RESILIENCE-BASED DESIGN OF PRECAST STEEL-CONCRETE COMPOSITES FOR RAILWAY TRACK SLABS

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KEYWORDS: Railway infrastructure, Track support structure, Components, Composites, Construction, Maintenance, Replacement, Slab track, and Bridge

ABSTRACT
The relatively high turnover of timber sleepers (crossties in a plain track), bearers (skeleton ties in a turnout), and transoms (bridge cross beams) is responsible for producing greenhouse gas emissions 6 times greater than an equivalent reinforced concrete counterparts. This paper presents an innovative solution for the replacement of aging timber transoms installed on existing railway bridges along with the incorporation of a continuous walkway platform, which is proven to provide environmental, safety and financial benefits. Recent developments for alternative composite materials to replace timber components in railway infrastructure construction and maintenance demonstrate some compatibility issues with track stiffness as well as structural and geometrical track systems. Structural concrete are generally used for new railway bridges where the comparatively thicker and heavier fixed slab track systems can be accommodated. This study firstly demonstrates a novel and resilient alternative by incorporating steel-concrete composite slab theory and combines the capabilities of being precast and modulated, in order to reduce the depth, weight and required installation time relative to conventional concrete direct-fixation track slab systems.

INTRODUCTION
In principle, track components in a railway system are designed to interact with each other in order to redistribute the imposed dynamic loads from the wheels of the railway vehicle to the foundation or support structure of the track [1-3]. These dynamic loads include both vertical loads influenced by the unsprung mass of the vehicles and lateral loads mobilized by centrifugal action of cornering or the momentum of breaking vehicles [4]. In general, two dominant forms of railway structures are ballasted and non-ballasted tracks. Bonnett [5] defined ‘ballasted tracks’ as incorporating an intermediate layer known as the ‘trackbed’ comprising ballast and sub-ballast (or called ‘capping layer’ in Australia) to effectively distribute the vehicle loads to the compacted soil layer called ‘sub-grade’ [6]. If the intermediate load distribution layer is forgone and the track supporting members bear directly on the sub-grade or the superstructure of a bridge or tunnel it is known as a non-ballasted track system. Based on the current design approach, the design life span of structural concrete components is around 50 years [7]. Figure 1a shows a typical railway infrastructure (i.e. railway transom bridge) with existing physical constraints. The rail track is built on timber cross beams, so-called ‘transoms’, which are supported by long-span steel girders between bridge piers. Recently, there has been a significant attempt to convert such transom bridges into direct-fixation track slab bridge as shown in Figure 1b. The design methodology and procedure for track slabs generally yields heavy concrete slabs with a thickness of over 220mm. As a result, the vertical levels (or heights) of adjacent systems such as fastening systems, rails, overhead wires, platforms and existing bridge girders must comply with such track slabs [8-14].
For many decades, concrete structures have been popularly used in railway tracks. However, it is being recognised that the demand to improve serviceability and functionality of rail infrastructure components is significantly increasing. In renewal and replacement of aging railway infrastructure systems and components, physical constraints are often exposed. Some examples are the limitation of bearing capacity of existing bridge steel girders, low platform and track clearances, insufficiency of spaces and cross sections of track structures, and so on. In addition, replacing new components within aging systems requires compatibility, compliance and consistency of strengths, properties and characteristics of those new components [15-17]. These specific situations have resulted in recent adoption of new alternative
materials in railway construction and maintenance. Taking into account constructability and maintainability, this study presents a novel design of resilient composite structures for railway applications. The iconic Sydney Harbour Bridge system was used to illustrate the innovative design of the composite railway track supports, as shown in Figure 2.

Figure 2: Sydney Harbour Bridge

RAILWAY BRIDGE CONFIGURATION
Bridge transoms or sleepers are the members oriented perpendicular to the rails and distribute the rail vehicle loads imposed through the rail to the ballast or superstructure below. Transoms also provide lateral separation of the rails and stability of gauge width between the rails. Currently the most common materials used for intermediate transoms on railway bridges are hardwood timbers. Feasible alternatives for the replacement of deteriorated hardwood timber transoms have been developed recently in Australia [16-17]. According to Manalo et al. [17], existing materials used for railway transoms are timber, concrete and steel with each having their own strengths and weaknesses. Their study disregards both concrete and steel as viable alternatives to timber transoms for the following reasons.

Due to high-frequency dynamic forces, high stiffness characteristics and reduced capacity to flex under load (poor tensile strength), traditional concrete transoms typically require a much deeper section than timber transoms [1, 15-17]. This depth makes traditional concrete transoms relatively expensive and quite heavy, with a typical weight of 285kg. It was also found that concrete structures tend to be the most cost-effective solution for the railway sleeper application in plain tracks. Benefit cost ratio of concrete sleepers was superior to composites [18-22]. In contrast, concrete transoms for railway bridge application fail to enter the rail market due to excessive weight and thickness. For railway bridges, design of track support components (e.g. transoms) is generally governed by vertical space. Many railway bridges have been converted to provide track support with concrete slabs. The precast concrete slabs are often heavy (requiring larger supporting bridge structures), but they are much thinner compared with concrete transoms. However, in many practical cases, such conversion is not always possible due to aging bridge structural systems and associated foundation [16].

Recent developments of new materials and composites are aimed at meeting this opportunity. Fibre composite transoms have been installed on a railway bridge in a coal network in Hunter Valley, NSW Australia. Field reports suggested that there were some technical issues associated with failed fixture bolts and helical spring washers used to pin the fibre composite down to steel girders [16-17]. The fastening system with significant vibration suppression characteristics has become the new area of research and development in a way that it could aid concrete and composite track supporting structures to withstand dynamic loading conditions. According to Shanmugananthan et al. [23], fibre composite transoms would not be a feasible alternative as the relatively new technology would require “intensive pre-testing prior to
installation on the bridge” and would not be cost effective to adopt new sets of such components. Manalo et al. [17] did not consider the use of composite concrete and steel as a replacement alternative to timber transoms. Likewise, none of the intermediate transom material replacement alternatives above address the issue of continuity of the walkway services platform on the railway bridge and these options will be evaluated in this study [4, 24].

RESILIENCE-BASED DESIGN OF COMPOSITE TRACK SLAB
This study has been initiated through consultation with industry partners RailCorp with the specific objective of determining the feasibility of and providing resilience-based design solutions for composite concrete and steel “slab panels” to replace the existing timber transoms within the rail corridor on the main span of the Sydney Harbour Bridge. Steel rails and guard rails will be installed using special baseplates and rail pads onto these modular composite panels. The replacement transom alternatives shall provide a solid rail corridor similar to that of slab track. Although unlike cumbersome slab tracks, the research herein has focused on using composite theory enabling the depth and hence, the weight of the modular replacement panels to be less than that of typical slab track systems. This solid and continuous surface will alleviate the need for the timber decking boards between the railways, prevent water egress to the girder top flanges and arrest foreign objects from falling to the habitable areas below the bridge. Installation time is required to be minimised to lessen the impact of excessive rail possession. This will be achieved by designing the panels as precast, piecewise and modular, so they can be transported to site and installed relatively quickly.

Figure 3: Configuration design for composite track slabs

Figure 3 illustrates an idealised cross-sectional view of the proposed track slab configuration with the existing intermediate timber transoms spaced at 550mm centres spanning between the stringers and providing support to the rails. The proposed configuration with the precast composite panels spans between the stringers and supports the rails while providing a continuous rail platform. The panel designs presented herein have disregarded the 563mm outstand as being ancillary to the design. To assist in retrofitting the precast panels to the existing stringer flanges, the panels shall be cast with holes at the headed shear stud locations. It is anticipated that the panels will be placed over the bridge stringers, the shear stud connectors will then be installed and the holes filled with non-shrink grout to secure the panels in place. The application of blind bolts as shear connectors can be adopted for conventional shear stud connectors [4, 24].

The panel design solutions shall ensure no increase in rail level occurs upon replacement of timber transoms. This is achieved by limiting the maximum depth of the panels beneath the rail pads to 180mm, which is in line with the depth of the current timber transoms. Maintaining the rail level will negate the need to adjust overhead systems and result in cost minimisation. Previous research into the application of composite steel and concrete panels over steel girder bridges conducted by Choi et al. [25] has not considered the application of vehicle derailment loads. The application of precast panels over the SHB herein requires a portion of the panels to cantilever over the supports. The application of derailment loads applied to these cantilevered ends induces considerable negative bending moments and shear forces over
the support. It is necessary for this study to provide a design and analysis for two variations of precast panels, one to accommodate derailment loads which will be referred to as “Derailment panels”; and the other to accommodate standard in-service loads which will be referred to as “In-service panels”.

**Load cases**
In principle, Rail Authority [25] outlines the minimum design criteria for new and existing underbridges and suggested loads applicable to railway components such as live and derailment loads be adopted as specified within Australian Standard AS5100.2 (2004). Design loads and load combinations specified within AS5100.2 (2004) are applicable for consideration in the design of the composite steel and precast concrete panels. The load combinations adopted from AS5100 [26] clause 22 can be tabulated as a case study in Table 1.

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Action</th>
<th>Load Combination (LC)</th>
<th>Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate</td>
<td>Moment</td>
<td>LC1 = 1.4 G_panel + 3.25 Q_300LA + 1.6 Q_breaking</td>
<td>Panel UDL + Pad UDL</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LC2 = 1.4 G_panel + 2.70 Q_300LA + 1.6 Q_breaking</td>
<td>Panel UDL + Pad UDL</td>
</tr>
<tr>
<td></td>
<td>All</td>
<td>LC3 = 1.4 G_panel + 1.5 Q_general</td>
<td>Panel UDL</td>
</tr>
<tr>
<td></td>
<td>All</td>
<td>LC4 = 1.4 G_panel + 1.6 Q_nosing</td>
<td>Panel UDL + Longitudinal Point</td>
</tr>
<tr>
<td></td>
<td>All</td>
<td>LC5 = 1.4 G_panel + 1.2 Q_derail</td>
<td>Panel UDL + Point</td>
</tr>
<tr>
<td></td>
<td>All</td>
<td>LC6 = 1.4 G_panel + 1.0 W_eq</td>
<td>Panel UDL</td>
</tr>
<tr>
<td></td>
<td>All</td>
<td>LC7 = G_panel + Q_300LA + Q_breaking + A_p</td>
<td>Panel UDL + Pad UDL</td>
</tr>
<tr>
<td>Service</td>
<td>Deflection</td>
<td>LC10 = 1.2 G_panel + 2.03 Q_300LA + 0.7 W_eq</td>
<td>Panel UDL + Pad UDL</td>
</tr>
</tbody>
</table>

**Concrete**
The concrete characteristic compressive cylinder strength after 28 days of curing (f'\_c) has been adopted as 50 MPa for durability. The exposure classification for the design of the SHB is considered for surface and exposure environment in “Coastal and any climatic zone”. Australian Standard AS3600 [27] suggests a minimum concrete strength f'\_c of 40 MPa. On this ground, two parameters can be modified in order to obtain the shallowest member profile possible, these are to increase the compressive strength of the concrete and to maximize the moment arm between the internal compressive and tensile forces (then reducing the load action). The latter is achieved by moving the conventional top reinforcement for hogging strength as close to the surface of the members as possible, which reduces the cover over the reinforcing steel. A minimum concrete cover over the conventional reinforcement of 35mm is recommended when using concrete with a f'\_c ≥ 50 MPa [27]. Note that the necessity to minimize cover and maximize f'\_c has led to the adoption of f'\_c = 50 MPa as stated above. Increasing f'\_c may yield greater flexural capacities however; f'\_c must not be less than 40 MPa due to durability concerns.
**Steel reinforcement for shear and tension**

According to Australian Standard AS / NZS 4671:2001 [28], steel reinforcing material of D500N grade has been adopted for all conventional reinforcing steel. The ‘D’ stands for deformed, ‘N’ stands for normal ductility and the 500 stands for 500 MPa, which is the yield stress of the reinforcement.

**Profiled steel sheeting**

The profiled steel sheeting adopted for design is the Bondek II profile, manufactured by BHP Building Products and has been selected for a number of reasons. The Bondek II section, commercially available, is manufactured from high tensile steel with yield strength of 550 MPa. It has a zinc coating for corrosion protection and durability and is available in 0.60mm, 0.75mm and 1.00mm base metal thickness [29]. 1.00mm BMT will be adopted for the design. The standard module width for Bondek II is 600mm with three troughs of approximately 180mm in width which makes the profile suitable to accommodate three equally spaced sets of shear stud fixings to the bridge stringer. Handling and installation manageability along with manufacturing and installation time have been considered in the selection of the panel width. While it may be easier to handle narrower panels, it is desirable to maximize the panel width to reduce the number required to be manufactured and installed. The 600mm panel width has been selected as an appropriate medium between the two criteria.

It is common practice to install fastening systems at 500mm interval over track support structures as the reduction in spacing reduces the reaction at each rail pad. However, to ensure symmetry of the design and analysis herein, the fastening systems have been designed to be spaced at 600mm centres.

![Figure 4: Plastic rectangular stress block theory](image)

**Design for flexural strength**

The negative moment regions of the panels have been designed as singly reinforced beams as per AS3600 (2009) with standard top reinforcement in tension and no contribution of the profiled steel sheeting in compression. The panels have been designed as under reinforced members by assuming tensile reinforcement has yielded to avoid brittle collapse at ultimate loading, this assumption is verified by calculations [24].

The design procedure for determining the positive flexural strength of the precast panels has been adopted from Goh et al. [29] and it incorporates the contribution of the profiled steel sheeting acting compositely with the concrete. Crucial design parameters of the profiled steel sheeting include ultimate yield strength, cross sectional area second moment of area and parameters relating to horizontal shear strength. The design approach adopts simple plastic rectangular stress block theory (see Figure 4). Full scale testing presented by Goh et al. [29] was conducted to determine the mechanical resistance or “rib shear strength” between the Bondek and the concrete in a similar manner to the ‘m – k’ method. This rib shear strength, which is nominated as ‘Hr’, [29], is used to determine the degree of shear interaction and in turn the tensile stress $T_{sh}$ within the sheeting at various distances from the supports. With $T_{sh}$ known, the positive flexural capacity can be obtained [24].

**Design for shear strength**

The design of composite slabs with regards to the vertical shear capacity of both positive and negative bending regions can be significantly affected by the reduction of concrete area due to the dimensions of
the profiled steel sheeting ribs. The design principle requires that the area of the concrete removed by the profiled steel sheeting ribs be not greater than 20 per cent of the total slab cross sectional area [4]. The negative shear capacity of the panels has been calculated in accordance with Australian Standard AS3600 [27] assuming similarity to a typical reinforced concrete beam incorporating conventional tensile reinforcement and disregarding any contribution from the steel sheet composite action [24, 29]. For the design herein, the positive shear actions consistently occur within the middle span of the panels meaning the profiled steel sheeting is anchored either side of the shear failure plane at the shear stud welding points of each support [30-37]. The positive shear capacity of the panels has therefore been calculated in accordance with AS3600 [27] assuming similarity to a typical reinforced concrete beam with the profiled steel sheeting acting as conventional tensile reinforcement at the full depth of the composite slab.

COMPOSITE TRACK SLABS

Figure 5 exhibits the resilience-based design outcome of steel-concrete composite panel for the track support structure. Derailment panels are required to be 180mm thick for the mid-span of the panels and below the fastener locations. The depth of the panel within 616mm from the edge is required to be increased to 230mm to accommodate the substantial negative bending moments induced by the derailment loads. The panels require 4 off N24 reinforcement bars throughout the length and 2 legged N10 shear fitments are also required throughout the length as shown. This is due to the fact that derailment loads could be applied at any location within the panel [24].

Table 2 provides a summary of the design actions imparted on the panels as well as the corresponding design capacities of the panel detailed within Figure 5 to resist each design action [38-40]. Figure 5a shows the design outcome for derailment panel and Figure 5b shows that of the in-service panel (excluding the derailment load). The design ratio shown in Table 2 equals the design capacity divided by the design action and must be greater than 1 for the design to be sufficient. It can be noted here that the 180mm depth of the-mid span is controlled by the shear capacity of the derailment panels while the edge depth is controlled by the negative flexural strength of the panel. Further research into the resistance of

![Figure 5: Precast composite panels for track support structure](image-url)
shear actions by the precast composite concrete and steel panels may be effective in yielding a reduction in mid span panel depth [24].

Table 2 Summary of composite panel design capacities

<table>
<thead>
<tr>
<th>Design Action</th>
<th>Design Capacity</th>
<th>Design Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>(+ve) M*</td>
<td>46.33 (kN.m)</td>
<td>φM⁺ᵤₒ 62.4 (kN.m)</td>
</tr>
<tr>
<td>(-ve) M*1</td>
<td>-108 (kN.m)</td>
<td>φM⁻ᵤ₁ 112 (kN.m)</td>
</tr>
<tr>
<td>(-ve) M*2</td>
<td>-58 (kN.m)</td>
<td>φM⁻ᵤ₂ 76 (kN.m)</td>
</tr>
<tr>
<td>(+ve) V*</td>
<td>142 (kN)</td>
<td>φV⁺ᵤ 142 (kN)</td>
</tr>
<tr>
<td>(-ve) V*</td>
<td>-217 (kN)</td>
<td>φV⁻ᵤ -217 (kN)</td>
</tr>
</tbody>
</table>

Note: * = Design shear capacities are conservative as the fitment spacing has been rationalised in accordance with slab depth. For example, the shear fitment spacing required to achieve the design capacities shown may be greater than the shear fitment spacing nominated due to design standards.

<table>
<thead>
<tr>
<th>Design Action</th>
<th>Design Capacity</th>
<th>Design Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>(+ve) M*</td>
<td>6.30 (kN.m)</td>
<td>φM⁺ᵤₒ 6.30 (kN.m)</td>
</tr>
<tr>
<td>(-ve) M*</td>
<td>-35.41 (kN.m)</td>
<td>φM⁻ᵤ -35.41 (kN.m)</td>
</tr>
<tr>
<td>(+ve) V*</td>
<td>94.48 (kN)</td>
<td>φV⁺ᵤ 94.48 (kN)</td>
</tr>
<tr>
<td>(-ve) V*</td>
<td>-94.48 (kN)</td>
<td>φV⁻ᵤ -94.48 (kN)</td>
</tr>
</tbody>
</table>

* = Design shear capacities are conservative as the fitment spacing has been rationalised in accordance with slab depth. For example, the shear fitment spacing required to achieve the design capacities shown may be greater than the shear fitment spacing nominated due to design standards.

CONCLUSIONS

Considerable needs to replace timber components in railway construction and maintenance requires different thinking, especially in some specific fit-for-purpose projects such as brown-field or aging railway transom bridges. This paper presents a new design of composite panels, which provide resiliency for either a spot replacement or a total renewal of aging timber components within railway infrastructure. Applications of composites to railway construction and maintenance require comprehensive considerations and systems thinking approach. This paper has demonstrates practical design issues and bridge-track requirements associated with the structural design of the composite slabs.

Using the Sydney Harbour Bridge as a case study, the limitation of bearing capacity of existing bridge steel girders or stringers, low track clearances, insufficiency of spaces and cross sections of track structures due to existing overhead wiring structures, and so on, has proven that the designed composite panels are a very attractive alternative. The composite panels have the capabilities of being precast and modulated in-order to reduce the depth, weight and required installation time relative to conventional slab track systems. This study also considers the design criteria for derailment loads since previous research and most transom designs in practice have neglected to consider. Two innovative designs have been proposed: composite panels with or without derailment concern (so-called ‘in-service’ and ‘derailment’ panel, respectively). Comparing with traditional concrete direct-fixation slabs, the weight of the composite panels could be reduced by 20-24% and 10-12% for in-service and derailment panels, respectively. Taking into account constructability and maintainability, this paper is aimed at providing a resilience-based design alternative and analysis guidance for railway engineers in order to safely and efficiently utilise new composite slabs as an essential part of railway infrastructure systems.

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