Fig. 2—CN Rail's main line, Jasper, Alberta, 1972

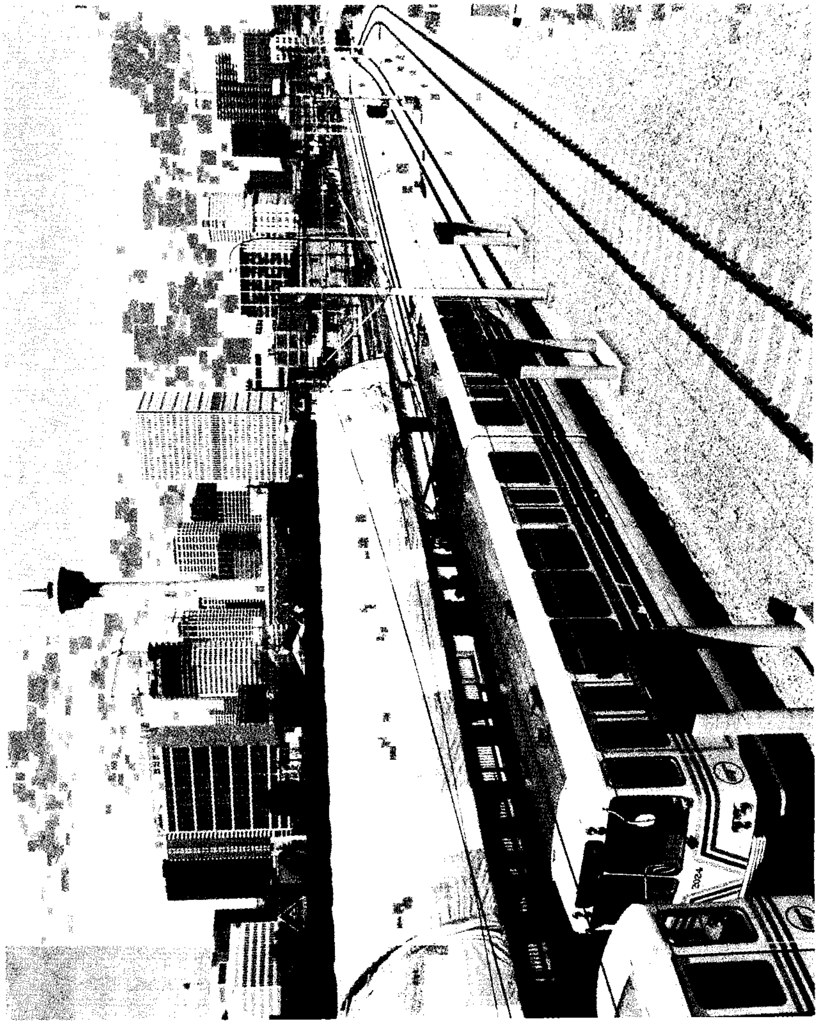

Fig. 3—Light Rail Transit, Calgary, Alberta, 1980

Fig. 4—CN Rail's number 20 turnout near Kamloops, B.C., 1984

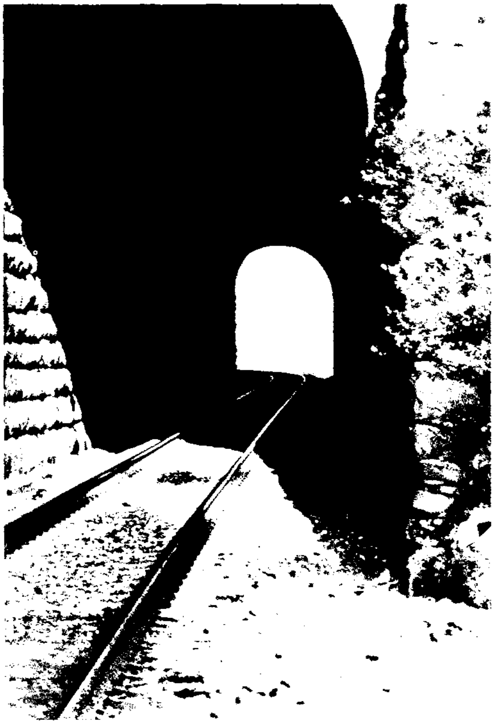

Fig. 5—CP Rail's main line, Yale, B.C., October 1983

SP 93-15

Development of a Precast Prestressed Concrete Railway Bridge Tie

by T. I. Campbell and M. S. Mirza

Synopsis: A research and development program on prestressed concrete ties for use on open deck railway bridge systems is described. This program involved the design and fabrication of prototype ties, studies of load distribution of a static wheel load among the ties, both by mathematical modelling and laboratory tests on a full-scale bridge system, determination of the strength of individual ties under static and repeated loadings, field testing of bridge systems in service, and a laboratory study of dynamic load distribution in the bridge system. It is shown that three parameters have a significant influence on the distribution of static wheel loads among the ties and that a linear relationship exists between the load taken by a tie and the tie spacing. Laboratory testing for static load distribution showed good correlation with an analyical model, while tests on individual ties indicated adequate strength under static and repeated loadings. Field tests, which are being conducted to determine the loading on individual ties and the behaviour of the ties under railroad traffic, are described and preliminary data are presented.

Keywords: loads (forces); mathematical models; precast concrete; prestressed concrete; railroad bridges; railroad ties; research; static loads; tests

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INTRODUCTION

Open decks are widely used for bridges on railway systems in Canada. Such a bridge comprises rails supported on ties spanning between two main longitudinal girders. The ties transfer the wheel loads directly from the rails to the main longitudinal supporting girders and thus are referred to as bridge ties. This structural system is economical in that the dead weight of ballast used to support the ties in a conventional railroad bridge is eliminated.

A deck with timber ties is susceptible to fire, which not only causes damage to the timber ties but also to other components of the bridge structure, resulting in major traffic disruptions and hazards to life and property, as well as in costly maintenance. Consequently, the Canadian railways (Canadian National (CN) and Canadian Pacific (CP)) are considering replacing the timber ties with precast, prestressed concrete ties.

An open-deck bridge system with concrete ties is shown in Figure 1. Elastometric pads are incorporated between the rails and the ties (rail-tie pads) and between the ties and the supporting girders (tie-girder pads) in order to control impact from wheel loading.

A research and development program on prestressed concrete bridge ties, which has been undertaken by the railways (CN and CP) in conjunction with the Canadian Prestressed Concrete Institute (CPCI), McGill University, Queen's University, and the Natural Sciences and Engineering Research Council of Canada, is described in this paper.

This program involved the design and fabrication of prototype ties, studies of load distribution of a static wheel load among the ties, both by mathematical modelling and laboratory tests on a full-scale bridge system, determination of the strength of individual ties under static and repeated loadings, field testing of bridge systems in service, and a laboratory study of dynamic load distribution in the bridge system.

DESIGN OF PROTOTYPE TIES

Four concrete bridge ties for evaluation were designed by the railways, two by CN (designated CN "A" and CN "B") and two by CP (designated CP "A" and CP "B"). Data used in design of the ties are given in Table 1 and basic dimensions of the ties are shown in Figure 2.

All ties are 12 feet (3.6 m) long and have a trapezoidal cross-section in which the top dimension and the side slopes are the same for all ties. It can be seen from Table 1 that the main difference in the design assumptions for the four ties is the value of the live load impact factor. A value of 60 per cent, the maximum value suggested in the concrete railway bridge code(1), was used for the CN "A" tie, while a value of 150 per cent was assumed for the other three ties. Cooper's E80(2) loading was used for all ties and the load distribution, which is the amount of an axle load taken by the tie, was assumed to be one-third for all ties except for CP "B" where a value of 40 per cent was used. Deflection of the tie was limited to not more than $S_g/900$, where S_g is the girder spacing, namely 8 feet (2.4 m) for the CN ties and 9 feet (2.7 m) for the CP ties.

The CN "A" tie was chosen for the initial evaluation and sixteen such ties were fabricated at the Genstar Costain Tie Company in Edmonton. Figure 3 shows that this tie was prestressed by 32 indented steel wires having a diameter of 0.193 inches (5 mm) and an ultimate tensile strength of 225 ksi (1,550 MPa).

STATIC LOAD DISTRIBUTION

Theoretical Study

The distribution of a wheel load to the ties depends on the properties of the various components of the system (see Figure 1), namely:

- location of the wheel load with respect to the ties
- stiffness of rail-tie pads
- differential elevation of the ties
- stiffness of the supporting girders
- type of tie
- spacing of supporting girders
- stiffness of tie-girder pads
- type of rail
- spacing of ties.

The relative effects of these variables were established from a parametric study using the analytical model of the system shown in Figure 4.

It was assumed that the main supporting steel girders were simply supported over a representative span of 30 feet (9.1 m). The length of the rails was taken as the same as the bridge span and both ends of each rail were assumed to be fixed to simulate continuity of the rail. A wheel load was applied simultaneously to each rail and positioned over the central tie since this represented the most critical condition of loading in the central tie. Both the rail-tie pads and the tie-girder pads were modelled as linear elastic springs. The computer program SAP IV(3) was used for the analysis. The rails and bridge girders were simulated by beam elements, while the pads were simulated by truss elements, whose properties were chosen to give the required stiffnesses.

Initially models containing three, five, seven and eleven ties were studied. It was concluded that the model with seven ties was adequate, since the load distributions among the ties for the seven- and the eleven-tie arrangements were similar. Further, with the eleven-tie system, the outermost ties were subjected to uplift. Also, it was found that the rail-tie pad could be assumed rigid. Consequently, a model incorporating seven ties and rigid rail-tie pads was used throughout the study. The variations considered for the parameters of the system are shown in Table 2. The rationale used in establishing these values is given in References 4 and 5.

This study showed that, of the above nine parameters, only three, namely the stiffness of the tie-girder pads, the type of rail, and spacing of the ties, influenced the distribution of a wheel load among the ties. A typical distribution of wheel load among the ties for a particular system in which the stiffness (K) of the tie-girder pads varies is shown in Figure 5. It is seen that the distribution of load is influenced by variations in the tie-girder pad stiffness and that the load taken by the tie under the wheel increases with pad stiffness.

Figure 6 shows the variation of load in the tie under the wheel loads with tie spacing for different types of rail and tie-girder pad stiffnesses. The load increases with tie spacing approximately linearly in each case and, consequently, a linear relationship may be used to relate the maximum wheel load in a concrete bridge tie to tie spacing for a particular type of rail and tie-girder pad stiffness.

It can be seen from Figure 6 that the commonly used design assumption(4) where the maximum load taken by a tie is equal to one-third the wheel load is reasonable, provided relatively soft tie-girder pads are incorporated in the system.

Laboratory Study

The set-up used for full-scale laboratory testing of the open deck bridge system is shown in Figure 7. Nine ties were supported on two longitudinal girders. These girders were spaced 8 feet (2.44 m) apart and simply supported over a span of 20 feet (6.1 m) on a steel grillage resting on the floor of the laboratory. The rails were attached to the ties using Pandrol fasteners and simulated wheel loads were applied to the rails by means of a loading beam. Load was applied to the loading beam by jacks reacting against a reaction beam to which were attached tie rods anchored to the substructure grillage. To ensure proper seating of the ties, subsequent to fastening the rail, shims were inserted between the ties and the tie-girder pads. This was necessary since improper seating could result from variation in elevation at the rail seats of adjacent ties and subsequent lifting of ties from the tie-girder pads when the rail was installed. This variation in elevation results from variation in thickness and differential camber of the individual ties. Both the analytical model and the load testing indicated that the presence of such gaps in the system significantly affected the distribution of load among the ties and that, in such a situation, the design assumption that the maximum load taken by a tie is equal to one-third of the wheel load is not acceptable. Consequently, the influence of potential differential elevation of the ties in an actual system was assessed during the field testing where care was taken to ensure proper seating of all ties in the system.

The load distribution among the ties was determined by measuring the flexural deformation of the nine ties by means of dial gauges. Each tie was calibrated prior to the load distribution tests by subjecting it to known simulated wheel loads and obtaining a load-deflection response curve.

The three significant parameters identified in the analytical study were varied in the load distribution tests. Tie spacings of 16 inches (406 mm) and 24 inches (610 mm) were used together with 132 RE rail and three different thicknesses, namely ½ inch (13 mm), ¾ inch (19 mm) and 1 inch (25 mm), of tie-girder pads. Results from a typical test are shown in Figure 8. Generally good agreement was obtained between the test data and the analytical predictions, thereby establishing the validity of the analytical model.

STRENGTH OF TIE

Static Loading

Three ties were tested to destruction under static load using the same loading frame as used for the load distribution testing. In this test a short length of rail was attached only to the tie directly below the loading beam. Strains were measured over the depth of the tie at mid-span using electrical resistance strain gauges both on the concrete surface of the tie and on two of the prestressing wires and also by mechanical strain gauges. Deflection was measured using dial gauges.

With increasing load, uniformly spaced flexural cracks appeared first in the bottom of the tie between the rail seats. As the load was increased further, flexural-shear cracks occurred in the tie between the rail seat and the tie-girder pad at the support. This was followed by the formation of wide diagonal cracks running from the rail seat to the tie support. Soon afterwards failure of the tie occurred in flexure in a brittle manner due to crushing of the concrete at the top surface of the centre of the tie. A strain compatibility analysis of the tie section confirmed that the tie was indeed over-reinforced.

Moment-curvature relationships for the three ties are shown in Figure 9. The maximum moments in each of the three ties at failure were 1,182 K.in (133 kN.m), 1,086 K.in. (123 kN.m), and 1,224 K.in. (138 kN.m), and the corresponding concrete strengths were 7,040 psi (48.5 MPa), 6,875 psi (47.4 MPa), and 7,240 psi (49.9 MPa), respectively. The maximum moment in the tie for Cooper's E80 loading with 60 per cent impact and one-third load distribution is 400 K.in. (45.2 kN.m). Thus, the load factor against Cooper's E80 loading with 60 per cent impact for the weakest of the three ties tested was 2.7 which is considered adequate. However, the lack of ductility demonstrated by the tie was a source of concern and attempts have been made to improve this.

Repeated Loading

Two ties were subjected to repeated loading, using the set-up shown in Figure 10. Each tie was loaded through short lengths of rail and supported as indicated in Figure 2(a).

The first tie was subjected to two million cycles of load at a frequency of 1.6 Hz with the load varying between 42.7 kips (190 kN) and zero. The maximum load level is equivalent to Cooper's E80 axle live load distributed one-third to the tie and considering an impact factor of 0.6. No cracking or evidence of fatigue damage was observed in the tie.

A number of different load levels were used in the test on the second tie. The tie was loaded beyond cracking prior to applying the repeated loading. Figure 11 shows the various load levels and the cumulative number of cycles at various load levels, as well as the observed mid-span deflections obtained from static load tests carried out at each stage. Failure occurred after 5.76 million cycles when the load level was 99.2 kips (441 kN) due to snapping of the prestressing wires followed by crushing of the concrete on the top surface at mid-span of the tie. Subsequent inspection of the prestressing wires in the failure zone indicated that eleven of the thirty-two wires showed typical fatigue failure in that no necking was apparent.