*Extended Abstract*

# STRUCTURAL ANALYSIS OF THE F-16 AIRCRAFT'S WEATHER SHELTER

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## 1 – INTRODUCTION

This thesis presents a study of the structural behavior of the Portuguese Air Force Weather Shelters, used by the F-16 aircraft. These shelters are reinforced concrete shells internally covered by profiled steel sheeting.

These structures were designed based on the portuguese national regulation, which is currently being replaced by the European codes, called Eurocodes. In addition, the design project didn't considerer the increase in structural strength offered by the profiled steel sheeting or accidental loads like an explosion near the structure.

The aim of this thesis is: 1) structural analysis of the shelter assuming reinforced concrete shell behavior and loads defined by the Eurocodes; 2) structural analysis of the shelter assuming a composite steel and concrete behavior and an accidental load corresponding to a nearby explosion.

The internal structure of this work is divided in the following subjects: 2) Aircraft shelter's evolution; 3) Cylindrical shell´s behavior and analytical formulation; 4) Load definition and design criteria relating the Weather Shelter; 5) Numerical analysis and safety evaluation of the Weather Shelter; 6) Strength evaluation of the composite steel and concrete shelter subjected to an accidental load; 7) Conclusions and future developments.

## 2 – AIRCRAFT SHELTER'S EVOLUTION

Aircraft shelters appeared in the 1970's with the primary objective being the protection of parked aircrafts. The main reason for the rapid expansion of these structures was the ongoing Cold War which was responsible for a permanent state of military tension during this period.

The first aircraft shelter was developed by the USAF (United States Air Force) and was called TAB-V (Theatre Air Base Vulnerability). It was a reinforced concrete semi-circular cylindrical shell with dimensions 30,7x14,6x7,2 m, internally covered by sinusoidal shape steel sheeting. One of the main features of the TAB-V shelter was the bow-shaped steel doors.

This shelter provided the basis to NATO (North Atlantic Treaty Organization) shelters, called HAS (Hardened Aircraft Shelter). NATO developed three different HAS denominated 1$^{st}$, 2$^{nd}$ and 3$^{rd}$ Generation HAS [Cocroft; 2003].

1$^{st}$ Generation HAS was almost identical to the TAB-V shelter. The only main difference was in the gates, which had a lateral and not outward opening procedure. 2$^{nd}$ Generation HAS was developed to accommodate bigger planes and had dimensions 37,6x25x10 m and a pseudo elliptical arch form. The gates were also much bigger than those of the 1$^{st}$ Generation HAS, with a total weight ascending to 167 ton. The 3$^{rd}$ Generation HAS was conceived to smaller planes like the F-15 or A-10 and had dimensions 36,6x21,6x9,1 m.

In the 1970’s and 1980’s hundreds of these shelters were build in Europe and USAF bases across the world. During this period, the nations of the Warsaw Pact, lead by the former USSR (Union of Soviet Socialist Republics), also constructed aircraft shelters, in many ways similar to those implanted in NATO bases. The main feature of these shelters was the earth cover that many of them had to provide extra strength and camouflage.

Other nations not directly involved in the Cold War developed their own aircraft shelters. One of best examples is Iraq. During Saddam Hussein’s regime, Iraq constructed several aircraft shelters called *Yugos* or *Trapezoids* because of their geometrical shape. These shelters were highly reinforced structures with alternate layers of concrete and sand, with a total thickness of approximately 2,5 m.

Since the 1990’s, this model of reinforced aircraft shelter is being put aside due to several changes in the concept of war and mostly due to advances in specialized bombs and missiles, capable of penetrating thick layers of highly reinforced concrete. Nowadays, most aircraft shelters are being made as portable structures with metallic frame covered by a tension resistant fabric. Though not providing the amount of resistance of a HAS type shelter, they still offer good protection from atmospheric hazards.

The Portuguese Air Force shelters used by the F-16 aircraft were conceived with this objective of atmospheric protection, being called ‘Weather Shelters’. Nevertheless, they have a structure similar to the HAS concept, with a reinforced concrete cylindrical shell with concrete portal frames on both ends, internally covered by a profiled steel sheeting. The dimension of the “Weather Shelters” are 25x16x6 m and the thickness of the concrete varies from 0,24 to 0,50 m (far from the values of NATO HAS type shelters) [Gestécnica; 2006].


**Fig. 2. 1 – Portuguese Air Force Weather Shelters in Monte Real Air Base.**

## 3 – CYLINDRICAL SHELL’S BEHAVIOR AND ANALYTICAL FORMULATION

Shell structures are one of the most efficient structural types. The equilibrium state that develops consists of 10 stress resultants (mainly in-plane forces, called the membrane forces, with some bending and twisting moments and shear forces).

The consideration of all these forces is not possible in a simple way. To overcome this difficulty the membrane theory was developed. This theory states that a shell’s behavior can be studied considering only the membrane forces. In order for this to be valid, some conditions must be verified [Montoya; 1969]: i) small thickness of the shell, in order to neglect the bending stiffness; ii) continuous curvature; iii) uniformly distributed loads; iv) reactions in the supports tangent to the shell’s middle surface and v) support conditions compatible with the shell’s free edge deformations.

Conditions iv) and v) are rarely verified and therefore bending and shear forces are developed. In this case a more general theory (bending theory) is required, with a much more complex treatment. However, it's proven by several authors that these forces have only relevant values in areas very close to the edges, making the membrane theory results very accurate in almost all the shell.

Cylindrical shells are also influenced by the support conditions near the linear and curved edges (called diaphragms). In fact, if the shell is supported only in linear edges, its behavior is very similar to that of an arch with the same dimensions as the arch of the shell. If supports are present on all edges, the shells behavior predicted by the membrane theory is similar to that of an arch-beam grid model.

In this thesis, analytical expressions are presented for both types of behavior. Since these expressions are only used in this work for validation of a finite element model, they correspond to simple loads, like a concentrated load (in case of the arch behavior) – expressions [3-1] to [3-3] [Leontovich; 1970] – and self-weight (in case of the membrane behavior) – expressions [3-4] to [3-6] [Ramaswamy; 1984].

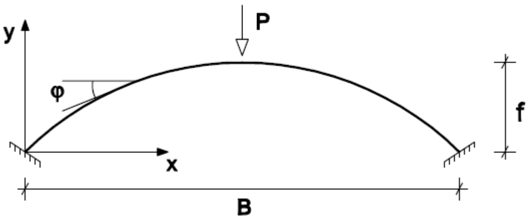


**Fig. 3.1 – Variables in expressions [3-1] to [3-3].**

$$N = -\frac{15}{64}\frac{P \times B}{f}\cos\varphi - \frac{P}{2}\sin\varphi \quad [3\text{-}1]$$

$$V = \frac{15}{64}\frac{P \times B}{f}\sin\varphi - \frac{P}{2}\cos\varphi \quad [3\text{-}2]$$

$$M = \frac{P}{32}\left(B + 16x - \frac{15B \times y}{2f}\right) \quad [3\text{-}3]$$

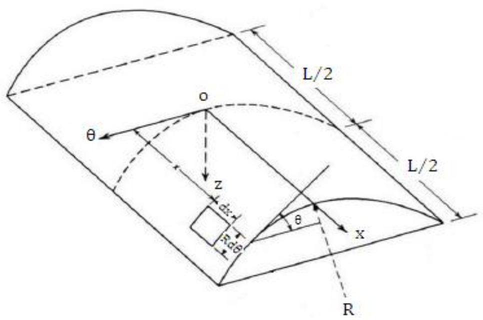


**Fig. 3.2 – Variables in expressions [3-4] to [3-6].**

$$N_\theta = -p_z \times R \quad [3\text{-}4]$$

$$N_{x\theta} = -Kx + f_2(\theta),\ with\ k = \frac{1}{R}\frac{dN_\theta}{d\theta} + p_\theta \quad [3\text{-}5]$$

$$N_x = \frac{x^2}{2R}\frac{dK}{d\theta} - \frac{1}{R}\frac{df_2(\theta)}{d\theta}x + f_1(\theta) \quad [3\text{-}6]$$

In [3-5] and [3-6], $f_1(\theta)$ and $f_2(\theta)$ correspond to integration constants. Parameters $p_z$ and $p_\theta$ correspond to the dead load components in the z and θ axis of Figure 3.2.

Expressions based on the general bending theory are not presented. The complexity of the formulation added to the accuracy of the membrane theory in almost all the shell and the presence of powerful finite element computer programs doesn't justify the use of this theory on the present work.

One aspect of the shell behavior that requires special treatment is structural stability. In shells of great span and/or small thickness this phenomenon can be important. Since the critical load of the structure is highly influenced by the load type and geometrical properties of the shell, no general expression can be used. In this thesis, results regarding the work of Timoshenko [Timoshenko; 1961] in arch structures are presented. These results, for common load types, are then extrapolated to cylindrical shell structures.

## 4 – LOAD DEFINITION AND DESIGN CRITERIA RELATING THE WEATHER SHELTER

The materials used in the shelter were grade C25/30 concrete, grade A400NR steel for rebar and grade Fe E 320G steel for the profiled sheeting. The loads considered on the structure, in accordance with the Eurocodes, were the following: self-weight; non-structural elements self-weight; imposed load; wind; snow; seismic load and shrinkage (the effect of shrinkage was only considered in the evaluation of Serviceability Limit States – SLS). Comparing these actions to the correspondent ones on the Portuguese regulation it's possible to conclude that, with the exception of wind action, the Eurocodes define loads that produce higher stresses in the structure.

The design combinations were defined in accordance with Eurocode 0 (EN1990). Two types of combinations were used for Ultimate Limit States (ULS) evaluation:

- For seismic load as main variable load:

$$A_d = \sum_{i=1}^{m} G_{k,i} + E_d + \sum_{j=2}^{n} (\psi_{2,j} \times Q_{k,j}) \qquad [4\text{-}1]$$

- For main variable load other than seismic load:

$$A_d = \sum_{i=1}^{m} (\gamma_{G,i} \times G_{k,i}) + \gamma_{Q,1} \times Q_{k,1} + \sum_{j=2}^{n} (\gamma_{Q,j} \times \psi_{0,j} \times Q_{k,j}) \qquad [4\text{-}2]$$

For the evaluation of SLS the quasi-permanent combination (QPC) was used:

$$A_{qpc} = \sum_{i=1}^{m} G_{k,i} + \sum_{j=2}^{n} (\psi_{2,j} \times Q_{k,j}) \qquad [4\text{-}3]$$

In these expressions $E_d$ corresponds to the seismic load design value; $Q_{k,1}$ and $Q_{k,j}$ are the main variable load and other variable loads' characteristic values; $G_k$ corresponds to the permanent loads' characteristic value; $\gamma_G$ is the safety factor of the permanent loads (1,35 for self-weight, 1,00 for shrinkage and 1,50 for non-structural elements self-weight) and $\gamma_Q$ the safety factor of the variable loads (equal to 1,50). The parameters $\psi_0$ and $\psi_2$ are the combination factors defined in EN1990 for each variable load.

In the evaluation of the ULS regarding material failure, a 'sandwich' model was used. This model assumes the existence of three 'fictitious' layers in a shell element. The outer layers are centered on the mid-plane of the rebar layers and are responsible for resisting in-plane forces, bending and twisting moments through an in-plane stress field (a pure membrane state). The stresses involved in this membrane state - $\sigma_{Edx}$, $\sigma_{Edy}$, $\tau_{Edxy}$ – are determined from equilibrium relations in the shell element. The middle layer is only responsible for carrying the shear forces.

In the design of the outer layers, a plastic strut-and-tie model is used. The optimum reinforcement stresses $f'_{tdx}$ and $f'_{tdy}$ according to this model and the corresponding compressive stress $\sigma_{cd}$ are given in Eurocode 2 Part 2 (EN1992-2) by the following expressions:

- If $\sigma_{Edx} \leq |\tau_{Edxy}|$:

$$f'_{tdx} = |\tau_{Edxy}| - \sigma_{Edx} \qquad [4\text{-}4]$$

$$f'_{tdy} = |\tau_{Edxy}| - \sigma_{Edy} \qquad [4\text{-}5]$$

$$\sigma_{cd} = 2 \times |\tau_{Edxy}| \qquad [4\text{-}6]$$

- If $\sigma_{Edx} > |\tau_{Edxy}|$:

$$f'_{tdx} = 0 \qquad [4\text{-}7]$$

$$f'_{tdy} = \frac{\tau_{Edxy}^2}{\sigma_{Edx}} - \sigma_{Edy} \qquad [4\text{-}8]$$

$$\sigma_{cd} = \sigma_{Edx} \times \left[1 + \left(\frac{\tau_{Edxy}}{\sigma_{Edx}}\right)^2\right] \qquad [4\text{-}9]$$

In the case of bi-axial compression, the expressions to determine the maximum and minimum compression ($\sigma_I$ and $\sigma_{II}$) in each of the outer layers are derived using the Mohr circle:

$$\sigma_I = \frac{|\sigma_{Edx}+\sigma_{Edy}|}{2} + \sqrt{\frac{\left(\sigma_{Edx}-\sigma_{Edy}\right)^2}{4} + \tau_{Edxy}^2} \qquad \text{[4-10]}$$

$$\sigma_{II} = \frac{|\sigma_{Edx}+\sigma_{Edy}|}{2} - \sqrt{\frac{\left(\sigma_{Edx}-\sigma_{Edy}\right)^2}{4} + \tau_{Edxy}^2} \qquad \text{[4-11]}$$

The compression values are compared to the maximum compression $\sigma_{cd,max}$ given in EN1992-2.

The middle layer's maximum shear force $v_{Ed0} = (v_{Edx}^2 + v_{Edy}^2)^{1/2}$ is compared to expression [4-12] or, in case transverse reinforcement is necessary, [4-13] (both from EN1992-2).

$$V_{Rd,c} = \left[C_{Rd,c} \times k(100\rho_L \times f_{ck})^{1/3} + k_1 \times \sigma_{cp}\right] \times b_w d \geq \left[0{,}035k^{3/2} \times f_{ck}^{1/2} + k_1 \times \sigma_{cp}\right] \times b_w d = V_{Rd,min} \qquad \text{[4-12]}$$

$$V_{Rd} = min\left\{\frac{A_{st}}{s} \times z_c \times f_{yd} \times cotg(\theta');\ \frac{b_w \times z_c \times 0{,}6\left(1-\frac{f_{ck}}{250}\right) \times f_{cd}}{cotg(\theta')+tan(\theta')}\right\} \qquad \text{[4-13]}$$

The variables in these expressions are defined in accordance with EN1992-2.

The finite element software SAP2000™ uses this methodology on the design of shells. Therefore, the program SAP2000™ is used for the safety evaluation described above. The only procedures that can't be executed directly on the program are the compression and shear force safety evaluations.

In the ULS regarding stability the IASS (International Association for Shell and Spatial Structures) procedure is considered [IASS; 1979]. The load adopted corresponds to the sum of the self-weight with the non-structural elements self-weight. The critical load of the structure must be superior to 3,5 times this value, and must have into consideration phenomenons like plastic behavior or creeping. Even adopting of a global reduction factor that includes all these phenomenons, the critical load of the shelter is still far superior to the safety limit.

The SLS considered were deformation and cracking. The limit value considered for deformation is L/500, with L being the distance between diaphragms. In order to account for creeping, the deformation in the structure is given by

$$\delta_\infty = \delta_{el} \times \left(1 + \varphi(\infty, t_0)\right),\ with\ \varphi(\infty, t_0) = 2{,}5 \qquad \text{[4-14]}$$

For cracking, the procedure involves determining if there are areas where the longitudinal stress (since shrinkage is the primary problem in this situation) surpasses $f_{ctm}$. If so, safety is verified if these areas possess a minimum reinforcement given by expression [4-15], defined in accordance with the Eurocode 2 Part 1-1 (EN1992-1-1):

$$A_{s,min} = k_c \times k \times A_{ct} \times \frac{f_{ct,ef}}{\sigma_s} \qquad \text{[4-15]}$$

**5 – NUMERICAL ANALYSIS AND SAFETY EVALUATION OF THE WEATHER SHELTER**

In order to analyze the shelter a finite element model using shell elements is used. The equivalent thickness ($h_{c,equiv}$) is determined so that each finite element has the same section area as the real corresponding section of the shelter. Two general areas of mesh refinement are considered: 1) shell elements situated above height 1,5 m – elements of approximately 1,0x1,0 m and

$h_{c,equiv}$=0,175 m; 2) shell elements below height 1,5 m – finer mesh, with elements of approximately 1,0x0,3 m and $h_{c,equiv}$=0,18, 0,21, 0,25, 0,31 and 0,39 m (the thickness of the shelter increases towards the linear edge of the structure). In the curved edges, the concrete portal frame is modeled with shell finite elements and the linear edges of the model, corresponding to the foundations, are assumed to have fixed supports.

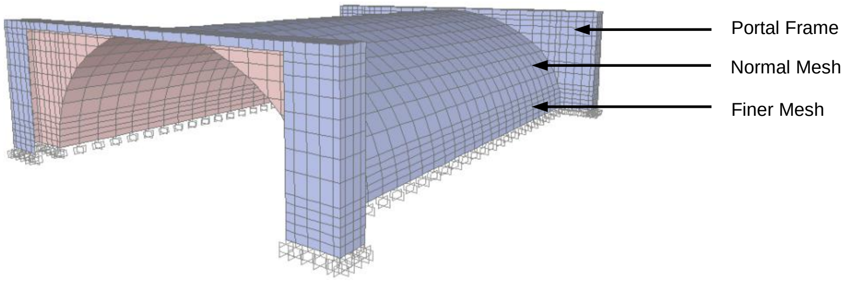


**Fig. 5.1 – Weather Shelter's finite element model in SAP2000™ software.**

To validate this model a comparison was made between the analytical results from expressions [3-1] to [3-6] and the numerical results from the model, in a mid-span section. In order for this comparison to be accurate, the model support conditions weren't the ones showed in Fig.5.1 but instead the conditions for which the expressions are valid. It was observed that the analytical and numerical results were similar except in the vicinity of linear edges. This difference is due to conceptual and geometric differences between the shelter model and the simplified structure assumed in the analytical expressions. Globally, the model provides a good approximation of analytical results.

The first ULS safety evaluation that is considered regards the existing shell's rebar, showed in Fig. 5.2.

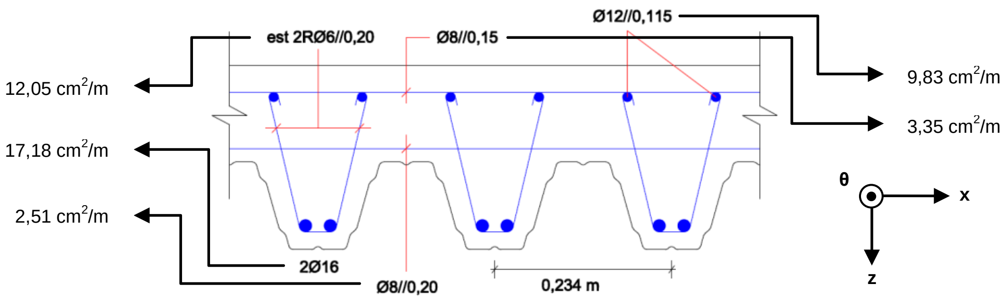


**Fig. 5.2 – Rebar adopted in the Weather Shelter – distribution and quantities per meter.**

The design combination that conditioned the rebar design was the one involving the wind load at 0º (wind load normal to the shell's longitudinal development), except for the lower layer of rebar in the θ direction, which was conditioned by the seismic combination.

In the θ direction, the values of rebar needed to verify the safety criteria are 3,33 and 8,15 $cm^2$/m, in the upper and lower layer, lower than the existing walls reinforcement.

In the x direction, the values of rebar needed to guarantee structural safety exceed the values showed in Fig.5.2, in a few localized areas near the portal frames. The maximum values are 4,06 and 4,84 $cm^2$/m, in the upper and lower layer. Taking into consideration the small difference to the shelter's existing reinforcement and the small areas where this problem occurs, one solution is to add locally some additional reinforcement in the horizontal direction

The compression ULS is also conditioned by the combination involving the wind load at 0º. The maximum compression values occur on the link between the shell and the portal frames, approximately 3 m from the base of the shell, and correspond to -2746,56 and -2145,25 kN/m$^2$ (upper and lower layer). Using a conservative approach, it's possible to prove that, with the grade of concrete and rebar used, the minimum value of $\sigma_{cd,max}$ in accordance with EN1992-2 is -8500 kN/m$^2$. Since this value is superior to the maximum compression observed in the structure, safety is guaranteed.

The shear resistance of the shelter, in accordance with expression [4-12], is a function of the shell thickness. Therefore, each different finite element has a shear resistance $V_{Rd,c}$>$V_{Rd,min}$ that must be compared with the maximum shear $v_{ed0}$ on that kind of element. $v_{ed0}$ is conditioned by the design combination with wind load at 0º, and occurs near the link between the shell and the portal frames.

As showed in Table 5.1, $v_{ed0}$ is inferior to $V_{Rd,min}$ in every type of finite elements, except $h_{c,equiv}$=0,39 m, in which the determination of $V_{rd,c}$ is needed. Since $v_{ed0}$ is inferior to $V_{rd,c}$ in all elements, safety is guaranteed.

**Table 5.1 – Maximum shear forces on the structure and shear resistance forces.**

| $h_{c,equiv}$ [m] | 0,175 | 0,18 | 0,21 | 0,25 | 0,31 | 0,39 |
|---|---|---|---|---|---|---|
| $V_{ed0}$ [kN/m] | 30,87 | 12,83 | 14,27 | 28,74 | 81,82 | 149,78 |
| $V_{Rd,min}$ [kN/m] | 47,02 | 49,50 | 64,35 | 84,15 | 108,13 | 131,36 |
| $V_{Rd,c}$ [kN/m] | - | - | - | - | - | 187,32 |

In relation to the shell's stability, a buckling analysis of the structure using SAP2000™ determined an elastic critical load of 831,92 kN/m$^2$. The limit value is 3,5x(self-weight+non-structural self-weight)=16,17 kN/m$^2$. Even assuming a global reduction factor of 0,43 (determined on the work of Cardoso [Cardoso; 2008], for a shell with a big span of 50 m and small thickness of 0,12 m, therefore highly susceptible to instability) the critical load is still 357,73 kN/m$^2$, which is about 23 times the limit value. Based on this, it's possible to assume that stability problems are not important in the Weather Shelter behavior.

For the SLS of deformation, the conditioning elastic value $\delta_{el}$ is at the mid-span section on the top of the shell, and corresponds to 8,39 mm. The deformation value accounting for creeping is obtained using expression [4-14], and has the value 29,4 mm. The limit value L/500, for a span of 25 m, is 50 mm which guarantees security in relation to this SLS.

The SLS of cracking involves the determination of the areas where longitudinal stress surpasses $f_{ctm}$. These areas are showed in Fig.5.3 for the top face of the shell (they're approximately the same in the bottom face) in dark-blue color.

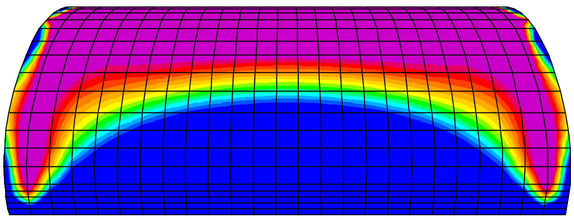

**Fig. 5.3 – Longitudinal stress on the top face of the shell.**

In order to verify safety to this SLS, these areas must have the minimum reinforcement given by [4-15]. However, none of the elements verify this condition since the minimum value of reinforcement corresponds to elements with $h_{c,equiv}$=0,175 m and is 11,38 cm$^2$/m (in the x direction total reinforcement in the section is 2,51+3,35=5,86 cm$^2$/m). Vertical cracking with large width is to be expected, and *in-situ* observations of the structure confirm this exact situation. Since this type of cracking is mainly due to shrinkage, this problem can be diminished or even solved with a new concreting plan.

## 6 – STRENGTH EVALUATION OF THE COMPOSITE STEEL AND CONCRETE SHELTER SUBJECTED TO AN ACCIDENTAL LOAD

In order to determine the strength of the composite structure some simplifying assumptions must be considered, due to limitations in the regulation (EN1994-1-1) and complexity of the structure: 1) a separate analysis of the structure in the x and θ is made; 2) in the θ direction the shell is considered with its real dimensions and with the profiled steel sheeting; in the x direction the profiled sheeting is not considered and the shell is assumed to have the thickness of the concrete between ribs (Fig. 6.1); 3) in the x direction the rebar considered is superior to the one existing on the shelter, in order to verify the SLS of cracking; 4) twisting moments are considered by including them in the bending moments; in/plane shear forces are not considered; 5) the shell is subjected to a combined state of axial compression and bending; assuming the axial compression is moderate, it isn't considered in the analysis; 6) transverse resistance in the θ direction is given by the ribs; 7) conexion between concrete and profiled sheeting is assumed to be complete; 8) the strength contribution of rebar or profiled sheeting subjected to compression is not considered.

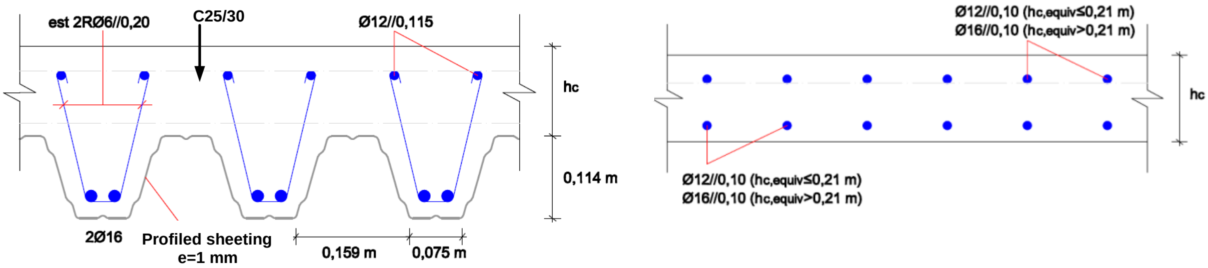


**Fig. 6.1 – Transverse section considered for strength evaluation: θ direction (in the left); x direction (in the right).**

The methodology used in the strength evaluation followed the work of Calado [Calado; 2006], based on the EN1994-1-1: in both directions a plastic analysis is considered, assuming that the stress distribution on the material is constant and equal to the yielding stress. The neutral axis is determined by equilibrium relations and the resistant bending moment is obtained by the binary between compression and tension forces. The shear resistance is obtained by the use of [4-12] for the x direction and [4-13] for the θ direction. The results are summarized in Table 6.1:

**Table 6.1 – Resistance bending moments and shear forces considering a composite steel and concrete shelter.**

| | $h_{c,equiv}$ [m] | 0,175 | 0,18 | 0,21 | 0,25 | 0,31 | 0,39 |
|---|---|---|---|---|---|---|---|
| | $h_c$ [m] | 0,12 | 0,125 | 0,155 | 0,195 | 0,255 | 0,335 |
| θ | $m^+_{pl,Rd}$ [kNm/m] | 175,321 | 180,974 | 214,894 | 260,119 | 327,957 | 418,408 |
| | $m^-_{pl,Rd}$ [kNm/m] | -59,426 | -61,135 | -71,389 | -85,060 | -105,566 | -132,908 |
| | $V_{Rd}$ [kN/m] | 127,056 | 130,686 | 152,467 | 181,508 | 225,070 | 283,153 |
| x | $m^+_{pl,Rd}$ [kNm/m] | 33,484 | 35,451 | 47,252 | 103,042 | 145,011 | 200,969 |
| | $m^-_{pl,Rd}$ [kNm/m] | -27,583 | -29,550 | -41,352 | -92,550 | -134,518 | -190,477 |
| | $V_{Rd,c}$ [kN/m] | 55,703 | 60,123 | 86,648 | 122,015 | 175,065 | 203,900 |

The accidental load considered in the shelter is a nearby explosion. From the various effects an explosion has, only the blast wave is considered. This blast wave consists on the sudden rise of air pressure from the atmospheric value $P_0$ (assumed as 131,325 kPa at sea level) to a peak incident value $P_{s0}$. When the blast wave encounters a structure it suffers a compression and the peak value $P_{s0}$ is augmented two to eight times, to what is called the peak reflected pressure $P_{r0}$. This value is fundamental in blast analysis because it's the responsible for the stresses in the structure [Shope; 2006].

To simplify the blast load some assumptions are considered: 1) although the blast load is a time-dependent phenomenon, its assumed in this work as a static action represented by the peak reflected value $P_{r0}$; 2) the blast wave occurs in lateral position of the shelter and has uniform distribution (Fig. 6.2).

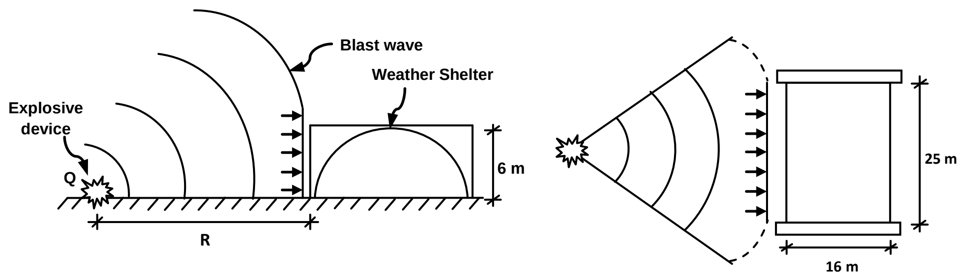


**Fig. 6.2 – Blast wave representation on the Weather Shelter: side view (in the left); top view (in the right).**

The explosive device considered for the academic purpose of this thesis is similar to an MK82 bomb, having an TNT equivalent explosive quantity Q of 95,23 kg. The relation between quantity Q and distance R is called Scaled Distance and is represented by $Z = R/Q^{1/3}$. This relation is used in the expressions [6-1] to [6-5] to determine blast wave pressure in MPa, in accordance with the Allied Ammunition Storage and Transport Publication (AASTP) methodology [AASTP; 2006].

$$P_{s0} = 1{,}3131379 \times Z^{-1{,}910441}, \quad 0{,}5 \leq Z \leq 0{,}75 \qquad [6\text{-}1]$$

$$P_{s0} = 1{,}330026 \times Z^{-2{,}218832}, \quad 0{,}75 \leq Z \leq 3{,}50 \qquad [6\text{-}2]$$

$$P_{s0} = 0{,}724571 \times Z^{-1{,}726565}, \quad 3{,}50 \leq Z \leq 8{,}50 \qquad [6\text{-}3]$$

$$P_{s0} = 0{,}293592 \times Z^{-1{,}295654}, \quad 8{,}50 \leq Z \leq 30{,}00 \qquad [6\text{-}4]$$

$$P_{r0} = 2 \times P_{s0} \times \left(\frac{7 \times P_0 + 4 \times P_{s0}}{7 \times P_0 + P_{s0}}\right) \qquad [6\text{-}5]$$

In order to determine the minimum distance R that guarantees structural safety (distance after which the maximum stress is inferior to the correspondent resistance value) several explosion actions were considered between 15 and 100 m, at 5 m intervals. The $P_{r0}$ values obtained were imposed to the finite element model of the shelter, using a accidental combination of 1,0xself-weight+1,0xnon-structural self-weight+1,0xExplosion.

It was verified that, in the θ direction, maximum stresses relating bending moments and shear forces were obtained near the linear edges of the shell. In the x direction, maximum stresses were obtained in the link of the shell with the portal frames, approximately 3 m from the base of the shell. Comparing the maximum stresses obtained with the resistance values of Table 6.1, it's possible to observe that the safety distance R is, in most cases, between 15 and 35 m. However, in three situations distance R exceeds 35 m: 1) negative bending moment in the θ direction for $h_{c,equiv}$=0,390 m

(R=47 m); 2) shear force in the θ direction for $h_{c,equiv}$=0,310 m (R=41 m) and 0,390 m (R=50 m); 3) shear force in the x direction for $h_{c,equiv}$=0,175 m (R=39 m). Based on these scenarios, it's possible to conclude the safety distance of an explosion of this kind is 50 m (corresponding to a $P_{r0}$ value of 27,9 kN/m$^2$).

In order to reduce this distance several solutions are presented to solve the mentioned issues: 1) assuming the rebar used is the more conventional Ø12//0,125 instead of Ø12//0,115, a reinforcement of Ø16//0,125 is considered in the conditioning region; 2) transverse reinforcement with superior diameter 2RØ8//0,20 is considered on the ribs of the conditioning region; 3) a thickness increase of the concrete wall of the shell near the portal frames, from 0,12 m to 0,15 m. If all of these solutions were adopted, the safety distance R would be reduced to 35 m (corresponding to $P_{r0}$=46,9 kN/m$^2$, a increase of 68% comparing to the current situation).

## 7 – CONCLUSIONS AND FUTURE DEVELOPMENTS

Regarding the first objective, it's possible to conclude the following: 1) in ULS structural safety is verified in all situations with exception of some small localized areas where a small reinforcement is needed in the x direction; the rebar solution in the θ direction is excessive; in the Weather Shelter stability problems aren't relevant; 2) In the SLS, only the crack safety is not verified since the minimum reinforcement for crack control is not considered (a specific concreting plan can be used to reduce shrinkage and therefore cracking).

With respect to the second objective the following conclusion can be taken: 1) structural safety is conditioned by localized material failure near the edges of the shell; the minimum safety distance of the explosion considered, if no modification is made in the shelter's structure (other than the ones considered for analysis purposes) is 50 m; 2) if the solutions presented in this work were implemented, the safety distance would be reduced to 35 m, corresponding to a gain of 68% on the maximum pressure $P_{r0}$ that the structure can withstand.

The work presented on this thesis can be complemented in the future by the following subjects: 1) analysis of other Weather Shelter with different support conditions; 2) analysis of the portal frames and their connection to the shell; 3) evaluation of the shelter's resistance to fire; 4) evaluation of the shelter's behavior to vibrations induced by the F-16's engine; 5) analysis of the shelter behavior to an inside explosion; 6) analysis of the shelter considering explosion effects other than blast wave; 7) evaluation of the shelter's resistance to small caliber ammunition; 8) evaluation of the shelter's resistance using a non-linear physical analysis using the mean values of material properties and real stress-deformation laws, in order to make an assessment on the real resistance of the Weather Shelter.

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