Conversion of AS1085.14 for prestressed concrete sleepers to limit states design format

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CONVERSION OF AS1085.14 FOR PRESTRESSED CONCRETE SLEEPERS TO LIMIT STATES DESIGN FORMAT

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Summary:  
Premature cracking of prestressed concrete sleepers has been found in railway tracks. The major cause of cracking is the infrequent but high-magnitude wheel loads produced by a small percentage of “bad” wheels or rail head surface defects which are crudely accounted for in AS 1085.14 by a single load factor. The current design philosophy, outlined in AS 1085.14, is based on assessment of permissible stresses resulting from quasi-static wheel loads and essentially the static response of concrete sleepers. In order to shift the conventional methodology to a more rational design method that involves more realistic dynamic response of concrete sleepers and performance-based design methodology, a significant research effort within the framework of the CRC for Railway Engineering and Technologies is currently underway to perform comprehensive studies of the loading conditions, the dynamic response, and the dynamic resistance of prestressed concrete sleepers.

The collaborative research between the University of Wollongong (UoW) and Queensland University of Technology (QUT) has addressed such important issues as the spectrum and amplitudes of dynamic forces applied to the railway track, evaluation of the reserve capacity of typical prestressed concrete sleepers designed to the current code AS 1085.14, and the development of a new limit states design concept.

This paper presents the results of the extensive investigations at UoW and QUT aimed at predicting wheel impact loads at different return periods (based on the field data from impact detectors) together with an experimental investigation of the ultimate impact resistance of prestressed concrete sleepers required by a limit states design approach. The paper also describes the reliability concepts and rationales associated with the development of limit states format codes and the issues pertaining to conversion of AS 1085.14 to a limit states design format.

Keywords: Prestressed concrete sleepers, design code, limit states, performance based design, reliability, probabilistic analysis, impact loading, Australian Standard AS1085.14

1.0 Introduction

Railway is the world’s safest transportation system for either passengers or merchandise across distant areas. Track structures guide and facilitate the safe, cost-effective, and smooth ride of trains. Figure 1 shows the main components constituting typical ballasted railway track [1]. Its components can be subdivided into the two main groups: superstructure and substructure. The most obvious components of the track such as the rails, rail pads, concrete sleepers, and fastening systems form a group that is referred to as the superstructure. The
substructure is associated with a geotechnical system consisting of ballast, sub-ballast and subgrade (formation) [2]. Both superstructure and substructure are mutually vital in ensuring the safety and comfort of passengers and a satisfactory quality of ride for passenger and freight trains. Note that in Australia, UK, 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 US and Canada. This paper will adopt the former term hereafter.

The main functions of sleepers are to: (1) transfer and distribute loads from the rail foot to underlying ballast bed; (2) hold the rails at the proper gauge through the rail fastening system; (3) maintain rail inclination; and (4) restrain longitudinal, lateral and vertical movements of the rails [2]. Typical load conditions on railway track structures have been presented previously in [3] while common design procedures for Australian railway tracks in [4]. The permissible stress design approach makes use of an empirical function taking into account the static wheel load \( P_0 \) with a dynamic impact factor \( \phi \) to account for dynamic vehicle/track interactions:

\[
P_D = \phi P_0
\]

where \( P_D \) is the design wheel load, \( P_0 \) is the quasi-static wheel load, and \( \phi \) is the dynamic impact factor (>1.0).

Significant research attention has been devoted to the forces arising from vertical interaction of train and track as these forces are the main cause of railway track problems when trains are operated at high speed and with heavy axle loads. It has been found 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. The summary of typical impact loadings due to train and track vertical interaction has been presented in [3] with particular reference to the shape, magnitude and duration of impact loads found in railway track structures.

Current Australian and international design standards for prestressed concrete (PC) sleepers are based on the permissible stress concept where various limiting values or reduction factors are applied to material strengths and load effects [3-5]. Empirical data
collected by railway organisations suggests that railway tracks, especially railway PC sleepers, might have untapped strength that could bring potential economic advantage to track owners. The permissible stress design approach does not consider the ultimate strength of materials, probabilities of actual loads, risks associated with failure, and other factors which could lead to overdesigning the PC sleepers. A research programme to investigate the actual load carrying capacity of PC sleepers was initiated as a collaborative project between UoW, QUT and the industry partners (QR, RailCorp, Austrak, Rocla) within the framework of the Australian Cooperative Research Centre for Railway Engineering and Technologies (Rail-CRC). The main objective was the conversion of the existing Australian design code for PC sleepers into limit states design format, in order to account for the statistical nature, probability and risk of failure.

Murray and Leong [6,7] proposed a limit states design concept and load factors for a revamped standard AS1085.14. The expressions for predicting the impact loads at different return periods (based on field data from impact detectors at two sites) were proposed. It was suggested that a simple pseudo-static (using factored load) approach can be used in the design procedures of PC sleepers under routine traffic. For concrete sleepers under non-routine traffic, a dynamic analysis was suggested as part of a design process. The research team of the Rail-CRC Project has undertaken statistical, probabilistic and experimental studies to investigate the ultimate resistance of the PC sleepers in a manner required by a limit states design approach [8-10].

In addition to experimental investigations in this project, conversion of the existing design standard into new limit states design format required a comparative examination of the safety margin and probability of failure of PC sleepers designed in accordance with both permissible stress and limit states provisions. It is well known that the performance of structural systems depends on the weakest element with lowest reliability [11]. To achieve uniform performance and reliability in structural designs for different design principles, the reliability-based approach is the most suitable, in order to either maintain consistent levels of desirable structural reliabilities or overcome the differences of such reliabilities [12]. From a review of the literature, very few studies were found devoted to the development of the limit states design method for PC sleepers. A preliminary reliability assessment exercise for PC sleepers has been discussed in [12].

The present paper proposes the use of reliability-based approach in the conversion of the existing design code for PC sleepers to limit states design format. Experimental results complementing the reliability concepts for the impulsive response and ultimate resistance of PC sleepers are also presented in this paper. An example 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.

2.0 Current standard: AS1085.14-2003

Australian Standard AS1085.14-2003 [4] prescribes a design methodology for PC sleepers. The life cycle of the sleepers based on this standard is 50 years. The design process relies 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 static wheel load. Load distribution to a single sleeper, rail seat load, and moments at rail seat and centre can be obtained using tables provided in AS1085.14. It should be noted that the ballast pressure underneath sleepers is not permitted to exceed 750 kPa for high-quality ballast as described by AS2758.7 [13].

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% to 60% reduction. However, the minimum pre-camber compressive stress at any cross-section through the rail seat area is set at 1 MPa after all losses (loaded only from prestress). It should be noted that 25% loss of prestress is to be assumed for preliminary design or when there is no test data. A lower level of 22% loss has been generally found in final design of certain types of sleepers (see details in ref. [4], Appendix E). The standard testing procedures in AS1085.14 have been
recommended for strength evaluation of PC sleepers. Past practice has indicated that utilisation of this standard is adequate for flexural strength design. AS1085.14 states that if the design complies with AS1085.14, there is no need for consideration to checking stresses other than flexural stresses, because the permissible stress design concept limits the strengths of materials to relatively low values compared to their true capacity. Under the design loads, the material is kept in the elastic zone so there is no permanent set. In particular, sleepers that comply with AS1085.14 have all cross sections of the sleepers fully in compression, under either pre-camber or design service loads. This approach ensures that an infinite fatigue life is obtained and no cracking occurs. Sleepers designed in this way therefore have a significant reserve of strength within their 50 year life cycle under normal service loads. In reality, impact forces due to wheel/rail interactions may subject the sleepers to dynamic loads that are much larger than the code-specified design forces. Large dynamic impact forces may initiate cracking in the concrete sleepers; indeed, testing at UoW has shown shear failure can also occur at or near the flexural limit. However, concrete sleeper flexural failures have rarely been observed in railway tracks, showing the conservative nature of the existing design process. To develop an ultimate limit states design approach, a study of the response of concrete sleepers to high-magnitude short-duration loading is required. The earlier proposal of allowing cracks in sleepers (by Wakui & Okuda [14]) could also be considered in a limit states design approach.

3.0 Statistics of dynamic loads on tracks

The Defined Interstate Network Code of Practice (Volume 5, Part 2 - Section 8, 2002) [15] prescribes a maximum allowed impact force of 230kN to be applied to the rail head by passing train wheels. That impact may come about from a variety of effects, including flats worn on the wheel tread, out-of-round wheels, and defects in the wheel tread or in the rail head. Leong [8] showed that the largest impact forces are most likely from wheel flats; because such flats strike the rail head every revolution of the wheel, severe flats have the potential to cause damage to track over many kilometres.

Despite the Code of Practice requirement, there is little published data able to be found showing the actual range and peak values of impact forces. Figure 2 shows the frequency of occurrence of impact forces, derived from [8].
normal operation of trains, and certainly none were found for the defined interstate network. The value of 230kN 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 by Leong [8] as part of the Rail CRC project at QUT. Over a 12 month period he gathered data from two Teknis Wheel Condition Monitoring stations located on different heavy haul mineral lines. The forces from a total of nearly 6 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 Figure 2. The vertical axis shows the number of axles on a log scale, while on the horizontal axis is the measured impact force from the Teknis station. Note that the impact force in Figure 2 is the dynamic increment above the static force exerted by the mass of the wagon on a wheel. Over 96% of the wheels created impact forces less than 50kN. The bulk of the graph in Figure 2 therefore, is comprised of only the remaining 4% of wheels. However, that small percentage still comprised over 100,000 wheels throughout the year of the study, and they caused impact forces as high as 310kN. The sloping dashed line in the graph represents a line of best fit to the data for these 100,000 wheel forces.

The vertical dotted line in Figure 2 represents the Code of Practice maximum impact force of 230kN – even though the heavy haul lines from which the data came are not part of the defined interstate network, it’s clear that in normal operation very large impact forces can occur which greatly exceed the Code of Practice specification.

The distribution of high impact wheel forces in the histogram columns of Figure 2 lies along the sloping, dashed straight line, which means the distribution would appear as a logarithmic curve on a graph with a linear scale on the vertical axis. Now, the vertical axis in Figure 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 representative of impacts over a longer period, then extrapolation of that sloping dashed line will provide the frequency of occurrence of impact forces greater than 310kN.

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

$$\text{Impact Force (kN)} = 53(5.8 + \log R) \quad (2)$$

where $R$ is the return period in years of a given level of impact. It should be emphasised that this impact force is that which is applied by a wheel to the rail head. To determine the impact force applied to components further down the track structure, such as the sleeper or ballast, appropriate measures should be applied which allow for force sharing amongst support elements and allow for the not insignificant dynamic behaviour of the track. Equation (2) is used later in this paper to help assess the probability of failure of concrete sleepers in the heavy haul lines which were monitored as part of this study.

### 4.0 Capacities of PC sleepers

The experimental programme to investigate the performance of PC sleepers under impulsive loads has been undertaken at UoW. The prestressed concrete sleepers were supplied by Australian manufacturers Rocla and Austrak, as part of the collaborative research project supported by the Australian Cooperative Research Centre for Railway Engineering and Technologies (Rail CRC). The sleepers were broad gauge with the gauge length of 1.60m commonly used in heavy haul coal lines. More details on the sleepers can be found in references [16-18].
A series of static tests on the concrete sleepers was 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 similar 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 corrected according to AS1012.14 [19]. The details of static responses, rotational capacity, post-failure mechanisms, and residual load-carrying capacity of the prestressed concrete sleepers under static loading can be found in references [16-18]. Figure 3 depicts the setup for static testing. A load cell was used to measure the applied load, while an LDVT was mounted at the mid-span to obtain the corresponding deflections. Strain gauges were affixed to the top and bottom surfaces of the test sleeper and on both sides. The transducers were connected to a computer for recording experimental data. The applied loading rate was 0.5mm per minute.

A new high-capacity drop-weight impact testing machine has recently been developed at the University of Wollongong, as depicted in Figure 4. To eliminate surrounding noise and ground vibration, the concrete sleepers were placed on a strong shock-isolated concrete floor in the laboratory. Thick rubber mats were used to replicate the ballast support. It was found that the test setup could accurately represent the support conditions for concrete sleepers found in typical track systems [4-5]. To apply impact loads, a drop hammer with a falling mass of 600kg 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.

The reliability of the drop hammer machine had been evaluated earlier through calibration tests using a high speed camera. It was found that the hammer’s experimental velocity was about 98% of the theoretical velocity. Experimental setup and impact tests were arranged in accordance with the Australian Standards. The in-situ conditions of railway concrete sleepers were replicated as shown in Figure 4. A separate study was performed in order to simulate the impact loads recorded in tracks by means of the drop hammer machine and numerical impact simulations [5].

![Figure 3 Static test setup](image_url)
A typical dynamic moment-deflection relationship at the railseat for PC sleepers is presented in Figure 5. The crack initiation load was detected 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 kNm. The maximum static load capacity was found to be about 583 kN, which is equivalent to bending moment at railseat of about 64 kNm.

Based on the statistical data of the frequency of occurrence of impact loads and their magnitudes (see Section 3), the impact tests on PC sleepers were designed to simulate wheel/rail interface forces by varying the height of drop and the

Figure 4 New high-capacity drop-weight impact testing machine at the University of Wollongong
contact stiffness to achieve the required magnitudes and durations of the load pulses. Typical impact force-time histories measured by the dynamic load cell are presented in Figure 7. Very small flexural cracks were initially detected starting from a drop height of 600 mm. Small shear cracks were also found after several impacts from a drop height of 800 mm. However, no major failure was observed in these single impact load experiments [22].

The PC sleepers were also subjected to gradually increasing impact loads until failure of the sleepers. Figure 8 depicts the progressive impact behaviour of a PC sleeper in the soft track environment. The crack widths at each stage were measured using the magnified telescope. For impact loads between 150 and 600 kN (see Figure 8a), the crack widths were about 0.01 to 0.02 mm. The crack widths increased from 0.02 to 0.08 mm when subjected to impact loads with magnitudes between 700 to 1,000 kN (see Figure 8b). At this stage, spalling of the concrete at the top of railseat section could be detected. When the impact forces were increased up to 1,500 kN, the crack widths also increased up to 0.5 mm (see Figure 8c). The ultimate impact load carrying capacity was reached at about 1,600 kN, when the sleeper railseat section has disintegrated. The failure mode was associated with both flexural and longitudinal splitting actions. The splitting fractures were aligned along the prestressing tendons as illustrated in Figure 8d.

Based on the probabilistic analysis of dynamic loading, 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.
Figure 6 Crack propagation of PC sleeper under static loading

Figure 7 Simulated impact forces
a) impact forces between 150 and 600 kN

b) impact forces between 700 and 1,000 kN

c) impact forces between 1,000 and 1,500 kN

d) impact failure at 1,600 kN

Figure 8 Progressive impact response of a PC sleeper in soft track environment
5.0 Reliability concept

The errors and uncertainties involved in the estimation of the loads on and the behaviour of a structure may be allowed for in strength design by using load factors to increase the nominal loads and using capacity factors to decrease the 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 earlier 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 by 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.

The specified maximum allowed stresses in AS1085.14 are expressions of ultimate strengths of isolated members divided by the factors of safety SF. Thus

\[
\text{Working stress} \leq \text{Permissible stress} \approx \frac{\text{Ultimate stress}}{\text{SF}} \quad (3)
\]

All structural design codes except AS1085.14 have been converted 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 \( \phi \), exceeds the sum of the nominal load effects multiplied by various load factors \( \gamma \), so that

\[
\Sigma (\gamma \times \text{Nominal load effects}) \leq \phi \times \text{Nominal capacity} \quad (4)
\]

or

\[
\text{Design load effect} \leq \text{Design capacity} \quad (5)
\]

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).

Although the limit states are described in 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 illustrated in Figure 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 \( \beta \), called the safety index or reliability index, by the transformation in equation (6) [24]

![Figure 9 Probabilistic density functions for reliability [23]](image-url)

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\[ \Phi(-\beta) = p_F \] (6)

where the function \( \Phi \) is the cumulative frequency distribution function. The relationship between \( \beta \) and \( p_F \) shown in Figure 10 indicates that an increase of 0.5 in \( \beta \) implies a decrease in the probability of failure by approximately one order of magnitude.

### 6.0 Conversion of AS1085.14

In the conversion of the existing design code AS1085.14 to a new limit states format, the concept of a safety index may be used to ensure that the use of the new code will lead to a satisfactory level of structural reliability. This could be done by first selecting typical prestressed concrete sleepers that had been designed according to the current working stress code. The safety indices of these sleepers would then be computed using idealised but realistic statistical models of their loads and structural capacities. These computed safety indices would be used to select target 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 met with reasonable precision. This procedure is called the code calibration procedure.

As an example, the calibration procedure and the safety indices \( \beta \) for ultimate limit state designs according to the Australian limit state code AS 4100 Steel Structures [25] are compared in Figure 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 of 3.0 to 3.5 compared to 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 related to specified limit states, and particularly to 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 rail-seat cross-section) are specified in terms of their characteristic values \( R_k, Q_k \) and \( S_k \) and associated design coefficients \( \phi, \gamma_Q \) and \( \gamma_S \) as follows:

\[ R' = \phi R_k \] (7)

![Figure 10 Relationship between safety index and probability of failure [24]](image_url)
Typically, extreme values such as 5 and 95 percentile values (of distributions similar to Figure 9) are chosen for characteristic values in specifying design values for checks concerned with ultimate limit states, while average values are typically used in checks concerned with serviceability limit states.

In the process of converting AS1085.14 to a new limit states format, it is proposed that the opportunity is taken to examine the structural reliability of both the existing and proposed concrete sleeper codes to endeavour to obtain some specified consistency in structural reliability in the formulation of the new design code, as demonstrated in the following paragraphs.

The statistical characteristics of resistance and loads may be stated in terms of random variables that will be denoted by $X_1, X_2, \ldots, X_N$. 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) may be solved for the probability of failure using the methods of structural reliability analysis. From the relationship between $\beta$ and $p_F$ given in Figure 10, the safety index could be determined.

It should be noted that equation (6) and Figure 10 lead to numbers for the safety index that are convenient for evaluations of comparative safety of various designs of prestressed concrete sleepers.

Conceptually, the conversion of the existing design code to a new code written in limit states format should be undertaken through a calibration procedure which could comprise the following steps:

1. Derive statistical models of structural resistances (concrete, prestressing steel) and loads (e.g. impact loads at wheel/rail interface) and load effects (e.g. bending moments at rail seat cross-section).
2. Using these models, safety indices could be evaluated for existing designs of concrete 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 a new limit states design code.
4. The load and resistance factors of the proposed new code could be selected so that, the associated safety indices are close to the chosen target values.

The essential information required for the calibration procedure that should be generated by the research teams at QUT and UoW is illustrated schematically below.

### 7.0 Limit states of PC sleepers

According to Leong [8], Australian railway organisations would condemn a sleeper when its ability to hold top of line or gauge is lost. These two failure conditions can be reached by the following actions:

- abrasion at the bottom of the sleeper causing loss of top;
- abrasion at the rail seat causing a loss of top;
- severe cracks at the rail seat causing the ‘anchor’ of the fastening system to move and spread the gauge;
- severe cracks at the midspan of the sleeper causing the sleeper to ‘flex’ and spread the gauge;
severe degradation of the concrete sleeper due to alkali aggregate reaction or some similar degradation of the concrete material.

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

A challenge in the development of a limit states design concept for prestressed concrete sleepers is the acceptance levels of the structural performances under design load conditions. Infinite fatigue life of sleepers cannot be retained after allowing cracks under impact loads. Degree of reliability is also an important factor that needs to be taken into account. The Australian Standard AS 5104-2005 [24] prescribes the general principles for reliability for structures, and indicates that limit states can be divided into the following two categories:

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

Leong [8] and Murray & Leong [6-7] noted that for railway concrete 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 an event of exceedance of a limit state. Therefore, the following three limiting conditions [8] have been proposed that would be relevant to the design of railway concrete sleeper:

**Ultimate Limit State**
A single once-off event such as a severe wheel flat that generates an impulsive load capable of failing a single concrete sleeper. Failure under such a severe event would fit within failure definitions causing severe cracking at the rail seat or at the midspan.

**Damageability (or Fatigue) Limit State**
A time-dependent limit state where a single concrete sleeper accumulates damage progressively over a period of years to a point...
where it is considered to have reached failure. Such failure could come about from excessive accumulated abrasion or from cracking having grown progressively more severe under repeated loading impact forces over its lifetime.

**Serviceability Limit State**

This limit state defines a condition where sleeper failure is beginning 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 failure of a cluster of sleepers, an operational restriction is usually applied until the problem is rectified.

For the purpose of this discussion paper, only the ultimate limit state for a single concrete sleeper is considered in the development of the reliability-based design procedure for concrete sleepers. An experimental programme will be developed to characterise the uncertainties of the calculation models for the resistances of concrete sleepers in the ultimate limit state.

If the ultimate limit state for a concrete sleeper is associated with the flexural failure, equation (6) could be defined as

$$M^* \leq \phi M_u$$

(11)

where the ultimate moment capacity, $M_u$, is determined from AS 3600 code [26], and $M^*$ is the design bending moment due to the design static wheel load combined with the design impact wheel load caused by wheel or rail irregularities (e.g. wheel flats). In the reliability analysis format, equation (8) can be represented by the following limit state function

$$g_{ul}(X) = \Theta_R M_u - \Theta_S \times M^*$$

(12)

where $M_u$ is the random variable that could be expressed as a function of the basic random variables describing the ultimate resistance of the selected cross section; the sleeper Applied moment is the random variable due to the design wheel impact load and described by a probability curve of flexural moments in sleeper; $\Theta_R$ and $\Theta_S$ are the model uncertainty coefficients.

As described in section 3 earlier, Murray and Leong [7] proposed a method by which the ultimate limit state wheel/rail impact design forces may be calculated based on data drawn from a QR WILD impact detector on a heavy haul coal line. But the problem with converting the design wheel/rail force to the design sleeper moment is still open for discussion.

Murray and Leong [7] emphasised the need for computer dynamic track analysis using such package as DTRACK to compute the design sleeper moment. While in principle this approach could be viable, it could lead to a complication with formulating statistical ultimate limit state models of concrete sleepers for their reliability assessment and for the model calibration in the conversion process to a new limit states design code format. Equation (12) will become

$$g_{ul}(X) = \Theta_R M_u - \Theta_S \times M^*$$

(13)

(applied moment, $M^*$ is to be determined from computer analysis - DTRACK) where the design sleeper moment does not have an analytical representation and equation (11) cannot be solved to find the safety indices $\beta$. Therefore, it is very important to carry out an experimental investigation of the relationship between impact wheel load and the resulting bending moments with a view to establishing a simplified analytical expression that could be incorporated in the limit state functions like equation (11) for conducting the reliability assessment studies on prestressed concrete sleepers.

The impact tests to establish the relationship between the impact load and the railseat bending moment have been carried out using a new drop hammer machine at UoW [16]. In the impact tests, the fall height of an anvil was increased step-by-step up to the maximum height from which the resulting bending moments would not exceed the cracking moment capacity. The duration of impact loads was kept almost constant at about 4-5 msec regardless of the fall height. To provide support in interpreting the data from the tests, finite-element modelling of sleepers subjected to impact loads and DTRACK simulations were also used. The findings from these studies showed that the results of UoW experiments were very close to those obtained from DTRACK [16-17].
8.0 Conclusion

The current design of railway prestressed concrete sleepers, stated in AS 1085.14, is based on the permissible stress concept. The design process is based on the quasi-static wheel loads and the static response of concrete sleepers. To shift to a more rational design method involves significant research effort within the framework of the CRC for Railway Engineering and Technologies.

The collaborative research between the University of Wollongong (UoW) and Queensland University of Technology (QUT) has involved all important facets such as the spectrum and amplitudes of dynamic forces applied to the railway track, evaluation of the reserve capacity of typical prestressed concrete sleepers designed to the current code AS 1085.14, and the development of a new limit states design concept. This paper presents the background information and some research outcomes of the Rail-CRC research project aimed at developing the new limit states design concept for prestressed concrete sleepers.

The paper also describes the reliability concepts and rationales associated with the development of limit states format codes and the issues pertaining to 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. The target reliability indices $\beta$ to be used for the code calibration can be obtained from the reliability analysis of existing design procedures and the newly proposed method to design the prestressed concrete sleepers.

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10.0 Notation

- $\beta$: safety index or reliability index
- $\phi$: dynamic impact factor, capacity factor, design coefficient for resistance
- $\Phi$: cumulative frequency distribution function
- $\gamma$: load factors
- $\gamma_Q$: design coefficient for loads
- $\gamma_S$: design coefficient for load effect
- $\theta_R$: model uncertainty coefficient of resistance
- $\theta_S$: model uncertainty coefficient of load effect
- $g(X_N)$: general limit function of the random variable $X_N$
- $g_{ult}(X)$: Limit function of the random variable $X$ at ultimate conditions
- $M^*$: design bending moment
- $M_u$: ultimate moment capacity (characteristic)
- $\rho_F$: possibility 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

11.0 References


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