Reliability-based conversion of a structural design code for railway prestressed concrete sleepers

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The manuscript was received on 25 July 2010 and was accepted after revision for publication on 12 July 2011.

DOI: 10.1177/0954409711418754

Abstract: Ballasted railway track is very suitable for heavy-rail networks because of its many superior advantages in design, construction, short- and long-term maintenance, sustainability, and life cycle cost. An important part of the railway track system, which distributes the wheel load to the formation, is the railway sleeper. Improved knowledge has raised concerns about design techniques for prestressed concrete (PC) sleepers. Most current design codes for these rely on allowable stresses and material strength reductions. However, premature cracking of PC sleepers has been found in railway tracks. The major cause of cracking is the infrequent but high-magnitude wheel loads produced by the small percentage of irregular wheels or rail-head surface defects; both these are crudely accounted for in the allowable stress design method by a single load factor. The current design philosophy, outlined in Australian Standard AS1085.14, is based on the assessment of permissible stresses resulting from quasi-static wheel loads and essentially the static response of PC sleepers. To shift the conventional methodology to a more rational design method that involves a more realistic dynamic response of PC sleepers and performance-based design methodology, comprehensive studies of the loading conditions, the dynamic response, and the dynamic resistance of PC sleepers have been conducted. This collaborative research between several Australian universities has addressed such important issues as the spectrum and the amplitudes of dynamic forces applied to the railway track, evaluation of the reserve capacity of typical PC sleepers designed to AS 1085.14, and the development of a new limit states design concept. This article presents the results of the extensive analytical and experimental investigations aimed at predicting wheel impact loads at different return periods (based on field data from impact detectors), together with an experimental investigation of the ultimate impact resistance of PC sleepers required by the limit states design approach. It highlights the reliability approach and rationales associated with the development of limit states and presents guidelines pertaining to conversion of AS 1085.14 to a limit states design format. The reliability concept provides design flexibility and broadens the design principle, so that any operational condition could be catered for optimally in the design.

Keywords: prestressed concrete sleepers, design code, allowable or permissible-stress design, limit states, performance-based design, reliability and safety, probabilistic analysis, impact loading, Australian Standard AS1085.14

1 INTRODUCTION

It is commonly believed that railway is the world's safest transportation system for either passengers or

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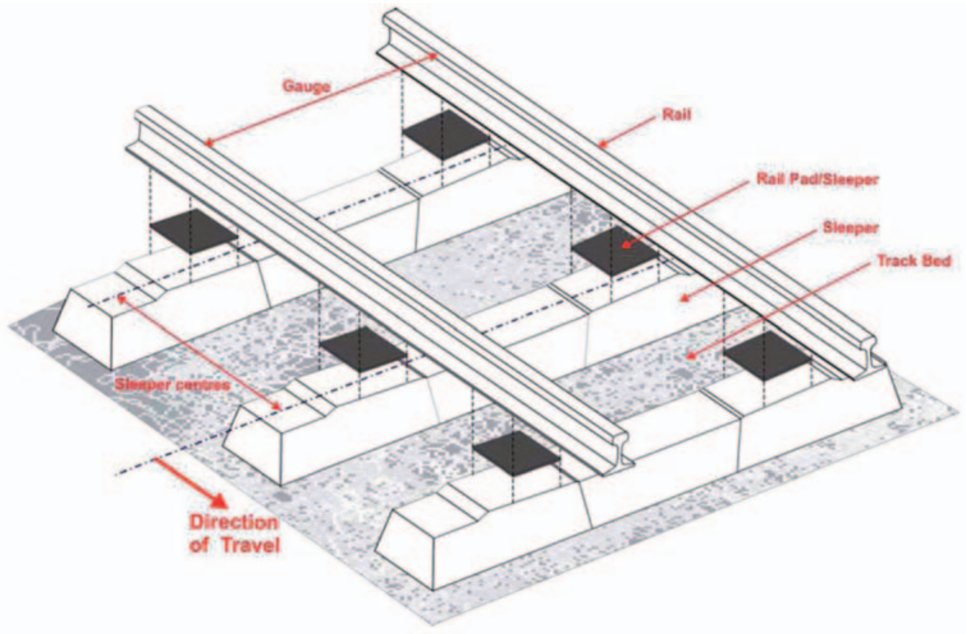

Fig. 1 Typical ballasted railway tracks

goods. Track structures guide the safe, economical, and comfort ride of trains. The key components of a typical ballasted railway track are shown in Fig. 1 [**1**]. Its components can be categorized into the two main groups: superstructure and substructure. The top-layer components of the track skeleton such as the rails, rail pads, prestressed concrete (PC) sleepers, and fastening systems form the superstructure group. The substructure includes a geotechnical system consisting of ballast, sub-ballast (or capping layer), and subgrade (formation) [**2**]. Both superstructure and substructure are vital in ensuring the safety and comfort of passengers and a satisfactory quality of ride for passenger and freight rolling stocks. In Australia, the United Kingdom, and Europe, the common term for the structural element that distributes axle loads from rails to the substructure is 'railway sleeper', while 'railroad tie' is the usual term used in the USA and Canada. This article will adopt the former term hereafter.

The main duties of sleepers are: (a) to transfer and distribute loads from the rail foot to the underlying ballast bed, (b) to hold the rails at the proper gauge through the rail fastening system, (c) to maintain rail inclination, and (d) to restrain longitudinal, lateral, and vertical movements of the rails [**2**]. Dynamic load conditions on railway track structures have been illustrated elsewhere [**3**] together with the common design procedures for Australian railway tracks [**4**]. The allowable or permissible-stress design method makes use of an empirical function taking into account the static wheel load (P_0) with a dynamic impact factor (ϕ) to account for dynamic vehicle/track interactions

$$P_{\mathrm{D}} = \phi P_0 \tag{1}$$

where P_{D} is the design wheel load, P_0 the quasi-static wheel load, and ϕ the dynamic impact factor (>1.0).

Significant research effort has been devoted to the forces arising from vertical interaction of train and track because these dynamic transient forces are the main cause of railway track problems, when trains are operated at high speed and with heavy axle loads. It is important to note that wheel–rail interactions induce much higher frequency and much higher magnitude forces than simple quasi-static loads. These forces are referred to as 'dynamic wheel–rail' or 'impact' forces. A summary of typical impact loadings (dynamic transient) due to train and track vertical interaction was presented in reference [**3**] with particular reference to the shape, magnitude, and duration of impact loads found in railway track structures.

The permissible-stress design concept has fundamentally dominated in current Australian and international design standards for PC sleepers, where various limiting values or reduction factors are imposed on material strengths and load effects [**3–5**]. Empirical data collected by railway organizations show that railway tracks, especially PC sleepers, might have untapped capacity that could bring potential economic advantage to infrastructure owners. It is well known that the permissible-stress design method does not consider the ultimate strength of materials, probabilities of actual loads, risks associated with failure, or other factors leading to overdesign of the PC sleepers. A research project to study the actual load-carrying capacity of PC sleepers was developed as a collaborative project between several Australian universities and the industry partners within the framework of the Australian Cooperative Research Centre for Railway Engineering and Technologies (Rail-CRC) [**6–8**]. The main objective was to develop guidelines for conversion of the existing Australian design code for PC sleepers into a more rational limit states design format, accounting for the statistical nature, probability, and realistic risk of failure.

A limit states design concept and load factors for a revamped standard AS1085.14 were proposed by Murray and Leong [**6**, **7**]. Expressions for predicting the impact loads at different return periods (based on field data from impact detectors at two locations) were proposed. It was suggested that a simple pseudo-static (using a factored load) approach can be used in the design procedures for PC sleepers under routine traffic. A dynamic analysis was recommended as part of a design process for PC sleepers under non-routine traffic. The research team has since carried out statistical, probabilistic, and experimental studies to study the ultimate resistance of the PC sleepers as required for a limit states design approach [**9–11**].

In addition to experimental capacity evaluations, the conversion of the existing design standard to a new limit states design format requires a comparative examination to reinforce the safety margin and to maintain the low probability of failure of PC sleepers designed in accordance with both permissible-stress and limit states provisions. The performance of structural systems depends on the weakest element with the lowest reliability. To achieve uniform performance and reliability in structural designs for different design principles, the reliability-based approach is the most suitable, either to maintain consistent levels of desirable structural reliabilities or overcome the differences in such reliabilities [**8**, **12**]. A survey of the literature found very few studies on the development of the limit states design method for PC sleepers. A preliminary reliability assessment exercise for PC sleepers was discussed in reference [**12**].

A proposal to use the reliability-based approach in the conversion of the existing design code for PC sleepers to limit states design format is highlighted in this article. Experimental studies towards reliability concepts for the impact response and the ultimate resistance of PC sleepers are also presented. A case study of the reliability assessment of an Australian-manufactured PC sleeper is presented to evaluate the influence of dynamic load amplification on the target reliability indices and probabilities of failure. This article reinforces the fundamental limit states design guideline for PC sleepers to suit any local track and operational parameters.

2 PERMISSIBLE-STRESS DESIGN CONCEPT

A design methodology for PC sleepers is prescribed by Australian Standard AS1085.14-2003 [**4**]. The *life cycle* of the sleepers based on this standard is 50 years. The design method is based on the permissible or allowable stress of materials. A load factor is used to increase the static axle load to incorporate dynamic effects. The design load is termed 'combined quasi-static and dynamic load' which has a specified lower limit of 2.5 times the static wheel load. Using tables provided in AS1085.14, the load distribution to a single sleeper, railseat load, and the moments at railseat and centre can be obtained. A constraint of the dimensional design is the ballast pressure underneath sleepers, which shall not exceed 750 kPa for high-quality ballast, as described by AS2758.7 [**13**].

The limiting factors to be used for strength reduction of concrete and steel tendons at transfer and after losses can be found in the standard, ranging between 40 per cent and 60 per cent reduction. On the other hand, the minimum pre-camber stress in compression at any cross-section through the railseat area is set at 1 MPa after all losses (loaded only from prestressing). Generally, 25 per cent loss of prestress is to be assumed for preliminary design or when there are no test data. A lower level of 22 per cent loss has been generally found in the final design of certain types of sleepers ([**4**], Appendix E). The standard testing procedures in AS1085.14 have been suggested for capacity evaluation of a PC sleeper.

Past practice has proven that the use of this standard is adequate for bending strength design. AS1085.14 also prescribes that if a design is compliant, there is no need to check stresses other than flexural stresses, because the permissible-stress design concept limits the strengths of materials to comparatively low values compared with their true capacity. Under the design service actions, the material is kept in the elastic zone, so that there is no permanent set. In particular, sleepers that comply with AS1085.14 have all cross-sections of the sleeper fully in compression, under either pre-camber or design service loads. This method ensures that an *infinite* fatigue life is obtained and *no* cracking occurs. Sleepers designed on this basis thus have a potential reserve of strength within their whole life cycle under *normal* service loads.

In fact, impact forces due to wheel–rail interactions may subject the sleepers to dynamic loads that are much higher than the code-specified design forces. Large dynamic impact forces may initiate cracking in the PC sleepers; indeed, the experiment at the University of Wollongong (UoW) described below has proved that shear failure can also occur at or near the flexural limit. However, PC sleeper flexural failures have seldom been observed in railway tracks, showing the conservative nature of the current design process. To develop an ultimate limit states design approach, a study of the response of PC sleepers to high-magnitude short-duration loading is required. Crack control in sleepers (by Wakui and Okuda [**14**]) could also be included in the limit states design approach.

3 PROBABILISTIC DYNAMIC LOADS ON TRACKS

A maximum allowed impact force of 230 kN to be applied to the rail head by passing train wheels is commonly accepted through the Defined Interstate Network Code of Practice (Volume 5, Part 2, Section 8, 2002) [**15**]. Such impact loads may be caused by a variety of effects, including flats worn on the wheel tread, out-of-round wheels, defects in the wheel tread or in the rail head, or a derailment. The most severe impact forces are most likely from wheel flats [**8**], because such flats strike the rail head every revolution of the wheel, and severe flats have the potential to cause damage to track over many kilometres before detection. Despite the Code of Practice requirement, there are little published data showing the actual range and peak values of impact in the normal operation of trains, and certainly none were found for the defined interstate network. The value of 230 kN is therefore a desired upper limit rather than a measure of real maximum forces encountered on track.

A comprehensive investigation of actual impact forces was undertaken as a part of the Rail CRC project at Queensland University of Technology (QUT) [**8**]. Over 1 year, data were gathered from two Teknis Wheel Condition Monitoring stations located on different heavy-haul mineral lines. The loading data from a total of nearly six million passing wheels were measured, primarily from unit trains with 26- to 28-tonne axle loads, in both the full and empty states. An analysis of Leong's data from one of those sites is shown as a histogram in Fig. 2. The vertical axis shows the number of axles on a log scale, while the horizontal axis the measured impact force from the Teknis station. This impact force is the dynamic increment above the static wheel force (140 kN) exerted by the mass of the wagon on a wheel. Over 96 per cent of the wheels created impact forces less

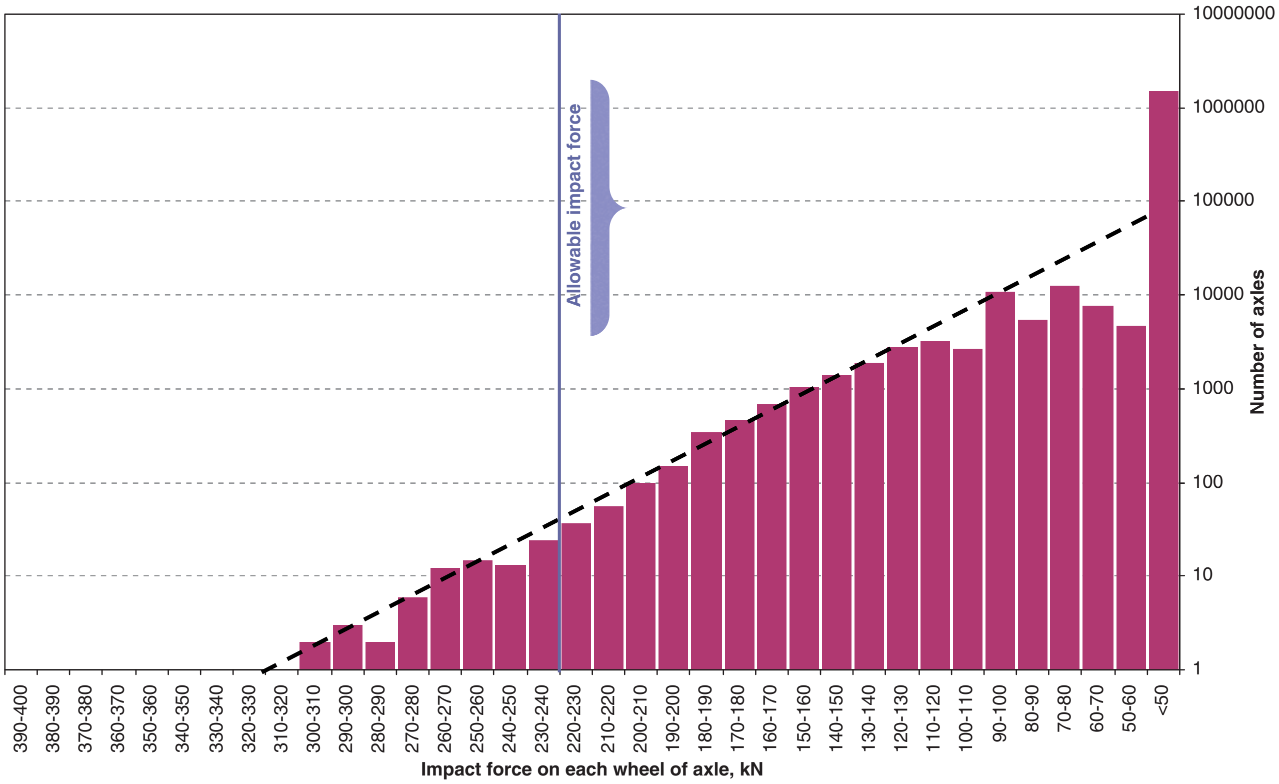

Fig. 2 Frequency of occurrence of impact forces per year

than 50 kN. The bulk of the graph in Fig. 2 therefore, is derived from only the remaining 4 per cent of wheels. However, that small percentage is still equivalent to more than 100 000 wheels throughout the year of the study, and they caused impact forces as high as 310 kN. The sloping dashed line in the plot represents a line of best fit to the data for these 100 000 wheel events.

In Fig. 2, the vertical line represents the Code of Practice maximum wheel impact force of 230 kN. Although the heavy-haul rail networks from which the data came are not part of the defined interstate network, it is clear that in normal operation, very large impact forces can occur and greatly exceed the Code of Practice specification. The frequency of high-impact wheel forces in the histogram columns of Fig. 2 lies along the sloping, dashed straight line, which means that the distribution would appear as a logarithmic curve on a graph with a linear scale on the vertical axis. In this case, the vertical axis in Fig. 2 is the number of impacting wheels per year, so if the rate of occurrence of such impacts over the year of the study is a representative of impacts over a longer period, then extrapolation of that sloping dashed line will provide an estimate of the frequency of occurrence of impact forces greater than the largest measured value of 310 kN.

On that basis, it could be predicted that an impact force of 380 kN would occur at the rate of 0.1 axles/year, or once in every 10 years; an impact of 450 kN would occur on an average once in every 100 years. This process naturally leads to the concept of a return period for impact force, which Murray and Leong [**7**] developed to produce equation (2)

$$\text{impactforce(kN)} = 53(5.8 + \log R) \tag{2}$$

where R is the return period in years of a given level of impact. It should be emphasized that these impact forces are applied by the wheel to the rail head. To determine the impact force applied to other components in the track structure, such as the sleeper or ballast, appropriate measures should be applied which allow for force sharing among support elements and also allow for the not-insignificant dynamic behaviour of the railway track. Equation (2) is utilized later in this article to help assess the probability of failure of PC sleepers in the heavy-haul lines which were monitored as part of this study. The effect of impact force characteristics and their roles on the PC sleepers were also demonstrated in reference [**5**]. The impulse duration plays a key role in amplifying the dynamic responses of the concrete sleeper at the corresponding resonance. The strain rate tends to increase dynamic stiffness of materials to some extent. However, the dynamic magnification factors can be up to double of the static response and cause the sleeper to crack [**5**].

4 CAPACITIES OF PC SLEEPERS

To evaluate the performance of PC sleepers under impact loads, an experimental programme was conducted at UoW. As a part of the collaborative research project supported by the Australian Cooperative Research Centre for Railway Engineering and Technologies (Rail-CRC), the PC sleepers were supplied by Australian manufacturers Rocla and Austrak. The sleepers were broad gauge (1.600 m) and standard gauge (1.435 m), both commonly used in heavy-haul coal lines [**16–18**]. A series of static tests on the PC sleepers were performed in accordance with the Australian Standards. A positive four-point bending moment test was conducted based on the assumption that the sleepers would behave similarly to those *in situ* [**4**]. It should be noted that the initial strain of prestressing wires is about 6.70 mm/m, and each prestressing wire has a specified minimum proof stress of 1860 MPa. The average compressive strength of cored concrete was 88 MPa. This value was adjusted according to AS1012.14 [**19**]. The details of static responses, rotational capacity, post-failure mechanisms, and residual load-carrying capacity of the PC sleepers under static loading can be found in references [**16–18**]. Figure 3 illustrates the setup for a static testing. A load cell was used to measure the applied load, while a linear variable differential transformer was installed at the midspan to obtain the corresponding deflections. Strain gauges were fixed to the top and bottom surfaces of the test sleeper and on both sides. The transducers were connected to a computer to record the experimental data.

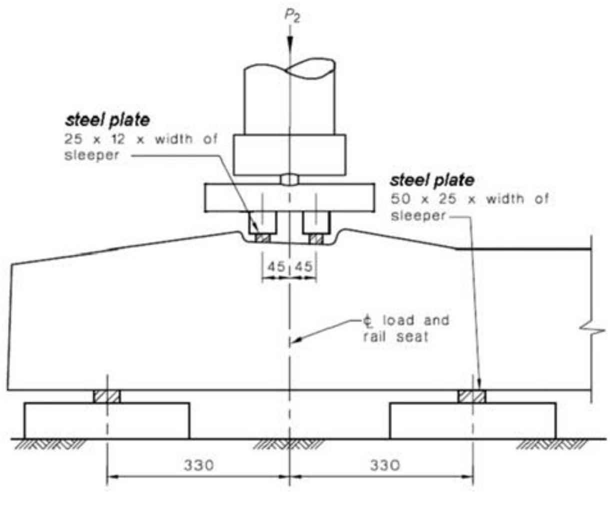

Fig. 3 Static test setup

The applied loading rate was a deflection control of 0.5 mm/min.

Figure 4 shows the new high-capacity drop-weight impact testing machine developed at UoW. To eliminate surrounding noise and ground vibration, the PC sleepers were placed on a strong shock-isolated concrete floor in the laboratory. Thick rubber mats were used to replicate the ballast support (static track spring rates: about 60 kN/mm for soft track and 135 kN/mm for hard track). It was found that the test setup could accurately represent the support conditions for PC sleepers found in typical track systems [**4**, **5**]. To apply impact loads, a drop hammer with a falling mass of 600 kg was used. The rail, with its fastening system for transferring the load to the specimens, was installed at the railseat. The drop hammer was hoisted mechanically to the required height and released. Impact load was recorded by the dynamic load cell.

Using a high-speed camera, the reliability of the drop hammer machine was first evaluated through calibration tests. It was found that the hammer's experimental velocity was about 98 per cent of the theoretical velocity. The experimental setup and the impact tests were arranged in accordance with the Australian Standards. The *in situ* conditions of PC sleepers were imitated, as shown in Fig. 4. A separate study was performed to simulate the impact loads recorded in tracks by means of the drop hammer machine and numerical impact simulations [**5**]. It is important to note that the experiments have contributed to the verification of numerical modelling and the simulations could also be used to evaluate the impact responses of other types of sleepers (e.g. differences in gauge, rail cant, topology, or shape), which is a better approach to the advanced analysis and design of PC sleepers. It should also be noted that the evaluations of bending and shear strengths of the PC sleepers are in accordance with Australian Standard AS3600 [**20**], which is the common analysis method for concrete structures. The ultimate bending capacity is based on the criteria whether the fibre stress in steel tendons approaches a yield point or the concrete strain reaches a crushing value under flexure. In terms of shear strength, it is determined by shear resistance of concrete section and steel reinforcement together with the contribution from prestressing force. The shear resistance takes into account the shear cracking and web crushing failures at the cross-section of the component.

Figure 5 shows a typical dynamic moment–deflection relationship at the railseat for PC sleepers. The crack initiation load was observed visually during each test as well as determined by the use of the load–deflection relationships. Figure 6 illustrates the crack propagation in a PC sleeper under static, monotonically increasing loading. The initial cracking moment was about 26 kN-m. The maximum

Fig. 4 New high-capacity drop-weight impact testing machine at the UoW

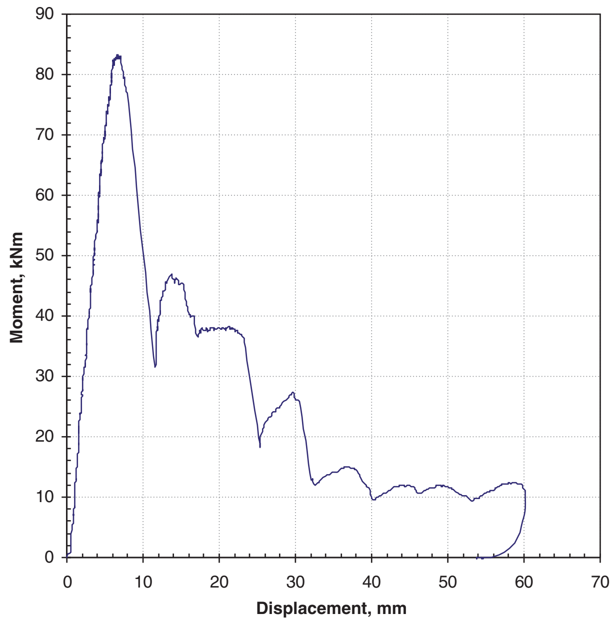

Fig. 5 Static moment–displacement relationship

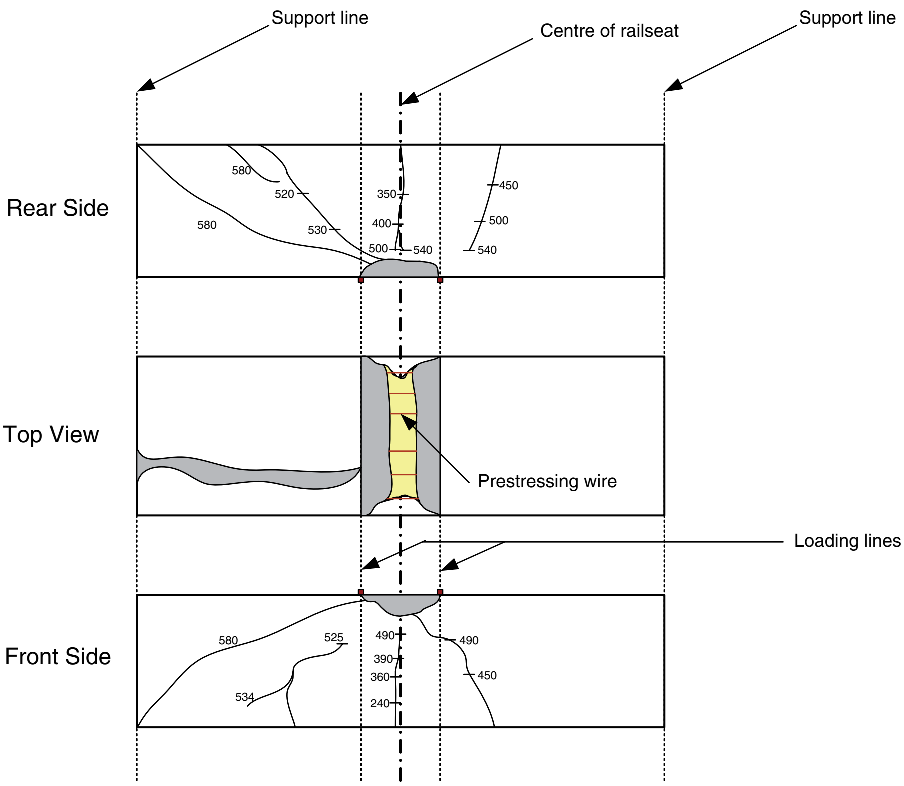

Fig. 6 Crack propagation of PC sleeper under static loading

static load capacity was about 583 kN, which is equivalent to the bending moment at the railseat of about 83 kN-m.

On the basis of the statistical data for the frequency of occurrence of impact loads and their magnitudes (Section 3), separate impact tests on PC sleepers were designed to simulate wheel–rail interface forces by varying the height of drop and the contact stiffness to achieve the desired magnitudes and durations of the load pulses. Figure 7 presents typical impact force–time histories measured by the dynamic load cell. Very small flexural cracks were initially detected starting from a drop height of 600 mm. Small shear cracks were also observed after several impacts from a drop height of 800 mm. However, no major failure could be observed in these single-impact load experiments [**21**].

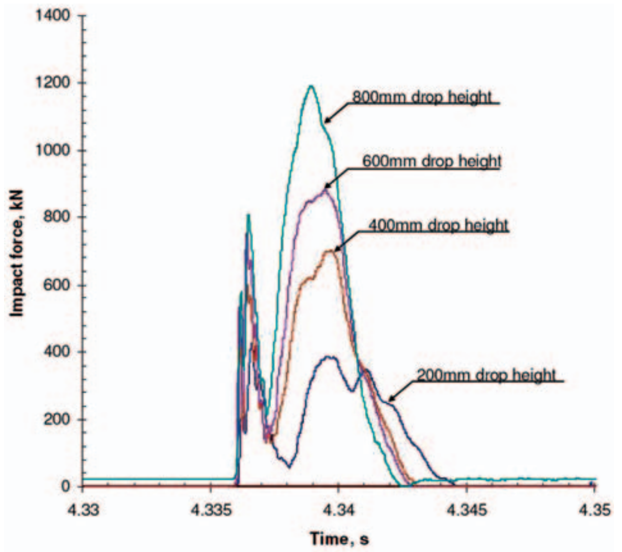

Fig. 7 Simulated impact forces

Later, the PC sleepers were also subjected to gradually increasing impact loads until they failed. The progressive impact behaviour of a PC sleeper in the soft track environment is presented in Fig. 8. The crack widths at each stage were measured using a magnifier telescope. The crack widths were about 0.01–0.02 mm for impact loads between 150 and 600 kN (Fig. 8(a)). When subjected to impact loads with magnitudes between 700 and 1000 kN, the crack widths increased from 0.02 to 0.08 mm (Fig. 8(b)). At this stage, spalling of the concrete at the top of railseat section could be detected, and the crack widths increased up to 0.5 mm when the impact forces were implemented up to 1500 kN (Fig. 8(c)). The ultimate impact load-carrying capacity was reached at about 1600 kN when the sleeper railseat section disintegrated. The failure mode was associated with both flexural and longitudinal splitting actions. The splitting fractures were aligned along

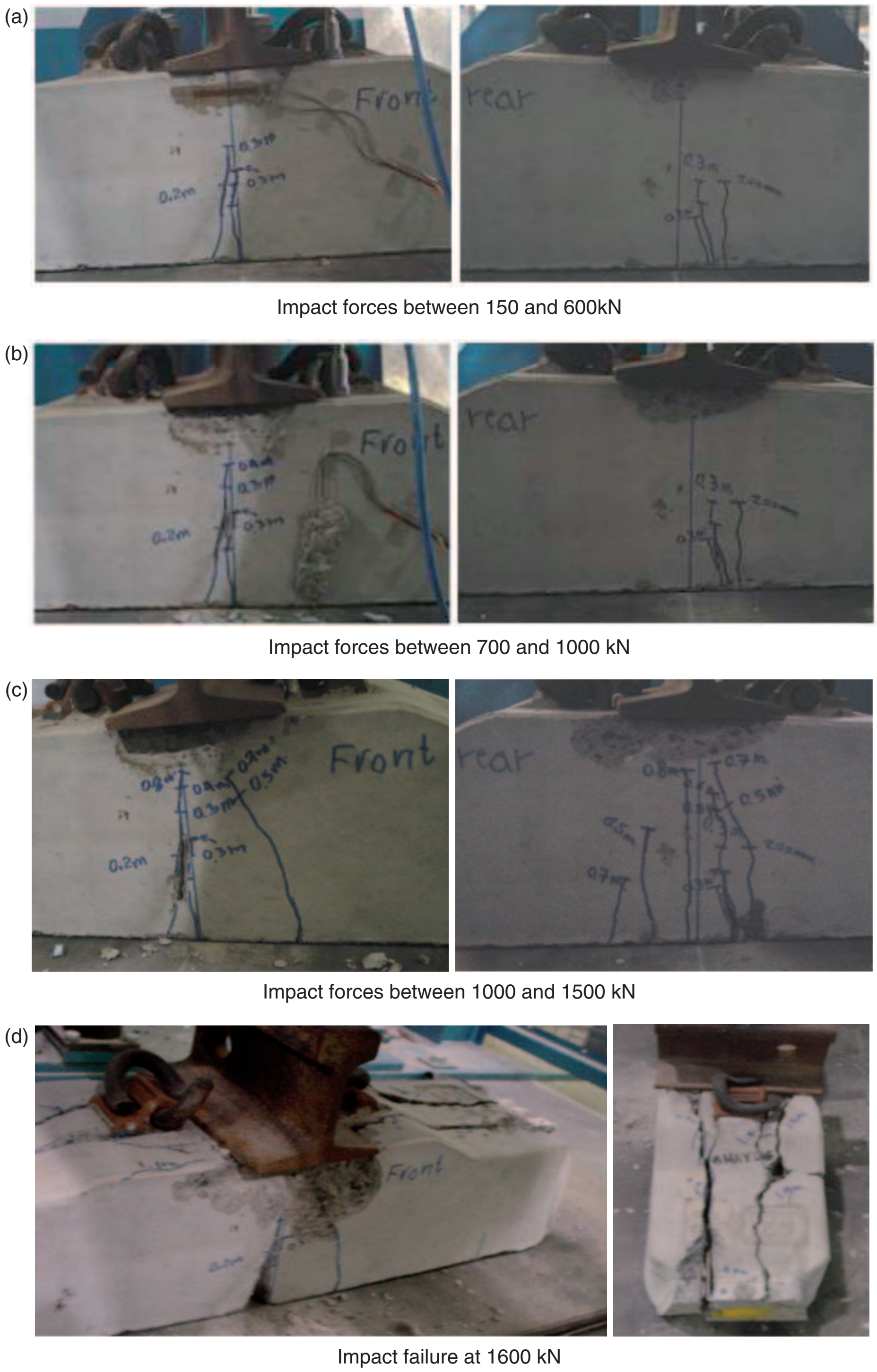

Fig. 8 Progressive impact response of a PC sleeper in a soft track environment [**22**]: (a) impact forces between 150 and 600 kN; (b) impact forces between 700 and 1000 kN; (c) impact forces between 1000 and 1500 kN; and (d) impact failure at 1600 kN

the prestressing tendons, as illustrated in Fig. 8(d). The probabilistic analysis of dynamic loading suggests that the magnitude of the ultimate impact load that caused failure of the PC sleeper would be equivalent to that with a return period of several million years.

5 STRUCTURAL RELIABILITY AND SAFETY CONCEPT

In principle, the errors and uncertainties involved in the estimation of the loading action and the capacity of a structure may be allowed for in strength design

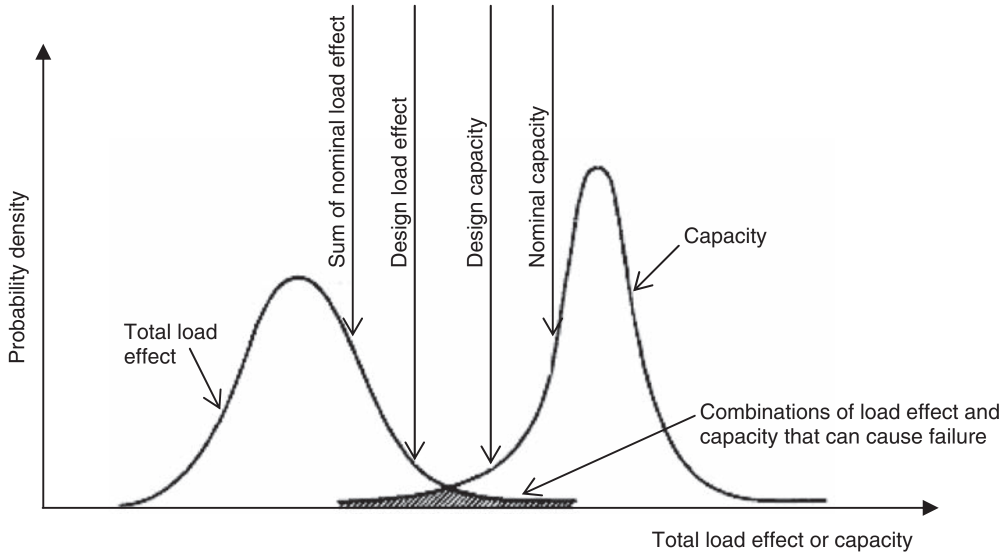

Fig. 9 Probability density functions for reliability [**23**]

using load factors to increase the nominal loads and using capacity factors to decrease the available structural strength. The purpose of using any factors is to ensure that the probability of failure under the most adverse conditions of structural overload remains very small, which may be implicit or explicit in the rules written in a code. In outdated structural design codes that employed the traditional *working stress design* (e.g. AS 1250-1981 Steel Structures [**23**]), and in the current AS1085.14 sleeper code, safe design was achieved using *factors of safety* to reduce the failure stress to permissible working stress values, but ultimately the purpose was to limit the likelihood of failure under *normal* services.

AS1085.14 prescribes the maximum allowed stresses, which are expressions of the ultimate strengths of isolated members divided by the safety factors (SFs). Thus

$$\text{working stress} \leq \text{permissible stress} \approx \text{ultimate stress/SF} \tag{3}$$

In Australia, all structural design codes except AS1085.14 have been amended to a limit state design approach. Limit state deems that the strength of a structure is satisfactory if its calculated *nominal capacity (resistance)*, reduced by an appropriate capacity factor ϕ, exceeds the sum of the nominal load effects multiplied by various load factors γ, so that

$$\Sigma(\gamma \times (\text{nominal load effects})) \leq \phi \times \text{nominal capacity} \tag{4}$$

or

$$\text{design load effect} \leq \text{design capacity} \tag{5}$$

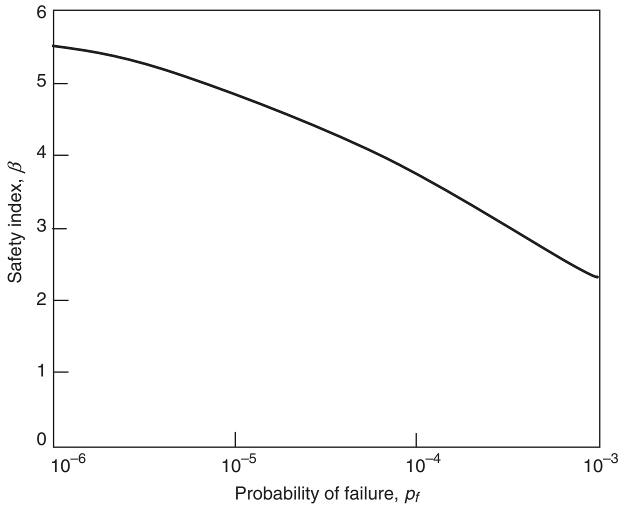

Fig. 10 Relationship between safety index and probability of failure [**24**]

where the nominal load effects are the appropriate bending moments, axial forces, or shear forces, determined from the nominal applied loads by an appropriate method of structural analysis (static or dynamic).

Even though the limit states are described in a deterministic form, the load and capacity factors involved are usually derived from *probabilistic models* based on statistical distributions of the loads and the capacities, as depicted in Fig. 9. The probability of failure p_F is indicated by the region for which the load distribution exceeds that for the structural capacity. In limit state codes, the probability of failure p_F is usually related to a parameter β, called the *safety index* or *reliability index*, by the transformation [**24**]

$$\Phi(-\beta) = p_F \quad (6)$$

where the function Φ is the cumulative frequency distribution function. The relationship between β and p_F shown in Fig. 10 indicates that an increase of 0.5 in β implies a decrease in the probability of failure by approximately one order of magnitude

6 CONVERSION OF CODES TO LIMIT STATES DESIGN FORMAT

The concept of a safety index should be used to ensure that the new code will lead to a satisfactory level of structural reliability and safety when the existing design code AS1085.14 is converted to a new limit states format. Safety indices could be obtained by first selecting typical PC sleepers that were designed according to the current working stress code. The safety indices of these sleepers would then be computed using idealized but realistic statistical models of their loads and structural capacities. These computed safety indices would be used to select target safety values for the limit state formulation. The load and capacity factors for the limit state design method would be varied until the target safety indices are achieved with reasonable precision. This generic procedure is called the *code calibration procedure*.

A classic example is the conversion of structural steel design methods in Australia. The calibration procedure and the safety indices β for ultimate limit state designs according to the new Australian limit state code AS 4100 Steel Structures [**25**] are compared in Fig. 11 with those of the previous working stress code AS 1250-1981 for steel beams and columns [**23**]. These comparisons show that the limit state formulations with a dead load factor of 1.25, live load factor of 1.5, and a capacity reduction factor of 0.9 offer designs with a reasonably consistent safety index in the range 3.0–3.5 compared with the working stress designs of steel beams and columns

An essential feature of the new limit states design format is that design criteria will be associated with specified limit states, and particularly with ultimate limit states such as structural collapse. Another feature of the new format is that the *design values* of resistance R^*, loads Q^*, and load effects S^* (such as for example, the bending moment at a railseat cross-section) are specified in terms of their *characteristic values* R_k, Q_k, and S_k and associated design coefficients ϕ, γ_Q, and γ_S as follows

$$R^* = \phi \times R_k \quad (7)$$

$$Q^* = \gamma_Q \times Q_k \quad (8)$$

$$S^* = \text{safety} \times \gamma_S \times S_k \quad (9)$$

In any typical case, extreme values such as the fifth and 95th percentile values (of distributions similar to Fig. 9) are selected for characteristic values in specifying design values for checks concerned with ultimate limit states, while average values are generally used in checks concerned with serviceability limit states. To convert AS1085.14 [**4**] to a new limit states format, it is proposed that the opportunity is taken to

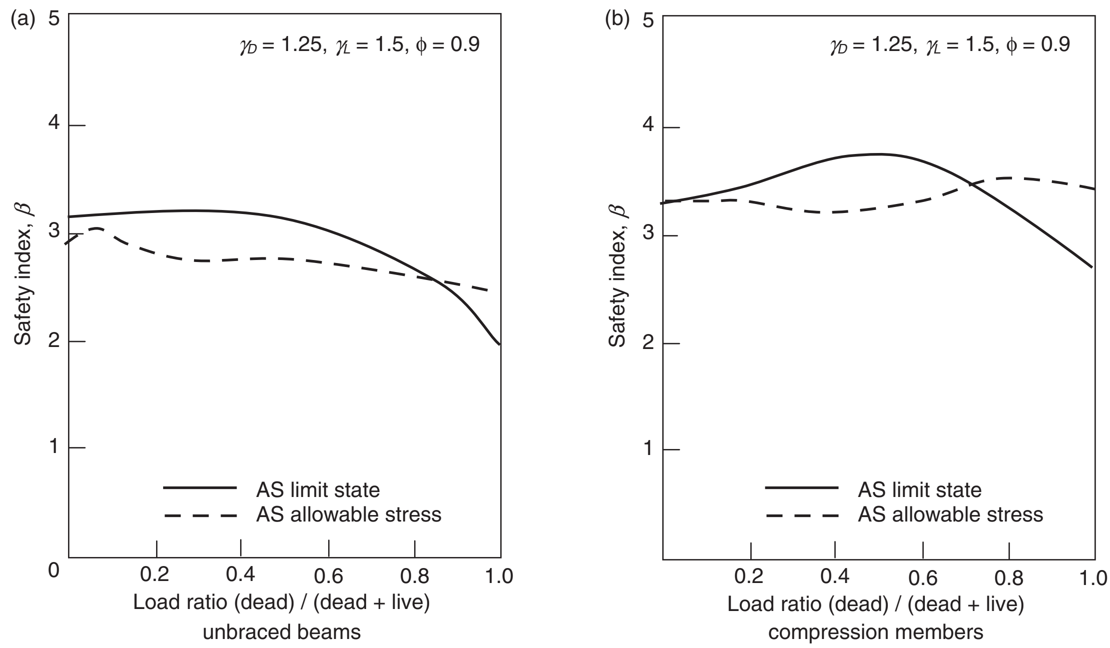

Fig. 11 Safety indices for steel beams and columns [**23**]: (a) unbraced beams and (b) compression members

ascertain the structural reliability and safety of both the existing and proposed PC sleeper codes to endeavour to maintain some specified consistency in structural reliability and safety in the formulation of the new design code, as demonstrated in the following paragraphs.

The *random variables* that will be denoted by X_1, X_2, ..., X_N represent the statistical characteristics of resistance and loads. For the simplified case where the parameters considered do not vary with time, the probability of failure is defined as

$$p_F = \text{Probability}\{g(X_1, X_2, \ldots, X_N) < 0\} \quad (10)$$

where $g(X_1, X_2, \ldots, X_N)$ may be a general function of the random variables $X_1, X_2, \ldots, X_N$ that represents the limit states equations for a selected structural member. If the statistical values of the random variables are known, equation (5) can be solved for the probability of failure using the methods of structural reliability analysis. From the relationship between β and p_F given in Fig. 10, the safety index could be determined.

Equation (6) and Fig. 10 lead to target numbers for the safety index that are convenient for evaluations of comparative safety of various engineering designs of PC sleepers. To summarize, the conversion of the existing design code to a new code written in limit states format should be implemented through a calibration procedure which could comprise the following steps.

1. Derive statistical models of structural resistances (concrete and prestressing steel), loads (e.g. impact loads at the wheel–rail interface), and load effects (e.g. bending moments at the railseat cross-section).
2. Using these models, safety indices could be evaluated for existing designs of PC sleepers according to the current code AS1085.14.
3. Using the values of safety indices obtained in Step 2, values of target safety indices could be chosen for the new limit states design code.
4. The load and resistance factors of the proposed new code (or method) could be selected so that the associated safety indices are close to the chosen target values.

The following schematic shows how research investigations could gather the essential information required for the calibration procedure.

This article proposes conceptual guidelines for the design method conversion, so that any structural design engineers can consider their operational and structural asset parameters as a part of the reliability and safety deliverables suitable to their customers.

7 CASE STUDY: RELIABILITY ASSESSMENT OF AUSTRAK BROAD-GAUGE SLEEPER

A preliminary study on the reliability analysis achieved preliminary target values. The 2700-mm long Australian-manufactured broad-gauge sleeper was originally designed for both metropolitan and country tracks with the following parameters:

(a) track gauge: 1600 mm;
(b) rail size: 53/60 kg;
(c) maximum axle load: 25 tonne;
(d) maximum train speed: 115 km/h;
(e) sleeper spacing: 685 mm; and
(f) design railseat load: 187 kN.

This sleeper was designed according to AS 1085.14 [**3**, **4**] to satisfy permissible stresses at transfer stage and at service stage. The sleeper design will be assessed using the reliability-based approach to calculate the safety index β. The limit state function $g(\mathbf{X})$ (equation (10)) with respect to permissible-stress criteria can be formulated as follows

$$g(\mathbf{X}) = \text{permissiblestress} - \text{fibrestress} \quad (11)$$

The railseat section is designed such that the extreme top and bottom fibres satisfy stress constraints, as prescribed by AS 1085.14 [**4**].

Concrete

At transfer: $f'_{cp} = 30$ MPa; $f_{ci} = 0.5$; $f'_{cp} = 15$ MPa; $f_{ti} = 0.25$; and $\sqrt{f'_{cp}} = 1.37$ MPa

At final stage: $f'_c = 55$ MPa; $f_c = 0.45 f'_c = 24.8$ MPa; $f_t = 0.4$; and $\sqrt{f'_c} = 2.97$ MPa

Prestressing steel

At transfer: $f_p = 1700$ MPa; $f_{pe@t} = 0.7$; and $f_p = 1190$ MPa

At final stage: $f_p = 1700$ MPa; $f_{pe@f} = 0.8$; and $f_p = 1360$ MPa

In general, the stresses at the top and bottom fibres (σ_t and σ_b, respectively) are

$$\sigma_t = -\frac{P}{A_g} \pm \frac{P \cdot e \cdot y_t}{I_g} \mp \frac{M \cdot y_t}{I_g} \quad (12)$$

$$\sigma_b = -\frac{P}{A_g} \mp \frac{P \cdot e \cdot y_b}{I_g} \mp \frac{M \cdot y_b}{I_g} \quad (13)$$

where P is the prestressing force, e the effective eccentricity, M the bending moment at the railseat, A_g the gross sectional area, I_g the gross moment of inertia of the cross-section, y_t the distance between

Table 1 Design results for the selected PC sleeper

Moment	Value of moment (kNm)	Location	Total stress (MPa)	Allowable stress (MPa)	Performance
M_R+	21.8	Top fibre	19.61	24.75	Functional
		Bottom fibre	−1.71	−2.97	Functional
M_R^-	14.6	Top fibre	−2.09	−2.97	Functional
		Bottom fibre	18.97	24.75	Functional

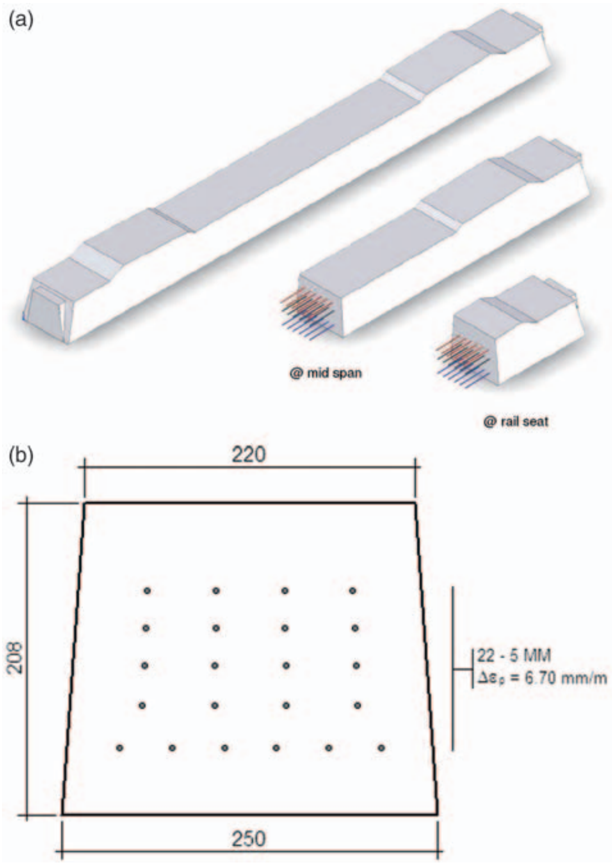

Fig. 12 Railseat section of a broad-gauge sleeper

the top fibre and the neutral axis of the cross-section, and y_b the distance between the bottom fibre and the neutral axis of the cross-section.

The current design procedure based on the QR PSC Design spreadsheet provides the designed railseat section, as shown in Fig. 12 with fibre stresses at each stage [**26**]. The design data are adopted from QR drawings.

The railseat load, R is defined: $R = j.Q$ (DF)/100 where j is the design load factor (2.5), Q the static wheel load (125 kN), and DF the axle load distribution factor (55 per cent for 600 mm spacing). For standard and broad-gauge sleepers, the positive moment at the railseat is $M_R^+ = R(L-g)/8$, while the negative moment $M_R^- = \max\{0.67M_R+, 14 \text{ MPa}\}$. The wheel load is 125 kN and the designed railseat load equal to 172 kN. Table 1 presents the sectional stresses of the Austrak broad-gauge sleeper at the final stage. It should be noted that the stresses σ_t and σ_b are calculated using equations (12) and (13). Detailed design criteria can be found in references [**4**, **5**].

Limit state functions for bending strength can be defined as

At the top fibre: $g_t(X) = \alpha_1\bar{\sigma}_t - \alpha_2\sigma_t$
At the bottom fibre: $g_b(X) = \alpha_1\bar{\sigma}_b - \alpha_2\sigma_b$

where $\bar{\sigma}_t$ and $\bar{\sigma}_b$ are the permissible stresses at the top and bottom fibres, respectively, at any stage (transfer/initial and final stages – f_{ci}, f_{ti}, f_c, f_t, $f_{pe@t}$, and $f_{pe@f}$) and α_1 and α_2 the model variation coefficients with respect to the resistance and the action, respectively.

The definition of a limit state function involves the use of appropriate strength models. These models should be realistic rather than code-based conservative approximations. In particular, the limit functions for M_R^+ of the railseat section at the final stage are

$$g_t(X) = \Theta_R(0.85f_c') - \Theta_S \left[\frac{0.76P}{A_g} - \frac{0.76P\cdot e\cdot y_t}{I_g} + \frac{R(L-g)}{8}\cdot\frac{y_t}{I_g}\right] \tag{14a}$$

$$g_b(X) = \Theta_R(0.6\sqrt{f'_c}) - \Theta_S \left[\frac{0.76P}{A_g} - \frac{0.76P\cdot e\cdot y_b}{I_g} - \frac{R(L-g)}{8}\cdot\frac{y_b}{I_g}\right] \tag{14b}$$

In equations (14), the compressive strength of concrete $R_c = 0.85f'_c$ and the characteristic flexural tensile strength $R_t = 0.6\sqrt{f'_c}$ are used as the permissible stresses to represent the material capacities more realistically [**20**]. Note that 0.76 is a coefficient that accounts for the prestressing losses (24 per cent) [**26**].

In addition, the limit functions for M_R^- of railseat section at the final stage are

$$g_t(X) = \Theta_R(0.4\sqrt{f'_c}) - \Theta_S \left[-\frac{0.76P}{A_g} + \frac{0.76P\cdot e\cdot y_t}{I_g} + \max\left\{\frac{0.67R(L-g)}{8}, 14\right\}\cdot\frac{y_t}{I_g}\right] \tag{15a}$$

$$g_b(X) = \Theta_R(0.45f_c') - \Theta_S \left[\frac{0.76P}{A_g} + \frac{0.76P\cdot e\cdot y_b}{I_g} + \max\left\{\frac{0.67R(L-g)}{8}, 14\right\}\cdot\frac{y_b}{I_g}\right] \tag{15b}$$

Table 2 Statistical model of the selected PC sleeper

Basic variables	Symbol	Distribution type	Units	Mean value	Standard deviation	Coefficient of variation
Loads						
Static wheel load	Qst	Log-normal	kN	125	31.25	0.25
Dynamic load factor	j	Log-normal		2.5	0.625	0.25
Axle load distribution factor	DF	Constant		0.55		
Resistances						
Permissible tension at transfer ($f_{cp} = 30$ MPa)	f_{ti}	Normal	MPa	1.37	0.246 6	0.18
Permissible compression at transfer ($f_{cp} = 30$ MPa)	f_{ci}	Normal	MPa	15.0	2.25	0.15
Permissible tension at service ($f_c = 55$ MPa)	f_t	Normal	MPa	2.97	0.534 6	0.18
Permissible compression at service ($f_c = 55$ MPa)	f_c	Normal	MPa	24.8	3.72	0.15
Concrete compressive strength	f'_c	Normal	MPa	66.0	9.9	0.15
Prestressing steel yield stress	f_p	Normal	MPa	1768	44.2	0.025
Area of prestressing steel	A_{ps}	Normal	m^2	432	5.4	0.012 5
Prestressing nominal force	P	Normal	kN	550.0	33	0.06
Sleeper dimensions						
Length	L	Constant	m	2.7		
Depth (railseat)	h	Constant	m	0.208		
Track parameters						
Track gauge	g	Constant	m	1.6		
Sleeper spacing	S	Constant	m	0.685		
Track stiffness	k_T	Constant	MN/m^2	100		
Railpad stiffness	k_P	Constant	MN/m^2	400		
Model uncertainties						
Uncertainty of resistance	Θ_R	Normal		0.99		0.06
Uncertainty of load effect	Θ_S	Normal		1.0		0.2

Note: Distribution patterns and coefficients of variation adopted from references [**25**, **27**, **28**].

Note that the coefficient 0.67 is used to calculate the negative railseat moment [**4**].

The railseat section is shown in Fig. 12. The basic random variables in this study are the permissible stresses at the transfer and final stages, the permissible prestressing force at the transfer and final stages, compressive and tensile strengths of concrete, allowable tensile stress of prestressing wires, sleeper dimensions (sectional area, width, length, and depth), effective eccentricity (e), area of prestressing wires, and model coefficients. The statistical properties of the basic random variables used in the reliability analysis of the selected PC sleeper are given in Table 2 [**26–29**]. These values have been adopted from previous studies, which developed and calibrated a structural design concept using the reliability approach [**5**, **12**]. (Note: these values could also be obtained from the field data from a location of operational interest for a localized application).

The reliability index β can be obtained using the stress limit functions:

(a) β_{ti} = reliability index for top fibre stress at initial stage;
(b) β_{bi} = reliability index for bottom fibre stress at initial stage;
(c) β_{tf} = reliability index for top fibre stress at final stage;
(d) β_{bf} = reliability index for bottom fibre stress at final stage;
(e) β_{wi} = reliability index for wire stress at initial stage;
(f) β_{wf} = reliability index for wire stress at final stage;
(g) β_{cf} = reliability index for cross-sectional stress at final stage,

where $\beta = \min\{\beta_{ti}, \beta_{bi}, \beta_{tf}, \beta_{bf}, \beta_{wi}, \beta_{wf}, \beta_{cf}\}$

In this example, only β_{tf} and β_{bf} will be determined. Five random variables include P, f'_c, Q, Θ_R, and Θ_S. Other parameters are treated as being deterministic in the reliability analyses

limit state function: $g(\mathbf{X}) = R - S$

Because the total design load acting on a sleeper includes both static and dynamic components, $Q = Q_{st} + Q_{dyn} = Q_{st}(1 + Q_{dyn}/Q_{st})$, the limit functions could be arranged such that the dynamic amplification factor Q_{dyn}/Q_{st} becomes the chief independent parameter with respect to which the reliability indices will be calculated.

Therefore, the limit functions for M_R^+ of railseat section at the final stage are

$$g_t(X) = \Theta_R(0.85f_c') - \Theta_S \left[\frac{P}{57671} - \frac{0.76P}{200\,363} + \frac{0.55 \times Q_{st}(1 + Q_{dyn}/Q_{st}) \times (2695 - 1680)}{8 \times 16\,57\,000}\right] \quad (16a)$$

$$g_b(X) = \Theta_R(0.6\sqrt{f'_c}) - \Theta_S \left[\frac{P}{57\,671} + \frac{0.76P}{209\,069} - \frac{0.55 \times Q_{st}(1 + Q_{dyn}/Q_{st}) \times (2695 - 1680)}{8 \times 17\,29\,000}\right] \quad (16b)$$

The limit functions for M_R- of railseat section at the final stage are

$$g_t(X) = \Theta_R(0.6\sqrt{f'_c}) - \Theta_S \left[\frac{P}{57\,671} - \frac{0.76P}{200\,363} - \frac{0.67 \times 0.55 \times Q_{st}(1 + Q_{dyn}/Q_{st}) \times (2695 - 1680)}{8 \times 16\,57\,000}\right] \quad (17a)$$

$$g_b(X) = \Theta_R(0.85f_c') - \Theta_S \left[\frac{P}{57\,671} + \frac{0.76P}{209\,069} + \frac{0.67 \times 0.55 \times Q_{st}(1 + Q_{dyn}/Q_{st}) \times (2695 - 1680)}{8 \times 17\,29\,000}\right] \quad (17b)$$

Using the structural reliability analysis program COMREL [**21**], the reliability indices can be calculated; the results are given in Table 3. The effect of the variation of the dynamic load factor on the reliability indices can be seen in Figs 13 and Fig 14. More results and the target reliability for limit state design of PC sleepers can be found in reference [**29**].

Table 3 Reliability indices of railseat section of selected PC sleeper

Moment	Reliability index	FORM[a]	Probability of failure, p_F
M_R+	β_{tf} (top fibre stress at final stage)	3.829	6.43E−5
	β_{bf} (bottom fibre stress at final stage)	1.872	3.06E−2
M_R-	β_{tf} (top fibre stress at final stage)	2.692	3.55E−3
	β_{bf} (bottom fibre stress at final stage)	3.998	3.19E−5

[a]First-order reliability method.

8 LIMIT STATES DESIGN CONCEPT FOR PC SLEEPERS

8.1 Definition of a 'failed' sleeper

Australian railway organizations commonly condemn a sleeper when its ability to hold top of line or gauge is lost [**9**]. These two failure conditions can be reached by the following actions:

(a) abrasion at the bottom of the sleeper causing loss of top;
(b) abrasion at the railseat causing loss of top;

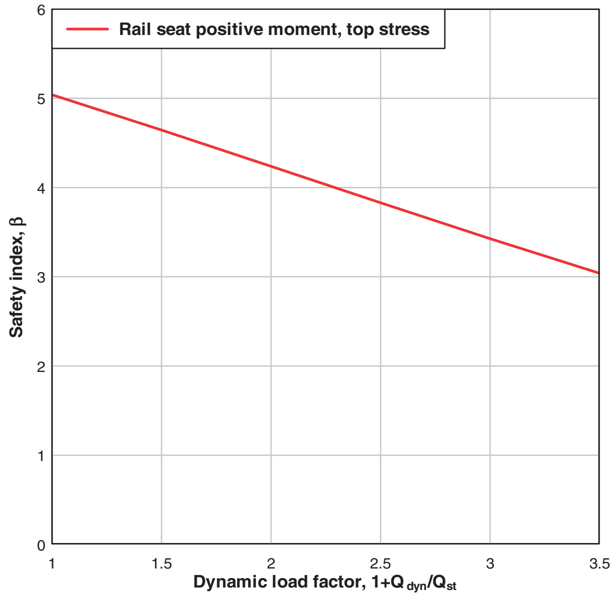

Fig. 13 Safety indices (railseat positive moment)

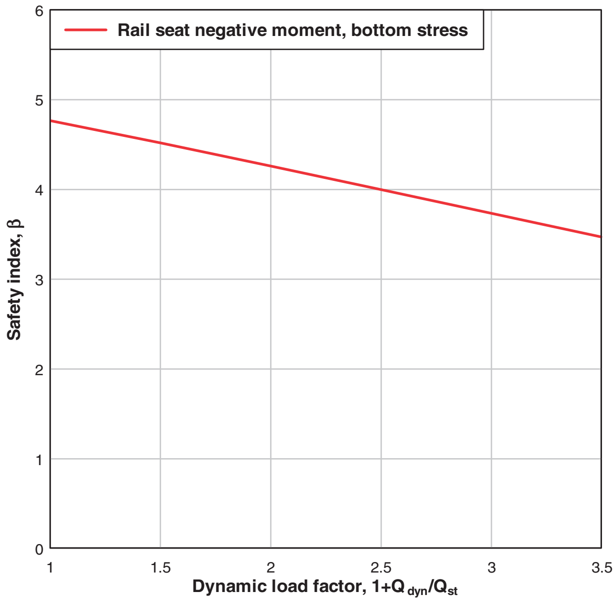

Fig. 14 Safety indices (railseat negative moment)

(c) severe cracks at the railseat causing the 'anchor' of the fastening system to move and spread the gauge;
(d) severe cracks at the midspan of the sleeper causing the sleeper to flex and spread the gauge;
(e) severe degradation of the PC sleeper due to alkali aggregate reaction or some similar degradation of the concrete material.

Only severe cracking leading to the sleeper's inability to hold top of line and gauge will be considered here as the failure conditions defining a limit state related to the operations of a railway system, because abrasion and alkali aggregate reaction are not structural actions causing failure conditions.

8.2 Limit states for PC sleepers

The acceptance of the structural performance under design load conditions is a major challenge in the development of a limit states design concept for PC sleepers. Infinite fatigue life of sleepers *cannot* be retained after allowing cracks under impact loads. The degree of reliability is also an important factor that must be considered. The Australian Standard AS 5104-2005 gives the general principles on reliability for structures [**24**]. According to AS 5104-2005, limit states can be divided into the following two categories.

(a) ultimate limit states, which correspond to the maximum load-carrying capacity or, in some cases, to the maximum applicable strain or deformation;
(b) serviceability limit state, which concerns normal use.

Note that for PC sleepers, the limit state categories could be different from the traditional structural approach and should take into consideration the track's ability to continue operating in the event of exceeding a limit state. Therefore, the following three limiting conditions have been proposed for the design of PC sleepers [**4–7**, **9**].

1. *Ultimate limit state:* A single one-off event such as a severe wheel flat that generates an impulsive load capable of failing a single PC sleeper. Failure under such a severe event would fit within failure definitions causing severe cracking at the railseat or at the midspan.
2. *Damageability (or fatigue) limit state:* A time-dependent limit state in which a single PC sleeper accumulates damage progressively over a period of years to the point where it is considered to have reached failure. Such a failure could come about from excessive accumulated abrasion or from progressively more severe cracking under repeated loading impact forces over the sleeper's lifetime.
3. *Serviceability limit state:* A condition in which progressive sleeper failure begins to impose some restrictions on the operational capacity of the track. The failure of a single sleeper is rarely if ever a cause of a speed restriction or a line closure. However, when there is a failure of a cluster of sleepers, an operational restriction is usually applied until the problem is rectified.

It has been reported that the majority of structural failure modes are associated with ultimate impacts [**9**]. The failure in damageability and serviceability limit states is generally caused by other degraded components, e.g. softened formation, pulverized ballast, or poor drainage. For factorized reliability design, the ultimate limit state for a single PC sleeper is considered in the development of the reliability-based design procedure for PC sleepers. An experimental programme has also been developed at UoW to characterize the uncertainties of the calculation models for the resistances of PC sleepers in the ultimate limit state. Other serviceability limit states are not the scope of this article. (Note: serviceability is dealt to a great extent by fatigue life calculation [**30**, **31**]).

8.3 Reliability analysis of PC sleepers for ultimate limit state

The limit state equation in *the partial factors format* is given by

$$S^* < \phi R_u \tag{18}$$

where S^* is the design action effect due to the factored design loads and φR_u the factored resistance capacity of the actual member

A reliability model for the ultimate limit states and the relationship between loading (S) and resistance (R) can be illustrated using the probability functions shown in Fig. 15. The design values of resistance and load effect in the new limit states design code are calculated using the characteristic resistance R_k and the characteristic load effects (e.g. sleeper bending moment) S_k which should be determined from statistical analyses of wheel load distributions and the experimental results on impact resistance of PC sleepers.

As an example of the statistical models required for the reliability-based code conversion procedure, Fig. 15 presents the probability density function of the wheel impact loads obtained by curve-fitting the data from a QR wheel impact load detector (WILD) impact detector [**26–29**]. It is apparent that one of

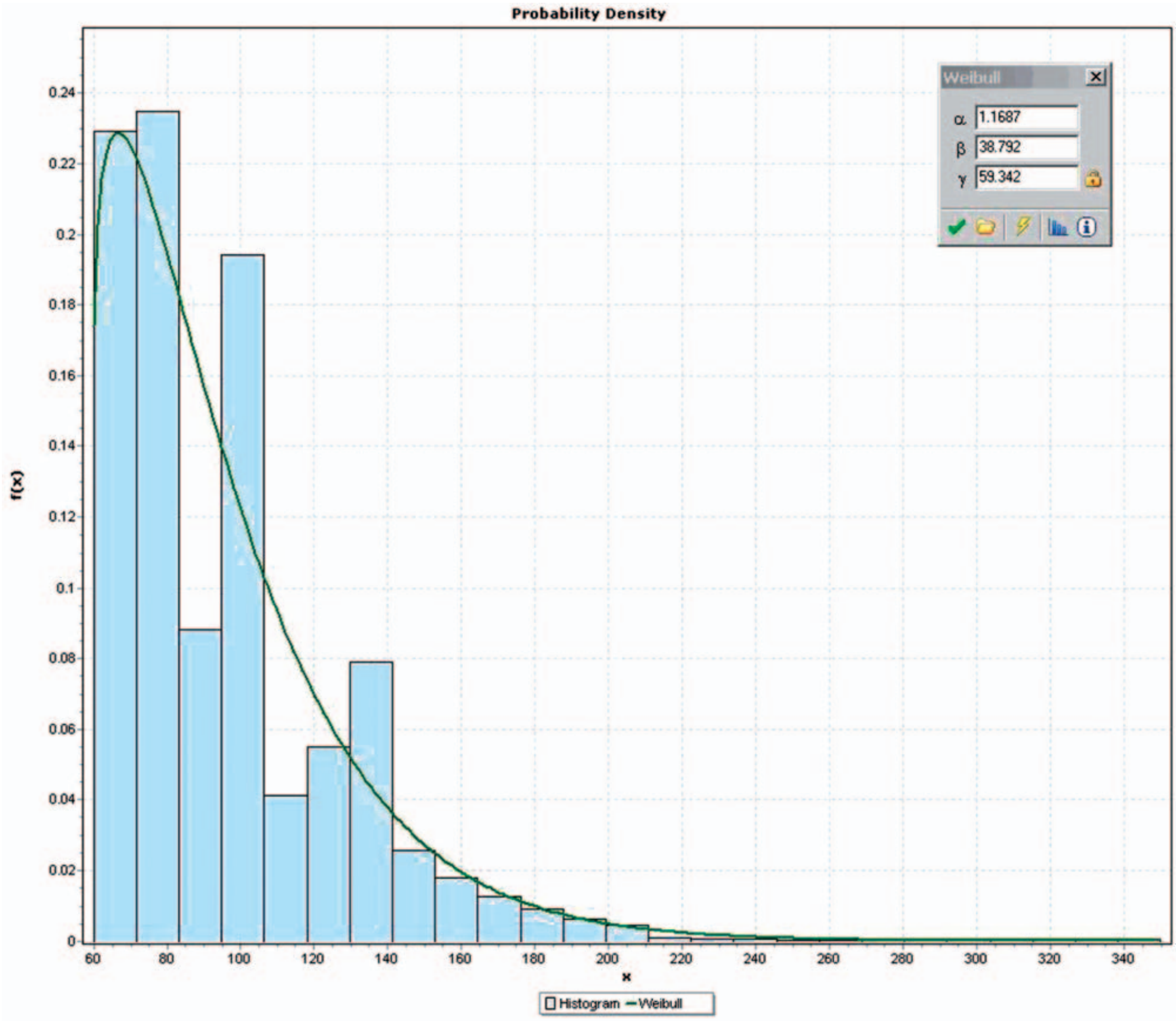

Fig. 15 Probability density function of wheel impact load

the standard statistical distributions (e.g. normal, log-normal, and Weibull) could be used to fit the data sets representing the loads and the resistance of PC sleepers so that the statistical models of PC sleepers could be formulated and analysed using methods of reliability analysis. In this case, the data were best fit using a Weibull distribution.

If the ultimate limit state for a PC sleeper is associated with the bending failure, equation (18) could be defined as

$$M^{*} \leq \varphi M_{u} \tag{19}$$

where the ultimate moment capacity, M_u is given by AS 3600 [**20**], and M^{*} the design bending moment due to the design static wheel load combined with the design impact wheel load caused by wheel irregularities (e.g. wheel flats). Equation (19) can be represented in the reliability analysis format by the following limit state function

$$g_{\mathrm{ult}}(\mathbf{X}) = \Theta_{\mathrm{R}} M_{\mathrm{u}} - \Theta_{\mathrm{S}} \times \text{applied moment} \tag{20}$$

where M_{u} is the random variable that could be expressed as a function of the basic random variables (Table 1) describing the ultimate resistance of the selected cross-section. The sleeper's *applied moment* is the random variable relating to the design wheel impact load; it is described by a probability curve of flexural moments in the sleeper; Θ_{R} and Θ_{S} the model's uncertainty coefficients [**29**].

A method has been proposed by which the ultimate limit state wheel–rail impact design forces may be calculated based on data collected from wayside WILDs installed on a heavy-haul coal line [**9**]. However, the problem with redistributing the design wheel–rail force to the design sleeper moment is still open for discussion. Accordingly, Murray and Leong [7] emphasized the need for computer dynamic track analysis using a package such as DTRACK to compute the dynamic design sleeper moment. While in principle, this method could be viable, this would lead in practice to complications in formulating statistical ultimate limit state models of PC sleepers for their reliability and safety assessment and for the model calibration in the conversion process to a new limit states design code format. In that case, equation (20) can be rewritten as

$$g_{\text{ult}}(\mathbf{X}) = \Theta_R M_u - \Theta_S \times \text{applied moment} \quad (21)$$

where M is to be determined from computer analysis. It was found that the design sleeper moment does not have an analytical representation or simplification, so equation (21) in practice cannot be solved to find the safety indices β. There is therefore a need to carry out an experimental investigation of the relationship between impact wheel load and the resulting bending moments to establish a simplified analytical expression that could be incorporated in the limit state functions such as equation (20) for conducting the reliability assessment studies on PC sleepers.

9 SIMPLIFIED RELATIONSHIP BETWEEN IMPACT LOAD AND SLEEPER BENDING MOMENTS

A series of tests based on the information about loading conditions on railway tracks [2] to verify the relationship between wheel impact loads and sleeper bending moments at the railseat position are required, allowing for the variety of rail pads and varying stiffness of the ballast support. To establish the relationship between railseat bending moment and the associated impact force, both numerical and experimental studies into the impact behaviours of PC sleepers were carried out [22]. The effect of impact force characteristics has also been studied [32]. The impact tests were identical for both support conditions: light and heavy tracks. The numerical and experimental relationships between the railseat bending moment (M^*) and impact force acting directly on the railseat (F) is shown in Fig. 16. The simplified relationship between the artificial impacts and railseat moment envelope for an initial design guideline for PC sleepers is

$$M^* = 0.08F \quad (22)$$

It is important to note that the best way to determine the bending moment along the sleepers is to employ a package that provides advanced dynamic analysis of railway tracks (i.e. D-TRACK). Numerical

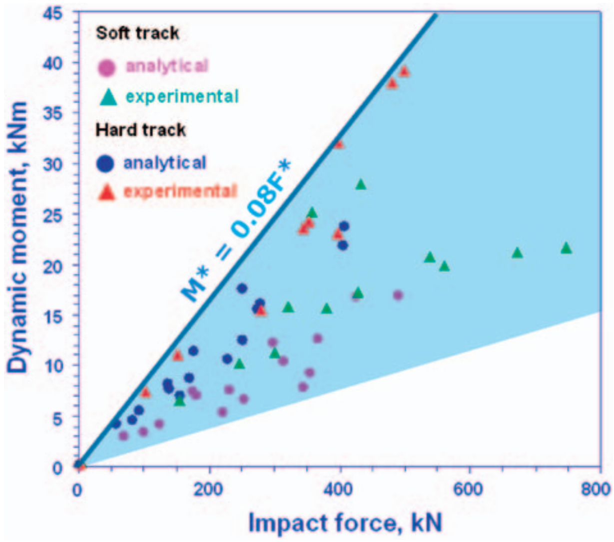

Fig. 16 Relationship between railseat bending moment and impact force

modelling of PC sleepers will help to optimize the design and reduce material waste. The design guideline presented is convenient but conservative, and would be preferable in railway practice as the analytical and experimental results in this study confirm. It would also provide faster yet adequate means to predict the bending moment on the sleepers from the anticipated wheel–rail interaction. A designer could also use a mean value of the coefficient and an estimate of the scatter of the relevant observed data about the predicted value in Fig. 16 for a more stringent reliability analysis. Note that the impact force on the sleeper railseat varies from about 50 per cent to about 70 per cent of the wheel–rail interaction force [29, 31] and the relationship between centre and railseat bending moments can be obtained *via* a dynamic analysis or an empirical method in accordance with AS1085.14 [4, 31, 33–34].

10 CONCLUSIONS

The permissible-stress design concept has long been used and is still specified in the current design of PC sleepers in AS1085.14. The design process relies on quasi-static wheel loads and the static response of the sleepers. Practical experience and scientific experiments have proven that PC sleepers possess significant untapped reserve capacity. To shift to a more rational design method requires significant research effort, which is being conducted within the framework of the CRC for Railway Engineering and Technologies. The collaborative project between UoW and QUT has considered all important aspects such as the spectrum and amplitudes of dynamic forces applied to the railway track, the true reserve capacity of typical PC sleepers, the impact behaviour of the sleepers, and the development of a new limit states design concept. This article presents the background information and focuses on some research outcomes of the Rail-CRC research project aimed at developing the new limit states design concept for PC sleepers. It is aimed at guiding design engineers to appropriate methods of reliability-based design for performance and to insights into the implication of limit states for PC sleepers.

This article proposes the reliability concepts and rationales associated with the development of limit states format codes and the practical issues associated with the conversion of AS 1085.14 to a limit states design format. The use of a reliability-based approach in the conversion of the existing code to the new limit states format has also been demonstrated as a case study. It reinforces the fundamental design guideline for PC sleepers to optimally suit any local track and operational parameters. The target reliability indices β to be used for the code calibration can be obtained from the reliability analysis of existing design procedures and the new method to design the PC sleepers. The demonstration provides design flexibility and choices to engineers, and the design guidelines in this article will enhance the reliability and safety of the track component. Sleeper manufacturers could apply the principle to their product designs; so, they are suitable and optimal to local train/track conditions as well as operational, structural, and environmental parameters.

FUNDING

This work was supported by Australian Cooperative Research Centre for Rail Engineering and Technologies – RailCRC [Grant Number 5/23].

ACKNOWLEDGEMENTS

This research project was funded by the Australian Cooperative Research Centre for Railway Engineering and Technologies (Rail-CRC) as part of Project 5/23. The support from industry partners Queensland Rail, RailCorp, Austrak and Rocla is gratefully acknowledged. The authors thank Alan Grant, Ian Bridge, Bob Roland, and Jason Knust for their technical assistance during the course of this project. The third author would also like to sincerely thank the Australian Government for an Endeavour Executive Award of a visiting fellowship at the Railway Technical Research Institute in Tokyo, Japan.

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APPENDIX

Notation

$g(X_N)$	general limit function of the random variable X_N
$g_{ult}(\mathbf{X})$	limit function of the random variable X at ultimate conditions
M^*	design bending moment
M_u	ultimate moment capacity (characteristic)
p_F	probability of failure
P_0	static wheel load
P_D	design wheel load
Q^*	design value of loads
Q_k	characteristic value of loads
R^*	design value of resistance
R_k	characteristic value of resistance
S^*	design load effects
S_k	characteristic load effects
SF	factors of safety
X_N	random variables
β	safety index or reliability index
γ	load factors
γ_Q	design coefficient for loads
γ_S	design coefficient for load effect
Θ_R	model uncertainty coefficient of resistance
Θ_S	model uncertainty coefficient of load effect
ϕ	dynamic impact factor, capacity factor, design coefficient for resistance
Φ	cumulative frequency distribution function