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Bridge deck analysis of transversely post-tensioned concrete box girder bridges

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ABSTRACT: For rural bridges in Australia, a common design practice is pouring in-situ concrete on top of beams in order to tie all the beams together and distribute load. However, pouring concrete on-site creates more risk and contractors prefer to avoid it. Another method is using transverse post tensioning to tie beams. This article investigated the behaviour of transverse post-tensioning bars in providing load distribution between beams and ultimately comment on their effectiveness compared to in-situ poured decks. Currently, the industry has not completely investigated this matter in order to design post-tensioning accurately. Conservative estimates are currently used in industry today. Current practice is 50% of the design load on the beam where the load is applied in their design assumptions which is quite high. The team modelled concrete box girder bridges with transverse post-tensioning using grillage method. Several factors were investigated including bridge length and width, bridge skew and beam type. From the models, the team concluded that increasing the bridge span increases the load distribution, the load distribution difference is negligible for skew between 0 and 20 degrees and larger shear actions are observed with increased skew and width. It was determined that the worst-case total load on the beam where the load as applied was found to be 40.5%, 9.5% less than current practice. It is recommended that a similar investigation is conducted using a finite element method to gain a deeper understanding.

KEYWORDS: bridge deck; grillage analysis; transversely post-tensioned concrete box girders

1 INTRODUCTION

1.1 *Background*

Bridges are one of the key infrastructures in urban development. The types of bridge infrastructures vary significantly depending on the enormous factors (AASHTO, 2017). Box girder bridges, super T girder bridges and segmental bridges are very popular in Australia. Over the last few decades, construction practice in bridges has changed significantly. The type of supporting components and type of decks of the bridges vary significantly depending on several factors. However, it can be seen that, the in-situ concreting of the bridge deck has not experienced many changes in Australian bridges. An in-situ concrete deck is poured over the supporting components, to tie the structure together, therefore creating a monolithic structure (TMR, 2021).

In Figure 1 below some common methods of constructing a bridge structure are shown.

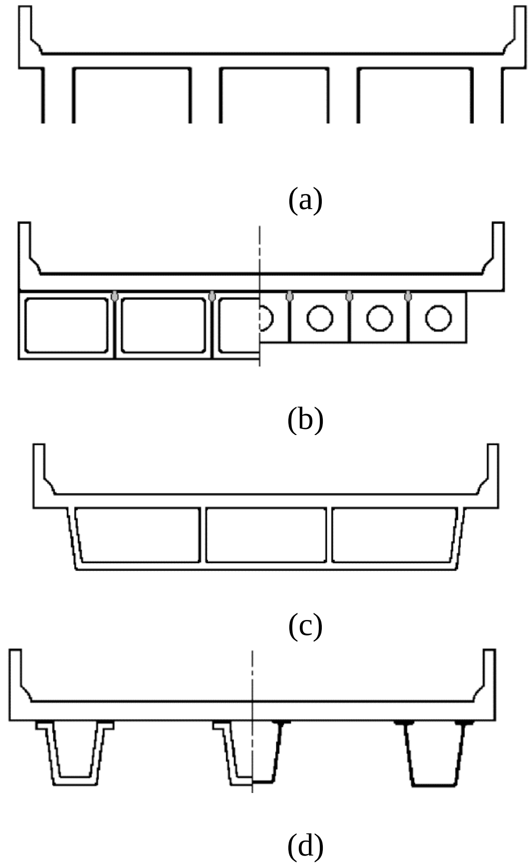
(a)
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Figure 1:a) Cast-in-place Concrete Tee Beam b) Precast Solid, Voided or Cellular Concrete Boxes with Shear Keys c) Cast-in-place Concrete multicell box d) Precast concrete Boxes with Cast-in-place concrete slab (AASHTO, 2017)

Modelling bridge structures is time-consuming for engineers practicing in the industry. A proper study based on Australian standards have not conducted a thorough investigation into the load distribution of these transversely post-tensioned deck units. According to the Tasmanian Government's Design Guide for superstructures with rectangular planks sated that 50% of the design load must be applied to each plank (Li et al., 2020). This assumption is currently using when designing its bridge decks. This assumption means that the wheel loads sitting on top of the beam are directly transferred to the beam below. Evidently, the bridge is designed to account for no transverse distribution between bridge deck units. Consequently, the design is robust in that failure of the transverse post-tensioning would still allow compliance with AS5100 (2017) and the risk of catastrophic failure would be acceptably low. At current times the pre-cast deck units are designed for larger stresses than what they realistically experience.

1.2 *Project Significance*

For the many new bridge projects, bridge contractors prefer to avoid pouring in-situ concrete on-site to reduce project costs and risks. It is now preferred to make as many of the bridge elements in the precast yard, then transport and erect those elements on site.

Often for smaller rural bridges around Australia, a technique is used where the precast beams are erected in place side by side, then grout is poured into the shear keys, followed by the post-tensioning rods being installed transversely through the beams and tensioned. This clamping effect allows the load distribution to be shared between the beams. Figure 2 shows a typical transversely post tensioned precast concrete beam units in a bridge.

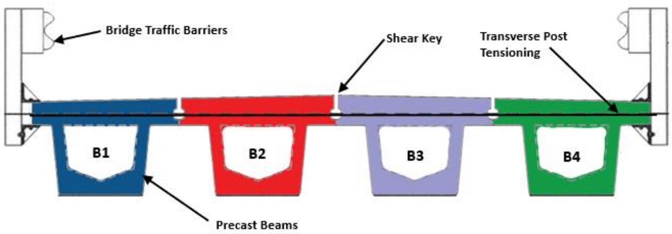


Figure 2: Transversely Post Tensioned Precast Concrete Beam Units

The most common method to ensure monolithic behavior and adequate load distribution in bridge decks is to pour an in-situ concrete deck on top of the precast beams. This method of construction has been used by engineers for decades and the structural performance of these bridges is well understood. The goal when designing a bridge is to make it behave as a monolithic structure, so that the loads are transferred evenly throughout the bridge deck. The in-situ concrete overlay is able to provide load transfer. Figure 3 shows the Super T beam bridge with an in-situ overlay.

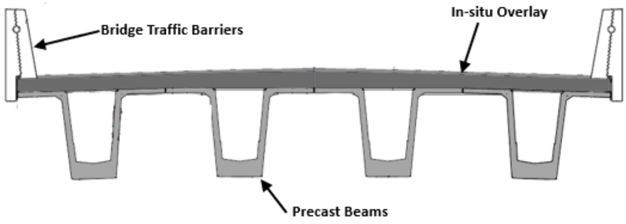


Figure 3: Super T beam bridge with an in-situ overlay

There are downsides of using an in-situ deck as the method of construction. Waiting for an in-situ concrete deck to reach adequate strength is timely and causes an inconvenience to residents that rely on this infrastructure. Not only is it an inconvenience, but it also exposes workers to risks on site and results in increased costs for the project.

Transversely post-tensioned precast concrete deck unit bridges are a brilliant result for contractors in terms of speed and simplicity of construction, aesthetics, risk and cost. However, it is not clear that

transversely post-tensioned precast concrete deck unit is effective in providing load distribution between beams compared to 'traditional in-situ deck bridges. Transverse post-tensioning provides a normal force to the interface that forms a clamping mechanism through friction to resist shear force (Fu et al., 2011). This will increase the shear strength and the performance of transverse connection will be enhanced.

Most of the bridge design firms in Australia have been designing transversely post-tensioned precast concrete deck unit bridges by making conservative assumptions with the load distribution. This study will give an indication of whether the lateral load distribution assumptions that they are making are conservative and guide them going forward with their future design of precast concrete deck unit bridges. This has the potential to reduce costs and materials when constructing bridges using this construction method.

This project will be a numerical study where the bridge decks will be modelled on analytical software. A suitable modelling technique and program will be chosen based on the presented research on the topic. Analysis of the bridge decks will be based on Australian Standards AS5100:2017 Bridge Design and research from previous authors and government bodies.

1.3 *Previous studies*

The most common type of transverse post-tensioned bridge system is an adjacent box girder bridge. In this bridge system, the precast boxes are butted against each other. There are some variations in the design of these bridges. The girders are connected at their interfaces by grouted shear keys and clamped together by full-width post-tensioning at diaphragm locations as shown in the following Figure 4. The diaphragm is created by having a full-depth shear key. In some cases, a structurally composite concrete overlay is used to provide further lateral load transfer.

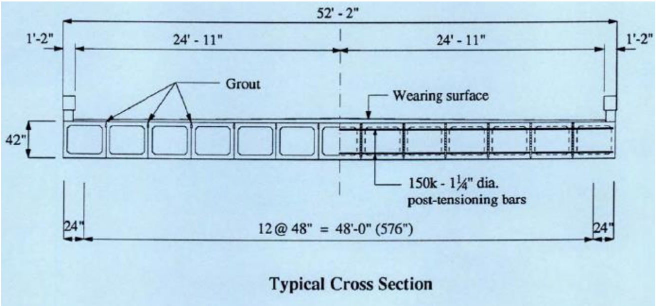
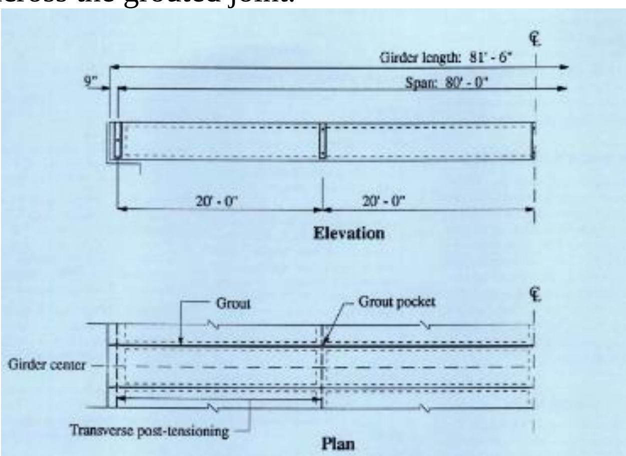


Figure 4: Typical style of adjacent box girder bridge in America (El-Remaily et al., 1996)

El-Remaily et al. (1996) implemented a simple grillage analysis to model the adjacent box girder bridge. The design methodology is based on the provision of rigid post-tensioned transverse diaphragms. The diaphragms serve as the primary load transfer mechanism between adjacent box girders. The bridge deck is modelled as a series of beam elements, the longitudinal members represent the precast box beams, while the transverse members represent the diaphragms.

The geometric properties of both the longitudinal and transverse members were calculated based on the full cross-section of the diaphragm and box beam. Transmission of shear, bending and torsion across the joint is accounted for in this grid analysis. Since the eccentricity created by the two post-tensioned rods as seen in Figure 5, a bending moment is transferred across the grouted joint.



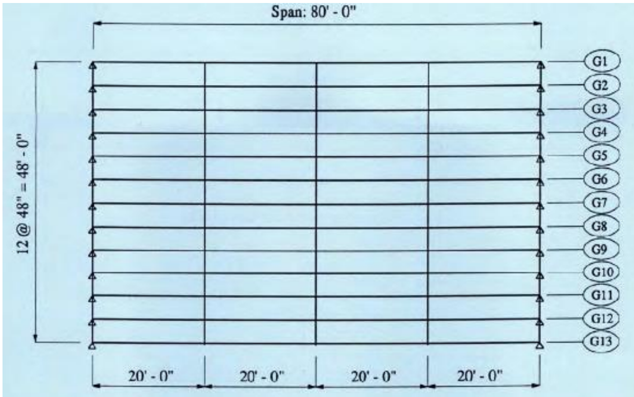


Figure 5: Grid Analysis development (El-Remaily et al., 1996)

In the grid analysis the post-tensioning bars are not modelled. The approach used in this paper was to calculate the transverse bending stresses in the diaphragm and then determine the required post-tensioning force to resist these stresses. Procedure described in El-Remaily et al. (1996) is now used in section 8.9 of PCI's Bridge design guide (PCI, 2014).

Post-tensioning is located in areas where the tensile stress is realised in the structure, in order to provide sufficient flexural strength. Various bridges of different spans, box depths and widths were tested in their study. It was found that "the required transverse post-tensioning force was found to be almost linearly proportional to the span length" and "varies significantly with the bridge width" (El-Remaily, 1996).

Hanna et al. (2009) was able to build on the procedure that El-Remaily et al. (1996) created. Hanna et al. (2009) concluded that the procedure was conservative because post-tensioning forces extend beyond the diaphragm area along the top and bottom flanges of the adjacent box girder. They suggest that transverse reinforcement (rather than diaphragms) is the primary means of distributing transverse loads and preventing differential displacement (Hanna et al., 2009).

For transversely post-tensioned multibeam bridges without diaphragms, this same approach to modelling cannot be used. But the way the authors determine the required PT force is an important conclusion.

Queensland Government's Department of Transport and Main Roads (TMR, 2021) has developed an annexure that compliments their Design Criteria for bridges and other Structures. The 2nd annexure which was developed in 2013 provides recommendations for developing models of deck unit bridge superstructures. A typical section of a prestressed concrete (PSC) deck unit bridge that is built in Queensland is shown below Figure 6.

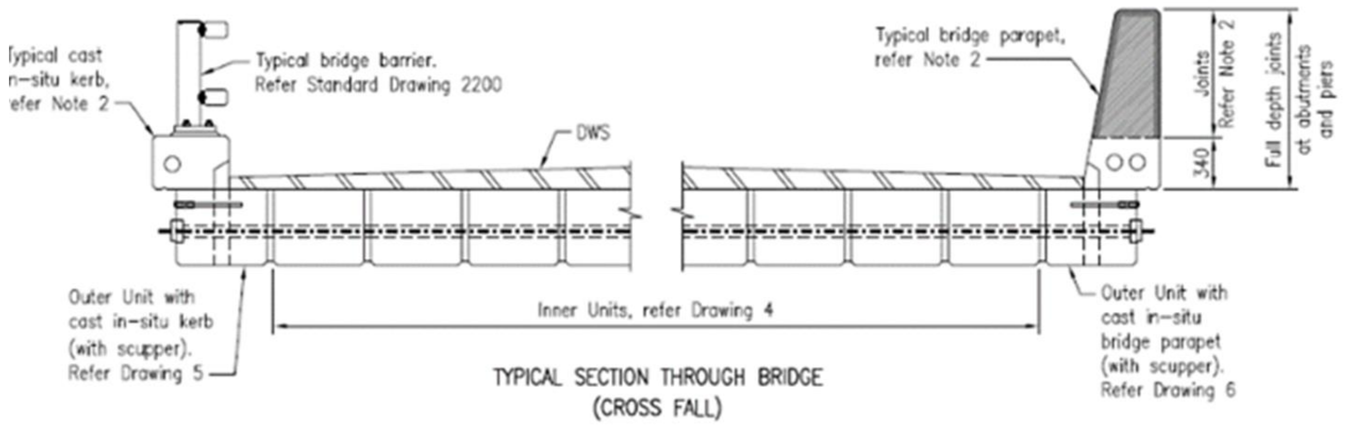


Figure 6: Typical bridge deck in Australia (TMR. 2021)

This type of bridge features transverse stressing bars with a low level of prestressing, stiff upright kerb units and no shear keys (Ngo et al., 2015).

Transport and Main Road's deck unit bridges have been in service since the 1950s and represent a significant portion of the road bridges in Queensland. Under general traffic loads it is assumed that the deck unit bridges behave as a slab with some level of stiffening provided by the kerb units. Bending moments, shear forces and torsions are transmitted as if the deck was a slab (TMR, 2021).

As stated in the document, evaluating how the structure will perform at high levels of overload is important from an assessment perspective (TMR, 2021). At extreme levels of loading, the joints between the deck units open on the tension side, as the bending stresses exceed the levels of transverse prestress (TMR, 2021). The transverse stiffness reduces in these areas and a larger portion of the load is distributed longitudinally. In regions where the bending effects are significant, it is expected that the behaviour will be dominated by the longitudinal cracks opening between the deck units.

The annexure recommends developing a model of the deck unit superstructure that approximates the distribution of load at the ultimate limit state. Modelling the bridge to match the behaviour at the serviceability limit state results in the kerb units and transverse members being overloaded at the ultimate limit state. Modelling the deck using relatively low torsion stiffness in the longitudinal members and modelling the transverse members with low moments of inertia is consistent with the strength of the transverse members at the ultimate limit state (TMR, 2021).

By reducing the torsion stiffness of the deck units, it

assumes the deck units to be cracked at the ultimate limit state loads which is consistent with the AS5100: Bridge Design Code (AS5100, 2017).

The following geometric properties are recommended when modelling the department's deck unit bridges:

- the transverse stiffness in bending is much lower than in the longitudinal direction – 3% of the longitudinal stiffness on a per metre basis

$$I_{trans_m} = \textit{Moment of inertia about the horizontal axis of transverse member used in model}$$

$$= 0.03 \times \frac{S_{trans}}{S_{long}} I_{z.long.du_m} \textit{ for 600 mm wide deck units.}$$

The hypothetical transverse member in the grillage model represents the stiffness created by the bridge slab, grouted joint connection and the clamped action from the stressing bar.

The Queensland Department of Transport and Main Roads (TMR) conducted load testing and in-service monitoring of transversely post-stressed deck unit bridges in 2015, as inconsistencies have been identified between the assessment models and the actual condition of the bridge decks (TMR, 2021). Figure 7 shows the typical loading testing on deck units in QLD.

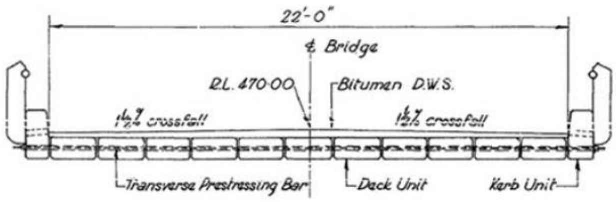


Figure 7: Load testing on deck unit bridge in QLD

The test program involved static and dynamic load testing with various vehicle types and long-term monitoring of the behaviour of a representative bridge under ambient traffic. The results of this program were able to determine that the live load distribution is similar to that of a flat slab and the stiff kerb units attract the majority of the load (Ngo et al., 2015). The bridge was able to perform better than theoretical model predictions. They concluded that the mechanism of how relative movements between deck units affect the transfer of transverse load requires further analysis.

In 2019, a performance assessment of transversely stressed deck units bridges was conducted by the Queensland's Department of Transport and Main Roads (TMR) (TMR, 2021). Load testing was conducted on a decommissioned deck unit bridge span in Queensland to investigate the effects of damaged stressing bars on the performance of the bridge.

Ngo et al. (2015) mentions that for this type of bridge the load transfer mechanism between deck units has not been fully understood and accurately quantified. He further mentions that it is challenging to accurately estimate the capacity of the bridge.

Figure 8: View of the tested bridge underneath

In this study, different levels of damage were introduced to the transverse stressing bars by severing the bars at various locations of the bridge deck. They concluded from this study that incrementally inducing damage to the transverse stressing bars reduced the overall capacity of the structure and reduced the lateral load distribution. An important conclusion was gained from this study. The changes in the load transfer between the deck units are proportional to the reduction in the areas of mortar joints rather than the level of transverse stressing bar damage. The integrity of the mortar joints plays a vital role in the lateral load transfer mechanism of the bridge deck, while the stressing bars contribute to maintaining the integrity of the mortar joints under loads (Ngo et al., 2015). This is an important conclusion, as the mortar joints provide the lateral load transfer mechanism between deck units.

For a deck unit bridge with damaged transverse stressing bars, damage to the mortar joints is highly likely when the bridge is overloaded beyond its ultimate limit state load. When the mortar joints are damaged to a severe level the deck units will carry the load separately, and there will be no transverse load distribution between adjacent units (Ngo et al., 2015).

This observation is supported by the works of Annamalai and Brown (1990), who conducted an experimental program to investigate the effect of transverse post-tensioning in small deck assemblies. They concluded that post-tensioning significantly improved the shear strength of grouted shear key connections. Post-tensioned grouted shear key connections exhibit a high degree of monolithic behaviour under service loads.

Badwan and Liang (2007) implemented a grillage analysis method introduced by Hambly (1991) to model a multi-beam bridge deck as shown in Figure 9. The bridge contains a longitudinal shear key that stretches the full span. Hambly (1991) explains that longitudinal shear key joints possess bending stiffness, therefore the shear keys can be represented by a short transverse member that simulates the dimensions and stiffness of the longitudinal shear key joint. The analysis assumes that the grouted joint has little transverse torsional stiffness.

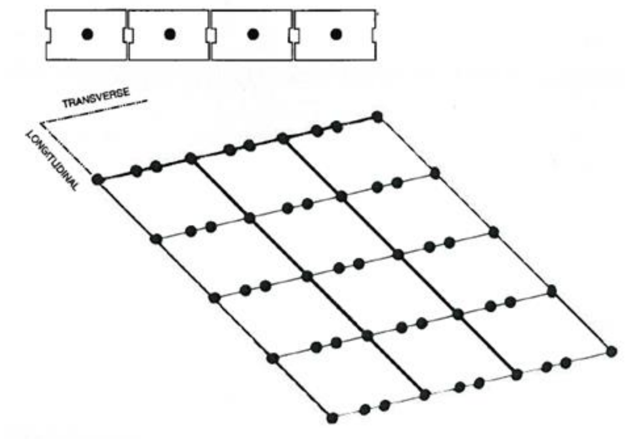


Figure 9: Modelling the bridge deck as grillage analysis (Badwan and Liang, 2007)

Badwan and Liang (2007) was able to calculate the bending stresses in the transverse members using this method. Baldwin came up with the following conclusions:

- The post-tensioning stress was mainly affected by the skew
    - 45-degree skew only required 0.19ksi (1.31MPa) post-tensioning stress whereas 0 degrees required 0.27ksi (1.86MPa).
- As the deck width increases, the post-tensioning stress required decreases.
- The required post-tensioning stress increases for span increases.

Labib et al. (2021) developed a three-dimensional non-linear finite element model using commercial software to investigate the effect of post-tensioning on lateral load distribution. He modelled the concrete, grout and steel elements using four-node tetrahedral elements.

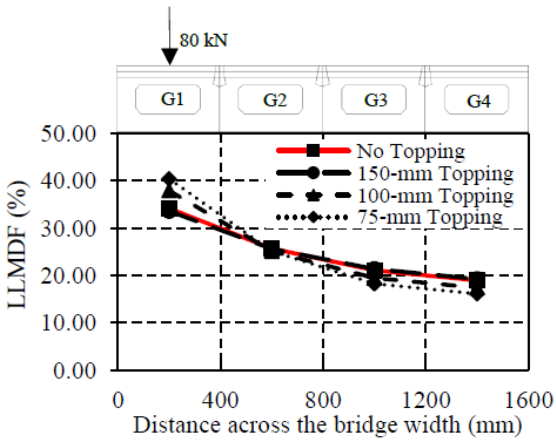


Figure 10: Live Load Moment Distribution Factors of 4-box girder beam bridge under edge loading with different thickness of toppings (Labib et al., 2021)

As expected, the presence of concrete topping enhanced the overall stiffness of the bridge model by

limiting deflections and adding to the transverse load-sharing mechanism. Live Load Moment Distribution Factors (LLMDFs) appeared to be insubstantially affected upon introducing concrete topping. A maximum LLMDF of 34.2 % was observed on the exterior girder for a 4-beam bridge as shown in Figure 10. The LLMDF were found to be reduced when the bridge width increased.

According to the Ministry of Transportation of Ontario’s (MTO’s) Ontario Highway Bridge Design Code the general design philosophy of adjacent-member systems assumes that the entire load between adjacent members is transferred by transverse shear, and the transverse flexural rigidity is completely ignored (Grace et al., 2012). Further experimental work concluded that shear joints cannot transmit flexural forces in the transverse direction, and it’s reasonable to assume that the load transfer mechanism between deck units takes place via vertical shear (Bakht et al., 1983).

Fu et al. (2011) highlights that “the concept of shear friction can be used to determine the level of transverse post-tensioning in adjacent precast multi-beam bridges without diaphragms”. He comments on how especially for adjacent precast solid multi-beam bridges without diaphragms, there are no theoretical justifications for designing the transverse post-tensioning. Fu et al. (2011) was able to come up with a simple approach for designing the post-tensioning for adjacent multi-beam bridges. Considering the worst case where the shear key is cracked, the friction interface between both cracked surfaces provided the mechanism for shear transfer. A prestressing force induces a force normal to the cracked surfaces. The amount of clamping force can be determined by considering the friction coefficient of the cracked interface and the action that needs to be resisted.

2 MODELLING STRATEGY

A number of modelling techniques are available for modelling bridges, such as frame analysis (Hambly, 1991) and finite element methods (Labib et al., 2021, Fu et al., 2011). Figure 12 illustrates the different modelling strategies of bridges.

Labib et al. (2021) developed a three-dimensional non-linear finite element model using commercial software to investigate the effect of post-tensioning on lateral load distribution. He modelled the concrete, grout and steel elements using four-node tetrahedral elements. Figure 11 shows the bridge deck modelled using brick elements. The Finite element method is the most powerful and versatile analytical method available at present as the elastic behavior of any structure can be analyzed accurately.

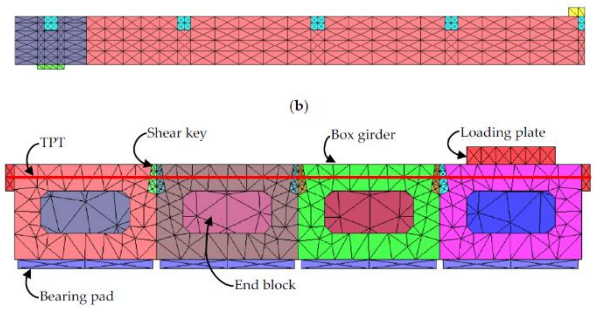


Figure 11: Bridge deck modelled using brick elements (Labib et al., 2021)

A fracture-plastic constitutive model was assigned to the geometric entities for the concrete and grout. The internal TPT reinforcement cable was modelled using truss elements embedded in the solid elements and given a coefficient of friction set to zero to simulate the unbonded TPT. A Mohr-Columb failure criterion was used to model the girder-to-girder interface (Labib et al., 2021).

A limitation of using the grillage method is the failure to account for the distortion of beam members (Badwan and Liang, 2007). The distortion of a box-girder deck can be analysed without complexity by various finite element and space frame analyses (Hambly, 1991).

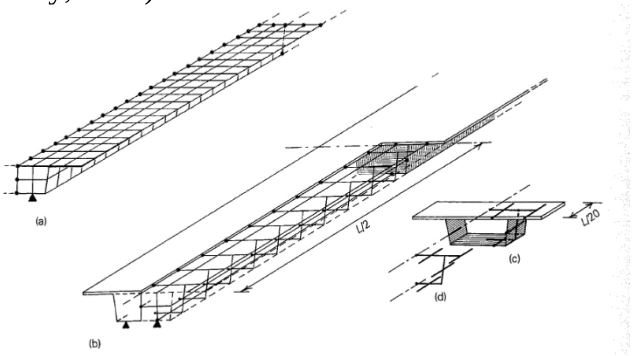


Figure 12: a) finite element models b) space frame model c) & d) illustrates the space frame members that reproduce the force system within the structure

A box-girder is very stiff in pure torsion and most of the twist of the deck is due to pure distortion unless the box is braced with diaphragms or cross-bracing (Hambly, 1991). Prestressed concrete box girders are relatively stiff against distortion, particularly if the webs are thick to accommodate prestressing tendons. The precast concrete beams that are being analysed in this project have a thick web so they are relatively stiff against distortion and a grillage model will not provide any limitations.

The most common analytical method to assess bridge decks is using a grillage analysis, which was developed by Hambly (1991). The grillage analogy has become a standard procedure for modelling bridge structures. In the grillage method, the bridge deck is idealized as an assemblage of longitudinal and transverse members rigidly interconnected at nodes located on the same plane. Figure 13 depicts Hambly's Grillage model for bridge deck analysis.

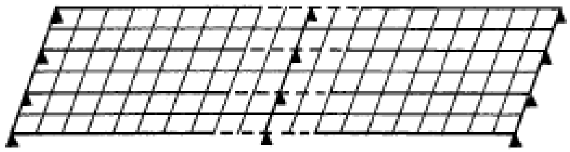


Figure 13: Hambly's Grillage Method (Hambly, 1991)

Hambly (1991) was able to show that the results from a grillage analysis of a box girder is comparable with the results obtained in a space frame analysis of the same box girder (Hambly, 1991). He highlights the attention that needs to be paid to the relevance of the approximations used in the grillage model and Figure 14 shows more details of the grillage analysis.

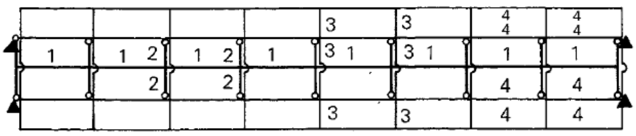


Figure 14: Grillage Analysis of a box girder (Hambly, 1991)

The box girder is represented by the longitudinal spine beam (1) with stiff outriggers (2). The stiffnesses of the box girder in bending, torsion and distortion are represented only by the spine beam. The slab is represented by the transverse members (3) which pass over the spine beam, and by longitudinal members (4)(Hambly, 1991). In this grillage, the twist rotation of the box longitudinal members represents the relative movement of the webs, and the slope of the transverse members reflect the slope of the slab.

This study adapted Hambly's grillage technique modelling the bridge decks (Hambly, 1991). Our team wanted to produce a model that was simplistic, a model where it was easy to validate the results and something that could be replicated by bridge design engineers.

There are drawbacks to using complex FEM software. The engineer must be trained in the use of the complex software to use it efficiently, often it is difficult to understand or verify the appropriateness of the element stiffnesses. It is significantly more challenging to extract member actions compared to grillage models. While FEM can address the level of transverse post-tensioning and shear transfer of the bridge deck it is resource intensive and thus unnecessary for a routine assessment of deck unit bridges.

The grillage method is popular method of analysis because beam behaviour is better understood by engineers than other methods (Badwan and Liang, 2007). For the purpose of our project, we determined that a grillage model would be the most appropriate method of analysis.

2.1 *Grillage Mesh*

Hambly's bridge deck analysis method was used to model our bridge decks on SpaceGass. A series of longitudinal beam elements represent the precast beams, and a series of transverse beam elements represent the slab (Hambly, 1991). Figure 15 shows the Grillage Technique used to model the bridge deck in this study.

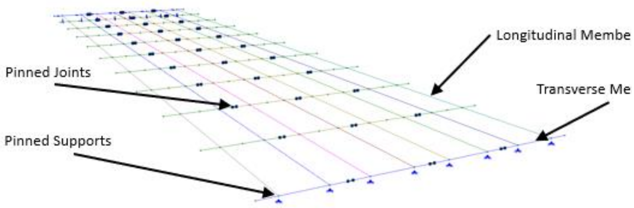


Figure 15: Grillage Technique was used to model the bridge deck

The longitudinal and transverse members are rigidly interconnected at nodes located on the same plane. The longitudinal beams represent half of the precast beam.

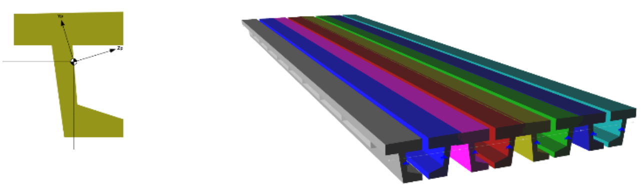


Figure 16: Longitudinal members - precast beams

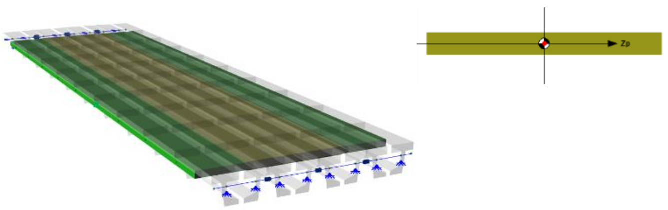


Figure 17: Transverse members - slab deck

Half of the precast beam is modelled as a longitudinal member, as seen in Figure 16. This decision was made so that the transverse member is supported at the webs of the box girders, which is how the top flange of the box-girder is supported realistically. If the slab was modelled from the centerline of the box girders, the actions in the slab wouldn't be representative of real behaviour. Figure 17 depicts the slab deck model developed in this study.

The post-tensioned deck unit bridges being studied in this project have post-tensioned bars located at 2 m centers along the bridge span, the transverse members are located at these locations in the model. According to Grace et al. (2012) the TPT forces are localized at the post-tensioning locations, and transverse pressure is not uniform over the bridge's span. This was a reason for this modelling decision.

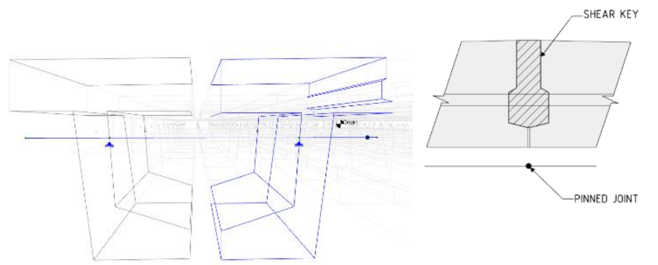


Figure 18: Shear keys of the bridge models

The load transfer mechanism for transversely post-tensioned adjacent multi-beam bridges over the shear keys takes place via vertical shear (Bakht et al., 1983, Fu et al., 2011, Annamalai and Brown, 1990). In practice, it is difficult to predict the bending stiffness of joint configurations and it is then sensible to assume for the purposes of design that the joints are flexible (Hambly, 1991). The shear key is modelled as hinge joint because of this reasoning. Figure 18 shows the shear keys adapted in these models. The precast beams are erected in place onto the crosshead. They are sitting on top of the abutment crosshead. A simply supported beam is an adequate approximation.

<u>Modelling Approximations:</u>
- Modelled half the beams
- Simply supported deck units
- Pinned joint to represent shear key
- Transverse members located at PT locations

2.2 *Sectional properties*

The section properties of the grillage members greatly affect the transverse load distribution. The reasons for factoring our sectional properties are based on provisions recommended in AS5100 (2017) and guidance given by the Queensland Government's Department of Transport and Main Roads (TMR, 2021). Table 1 shows the sectional properties of the models.

Table 1: Sectional properties adopted

	Transverse Members	Longitudinal Members
Torsion Constant (J)	0%	20% (10%)
Moment of Inertia about z axis (Iz)	2%-100%	100%
Area	Nominal	Nominal
Moment of Inertia about y axis (Iy)	Nominal	Nominal

All of the other factors were not touched when inputting sections into SpaceGass. Modelling the deck with a relatively low torsional stiffness in the longitudinal members and modelling the transverse members with low flexural stiffness is consistent with the strength of the transverse members, i.e.. the slab, at the ultimate limit state (TMR, 2021). The uncracked section modulus (iz) can be used for the precast beam as the prestress tendons allow the cross section to remain in compression under load.

A parametric study was conducted to determine the influence of transverse stiffness on the live load distribution. There is little theoretical guidance on what stiffness values to give transverse members (Fu et al., 2011).

2.3 *Loading*

The design loads were determined according to (AS5100, 2017). Serviceability Limit State (SLS) and Ultimate Limit State (ULS) loads on the deck were determined following AS5100.2. The following live load and dead load combinations are based on AS5100.2.

ULS = 1.2 G + 2.34 Q

SLS = G + Q

2.4 *Live loads*

The bridge code specifies the live loading to be applied,

- An SM1600 live load. Which consists of an M1600 moving load and a S1600 stationary load.

The moving load analysis on SpaceGass was used to generate the SM1600 loads on the grillage. Figure 19 shows the SM1600 loads on the grillage.

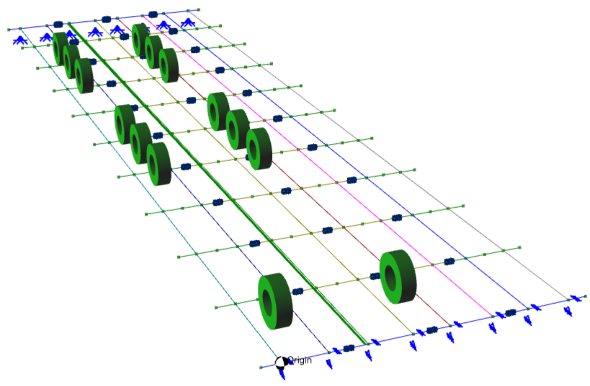


Figure 19: SM1600 loads on the grillage

The position of the SM1600 Live load was altered. As seen below the live load was positioned in the middle deck and at the edge position for the single lane bridge. And there were four loading positions investigated for the double lane bridge. Figures 20 & 21 show the loading positions considered in this study.

Figure 20: Loading positions for single lane

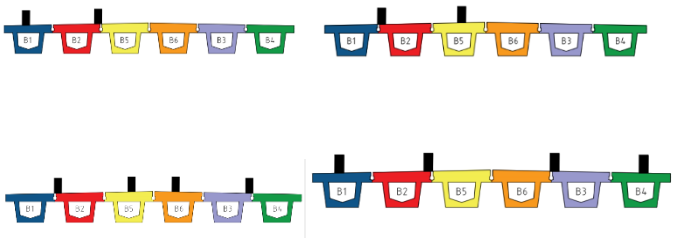


Figure 21: Loading positions for double lane bridge A single lane loaded with a SM1600 load on the external edge, two SM1600 loads on the external edge, one SM1600 load in the middle and two SM1600 loads in the middle

The number of standard design lanes loaded, and the load patterning shall be selected to produce the most adverse effect on the structure in accordance with the bridge code (AS5100, 2017).

2.5 *Dead Loads*

The dead load of the structure was broken into three components:

- Self-weight precast beam
- Self-weight slab
- Self-weight Barrier

See appendix X for loads prescribed for each of these elements.

2.6 *Method*

1. Developed the grillage model
2. Applied dead loads (DL)
3. Loading simulation (LL) at different positions on bridge deck
4. Validation of model
5. Performed a parametric study to see the effect of changing the transverse stiffness value (Iz,trans)
6. Determine Live Load Moment Distribution (LLMD) between longitudinal beams
7. Loading Simulation (ULS) at different positions to determine maximum shear force (V*) in shear key joint
8. Repeated procedure for 9 different bridges

Three aspects of the bridge's geometry were adjusted to see the effect of Live Load Moment Distribution (LLMD):

1. Bridge width: double lane (6 beams), single lane (4 beams)
2. Bridge span: 12m span, and20m span
3. Skew: 20^0 angles, 0^0 angle

2.7 *Determining the PT force*

Modelling the transverse post-tensioning was not deemed critical with our modelling approach. The performance assessment conducted by Ngo et al. (2015) alongside the Queensland's Department of Transport and Main Roads (TMR, 2021) concluded that, the changes in the load transfer between the deck units are proportional to the reduction in the areas of mortar joints rather than the level of transverse stressing bar damage. The integrity of the mortar joints plays a vital role in the lateral load transfer mechanism of the bridge deck, while the stressing bars contribute to maintaining the integrity of the mortar joints under loads (Ngo et al., 2015).

A conservative design approach which was done in previous works by Badwan and Liang (2007) and El-Remaily et al. (1996) was adopted. Our approach required modelling the bridge deck as a grillage then determining the required post-tensioning force to resist the shear action at the shear key location.

The shear key possesses shear capacity, but we have decided to consider the case when the shear key develops a longitudinal crack throughout the full depth as shown in Figure 22. The mechanism creating a connection between the deck elements is friction between concrete interfaces. Eq (1) gives maximum shear capacity of the shear key.

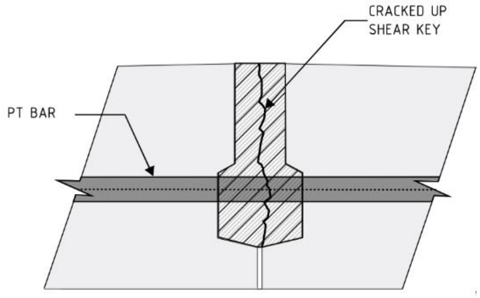


Figure 22: Cracking of shear key

$$V = \mu N_{PT\ force} \qquad (1)$$

2.8 *Validation of the models*

The grillage models were validated by analysing the models and checking the results obtained. Checking that the magnitude of the reactions at the support equals the total applied load.

A deflection check was carried out on each model to verify that the order of deflections was reasonable. Bridges are designed for deflections to be typically less than span/800. A check was performed where only the unfactored live load (LL) was applied to the model to see if the deflection magnitude was less than the allowable deflection.

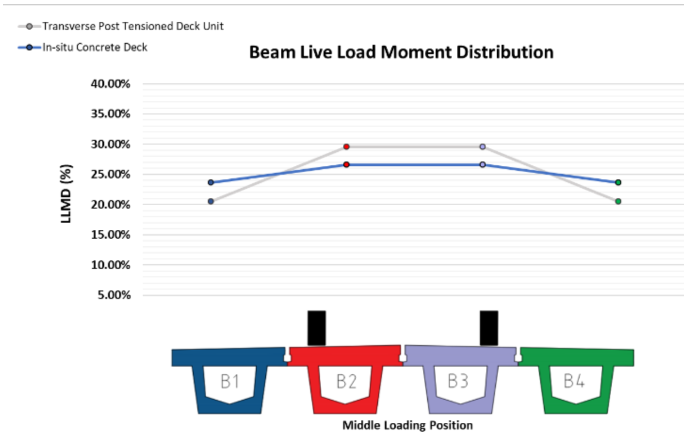
The beams are designed with a camber to allow for Superimposed deal load (SDL) and dead loads. Therefore, the serviceability check was conducted with only unfactored Live Load (LL) applied.

3 RESULTS AND ANALYSIS

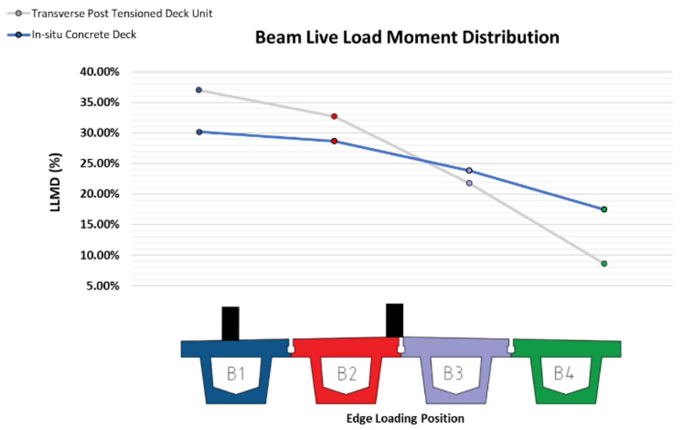
Live load moment distribution (LLMD) will be presented for differing beam types, bridge widths, bridge skews and spans. Live load moment distribution tables are presented in the appendix.

3.1 *In-Situ Deck vs Post Tensioned Deck Units*

A grillage model was created for a 20 meter, single-lane bridge without any skew for both an in-situ deck and a deck unit bridge with TPT. Figure 23 shows the comparison of live load moment distribution in in-situ and transverse post-tensioned deck units for 20 m 4 beam bridge.



(a) Middle loading position



(b) Edge loading position

Figure 23: Live Load Distribution for 20 m, 4-Beam Bridge comparing to the grillage modelling for an-situ deck grillage and TPT deck grillage

From Figure 23, an in-situ deck distributes the moment distribution more evenly than a transverse post-tensioned unit. This difference in distribution is due to the sectional properties of the grillage and having no pinned connection between deck units, hence allowing moment transfer across the shear key. The in-situ deck has no reduction in the transverse stiffness, but still has a 20% reduction in the torsional constant of the longitudinal member as per AS5100.5. This increased transverse stiffness and torsional stiffness of the transverse members allow the LLMD to be significantly higher. The torsional stiffness is still reduced for both members in accordance with AS5100 (2017).

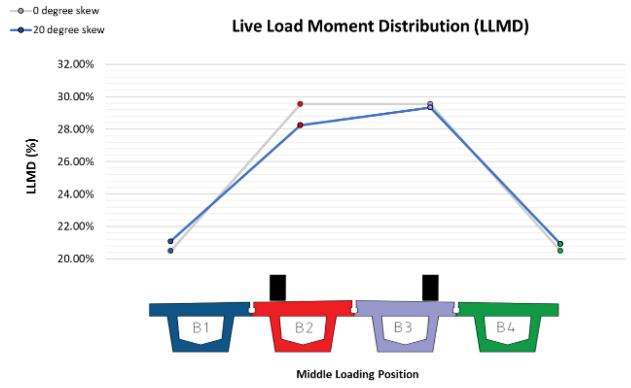
An in-situ deck can be assumed to have a moment transfer between precast beams as there is a continuity of the slab deck between adjacent beams, and furthermore reinforcement provided in the transverse direction.

3.2 *Effects of Skew*

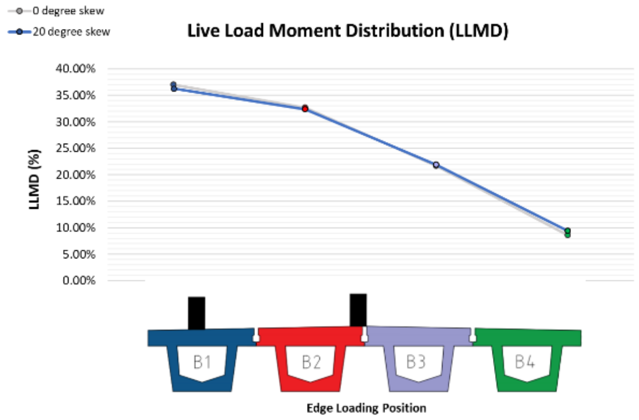
3.2.1 *Single Lane Bridge*

The effects on the lateral moment distribution when skewing the bridge at 20 degrees were investigated for all the different bridge geometries.

The effects of skewing a single-lane bridge at 20 degrees were investigated for an M1600 live load in the middle and at the edge of a single-lane bridge. Figures 24 & 25 depict the effect of skewness on the LLMD of the 20 m and 12 m 4- beam bridges, respectively.



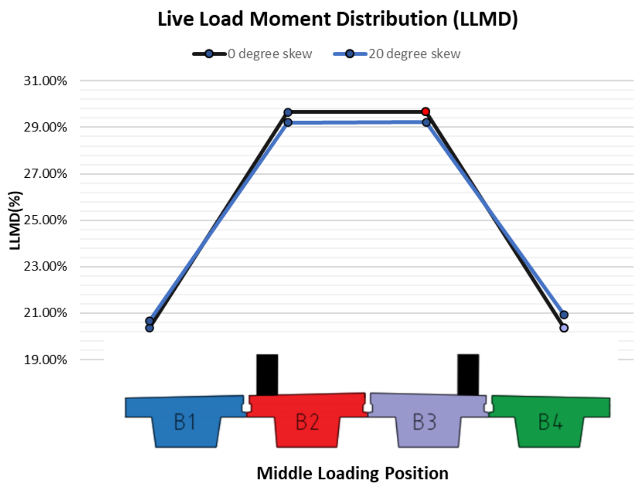
(a) Middle loading position



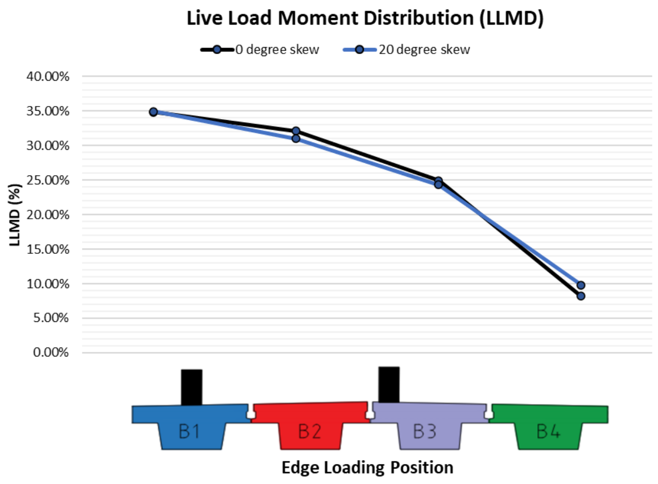
(b) Edge loading position

Figure 24: Effect of Skew on Load Distribution for 20 m, 4-Beam Bridge

It is clear from Figure 24 that there is a minimal effect from skew. There is a slightly different load distribution on Beam 2 where the load is applied, which is approximately 2%.



(a) Middle loading position



(b) Edge loading position

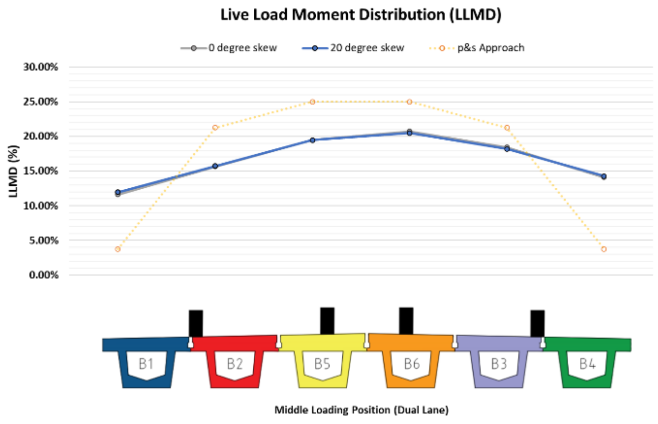
Figure 25: Effect of Skew on Load Distribution for 12m, 4-Beam Bridge

Similarly, the effect of skew on load distribution is negligible. There is a less than 1% difference on load distribution as seen in Figure 25.

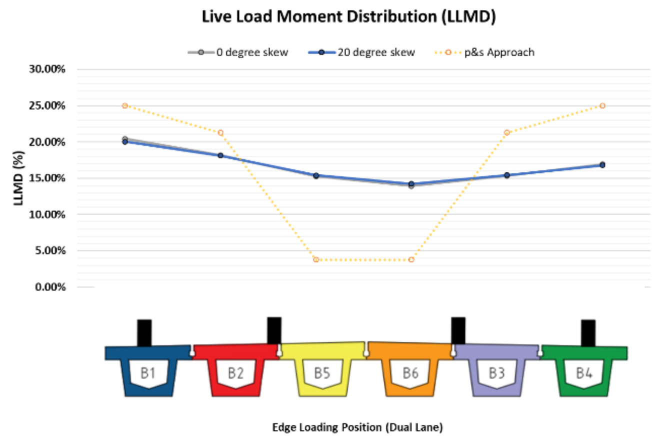
3.2.2 *Double Lane Bridge*

There are four loading scenarios on a double-lane bridge that were investigated.

In Figure 26, a 20-meter 6 beam bridge (double lane) was modelled. The loading positions changed from the middle of the bridge to the edges. The skew was changed from 0 degrees to 20 degrees. The load distribution (LLMD) was compared to the current conservative approach used in the industry (p&s approach) on these Figures 26 & 27.



(a) Middle loading position

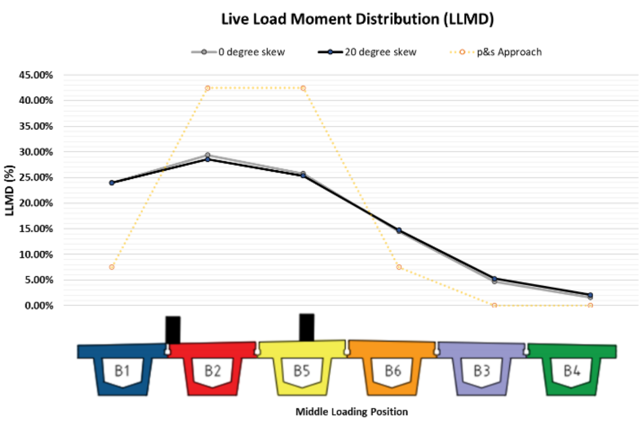


(b) Edge loading position

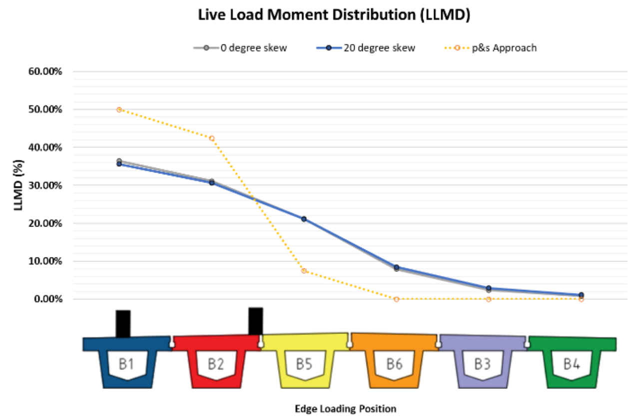
Figure 26: Effect of Skew on Load Distribution for 20m, 6-Beam Bridge (Dual Load)

In Figure 26 above, it is clear that the industry approach was unable to estimate the LLMD on the beams. When the load is more centralised as in Figure 27(a), the load on the outer beams is underestimated by approximately 10% and the middle beams by approximately 5%. When the loading is towards the edges, beam 1, 2 and beam 3 and 4 are overestimated by approximately 5%. beam 5 and 6 are underestimated by approximately 10%.

In Figure 27, LLMD of the 20m 6-beam bridge with only one loading position is described. In this model, the same bridge was used, however, there is only one load, compared to two. The same positions and skew were tested.



(a) Middle loading position



(b) Edge loading position

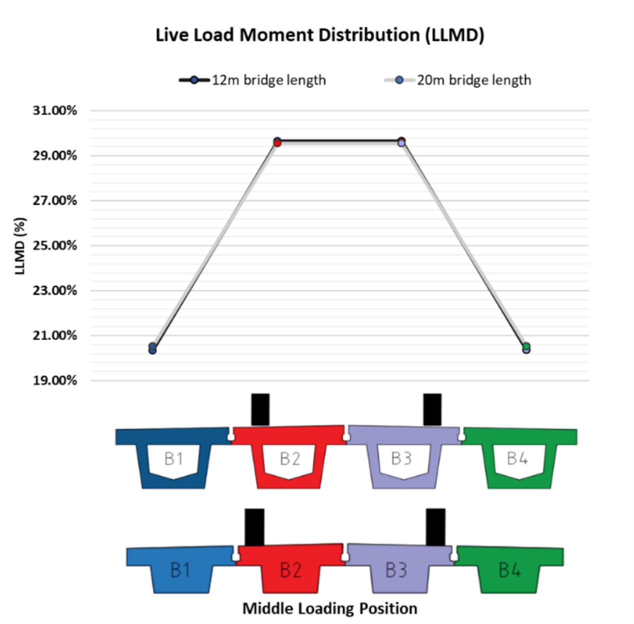
Figure 27: Effect of Skew on Load Distribution for 20m, 6-Beam Bridge (Single Load)

In this Figure 27, industry practices underestimate the middle load on beam 1 by approximately 15% and underestimates beam 4, 3 and 6 slightly. Beams 2 and 4 have been overestimated by approximately 15%. For an edge load, Beams 1 and 2 are overestimated by approximately 5% and the remainder are underestimated. The grillage analysis is able to show that the live load distribution is worst on the external beams and its LLMD is approximately 38% which lower than the assumed percentage of 50%.

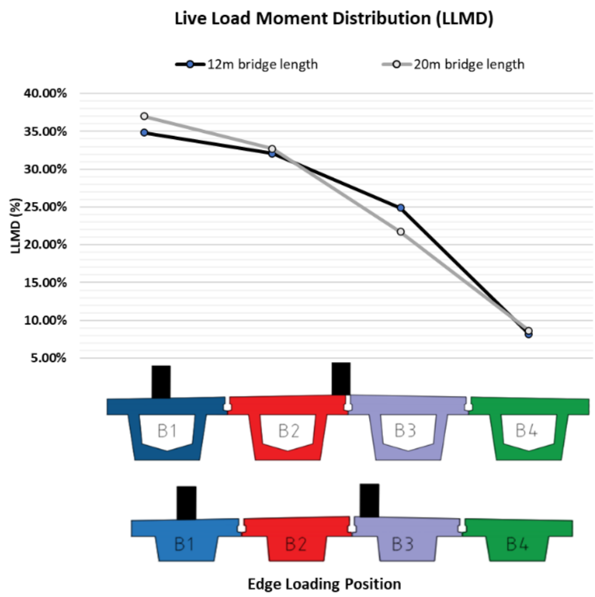
The results have been represented for the larger beam type. Analysis was conducted on the smaller beam type, once again no significant effect was overserved when the bridge deck was skewed 20 degrees.

3.3 *Effects of Beam Type*

The following Figure 29 shows the difference in load distribution for a 4-beam, 0 skew bridge with different beam types and spans.



(a) Middle loading position



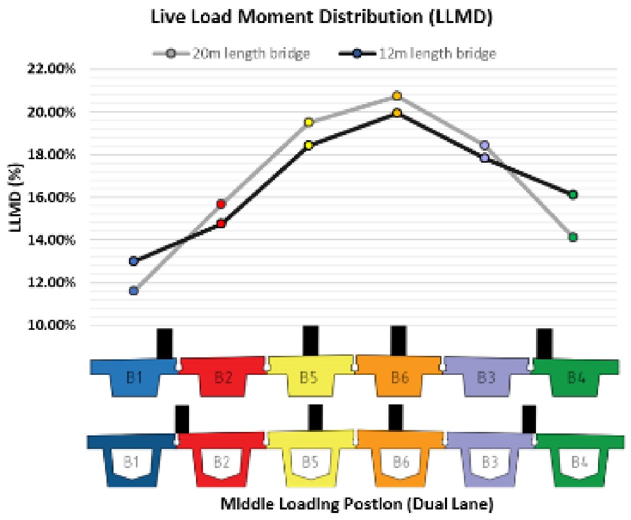
(b) Edge loading position

Figure 28: Effect of Length on Load Distribution for 12m and 20m 4-Beam Bridge. The smaller beam type was 12m span, while the larger beam type had a 20m span

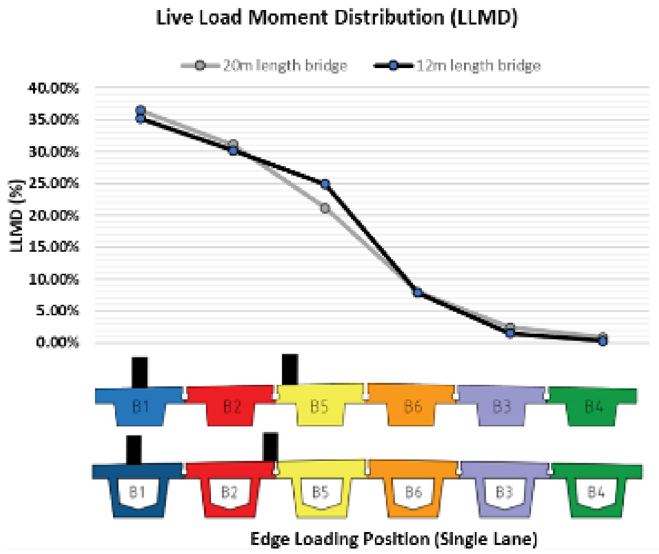
For the middle loading, the results demonstrate that the load distribution is more even than previously thought. Beams 2 and 3 take 30% of the load whilst Beam 1 and 4 take 20%. The LLMD is not significantly different between the two. No significant conclusions can be reached when comparing two parameter changes.

For edge loading, the length made a slight difference in moment distribution between beams. The 12 m bridge was able to share its load more evenly amongst longitudinal members. This may have been partly to do with the wheel locations on the bridge deck.

The following Figure 29 compares the live load moment distribution for a double lane bridge with 0 degrees skew by changing the beam type and length.



(a) Middle loading position



(b) Edge loading position

Figure 29: Effect of Length on Load Distribution for 12m and 20m 6-Beam Bridge (Dual and Single Load)

The 500 deep beam type with a 12m span is able to share its load more evenly amongst precast beam members over the 800 deep, 20m bridge deck.

3.4 *Effects of Span*

The following Figure 30 is comparing a bridge deck with the same beam type, same skew and width, but changing the span of the members.

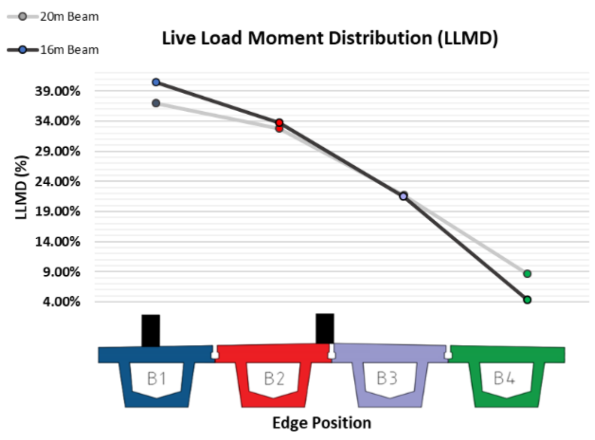


Figure 30: Effect of Span on Load Distribution (LLMD) for 16m span and 20m of the same beam type

Increasing the bridge deck's span causes the load distribution to be more even amongst the beams. This likely cause is the reduced stiffness amongst the deck as two transverse members, i.e., post-tensioning bars, have been removed from the grillage.

3.5 *Effects of Width*

The effect of bridge width has been investigated and is shown in Figure 31.

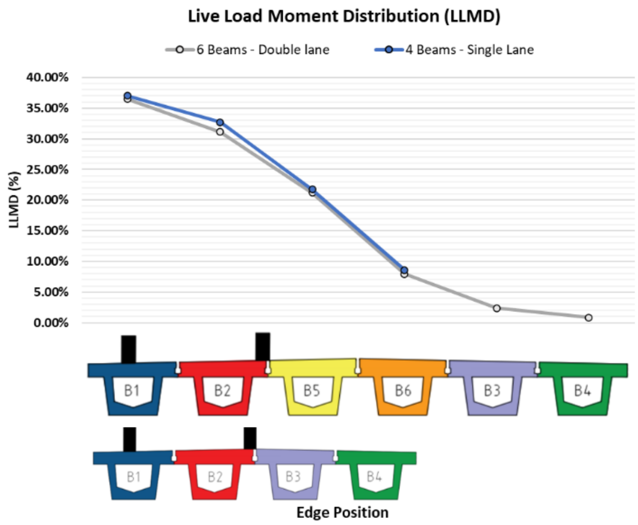


Figure 31: Effect of Width on Load Distribution (LLMD) for 20m Span bridge, changing the width of the deck

By increasing the bridge width, the LLMD percentage is reduced at the critical external beam. For the purpose of design, the beams can be designed for a reduced moment.

3.6 *Shear Demand at Joints*

A result that we did check was the shear transfer across the shear key locations i.e., the pinned joint. To be able to make sure that these moment distribution percentages are compatible with shear demand at joints, maximum shear force at joints all possible loading positions were considered and Table 2 shows the summary of the analysis. An ultimate limit state load was applied to the bridge deck for this check.

The maximum shear force in the joints was realised for the load case when the M1600 moving load was situated in the middle of the span and positioned on the exterior beam. For the double lane bridges the load case that contributed to the worst shear in the joints was when both design lanes where load as close to the edge as deemed acceptable by AS5100.2.

The largest shear forces were realised in the transverse members located close to the supports.

Table 2: Maximum Shear Force (V*) at Joints - ULS

Span		20m		12m	
Width		4-Beam	6-Beam	4-Beam	6-Beam
Skew (deg)	0^0	118.82 kN	164.67 kN	83.71 kN	135.15 kN
	20^0	108.25 kN	209.41 kN	102.79 kN	143.2 kN

The shear force increases across the joint when the bridge is skewed. The wider the bridge deck the larger the shear forces in the joints.

The shear action is proportional to the load applied (single vs double loaded) and the amount of load distribution. Shear transfer is the load transfer mechanism.

3.6 Capacity of Joints

Shear keys provide the load transfer mechanism between beams. The capacity of the shear key is approximately 767kN using the Eq (1). If we assume the shear has longitudinal cracks along the full length and assume a coefficient of friction of 0.9 (AS5100.5). The shear resistance is 318.6kN.

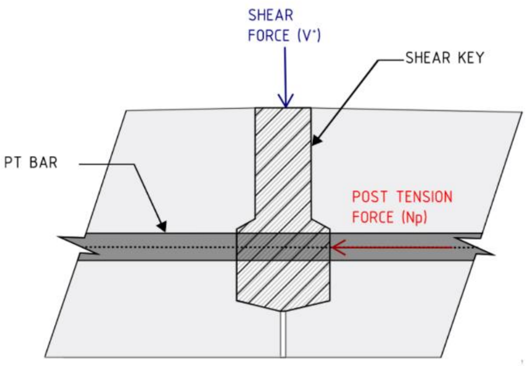


Figure 32: Capacity of the joint

The moment distribution values that we have determined are adequate, even for the worst case when the shear key is cracked up. The friction between the concrete interfaces still allows 318.6kN of shear force to be transferred across the joint. All our models had a shear load of less than 210 kN.

4 PARAMETRIC STUDY

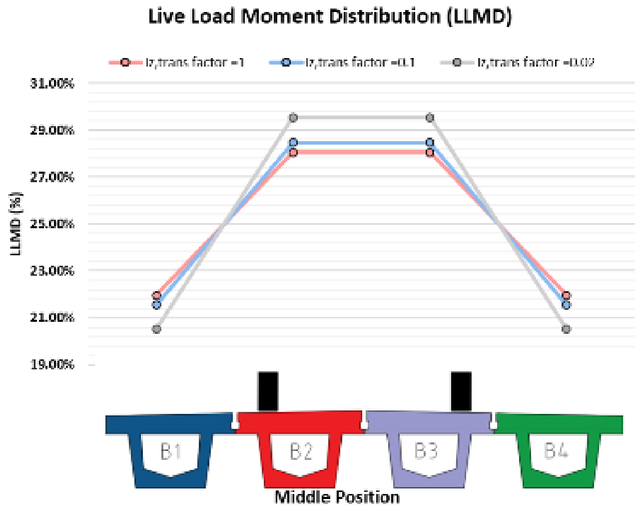
A parametric study was conducted to determine the influence of the torsion constant (J) and moment of inertia (Iz) on live load moment distribution.

The torsional constant (J) and the moment of inertia (Iz) influence the behaviour of the bridge deck significantly. The reduction in torsional constants and moment of inertia are based around the provisions from AS5100.5 and the recommendations from the Queensland Government annexure on modelling deck units (TMR, 2021). Clause. 8.2.1.2 of AS5100.5 states that for checking flexure, shear and torsion under ULS, torsional constants shall be reduced to 20% of the full value and designed for reduced torsion and corresponding moment and shear (AS5100, 2017).

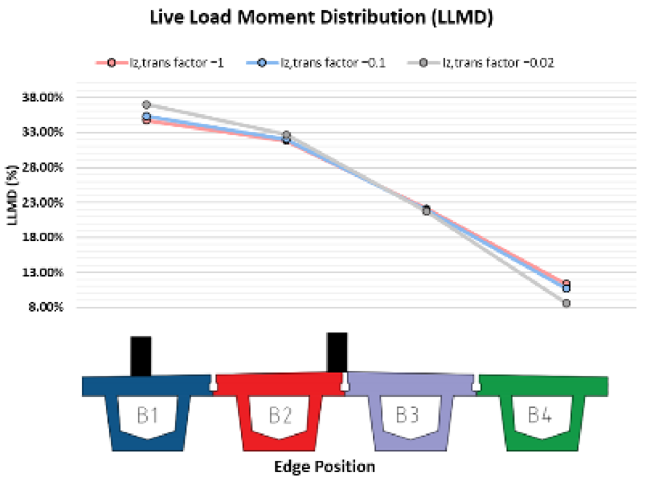
Its recommended reducing the moment of inertia (Iz) to 3% of the original value per meter basis to model the bridge deck under extreme levels of load.

$$I_{Z\ trans} = 0.03\ \frac{s_{trans}}{s_{long}} I_{Z\ long.dum} \qquad (2)$$

With our model, we had a transverse spacing of 2000mm and a longitudinal spacing of 1335mm. Based on spacing of our grillage model, it would be recommended to reduce the stiffness to 4.5% of the original moment of inertia about the z axis.



(a) Middle loading position



(b) Edge loading position

Figure 33: Live Load Moment Distribution for 20m, 4-Beam Bridge, adjusting the transverse stiffness factor

As seen in Figure 33, the lower the transverse stiffness, the lower the amount of moment distribution there is between beams. Changing the Iz factor from 100% to 10% has little change in the distribution compared to the difference between 10% and 2%.

The only context we have for the performance of the TPT deck units in practice is based on the studies conducted by the QLD TMR in 2015 and 2019 (TMR, 2021). There studies showed that the "bridge performed better than theoretical model predictions" in the field tests (Ngo et al., 2015). Even though this conclusion was drawn, the transverse stiffness was reduced to 2% of the original value for the purpose of this numerical study.

The shear force reduces from 219.82 kN to 86.32 kN when the transverse stiffness is reduced from 100% to 2%. The load is distributed towards the stiffer elements, the longitudinal beam.

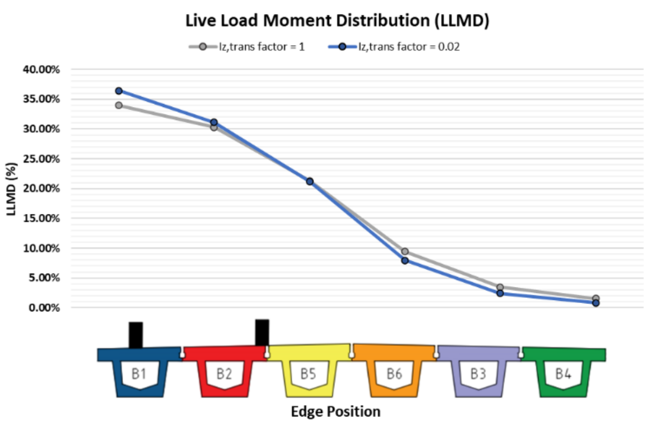


Figure 34: Live Load Moment Distribution for 20m, 6-Beam Bridge, the transverse stiffness was altered.

As can be seen in Figure 34, changing the transverse stiffness from 100% to 2% in 6-beam bridge, only changes the LLMDF by 7.3%. (33.97% vs 36.46%). The shear force action in the joint reduces from 280.21 kN to 164.67 kN when transverse stiffness is reduced.

5 DISCUSSION

The results are able to show the effect of changing the bridge span, changing the beam type, skewing the deck and altering the bridge width on the live load moment distribution (LLMD). The results show that the LLMD is not significantly affected by skewing the bridge deck by 20 degrees. For the purpose of design going forward, grillage models do not need to be skewed when the bridge skew is less than 20 degrees.

Increasing the span improves the load distribution between longitudinal members. The same effect can be seen when the bridge width is increased. The LLMD percentage is more evenly shared amongst the precast elements. It was also observed that larger shear forces (V*) are observed for bridge decks with larger width. Based on this, the post tensioning force should be larger for bridge decks that are wider, but industry practice is to specify the same tension force, 354kN for all their bridge geometries. No enlightening results can be obtained from comparing the bridge decks with different beam types as the beams are different lengths.

The reason why the LLMD is required for engineers is it determines the moment action that the precast prestressed beams need to be designed for. It is common practice for designers to design the beam that is the worst loaded and provide the same prestress and reinforcement for all the beams. The exterior girder exhibited the maximum LLMDF when the load was placed at the farthest possible point across the bridge width, this was consistent with the works found by Labib et al. (2021) in his studies.

The results found are important for bridge design engineers as they will be able to use the LLMD factors to be able to design the precast deck elements in their future bridge projects. Tables that present the different LLMD percentages will be the deliverables out of this project.

Based on the team's ten bridge models, the external beam that was exposed to the largest percentage of the live load was when the bridge was 16m, consisted of 4 beams and no skew. A LLMD factor of 40.46% was observed when the M1600 load was located in the middle of the external beam. The current design practice of 50% of the design load being applied to each plank is more conservative. They have built some redundancy into the bridge system so that if there is failure of the post-tensioning bars, the bridges are designed to be able to adequately carry the ultimate limit state design loads. The integrity of the

PT bars is important for load transfer, without the clamping force the shear key will likely be damaged when overloaded. Not relying on the integrity of the PT bars to provide load transfer is a consideration that should be taken into account for the designers, as these rural bridges will not be inspected on a regular basis.

A comparison was made between the in-situ deck and a TPT deck unit structure. The in-situ deck produces a more desirable lateral moment distribution because of few reasons. The changes made to the grillage model where the reduction factors to the sectional properties and the changes to the joint connection. The torsional stiffness and the bending stiffness were reduced as previously discussed in the results. (Ngo et al., 2015) showed TPT deck units perform better than what the theoretical models predicted when conducting load testing. The theoretical models were based off the modelling recommendations from the Queensland Government department of Transport (TMR, 2021). The team has based the reduction in transverse stiffness in accordance with this recommendation. As a result of this, the results of this project are limited in terms of research but provide good guidance for the bridge industry.

The Queensland Government Department of Transport have not updated the annexure (TMR, 2021) on modelling deck units since 2013, it'd be interesting to see if they change the modelling recommendations based on their load testing and performance assessments recently conducted. The sectional properties of the grillage members affect the load distribution amongst the deck units as seen in the parametric study. Load testing needs to be conducted on this style of bridge to be able to fully understand the behaviour of the deck. Load testing has been done on deck unit planks by the Queensland Government, but load testing needs to be done on this style of deck units with deep beams and with shear keys. Further guidance needs to be produced to inform designers on how to model these structures using grillage models and determining a hypothetical stiffness of the transversely post tensioned deck.

These grillage models do not represent the actual behaviour of these bridge decks, but rather there is an approximate representation of the deck behaviour. The team is able to show with conservative sectional properties for the transverse and longitudinal members of the grillage model that the bridge deck is able to exhibit moment distribution between deck units.

The team did intend on checking the lateral shear distribution of the bridge decks, but the grillage model did not allow for this distribution to be adequate. The shear forces in grillage members are sensitive to how the wheel loads are applied to the grillage structure (TMR, 2021). This is a consequence of approximating a continuum as a series of discrete line elements. Having closer spaced transverse members near supports in order to ensure the peak shears are realised in the critical regions.

The coefficient of friction of 0.9 is advised by AS5100 (2017) for shear planes that are "deliberately roughened by providing a shear key" or "deliberately roughened by texturing the concrete to give a pronounced profile". Our team has used this value only for the purpose of checking the adequacy of the post tensioning force. The coefficient of friction plays an important part in the analysis of the required posttensioning, so additional theoretical and experimental research is recommended to obtain an appropriate design value of the coefficient of friction.

It is recommended following this study that a finite element model (FEM) be developed so that the nonlinear behaviour of the structure can be fully understood. Modelling the structure has as beam elements has its limitations as previously discussed above. By modelling the structure as solid elements, modelling the shear key interface and modelling the compression force created by the unbonded post-tensioning bar, a more accurate representation of the system would be created. If this project was to be pursued further, that would be the direction the project would go.

It is recommended a study be performed to see if skewing the bridges more than 20 degrees affects the load distribution.

6 CONCLUSIONS

This study aimed to investigate the load distribution of transversely post-tensioned precast concrete deck unit bridges of differing beam numbers. The load distribution of the post tensioned system with four beams was then compared to an in-situ deck system with the same number of beams. A six beam transversely post tensioned system was then modelled with differing width, length and skew and the live load moment distribution was again investigated.

The team conducted extensive research in order to determine the best method to model said bridges. A grillage model was adopted in the method.

With the different models, the following can be concluded:

- Increasing the span improves load distribution.
- For bridges skewed between 0 and 20 degrees the difference in load distribution is negligible.
- Larger shear actions (v*) are observed for bridges with increased skew and width.

- Assumptions for load distribution are improved - the worst case is 40.5% of the design load is applied to each beam rather than the previously used, 50%.

However, grillage analysis is only an approximate method for determining the lateral load transfer. So, it is recommended that a finite element model be created to model the interface shear friction between adjacent decks in future investigations. It is recommended that further modelling and research is conducted in order to compare the two types of bridges and gain even more accurate results. Different bridge types, and attributes with transverse post-tensioning present should be further investigated using the method the team has outlined in order to understand the behaviour of post-tensioned deck units more thoroughly.

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