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STRUCTURAL STABILITY STUDY AND DESIGN OF STEEL-GLASS COMPOSITE STRUCTURES UNDER EARTHQUAKE ACTIONS

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Abstract.*The present paper studies the structural behavior and the necessary design aspects of steel and structural glazing composite buildings in areas with high seismic risk. In recent years, the great progress in the design of specialized types of glass structures reinforced with metal frames due to their high architectural impact created the need of additional design requirements against seismic action. A structural glass system when exposed to intense seismic conditions could experience significant damage that would reduce its bearing capacity and downgrade the life safety level. Thus, the enhancement of the stability and the structural performance of such structures subjected to seismic actions could lead to an optimal configuration of the composite steel and glass structural system. In the present study individual design specifications according to the Eurocodes Framework and Normative Special Guides are assessed, and a methodology to evaluate the earthquake resistance of composite buildings with structural systems of steel and structural glazing is proposed. This methodology is applied to a case study and in particular, to the design of an Orthodox Church constructed with a load bearing system of steel and glass. The overall system of the building has been numerically simulated using SCIA Engineer Software by a complex 3-D FEM model. This model includes all the structural steel frame elements as well as multiple laminated glass shell elements. The principal focus of the study is the structural stability of the system considering the interaction of glass facades with the steel elements during intense seismic conditions, taking into account specifications for the resistance of a single structural glass member according to the CNR-DT 210/2013-Guide. Applying the proposed method an appropriate percentage overlay could be achieved by choosing the right material distribution of structural glass that leads to a safe increase of the load-bearing capacity of the steel members, leading to less material usage of steel and increasing the transparency of the building.*

1 INTRODUCTION

Technological advances during the last decades in Structural Engineering technology have given rise to radical changes in modern constructions and to architectural ideas that propose extended glass surfaces and increased transparency. The availability to use high-strength building materials, such as steel and aluminium, helps in many cases to install a strong load bearing system to support without failures larger than in the past, glass structural surfaces. Buildings of this type in the past was limited worldwide such as are the San Sebastian Church (Manila –Philippines, 1891) and the Palacio De Crystal (Madrid-Spain,1883) [11]; however, nowadays steel-glass constructions are popular in modern architecture for a wide scale of special structures and important public buildings. In addition, a demand for lighter and more transparent buildings, allowed larger opening for a stunning expansion into the steel - glass applications by assembling innovative load bearing support systems. Instead of trying to disappear or hide the bearing

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supporting system, the design of a hybrid structure composed of glass and metal frame could optimize the arrangement of load bearing elements increasing the free glass surfaces.Glass is an elastic and brittle material without any capacity for plastification [1], but steel resistance to deflection can overcome the brittle behavior of glass under intense tensile stress; by this property a hybrid system of the two materials could provide resilience and sustainability. The steel lateral buckling resistance [3] is different than the respective of glass, being a thin surface member that demands direct interaction with the supporting frame of the structure. Therefore, it is necessary to study the applicability of the Eurocodes design rules [2], [4] for steel structures and the respective European Normative special Guidelines for structural glass [6] on a steel-glass composite structure, and in particular, the guidelines that refer to the resistance against extreme wind action and seismic hazard.

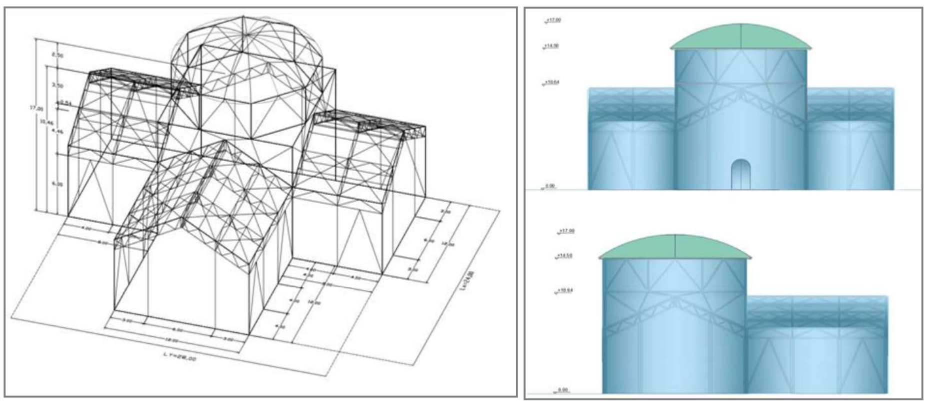

Figure 1. Sketch of the case study (A steel-glass Orthodox Church).

The herein proposed research study is illustrated by means of a design example of a memorial building located in an extreme seismic area using innovative design methodology for a composite structure made of steel and glass elements. This case study concerns an Orthodox Church composed by a hybrid load bearing system of steel and multi-laminated glass elements. The steel-glass Church under study is located in the city of Thessaloniki, Greece, that is an area with remarkable wind and seismic risk.

2 STRUCTURAL OF STEEL SUPPORTED GLAZING SYSTEMS

2.1 Glass structural response

Glass is a very hard material, with strength values similar to those of steel. The thermal conductivity of glass is very low 0.8W/m°C and closer to 0.6W/m°C of the water. The special heat of glass justifies its use as an efficient building material. This parameter is about 0.85-1.0KJ/kg°C, compared to 0.5KJ/kg°C for steel and 4.19KJ/kg°C for water. It is therefore obvious that glass retains heat to about the same degree as other thermal insulation materials. Glass in relation to ductile materials such as steel and aluminum, shows a completely different behavior under the influence of tensile stress as a brittle material because it deforms linearly elastic until their break point. The brittle fracture of a glass section starts in a direction approximately perpendicular to the direction of an applied tensile stress near to 120MPa.From the comparison of the stress-strain relations of steel and glass (see Figure 2), it can be understood that in relation to steel, glass section has the 1/3 of their strength, but in much greater strain. Thus, an optimal design for a steel glass structure leads to a design with strain for the glass structure equal to the limit of the elastic strain of steel (yield point).Glass sections can be deformed due to bending or lateral buckling in their elastic range and returns to its original position after the end of action; due to this property glass is particularly appropriate in relation to the supporting metal frame which also responds mainly elastically.

The fracture limit of glass does not only depend on their material properties, but also depends on the surface compression tendency, the connection between multiple layers and the protection layer of the surface (protective coating etc.). A characteristic case is the synthetic safety glass (Triplex), where the poly-vinyl-butyl (PVB)

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intermediate layer acts as a shear bond and increases its bearing capacity.

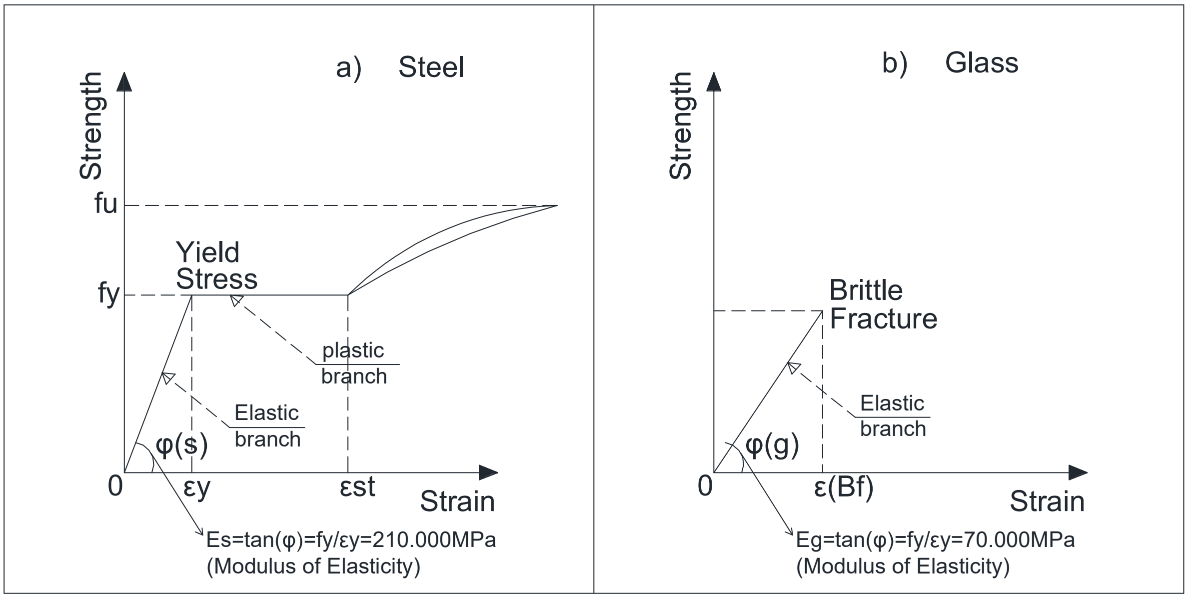

Figure 2. Stress strain relations of a) steel and b) glass.

Semi-structural glazing systems could be used for composite of steel and glass structures. The structures with such systems must be designed and simulated to be totally constrained at the corners of a single glass panel by four Spider point-fixings and at the edges connected by a secondary light metal frame, along the respective edges of the glass area unit. In many cases the role of the light metal frame in the longitudinal edges of the glass unit plays continuous secondary beams of the whole steel frame [10].For the analysis in **Phase 1-PreBreakege Behavior**, taking into account initial imperfections and different type and duration of loadings, the glass surface elements were modeled by their effective thickness, in accordance with EN 1991 - Eurocode 1 - Actions on structures and the more detailed European directive prEN16612/13-9.2.2. Deflection and stress calculations for all Serviceability Limit States (SLS) must be considered by taking into account an effective thickness of the whole multilayered glass of anominal thickness hi (for each layer) and a respective coefficient ω(which depend of the arrangement of layers), by the form (1):

$$hef,_{w} = \sqrt[3]{(\sum hi^{3}) + 12\cdot\omega\cdot(\sum_{i} hi\cdot hm,_{k})^{3}} \quad (1)$$

Here, a value of ω=0.3 for the respective coefficient could be used for structural glass which is exposed to extreme wind action. The main limitation for the maximum permissible deformations when we do not have special requirements is that it should not exceed as value W_d=(Length/65(mm) or 50mm).Moreover, stress calculation for Ultimate Limit States(ULS) must be considered by taking into account an effective thickness of the single internal layer of a glass unit by the form (2):

$$hef,_{\sigma,j} = \sqrt{\frac{hef,_{w}{}^{3}}{hi+2\cdot hm,_{j}\cdot\omega}} \quad (2)$$

An Important part of the ULS checks, is the performance of glass plated structural units in in-plane compression, in-plane shear and in lateral torsional buckling using the foregoing calculated values for the thickness of the glass unit or layer. As different values of the intermediate layer shear modulus were used according to the considered action, i.e., its characteristic duration and the working temperature. Therefore, it is necessary to calculate the effect of the actions separately, and then to combine them. On the contrary, for the analysis in **Phase 2-PostBreakege Behavior,**

initial fractures that converts the type of section, as well as different type of loading, in accordance to [9]§3.1-CNR-DT 210/2013, could be contributed in the calculation of deformation and stress, for Collapse Limit State (CLS), according to the effective thickness calculated by the form (1) of the remaining intact glass panel.
The calculation of the design strength is carried out according to the technical instructions prEN16612-8.2.1, by the form:

$$f_{gd} = k_{mod}\cdot k_{sp}\cdot(f_{g,k})/\gamma_{MA}+k_v\cdot(f_{b,k}-f_{g,k})/\gamma_{Mv} \quad (3)$$

where for a design time duration of loading t, k_{mod}=1.0 for wind, 0.44 for snow and $k_{mod-min}$=0.25, γ_{MA}=1.80 is the material coefficient of glass, k_{sp}=1.0 is coefficient of roughness of glass surface, $f_{g,k}$=45MPathe tensile stress of glass, $f_{g,v}$ the tensile stress of prestress glass (if it is used prestressed glass unit).

2.2 Composite Steel Glass structural design

A composite load-bearing system of steel and glass corresponds to a system of bearing mainly vertical loads. As glass can break suddenly due to accidental causes without a general design error, it should be designed and analyzed both as a standalone element and as an associated element to the support metal frame. To carry out a comprehensive analysis of a composite system of glass and steel members, the Eurocodes framework do not fully cover this case as they do not yet contain design rules for glass. Therefore, technical European and country guides and provisions [7], [8] must be applied. However, the general design principles of [5] EN1990 can be taken into account, as well as further Eurocodes provisions for the application of loads and their combinations, suitable for the steel-glass system. Therefore, for the implementation of Eurocode, different categories of construction materials such as glass in the case under consideration should be taken into account in advance and classified in the above reliability classes based on the probability of failure. For the purpose of reliability differentiation, consequences classes (CC) of glass could be established by considering the consequences of failure of the structure. Such classification into categories of special effects for glass structures could be:
• CC1: High consequence for loss of human life and structures where social or environmental consequences of failure are low. There are rather small consequences when the glass unit fails.
• CC2: Medium consequence for loss of human life and structures where social or environmental consequences of failure are medium. There are medium consequences when the glass unit fails.
• CC3: High consequence for loss of human life and structures where social or environmental consequences of failure are high. There are serious consequences when the glass unit fails.
In the case of composite of steel and glass buildings, the loading combination using [5] EN1990 and the respective limit states using at least CC2 reliability class for glass must be introduced by different coefficients ψ_{2i}according the European technical guides. This difference expresses the percentage of the characteristic value of an action such as wind, which, for the considered limit state, has a high probability of temporal identification with other actions.

2.3 Steel structures including load bearing glass members under seismic action

Steel and metal buildings whose load-bearing structure contains glass elements, must be designed against seismic risk so that the glass structure reacts without breaking and without any exceedance of the allowable stresses at the seismic load combination. The interaction between glass structures and the steel frame structure of the building must always be considered, along with the local behavior of the glass elements taking into account the respective actual deformation.
A nominal technical life cycle equal to a conventional lifetime of V_N=50 years, lead to the importance classes periods I, (V_R=35 years), II (V_R=50 years), III (V_R=75 years) and IV (V_R=100 years), according to the Design Guidelines, Construction and Control of Buildings with Structural Glass Elements of NRCIAC [9]. Moreover, the adoption of an assessment objective with a probability of exceedance of the seismic action generally leads to four different performance levels of probability of exceedance such as SLO serviceability level, SLD Limited damage level with, [SLV] life safety level and [SLC] collapse prevention level. The combination of the two above assessment categories lead to an assessment of final performance levels as [ND] No Damage, [SD] Slight Damage, [HD] Heavy Damage and [F] Failure respected to a return period (see Tab. 1), which can assess the seismic performance of a steel glass structure under consideration.

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Level	Importance Class			
PL	I	II	III	IV
SLO	-	-	$ND_{45\ years}$	$ND_{60\ years}$
SLD	$SD_{35\ years}$	$SD_{50\ years}$	$SD_{75\ years}$	$SD_{100\ years}$
SLV	$HD_{333\ years}$	$HD_{475\ years}$	$HD_{713\ years}$	$HD_{950\ years}$
SLC	-	-	$F_{1463\ years}$	$F_{1950\ years}$

Table 4:Seismic assessment objectives of a steel glass structure

On the other hand, the known process of modal response spectrum analysis using EN1998 for regular buildings or the performance based method of limitation of interstorey drifts for non-regular buildings, could be taken under evaluation to cover all possible seismic design cases.

3 NUMERICAL ANALYSIS MODELLING

3.1 Detailing Conditions and action analysis

The analysis about the structural performance against wind and seismic hazard of a composite steel – glass structure is illustrated in the following case study. This case study concerns a known model of an Orthodox Church located in the city of Thessaloniki, Greece, composed with a hybrid load bearing system of steel and multi-laminated glass elements. The structure modeled by using Scia Engineer software, based on the known architectural plans of the Church, having total dimensions Ly=28m, Lx=24m, H=17m as shown in Figure 3. The structural system constituted by the main part of the steel frame and covered by glass surfaces both on the sides and the roof with a total height of h1=10.46m and a central dome part made of steel shell system and covered of Nordic Green cooper sheets that provide durability and aesthetic uniformity with the main glass surfaces. The whole structure can transfer vertical loads from the roof to the ground via the columns and as well the horizontal forces via the inverted V shape vertical bracings. As a first part of the typical design methodology for a steel structure like this based on the Eurocodes is the composition and pre-dimensioning of the steel structural system. That led to acceptable cross sections and in a second part determined the required dimensions of vertical and horizontal glass panels.

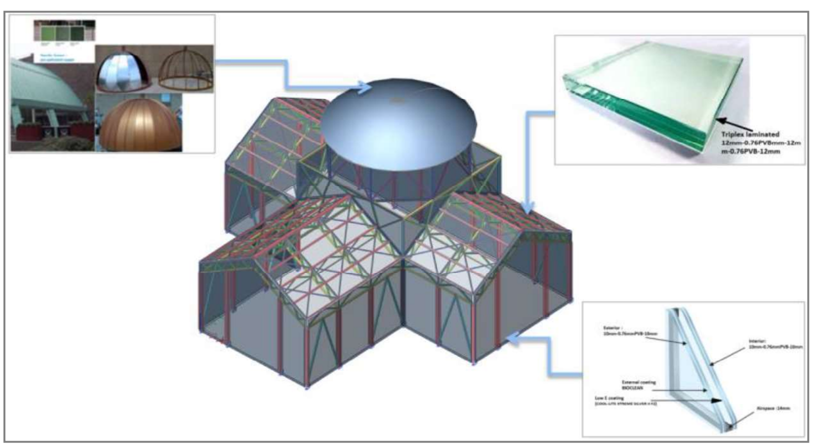

Figure 3.On the 3D model of the case study (Orthodox Church with glass shell elements).

At this point, it was proposed for the wall glazing, to use double insulated units of laminated glass panels (inner panel:10-0.76PVB-10mm,cavity:14mm Argon, outer panel:10-0.76-10mm), which combined with special low E-coating can offer excellent conditions of thermal and sound insulation, necessary for crowded memorial buildings. On the contrary, the glass rooftop was decided to be constructed by triplex laminated glass panels (12-0.76 PVB -12-0.76PVB-12mm), that could ensure the strength resistance of the glazing, both at strong transverse loads and horizontal seismic loads, as well as durability in long-term failure and safety in cases of extreme loading or breakage of the inner layers.

A regular PVB-laminated glass units of the insulating wall glazing was designed of 4×2m and simulated by 20-node solid shell elements of tempered glass with three layers of corresponding effective thickness for (inner panel, cavity, outer panel) and examined in both (Phase1-Pre-Breakage behavior+ Phase2-Post-Breakage behavior).The software used automatically formulates the self-weight of the structural components by using the density of steel members and glass, whereas the variable surface loads are applied as pressure distributed correctly on each glass panel of the double wall insulating glazing system. As far it concerns **Phase 1-PreBreakege Behavior** , glass panels were investigated as intact in order to be able to transfer **a)** its own G-self weight as vertical load for each panel of0.508kN/m^2,**b)**p_{w3sec} transverse wind loads of 3-second peak kinetic pressure 1.32kN/m^2, applicable for short term loading of 5 sec during wind actions, according to the technical regulations [9], §4.5.2 CNR-DT 210/2013),**c)**p_{w10min}transverse wind loads kinetic pressure of 0.60kN/m^2 averaged over 10 minutes, applicable for long term loading of 10 min during wind actions **d)**transverse internal isochoric pressure of insulating glass units 0.204kN/m^2(§4.8 CNR-DT 210/2013).Although, in the **Phase 2-PostBreakage Behavior**, where there is the case of failure, the laminated panel had to be able to retain its integrity, thus were considered and tested one fractured panel and one intact, under **a)** the same own G-self weight **b)** $p_{w10'3sec}$ transverse wind loads of 3-second peak kinetic pressure 1.076kN/m^2(with a different return period TR = 10 years, §8.2.5 CNR-DT 210/2013) , **c)**p_{w10min}transverse wind loads kinetic pressure of 0.49kN/m^2 , **d)** same p_o internal isochoric pressure.

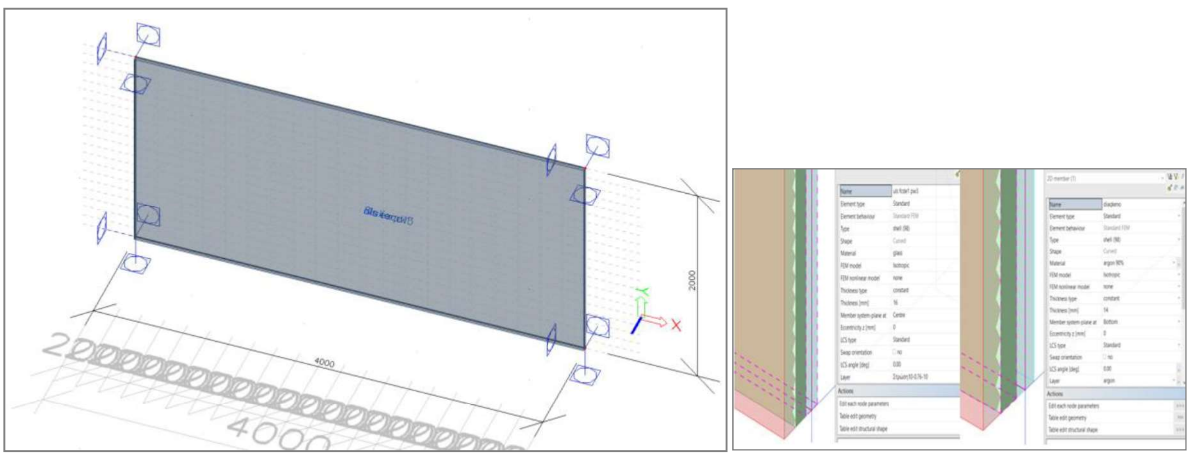

Figure 4.3-D Finite Element Model of a double insulating wall glazing system

A Triplex laminated glass unit of the roof glazing was simulated by 20-node solid shell elements of heat strengthened glass and designed as an orthogonal glass plate of 4×1.73m with its corresponding effective thickness and examined also in both phases of Pre-Breakage and Post-Breakage behavior. In that case, the horizontal glass panels were designed to receive in **Phase 1-PreBreakege Behavior: a)** its own G-self weight as transverse load of 0.92kN/m^2 , **b)** pw_{3sec} wind loads of 3-second peak kinetic pressure 1.28kN/m^2 , **c)** pw_{10min} wind loads kinetic pressure of 0.58kN/m^2 averaged over 10 minutes, **d)**S-snow load 0.67kN/m^2 for a conventional load duration of 3 months, **e)** Q live anthropic load (maintenance) for a conventional load duration of 30 seconds of 2kN/m^2 that is distributed over an area of 50 × 50 mm, at the midpoint of the shorter not supported edge of the panel. Though in **Phase 2-PostBreakage Behavior**, the analysis referred to the possibility of getting fractured one layer of the Triplex glass element and being modified to a simple laminated. As a result, changing the loads that could transfer, especially: **a)** its own G-self weight converting to 0.61kN/m^2**b)** $pw_{10'3sec}$wind short term loads 1.04kN/m^2(with a different return period TR = 10 years, §8.2.5 CNR-DT 210/2013) , **c)**pw_{10min}wind long term loads of 0.47kN/m^2**d)**S΄-snow load 0.785kN/m^2 (for a different return period n=10 years according to EN1991-1-3,Annex D).

Roof glazing designed and simulated to be constrained at intervals by Spider point-fixings on their longest edge, in order to be connected and supported long wisely by the purlins (see Figure 5). The point fixing holes, of the connections, have diameter 36.5mm and distances 177mm in x-x and y-y from the corners, allowing the connection with the horizontal elements. The practical impact on the roof glass panels supported and hinged only in 2 edges of the 2Dshell element. On the contrary, the roof glazing of the dome construction as it obtained triangular glass panels supported by horizontal steel members on all three edges had to be examined as equivalent orthogonal glass plate that was fully hinged on all four edges. Consequently, we could conclude that both horizontal and vertical glass panels could be examined as main structure with CC2 -class consequence.

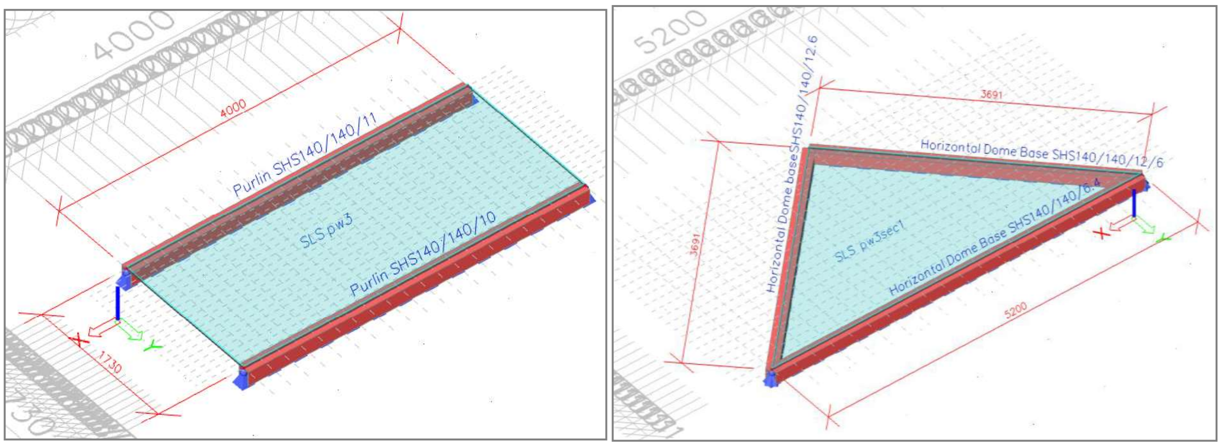

Figure 5. 3-D Finite Element Model of a Triplex Laminated Roof glazing system

At that point was obvious, the need for separate review of design strength and maximum deflection and for each type of load, since the application leads to different glass sections and results.

3.2 Evaluation of results

For the evaluation of maximum stresses and deflections of glass shell elements for SLS, ULS and CLS, the results of a 3D finite element model of the whole structure conducted with similar results , using Linear FEM for single glass panel models (see Figs. 6, 7), in respect to design limits based on the European directive prEN16612 and the technical guide CNR-DT 210/2013.For the wall glazing, structural analysis results, proved as where the dominating variable action, the critical value of the wind load w, whereas self-weight was carried from the point fixings, leading to an optimal bearing capacity of the system as shown in Table 2.

	Maximum deflection at the SLS(mm)	Maximum Stress at the ULS (MPa)	Maximum deflection at the CLS(mm)	Maximum Stress at the CLS (MPa)
FEM Method	$6.38<\mathbf{d}=14<\mathbf{w}_{max}=25.28$	$9.84<\mathbf{fd}_{w3sec}=87.5$	$4.2<\mathbf{d}=14<\mathbf{w}_{max}=25.28$	$16.33 <\mathbf{fd}_{w3sec}=87.5$
3D Analysis	$7.6<\mathbf{d}=14<\mathbf{w}_{max}=25.28$	$7.1<\mathbf{fd}_{w3sec}=87.5$	$5<\mathbf{d}=14<\mathbf{w}_{max}=25.28$	$7.8<\mathbf{fd}_{w3sec}=87.5$

Table 2: Maximum stress and Maximum deflection of comparative results between methods for critical wind loads.

However, in roof glazing, since uniformly distributed loads caused by self-weight and snow acted on the whole plate, deformed into an almost cylindrical surface. As it was observed, the stress distribution, was uniform at the generators parallel to the longer edges of the plate and in combination with an increase of stress at the edges due to the Poisson effect, resulted to the shorter edges (not supported) of the plate become the most stressed. On the other hand, live service load acted on a reduced area, 50x50 mm that did not really affect the plate behavior. Consequently, also in that

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case, the most critical variable action emerged the wind load and especially when applied in the dangerous position of midway along the free edge, causing maximum stress and maximum deflection in this area as in Table 3.

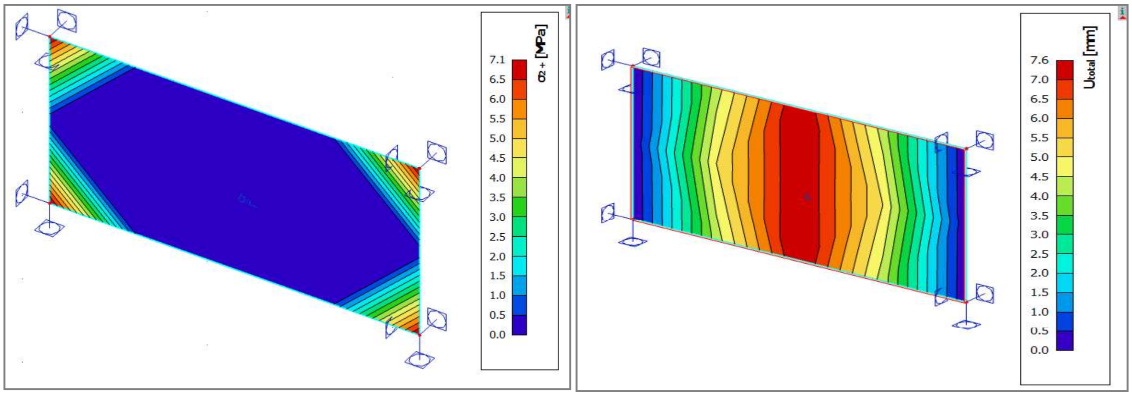

Figure 6.Maximum stress and maximum deflection of double insulating units for critical wind loading

	Maximum deflection at the SLS(mm)	Maximum Stress at the ULS (MPa)	Maximum deflection at the CLS(mm)	Maximum Stress at the CLS (MPa)
FEM Method	2.47<$\mathbf{w}_{max}$=40	6.38<$\mathbf{fd}_{w3sec}$=45.83	4.26<$\mathbf{w}_{max}$=40	10.06<$\mathbf{fd}_{w3sec}$=45.83
3D Analysis	0.8 <$\mathbf{w}_{max}$=40	6.7 <$\mathbf{fd}_{w3sec}$=45.83	1.4 <$\mathbf{w}_{max}$=40	10.05<$\mathbf{fd}_{w3sec}$=45.83

Table 3:Maximum stress and Maximum deflection of comparative results between methods for critical wind loads.

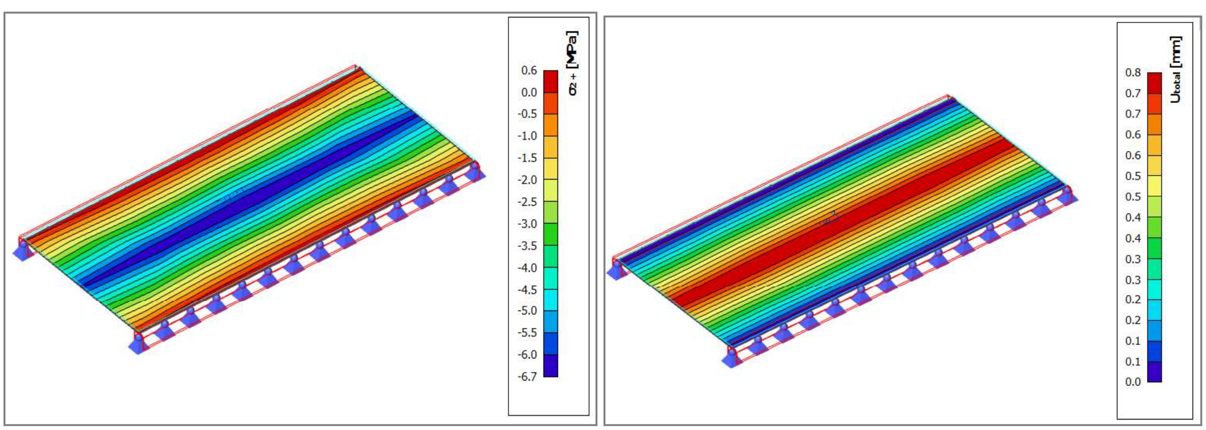

Figure 7. Stresses and deformations arrangement on a triplex laminated glass unit under critical wind loading.

On the one hand, has been attempted an optimal analysis to check the respective stresses and deformations of steel frame members in accordance to the standards of EN1993. In order to compare the effectiveness of the proposed methodology and values of the directives and technical guides must be considered for the glass elements the actual thicknesses, eccentricities and structural connection on the steel frame. With the known bearing capacity of the glass as an independent structure the next step is to proceed to the static and dynamic analysis of the complex 3-D FEM church model, as a unified system of steel and glass. Here a complete analysis of a composite glass building structure and their simulation with SCIA ENGINEER is taken under evaluation. Through the detailed calculated actions for each steel frame section it is observed that for the typical loading combinations at SLS and ULS they corresponded with smaller values of actions than in the phase of pre-dimensioning as members of glass panel envelope.

4 SEISMICDESIGNS

4.1 Seismic Conditions

As main part of the study, a complete seismic analysis is conducted for the evaluation of the seismic behavior of the composite steel – glass structure where the assessment of the combination of actions on glass elements following the known procedure using EN1998 for the seismic actions and [5] EN1990 for the Seismic combination compared with the respective by the special guide [6] §3.2.1 -CNR-DT 210/2013.

On one hand, they are considered seismic combinations, based on Eurocode 8 [G+0.3W±Ex±0.3Ey, G+0.3W±Ey±0.3Ex] where the partial factor ψ2 for quasi-permanent variable actions as is the dominating action of wind pressure w, or snow might be $\psi_{2,i}$=0 in accordance to the Annex A of EN 1990. Although, for the purpose of the present study, the horizontal wind action (as variable action), combined with a value for the partial factor $\psi_{2,i}$=0.3, during a possible seismic design state at the system of steel-glass elements as a composite structure.

On the other hand, in accordance to the Italian technical guidelines [9], CNR-DT 210/2013 proposed the seismic combinations of [G+0.6W±Ex±0.3Ey] and [G+0.6W±Ey±0.3Ex].As it is suggested for a country with higher seismic hazard, following this procedure is taken into account the extreme case where the structure is loaded simultaneously from strong wind and seismic actions, using in the seismic combination a higher partial factor ψ_{2i}=0.6 for wind. As it was evident, was taken two separate dynamic analysis cases with their corresponding seismic combination, under the same 3-D linear analysis model and with the same process in order to be comparable.

Adequate Seismic response of steel columns in x-x direction					
1.35G+1.5Q [kN]	G+0.3W [kN]	G+Ex+0.3Ey+Eccx [kN]	G+Ey+0.3Ex+Eccy [kN]	G+0.6W+ Ey+0.3Ex+Eccy [kN]	G+0.6W+ Ey+0.3Ex+Eccy [kN]
91	16.83	31.62	60	38.79	69.19
29.85	15.59	32.4	59.93	35.97	64.44
2.51	6.93	23.33	29.7	23.63	32.04
12.2	9.48	19.51	29.7	20.41	32.47
15.49	9.1	17.02	28.8	17.90	31.27
21.07	13.31	22.52	27.59	25.5	32.18
Adequate Seismic response of steel columns in y-y direction					
1.35G+1.5Q [kN]	G+0.3W [kN]	G+Ex+0.3Ey+Eccx [kN]	G+Ey+0.3Ex+Eccy [kN]	G+0.6W+ Ey+0.3Ex+Eccy [kN]	G+0.6W+ Ey+0.3Ex+Eccy [kN]
33.65	21.51	33.97	32.35	43.59	43.15
15.22	11.21	23.11	17.61	29.42	24.19
33.69	21.01	31.83	31.31	39.39	39.78
15.72	8.78	21.9	17.21	23.26	19.37

Table 5: Maximum compressive axial forces for the comparative loading senarios for the front columns in the seismic combinations

4.2 Seismic Analysis

Specifically, the steps that were followed is the calculation of eigenfrequencies (ω_i) and eigenmodes (φ_i) and also the calculation of generalized mass, their participation rate and the total moving masses. In conclusion, as observed in both case studies, the first and second fundamental eigenmodes were transport, so as to characterized the building as

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structural stable or torsional insensitive and with the same fundamental period of vibration T1=0.77sec. Moreover, based on the design spectrum, examined the distribution of the horizontal seismic forces and calculated the maximum responses for each seismic excitation. Additionally, had been inquired an estimation of second-order effects and of limitation of interstorey drift according to the provisions of EN1998.
Remarkable section of the research is the satisfactory design seismic response of steel members (see Tab. 5) which obtain an important role in the dynamic behavior of the building.

Obviously, the assessment of glass structural element's seismic behavior could not be dismissed from the study. In this procedure, was decided to follow the simplified method proposed in [6], CNR-DT 210/2013. According to which, based on the known seismic data, the elastic acceleration response spectrum SDe (T) =1.65 should be converted and expressed as a horizontal movement. Subsequently, the fundamental eigenmode T1=0.77sec, can be evaluated that the most important response spectrum expressed in displacement of Sd(T) by taking the threshold corresponding to the horizontal plateau for each of the limit states (SLO, SLD, SLV and SLC). Taking into account the results of this procedure, observed that are covered through all possible limit states and meet the most critical state of SLC, (Limit State of preventing Collapse). That leads to the single-stage oscillator, where Sde=dmax(G)=0.029m can be defined as the maximum design ground displacement and as the most critical and reference value dmaxG=dmax(SLC).

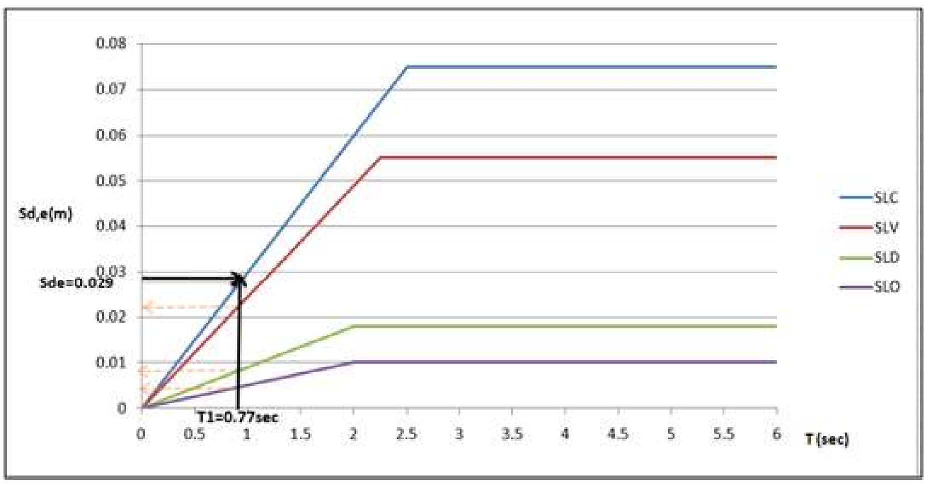

Figure 7. Graph of response spectrum in terms of displacement Sd(T) for SLO, SLD, SLV and SLC limit states depending on the respective eigenmode.

As a next step, using the ratio coefficients of the directives [6], are defined the critical maximum base displacement dmaxG = dmaxSLC and the maximum spectral inderstorey Drifts for all possible limit states. This seismic analysis concerned a composite of steel and glass deformed building, so the values of the marginal maximum ground movements from the single-stage, had to be transferred to the total multistage oscillator of the steel-glass structure. By that means, using the following expression could be calculated the maximum permissible displacements at the top of the frame for each limit state (see Table 6):

$$\mathrm{dmax}_{\mathrm{MDOF}} = \mathrm{dmax}_{\mathrm{G}} \cdot \Gamma \, , \Gamma = \left(\frac{3\mathrm{n}}{2\mathrm{n}+1}\right) \tag{4}$$

Additionally, the permitteable interstorey drift Dp (taking into account n=3 as a representative number of floors for this building) can therefore be estimated for each limit state too, by the following expression (see Table 6):

$$\mathrm{Dp} = \frac{\mathrm{dmax}_{\mathrm{MDOF}}}{n} \tag{5}$$

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Maximum permissible displacements	Permitteable interstorey drift Dp
dmax$_{MDOF}$(SLC)= 0.029·1.286=0.0383m=38.3mm	Dp-(SLC)=0.01276m= 12.76mm
dmax$_{MDOF}$(SLV)=0.021·1.286=0.0257m=25.7mm	Dp-(SLV)=0.00857m=8.57mm
dmax$_{MDOF}$(SLD)=0.0064·1.286=0.0077m=7.7mm	Dp-(SLD)=0.00256m=2.56mm
dmax$_{MDOF}$(SLO)=0.0025·1.286=0.0032m=3.2mm	Dp-(SLO)=0.00107m=1.07mm

Table 6: Critical maximum permissible displacements and Permitteable intersorey drifts for several limit states.

Having calculated the maximum allowable seismic displacements of the entire structure for each possible limit state, is possible to require from the results of the computer program the maximum displacements u_x, u_y for both seismic combinations under consideration, 1) G+0.3W±EY±0.3EX and 2) G+0.6W±EY±0.3EX and evaluate the behavior of 3D glass-steel system. Unexpectedly, the maximum displacements, according to the analysis of EN1998, appeared $u_{y,max}$=36.7mm on the critical position 1, for the main glass panel at the location of the central entrance, concerning about transversal earthquake in direction Y(see Figure 8), while in the context of the directive CNR-DT 210/2013 was $u_{y,max}$=40mm.

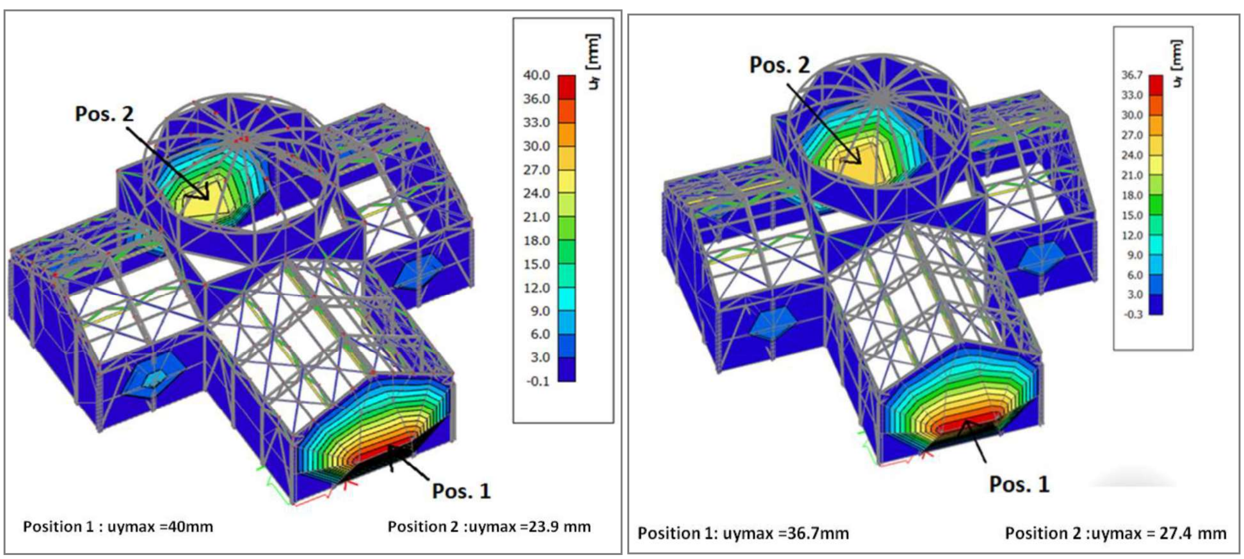

Figure 8. Displacements u_y on the steel-glass structure for 1) G+0.3W+EY+0.3EX (left) and 2) G+0.6W+EY+0.3EX (right).

In Figure 9, are shown the maximum horizontal displacements of the 3D glass church, on the one hand in the seismic analysis of EN1998 leaded to SLV state and on the other hand, the seismic analysis of technical guide with the extreme case of seismic action combined with stronger wind leaded to SLC.

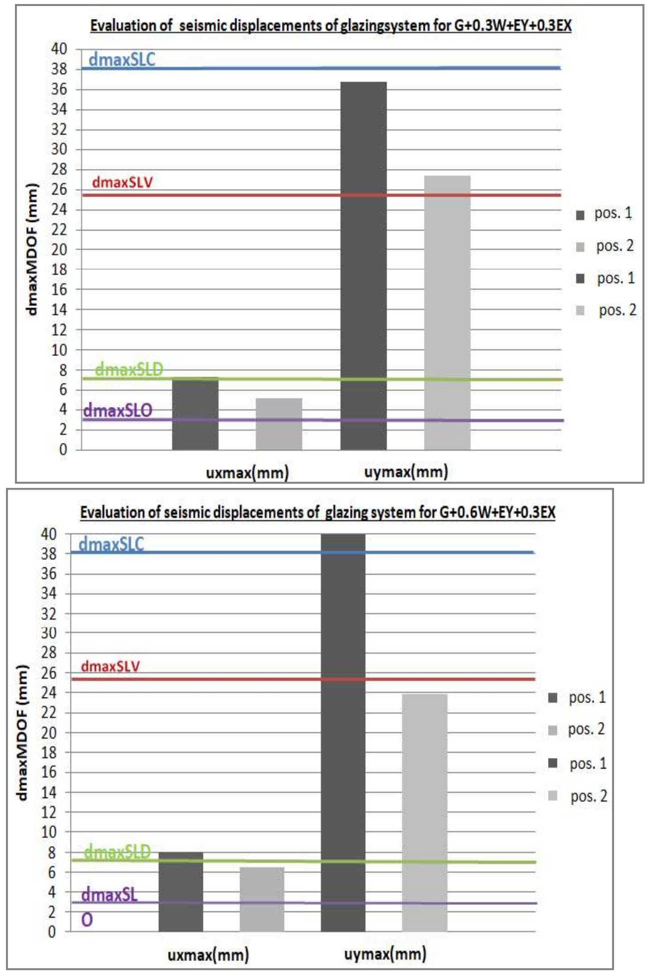

Figure 9. Comparison of the maximum displacements of the glass panels as primary elements of the structure.

Following the proposed evaluation method of the directive for the two possible seismic scenarios and for a building structure of a church, it is estimated that the required level of efficiency according to EN1998 for SLV is the emerged Heavy Damage (HD).Which although, there is some operational damages in glass members there is no risk of breakage and falling of materials that will cause risky situations, offering enough bearing capacity to the glazing system against seismic conditions. On the contrary, in accordance with standards of CNR-DT 210/2013 for SLC observed to be set to Failure performance level. Where the glazing system and the frame supported structure are severely damaged and show extensive signs of failure, with a high enough risk that any material fall will be more dangerous, meaning not at all bearing capacity at extreme seismic conditions. Consequently, the openings would necessarily be reduced and by using smaller individual glass panels, the load-bearing capacity will be increased and could allow to more acceptable and safe seismic response displacements. Taking into consideration, although, the worst case scenario of SLC and level performance of failure and construction class 3, is estimated reference period of VR=75years and return period for that extreme seismic condition of TR=1463 years with probability of exceedance PVR =5%.

11 CONCLUSIONS

As the glass is a structural material when used in building structures in load-bearing elements does not offer a functional separation of the frame from the total steel frame, the cladding of the structure must be examined as a composite system of structure. A study in such large-scale composite buildings is a complicated task and in particular, when this concerns the cooperation of two different materials such as steel and glass. Therefore, for the design and analysis of composite steel and glass structures it is particularly useful to perform dimensioning and evaluation of glass elements firstly as isolated parts on a unit scale.

In the design of glass components, there is a lack of design provisions and principles in the references of Eurocodes. As a result, there is a need of an extensive analysis of complicated steel glass structures by using individual European provisions, technical instructions, and manuals for the implementation of the correct pre-dimensioning of glass sections. Although, by combining the applied method and the design principles proposed by the technical directives and the annexes of the European Provisions. It is evident there is necessity for parallel pre-dimensioning of the main steel frame components and the respective of structural glass, as they are linked and interacted in the buildings. The results from this study of the composite of steel and glass study using an non-regular structure such as a church showed that the cross-sections that obtained during the pre-dimensioning phase of the steel frame, occurs to be significantly reduced when considered together with the glass envelope. More specifically, the actual stresses in compression, tension and the interaction of bending and compression of the glass are doubled in the pre-dimensioning phase of the glass, but to the extent of not reaching failure. Therefore, if an optimal combination of the two materials is done in terms of quantities on the building taking into account the architectural requirements can be obtained a sufficient bearing capacity for all members.

This investigation could ideally lead, to less steel building material and more glass element, to lighter constructions and break down the transparency limits. However, this process of finding the minimum required final cross sections, due to the interdependence of the glass-steel system is a time consuming and laborious process and must be done at the highest possible level of performance. Although, for the design of the glass shell elements as assessed, technical instructions and special provisions performed most critical resistances than the using Eurocodes framework for the steel part of the structure.

For the evaluation of the glass system's seismic movements on the one hand by the seismic combination of EN1990 (G+0.3W±EY±0.3EX), when examined with the proposed methodology of the technical directive [6], we are led to the desired SLV limit state with a level of performance of Heavy Damage. On the other hand, the assessment of seismic movements during the extreme seismic action of the special Italian directive [9] CNR-DT 210/2013 (G+0.6W±EY±0.3EX), has led to a level of performance Failure, where we do not have the adequacy of glazing systems under an earthquake action. This means that serious damage is expected, extensive signs of failure, thus degrading the level of safety. Consequently, the distances between steel members would be reduced and by using smaller individual glass panels, the load-bearing capacity will be increased and could allow to more acceptable and safe seismic response displacements as the respective by the classic procedure according EN1998.

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