RELAZIONE TECNICO STRUTTURALE

C.P.A. S.R.L.

STRUCTURAL REPORT MODULAR POOL IN AEGEAN STEEL PANELS - PANEL L. MAX 1.35 M, H.1.50 M

ATTENTION: this document represents a mere development of the calculation methodology. The actual project values will be evaluated for each single case study.

C.P.A. srl
Via Don Demetrio Castelli, 71 - 12060 RODDI (CN)
Tel. 0173.615693 - Fax 0173.620922
www.cpa-piscine.it
cpa@cpa-piscine.it

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SUMMARY

Introduction

The subject of this report is the swimming pools, for private or public use, built through the use and assembly of galvanized steel panels called "Egeo Steel".
The supporting structure of the swimming pool will then be adequately connected on a reinforced concrete base, the verification of which will not be the subject of this study, since, in this report, we will limit ourselves only to the analysis of the steel structure.

1. Description of the project procedures pursuant to point 10.1 of the Ministerial Decree 17/01/2018.

The structural calculation report includes:
a) *the general illustrative description of the work, its use, its function as well as the specific safety regulatory criteria of the type of construction with which the designed structure must be compatible. It contains a description of the work, with the definition of the characteristics of the building such as destination, type and main dimensions.*
b) *the reference regulations;*
c) *the report on materials, as required by Chapter 11 of the NTC18, describes in a discursive way the materials to be used in the construction and the project specifications that provide for their identification and qualification.*
d) *the evaluation of the safety and performance of the structure or of a part of it in relation to the limit states that may occur, in particular in seismic areas, bearing in mind that the level of safety must always be guaranteed for each new or existing work required by the NTC in relation to the nominal life, the class of use, the reference period, the actions including seismic and exceptional actions and their combinations, for each type of structure: reinforced concrete, cap, steel, steel-concrete composite, other materials, with reference to the specific chapters of the NTC;*

a. General illustrative description of EGEO STEEL modular panels

The Egeo Steel panels are made by processing steel sheet, thickness 20/10 mm: the sheet is folded to form a box-like structure with an overall thickness of 100 mm. Subsequently, the panel is welded on the corners and assembled with L profiles 95x50x2 mm with a pitch never greater than 675 mm, one from the other. The development of the single panel can never exceed the maximum length of 1.35 m.
The panels consist of a steel substrate on which a coating consisting of aluminum (55%), zinc (43%) and silicon (1.6%) is applied by continuous hot immersion. Their main feature lies in their excellent resistance to corrosion in acidic environments.
The connections between the different panels are made through bolted joints, with union flanges made in the side shapes of each element; a thrust buttress is installed at each junction, made with press-bent sheet metal profiles, also 20/10 mm thick.
Each panel is assembled with a series of bolts, nuts and washers of the M10 type, also in galvanized steel.
The modularity of the structure allows you to easily create any type of shape, linear or curved, with full customization possibilities.

Reference legislation

The checks were carried out in accordance with the following Technical Standards:

- Ministerial Decree 17/01/18 Infrastructure Min. Interior and Pro. Civil and annexes "Technical standards for construction"
- Circular 7 of 21 January 2019: "Instructions for applying the update of the technical standards for construction Ministerial Decree 17 January 2018 NTC 2018 "
- CNR 10022/84 "Cold formed steel sections. Instructions for use in construction "
- Ministerial Decree 11/03/1988 "Technical standards concerning investigations on soils and rocks, the stability of natural slopes and escarpments, the general criteria and requirements for the design, execution and testing of the support works of the earth and foundation works "
- EN 1993 Eurocode 3: "Design of steel structures. Part 1-1: General rules and rules for structures "
- EN 1993 Eurocode 3: "Design of steel structures. Part 1-3: General Rules - Rules
- additional for the use of cold-bent profiles and thin sheets "
- EN 1998 Eurocode 8: "Design indications for the seismic resistance of structures. Part 1-1: General rules - Seismic actions and general requirements for structures "
- EN 1998 Eurocode 8: "Design indications for the seismic resistance of structures. Part 1-2:
- General rules for structures "
- UNI EN 16582: "Domestic swimming pools: general requirements including safety and test methods"

b. Relation on materials: characteristics and propertie

The steel used is of the S235 Z275 type and has the following characteristics:

- Young's modulus (E) equal to 2100000 daN / cm2

- Poisson's ratio (ν) equal to 0.3

- tangential modulus of elasticity (G) of the value 8.077 * 105 daN / cm2

- coefficient of thermal expansion (α) of the value of 1.0 * 105 daN / cm2.

The mechanical resistance characteristics of the S235 steel in question are as follows:

- ftd (breaking tension in traction) = 3600 daN / cm2

- fyd (yield stress) = 2350 daN / cm2

- sadm (admissible tension) = 1600 daN / cm2

c. For the bolted connections, class 8.8 galvanized bolts made of A2-70 steel were used, compliant with point 11.3.4 of the NTC 2018 currently in force and in particular the fyB and fTB values compliant with point 11.3.4.6 (fyb = 640N / mm2, ftb = 800 N / mm2).

d. Evaluation of the safety and performance of the facility

The safety and performance of a work or part of it must be assessed in relation to the limit states (ULS, SLE, fire protection, durability and safety) that may occur during the nominal design life of the structure. The nominal project life VN of a work is conventionally defined as the number of years in which it is expected that the work, as long as it is subject to the necessary maintenance, maintain specific performance levels. Through this parameter, the class of use, the reference period for the simic action and the coefficient of use, it is possible to evaluate the safety level of any structure. The parameters considered are:

- Class of use I
- Nominal life VN 50 years
- Coefficient of use 0.7
- VR period 35 years

2. Load condition

The structure must be sized to reasonably withstand foreseeable load configurations, such as for example:

- Ground and additional pressures on partially or totally underground and empty pools;
- Hydrostatic pressures on pools filled before backfilling or above ground; i

loads taken into consideration are listed below:

- Permanent Loads:

- G: Own weight of the steel structure (G is zero in the case of completely buried structures)
- P_{suolo}: Ground pressure;

- Variable loads:

- P_{acqua}: Water pressure;
- q_h: Additional pressure exerted on the pool wall;

- Seismic loads

2.1 Actions acting on the structure

The actions acting on the structure and the value of the parameters to be considered are specified by the UNI EN 16582-2 standard in which they are defined:

- The pressure exerted by the ground on the structure varies with depth, indicated by z:

$$P_{suolo} = K * \rho * g * z$$

where:

$\rho * g = 18\ KN/m^3$ represents the unit weight of the soil[1];
$K = 0.3$ coefficient corresponding to a sliding angle of 30 °;
Z = depth or height of the wall.

- The pressure exerted by water on the wall varies with depth, indicated by z:

$$P_{acqua} = \rho * g * z$$

where :
$\rho * g = 10\ KN/m^3$ represents the unit weight of water;

- The additional pressure exerted on the pool wall is induced by a vertical overload qv acting at the top:

$$q_h = K'' * q_v$$

[1] Average value recommended by standard - UNI EN 16582. In the executive phase refer to the actual project values, not to the technical data of this.

where :
$q_v = 250\ \mathrm{Kg/m^2}$
$K' = \tan\left(\frac{\pi}{4} - \frac{\varphi}{2}\right)^2$

Considering φ=30° you get K'=0.333. result:
$$q_h = 0.83\ \mathrm{KN/m^2}$$

Define the actions that act on the structure and consider the partial safety factors γ_G=1.35, γ_Q=1.50, γ_I=1.20, the following load combinations are considered:

Operating limit states (SLS)

- SLS1: Embankment on the wall and
  empty tank load SLS1: G + P_{suolo} + q_h

- SLS2: Pool filled not backfilled load
  SLS2: G + P_{acqua}

At SLS all the safety factors take on a unitary value (γ=1)

Ultimate limit states (SLU)

- SLU1: Embankment on the wall and
  empty tank load
  SLU1:1.35 G+1.35P_{suolo}+1.5q_h

- SLU2: Pool filled not backfilled load
  SLS2: 1.35G+1.20 P_{acqua}

Following the structural analysis, the worst load conditions to which the structure will be subjected will be evaluated and structural checks will be carried out.

With the use of curved, concave or convex paneling, the structure does not undergo any dimensional variation and is always strengthened by ribs. In this last case the stiffness of the structure undergoes a considerable increase due to the shape.

2.2 Seismic analys

The seismic analysis is carried out taking into consideration the reference Seismic Zone, in which the structure falls and applying the prescriptions dictated by the D.M. 17-01-2018.

Zona sismica	Descrizione	accelerazione con probabilità di superamento del 10% in 50 anni [a_g]	accelerazione orizzontale massima convenzionale (Norme Tecniche) [a_g]	numero comuni con territori ricadenti nella zona (*)
1	Indica la zona più pericolosa, dove possono verificarsi fortissimi terremoti.	a_g > 0,25 g	0,35 g	703
2	Zona dove possono verificarsi forti terremoti.	0,15 < a_g ≤ 0,25 g	0,25 g	2.229
3	Zona che può essere soggetta a forti terremoti ma rari.	0,05 < a_g ≤ 0,15 g	0,15 g	2.807
4	E' la zona meno pericolosa, dove i terremoti sono rari ed è facoltà delle Regioni prescrivere l'obbligo della progettazione antisismica.	a_g ≤ 0,05 g	0,05 g	2.224

Figure 1: Classification of seismic areas

Placing ourselves in favor of safety, the stresses acting were calculated in the most critical situation by taking into consideration the highest horizontal seismic acceleration value ag for each reference seismic zone (ex: Zone 1・ ag = 0.35g).
The seismic action is represented by a set of horizontal and vertical static forces given by the product of the forces of gravity by the seismic coefficients Kh and Kv thus calculated:

$$K_h = \beta_m \frac{a_{max}}{g}$$

$$K_v = \pm 0{,}5 K_h$$

where:

β_m is a reduction coefficient that depends on the type of soil (type B--> βm = 0.31);

$a_{max} = S_S S_T a_g$

S_S, S_T are respectively the stratigraphic and topographical amplification coefficient (S_S=1,4 S_T=1);.

By doing so it will be possible to derive the seismic forces, deriving them from the static ones:

$$F_{sismiche} = F_{statiche} * K$$

3. Calculation of stress

The calculation of seismic stresses and the verification of structural elements is performed using the Pseudo-Static analysis proposed by the NTC 2018.

The reference model for the analysis consists of the support work and a wedge of soil, upstream, which is supposed to be in an active limit equilibrium state. Furthermore, since the weight of the structure is totally negligible, compared to the forces involved, the seismic coefficients previously introduced only to the wedge of soil acting on the work will be applied.

Considering the mechanical characteristics of the filling soil[2] as reasonable values according to the UNI EN 16582-2 standard, it is possible to assume:

- Specific soil weight: $\gamma = 18$ KN/m^3
- Effective ground friction angle: $\phi_d = 30{,}0$ °
- Structure-ground friction angle: $\delta_d = 15{,}0$ °
- Structure inclination: $\psi = 90{,}0$ °
- Profile inclination of the upstream embankment: $\beta = 0{,}0$ °
- Valley profile inclination: $\beta_{valle} = 0{,}0$ °
- Inclination of the resultant of the volume forces: $\theta = 9°$

where:

$$\theta = \tan^{-1}\left(\frac{K_h}{1 \pm K_v}\right)$$

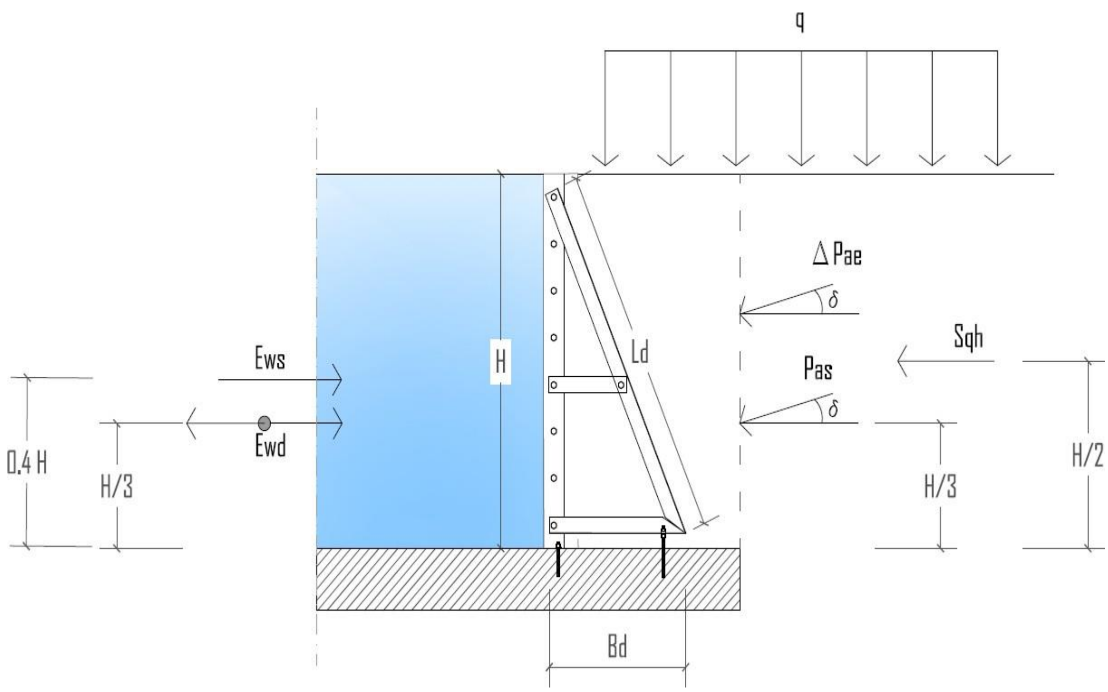


Figure 2: Forces acting on the structure

[2] Value recommended by standard - UNI EN 16582. In the executive phase refer to the actual project values, not to the technical data of this.

3.1 Static and dynamic active ground thrust

The sub-horizontal PAE thrust (static and dynamic) exerted by the ground and acting on the work of support with inclination , is given by:

$$P_{AE} = \frac{1}{2}\gamma * (1 \pm K_v) * K_{AE} * H^2$$

In order to determine the thrust exerted by the upstream embankment it is necessary to calculate the dynamic coefficient KAE, valid for the active thrust states.
In the case of $\beta < (\phi-\theta)$, the following formula is adopted (**Mononobe e Okabe,1929**):

$$K_{AE} = \frac{\sin(\phi + \psi - \mathrm{P})^2}{\cos \mathrm{P} \sin \psi^2 \sin(\psi - \mathrm{P} - \delta)\left[1 + \sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta-\mathrm{P})}{\sin(\psi-\mathrm{P}-\delta)\ \sin(\psi+\beta)}}\right]^2}$$

The fundamental assumption of the pseudo-static method consists in breaking down the thrust exerted by the ground, PAE, into two contributions:

- a static given by the PAS thrust applied to one third of the height of the structure;
- a dynamic one due to the seismic action ΔPAE acting in the center line of the

structure

- Once the global thrust PAE has been calculated, it is possible to obtain ΔPAE with

the following relationship:

$$\Delta P_{AE} = P_{AE} - P_{AS}$$

3.2 Hydrodynamic thrust

In the presence of free water on the face of the panel, the dynamic contribution due to the earthquake that the water can exert must be taken into account. Reference was made to Westergaard's theory (1933) in which the dynamic increase $\mathbf{E_{wd}}$, resulting from a curvilinear load distribution q (z), it acts with an arm equal to 0.4 * H, in which H represents the height of the structure.

$$q(z) = \pm\frac{7}{8}K_h\gamma_w\sqrt{H * z}$$

$$E_{wd} = \pm\frac{7}{12}K_h\gamma_w H^2$$

The Ewd dynamic thrust must be considered in both directions, thus adding to and subtracting from the Ews buoyancy.

The structural checks of all the elements are carried out below, assuming to carry it out in seismic zone 1. In this way we want to demonstrate how the prefabricated steel panel structure is able to withstand the maximum seismic thrust on Italian territory.

4. Structural verifications

4.1 Verification of stresses and deformations on the panel

The panel is studied as a thin, flexible membrane under transverse load. Its behavior is not comparable to that of a beam, but to a two-dimensional catenary in two directions perpendicular to each other. The result is a behavior that allows a better distribution of the stresses in operation. The thin wall will therefore have very different stiffness values: in a direction perpendicular to the mean plane it will oscillate around discrete values, depending on the length of the external wall and the influence of the joint at the ends. Parallel to the medium level, it is instead characterized by important values that allow excellent management of horizontal actions

Assuming constant the pressure q on the membrane, the following formulas apply[3]:

$$\sigma = Z_1 \sqrt[3]{\left(\frac{ql}{s}\right)^2 \frac{E}{(1-\nu^2)}} \qquad f = Z_3 l \sqrt[3]{(1-\nu^2)\frac{ql}{Es}}$$

where :

- f= freccia massima;
- σ=tensione massima;
- $Z1$, $Z3$ = coefficienti per membrana piatta con contorno appoggiato;
- L=1,50 m altezza pannello;
- l=1.35 m larghezza pannello;
- q= 14050 N/m2 pressione lineare costante;
- s=0.002 m spessore pannello;
- E=210000000000 N/m2 Modulo di Young;
- ν= 0.3 modulo di Poisson;

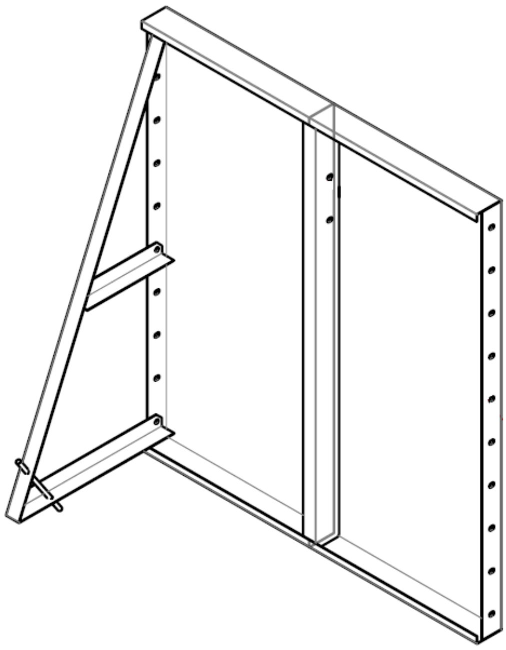


Figure 3: 3D view of Egeo Steel panel - Variable length

[3] Ref. Metallic constructions by Vittorio Zignoli, 1957, UTET (Extremely flexible sheets - membranes)

For the exemplified calculation of the stress on the panel and its deformation, the most unfavorable condition is assumed, i.e. the one in which the pool is empty with the ground pushing, behind the structure. Part of the deformation calculated here will be recovered with the counterthrust of the water. Using the formulas indicated above and considering, for safety purposes, a uniform pressure on the panel equal to the thrust of the ground at a depth of 1.50m, we obtain that:

Z_1=(l/L=0.9) =0.161

Z_3=(l/L=0.9) =0.304

Where Z1 and Z3 are two coefficients for a rectangular membrane fixed along the perimeter. Their value varies in relation to the ratio between the smaller side and the larger side of the panel.

The maximum stress on the membrane will therefore be:

$$\sigma = Z_1 \sqrt[3]{\left(\frac{ql}{s}\right)^2 \frac{E}{(1-\nu^2)}} = 0.161 \cdot \sqrt[3]{\left(\frac{14050 \cdot 1.35}{0.002}\right)^2 \frac{210000000000}{(1-0.3^2)}} = 44.26 N/mm^2$$

44,26 N/mm^2 ≪235 N/mm^2

As for the deformations, in the critical case of an empty pool and thrust of the ground:

$$f = Z_3\, l \sqrt[3]{(1-\nu^2)\frac{ql}{Es}} = 0.304 \cdot 1.35 \sqrt[3]{(1-0.3^2)\frac{14050 \cdot 1.35}{210000000000 \cdot 0.002}} = 0.014m = 14mm$$

The maximum horizontal translation is therefore equal to 14 mm.
This shift is of modest magnitude. It is important to underline that, in operating conditions, a large part of the deformation will be recovered thanks to the counterthrust of the water.
Despite this, in order not to compromise the use of the structure and not cause damage of an aesthetic nature, or as regards the waterproofing, there is a stiffener on the panel that provides a significant contribution to limiting the deformation.

4.2 Verification of the stresses on the stiffeners

Il pannello di lunghezza 1,35 m, oltre ad essere vincolato in corrispondenza del contrafforte, risulta essere nervato, ovvero viene inserito un angolare (elemento in acciaio con sezione ad L di dimensione 95x50x2) che ha la funzione di irrigidimento, in maniera da contrastare gli effetti che potrebbero provocare l'ingobbimento della lastra fuori dal piano. Questa nervatura, resa solidale al pannello tramite saldatura, è posizionata a metà della lunghezza totale del pannello, con un interasse circa pari a 675 mm.
Per verificare la nervatura, si fa l'ipotesi che il contrafforte sia ininfluente ai fini dell'equilibrio e che la totalità del carico sia assorbita dall' irrigidimento.
Inoltre, essendo quest' ultimo saldato al pannello, il loro comportamento strutturale è assimilabile a quello di una trave incernierata alle due estremità, soggetta ad un carico distribuito.

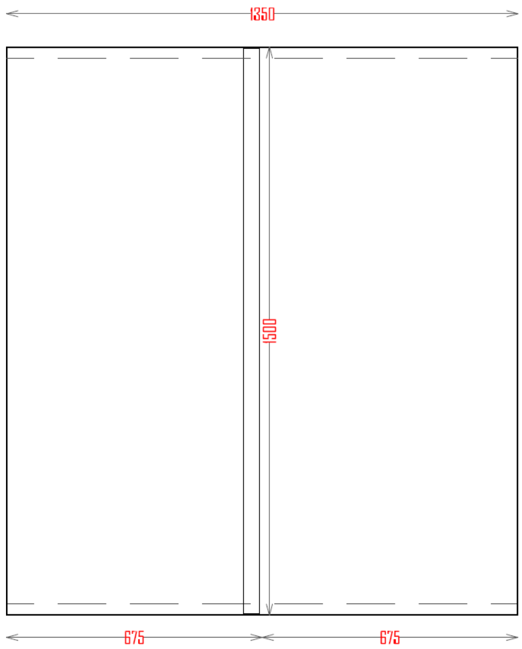


Figure 5: Elevation of the panel l.1.35m

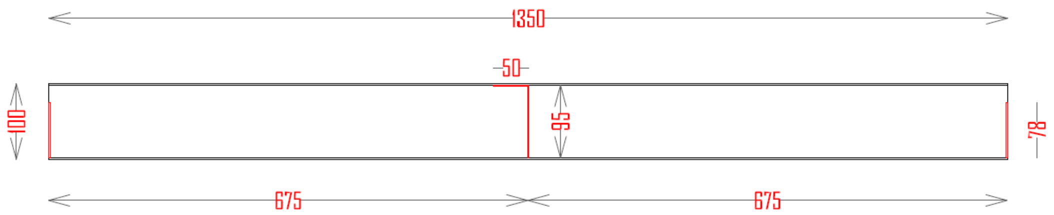


Figure 6: Plan view of the 1.35m panel

Considering the resistant section it is possible to calculate the geometric characteristics. It is obtained:

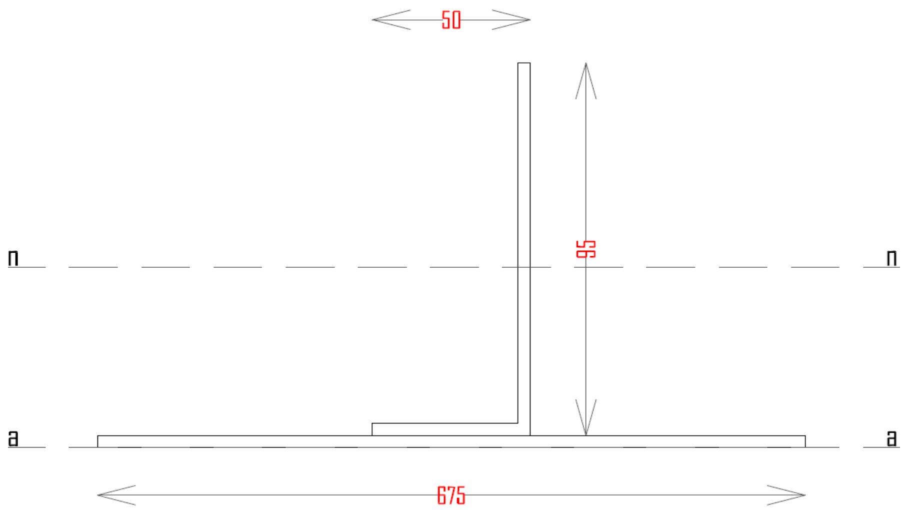


Figure 7: Detail of the stiffening rib section

- *Area*=1640 mm^2;

- *Static moment: 11435 mm3*
- *Center of gravity moment of inertia: 553376 mm4*
- *Neutral axis position: 7 mm*

Prendendo come riferimento una lunghezza di influenza pari ad 675 cm, le caratteristiche della sollecitazione sono di seguito riportate:

$$V_{SD} = 10{,}33\ KN/m$$

$$M_{SD} = 5.1\ KNm/m$$

In condizione ultima, al raggiungimento della completa plasticizzazione della sezione, il momento resistente risulta:

$$M_{RD} = W_{pl} * \frac{f_{yd}}{\gamma_{M0}} = 2 * 11435 * \frac{235}{1.05} = 5{,}50\ KNm$$

La sezione risulta verificata ($M_{RD} > M_{SD}$).

4.3 Verifica degli sforzi sul contrafforte

Il contrafforte è realizzato con un profilato ad L52x52x2 presso-piegato a freddo e presenta una sezione resistente pari a A=208 mm^2.
Considerando la condizione di carico più gravosa, questo elemento risulta soggetto unicamente ad uno sforzo di trazione, in quanto tutto il carico di compressione viene scaricato in fondazione direttamente al piede del pannello.
Ne consegue che l'elemento non presenterà fenomeni di instabilità e quindi la verifica potrà basarsi solamente sulla resistenza della sezione trasversale e dei giunti strutturali.
Per le membrature soggette a trazione assiale deve essere verificata tale condizione (UNI ENV 1993-1-1: 2004):

$$N_{sd} \leq N_{pl,Rd}$$

In cui il valore di progetto della forza di trazione N_{sd} deve risultare inferiore alla resistenza di progetto della sezione trasversale $N_{t,Rd}$.

- Resistenza plastica di progetto della sezione lorda:

$$N_{pl,Rd} = \frac{(208 \cdot 235)}{1.1} = 44.45\ KN$$

- Resistenza ultima di progetto della sezione netta in corrispondenza dei fori

Per quanto riguarda angolari collegati su una sola ala attraverso un solo bullone (sezione ad L ancorata su un solo lato) si fa riferimento al punto 6.5.2.3 dell'Eurocodice 3 sopracitato per la determinazione di:

$$N_{t,Rd} = \frac{2 \cdot (e_2 - 0.5 d_0) \cdot t \cdot f_u}{\gamma_{M2}} = \frac{2 \cdot (26 - 0.5 \cdot 10) \cdot 2 \cdot 235}{1.25} = 15.79\ KN$$

Si ottiene un valore di resistenza, considerando il valore inferiore tra i due sopra riportati, pari a 15,79 KN.
La risultante della spinta complessiva agente sulla parete del singolo pannello di lunghezza 1,00 m, in condizione di stato limite ultimo risulta essere pari a N_{Sd}=14,05 KN.

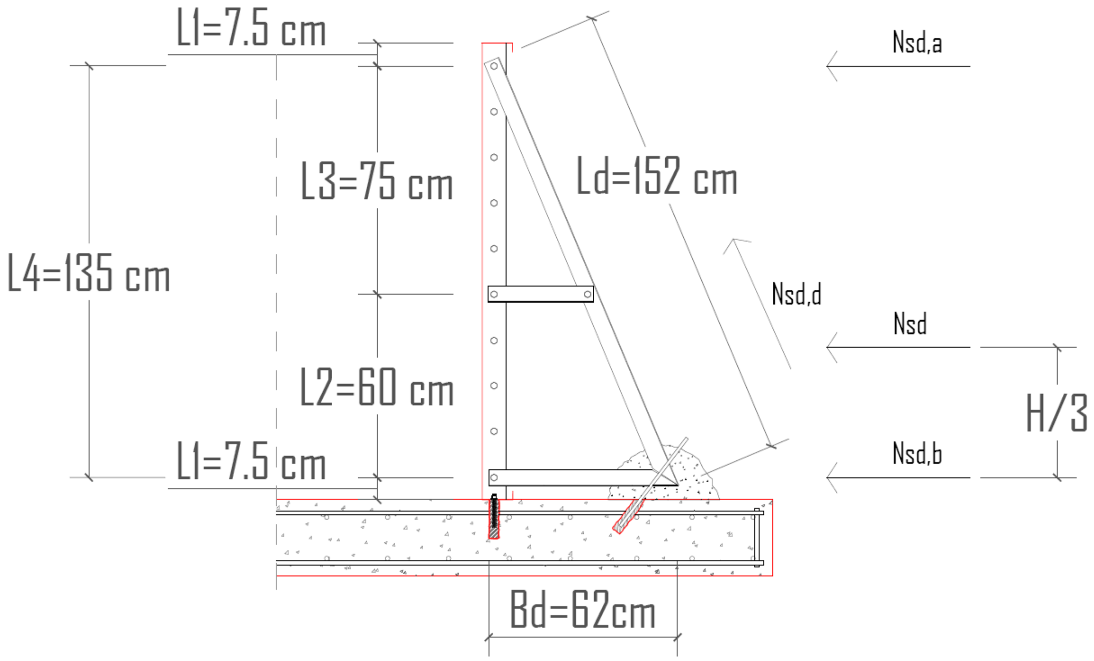


Figure 4: Distribution of forces on the buttress

By breaking down this force along the resistant elements of the structure we have:

$$N_{sd,a} = N_{sd} \cdot \frac{L_2}{L_4} = 14.05 \cdot \frac{60}{135} = 6.24\ KN$$

$$N_{sd,d} = N_{sd,a} \cdot \frac{L_d}{B_d} = 6.24 \cdot \frac{152}{62} = 15.29\ KN$$

$$N_{sd,b} = N_{sd} - N_{sa} = 14.05 - 6.24 = 7.81\ KN$$

You will therefore have that:

$$N_{sd,d} = 15.29\ KN \leq N_{t,Rd} = 15.79\ KN$$

The buttress will therefore be stressed by a traction effort spread over an effective area of 1.50 cm2.

The tension along the inclined element of the buttress will be equal to:

$$\sigma_{sd,d} = \frac{N_{sd,d}}{A_{eff}} = \frac{15.29}{1.50} = 10.13 \frac{KN}{cm^2} \ll 16 \frac{KN}{cm^2}$$

and along the horizontal element:

$$\sigma_{sd,b} = \frac{N_{sd,b}}{A_{eff}} = \frac{7.81}{1.50} = 5.20 \frac{KN}{cm^2} \ll 16 \frac{KN}{cm^2}$$

Le tensioni risultano inferiori al valore della tensione ammissibile dell'acciaio S235 pari a 16 KN/cm^2.

4.4 Verification of the stresses on the bolts

The panel opposes the shear stresses by means of the log bolts or the fixing bars, which bind the entire structure with the foundation slab, and the bolts that connect the individual panels to each other. Placing ourselves in the most unfavorable condition, in which the first type of constraint is unable to make any contribution to the absorption of the cutting effort, the load would be entirely distributed on the side bolts.
The connection between the vertical flange of the panel and the buttress is made by means of 3 bolts. M12x25 bolts with the following characteristics are used:

- $class\ 8.8;$
- $f_{yb} = 640\ N/mm^2$
- $f_{tb} = 800\ N/mm^2$
- $d = 12\ mm;$
- $Ares = 84\ mm^2$

Considering a 1.35m light panel, the resultant of the load will be equal to $Fv,\ sd$ = 18.05 KN. Each bolt will therefore be subjected to a shear and tensile force equal to $Fv,\ sd$ = 6.02 KN.
If bolts are subjected to combinations of shear and tension they must also satisfy the following equation:

$$F_{t,Rd} = \frac{F_{v,sd}}{F_{v,Rd}} + \frac{F_{t,sd}}{1.4 \cdot F_{t,Rd}} \leq 1$$

where $F_{v,Rd}$ represents the design shear resistance and is calculated:

$$F_{v,Rd} = n \cdot \frac{0.6 \cdot f_{tb} \cdot A_{res}}{\gamma_{M2}} = \frac{0.6 \cdot 800 \cdot 84}{1.25} = 32.26\ KN$$

e $F_{t,}\ Rd$ represents the design tensile strength and is calculated:

$$F_{t,Rd} = \frac{0.9 \cdot f_{tb} \cdot A_{res}}{\gamma_{M2}} = \frac{0.9 \cdot 800 \cdot 84}{1.25} = 48.38\ KN$$

The partial safety factor related to bolted connections is equal to: $\gamma M2$ = 1.25,
n = 1 represents the number of connected faces.

Follow:

$$F_{t,Rd} = 0.222 + 0.106 = 0.328 \leq 1$$

verified.

4.5 Bearing check of the vertical flange / buttress joint

The junction of the flange with the buttress occurs. This connection is made with an M12 bolt. It is loaded by an axial action equal to 13.93 KN. The bearing stress on the square-buttress junction must be calculated on the critical section of the buttress of 2mm and the calculation is carried out according to the requirements of Eurocode 3, table 6.5.3 of UNI ENV 1993 1-8.

La seguente equazione riporta la resistenza a rifollamento:

$$F_{b,Rd} = k \cdot \alpha \cdot \frac{f_{tb} \cdot d \cdot t}{\gamma_{M2}}$$

dove :

- d = diametro del bullone: 12 mm;
- d_0= diametro del foro: 14 mm;
- t= spessore lamiera: 2 mm;
- γ_{M2}=coefficiente parziale di sicurezza collegamenti bullonati: 1.25;
- e_1= distanza minima dall'estremità;
- e_2= distanza minima dal bordo;
- $k = min\{\frac{2.8e_2}{d_0} - 1.7, 2.5\} = 2.5$;
- $\alpha = min\{\frac{e_1}{3d_0}, \frac{f_{tb}}{f_{tk}}, 1\} = 1$;

La resistenza a rifollamento vale:

$$F_{b,Rd} = 17.30\ KN$$

L'Eurocodice 3 raccomanda, qualora si utilizzino bulloni M12 in fori con 2 mm di gioco, una riduzione della resistenza al taglio pari al 0.85 volte il valore sopra indicato. Si avrà:

$$F_{b,Rd,rid} = 0.85 \cdot 17.30 = 14.70\ KN$$

La capacità di resistenza ultima a rifollamento del contrafforte da 2 mm a stato limite ultimo risulta superiore all'azione sollecitante. La giunzione risulta verificata.

4.6 Verifica delle reazioni vincolari

Per ogni contrafforte sono predisposti due tirafondi per collegare in maniera adeguata la struttura in pannelli d'acciaio con la platea di fondazione sottostante. In maniera cautelativa, ipotizzando che la totalità del carico sia contrastato interamente dai pioli, sarà opportuno effettuare la verifica di resistenza al taglio (EN 1998 6.5.3) degli elementi di connessione.

$$F_{v,Rd} = \frac{0.6 \cdot f_{tb} \cdot A_{res}}{\gamma_{M2}}$$

Considerando due tirafondi M12, con le medesime caratteristiche meccaniche dei bulloni sopraelencate, si ottiene:

$$F_{v,Rd} = \frac{0.6 \cdot 800 \cdot 84}{1.25} = 3226\ N = 32.26KN$$

E risulta:

$$F_{v,Sd} = 18.05KN < F_{v,Rd} = 32.26KN$$

La tensione tangenziale r del singolo piolo è pari a 10,74 KN, inferiore al valore ammissibile a SLE.

5. Construction details

In the executive phase, there are different ways of installing the Egeo Steel galvanized steel panel. The connection with the underlying reinforced concrete slab can be made as follows:

1) Structure - floor connection through the use of M12x25 log bolts;

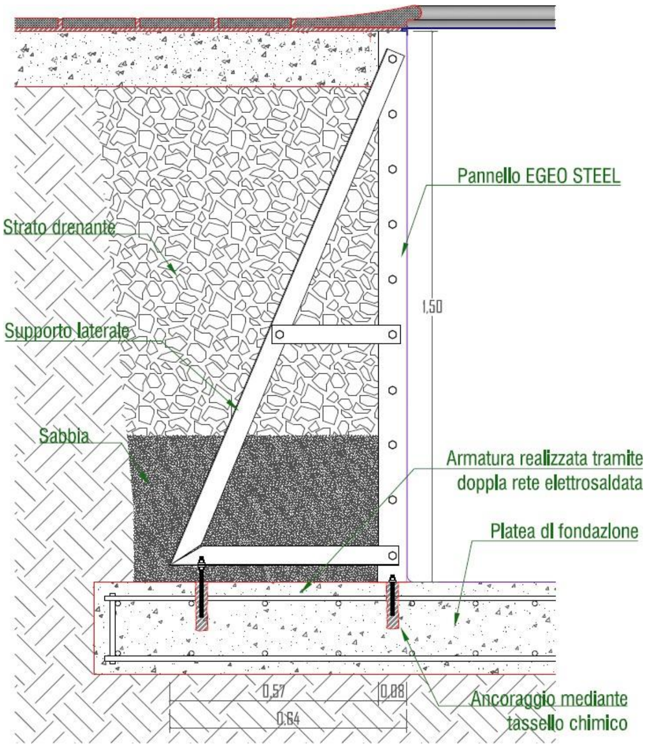
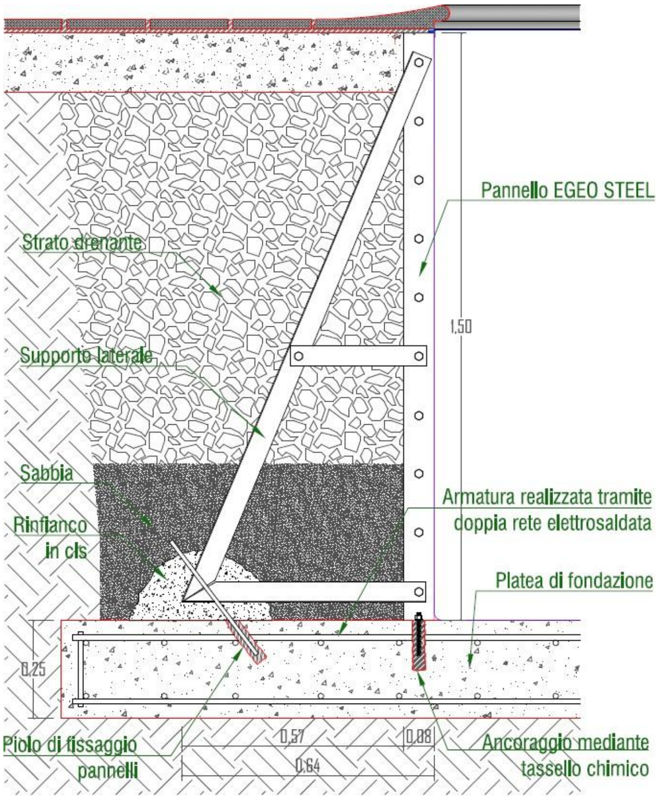


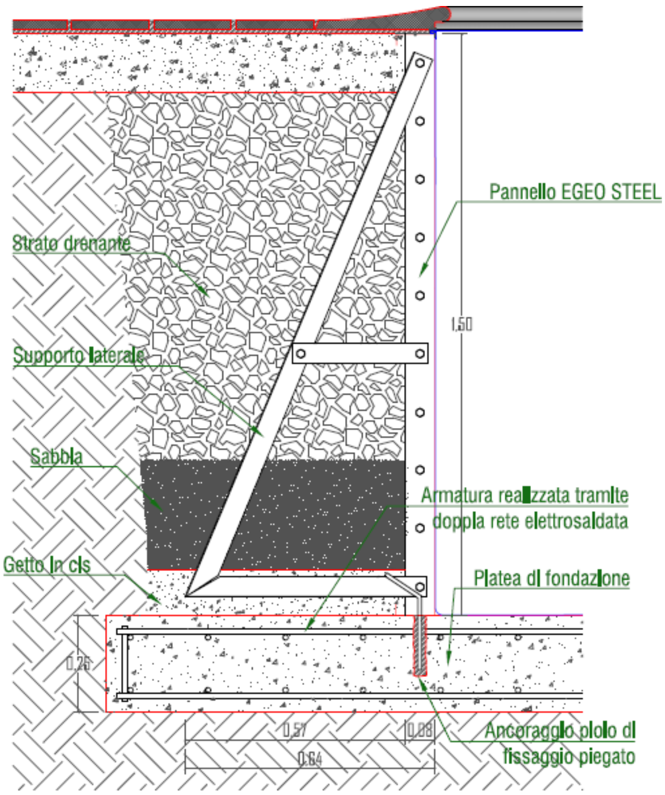
Figura 8: Sezione pannello con tirafondi

2) Structure - floor connection by means of prefixing bars made with reinforcing rods φ12 mm B450C and concrete backing;

3) Connection structure - base by means of a bent peg made with reinforcing rod φ12 mm B450C and concrete casting.



Panel section with reinforcing bars and concrete support



Panel section with bent pin and concrete casting

Note:

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