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Normanby Overpass, Bridge Strike: Assessment, Management and Rehabilitation

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Abstract

A truck carrying an over-height load travelling on Bruce Highway (now Old Bruce Highway) struck the superstructure of the Normanby Overpass in Gympie, Queensland, Australia. The overpass is a 13.5 m long single span bridge with signs attached indicating a 5.0 m clearance. The impact damaged the steel girders.

Load restriction of 17 tonnes Gross Vehicle Mass (GVM) and reduced speed limit were then imposed to keep the overpass operational while an investigation into the preferred rehabilitation method was conducted.

Nonlinear finite element analysis (NLFEA) was used to assess the load rating and remaining fatigue life of the as-damaged girders, and to derive repair options to carry Regulation Mass Vehicles (62.5 tonne B-Doubles) and AS 5100 design traffic loads (SM1600 and HLP400). 3D laser scan data of the damaged overpass was used to accurately model the damaged girders for the NLFEA assessment.

Traffic monitoring by cameras detected that load restrictions were being exceeded, largely by 22.5 tonnes GVM rigid trucks. Fatigue damage due to these rigid trucks was estimated using NLFEA, and the remaining fatigue life was predicted to be less than a year. A rigorous monitoring strategy was put in place to check for any deterioration in the damaged girders to ensure safety while keeping the overpass operational until an optimal rehabilitation method was determined and implemented. The monitoring strategy included regular visual inspections and non-destructive testing (e.g. magnetic particle testing).

Despite the estimated design, construction, and future inspection cost of superstructure replacement being 55% more than the preferred repair option, replacement was adopted because:

- 100 years of design life could be achieved, compared to approximately 31 years for the repair
- design and construction of the repair were complex due to the extent and severity of the damage, and
- satisfactory site welding quality could be difficult to achieve for the repair.

Keywords: assessment, rehabilitation, steel, FEA, testing, monitoring

1. Background

On the morning of 12th July 2018, a truck carrying an over-height load travelling north on the Bruce Highway (Brisbane – Gympie) towards Gympie in Queensland, Australia, struck the superstructure of the Normanby Overpass. Figure 1 shows the aerial view of the overpass and surroundings. The over-height load was a Liebherr T282 dump truck chassis and engine.

This single-span, 13.5 m long overpass was constructed circa 1960. The original superstructure was replaced in 2009, after it was severely damaged by an over-height load (a steel tank) on a truck. The restored superstructure comprised a 200 mm minimum thick reinforced concrete deck slab supported by five 610UB125 steel girders; refer Figure 2 for further overpass details. During the same rehabilitation, the road/highway level below the overpass was also lowered by approximately 400 mm, increasing the signed vertical clearance from 4.6 m to 5.0 m.

The overpass is owned by the Queensland Department of Transport and Main Roads (TMR), which currently has preferred and absolute minimum vertical clearances over highways and motorways of 6.0 m and 5.5 m respectively (note the same minimum clearance requirement existed in 2018).

This paper summarises the actions taken to rehabilitate the superstructure damaged by the strike in 2018. The actions included:

- inspect and assess damage
- develop and assess rehabilitation options
- utilise advanced Finite Element analysis (FEA) to determine structural strength and remaining life of the damaged superstructure
- design preferred rehabilitation method
- develop and implement measures to keep the damaged overpass operational while undertaking the above actions, and
- implement long-term measures to prevent future strikes.

The strike damaged the superstructure's steel girders, intermediate reinforced concrete cross girders and some steel bracings. Figure 3 and 4 show the damage.

The overpass was closed to traffic following the strike. After assessing the structural capacity, the traffic closure was lifted on the afternoon of the strike, with the following traffic restrictions imposed on the overpass:

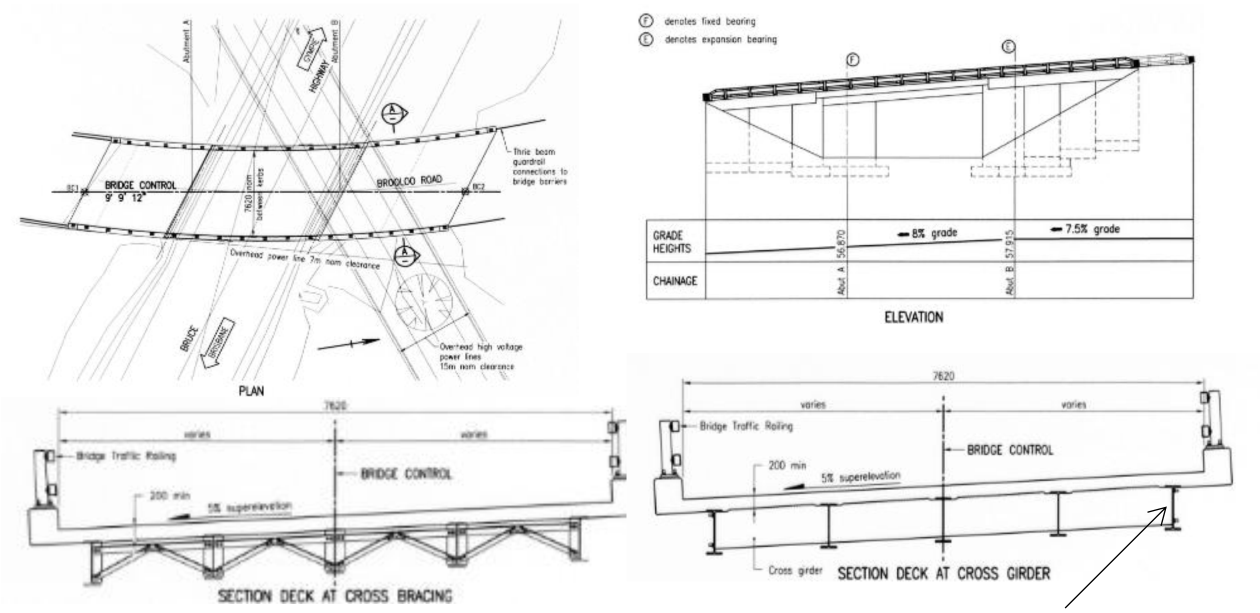
- a 17-tonne GVM load limit, which is equivalent to a two-axle bus/coach. This load limit allowed for school bus usage after consultation with local bus and coach companies that service schools in the area, and
- a speed limit of 40 km per hour.

Even though the vertical clearance was increased in 2009, the overpass remained vulnerable to bridge strikes. Based on the records of the Level 1 and 2 Inspections conducted after 2009 and prior to the 2018 strike, the superstructure was damaged by multiple strikes (Hu & Wong[1]). The exact number of strikes was unknown as these strikes went unreported to the asset owner, TMR. Almost all of the observed damage was located above the northbound lane of the Bruce Highway. This side of the overpass has less clearance than the southbound side of the Bruce Highway due to the deck/road grade of the overpass. Figure 2 shows the deck/road grade. Corrosion in the damaged areas of the steel girders indicated that the damage was caused by impacts prior to the 2018 strike. Figures 3 and 4 show the damage.

Figure 1 Aerial view of Normanby Overpass and surroundings

Source: Google Maps

Figure 2 General details of Normanby Overpass



610UB125 girders with 20mm thick x 250mm wide doubler at bottom flange (typical)

Figure 3 Damaged bridge components

a) Damaged steel girders due to strike in 2018

(b) Buckled steel bracing

(c) Damaged reinforced concrete cross girder

(d) Deformed steel girder

e) Damaged girder due to strike prior to 2018

Figure 4: Damage to steel girders above northbound lane – as viewed from the road

2. Design traffic loading class rating and assessment vehicles

The original design traffic loading class for the overpass was H20-S16-44 (33 tonne vehicles) as defined by NAASRA[2]. The superstructure replacement in 2009 was designed to the latest and higher design traffic loadings, SM1600 and HLP400 in accordance with the Australian Standard, AS 5100-2004 Bridge Design (Standards Australia[3]), and the design traffic loading class of the overpass was subsequently updated to reflect these loadings.

Repairing the superstructure to restore the design traffic loading class rating (SM1600 and HLP400) was considered to be structurally unfeasible, impractical and/or uneconomical. TMR indicated that the following Regulation Mass Vehicles were the preferred lower load rating for rehabilitation:

- 42.5 tonne Semi-trailers
- 62.5 tonne B-doubles (BD62.5t), and
- 48 tonne Cranes.

3. Structural assessment of damaged superstructure

The initial determination by TMR was to allow school buses and trucks with a GVM of up to 17 tonnes and prohibit trucks with greater GVMs. TMR was however interested in learning whether cement mixer trucks with a GVM of 32 tonnes from two nearby batching plants could use the damaged overpass in the short term, as well as the long-term remediation options that would allow the overpass to safely carry Regulation Mass Vehicles for the remaining design life of the overpass.

Severe damage was largely to the bottom flange doubler plate of the steel girders, while the girder bottom flanges and webs had undergone twisting and warping (refer to Figures 3, 4 and 6**Error! Reference source not found.**). A simple grillage analysis that excluded the doubler plates indicated the girders had sufficient capacity to carry BD62.5t, but not HLP400 or SM1600 design traffic loads (Kandadai & Timms[4]). The grillage assessment was deficient in the sense that it ignored the plastic straining of the bottom flange of the steel girders, and the presence of cracks (refer to Figure 5). The presence of cracks sparked concerns that these cracks could spread through the entire width of the bottom tension flange of the girders, leading to catastrophic failure of the superstructure. The high level of plastic straining implied that the bottom flange had reduced ductility, and brittle failure could occur if overstressed. The simplistic grillage analysis could not be relied upon to predict the capacity of the damaged girders or the risk of the cracks spreading to the full width of the flange and causing failure. Nonlinear finite element analysis (NLFEA) of the superstructure was therefore carried out to determine the structural strength and remaining life of the damaged superstructure.

Figure 5: A closer look at the damage to the bottom flange doubler plate of the girders. Cracking and shearing of metal can be seen

Investigation of cracks

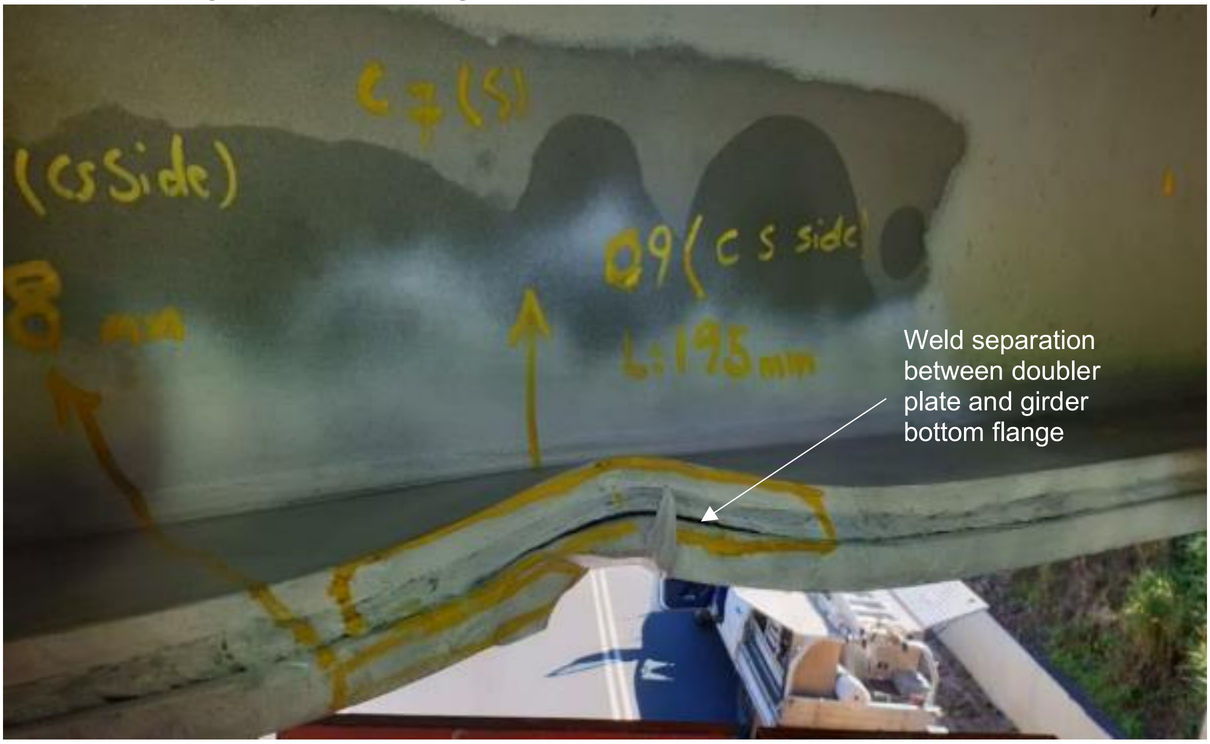
While there were visible cracks up to approximately 2 mm wide on the bottom flange doubler plates, inspection revealed no visible cracks on the upper surface of the bottom girder flanges. A key question was whether there was cracking in the hidden bottom surface of the 610UB125 girder flanges. Non-destructive testing (NDT) was adopted to investigate this possibility.

Manual phased array ultrasonic (PAUT) and encoded time of flight diffraction (TOFD) investigations confirmed that the cracking was restricted to the doubler plates (Rostami[5]). This implied that the girders were not in immediate danger of failing, as there were no cracks in the bottom flange of the 610UB125 girders, which had previously been assessed to have sufficient capacity to carry the Regulation Mass Vehicles.

Magnetic particle investigation (MPI) performed on the exposed surfaces of the girders revealed poor fusion and widespread separation of the fillet welds between the girder bottom flange and the doubler plate (Rostami[5]) (refer to Figure 6). Poor quality weld, intermittent weld regions, and pre-existing cracks in the bottom tension flange increased the risk of continued crack propagation and subsequent failure.

Traffic monitoring by cameras carried out from 9th to 11th September 2019 revealed that the most common vehicles exceeding the 17-tonne load limit were 3-axle rigid trucks (22 tonnes GVM). A fatigue assessment of the damaged overpass using this vehicle load estimated a short fatigue life of just over one year at the cracked weld between the doubler plate and the girder bottom flange (Kandadai & Timms[4]). As determined by computations, as long as cracks were monitored regularly and there was no growth in the cracks until remediation could be carried out, the risk of bridge failure was minimal, and the overpass could remain operational.

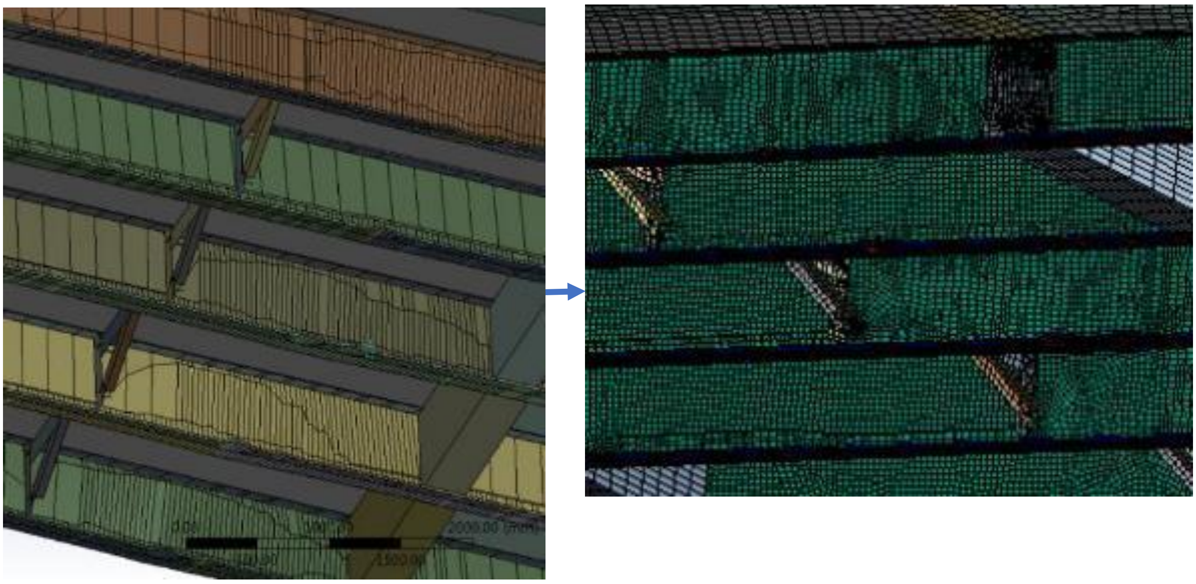
Figure 6 MPI showing weld separation at fillet weld to doubler plate, but no cracks in the 610UB125 girder bottom flanges



Modelling the damaged superstructure

3D laser scans of the superstructure produced a cloud of points, which were then converted into geometric surfaces. Ansys[6], an engineering software, was then used to create a finite element model of the girders, including damage, distortions and cracks. Figure 7 shows the geometric surfaces and finite element mesh of the damaged superstructure.

Figure 7 3D laser scan data converted to geometric surfaces, and then to finite element mesh

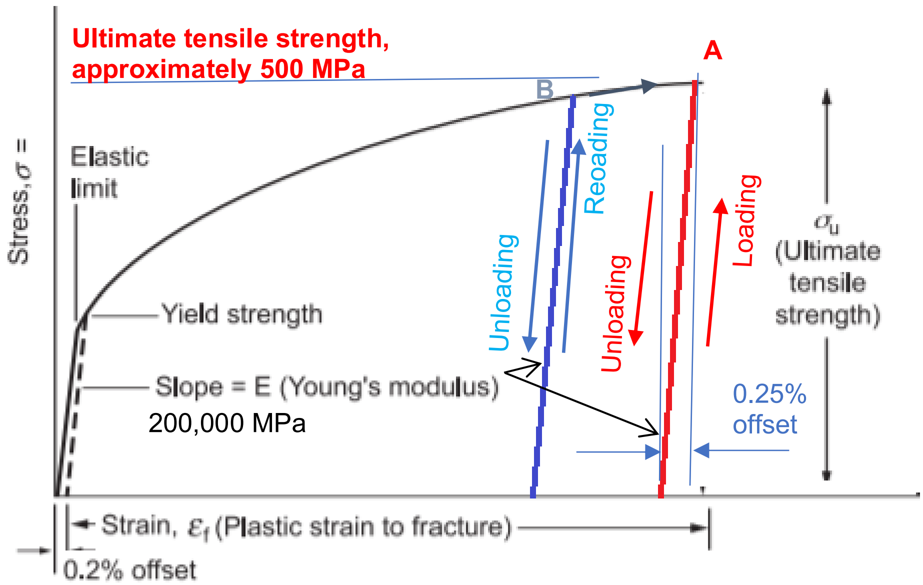


(a) Geometric surface *(b) Finite element mesh*

Loss of ductility and design criteria

Sections of the damaged flanges had undergone plastic deformation due to vehicle impacts. In ductile strain hardening materials, such as the steel in the girder flanges and the doubler plates, this will lead to an increase in yield strength but could also lead to a reduction in subsequent strain to failure. For example, if the steel in the 610UB125 flange was plastically strained due to damage to Point B in the stress-strain curve (refer to Figure 8), and if load was then removed and re-applied, stress would travel linearly along the slope having a gradient corresponding to the Young's Modulus, E, shown by the blue arrows and would then flow along the original stress-strain curve until failure due to rupture. If, however, the plastic strain due to the impacts caused the steel to nearly reach the point of rupture (refer to Point A in Figure 8), then reloading would cause strain to increase linearly with a slope equivalent to E until the ultimate tensile strength (UTS) of approximately 500 MPa was reached, but could then rupture soon after since it was already close to the failure limit on the stress-strain curve. It is difficult to determine the residual plastic strain in the flanges (see note in Figure 8). The conservative assumption would be to assume straining due to impact up to Point A. A conservative measure of rupture strain would then be 0.25% (UTS of approximately 500 MPa divided by elastic modulus of 200,000 MPa). This criterion was applied to the average of the strain across the flange section at the high strain location, when determining the residual capacity of the damaged sections of the girders. The criterion is conservative since monotonic loading was assumed with yield occurring at yield strength rather than at an increased value due to strain hardening.

Figure 8 Strain to failure in undamaged and damaged steel with plastic yield



Note: A nonlinear analysis was carried out to replicate the deformed shape of the damaged flange. The quasi-static analysis did not converge to the fully deformed value but predicted a plastic strain of 17% at about 85% of the deformation. Therefore, residual strains of the order of 20% or more could potentially exist in the damaged girders.

Load rating of damaged overpass

Table 1 summarises results from grillage and 3D finite element models. The best-case scenario, i.e., no loss of ductility in the bottom flange (Assumption C. in Table 1), predicted sufficient capacity to resist the BD62.5t vehicle load. The worst-case scenario of near complete loss of ductility, with a conservative assumption of 0.25% strain to failure post yield (i.e. Assumption B in Table 1), predicted a load rating of 0.57 for the B-Double. In Load Cases 1 to 3, predicted load ratings are below unity for the design M1600 and HLP400 vehicles.

The results indicated that remediation was needed to restore the overpass' design capacity to carry at least Regulation Mass Vehicles.

Table 1 Summary of bridge load ratings using grillage and 3D FEA models

Ultimate Limit State design load case	$\text{Load Rating} = \dfrac{\textbf{Capacity of damaged girder}}{\textbf{Total factored load effects}}$		
	Grillage	**3D FEA**	**3D FEA**
Assumptions →	**A. Loss of doubler plate and half of 610UB125 bottom flange. No loss of ductility**	**B. Crack in double plate and loss of ductility in damaged 610UB125 flange**	**C. Crack in doubler plate and ductile 610UB125 flange**
1) PE* + M1600	**0.74**	**0.49**	**0.9**
2) PE+ HLP400	**0.79**	**0.47**	**0.9**
3) PE + BD62.5t	**1.1**	**0.57**	**1.2**
4) PE + 17t Bus	**Not computed**	**Greater than 1**	**Greater than 1**

* PE stands for permanent effects.

4. Management of damaged overpass

As previously described, the damaged girders had cracked components and welds, as well as a predicted fatigue life of just over one year. The following management strategy was implemented to maintain safety while keeping the overpass operational until an optimal rehabilitation method was determined and carried out:

- continued imposing traffic restrictions
    - 17 tonnes GVM load limit, and
    - 40 km per hour speed limit
- a rigorous monitoring strategy was put in place to detect any deterioration of the defects in the damaged girders. This involved taking baseline measurements, performing monthly visual inspections and monitoring, and using NDT methods every 6 months, such as PAUT, TOFD and MPI, to check for crack propagation.

5. Rehabilitation design

Repair options

The following four repair options were assessed for structural strength and fatigue life (Kandadai & Timms[4]) (Hu & Wong[7]).

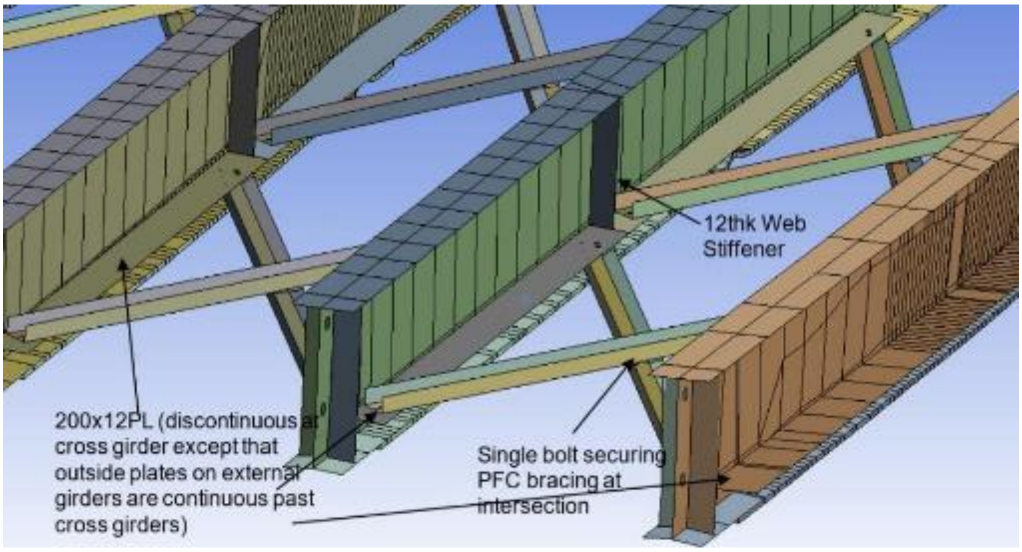
1) Option 1: Install steel bracings with flange plates welded to the webs

   Features include:

   - additional 'bottom flange' plates welded to the web 90 mm above the existing damaged flange
   - additional 125PFC horizontal bracings added between girder flanges
   - additional 12 mm thick web stiffeners, and
   - no repair of cracks or rectification of deformed girders.

   Refer to Figure 9 for visualisation.

Figure 9 Option 1: additional 'bottom flange' plates welded to web and bracing. Reinforced concrete cross girders not shown for clarity



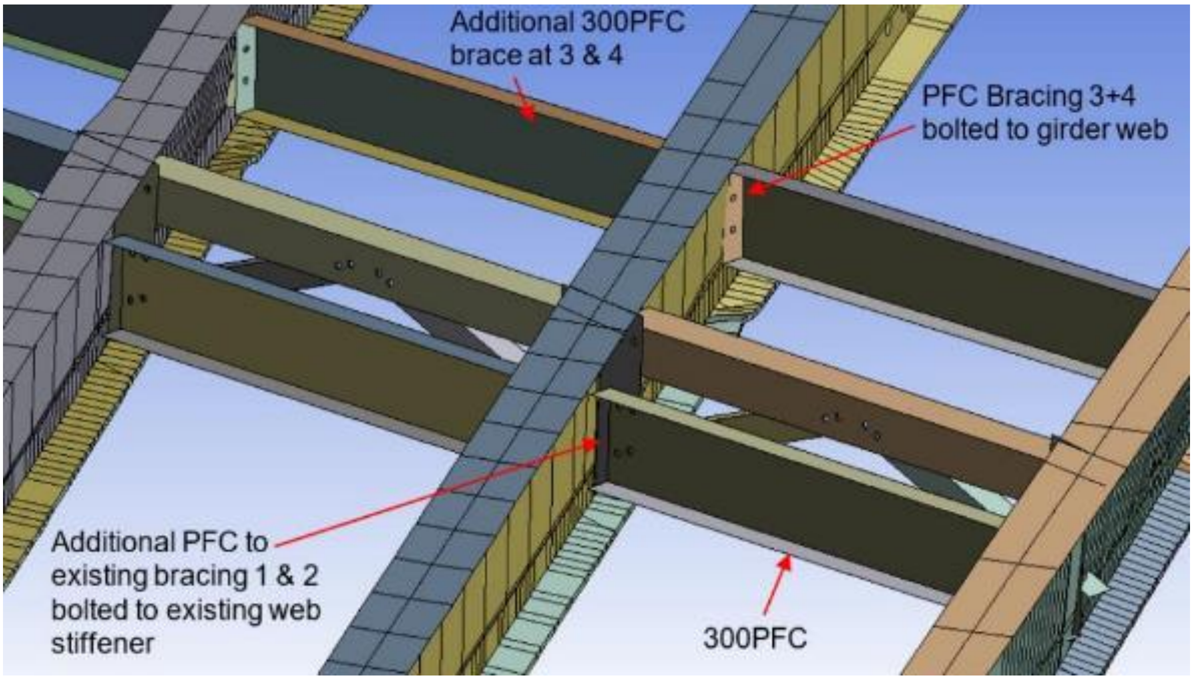
2) Option 2: Install steel bracings bolted to the webs

   Features include:

   - additional channel section bracings between girders both in the vertical and in the horizontal plane, and
   - no repair of cracks or rectification of deformed girders.

   Refer to Figure 10 for visualisation.

Figure 10 Option 2 – additional channels bolted to webs adjacent to existing cross-girders



3) Option 3: Repair cracked doubler plates and welds by field welding

   Features include:

   - cut cracks on the doubler plates for their entire length, then repair the cuts with full penetration butt welds
   - grind out cracked fillet welds between girder bottom flange and doubler plate and reapply fillet welds, then perform NDT to confirm fusion and quality, and
   - no other remediation.

4) Option 4: Combine Options 1 and 3 above

Ultimate Limit State (ULS) load ratings

ULS load ratings for the repair options are presented in Table 2. ULS strength ratings are lower than required by AS 5100 for Options 2 and 3 and marginally lower for Option 1. ULS ratings are greater than unity for Option 4.

Table 2 Summary ULS load ratings for repair options

ULS Design Load Case	$\text{Load Rating} = \dfrac{\textbf{Capacity}}{\textbf{Total factored load effects}}$				
	As Damaged	Option 1	Option 2	Option 3	Option 4
1) PE* + M1600	0.49	0.97	0.64	0.85	1.08
2) PE + HLP400	0.47	0.93	0.64	0.89	1.08
3) PE + BD62.5t	0.57	1.07	0.78	1.10	1.37

* PE stands for permanent effects.

Fatigue Life

Fatigue limit state ratings of the repair options were computed using the fatigue load as defined in the Australian Standard, AS 5100:2017 Bridge Design (Standards Australia[7]). The number of heavy vehicles was based on annual average daily traffic values. The majority of the vehicles on the road of the overpass are light vehicles, accounting for around 94% of traffic, while heavy vehicles account for approximately 6%. Heavy load platforms (HLP) do not travel on this road, therefore HLP vehicle loads were excluded from the fatigue assessment.

The weld between the 610UB125 bottom flange and the doubler plate was assigned a low “end zone” detail category, because damage had caused cracks/discontinuities in the welds between these two plates. Table 3 shows the summary of the predicted fatigue lives of the repair options for carrying the M1600 vehicle load.

Very short fatigue lives were predicted for Options 1 and 2 since the options did not include remediation of cracked welds. Option 4 had the longest predicted fatigue life of 22 years, with an even longer predicted fatigue life of 31 years if designed to carry the BD62.5t vehicle load.

The short fatigue life for Option 2 implied that the damaged superstructure needed to remediated quickly within the next 12 months. Even with Option 4 repairs, the bridge superstructure would need to be replaced again or remediated around the year 2040.

Table 3 Predicted fatigue life according to AS5100 using M1600 based fatigue vehicle

Fatigue critical location	Fatigue life (years)			
	Option 1	Option 2	Option 3	Option 4
Location 1 - where the doubler plate is cracked	Not determined	Not determined	7	22*
Location 2 – where the weld between the girder bottom flange and the doubler plate is cracked	7	1	16	48

* Option 4 had a predicted fatigue life of 31 years, if designed to carry the BD62.5t vehicle load.

Determination of preferred rehabilitation method

Due to the expected design complexity of repair Option 4, it was uncertain whether the predicted life could be achieved. A costly and rigorous inspection and monitoring regime, including the use of NDT, would be required to assess fatigue performance over the structure’s life after the repairs. Therefore, the repair option was compared to a full superstructure replacement to determine the preferred rehabilitation option, bearing in mind that superstructure replacement could achieve a design life of 100 years and accommodate SM1600 and HLP vehicle loads. A like for like replacement using the 2009 superstructure replacement design was considered due to its simplicity in design and construction.

The comparison was based on costs, efficiency and risks between these two options, as well as the urgency of rehabilitation. Table 4 shows the comparison. Despite the fact that the estimated cost of replacing the superstructure was 55 percent higher than repair Option 4, superstructure replacement was preferred over repairs. The superstructure replacement design followed the 2009 replacement design, with some details updated before issuing for construction, e.g. general notes and welding requirements.

Table 4 Compare repair Option 4 and superstructure replacement

Feature	**Rehabilitation Options**	
	Repair Option 4	Superstructure replacement
Predicted/design life	31 years due to fatigue load BD62.5t vehicle load	100 years Design to SM1600 and HLP400 vehicle loads
Cost comparison		
Estimated **total cost over 31 years**	Costs for superstructure replacement^ over 31 years were 55 percent more than repairs#. ^ The costs included detailed design, construction, and future routine condition inspections. # The costs included detailed design, construction, future routine condition inspections, and inspection and monitoring using NDT.	
Risk comparison		
Estimated **time for** detailed design	At least 12 weeks to complete	Minimal Reusing and updating existing 2009 superstructure rehabilitation design
Design complexity	The design was expected to be complex due to the extent and state of the damage. It was uncertain whether the predicted life could be achieved, therefore a rigorous inspection and monitoring regime would be required to assess fatigue performance over the superstructure's life after the repairs	Simple design due to reusing the existing 2009 superstructure rehabilitation design
Estimated time for construction	Approximately 7 weeks	Completed in approximately 2 months in 2009 rehabilitation
Construction complexity and quality	Due to the expected complex design, rigorous construction planning and site supervision would be required. Satisfactory site welding quality could be difficult to achieve	Simple and carried out in 2009 rehabilitation TMR personnel who worked on the 2009 rehabilitation were still available to provide input

Feature	Rehabilitation Options	
	Repair Option 4	Superstructure replacement
Summary		
Superstructure replacement using the existing 2009 rehabilitation design would take **less time** to deliver **than adopting repair Option 4.** Furthermore**,** superstructure replacement was considered to have lower **level of risks and uncertaint**y than repair options**.**		

Sustainability

The existing bridge traffic barriers and girder restraint steel brackets were reused and reinstalled. The initiative was made to preserve the environment by conserving resources and reducing waste, also these components were still in good condition. Additionally, this approach reduced construction costs and time, allowing the project to be completed more efficiently and economically.

6. Construction

The superstructure re-construction proceeded effectively, and was completed in just over 2 months, from February 2021 to April 2021. There were minimal changes to the design details during the construction. Figure 11 shows the demolition of the damaged superstructure.

A section of the Bruce Highway was fully closed for superstructure re-construction. The primary focus was on traffic management to divert traffic and maintain an efficient flow on the highway's exit and entry ramps. Detours were implemented to facilitate this.

Figure 11 Superstructure re-construction

(a) Bridge concrete deck/slab demolition

(b) Damaged steel girder removal

7. Strike prevention measures and road safety enhancement

The upcoming Bruce Highway Upgrade, Section D between Woondum and Curra (Gympie Bypass) will divert the majority, if not all, of oversized or heavy vehicles to this future route of the highway. This will reduce the risk of strikes to the overpass from over-height vehicles.

The vertical clearance was increased from 4.6 m to 5.0 m in the last superstructure rehabilitation due to the strike in 2009. Increasing the vertical clearance further by the following methods was considered but rejected:

- raising the deck level of the overpass:

  this would require raising the level of the entry and exit ramps, neighbouring roads and streets, and the Normanby Bridge over the Mary River. This would require substantial cost and cause significant impact on the nearby residents when accessing driveways.

- lowering the Bruce Highway further:

  this would have an impact on three neighbouring water main crossings and a capped mine shaft. The bridge abutments were built up from the existing ground surface and are not supported by piles. Lowering the highway would destabilise the abutments, therefore strengthening would be required.

Bridge strike protection beams were deemed unsuitable due to the potential secondary effects on road users from strikes.

The following strike prevention measures were implemented to further reduce the risk and enhance road safety:

- laser height detection and alert system
- new improved signage, and
- bollards.

Laser height detection and alert system

A laser height detection system was installed on the Bruce Highway on either side of the overpass, approximately 350m away. This system consists of static signs with a light-emitting diode (LED) panel. When an over-height vehicle is detected, a message will display on the LED panel alerting and directing the driver to take the exit ramp off the Bruce Highway before the overpass and follow the detour route. Simultaneously, a pre-recorded message will be broadcast over Ultra High Frequency (UHF) Radio Channel 40 instructing the driver to take the detour. UHF Channel 40 is an Australia-wide road safety channel that is commonly used by truck, oversized vehicle or heavy vehicle drivers. Variable message boards were also installed to provide a dynamic display that directs the driver to take the detour. Figure 12 shows the laser height detection system.

New improved signage

New improved signage was installed to indicate the vertical clearance and detour. All new signs are larger and more noticeable as they have bright orange borders to highlight the message to drivers. Figure 13 shows the new signage.

Bollards

The southbound lane of the Bruce Highway has more vertical clearance than the northbound lane due to the overpass sloping grade. Some drivers are aware of this and intentionally avoid taking the detour by travelling into the southbound lane when heading north. To discourage this behaviour and improve road safety, bollards were placed along the central lane marking. Figure 13(b) shows the bollards.

Figure 12 Laser height detection system

(a) Laser height detection unit

(b) Static sign with LED panels

Source: Google Maps

(c) Variable message board

Figure 13: New signage and bollards

(a) New signage indicating detour for over-height vehicles

Source: Google Maps

(b) Bollards and new signage indicating vertical clearance

8. Current Status

The Bruce Highway Upgrade (Gympie Bypass) opened to traffic in October 2024. At the time of writing, there had been no known strikes on the Normanby Overpass since the superstructure was reconstructed, strike prevention measures were implemented, and the new Bruce Highway route was opened.

9. Conclusions

The steel girders of the single span composite overpass suffered substantial damage as a result of a major strike by a truck carrying an over-height load in 2018, which contributed to existing damage from earlier unreported strikes. Traffic restrictions were imposed shortly after the strike to maintain safety while keeping the overpass operational.

Advanced NLFEA methods were used to determine the strength of the damaged structure. A strain criterion was developed to account for loss of ductility in the damaged members. 3D laser scan data was used to accurately model the damaged and cracked structure. The NLFEA model was then used in conjunction with the strain criterion to determine the strength and remaining life of four repair options. Three of the repair options were expected to restore bridge capacity to carry BD62.5t vehicle load. However, the most feasible repair option was anticipated to have a fatigue life of only 22 years.

The most feasible repair option was Option 4, installation of additional steel bracings and repair of cracked doubler plates and welds by field welding.

The presence of cracks and distortions on the bottom flange doubler plates indicated a risk of failure if the cracks spread. An initial NDT investigation revealed that the cracks were restricted to the bottom flange doubler plates, with no cracks on the bottom flange of the girders. Furthermore, due to the expected design complexity of repair Option 4, it was uncertain whether the predicted life could be achieved.

The combination of NLFEA derived strength and fatigue life provided confidence in the permissible load that could be carried by the damaged structure. Combining the NLFEA results with a rigorous monitoring strategy that included visual inspections and NDT to detect any crack growth provided assurance of safety until an optimal rehabilitation method was implemented.

A cost benefit and risk analysis was carried out to compare repair Option 4 with full superstructure replacement for the optimal rehabilitation method. The analysis considered design life, estimated design construction costs, future inspection and monitoring expenses, design and construction timeframes, complexity etc. Although the estimated cost of superstructure replacement was 55 percent higher than repairs, it was chosen as the preferred option. Replacement had a far longer design life of 100 years, compared to 22 years for repair Option 4, and the level of risks and uncertainties was lower than with the repair options.

The superstructure re-construction proceeded effectively and was completed in just over two months.

The following prevention measures were implemented to reduce the risk of bridge strikes and enhance road safety:

- laser height detection and alert system
- new improved signage, and
- bollards.

The new highway route, Gympie Bypass, reduces the risk of bridge strikes further by diverting the majority, if not all, of oversized or heavy vehicles away from the Normanby Overpass.

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