Reliability-Based Progressive Collapse and Redundancy Analysis of Suspension Bridges

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**ABSTRACT:** This paper describes a reliability-based approach for evaluating the performance of suspension bridges susceptible to local damage from rare extreme events. The approach accounts for the uncertainties in evaluating the ambient loads during the occurrence of sudden damage and in evaluating the load carrying capacities of the members as well as the damaged system. A method to develop appropriate reliability-based and equivalent deterministic criteria that can be used to assess the potential for progressive collapse are proposed based on risk tolerance levels. Latin Hypercube Simulations (LHS) are used for the system reliability analysis and the consequences of failure are estimated in monetary terms accounting for the cost of rebuilding the bridge, loss of life, traffic and user costs. The risk analysis methodology and the determination of equivalent reliability and deterministic criteria are illustrated using an example suspension bridge subjected to different initial damage scenarios.

1. **INTRODUCTION**

The importance of providing progressive collapse analysis guidelines for all types of long span bridges has been gaining increased awareness due to the risks of accidental failures and intentional sabotage that such structures may be under due to their symbolic as well as their economic importance. The goal is to ensure that these bridges are able to sustain significant levels of damage without collapse. The insensitivity of a structure to an initial sudden damage has been recently defined as structural robustness by Starossek and Haberland (2010) who emphasized the need to improve current structural design standards in order to directly account for system redundancy and robustness in all types of structural systems including long span bridges. The focus of this paper is on suspension bridges because they represent the most common type of long span cable supported bridges in the U.S.

A limited number of previous studies have performed preliminary investigations of the performance of damaged suspension bridges. Researchers have found that the critical components of a suspension bridge are the suspender cables because they are vulnerable and exposed (Bontempi et al., 2004; Giuliani, 2008 and Haberland et al., 2012). They reported that even the failure of one suspender may lead to a progressive collapse of the entire bridge. For these reasons, Zoli and Steinhouse (2007) suggested that in general a large factor of safety should be used for the design of suspender cables compared to that of other structural components, but they did not explain how the factor can be determined. Imai and Frangopol (2001) studied a suspension bridge from a member and system reliability points of view and made similar recommendations but they did not consider system robustness.

The objective of this paper is to develop a methodology that will help establish non-subjective criteria that can be applied to evaluate the redundancy and robustness of suspension bridges. The focus is on evaluating the structural robustness of the system following sudden damage to the suspender cables. The performance of
the structure is evaluated using system reliability concepts and target reliability levels are used to recommend deterministic static analysis criteria based on concepts of risk tolerance. Such deterministic criteria will help engineers evaluate the ability of bridges to sustain sudden single point failures using pseudo-static methods.

2. PROGRESSIVE COLLAPSE ANALYSIS METHODOLOGY
In this paper, bridge system reliability is evaluated using the Latin hypercube sampling (LHS). The LHS is used in conjunction with a three-dimensional, nonlinear time history dynamic analysis of the bridge system. Figure 1 gives a schematic representation of the dynamic analysis process. Specifically, the progressive collapse analysis procedure first identifies the members susceptible to sudden damage. The internal forces in those members are calculated under ambient loading conditions. Impulsive loads equal and opposite to those calculated are then applied to simulate the sudden removal of the members. The dynamic response of the system is studied to evaluate the extent of structural damage. The suddenness of the removal is represented by the impulse duration $T_d$ which depends on the damage scenario considered. Values in the range $T_d=1\times10^{-3}$~$1\times10^{-2}$ s may represent damage scenarios caused by blast or impact. In this paper, a sensitivity analysis considered the effect of rupture time of the suspenders and it was found that a value for $T_d=0.01$s provides an optimal balance between accuracy and computational efficiency.

3. STRUCTURAL MODEL OF BRIDGE
A suspension bridge with span lengths 250m-830m-250m, as shown in Figure 1(a), is analyzed to illustrate the dynamic progressive collapse analysis process. The steel box girder is 28.2m wide supported by suspenders in the middle span with spacing of 12.5m+70x11.5m+12.5m. The two side spans have no suspenders.

3.1. Main Cable
Zinc-coated parallel wire structural strands, ASTM A586 are used for the main cables of the bridge. The cross section of the main cable is determined by the combination of dead load, live load, static wind load and temperature change. The design of the main cables was based on a safety factor of 2.5 with respect to the rupture strength of 1,600 MPa. The main cables consist of 9,271 parallel wire cables providing a total cable cross section area=0.1963 m$^2$. Previous studies have determined that the uncertainties in estimating the strength of main cables are relatively low for newly constructed bridges which have not yet been exposed to deterioration. The statistical data for the main cables, suspenders and stiffening girder are summarized in Table 1.
3.2. Suspender Rope
For the original design, the cross section area of the suspender rope shown in Figure 2 is determined based on the safety factor of 3.0 with respect to rupture strength. Each connection of suspender and main cable has two ropes. Based on available data, this study uses a bias factor of 1.15 and a coefficient of variation of 0.08 to account for the uncertainties in the strength of suspenders (Imai and Frangopol, 2001).

3.3. Stiffening girder
The stiffening girder is a single cell steel box girder. The steel box height is 3.0 m at the center line and the transverse slope of the deck is 2%. The width of the box is 28.2 m and the yield strength of the steel is 345 Mpa. The thickness of upper flange is 12 mm. The lower flange and webs are 10 mm. The upper and lower flanges are stiffened by U-shaped stiffening ribs. Based on available data, this study uses a bias factor of 1.05 and a coefficient of variation of 0.10 on the ultimate moment capacity which is calculated to be equal to 500,400 kN-m (Imai and Frangopol, 2001).

![Figure 2: Suspenders and their connections to cable](image)

<table>
<thead>
<tr>
<th>Table 1: Random variables for bridge members</th>
</tr>
</thead>
<tbody>
<tr>
<td>Member</td>
</tr>
<tr>
<td>--------</td>
</tr>
<tr>
<td>Main cable</td>
</tr>
<tr>
<td>Suspender</td>
</tr>
<tr>
<td>Girder</td>
</tr>
</tbody>
</table>

4. LOAD MODELS

4.1. Dead load
Two types of dead loads are considered in the design of the suspension bridge: (1) permanent load (self-weight of the steel box and other structural members); and (2) superimposed load (pavement and other secondary members). Table 2 shows the dead loads and live loads that were used for the bridge. According to Imai and Frangopol (2001), the distributions of both the permanent and the superimposed loads follow normal distributions with bias factors and coefficients of variation equal to 1.03 and 0.08, respectively.

4.2. Live load
According to the AASHTO LRFD Bridge Design Specifications (2012), a uniformly distributed design live load of 0.64kip/ft per lane is applied on the suspension bridge. As the bridge has two traffic lanes in each direction, the uniform design traffic load, p_u, for the bridge considering a 0.85 multiple presence factor for each passing lane, the nominal load is calculated to be 2.368kip/ft (34.56kN/m).

To consider uncertainties in the live load applied on the suspension bridge, weigh-in-motion (WIM) data collected over a one year period in the vicinity of a suspension bridge site was used to calculate the expected maximum load over different bridge service periods. The maximum loads for 1-week, 1-year, 5-year and 75-year service periods were checked under congested traffic conditions. Based on previous studies, Ghosn et al (2013) observed that the statistical projections of maximum live load data would approach a Gumbel Type I probability
distribution with a COV that varies depending on projection period. For this problem the COV’s of the projected maximum live load are shown in Figure 3. The live load should also account for epistemic uncertainties to account for the limitations in the WIM data projection procedure using a modeling variable $\lambda_{\text{data}}$ (Normal distribution: $\lambda_{\text{data}} = 1.0$ and $V_{\text{data}} = 2\%$) and for site variability using $\lambda_{\text{site to site}}$ (Normal distribution: $\lambda_{\text{site to site}} = 1.0$ and $V_{\text{site to site}} = 10\%$).

Performing a Monte Carlo simulation, it was found that the final live load including modeling uncertainties would approximately follow a normal probability distribution with the statistical data shown in Table 2. Figure 3 illustrates how the epistemic uncertainties dominate the probability distribution of the live load.

### Table 2: Statistical parameters for dead loads and live loads

<table>
<thead>
<tr>
<th>Variable</th>
<th>Nominal Loads (kN/m)</th>
<th>Bias</th>
<th>COV</th>
<th>Distribution Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent load</td>
<td>108.00</td>
<td>1.03</td>
<td>8%</td>
<td>Normal</td>
</tr>
<tr>
<td>Superimposed dead load</td>
<td>48.00</td>
<td>1.03</td>
<td>8%</td>
<td>Normal</td>
</tr>
<tr>
<td>1-week live load</td>
<td>34.56</td>
<td>1.136</td>
<td>10.4%</td>
<td>Approximately Normal</td>
</tr>
<tr>
<td>1-year live load</td>
<td>34.56</td>
<td>1.206</td>
<td>10.3%</td>
<td>Approximately Normal</td>
</tr>
<tr>
<td>5-year live load</td>
<td>34.56</td>
<td>1.230</td>
<td>10.3%</td>
<td>Approximately Normal</td>
</tr>
<tr>
<td>75-year live load</td>
<td>34.56</td>
<td>1.266</td>
<td>10.3%</td>
<td>Approximately Normal</td>
</tr>
</tbody>
</table>

![Figure 3: Probability distribution of live load $L_{\text{max}}$ including epistemic uncertainties](image-url)
5. PROBABILISTIC ANALYSIS OF SYSTEM REDUNDANCY AND PROGRESSIVE COLLAPSE

Probabilistic analyses of the suspension bridge are performed using the Latin Hypercube Sampling (LHS) method. The probability distribution of each variable was divided into 180 bins. The bins were randomly selected and a random sample was extracted from each bin for each variable. At first, a static reliability analysis of the entire originally intact system is performed. The reliability index for the first member is found to be 8.0 and that of the system is 10.0 when the 75-yr maximum load is applied. These values are compared to $\beta_{\text{member}}=9.1$ and $\beta_{\text{system}}=5.99$ (governed by stiffening girder) found by Imai and Frangopol (2001). The girder of the bridge analyzed in this study was overdesigned which explains the higher reliability index for the system obtained in this study.

Subsequently, three damage scenarios for suspenders are consider, (1) one suspender suddenly removed from the center of the span; (2) three suspenders suddenly removed; (2) five suspenders suddenly removed. The dynamic reliability analysis of the bridge response due to the sudden damage follows the approach described in Figure 1 for the bridge loaded by service live loads which are conservatively represented by the 1-week maximum load. The analysis included both material and geometrical nonlinearities. The suspenders are considered to be brittle members. The deck is ductile and its nonlinear behavior is modeled by a moment-curvature relationship. Member safety is evaluated based on the reliability index of the hanger next to the removed one. System safety is evaluated based on the reliability of the fifth hanger away from the removed one. The system safety criterion was established following a sensitivity analysis which showed that the brittle nature of the suspenders and the wide 11.5 m distance between them lead for a rapid unzipping of the hangers when six consecutive hangers are removed. The LHS Simulation results are summarized in Table 3 for the reliability index of one member and that of the system for the case when the suspenders are designed with safety factors S.F.=3.0 and 3.5.

<table>
<thead>
<tr>
<th>Safety factor for design of suspender</th>
<th>No. of suspenders ruptured</th>
<th>Reliability index of Adjacent Suspender</th>
<th>System Reliability index</th>
<th>Live Load factor for pushdown analysis- member failure</th>
<th>Live Load factor for pushdown analysis-system collapse failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.0</td>
<td>N</td>
<td>$\beta_{\text{member,progressive}}$</td>
<td>$\beta_{\text{system,progressive}}$</td>
<td>Live load factors</td>
<td>Live load factors</td>
</tr>
<tr>
<td>1</td>
<td>7.15</td>
<td>8.80</td>
<td>7.08</td>
<td>8.61</td>
<td></td>
</tr>
<tr>
<td>3*</td>
<td>2.90</td>
<td>7.26</td>
<td>2.31</td>
<td>5.87</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>1.49</td>
<td>2.79</td>
<td>--*</td>
<td>1.87</td>
<td></td>
</tr>
<tr>
<td>3.5</td>
<td>N</td>
<td>$\beta_{\text{member,progressive}}$</td>
<td>$\beta_{\text{system,progressive}}$</td>
<td>Live load factors</td>
<td>Live load factors</td>
</tr>
<tr>
<td>1</td>
<td>8.06</td>
<td>9.47</td>
<td>9.11</td>
<td>9.78</td>
<td></td>
</tr>
<tr>
<td>3*</td>
<td>3.96</td>
<td>8.87</td>
<td>3.51</td>
<td>8.77</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>1.59</td>
<td>4.91</td>
<td>--*</td>
<td>3.51</td>
<td></td>
</tr>
</tbody>
</table>

* The two suspenders adjacent to damaged area fail under dead load before any live load is applied when 5 suspenders are removed.
6. CALIBRATION OF LOAD FACTORS FOR USE IN DETERMINISTIC ANALYSIS

Because reliability-based progressive collapse analyses are difficult to implement in routine bridge engineering practice, following the GSA (2000) procedures for the progressive collapse analysis of buildings, bridge engineers have been using a push down analysis to study the capacity of a damaged system and comparing the capacity of the damaged system to carry some level of live load in addition to the dead loads. A dynamic amplification factor is also used to account for the release of energy that may accompany a sudden removal of members susceptible to extreme hazards. However, no recommendations are currently available to advise engineers on the criteria, dynamic amplification, or safety factors that should be used to assess the safety against progressive collapse of bridges (Miao and Ghosn, 2013). It is herein argued that these criteria can be established using risk analysis principles. For example, the relationships between the live load multiplier that can be used to determine whether the bridge has sufficient levels of safety when performing a nonlinear static pushdown analysis and the damaged system’s reliability index are shown in Figure 4. Each curve is obtained by comparing the reliability index of the member or system for the three damage scenarios of one, three or five hangers suddenly removed and the results of a static push down analysis where the design live load is incremented until either member or system failure. For example, if the member reliability index is found to be equal to $\beta_{\text{member}} = 4.5$ as a result of the sudden failure of one suspender, then structural safety is assured by verifying that under a static pushdown analysis with one suspender removed, the bridge will be able to sustain 4 times the design live load before the next suspender fails. Similarly, a system reliability index $\beta_{\text{system}} = 5.3$ would correspond to the case where the nonlinear static pushdown analysis would require 4 times the design load on a damaged system to cause collapse. These live load factors can then be used by a bridge engineer to check the safety against next member failure or system collapse by executing a static nonlinear pushdown analysis without having to perform a dynamic reliability analysis.

![Figure 4: Live load factor versus reliability index for different bridge damage scenarios](image)

7. RISK ANALYSIS METHOD FOR ESTABLISHING TARGET RELIABILITY LEVELS

The target reliability levels can be established using a risk analysis procedure using the probability of system failure given an initial damaged caused by an extreme hazard and the expected consequence of failure expressed in monetary terms from an equation of the form (Ellingwood, 2005):

$$\text{Risk} = E(\text{cost}) = \sum_{h} \sum_{d} P(C/D)P(D/H)P(H) \times \text{consequence}$$

Eq. (1)

Where $H$ is the hazard, $D$ is the expected damage due to hazard $H$ and $C$ is the system failure due to damage $D$. By summing over all possible hazards and damage scenarios risk is calculated as the expected cost. In this example, we assume that bridge suspenders could fail due to metal fatigue with a probability $P(H) = 1.75 \times 10^{-2}$
corresponding to a reliability index $\beta = 2.5$ which is the normal target for the design of steel bridge members. Additionally, we study the probability that the bridge will be subject to an accidental plane impact in an active airport region such as New York metropolitan area. In this case the rate of an impact from a plane is estimated to be $P(H) = 1.5\%$ over a 100-yr period. Depending on the plane size, it is estimated that the probability of a single suspender failure is 30%, the probability of three suspenders failing is estimated to be 40% and the probability of five suspenders is 30%. For each of these cases, the reliability analysis estimates the probability that another suspender fails or the probability of unzipping of the all the suspenders leading to catastrophic system collapse. The consequence of bridge failure is obtained using the following equation (NCHRP web-only document 107):

$$Cost = C_eWL + \left[ C_2 \left( 1 - \frac{T}{100} \right) + C_3 \left( 1 - \frac{T}{100} \right) \right] DAd + \left[ C_4 \left( 1 - \frac{T}{100} \right) + C_5 \left( 1 - \frac{T}{100} \right) \right] \frac{DAd}{S} + C_eX$$

Eq. (2)

where $D$=detour length, $A$=average daily traffic, $d$=duration of detour, $S$=average detour speed, $T$=average daily truck traffic (ADTT), $O$=average occupancy rate, $e$=cost multiplier for early replacement, $W$=bridge width and $L$=bridge length.

The equation is the sum of costs related to the following issues: the cost of reconstruction (unit cost $C_1$), detour-related consumer costs ($C_2$, $C_3$, $C_4$, and $C_5$), and the potential cost of fatalities ($C_6$). For more details please refer to NCHRP web-only document 107.

The risk analysis follows the fault tree shown in Figure 5. By summing all the consequences for each branch of the tree the total estimated cost is obtained for the original bridge design where the suspenders were detailed using a safety Factor S.F. =3.0. Similar analyses were conducted for different safety factors and the costs of failure versus safety factor are obtained as shown in Figure 6. To determine the optimum safety factor for design, the cost of failure curve is compared to the cost of constructing the suspenders to higher standards which is also shown in Figure 6. The optimal design lies at the intersection of the two curves. Showing that at a safety factor S.F. =3.5, the consequences of failure are equal to the cost of the suspenders to meet that safety factor requirement.

Table 3 lists the reliability indices that correspond to each type of failure obtained from the fault tree corresponding to the optimum safety factor S.F. =3.5. Figure 4 is used to extract the corresponding live load factors as listed in the last two columns of Table 3. These live load factors can be used in engineering practice in a deterministic performance based design method that will meet an optimum risk objective.
8. CONCLUSIONS
This paper described a procedure to evaluate the reliability of suspension bridges and perform a probabilistic analysis of progressive collapse where one or more suspenders are suddenly damaged.

Because performing reliability analyses are beyond the day-to-day practice of bridge engineers, a methodology is presented that can be used to calibrate progressive collapse analysis criteria to help bridge engineers perform traditional deterministic nonlinear pushdown analyses to assess the ability of a bridge system to resist progressive collapse.

The calibration process is illustrated for an example bridge designed with various suspenders’ safety factors and for different damage scenarios. The paper outlines how deterministic risk-informed and performance-based design criteria can be developed for routine use in bridge engineering practice.

9. REFERENCES


