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Paper 274

The Collapse of a Skylight Shell Structure caused by a Blast

A. Angotzi[1], A. Mussi[1], E. Trolese[2], M. Zanchin[2]
R. Gori[3] and A. Mastropasqua[3]
[1] Arup Italy srl, Milan, Italy
[2] Mastropasqua-Zanchin & Associates Structural Engineering Treviso-Udine, Italy
[3] Department of Civil, Environmental and Architectural Engineering (DCEAE) Padua University, Italy

Abstract

This paper describes the non-linear mechanical response of a skylight shell structure, created using a glass envelope and structural steel members, in the event of an explosion. The system, currently under construction, has a geometrical axis of 52.50m (maximum) and 40.00m (minimum), and the span ratio n = 6.75 %. Several pressure wave signals were obtained, according to the American specifications of TM5-1300 [1], and the consequent non-linear time-history response of the system were investigated.

Keywords: steel structures, non-linear analysis, blast response.

1 Introduction

Following the collapses caused by explosions which have involved both civil and military buildings in recent years, the dimensioning of structures subjected to explosive forces and the verification of the resistance to collapse of one or more structural elements have become of utmost importance during the design phase.
In the past, building design aimed at preserving the safety of people following an explosion only applied to buildings used for military purposes or those that were located near sites where explosives were stored or flammable materials were processed.
However, in the current socio-political climate, particularly after the attacks on the Twin Towers and the Pentagon, the risk of building collapse is no longer considered remote for those buildings not necessarily used for military or strategic purposes. Attention is currently focused on branches of banks and other financial institutions, centers for political activity, religious sites and large commercial buildings. Therefore, with reference to the structural properties of a system as a general requisite for resistance to explosions, researchers in the last few years have concentrated on *robustness*, even though an explicit method for the evaluation of

this characteristic has yet to be determined. We will leave the methods for the quantification of the *index of robustness* (I_r) to more specialized treatises (see the guidelines set by JCSS [2], as well as recent proposals overseen by Formisano-Mazzolani [3]). Instead, we have focused on highlighting the lack of design for blast resistance, associated with lack of robustness, with particular reference to the progressive collapse of the Alfred P. Murrah Federal Building in Oklahoma City on 19 April 1995 (Figure 1), where the blast from a car bomb near the building caused the death of 168 people. In particular, subsequent investigations revealed that approximately 80% of the victims were not killed as a direct result of the explosion, but rather because of the collapse of the north facade of the building. Also, even though it was shown that the resistance of the materials used was well over the minimum design requirements, the structure in fact had little ductility and was redundant. There were numerous consequences of this attack, including the extension of the pre-existing regulations for the construction of government buildings in foreign countries to all new buildings constructed in the USA. These include both passive design safety measures (for example preventing unknown vehicles from nearing buildings) and active design safety measures (aimed at increasing the ductility and resistance of structures). Additionally, a proposal was made to adopt anti-seismic regulations for all new buildings at risk of attack, even in areas not classified as seismic [4].

Figure 1: Oklahoma City (1995).

Therefore, inserting the actions provoked by a blast into the conceptualizing process of structures is an active design defense against the potential effects of an attack. The characteristics of an explosion are therefore managed as a design parameter in structural analysis in order to determine its consequences. In particular, analysis of the behaviour of a structure in the event of a malicious explosion can reveal damping systems and deviations from the effects which could diminish the direct consequences of such an attack [4]. It could also be extremely significant to

intercept any potential kinematic mechanism associated with structural failure, by means of a suitable probability rate.
This work therefore evaluates suitable considerations, with reference to a specific structural system currently under construction. On one hand, there is the probability and potential magnitude of a terrorist attack, and on the other hand, there is the possibility of structural failure, which is investigated using numerical analysis of the transitional non-linear mechanical response.

2 The shell structure and its structural geometry

Our analysis focuses on a skylight structure, used as the roof of a large atrium of a commercial building which often contains large crowds (Figure 2). The metal skeleton is constructed using: principal beams (RHS600*250*10 type) arranged along the minor geometrical axis (a = 40.00m) of the ellipse's rim; secondary beams (RHS300*200*10 type) arranged transversally (therefore with length b = 52.50m) and, in alternation, connected with welds and bolted to the principal beams (in order to simplify the lifting operation and the mounting of the structural pre-assembled subparts); an HE600M type beam reversed along the rim to reduce thrust; and a set of bracings (CHS 139.7*8 profile) (Figure 2). All the elements were designed in S355J0 steel [5]. The *skylight* is enclosed by glass modules of approximately 300mm x 1500 mm, made of 10.10.4 laminated glass where the external and internal monolithic sheets are tempered and hardened, with 4 layers of shatterproof PVB. The presence of this laminated glass, although respecting the current anti-fall safety regulations, also constitutes an unbreakable barrier which is unable to dissipate energy by means of the instant opening of the glass area and the dispersion of the shock wave.

a = 40.00 m (minor axis)   b = 52.50 m (major axis)
f = 2.70 m (height)   n = f/L = 6.75% (span ratio)

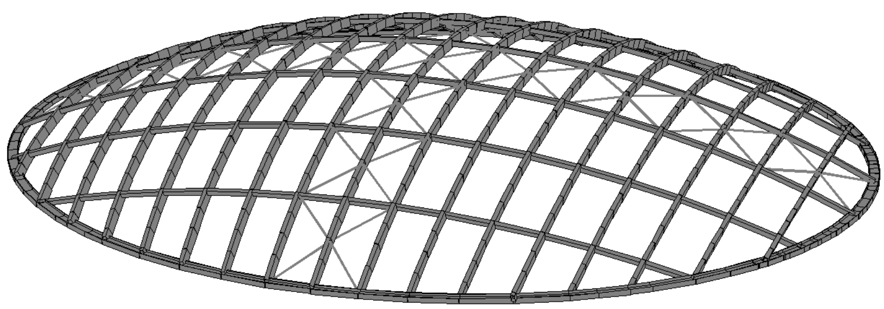

Figure 2: 3D view of the structural system.

The elliptical surface area covered is 1658 m^2, while the glass surface area is 1750 m^2; the approximate final weight of the structural steel work is 1500 kN, thus obtaining the resultant average force along the surface (projected vertically) of 0.90 kPa. From the standpoint of the static constraint, the skylight can be considered a closed structural system in compression, due to the absorption of forces at the dome's flat plane by the HE600M beams on the rim. The constraints supporting the beams along the rim have thus been designed to allow for free expansion and contraction along the horizontal plane under any load, including thermal loads. With regard to the constraint for horizontal translation, an isostatic distribution was used throughout the structure. The finite element model, generated by directly importing the design geometries, was created with the software Straus7-Non-Linear rel.2.4 *beta version for tester* [6].

3 Action applied to the structure

3.1 Static design actions

In accordance with EN1990 and EN1991, the following were used in the design phase for static verification: G_1 = 0.90 kPa (actual weight of structure), G_2 = 0.60 kPa (weight of glazing), Q_1 = 1.00 kPa (maintenance of the skylight), Q_2 = 0.25 ÷ 1.00 kPa (action of the wind, quantified in zones under pressure and zones predominantly under suction), Q_3 = ± 35°C (uniform thermal action), Q_4 = 0.16 kPa (action of snow, according to technical specifications). The seismicity classification was characterized by a peak ground acceleration of a_g = 0.08 g (according to technical specifications).

3.2 Blast actions: the evaluation of the shockwave

The type of explosive considered was *trinitrotoluene* (TNT). The maximum charge considered was 20 kg, which is a plausible maximum load that could be carried by one person without the use of transport (motorbikes, cars). This is in virtue of the fact that the nearest access to the skylight, as shown in Figure 3, can be reached from the second raised floor of the structure. As a result, other types of explosions were not taken into consideration. Four charge quantities of TNT were taken into account during the design phase, of 5 kg, 10 kg, 15 kg and 20 kg respectively.
The detonation effects, in terms of *peak reflected pressure* (P_{R0}), *shockwave arrival time* (t_A), *duration of the positive phase* (t_0), *duration of the negative phase* (t_0^-), and *negative peak reflected pressure* (P_{R0}^-) were determined assuming, conventionally, a direct *pressure* relationship $P_S(t)$ using the polynomial equations experimented by Kingery and Bulmash [7], which form the basis of the US manual TM5-1300 [1]. This assumption was permitted by the *job technical specifications* which considered the distance between the detonation point and the glass system, therefore ignoring the side-wall reflection effects. Also the use of TM-1300 was indicated in the *job technical specifications*, in place of a more recent UFC_3-340-02 [8]. In order to utilize the above mentioned equations, having assigned a generic position to the source of the blast, the following variables need to be supplied: W (quantity of TNT, in *kg*), d (in *m*, distance from the source of the blast to the zone for the calculation of the pressure at the intrados of the glass enclosure) and α (angle of incidence at which

the shockwave hits the surface). The typological shape of the shockwave curve over time, for a generic point *R* of the skylight is characterized by W_R, d_R and α_R is thus illustrated in Figure 4, with the definitions for the symbols: P_S, P_{S0}, P_{S0}^-, P_0, t_A, t_0, t_0^- respectively: *shockwave pressure*, *peak pressure* (*positive*), peak *pressure* (*negative*), *atmospheric pressure*, *time of arrival of the shockwave*, *duration of the overpressure*, *duration of the shockwave under-pressure (pressure drop)*.

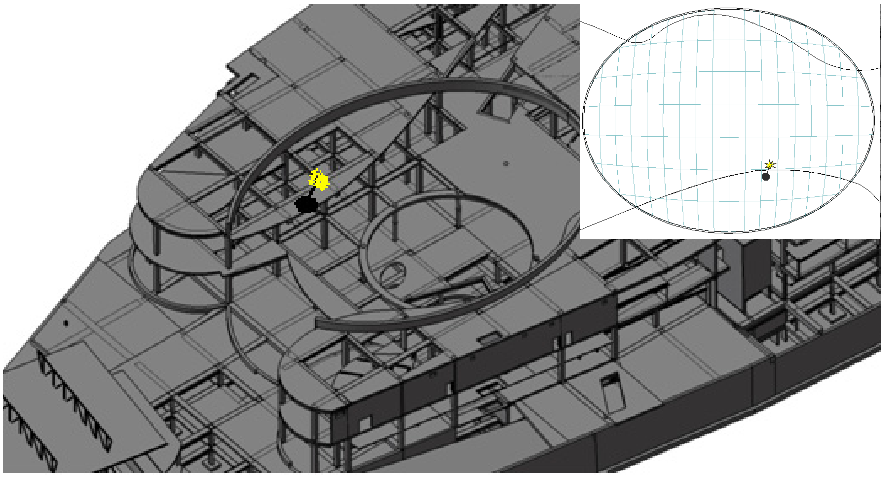

Figure 3: Structural support of the skylight and positioning of the explosive (yellow)

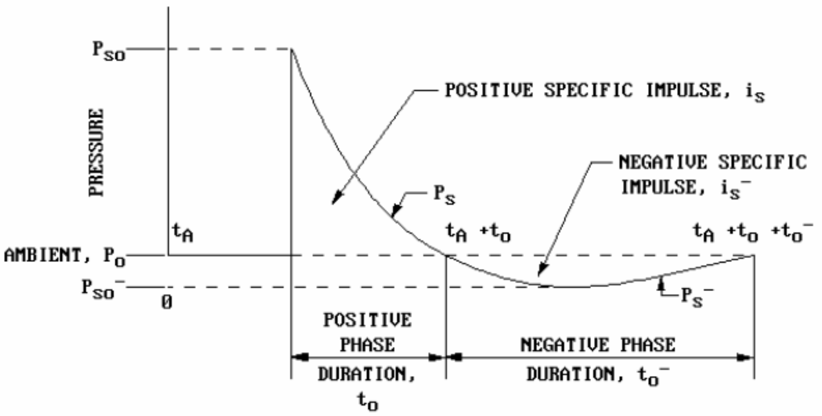

Figure 4: Overpressure wave: time trend.

The localized application to each glazing unit of the above-mentioned history of pressure in terms of time $P_S(t)$ was calculated for the 4 charge quantities hypothesized. For example, Figure 5 represents the spatial distribution of peak pressure (P_{S0}), obtained by denominating the 7 rows of glazing panels with the letters A÷F along the minor axis of the skylight (a = 40 m),.

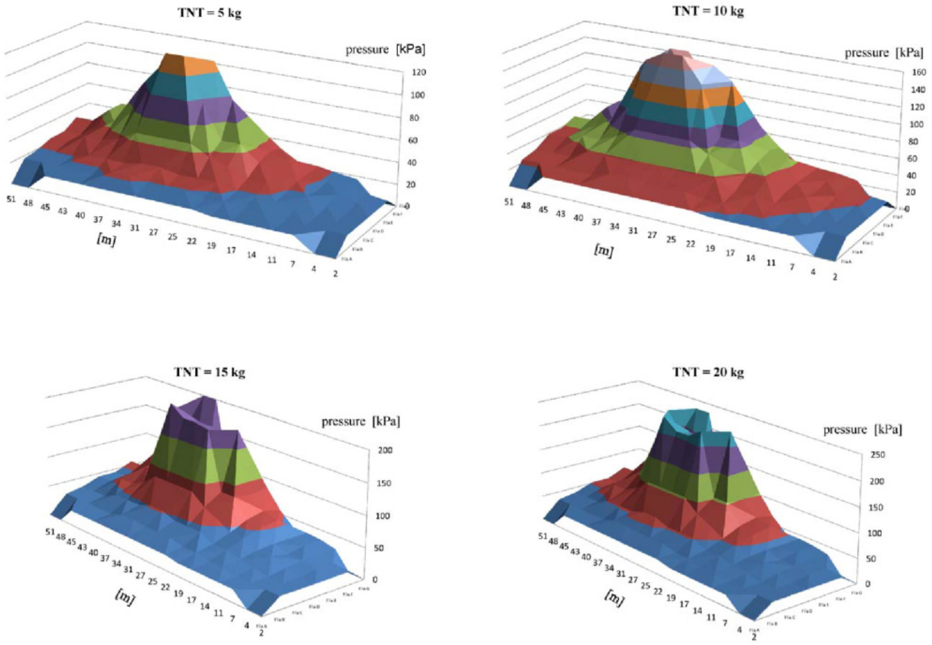

Figure 5: Spatial distribution of pressure peak P_{S0}.

The space-time distributions $P_S(t)$ were thus applied to a finite element model, the analysis of which will be described in the following paragraph.

4 The mechanical response to the explosion

The characteristic equation associated with the non-linear transient dynamic problem is:

$$Mu\ddot{(t)} + C\dot{u}(t) + [K_0 + K_G + K_{NL}]u(t) = G_1 + G_2 + E_b(t) \qquad (1)$$

where: M mass matrix, C damping matrix, $[K_0 + K_G + K_{NL}]$ non-linear tangent stiffness matrix (for material and geometry), $u(t)$ nodal displacement vector, $\dot{u}(t)$ nodal velocity vector, $\ddot{u}(t)$ nodal acceleration vector, G_1 steel dead load, G_2 glass dead load, $E_b = \iint_{A_g} P_S(t)dxdy$ blast force (time depending) obtained by integrating the blast pressure on the glass surface (A_g). For the calculation of the damping matrix C contained in (1), the following equations were used in accordance with the Rayleigh decomposition of the same [9][10][11]:

$$C = \alpha M + \beta[K_0 + K_G + K_{NL}] \qquad (2)$$

$$\alpha = 2\omega_1\omega_2(\varepsilon_2\omega_1 - \varepsilon_1\omega_2)/(\omega_1^2 - \omega_2^2) \qquad (3)$$

$$\beta = 2(\varepsilon_1\omega_1 - \varepsilon_2\omega_2)/(\omega_1^2 - \omega_2^2) \tag{4}$$

where α, and β are defined as the coefficients of *proportionality of mass* and *stiffness*, ω_1 and ω_2 the angular pulsations of the system in the immediate post-critical oscillation phase, ε_1 and ε_2 the damping coefficients associated with the same angular pulsations mentioned above (Table 1).

α and β factors							
Natural frequencies ν		Angular frequencies ω		Damping factors ε		Factors	
ν_1 [Hz]	ν_2 [Hz]	ω_1 [rad/s]	ω_2 [rad/s]	ε_1	ε_2	α	β
2.18	5.08	13.70	31.90	0.08	0.08	1.533	0.003509

Table 1: damping factors.

The differential equation (3) is solved by direct numerical integration according to the Newmark method. In accordance with this procedure, the numerical solution at every time step is supplied by the following system of equations:

$$u(t_{k+1}) = u(t_k) + \Delta t\dot{u}(t_k) + \left[\left(\tfrac{1}{2} - \theta\right)\ddot{u}(t_k) + \theta\ddot{u}(t_{k+1})\right](\Delta t)^2 \tag{5}$$

$$\dot{u}(t_{k+1}) = \dot{u}(t_k) + \tfrac{1}{2}\Delta t[\ddot{u}(t_k) + \ddot{u}(t_{k+1})] \tag{6}$$

where, supposing that θ = ¼, the method becomes unconditionally stable from a numerical point of view [9][10]. Furthermore, assuming the interval $\Delta t = 0.002$ s $\leq (5\nu_6)^{-1}$ where $\nu_6 = 4.93$ Hz is the sixth natural frequency of the structure, a significant number of modal shapes can be taken into account in the numerical response. In terms of the numerical output relative to the structure, the 6 characteristic points in Figure 6 were extracted. These 6 nodes were defined as: *maximum vertical oscillation* (Node 1), *point above the point of detonation* (Node 2), the *diametral position opposite the detonation point* (Node 3), and *critical points for the study of the ring beam* (Nodes 4, 5, 6). The applied moment of the explosion was set at t_b = 3.00 s. The following structural responses were highlighted:

a) TNT = 5.00 kg: global elastic response ($f<f_{yd}$);
b) TNT = 10.00 kg: global elastic response ($f<f_{yd}$);
c) TNT = 15.00 kg: non-linear response with reduced diffusion ($f\leq f_{yd}$), by the formation of a limited number of partial plastic hinges at the skylight beams and reduced degree of plastic ratio of sections;
d) TNT = 20.00 kg: non-linear response ($f\leq f_{yd}$) with moderately reduced diffusion, by the formation of a greater number of partial plastic hinges on the skylight, and plastic responses of the HE600M ring beam (Figure 9 and 10).

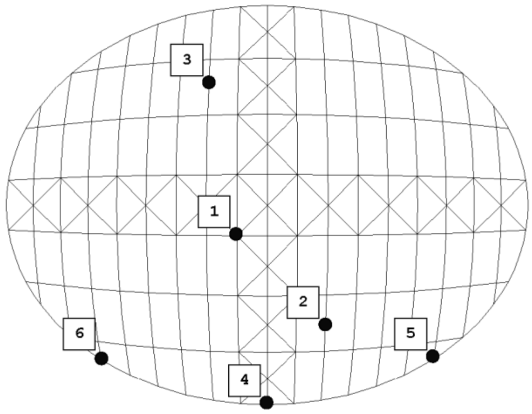

Figure 6: Characteristic points of study.

Moving to the fourth and worst case scenario where TNT = 20 kg, a comparison between a hypothetical *non-linear elastic* behaviour (with no energy dissipation) of the material and the realistic *non-linear elastic-plastic* response (with energy dissipation) was considered for certain nodes that resulted more stressed, and some indications about the damping response of the material were obtained. Figure 11 shows the output obtained from the plasticized beam n° 551 which is exactly above the detonation point and where the greatest local plastic dissipation occurred. It is possible to observe the structure's complete energy response curve (blue line, characterized by $\varepsilon_{tot}[t] = \varepsilon_{el}[t] + \varepsilon_{pl}[t]$) and the elastic response curve (red line, $\varepsilon_{el}[t]$) and deduce the following considerations:

e) the material's elastic limit was exceeded for TNT ≥ 15 kg. However, due to the reduced mass of the system and because of the impulse method of applying the blast energy, it occurs in reduced charge cycles during the moments immediately after the explosion (t_b = 3.10 ÷ 3.30 s); this time period is characterized by loss of energy due to plastic deformation. Furthermore, during the subsequent oscillations, a complete return to the linear behaviour is observed;
f) the plastic damping phenomenon is clearly visible only for the points that significantly enter into the plastic region (Figure 11, time interval t = 3.10 ÷ 3.30 s), but it is ultimately strictly local;
g) the integration to the total number of plastic hinges of the energy quantity (kJ) used by plastic dissipation was limited, in engineering terms. Hence the global damping can definitely not be compared to the results frequently obtained from non-impulsive dynamic analysis (for instance seismic or harmonic), where the repeated completion of the hysteretic cycles significantly affects the energy balance of the system.

Finally, an additional investigation with TNT = 25 kg was conducted with the aim of studying more critical situations of system equilibrium. There were no numerical convergences during the entire post-explosion response (t > 3.000 s). However,

Figure 12 shows the state of plasticity of the system in the last moment of equilibrium of the analysis (precisely 86 ms after the blast), and it was possible to observe a significant enlargement of the post-elastic region. This was characterized by a marked diffusion of plastic response of the ring beam, in addition to the area above the detonation, which deteriorated considerably due to a loss of stiffness of the sections. These indications, although they are only partial and cannot describe the collapse of the system, were nonetheless able to provide valid indications in terms of the hypotheses of possible locations of plastic mechanisms which were responsible for the progressive *structural failure*.

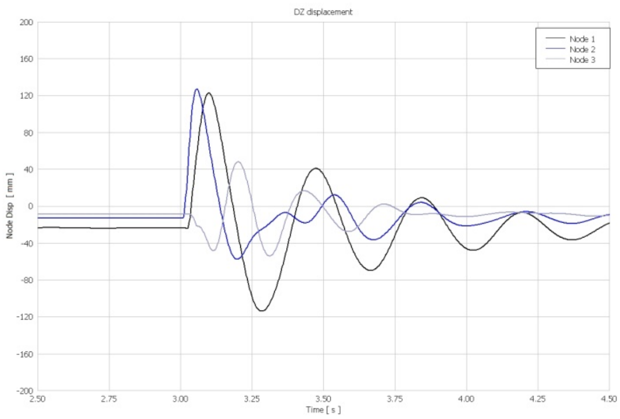

Figure 7: Vertical displacements (TNT = 15 kg).

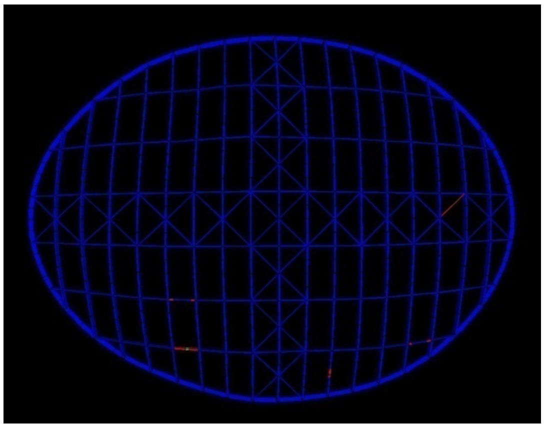

Figure 8: Localized plastic hinges (TNT = 15 kg).

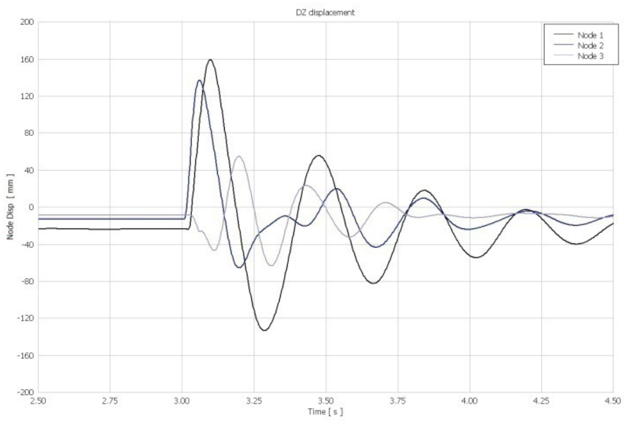

Figure 9: Vertical displacements (TNT = 20 kg).

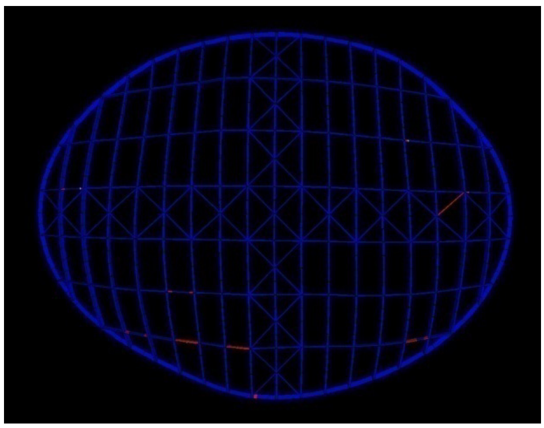

Figure 10: Localized plastic hinges (TNT = 20 kg).

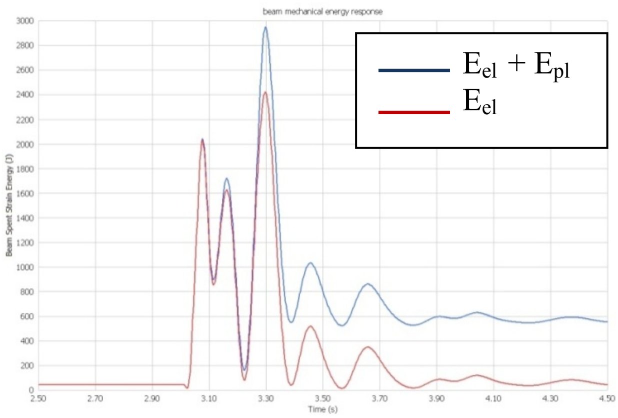

Figure 11: Energy dissipation of beam attached to Node 2 (above the detonation point, TNT = 20 kg)

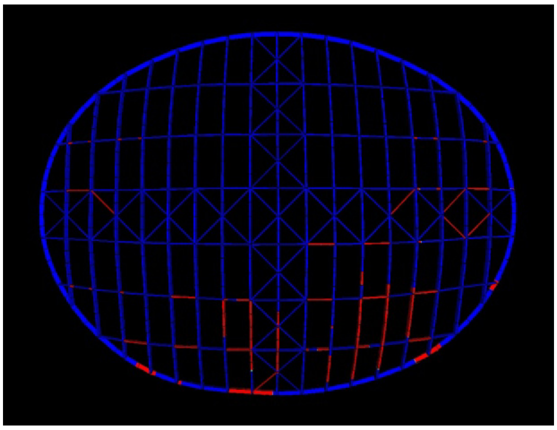

Figure 12: Localized plastic hinges (TNT = 25 kg).

6 Conclusion

A non-linear analysis (geometrical and material) was conducted of a steel and glass domed skylight subjected to an explosion. The glass was considered shatterproof, as required by the technical specifications of the construction in progress, and the steel was S355J0 alloy. Also, a hypothesis was made and quantified, in accordance with the American Manual TM5-1300 [1], for explosions caused by terrorist activities characterized by 4 different quantities of *trinitrotoluene* (TNT), specifically 5, 10, 15, and 20 kg. From the analysis of the non-linear response, the following results were noted:

a) the use of S355J0 steel reveals that the structure reaches a plastic deformation phase following an explosion caused by TNT ≥ 15kg;
b) nevertheless, the system seems to meet the blast-resistant performance requirements up to TNT = 20kg;
c) it was possible to identify a chronological series of plastic hinge formations for the skylight and the ring beams, and the relative degree of stiffness loss in the sections;
d) following additional analysis performed with a hypothesis of TNT = 25 kg, it was possible to obtain an indication (albeit a partial one) of the possible path of plastic hinges in the structure and, consequently, it was possible to hypothesize several possible structural subparts responsible for failure when TNT > 20kg;

When comparing the *non-linear elastic-plastic* response (energy dissipation) with the *non-linear elastic response* of material (no energy dissipation), it was possible to make the following conclusions regarding the dissipative capacity of the system:

e) the plastic behaviour of the system during the post-critical phase highlighted dissipative behaviour (for TNT $\leq$ 20kg) of a strictly localized type, therefore with reduced overall energy dissipation. This was claimed in the event of an explosion (impulsive energy release), the nature of which was unable to trigger cyclic hysteretic mechanisms, and with the reduced structural and non-structural mass of the skylight system characterized by reduced kinetic energy during the oscillation phase.
f) plastic deformation occurred, according to the behaviour of similar architectural systems, at the beam-beam connection (nodal points). It is therefore necessary to conduct a detailed study of the connections and relative non-linear properties.

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