SAFETY, RISK AND UNCERTAINTIES IN TRANSPORTATION AND TRANSIT SYSTEMS

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SAFETY, RISK AND UNCERTAINTIES IN TRANSPORTATION AND TRANSIT SYSTEMS

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EDITORIAL
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Editorial: Safety, Risk and Uncertainties in Transportation and Transit Systems

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Editorial on the Research Topic

Safety, Risk and Uncertainties in Transportation and Transit Systems

Disruptions in the operation of transportation and transit infrastructure may put at risk the functioning of our societies and their economies. Such disruptions may result from many kinds of hazards and physical and/or cyber-attacks on installations and systems. Safety is the first priority in operating transportation and transit systems. The public and customers rely on operators to assure them the reliable and safe day-to-day uses of public transports. To improve safety and reliability of transportation and transit systems, many key engineering implementations on board and on site have been innovated. In addition, based on recent facts and evidences, extreme physical and cyber threats become more common and even more dangerous to the public. Such examples are the terrorist attacks in St Petersburg in 2017, in London in 2017, in Stockholm in 2017, in Brussel in 2016, in Nice in 2016, and so many more. These examples have one thing in common. They all targeted at transportation and transit system, either on rail, bus, car, or truck, etc. This research topic will further promote and encourage research, development, policy, and innovation in improving safety, managing risks, and mitigating uncertainties in transportation and transit systems where perspectives from the humanities and the operations are also included.

It will be aligned with United Nation's Sustainable Development Goals, especially:

- Goal 11. Make cities and human settlements inclusive, safe, resilient and sustainable;
- Goal 16. Promote peaceful and inclusive societies for sustainable development, provide access to justice for all and build effective, accountable and inclusive institutions at all levels; and
- Goal 17. Strengthen the means of implementation and revitalize the global partnership for sustainable development.

The topic attracts very recent research work and very best discussions over a wide range of timely issues on technologies and innovations focusing on broad aspects of safety, risk, and uncertainties in order to address global grand challenges and UN's sustainable development goals with great social and economic importance. One of the papers published has won the inaugural Professor Joseph M Sussman Best Paper Prize in early 2019.

Along this line, Goto et al. shared extensive Japanese experience on the real service lives of railway concrete sleepers where the presence of fatigue failure is rarely observed in reality. The insight will underpin the sustainable development of new ISO standard for railway track concrete sleepers, taking into account safety, risk, and uncertainties in operational and maintenance.

Bin Osman and Kaewunruen presented a novel data-driven risk cascade using twitter data that help inform the stakeholders and rail operators to improve safety and risk of tram operations. A real case study using data fusion technique reveals an impetus to railway industry to effectively partake in data exploration.

Ngamkhanong et al. highlighted the effects of far-field earthquakes on the cantilever mast structure and the response of OHLE. The insight in this earthquake response of OHLE and its support has raised the awareness of engineers for better design of cantilever mast structure and its support condition, to mitigate multi-hazard risks and to enhance resilience in built environments.

Mirza and Kaewunruen investigated resilience and robustness of railway track slabs exposed to train derailments. The structural safety has been analyzed critically to provide the insight into the robustness of the infrastructure. The profound insight is essential for risk and safety management of transportation infrastructure, from the bottom up.

These papers provide an insight on advanced methods and concepts for the prevention, mitigation, assurance and development toward safer built environments via transportation systems. The topic editors are in significant debt with the review editors and associated editors. We also wish to congratulate the authors of the 2019 best paper (Ngamkhanong et al.) and hope to see more submissions on this research topic in the future.

AUTHOR CONTRIBUTIONS

All authors listed have made a substantial, direct and intellectual contribution to the work, and approved it for publication.

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ORIGINAL RESEARCH
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Resilience and Robustness of Composite Steel and Precast Concrete Track Slabs Exposed to Train Derailments

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Sydney Harbor Bridge (SHB) is an iconic structure constructed in 1923 in Sydney, connecting North Sydney and Sydney CBD. It is a steel-arch structure that caries eight road lines and two rail tracks. Rail tracks of the bridge presently use conventional timber transoms (or sleepers) as track support structure. Replacement of aging and failing timber transoms annually cost public tax money significantly as a result of the high turnover rate of components caused by salt spray, electrolysis, humidity, aggressive dynamic condition, and so on. Steel-concrete composite transoms have been found to be a feasible alternative to timber transoms, which enable systems compatibility with the structural configuration of SHB. However, critical literature review on composite transoms reveals that their impact failures due to train derailments have not yet been considered. In fact, the derailment impacts can cause progressive failure to the bridge structure system. This paper therefore investigates the unprecedented impact damage and failure modes of composite transoms during train derailments. The derailment loads was considered in accordance with Australian Bridge Standard AS5100. The train derailment load was simulated using three dimensional non-linear finite element modeling by ABAQUS. The damage and failure behavior of the precast concrete-steel composite transoms was then analyzed. The ductility of composite slabs can be observed from the yielding of both reinforcements and steel sheets during the impact loading. The dynamic finite element analysis is found to be capable of making reasonable predictions by determining the possible failure modes of steel-concrete composite track slabs subjected to impact loads.

Keywords: railway bridge, steel-concrete composite, transom, impact damage, failure analysis, dynamic finite elements, resilience, robustness

INTRODUCTION

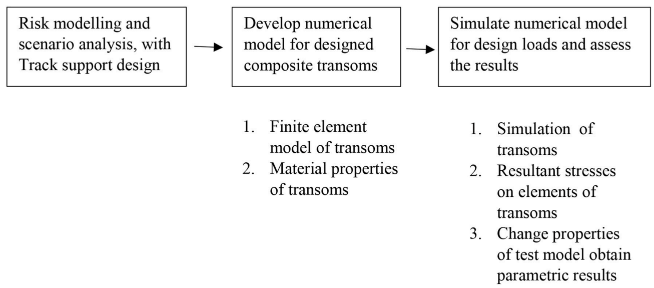
Ninety-two percent of Australia's aged railway bridges are using timber transoms (Krezo et al., 2016a). At present, railway infrastructure managers require to renew around 3.5 million of aging and failing timber transoms in Australia in order to upgrade the railway lines and to maintain existing rail lines. Timber transom are generally replaced within 10–20 years' period of time (Griffin et al., 2014, 2015). This research was undertaken to explore the replacement of timber transoms in the iconic Sydney Harbor Bridge (SHB) using new materials and design. Composite transoms are a good alternative to timber transoms due to the production of greenhouse gas emission for timber transoms are six times higher than in the concrete and composite transom (Krezo et al., 2016b). Their design life time can be longer than 50 years. An alternative for the replacement of timber transoms shall be compatible with the existing structural system. Furthermore, it should also be able to be replaced without much effort in order to enhance resilience and maintainability (Kaewunruen et al., 2015a,b). Therefore, composite precast concrete retrofitted on the existing steel girder would be a good alternative in order to provide safe, resilient and reliable track supports.

Derailments keep presenting challenges to the railways system continuously around the world. A train derailment can cause the losses of money, damage, human injuries, and even life casualties. A recent example was the Kalinga-Utkal Express train derailment in November 2017 that killed 23 passengers on board[1] In fact, the derailments could also occur on a straight viaduct due to broken bogie axles, which could potentially lead to serious casualties (Australian Transport Safety Bureau, 2014). The objective of this paper is to investigate the complex and unprecedented behavior of composite transoms caused by derailments. Failure modes of the rail transoms due to train derailments need to be well-understood for public safety protection and fail-safe design principle of critical infrastructures. This is to be able to improve a performance-based design methodology for railway bridge transoms using steel-precast concrete composite members. Possible failure modes can be identified by previous failure investigations of railway bridges. Grayigg, Savannah River site, is a clear example for progressive failure of railway structures triggered by derailments (The Guardian, 2016). Therefore, train derailment is a scenario, which cannot be ignored or underestimated in designing structural components of a railway system (Kaewunruen et al., 2016).

The steel-concrete composite transoms can be tested in laboratories. However, carrying out experimental investigations is a very expensive and time consuming process. Therefore the paper herein proposed using numerical analysis to investigate the failure modes and behavior caused by derailment. Brabie and Andersson (2006) have investigated high speed derailments through various computer models. This simulation has been done on the wheel–sleeper impact when derailments occurred. They have further enhanced the model to simulate a post derailment scenario. Gu and Franklin (2010) created a model to analyze dynamic impact loading accurately. They considered the response of the railway bridge over the traveling speed of the train. The finite element method in combination with multi-body simulation has been adopted for derailment analysis. The multi-body simulation is often used to idealize train components and some of track elements (e.g., rail and fastening system). The track structures (e.g., track slab, sleepers, structural components, support structures) are commonly modeled using finite elements to enable failure mode investigations.

Ju (2014) studied non-linear behavior of the wheel rail interaction. Effects of the profile of the track on derailment failure have been studied by Xiao (Brabie and Andersson, 2008) through a wheel track dynamic model. Fang and Zhang (2013) developed a model to investigate the feasibility of using fiber concrete in transoms. In this study, a detailed parametric study was carried out by changing selected material properties of the fiber concrete to simulate different test models. However, all the above literatures did not considered the newly retrofitting pre-cast concrete slab into existing steel girders. The paper herein will look at the behavior and failure modes when derailment load is subjected to the retrofitted system.

METHODOLOGY



Composite steel-concrete transoms have been designed based on structural guidelines given in the Australian Standard AS3600 (Standards Australia, 2009). Design loads considering the impact of derailments were obtained from the Australian Standard AS5100.2 (Standards Australia, 2004). The precast concrete transoms were numerically modeled to verify the proposed design. A finite element model (FEM) was created using a finite element package, ABAQUS. The validation was previously carried out by very good agreement between numerical results and field measurements (Griffin et al., 2014, 2015). Furthermore, the interactions and internal forces in the steel-precast concrete composite transoms under impact conditions were considered. The risk assessment portrays two possible scenarios with medium and high risk on SHB (see **Figure 1**). Such the two train derailment situations involve the derailment with train speeds of 5 and 50 km/h, which will be considered in the analyses herein. These speed ranges are very common for metro and suburban rail networks and they could be the representative derailment speeds for the networks. The low derailment speed (5 km/h) correlates to the risk of wheel climbing derailment while the high derailment speed relates to potential damage of wheel sets when traveling around the median train speed (50 km/h). The robustness and resilience of the track slabs are critical to railway systems, especially for metro-suburban passenger traffics. It is thus essential to identify them in order to help railway engineers manage the unlikely crisis better.

Structural System of the Sydney Harbor Bridge

Rails are mounted on transoms and transoms are supported on steel stringers in the SHB. There exists neither ballast nor aggregate materials used under the transoms, in order to transfer and suppress dynamic loadings. Ballast and granular materials play a vital role in uniformly transferring the impact loads

[1] https://www.thetimes.co.uk/article/23-killed-in-kalinga-utkal-express-train-derailment-en-route-to-haridwar-hn5cl3dtm.

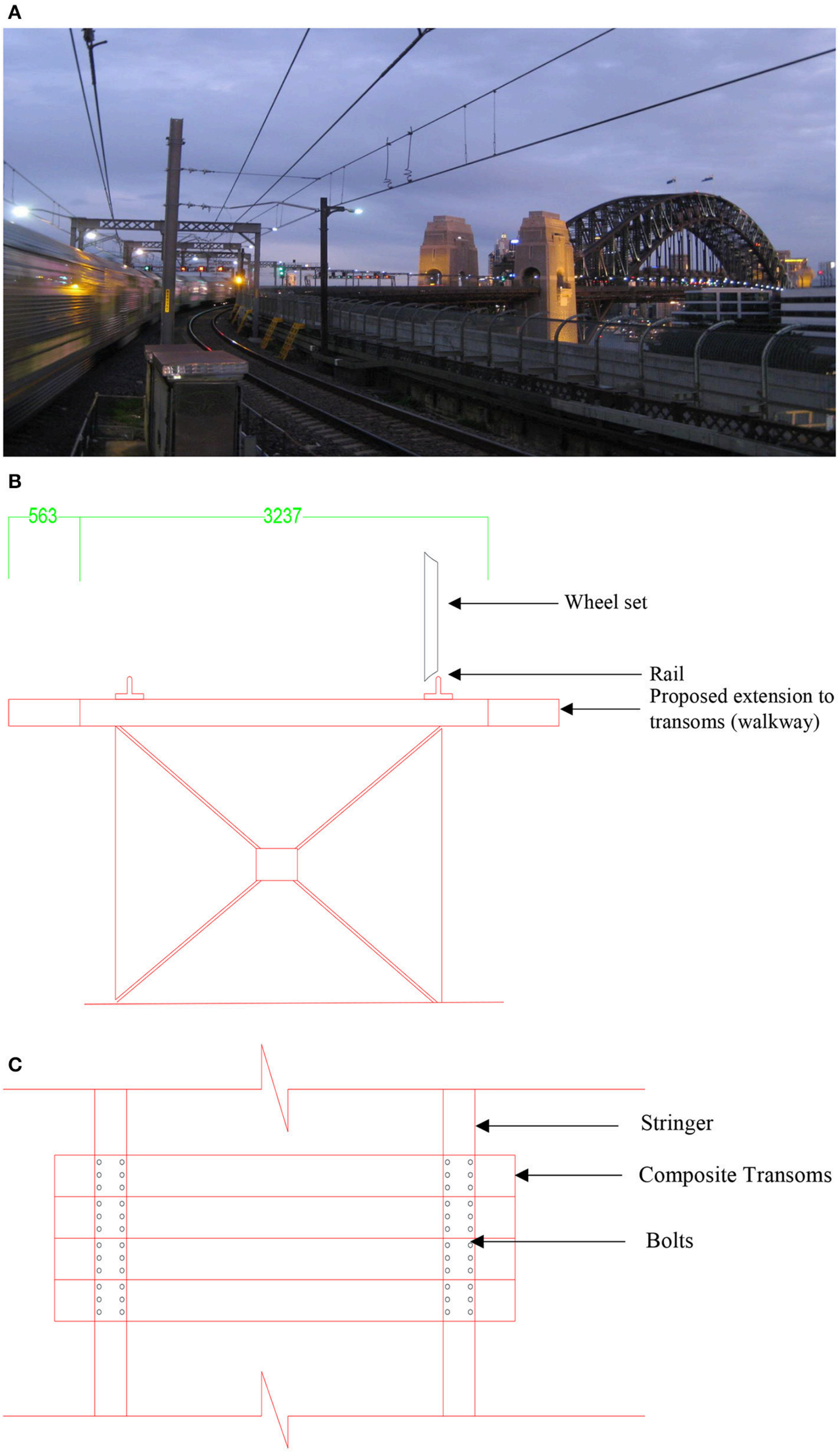


FIGURE 1 | Sydney Harbor Bridge. **(A)** General view of Sydney Harbor Bridge. **(B)** Cross section for the structural system of Sydney Harbor Bridge. **(C)** Top view of the structural system.

from transoms to the ground. It acts as a tensionless spring support. Ballast and granular materials have good damping properties against vibrations. Taking these facts in to account, a simplified structural system was developed for the analysis under dynamic action of loads on transoms in this research as is shown in **Figure 1**. Transoms are simply supported on stringers and the walk-way has been designed as a cantilever. The proposed composite transoms are connected to the existing steel stringers using headed shear stud connectors. Existing rails then be fastened to the composite transoms using rail pads installed on steel plates.

Design Load Actions on Transoms

Actions of the design loads on transoms shall either be considered in their serviceability and ultimate limit states. LC1, LC2, LC5, and LC10, which will be described later, are considered as the most unfavorable load combinations or the set of worst case scenarios for design according to precedent research (Griffin et al., 2014, 2015). The maximum axial load acting on a transom during a train derailment was taken as 20 tons as per the guide lines established by the transport for New South Wales (Standards Australia, 2004; Remennikov and Kaewunruen, 2008; Kaewunruen and Remennikov, 2015).

Dead Load

The maximum thickness of composite panel will not be increased more than 180 mm due to the limitations of allowable spacing for transoms in the existing structure (limited clearance due to existing overhead wiring structure). Density of concrete is 2,400 kg/m^3. Self-weight of concrete transoms is 4.23 kPa. Dead load of services hanged on track was not considered.

Dynamic Load Allowance (DLA)

Dynamic Load Allowance (DLA) is in proportion to the traffic rail way live load (Standards Australia, 2004). This factor increases the live load to be equivalent to the impact of dynamic behavior of the locomotive which induces the live load.

Live Load

Transport of New South Wales recommends a model for the most unfavorable axial loading arrangement live load imposed by a train (Standards Australia, 2004). Model 300LA (a 30t-axle train loading model) shall incorporate a factor for with dynamic load allowance (DLA) to represent the dynamic behavior of the train live load. 300LA Loading arrangement model shown in **Figure 2**. The load case (300LA) is recommended by Australian Standard AS5100 (Standards Australia, 2004).

Design and numerical results obtain by Griffin et al. (2015) were relevant to 300LA, and have revealed the most unfavorable loading arrangement long the rail (worst case scenario).

Loads Due to Thermal Effects

Thermal stresses are also critical in bridges. Thermal movements between various components in deferent material become significant due to the deferent thermal properties of those materials. Horizontal load (F) for L (503 m) > 50 m = 100 + 15(L − 50) = 6,895 kN was considered as braking force; and, as the panels are 0.6 m wide and the breaking force is distributed to two tracks, $Q_{(breaking)}$ of 4.11 kN/m (0.5 × 0.6 × 6,895 kN/503 m) is considered to be transferred to the transom. According to the AS5100.2 (2004), 300LA traffic load produces 100 kN nose load to directions to the structure. Nose load should not be increased in Dynamic Load Allowance (DLA). $Q_{(nose)}$ was considered to be 50 kN/m per track (Mirza et al., 2016a).

Derailment Load

Derailment load was considered in the ultimate load state and would not be increased by DLA. The derailment load is considered as a quasi-static load applying over an area prescribed by Australian Standard AS5100 (Standards Australia, 2004) as shown in **Figure 3**. The calculation of forces, pressure area and the locations of derailment action can be obtained from the standard to enable the worst-case scenario analyses. Based on the standard, $Q_{(derail)}$ is 133.33kN (Standards Australia, 2004).

Wind Load

According to the AS5100.2 (2004), design wind speed for the ultimate state is 48 m/s and the design wind speed for the serviceability limit state would be 37 m/s (Standards Australia, 2001). Only the vertical wind load was considered for SHB due to the inclination is less than 5 degrees (Standards Australia, 2004). Ultimate wind pressure was calculated as 1.0368 kPa and the service wind pressure was found to be 0.616 kPa.

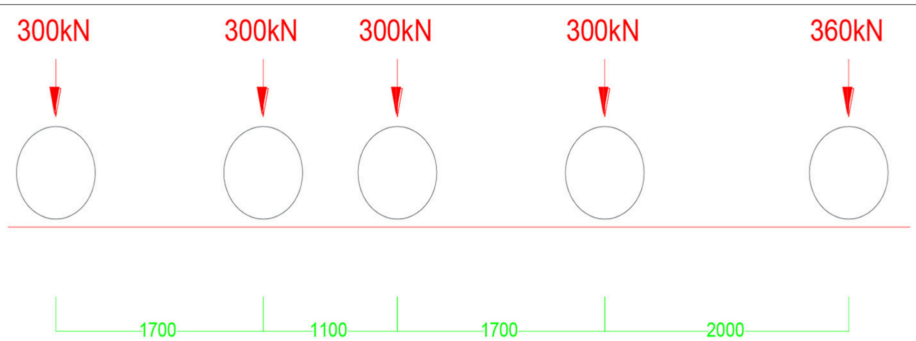


FIGURE 2 | 300LA Axial Loading Configuration (after Standards Australia, 2004).

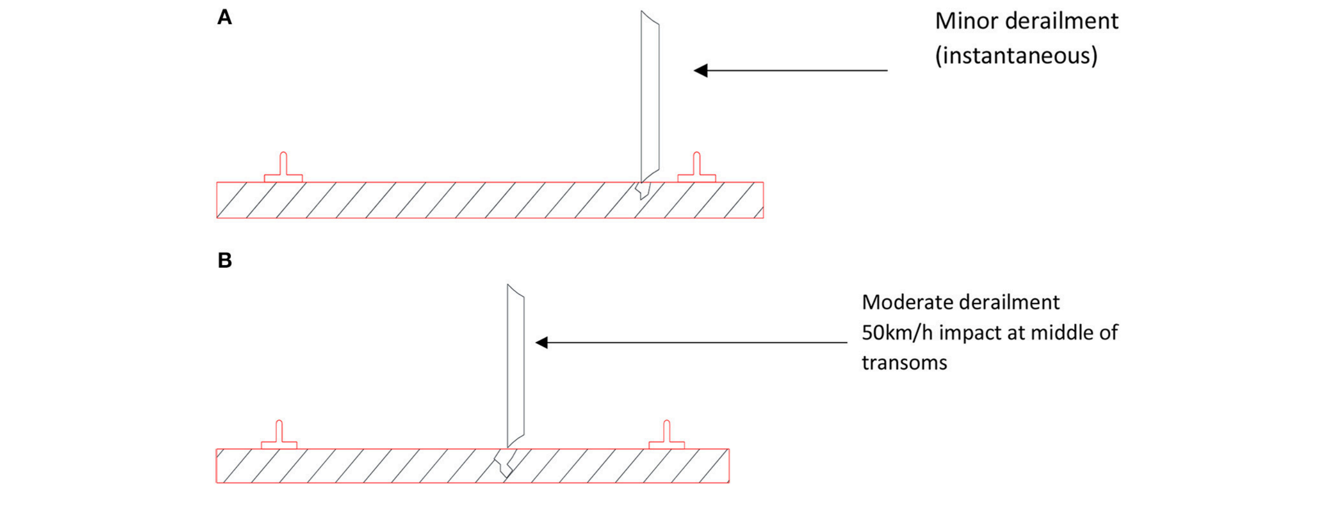


FIGURE 3 | Derailment loading. **(A)** Minor derailment (high risk). **(B)** Moderate derailment (moderate risk).

Load Combinations

Ultimate Limit States

LC1 = 1.4 Gpanel + 3.2 Q200LA + 1.6 Qbreaking (Panel UDL + Pad UDL)
LC2 = 1.4 Gpanel + 2.67 Q200LA + 1.6 Qbreaking (Panel UDL + Pad UDL) LC3 = 1.4 Gpanel + 1.5 Qgeneral (Panel UDL)
LC4 = 1.4 Gpanel + 1.6 Qnose (Panel UDL and Longitudinal point load)
LC5 = 1.4 Gpanel + 1.2 Qderail (Panel UDL and point load)
LC6 = 1.4 Gpanel + 1 W*vu (Panel UDL + wind)
LC7 = Gpanel + Q200LA + Qbreaking + W*vu (Panel UDL and Pad UDL.

Serviceability Limit State

LC10 = 1.2 Gpanel + 2 Q200LA + 0.7 W*vs.

Resultant live load arrangement on transom is given in **Figure 4A**. Live loads would act as uniformly distributed loads.

The load arrangement due to the impact of derailment is given in **Figure 4B**. Derailment load would be applied as point loads on locations shown in the figure.

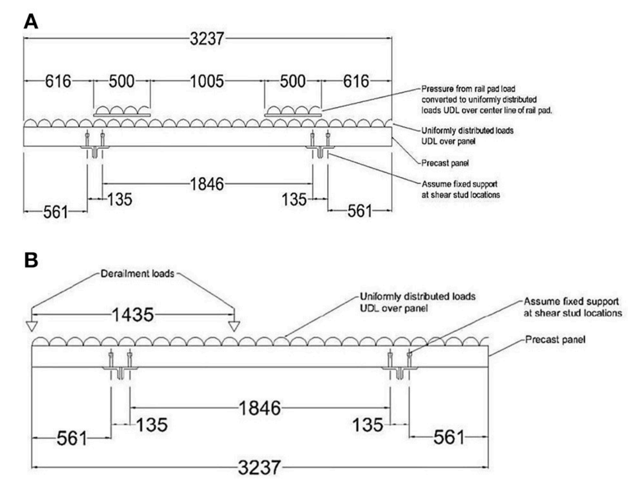


FIGURE 4 | Load Arrangement on transoms. **(A)** Live Load in accordance with AS5100. **(B)** Derailment Loading in accordance with AS5100.

Structural Properties of Elements

Rails, Rail pads, and stingers are the existing structural elements in the model and Composite transoms are only the new element (Kaewunruen and Remennikov, 2009, 2010). Following structural properties for steel (in **Table 1**) and concrete (in **Table 2**) are considered.

Structural Analysis and Design of Transoms

Transoms are analyzed for the load cases developed earlier. Linear elastic analysis have been investigated using Microstran to assess load actions and corresponding critical stress locations. Bending moment and shear force diagrams, shown in **Figures 5A,B** respectively, were obtained using Microstran for the ultimate load cases (LC1/2 and LC5, respectively). The structural design of the composite transoms was then carried out in accordance with the AS5100.2 and AS3600 for the composite sections (Standards Australia, 2001, 2004, 2009). **Table 3** lists the maximum design values of bending moments and shear forces obtained from the analysis. The design calculation for the proposed composite steel-concrete transoms is summarized in **Table 4**.

Summary of the Structural Design Calculation

Table 5 illustrates the major finding in structural design of composite transoms according to the guideline of AS 3600 (Standards Australia, 2009).

Design Details

The details of the arrangement of the components of the proposed composite steel-concrete transoms are given in **Figure 6**.

TABLE 1 | The properties of steel elements used in design.

Material	Standard	Compressive strength (MPa)	Yield strength (MPa)	Young's modules
COMPOSITE TRANSOMS				
Tensile reinforcements	AS/NZS4671 Standards Australia, 2001 D500N	500	500	210,000
Profile steel sheeting	Bondek II BHP	550	550	210,000
Shear connectors			350	210,000
BRIDGE COMPONENT				
Stringer	AS/NZS4671 Standards Australia, 2001 D500N	300	300	210,000
RAIL				
Rail (standard carbon rail)	AS/NZS4671 Standards Australia, 2001 D500N	250	250	210,000

TABLE 2 | The properties of concrete elements used in design.

Material	Standard	Compressive strength	Tensile strength	Young's modules
COMPOSITE TRANSOMS				
Concrete	AS3600 (Standards Australia, 2009)	50 MPa	5 MPa (ignored)	30,000

Bondek Sheets are commonly available in sizes of 0.6, 0.75, and 1.00 mm.

FINITE ELEMENT MODELING

Numerical model was developed using ABAQUS Explicit to simulate the impact of derailment loadings on the proposed steel-concrete composite panel. ABAQUS Explicit can be used to simulate non-linear behavior of the composite transoms in concrete crushing scenarios (Griffin et al., 2015).

Material Properties

In order to benchmark failure modes with structural design code, the material properties used for the design and analysis using the FEM of the composite transoms were obtained from previous studies and assessments (Griffin et al., 2015). The mechanical behaviour of the steel materials is summarized in **Table 1**. Concrete compressive strengths and elastic moduli of concrete results are summarized in **Table 2**. The mechanical behavior of the steel materials is summarized in **Table 1**. The material properties of the components of the FEMs have been represented by constitutive laws and actual material property test data (Remennikov and Kaewunruen, 2006; Zerbst and Beretta, 2011; Klinger et al., 2014; Wu et al., 2014).

Concrete Properties

A concrete damage plasticity model was utilized to describe the actual material behavior for concrete. The material model was found to be suitable for design and modeling (Griffin et al., 2015; Mirza et al., 2016b; Kaewunruen and Kimani, 2017; Kimani and Kaewunruen, 2017). The evolution of the yield (or failure) surface is controlled by tensile and compressive equivalent plastic strains linked to failure mechanisms under tensile and compressive loading. The Concrete Damage Plasticity option is used in conjunction with Concrete Tension Stiffening and Concrete Compression Hardening options in ABAQUS. The flow potential and yield surface parameters have been defined using default values of ABAQUS in the concrete damage plasticity option. The material model for normal weight concrete is used to define the elastic-plastic or curvilinear behavior of concrete for the compressive region. The model is expressed by the following equations, and compression is assumed to be linear elastic up to $0.4f'_c$.

$$\sigma_c = \frac{f'_c \gamma \left(\varepsilon_c/\varepsilon'_c\right)}{\left[\gamma - 1 + \left(\varepsilon_c/\varepsilon'_c\right)^{\gamma}\right]} \tag{1}$$

$$\gamma = \left[\frac{f'_c}{32.4}\right] + 1.55 \quad \text{and} \quad \varepsilon'_c = 0.002 \tag{2}$$

where f'_c is the characteristic uniaxial compressive strength of concrete, σ_c is the uniaxial compressive stress, and ε_c is the uniaxial strain of the concrete.

In this study, the stress-strain relationship of concrete in tension was assumed to be linear. The tensile stress of concrete increases linearly until concrete begins cracking in tension and decreases linearly to zero from that point. The ratio of the uniaxial tensile stress to the uniaxial compressive stress at failure is evaluated as 0.1. **Figure 7A** illustrates the stress-strain relationship of concrete according to the compressive behavior and the uniaxial tensile stress-strain behavior of concrete. Strain-rate effect of high strength concrete has been adopted as follows (Kaewunruen and Remennikov, 2011):

$$\frac{f'_{c,dyn}}{f'_{c,st}} = 1.49 + 0.268\log_{10}\dot{\varepsilon} + 0.035[\log_{10}\dot{\varepsilon}]^2 \tag{3}$$

where $f'_{c,dyn}$ is the dynamic compressive strength, $f'_{c,st}$ is the static compressive strength of concrete, ε is the dynamic strain, $\varepsilon_{c0,st}$ is the static ultimate strain, and $\dot{\varepsilon}$ is the strain rate in concrete fiber.

Steel Properties

Figure 7B illustrates the stress-strain relationships of the steel materials used to model the profile sheeting, rail, train wheels, reinforcing steel, and welded shear studs respectively. Steel materials generally exhibit elastic behavior up to their yield points and this is followed by further yielding or strain hardening before fracture. The stress-strain relationships of the materials were converted into piecewise linear curves. Strain-rate effect of high

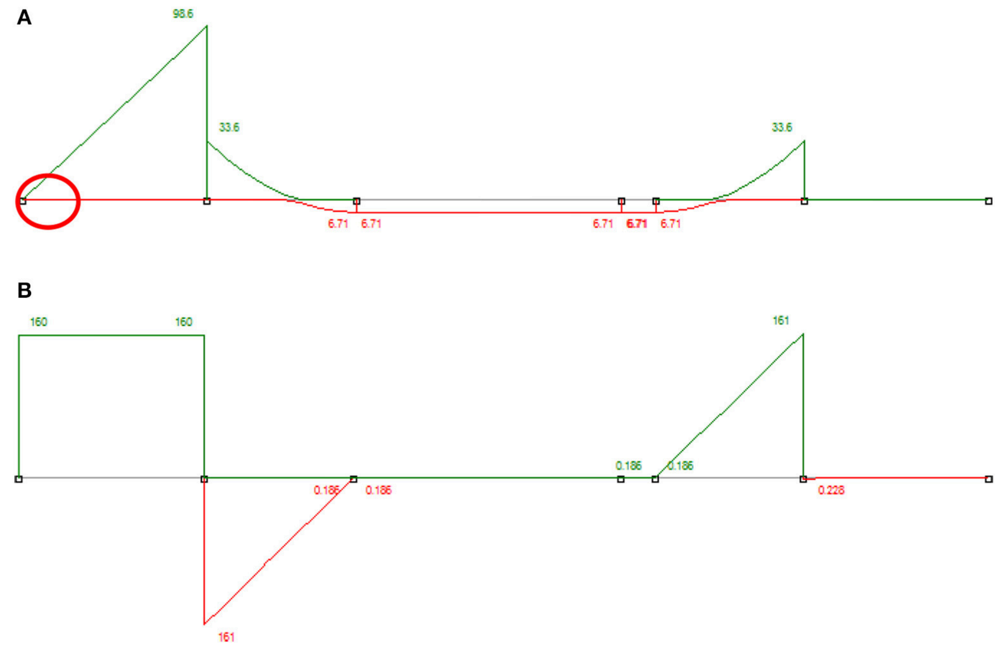


FIGURE 5 | Load effect on the transom. **(A)** bending moment envelop. **(B)** Shear force envelop.

TABLE 3 | Design bending moment envelops.

Design action	Max value	Distance from support	Load combination
(+)M*	6.71	Mid span	LC1
(−)M*	−33.6	0	LC1
(+)M*	135	0	LC2
(−)M*	−135	0	LC2

*The maxium design values obtain by structral analysis using microstrain.

TABLE 4 | Summary of design bending moments.

Design action	Max value	Distance from support	Load combination
(+)M*	33.5	Mid span	LC5
(-)M*	−98.6	0	LC5
(+)M*	160	0	LC5
(-)M*	−161	0	LC5

strength steel has been adopted as follows (Kaewunruen and Remennikov, 2011):

$$\frac{f_{y,dyn}}{f_{y,st}} = 10^{0.38\log_{10}\dot{\varepsilon}^{-0.258}} + 0.993 \tag{4}$$

where $f_{y,dyn}$ is the dynamic upper yield point stress, $f'_{c,st}$ is the static upper yield point stress, and $\dot{\varepsilon}$ is the strain rate in high strength steel. In this study, high strain rate properties of materials have not been fully considered. This is because this study focuses on a scenario when the damaged train on a straight viaduct often reduces its speed significantly before it derails (Australian Transport Safety Bureau, 2014). The strain rate in these cases tends to be relatively low. However, more work on transient impact and high strain rates will be investigated in the future (Kaewunruen and Remennikov, 2008; Choi et al., 2010; Crawford, 2011).

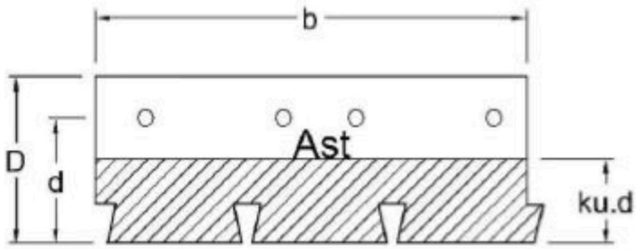
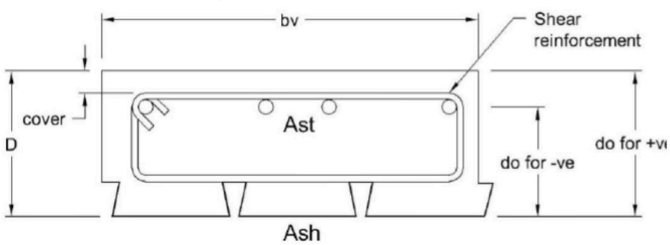
Geometry, Element Type, and Mesh

For design purpose, the maximum influences on the composite transoms of single wheel derailment have been considered in the derailment model as is illustrated in **Figure 5**. Due to the loading, which is applied on the stringer not being of importance, the project being conducted will only need the use of half model to produce accurate results. The concrete slab and steel components were modeled using the eight-node linear hexahedral solid elements with reduced integration and hourglass control (C3D8R). Elements with reduced-integration have been adopted as they could reduce computing run time. These elements were incorporated in a reasonably fine mesh in order to improve the accuracy of these models. The mesh sizes were also verified by carrying out a sensitivity analysis. The shear connectors were modeled using second order three-dimensional 20-node quadratic brick elements with reduced integration (C3D20R). The connectors were modeled to represent the actual geometric size and shape within the limitations of the application. The reinforcing bars were modeled with two-node linear three-dimensional truss elements (T3D2).

Contact Properties, Boundary Conditions, and Load Application

The interactions between different parts of the FEM were modeled using general interaction and constraint options

TABLE 5 | Design criteria of composite transoms.

	Calculation	References
Geometrical properties	 D = 180 mm d = 119 mm b = 600 mm	
Assumptions	Ast = 3217 mm^2 (4 N32)	
Material properties	fc = 50 MPa fsy = 500 MPa	
Bending moment capacity	ΦMu = 112.55 kNm	AS 3600 Standards Australia, 2009
Minimum RF	Ast min = 366 mm^2 < Ast prov  N 10	
Shear force capacity	Vuc = 331.9 kN V * < 0.5 Vcu	AS 3600 Standards Australia, 2009

available in ABAQUS. **Table 6** details the contact behavior between different parts of the FEM. The symmetrical model considered in this project is the derailment panel. The derailment model was considered as symmetrical and all boundary conditions were applied on the section which has been cut. The derailment model as illustrated in **Figure 8** is symmetrical about the x-axis. In addition, the Surface 1 of the model is also defined as a symmetrical surface and the nodes of the concrete and Bondek II that lies on that surface was restricted from translating in the z-direction. The sliced section of the stringer is labeled as Surface 2 and the nodes of this area have been restrained from rotation and translation in all three axes, which is defined as *Encastre boundary conditions* in ABAQUS. The load combination LC5 produced the worst case design actions due to the applied derailment loads in microstran. Therefore, the loads resulting from LC5 have been adopted for the simulations of derailment models. The expected impact loading scenarios were created using dynamic explicit step functions. Loads applied onto the panel are representative of the wheel of the train derailing and causing impact on the panel itself.

RESULTS AND DISCUSSION

Developed design methodology was tested by a simulation of train wheel derailments using a FEM developed in ABAQUS. Results obtained from the FEM, were compared with the proposed design results. The impact of design parameters on the design strengths of the steel-concrete composite transoms was studied by carrying out a detailed parametric analysis. The maximum stresses developed in the composite transom were studied.

Panel Design Results

The composite panels are 180 mm in depth maintaining the same dimensions of the current hardwood timber panels which are not designed to take derailment loadings. The transoms require 4 N32 reinforcement bars throughout the length of the transom and must have a minimum of 54 mm of cover to avoid corrosion. There is no shear reinforcement needed in the panels due to achieving a low enough design shear force (V*). N10 minimum shear reinforcement will be used to fix the tensile reinforcements in place before pouring concrete. **Table 7** displays the design actions and design capacities which were obtained

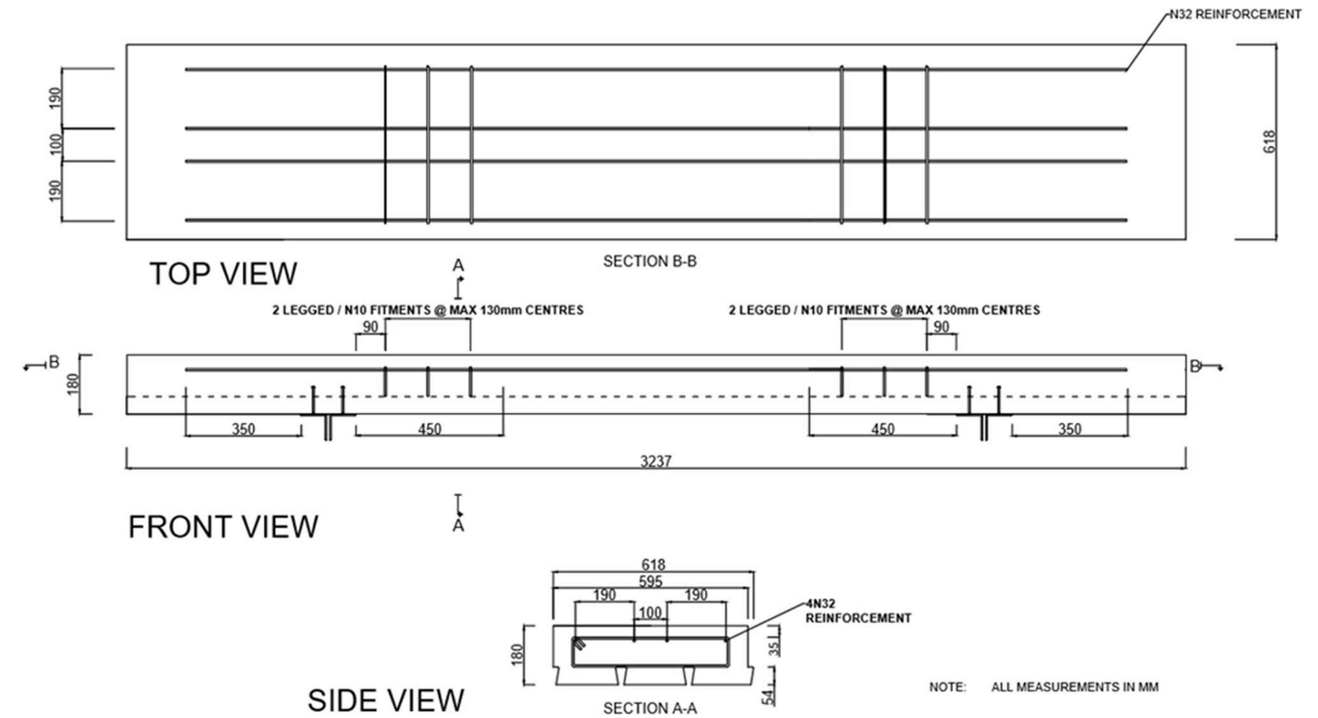


FIGURE 6 | Details of the proposed composite transoms.

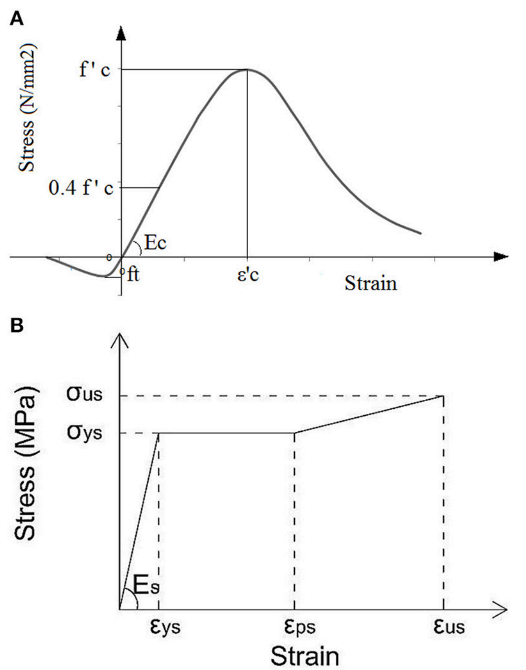


FIGURE 7 | Material properties. **(A)** Stress-strain curve for normal concrete. **(B)** Stress-strain curve for structural steel.

from the transom. The design ratio is a quick indication that the design capacity is greater than the design action and must be >1.

Numerical Analysis Results

Stresses Developed in the Reinforcing Steel and Concrete Panel

The applied dynamic impact was represented by a function of time. The force applied to the transom by the derailment was modeled as an incremental force with the time. **Figures 9A,B** illustrate the development of the stresses in the reinforcing steel and concrete respectively. The reinforcing steel commenced yielding before concrete started cracking due to compression failure.

Stresses Developed in the Profiled Sheet (Bondek II)

The graph distribution in **Figure 10A** displays the resulting stress levels carried by the Bondek II at impact caused by derailment. The derailment relationship of the Bondek II is shown as stress vs. time. From initial impact at time 0, the graph displays a linear curve up until about time 0.004, this is due to the combination of concrete and steel taking load. The maximum stress is reached at time 0.005 with a value of 550 MPa and fluctuates up until time 0.015. Beyond time 0.02 the stress level is reached and is maintained constant. **Figure 10B** displays the deformed model of the Bondek II caused at the impact once the wheel interacts with the panel. The diagram clearly depicts deformations in the region which the train wheel impacts the panel, this region is highlighted in the mesh with red color located in areas labeled (Y). This maximum stress distribution travels from the point of impact at deformation of the Bondek II and travels toward the shear stud locations. The maximum stress exerted on the Bondek II reaches 550 MPa. This value is the maximum yield stress of the

TABLE 6 | Interaction between components of FEM.

Type	Interface	Interface type	Master surface	Slave surface	Friction coefficient
A	Reinforced steel in concrete	Embedded	Reinforced Steel	Concrete	N/A
B	Concrete to Bondek II	Tie Constraint	Bondek II	Concrete	N/A
C	Shear stud bolt in concrete		Shear Stud Bolt	Concrete	N/A
D	Shear Stud weld to Bondek II	Tie Constraint	Bondek II	Stringer	N/A
E	Bondek II weld to stringer		Bondek II	Stringer	N/A
F	Bondek II on stringer	Surface to Surface	Bondek II	Stringer	0.3
G	Wheel to concrete	General contact explicit	Hard contact overclosure		0.5

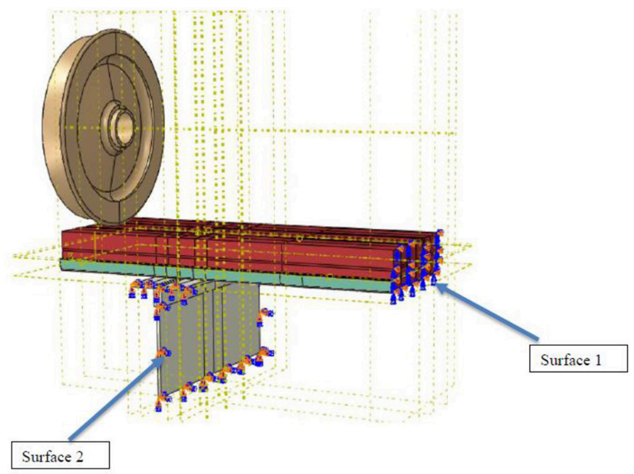


FIGURE 8 | Boundary conditions in the finite element model.

TABLE 7 | Derailment design capacities.

Design action		Design capacity		Design ratio
M*	98.6 kNm	ΦMu	112.55 kNm	1.14
V*	160 kN	ΦVu	165.5 kN	1.03

Bondek II. Therefore the yield limit for the Bondek II is reached, and notably does not surpass this point.

Parametric Studies

A detailed parametric study was carried out using the FEM to assess the effect of concrete compressive strength, tensile reinforcement strength, and Bondek II strength on the failure behavior of the composite transoms under impact loading due to train derailments. Prototype model developed by ABAQUS will be tested using a displacement control method. The maximum allowable stresses developed in the structural element was determine by controlling the displacement in allowable limit. This was repeated for deferent strength parameters of concrete, reinforcement steel and Bondek sheets.

The Effect of Concrete Compressive Strength

The concrete strengths including 32, 40, 50, 65, 80, and 100 MPa were modeled. **Figure 11** illustrates the relationship between the

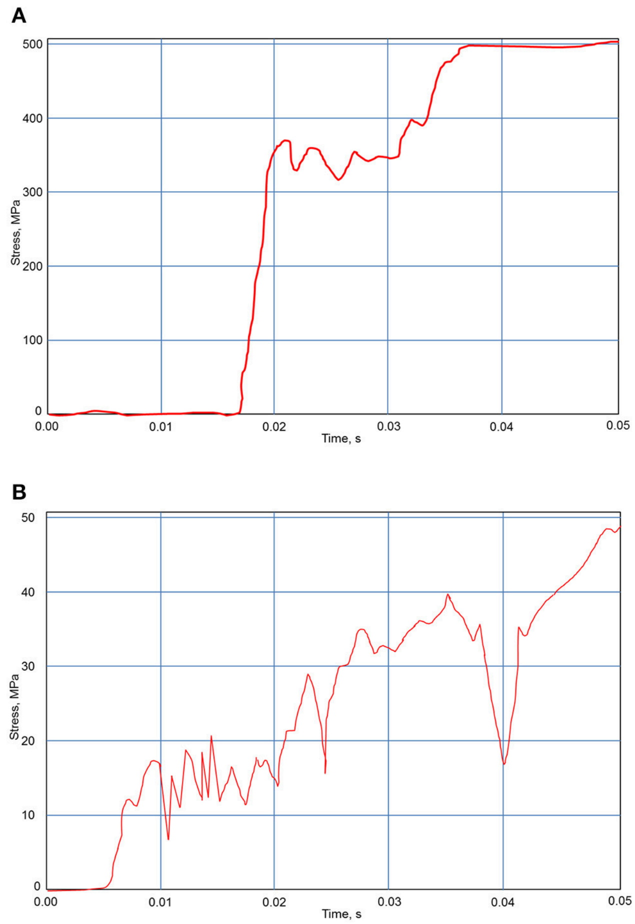


FIGURE 9 | Stresses developed in structural elements. **(A)** Stresses developed in reinforcing steel (worst-case location). **(B)** Stresses developed in concrete (worst-case location).

maximum stress developed in concrete of the transoms and the characteristic compressive strength of the concrete used. The results suggest that the maximum stress in concrete of the transoms increases with increasing concrete compressive strength. However, an unusual pattern was noticed that at 50 MPa concrete strength the maximum stress of the transoms was quite higher than that with the 65 MPa concrete. Furthermore, the transoms with 65 MPa concrete displayed a higher maximum

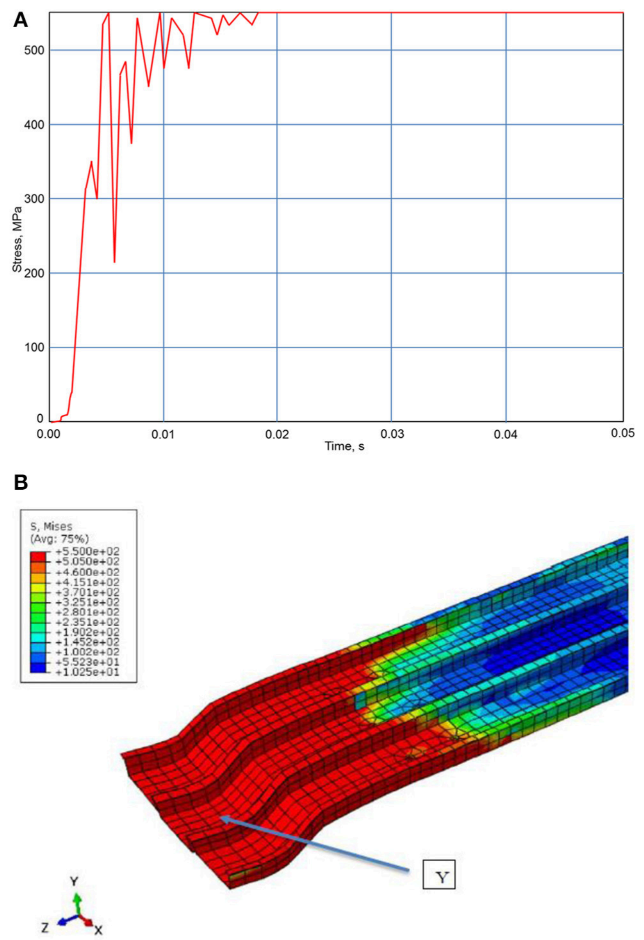


FIGURE 10 | Load effects in the profile element. **(A)** Stresses developed in the profile sheet. **(B)** Deformed model of the profile sheet.

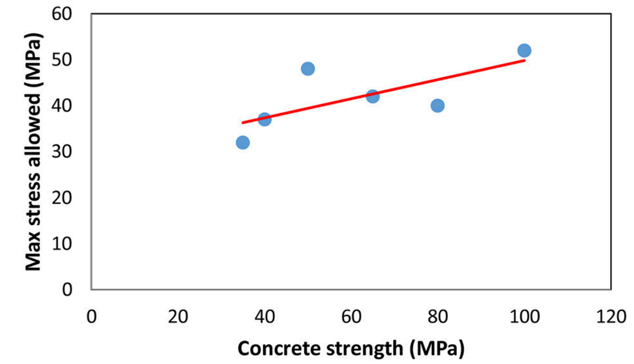


FIGURE 11 | Variation of stresses in concrete vs. characteristic strength of concrete.

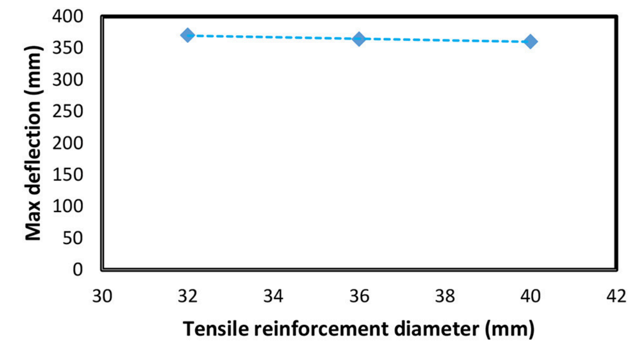


FIGURE 12 | Maximum deflection vs. Diameter of reinforcements.

stress than that with 80 MPa concrete and that difference was approximately 2 MPa. Theoretically however, since the transom with 50 MPa has a maximum stress of 48–49 MPa, selecting this option in practice would be realistically better in real life application, however needs experimental testing to validate further.

The Effect of the Reinforcing Steel

A parametric study for area of tensile reinforcement was explored, whether a higher value of tensile reinforcement will increase the strength and most importantly the serviceability limit. The diameters of steel tensile reinforcement include 33, 36, and 40 mm diameters. The results as shown in **Figure 12** indicated that changing of tensile steel reinforcement size does not have a significant effect on the deflection of the panel.

The Effect of the Yield Strength of the Profile Sheet

Different yield strengths of the profile sheet were considered in the parametric analysis in order to study the maximum stresses developed in the panel. The material properties change from 550 to 800 MPa. The relationship of maximum allowable stresses vs. Bondek II properties was proposed and is shown **Figure 13**. The study demonstrated that the maximum allowable stress increases linearly with increasing Bondek II yield strength. However, there was a maximum allowable stress value for the profile sheet could take, that was 700 MPa. This was shown in the FE results when 800 MPa Bondek II strength was used, the allowable stress did not increase. Note that the stress over the profile sheet can be reduced by increasing depth of the concrete or the steel sheet itself. With higher strength of concrete, the stress can also be further alleviated.

CONCLUSION

Composite panels can be effectively used as a replacement of timber transoms in SHB. There are various approaches in designing the composite transoms, which can be adopted in finding design solution. In this study, primarily provisions given in the Australian standards for structural concrete design was adopted in order to design the proposed composite steel-concrete transoms. The same depth of 180 mm in the existing system was resembled for composite slabs to avoid any changes to the level of existing structure. Moderate derailment condition while the train is traveling in 50 km/h speed was considered in the design based on the systems risk assessment. The design should be verified by testing a prototype. Numerical model was developed for the prototype testing.

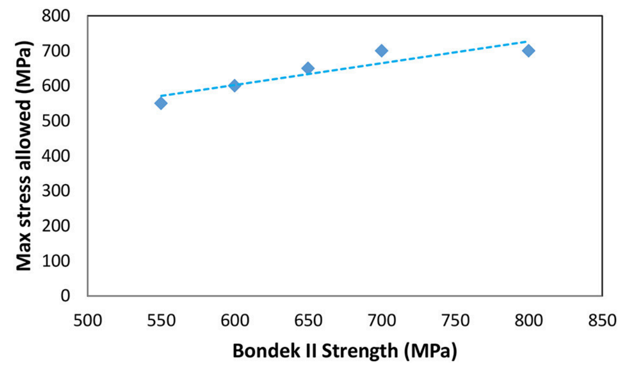


FIGURE 13 | Maximum stress in Bondek sheets vs. Tensile strength of Bondek.

The FEM results suggested, that the reinforcing steel commenced yield before the compressive stress in concrete arrived its characteristic strength due to the impact loading. Therefore, this result indicates that the designed composite transoms are ductile. The Bondek sheets commenced yielding even before the yielding of reinforcement commenced. Stain compatibility was not observed between Bondek and concrete. It was sign of slap between the Bondek sheet and the concrete panel. A detailed parametric study was carried out using the FEM, to understand the effect of concrete compressive strength, tensile reinforcement area and Bondek II strength on the behavior of the composite transoms under impact loading due to train derailments. The deflection of the composite transom under the design load would govern the strength of transom. This was the deflection control method of determination of allowable stress in the structural component. Allowable stresses in components of the composite transom could also be determined experimentally in the simulation by controlling the deflection. Stresses developed in various components were observed for deferent characteristic strength of the materials. The maximum allowable stress in concrete increased with increasing characteristic compressive strength of concrete. Elastic moduli of concrete increase with the increment of the characteristic strength of concrete.

There was no relationship between area of reinforcements and the deflection of the composite ransom. These critical design parameters, which govern the deflection, would rather be some other parameter than the reinforcement area. Theoretically, area of reinforcement has nominal effect on stiffness of panel. In order to decrease the deflection of the composite transom panels, that future studies explore the possibility of incorporating pre-stressed steel in the panel could be suggested. Defection control test simulation was done to find out the relationship between strength of Bondek sheet (yield strength) and the allowable stress. The allowable stress increased linearly until the yield strength of Bondek sheet increased to 700 MPa.

Further Research

This research has been limited to equivalent static load analysis to the dynamic impact of the loads over the bridge. This method could be justified by AS5100.2 (Standards Australia, 2004). In a Further research the entire bridge can be modeled and modal analysis can be performed to understand the response of the structure to the dynamic loads imposed by train live loads and derailments (Kaewunruen and Kimani, 2018; Mirza and Kaewunruen, 2018). Response Spectrum Density Function could be developed to model the train live loads based on actual data.

AUTHOR CONTRIBUTIONS

OM and SK developed the design concept. OM performed the analyses. SK provided field data for model validation. OM and SK wrote and contributed their thoughts and experience to the paper.

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Far-Field Earthquake Responses of Overhead Line Equipment (OHLE) Structure Considering Soil-Structure Interaction

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Overhead line equipment (OHLE) is the components for the electric train which supply the electric power to the train. For one or two tracks, OHLE is normally supported by cantilever mast. The cantilever mast, which is made of H-section steel, is slender and has a poor dynamic behavior by nature. Nonetheless, the mast structures, which located alongside the railway track, have not been fully studied on the dynamic behavior. This paper presents the effects of far-field excitations on cantilever mast and overhead contact wire. The five far-field earthquake records at various magnitudes between 6.5 and 8 Mw are considered. A three-dimensional mast structure with varying support stiffness is made using finite element modeling. It is interesting that support stiffness plays a role in the dynamic responses of OHLE during far-field earthquakes due to the change of its properties. Surprisingly, the earthquakes can cause damage to the overhead contact wire which lead to the failure of electric system. In this case, the train cannot run until the broken wire and electric system is cleared. This occurs when there are the losses of support stiffness due to the failure of support connection or soil degradation. Moreover, beating phenomenon, which normally occurs in the tall building, is obviously observed in OHLE during the occurrence of earthquake. This is the world first to demonstrate the effects of far-field earthquakes on the cantilever mast structure and the response of OHLE. The insight in this earthquake response of OHLE and its support has raised the awareness of engineers for better design of cantilever mast structure and its support condition. The outcome of this study will provide a new earthquake detection method using OHLE.

Keywords: far-field earthquake, overhead line equipment, cantilever mast, overhead contact wire, soil-structure interaction, resonance phenomenon, beating phenomenon

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INTRODUCTION

At present, railway infrastructure experiences harsh environments and aggressive loading conditions from increased traffic and load demands. Due to the rapid growth in population, the passenger journeys have increased by nearly 100% and freight by 60% (Baxter, 2015). The provided extra capacity is needed for the economic growth in the future (RailCorp, 2011a). Consequently, the electric train has become the efficient railway systems. The electric train is allowed to run frequently and quickly. Also, an electric train is cheaper than diesel train in terms of both construction and maintenance. For electric train, the overhead line equipment (OHLE) is an important asset of the railway infrastructure and is one of the most vulnerable ones. OHLE is an equipment to supply power to make electric trains move, as shown in **Figure 1**. The support of OHLE is cantilever mast or portal frame depending on the number of track. The cantilever mast, which is normally made of H-section steel column, is a support of OHLE for only one or two tracks. Although the concept of OHLE is simple, the problem is poor dynamic behaviors of OHLE are needed to develop (Beagles et al., 2016). Due to the extreme environmental events and severe periodic force, such as an earthquake, in surrounding area may cause damage to the track and OHLE structure especially mast structure, this can lead to the failure of the electrical system (Shing and Wong, 2008; Robinson and Bryan, 2009; Taylor, 2013). Therefore, the dynamic behaviors of cantilever mast structure and its monitoring system are needed to take into account during train operation and the occurrence of extreme environmental events. Although the effects of ground borne vibration generated by passing train on cantilever mast have been studied (Kurzeil, 1979; Madshus et al., 1996; Jonsson, 2000; Kouroussis et al., 2013, 2014, 2015; Vogiatzis and Mouzakis, 2017), the effects of earthquake have not been introduced.

Based on a review of open literature, far-field earthquakes on seismic responses of SDOF system with considering soil-structure interaction have been studied (Davoodi and Sadjadi, 2015). In addition, the responses of building under far-field earthquake have been investigated as can be seen in many studies (Ngamkhanong and Pinkaew, 2015). It was noted that large structures or high rise buildings were more affected due to the long duration and narrow band nature of far-field excitation. Resonance phenomenon is the effect occurred when the frequency of ground motion matches the natural frequency of a structure. It will suffer the damage and large oscillations. Even though the cantilever mast structure is the small structure compare to the building, this structure may experience the resonance effect due to the adaptation of soil and support conditions beneath the structure which leads to the change of its properties. In practical work, the structures are designed with the assumption of having fixed support. In fact, there is a small displacement created by the supporting soil. Based on the revealed literature (NEHRP Consultants Joint Venture, 2012; Prum and Jiravacharadet, 2012), different soil support conditions were taken into account. It was noted that soil-structure interaction affected the overall response of the structure. As for mast structure, it was noticeable that the rotational stiffness affected the natural frequencies and mode shape of vibration in a lower mode but rarely affected the fundamental mode in a higher mode (Ngamkhanong et al., 2017). This was because the dynamic behavior was characterized by coincident eigenfrequencies, mode order change, while the eigenfunctions remain associated with the corresponding eigenvalues (Pierre, 1988; Benedettini et al., 2009; Sari et al., 2017). In addition, soil-structure interaction has a crucial influence on the seismic response of structures especially founded on soft soils (Davoodi and Sadjadi, 2015).

FIGURE 1 | Electric train and overhead contact wire on West Midland railway line, UK.

The present paper aims to present a new study into the effect of far-field earthquakes on mast structure and overhead line equipment (OHLE) taking into account its underlying soil properties. Finite element model is employed to calculate the structural responses. The five far-field ground motion records with the magnitude of 6.5–8 Mw and the distance greater than 150 km were extracted from PEER Strong Motion Database. The obtained simulation results reveal that the support condition, earthquake magnitude, and its characteristics influence on the dynamic responses of mast structure. The insight in the earthquake response of overhead line equipment and its support has raised the awareness of engineers for better design of cantilever mast structure. The outcome of this study is to propose the possibility of using OHLE for earthquake detection.

METHODOLOGY

Modeling

In this study, the 3-dimensional finite element modeling is considered using a general-purpose finite element package STRAND7 (G+D Computing, 2001). OHLE is normally supported from cantilever masts, typically made of H-section steel, with a fixed base. The catenary cable and the pull/push-off arms supporting the contact wire are attached to the ends of the cantilever. The modeling of mast structure is shown in **Figure 2**, where consist of the two force member only. The young modulus of steel is 2×10^5 MPa with the density of 7850 kg/m^3. Poisson's ratio is 0.25.

In this study, support condition is taken into account. **Figures 3A,B** show the support condition of cantilever mast and frame mast, respectively, which are the embedded steel to foundation connection. The translational stiffness in three directions is assumed to be fixed in order to restraint the translation displacement. Based on soil and support conditions, although translational stiffness, denoted by k_x and k_y in is not taken into account, the rotational stiffness, denoted by k_{zz} in **Figure 3C**, of support conditions is varied from 100 kNm/rad to infinite value (fully fixed support or rigid soil). The schematic load to structure and its base support is shown in **Figure 3C**.

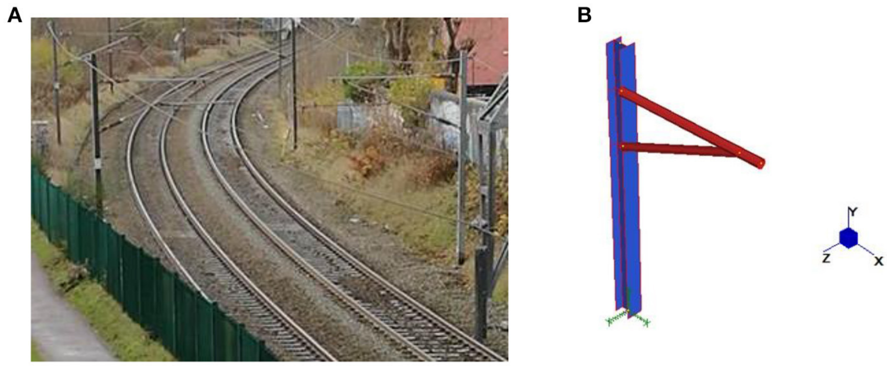

FIGURE 2 | (A) OHLE for double tracks and (B) 3-D modeling.

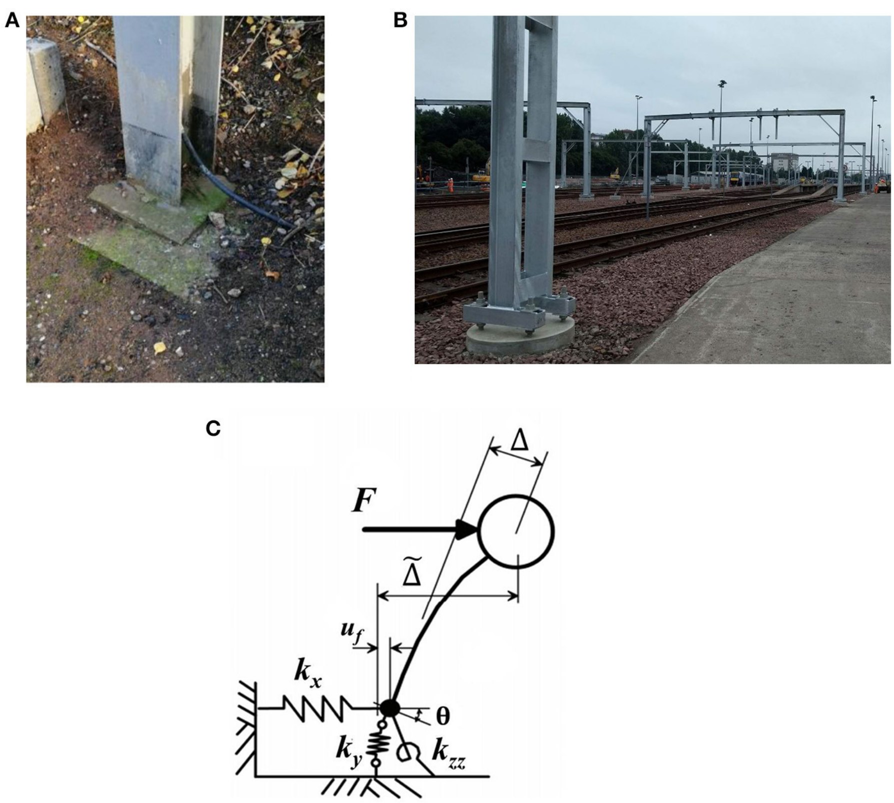

FIGURE 3 | Support of (A) cantilever mast (B) frame mast and (C) Schematic load to structure with rotational flexibility at support.

It should be noted that the change of soil-structure interaction leads to the decrease in the natural frequency as tabulated in **Table 1** and have more flexible compared to the corresponding structure supported by rigid soil or fully fixed support. The rotational support stiffness relates to the ability of connection to resist moment and curvature (in general, hinge support is 0 and fixed support is the infinity). It is noted that the support stiffness depends on the quality of connection and soil condition. The viscoelastic model of soil is adopted in this study.

Far-Field Earthquake Excitations

The 5 far-field earthquake records from PEER ground motion database having moment magnitudes (Mw) of 6.5–8.0 and the distance greater than 150 km are considered as shown in **Table 2**. The response spectra of earthquakes in the frequency domain are shown in **Figure 4**. Generally, the earthquakes recorded have two directions (N-S and E-W). However, the only stronger direction of the earthquake is used as an inputted earthquake. It should be noted that the frequencies of earthquakes in both directions are

TABLE 1 | Mode shape and natural frequency of cantilever mast at various soil stiffness in kNm/rad (Ngamkhanong et al., 2017).

Mode no.	Mode shape	Resonance (Hz)				
		K = 100	K = 1,000	K = 10,000	K = 100,000	Fully fixed
1	1st twisting	0.33	0.87	1.06	1.07	1.07
2	1st bending abt x-axis	0.94	1.37	3.13	5.08	5.60
3	1st bending abt z-axis	0.32	1.02	2.99	6.20	7.65
4	2nd bending abt z-axis	29.32	29.39	29.95	32.51	34.82
5	2nd bending abt x-axis	35.86	36.05	37.51	41.04	42.53
6	2nd twisting	48.46	48.46	48.46	48.46	48.46
7	3rd twisting	83.44	83.44	83.42	83.43	83.43
8	3rd bending abt z-axis	115.28	115.30	115.50	116.43	117.35
9	3rd twisting	144.52	144.52	144.52	144.52	144.52
10	3rd bending abt x-axis	185.69	185.77	186.35	187.89	188.59

TABLE 2 | Far-field earthquakes.

	Earthquake	Station	Year	M_W	5–95% Duration (sec)	Distance (km)	V_{s30} (m/s)
1	Denali Alaska	"Anchorage—K2-04"	2002	7.90	148	274	240
2	"Nenana Mountain Alaska"	"Anchorage International Airport"	2002	6.70	91	273	342
3	Chi-Chi Taiwan	"KAU046"	1999	7.62	43	163	204
4	Kobe Japan	"FUK"	1995	6.90	46	159	256
5	Landers	"Northridge—17645 Saticoy St"	1992	7.28	35	172	281

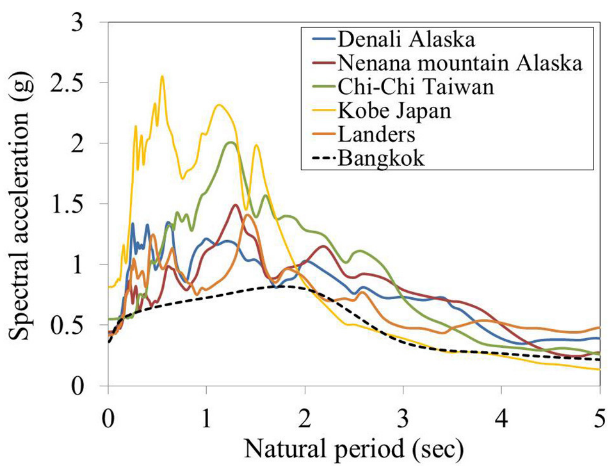

FIGURE 4 | Far-field earthquake response spectra.

nearly the same. As expected, the critical case that causes higher response is in the longitudinal direction to the track because the first two fundamental modes are twisting and bending about transverse (X-axis). However, firstly, both directions are considered because the sensitivity of soil stiffness, that causes the change in natural frequency and mode of vibration, is taken into account. It is noted that the earthquakes used are scaled up to match the predicted ground motions in Bangkok, Thailand.

RESULTS AND DISCUSSIONS

Non-linear Static Analysis (Push Over Analysis)

Firstly, push over analysis is used to define the direction of earthquake that can affect highest response of OHLE by applying static force to the structure. The displacements are measured at the end of cantilever which is the location of overhead wire to supply the electric power. In fact, the lowest absolute displacement occurs when the earthquake takes place in transverse direction because this axis has more stiffness to resist deformation than another direction. However, the overhead wire displacement in transverse direction affects overhead line system which has limitation of sway movement. Generally, the contact wire runs in a zig-zag path (also called "stagger"), as shown in **Figure 5**, above the track to avoid wearing a groove in the pantograph. The allowable displacement is assumed to be a construction tolerances of contact wire. Hence, 50 mm construction tolerance of contact wire is used as the maximum displacement at the end of cantilever mast in transverse direction (RailCorp, 2011b).

Even though earthquake occurred in transverse direction should create highest displacement in this direction, this axis has the highest stiffness. Due to the sensitivity of this structure, longitudinal force can also create displacement in transverse if there is an occurrence of coupling mode. **Figure 6** shows the maximum displacement occurred at the end of cantilever at various angle of force from 0 to 90 measured from longitudinal direction. However, the maximum transverse displacement occurs when the force is inputted in transverse direction as

FIGURE 5 | Zig-zag path (stagger) on overhead contact wire.

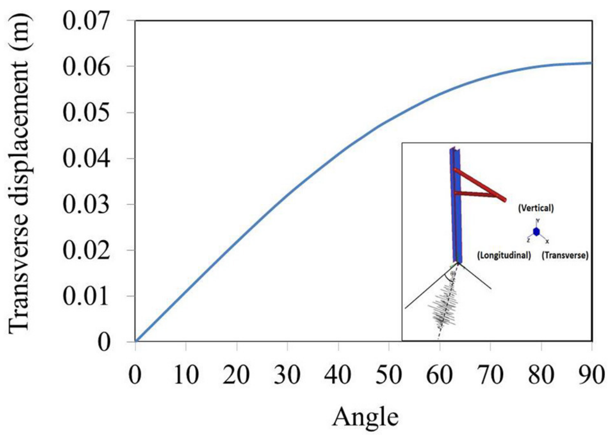

FIGURE 6 | Transverse displacement at various angles caused by static force applied to structure.

expected. Therefore, the earthquake are applied in transverse direction.

Time History Responses

The maximum displacement responses at the end of cantilever of the mast caused by various far-field earthquakes and its support stiffness are presented. This is derived from the dynamic response spectra from the nonlinear analysis over time domain. In theory, the dynamic response spectra correspond to the Earthquake excitation frequency and structural collapse can occur when internal resonance exists. In this study, the Earthquake spectra based on Chi-Chi and Kobe records are considered. This position is chosen because this is the location of overhead contact wire which supplies the electric power to the train. It should be noted that earthquake is applied in the transverse direction as previously describe. **Figures 7**, **8** show the displacement of OHLE in transverse directions during Chi-Chi and Kobe earthquakes at various support stiffness.

It is observed from **Figures 7A**, **8A** that structure located in the very soft soil condition or bad support (K = 100 kNm/rad) is affected by earthquakes because of the loss of stiffness and contact between soil and structure. It is shown that the failure of contact wire occurs at around 30 and 10 s after Chi-Chi and Kobe earthquakes, respectively (**Figures 7A**, **8A**). However, in case of support stiffness of 1,000 kNm/rad, it is observed that about 15 s after both earthquakes is presented as a failure time of system (**Figures 7B**, **8B**). For the structure with the support stiffness of 10,000 kNm/rad, it can be seen that Chi-Chi earthquake cannot affect the failure of contact wire. Meanwhile, the failure of contact wire on OHLE with its support stiffness of 10,000 kNm/rad is observed during Kobe earthquake, as shown in **Figure 8C**. However, the responses of OHLE can be reduced and vanish immediately by increasing the stiffness of soil. Therefore, the failure of contact wire can be detected within 50 s after earthquake occurs. The train can decelerate to stop before the failure of the electric system.

Maximum Displacement

Figure 9A shows the maximum displacement occurred at the overhead wire in transverse directions at various support stiffness. The 4-earthquakes have the same trends on structural responses which is the increase of displacement when the stiffness decreases. The amplitudes of OHLE responses are related to the magnitude and dominant frequencies of earthquakes. As for Kobe earthquake, the maximum displacements show the different trend as others. The maximum displacement at the soil stiffness of 1,000 kNm/rad is greater than that of 100 kNm/rad. Due to the change of natural frequency, even the mast structure is more flexible, the frequency of earthquake does not match the fundamental frequency of mast structure as can be seen in **Figure 4** so that the displacement responses of OHLE at the support stiffness of 100 kNm/rad show the lowest displacement comparing to earthquakes. Moreover, in case of Kobe earthquake, the response spectra curve shows that the PGA has a slight increase between 0 and 0.8 Hz until it reaches the peak at the frequency of 0.9 Hz which is close to the fundamental frequency of mast structure at the support stiffness of 1,000 kNm/rad. Hence, the resonance can be observed in this condition.

Overall, it is clearly seen that earthquake can affect the failure of electrical system based on the assumption that the system failure occurs when the transverse displacement of overhead contact wire is greater than 50 mm. **Figure 9** shows that about 30 mm average displacement is observed when the cantilever is located in the support stiffness of 10,000 kNm/rad. When the support stiffness decreases to 1,000 and 100 kNm/rad, about 480 and 1,100 mm, respectively, average displacement are observed which are much larger than maximum value. Hence, In this case, no trains can run until the broken wire is fixed. In fact, the design stiffness of soil beneath the structure must have the value more than this value. However, the soil stiffness might be reduced due to the strength degradation or environmental events such as erosion, seepage etc. which lead to the loss of soil stiffness. Moreover, the

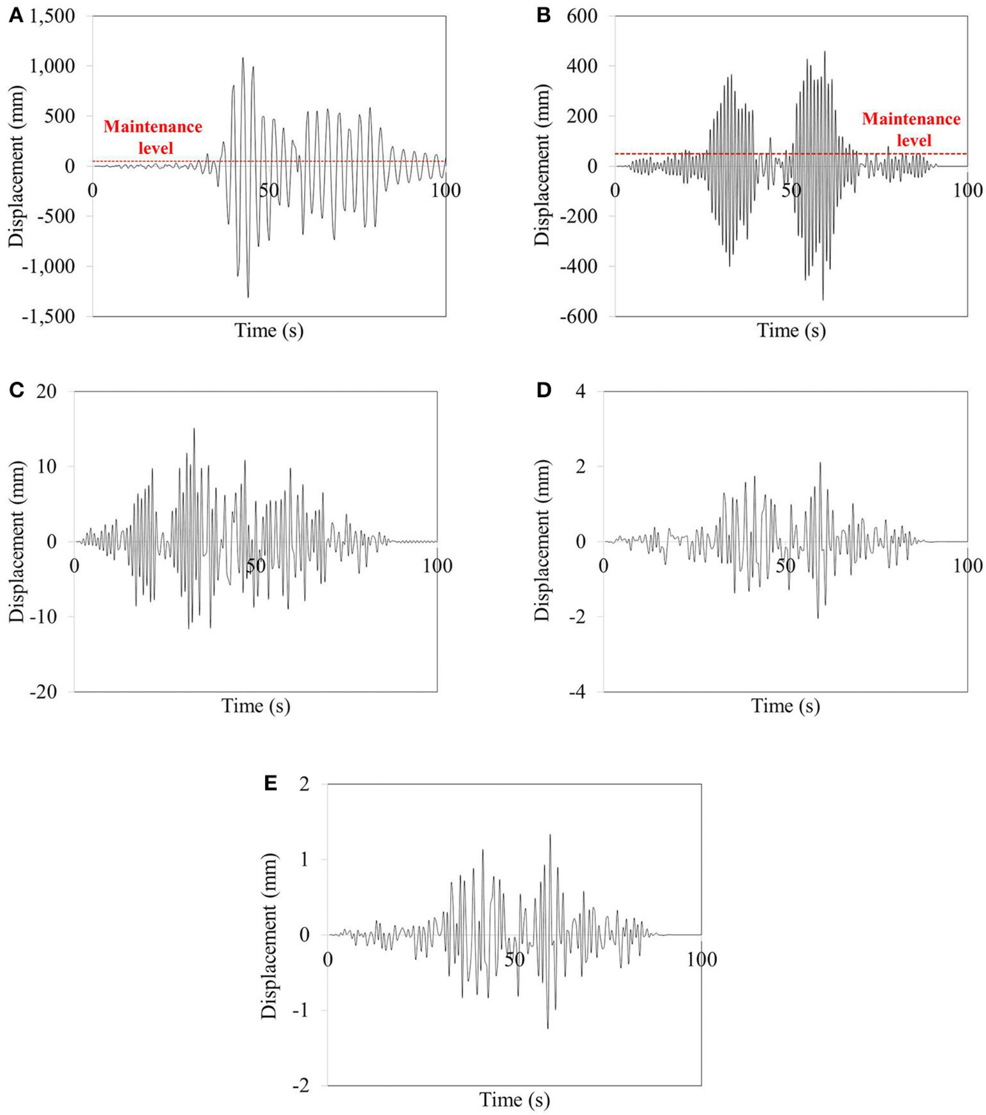

FIGURE 7 | Displacement responses under Chi-Chi earthquake at the position of overhead contact wire on cantilever mast at support stiffness of **(A)** 100 kNm/rad **(B)** 1000 kNm/rad **(C)** 10000 kNm/rad **(D)** 100000 kNm/rad **(E)** Fully fixed.

steel to foundation connection is also a reason of support stiffness reduction due to the connection failures such as broken bold, yielding weld, improper design and construction etc.

Beating Phenomenon

Even though the direction concerned for overhead contact wire monitoring is transverse direction, the beating phenomenon, which normally observed in tall building, is clearly detected in cantilever mast structure when earthquake is applied in longitudinal direction. The beating phenomenon is a periodic vibration caused by distinctive coupling between translational and torsional modes which have close natural frequencies (Çelebi, 1994, 2007; Suhairy, 2000; Mayoof, 2009; Çelebi et al., 2016). This phenomenon has a significant effect on the elongation of structure shaking especially lightly damped structure. It is noted that the stiffness of soil and natural frequency have a direct variation with damping coefficient.

The beating phenomenon can be clearly seen from the response in term of frequency domain. The fast fourier transform (FFT) is used to transform the response from time domain to frequency domain. The two peaks can be obviously observed. The beating period can be computed by the following equation (Boroschek and Mahin, 1991):

$$T_b = \frac{1}{f_b} = \frac{1}{|f_1 - f_2|} = \frac{2\pi}{|\omega_1 - \omega_2|} \tag{1}$$

Where f_b is the beating frequency, difference between two peak frequencies (f_1, f_2).

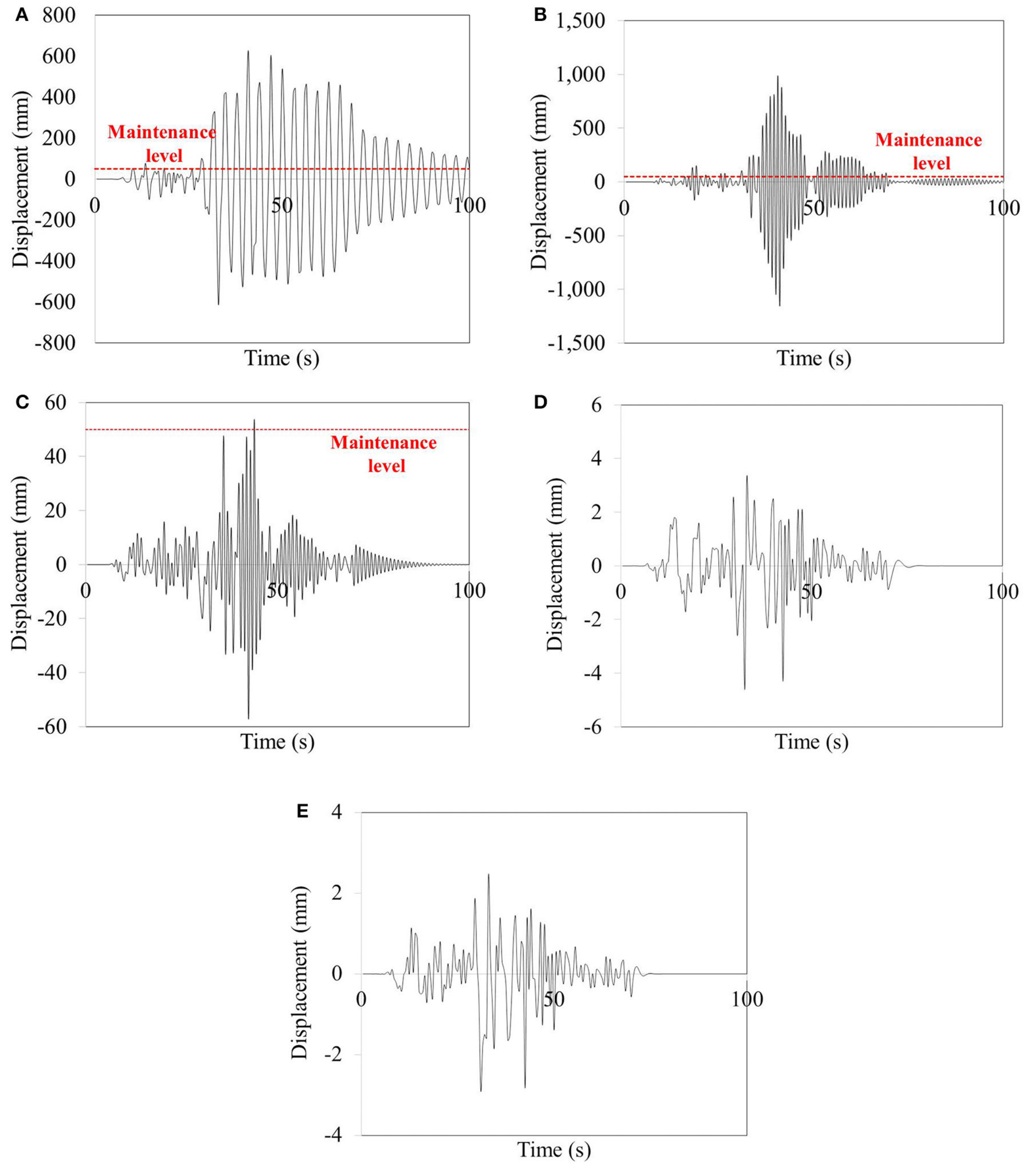

FIGURE 8 | Displacement responses under Kobe Japan earthquake at the position of overhead contact wire on cantilever mast at support stiffness of **(A)** 100 kNm/rad **(B)** 1000 kNm/rad **(C)** 10000 kNm/rad **(D)** 100000 kNm/rad **(E)** Fully fixed.

Figure 10 shows the displacement responses of OHLE at rigid soil condition under Kobe earthquake in term of time domain and frequency domain, respectively. The two major peaks at around 0.93 and 1.07 Hz are clearly appeared, as shown in **Figure 10B**. To confirm this phenomenon, beating period and oscillation period should be calculated by the relationship of time and the number of cycles in order to obtain the two peak frequencies. The beating period calculation is shown in **Figure 10B** while, the oscillation period is computed by beating period divided by the number of cycles in one beating period.

$$\text{Beating period, } T_b = \frac{2\pi}{|\omega_1 - \omega_2|} = \frac{1}{|f_1 - f_2|} = 66.5 - 60.3 = 6.2\ s$$

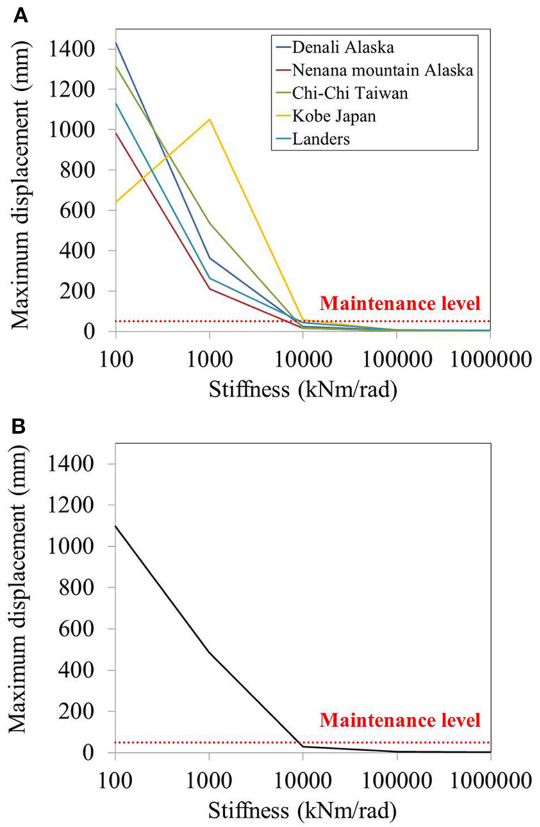

FIGURE 9 | Maximum displacements at the position of overhead contact wire on cantilever mast at various stiffness in transverse direction under **(A)** earthquakes **(B)** average displacement.

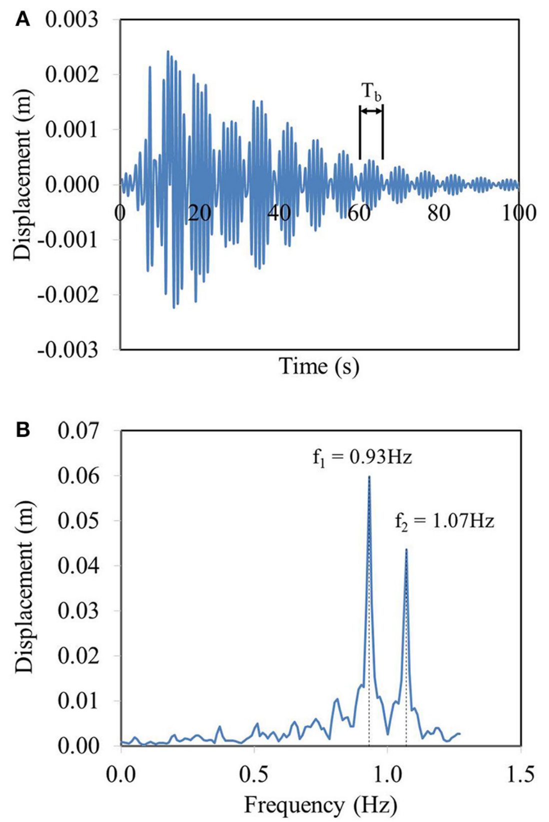

FIGURE 10 | Displacement of OHLE at rigid support in longitudinal direction under unscaled Kobe earthquake in term of **(A)** time domain **(B)** frequency domain.

$$\text{Oscillation period} = \frac{4\pi}{|\omega_1 + \omega_2|} = \frac{2}{|f_1 + f_2|}$$
$$= \frac{66.5 - 60.3}{6} = 1.033\, s$$

After solving the system of two equations, the two frequencies are $f_1 = 1.05$ s and $f_2 = 0.89$ s. It is noted that the differences between calculation of time domain response and frequency domain response are less than 5%. Therefore, the calculation shows a consistent between the time domain and the frequency domain. It is concluded that the beating phenomenon can occur in the OHLE system during far-field earthquakes.

CONCLUSIONS

The overhead line equipment (OHLE) is an important asset of the railway infrastructure and is one of the most vulnerable ones. For one or two railway tracks, the support of OHLE is cantilever mast which is normally made of H-section steel column. Although the concept of OHLE is simple, the problem is poor dynamic behavior of OHLE are needed to develop. The dynamic behavior of cantilever mast has not been fully investigated. The three-dimensional cantilever mast structure is created using finite element package STRAND7 with the consideration of soil-structure interaction. The five far-field earthquakes with the magnitude of 6.5–8 Mw are considered in this study. This paper indicates the effects of far-field earthquake excitations on cantilever mast structure located alongside the railway track. The support stiffness underneath the structure is considered. The position concerned is the end of the cantilever which is the location of overhead contact wire. The finding shows that beating phenomenon can be observed not only in tall or mid-rise building but also in small structure such as cantilever mast. This phenomenon is observed when the earthquake is inputted in longitudinal direction or oblique angles. Beating phenomenon is a dynamic behavior caused by the distinctive coupling of translational mode and torsional mode. This makes an elongation shaking of cantilever mast structure during earthquake excitations. Nevertheless, the direction considered is transverse because the movement of contact wire in this direction can cause the broken wire which lead to the failure of electrical system. The responses indicate that the maximum displacement increases when the support stiffness decreases. Moreover, in case of Kobe earthquake, the responses occurred show that the maximum displacement of OHLE at the soil stiffness of

1,000 kNm/rad is greater than that at the soil stiffness of 100 kNm/rad which is different from other earthquake responses. This is because the change of natural frequency of cantilever mast due to the increase of soil stiffness from 100 to 1,000 kNm/rad seems to have the frequency close to a dominant frequency of Kobe earthquake which leads to the appearance of resonance phenomenon. The results clearly show that the responses of the cantilever mast are depended on the frequencies of earthquake excitations which can be seen in the response spectral curve. Surprisingly, the displacements observed at the contact wire are greater than the allowable value when the support stiffness is less than around 9,000 kNm/rad. The reasons of the losses of support stiffness are the connection failure, such as broken bold, yielding weld, and soil erosion and degradation. Therefore, the connection at support should be carefully designed and constructed. The insight shows that condition monitoring of OHLE can be used for earthquake detection so the train could be stopped even faster before the failure of system. This is the world first to investigate the effect of far-field earthquake on overhead line equipment. The outcome of this study will help a better understanding of the critical responses and dynamic behavior of cantilever mast structure and its support under earthquake in order to improve the design standard of this structure.

AUTHOR CONTRIBUTIONS

CN and SK developed the concept and model. CN performed the seismic analysis and model verification. SK and CB reviewed the results. CN, SK, and CB wrote the manuscript.

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frontiers in Built Environment

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Role of Pre-processing in Textual Data Fusion: Learn From the Croydon Tram Tragedy

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Tram/train derailment subject to human mistakes makes investments in an advanced control room as well as information gathering system exaggerated. A disaster in Croydon in year 2016 is recent evidence of limitation of the acquired systems to mitigate human shortcoming in disrupted circumstances. One intriguing way of resolution could be is to fuse continuous online textual data obtained from tram travelers and apply the information for early cautioning of risk discovery. This resolution conveys our consideration regarding a resource of data fusion. The focal subject of this paper is to discuss about role of pre-processing ventures in a low-level data fusion that have been distinguished as a pass to avoid time and exertion squandering amid information retrieval. Inclines in online text data pre-processing is reviewed which comes about an outline suggestion that concede traveler's responses through social media channels. The research outcome shows by a case of data fusion could go about as an impetus to railway industry to effectively partake in data exploration and information investigation.

Keywords: data fusion, text processing, social media, alert system, risk mitigation, disruption, tram accident, microsleep

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INTRODUCTION

Data is enormous in railway industry, covering both train/tram operations and infrastructure management dimensions. Acknowledging many successful stories from outside railway domain associated with an adaptation of data-driven innovation, British railway infrastructure manager has launched the "Challenge Statements" program series which recognize data exploration/exploitation is one of their agenda (Network Rail, 2017). One element which has a great potential to accelerate the journey of worthwhile state is data fusion.

Data fusion is famously defined as (White, 1987): "A multi-process dealing with the association, correlation, combination of data and information from single and multiple sources to achieve refined position, identify estimates and complete and timely assessments of situations, threats and their significance." The definition grasps a spirit of encouragement and inspirations for data analysts to continuously conduct data exploration/exploitation to appropriately enrich the value of existing information about an object of interest. In regard to application-wise, data fusion is nowadays beyond the dominant and matured domain; remote sensor and signal processing, as case studies can be found in condition monitoring (Raheja et al., 2006), crime analysis (Nokhbeh Zaeem et al., 2017), forest management (Chen et al., 2005), and engineering (Steinberg, 2001). The key of data fusion being applied in diverse research domains is about the way the problem is formulated and the choice of methods (Hannah et al., 2000; Starr et al., 2002) recognize the capacity of identifying a parallel between problems under consideration with data fusion models is the key to success. These influential factors drag our regard to the classical data fusion framework, known as the JDL (an acronym for Joint Directors of Laboratories) framework.

The JDL framework put major elements of data fusion definition into five successive steps (a pre-processing, object refinement, situation assessment, threat assessment, and process refinement) to aid the encouragements expressed in the definition in a systematic and efficient way. The framework could be conveniently attended in hierarchy sense upon user's capacity to accommodate the current problem at hand. For example, the low-level data fusion comprises of a pre-processing step is significant for a new application (or project) in a context of data refinement. Hence, this study is carried out to address design issues of low-level data fusion, especially pre-processing steps regarding textual data. The discoveries are relied upon to serve the future application of online text data-driven alerting system inspired from the tragedy of Croydon tram derailment.

On 9th November 2016, a commuter community particularly in London, was stunned when the unfortunate tram No. 2551 derailed and killed seven passengers. The tragedy can be delegated as a *black swan* in the clean track records of the tram operator company following 15 years of re-operations in London. According to early investigation report, a human-error is identified as a primary contributor to the derailment. The tram driver endured *black out* which prompt to an inability to reduce the tram speed as far as possible before entering the accident area. The report proclamation sparks our interest to discover a data-powered solution to mitigate the derailment risk root from human weakness.

Having text as data source creates a unique data pre-processing challenge, as it is unstructured data. Text appears in non-uniform length of words that does not reside in fixed row-column database. This causes a dimensionality in terms of words space of each text is vary from one sender to another which requires a robust algorithm for text classification. Consequently, assigning conventional pre-processing tools that are designed for structured data, to textual data might fail to optimally exploit hidden information.

In the following section, role of pre-processing steps in a data fusion model is disclosed to highlight adaptability of the steps for different research motives. An investigation report of the Croydon Tram tragedy was analyzed in parallel to author's viewpoint to underpinning a motivation of a passenger-participation warning system proposal. For a set of identified system requirements, a suitable design of online text pre-processing steps to be embedded in the system under purview is presented. A conclusion remark about synergy among low-level data fusion, text pre-processing and online data source is stated in the Conclusion section.

ROLE OF PRE-PROCESSING

Pre-processing is the lowest level activities in the process of combining input data from multiple sources to gather information in order to achieve inferences. Some authors call it as source processing due to high workload is applied on data source. Depending on the problem under consideration, data sources can be sensors, a prior knowledge, databases, or human input. The step is metaphorically viewed as a bridge connecting external elements with data fusion framework that must be traversed before subsequent data fusion steps, more complex in nature, can be performed. By imposing barriers to free-flow of data streaming, a compact representation of raw data is sufficient to produce brief but reliable decision making process (Eggers and Khuon, 1990). In addition, an efficiency and scalability of an object refinement; a subsequent step applied to processed data, can be improved through a proper pre-processing (Mitali et al., 2003).

However, this particular benefit is challenging to gain when online text as a data source. Online text is delivered from vast points of network to the servers at different arrival time and in various styles which is very informal in nature. Language dependent factors which do not have an impact to information retrieval are also identified obstacle (Singh and Kumari, 2016; Nokhbeh Zaeem et al., 2017) points out the challenge in dealing with social media text data for fortification of sentiment classification especially in terms of short length and internet slang word. In addition, an existence of uninformative text such as HTML tags, scripts, internet abbreviations and advertisements rises the computational complexity to an upper level as compared to a well-presented text (Petz et al., 2012; Dos Santos and Ladeira, 2014) underline the significant of having a language detector as an additional component to standard text pre-processing process in case of multilingual responses are eligible to a system of response. In (Hamouda and Ben Akaichi, 2013), major text processing issues when dealing with non-English data source was a topic of discussion.

CROYDON TRAM DERAILMENT

The Tragedy

More than 30 million passengers use the lime green-and-blue Croydon trams in 2013/2014 and this number is expected to increase to up to 60 million in 2030 (Transport for London, 2014). The projection is expected to face a small degradation in a period of December 2016 to February 2017 due to a fatal tram derailment which killed seven passengers and more than 51 suffered from various degree of injury. The tragedy took place at the first track junction (diverging track) which is located approximately 200 m from the Sandilands stop toward the Wimbledon station (refer to a star marker in **Figure 1**).

Motivation for Pursuing the Research

An earlier investigation conducted by U.K Rail Accident Investigation Branch found that the tram was speeding excessively when entering the junction (Rail Accident Investigation Branch, 2016). Theoretically, a significantly large difference in speed between traveling speed and the speed limit increases the risk of train or tram to over-tuning at a transition curve (Bearfield and Marsh, 2005). Routine tram passengers who survived the tragedy agree with the initial finding. The

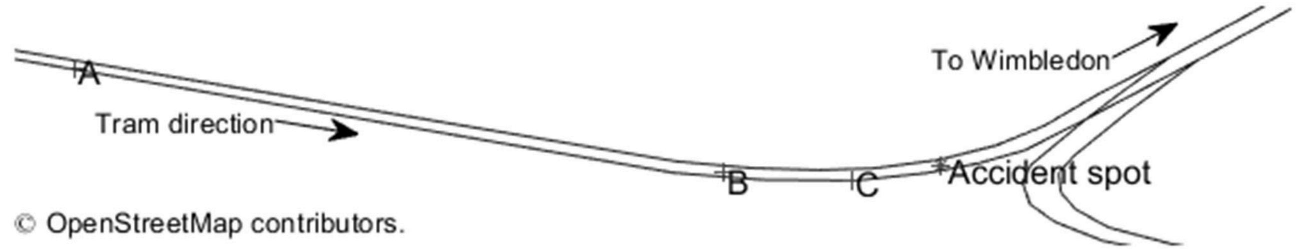

FIGURE 1 | Three points of interest to a maximum speed and braking distance analysis. The OpenStreetMap representation of the accident area is downloaded from www.openstreetmap.org.

passengers felt that the tram traveled at unusual speed along a tangent track- and are possibly, a track segment that connects a point A and B shown in **Figure 1** (Christodoulou, 2016). Their latter claim may make sense if the tram speed was over the speed restrictions applied to that section. Regardless of whether the London Arms, the Croydon Tramlink operator, implements a speed restriction to the tangent line or not, technically speaking, a tram needs to reduce an initial speed when approaching the point B. Failure to do so will cause a tram to make a sharp/hard turn at the transition curve, point C In extreme cases (i.e., when trams travel too fast), a tram derailment is possible to occur. One may query how an experienced tram driver was unable to control the over speeding tram. Unfortunately, the tram driver was reportedly suffering *black out* which could be grouped in the same category with explosion, sabotage and sinkhole as a disruption i.e., rare events but have high consequences.

Speaking about technology, Croydon Tramlink equips their network with a sophisticated computer-based system known as SCADA (Supervisory Control and Data Acquisition) (Parascandolo, 2007). Technically speaking, controllers in the control room should have no problem stopping or reducing the speed of any misbehavior/suspicious trams under their supervision remotely by reducing or cutting-off electric supply. Indeed, we believe the company has a standard procedure to trigger that action. In respect to the recent tram crash were over-speeding is greatly believed to be the cause of the tram derailment, the system or its associated system should detect an unusual power usage by the trams only seconds before the crash (particularly in the braking zone) unless the system is not programmed to execute that kind of proactive action. This situation motivates us to introduce a system patch which capable to warn a control office in the event of hazard detection. Interestingly, tram passengers participation and social media will be the core of the proposed data-driven innovation.

TEXTUAL DATA PRE-PROCESSING

Four components are proposed to increase quality of online text raw data for the use of an alerting system. **Figure 2** depicts their sequence in a basic data fusion model i.e., input, pre-processing and decision.

Input

The system is only function with a participation of tram passengers. Limited Wi-Fi access and/or traveler's phone apps should be provided at no cost to all passengers in order to channel the system with various sources of online textual data.

System users, who initially agree to give permission for the system to extract his/her online data, could feel their privacy is threatened when the exclusivity as an individual deteriorates. In this context, Van Wel and Royakkers (2004) describes such circumstance as "de-individualisation"—an occurrence when group profiles are often used to judge, treat and possibly discriminate people instead of gratifying them as per individual characteristics. However, there tends to exist a group of users who are voluntarily responsive toward deals and promotions offered by companies—such as fast speed internet connection, ticket discounts or meal vouchers— at the expense of their own data privacy. Surprisingly, even though they are informed of losing their own privacy, this doesn't stop them from blindly responding and agreeing to the so-called "online privacy-policies protection."

To reduce unnecessary stress related to online privacy protection between stakeholders of the system, privacy policies are normally equipped with an instrument used for online data collection, processing and storage (Van Wel and Royakkers, 2004; Dean et al., 2016) highlights the need for a policy that is short and precise, easy to understand and consistent in its contents. In fact, an enterprise can avoid a myopic policy by adopting a sound policy which included legal, consumer and social, business perspectives into consideration (Dean et al., 2016). On the other hand, transparency in the privacy policy can be promoted by letting users to have control over data distribution and data protection. Achieving these features is possible from the users' side which is only sending encrypted data to the web-based system. Furthermore, to facilitate textual data encryption technology, a browser plugin called ShadowCrypt (He et al., 2014), is recommended for user's consideration.

ShadowCrypt performs the so-called diplomatic role between the user and the web application. User input is captured and will be encrypted either by using a deterministic or random encryption before the application is allowed to access the data. Encrypted data is only accessible to the web apps

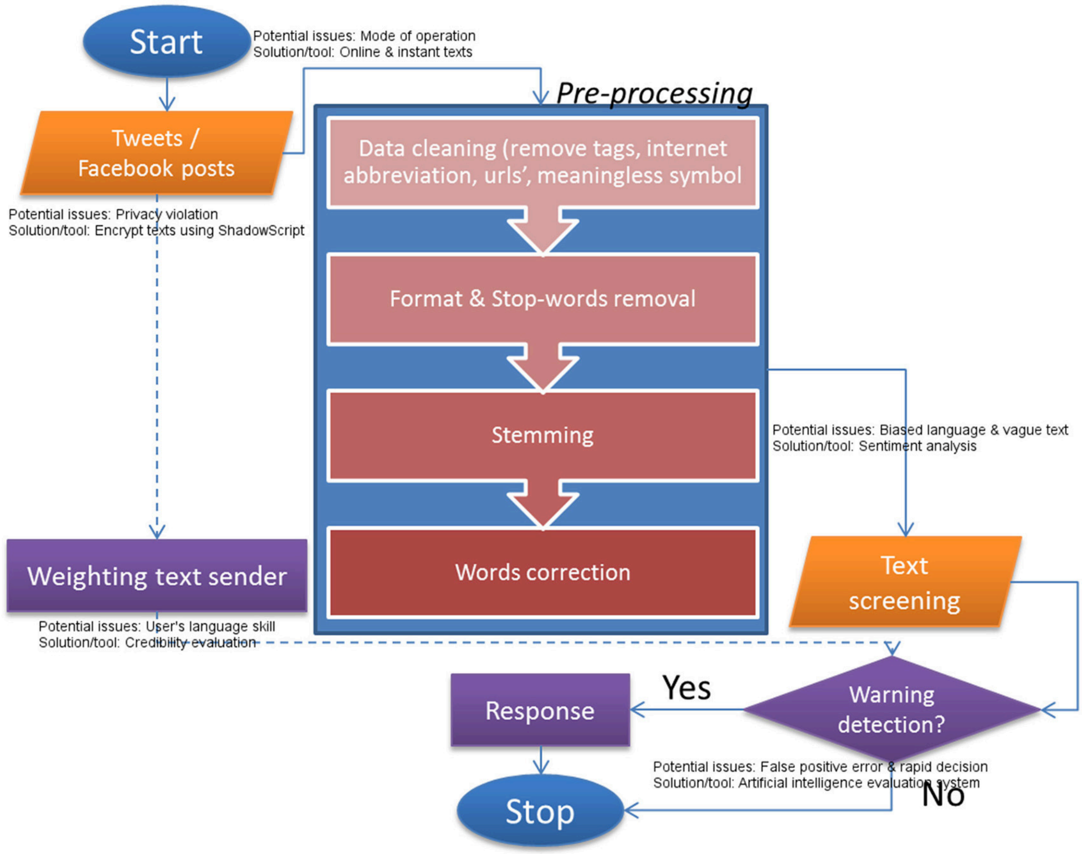

FIGURE 2 | A position of the proposed online text pre-processing steps in the passenger-participation warning system.

with decryption keys supplied by the data owner who is also the system user. Interestingly, that keys are only stored in the ShadowCrypt of the user's computer. This implies that the administrator of the web apps cannot simply access the data as he must be physically present at the location of the user's computer. This smart mechanism shifts the threat level of data breach and privacy violation from high to low risk (possible but not expected) while still participating in the alert system.

Weight Assignment

When the system is opened to every tram passengers, it is necessary to take credibility of text sender into consideration. Users annotated with low credibility might post vague texts and this drives the system to trigger a false alarm. Fortunately, user's credibility can be measured quantitatively by applying a reputational ranking model to user's profile (Alrubaian et al., 2017). User's profile features such as number of followers, number of positive and negative posts, etc. being feed to the model in order to generate credible scores. The scores weighs user's texts and it will be used to prioritize resources when verifying the content of text.

Pre-processing

Data Cleaning

Not only regular computer program expressions, html characters, banners, and graphical images but non-text data will be rid out from the messages. However, emoticons are given exemption at this phase as it has been recognized as a trend in non-verbal expressions. An effective data cleaning significantly reduce data dimensionality of term space which has a great impact on information extraction efficiency.

Stop-Words Removal

At this stage, an input vector will consist of words and emoticons (if any) only. Stop-words which are a bunch of words that insignificant for analysis i.e., non subject keywords, including high frequency word categories such as articles, prepositions, and pro-nouns will be removed from the text vector. As the designed system expects text data which illustrates non-positive side of passenger's reactions about their on-board experiences, positive words and emoticons are also labeled as stop-words.

Stemming

Further number of words reduction can be gained from stemming the remaining inflected words to its stem or base form. Stemming the words with similar meaning also offers time savings and larger memory space. Note that, this process should not be applied to words that are not semantically related. Hence, words that do not have the same meaning should be kept separate. In fact, a text classifier can be sometimes negatively affected from text stemming (Nokhbeh Zaeem et al., 2017).

In the context of alert/warning system, biases in language of texting should be treated with care for the system to have a low rate of false positive. False positive case occurred when the system triggered an alert for no apparent reason. To address this issue, it is necessary to equip the system with a stemming algorithm that is capable to deal with biased language in a framing situation e.g., a specific train journey. Referred to as framing bias in literature of linguistic subjectivity, this class of bias is considered by the fact that people use subjective word, phrase or sentence to express his/her opinion toward a particular frame of event. For instance, a phrase "train makes me dizzy" unlike "train is wavering," seems to be an excessive expression to describe uncomfortable feeling of a train journey.

Subjective expression which is commonly found in people's experiences and opinions can be treated by performing sentiment analysis. The analysis is an automatic process which begins with a classification of word, phrase, emoticon, and slags that occurs in a text into a positive or negative category. Following the classification, sentiment of each text entry is calculated with respect to the semantic orientation or polarity of the text. In the case of Twitter messages, adjectives would be the indicators of the orientation (Taboada et al., 2011). For a topic of interest in single domain, lexicon-based approach that need for a dictionary of (positive and negative) word rankings is preferably adopted to calculate sentiment value for any given text. Hence, a dictionary of (positive and negative) word rankings will be built from a list of adjectives and corresponding scores of semantic orientation. A list of ready-to-use dictionaries can be found in (Taboada et al., 2011).

Words Correction

Before leaving the pre-processing phase, any abbreviations or misspelling words are replaced. Association rules and online dictionary could be applied here to increase a transformation success rate.

Text Screening (Decision)

An output of pre-processing can be treated as a cleaned, filtered and compact version of user's textual response. The final contents are then screened thoroughly to unsurfaced words or phrases related with ride comfort, motion, safety, noise and vibration and bad or suspicious feelings. The process loop is closed i.e., calling for subsequent texts, unless the system triggers an early warning in a specific identified tram. An expert assessment is called upon for human interpretation before any responsive action such as an immediate call to an involved tram driver is performed.

CONCLUSION

Trams in Britain are still manned-operated mode of transport, which means it is vulnerable to human errors as appeared in a recent Croydon tram derailment tragedy. Besides physical technologies adaption for human-related risk reduction in tram/train operations such as radio communication systems, one interesting aspect that has huge potential but not been explored extensively is passenger's participation in hazard detection. This study introduces a concept of data fusion in which processed passenger's negative reactions about real-time tram operations posted in media social medium is treated as complementary information to the existing safety system. A key element to successfully use text as data source is having a proper design of pre-processing steps which was a topic of discussion. Data cleaning, removal, stemming, and correction are necessary to refine raw data before high-level data fusion is considered. To analyse parameters sensitivity of the recommended design, a training dataset is required and its preparation has been identified as the next agenda. On top of that, an identification of suitable incentive-driven methodology to attract passenger's participation must be solved in parallel, otherwise the proposed system has nothing to improve.

AUTHOR CONTRIBUTIONS

MB developed the presented idea and wrote the first draft of manuscript with support from SK. All authors contributed to the final version of the manuscript. SK supervised the writing.

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frontiers in Built Environment

GENERAL COMMENTARY
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Commentary: Fatigue Life Assessment Method for Prestressed Concrete Sleepers

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Keywords: railway, prestressed concrete sleeper, fatigue life, assessment method, ballasted track

A commentary on

Fatigue Life Assessment Method for Prestressed Concrete Sleepers
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We have closely read with interest the paper published by You et al. (2017) about the fatigue life assessment method for prestressed concrete sleepers. This article shows that there is a possibility that the assessment method, which is commonly used to evaluate the fatigue life of the reinforcing bar and prestressed steel for railway concrete structures, is also applicable to the prestressed steel of the concrete sleepers. Establish of the assessment method for the prestressed steel on the concrete sleepers can be benefit when it comes to maintenance (e.g., replacement cycles and priority of replacement locations, etc.).

However, in the experiments conducted by Parvez (2015) and Parvez and Foster (2017) to verify the applicability of the assessment method for the prestressed steel of prestressed concrete sleepers, the upper limit of the cyclic load was set to 240 kN to make sure that the prestressed steel fractures. The use of this load shows that very heavy vehicles with severely flat wheels may run repeatedly on poorly maintained tracks. In Japan, the ballasted track, in which prestressed concrete sleeper is a vital component, is selected under the assumption of regular maintenance (Railway Technical Research Institute, 2014). Hence, such a high load which makes prestressed concrete sleepers cracked rarely occurs. For this reason, it should be discussed whether fatigue fracture of a prestressed steel can occur in the actual environment during the service life of prestressed concrete sleepers.

In the design of Japanese prestressed concrete sleepers, as shown **Table 1**, dynamic wheel load for the serviceability limit state is specified as twice as high as the static wheel load. The concrete stress of the prestressed concrete sleeper is normally needed to be in compressive region against that dynamic load, which is called full prestress design (Antoine, 2012). In addition, the event probability of the dynamic load is estimated to be about 0.1% or less (Wakui and Okuda, 1999). Therefore, it is rare that cracks on prestressed concrete sleepers will occur. Even if high load over that dynamic force is added to the tracks, it is easy to imagine that the number of the occurrences is limited within design service life of the prestressed concrete sleepers. In evidence, there are no reports that a sleeper has broken due to fatigue fracture of the prestressed steel. Thus, the check about fatigue fracture of prestressed steels of the prestressed concrete sleeper is almost omitted in the design of Japanese prestressed concrete sleepers.

On the other hand, when focusing on prestressed concrete sleepers in service, some of them exceed the design service life which is generally set to 50 years (Railway Technical Research Institute, 2014). The fatigue life of prestressed steels may be an important indicator for determining the

TABLE 1 | Comparison of design methods among countries.

Country Design code	Japan Design standards for railway structures and commentary (track structures)	EU EN 13230	United States of America American Railway Engineering and Maintenance-of-Way Association	Australia AS 1085.14
Structural design	Full prestress	Partial prestress	Partial prestress	Partial prestress
Impact factor (IF) (P_d = IF*P_s)	2	1.61–2.36	0.7–1.2	2.5
Load distribution factor (*D*)	0.5	0.5	0.43–0.61	0.45–0.60
Tonnage factor (*T*)	–	–	0.7–1.1	–
Total value [IF*D(*T)]	1	0.81–1.18	0.21–0.81	1.13–1.50

P_d, dynamic wheel load; P_s, Static wheel load.

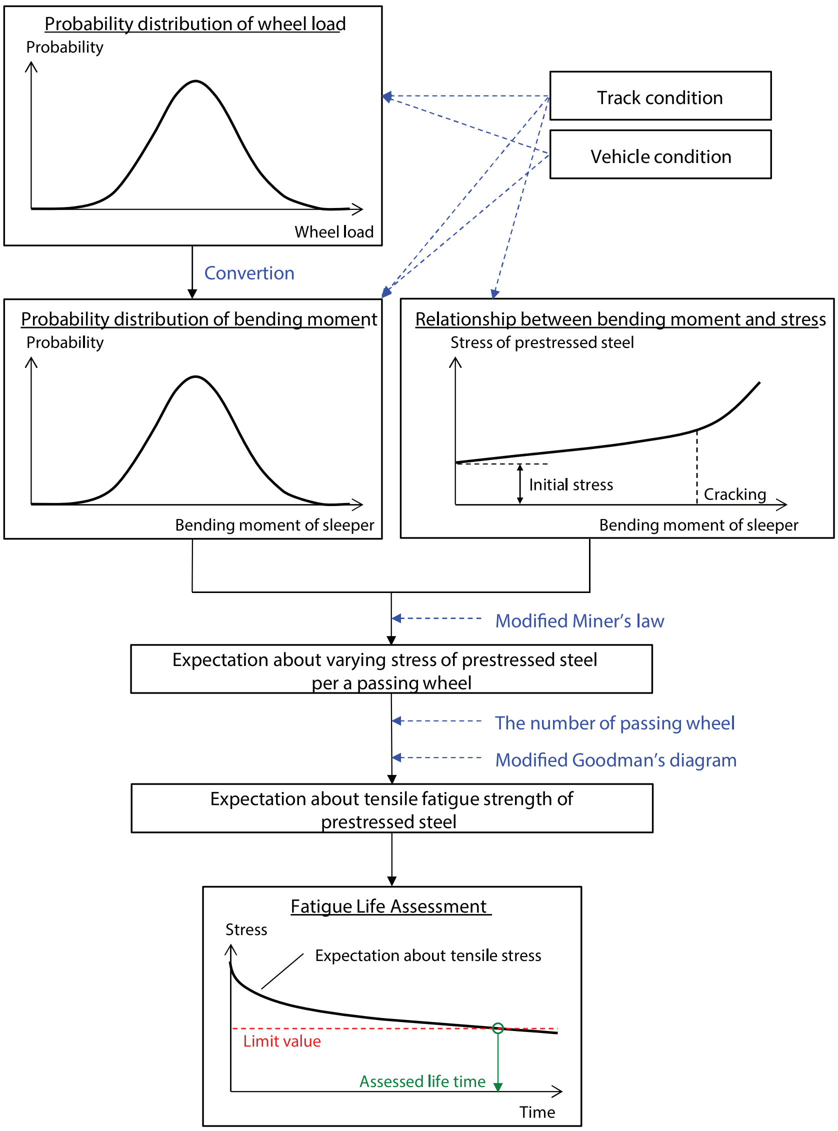

FIGURE 1 | Fatigue life assessment method for prestressed concrete sleepers.

appropriate replacement time of prestressed concrete sleepers. In other countries, the importance of this indicator increases because prestressed concrete sleepers are normally designed with partial prestress design (FIP Commission on Prefabrication, 1987; Antoine, 2012) which allows generation of tension stress on sleeper's concrete when train load is added as shown **Table 1**. Therefore, developing fatigue life assessment method for prestressed steels of prestressed concrete sleepers will become more important in the future.

To assess the fatigue life of prestressed concrete sleepers on service tracks more accurately and reasonably, it is necessary to establish fatigue life assessment method according to the situation of the individual service line. **Figure 1** shows the image of the assessment method. First of all, it is important to obtain the relationship between wheel load acting to tracks and the probability. Secondly, the relationship is needed to be converted to the relationship between bending moment of the prestressed concrete sleeper and the probability. At the same time, it is also important to ascertain the between the wheel load and the stress of prestressed steel. And then, by multiplying these, expectation about the stress of prestressed steel per a passing wheel is calculated through modified Miner's law (Miner, 1945). In addition, taking into account the number of passing wheel and modified Goodman's diagram (Nordby, 1958), expectation value about the tensile fatigue strength of prestressed steel at certain period can be calculated. Finally, the expected value is compared to the limit value for fatigue fracture of prestressed steel (Wakui and Okuda, 1999; Goto et al., 2012, 2017). In that time, it is necessary to sufficiently grasp the influence of each parameter (Watanabe et al., 2016) which is concerning the track and the vehicle condition on expectation value of the tensile fatigue strength of prestressed steel.

AUTHOR CONTRIBUTIONS

All the authors equally contributed to the manuscript. TW, MH, and SM searched for literature to support main content of the manuscript. KG and TW drafted the manuscript. KG and MH revised manuscript and gave final approval to submit this manuscript. However, all the authors agreed to submit this manuscript.

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