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Spring March 13, 2011

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Disproportionate Collapse Analysis of Cable-stayed Steel Roofs for Cable Loss

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Abstract

Disproportionate collapse has been identified lately as a real cause of failure for structural engineering projects. Rare and unexpected, the phenomenon of disproportionate collapse usually results to many fatalities and thus, its analysis and mitigation is deemed necessary. This work describes the analysis of a cable-stayed steel roof under the scenario of a cable loss. The event of a cable loss is assumed to be brittle, while relevant recent recommendations suggest the application of a scaled equivalent static force at the points of the anchorage of the cable but in the opposite direction of the original cable force. In this paper, three different conditions have been considered in order to study the effect of the cable loss into the overall structural response of a typical cable-stayed roof; the level of the equivalent nodal load in the opposite direction of the original cable force varies. The steel structure of the roof, in its complexity, closer to responding as a cable-stayed bridge rather than a steel roof provides a useful template for conclusions; several topics regarding disproportionate collapse and cable losses are discussed.

Keywords: cable-stayed structure, robustness, disproportionate collapse, cable loss, dynamic amplification factors, load increase factors

1. Introduction

Although disproportionate collapse has been an area of significant research lately (Ellingwood *et al.*, 2007; Starossek, 2007; Structures Congress SEI, 2009), most efforts have focused on the effects of the phenomenon on buildings (Tomasetti *et al.*, 2005; Byfield *et al.*, 2007; Demonceau and Jaspert, 2008; Izzuddin *et al.*, 2008; Gerasimidis *et al.*, 2009; Kim and Kim, 2009; Knoll and Vogel, 2009; Dubina *et al.*, 2010) rather than roofs or bridges. It is not surprising that most of the guidelines produced so far (GSA, 2003; DoD, 2009) include mainly criteria and requirements for building structures. Undoubtedly, there has been an increasing series of events where the phenomenon appeared in building structures such as the Ronan Point apartment building in London, the World Trade Center in New York, the 130 Liberty Street building in New York and many others. A common point in all these cases was the unexpected nature of the events and the accidental loading which

triggered the sequence of the collapse.

However, the resistance of bridges or roofs against disproportionate collapse may be even more crucial since a possible appearance of the phenomenon in such structures can have many and dreadful consequences. Not only can the casualties of a possible collapse of a big structure be very high, but also the impact of their malfunction to the general infrastructure net of urban environments can rapidly escalate the economic losses of several stakeholders and labor any kind of traffic.

During the last decades several affined cases have been recorded, one of which was the collapse of the I-35W Mississippi River bridge in Minneapolis, Minnesota, in 2007. The bridge was designed and constructed using steel arched trusses and provided an 8 lane traffic passing over the river. The sudden and completely unexpected collapse of the bridge during rush hour resulted in the killing of 13 people and injuring another 145 (Abolhassan, 2008); the bridge was not even 40 years old. The consequential effects of the collapse on business, traffic and transportation funding were enormous.

In addition to that event there has been a series of references lately (Starossek, 2006) about failing cables in bridge structures the most prominent of which was the sudden and unforeseen loss of a cable at the newly constructed Rio-Antirio cable-stayed Bridge in Greece, in 2005. The event was caused by a lightning strike on the cable igniting it, which resulted in its complete failure

Note.-Discussion open until August 1, 2011. This manuscript for this paper was submitted for review and possible publication on July 2, 2010; approved on February 10, 2010.
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and collapse on the deck of the bridge. Luckily in that case, there was no disproportionate collapse mechanism triggered by the cable loss, mainly due to the design of the bridge and thus, there was enough time for its replacement.

Another similar cable failure occurred in the Mezcala bridge in Mexico (Zoli and Steinhouse), where after a car accident on the bridge, a fire was initiated causing the failure of one of the bridge's cables. Again, no serious damage was induced at the bridge, but undoubtedly both of these events have alarmed the structural engineering community regarding the vulnerability of cable-stayed structures.

Additionally, the reliability of cables against fire combined with the accidental nature of these events have created a vague environment related to cable resistance. In this framework, the cable-stayed roof under review in this paper provides a well-suited template for a study regarding its disproportionate collapse resistance for a cable loss. The work presented herein focuses on the simulation of the cable loss event using static elastic models, in an attempt to identify a proper simple method to perform the robustness evaluation of the structure for the case of cable loss. The general approach of already accepted guidelines for building structures is adopted by applying the force of the failed cable to the remaining structure increased by different amplification factors.

2. A Typical Structural System of a cable-stayed Roof

Among the various types of cable-stayed roofs, the one under review consists of a couple of bent curves which cover an area of around 28000 m²; the curves form a cable stayed bridge-like steel structure. The roof has four independent supports on the ground, separated by 300 m in the longitudinal direction and by 145 m in the transversal one. The initial design of the roof was performed according to the relevant Eurocodes. A plan view of the roof is shown in Fig. 1.

The main steel structure of each bent curve is composed of two large tied arches which run between the supports along the longitudinal direction. Both arches are designed with tubular sections, belong to the same plane, perpendicular to the ground level and are connected by a series of prestressed cables (principal cables). For that purpose, the lower arch is heavily reinforced by diaphragms which cover its section. An elevation of the two arches is shown in Fig. 2.

The cladding of the roof is meshed by triangular cantilever beams or cross girders, all "I" in section, which are attached to the lower arch and are suspended from the upper arch by another series of prestressed cables (secondary cables).

Overall, the presence of 248 cables on the roof with a total cable length of approximately 9800 m highlights

Figure 1. Plan view of the roof.

Figure 2. Elevation of the structural system of the roof.

their importance for the reliability of the whole structure. Taking into account the incidents described in the introduction, the steel roof, just as any cable-stayed structure, is strongly dependent on the reliability of its cables.

3. Disproportionate Collapse Analysis

3.1. DoD guidelines-alternate path method

During the last decades, there have been several efforts aiming at the production of guidelines regarding disproportionate collapse. As for any other collapse related problem there is an increasing need for structural engineers to reference to commonly accepted standards which can be followed during the design process.

Among the many approaches to the problem, the United Facilities Criteria by the DoD has been accepted by many as the most dominant document. The DoD criteria, based on the standard of ASCE 7, defines the problem such that any structure should be designed "to sustain local damage with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage". In this framework, the proposed methods of design are the direct and the indirect design methods; the direct design methods are further explicitly divided in the alternate path (AP) method and the specific local resistance (SLR) method, the former of which is loosely adopted in this example.

However, the DoD's major focus on buildings does not allow for any reference to other types of structures such as bridges or roofs. Therefore, the loss of a cable in a bridge-like structure such as the cable-stayed roof under review is not included in the provisions. However, it can

be treated in a comparable manner. Indeed, the conceptual approach of the alternate path method in a broader manner can cover all types of structural systems.

The loss of a cable has been described in recent literature as a brittle phenomenon which requires the consideration of dynamic phenomena when incorporated in the analysis of a structure. For that reason many authors (Starossek, 2006, Wolff and Starossek, 2008, PTI, 2007) have recommended the so-called dynamic amplification factors (DAFs) used for the static analysis of the phenomenon, which are assumed to be close to the value of 2.0 and affect the cable force which is applied to the remaining structure.

For the same reason, the DoD criteria define, for building structures, the load increase factors (LIFs) which appear in reference to a column failure. The LIFs affect the loading of the elements which are at the areas above the removed column, increasing it by a scale closely ranging around the value of 2.0. Both factors, the DAFs and the LIFs consist of a conservative technique in order to incorporate the dynamic response of the structure to a sudden failure; their application allows therefore a non-linear static approach which is also the method of analysis for this example.

However, the only difference between the two approaches is that the LIFs are related to a given parameter of the problem, i.e. the vector of loading, while the DAFs are related to the value of the cable force just before the failure which can only be extracted after the structural analysis and is dependent on the vector of loading, the stiffness of the structure and its boundary conditions.

Therefore, the appearance of several different load cases creates a difficulty for the designer regarding the correlation of the DAFs with their reference value. This work considers the DAFs to be directly related either to the pretension force of the cables or to their ultimate load capacity, both of which are parameters not dependent on the analysis of the structure.

3.2. Method of analysis

The disproportionate collapse mechanism for cable-stayed roofs or bridges is described in the literature as a very complicated event. Although many authors suggest the simultaneous loss of more than one cables (Wolff and Starossek, 2008) or the zipper-type collapse (Starossek, 2007), the analysis of the cable-stayed roof in this paper was limited to a single cable loss. The number of cables in the roof and their topological distribution along its length and surface converge towards this assumption (see Fig. 3).

The comparative analyses conducted herein can be categorized into three different approaches to the problem which correspond to three different levels of cable force applied to the remaining structure. The first two analyses are described below and are intended to highlight the importance of the DAFs to the response of the structure

Figure 3. Intact structure.

Figure 4. Analysis 1, introducing the removal of the cable in the geometry of the model.

Figure 5. Analysis 2, introducing the removal of the cable in the geometry of the model combining it with a nodal load equal to two times the pretension force of the cable.

for the event of a cable loss:

- Analysis 1 Introducing the removal of the cable in the geometry of the structure without any DAF or any nodal load at the remaining structure (shown in Fig. 4).
- Analysis 2 Combining the removal of the cable with an equivalent nodal load in the opposite direction of the cable force, equal to two times the pretension force of the cable (shown in Fig. 5).

The definition of the third analysis was determined by the problem's inherent difficulty regarding the simulation of the phenomenon.

For the sake of simplicity, the cable-stayed structure roof is assumed to be under the operational loading which can be described by the serviceability combinations of EC3. Under these conditions and for an unexpected

Figure 6. Anticipated failure mode of the second cable. Load to be inserted for the second cable failure should be equal to the ultimate capacity load of the cable.

Figure 7. Analysis 3, introducing the removal of the cable in the geometry of the model combining it with a nodal load equal to its ultimate breaking force of the cable.

reason, it is assumed that a thunderstorm or any other accidental event causes the sudden loss of one of the cables of the structure.

According to the current recommendations, the response of the structure to that event should be analyzed by removing the cable from the structure and by applying an equivalent nodal static load at the cable anchorage nodes but in the opposite direction of the cable force, equal to two times the static force of the cable under the serviceability combination.

The analysis of the input previously described certainly provides results which would increase the forces and the stresses of the remaining structure, probably causing another cable to fail. However, in that case this consequent failure is completely different than the first one; the second failure appears due to the increased loading and therefore the cable fails due to high stresses. Actually, at the time of failure, the second cable has probably reached its ultimate loading capacity. Thus, the relation of the DAFs to the static force of the cable should now be associated with the ultimate capacity of the cable, rather than the cable force under the serviceability combinations as above.

This difference notifies that even a nonlinear static analysis of the phenomenon would prove to be inefficient for the case of a cable-stayed structure. Therefore, for the sake of comparison to the first and second analyses, analysis 3 was performed in order to provide a qualitative response to this level of cable forces:

Analysis 3 Combining the removal of the cable with an equivalent nodal load in the opposite direction of the cable force, equal to its ultimate breaking force.

It should be further explained that analysis 3 is comparable to analysis 1 or 2 in the sense that it simulates a different type of cable failure and does not intent to substitute any design method. Additionally, this paper does not intend to follow the possible failure pattern of the roof but instead is aiming at highlighting the major differences in the response of the roof for different types of cable failure. Nevertheless, the results from analysis 3

provide a useful comparison for the response of the structure regarding the nature of the failure.

4. Results

The results extracted from the analysis model were categorized in three different levels of loading conditions relative to their probabilistic manner of appearance, according to EC3. Therefore, for the sake of completeness, the serviceability limit state combinations and the ultimate limit state combinations were distinguished, while the dead load combination was also sorted out for a better depiction of the response of the structure. The results are presented in Table 1 and show the demand-to-capacity ratio (i.e. Cable load/Cable capacity) for the remaining cables of the structure as well as the stress analysis for the two major elements of the structure, the two arches.

Although the cable loss for the roof would be expected to be critical for the survival of the structure, the response of the structure is considered adequate to resist the phenomenon of disproportionate collapse. The results of analysis 1 and 2 regarding the dead load and the serviceability limit state combinations are all inside the acceptable stress and design criteria. Regarding the results of the ultimate limit state combinations, they provide lower overstresses than the ones expected and in any case they do not provide a useful template for conclusions since they should not be connected to any disproportionate collapse events. Undoubtedly, the condition best described by the criteria of current codes and provisions which is the serviceability limit state combinations in analysis 2, does not produce any overstress in the structure.

On the other hand, the application of a nodal load as high as the ultimate capacity of the failed cable provides results with much higher values for all the elements. It should be mentioned though that analysis 3 was performed only for qualitative conclusions and is not described by any code or recommendation. The results of analysis 3 show that a possible cable failure at its ultimate capacity would cause much more damage to the structure. These

Figure 8. Stresses distribution of the affected upper arch.

results emphatically point out the necessity to limit the cable losses to the first cable, since the possible consequent cable loss could be the catastrophic one. The reason for this sequence is the brittle nature of cable failures, characterized with almost no plastic behavior.

A clearer depiction of the structure's response is presented in figures 3 and 4 which show the distribution

Table 1. Demand-to-capacity ratios for remaining cables and stress analysis for both arches, a. Analysis 1, a. Analysis 2, a. Analysis 3.

Analysis 1			
Element	Dead	Serviceability	Ultimate
Principal cables $\Phi 90$	0.57	0.682	0.896
Principal cables $\Phi 104$	0.87	0.9747	1.23
Secondary cables $\Phi 40$	0.66	0.818	1.177
Upper arch (MPa)	221.47	254.8	361.6
Lower arch (MPa)	227	269.4	355.4
a			
Analysis 2			
Element	Dead	Serviceability	Ultimate
Principal cables $\Phi 90$	0.57	0.682	0.896
Principal cables $\Phi 104$	0.876	0.983	1.243
Secondary cables $\Phi 40$	0.69	0.844	1.21
Upper arch (MPa)	221.47	255.75	362.83
Lower arch (MPa)	231.95	274.8	362.64
b			
Analysis 3			
Element	Dead	Serviceability	Ultimate
Principal cables $\Phi 90$	0.56	0.683	0.897
Principal cables $\Phi 104$	1.13	1.24	1.59
Secondary cables $\Phi 40$	1.49	1.64	2.28
Upper arch (MPa)	248.5	283	397.4
Lower arch (MPa)	393.9	436.3	579.5

of von Mises stresses along the two arches due to the cable loss, for the dead load combination. Figure 3 shows the stresses of the upper-affected arch after the loss of the cable, while Fig. 4 the upper-away arch. There is a slight increase of the stresses along the upper-affected arch, while the symmetrical distribution of the stresses is clearly disturbed. On the other hand, Fig. 5 shows the stresses of the lower-affected arch after the cable loss and Fig. 6 the stresses of the lower-away arch. The increase in stresses in Fig. 5 is significant, especially close to the area around the cable loss. There is an area close to the center of the arch which appears to have uneven high stresses. However all the elastic stresses are inside the acceptable limits.

A qualitative comparison of the three analyses can be derived from Figs. 7, 8 and 9, which emphasize the criticality of the consequential cable failure carrying its ultimate load. These figures present the demand-to-capacity ratios of the remaining secondary cables of the structure and their response to the cable loss. Figures 7 and 8 show the response in analysis 1 and 2 while figure 9 shows the response in analysis 3 where the applied load to the remaining structure is the ultimate bearing capacity of the cable. The difference between the three graphs shows a higher increase in the cable force for analysis 3, justifying that a cable failure due to ultimate load has a much bigger impact to the structure than the failure due to unexpected accidental reasons such as lightning or fire.

5. Conclusions

Although the approach of using a dynamic amplification factor of 2.0 for the first cable loss is justified, a possible consecutive failure which is triggered from the first one, should be treated completely differently and should directly refer to the ultimate load capacity of the failing cable. This consecutive cable loss due to its carrying load at the time of the failure can potentially prove to be the

Figure 9. Stresses distribution of the away upper arch.

Figure 10. Stresses distribution of the affected lower arch.

Figure 11. Stresses distribution of the away lower arch.

critical one for any cable-stayed structure. Therefore, the cable loss of such structures must remain localized at the loss of one cable, otherwise it could evolve to unexpected

consequences.

It should be mentioned here that among the three methods, the most realistic redistribution of the stresses is

Figure 12. Cable capacity along arches, analysis 1.

Figure 13. Cable capacity along arches, analysis 2.

Figure 14. Cable capacity along arches, analysis 3.

achieved with a combination of analysis 2 and analysis 3. Analysis 1, does not simulate the cable failure in any case, since it does not incorporate any dynamic effects. On the other hand, analysis 2 incorporates dynamic effects relative to the serviceability condition of the structure, while the results of analysis 3 present the response of the roof in a possible sequential failure. The best way to account for the distribution of the stresses in the roof

would be to consider analysis 2 for the first cable failure and analysis 3 for the cable failures which would follow afterwards. For the purposes of this paper, analyses 2 and 3 provide the comparison of the first cable failure of the roof with possible cable failures which could possibly follow.

The robustness level of the roof in particular was shown to be high with the dynamic amplification factor

fixed to 2.0, if the reference load is the pretension force of the cable. However, the analysis conducted here does not provide a detailed understanding of the disproportionate collapse response of the structure and is more qualitative rather than quantitative.

More studies in that direction, with the incorporation of limit analysis or collapse load analysis are necessary in order to justify and better depict the robustness limit of the structure at hand under a sudden cable loss.

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