Time history analysis of a tensile fabric structure subjected to different seismic recordings

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Abstract. The structural behavior of a tensile fabric structure, known as hypar, is investigated. Seismic-induced stresses in the fabric and axial forces in masts and cables are obtained using accelerograms recorded at different regions of the world. Time-history analysis using each recording are performed for the hypar by using finite element simulation. It is found that while the seismic stresses in the fabric are not critical for design, the seismic tensile forces in cables and the seismic compressive forces in masts should not be disregarded by designers. This is important, because the seismic design is usually not considered so relevant, as compared for instance with wind design, for these types of structures. The most relevant findings of this study are: 1) dynamic axial forces can have an increase of up to twice the static loading when the TFS is subjected to seismic demands, 2) large peak ground accelerations seem to be the key parameter for significant seismic-induced axial forces, but not clear trend is found to relate such forces with earthquakes and site characteristics and, 3) the inclusion or exclusion of the form-finding in the analysis procedure importantly affects results of seismic stresses in the fabric, but not in the frame.

Keywords: tensile fabric structure; time-history analysis; finite element simulation; form-finding; seismic-induced forces

1. Introduction

Tensile fabric structures (TFSs) are used in construction for spanning large lengths while allowing impressive architectonical designs. However, they are challenging structures for engineering design, since their behavior is highly non-linear, their material properties may change in different directions, they exhibit wrinkling when subjected to compressive loads (i.e., they only have tensile stresses), among other issues. Conceptually, the beginnings of TFSs are referred to shapes of equilibrium unattainable with computational methods; so, authors as Gaudi from Barcelona, Heinz Isler from Switzerland and Frei Otto from Germany are cited in the literature for prestressed and hanging structures before the advent of computer-based methods (Linkwitz 1999, Bletzinger and Ramm 1999). Finding the shape of a TFS can be considered as an inverse engineering problem (Bletzinger and Ramm 1999), in which the usual methodology to find a stress field from a given section under prescribed loads is inverted, i.e., for a given stress field the geometry of a TFS is found, so that it satisfies the prescribed stresses; however, not all the shapes are executable in practice or desirable for architectonical interests. Currently, computer-aid methods are used to find the desired shapes of TFSs with the so-called form-finding approach. Form-finding procedures can be divided in three main families (Veenendaal and Block 2012), which are force density methods, dynamic relaxation methods and stiffness matrix methods. The development of the force density method is linked to the project for the Olympic Stadium of Munich 1972 in the literature (e.g., Linkwitz and Scheck 1971, Scheck 1974, Linkwitz 1999). Dynamic relaxation methods are listed by Wüchner and Bletzinger (2005) and include the studies by Barnes (1999), Wakefield (1999) and Wood (2002). Veenendaal and Block (2012) refer to the stiffness matrix methods of Haug and Powell (1972), Argyris et al. (1974) and Tabarrok and Qin (1992).

The stiffness matrix method, which is also implemented in commercial software and recommended for analysis and design (Huntington 2013), is used in the present study. It is noted that when mechanical approaches based on the virtual work of surface stress fields are considered, at the stage of form-finding the material specifications are not important, since the problem is to find a geometry which results in equilibrium for a given stress field, regardless of how such field was generated (Bletzinger and Ramm 1999); in the
form determination stage, the response of the structure is not very dependent on employed stress-strain functions, Huntington (2013).

The impact of the form-finding procedure in the seismic analysis of TFSs is not reported in the literature. In fact, in general there are scarce studies of TFSs under earthquake-induced loads; nevertheless, Valdés-Vázquez et al. (2019) report a parabolic hyperboloid (hypar) formed of prestressed fabric and cables, over a supporting frame of cables and masts, subjected to an Italian seismic record. To inspect the seismic response of other TFSs is desirable, especially if different recordings worldwide are used, so that possible differences can be investigated. This can be achieved by using an adequate finite element method (FEM) which allows the time-history non-linear analysis of TFSs.

The main objective of the present study is to investigate the structural behavior of a TFS (a hypar) on a supporting frame when subjected to seismic loading by using time-history dynamic analysis with the aid of finite element simulation. The effect of the form finding, as well as that of the different recordings on the results, is reported.

2. Theory

The finite element simulation is based on the theoretical background briefly described in the following. The membrane theory used in this work is based on vectors and tensors using a curvilinear coordinate system, where Greek indices on membrane middle surface take on values of 1 and 2 in a plane stress state in Euclidean space, as shown in Fig. 1. The position vector \( \mathbf{X} \) on the membrane in the reference configuration \( \Omega_0 \) is defined by

\[
\mathbf{X} = \mathbf{X} (\xi^1, \xi^2)
\]

while its corresponding position vector \( \mathbf{x} \) in the current configuration \( \Omega \) is given by

\[
\mathbf{x} = \mathbf{x} (\xi^1, \xi^2)
\]

With these position vectors, the covariant base vectors on \( \Omega_0 \) and \( \Omega \) are defined respectively as

\[
\mathbf{G}_\alpha = \frac{\partial \mathbf{X}}{\partial \xi^\alpha}
\]

and

\[
\mathbf{g}_\alpha = \frac{\partial \mathbf{x}}{\partial \xi^\alpha}
\]

The covariant components of the metric tensors are expressed as

\[
G_{\alpha\beta} = \mathbf{G}_\alpha \cdot \mathbf{G}_\beta
\]

and

\[
g_{\alpha\beta} = \mathbf{g}_\alpha \cdot \mathbf{g}_\beta
\]

Using the above definitions, components of the Green-Lagrange strain tensor can be found using

\[
E_{\alpha\beta} = \frac{1}{2} (g_{\alpha\beta} - G_{\alpha\beta})
\]

Finally, with appropriate constitutive equations, the second Piola-Kirchhoff components \( S^{\alpha\beta} \) are obtained and virtual internal work \( \delta W^{\text{int}} \) in curvilinear coordinates is

\[
\delta W^{\text{int}} = \int_{\Omega_0} \delta E_{\alpha\beta} S^{\alpha\beta} d\Omega_0
\]

where \( \delta E_{\alpha\beta} \) stands for the virtual strain tensor. From the virtual internal work, the internal forces \( \mathbf{F}^{\text{int}} \) can be obtained and the dynamic equation to be solved yields

\[
\mathbf{F}^{\text{int}} + \mathbf{M} \ddot{\mathbf{u}} = \mathbf{F}^{\text{ext}}
\]

where \( \mathbf{M} \) is the mass matrix of the system, \( \mathbf{a} \) is the acceleration vector and \( \mathbf{F}^{\text{ext}} \) are the external forces given by the prestressed forces and seismic records; the recordings used in this study are described in detail later. More details of the formulation and discretization with the finite element method can be found in Valdes et al. (2009), where, for a particular direction \( i \) and node \( I \), the internal forces in tensorial form are

\[
f_{ij}^{\text{int}} = \int_{\Omega_0} B_{a\hat{a}ij} S^{\alpha\beta} d\Omega_0
\]

with the second Piola-Kirchhoff components \( S^{\alpha\beta} \) obtained using proper constitutive equations, and the fourth-order strain-displacement tensor given by
\[ B_{\alpha\beta ij} = \frac{1}{2} \left( \frac{\partial N_j}{\partial \xi^\alpha} x^h_{\beta i} + \frac{\partial N_i}{\partial \xi^\beta} x^h_{\alpha j} \right) \]  

(11)

where \( N_i(\xi) \) are the element shape functions and

\[ x^h_{\alpha,i} = \sum_{j=1}^{n_{\text{node}}} \frac{\partial N_j}{\partial \xi^\alpha} x_{ij} \]

(12)

is the discretized form for the derivative of the shape functions times the element nodal coordinates.

### 3. Geometry definition

The geometry used in this study is defined by a hyperbolic-paraboloid (hypar) of 8 meters of side in a plan view, as shown in Fig. 2, with a difference between its upper U and lower L corners of 3 meters in height. In this figure, the upper corner nodes of the membrane fabric structure are shown with filled black circles, while the lower corner nodes are depicted with empty circles. The lower fabric nodes in Fig. 2 are 4.5 meters high from the ground level, while nodes H for the frame have a height of 9 meters from ground level. Projected dimensions of the supporting structure in a plan view are also depicted in Fig. 2 (i.e., cables in blue and masts in red in Fig. 2 and Fig. 4). It is point out that the geometry in Fig. 2 corresponds to the TFS before the so-called form-finding procedure is performed.

The manufacture warp direction of the membrane is defined along the diagonal between upper nodes U of the fabric, while the fill direction is perpendicular to the warp direction (i.e., along the diagonal going from one L corner to the other L corner). The fabric is surrounded by a cable (yellow lines in Fig. 2). The membrane fabric and the surrounding cable are prestressed. As mentioned before, the hypar frame consists of cables and masts, whose material properties, together with those of rest of the TFS, are given in Table 1, where \( E_w = E_t \) is the tensile stiffness for warp and fill respectively, \( \nu \) is the Poisson ratio, \( t \) is the fabric thickness, \( \rho \) is the density, \( E_a \) is the elastic modulus for the surrounding cable with \( d_a \) as its diameter, \( E_m \) is the elastic modulus for the mast with a cross section defined by \( A_m \) and \( E_c \) is the elastic modulus for the frame cable with a diameter given by \( d_c \).

#### 3.1 Form-finding

In this work the direct stiffness method, also known as matrix stiffness method (Tabarrok and Qin 1992) was used for the form-finding. Since the minimum surface of the structure must be independent of the material properties, the assumed material properties for the membrane and surrounding cable are given in Table 2; the fictitious values of some properties in Table 2 (e.g., a modified modulus of elasticity, after Tabarrok and Qin, 1992) are used simply for convenience, so that the direct stiffness method can be applied in the form-finding stage, which does not impact on the finding of the minimum surface (Bletzinger and Ramm 1999, Huntington 2013). Material properties for the frame remain unchanged. Note that the self-weight of the structure is added in the form-finding.

The resulting deformed shape of the membrane fabric and surrounding cable is shown in Fig. 3, after the form-finding analysis is carried out. This deformed structure is taken as the initial geometry to perform the dynamic seismic analysis in the next section and it is shown in Fig. 4 in a 3D view.

### 4. Seismic records

Once the form finding analysis is performed, a dynamic time history analysis using accelerations of four recordings is carried out for the hypar in Fig. 4. The four seismic records to be used in the dynamic analysis are listed in Table 3, where the earthquake characteristics are also included. These records are briefly discussed and results of the dynamic analysis are described in the next sections, first for a record from an earthquake in Alaska, then for the remaining events.
The first considered record was obtained during the last of a sequence of upper crustal normal-faulting earthquakes which struck the Apenines in Central Italy (D’Amico et al. 2019, Civico et al. 2018). It was the Norcia October 30th, 2016 earthquake, with a moment magnitude, Mw 6.5, a focal depth of 9.2 km, and an epicentral distances, $R_{epi}$, ranging from 4.6 to 66.6 km (D’Amico et al. 2019). It was recorded at 60 sites, including at the station known as Forca Canapine located at rock site class A* (EC8 code, Luzi et al. 2016) and $R_{epi}$=11 km; A* means that site classification is not based on the shear wave velocity at 30 m, $V_{S30}$, (Clementi et al. 2020). We have selected the processed record from Forca Canapine retrieved from https://strongmotioncenter.org/ because of the maximum PGA recorded at the site. Note that according to a preliminary report (ReLUIS-INGV Workgroup 2016), the records from Forca Canapine were under technical revisions at the time but deemed adequate and, if confirmed, the strongest records ever recorded in Italy (close to a maximum peak ground acceleration, PGA, of 1 g). More recently, this record seems to be included to investigate the fling effects from near-source strong-motion records (D’Amico et al. 2019) and it is certainly included in a study to assess the seismic structural response of ancient Italian churches (Clementi et al. 2020). We include this seismic record in the present study as input for the time-history analysis of the TFS previously described, since this record was also used in a previous study, Valdés-Vázquez et al. (2019), resulting in large seismic demands for some elements. Other large PGAs recorded during the Norcia Mw6.5 earthquake are reported for the vertical component being 869 and 782 cm/s² (absolute values) at stations known as T1213 and CLO, respectively (Luzi et al. 2017). It would be interesting to inspect the vertical response of TFSs using these records, as well as the simultaneous effect of two or three orthogonal components, which is recommended in future studies.

The second record listed in Table 3 was generated by an intraslab normal-faulting event, the November 30th, 2016, Mw7.1 Anchorage earthquake, in Alaska (West et al. 2019), at a depth of 47 km. The record corresponds to station known as Chugach Park located over till and glacial deposits with bedrock at 34 m; the horizontal maximum PGA of approximately 2 g at the site is partly attributed to topographic and radiation effects (Cramer and Jambo 2020). The processed record was obtained from https://strongmotioncenter.org/, where a distance to the epicentre for Chugach Park station of approximately 27 km is given. The inclusion of the record is based on previous evidence of important stresses in the supporting frame of a TFS when large PGAs are used (Valdés-Vázquez et al. 2019).

The third listed accelerogram in Table 3 was recorded during the September 16th, 2015 Mw8.3 Illapel earthquake in Chile. This was a megathrust subduction earthquake recorded at several stations, including C11O station, whose record is used in the present study. Seismic intensities, over the median plus one standard deviation, were recorded for structural periods under 0.4s at this station, which was attributed to possible site amplification (Candia et al. 2017).

Station C11O is located at Monte Patria, over Quaternary alluvial deposits surrounded by volcanic rocks ($V_s30$=625 m/s), at a rupture distance $R_{epi}$ of approximately 55 km; an unexpectedly very high maximum PGA of 0.77 g was recorded at the site (Fernández et al. 2019). The seismic signal used in this study for this Chilean record was retrieved from http://evtdb.csn.uchile.cl.

The last record in Table 3 was obtained during the November 14th, 2016, Mw 7.8 Kaikoura earthquake in New Zealand; the rupture at 15 km depth, with ground motions exceeding 1g for the horizontal and vertical components, from the epicentral region at Station Waiau to Station Ward at approximately 125 km away from the epicenter (Kaiser et al., 2017). Although the preliminary study stated that the failure mechanism was oblique thrust (Kaiser et al. 2017) other studies describe it as a mechanism which possibly represents the most complex multiault rupture ever recorded (Crowell et al. 2018). In other study it is considered that a simultaneous rupture of the subduction interface and overlying faults occurred (Wang et al. 2018). The selected station is in the town of Ward about 6 km from the Kekerengu fault over deep soil (Allstadt et al. 2018, Kaiser et al. 2017). The processed record used was also obtained from https://strongmotioncenter.org/, where a maximum PGA of around 1.2 g is reported.

The records in Table 3 are selected because of their large PGA; all of them lead to important structural response of the TFS frame structure. Plots for these seismic records are shown in Fig. 5, where it can be observed that their maximum PGA is significant. They can be near-source recordings, like the Norcia one, but also have source-to-site distances of up to 125 km, like the New Zealand record. Some recordings with PGAs under 0.5g were also inspected but did not lead to significant increases in the seismic demand. Thus, only records in Table 3 are considered for the time-history analysis. Nonetheless, research should be extended using accelerograms from different tectonic environments, different frequency contents, etc. (even if they do not have large maximum PGAs), to further inspect this aspect.

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**Fig. 4** Geometry after form finding analysis and frame elements.
The original structured, mesh, form as recordings simply earthquake by 5.

Table 3 Earthquakes and related seismic records characteristics

<table>
<thead>
<tr>
<th>Record number</th>
<th>Earthquake location</th>
<th>Country</th>
<th>Station</th>
<th>Mw</th>
<th>Date</th>
<th>$R_{ep}$ or $R_{hyp}$ (km)</th>
<th>Depth (km)</th>
<th>Earthquake type/ Focal mechanism</th>
<th>Focal mechanism</th>
<th>Station Site Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Norcia</td>
<td>Italy</td>
<td>Forca Canapine</td>
<td>6.5</td>
<td>Oct 30th, 2016</td>
<td>11</td>
<td>9.2</td>
<td>Upper crustal normal-faulting</td>
<td>Intraslab normal-faulting</td>
<td>A* (EC8 Code), rock site</td>
</tr>
<tr>
<td>2</td>
<td>Anchorage</td>
<td>Alaska, U.S.A</td>
<td>Chugach Park</td>
<td>7.1</td>
<td>Nov 30th, 2018</td>
<td>27.4</td>
<td>47</td>
<td>Megathrust subduction earthquake</td>
<td>Subduction interface-overlying faults-multifault rupture</td>
<td>On till and glacial deposits with bedrock at 34 m (e.g., very dense soil and soft rock, NEHRP)</td>
</tr>
<tr>
<td>3</td>
<td>Illapel</td>
<td>Chile</td>
<td>Monte Patria</td>
<td>8.3</td>
<td>Sep 16th, 2015</td>
<td>55</td>
<td>11</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Amberley</td>
<td>New Zealand</td>
<td>Ward Fire</td>
<td>7.8</td>
<td>Nov 13th, 2016</td>
<td>125</td>
<td>15</td>
<td></td>
<td></td>
<td>Deep soils</td>
</tr>
</tbody>
</table>

Fig. 5 Acceleration recordings used in the dynamic analysis

5. Finite element discretization and results for the Alaska record

The seismic dynamic analysis of the TFS is carried out by using three types of finite element meshing, first for the earthquake recorded in Alaska in the x-direction (Fig. 2), simply because it has the maximum recorded PGA of the recordings depicted in Fig. 5. The first meshing is denoted as FF-S-Mesh and refers to a structured mesh from the form-finding results. The second one is denoted as FF-NS-Mesh, a non-structured mesh, that is also generated from the form-finding results. Finally, a third mesh which is structured, and not based on the form-finding, but in the original geometry denoted as NFF-S-Mesh is considered. The three types of meshing are shown in Fig. 6. The frame elements are the same for the three meshing cases (Figs. 6(a), 6(b) and 6(c), respectively).

As shown in Fig. 6, the type of finite element used for the analysis consists of membrane triangular elements, each one with three nodes. The mass matrix discretization for each finite element consists in computing the finite element mass and dividing it with equal parts to its three nodes. In structural analysis, usually the damping matrix is associated with the Rayleigh damping that requires two periods to compute the constants for the mass and stiffness matrices, in such a way, the vibration modes of the structure need to be previously calculated. In this work, the eigenperiods have not been computed and instead a parametric analysis with different damping coefficients for the mass is studied, and the coefficient that produces an exponential decay in the structural response has been chosen once the external loads are withdrawn.

In Figs. 6(a) and 6(b) it is clearly observed that the perimeter of the surrounding cable is longer than for the one in Fig. 6(c), since in the latter the cable is straight leading to a shorter length. This difference in length affects very
slightly the results because it leads also to a decrease in the weight for NFF-S-Mesh.

In Figs. 7, 8 and 9 the principal stresses at the membrane center, upper corner and lower corner, respectively, are presented. Figs. 7(a)-9(a) show the global behavior; Figs. 7b-9b show extreme (maximum and minimum) dynamic values and the static loading before imposing the seismic demand (i.e., that due to the self-weight and the prestress). Other figures shown later follow the same format. In Figs. 7(a), 8(a) and 9(a), the straight inclined line at the beginning of the time represents a monotonic increment in the membrane prestress, which reaches a static prestressed state (the horizontal line), before the hypar is subjected to the seismic excitations; the dynamic response can be clearly observed when the stresses fluctuate around the static prestress as a function of time.

In Fig. 7(b), it is observed that, when the hypar is analyzed from the form-finding mesh FF-S, the principal...
stresses are increased approximately 10% with respect to the static analysis, while for the FF-NS mesh the increase is approximately of 13%; the order of the stresses is similar for these two cases. Nonetheless, when the analysis is performed without considering the form-finding (NF-SS mesh), the change is the principal stresses is noticeable and the variation in stresses surpass 160% with respect to the static prestress values. This means that the stresses at the membrane center is not realistic without a previous form-finding procedure.

With regards to Figs. 8 and 9, it is observed that for the membrane corners behavior (in terms of principal stresses) similar conclusions can be drawn, but with a lower percentage. Overall, results presented in this section for the dynamic response, when the hypar is subjected to the Alaska recording, indicate that the form-finding is
important to adequately assess the membrane stresses and should be taken into consideration.

The axial force for one of the principal tensors is shown in Fig. 10, where the same three mesh cases are shown and it can be observed that, broadly speaking, the behavior is very similar for all cases, being the main difference that of the static case; this occurs because the surrounding cable perimeter is less when the form finding is not accounted for. However, the difference between the static forces and the maximum reached dynamic forces is in the order of 5%.

Fig. 11 is analogous to Fig. 10, except that the axial forces correspond to the compressive forces in one of the masts. Qualitatively, results in Fig. 11 resembles results in Fig. 10, but quantitatively an important difference is observed, since the increment in axial forces with respect to the static compressive forces is approximately 54%, 53% and 55% for cases FF-S, FF-NS and NFF-S, respectively. It is point out that this increase is important from a design standpoint, since it surpasses the 50% in all cases.

Figs. 12 and 13 show tensor axial forces for “x” and “y” tensors (see Figs. 2 and 4), respectively; these tensors are those used to give the masts support. Tensors behave alike in both directions, having increases in the axial force with respect to the static case of 167% and 179% when the form-finding is considered or not, respectively. Conclusions that can be drawn are, in one hand, that while the form-finding has an important effect in adequately computing the membrane stresses, it does not significantly change the resulting axial forces and, in the other hand, that such significant increment in the axial load imposed by the seismic effects should not be neglected in the design of the frame elements.

In the next section, results when the TFS is subjected to the remaining records are presented. Figures are like those reported in this section. Descriptions of results are given, and a discussion, focused on any possible effect of considering earthquakes from different tectonic environments worldwide, is also presented.

6. Results and discussion including Chile, Italy and New Zealand seismic records

In this section a comparison of the structural response for all the records in Fig. 5 is carried out. Even though in the previous section no significant difference, whether the dynamic analysis includes the form-finding or not, was found for the axial forces (i.e., FF-S-Mesh or FF-NS-Mesh), in this section the results are compared using only the FF-S-Mesh (i.e., including the form-finding), since the results are
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Fig. 14 Mast axial force

Fig. 15 Mast axial force limits

Fig. 16 Tensor x-axis axial force
more reliable, especially for the fabric stresses.

In Figs. 14 and 15 the variations in axial force in a mast are presented. The increases with respect to the static load (self-weight and prestress) in percentage are shown for the different seismic events in Figs. 14 and 15, resulting in values 54%, 30%, 30% and 69% for Alaska, Chile, Italy and New Zealand earthquakes, respectively. It is noteworthy that for the Mw6.5 Italy event as well as for the Mw8.3 New Zealand event the increase is only 30%, while the other events lead to increases of approximately twice the static loads.

In Figs. 16 and 17 the increments for the tensile force in a tensor, after the seismic dynamic loads are applied, are reported. Values of 156%, 205%, 189% and 143% for recordings from Alaska, Chile, Italy and New Zealand are obtained. In this case the maximum increment does
coincide with the event with the largest Mw, the Mw8.3 Chile earthquake, leading to an increment of 205%. From the design standpoint, no element designed for the static loads would be able to withstand twice the demand as compared to the static design. This proves that the seismic design of TFS should not be neglected.

Figs. 18 and 19 show the results for the principal tensor. In this case the increments are not significant, since values of 5%, 4%, 7% y 5% for the Alaska, Chile, Italy and New Zealand cases, respectively, are obtained. In this case an original design from the static loads may cover the additional axial forces induced by the seismic activity.

Finally, Figs. 20 and 21 present resulting stresses for the upper corner of the fabric. It is observed that also for this case the increments can be deemed somewhat unimportant, like the case of axial forces for the principal tensor, with values equal to 8%, 4%, 5% and 12% for the recordings from the Alaska, Chile, Italy and New Zealand earthquakes. It should be highlighted tough, that the shown stresses are principal stresses at the corners (the maximum stresses occur at the corners), where it is recommended to have additional strengthening (thus the common use in practice of double fabric at the corners is justified).

By considering the earthquakes and site characteristics of the employed recordings, which cover an important range of moment magnitude (6.5≤Mw≤8.3), epicentral or rupture distances (11 km - 125 km), focal depths (9.2 km - 47 km), as well as tectonic environments, failure mechanisms and site soils classifications, the only clear conclusion that can be drawn, is that all of them have very large maximum PGAs (approximately 0.7 g - 2.0 g) and this is the main feature which can be related to significant increases in the axial forces imposed by the seismic demand in TFSs, if compared to the static loading. An attempt was made to inspect whether higher increments are a function of increasing maximum PGA, but not clear trend was found. For instance, while the largest seismic demand increment for the mast compressive force is due to the New Zealand record, the maximum increase in the tensile force for the tensor is imposed by the Chile earthquake (whose maximum PGA is the smallest considered), followed close by the Norcia Earthquake, which nor has one of the largest
maximum PGAs considered, neither a large magnitude (the smallest indeed), albeit it has the shortest source-to-site distance. Similar unclear trends are found for the principal tensor and membrane stresses. Therefore, simple recipes for design are not recommended. Nevertheless, further research is required to inspect whether the PGA could be solely the main parameter for design (for instance in probabilistic rather than in deterministic terms), since it is usually one of the most relevant parameters of ground motion for design purposes. The seismic structural response of TFSs seem to be a complex combination of source, path, and site characteristics, plus perhaps frequency content of the signal and even the supporting frame configuration (Valdés-Vázquez et al. 2019). If seismic risks analysis of TFSs is pursued in the future, the seismic hazard from different contributing sources and seismic scenarios, as well as the vulnerability of different TFS geometries and configurations of their supporting frames, may be necessary. The finite element formulation, the meshing and other structural analysis aspects should also be selected carefully, so that time-history analyses can be performed adequately in TFSs.

8. Conclusions

The seismic structural response of a tensile fabric structure, in terms of membrane stresses and axial forces in the supporting frame, is investigated by considering a geometrical configuration commonly known as hypar. Recordings from several parts of the world are used. They cover wide ranges of moment magnitude (6.5≤Mw≤8.3), epicentral or rupture distances (11 km - 125 km) and focal depths (9.2 km - 47 km), and they come from different tectonic environments with different failure mechanisms, site soil classifications and frequency contents. However, they have in common large maximum PGAs. Finite element simulation is employed to perform time-history analyses using the seismic records. The analyses include the so-called form-finding procedure.

The main findings of this study are:

- The maximum seismic tensile forces in cables and compressive forces in masts should be incorporated in design; especially if considering that seismic loading (unlike wind loading) is usually not considered relevant in practice for TFSs. The opposite occurs for the membrane earthquake-induced stresses, which do not increase importantly under seismic loading.

- Records with large maximum peak ground accelerations (> 0.7 g) induce axial forces in the masts and cables up to twice as large as those imposed by the static case (i.e., self-weight and prestress).

- Although, the maximum PGA is a key parameter correlated to significant seismic-induced axial forces, there is not a clear trend to express the latter as a function of the former.

- The exclusion of the form-finding procedure, before the dynamic analysis, significantly impacts the resulting fabric stresses; the opposite occurs with respect to the compressive and tensile forces in masts and tensors, respectively (i.e., resulting axial forces in the time-history analysis are relatively unaffected by excluding the form-finding procedure).

- Other than large axial forces linked to large maximum PGAs, no clear trends were found between the different earthquakes and recordings characteristics and the largest obtained axial forces.

It is concluded that simplistic rules and analyses for designing support frame elements of TFSs subjected to seismic loads are not recommended. It is also concluded that the seismic structural response of TFSs is a complex combination of earthquake failure mechanism, frequency content and site effects, among other possible aspects. Seismic hazard from different contributing sources, as well as the capacity of different TFS geometries and configurations, should be considered if seismic risk analyses are desired in the future. Not every finite element formulation can capture the complex, highly non-linear, behaviour of TFSs, therefore care should be exercised to select the adequate tools for the analysis.

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References


