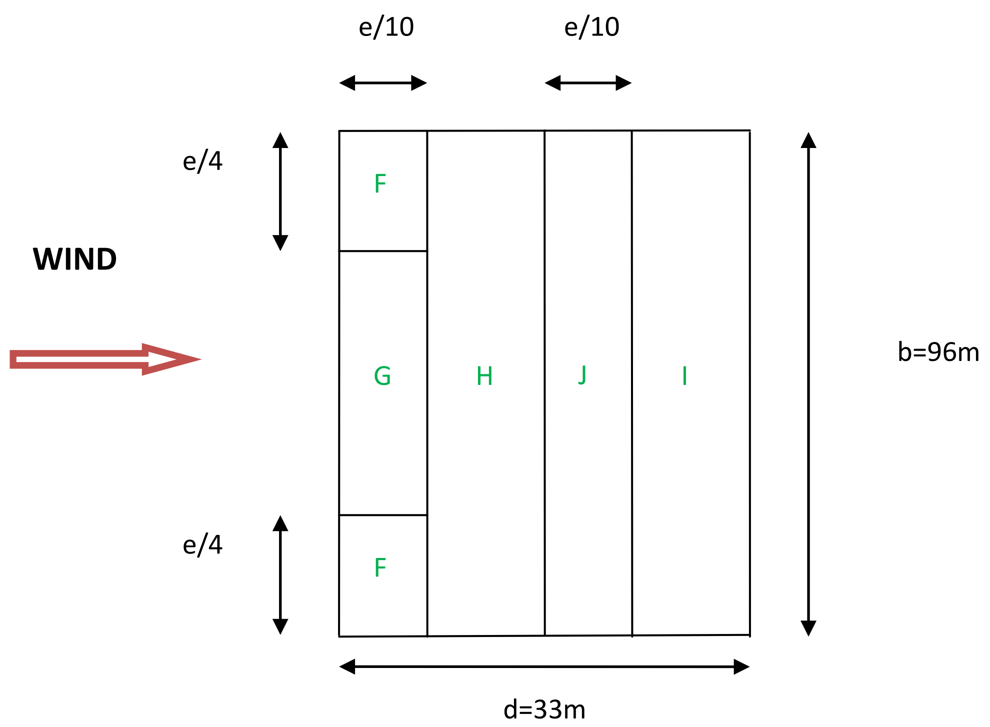
Table 2.6-Wind pressure for closed doors

The units in the table are kN/m^2

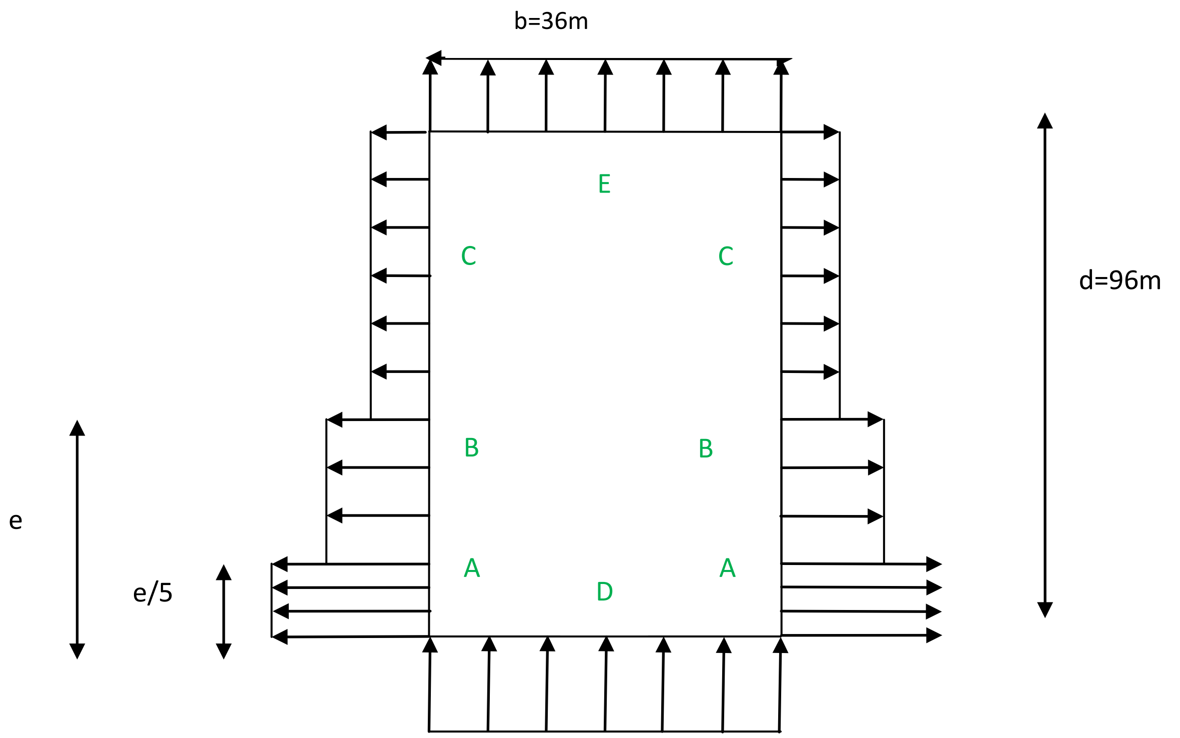
•CLOSED DOORS, θ=0 (VERTICAL WALLS)



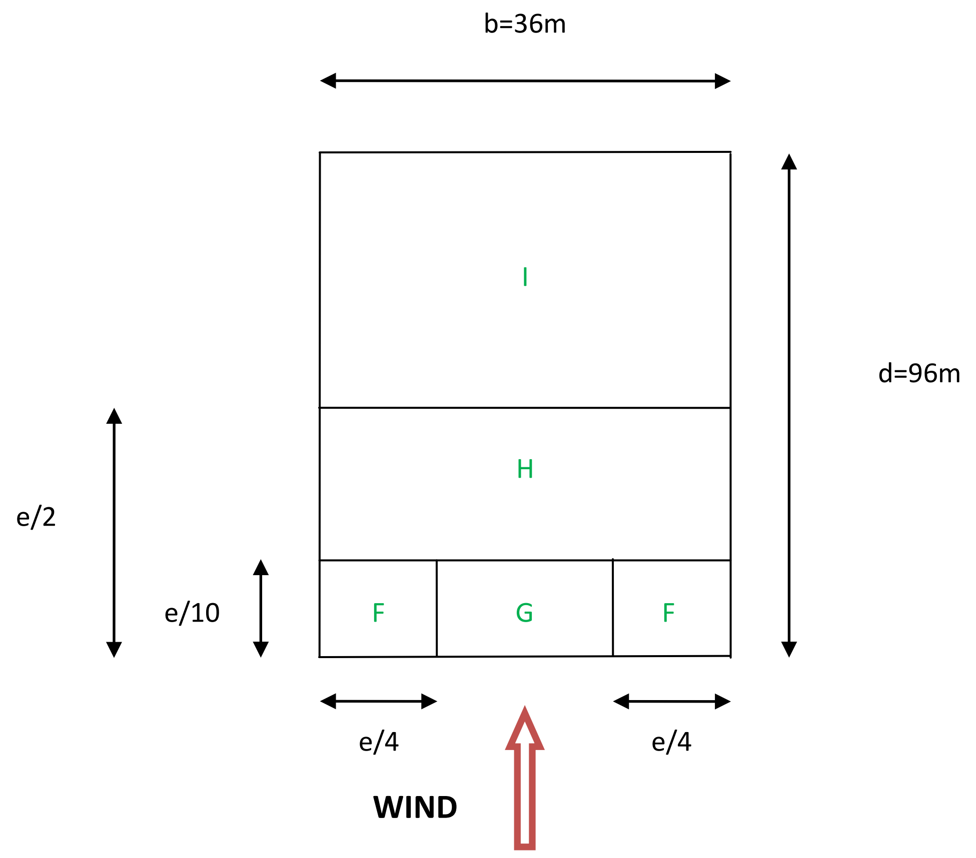
•CLOSED DOORS, θ=0 (DUOPITCH ROOF)



•CLOSED DOORS , θ=90° OR θ=-90° (VERTICAL WALLS)



•CLOSED DOORS , θ=90° OR θ=-90° (DUOPITCH ROOF)



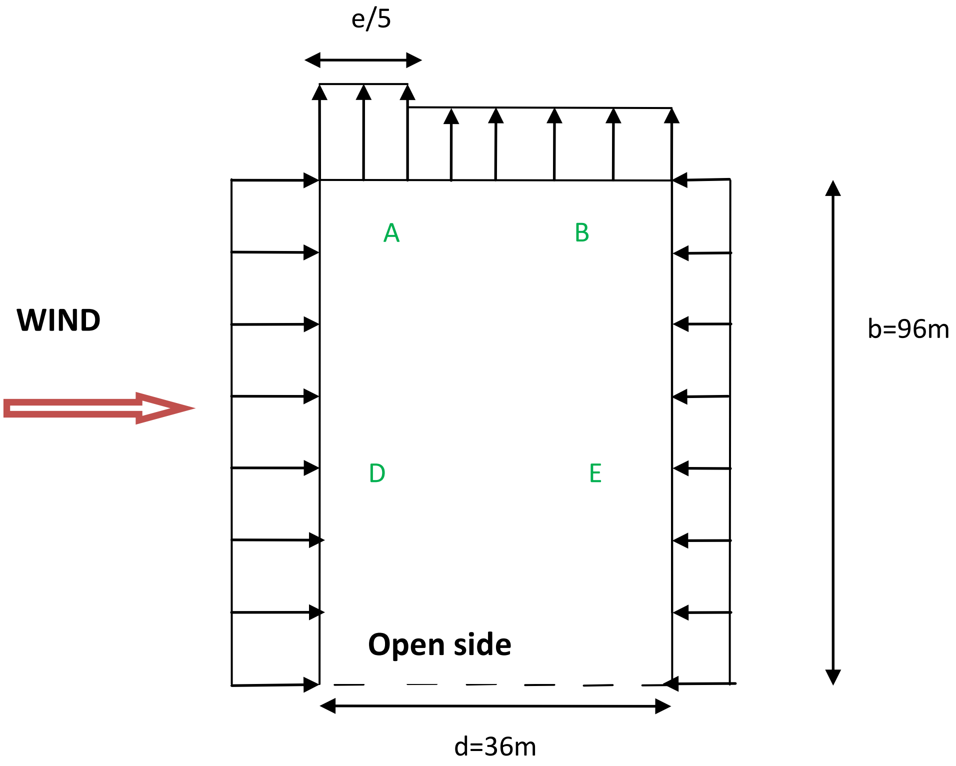
OPEN DOORS

Zones / Wind direction	A	B	C	D	E	F	G	H	I	J
0°	- 1,028	- 0,184	-	+3,192	+0,407	- 1,526 +0,53	- 1,020 +0,53	- 0,110 +0,53	- 0,028	- 0,260
90°	- 3,654	- 2,810	- 2,177	-	-1,799	- 5,103	- 4,881	- 3,478	- 3,267	-
-90°	- 2,025	- 1,118	- 0,548	+2,00	-	- 2,120	- 1,898	- 0,495	- 0,284	-

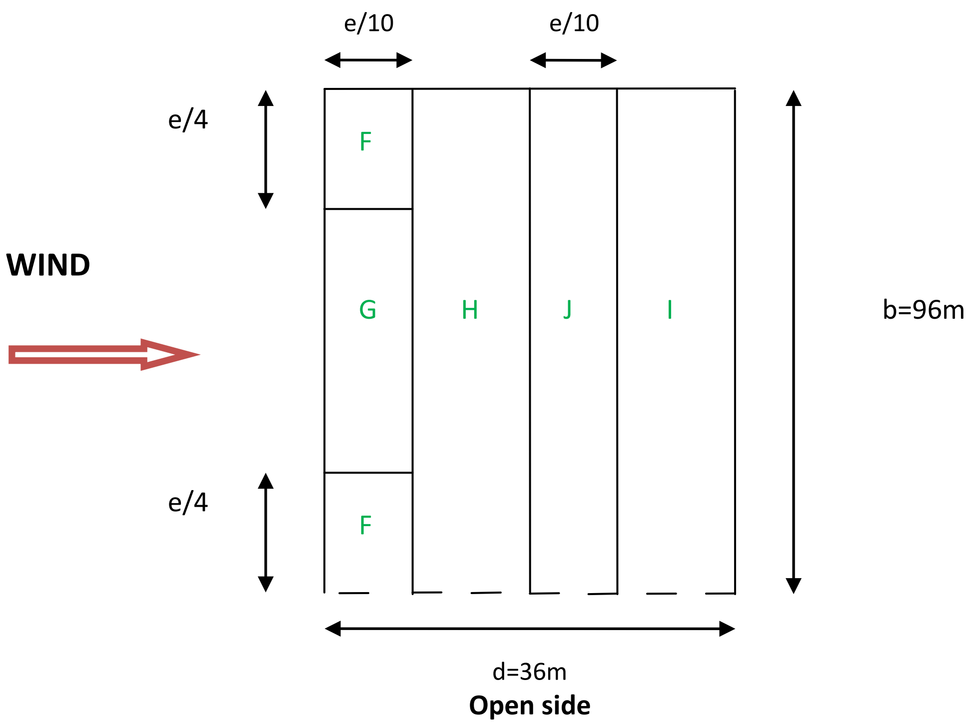
Table 2.7-wind pressure for open doors

The units of the table are kN/m^2

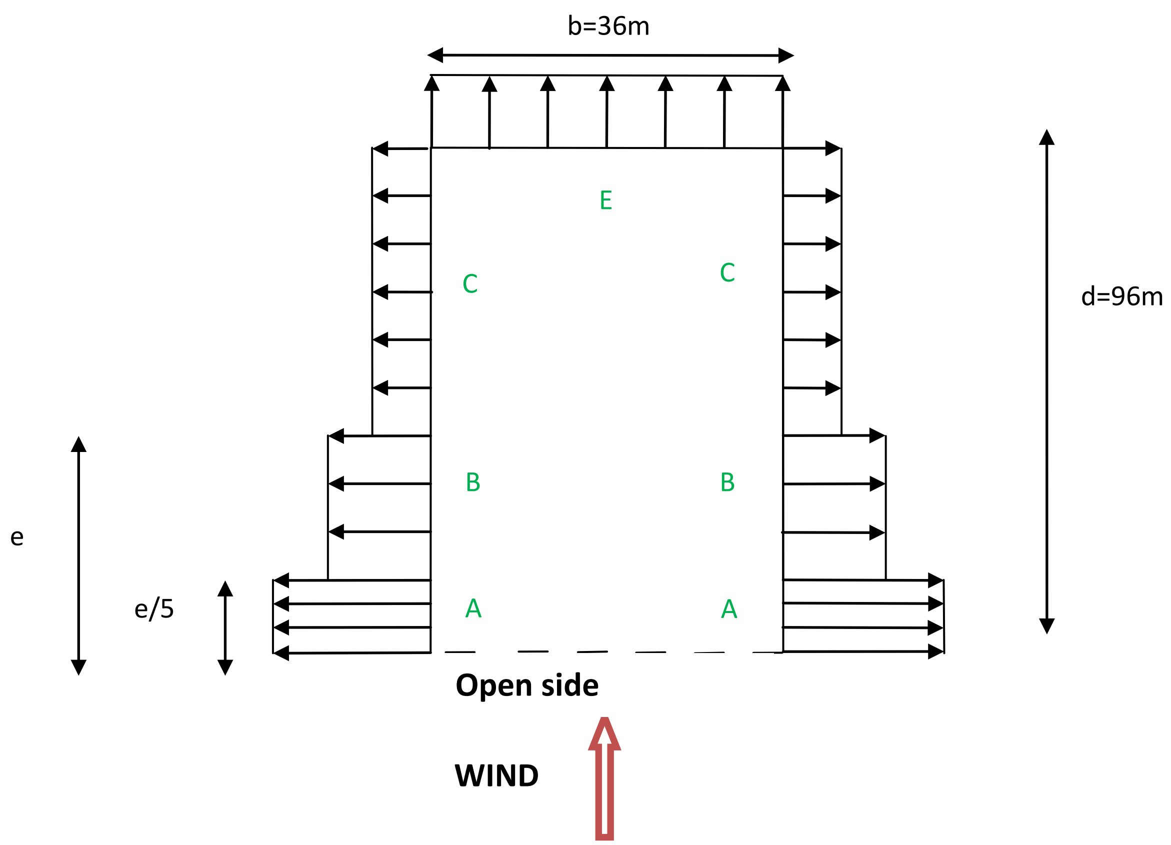
●OPEN DOORS, θ=0 (VERTICAL WALLS)



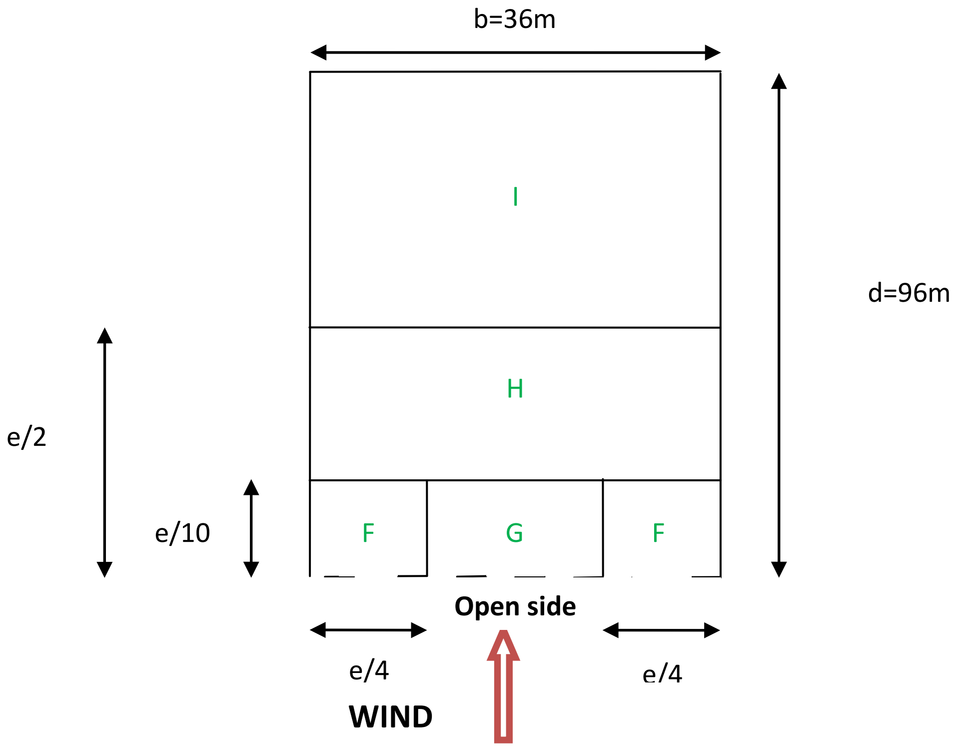
•**OPEN DOORS, θ=0 (DUOPITCH ROOF)**



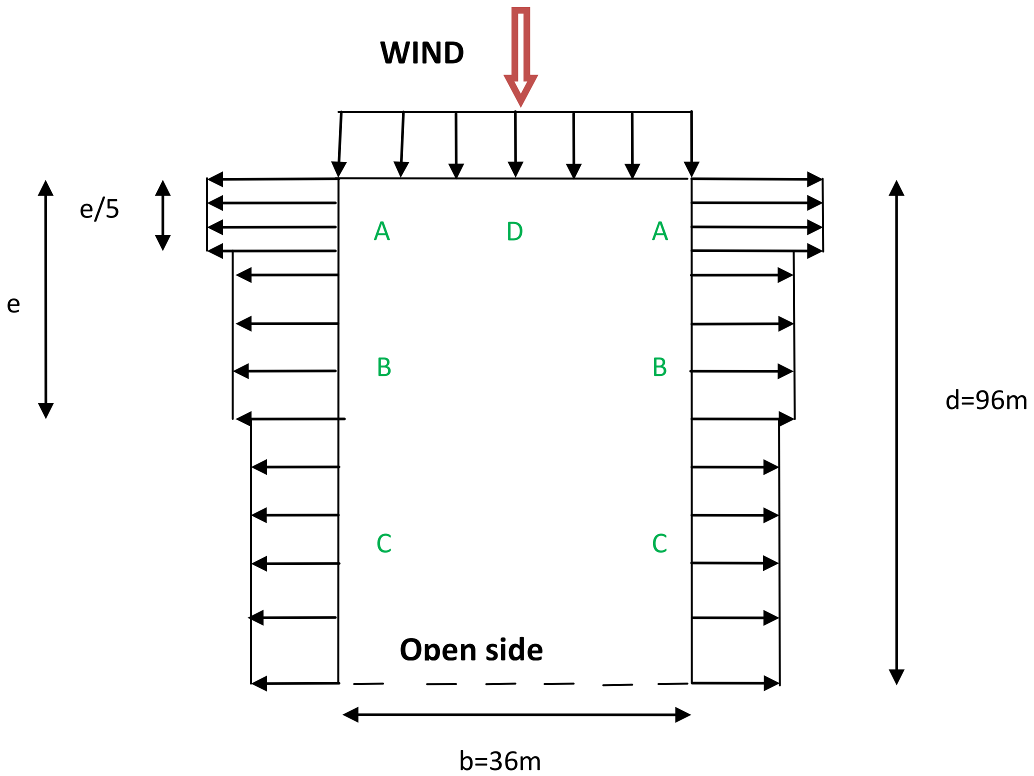
•**OPEN DOORS , θ=90° (VERTICAL WALLS)**



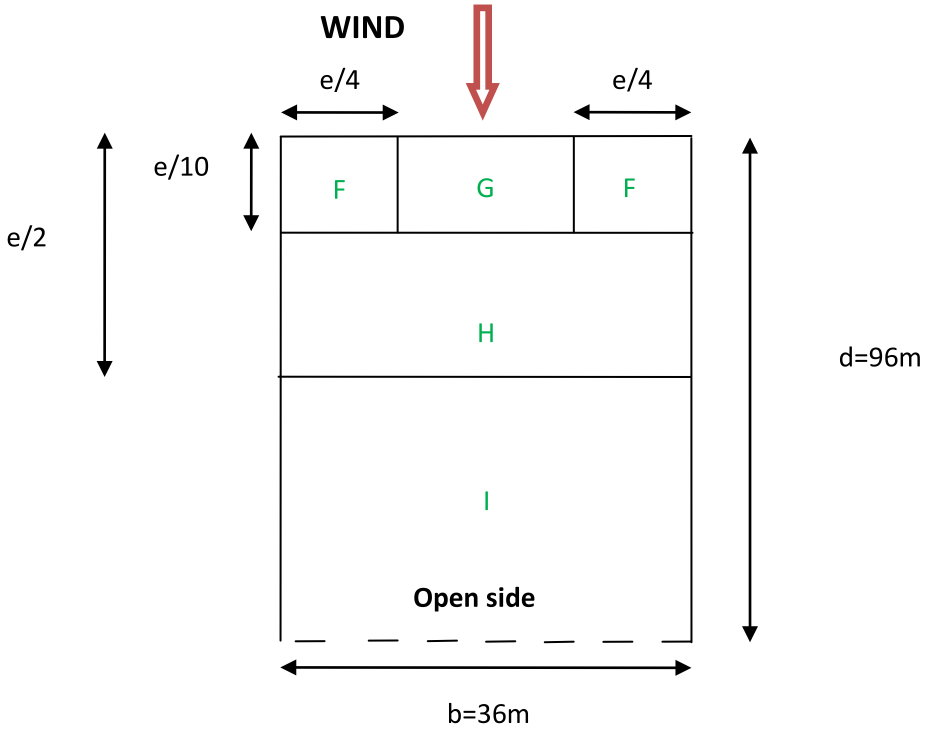
●**OPEN DOORS , θ=90 (DUOPITCH ROOF)**



●**OPEN DOORS , θ=-90° (VERTICAL WALLS)**



●**OPEN DOORS , θ=-90 (DUOPITCH ROOF)**



2.4 EARTHQUAKE LOADS

During the earthquake, the ground and so the base of a structure , moves fast, with an alternate sign. On a dynamic approach, the important quantity is the acceleration that the structure receives. The mass of the structure, due to inertia, doesn't follows the motion of the base, but follows its own oscillation. That different motion of the mass and the base of the structure creates deformations. The necessary conditions for the creation of seismic forces is mass.

The structure receives alternate displacements due to the displacement of the ground. It reacts to these displacements with its resistance to bend, tension, compression etc. So the earthquake doesn't create forces but displacements and the structure reacts by internal forces. That's why earthquake loads are considered as indirect actions.

According to the Greek anti seismic code EAK 2000, we use the Dynamic Spectrum Method, according to which, we examine our structure as a space model, not as a plan model.

The design spectrum is described by the expressions:

$$0 \leq T \leq T_1 \quad : \quad \Phi_d(T) = \gamma_1 \cdot A \left[1 + \frac{T}{T_1} \cdot \left(\eta \cdot \frac{\theta}{q} \cdot \beta - 1\right)\right]$$

$T_1 \leq T \leq T_2$: $\Phi_d(T) = \gamma_1 \cdot A \cdot \eta \cdot \frac{\theta}{q} \cdot \beta_0$

$T_2 \leq T$: $\Phi_d(T) = \gamma_1 \cdot A \cdot \eta \cdot \frac{\theta}{q} \cdot \beta_0 \cdot \left(\frac{T_2}{T}\right)^{2/3}$

Where:

A :is the maximum seismic ground acceleration. $A = \alpha \cdot g$, where α is the ground acceleration related to the gravity acceleration. Values for α in the table

γ_1 : is the factor of the importance of the building, given in the table

q : is the factor of ductility or behaviour, given in the table

θ :is the factor of the influence of the foundation and it depends on the depth and the stiffness of the foundation. For terrain category A and B, $\theta=1$

T_1 και T_2 : are the phasma periods related to the foundation ground dangerousness, given in the table

- structure with $T<T_1$, is considered with high stiffness, as $T \sim 1/K$
- structure with $T>T_2$ is considered with low stiffness.

β_0 : is the factor of spectrum acceleration.

Terrain category	**A**	**B**	**Γ**	**Δ**
T_1	0,10	0,15	0,20	0,20
T_2	0,40	0,60	0,80	1,20

Table 2.8-Periods T_1 , T_2 (sec)

The terrain category for our structure is B. So : **T_1 = 0,15 sec** και **T_2 = 0,60 sec**

The following figure shows the three zones of seismic dangerousness in Greece:

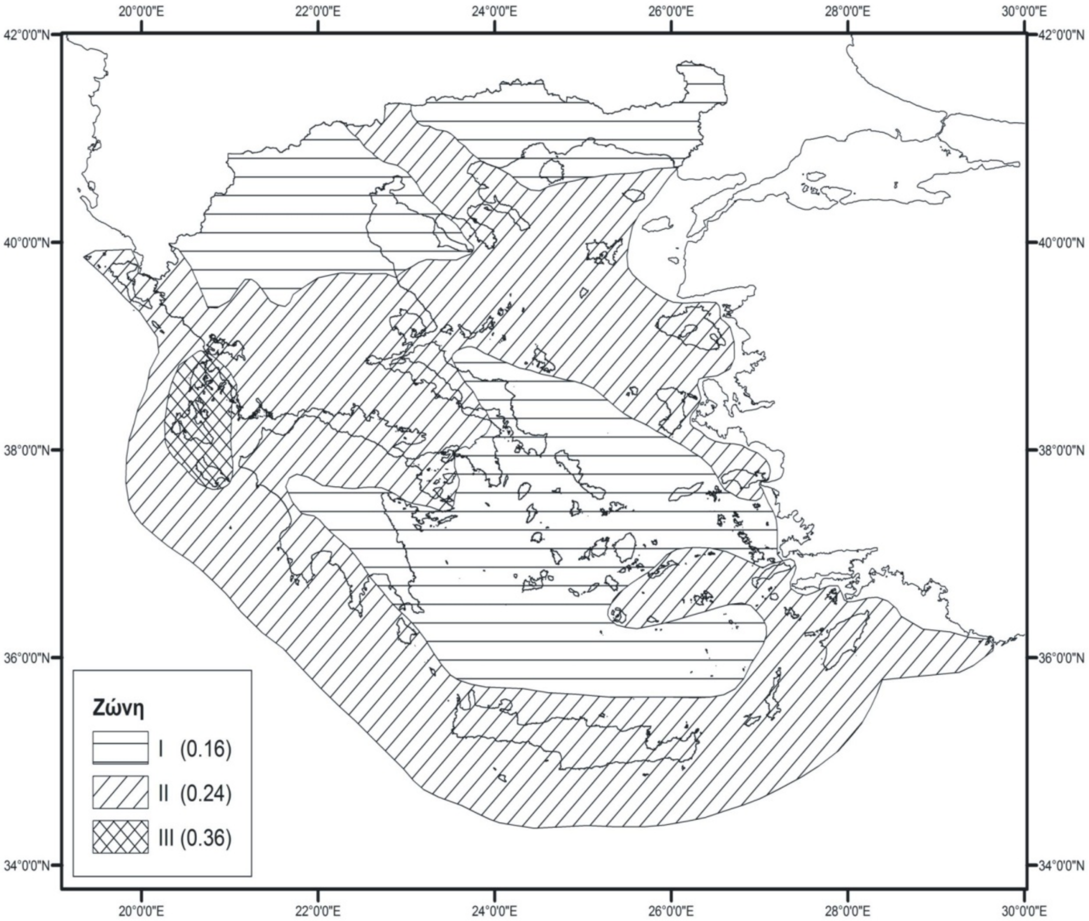


Figure 2.9-Zones of seismic dangerousness in Greece

Zone of seismic danderousness	**I**	**II**	**III**
α	0,16	0,24	0,36

Table 2.9-ground acceleration related to zones of seismic dangerousness

Our structure is located in the region of Scaramangas, in the west Athens. According to the map above is in the Zone I:

$$\mathbf{A = \alpha \cdot g = 0{,}16g}$$

Importance class		γ_1
S_1	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.	0,85
S_2	Ordinary buildings, houses, offices, industrial buildings, hotels, etc.	1,00
S_3	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions, etc.	1,15
S_4	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.	1,30

Table 2.10-factors of building importance

Our industrial building belongs to the category S_2, so **γ_1=1,00.**

The building has bolt connections, so **ζ=4%.** The correction factor is:

$$\eta = \sqrt{\frac{7}{2+\zeta}} = \sqrt{\frac{7}{2+4}} = 1{,}08 \geq 0{,}7$$

Finally:

- $A = 0{,}16g$
- $\gamma_1 = 1{,}00$
- $q = 1{,}00$
- $\theta = 1{,}00$
- $\beta_0 = 2{,}5$
- $T_1 = 0{,}15$sec and $T_2 = 0{,}60$sec
- $\eta = 1{,}08$ (for $\zeta = 4\%$)

$0 \leq T \leq 0{,}15$sec : $\Phi_d(T) = \gamma_1 \cdot A \left[1 + \frac{T}{T_1} \cdot \left(\eta \cdot \frac{\theta}{q} \cdot \beta - 1\right)\right] = 1{,}6 + 18{,}13 \cdot T$

$0{,}15\text{sec} \leq T \leq 0{,}60$sec: $\Phi_d(T) = \gamma_1 \cdot A \cdot \eta \cdot \frac{\theta}{q} \cdot \beta_0 = 4{,}32 \text{ m/sec}^2$

$0{,}60\text{sec} \leq T$: $\Phi_d(T) = \gamma_1 \cdot A \cdot \eta \cdot \frac{\theta}{q} \cdot \beta_0 \cdot \left(\frac{T_2}{T}\right)^{2/3} = 3{,}068/T^{2/3}$

For $T=0$: **min$\Phi_d(T)$=1,6m/sec^2**

For $0{,}15\text{sec} \leq T \leq 0{,}60$sec : **max$\Phi d(T)$=4,32m/sec^2**

CHAPTER 3: OVERHEAD CRANE-RUNWAY BEAM

3.1 CONSTRUCTION ELEMENTS

The crane runs to the y dimension of the building and has a wire rope hoist that moves in the x dimension, making possible the transportation of the loads in every point of the interior space of the building. It’s consisted of two parallel bridge girders that end up to the trolleys. Every trolley has two wheels that run into rails upon the runway beam. The following picture shows a typical overhead crane:

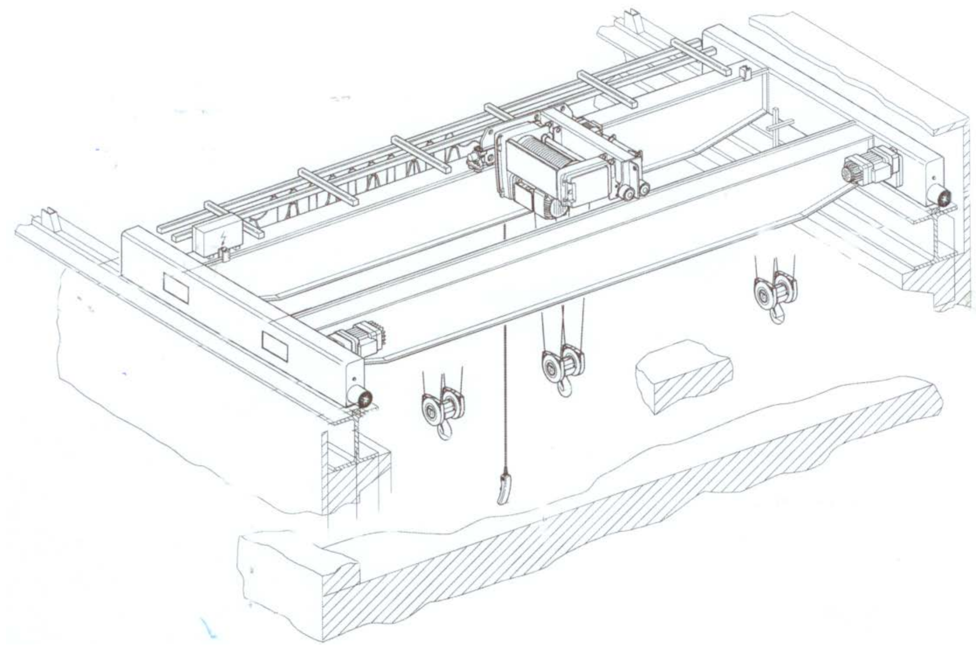


FIGURE 3.1-TYPICAL OVERHEAD CRANE

In the figure above we can see the bridge girder, the wire rope hoist, the grab, the trolleys, the rails, the buffers and the two runway beams.

Depending on the length and the lifting capacity every manufacturer gives the geometrical characteristics of the crane. We use an overhead crane of the DEMAG company type ZKKE,30m long, category HC3, total weight 218,4 kN and lifting capacity 40t. The crane moves parallel to the dimension of 96m on two wheels in every trolley. The distance between the two wheels is 3,5m. The runway beams are consisted of simple supported members 6m long. The lifting speed of the load is 8m/min and the distance between the extreme position of the hook and the axis of the runway beam is 1243mm. The weight of the crab is 2t(=20kN).The table below shows the characteristics of the crane. More details from the company DEMAG are given in the annex.

LIFTING CAPACITY	TOTAL WEIGHT	LENGTH	DISTANCE BETWEEN EXTREME HOOK POSITION AND RUNWAY BEAM	DISTANCE BETWEEN WHEELS	LIFTING SPEED	OVERHEAD CRANE CATEGORY
(t)	**(kN)**	**S(m)**	**a_1(mm)**	**e_1(mm)**	**u(m/min)**	**HC3**
40	218,40	27	1243	3.500	8	β_2=0,51 $\varphi_{2,min}$=1,15

TABLE 3.1 CHARACTERISTICS OF CRANE

In order to avoid lateral and lateral torsional buckling, the compressive flange of the runway beam is connected to the column through a truss, which gives lateral support to the beam. The following figure shows this lateral support of the beam:

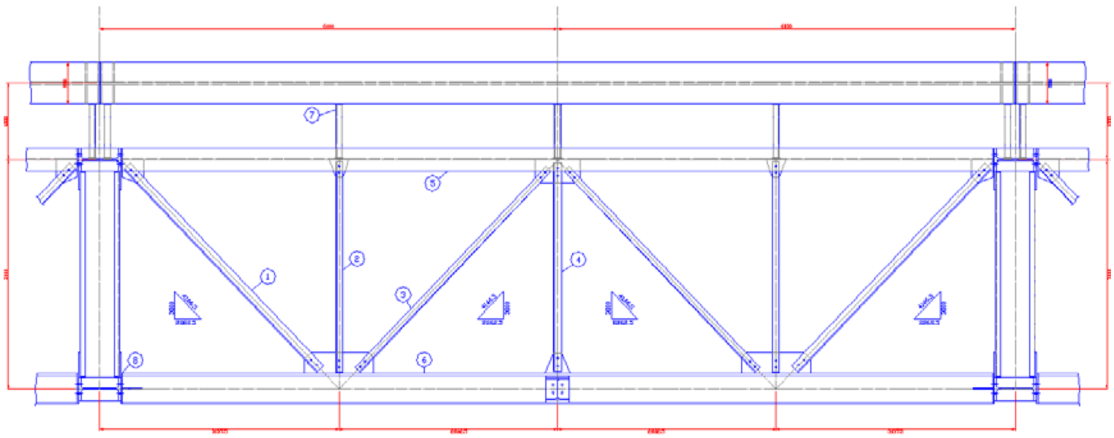


FIGURE 3.2- LATERAL SUPPORT OF THE RUNWAY BEAM

3.2 CLASSIFICATION OF ACTIONS

The actions induced by cranes will be presented according to the third part of the eurocode EC1.Actions induced by cranes are classified as variable and accidental actions which are represented by various models.

VARIABLE ACTIONS

For normal service conditions variable crane actions result from variation in time and location. They include gravity loads including hoistloads, inertial forces caused by acceleration/deceleration and by skewing and other dynamic effects.

The variable crane actions should be separated in variable vertical crane actions caused by the selfweight of the crane and the hoist load and in variable horizontal crane actions caused by acceleration or deceleration or by skewing or other dynamic effects.

The various representative values of variable crane actions are characteristic values composed of a static and a dynamic component.

Dynamic components induced by vibration due to inertial and damping forces are in general accounted by dynamic factors φ to be applied to the static action values.

$F_{\varphi,k} = \varphi_i \bullet F_k$

Where:

$F_{\varphi,k}$ is the characteristic value of a crane action

φ_i is the dynamic factor

F_k is the static component of a crane action

ACCIDENTAL ACTIONS

Cranes may generate accidental actions due to collision with buffers (buffer forces) or collision of lifting attachments with obstacles (tilting forces). These actions should be considered for the structural design where appropriate protection is not provided.

DYNAMIC FACTORS

The various dynamic factors and their application are listed in Table 3.2

Dynamic factors	Effects to be considered	To be applied to
φ_1	-Vibrational excitation of the crane structure due to lifting hoist load off the ground	Selfweight of the crane
φ_2 **or** φ_3	-dynamic effects of transferring the hoistload from the ground to the crane -dynamic effect of sudden release of the payload if for example grabs or magnets are used	hoistload
φ_4	-dynamic effects induced when crane is travelling on rail tracks or rynways	Selfweight of the crane and hoistload
φ_5	-dynamic effects caused by drive forces	Drive forces
φ_6	-when a test load is moved by the drives in the way	Test load

	the crane is used	
Φ_7	-considers the dynamic elastic effects of impact on buffers	Buffer loads

DYNAMIC FACTOR φ_1

$0,9 < \varphi_1 < 1,1$

For our structure we choose **$\varphi_1=1,1$**

DYNAMIC FACTOR φ_2

$\varphi_2 = \varphi_{2,min} + \beta_2 \bullet v_h$

where:

v_h is the steady hoisting speed in (m/s)

Hosting class of appliance	**β_2**	**$\varphi_{2,min}$**
HC1	0,17	1,05
HC2	0,34	1,10
HC3	0,51	1,15
HC4	0,68	1,20
NOTE: Cranes are assigned to Hoisting Classes HC1 to HC4 to allow for the dynamic effects of transferring the load from the ground to the crane.		

Table 3.3- values β_2 and $\varphi_{2,min}$

The category of our crane is HC3 so:

$\beta_2 = 0,51$

$\varphi_{2,min} = 1,15$

$\varphi_2 = \varphi_{2,min} + \beta_2 \cdot v_h = 1,15 + 0,51 \cdot (8/60) \Rightarrow$ **$\varphi_2=1,218$**

DYNAMIC FACTOR φ_3

$$\varphi_3 = 1 - \frac{\Delta m}{m} \cdot (1 + \beta_3)$$

where:

Δm released or dropped part of the load m total hoisting load

β_3=0,5 for cranes equipped with grabs or similar slow-release devices

β_3=1 for cranes equipped with magnets or similar rapid-release devises

In our case there is not a possibility of released or dropped part of the load (Δm=0), so:

φ_3 = 1,0

DYNAMIC FACTOR φ_4

φ_4 = 1,0

provided that the tolerances for rail tracks as specified in part 6 –eurocode 3 are observed.

DYNAMIC FACTOR φ_5

Dynamic factor φ_5	**Specific use**
φ_5 =1,0	For centrifugal forces
1,0 $\leq \varphi_5 \leq$ 1,5	Correspond to systems in which forces change smoothly
1,5 $\leq \varphi_5 \leq$ 2,0	When sudden changes occur
φ_5 =3,0	For drives with considerable backlash

Table 3.4- dynamic factor φ_5

In our case forces change smoothly so:

1,0 $\leq \varphi_5 \leq$ 1 ,5

We take the most adverse case: **φ_5=1,5**

DYNAMIC FACTOR φ_6

In our case is not taken into account.

DYNAMIC FACTOR φ_7

We choose the most adverse value:

φ_7 = 1,6

LOAD SIMULTANEITY

The simultaneity of the crane load components may be taken into account by considering groups of loads as identified in Table 3.5. Each of these groups of loads should be considered as defining one characteristic crane action for the combination with non-crane loads.

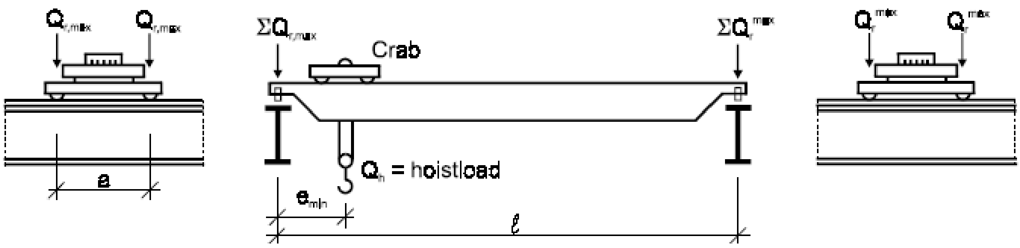
		symbol	Groups of loads										
			ULS							Test load		accidental	
			1	2	3	4	5	6	7	8α	8β	9	10
1	Selfweight of crane	Q_C	φ_1	φ_1	1	φ_4	φ_4	φ_4	1	φ_1	φ_1	1	1
2	Hoist load	Q_h	φ_2	φ_3	-	φ_4	φ_4	φ_4	η	-	-	1	1
3	Acceleration of crane bridge	H_L,H_T	φ_5	φ_5	φ_5	φ_5	-	-	-	φ_5	-	-	-
4	Skewing of crane bridge	H_S	-	-	-	-	1	-	-	-	-	-	-
5	Acceleration or braking of crab or hoist block	H_{T3}	-		-	-	-	1	-	-	-	-	-
6	In service wind	F_W	1	1	1	1	1	-	-	1	1	-	-
7	Test load	Q_T	-	-	-	-	-	-	-	φ_6	φ_6	-	-
8	Buffer force	H_B	-	-	-	-	-	-	-	-	-	1,6	-
9	Titling force	H_{TA}	-	-	-	-	-	-	-	-	-	-	1

Table 3.5- Groups of loads and dynamic factors to be considered as one characteristic crane action

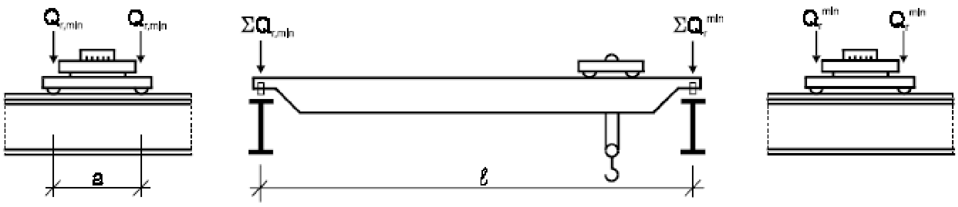
Note: η is the part of the hoist load that remains when the payload is removed, but is not included in the selfweight of the crane.

3.2.1 VERTICAL LOADS

For the calculation of the vertical loads we consider that the crane bridge obtains the maximum load 40t=400KN and this load is at the extreme possible place. The one runway beam is distressed more than the other.



a) Load arrangement of the loaded crane to obtain the maximum loading on the runway beam



b) Load arrangement of the unloaded crane to obtain the minimum loading on the runway beam

Where:

$Q_{r,max}$ is the maximum load per wheel of the loaded crane

$Q_{r,(max)}$ is the accompanying load per wheel of the loaded crane

$\Sigma Q_{r,max}$ is the sum of the maximum loads *Qr,max* per runway of the loaded crane

$\Sigma Q_{r,(max)}$ is the accompanying sum of the maximum loads *Qr,max* per runway of the loaded crane

$Q_{r,min}$ is the minimum load per wheel of the unloaded crane

$Q_{r,(min)}$ is the accompanying load per wheel of the unloaded crane

$\Sigma Q_{r,min}$ is the sum of the minimum loads *Qr,min* per runway of the unloaded crane

$Q_{h,nom}$ is the nominal hoistload

a is the distance between the wheels (4m).

e_{min} is the distance between extreme position of the hook and the runway beam(1243mm).

- **CRANE BRIDGE UNLOADED-MINIMUM VERTICAL LOAD VALUES (CRAB IN THE POSSIBLE EXTREME POSITION)**

A) COMBINATION OF LOADS 1,2

-Load of crane, apart from crab, converted into distributed load:

$q_c=\frac{218{,}40-20}{30}$=6, 613 kN/m

Dynamic factor:

$\varphi_1 \cdot q_c$=1,1·6,613=7,274 kN/m

Crab point load:

$\varphi_1 \cdot G_c$=1,10·20=22kN

-load in the most loaded runway beam:

$\Sigma Q_{r,(min)} = \frac{1}{2}\ 7{,}274 \cdot 30 + 22 \cdot \frac{30-1{,}243}{30}$=130, 20 kN

In every wheel:

$Q_{r,(min)} = \frac{1}{2}\ \Sigma Q_{r,(min)} = \frac{1}{2}\ 130{,}20$=65,10 kN

-Load in the less loaded runway beam:

$\Sigma Q_{r,min} = \frac{1}{2}\ 7{,}274 \cdot 30 + 22\ \frac{1.243}{30}$ =99,46 kN

In every wheel:

$Q_{r,min} = \frac{1}{2}\ \Sigma Q_{r,(min)} = \frac{1}{2}\ 99{,}46$= 49,73 kN

B) COMBINATION OF LOADS 3,4,5

-Dynamic factor φ_1=1,00 instead of 1,10

$\varphi_1 \cdot q_c$=6, 613 kN/m

$\varphi_1 \cdot G_c$=1 ·20 =20 kN

-load in the most loaded runway beam:

$\Sigma Q_{r(,min)} = \frac{1}{2}\ 6{,}613 \cdot 30 + 20 \cdot \frac{30-1{,}243}{30}$=118,37 kN

In every wheel:

$Q_{r,(min)} = \frac{1}{2}\ \Sigma Q_{r(min)} = \frac{1}{2}\ 118{,}37$=59,19 kN

-Load in the less loaded runway beam:

$\Sigma Q_{r,min} = \frac{1}{2}\ 6{,}613 \cdot 30 + 20\ \frac{1.243}{30}$ =100, 02 kN

In every wheel:

$Q_{r,min} = \frac{1}{2}\ \Sigma Q_{r,min} = \frac{1}{2}\ 100{,}02$=50,01 kN

- **CRANE BRIDGE LOADED-MAXIMUM VERTICAL LOAD VALUES (CRAB IN THE POSSIBLE EXTREME POSITION)**

A) COMBINATION OF LOADS 1

-Hoist load:

$$Q_H = \varphi_2 \cdot Q_{r,nom} = 1{,}218 \cdot 400 = 487{,}2kN$$

-load in the most loaded runway beam:

$\Sigma Q_{r,max} = \frac{1}{2}$ 7,274 · 30 + (22+487,20) · $\frac{30-1,243}{30}$=597,21 kN

In every wheel:

$Q_{r,max}=\frac{1}{2}$ $\Sigma Q_{r,max}$ = $\frac{1}{2}$597,21 =298,61 Kn

-Load in the less loaded runway beam:

$\Sigma Q_{r,(max)}$ = $\frac{1}{2}$ 7,274 · 30+(22+487,20) $\frac{1.243}{30}$ =130,21 kN

In every wheel:

$Q_{r,(max)}=\frac{1}{2}$ $\Sigma Q_{r,(max)}$ = $\frac{1}{2}$ 130,21=65,11 kN

B) COMBINATION OF LOADS 2

-Hoist load:

$$Q_h = \varphi_3 \cdot Q_{r,nom} = 1,0 \cdot 400 = 400kN$$

-load in the most loaded runway beam:

$\Sigma Q_{r,max} = \frac{1}{2}$ 7,274 · 30 + (22+400) · $\frac{30-1,243}{30}$=513,63 kN

In every wheel:

$Q_{r,max}=\frac{1}{2}$ $\Sigma Q_{r,max}$ = $\frac{1}{2}$513,63=256,82 kN

-Load in the less loaded runway beam:

$\Sigma Q_{r,(max)}$ = $\frac{1}{2}$ 7,274 · 30+(22+400) $\frac{1.243}{30}$ =126,59 kN

In every wheel:

$Q_{r,(max)}=\frac{1}{2}$ $\Sigma Q_{r,(max)}$ = $\frac{1}{2}$ 126,59=63,30

C)COMBINATION OF LOADS 4,5 (φ_4=1,0)

-Hoist load:

$$Q_h = \varphi_3 \cdot Q_{r,nom} = 1,0 \cdot 400 = 400kN$$

$\varphi_4 \cdot q_c$=1,00·6,613=6,613 kN/m

Crab point load:

$\varphi_4 \cdot G_c$=1,00·20=20 kN

-load in the most loaded runway beam:

$\Sigma Q_{r,max} = \frac{1}{2}\ 6{,}613 \cdot 30 + (20+400) \cdot \frac{30-1{,}243}{30} = 501{,}79$ kN

In every wheel:

$Q_{r,max} = \frac{1}{2}\ \Sigma Q_{r,max} = \frac{1}{2} 501{,}79 = 250{,}90$ kN

-Load in the less loaded runway beam:

$\Sigma Q_{r,(max)} = \frac{1}{2}\ 6{,}613 \cdot 30 + (20+400) \frac{1.243}{30} = 116{,}60$ kN

In every wheel:

$Q_{r,(max)} = \frac{1}{2}\ \Sigma Q_{r,(max)} = \frac{1}{2} 116{,}60 = 58{,}3$ kN

ECCENTRICITY OF VERTICAL LOADS

The eccentricity of application *e* of a wheel load Qr to a rail should be taken as equal to a quarter of the width of the rail head br, see Figure 3.3.

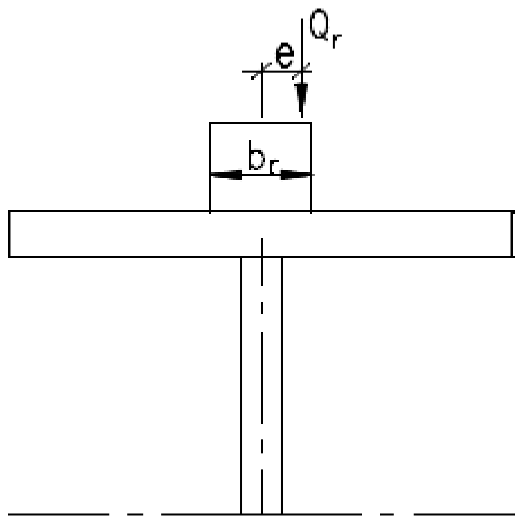


FIGURE 3.3-ECCENTRICITY OF VERTICAL LOAD

In our case b_r=50mm, so the eccentricity is:

$e = 0{,}25 \bullet b_r = 0{,}25 \cdot 50 = 12{,}5$mm

3.2.2 HORIZONTAL LOADS

The following types of horizontal loads from overhead travelling cranes should be taken into account:

a) horizontal loads caused by acceleration or deceleration of the crane in relation to its movement along the runway beam

b) horizontal loads caused by acceleration or deceleration of the crab or underslung trolley in relation to its movement along across the crane bridge,

c) horizontal loads caused by skewing of the crane in relation to its movement along the runway beam

d) buffer forces related to crane movement

e) buffer forces related to movement of the crab or underslung trolley.

Only one of the five types of horizontal load (a) to (e) should be included in the same group of simultaneous crane load components.

LOADS CAUSED BY ACCELERATION OR DECELERATION OF THE CRANE BRIDGE

A)LONGITUDINAL LOADS

The longitudinal loads *HL,i* caused by acceleration and deceleration of crane structures result from the drive force at the contact surface between the rail and the driven wheel, see Figure 3.4.

The longitudinal loads *HL,i* applied to a runway beam may be calculated as follows:

$$H_{L,i} = \varphi_5 \cdot K \cdot \frac{1}{n_r}$$

where:

nr is the number of runway beams

K is the drive force

φ_5 is the dynamic factor

i is the integer to identify the runway beam (i = 1,2).

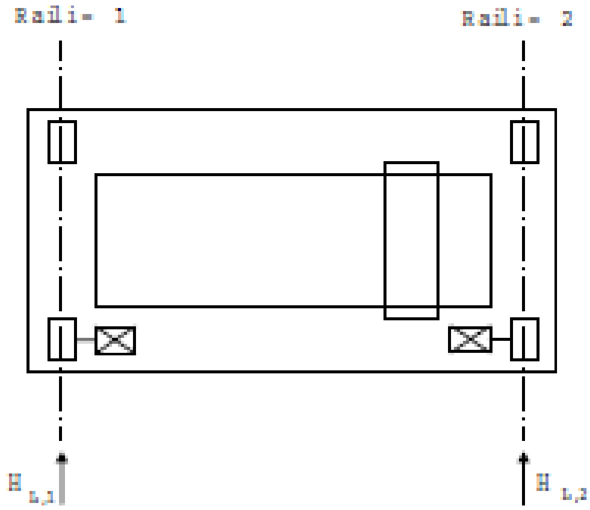


FIGURE 3.4-Longitudinal horizontal loads *HL,i*

B)TRANSVERSE LOADS

The moment *M* resulting from the drive force which should be applied at the centre of mass is equilibrated by transverse horizontal loads *HT,1* and *HT,2*, see Figure 3.5. The horizontal loads may be obtained as follows:

$$H_{T,1} = \varphi_5 \cdot \xi_2 \cdot \frac{\mathrm{M}}{\alpha} \qquad\qquad H_{T,2} = \varphi_5 \cdot \xi_1 \cdot \frac{\mathrm{M}}{\alpha}$$

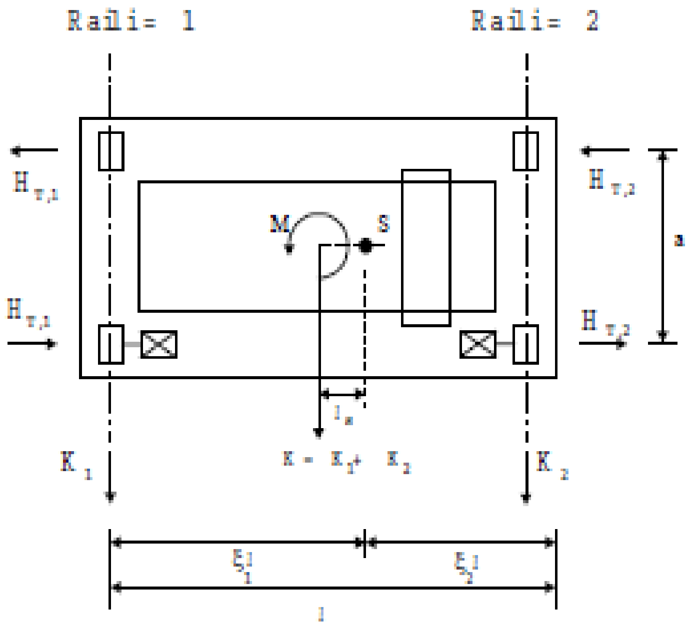


FIGURE 3.5-DEFINITION OF THE TRANSVERSE LOADS

Where:

$$\xi_1 = \frac{\sum Q_{r,\max}}{\sum Q_r}$$

$$\xi_2 = 1 - \xi_1$$

$$\sum Q_r = \sum Q_{r,\max} + \sum Q_{r,(\max)}$$

a the distance between the wheels

$$M = K \cdot l_s$$

$$l_s = (\xi_1 - 0,5) \cdot l$$

l the length of crane bridge

φ_5 the dynamic factor

K the drive force

If the drive force hasn't been given by the manufacturer of the crane, it can be calculated:

$K = K_1 + K_2 = \mu \cdot \Sigma Q_{r,min}$

Where :

μ the friction factor

$\mu=0,2$ for steel-steel

$\mu=0,5$ for steel-rubber

$\Sigma Q_{r,min}$:

$\Sigma Q_{r,min} = m_w \cdot Q_{r,min}$ for single wheel drive.

$\Sigma Q_{r,min} = Q_{r,min} + Q_{r,(min)}$ for central wheel drive

•

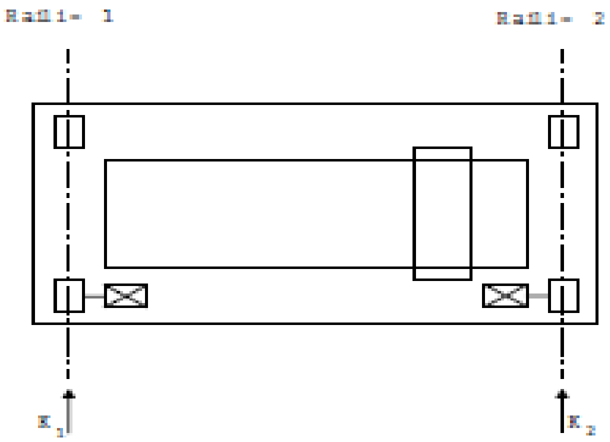


FIGURE 3.6-DEFINITION OF THE DRIVE FORCE

In this project the above loads are:

A)LONGITUDINAL LOADS

Friction factor:

μ=0,2 (steel-steel)

$\Sigma Q_{r,min} = m_w \bullet Q_{r,min}$ and $m_w = 2$.

So, the driving force is:

K=0,2 · 2 · 50,01= 20,004 kN

Horizontal longitudinal loads:

$H_{L,1}$=$H_{L,2}$=φ_5 K $\frac{1}{n_r}$ = 1,50 · 20,004 · $\frac{1}{2}$ = 15,003 kN

B)TRANSVERSE LOADS

Factor ξ_1 :

$\xi_1=\frac{\Sigma Qr,max}{\Sigma Qr}=\frac{\Sigma Qr,max}{\Sigma Qr,max+\Sigma Qr,(max)}=\frac{501,79}{501,79+116,60}$ = 0,811 kN

ξ_2 =1- ξ_1 =1-0,811=0,189m

$l_s=(\xi_1-0,5)\ l$ = (0,811-0,5) 30= 9,33m

M=K · l_s = 20,004 · 9,33 = 186,64 kNm

Horizontal transverse loads in the less loaded rail:

$H_{T,1}$=φ_5 ξ_2 $\frac{M}{\alpha}$ =1,50 $H_{T,1}$=φ_5 ξ_2 $\frac{M}{\alpha}$ 0,189 $\frac{186,64}{3,5}$ =15,12kN

Horizontal transverse loads in the most loaded rail:

$H_{T,2}$=φ_5 ξ_1 $\frac{M}{\alpha}$= 1,50 · 0,811 $\frac{186,64}{3,5}$ =64,87 kN

HORIZONTAL LOADS CAUSED BY SKEWING OF THE CRANE

The guide force *S* and the horizontal forces *HS,i,j,k* caused by skewing may be obtained from:

$$S = f \bullet \lambda_{S,j} \bullet \Sigma Q_r$$

$$H_{S,1,j,L} = f \bullet \lambda_{S,1,j,L} \bullet \Sigma Qr$$

$$H_{S,2,j,L} = f \bullet \lambda_{S,2,j,L} \bullet \Sigma Qr$$

$$H_{S,1,j,T} = f \bullet \lambda_{S,1,j,T} \bullet \Sigma Qr$$

$$H_{S,2,j,T} = f \bullet \lambda_{S,2,j,T} \bullet \Sigma Qr$$

where:

f is the non-positive factor

$\lambda_{s,1,j,L}$ is the force factors,

i is the rail *i*;

j is the wheel pair *j*;

k is the direction of the force (L = longitudinal, T = transverse)

The non-positive factor may be determined from:

f = 0,3 [1 - exp(-250a)] $\leq$ 0,3 where a: is the skewing angle

The force factor $\lambda_{s,1,j,L}$ depends on the combination of the wheel pairs and the distance *h* between the instantaneous centre of rotation and the relevant guidance means, which is the front guidance means in the direction of motion, see Figure 3.7. The value of the distance *h* may be taken from Table 3.5. The force factor 8*S,i,j,k* may be determined from the expressions given in Table 3.6.

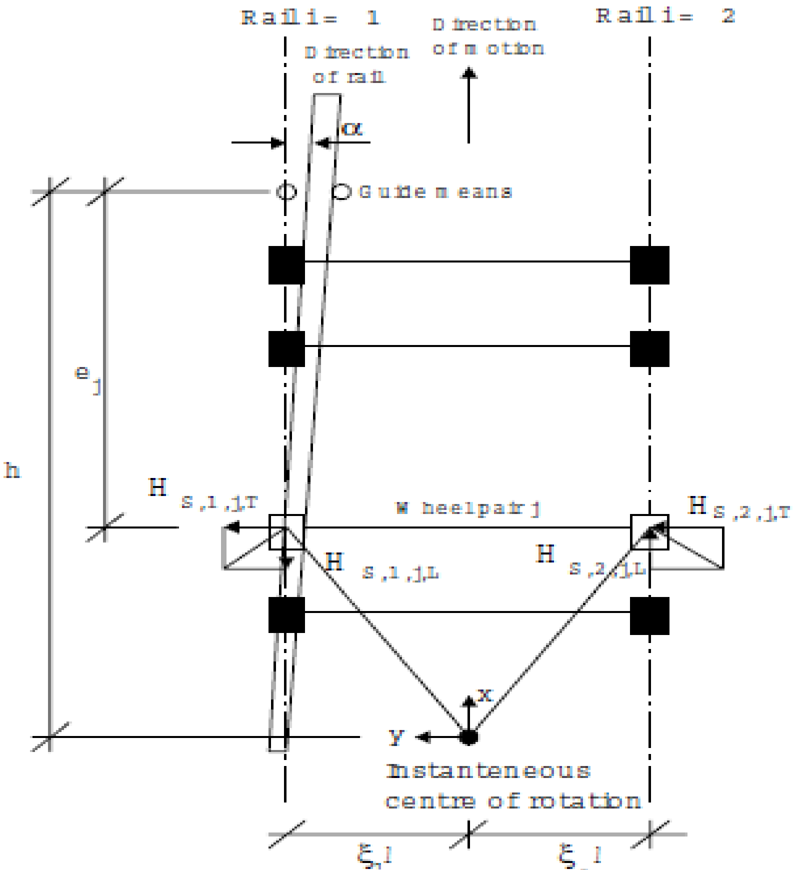
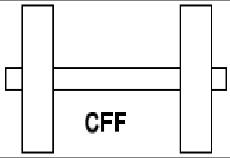
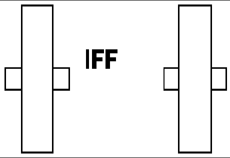
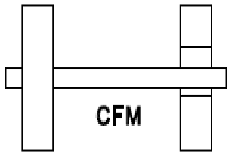
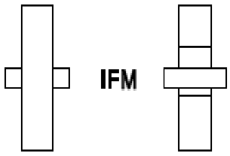


FIGURE 3.7-DEFINITION OF THE ANGLE a AND THE DISTANCE

	Combination of wheel pairs		h
	coupled (c)	independent (i)	
Fixed/Fixed FF			$\frac{m\xi_1\xi_2\ell^2 + \Sigma e_j^2}{\Sigma e_j}$
Fixed/Movable FM			$\frac{m\xi_1\ell^2 + \Sigma e_j^2}{\Sigma e_j}$

Where:
h is the distance between the instantaneous centre of rotation and the relevant guidance means;
m is the number of pairs of coupled wheels (m = 0 for independent wheel pairs);
$>_1P$ is the distance of the instantaneous centre of rotation from rail 1;
$>_2P$ is the distance of the instantaneous centre of rotation from rail 2;
P is the span of the appliance;
e_j is the distance of the wheel pair j from the relevant guidance means.

TABLE 3.5-DETERMINATION OF THE DISTANCE h

System	$8_{S,j}$	$8_{S,1,j,L}$	$8_{S,1,j,T}$	$8_{S,2,j,L}$	$8_{S,2,j,T}$
CFF	$1 - \frac{\Sigma e_j}{nh}$	$\frac{\xi_1\xi_2}{n}\frac{\ell}{h}$	$\frac{\xi_2}{n}\left(1 - \frac{e_j}{h}\right)$	$\frac{\xi_1\xi_2}{n}\frac{\ell}{h}$	$\frac{\xi_1}{n}\left(1 - \frac{e_j}{h}\right)$
IFF		0	$\frac{\xi_2}{n}\left(1 - \frac{e_j}{h}\right)$	0	$\frac{\xi_1}{n}\left(1 - \frac{e_j}{h}\right)$
CFM	$\xi_2\left(1 - \frac{\Sigma e_j}{nh}\right)$	$\frac{\xi_1\xi_2}{n}\frac{\ell}{h}$	$\frac{\xi_2}{n}\left(1 - \frac{e_j}{h}\right)$	$\frac{\xi_1\xi_2}{n}\frac{\ell}{h}$	0
IFM		0	$\frac{\xi_2}{n}\left(1 - \frac{e_j}{h}\right)$	0	0

Where:
n is the number of wheel pairs;
$>_1P$ is the distance of the instantaneous centre of rotation from rail 1;
$>_2P$ is the distance of the instantaneous centre of rotation from rail 2;
P is the span of the appliance;
e_j is the distance of the wheel pair j from the relevant guidance means;
h is the distance between the instantaneous centre of rotation and the relevant guidance means.

TABLE 3.6-DETERMINATION OF $\lambda_{s,1,j,L}$ VALUES

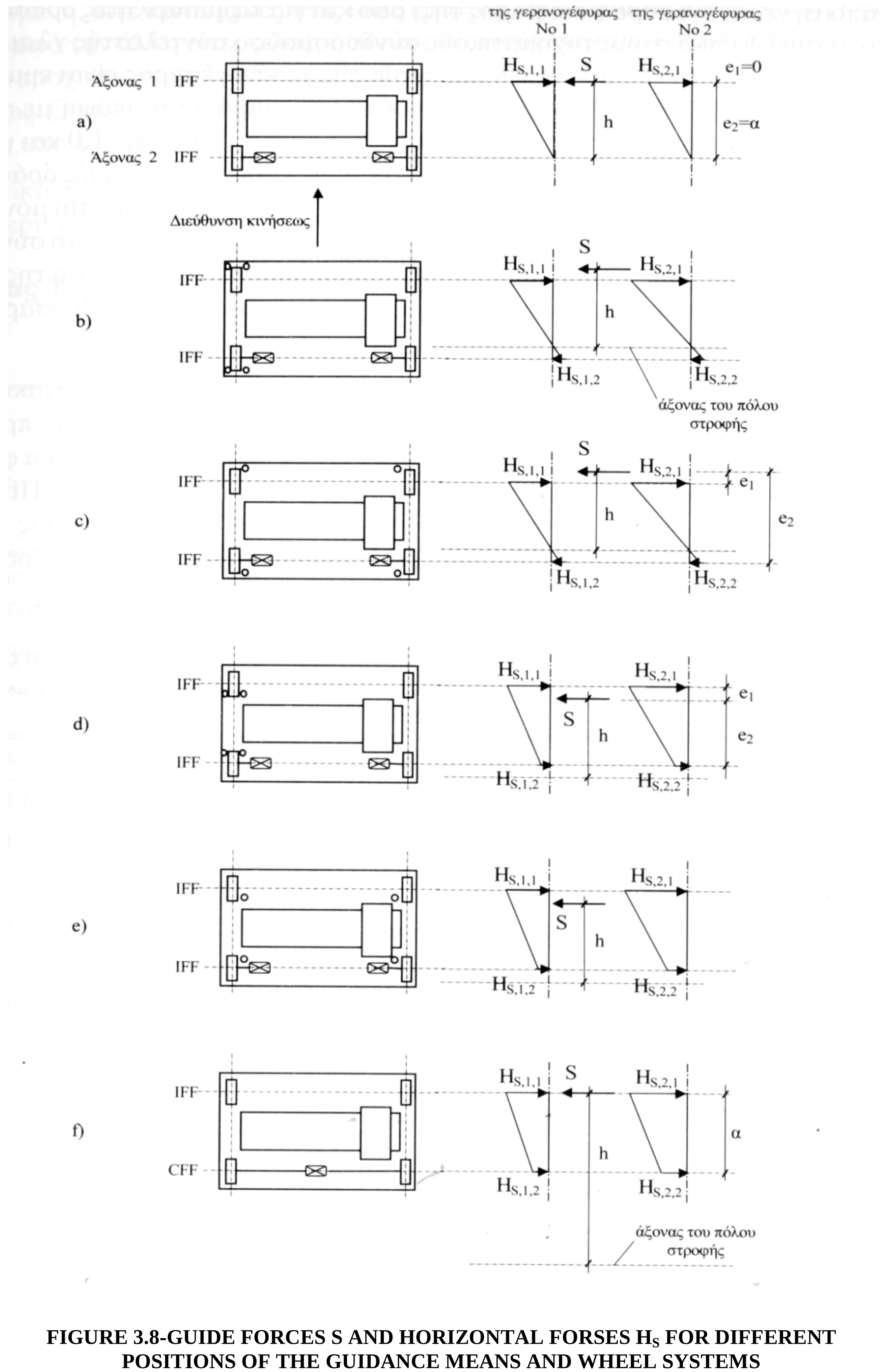


FIGURE 3.8-GUIDE FORCES S AND HORIZONTAL FORSES H_S FOR DIFFERENT POSITIONS OF THE GUIDANCE MEANS AND WHEEL SYSTEMS

For our crane we consider the adverse skewing angle :

$$\alpha = 0{,}015 \text{ rad}$$

The non-positive factor is:

f = 0,3 [1 - exp(-250a)] = 0,30 [1- exp(-250•0,015)] = 0,293 $\leq$ 0,3

The wheels are equipped with relevant guidance means, so e_1=0. For the second pair of wheels is e_2=a=3,50m. The wheel pairs are independent so m=0. Finally the system of the crane is IFF, so:

Calculation of the distance h:

$$h = \frac{m \cdot \xi_1 \cdot \xi_2 \cdot l^2 + \sum e_j^2}{\sum e_j} = \frac{\sum e_j^2}{\sum e_j} = \frac{3{,}5^2}{3{,}5} = 3{,}5m$$

The system of the wheels and the position of the guidance means corresponds to the case a of the figure 3.8.

Calculation of factor λ_s:

$$\lambda_s = 1 - \frac{\sum e_i}{n \cdot h} = 1 - \frac{3{,}5}{2 \cdot 3{,}5} = 0{,}5$$

$$\lambda_{S,1L} = \lambda_{S,2L} = 0$$

For the first pair of wheels:

$\lambda_{s,1,1T} = \frac{\xi_2}{n}(1-\frac{e_1}{h}) = \frac{0{,}189}{2}(1-0) = 0{,}094$ (axis 1)

$\lambda_{s,2,1T} = \frac{\xi 1}{n}(1-\frac{e_1}{h}) = \frac{0{,}811}{2}(1-0) = 0{,}406$ (axis 2)

For the second pair of wheels:

$$\lambda_{S,1,2T} = \frac{\xi_2}{n} \cdot (1 - \frac{e_2}{h}) = \frac{0{,}192}{2} \cdot (1 - \frac{3{,}50}{3{,}50}) = 0$$

$$\lambda_{S,2,2T} = \frac{\xi_1}{n} \cdot (1 - \frac{e_2}{h}) = \frac{0{,}808}{2} \cdot (1 - \frac{3{,}5}{3{,}5}) = 0$$

For the forces H_s are:

S = f • λ_S • $\Sigma Q_{r,max}$=0,293 • 0,5 • 501,79=73,51 kN

$H_{S,1,1T} = f \cdot \lambda_{S,1,1T} \cdot \Sigma Qr_{max} = 0{,}293 \cdot 0{,}094 \cdot 501{,}79 = 13{,}82$ kN

$H_{S,2,1T} = f \cdot \lambda_{S,2,1T} \cdot \Sigma Qr_{max} = 0{,}293 \cdot 0{,}406 \cdot 501{,}79 = 59{,}69$ kN

$H_{S,1,2T} = H_{S,2,2T} = 0$

So in the first pair of wheels, on the first rail :

$H_{s,1T} = H_{s,1,1T} - S = 13{,}82 - 73{,}51 = -59{,}69$ kN

And in the second rail:

$H_{s,2T} = H_{s,2,1T} = 59{,}69$ kN

The above forces H_s will be taken into account only in the combination of loads 5.

FINAL RESUMING TABLE-DESIGN LOADS

			1	**2**	**3**	**4**	**5**
Vertical loads	Selfweight of crane bridge	$Q_{r,(min)}$	65,1 87,89	65,1 87,89	59,19 79,91	59,19 79,91	59,19 79,91
		$Q_{r,min}$	49,73 67,14	49,73 67,14	50,01 67,51	50,01 67,51	50,01 67,51
	Selfweight of crane bridge + Hoist load	$Q_{r,max}$	298,61 403,12	256,82 346,70	-	250,90 338,72	250,90 338,72
		$Q_{r,(max)}$	65,11 87,90	63,3 85,50	-	58,3 78,71	58,3 78,71
Horizontal loads	Acceleration Braking of crane bridge	H_{L1}	15 20,25	15 20,25	15 20,25	15 20,25	-
		H_{L2}	15 20,25	15 20,25	15 20,25	15 20,25	-
		H_{T1}	15,12 20,41	15,12 20,41	15,12 20,41	15,12 20,41	-
		H_{T2}	64,87 87,57	64,87 87,57	64,87 87,57	64,87 87,57	-
	Skewing of crane bridge	$H_{S,1T}$	-	-	-	-	59,69 80,58
		$H_{S,2T}$	-	-	-	-	59,69 80,58

TABLE 3.6-FINAL RESUMING TABLE-DESIGN LOADS

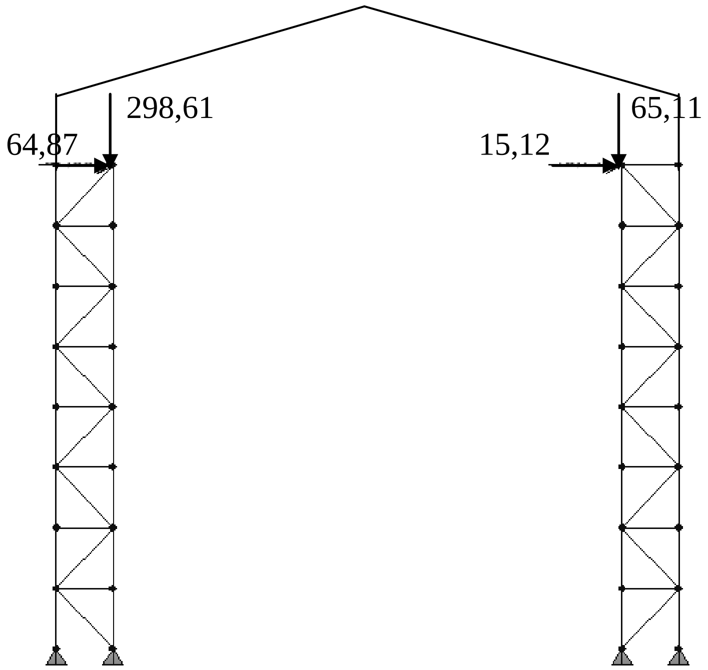
In the table above the first numbers are function loads (γ=1,0) and the second are design loads (γ=1,35). In every column we have a group of vertical and horizontal loads, that is considered as a characteristic action of the crane Each of these groups of loads should be considered as defining one characteristic crane action for the combination with non-crane loads according to the eurocode 1.

3.2.3 APPLICATION OF THE LOADS

The loads were indicial applied in the two most representative positions of the structure, in the outer and the middle panel (panel 1-2 and 8-9).For each one of these positions we have four cases:

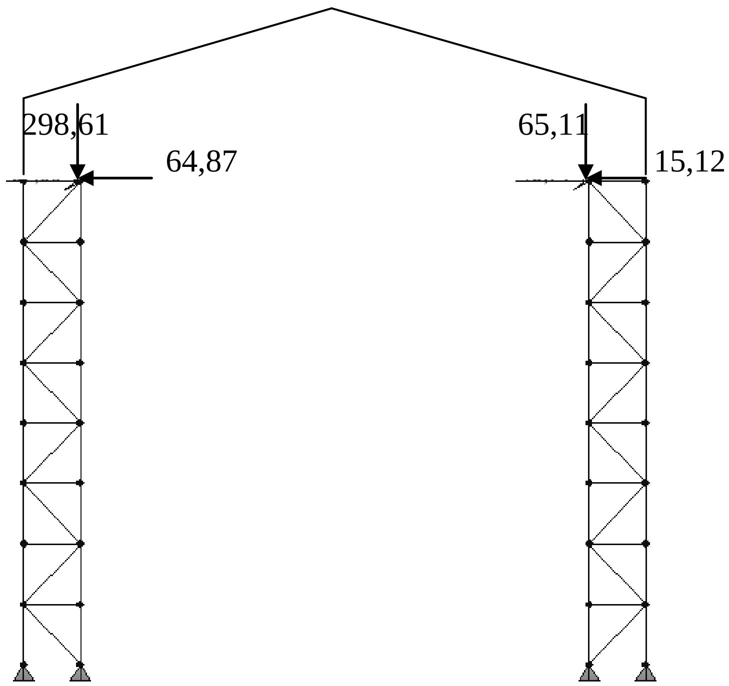
FIRST CASE

The left beam is the most loaded and the right less. The horizontal force is pointing to the right.



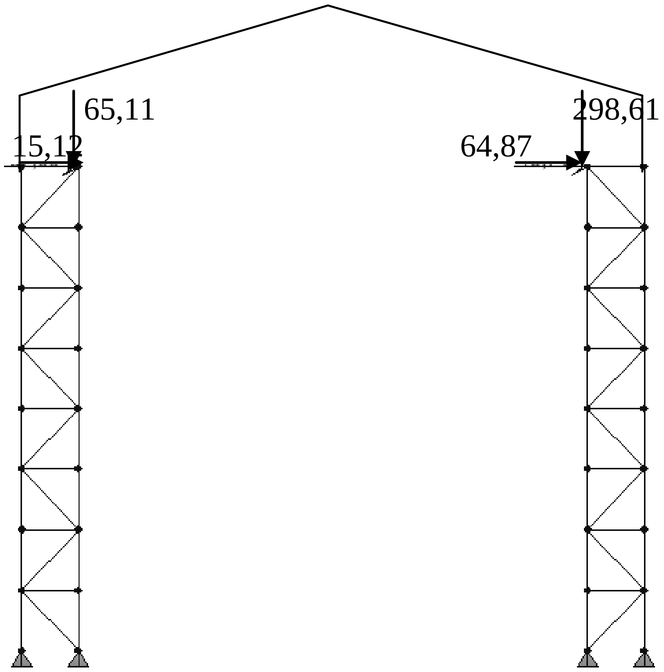
SECOND CASE

The left beam is the most loaded and the right the less. The horizontal force points to the left.



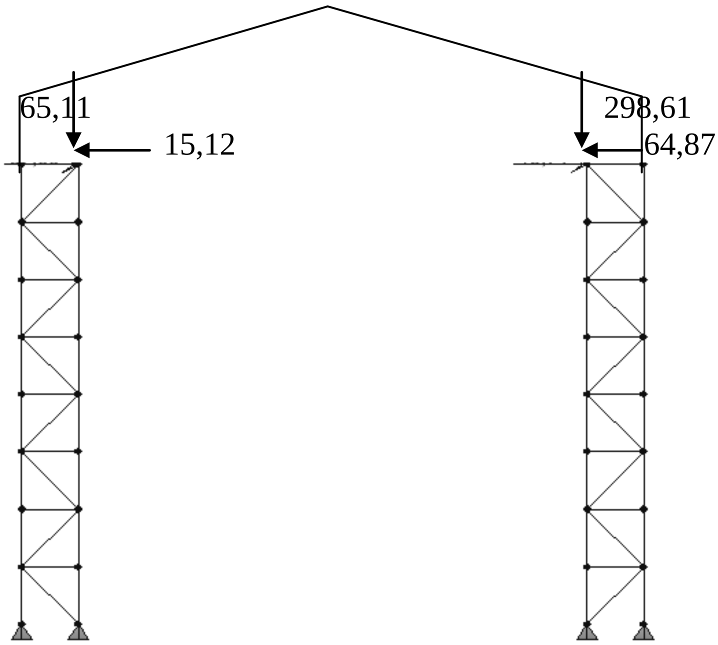
THIRD CASE

The right beam is the most loaded and the left the less. The horizontal force points to the right.



FOURTH CASE

The right beam is the most loaded and the left the less. The horizontal force points to the left.

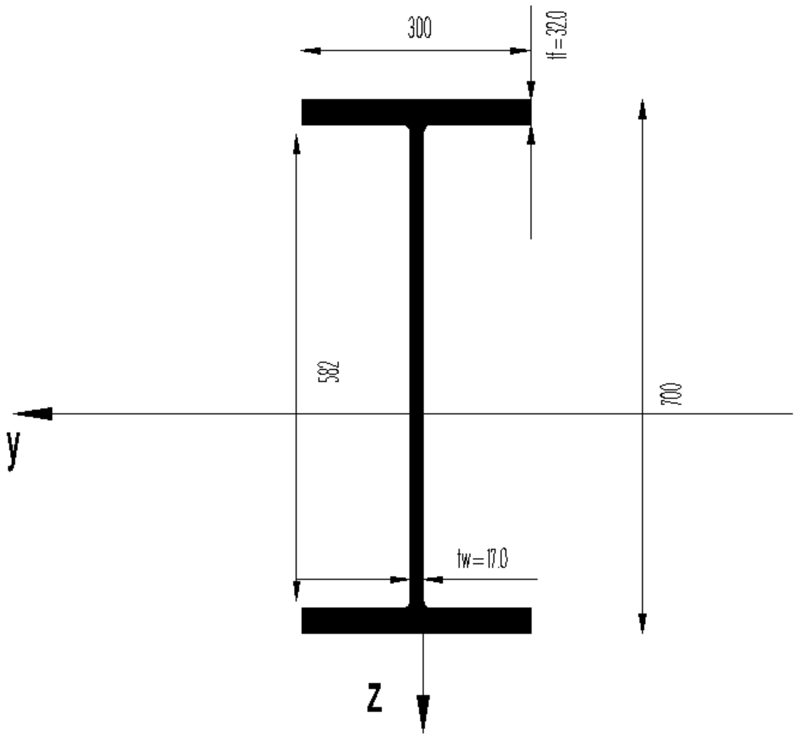


3.3. RUNWAY BEAM

The runway beam will be checked for the Ultimate Limit State, the Serviceability Limiting State and Fatigue.

3.3.1. ULTIMATE LIMIT STATE

SECTION CHARACTERISTICS (HEB 700)



Sectional area:

A = 306,40 cm^2

Moments of inertia:

I_y = 256.900 cm^4

I_z = 14.440 cm^4

Plastic modulus:

$W_{pl,y}$ = 8.327 cm^3

$W_{pl,z}$ = 1.495 cm^3

Section modulus:

$W_{el,y}$ = 7.340 cm^3

$W_{el,z}$ = 962,7 cm^3

Torsional constant:

$$J = \frac{1}{3}\Sigma b_i t_i^3 = \frac{1}{3}(30 \cdot 3,2^3 \cdot 2 + 63,6 \cdot 1,7^3) = 759,52cm^4$$

$$J_w = \frac{1}{4} \cdot h^2 \cdot I_z = \frac{1}{4} \cdot 70^2 \cdot 14.440 = 17.689.000cm^4$$

SECTION CLASSIFICATION

Web:

$$\frac{d}{t_w} = \frac{582}{17} = 34 < 72\varepsilon = 72$$

Flange:

$$\frac{c}{t_f} = \frac{(300-17)/2}{32} = 4,42 < 9\varepsilon = 9$$

So the section is category 1.

DESIGN ACTIONS

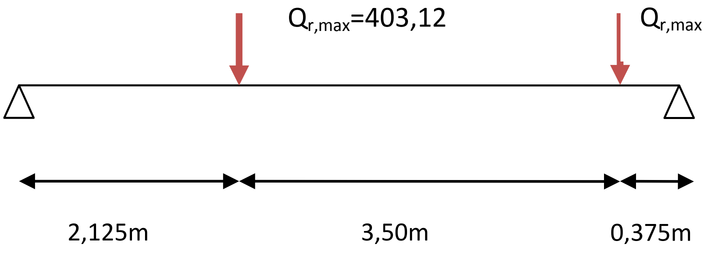
A)COMBINATION 1-MAXIMUM BENDING MONUMENT IN THE STRONG AXIS

max $M_y = \frac{Qr,max}{8l}(2\,l - \alpha)^2 = \frac{403,12}{8 \cdot 6}(2 \cdot 6 - 3,5)^2 = 606,78$ kNm

and $x = \frac{2\,l - a}{4} = \frac{2 \cdot 6 - 3,5}{4} = 2,125$m

$\alpha = 3,50\text{m} < 0,586\text{l} = 0,586\text{x}6,00 = 3,516\text{m}$

The beam takes the weight of the crane and the hoist load(most loaded beam).



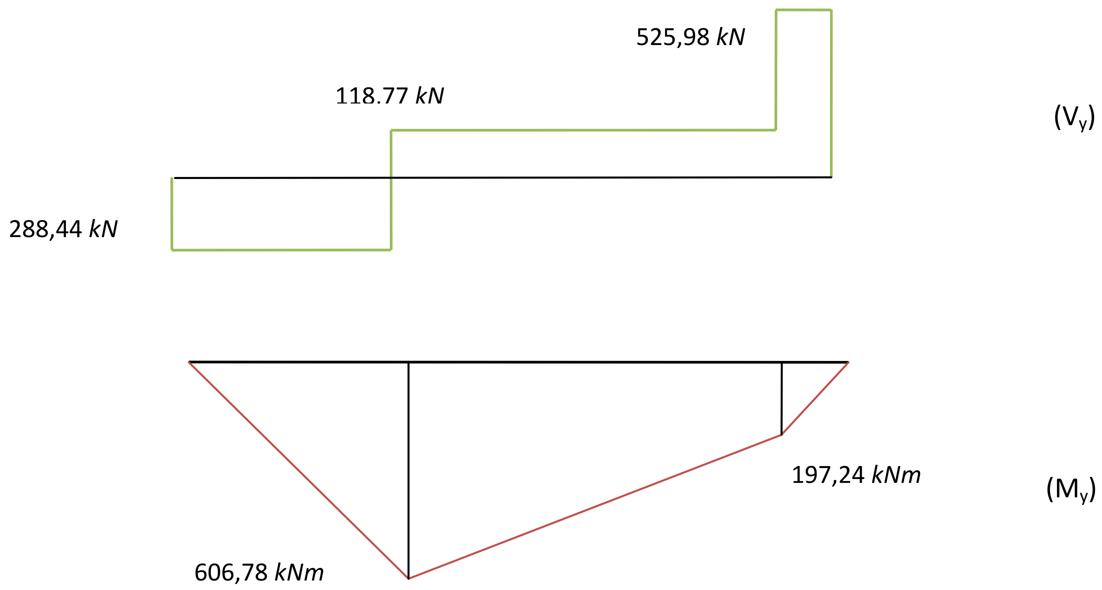


FIGURE 3.9-POSITION OF LOADS FOR THE MAX BENDING MONUMENT IN THE STRONG AXIS

At the same time, due to the acceleration and braking of the crane we have actions H_T that create bending monuments in the weak axis of the runway beam.

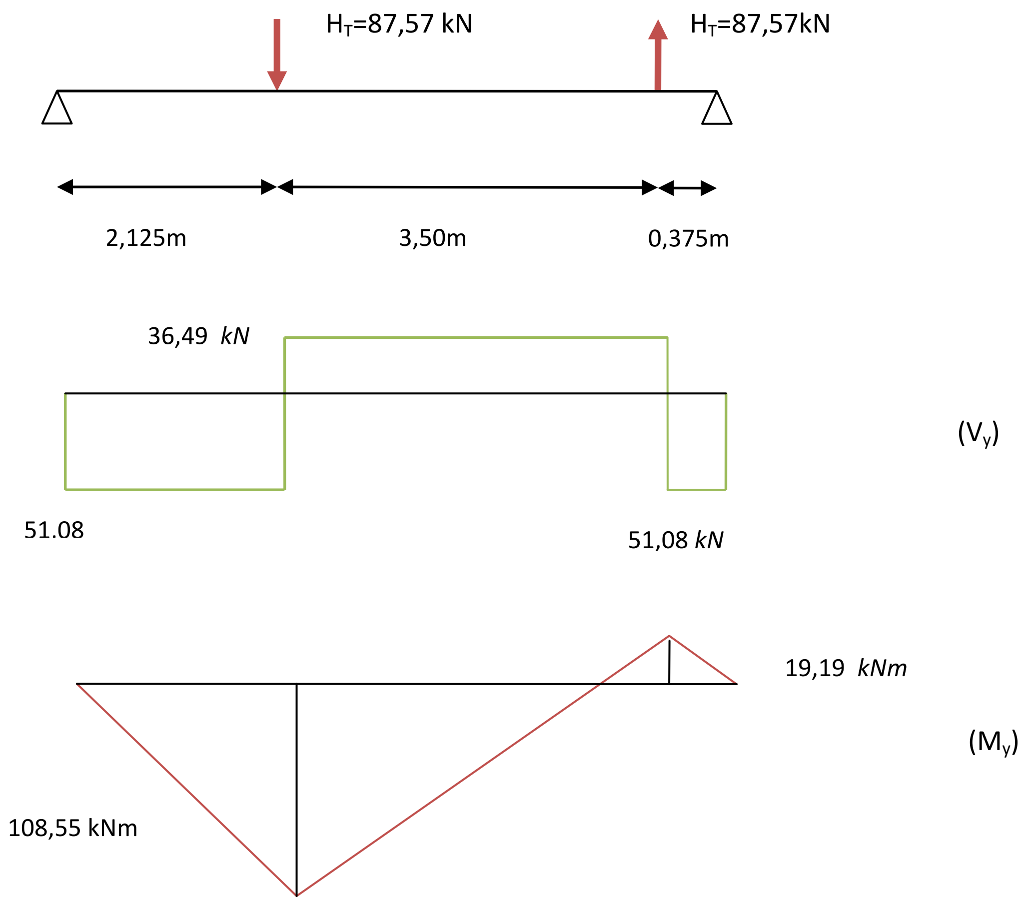


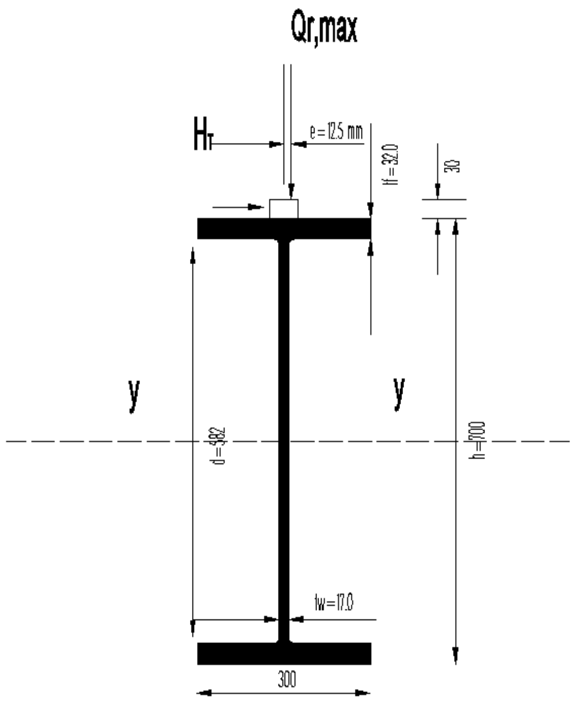
FIGURE 3.10-BENDING MONUMENTS DUE TO TRANSVERSE LOADS H_T

Due to the eccentricity of the loads we also have torsional monuments:

M_{ta}= 403,12 ·1,25 + 87,57 ·(35+3) = 3831,56 kNcm = 38,32 kNm

M_{tb}=403,12 ·1,25 – 87 ,57 ·(35+3) = - 2823,76 kNcm = 28,24 kNm

The horizontal forces H_T are placed to the higher point if the rail.



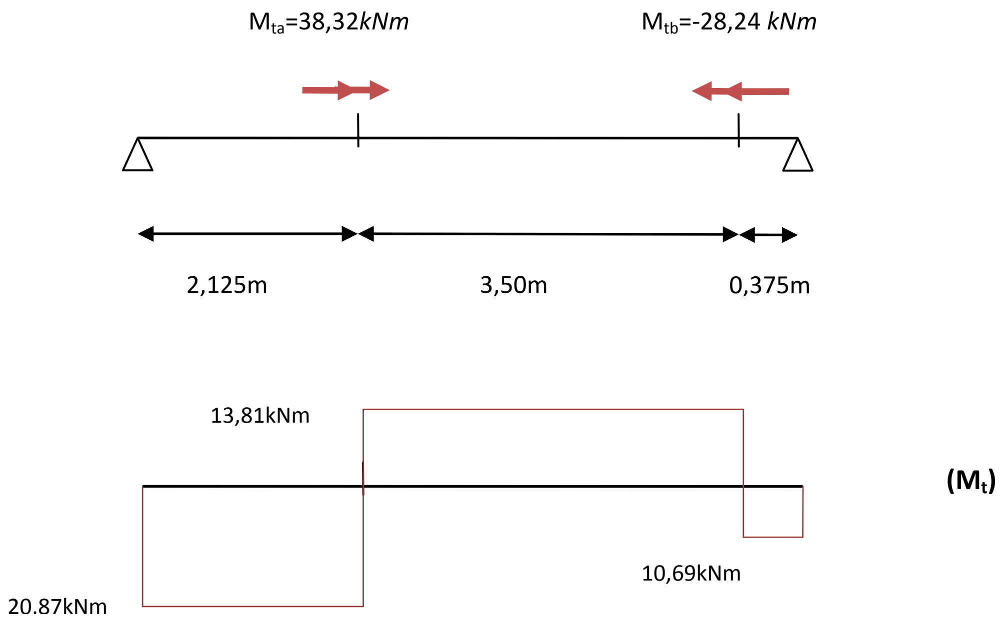


FIGURE 3.11-DIAGRAM OF TORSIONAL MONUMENTS

B)COMBINATION 1-MAXIMUM SHEAR FORCE

The maximum shear force appears in the edge of the beam, when one of the wheel vertical loads are in the edge of the beam.

max $V_{y,sd}$ = 403,12 (1+$\frac{2,50}{6,00}$) = 571,09 kNm

C)COMBINATION 5-MAXIMUM BEMDING MONENT IN THE STRONG AXIS

Max M_y = 606,78 $\frac{338,72}{403,12}$ = 509,84 kNm

In the weak axis:

M_z= 80,58 · 2,125 $\frac{(6,00-2,125)}{6,00}$ = 110 59 kNm

D)COMBINATION 5-MAXIMUM BENDING MOMENT IN THE WEAK AXIS

When the load is in the middle of the beam.

Max M_z = $\frac{1}{4}$ · 80,58 ·6,00 = 120,87 kNm

In the strong axis:

M_y = $\frac{1}{4}$ 338,72 ·6,00 + $\frac{1}{4}$ 338,72 · (6,00 – 2 · 3,5) = 423,40 kNm

E)SELF WEIGHT OF RUNWAY BEAM

The selfweight of the beam including the rail is 2,53 KN/m

$$M_{g,sd} = \frac{1}{8} \cdot 2,53 \cdot 6,00^2 \cdot 1,35 = 15,37 KNm$$

$$V_{g,sd} = \frac{1}{2} \cdot 2,53 \cdot 6,00 \cdot 1,35 = 10,25 KN$$

WEB SHEAR RESISTANCE

Shear area $A_v = 108,12 cm^2$

Shear resistance:

$$V_{y,Rd} = A_v \cdot f_y / (\sqrt{3} \cdot \gamma_M) = 108,12 \cdot 23,50 / 1,10 \cdot \sqrt{3} = 1333,58 kN$$

Check:

$V_{y,sd}$ = 571,09 + 10,25 = 581,34 kN < 133,58 kN

UPPER FLANGE SHEAR RESISTANCE

We consider that the horizontal loads are obtained only by the upper flange of the beam. We have the most adverse shear force V_z when the load H_s is in the edge of the beam.

Shear area $A_v = 30 \cdot 3{,}2 = 96\ cm^2$

$V_{z,sd} = 80{,}58$ kN

$V_{z,Rd} = 96 \cdot 23{,}50 / \sqrt{3} \cdot 1{,}10 = 1184{,}09 > 80{,}58 = V_{z,sd}$

LATERAL AND LATERAL-TORSIONAL BUCKLING RESISTANCE

In order to avoid lateral and lateral torsional buckling, the compressive flange of the runway beam is connected to the column through a truss, which gives lateral support to the beam. So there is no need for this check

3.3.2. SERVICEABILITY LIMIT STATE

For the permanent and variable loads we use the partial safety factor γ=1.

DEFLECTION DUE TO VERTICAL LOADS

a)Deflection due to point variable loads:

$$\delta_{lz} = \frac{P \cdot c}{24EIy}(3l^2 - 4c^2) = \frac{298{,}61 \cdot 125}{24 \cdot 21000 \cdot 256900}(3 \cdot 600^2 - 4 \cdot 125^2) = 0{,}30\ cm$$

The maximum vertical deflection is when the two wheel loads are symmetric to the middle of the beam, if $\alpha \leq 0{,}65l$.

b)deflection due to self-weight of runway beam

$$\delta_{2Z} = \frac{5}{384} \cdot \frac{2{,}53 \cdot 600^4}{21000 \cdot 256900 \cdot 100} = 0{,}0079cm$$

c)total deflection

$$\delta_Z = \delta_{1Z} + \delta_{2Z} = 0{,}30 + 0{,}0079 = 0{,}31cm$$

Total deflection is less than 25mm and less than *l*/600=1cm. So it’s acceptable.

DEFLECTION DUE TO HORIZONTAL LOADS

The most adverse case appears when the load H_s is in the middle of the beam. We assume that the load is obtained by the upper flange of the beam.

Moment of inertia of the flange:

$$I_Z = \frac{3,2 \cdot 30^3}{12} = 7200 cm^4$$

$$\Delta_y = \frac{P\, l^3}{48 \cdot E \cdot I} = \frac{59,69 \cdot (\frac{600}{2})^3}{48 \cdot 21000 \cdot 7200} = 0,22 \text{ cm}$$

It's acceptable because it's less than l/600=1,00 cm.

Due to the lateral support, the length of the beam was reduced to the half.

3.3.3. FATIGUE

For normal service condition of the crane the fatigue loads may be expressed in terms of fatigue damage equivalent loads *Qe* that may be taken as constant for all crane positions to determine fatigue load effects. The fatigue damage equivalent load *Qe* may be determined such that it includes the effects of the stress histories arising from the specified service conditions and the ratio of the absolute number of load cycles during the expected design life of the structure to the reference value $N = 2 \cdot 10^6$ cycles.According to the Annex B, table B.1 of the eurocode 1 the category of our crane is S_6.

The fatigue load may be specified as:

$$Q_{e,i} = \lambda_\iota \cdot \varphi_{fat} \cdot Q_{\max,i}$$

The factor λ_i, for crane category S6, takes the values:

λ_i=0,794 for normal stress

λ_τ=0,871 for shear stress

The wheel load is without the dynamic factor: $Q_{max,I}$ =250,90 kN

The damage equivalent dynamic impact factor ν*fat* for normal conditions may be taken as:

$$\varphi_{fat} = (1+\varphi_2)/2 = (1+1,24)/2 = 1,12$$

Finally the fatigue loads are:

For normal stress

$Q_{e,I}$ = 0,794 ·1,12 ·250,90 = 223,12 kN

For shear stress

$Q_{e,I}$ = 0,871 ·1,12 ·250,90 = 244,76 kN

For constant amplitude loading the fatigue assessment criterion is:

$$\gamma_{Ff} \Delta\sigma_{E2} \leq \frac{\Delta\sigma_c}{\gamma_{Mf}}$$

Where

$\Delta\sigma_{E2}$ is the nominal stress range

and $\Delta\sigma_c$ is the fatigue strength for the relevant detail category for the total number of stress cycles N during the required design life.

$\gamma_{Ff} = 1,0$

$\gamma_{Mf} = 1,25$

CHAPTER 4: COMBINATIONS OF ACTIONS

4.1 GENERAL

Limit states are states beyond which the structure no longer satisfies the design performance requirements.

Limit states are classified into:

• ultimate limit states

• serviceability limit states.

Design situations are classified as:

• persistent situations corresponding to normal conditions of use of the structure

• transient situations, for example during construction or repair

• accidental situations.

The actions in the structure are:

•Permanent actions

a)self-weight of structural members G

b)self-weight of non structural elements

•Variable actions

a)wind W

b)snow S

•crane loads C

•Earthquake E

•temperature change ±20°C

The wind is calculated for two cases, for open and closed doors and for each one of the cases we consider four load cases, as it was mentioned in the chapter 2.

Wind +X: acts in the left side of the building

Wind –X: acts on the right side of the building

Wind +Y: acts in the front side of the building

Wind –Y: acts in the back side of the building

The crane loads were calculated in two different positions of the building, in the outer and the middle panel. For every case we consider four different cases, described in chapter 3. In addition the loads were placed in positions in the runway beam for maximum monument and maximum shear force.

4.2 COMBINATIONS OF ACTIONS

4.2.1 ULTIMATE LIMIT STATES

$\mathbf{E_d \leq R_d}$

Where:

E_d : is the design value of an internal force or moment (or of a respective vector of several internal forces or moments).

R_d : is the corresponding design resistance, each taking account of the respective design values of all structural properties

The combinations of actions have the forms:

- Persistent and transient situations

$$\sum_{j\geq1}\gamma_{Gj}\cdot G_{kj}\text{"+"}\gamma_P\cdot P_k\text{"+"}\gamma_{Q1}\cdot Q_{k1}\text{"+"}\sum_{i>1}\gamma_{Qi}\cdot\psi_{0i}\cdot Q_{ki}$$

- Accidental design situations

$$\sum_{j\geq1}\gamma_{GAj}\cdot G_{kj}\text{"+"}\gamma_{PA}\cdot P_k\text{"+"}A_d\text{"+"}\psi_{11}\cdot Q_{k1}\text{"+"}\sum_{i>1}\psi_{2i}\cdot Q_{ki}$$

- Earthquake design situations

$$\sum_{j\geq1}G_{kj}\text{"+"}P_k\text{"+"}\gamma_1\cdot A_{Ed}\text{"+"}\psi_{21}\cdot Q_{k1}+Q_2$$

where:

$G_{k,j}$ are the characteristic values of the permanent actions

$Q_{k,l}$ is the characteristic value of one of the variable actions

$Q_{k,i}$ are the characteristic values of the other variable actions

A_d is the design value (specified value) of the accidental action

$A_{E,d}$ is the design value for the earthquake action

$\gamma_{G,j}$ is the partial safety factor for the permanent action Gk,j

$\gamma_{Q,i}$ is the partial safety factor for the variable action Qk,i

$\psi_{0,i}$, $\psi_{1,i}$, $\psi_{2,i}$ are factors

4.2.2 SERVICEABILITY LIMIT STATE

$\mathbf{E_d \leq C_d}$

Where:

E_d :is the design effect of actions, determined on the basis of one of the combinations defined below

C_d : is a nominal value or a function of certain design properties of materials related to the design effect of actions considered.

Three combinations of actions for serviceability limit states are defined by the following expressions:

- Rare combination

$$\sum_{j\geq 1} G_{kj} "+" P_k "+" Q_{k1} "+" \sum_{i>1} \psi_{0i} \cdot Q_{ki}$$

- Frequent combination

$$\sum_{j\geq 1} G_{kj} "+" P_k "+" \psi_{11} \cdot Q_{k1} "+" \sum_{i>1} \psi_{2i} \cdot Q_{ki}$$

- Quasi-permanent combination

$$\sum_{j\geq 1} G_{kj} "+" P_k "+" \sum_{i\geq 1} \psi_{2i} \cdot Q_{ki}$$

Below are presented tables of the partial safety factors according to EC1.

Action	γ_f	**Ultimate limit state**		**Serviceability limit state**	
		Unfavorable effect	Favorable effect	Unfavorable effect	Favorable effect
Permenant	γ_G	1,35	1,0	1,35	1,0
Variable	γ_Q	1,50	0,0	1,0	0,0
Imposed deformation	γ_{Qind}	1,20-1,50	0,0	1,0	0,0

TABLE 4.1-SAFETY FACTORS γ

ACTION	ψ_0	ψ_1	ψ_2
Imposed loads to buildings Category A: Residence Category B: Officies Category: Areas where people may Congregate Category D: Shopping areas Category E: Storage areas	 0,7 0,7 0,7 0,7 0,1	 0,5 0,5 0,7 0,7 0,9	 0,3 0,3 0,6 0,6 0,8
Traffic loads in buildings			

Category F: vehicle weight ≤ 30 KN Category G: 30 KN ≤ vehicle weight ≤ 160KN Category H: roofs	0,7 0,7 0	0,7 0,5 0	0,6 0,3 0
Snow loads in buildings For altitude 1000m < H < 1500m For altitude H ≤ 1000m	 0,7 0,5	 0,5 0,2	 0,2 0
Wind loads in buildings	0,6	0,5	0
Temprature on buildings	0,6	0,5	0

TABLE 4.2-FACTORS Ψ

Specially for Greece, according to EAK:

ACTION	ψ_2
Residence Areas where people may Congregate Parking aereas Storage aereas Non passable roofs	0,3 0,5 0,6 0,8 0,0
2. Wind	0,0
3. Snow (passable roofs) Snow(non passable roofs)	0,3 0

TABLE 4.3-FACTOR Ψ_2 ACCORDING TO EAK

For crane loads:

action	symbol	ψ_0	ψ_1	ψ_2
Crane loads	Q_r	1,00	0,90	*

TABLE 4.4-FACTORS Ψ FOR CRANE LOADS

4.3 COMBINATIONS OF ACTIONS IN THE PRESENT PROJECT

• Combinations without earthquake actions

54 combinations (S1 – S54)

•Combinations with earthquake actions

8 combinations (E1 – E8)

•Dynamic combinations for the two directions of earthquake loads

2 combinations (DYNAX,DYNAY)

•Combinations of serviceability

16 combinations (L1 –L16)

In all the combinations above we have to add the crane loads for the four different load cases. With the symbols GERAN1 and GERAN2, we have maximum vertical loads on the left runway beam and horizontal loads to the right and left correspondingly. With the symbols GERAN3 and GERAN4 we have maximum vertical loads on the right runway beam and horizontal loads to the right and to the left correspondingly.

The final combinations are:

• Combinations without earthquake actions

216 combinations (S1 – S216)

•Combinations with earthquake actions

32 combinations (E1 – E32)

•Dynamic combinations for the two directions of earthquake loads

8 combinations (DYNAX1-DYNAY4) and (DYNAY1-DYNAY4)

•Combinations of serviceability

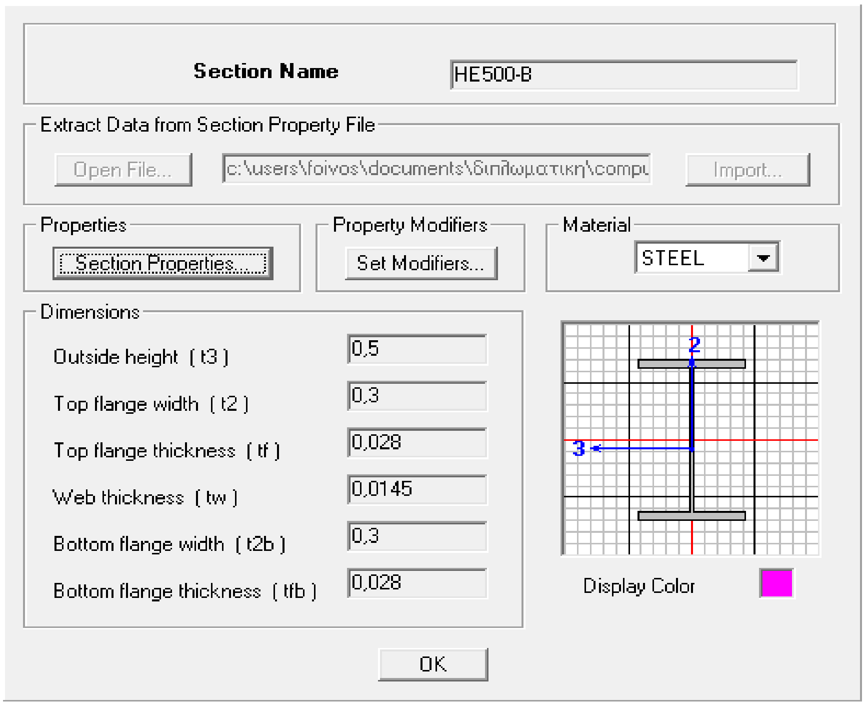
64 combinations (L1 –L64)

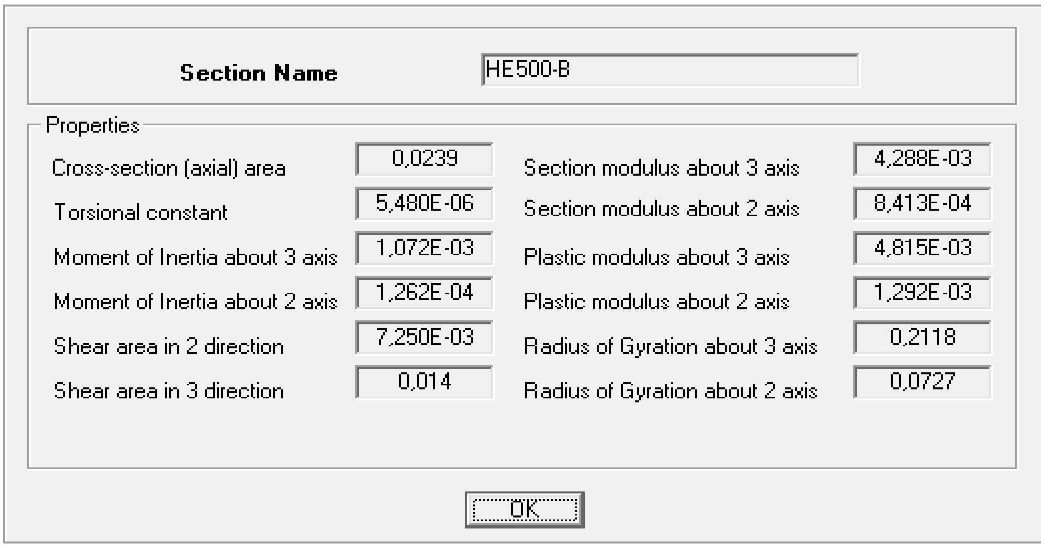
NOTE: More analytically the above combinations are presented in the ANNEX.

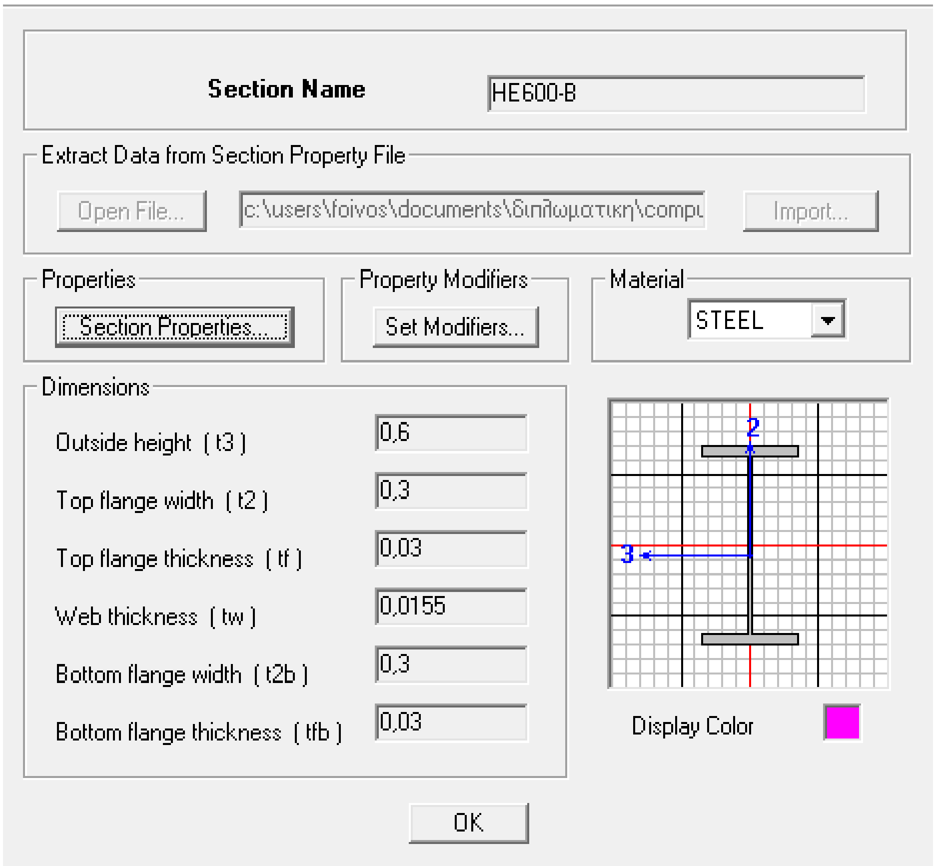
CHAPTER 5: ANALYSIS-DIMENSIONING OF MEMBERS

The analysis is made by the program ETABS. It calculates the forces of all members for all the combinations of the loads. The analysis is made according to the eurocode 3, checking the members in tension, compression, one-axis bending, shear force, bending + shear force, bending + axial force, biaxial bending + axial force, buckling, lateral-torsional buckling (ultimate limit state) and deflections (ultimate serviceability state).

5.1 SECTION PROPERTIES



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Top flange thickness (tf)
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Bottom flange thickness (tfb)
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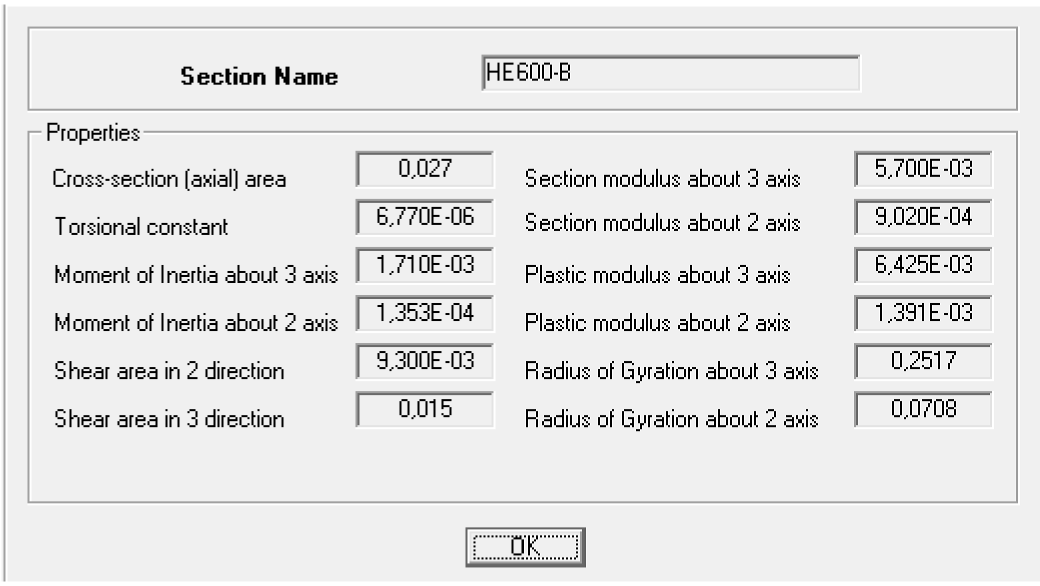
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Shear area in 3 direction
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Section modulus about 3 axis
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Section modulus about 2 axis
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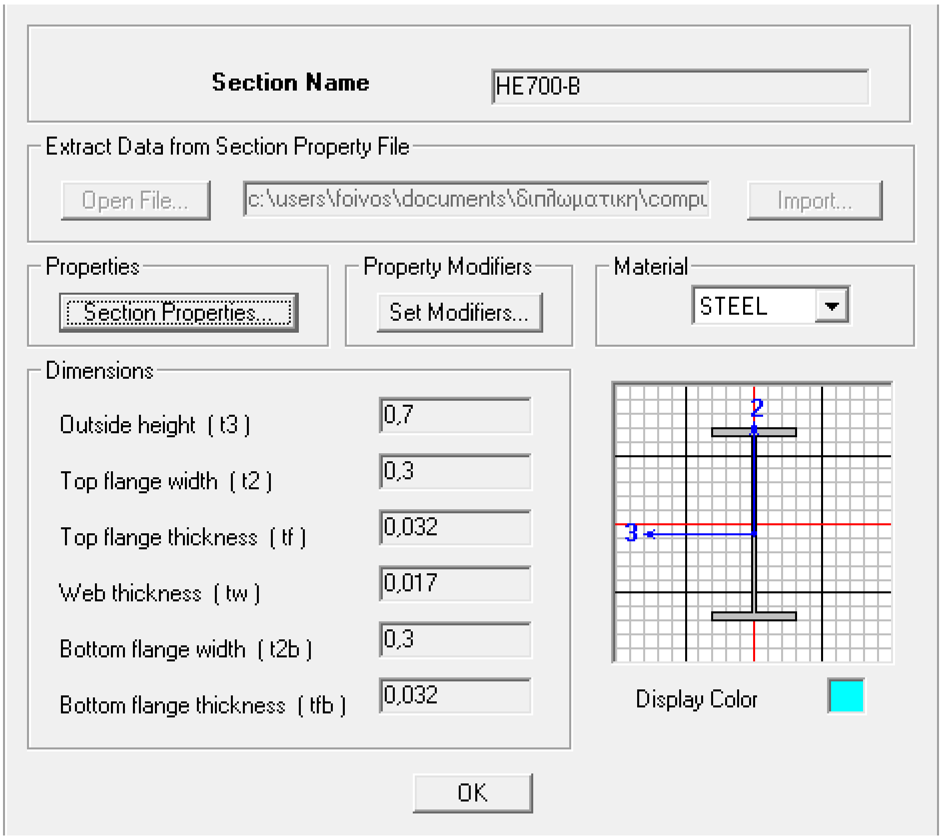
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Top flange thickness (tf)
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Web thickness (tw)
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Bottom flange width (t2b)
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Bottom flange thickness (tfb)
0,032
2
3
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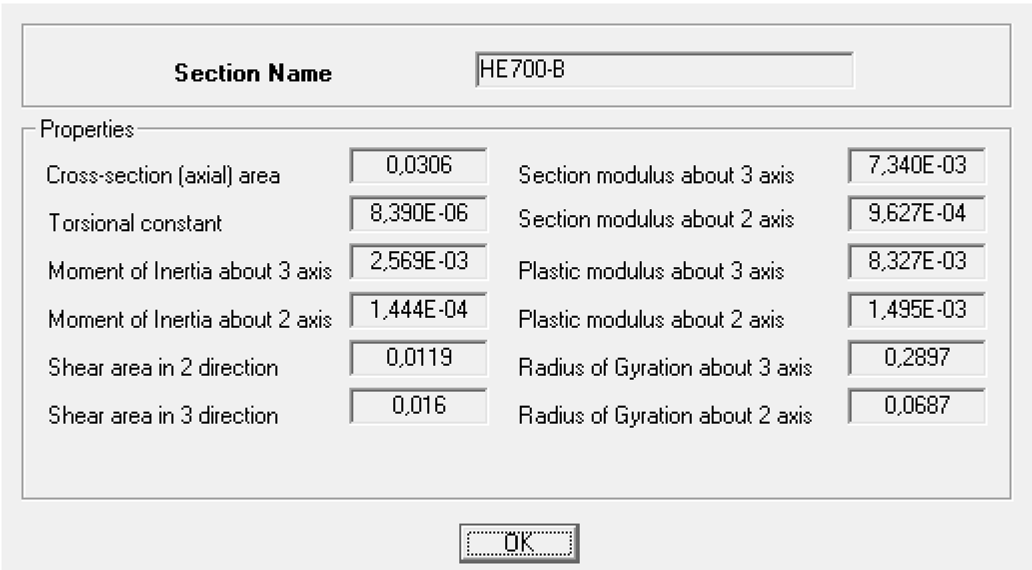
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Moment of Inertia about 2 axis
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Shear area in 2 direction
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Shear area in 3 direction
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Section modulus about 3 axis
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Section modulus about 2 axis
9,627E-04
Plastic modulus about 3 axis
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Plastic modulus about 2 axis
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Radius of Gyration about 3 axis
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Radius of Gyration about 2 axis
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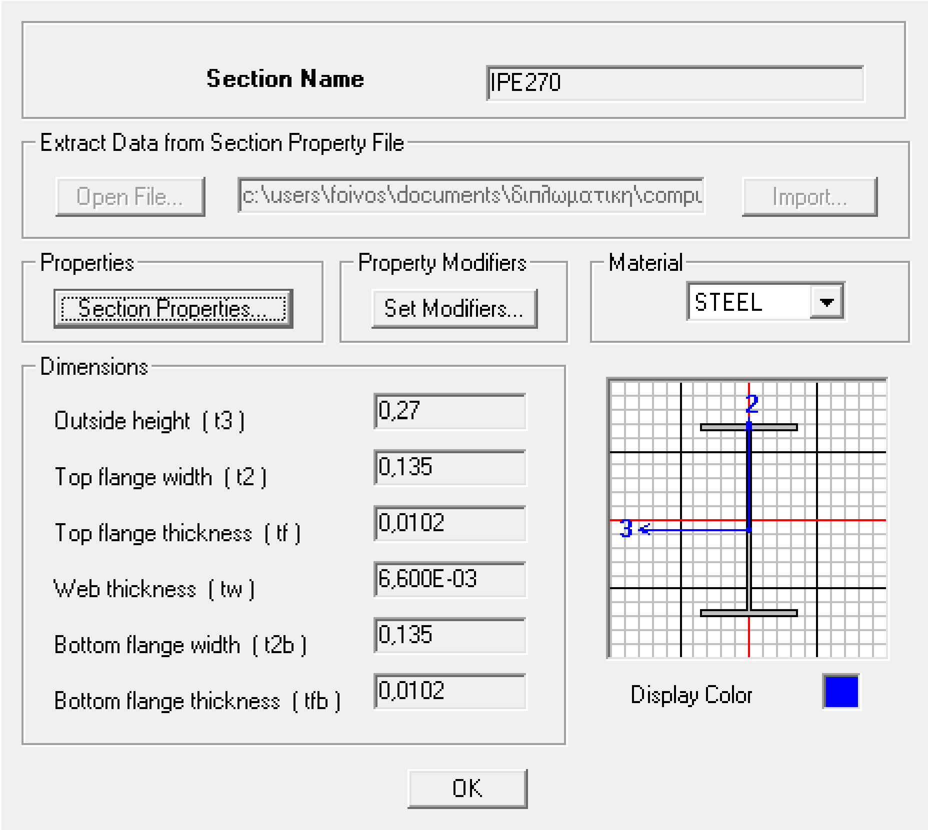
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Bottom flange thickness (tfb)
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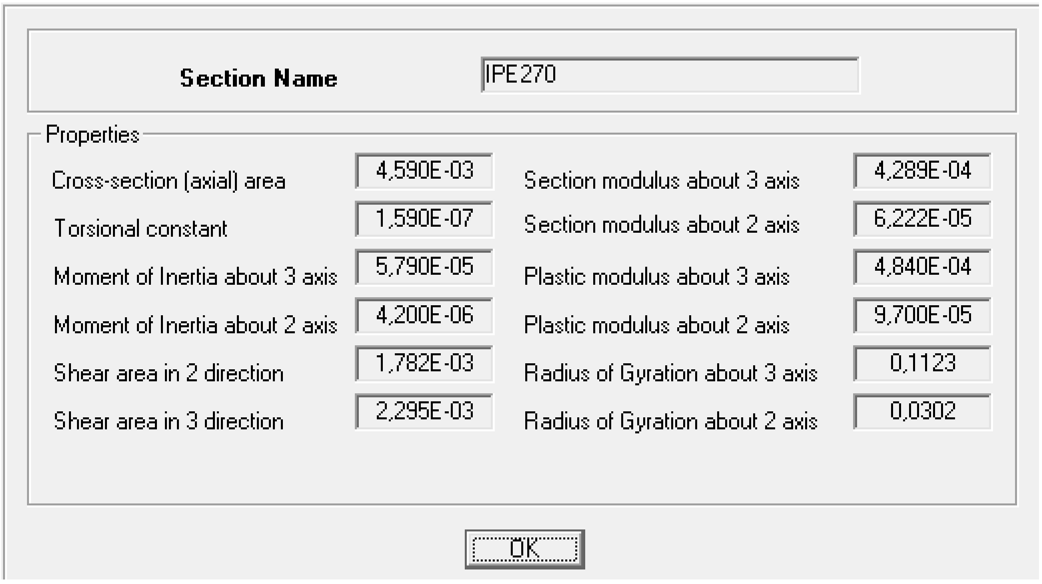
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1,782E-03
Shear area in 3 direction
2,295E-03
Section modulus about 3 axis
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Section modulus about 2 axis
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Plastic modulus about 3 axis
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Plastic modulus about 2 axis
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Radius of Gyration about 2 axis
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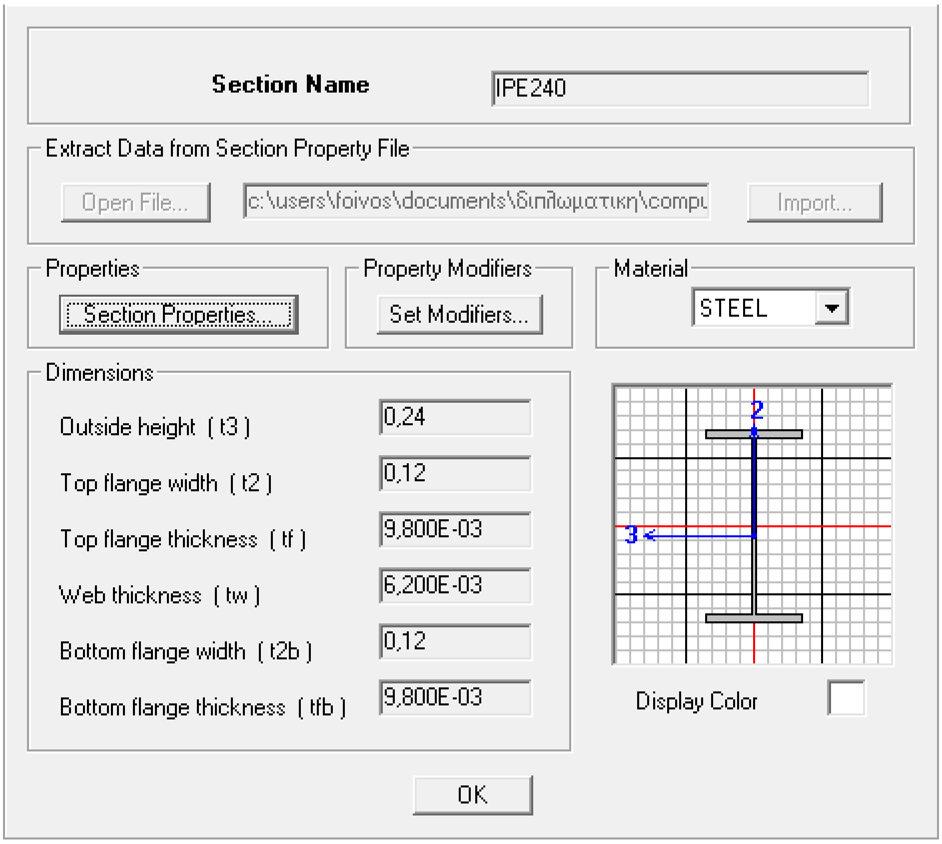
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Web thickness (tw)
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Bottom flange width (t2b)
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Bottom flange thickness (tfb)
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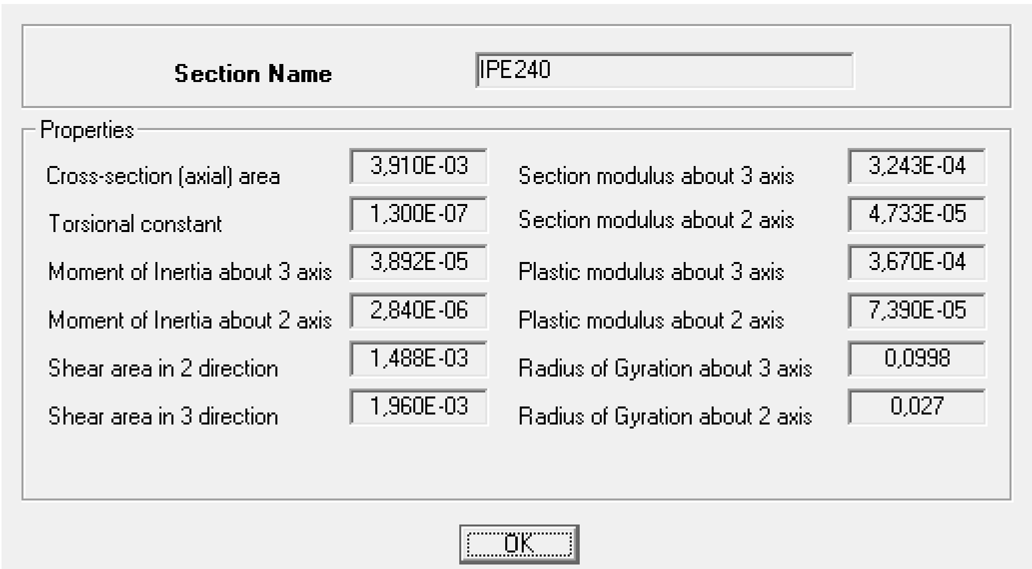
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Moment of Inertia about 2 axis
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Shear area in 3 direction
1,960E-03
Section modulus about 3 axis
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Section modulus about 2 axis
4,733E-05
Plastic modulus about 3 axis
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Plastic modulus about 2 axis
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Radius of Gyration about 3 axis
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Radius of Gyration about 2 axis
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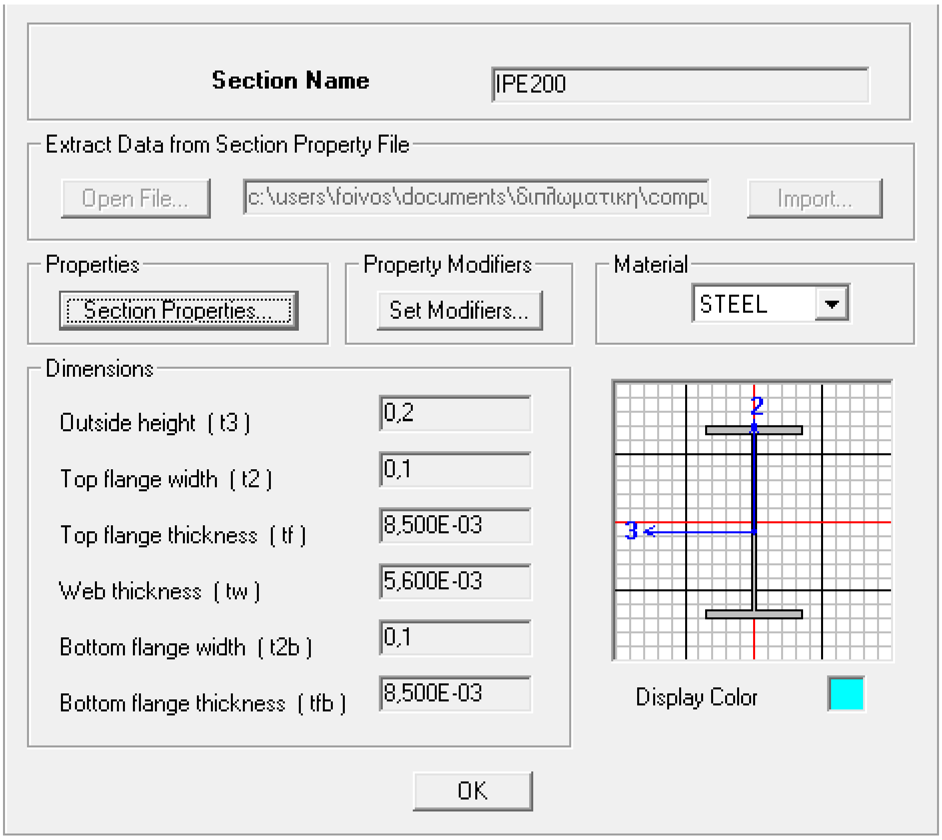
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Top flange width (t2)
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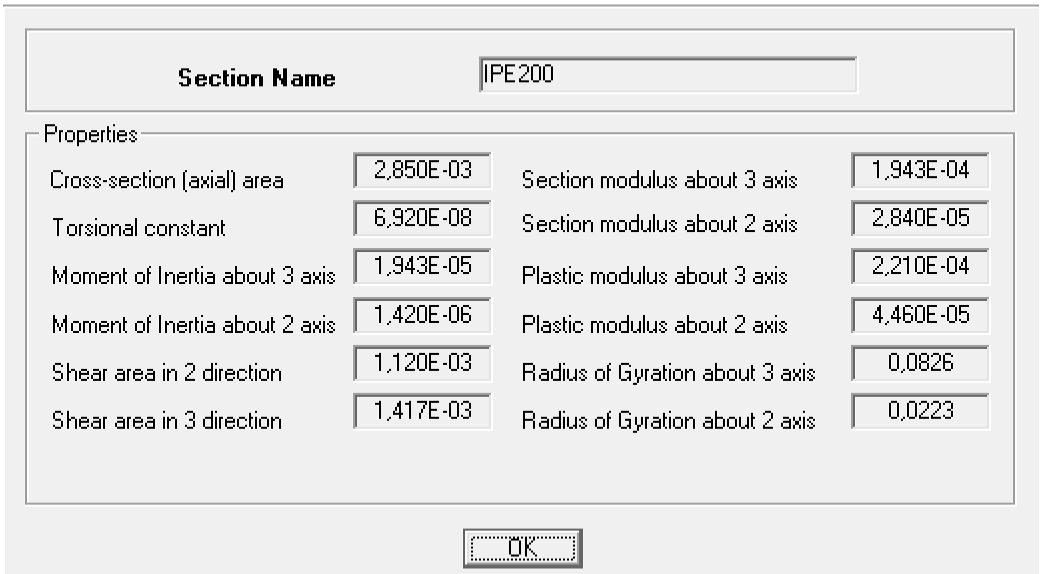
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Shear area in 3 direction
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Section modulus about 3 axis
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Section modulus about 2 axis
2,840E-05
Plastic modulus about 3 axis
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Radius of Gyration about 2 axis
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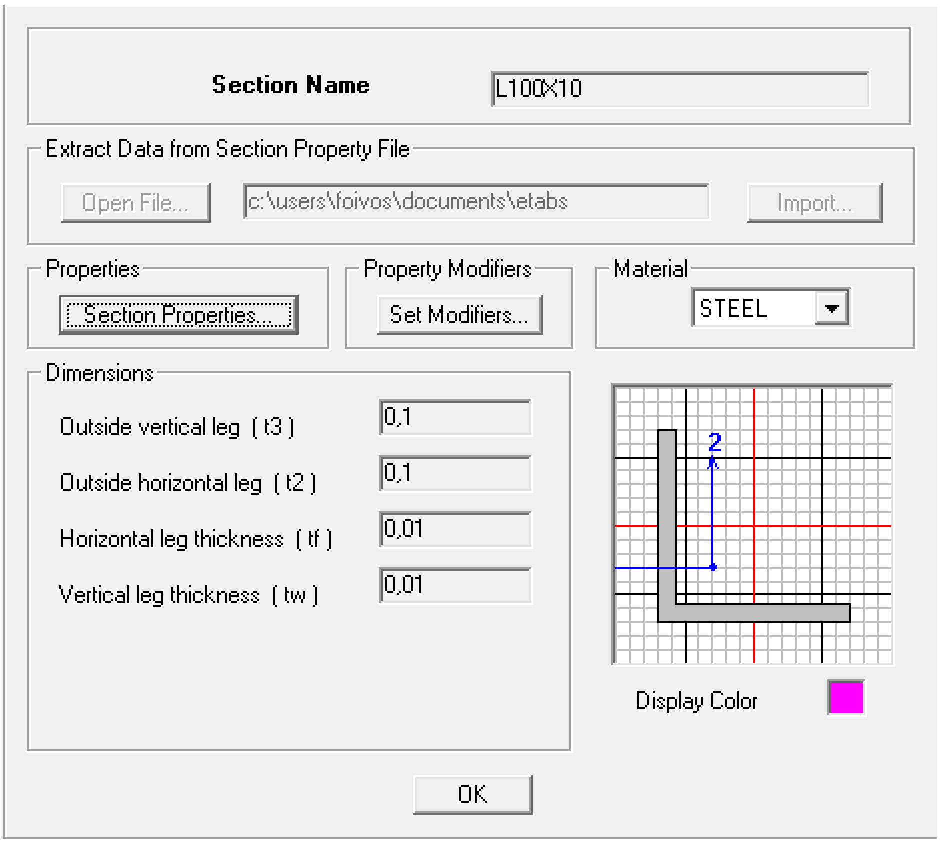
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Vertical leg thickness (tw)
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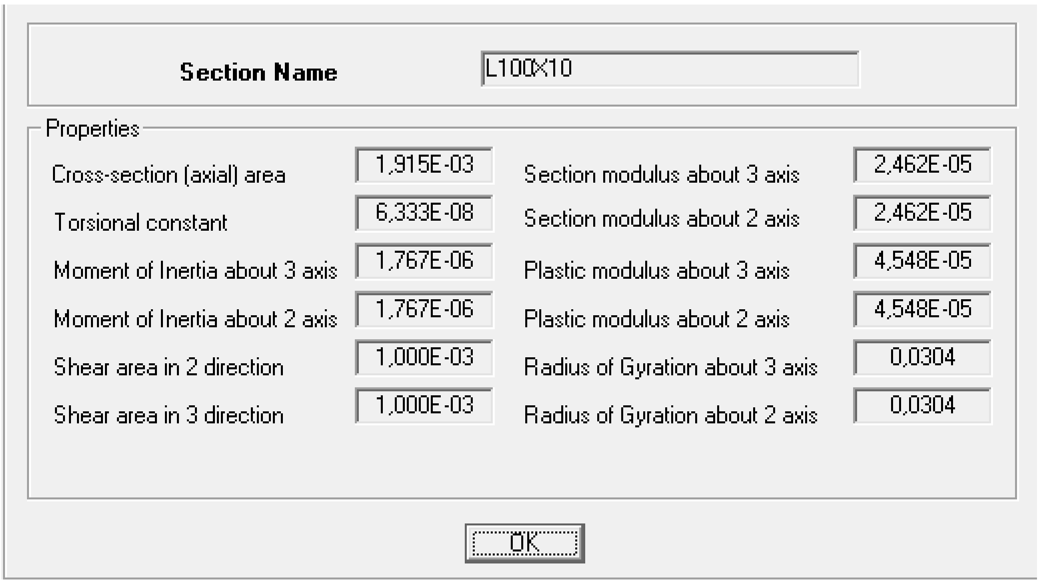
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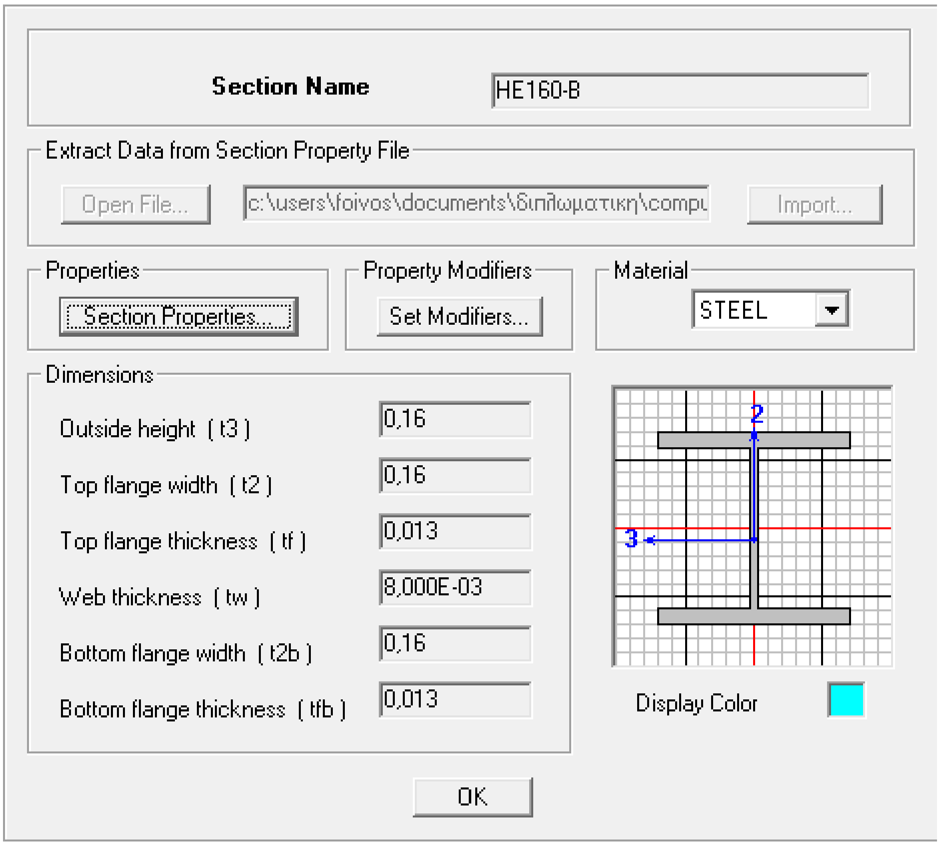
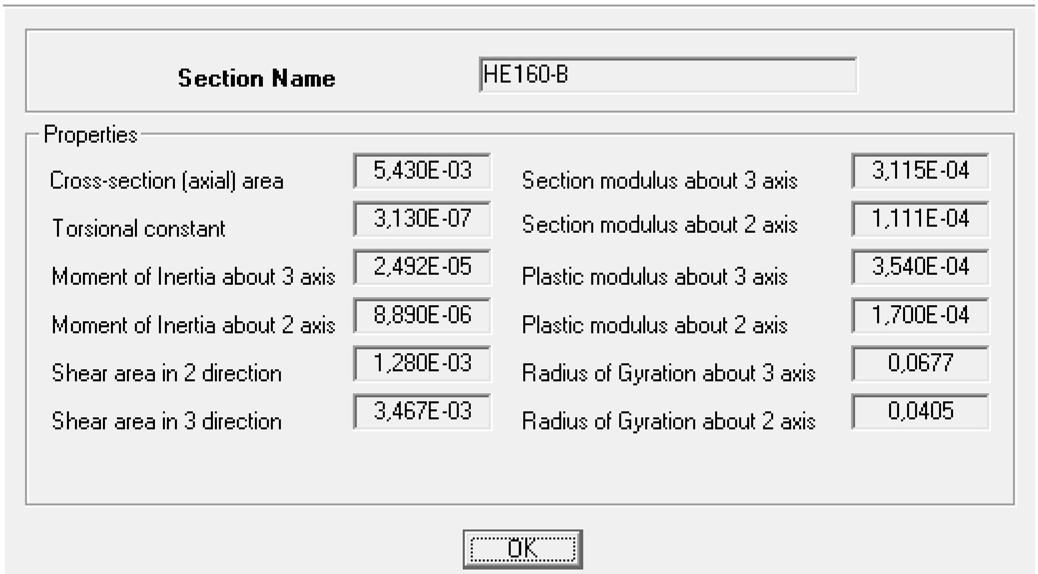
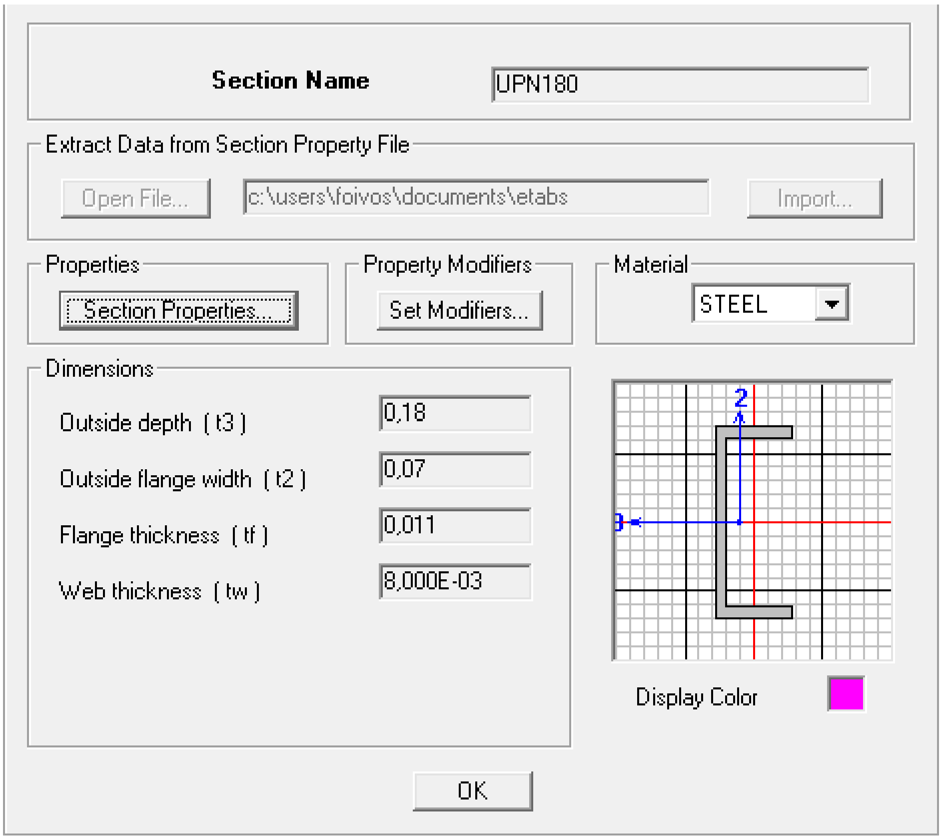
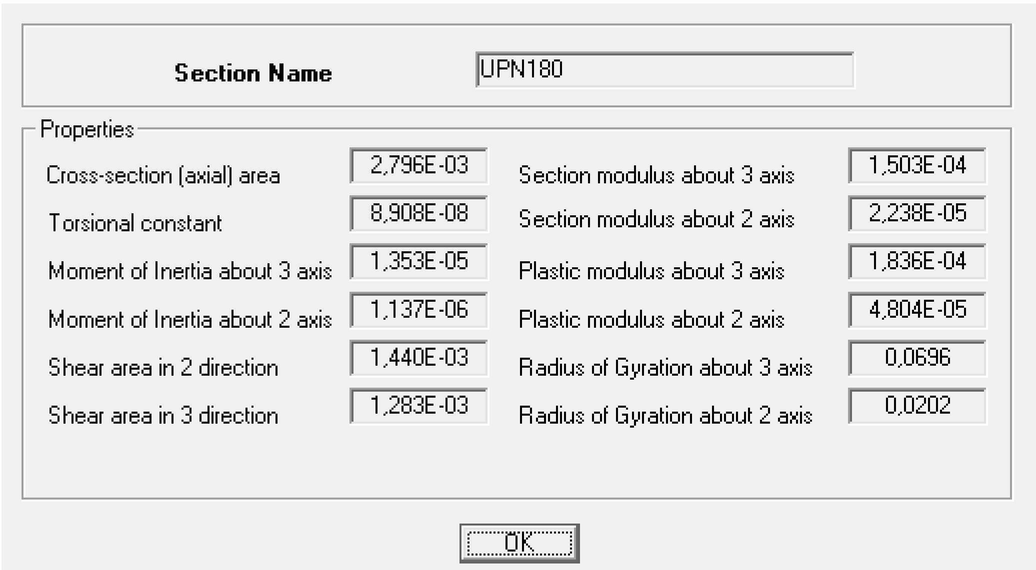
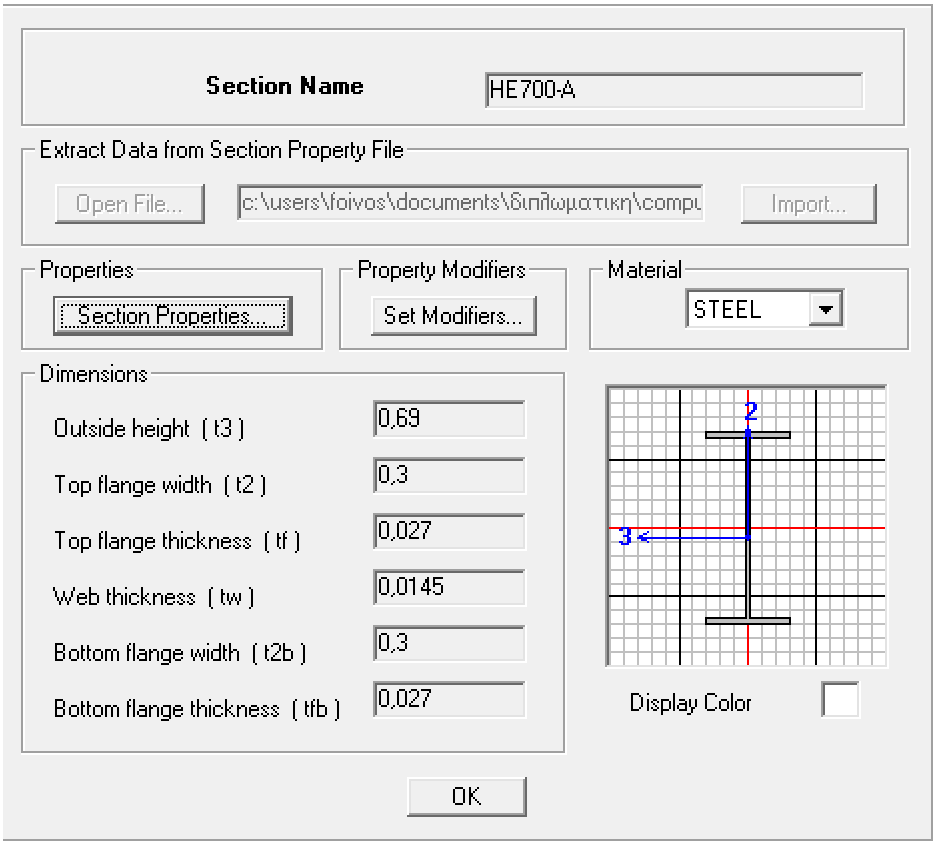
6,333E-08
Moment of Inertia about 3 axis
1,767E-06
Moment of Inertia about 2 axis
1,767E-06
Shear area in 2 direction
1,000E-03
Shear area in 3 direction
1,000E-03
Section modulus about 3 axis
2,462E-05
Section modulus about 2 axis
2,462E-05
Plastic modulus about 3 axis
4,548E-05
Plastic modulus about 2 axis
4,548E-05
Radius of Gyration about 3 axis
0,0304
Radius of Gyration about 2 axis
0,0304
OK


Section Name
HE160-B
Extract Data from Section Property File
Open File...
c:\users\foivos\documents\διπλωματικη\compu
Import...
Properties
Section Properties...
Property Modifiers
Set Modifiers...
Material
STEEL
Dimensions
Outside height (t3)
0,16
Top flange width (t2)
0,16
Top flange thickness (tf)
0,013
Web thickness (tw)
8,000E-03
Bottom flange width (t2b)
0,16
Bottom flange thickness (tfb)
0,013
2
3
Display Color
OK


Section Name
HE160-B
Properties
Cross-section (axial) area
5,430E-03
Torsional constant
3,130E-07
Moment of Inertia about 3 axis
2,492E-05
Moment of Inertia about 2 axis
8,890E-06
Shear area in 2 direction
1,280E-03
Shear area in 3 direction
3,467E-03
Section modulus about 3 axis
3,115E-04
Section modulus about 2 axis
1,111E-04
Plastic modulus about 3 axis
3,540E-04
Plastic modulus about 2 axis
1,700E-04
Radius of Gyration about 3 axis
0,0677
Radius of Gyration about 2 axis
0,0405
OK


Section Name
UPN180
Extract Data from Section Property File
Open File...
c:\users\foivos\documents\etabs
Import...
Properties
Section Properties...
Property Modifiers
Set Modifiers...
Material
STEEL
Dimensions
Outside depth (t3)
0,18
Outside flange width (t2)
0,07
Flange thickness (tf)
0,011
Web thickness (tw)
8,000E-03
2
3
Display Color
OK


Section Name
UPN180
Properties
Cross-section (axial) area
2,796E-03
Torsional constant
8,908E-08
Moment of Inertia about 3 axis
1,353E-05
Moment of Inertia about 2 axis
1,137E-06
Shear area in 2 direction
1,440E-03
Shear area in 3 direction
1,283E-03
Section modulus about 3 axis
1,503E-04
Section modulus about 2 axis
2,238E-05
Plastic modulus about 3 axis
1,836E-04
Plastic modulus about 2 axis
4,804E-05
Radius of Gyration about 3 axis
0,0696
Radius of Gyration about 2 axis
0,0202
OK


Section Name
HE700-A
Extract Data from Section Property File
Open File...
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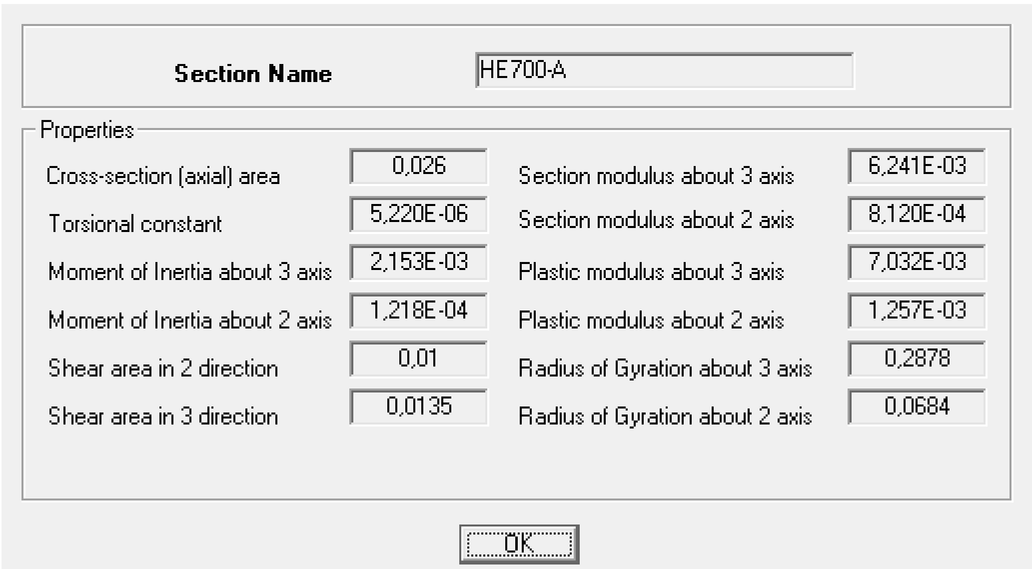
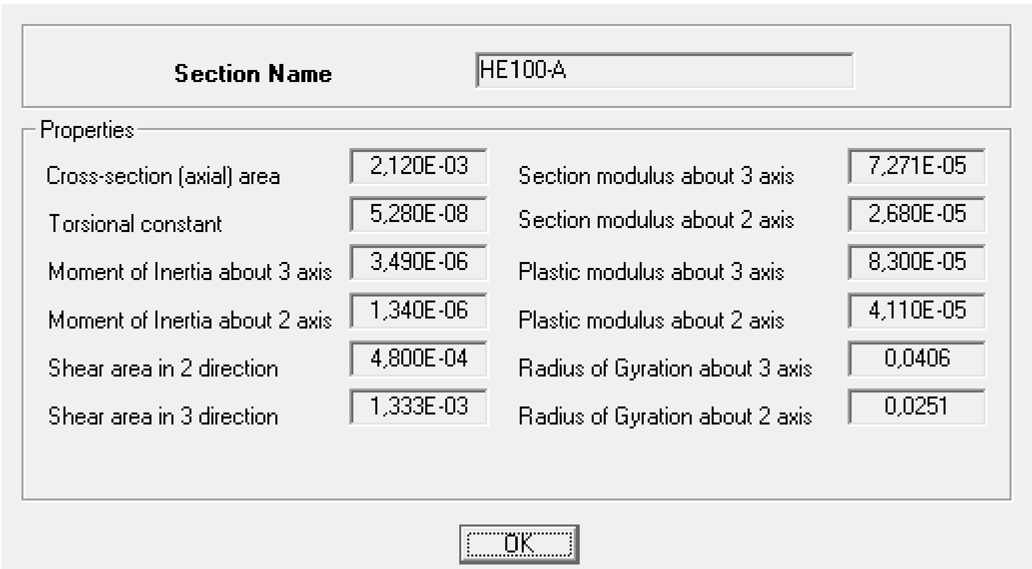
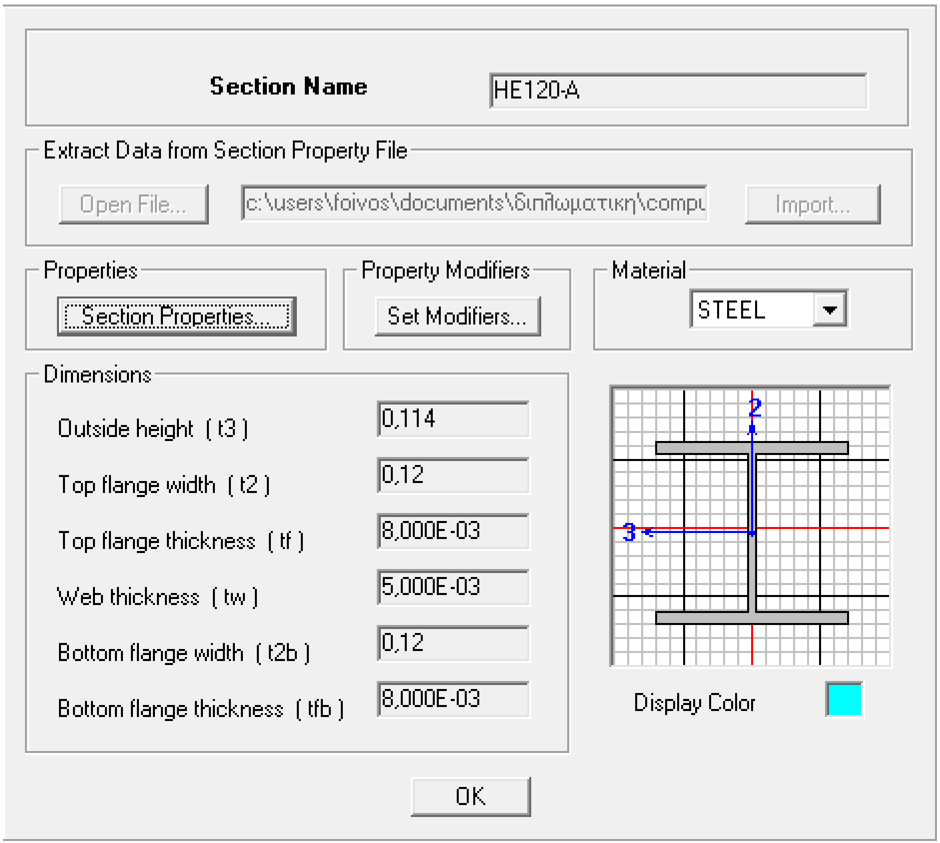
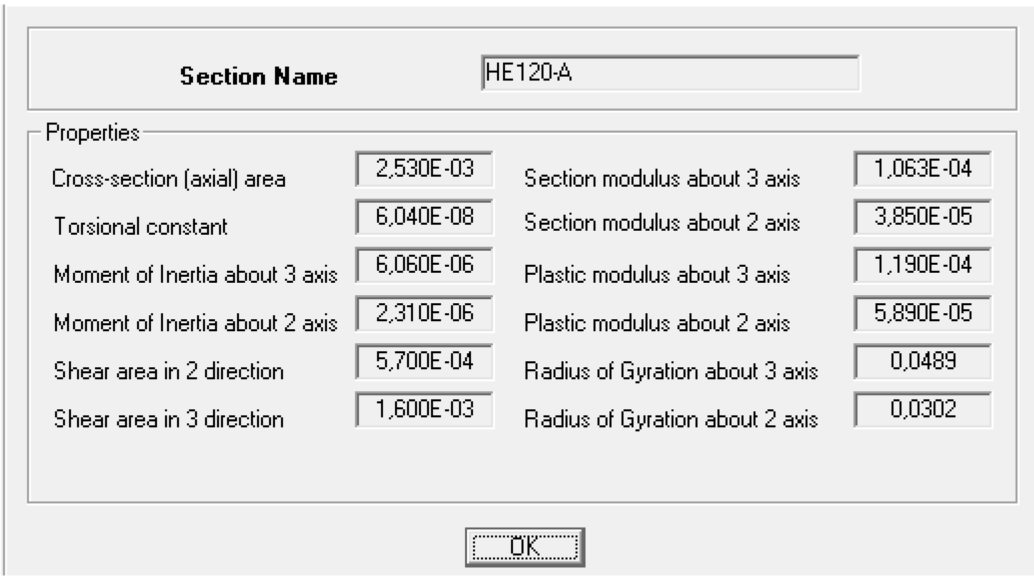
Import...
Properties
Section Properties...
Property Modifiers
Set Modifiers...
Material
STEEL
Dimensions
Outside height (t3)
0,69
Top flange width (t2)
0,3
Top flange thickness (tf)
0,027
Web thickness (tw)
0,0145
Bottom flange width (t2b)
0,3
Bottom flange thickness (tfb)
0,027
2
3
Display Color
OK

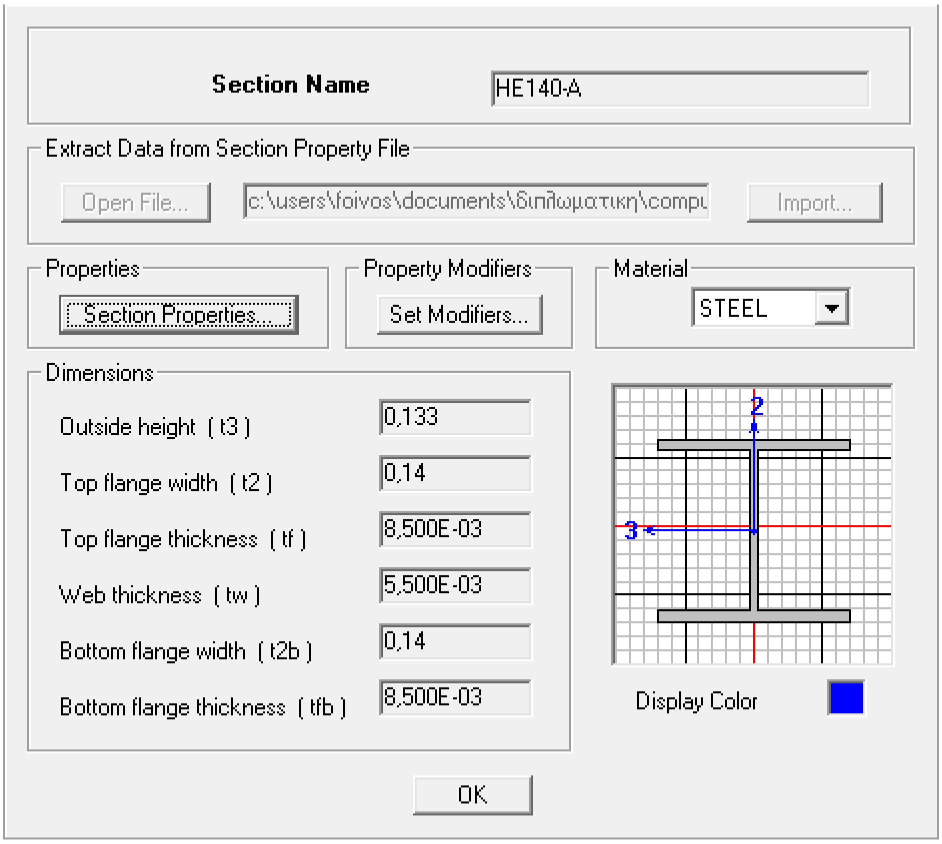

Section Name
HE700-A
Properties
Cross-section (axial) area
0,026
Torsional constant
5,220E-06
Moment of Inertia about 3 axis
2,153E-03
Moment of Inertia about 2 axis
1,218E-04
Shear area in 2 direction
0,01
Shear area in 3 direction
0,0135
Section modulus about 3 axis
6,241E-03
Section modulus about 2 axis
8,120E-04
Plastic modulus about 3 axis
7,032E-03
Plastic modulus about 2 axis
1,257E-03
Radius of Gyration about 3 axis
0,2878
Radius of Gyration about 2 axis
0,0684
OK

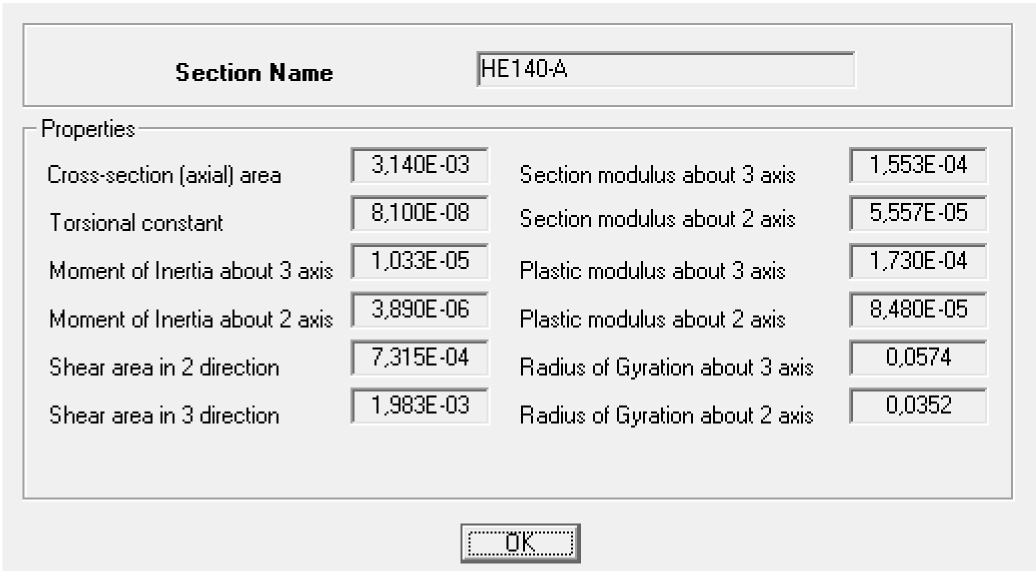

Section Name
HE100-A
Extract Data from Section Property File
Open File...
c:\users\foivos\documents\διπλωματικη\compu
Import...
Properties
Section Properties...
Property Modifiers
Set Modifiers...
Material
STEEL
Dimensions
Outside height (t3)
0,096
Top flange width (t2)
0,1
Top flange thickness (tf)
8,000E-03
Web thickness (tw)
5,000E-03
Bottom flange width (t2b)
0,1
Bottom flange thickness (tfb)
8,000E-03
2
3
Display Color
OK

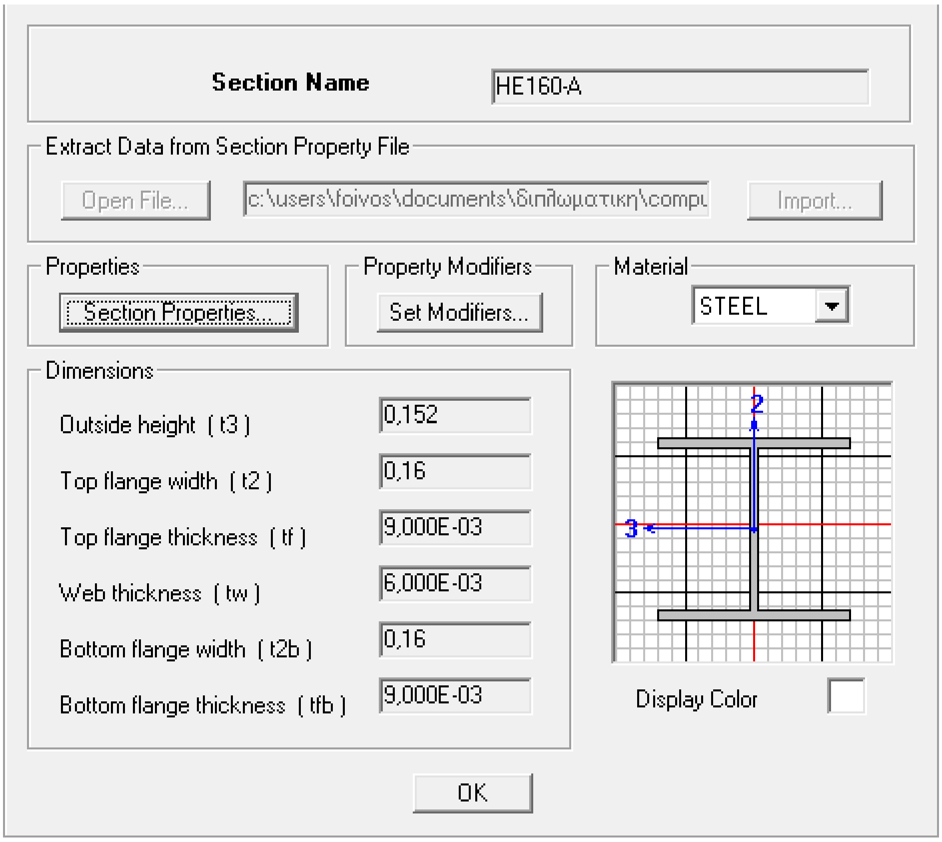

Section Name
HE100-A
Properties
Cross-section (axial) area
2,120E-03
Torsional constant
5,280E-08
Moment of Inertia about 3 axis
3,490E-06
Moment of Inertia about 2 axis
1,340E-06
Shear area in 2 direction
4,800E-04
Shear area in 3 direction
1,333E-03
Section modulus about 3 axis
7,271E-05
Section modulus about 2 axis
2,680E-05
Plastic modulus about 3 axis
8,300E-05
Plastic modulus about 2 axis
4,110E-05
Radius of Gyration about 3 axis
0,0406
Radius of Gyration about 2 axis
0,0251
OK

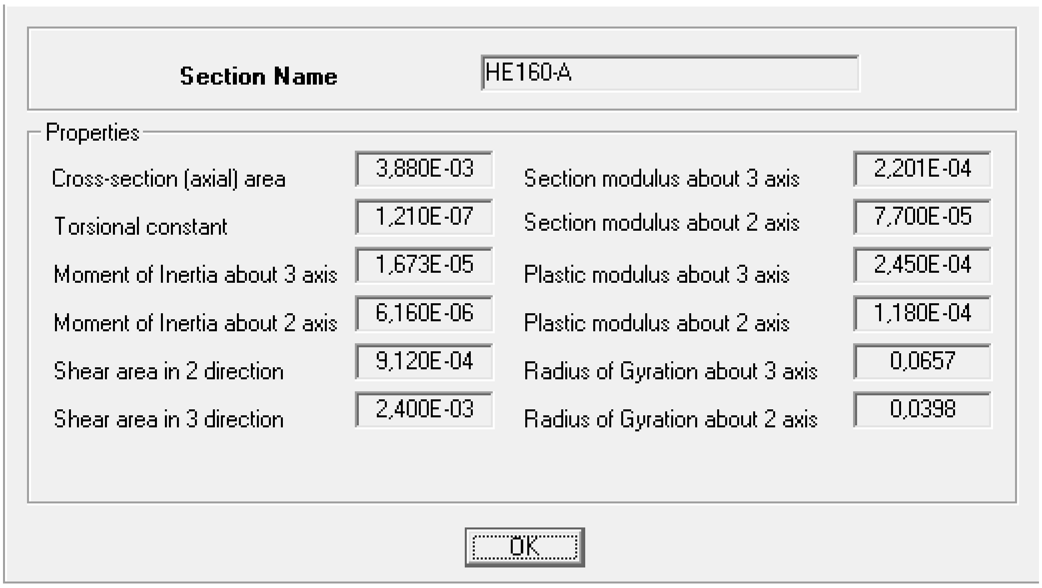

Section Name
HE120-A
Extract Data from Section Property File
Open File...
c:\users\foivos\documents\διπλωματικη\compu
Import...
Properties
Section Properties...
Property Modifiers
Set Modifiers...
Material
STEEL
Dimensions
Outside height (t3)
0,114
Top flange width (t2)
0,12
Top flange thickness (tf)
8,000E-03
Web thickness (tw)
5,000E-03
Bottom flange width (t2b)
0,12
Bottom flange thickness (tfb)
8,000E-03
2
3
Display Color
OK

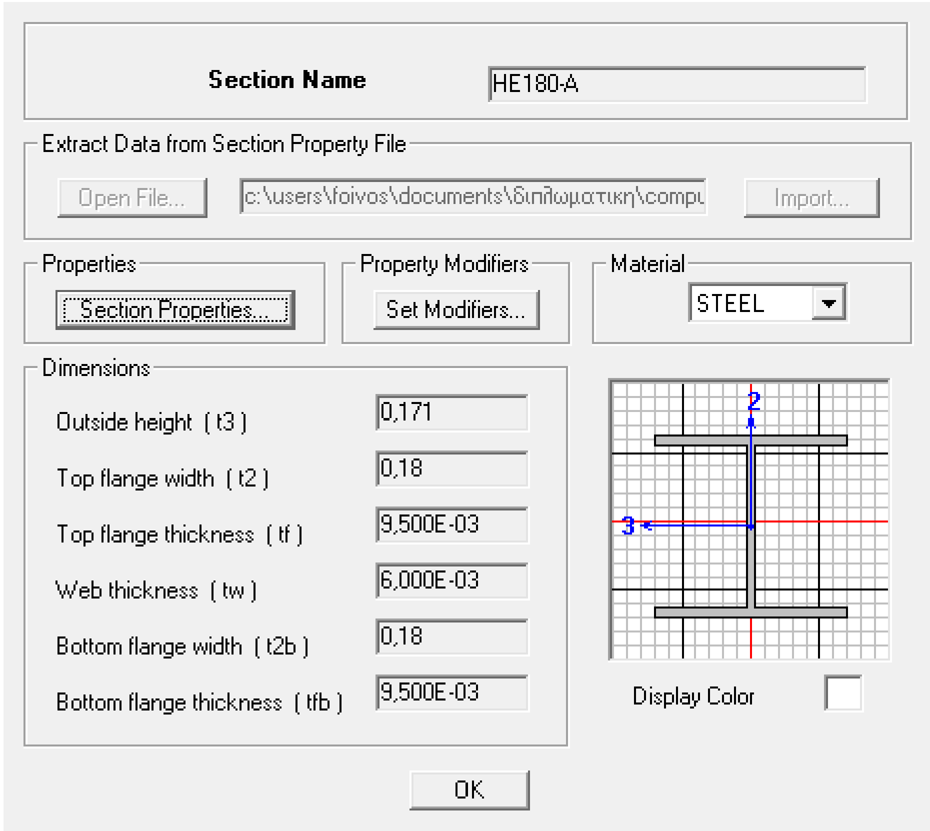

Section Name
HE120-A
Properties
Cross-section (axial) area
2,530E-03
Torsional constant
6,040E-08
Moment of Inertia about 3 axis
6,060E-06
Moment of Inertia about 2 axis
2,310E-06
Shear area in 2 direction
5,700E-04
Shear area in 3 direction
1,600E-03
Section modulus about 3 axis
1,063E-04
Section modulus about 2 axis
3,850E-05
Plastic modulus about 3 axis
1,190E-04
Plastic modulus about 2 axis
5,890E-05
Radius of Gyration about 3 axis
0,0489
Radius of Gyration about 2 axis
0,0302
OK

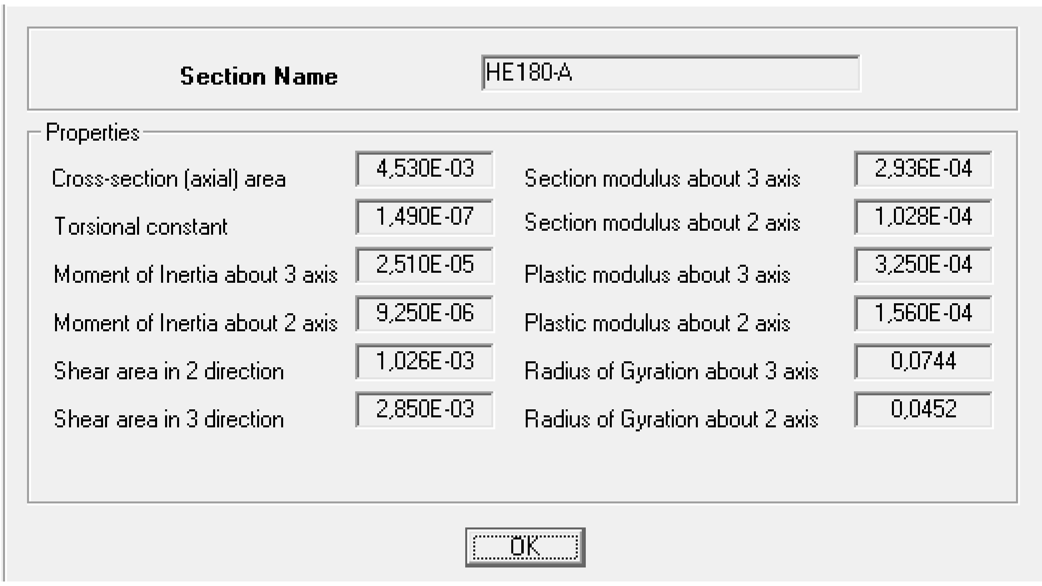

Section Name
HE140-A
Extract Data from Section Property File
Open File...
c:\users\foivos\documents\διπλωματικη\compu
Import...
Properties
Section Properties...
Property Modifiers
Set Modifiers...
Material
STEEL
Dimensions
Outside height (t3)
0,133
Top flange width (t2)
0,14
Top flange thickness (tf)
8,500E-03
Web thickness (tw)
5,500E-03
Bottom flange width (t2b)
0,14
Bottom flange thickness (tfb)
8,500E-03
2
3
Display Color
OK

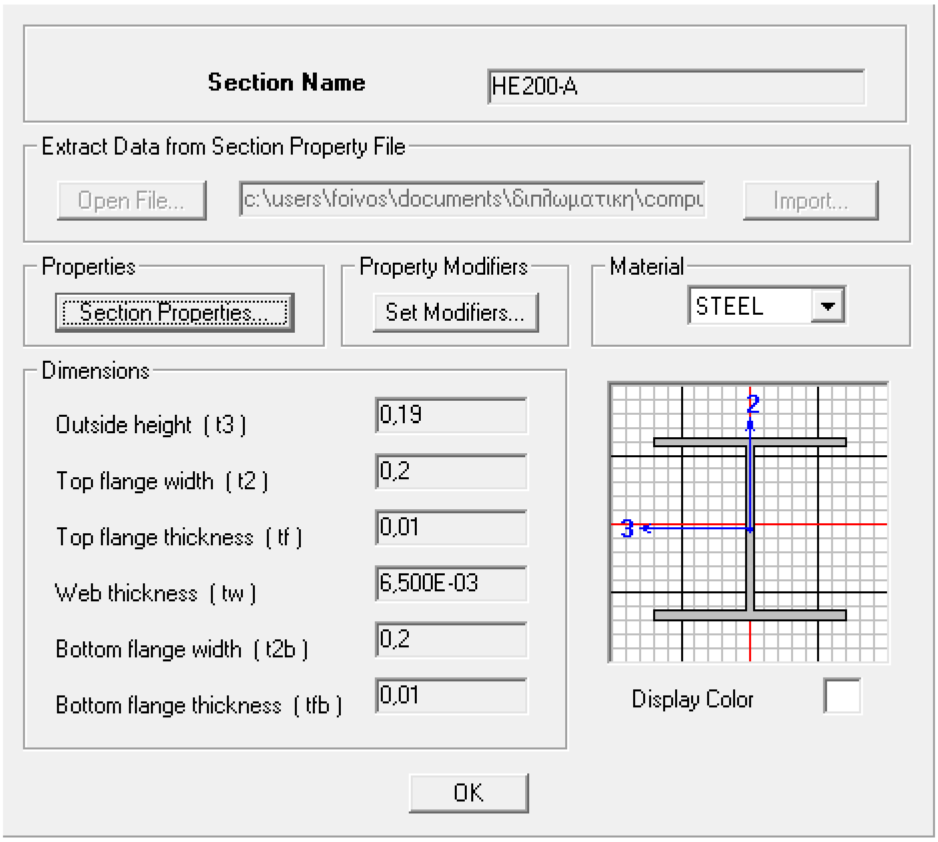

Section Name
HE140-A
Properties
Cross-section (axial) area
3,140E-03
Torsional constant
8,100E-08
Moment of Inertia about 3 axis
1,033E-05
Moment of Inertia about 2 axis
3,890E-06
Shear area in 2 direction
7,315E-04
Shear area in 3 direction
1,983E-03
Section modulus about 3 axis
1,553E-04
Section modulus about 2 axis
5,557E-05
Plastic modulus about 3 axis
1,730E-04
Plastic modulus about 2 axis
8,480E-05
Radius of Gyration about 3 axis
0,0574
Radius of Gyration about 2 axis
0,0352
OK

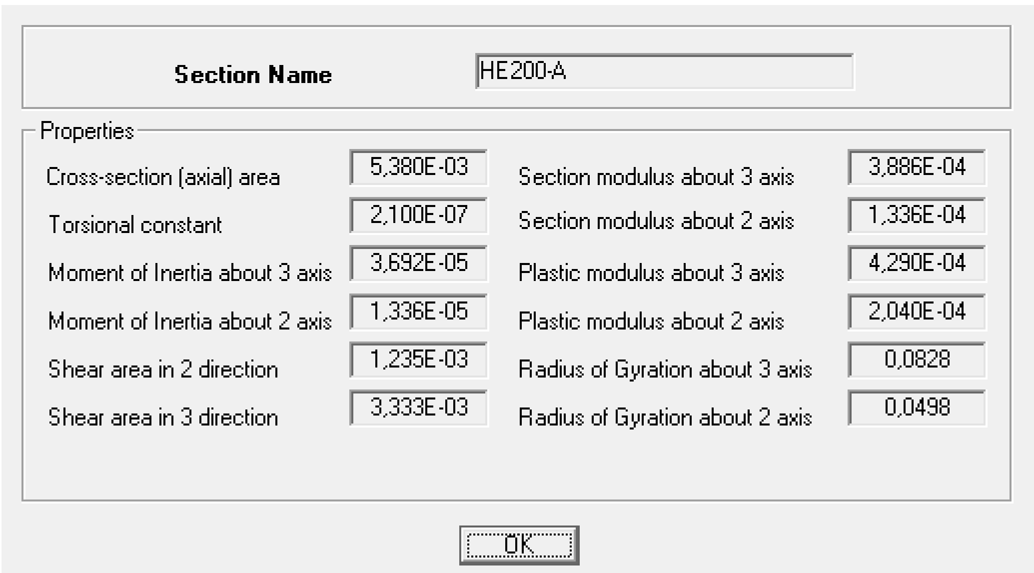

Section Name
HE160-A
Extract Data from Section Property File
Open File...
c:\users\foivos\documents\διπλωματικη\compu
Import...
Properties
Section Properties...
Property Modifiers
Set Modifiers...
Material
STEEL
Dimensions
Outside height (t3)
0,152
Top flange width (t2)
0,16
Top flange thickness (tf)
9,000E-03
Web thickness (tw)
6,000E-03
Bottom flange width (t2b)
0,16
Bottom flange thickness (tfb)
9,000E-03
2
3
Display Color
OK


Section Name
HE160-A
Properties
Cross-section (axial) area
3,880E-03
Torsional constant
1,210E-07
Moment of Inertia about 3 axis
1,673E-05
Moment of Inertia about 2 axis
6,160E-06
Shear area in 2 direction
9,120E-04
Shear area in 3 direction
2,400E-03
Section modulus about 3 axis
2,201E-04
Section modulus about 2 axis
7,700E-05
Plastic modulus about 3 axis
2,450E-04
Plastic modulus about 2 axis
1,180E-04
Radius of Gyration about 3 axis
0,0657
Radius of Gyration about 2 axis
0,0398
OK


Section Name
HE180-A
Extract Data from Section Property File
Open File...
c:\users\foivos\documents\διπλωματικη\compu
Import...
Properties
Section Properties...
Property Modifiers
Set Modifiers...
Material
STEEL
Dimensions
Outside height (t3)
0,171
Top flange width (t2)
0,18
Top flange thickness (tf)
9,500E-03
Web thickness (tw)
6,000E-03
Bottom flange width (t2b)
0,18
Bottom flange thickness (tfb)
9,500E-03
2
3
Display Color
OK


Section Name
HE180-A
Properties
Cross-section (axial) area
4,530E-03
Torsional constant
1,490E-07
Moment of Inertia about 3 axis
2,510E-05
Moment of Inertia about 2 axis
9,250E-06
Shear area in 2 direction
1,026E-03
Shear area in 3 direction
2,850E-03
Section modulus about 3 axis
2,936E-04
Section modulus about 2 axis
1,028E-04
Plastic modulus about 3 axis
3,250E-04
Plastic modulus about 2 axis
1,560E-04
Radius of Gyration about 3 axis
0,0744
Radius of Gyration about 2 axis
0,0452
OK


Section Name
HE200-A
Extract Data from Section Property File
Open File...
c:\users\foivos\documents\διπλωματικη\compu
Import...
Properties
Section Properties...
Property Modifiers
Set Modifiers...
Material
STEEL
Dimensions
Outside height (t3) 0,19
Top flange width (t2) 0,2
Top flange thickness (tf) 0,01
Web thickness (tw) 6,500E-03
Bottom flange width (t2b) 0,2
Bottom flange thickness (tfb) 0,01
2
3
Display Color
OK


Section Name
HE200-A
Properties
Cross-section (axial) area 5,380E-03
Torsional constant 2,100E-07
Moment of Inertia about 3 axis 3,692E-05
Moment of Inertia about 2 axis 1,336E-05
Shear area in 2 direction 1,235E-03
Shear area in 3 direction 3,333E-03
Section modulus about 3 axis 3,886E-04
Section modulus about 2 axis 1,336E-04
Plastic modulus about 3 axis 4,290E-04
Plastic modulus about 2 axis 2,040E-04
Radius of Gyration about 3 axis 0,0828
Radius of Gyration about 2 axis 0,0498
OK

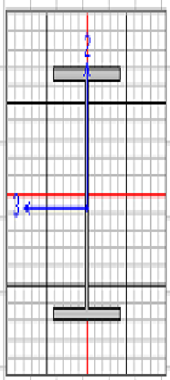
5.2 ANALYSIS OF CRITICAL SECTIONS

A) ANALYSIS OF CRITICAL COLUMN (HEB 500)

The critical column is in the first panel in the case that the first wheel of the crane bridge is upon it and the second wheel after 3,5m:

File

Units KN-m



AISC-LRFD93 STEEL SECTION CHECK Units: KN-m (Summary for Combo and Station)
Level: STORY1 Element: C3 Station Loc: 3,000 Section ID: HE500-B
Element Type: Moment Resisting Frame Classification: Compact

L=24,000
A=0,024 i22=1,262E-04 i33=0,001 z22=0,001 z33=0,005
s22=8,413E-04 s33=0,004 r22=0,073 r33=0,212
E=210000000,00 Fy=235000,000
RLLF=1,000

P-M33-M22 Demand/Capacity Ratio is 0,632 = 0,610 + 0,000 + 0,021

STRESS CHECK FORCES & MOMENTS

		P	M33	M22	V2	V3
Combo	SC201	-1640,477	0,059	-6,440	-0,019	2,147

AXIAL FORCE & BIAXIAL MOMENT DESIGN (H1-1a)

	Pu Load	phi*Pnc Strength	phi*Pnt Strength
Axial	1640,477	2687,990	5054,850

	Mu Moment	phi*Mn Capacity	Cm Factor	B1 Factor	B2 Factor	K Factor	L Factor	Cb Factor
Major Bending	0,059	1018,373	1,000	1,669	1,000	1,000	0,971	1,139
Minor Bending	6,440	266,913	0,850	1,000	1,000	1,000	0,125	

SHEAR DESIGN

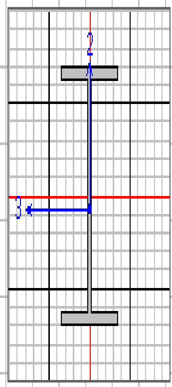
	Vu Force	Phi*Vn Strength	Stress Ratio
Major Shear	0,019	920,025	2,074E-05
Minor Shear	2,147	1776,600	0,001

B) ANALYSIS OF CRITICAL COLUMN (HEB 600)

It is in the third panel and the critical combination is DSTLS16:

File

Units KN-m



AISC-LRFD93 STEEL SECTION CHECK   Units: KN-m (Summary for Combo and Station)
Level: STORY2 Element: C9 Station Loc: 5,840 Section ID: HE600-B
Element Type: Moment Resisting Frame   Classification: Compact

L=6,000
A=0,027 i22=1,353E-04 i33=0,002 z22=0,001 z33=0,006
s22=9,020E-04 s33=0,006 r22=0,071 r33=0,252
E=210000000,00 fy=235000,000
RLLF=1,000

P-M33-M22 Demand/Capacity Ratio is   0,932 = 0,027 + 0,904 + 0,000

STRESS CHECK FORCES & MOMENTS

	P	M33	M22	V2	V3
Combo DSTLS16	-287,710	1229,036	-0,015	-250,825	-0,345

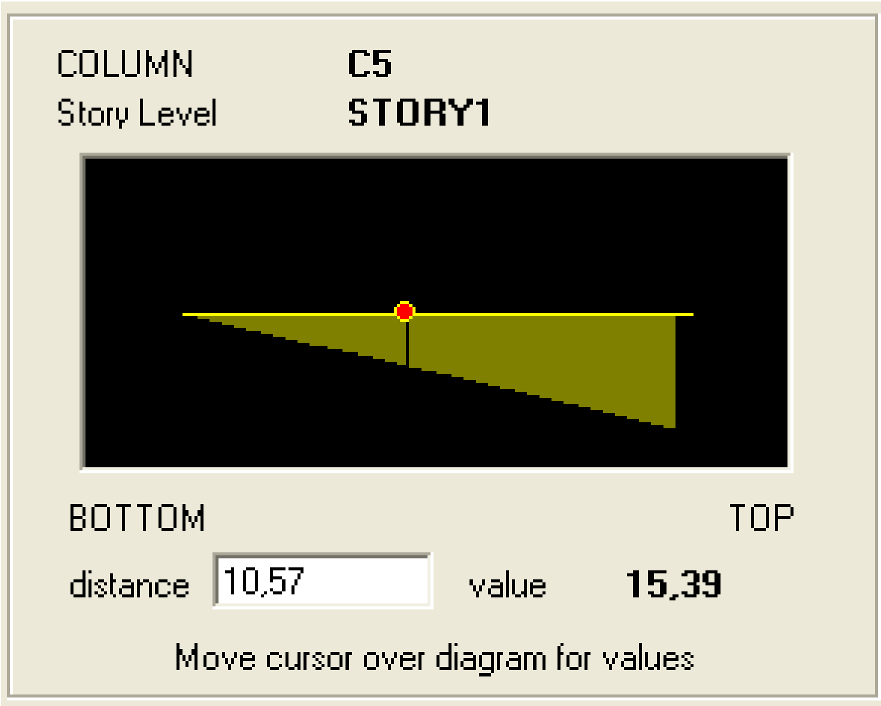
AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-1b)

	Pu Load	phi*Pnc Strength	phi*Pnt Strength
Axial	287,710	5257,167	5710,500

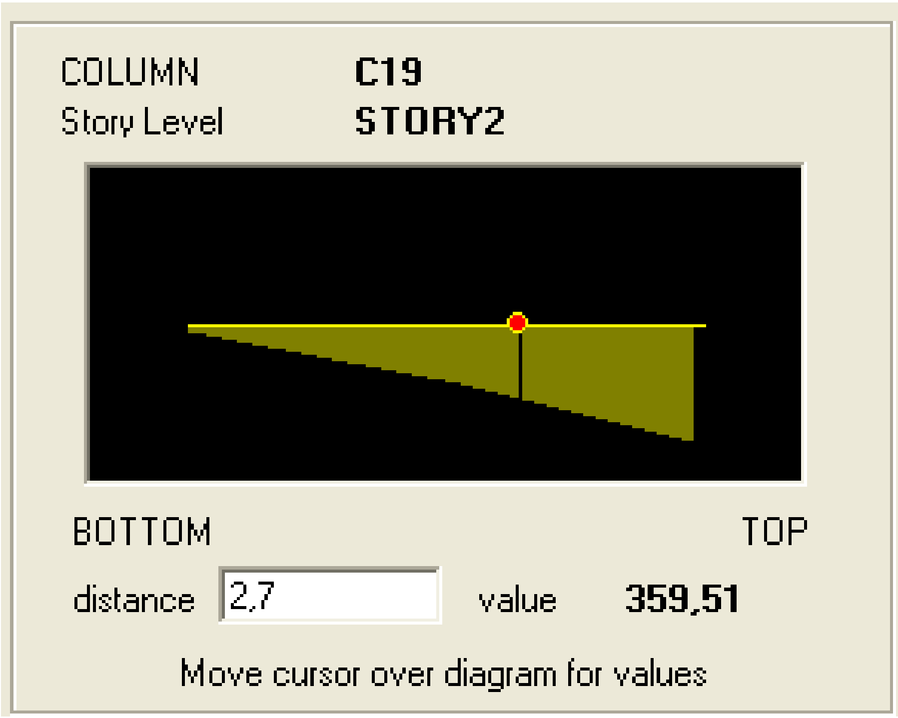
	Mu Moment	phi*Mn Capacity	Cm Factor	B1 Factor	B2 Factor	K Factor	L Factor	Cb Factor
Major Bending	1229,036	1358,888	0,850	1,000	1,000	1,000	0,973	1,223
Minor Bending	0,015	286,160	0,850	1,000	1,000	1,000	0,250	

SHEAR DESIGN

	Vu Force	Phi*Vn Strength	Stress Ratio
Major Shear	250,825	1180,170	0,213
Minor Shear	0,345	1903,500	0,000



MOMENT DIAGRAM OF COLUMN HEB 500 FOR COMBINATION SC201



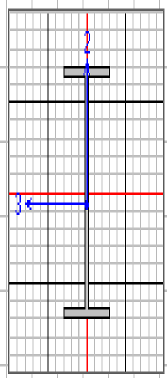
MOMENT DIAGRAM OF COLUMN HEB 600 FOR COMBINATION DSTLS16

C) ANALYSIS OF CRITICAL RUNWAY BEAM

It is in the first panel when the wheel of the crane bridge is in distance of 2,125 from the first column:

File

Units KN-m



AISC-LRFD93 STEEL SECTION CHECK Units: KN-m (Summary for Combo and Station)
Level: STORY1 Element: B835 Station Loc: 3,000 Section ID: HE700-B
Element Type: Moment Resisting Frame Classification: Compact

L=6,000
A=0,031 i22=1,444E-04 i33=0,003 z22=0,001 z33=0,008
s22=9,627E-04 s33=0,007 r22=0,069 r33=0,290
E=210000000,00 fy=235000,000
RLLF=1,000

P-M33-M22 Demand/Capacity Ratio is 0,861 = 0,020 + 0,265 + 0,576

STRESS CHECK FORCES & MOMENTS

		P	M33	M22	V2	V3
Combo	SC57	-226,443	-467,238	-175,975	-165,440	-63,991

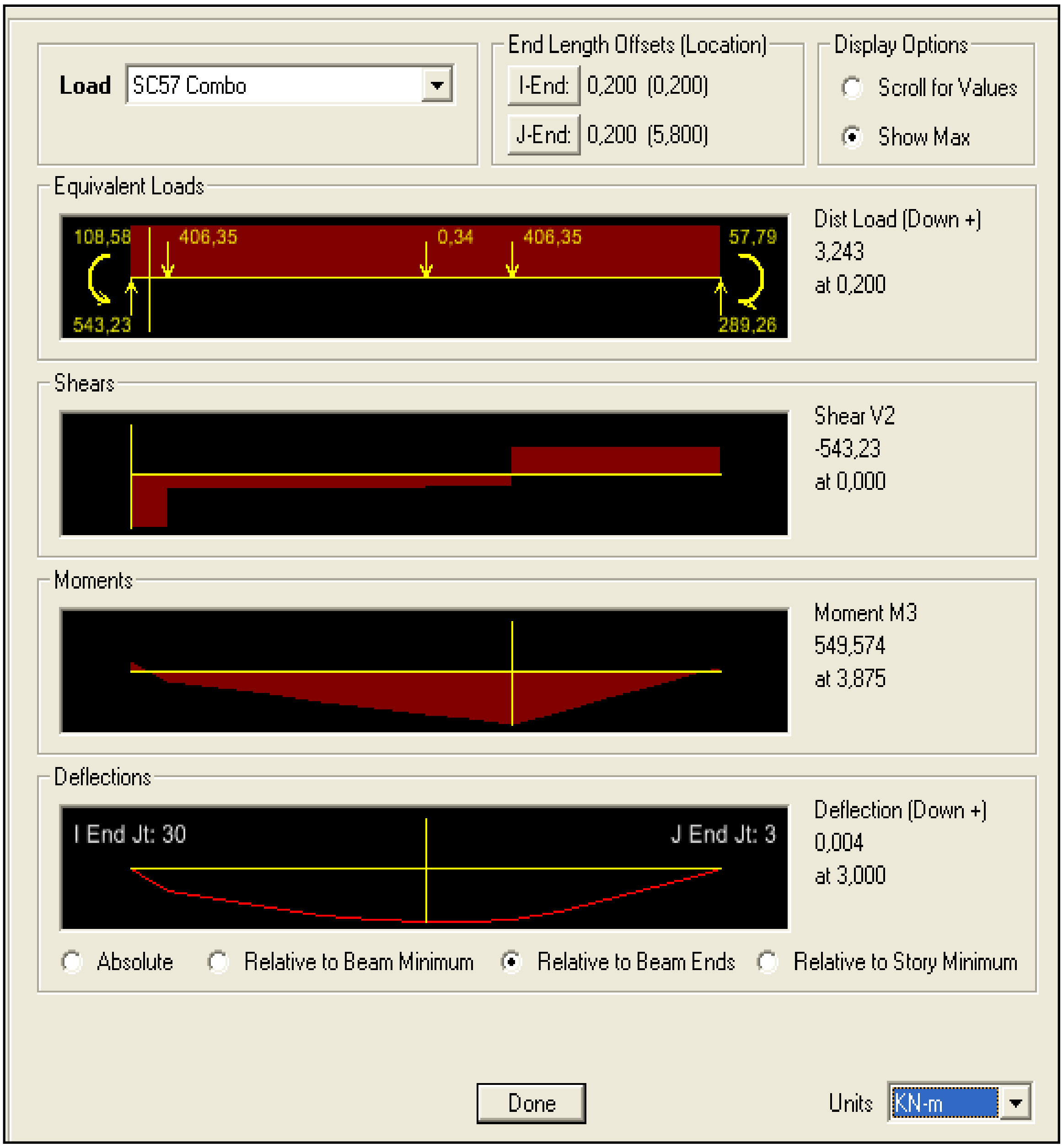
AXIAL FORCE & BIAXIAL MOMENT DESIGN (H1-1b)

	Pu Load	phi*Pnc Strength	phi*Pnt Strength
Axial	226,443	5664,725	6471,900

	Mu Moment	phi*Mn Capacity	Cm Factor	B1 Factor	B2 Factor	K Factor	L Factor	Cb Factor
Major Bending	467,238	1761,161	1,000	1,001	1,000	1,000	0,917	1,104
Minor Bending	175,975	305,406	0,850	1,000	1,000	1,000	0,458	

SHEAR DESIGN

	Vu Force	Phi*Vn Strength	Stress Ratio
Major Shear	165,440	1510,110	0,110
Minor Shear	63,991	2030,400	0,032

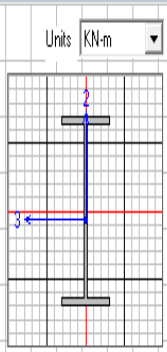


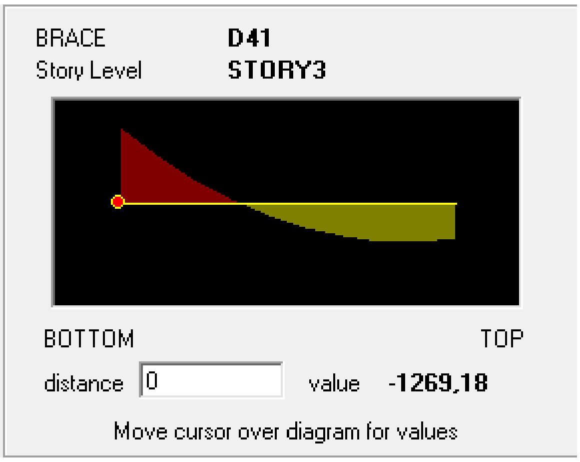
DIAGRAMS FOR RUNWAY BEAM HEB 700

D) ANALYSIS OF CRITICAL RAFTER (HEA 700)

It is in the third panel for the load combination DSTLS16:

```
File

AISC-LRFD93 STEEL SECTION CHECK    Units: KN-m  (Summary for Combo and Station)
Level: STORY3  Element: D41  Station Loc: 0,000  Section ID: HE700-A
Element Type:  Moment Resisting Frame   Classification:  Compact

L=18,282
A=0,026  i22=1,218E-04  i33=0,002  z22=0,001  z33=0,007
s22=8,120E-04  s33=0,006  r22=0,068  r33=0,288
E=210000000,00  fy=235000,000
RLLF=1,000

P-M33-M22 Demand/Capacity Ratio is   0,889 = 0,036 + 0,853 + 0,000

STRESS CHECK FORCES & MOMENTS
                           P         M33         M22          V2          V3
    Combo DSTLS16   -309,304   -1269,181       0,007    -231,455       0,259

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-1b)
                          Pu     phi*Pnc     phi*Pnt
                        Load    Strength    Strength
    Axial            309,304    4288,156    5499,000

                          Mu     phi*Mn      Cm      B1      B2       K       L      Cb
                      Moment   Capacity  Factor  Factor  Factor  Factor  Factor  Factor
    Major Bending   1269,181   1487,268   0,850   1,000   1,000   1,000   1,000   1,249
    Minor Bending      0,007    257,607   0,850   1,000   1,000   1,000   0,111

SHEAR DESIGN
                          Vu     Phi*Vn      Stress
                       Force   Strength       Ratio
    Major Shear      231,455   1270,269       0,182
    Minor Shear        0,259   1713,150       0,000
```





MOMENT DIAGRAM FOR RAFTER HEA700

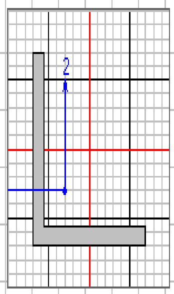
D) ANALYSIS OF HORIZONTAL X-BRACING L100.10

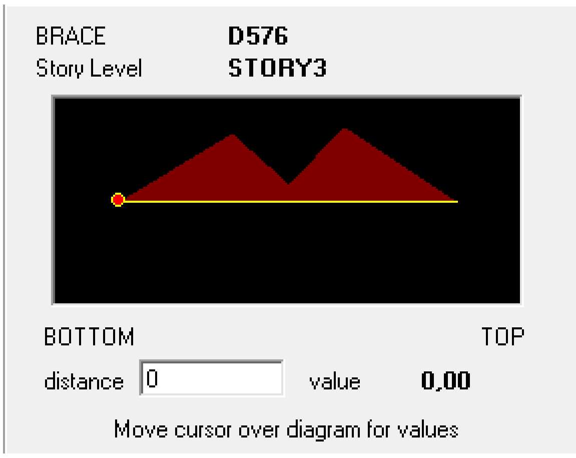
It is in the last panel for the load combination SC209:

File

```
AISC-LRFD93 STEEL SECTION CHECK    Units: KN-m  (Summary for Combo and Station)
Level: STORY3  Element: D616  Station Loc: 2,851  Section ID: L100X10
Element Type:  Moment Resisting Frame   Classification:  Non-Compact

L=8,552
A=0,002  i22=1,767E-06  i33=1,767E-06  z22=4,548E-05  z33=4,548E-05
s22=2,462E-05  s33=2,462E-05  r22=0,030  r33=0,030  alpha=45,000
E=210000000,00  fy=235000,000
RLLF=1,000

    Stress Check Message - kl/r > 200

P-M33-M22 Demand/Capacity Ratio is   0,594 = 0,542 + 0,020 + 0,032

STRESS CHECK FORCES & MOMENTS
                         P        M33        M22        V2        V3
    Combo   SC209   -8,496     -0,167     -0,156     0,384    -0,017

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (SAM 6-1a)
                        Pu     phi*Pnc     phi*Pnt
                      Load    Strength    Strength
    Axial            8,496      15,687     405,023

                        Mu     phi*Mn      Cm      B1      B2       K       L      Cb
                    Moment    Capacity  Factor  Factor  Factor  Factor  Factor  Factor
    Major Bending    0,156       6,967   1,000   1,118   1,000   1,000   1,000   1,000
    Minor Bending    0,167       4,607   1,000   1,747   1,000   1,000   0,333

SHEAR DESIGN
                        Vu     Phi*Vn      Stress
                     Force   Strength       Ratio
    Major Shear      0,384    126,900       0,003
    Minor Shear      0,017    126,900       0,000
```

Units KN-m





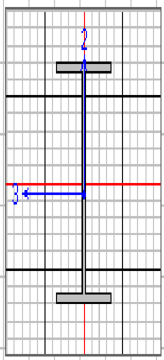
MOMENT DIAGRAM OF HORIZONTAL X-BRACING

E) ANALYSIS OF CTRITICAL PURLIN (IPE 270)

File

Units KN-m

AISC-LRFD93 STEEL SECTION CHECK   Units: KN-m (Summary for Combo and Station)
Level: STORY3 Element: B1096 Station Loc: 3,000 Section ID: IPE270
Element Type: Moment Resisting Frame   Classification: Compact



L=6,000
A=0,005 i22=4,200E-06 i33=5,790E-05 z22=9,700E-05 z33=4,840E-04
s22=6,222E-05 s33=4,289E-04 r22=0,030 r33=0,112
E=210000000,00 fy=235000,000
RLLF=1,000

P-M33-M22 Demand/Capacity Ratio is   0,707 = 0,266 + 0,441 + 0,000

STRESS CHECK FORCES & MOMENTS

	P	M33	M22	V2	V3
Combo DSTLS16	-47,878	32,475	0,000	0,000	0,000

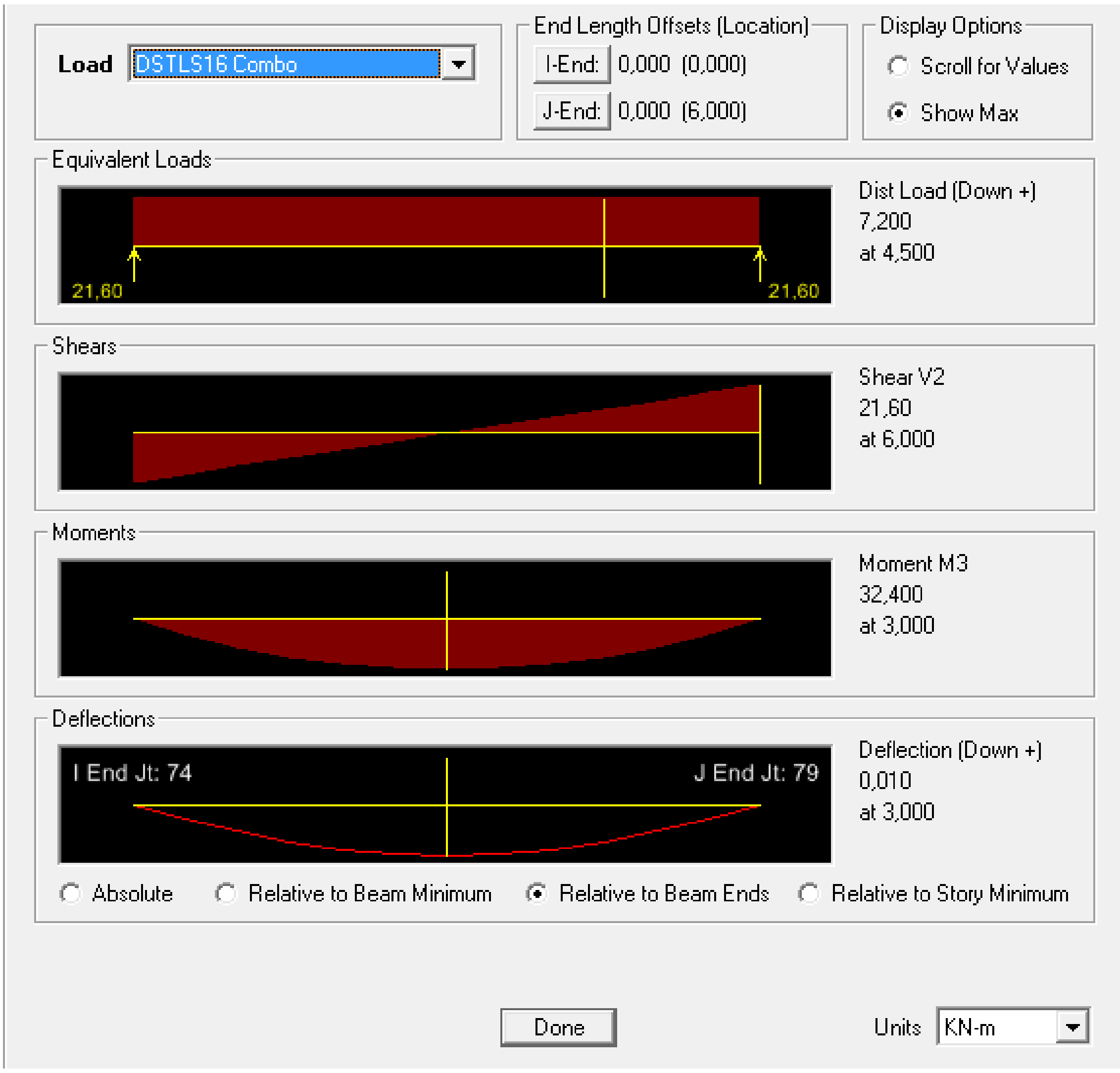
AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-1a)

	Pu Load	phi*Pnc Strength	phi*Pnt Strength
Axial	47,878	180,254	970,785

	Mu Moment	phi*Mn Capacity	Cm Factor	B1 Factor	B2 Factor	K Factor	L Factor	Cb Factor
Major Bending	32,475	65,422	1,000	1,015	1,000	1,000	1,000	1,136
Minor Bending	0,000	19,740	1,000	1,247	1,000	1,000	1,000	

SHEAR DESIGN

	Vu Force	Phi*Vn Strength	Stress Ratio
Major Shear	0,000	226,136	0,000
Minor Shear	0,000	291,236	0,000



DIAGRAMS OF FORCES FOR PURLIN (IPE 270)

F) ANALYSIS OF CRITICAL GIRD (IPE 240)

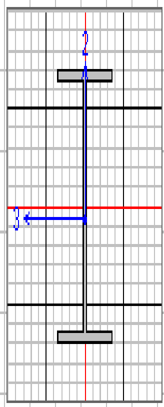
File

AISC-LRFD93 STEEL SECTION CHECK   Units: KN-m (Summary for Combo and Station)

Level: STORY1 Element: B712 Station Loc: 3,000 Section ID: IPE240

Element Type: Moment Resisting Frame   Classification: Compact

Units KN-m



L=6,000

A=0,004 i22=2,840E-06 i33=3,892E-05 z22=7,390E-05 z33=3,670E-04

s22=4,733E-05 s33=3,243E-04 r22=0,027 r33=0,100

E=210000000,00 fy=235000,000

RLLF=1,000

Stress Check Message - kl/r > 200

P-M33-M22 Demand/Capacity Ratio is   0,703 = 0,023 + 0,551 + 0,129

STRESS CHECK FORCES & MOMENTS

		P	M33	M22	V2	V3
Combo	SC204	-5,645	-25,650	-1,931	0,000	0,000

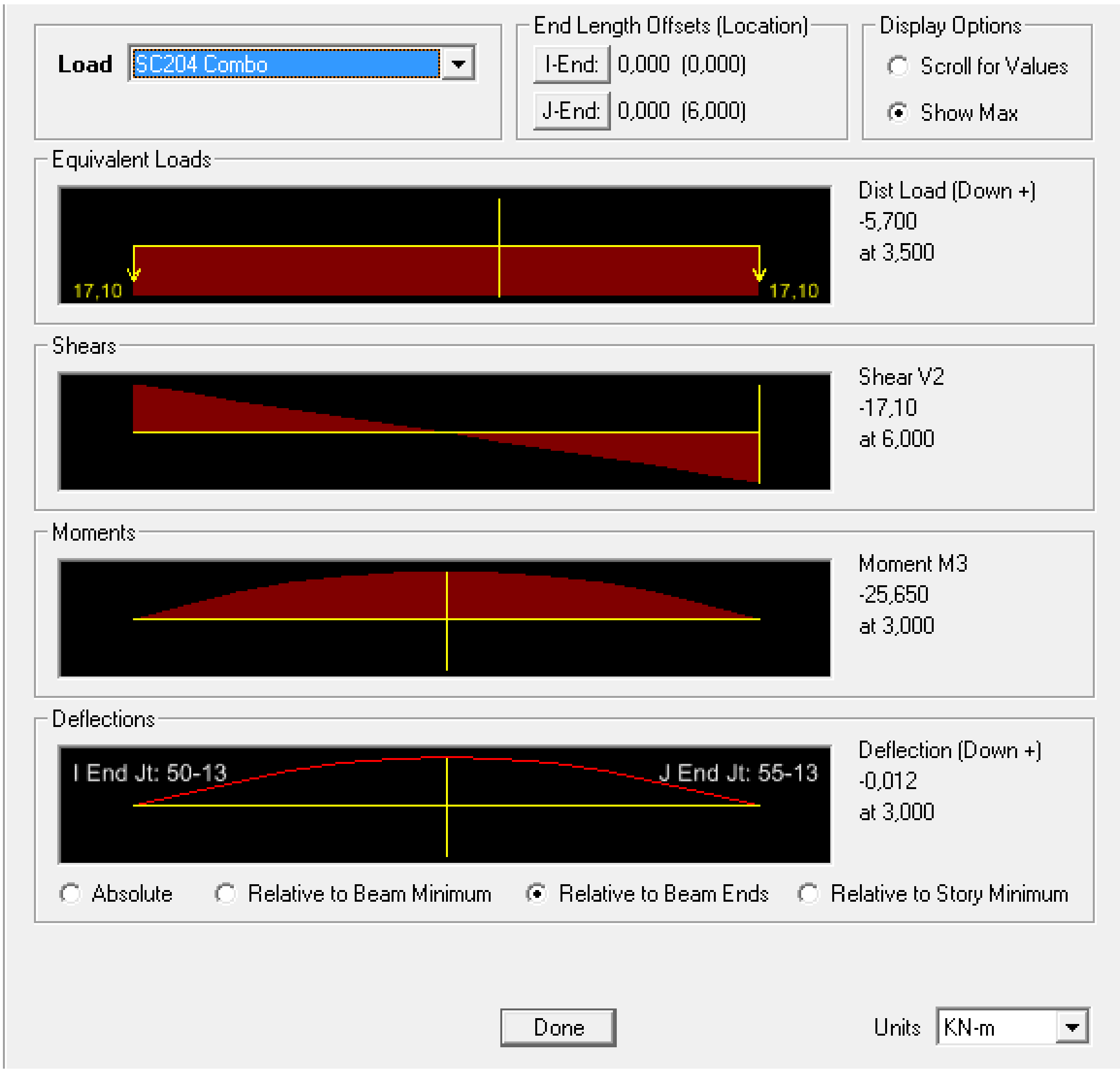
AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1-1b)

	Pu	phi*Pnc	phi*Pnt
	Load	Strength	Strength
Axial	5,645	121,886	826,965

	Mu	phi*Mn	Cm	B1	B2	K	L	Cb
	Moment	Capacity	Factor	Factor	Factor	Factor	Factor	Factor
Major Bending	25,650	46,542	1,000	1,003	1,000	1,000	1,000	1,136
Minor Bending	1,931	15,016	1,000	1,036	1,000	1,000	1,000	

SHEAR DESIGN

	Vu	Phi*Vn	Stress
	Force	Strength	Ratio
Major Shear	0,000	188,827	0,000
Minor Shear	0,000	248,724	0,000



FORCE DIAGRAMS FOR GIRD IPE 240

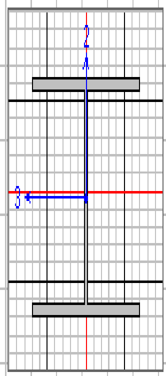
G) ANALYSIS OF CRITICAL LATTICE COLUMN TRUSS (HEA 200)

It is in the second panel for the load combination of SC202:

File

Units KN-m

AISC-LRFD93 STEEL SECTION CHECK   Units: KN-m (Summary for Combo and Station)
Level: STORY1 Element: D24 Station Loc: 2,121 Section ID: HE200-A
Element Type: Moment Resisting Frame Classification: Compact



L=4,243
A=0,005 i22=1,336E-05 i33=3,692E-05 z22=2,040E-04 z33=4,290E-04
s22=1,336E-04 s33=3,886E-04 r22=0,050 r33=0,083
E=210000000,00 fy=235000,000
RLLF=1,000

P-M33-M22 Demand/Capacity Ratio is 0,894 = 0,884 + 0,011 + 0,000

STRESS CHECK FORCES & MOMENTS

		P	M33	M22	V2	V3
Combo	SC202	-673,337	1,078	0,000	0,000	0,000

AXIAL FORCE & BIAXIAL MOMENT DESIGN (H1-1a)

	Pu Load	phi*Pnc Strength	phi*Pnt Strength
Axial	673,337	761,862	1137,870

	Mu Moment	phi*Mn Capacity	Cm Factor	B1 Factor	B2 Factor	K Factor	L Factor	Cb Factor
Major Bending	1,078	90,734	1,000	1,188	1,000	1,000	1,000	1,316
Minor Bending	0,000	42,385	1,000	1,778	1,000	1,000	1,000	

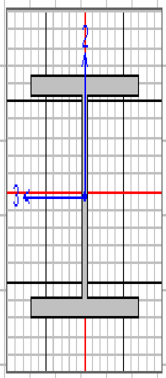
SHEAR DESIGN

	Vu Force	Phi*Vn Strength	Stress Ratio
Major Shear	0,000	156,722	0,000
Minor Shear	0,000	422,958	0,000

H) ANALYSIS OF CRITICAL RUNWAY TRUSS (HEA 100)

File

Units KN-m



AISC-LRFD93 STEEL SECTION CHECK Units: KN-m (Summary for Combo and Station)
Level: STORY1 Element: B1200 Station Loc: 2,246 Section ID: HE100-A
Element Type: Moment Resisting Frame Classification: Compact

L=4,243
A=0,002 i22=1,340E-06 i33=3,490E-06 z22=4,110E-05 z33=8,300E-05
s22=2,680E-05 s33=7,271E-05 r22=0,025 r33=0,041
E=210000000,00 fy=235000,000
RLLF=1,000

P-M33-M22 Demand/Capacity Ratio is 0,714 = 0,684 + 0,030 + 0,000

STRESS CHECK FORCES & MOMENTS

	P	M33	M22	V2	V3
Combo DSTLS28	-86,589	0,584	0,000	0,052	0,000

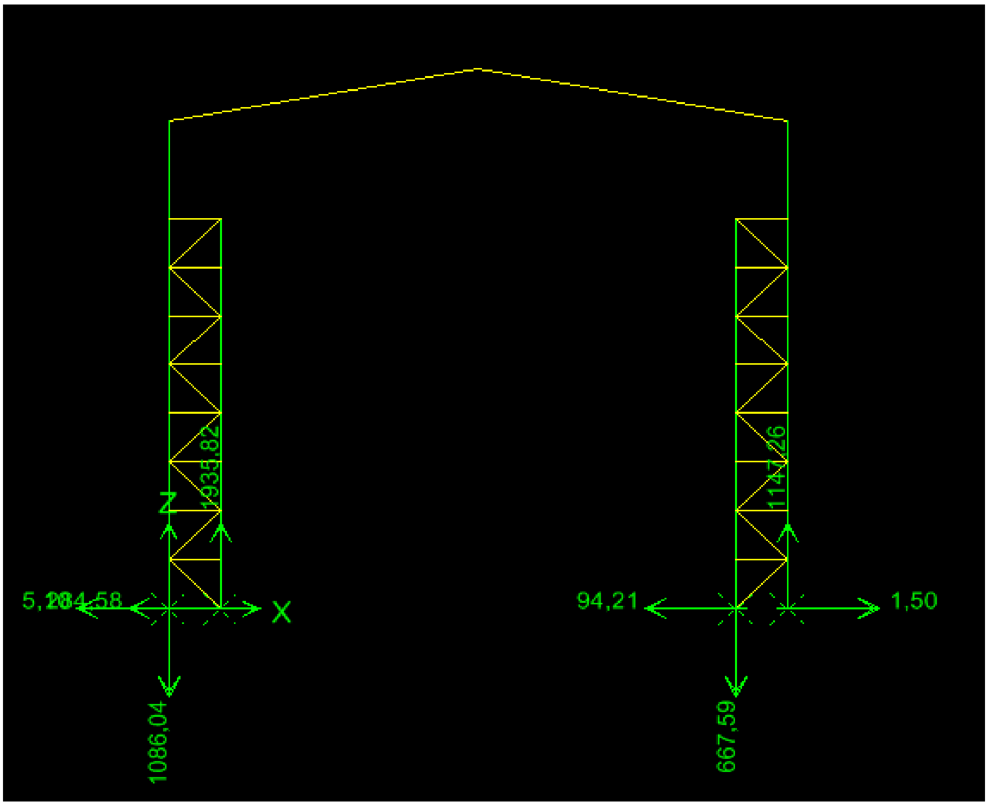
AXIAL FORCE & BIAXIAL MOMENT DESIGN (H1-1a)

	Pu Load	phi*Pnc Strength	phi*Pnt Strength
Axial	86,589	126,681	448,380

	Mu Moment	phi*Mn Capacity	Cm Factor	B1 Factor	B2 Factor	K Factor	L Factor	Cb Factor
Major Bending	0,584	17,265	1,000	1,243	1,000	1,000	0,953	1,136
Minor Bending	0,000	8,502	1,000	2,039	1,000	1,000	0,953	

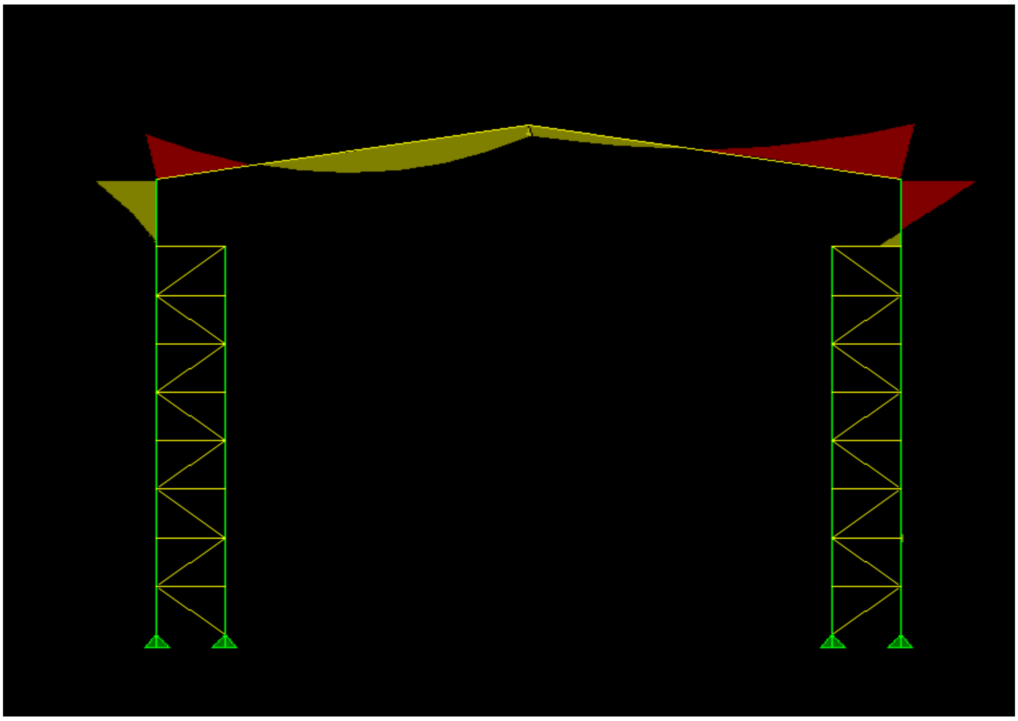
SHEAR DESIGN

	Vu Force	Phi*Vn Strength	Stress Ratio
Major Shear	0,052	60,912	0,001
Minor Shear	0,000	169,158	0,000



SUPPORT REACTIONS FOR LOAD COMBINATION SC201

1,35 G+1,35 WINDX+1,35 GERAN1

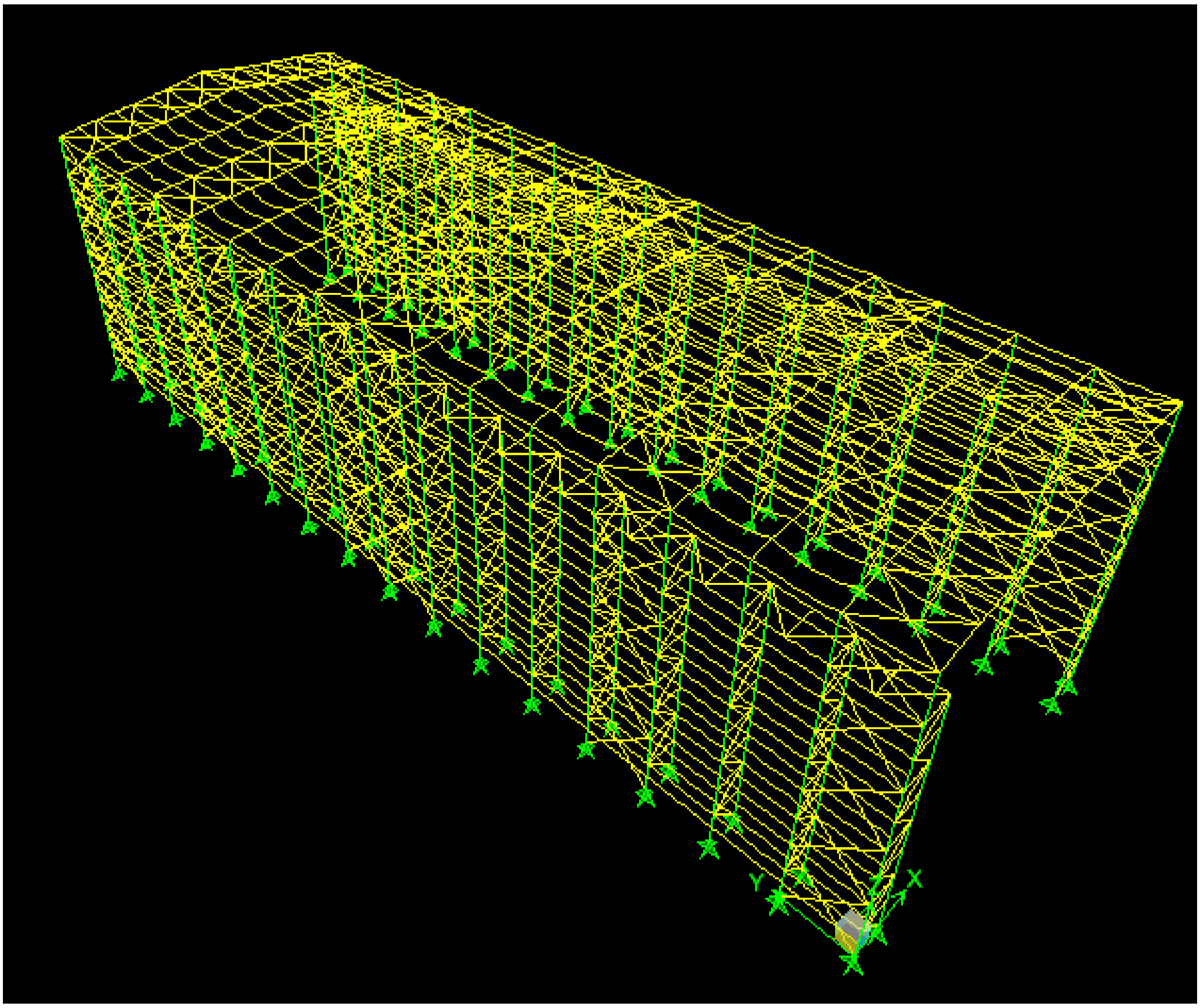


MOMENT DIAGRAM FOR LOAD COMBINATION SC201

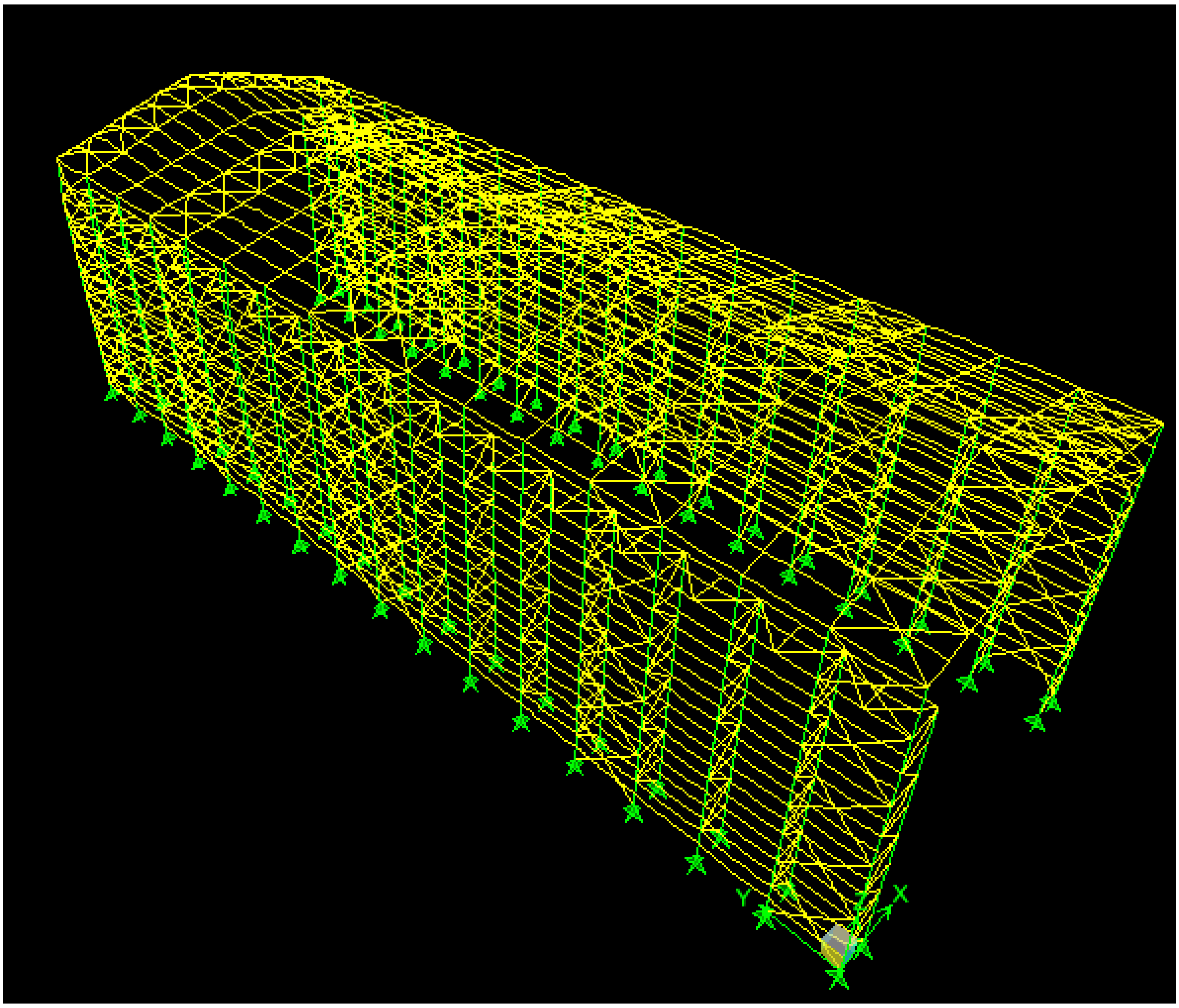
1,35 G+1,35 WINDX+1,35 GERAN1

5.3. DEFORMATIONS OF STRUCTURE

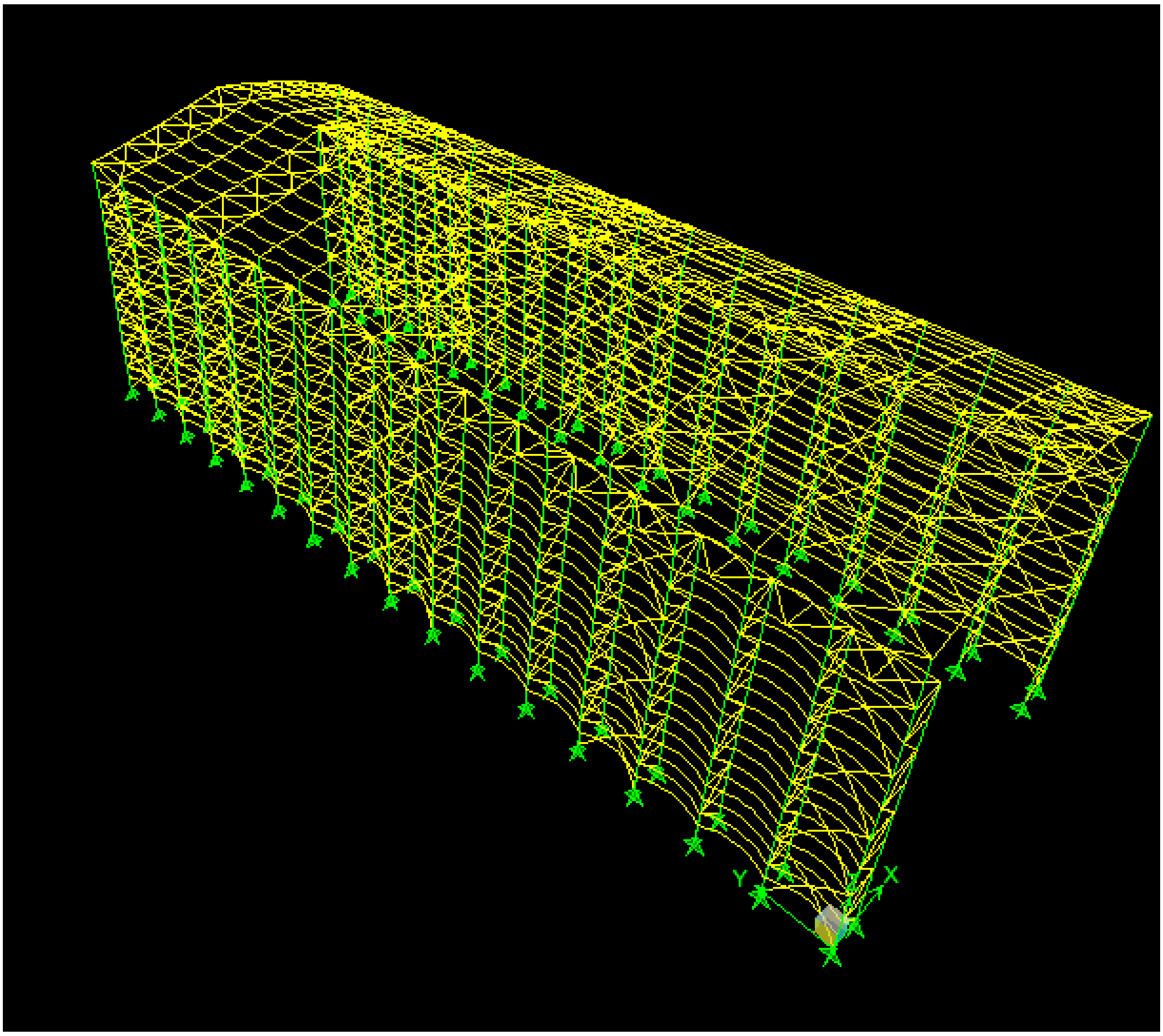
a)Deformation due to vertical loads of covering. The same deformation appears from the loads of snow.



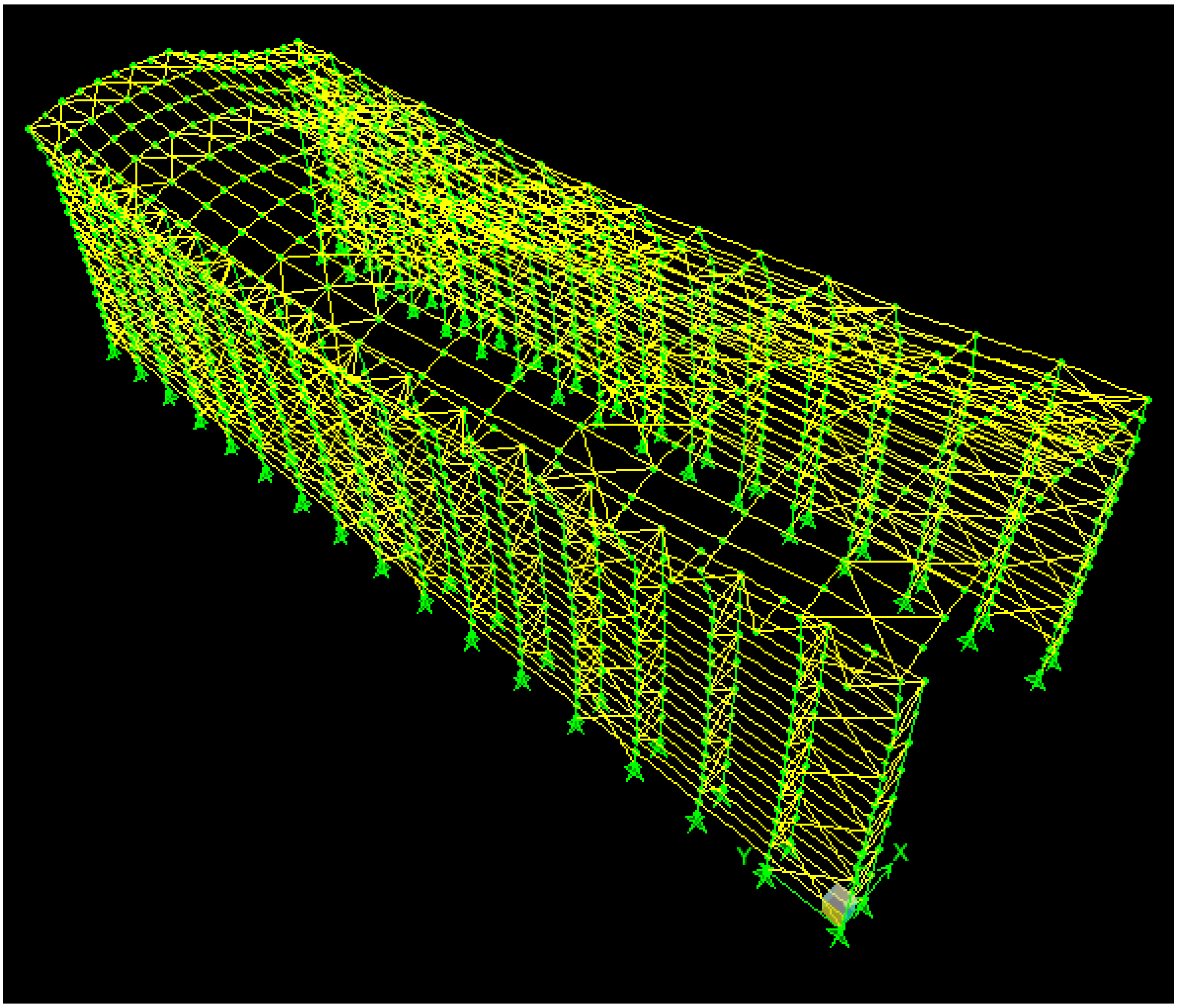
b)Deformation due to the loads of wind in the direction of α=-90°



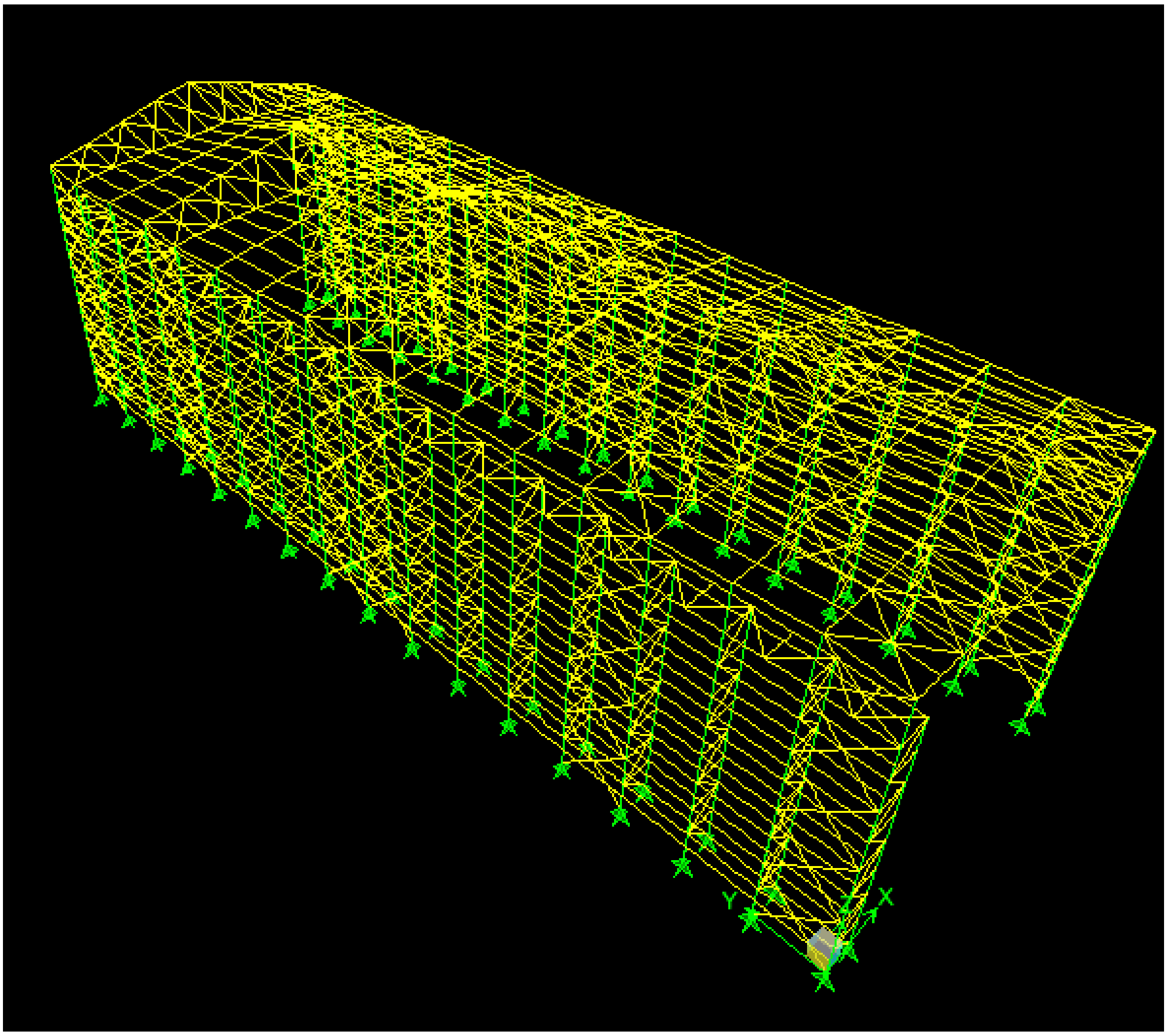
c)Deformation due to wind loads in the direction of α= 0°



d)Deformation due to seismic load for the second mode T=0,5890sec



e)Deformation due to seismic load for the first mode T=0,5905sec



CHAPTER 6: GATES

For the better serviceability of the building gates are placed in the front and in the sides. They contribute to the main services of the building, such as the entrance of big ships, the transportation of machinery and vehicles and the entrance of the employees.

6.1 FRONT GATES

6.1.1 CONSTRUCTION CHARACTERISTICS

The front gate is consisted from four parts 7,5m long, each one runs along it's own rail.

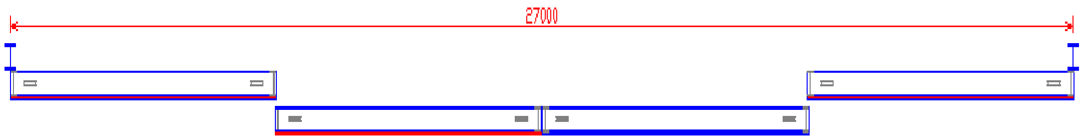


FIGURE 6,1-FOUR PANEL FRON GATE

In the bottom of every panel there is a crab with two wheels and all along the up and down part are placed guidance means. The loads are transferred in the base and finally to a special designed foundation, shown in the figure below, to avoid deferential subsidence due to the weight of the gate.

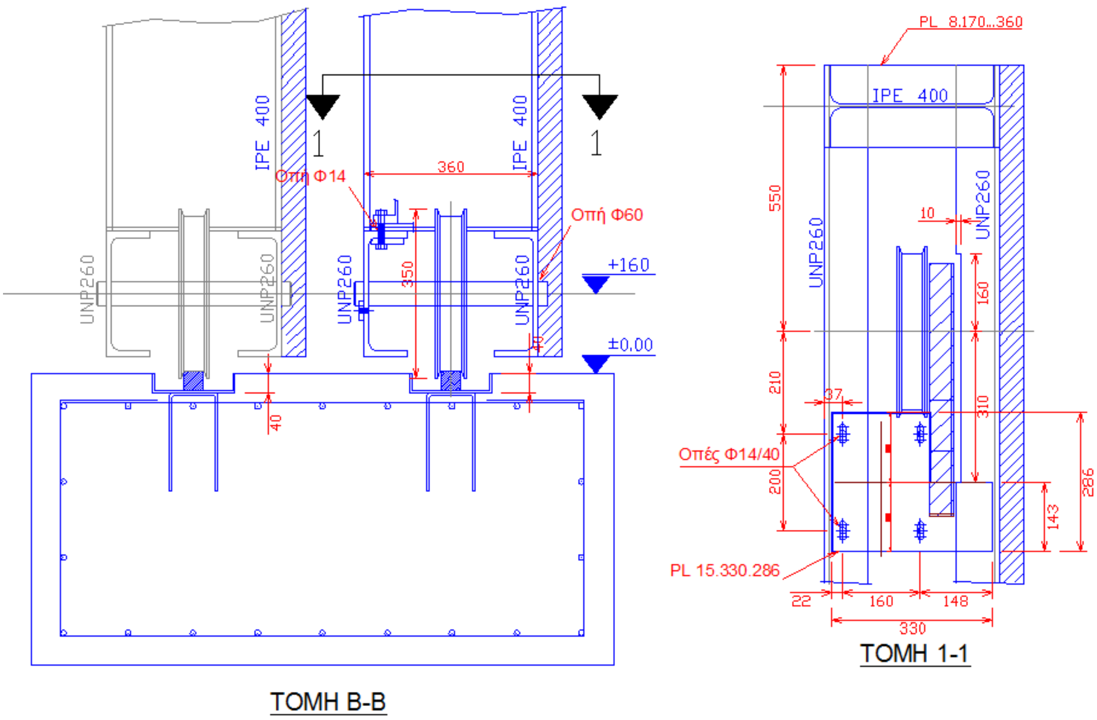


FIGURE 6.2-GUIDANCE MEANS OF GATE

Each part is consisted of three panels, created by four vertical IPE400 sections. These sections are connected in the top with an IPE400 section, while in the horizontal direction they are welded with sections IPE120 in equal spaces of 2,2m. Finally in the two outer panels they are welded diagonal sections L60x6. The figures below show the details:

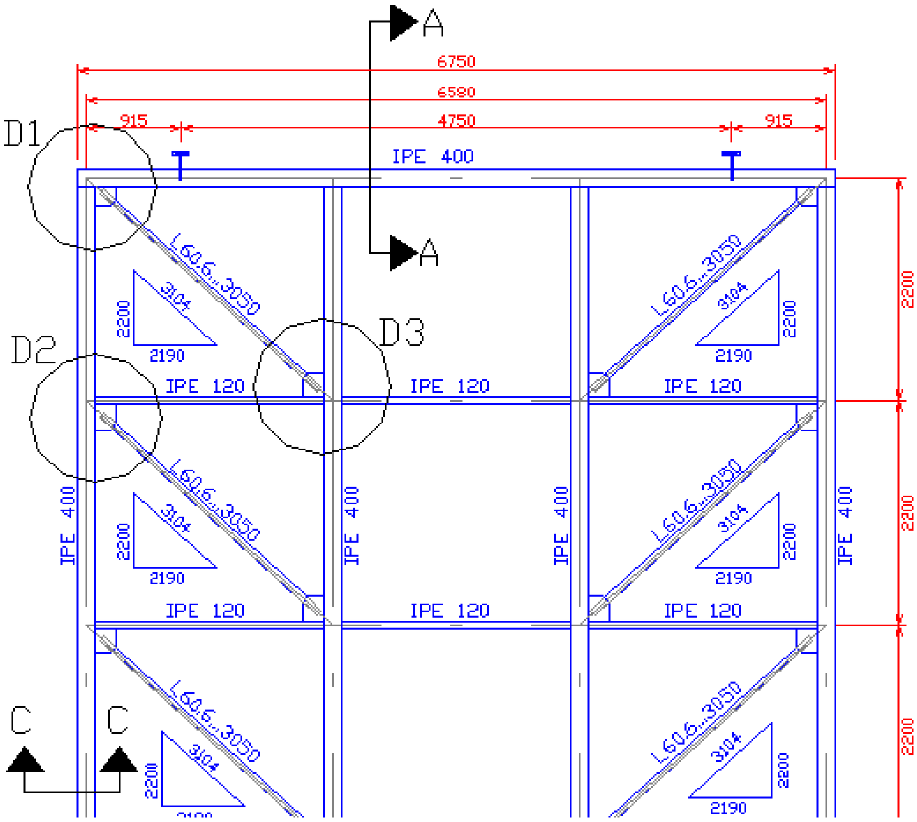


FIGURE 6.3- DETAILS OF UPPER PART

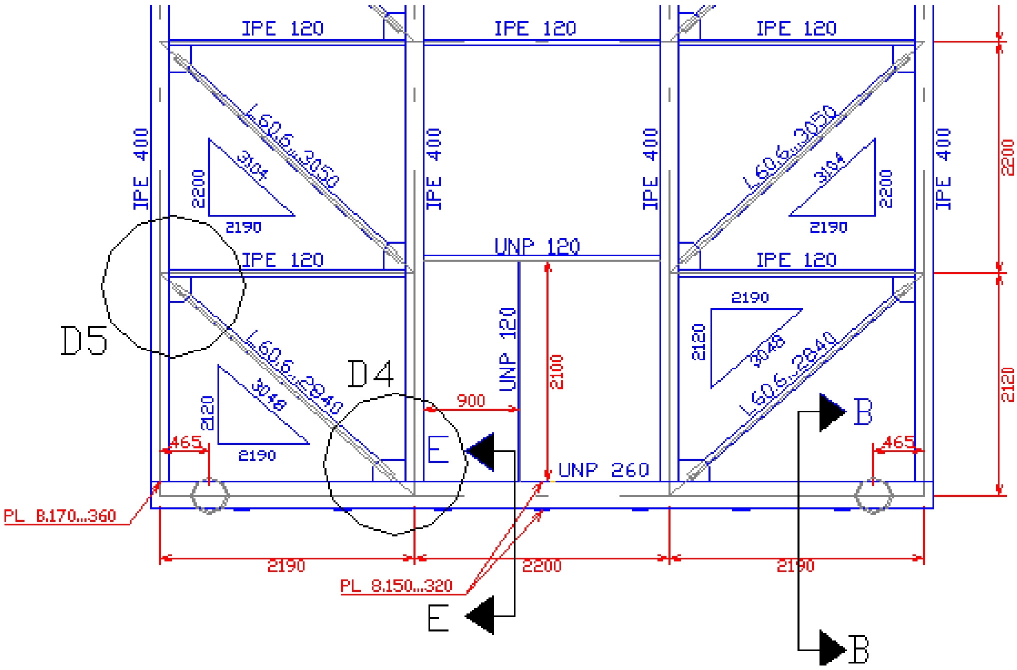


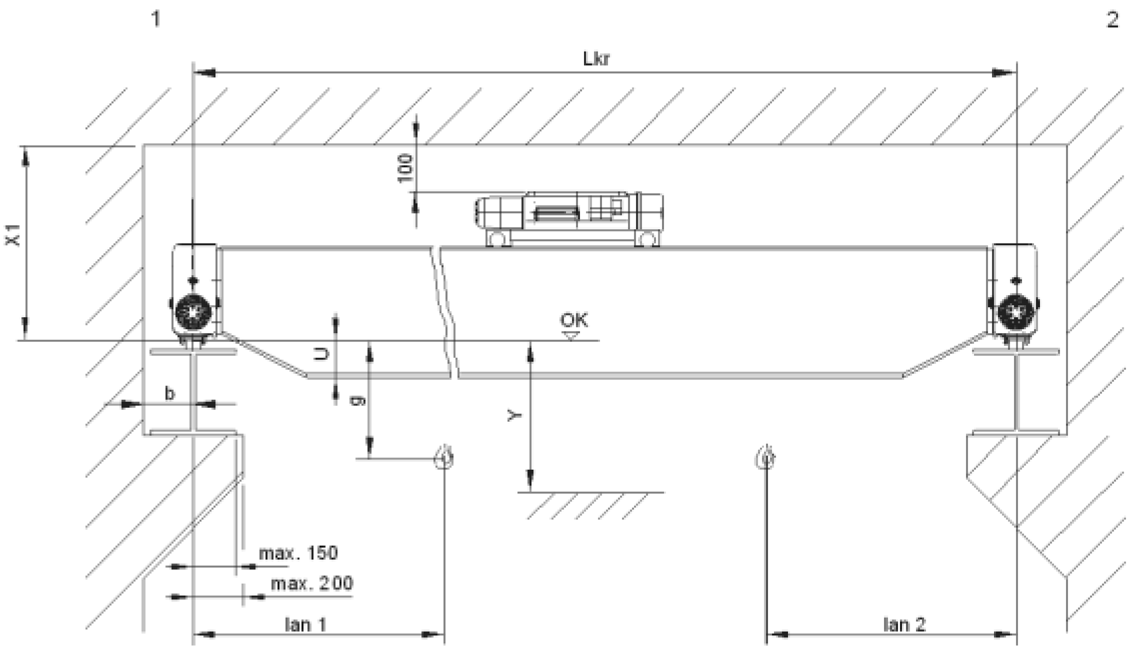
FIGURE 6.4-FIGURE OF THE DOWN PART

ANNEX

CRANEBRIDGE PROPERTIES FROM DEMAG

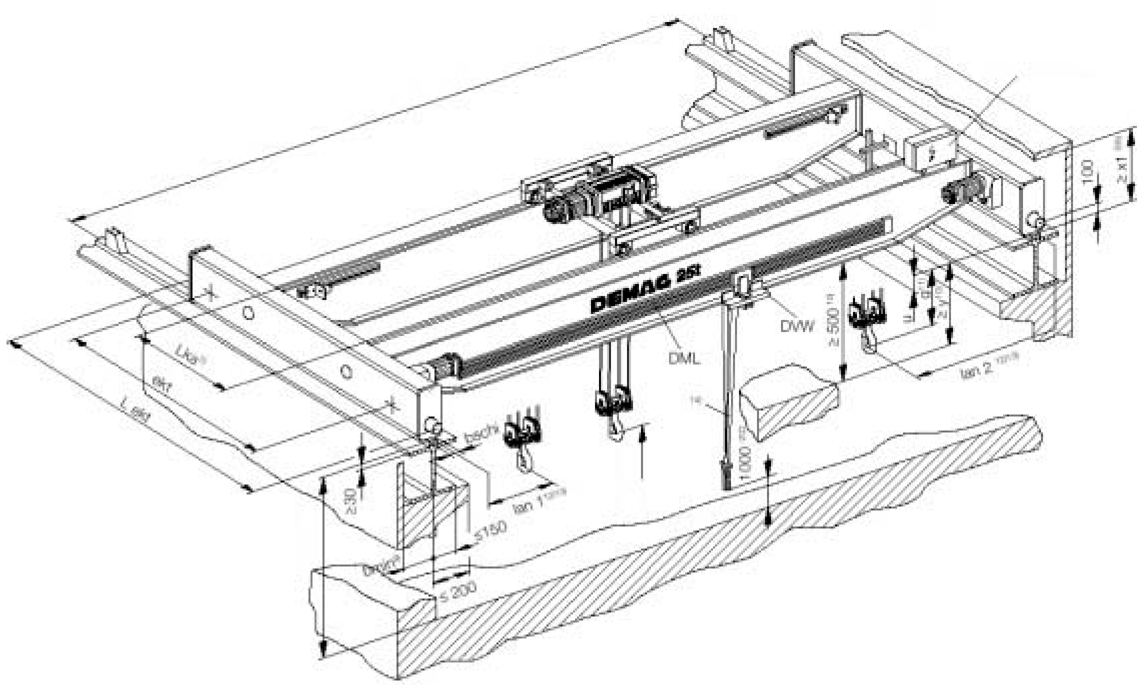
- **DIMENSIONS**

<u>Type ZKKE</u>



SWL:	40 t
Span:	27 m
Model:	NORM1
Crane bridge X1:	1792 mm
Crane bridge X2:	1792 mm
Travel unit Ø d:	400 mm
Trolley eKT:	4000 mm
Trolley LeKT:	4899 mm
Trolley buffer:	160 DPZ
Trolley b:	196 mm
Travelling hoist y:	1385 mm

Travelling hoist g:	203 mm
Travelling hoist lan1:	1243 mm
Travelling hoist lan2:	1301 mm



Requested data

Crane type: ZKKE

SWL : 40t

Span : 27m

Hook path : 9m

Required Hook path : 24m

Operating voltage : 400V

Control voltage :230V

Frequency :50Hz

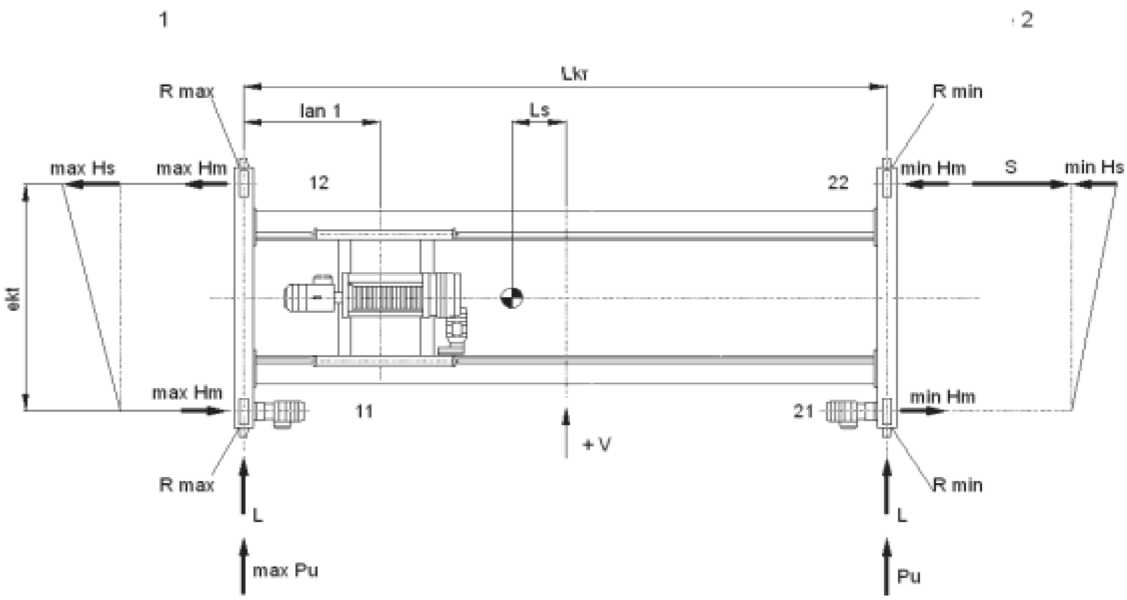
Control : Verfahrbar

Power supply : KBK

Model : NORM1

Crane travel speed :10/40 m/min

- **LOAD / FORCES TO DIN 4132**



SWL:	40 t
SWL:	27 m
Model:	NORM1
maxR:	25264 kg
minR:	5746 kg
L:	7.28 kN
maxHM:	24.50 kN
minHM:	5.81 kN
S:	76.02 kN
maxHs:	61.29 kN

minHs:	14.56 kN
Buffer type:	150 DPZ
maxPu:	71.50 kN
Crane weight:	21840 kg