Some Outcomes from Load Testing of Bridge 631 in Western Australia

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SYNOPSIS

Bridge 631 is a 190m long timber bridge with 31 spans over the Avon River in Toodyay (a small town approximately 100 km from Perth the capital city of Western Australia). The bridge was constructed from local hardwood timbers (Jarrah and Wandoo) in 1950 and has had a series of repairs in 1965, 1980, 1994 and 1998. However as the width between kerbs was only 5.5 metres and it did not have a footpath it was decided to replace the bridge with a new wider concrete structure during 2001-2002.

The proposed removal of the existing timber bridge provided an opportunity to pursue a program of research into aspects of bridge inspection, timber material properties, structural failure and modelling and improved repair techniques and strengthening methods. This paper describes and discusses some of that research.

Non destructive evaluation of the bridge prior to construction of the new bridge was conducted during May 2000 using trucks to provide both static and dynamic loadings. Additional static tests were conducted in March 2002 prior to the removal of the bridge in April 2002.

Outcomes from the research could be applied to improved maintenance and load rating of the remaining 1400 timber bridges in the road network. As such this research work offers significant benefits to the community, including local government, road users, road freight industry and tertiary institutions.

1 INTRODUCTION

Since 1992, Curtin University of Technology has conducted research with Main Roads Western Australia into the structural performance of bridges with an emphasis on understanding the behaviour of timber bridges (Putt, et al (1), Chandler and Van Kleef (2), Chandler and Haritos (3)).

The proposed removal of a large existing timber bridge provided an opportunity to pursue a program of research into aspects of bridge inspection, timber material properties, structural failure and modelling and improved repair techniques and strengthening methods. This paper describes and discusses some of the preliminary outcomes of that research.

2 BACKGROUND DETAILS OF BRIDGE 631

Bridge 631 was a 190m long timber bridge with 31 spans (each approximately 6 metres) over the Avon River in Toodyay (a small town approximately 100 km from Perth the capital city of Western Australia) - see Figure 1. The bridge had round log stringers supporting sawn timber bearers and a longitudinal timber deck overlain with a concrete deck approximately 100 mm thick. It was constructed from local hardwood timbers (predominantly Wandoo with some Jarrah) in 1950 and had a series of repairs in 1965, 1980, 1994 and 1998. It is the fourth bridge on the site after flooding destroyed the previous bridges. However as the width between kerbs was only 5.5 metres and it did not have a footpath it was decided to replace the bridge with a new wider concrete structure during 2001.

Non destructive evaluation of this bridge prior to construction of the new bridge was conducted during May 2000 using trucks to provide both static and dynamic loading. Further evaluation took place in March 2002 just prior to removal of the bridge and after the replacement bridge had been opened to traffic. Constraints limited this second evaluation to static testing of two sections of the bridge.

Figure 1: General View of Bridge 631

3 LOAD TESTING OF BRIDGE 631 IN MAY 2000

3.1 Details of the First Load Testing Program – May 2000

Static and dynamic load testing of the bridge with two trucks took place over two weekends and allowed strain and deflection measurements to be recorded for 17 of the 31 spans. Over 600 Mbytes of data was collected from the tests.

For the static tests the trucks were positioned in 3 lateral positions across the width of the bridge and multiple positions along the bridge with the rear axle group over a pier or midspan. Dynamic tests were also conducted with the trucks crossing the bridge at designated speeds of walking pace, 25, 40, 50, 60, 70 and 80 km/hr and in one of the three lateral positions.

The trucks were chosen to represent the extremes of the Austroads (5) T44 axle spacings with the maximum load limited by MRWA concerns about the load rating (capacity) of some bridge elements. The shorter of the trucks had axle spacings of 3.05, 1.36, 3.42, 1.30 metres and an overall length between axles of 9.13 metres. The dual axles on the trailer were loaded to 16.4 tonnes and the gross vehicle mass (GVM) was 35.6 tonnes. The longer truck had axle spacings of 3.36, 1.32, 8.70, 1.37, 1.36 metres and an overall length between axles of 16.11 metres. The triaxles on the trailer were loaded to 21.9 tonnes and the GVM was 41.2 tonnes.

Measurements were recorded electronically via a purpose built data logging system with 32 channels. Linearly Variable Displacement Transducers (LVDT's) were used to measure deflections at midspans of stringers, ends of corbels and various locations on halfcaps and piles. Strains were recorded at midspan of stringers and various locations on the substructure.

It is difficult to install instruments and move them between set-ups with the bridge deck 4 to 6 metres above the ground. Therefore, a series of aluminium tube descenders were attached to the bridge elements transferring all vertical movements closer to the ground and allowing instruments to be installed at a convenient height on standard scaffolding reference frames (more details of this arrangement can be found in Chandler (4)).

Strain readings were obtained by measurement of the movement between two 6mm diameter steel pins inserted in drilled holes in the timber at 500 mm centres. A purpose built extensometer was attached to the pins and movement recorded via a short travel LVDT (Putt et al (1)). These instruments have been successfully used on a number of tests, but it is time consuming installing and shifting the instruments.

3.2 Results from the First Load Testing Program – May 2000

The maximum measured deflections under static loads ranged from 6 mm to 12 mm for the various spans tested and the maximum measured strains were 150 to 200 microstrain. These values were usually produced by the triaxle group of the longer wheelbase truck and when the truck was located in either the North or South loading positions near the outside stringers.

A number of factors contribute to the large variation in deflection response of the bridge including variations in stringer stiffness and variations in repairs to the spans with up to three additional stringers in a few spans. However the factor that was seen to be particularly significant in this bridge was the variations in stiffness of the corbel and halfcap interface with some corbels rotating and bending so that the end of the corbel deflected as much as 30% of the midspan deflection. This has a significant influence on load sharing between the stringers and warrants further investigation.

The bridge has very visible undulations in the deck surface at approximately 6 metre intervals. It shook and vibrated noticeably under traffic and all vehicles bounced up and down due to the uneven surface and flexible deck of the bridge. The length of the bridge and the repeated undulations increase this motion as vehicles travel along the bridge and make vertical bounce quite dramatic for trucks around 60 km per hour – the legal speed limit on the bridge. As a result, maximum dynamic effects can occur for short vehicles and lighter loads. Figure 2 shows that the drive axles of the short truck with only 14.1 tonnes produced a larger deflection than any of the other axle groups, even more than the triaxle with 21.9 tonnes.

The dynamic load allowance varied considerably with vehicle speed and was affected by the matching of vehicle motion with surface undulation. It was greater for the short truck and values of 45% to 50% were recorded. The natural frequency for this bridge was in the range 9 to 9.5 Hz giving an Austroads Bridge code (5) value for dynamic magnification of 25% - significantly less than that observed.

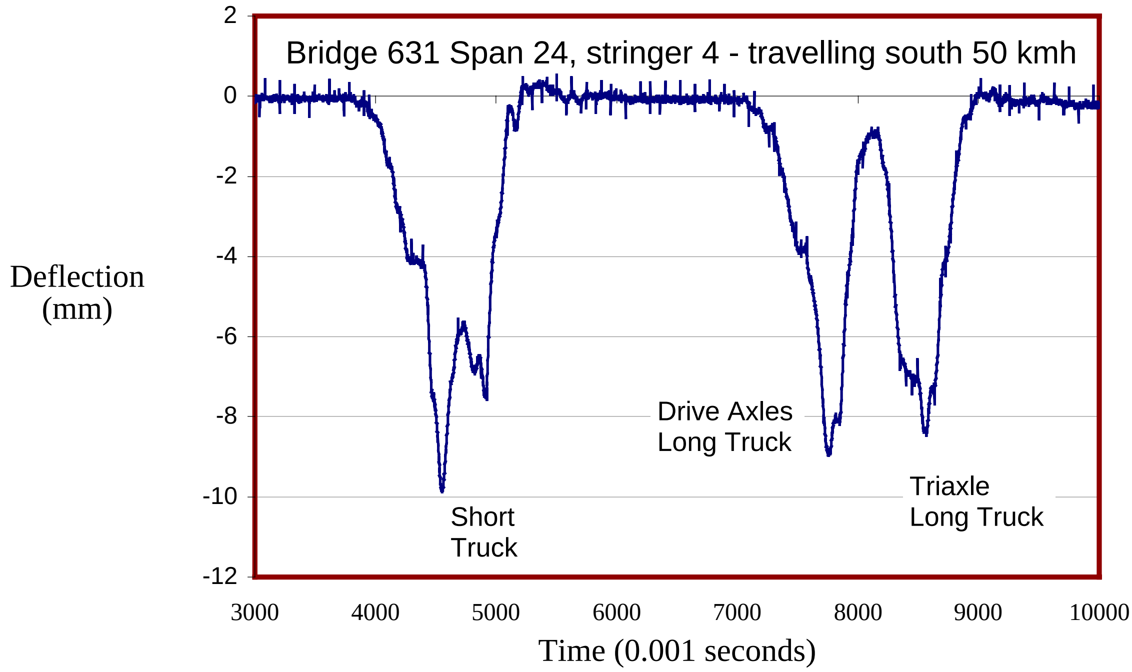


Figure 2: Typical Dynamic Deflection Response for Bridge 631 – Trucks Travelling in Convoy

4 LOAD TESTING OF BRIDGE 631 IN MARCH 2002

4.1 Details of the Second Load Testing Program – March 2002

Bridge 631 was tested statically for the second time during March 2002. Two sections of the bridge were chosen for testing including, piers 1 to 4 and piers 21 to 24. These two sections were chosen as they contained the only groups of spans where the stringers had not been modified by bridge maintenance. This was significant as the main aim of the second set of testing was to determine the load sharing between the half-caps for a range of vehicle loads by measuring the load transferred at each of the corbels to the half-caps. To achieve this a number of loadcells were needed to be inserted between the corbels and halfcaps.

4.1.1 Preparations and Planning

Sixty five purpose built load cells were used to measure the load that was transferred from the corbels to the half-caps on Bridge 631. The load cells were manufactured at Curtin University of Technology to support a maximum axial load of 200 kN each to allow for destructive (ultimate strength) testing. The load cells were constructed from mild steel consisting of two end plates and a central hollow tube connecting the plates. A minimum height of 100mm was established for the loadcell design with a diameter of 130 mm.

The load cells were intended to represent a similar contact area to the normal situation when the corbels rested directly upon the half-caps, where the width of contact ranges between 200mm and 300mm and the width of the halfcaps was 170 mm. Therefore two load cells were used on each corbel, totalling 16 load cells per pier with 64 load cells required for the four piers.

Each load cell had four electrical resistance strain gauges, two pairs of longitudinal and transverse gauges diametrically opposite and connected in a full bridge arrangement. Individual calibrations were established for each cell using Vishay strain measuring equipment.

As the bridge had been closed to traffic the loading applied to the bridge did not need to comply with MRWA rating requirements but needed to stay within safe limits for the testing personnel. Based on the results from the first series of tests, the author considered it significant to maximise the loading on a given span and to be able to reach a load of 40 to 45 tonnes. To monitor the bridge response and to ensure a safe test environment it was decided to use five load increments to reach the maximum.

For simplicity and speed of testing, the loading was again applied to the bridge by driving a truck onto the bridge with five different load magnitudes. Constraints on individual wheel loads and on transport to and from the site limited the choice of truck. The truck selected was a low loader with a split spreadable trailer and a triaxle with four sets of dual tyres across the width. In the test configuration the width of road contact was 2.53 metres and the axle spacings from the front of the vehicle were 4.55, 1.3, 10.0, 1.8, 1.8 metres and an overall length between axles of 18.8 metres.

A large steel beam of 14 tonnes formed the basic load for the truck and a series of steel and concrete weights was used to provide five increments to the maximum load. Table 1 shows the loads that each axle group of the truck exerted on the bridge for the five load increments.

Table 1: Load Exerted by each Axle Group for Five Truck Configurations

Axle Group	Load 1 (tonnes)	Load 2 (tonnes)	Load 3 (tonnes)	Load 4 (tonnes)	Load 5 (tonnes)
Steer	6.5	6.5	6.6	6.5	6.6
Drive	16.5	16.6	16.6	16.2	16.8
Trailer	18.2	21.1	30.6	38.0	42.4
TOTAL (GVM)	41.2	44.2	53.8	60.7	65.8

4.1.2 Site Preparation for Testing

The testing on Bridge 631 was carried out over a continuous period of 15 days. This time included the set-up and pack-up as well as the two days of testing. The first test was on the Toodyay end of the bridge and the second test was on the Goomalling end of the bridge. The procedure for the two tests was essentially identical, because at both ends there were four piers that were tested and set-up the same.

Testing of Bridge 631 to determine the half-cap load sharing required the load cells to be inserted between the corbels and half-caps prior to the day of testing. The insertion of the load cells between the half-caps and the corbels required the superstructure to be lifted up using eight 30-tonne hydraulic jacks. Only one pier was lifted at a time and the full width of the superstructure was raised approximately 130mm to create a gap for insertion of the load cells.

Frames were used to support the jacks from the half-caps so that the corbels and deck structure could be jacked by reacting against the half-caps. Each frame was assembled from two halves where one half could rest upon one half-cap and the other half of the frame on the other half-cap (see Figure 3). The frames were connected at the top using two high tensile bolts and at the bottom with at least one bolt. The connected frames were then grouped in pairs to support the jacks. A connecting steel beam section was inserted between each of the pairs on either side of the pier to provide a flat base surface for the jacks to rest upon.

The height of the bridge above the ground level varied between 3 metres and 6 metres in the test locations. Scaffolding was required to access the underside of the bridge for preparation and testing. The scaffolding was positioned to surround each of the piers for easy access to any point. The scaffolding was also linked on one side between each of the piers. This was designed to allow the hydraulic jacks and the jacking frames to be moved easily from pier to pier during the site preparation.

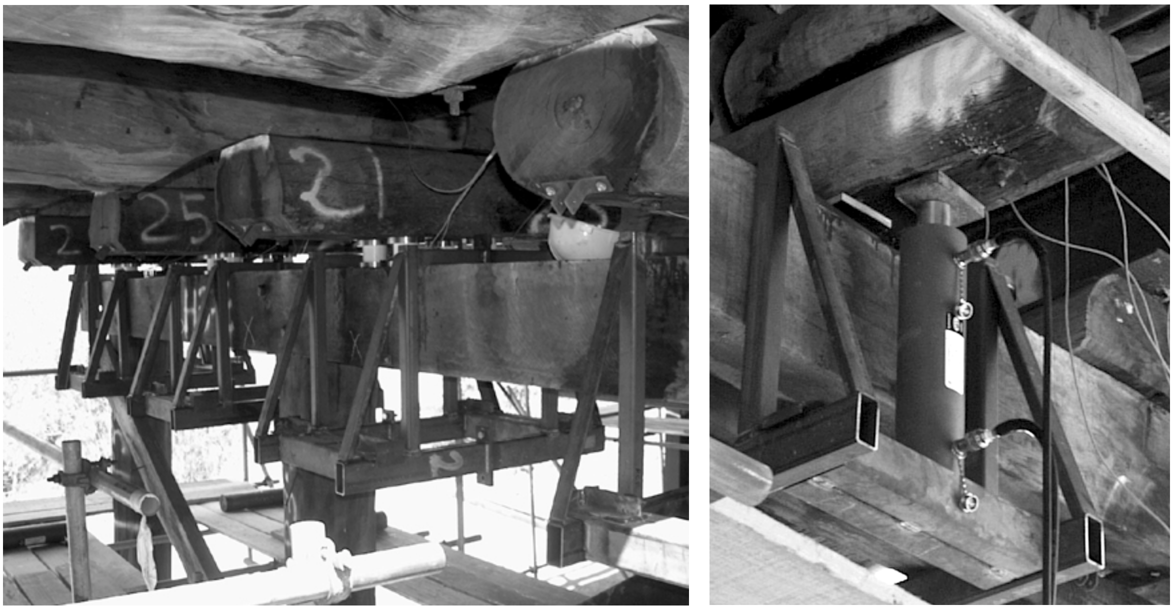


Figure 3: Purpose-Built Frames Used To Lift the Bridge Superstructure To Insert Loadcells

Jacking of the bridge involved lifting the corbels from the half-caps, before this could occur it the nuts were removed from the bolts tying the corbels to the half-caps to allow vertical movement as they separated. Jacking of the bridge had to be very controlled so that all the jacks moved the same distances so that the bridge was jacked up evenly across the pier. To enable controlled jacking, the pump had separate controls to each pair of jacks allowing individual corbels to be raised or lowered if required. To ensure that the contact surfaces remained parallel and to monitor the rise of the bridge, a person watched each jack. The bridge was jacked to approximately 130mm so that the load cells could be inserted between the corbels and the half-caps.

Once the superstructure was lifted, temporary packers were inserted before the load cells were placed between the half-caps and the corbels - typically four load cells under each corbel, however in a few cases only two load cells could fit under narrow corbels. The bolts connecting the corbels to the half-caps were left in place (only the lower nut was removed to allow lifting), therefore the load cells had to be positioned around the bolts, one on each side of the bolt. Figure 4 shows the load cells in position between the half-caps and the corbels. To ensure that the load was transferred axially through the load cells, care was taken to ensure that the surfaces bearing on the load cell plates were smooth and flat. After all of the load cells were inserted, the bridge superstructure was slowly lowered so the corbels rested upon the load cells. The lifting procedure was repeated for each pier in turn until all four piers associated with the test location had load cells under every corbel.

During the installation process the loads in the cells were monitored to assess the distribution of dead weight between the various corbels and the changes as the next pier was lifted.

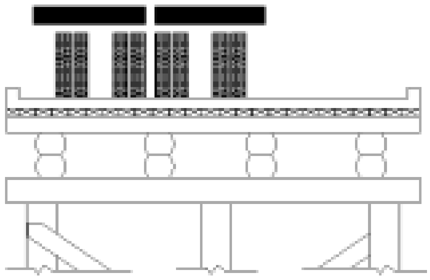
Figure 4: Typical Arrangement of Four Loadcells Inserted between a Corbel and Halfcaps

4.1.3 Testing Methodology

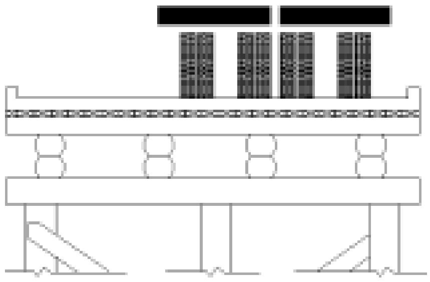
The testing was designed to collect the maximum possible amount of data within a full day. As well as the loadcell readings a comprehensive set of deflection readings was recorded using LVDT's (as for the first test) and photogrammetric targets with a digital still camera recording six alternate images for each load position. The main constraint was the time taken for adding load to the truck and the number of load cases at various longitudinal and transverse positions on the bridge deck.

The transverse loading positions included north, south and centre loading of the bridge deck as shown in Figure 5. When the truck was on the north or south sides it was positioned with the outer edge of the wheels 0.5 m from the kerb of the bridge.

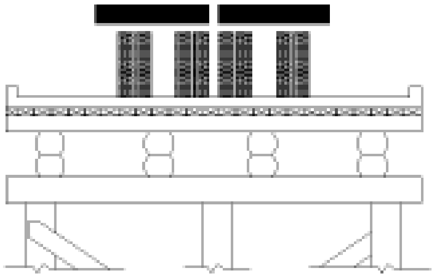
For each of the centre, north and south runs the truck was located with an axle of the trailer at various positions over the piers or in the centre of the spans. These combinations included loading with the centre trailer axle at the centre of each span, and with each of the trailer axles positioned directly over each of the piers. A few key load cases were repeated to assess the repeatability of the bridge response and the instrumentation.



(a): North Loading



(b): South Loading



(c): Centre Loading

Figure 5: Lateral Loading Positions of Truck across the Bridge Deck

4.2 Results from the Second Load Testing Program – March 2002

The analysis of the measurements of loads in the loadcells on the timber half-caps on Bridge 631 included determination of the load sharing of the dead load of the structure and the live load applied to the structure. The loading where the axles of the truck were positioned in the centre of the span on the structure were expected to be the critical load cases as these loads cause deflection of the deck and stringers which in turn cause rotation of the corbels at the piers and alter the distribution of the load to the half-caps.

The distribution of the live load at the piers was analysed for the distribution of the load along the length of the half-caps, the distribution of the load at each of the corbels and the load sharing between the two beams. The determination of the load sharing capabilities of the timber half-caps is essentially the load sharing between the two beams, however the analysis of the distribution of load along the pier and at the corbels gives insight into other issues of load sharing within the timber bridge structure.

It was found from analysis of the measurements that the critical load sharing between the half-caps was for the triaxle of the truck in the middle of the span. The distribution of the live load was found to be 15% to one halfcap and 85% to the other, on average for a typical pier. However the range of possible load sharing includes one half-cap supporting the entire live load and a portion of the dead load. Figure 6 illustrates the distribution of the live load at a pier for span loads where the corbels rotate due to the deflection at the centre of the stringers.

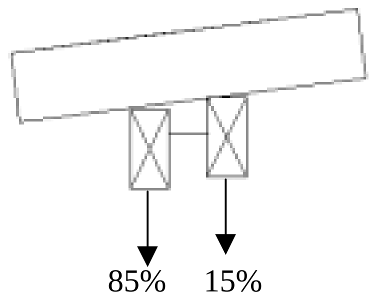


Figure 6: Average Percentages of Timber Half-Cap Load Sharing Capabilities

5 ANALYTICAL MODELS OF BRIDGE 631

Spans 2 to 11 of the bridge were modelled using the finite element analysis software package Strand 7. The model consisted of beam elements with user defined cross-sections to match the individual stringers, beams for the bearers and corbels, plate elements for the deck and three dimensional brick elements for the half caps. Overall 18500 elements were used in the model.

Data for the model was obtained from site surveys with a total station giving cartesian coordinates of key points throughout the structure. Visual inspection data was available giving the vertical and horizontal dimensions of each stringer at both ends and at midspan. However to simplify the model it was decided that the section at midspan would be used throughout the stringer length.

The real deck was constructed of longitudinal timber planks 250 mm wide by 125 mm thick overlayed with concrete of varying thickness. This was represented by a single layer of plate elements approximating the combined stiffness. Othotropic properties were used to reduce the lateral stiffness due to the discontinuities in the deck timbers. The deck timbers were joined longitudinally in a butt joint at the middle of each span and this was modelled by a reduced thickness of elements at that point.

A series of link elements were used to join the various components together without over constraining the connections. Master-slave links at the end of the corbels were used to connect stringers to corbels and also to connect the bearers to the stringers. It was found that the rotational degree of freedom about the longitudinal axis of the model had to be released to match bearer and deck displacements with measured results.

Results obtained from this model generally matched the measured deflections to within 5% although there were some local anomalies. The lateral stiffness of the deck and the method of linking different parts of the model were significant influences on the results. A better match could have been obtained by separate modelling of the concrete and timber portions of the deck at the expense of increased model complexity and increased solution time.

The model indicated maximum stresses in the timber stringers of 5 MPa and maximum stresses in the steel beams of 60 MPa. These results were in general agreement with the strain measurements on steel and timber stringers.

The longitudinal deck stiffness was varied to use the model to test predictions about the lateral load sharing and relative deflection of the stringers. The model indicated that the relative deflections remained unchanged with significant changes in the deck stiffness.

6 SUMMARY OF THE OUTCOMES FROM TESTING

In the short length of this paper it has only been possible to briefly mention some of the outcomes from the testing program and subsequent analysis. Further analysis and testing is needed to clarify the behaviour of the complex structural system represented by timber stringer bridges.

The testing did identify that steel stringer inserts were carrying significant proportions of the vehicle loads and showed that the corbel movement was larger and more significant than previously identified.

Dynamic magnification of the truck loads was found to be significantly greater than Austroads code predictions and confirmed the discomfort felt by occupants of many vehicles and trucks when crossing the bridge even at speeds of 60 km per hour or less.

It was found that the load sharing between the half-caps of the truck load was 15% to 85% on average for a typical pier. However the range of possible load sharing that was measured included the situation where one half-cap supported the entire live load and a portion of the dead load. There is still much to be learnt about the load sharing capabilities of the timber half-caps on Bridge 631 and other timber bridges.

Finite Element modelling proved valuable in matching analytical stresses and deflections to the observed behaviour and identifying effects, such as the bearer location, that had a large influence on the modelled behaviour.

The research work presented in this paper offers significant benefits to the community, including local government, road users, the road freight industry and tertiary institutions. However, further analysis of the test data is required and the finite element model needs to be applied to the eastern end of the bridge to verify its consistency with the observed behaviour. A series of additional laboratory tests on various components from this bridge are also underway and will extend our understanding of key parameters influencing the structural response.

7 ACKNOWLEDGEMENT

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