Reliability assessment of a typical steel truss bridge

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SYNOPSIS

The paper presents the structural reliability analysis of a multilane steel truss bridge to assess the safety of the bridge, both in its current configuration and with the addition of a new lane. Probabilistic models of tensile and compressive resistance, as well as dead load, were obtained from existing literature. The probabilistic model of the peak 50-year live load was derived from traffic survey data obtained from the bridge site which enabled an Extreme Value Type I (EV-Type 1 or Gumbel) distribution to be fitted to the upper tail of the distribution of load effects due to traffic loading. The reliability analysis also included: (i) loading model uncertainty which allows for uncertainty in the selection of load and the structural analysis, and (ii) the effect of resistance updating based on proven service performance.

The reliability analysis found that the reliability index for the current bridge is 3.1. The reliability index for the new clip-on bus lane reduces to 2.9. If the weakest structural member is strengthened by 10%, 15% or 20% then the reliability indices increase to 3.18, 3.26 and 3.30 respectively. The reliability indices are compared to the target reliabilities recommended in the Australian Standards. This paper aims to determine the realistic bridge load capacity and the appropriate strengthening to carry maximum traffic load without minimising risk. The structural reliability analysis provides very useful risk management tool for assessing the safety of existing bridges.
1. INTRODUCTION

Reliability-based safety assessment is used frequently in Europe, and the Danish Roads Directorate (DRD). DRD is one of the few authorities to provide very specific guidelines on the reliability-based assessment of existing bridges (DRD, 2004). While deterministic safety assessments are appropriate for most bridge assessments, if an assessment recommend bridge closure, load restriction or extensive and costly strengthening it is often useful to undertake a more detailed reliability-based assessment. For example, the DRD now pursues reliability-based assessment as a matter of course for all structures that have failed a deterministic assessment and probability-based assessments on 11 bridges has saved the DRD over $35 million (O’Connor and Enevoldsen 2007). Note that Australian Standards now provide guidance on reliability-based assessment of existing structures (AS5104-2005, AS ISO 13822-2005).

2. BACKGROUND AND BRIDGE DESCRIPTION

The Roads and Traffic Authority of NSW (RTA) manages more than 5000 bridges in its network and 86 of them are steel truss bridges as shown in Figure 1B. These bridges were built over 125 years using various materials, technology and five different design standards. These steel truss bridges are exposed to different environments and subject to increased traffic loading and frequencies. There 86 steel truss bridges and there distribution by design age are shown in Figure 1B.

The Bridge over Iron Cove is a major bridge on a major arterial route in Sydney. It was built in 1955 to accommodate 4 traffic lanes. It has 4 plate girder spans and 7 steel truss spans. In 1970s, one external traffic lane was added to the upstream side of the bridge, mostly for the use of buses. This external lane is supported on cantilevers from the original bridge.

The elevation and cross-section of the bridge are shown in Figure 2 and 3 below. The sequence of span is 2 x 18m continuous plate girders, 7 x 52m steel trusses simply supported and 2 x 18m continuous plate girders. The plate girders spans...
consist of two main built-up girders supporting the cross girders. The truss spans have 7 panels each and the truss members are built from welded members.

![Figure 2: Bridge Elevation](image1)
![Figure 3: Cross section of Bridge looking towards Sydney](image2)
Generally, strengthening of bridges is carried out in accordance with the current 1996 AUSTROADS Bridge Design Code ('96 ABDC). However, because of the earlier studies conducted in assessing the bridge, it was evident that strengthening the bridge as per '96 ABDC would have been exceptionally expensive. In addition, it was thought that the bridge would not experience the live loads stipulated in the '96 ABDC for the next 50 years or during its expected life.

Therefore, the RTA Bridge Engineering proposed a realistic method of determining live loads for strengthening the bridge for legal loads based on the current legal loads experienced by the bridge, the probability of multiple presences of legal loads and the predicted future growth of legal loads over the route. Based on traffic survey at bridge site, the probability of occurrences for worst load combination has been estimated and subsequently recommended the future design legal load combination for this multiple lane bridge. This approach will result a significant saving in the strengthening of the bridge.

In order to determine the reliability of this approach for a steel truss bridge on a critical network it was decided to conduct a reliability assessment of the critical members. The reliability analysis is limited to the truss members only (see Figure 3), and not to the deck or other structural members. The capacity and reliability of connections, as well as deterioration and fatigue, are not considered in this study.

This paper discusses the use of the approach and it is believed that the use of such an approach would be beneficial in minimising strengthening of other similar bridges.

![Figure 4. Truss](image)

2. PROBABILISTIC MODELLING OF RESISTANCE

The probabilistic modelling of resistance for structural steel members is dominated by the statistics (mean, standard deviation, distribution type) for yield strength $f_y$. As yield strength is a highly variable parameter, it is often recommended that tests or samples be taken from the assessed structure to better characterise the yield strengths for the structure under consideration (e.g. Diamantidis 2001). The incorporation of test data into a Bayesian statistical framework often results in the selection of higher mean strengths with lower Coefficient of Variation (COV). The RTA have done limited testing of the bridge elements and the calculated recommended yield strength varies between 230-255 MPa based on Hardness testing. Hence, in the analysis to follow, the specified yield strength for all truss elements is assumed to be 230 MPa.
2.1 Tension Capacity

The sources of variability for tension yielding (T) are model error, yield strength and cross-sectional area (Pham 1987). It follows that \( \text{mean}(T) = 1.17 N_t \) and \( \text{COV}(T) = 0.10 \) where \( N_t \) is the design tension yielding capacity specified by AS5100.6-2004. The tensile capacity is lognormally distributed.

2.2 Compression Capacity

The sources of variability for compression capacity (C) are model error, yield strength and cross-sectional area (Pham and Bridge 1985). The model error is dependent on member slenderness ratio. The statistics provided by Pham and Bridge (1985) were based on working stress code provisions (AS1250-1981), rather than the current limit state codes AS4100-1998 and AS5100.6-2004. The statistics provided Pham and Bridge (1985) are corrected for limit state design capacities \( N_c \) obtained from AS5100.6-2004, see Table 3. These statistics will be used in the reliability analysis to follow. The axial compression capacity is lognormally distributed. The top chords have \( L_e/r < 46.3 \), whereas all other compression members have slenderness ratio within the limits \( 46.3 < L_e/r < 92.6 \). While tensile and axial compression strength statistics were derived for reliability-based calibration of hot rolled sections, Ellingwood et al. (1980) shows that statistics for plate girders are not significantly different than those for hot rolled sections.

3. LOAD MODELLING

3.1 Dead Loads

Dead load is the permanent load comprising the weight of the structure. For structural assessment purposes, the Danish Road Directorate guidelines for reliability assessment of existing structures (DRD 2004) recommends that:

1. Dead load of structure: \( \text{mean}(G) = G_n \) \( \text{COV}(G) = 0.05 \)
2. Superimposed dead load (asphalt) \( \text{mean}(G_w) = G_{nW} \) \( \text{COV}(G_w) = 0.10 \)

where \( G_n \) and \( G_{nW} \) are the design (specified) dead loads of the structure and superimposed loads, respectively. The dead load is normally distributed. In the present analysis, information about the contribution of asphalt self-weight to total dead load is not available. It is expected that asphalt self-weight is small compared to total dead weight so the variability of total dead load is assumed similar to that recommended for the dead load of the structure.

3.2 Live Loads

Traffic live load modelling is complicated by the high number of traffic lanes and mix of heavy vehicular traffic. Frameworks for the probabilistic modelling of extreme loading, such as peak load effects over 50-100 years, are quite well developed and used for assessment safety decisions (e.g. DRD 2004). While generic load and traffic data may be used, it is preferable if detailed information about the frequency of single and multiple vehicle presence, variability of individual vehicle load, peak load effect caused by each vehicle class in each lane, and growth in traffic volume and vehicle
mass is used. Load history is required for the future or intended service life, as well as from the time of bridge opening to the present day (to assess effect of service proven performance on updating resistance modelling). Such information requires multiple structural analyses for many combinations of vehicle type and location. The reliability-based load-rating procedure for existing bridges proposed by Reid (2004) is potentially a useful framework. However, the statistical information required for bridges with multiple lanes with a large mix of vehicle types is difficult to implement in practice for reasons given above. Weigh-in motion (WIM) data is also recommended to characterise the likelihood of vehicle mass higher than that for the design vehicle, and the extent of such overloading. The integration of generic and site-specific traffic and loading data into a structural reliability assessment is an area for further study.

As WIM data is not available for the bridge, this analysis will require some approximations and assumptions to enable development of a probabilistic model for live loading. The RTA have provided the worst five load cases of semi-trailers, B-doubles and buses that will cause maximum loads on the bridge. For each load case the RTA calculated member forces and the probability of each of these load cases occurring. This will enable an Extreme Value Type I (EV-Type 1 or Gumbel) distribution to be fitted to the upper tail of the distribution of member forces due to traffic loading. It is the upper tail of the distribution of member forces that has the most influence on structural reliability. As the time period for operation of the bridge is until 2058, a probabilistic model of the peak 50-year live load is needed.

The reliability analysis also considers the effect of resistance updating based on prior service proven performance (e.g., Stewart and Val 1999). The RTA appears to have little statistics on traffic mix, frequency and loading for the years since bridge opening to the present date. It is reasonable to assume that as traffic volume and mass has been increasing since bridge opening in 1955, the bridge is most likely to have experienced its peak vehicle live load in the past five to ten years. As such, the peak vehicle live load since 1955 is based on a time period of five years. The RTA traffic data assumes a 50% increase in traffic volume over present levels, but this effect is likely to be outweighed by considering only a five year time period for a bridge that has experienced over 50 years of service proven performance.

Axial forces were calculated for the western truss (supports clip-on bus lane) for the five most heavily loaded load cases LC1 to LC5. The load effect data provided by the RTA used a Dynamic Load Allowance ($D$) = 0.0 and Accompany Lane Factor of 1.0. The RTA also provided data on the probability of load case occurrence for reference periods of 10 to 1,000 years obtained from their traffic survey. These probabilities of occurrence are likely to be conservative (high) as the load cases are based on a vehicle speed of 20 km/hr to obtain highest multiple presence of heavy vehicles at any position on different lanes of a span (Ariyaratne et al. 2004). The probabilities of occurrence for five load cases are summarised in Table 2, for time periods of five and 50 years.
Table 1. Probabilities of Occurrence for Five Load Cases.

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Probability of Occurrence in 50 Years (bus + 4 lane traffic)</th>
<th>Probability of Occurrence in 5 Years¹ (bus + 4 lane traffic)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LC1</td>
<td>0.01537</td>
<td>0.001537</td>
</tr>
<tr>
<td>LC2</td>
<td>0.02952</td>
<td>0.002952</td>
</tr>
<tr>
<td>LC3</td>
<td>0.00015</td>
<td>0.000015</td>
</tr>
<tr>
<td>LC4</td>
<td>0.01457</td>
<td>0.001457</td>
</tr>
<tr>
<td>LC5</td>
<td>0.00015</td>
<td>0.000015</td>
</tr>
</tbody>
</table>

¹Modelled as a Poisson Process – Probability is 10% of Pr(50 years).

The Gumbel distribution is often used for live load modelling (Pham 1985; AS5104-2005) and so it is assumed that traffic live load will also follow a Gumbel distribution. For each member, one Load Case (LC) produces the largest force in a member. Hence, it is assumed that the probability of exceeding this action in a 50 year period is the probability of occurrence for that load case. For example, for vertical member U2L2 LC5 produces the largest member force, so the probability of exceeding this action in a 50 year period is 0.00015 (see Table 2). As occurrence of load cases is not mutually exclusive (i.e., events are independent), it follows that the probability of exceeding the smallest force (of the five load cases) in a 50 year period is 0.0499. This is based on the well known principle:

\[
\Pr(A \cup B) = \Pr(A) + \Pr(B) - \Pr(A \cap B)
\]  

Gumbel statistical parameters for each western truss member can then be derived if two percentiles are known. In this case, the upper percentile is taken from the load case for the maximum force (out of the five load cases), and the 95\textsuperscript{th} percentile is taken as the minimum force out of the five load cases. RTA traffic data was also used to estimate probability of exceeding the largest force and smallest force (out of five load cases) for five-year peak loadings needed to update resistance based on service proven performance. For example, Figure 2 shows the resulting probability distribution of 5-year and 50-year peak live loads for the vertical member U2L2.
The member forces provided by the RTA are based on current general access vehicle T42.5t and BD65t vehicle loads. Whether an analysis based on worst combinations of access vehicle loads is more conservative than one that considers probabilistic modelling of actual individual vehicle loads (including the potential for higher non-legal loads) is not clear.

It should be noted that there is considerable uncertainty with the developed probabilistic model for traffic live loading. This model has been based on design loads, whereas information regarding variability of vehicle mass and likelihood and extent of overloading is also preferable. The following section considers loading model uncertainty, which should help capture some of the sources of traffic load variability not considered in this section.

### 3.3 Loading Model Uncertainty

A model uncertainty (ME) exists for dead and live loads. This allows for uncertainty in the selection of load and the structural analysis. The Danish Road Directorate guidelines for reliability assessment of existing structures (DRD 2004) recommends the statistics (normally distributed) given in Table 2. The statistics assume that there is no bias (systematic variation) in the modelling techniques used.
Traffic volumes are multiplied by a factor of 1.5 to account for future traffic growth and some inaccuracies in traffic survey (Ariyaratne et al. 2004). This factor is unlikely to fully capture the likelihood of overloading and variability of heavily loaded vehicles – information essential in a reliability analysis. Hence, it is reasonable to assume that there is high uncertainty in traffic modelling.

3.4 Dynamic Load Allowance ($\alpha$)

The Dynamic Load Allowance ($\alpha$) used for the load rating of the bridge is 40%. However, this is conservative as the load cases are based on a vehicle speed of 20 km/hr. To be consistent with reliability-based code calibration procedures, statistics used for development of U.S. AASHTO LRFD specifications are used (Hwang and Nowak 1991): mean($\alpha$)=1.15, COV($\alpha$)=0.10. The distribution of $\alpha$ is normally distributed and truncated at 1.0.

4. RELIABILITY ANALYSIS

4.1 Computational Procedure

Failure of a structural element occurs when the load effect ($S$) exceeds the resistance ($R$). The probability of failure ($p_f$) is

$$p_f = Pr(R \leq S) = \int_{0}^{\infty} F_R(r)f_S(r)dr$$

(2)

where $R$ and $S$ are statistically independent random variables, $f_S(r)$ is the probability density function of the load and $F_R(r)$ is the cumulative probability density function of the resistance. For many realistic problems the simplified formulation given by Eqn. (2) is not sufficient as the limit state function often contains more than two variables. Equation (2) can be generalised to

$$p_f = \Phi(-\beta) = Pr(G(X) \leq 0)$$

(3)

where $\Phi$ is the standard normal distribution function, $G(X)$ is the “limit state function” [defines “failure”: in the present case this is equal to $R-S$], the vector $X$ represents the basic variables in the limit state and $\beta$ is the “reliability index”. For reliability-based safety assessment the reliability index is used as the measure of safety.
In the present case, the limit state for the $i^{th}$ member in a truss is

$$G_i(X) = R_i - ME_G G_i - ME_Q \alpha Q_{50i}$$ (4)

where $Q_{50}$ is the peak 50-year live load. The statistical parameters are summarised in Table 3.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Mean</th>
<th>COV</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Capacity (R)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension</td>
<td>1.17 N_t</td>
<td>0.10</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Compression (L_e/r&lt;46.3)</td>
<td>1.19 N_c</td>
<td>0.15</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Compression (46.3&lt;L_e/r&lt;92.6)</td>
<td>1.22 N_c</td>
<td>0.16</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Dead Load (G)</td>
<td></td>
<td>0.05</td>
<td>Normal</td>
</tr>
<tr>
<td>Peak 50-year Live Load (Q_{50})</td>
<td>Appendix F</td>
<td></td>
<td>Gumbel$^+$</td>
</tr>
<tr>
<td>Peak 5-year Live Load (Q_5)</td>
<td>Appendix F</td>
<td></td>
<td>Gumbel$^+$</td>
</tr>
<tr>
<td>Dead Load Model Uncertainty (ME_G)</td>
<td>1.0</td>
<td>0.10</td>
<td>Normal</td>
</tr>
<tr>
<td>Live Load Model Uncertainty (ME_Q)</td>
<td>1.0</td>
<td>0.20</td>
<td>Normal</td>
</tr>
<tr>
<td>Dynamic Load Allowance (a)</td>
<td>1.15</td>
<td>0.10</td>
<td>Normal$^*$</td>
</tr>
</tbody>
</table>

Note: $^*$ Truncated at $a=1.0$, $^+$ Truncated at zero.

Table 3. Summary of Statistical Parameters for Reliability Analysis.

As the loads applied on each member or the capacity of each member are correlated, then assessing the reliability of a truss system is complicated. In this case, the precise probability of failure for the system cannot be obtained unless sophisticated simulation methods are used. Hence, event-based Monte Carlo simulation methods will be used herein. The truss is a non-redundant structure where failure of any member means significant structural damage or collapse of the bridge – this makes the truss bridge a series system. It follows that the probability of failure is

$$p_f = 1 - \Pr[G_1(X) > 0 \cap G_2(X) > 0 \cap \ldots \cap G_n(X)]$$ (5)

where $G_i(X)$ is given by Eqn. (5) and $n$ is the number of elements in both trusses that support the span. Failure of any member in either of these trusses will lead to significant structural damage and possibly collapse of the bridge. As member forces have been provided for the ten most heavily loaded members per truss, then $n=20$.

The loads in truss members will be strongly correlated as a high vehicle load will cause high actions in most truss members. Hence, in the Monte Carlo simulation analysis the actions in all members in both truss are assumed fully correlated. The resistance in each truss member is statistically independent. When considering dead and live loads the member forces in the eastern truss are reduced by 28% (dead loads) and 20% (live loads).
Service loads (dead loads and live/traffic loading) may be treated as a proof load with uncertainty and so the updated distribution of structural resistance is

\[
f_R(r) = \frac{F_S^T(r)f_R'(r)}{\int_{-\infty}^{\infty} F_S^T(r)f_R'(r) \, dr}
\]

where \(F_S^T(r)\) is the cumulative distribution function of the maximum actual load effect experienced during service proven performance and \(f_R'(r)\) is the distribution of resistance before the structure is loaded (statistics for \(R\) given in Table 6), see Stewart and Val (1999). The maximum load effect is based on the past five years of service proven performance - i.e. dead load (\(G\)) + peak 5-year live load (\(Q_5\)).

4.2 Suggested Target Reliability Index (\(\beta_T\))

The Australian Standard ‘General Principles on Reliability of Structures’ AS5104-2005 suggests that the lifetime target reliability index (\(\beta_T\)) is 3.1 to 4.3 for ultimate (strength) limit states design (see Table 4). In structures where there is little redundancy a higher target reliability index may be selected. Such target values are ‘informative’ only, as the selection of the target reliability level depends on the different parameters such as type and importance of the structure, possible failure consequences, socio-economic criteria, etc. The term ‘relative costs of safety measures’ referred to in Table 4 is the cost needed to reduce the risk and its effect on the target reliability index is an indirect measure of cost versus benefit of risk reduction. Ultimately, it is the responsibility of the asset owner to decide on the level of risk acceptability for its assets.

<table>
<thead>
<tr>
<th>Relative Costs of Safety Measures</th>
<th>Consequences of Failure</th>
<th>Small</th>
<th>Some</th>
<th>Moderate</th>
<th>Great</th>
</tr>
</thead>
<tbody>
<tr>
<td>High</td>
<td>0</td>
<td>1.5</td>
<td>2.3</td>
<td>3.1</td>
<td></td>
</tr>
<tr>
<td>Moderate</td>
<td>1.3</td>
<td>2.3</td>
<td>3.1</td>
<td>3.8</td>
<td></td>
</tr>
<tr>
<td>Low</td>
<td>2.3</td>
<td>3.1</td>
<td>3.8</td>
<td>4.3</td>
<td></td>
</tr>
</tbody>
</table>

Table 4. Suggested Lifetime Target Reliabilities (\(\beta_T\)) (AS5104-2005)
5. RESULTS

5.1 System Reliability

The reliability index ($\beta$) for the structural system (bridge) is $\beta = 3.10$. Table 5 shows the proportion of failure for each member. Table 5 shows that 77.8% of bridge failures are due to failure of the second vertical member U2L2 in the western truss. This member is clearly critical to bridge reliability. As the eastern truss experiences lower member forces then it is not surprising that very few members (0.54%) in the eastern truss fail.

The reliability results are sensitive to the load modelling. If predicted loads are increased by 20% then the reliability index reduces to 2.70. If predicted loads are decreased by 20% then the reliability index increases to 3.55. The reliability results are also sensitive to the updating of resistance based on service proven performance. For example, if the probabilistic model of service proven traffic load is increased from a 5 year to a 10 year period, then the reliability index increases from, 3.10 to 3.17.

<table>
<thead>
<tr>
<th>Members</th>
<th>Member</th>
<th>Percentage of Failures</th>
</tr>
</thead>
<tbody>
<tr>
<td>Western Truss:</td>
<td>End Post</td>
<td>LoU1 5.85 %</td>
</tr>
<tr>
<td></td>
<td>Top Chord</td>
<td>U1U2 2.31 %</td>
</tr>
<tr>
<td></td>
<td>Top Chord</td>
<td>U2U3 5.99 %</td>
</tr>
<tr>
<td></td>
<td>Top Chord</td>
<td>U3U3' 4.16 %</td>
</tr>
<tr>
<td></td>
<td>Vertical</td>
<td>U1L1 0.00 %</td>
</tr>
<tr>
<td></td>
<td>Vertical</td>
<td>U2L2 77.83 %</td>
</tr>
<tr>
<td></td>
<td>Vertical</td>
<td>U3L3 0.00 %</td>
</tr>
<tr>
<td></td>
<td>Inside Diagonal</td>
<td>U1L2 3.03 %</td>
</tr>
<tr>
<td></td>
<td>Inside Diagonal</td>
<td>U2L3 1.18 %</td>
</tr>
<tr>
<td></td>
<td>Inside Diagonal</td>
<td>U3L3' 0.00 %</td>
</tr>
</tbody>
</table>

Table 5. Proportion of Failures for Each Member.

5.2 Member Reliability

The reliability index for a 50 year service life was also calculated for each member, see Table 6. The Live Load Factor (LLF) calculated by the RTA is shown for comparative purposes (Ariyaratne et al. 2004) where a LLF of less than 2.0 indicates need for strengthening. If the target reliability index is assumed as $\beta_T = 4.3$, then Table 6 also shows the amount of strengthening needed for a member so that its reliability equals $\beta_T$. Table 6 shows that where the LLF is less than 2.0 the reliability index is also less than the target value. However, there is no clear link between LLF and strengthening needed. For example, a low LLF of 1.3 (which according to this metric indicates a least safe member) requires only very minor (1-4%) strengthening. Whereas members with a higher LLF of 1.7 actually require more strengthening (7-
This occurs because of the higher uncertainties associated with prediction of compressive capacity in the top chords (when compared to tensile capacity of the diagonal members), leading to lower reliabilities for compression members and so more strengthening to reach the target reliability. As expected, the most critical member identified in Table 5 (vertical U2L2) requires the largest amount of strengthening. If a shorter service life is chosen then amount of strengthening may be reduced as reliabilities would increase.

<table>
<thead>
<tr>
<th>Members</th>
<th>Member</th>
<th>Tension (T) / Compression (C)</th>
<th>Live Load Factor (LLF)</th>
<th>Member Reliability Index (β)</th>
<th>Member is Safe (β&gt;4.3)</th>
<th>Member Strengthening (β=4.3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Western Truss:</td>
<td>End Post</td>
<td>C</td>
<td>-</td>
<td>3.86</td>
<td>10%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Top Chord</td>
<td>U1U2</td>
<td>C</td>
<td>1.7</td>
<td>4.09</td>
<td>5%</td>
</tr>
<tr>
<td></td>
<td>Top Chord</td>
<td>U2U3</td>
<td>C</td>
<td>1.7</td>
<td>3.85</td>
<td>10%</td>
</tr>
<tr>
<td></td>
<td>Top Chord</td>
<td>U3U3’</td>
<td>C</td>
<td>1.7</td>
<td>3.95</td>
<td>7%</td>
</tr>
<tr>
<td></td>
<td>Vertical</td>
<td>U1L1</td>
<td>T</td>
<td>-</td>
<td>5.75</td>
<td>β</td>
</tr>
<tr>
<td></td>
<td>Vertical</td>
<td>U2L2</td>
<td>C</td>
<td>1.1</td>
<td>3.18</td>
<td>28%</td>
</tr>
<tr>
<td></td>
<td>Vertical</td>
<td>U3L3</td>
<td>C</td>
<td>1.7</td>
<td>6.30</td>
<td>β</td>
</tr>
<tr>
<td></td>
<td>Inside Diagonal</td>
<td>U1L2</td>
<td>T</td>
<td>1.3</td>
<td>4.03</td>
<td>4%</td>
</tr>
<tr>
<td></td>
<td>Inside Diagonal</td>
<td>U2L3</td>
<td>T</td>
<td>1.3</td>
<td>4.24</td>
<td>1%</td>
</tr>
<tr>
<td></td>
<td>Inside Diagonal</td>
<td>U3L3’</td>
<td>C</td>
<td>-</td>
<td>6.45</td>
<td>β</td>
</tr>
</tbody>
</table>

Table 6. Member Reliability Indices (β) and Required Strengthening so that β=4.3.

CONCLUSIONS:

This paper aims to determine the realistic bridge load capacity and the appropriate strengthening to carry maximum traffic load without minimising risk. The structural reliability analysis provides very useful risk management tool for assessing the safety of existing bridges.

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11 DISCLAIMER

The opinions expressed in this paper are entirely those of the authors, and do not necessarily represent the Policy of the Roads and Traffic Authority of NSW.

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