

Load Rating of Impaired Bridges Using a Dynamic Method

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ABSTRACT: Local Government in Australia is responsible for the operational management and maintenance of over 20,000 bridges. More than 70% of these bridges comprise aging timber bridges, the load capacity and structural adequacy of many of which have been impaired over time. This is partly due to increased vehicular loads with little attention to consequence of such increases. It is now necessary to determine the load carrying capacity of these bridges using simple yet reliable methods to allow local authorities to upgrade, replace or sign post at-risk bridges. In this paper a novel dynamic based method is presented by which the in-service stiffness of the bridge is estimated first. From this stiffness the load carrying capacity of the bridge is estimated following a statistically based analysis.

1 INTRODUCTION

A major challenge facing Local Government in Australia is to develop effective strategies for the maintenance and rehabilitation of the extensive timber bridge stocks which form a key component of the road network under its control. Raising the efficiency and reliability of bridge maintenance practices of local government has the potential not only to minimize costly unscheduled emergency repairs, but also to reduce the overall maintenance costs, whilst improving the operational effectiveness of its road network.

The field testing of several timber bridge spans in NSW has been undertaken successfully using a novel and simple dynamic method to estimate the in-service stiffness of the bridge, from which its load carrying capacity is estimated. Coupled with specially developed analysis software, the method provides a measure of the structural adequacy of the bridge and a reliable basis for devising appropriate maintenance or remedial measures.

2 BASIC CONCEPTS UNDERPINNING THE PROPOSED TESTING PROCEDURE

The proposed dynamic bridge assessment procedure involves the attachment of a few accelerometers underneath the bridge girders and the measurement of the vibration response of the bridge superstructure unloaded and with one or more loads (such as a truck, water tanker, grader, concrete blocks, etc, of known weight) applied at midspan. The excitation is generated by a modal impact hammer. The resulting

dynamic responses are measured with uniaxial accelerometers which are robust and simple to attach. The data is logged and the bridge deck properties evaluated, using a dynamic signal analyser or a standard computer with special software.

Two sets of bending frequencies are measured for the bridge, 'as is', and when loaded by the extra weight. By loading the bridge, the bending frequency of the bridge decreases. From the resulting frequency shift due to added weight, the flexural stiffness of the bridge can be calculated.

User friendly software has also been developed which allows the estimation of bridge load carrying capacity from calculated stiffness, adopting a statistically based approach. The proposed test does not require the precise measurement of deformations, as is the case for static load tests. It is also much quicker to conduct compared with load testing, and hence less expensive and much more affordable than load testing. It is also safer than load testing, particularly with respect to old bridges where applying a large load may further jeopardise the integrity of the bridge.

3 TEST RESULTS FOR A TYPICAL BRIDGE

As a direct result of the modal analysis, the dynamic properties of a bridge, such as the natural frequencies, damping ratio and mode shapes, can be obtained. However, the proposed dynamic method requires only the first flexural natural frequency for both with and without added mass cases. Figures 1 and 2 show the comparison of Frequency Response

Functions (FRFs) with and without added mass for a two span tested bridge, respectively.

2.6 8.65 20.2

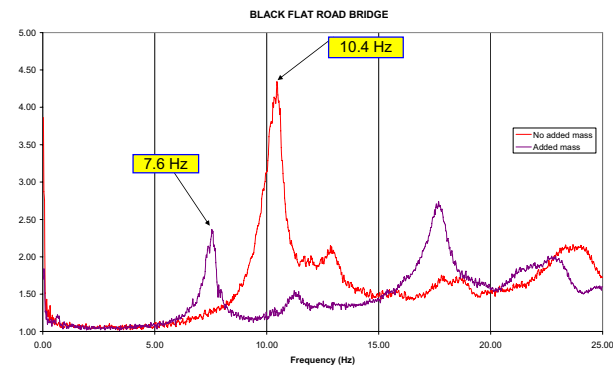


Figure 1. Comparison of sum FRFs for span 1 with and without added mass

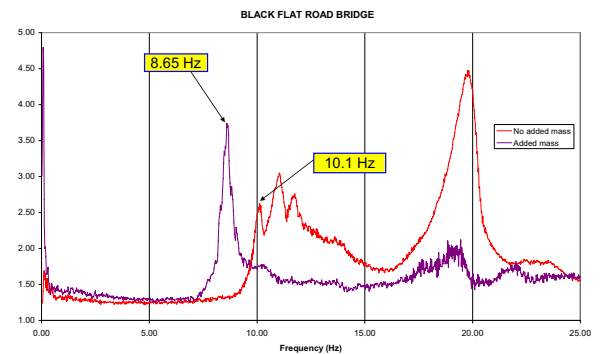


Figure 2. Comparison of sum FRFs for span 2 with and without added mass

3.1 Flexural Stiffness of the Tested Bridge

With added mass given, and frequencies with and without the mass known, the flexural stiffness of the tested bridge can be easily calculated. Table 1 shows the amount of mass added, the first natural frequency with and without mass and prediction of the flexural stiffness of tested bridge.

The first span stiffness is calculated as 12.7 kN/mm and the corresponding calculated stiffness for span 2 is 20.2 kN/mm. It can be seen that the stiffness results for both spans are different, partly due to varying span length.

3.2 Load Capacity of the Tested Bridge

The determination of strength of in-service bridge girders is extremely difficult and complex, unless of course the girder is broken and the failure load and loading pattern is known.

Table 1 - Results of predicted stiffness using the proposed dynamic method

Span No.	Mass added (tonnes)	First natural frequency (Hz)	Predicted stiffness (kN/mm)
1	0	10.4	12.7
	2.6	7.6	
2	0	10.1	

Current “best practice” in Australia generally assumes that the fibre strength of any girder is 80 to 100 MPa (depending upon the species). Bending capacity is predicted by multiplying the assumed section modulus "Z" (based on the gross section) by the assumed fibre strength.

Proof loading of timber bridges is expensive and inherently risky, since it is a well established fact that high load levels cause permanent and irrecoverable damage to the wood fibres. This may result in subsequent failure of a timber girder at load levels significantly less than that indicated by the proof test. It is for this reason that most rating procedures of timber bridges have been based on applying serviceability load levels (such as from a water tanker), measuring the deflections in order to estimate the stiffness and then using an assumed relationship between strength and stiffness to predict the load carrying capacity of each girder.

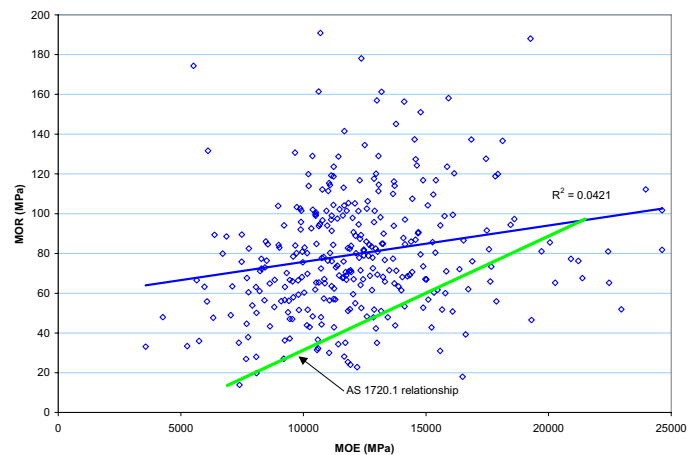


Figure 3. MOR vs MOE for round timbers

The relationship between strength and stiffness used in current load assessment methods is based on the assumed relationship between Modulus of Rupture (MOR) and Modulus of Elasticity (MOE) defined in the Australian Timber Structures Code, AS1720.1. However, it is not commonly understood that this relationship is both idealized and theoretical. Figure 3, illustrates the problems associated with this approach. This figure presents a plot of MOR vs MOE data obtained from full scale testing of round timbers extracted from service with an average life of 30 years and also compares this with the AS1720 relationship. It is obvious from the linear regression co-efficient for the test data that the relationship between strength and stiffness for aged poles/girders is not statistically significant. Furthermore, the theoretical relationship assumed in AS1720 is not reliable for these timbers, as many round timbers have a rupture strength significantly

lower than that predicted by the Code relationship. For example, extensive testing of some 1200 round timber poles indicates that the actual 5th percentile strengths for strength group 1 & 2 timbers range between 30 and 55 MPa, not 80 to 100 MPa as previously assumed.

In order to assess the strength of timber bridge girders with any degree of reliability, it is necessary to develop strength models, which reflect the actual bending capacity of timber. This should take into account the uncertainties associated with determination of the geometric section properties and the actual strength properties. Such a model has been developed to form the basis of the proposed load testing system developed in this paper. Using test data obtained from extensive testing of full scale round timbers, a relationship between actual measured stiffness (EI) and actual bending capacity has been derived, with correlation coefficients in the range 0.4 to 0.5 for lower bound 5th percentiles, as indicated in Figure 4.

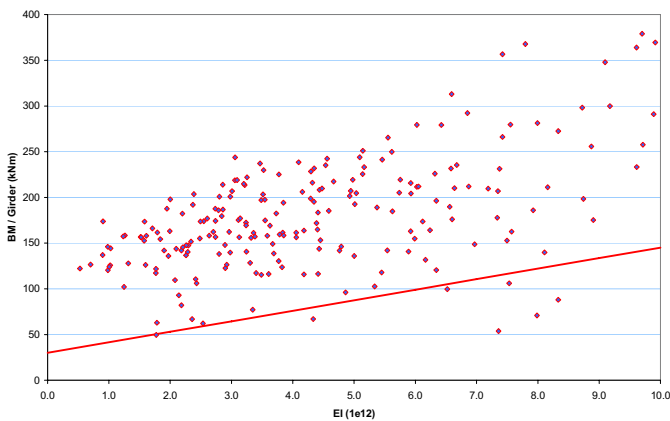


Figure 4. Bending strength vs gross stiffness for round timbers

Using a probabilistic approach, this relationship can be used in reliability-based models to predict the load capacity of a deck from the stiffness data obtained from the dynamic frequency method, with acceptable and transparent degrees of uncertainty.

Applying the probabilistic approach described above, the estimated live load factor (defined as the ratio of the net factored moment capacity and the moments, including live load allowance, caused by a T44 truck weighing 42.5 tonnes per lane) is 1.2 for span 1 and 1.5 for span 2. This means that the maximum load carrying capacity of the bridge is estimated at 50 tonnes.

4 PRACTICAL CONSIDERATIONS

4.1 A new exciter for larger bridges

The method proposed above is not limited to timber bridges. The method has been extended to test other and larger bridges made of concrete and steel. To do

so a modal hammer is usually incapable of imparting enough energy to excite the bridge, hence a larger exciter in the form of a drop mass was designed by researchers at the University of Technology Sydney (Figure 5). This was undertaken to increase the precision of the measured modal frequencies.

As mentioned, the modal hammer has been proven to excite timber bridges adequately but in the case of concrete, steel or other stiff bridges, did not provide enough energy to adequately excite the lower frequency spectrum of 5 to 30 Hz. Thus, the first vibration modes of the structures have not been as clear, and as definable, as desired.

The new exciter consists of a PCB 200C50 Quartz Impact Force Sensor to sense the force and frequency applied to the bridge from the exciter; a safe lifting mass of 20kg to provide the impact; replaceable rubber tips to provide low frequency excitation and reduce second impact; and a steel frame and base plate. The mass is guided by three vertical rods to where it impacts the sensor at the bottom of the guides. The force is then transferred to the structure through the base plate. The mass can be dropped from any height up to 1.5m depending on the level of excitation needed.

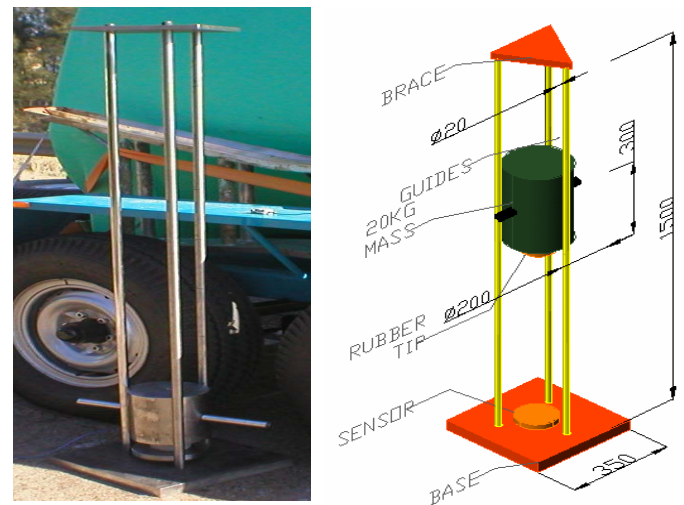


Figure 5. The new exciter and its schematic

4.2 Alternate locations for added mass

Based on the Dynamic Frequency Analysis (DFA) procedure, prediction of the flexural stiffness of the bridge requires determination of the frequency shift produced as a result of added mass, which is distributed at midspan of the bridge. Theoretically speaking, the DFA procedure should produce accurate prediction of the flexural stiffness of any given bridge. However, in practical situations, a few problems do arise in field testing which may severely impact on the accuracy of the predictions using a DFA procedure. In addition, the possibility of varying the added mass locations may have

significant benefits in terms of reducing the testing time and easing the traffic flows during testing. In the following, a few possible field testing situations are described, which may adversely impact on the results obtained from the standard ‘midspan mass addition’ procedure:

- (i). Due to the existence of the kurbs, added mass cannot be completely and uniformly distributed along the midspan;
- (ii). The physical size of the blocks precludes them as being point masses as assumed in the theory; in addition, for practical convenience it may be required to arrange the blocks in a more distributed fashion;
- (iii). Some extreme situations, such as when the test bridge is in a very poor condition (eg, some old and deteriorating timber bridge), may prohibit the use of a crane truck to carry mass blocks to the midspan. The only possible location for adding mass may be quite far from the midspan of the bridge;
- (iv). As using a trailer as added mass has practical advantages. The mass of a trailer cannot be distributed evenly along the midspan, and hence, the impact of this on the accuracy of results needs to be investigated.
- (v). There are possible arrangements for adding mass on the bridge which may have advantages in terms of easing traffic flow. These need to be also considered.

In summary, an investigation of the resulting modal mass for each arrangement of added mass is required to assess the accuracy of stiffness predictions, due to these variations, and to find possible solutions to further improve the accuracy for various practical situations. The following section attempts to address this particular issue.

4.3 Calculation of Flexural Stiffness with Mass Located at Midspan

The principle of the DFA procedure in bridge testing has been discussed extensively in previous papers by authors (eg, Champion, et al. 2002; Crews, et al. 2004a,b,c; Crews et al. 2005; Li et al. 2004; Li et al. 2005; Samali, et al. 2002) and hence this paper will avoid unnecessary details. In summary, by knowing the natural frequencies before and after adding mass, the flexural stiffness of the bridge can be estimated as follows:

$$k = \frac{\omega_1^2 \omega_2^2}{\omega_1^2 - \omega_2^2} \Delta M \quad (1)$$

where k is the flexural stiffness of the bridge, ΔM is the total added mass and ω_1 and ω_2 are, respectively, the natural frequencies before and after adding mass in rad/sec.

In this case, because the added mass, ΔM , is located at the midspan of a bridge, and as it contributes directly to modal mass of the first mode, there is no need for mass compensation.

4.4 Calculation of Flexural Stiffness with Modal Mass Compensation

Considering the theory of DFA, it is clear that the first modal mass is the key for improving the accuracy of the method. The current DFA procedure requires the ‘added mass’ to be positioned at midspan to ensure the first modal mass of the structure is increased by the total added mass. In Equation 1, if ΔM is considered as equivalent first modal mass instead of physical ‘added mass’, it is then possible to increase the accuracy of the stiffness prediction for various scenarios mentioned above. To determine the modal mass for each case of added mass, the added mass matrix is normalized with respect to the mode shape. The modal mass is therefore calculated from,

$$\hat{m}_i = \phi_i^T \Delta M \phi_i \quad (2)$$

where ΔM is the added lumped mass matrix and ϕ_i the mode shape corresponding to the added mass mode.

In Equation 1, the added mass ΔM can then be replaced with the modal mass \hat{m}_i to yield the following,

$$k = \frac{(2\pi)^2 f_1^2 f_2^2}{f_1^2 - f_2^2} \Delta \hat{m}_i \quad (3)$$

Equation 3 can predict the flexural stiffness with the added mass, compensated with the modal mass $\Delta \hat{m}_i$. In Equation 3, f_1 and f_2 are, respectively, the natural frequencies of the bridge before and after adding mass but expressed in Hertz.

4.5 Case Study - a previously tested bridge

To confirm and evaluate the proposed modal compensation method, a five girder composite steel/concrete bridge was selected for modelling and calculation. The bridge over Redbank creek in New South Wales near Sydney was chosen for this purpose.

The composite steel/concrete bridge structure was built in 1945, and consists of three simply supported spans of length 10.46m, 10.67m and 10.46m, respectively, with a carriageway width of 6.1m. The concrete slab, with an average thickness of 160mm, is supported by five RSJ (22”x7”) girders spaced at

1.37 meter centres. Transverse reinforced concrete diaphragms are located at each support and at midspan (Figure 6).



Figure 6. Underside of Redbank Creek bridge

Microstran computer program was chosen to model the bridge structure due to its relative simplicity. The bridge structure was modelled using space frame elements with six degrees of freedom at each node. The space frame elements were chosen over the grillage elements as they provide greater flexibility in modelling the composite bridge structure.

The structural elements modelled included the Rolled Steel Joist (RSJ) girders, the reinforced concrete deck and the transverse reinforced concrete diaphragms. The concrete kerbs and barriers were not modelled as it was assumed that their contribution to the overall global stiffness is negligible.

The same model was later used to estimate the load carrying capacity of the bridge using current code provisions and after calibrating it using the experimental stiffness results.

4.5.1 Modelling Assumptions

The modeling of the composite action between the steel girders and the concrete deck was through additional dummy members. These dummy members model the composite action through a rigid vertical member that connects the steel girder to the concrete deck rigidly. These members have a relatively large moment of inertia compared to surrounding members. This ensures compatibility of deformation is maintained through the rigid connection.

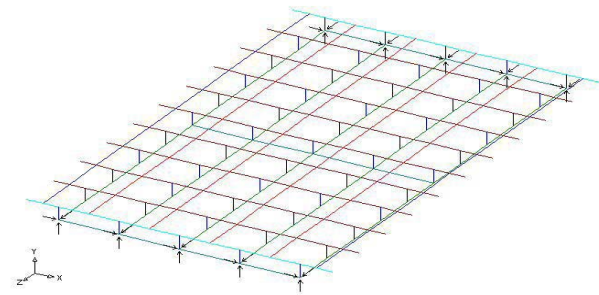


Figure 7. Microstran model of Redbank Creek bridge

The concrete deck is modelled using a grid of beams in the X-Z plane. The total area of these beams equals the total area of the concrete deck. This is important in the dynamic analysis as it ensures that the mass of the model is equal to the actual mass of the concrete deck. The grid work assumes the relative stiffness the deck provides is equal in both the X and Z directions. This is a reasonable assumption as the concrete deck is uniform in depth across the entire bridge.

The structure is assumed to be simply supported and, therefore, the supports are modelled with simple pinned supports. This ignores any contributing effects from adjoining spans as only one typical span is modelled.

4.5.2 Material Properties

The material properties adopted for the model were:

Concrete: $E = 30,000 \text{ MPa}$	Modulus of Elasticity
$\nu = 0.15$	Poisson's ratio
$\rho = 2.4 \text{ tonne/m}^3$	Mass Density

Steel: $E = 200,000 \text{ MPa}$	Modulus of Elasticity
$\nu = 0.33$	Poisson's ratio
$\rho = 7.85 \text{ tonne/m}^3$	Mass Density

4.5.3 Static analysis

A linear elastic static analysis of Redbank Creek Bridge by Microstran will determine a value for the global flexural stiffness of the structure. Static loading was modelled as point loads over each girder at midspan (Figure 8).

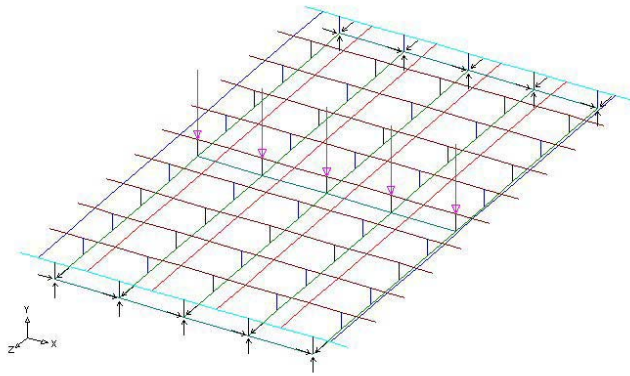


Figure 8. Static load applied to the model at midspan

The determination of the flexural stiffness of the bridge structure through static means is the familiar force displacement relationship where

$$k = \frac{\text{Load}}{\text{Displacement}} = 98,355 \text{ kN/m} \quad (4)$$

The results of the static analysis are shown in Table 2.

Table 2. Results of the static analysis

Girder	Load	Vertical Displacement
	kN	mm
1	1,000	50.8
2	1,000	50.9
3	1,000	50.9
4	1,000	50.9
5	1,000	50.8
Total	5,000	50.8

4.5.4 Dynamic analysis

The dynamic analysis requires the stiffness and mass matrices to be determined from the Microstran model. Forty five ($n = 45$) nodes or DOF (Figure 9) were used in the analysis.

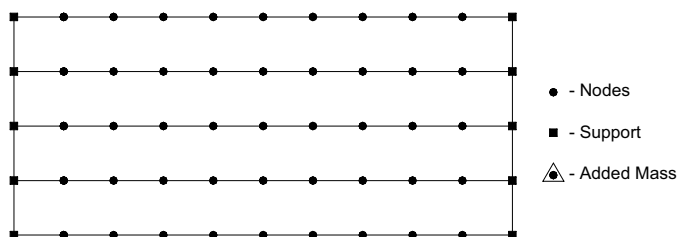


Figure 9. Bridge model with $n = 45$ DOF

4.6 Added mass cases

The determination of flexural stiffness of the bridge structure is through a shift in the natural frequency. This frequency shift is due to the additional mass on the structure. The standard DFA procedure requires the added mass to be evenly distributed at midspan. The possibility of varying the location of the added mass offers significant and practical benefits to the testing procedure. A total of ten varying patterns of added mass have been investigated. The common factor to each type of added mass is that the total additional mass remains constant to ensure the results are comparable. For the case investigated here the total added mass is 10.6 tonnes.

4.6.1 Type 1 added mass

The added mass is lumped at midspan over each girder. This is based on the current theory where the total added mass increases the modal mass of the structure (Figure 10).

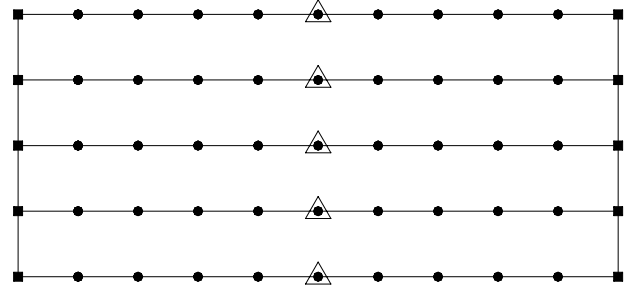


Figure 10. Type 1 added mass

4.6.2 Type 2 added mass

The added mass is lumped at midspan but only on half the width with girders 1, 2 and 3 loaded. The advantage of such a configuration is the ability to allow traffic to continue through one lane as the structure is loaded and unloaded. This reduces the bridge closure time, hence allowing bridges with large traffic volumes to be tested with minimal disruption to traffic (Figure 11).

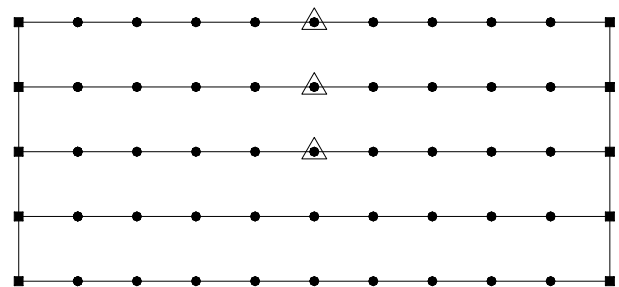


Figure 11. Type 2 added mass

4.6.3 Type 3 added mass

The added mass is identical to that of Type 2, only mirrored. The position of the added mass is on the opposite side (Figure 12).

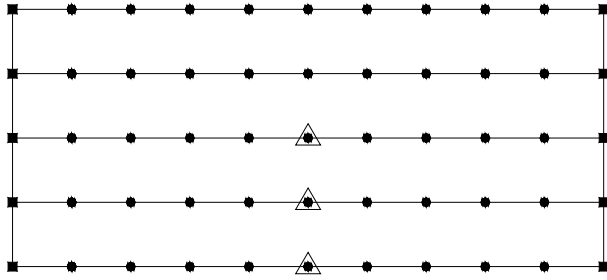


Figure 12. Type 3 added mass

4.6.4 Type 4 added mass

The added mass is located at 1/4 spans on girders 2 and 4. This type of loading represents a vehicle which may be used as the added mass. The total lumped mass is proportioned to each wheel location. This greatly reduces the loading and unloading time of the bridge (Figure 13).

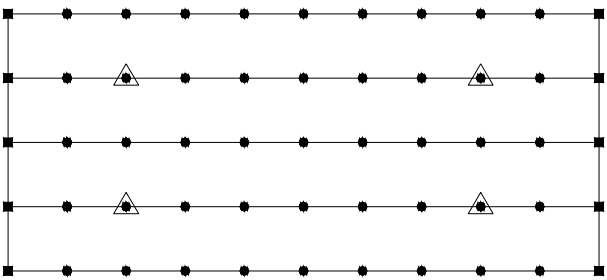


Figure 13. Type 4 added mass

4.6.5 Type 5 added mass

The added mass is located at midspan but only on the central girders 2, 3 and 4. This type of loading is considered for bridge structures that do not allow a lumped mass on the exterior girders. This may be due to the location of the barriers or kerbs on the bridge (Figure 14).

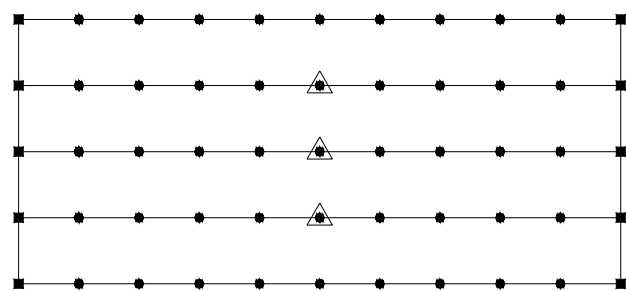


Figure 14. Type 5 added mass

4.6.6 Type 6 added mass

The added mass is distributed over the entire structure. The total added mass is distributed and

lumped at every DOF. This is only considered for theoretical benchmarking (Figure 15).

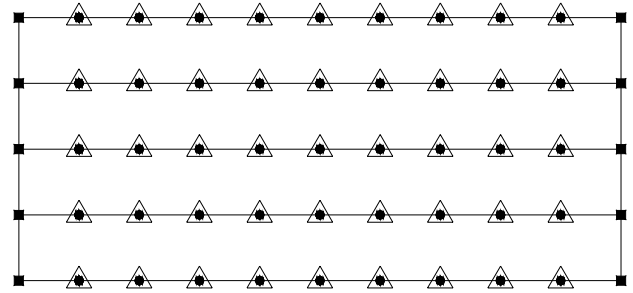


Figure 15. Type 6 added mass

4.6.7 Type 7 added mass

The added mass is distributed to each girder as in Type 1, but offset from midspan. This type of loading has practical implications in that the lumped mass may not be able to be placed exactly at midspan; therefore, this type of loading will investigate the sensitivity of the current method, to the loading location (Figure 16).

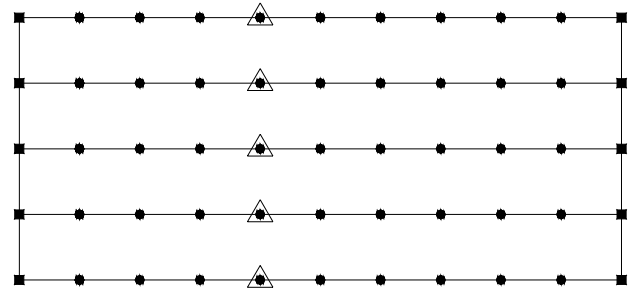


Figure 16. Type 7 added mass

4.6.8 Type 8 added mass

The added mass is identical to that of Type 7 only with a greater offset from midspan. This case has a theoretical value when investigating the results (Figure 17).

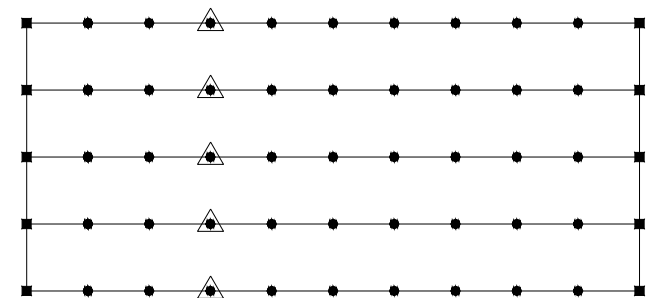


Figure 17. Type 8 added mass

4.6.9 Type 9 added mass

The added mass is located with two masses on girders 2 and 4 on either side of midspan. They are

closely spaced and resemble the added mass of wheels from a trailer or vehicle. The advantage of this type of added mass configuration is in the reduction of time to load and unload the bridge (Figure 18).

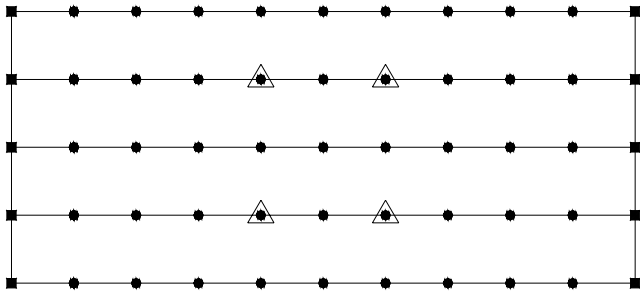


Figure 18. Type 9 added mass

4.6.10 Type 10 added mass

The added mass is located at midspan but only on the exterior girders. This configuration would limit the disruption to traffic, allowing a single lane through the centre of the bridge to remain open (Figure 19).

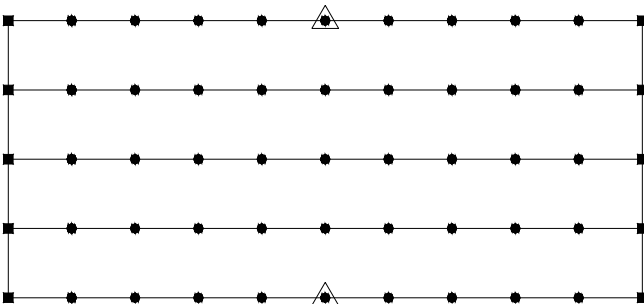


Figure 19. Type 10 added mass

4.7 Natural Frequency and Mode Shapes

The natural frequencies and mode shapes are derived using the MATLAB program. The number of natural frequencies calculated will equal the number of DOF. Therefore, for the case $n = 45$ there will be forty five natural frequencies and corresponding mode shapes calculated. The first ten frequencies are of most interest to the analysis and hence are shown in Table 3. The higher frequencies and corresponding mode shapes become too complex and somewhat irrelevant.

The first frequency represents the first flexural or bending mode, the second frequency represents the first torsion or twisting mode and the third frequency represents the transverse flexural or bending mode. These are the basic representations of the modes with all higher modes exhibiting some correlation to the fundamental modes.

Table 3. Calculated natural frequencies for Redbank Creek bridge model

Mode	Natural Frequency, Hz
1	12.00
2	12.98
3	31.66
4	39.40
5	41.34
6	47.46
7	57.33
8	59.05
9	76.50
10	79.56

4.8 Natural Frequency and Mode Shapes with Added Mass

The natural frequencies and mode shapes are calculated using the MATLAB program for each type of added mass. The results for only the first ten modes are shown in Table 4.

Table 4. Natural frequencies for ten cases of added mass

Mode	Natural Frequency (Hz)				
	Type 1	Type 2	Type 3	Type 4	Type 5
1	9.47	8.77	8.77	10.94	9.41
2	10.26	11.83	11.83	12.35	11.81
3	25.69	25.65	25.65	29.79	28.16
4	39.40	39.40	39.40	30.55	39.40
5	41.34	41.34	41.34	31.99	41.34
6	47.46	47.46	47.46	41.12	47.46
7	50.54	51.48	51.48	46.29	49.35
8	59.04	59.04	59.04	48.49	59.04
9	71.25	68.79	68.79	56.45	69.06
10	72.96	71.86	71.86	65.61	70.88

Mode	Natural Frequency (Hz)				
	Type 6	Type 7	Type 8	Type 9	Type 10
1	10.42	9.64	10.15	9.63	8.74
2	11.30	10.45	11.00	11.46	9.33

3	27.49	25.30	25.39	30.25	26.59
4	34.04	36.92	33.35	34.90	39.40
5	35.76	38.71	34.95	37.50	41.34
6	41.03	44.84	40.07	44.61	47.46
7	49.59	44.95	41.79	46.64	54.61
8	50.99	58.44	52.67	50.90	59.04
9	66.01	60.97	58.14	67.65	69.28
10	68.70	77.95	78.12	69.11	70.22

Added	10.6	10.6	10.6	10.6	10.6
Mass (t)					
Stiffness	185	110	151	109	92.2
(MN/m)					
Error (%)	85.5%	10.2%	51.6%	9.4%	-7.5%

The MATLAB program performs the required transformation and calculation to determine the FRF for the case of bridge ‘as is’ and for each case of added mass. It is from these FRFs that the frequency shift required to determine the flexural stiffness is measured.

The predictions of the flexural stiffness for each added mass case are presented in Table 5. The frequency shift error is expressed considering Type 1 added mass as the benchmark. This is also the case for the stiffness error calculation.

It can be seen that added mass cases 5, 9 and 10, in addition to benchmark case 1, can produce accurate results with errors of less than 7%. This provides lots of flexibility when choosing the most appropriate and convenient mass locations without compromising the overall accuracy.

5 CONCLUSION

A new method, based on dynamic response of bridges to an impact load, is proposed to measure the in-service flexural stiffness of bridges. Utilizing a statistically based analysis, the knowledge of flexural stiffness can be converted into an estimate of the load carrying capacity of the bridge (for timber bridges). The reliability and simplicity of the proposed methodology has been demonstrated by testing over 200 bridge spans covering a wide range of single and multi-span timber bridges. The results pertaining to two spans of one of these bridges are reported in this paper, along with the underlying principles and methodology adopted.

The methodology was refined and extended to larger concrete and steel bridges but using the same principles. This necessitated the design and fabrication of a larger exciter capable of imparting larger impacts to the bridge to excite it.

The sensitivity of varying mass locations on the accuracy of measured frequency and hence the predicted stiffness was also investigated. It is found that in addition to the optimal location to add mass, other alternatives also exist which allow the addition of mass with more ease and less obstruction to traffic without sacrificing the accuracy of results.

For the case of larger concrete and steel bridges, the developed computer model is also used to estimate the load carrying capacity of the bridge using current code provisions and after calibrating it using the experimental stiffness results.

4.9 Frequency Response Function of the Bridge

The transient analysis of the bridge model with the impact force imparted by a tuned modal impact hammer, allows the calculation of the acceleration responses of the structure. The acceleration response data, when processed using the fast Fourier transform (FFT) allows the Frequency Response Function (FRF) to be obtained. This method replicates the Dynamic Frequency Analysis (DFA) procedure applied to bridge structures and allows the determination of the first flexural frequencies from the appropriate FRF.

Table 5. Flexural stiffness prediction for different mass types

	as is	Type 1	Type 2	Type 3	Type 4	Type 5
Frequency (Hz)	12.00	9.47	8.77	8.77	10.94	9.41
Shift (%)	0.0%	26.6%	36.8%	36.8%	9.7%	27.5%
Error (%)	N/A	0.0%	38.0%	38.0%	-63%	3.2%
Added Mass (t)		10.6	10.6	10.6	10.6	10.6
Stiffness (MN/m)		99.7	69.2	69.2	296	96.3
Error (%)	N/A	0.0%	-30.0%	-30.0%	197%	-3.5%
	Type 6	Type 7	Type 8	Type 9	Type 10	
Frequency (Hz)	10.42	9.64	10.15	9.63	9.33	
Shift (%)	15.1%	24.4%	18.2%	24.6%	28.6%	
Error (%)	-43%	-8.4%	-31%	-7.8%	7.2%	

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Load Rating of Impaired Bridges Using a Dynamic Method

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ABSTRACT: Local Government in Australia is responsible for the operational management and maintenance of over 20,000 bridges. More than 70% of these bridges comprise aging timber bridges, the load capacity and structural adequacy of many of which have been impaired over time. This is partly due to increased vehicular loads with little attention to consequence of such increases. It is now necessary to determine the load carrying capacity of these bridges using simple yet reliable methods to allow local authorities to upgrade, replace or sign post at-risk bridges. In this paper a novel dynamic based method is presented by which the in-service stiffness of the bridge is estimated first. From this stiffness the load carrying capacity of the bridge is estimated following a statistically based analysis.

1 INTRODUCTION

A major challenge facing Local Government in Australia is to develop effective strategies for the maintenance and rehabilitation of the extensive timber bridge stocks which form a key component of the road network under its control. Raising the efficiency and reliability of bridge maintenance practices of local government has the potential not only to minimize costly unscheduled emergency repairs, but also to reduce the overall maintenance costs, whilst improving the operational effectiveness of its road network.

The field testing of several timber bridge spans in NSW has been undertaken successfully using a novel and simple dynamic method to estimate the in-service stiffness of the bridge, from which its load carrying capacity is estimated. Coupled with specially developed analysis software, the method provides a measure of the structural adequacy of the bridge and a reliable basis for devising appropriate maintenance or remedial measures.

2 BASIC CONCEPTS UNDERPINNING THE PROPOSED TESTING PROCEDURE

The proposed dynamic bridge assessment procedure involves the attachment of a few accelerometers underneath the bridge girders and the measurement of the vibration response of the bridge superstructure unloaded and with one or more loads (such as a truck, water tanker, grader, concrete blocks, etc, of known weight) applied at midspan. The excitation is generated by a modal impact hammer. The resulting

dynamic responses are measured with uniaxial accelerometers which are robust and simple to attach. The data is logged and the bridge deck properties evaluated, using a dynamic signal analyser or a standard computer with special software.

Two sets of bending frequencies are measured for the bridge, 'as is', and when loaded by the extra weight. By loading the bridge, the bending frequency of the bridge decreases. From the resulting frequency shift due to added weight, the flexural stiffness of the bridge can be calculated.

User friendly software has also been developed which allows the estimation of bridge load carrying capacity from calculated stiffness, adopting a statistically based approach. The proposed test does not require the precise measurement of deformations, as is the case for static load tests. It is also much quicker to conduct compared with load testing, and hence less expensive and much more affordable than load testing. It is also safer than load testing, particularly with respect to old bridges where applying a large load may further jeopardise the integrity of the bridge.

3 TEST RESULTS FOR A TYPICAL BRIDGE

As a direct result of the modal analysis, the dynamic properties of a bridge, such as the natural frequencies, damping ratio and mode shapes, can be obtained. However, the proposed dynamic method requires only the first flexural natural frequency for both with and without added mass cases. Figures 1 and 2 show the comparison of Frequency Response

Functions (FRFs) with and without added mass for a two span tested bridge, respectively.

2.6 8.65 20.2

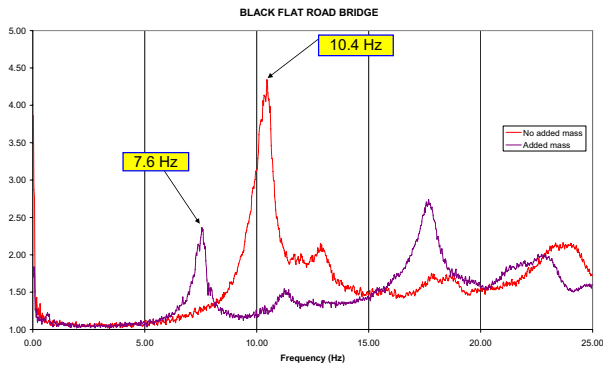


Figure 1. Comparison of sum FRFs for span 1 with and without added mass

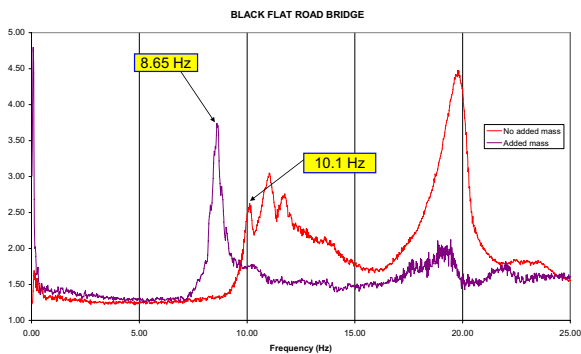


Figure 2. Comparison of sum FRFs for span 2 with and without added mass

3.1 Flexural Stiffness of the Tested Bridge

With added mass given, and frequencies with and without the mass known, the flexural stiffness of the tested bridge can be easily calculated. Table 1 shows the amount of mass added, the first natural frequency with and without mass and prediction of the flexural stiffness of tested bridge.

The first span stiffness is calculated as 12.7 kN/mm and the corresponding calculated stiffness for span 2 is 20.2 kN/mm. It can be seen that the stiffness results for both spans are different, partly due to varying span length.

3.2 Load Capacity of the Tested Bridge

The determination of strength of in-service bridge girders is extremely difficult and complex, unless of course the girder is broken and the failure load and loading pattern is known.

Table 1 - Results of predicted stiffness using the proposed dynamic method

Span No.	Mass added (tonnes)	First natural frequency (Hz)	Predicted stiffness (kN/mm)
1	0	10.4	12.7
	2.6	7.6	
2	0	10.1	

Current “best practice” in Australia generally assumes that the fibre strength of any girder is 80 to 100 MPa (depending upon the species). Bending capacity is predicted by multiplying the assumed section modulus "Z" (based on the gross section) by the assumed fibre strength.

Proof loading of timber bridges is expensive and inherently risky, since it is a well established fact that high load levels cause permanent and irrecoverable damage to the wood fibres. This may result in subsequent failure of a timber girder at load levels significantly less than that indicated by the proof test. It is for this reason that most rating procedures of timber bridges have been based on applying serviceability load levels (such as from a water tanker), measuring the deflections in order to estimate the stiffness and then using an assumed relationship between strength and stiffness to predict the load carrying capacity of each girder.

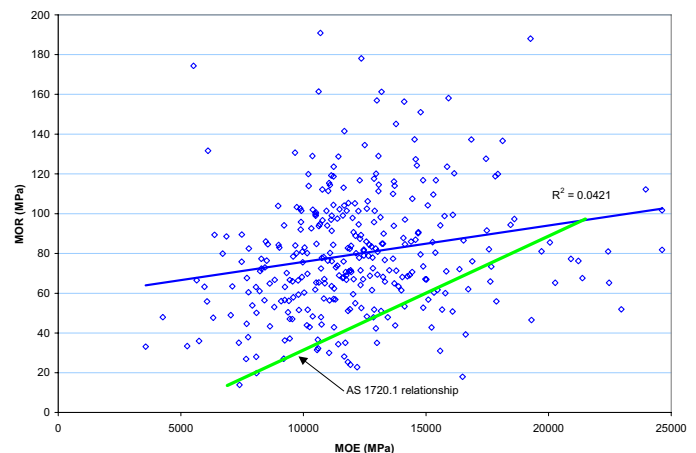


Figure 3. MOR vs MOE for round timbers

The relationship between strength and stiffness used in current load assessment methods is based on the assumed relationship between Modulus of Rupture (MOR) and Modulus of Elasticity (MOE) defined in the Australian Timber Structures Code, AS1720.1. However, it is not commonly understood that this relationship is both idealized and theoretical. Figure 3, illustrates the problems associated with this approach. This figure presents a plot of MOR vs MOE data obtained from full scale testing of round timbers extracted from service with an average life of 30 years and also compares this with the AS1720 relationship. It is obvious from the linear regression co-efficient for the test data that the relationship between strength and stiffness for aged poles/girders is not statistically significant. Furthermore, the theoretical relationship assumed in AS1720 is not reliable for these timbers, as many round timbers have a rupture strength significantly

lower than that predicted by the Code relationship. For example, extensive testing of some 1200 round timber poles indicates that the actual 5th percentile strengths for strength group 1 & 2 timbers range between 30 and 55 MPa, not 80 to 100 MPa as previously assumed.

In order to assess the strength of timber bridge girders with any degree of reliability, it is necessary to develop strength models, which reflect the actual bending capacity of timber. This should take into account the uncertainties associated with determination of the geometric section properties and the actual strength properties. Such a model has been developed to form the basis of the proposed load testing system developed in this paper. Using test data obtained from extensive testing of full scale round timbers, a relationship between actual measured stiffness (EI) and actual bending capacity has been derived, with correlation coefficients in the range 0.4 to 0.5 for lower bound 5th percentiles, as indicated in Figure 4.

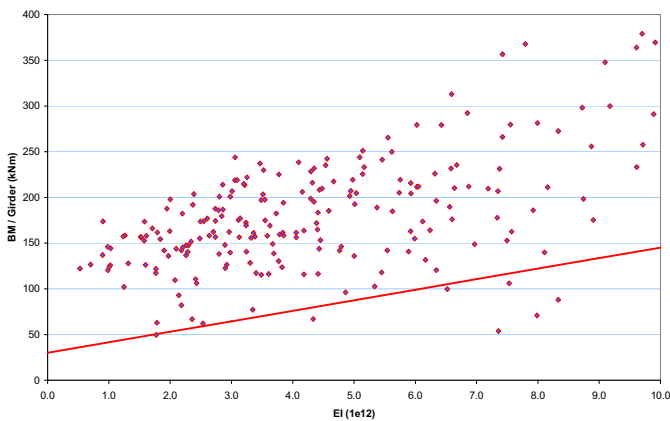


Figure 4. Bending strength vs gross stiffness for round timbers

Using a probabilistic approach, this relationship can be used in reliability-based models to predict the load capacity of a deck from the stiffness data obtained from the dynamic frequency method, with acceptable and transparent degrees of uncertainty.

Applying the probabilistic approach described above, the estimated live load factor (defined as the ratio of the net factored moment capacity and the moments, including live load allowance, caused by a T44 truck weighing 42.5 tonnes per lane) is 1.2 for span 1 and 1.5 for span 2. This means that the maximum load carrying capacity of the bridge is estimated at 50 tonnes.

4 PRACTICAL CONSIDERATIONS

4.1 A new exciter for larger bridges

The method proposed above is not limited to timber bridges. The method has been extended to test other and larger bridges made of concrete and steel. To do

so a modal hammer is usually incapable of imparting enough energy to excite the bridge, hence a larger exciter in the form of a drop mass was designed by researchers at the University of Technology Sydney (Figure 5). This was undertaken to increase the precision of the measured modal frequencies.

As mentioned, the modal hammer has been proven to excite timber bridges adequately but in the case of concrete, steel or other stiff bridges, did not provide enough energy to adequately excite the lower frequency spectrum of 5 to 30 Hz. Thus, the first vibration modes of the structures have not been as clear, and as definable, as desired.

The new exciter consists of a PCB 200C50 Quartz Impact Force Sensor to sense the force and frequency applied to the bridge from the exciter; a safe lifting mass of 20kg to provide the impact; replaceable rubber tips to provide low frequency excitation and reduce second impact; and a steel frame and base plate. The mass is guided by three vertical rods to where it impacts the sensor at the bottom of the guides. The force is then transferred to the structure through the base plate. The mass can be dropped from any height up to 1.5m depending on the level of excitation needed.

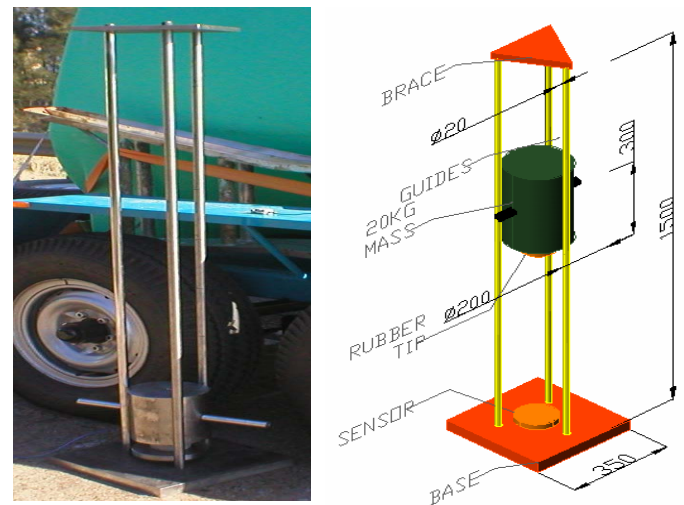


Figure 5. The new exciter and its schematic

4.2 Alternate locations for added mass

Based on the Dynamic Frequency Analysis (DFA) procedure, prediction of the flexural stiffness of the bridge requires determination of the frequency shift produced as a result of added mass, which is distributed at midspan of the bridge. Theoretically speaking, the DFA procedure should produce accurate prediction of the flexural stiffness of any given bridge. However, in practical situations, a few problems do arise in field testing which may severely impact on the accuracy of the predictions using a DFA procedure. In addition, the possibility of varying the added mass locations may have

significant benefits in terms of reducing the testing time and easing the traffic flows during testing. In the following, a few possible field testing situations are described, which may adversely impact on the results obtained from the standard ‘midspan mass addition’ procedure:

- (i). Due to the existence of the kurbs, added mass cannot be completely and uniformly distributed along the midspan;
- (ii). The physical size of the blocks precludes them as being point masses as assumed in the theory; in addition, for practical convenience it may be required to arrange the blocks in a more distributed fashion;
- (iii). Some extreme situations, such as when the test bridge is in a very poor condition (eg, some old and deteriorating timber bridge), may prohibit the use of a crane truck to carry mass blocks to the midspan. The only possible location for adding mass may be quite far from the midspan of the bridge;
- (iv). As using a trailer as added mass has practical advantages. The mass of a trailer cannot be distributed evenly along the midspan, and hence, the impact of this on the accuracy of results needs to be investigated.
- (v). There are possible arrangements for adding mass on the bridge which may have advantages in terms of easing traffic flow. These need to be also considered.

In summary, an investigation of the resulting modal mass for each arrangement of added mass is required to assess the accuracy of stiffness predictions, due to these variations, and to find possible solutions to further improve the accuracy for various practical situations. The following section attempts to address this particular issue.

4.3 Calculation of Flexural Stiffness with Mass Located at Midspan

The principle of the DFA procedure in bridge testing has been discussed extensively in previous papers by authors (eg, Champion, et al. 2002; Crews, et al. 2004a,b,c; Crews et al. 2005; Li et al. 2004; Li et, al. 2005; Samali, et al. 2002) and hence this paper will avoid unnecessary details. In summary, by knowing the natural frequencies before and after adding mass, the flexural stiffness of the bridge can be estimated as follows:

$$k = \frac{\omega_1^2 \omega_2^2}{\omega_1^2 - \omega_2^2} \Delta M \quad (1)$$

where k is the flexural stiffness of the bridge, ΔM is the total added mass and ω_1 and ω_2 are, respectively, the natural frequencies before and after adding mass in rad/sec.

In this case, because the added mass, ΔM , is located at the midspan of a bridge, and as it contributes directly to modal mass of the first mode, there is no need for mass compensation.

4.4 Calculation of Flexural Stiffness with Modal Mass Compensation

Considering the theory of DFA, it is clear that the first modal mass is the key for improving the accuracy of the method. The current DFA procedure requires the ‘added mass’ to be positioned at midspan to ensure the first modal mass of the structure is increased by the total added mass. In Equation 1, if ΔM is considered as equivalent first modal mass instead of physical ‘added mass’, it is then possible to increase the accuracy of the stiffness prediction for various scenarios mentioned above. To determine the modal mass for each case of added mass, the added mass matrix is normalized with respect to the mode shape. The modal mass is therefore calculated from,

$$\hat{m}_i = \phi_i^T \Delta M \phi_i \quad (2)$$

where ΔM is the added lumped mass matrix and ϕ_i the mode shape corresponding to the added mass mode.

In Equation 1, the added mass ΔM can then be replaced with the modal mass \hat{m}_i to yield the following,

$$k = \frac{(2\pi)^2 f_1^2 f_2^2}{f_1^2 - f_2^2} \Delta \hat{m}_i \quad (3)$$

Equation 3 can predict the flexural stiffness with the added mass, compensated with the modal mass $\Delta \hat{m}_i$. In Equation 3, f_1 and f_2 are, respectively, the natural frequencies of the bridge before and after adding mass but expressed in Hertz.

4.5 Case Study - a previously tested bridge

To confirm and evaluate the proposed modal compensation method, a five girder composite steel/concrete bridge was selected for modelling and calculation. The bridge over Redbank creek in New South Wales near Sydney was chosen for this purpose.

The composite steel/concrete bridge structure was built in 1945, and consists of three simply supported spans of length 10.46m, 10.67m and 10.46m, respectively, with a carriageway width of 6.1m. The concrete slab, with an average thickness of 160mm, is supported by five RSJ (22”x7”) girders spaced at

1.37 meter centres. Transverse reinforced concrete diaphragms are located at each support and at midspan (Figure 6).



Figure 6. Underside of Redbank Creek bridge

Microstran computer program was chosen to model the bridge structure due to its relative simplicity. The bridge structure was modelled using space frame elements with six degrees of freedom at each node. The space frame elements were chosen over the grillage elements as they provide greater flexibility in modelling the composite bridge structure.

The structural elements modelled included the Rolled Steel Joist (RSJ) girders, the reinforced concrete deck and the transverse reinforced concrete diaphragms. The concrete kerbs and barriers were not modelled as it was assumed that their contribution to the overall global stiffness is negligible.

The same model was later used to estimate the load carrying capacity of the bridge using current code provisions and after calibrating it using the experimental stiffness results.

4.5.1 Modelling Assumptions

The modeling of the composite action between the steel girders and the concrete deck was through additional dummy members. These dummy members model the composite action through a rigid vertical member that connects the steel girder to the concrete deck rigidly. These members have a relatively large moment of inertia compared to surrounding members. This ensures compatibility of deformation is maintained through the rigid connection.

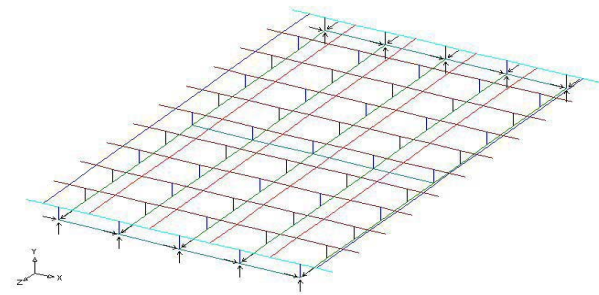


Figure 7. Microstran model of Redbank Creek bridge

The concrete deck is modelled using a grid of beams in the X-Z plane. The total area of these beams equals the total area of the concrete deck. This is important in the dynamic analysis as it ensures that the mass of the model is equal to the actual mass of the concrete deck. The grid work assumes the relative stiffness the deck provides is equal in both the X and Z directions. This is a reasonable assumption as the concrete deck is uniform in depth across the entire bridge.

The structure is assumed to be simply supported and, therefore, the supports are modelled with simple pinned supports. This ignores any contributing effects from adjoining spans as only one typical span is modelled.

4.5.2 Material Properties

The material properties adopted for the model were:

Concrete: $E = 30,000 \text{ MPa}$	Modulus of Elasticity
$\nu = 0.15$	Poisson's ratio
$\rho = 2.4 \text{ tonne/m}^3$	Mass Density

Steel: $E = 200,000 \text{ MPa}$	Modulus of Elasticity
$\nu = 0.33$	Poisson's ratio
$\rho = 7.85 \text{ tonne/m}^3$	Mass Density

4.5.3 Static analysis

A linear elastic static analysis of Redbank Creek Bridge by Microstran will determine a value for the global flexural stiffness of the structure. Static loading was modelled as point loads over each girder at midspan (Figure 8).

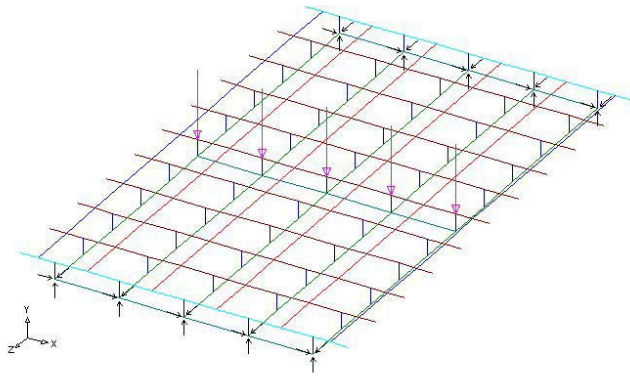


Figure 8. Static load applied to the model at midspan

The determination of the flexural stiffness of the bridge structure through static means is the familiar force displacement relationship where

$$k = \frac{\text{Load}}{\text{Displacement}} = 98,355 \text{ kN/m} \quad (4)$$

The results of the static analysis are shown in Table 2.

Table 2. Results of the static analysis

Girder	Load	Vertical Displacement
	kN	mm
1	1,000	50.8
2	1,000	50.9
3	1,000	50.9
4	1,000	50.9
5	1,000	50.8
Total	5,000	50.8

4.5.4 Dynamic analysis

The dynamic analysis requires the stiffness and mass matrices to be determined from the Microstran model. Forty five ($n = 45$) nodes or DOF (Figure 9) were used in the analysis.

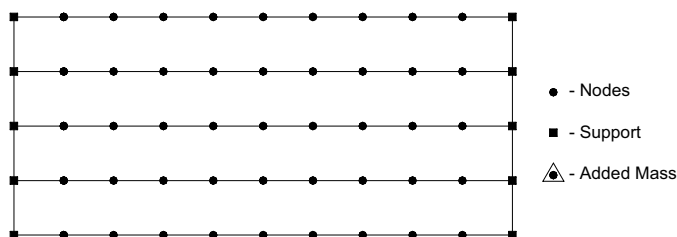


Figure 9. Bridge model with $n = 45$ DOF

4.6 Added mass cases

The determination of flexural stiffness of the bridge structure is through a shift in the natural frequency. This frequency shift is due to the additional mass on the structure. The standard DFA procedure requires the added mass to be evenly distributed at midspan. The possibility of varying the location of the added mass offers significant and practical benefits to the testing procedure. A total of ten varying patterns of added mass have been investigated. The common factor to each type of added mass is that the total additional mass remains constant to ensure the results are comparable. For the case investigated here the total added mass is 10.6 tonnes.

4.6.1 Type 1 added mass

The added mass is lumped at midspan over each girder. This is based on the current theory where the total added mass increases the modal mass of the structure (Figure 10).

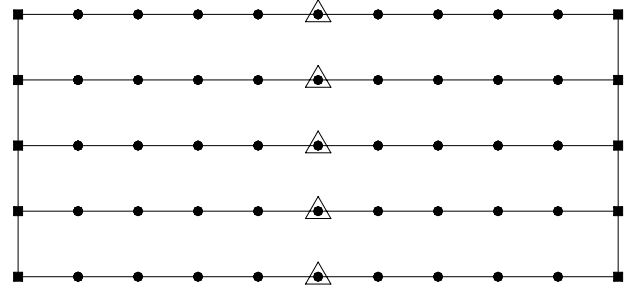


Figure 10. Type 1 added mass

4.6.2 Type 2 added mass

The added mass is lumped at midspan but only on half the width with girders 1, 2 and 3 loaded. The advantage of such a configuration is the ability to allow traffic to continue through one lane as the structure is loaded and unloaded. This reduces the bridge closure time, hence allowing bridges with large traffic volumes to be tested with minimal disruption to traffic (Figure 11).

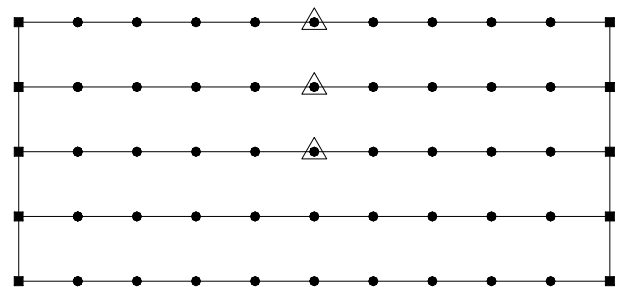


Figure 11. Type 2 added mass

4.6.3 Type 3 added mass

The added mass is identical to that of Type 2, only mirrored. The position of the added mass is on the opposite side (Figure 12).

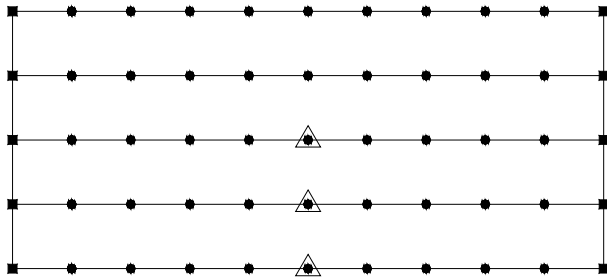


Figure 12. Type 3 added mass

4.6.4 Type 4 added mass

The added mass is located at 1/4 spans on girders 2 and 4. This type of loading represents a vehicle which may be used as the added mass. The total lumped mass is proportioned to each wheel location. This greatly reduces the loading and unloading time of the bridge (Figure 13).

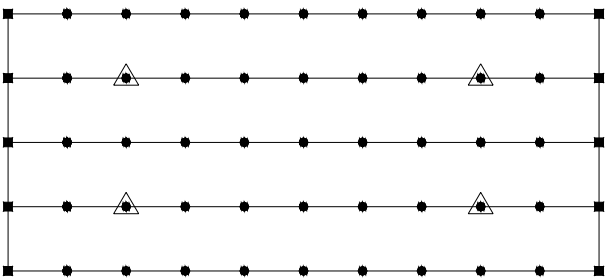


Figure 13. Type 4 added mass

4.6.5 Type 5 added mass

The added mass is located at midspan but only on the central girders 2, 3 and 4. This type of loading is considered for bridge structures that do not allow a lumped mass on the exterior girders. This may be due to the location of the barriers or kerbs on the bridge (Figure 14).

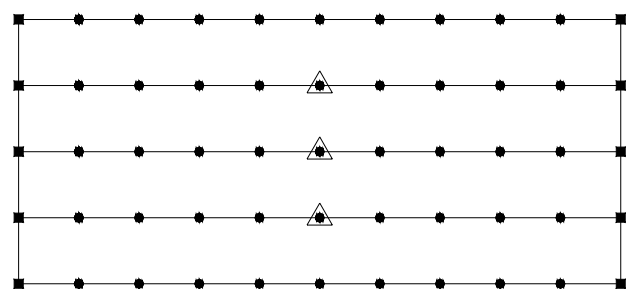


Figure 14. Type 5 added mass

4.6.6 Type 6 added mass

The added mass is distributed over the entire structure. The total added mass is distributed and

lumped at every DOF. This is only considered for theoretical benchmarking (Figure 15).

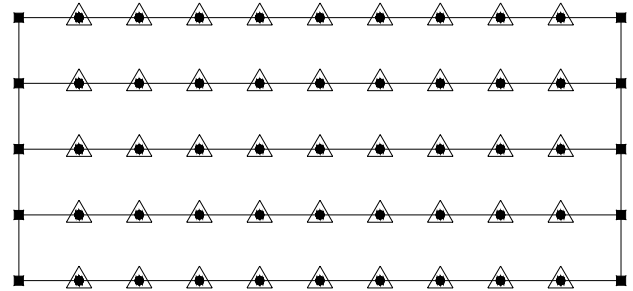


Figure 15. Type 6 added mass

4.6.7 Type 7 added mass

The added mass is distributed to each girder as in Type 1, but offset from midspan. This type of loading has practical implications in that the lumped mass may not be able to be placed exactly at midspan; therefore, this type of loading will investigate the sensitivity of the current method, to the loading location (Figure 16).

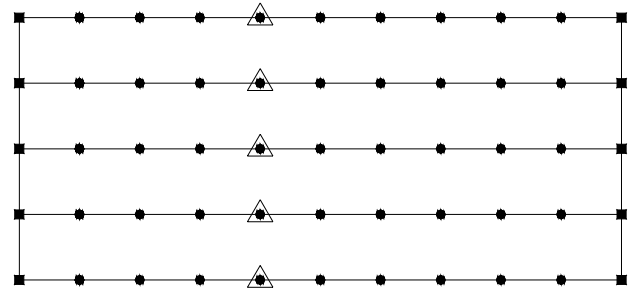


Figure 16. Type 7 added mass

4.6.8 Type 8 added mass

The added mass is identical to that of Type 7 only with a greater offset from midspan. This case has a theoretical value when investigating the results (Figure 17).

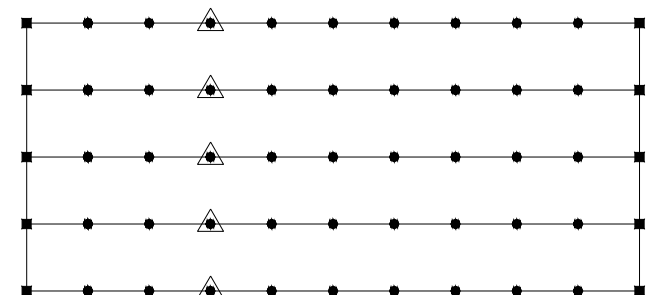


Figure 17. Type 8 added mass

4.6.9 Type 9 added mass

The added mass is located with two masses on girders 2 and 4 on either side of midspan. They are

closely spaced and resemble the added mass of wheels from a trailer or vehicle. The advantage of this type of added mass configuration is in the reduction of time to load and unload the bridge (Figure 18).

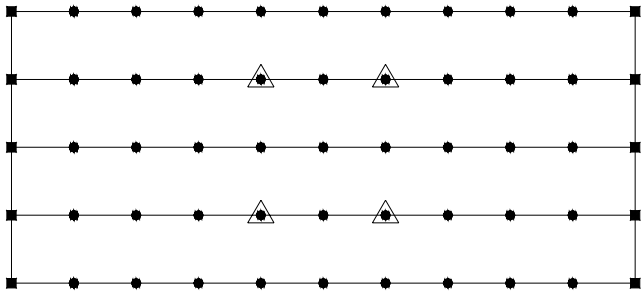


Figure 18. Type 9 added mass

4.6.10 Type 10 added mass

The added mass is located at midspan but only on the exterior girders. This configuration would limit the disruption to traffic, allowing a single lane through the centre of the bridge to remain open (Figure 19).

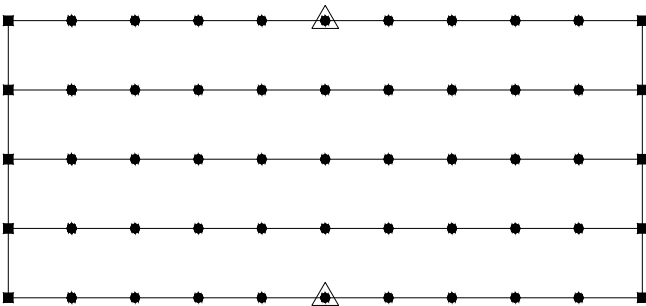


Figure 19. Type 10 added mass

4.7 Natural Frequency and Mode Shapes

The natural frequencies and mode shapes are derived using the MATLAB program. The number of natural frequencies calculated will equal the number of DOF. Therefore, for the case $n = 45$ there will be forty five natural frequencies and corresponding mode shapes calculated. The first ten frequencies are of most interest to the analysis and hence are shown in Table 3. The higher frequencies and corresponding mode shapes become too complex and somewhat irrelevant.

The first frequency represents the first flexural or bending mode, the second frequency represents the first torsion or twisting mode and the third frequency represents the transverse flexural or bending mode. These are the basic representations of the modes with all higher modes exhibiting some correlation to the fundamental modes.

Table 3. Calculated natural frequencies for Redbank Creek bridge model

Mode	Natural Frequency, Hz
1	12.00
2	12.98
3	31.66
4	39.40
5	41.34
6	47.46
7	57.33
8	59.05
9	76.50
10	79.56

4.8 Natural Frequency and Mode Shapes with Added Mass

The natural frequencies and mode shapes are calculated using the MATLAB program for each type of added mass. The results for only the first ten modes are shown in Table 4.

Table 4. Natural frequencies for ten cases of added mass

Mode	Natural Frequency (Hz)				
	Type 1	Type 2	Type 3	Type 4	Type 5
1	9.47	8.77	8.77	10.94	9.41
2	10.26	11.83	11.83	12.35	11.81
3	25.69	25.65	25.65	29.79	28.16
4	39.40	39.40	39.40	30.55	39.40
5	41.34	41.34	41.34	31.99	41.34
6	47.46	47.46	47.46	41.12	47.46
7	50.54	51.48	51.48	46.29	49.35
8	59.04	59.04	59.04	48.49	59.04
9	71.25	68.79	68.79	56.45	69.06
10	72.96	71.86	71.86	65.61	70.88

Mode	Natural Frequency (Hz)				
	Type 6	Type 7	Type 8	Type 9	Type 10
1	10.42	9.64	10.15	9.63	8.74
2	11.30	10.45	11.00	11.46	9.33

3	27.49	25.30	25.39	30.25	26.59
4	34.04	36.92	33.35	34.90	39.40
5	35.76	38.71	34.95	37.50	41.34
6	41.03	44.84	40.07	44.61	47.46
7	49.59	44.95	41.79	46.64	54.61
8	50.99	58.44	52.67	50.90	59.04
9	66.01	60.97	58.14	67.65	69.28
10	68.70	77.95	78.12	69.11	70.22

Added Mass (t)	10.6	10.6	10.6	10.6	10.6
Stiffness (MN/m)	185	110	151	109	92.2
Error (%)	85.5%	10.2%	51.6%	9.4%	-7.5%

The MATLAB program performs the required transformation and calculation to determine the FRF for the case of bridge ‘as is’ and for each case of added mass. It is from these FRFs that the frequency shift required to determine the flexural stiffness is measured.

The predictions of the flexural stiffness for each added mass case are presented in Table 5. The frequency shift error is expressed considering Type 1 added mass as the benchmark. This is also the case for the stiffness error calculation.

It can be seen that added mass cases 5, 9 and 10, in addition to benchmark case 1, can produce accurate results with errors of less than 7%. This provides lots of flexibility when choosing the most appropriate and convenient mass locations without compromising the overall accuracy.

5 CONCLUSION

A new method, based on dynamic response of bridges to an impact load, is proposed to measure the in-service flexural stiffness of bridges. Utilizing a statistically based analysis, the knowledge of flexural stiffness can be converted into an estimate of the load carrying capacity of the bridge (for timber bridges). The reliability and simplicity of the proposed methodology has been demonstrated by testing over 200 bridge spans covering a wide range of single and multi-span timber bridges. The results pertaining to two spans of one of these bridges are reported in this paper, along with the underlying principles and methodology adopted.

The methodology was refined and extended to larger concrete and steel bridges but using the same principles. This necessitated the design and fabrication of a larger exciter capable of imparting larger impacts to the bridge to excite it.

The sensitivity of varying mass locations on the accuracy of measured frequency and hence the predicted stiffness was also investigated. It is found that in addition to the optimal location to add mass, other alternatives also exist which allow the addition of mass with more ease and less obstruction to traffic without sacrificing the accuracy of results.

For the case of larger concrete and steel bridges, the developed computer model is also used to estimate the load carrying capacity of the bridge using current code provisions and after calibrating it using the experimental stiffness results.

4.9 Frequency Response Function of the Bridge

The transient analysis of the bridge model with the impact force imparted by a tuned modal impact hammer, allows the calculation of the acceleration responses of the structure. The acceleration response data, when processed using the fast Fourier transform (FFT) allows the Frequency Response Function (FRF) to be obtained. This method replicates the Dynamic Frequency Analysis (DFA) procedure applied to bridge structures and allows the determination of the first flexural frequencies from the appropriate FRF.

Table 5. Flexural stiffness prediction for different mass types

	as is	Type 1	Type 2	Type 3	Type 4	Type 5
Frequency (Hz)	12.00	9.47	8.77	8.77	10.94	9.41
Shift (%)	0.0%	26.6%	36.8%	36.8%	9.7%	27.5%
Error (%)	N/A	0.0%	38.0%	38.0%	-63%	3.2%
Added Mass (t)		10.6	10.6	10.6	10.6	10.6
Stiffness (MN/m)		99.7	69.2	69.2	296	96.3
Error (%)	N/A	0.0%	-30.0%	-30.0%	197%	-3.5%
	Type 6	Type 7	Type 8	Type 9	Type 10	
Frequency (Hz)	10.42	9.64	10.15	9.63	9.33	
Shift (%)	15.1%	24.4%	18.2%	24.6%	28.6%	
Error (%)	-43%	-8.4%	-31%	-7.8%	7.2%	

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