Statistical Description of Service Loads for Concrete Crosstie Track

Robert H. Prause, Applied Dynamics and Acoustics Section, Battelle Columbus Laboratories, Columbus, Ohio
Andrew Kish, Transportation Systems Center, Cambridge, Massachusetts

Measurements of loads and bending moments on concrete crossties for several days of revenue traffic were used to develop a statistical description of track loads for tangent and curved tracks that have variable tie spacing. The measured data show large tie-to-tie variations in loads and a load-dependent tie support condition. Many ties were center-bound for loads from light or empty cars, but the tie support became more uniform for heavy wheel loads. Maximum tie bending moments measured on curved track were considerably higher than those on tangent track because of the increase in vertical and lateral loads on the high rail when trains exceed the balance speed of the curve. Tie bending moments measured in this program were considerably lower than the current static flexural strength requirements for a probabilistic prediction of maximum load for a 50-year life. These and data from other concrete-tie test installations indicate a need to identify the failure mechanism for concrete ties so that statistical load descriptions can be used for future design and testing. Low-probability maximum loads will be very important if failures result from infrequent loads that exceed the static strength. However, the higher probability mean cyclic loads will be the more important factor if fatigue is identified as the governing failure mechanism.

There is currently considerable interest in the development of concrete crossties for main-line use in North America. Experience in several other countries has indicated that these ties have the potential advantages of longer tie life, reduced lining and surfacing maintenance, and increased rail life on curves. However, the premature cracking of concrete ties at several U.S. test installations during the past decade has prevented these ties from becoming a workable alternative to wooden ties.

Much of the difficulty in obtaining acceptable performance from concrete ties results from a lack of knowledge about tie loading and the effective support provided by the ballast. Tie center binding and end binding are familiar conditions for wooden-tie track, but the inherent resilience of wood minimizes the damage that results from these undesirable loading conditions. Concrete, however, is a very brittle material that is, therefore, less forgiving when stressed beyond its design limits.

The development of concrete ties in the United States has followed the development of the American Railway Engineering Association (AREA) specifications (1). Those specifications have evolved through several modifications in which tie-strength requirements have been gradually increased because of premature tie cracking. Specifications for the minimum bending strength at the rail seat and tie center and the corresponding static acceptance tests are the major considerations. The lack of sufficient field-test data to provide accurate descriptions of tie service loads that reflect realistic variations in support and loading conditions has been a major deterrent to the development of these specifications. This paper presents some statistical data on service loads for concrete ties and rail-fastener assemblies for typical main-line revenue railroad traffic.

TEST-SITE DESCRIPTION

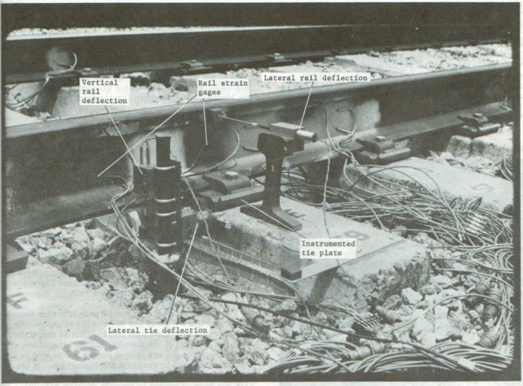
The test sites selected for this extensive measurement program were on the Florida East Coast Railway (FEC) about 32 km (20 miles) north of West Palm Beach. This track was selected from among the several available sites (such as the Kansas test track; the Atchison, Topeka, and Santa Fe Railway test track at Streator, Illinois; the Chessie System test track at Lorraine, Virginia; or the Norfolk and Western Railway test track at Roanoke, Virginia), because it provided the best combination of track variables required for this program. These included tangent and curved tracks, tie spacings of 0.51 and 0.56 m (20 and 22 in) for comparison with the 0.61-m (24-in) standard spacing, and mixed freight loadings that included 90.7-Mg (100-ton) cars and speeds up to 96.5 km/h (60 mph). Two sections of tangent track that had 0.51- and 0.61-m tie spacings were instrumented to evaluate the effect of tie spacing, a major track design variable. A third test site that had 0.61-m tie spacing on a 3° 52′ curve and a 72.4-km/h (45-mph) balance speed was selected for a comparison of loads on tangent and curved tracks. Track construction consisted of 60-kg (132-lb) rail, Railroad Concrete Crosstie Corporation (RCCC) ties with Cliploc fasteners and polyethylene rail pads, and granite ballast. The RCCC tie, a modification of the original MR-2 design, is somewhat smaller than the ties designed according to the most recent AREA specifications, but this was not detrimental to the objective of measuring tie and fastener loads. Also, the fact that the temperate Florida climate is not a typical North American environment was not considered critical for obtaining load data over a short time period.

The tangent-track sites had been in service for about 1 year and the curve site had been in service about 6 years when the measurements were begun during July 1976. However, the curve had been surfaced and lined at the same time that the tangent track was constructed, and measurements from the U.S. Department of Transportation track-geometry car showed that track geometry was excellent throughout. This track was located on old roadbed that had been scraped to provide an even surface and to remove the old limestone ballast. Excavations at each of the tangent-track test sites showed a ballast depth of about 16.5 cm (6.5 in) under the tie and a clear demarcation between the new granite ballast and the old roadbed. It was apparent that the old roadbed (subgrade) was actually a well-compacted mixture of sandy soil and old limestone ballast, which provided a very stable and relatively stiff foundation.

TRACK INSTRUMENTATION

The selection of the measurement parameters, instrumentation, and data requirements for meeting the objectives of this program are discussed elsewhere (2). As shown in Figure 1, the instrumentation at the test sites was extensive. As many as 72 different measurements were recorded for a few trains at each site. About 30 measurements were recorded continuously for several days of traffic. The major types of instrumentation used are described below:

Figure 1. Typical track instrumentation.



1. Strain gauge circuits applied to the rail web were used to measure the maximum (peak) vertical and lateral rail load for each passing axle. The signals from these circuits were also used to determine train speeds and approximate car loads. The vertical-load circuits were calibrated by using empty and loaded cars. A hydraulic ram placed between the two rails was used to calibrate the lateral load circuits.

2. Special-design instrumented tie plates were installed between the rail and the tie to measure the vertical rail-seat loads and the rail-seat rollover moments on five adjacent ties in each section. The load-cell washers in the tie plates were calibrated by using a laboratory loading fixture.

3. Strain gauge circuits were installed on several ties to measure the bending moments at the rail seat and the bending and torsional moments at the tie center. A full bridge with four active gauges was used for each measurement. Bridge output was calibrated directly in moment by using equivalent concrete ties in the laboratory.

4. Three FRA-Portland Cement Association (PCA) load-cell ties (see Figure 2) were installed to measure tie-support reactions at the interface of the tie and the ballast. The load-cell ties are steel and have a bending stiffness similar to that of concrete ties; they have 10 instrumented segments along the tie bottom to measure tie-to-ballast pressure.

5. Displacement transducers were used to measure the vertical track deflections and the lateral deflections of the rail head relative to the tie.

6. Instrumented load washers were used to record load variations on rail-fastener bolts.

7. Movable accelerometers were used to measure rail and tie vertical accelerations at several locations.

All three of the test sites included a main instrument array that extended over seven adjacent ties so that a complete set of load and response data could be obtained at one location. Additional instrumentation was located at random in a 15.2-m (50-ft) zone on either side of the main array and used to record load variations caused by dynamic motions of the cars as they passed the test site. The instrumented tie plates, which required lowering the ties in the main array about 2.54 cm (1 in), and the load-cell ties were all installed in the track 1 month before the measurement program was started to allow reconsolidation of the ballast under traffic.

STATISTICAL DATA ANALYSIS

Time-history records of track loads were recorded on frequency modulation tape for all trains passing during several days of revenue service. A special-purpose computer program was used to digitize these data and store a single peak value of load for each wheel (axle) that passed a particular measurement location. An identification for car load and car speed was used to separate the data into 16-km/h (10-mph) speed bands and into three car load categories before the data were stored on a disk file for subsequent analysis. Car load was determined from the vertical wheel-rail load circuits in the main array, and car speed was determined from the transit time for a wheel to pass over a premeasured track section.

The final step in the data processing was to perform the statistical calculations needed to obtain mean values, SDs, probability densities, and probability distributions for the peak-value data from each measurement. The data in each of the speed and load categories were analyzed separately for each measurement (channel), and summations could be made for any category. Data from selected categories at different measurement locations could also be combined to form new data sets.

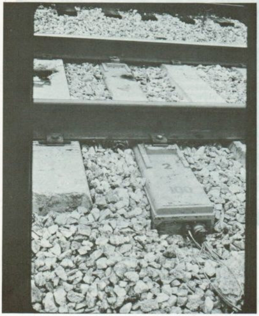


Figure 2. Load-cell tie.

For example, data from the five wheel-rail load circuits at site one could be combined for heavy cars in the 80- to 97-km/h speed range to include spatial variation effects.

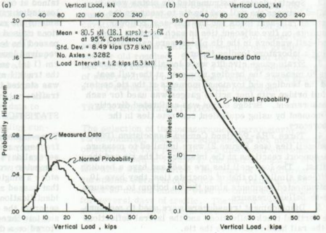
Statistical calculations were made by dividing the total expected data range into 200 equal intervals and summing the number of peak values (wheels) falling in each interval. Graphs of probability density (histograms) and probability distribution functions were then plotted by using an interactive graphics terminal and the identification numbers for single categories and combinations.

The format for the results of the statistical analysis is shown in Figure 3 for measurement of the peak vertical wheel-rail loads. These data are for all cars and all speeds (all trains) at one measurement location. The probability density histogram shows the ratio of the number of peak loads within each of the fifty 5.3-kN [1200-lbf (1.2-kip)] load intervals that cover the total range of 267 kN (60 000 lbf). It is important to note that the quantitative results for the histogram depend on the load interval selected and are therefore not unique. Increasing the load interval (reducing the number of intervals) increases the number of occurrences at a particular load level. This improves the averaging used for the estimate but reduces the resolution—a trade-off decision. Load intervals that are too small for the data base cause irregularities in the density curve at extreme loads because there are insufficient data points to provide a reliable average for these low-probability events.

The amplitude of the probability-distribution function shown in Figure 3 gives the percentage of peak loads that exceed a specified load level. This is calculated from the integral of the density function; therefore, the quantitative results are unique and do not depend on the load interval used to generate the histogram. In the probability-distribution function format, the vertical axis has been expanded to provide greater resolution of the extreme values. Insufficient data points to provide reliable estimates for low-probability events appear in the distribution function as horizontal segments in some of the later figures. This shows that there were no data points at that load level. The accuracy of the estimates at these points is questionable.

Statistical data that have a normal (Gaussian) distribution will appear as the familiar bell-shaped curve

Figure 3. Formats for results of statistical analyses: (a) probability-density histogram and (b) probability distribution function.



on the density plot and as a straight line on the scale used for the distribution curve. These curves are shown in Figure 3 for comparison. The 50 percent probability level gives the median load (50 percent higher and 50 percent lower) for any distribution. If the experimental data had had a perfectly normal distribution, then the median peak load would have been identical to the mean peak load [which is 80.5 kN (18 000 lbf) in this example]. The theoretical curves for the normal distribution shown in the figure have the same mean value and SD as the measured data. For this particular measurement, the normal curve gives a better estimate of the data at low probability levels than it does in the vicinity of the mean load because of the distortion caused by large numbers of empty cars. Other distributions such as a beta or a log-normal distribution may give a better estimate of extreme-value statistics, but these were not investigated.

The following sections of this paper summarize some of the more interesting results derived from statistical data on tie loads and show the effects of tie-to-tie spatial variations and of variations from the mix of vehicle types and operating conditions. For reference purposes, the vertical rail-seat loads and tie bending-moment requirements of the current AREA specifications are listed below (1 m = 3.3 ft, 1 kN = 225 lbf, and 1 kN·m = 8852 lbf·in).

	Vertical Rail-Seat Load		Bending-Moment Requirement (kN·m)			
Tie Spacing (m)	Percentage of Wheel Load	Value (kN)	Rail Seat (+)	Rail Seat (-)	Tie Center (-)	Tie Center (+)
0.533	46.5	214.5	25.4	13.0	22.5	10.1
0.61	51	234.1	28.4	13.0	22.5	10.1
0.69	55.5	254.1	31.0	13.0	22.5	11.3
0.76	60	273.7	33.8	13.0	22.5	12.4

The RCCC concrete tie used on the FEC is designed to have a minimum flexural strength of 17.0 kN·m (150 000 lbf·in) at the rail seat, and one of every 200 ties is checked to this limit after curing for 18 h. Although some additional increase in strength will occur with time, this smaller tie cannot meet the 28.3 kN·m (250 000 lbf·in) positive moment currently required for 0.61-m tie spacing. More detailed data from the measurement program are given elsewhere (3).

Vertical Wheel-Rail Loads

Figure 4 shows typical statistical distributions for all five measurements of vertical wheel-rail load at one site. There was no significant spatial variation in load measurements at this site, and the data for the other sites were similar. The 0.1 percent exceedance load levels for the most severely loaded location at each site are given below.

				Bending Moment (kN·m)	
Test Site	Tie Spacing (m)	Vertical Wheel-Rail Load (kN)	Tie-Rail-Seat Load (kN)	Tie Rail Seat (+)	Tie Center (-)
Tangent	0.61	200	107	7.4	3.4
Tangent	0.51	205	93	8.7	6.3
Curve	0.61	222	138	8.8	4.7

The 0.1 percent load level is exceeded by only 1 of each 1000 axles but, the annual traffic of 2.4 Tg (20 million gross tons) averages about 4000 axles/d. Therefore, the 0.1 percent load level would be exceeded about 4 times/d for this traffic.

Vertical Rail-Seat Loads

The data shown in Figure 5 for vertical rail-seat loads on several adjacent ties show that there is considerable tie-to-tie variation, which reflects local variations in support conditions. This causes a larger percentage variation in the average load than it does in the less frequently occurring high loads where the ties are firmly seated in the roadbed. The 0.1 percent exceedance rail-seat load of 138 kN (31 000 lbf) at the curve site given above occurred under the higher rail (where vertical loads are higher than those on tangent track when trains operate consistently above the balance speed for the curve).

Tie-Rail-Seat Bending Moments

Figure 6 shows the statistical distributions of the rail-seat bending moments measured on several different ties at site 1. A characteristic of tie bending-moment data is the large tie-to-tie variation in the mean and 0.1 percent moments. Also, all ties except one showed both positive and negative peak bending moments (which indicates a load-dependent ballast support condition). Negative rail-seat bending moments can be caused by a center-bound condition. Positive moments are expected for a uniform support condition, an end-bound support condition, or one in which a ballast pocket may have formed under the rail seat.

Figure 7 shows a typical load-dependent effect by comparing the bending-moment data for a single tie; locomotives, light cars [less than 45.5-Mg (50-tons) gross mass], and heavy cars [more than 45.5-Mg (50-tons) gross mass] are identified separately. For this particular tie, the peak rail-seat bending moment was positive for all of the locomotives and heavy cars, but some negative values were recorded for light cars. It is also evident that, as a class, locomotives cause the highest mean loads but heavy freight cars cause loads that are as high or higher at the 0.1 percent probability level. Also, the presentation of the data as percentage of wheels can obscure an important point. Because there are 10 to 15 heavy cars for every locomotive in a typical train, track damage from high vertical loads will occur much more frequently from heavy cars than from locomotives. It also appears that the probability-distribution curves for heavy cars and locomotives cross near the 0.1 percent load level so that the loads from heavy cars will dominate the high-load, low-probability tail of the probability-distribution curve.

The maximum 0.1 percent rail-seat bending moments listed above are quite similar for all three measurement sites, but the highest loaded tie at the curve site has a higher SD than any of those measured at the other sites. Table 1 gives the low-probability statistics that would be predicted by using the measured mean and SD for the highest loaded tie at site 3 and assuming a normal probability distribution and the corresponding number of axles between occurrences; e.g., a bending moment of 9 kN·m (79 300 lbf·in) would be exceeded by 0.1 percent of the axles or 1 of every 1000 axles. The comparison between the bending moments predicted by using a normal distribution and the actual measured distribution of moments shows very good agreement over the limited range of this particular measurement, but other theoretical distributions might be more appropriate for extreme-value estimates.

For reference purposes, Table 1 also lists the estimated number of days between exceedances for different annual traffic densities. These data indicate that bending moments greater than about 13 kN·m (115 000 lbf·in) would not be expected during a 50-year

Figure 4. Statistics of wheel-rail loads: typical railroad traffic.

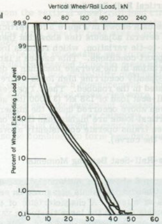


Figure 5. Statistics of rail-seat loads: typical railroad traffic.

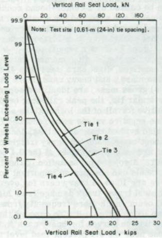
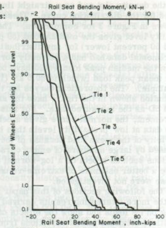
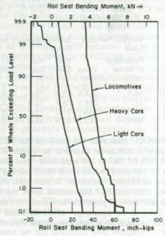


Figure 6. Peak tie-rail-seat bending-moments: all traffic at site 1.



Note: 0.27 kN·m (2400 lbf·in) load interval.

Figure 7. Peak tie bending moments for different car sizes.



life at any normal traffic level, assuming that the predicted distribution is valid for this period of time. This is less than 50 percent of the 28-kN·m bending-moment requirement given in the current specifications. However, it should be cautioned that this extrapolation is based only on vehicle-load statistics for a specific, heavily loaded tie. The additional statistics for tie-to-tie variations have not been included. Also, the question of whether the normal distribution, or some other distribution, will give a conservative estimate of the very low probability high bending moments that might be caused by severe wheel-flat impacts cannot be answered completely without collecting data for a much longer time period. Experience at test installations where ties have failed, however, shows that a considerable number of ties crack within a few months after installation, which tends to dispute the hypothesis that cracking is due to very infrequent occurrences of high loads.

Tie-Center Bending Moment

The statistical data for the bending moments measured at the center of five different ties at each site showed considerable tie-to-tie variation. All ties except one had both positive and negative peak bending moments. Negative center bending moments represent a center-bound support condition and cause tension in the top surface of the tie. Bending cracks in the middle of concrete ties almost always start at the top surface; thus, negative bending moments have historically been of major importance. Positive bending moments at the tie center can be caused by an end-bound support condition. If the rail-seat loads were distributed symmetrically on a well-compacted support region under each rail seat, the bending moments in the tie center would be quite low.

The maximum bending moments at the tie center summarized above show a high value of 6.3 kN·m (56 000 lbf·in) at site two, and this was exceeded by a maximum positive moment (not listed) of 7.6 kN·m (67 000 lbf·in) on one tie at site three. These maximum moments at the tie center are only about 15 percent lower than the maximum positive moments in the rail-

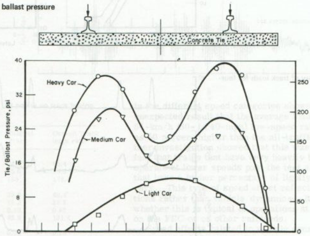
Table 1. Extrapolated statistics for rail-seat bending moments based on most severe tie loading.

Percentage Level Exceeded	Rail-Seat Bending Moment (kN·m) Predicted	Rail-Seat Bending Moment (kN·m) Measured	No. of Axles Between Exceedances	Estimated Time Between Exceedances (years)* 2.4-Tg (20 million-ton) Load	4.8-Tg (40 million-ton) Load	7.2-Tg (60 million-ton) Load
50	4.3	4.2	2			
1.0	7.8	7.7	100			
0.1	8.9	8.8	1000	7.5×10^{-4}	3.7×10^{-4}	2.5×10^{-4}
0.01	9.4		10^4	7.5×10^{-3}	3.7×10^{-3}	2.5×10^{-3}
0.001	10.7		10^5	7.5×10^{-2}	3.7×10^{-2}	2.5×10^{-2}
10^{-4}	11.4		10^6	7.5×10^{-1}	3.7×10^{-1}	2.5×10^{-1}
10^{-5}	12.1		10^7	7.5	3.7	2.5
10^{-6}	12.7		10^8	75	37	25
10^{-7}	13.2		10^9	750	375	250

Notes: 1 kN·m = 8852 lbf·in.
Mean moment = 4.3 kN·m (38 400 lbf·in) and SD = 1.5 kN·m (13 200 lbf·in).

*Based on 3700 axles/d for 2.4-Tg annual traffic.

Figure 8. Load-dependent distribution of ballast pressure on bottom of load-cell tie.



seat region. However, they are considerably lower than the 22.5 kN·m (200 000 lbf·in) negative and 10.2 to 12.4 kN·m (90 to 110 00 lbf·in) positive strength requirements in current specifications.

The data from the individual load categories show that the bending moment at the tie center is practically independent of car load for many ties. This indicates a nonlinear support condition in which the distribution of reaction loads along the tie length changes with load to maintain a relatively constant bending moment. For example, a center-bound tie that has voids under each end but is supported in the middle will develop negative bending at both the center and the rail seats with light loads. However, increased wheel loads will cause the tie to bear more fully on the ballast and shift the reaction load toward the tie end. This will produce positive bending at the rail seat and very little change in the bending moment at the tie center.

Tie-Ballast Pressure Distribution

The load-dependent support condition observed in the bending moments of several concrete ties was confirmed by load-cell-tie data. The graph of tie-ballast pressures along the tie length (Figure 8) shows a noticeably center-bound condition for light wheel loads [35.6 kN (8000 lbf)], whereby most of the tie load is supported by the middle of the tie. But for higher wheel loads [89 to 160 kN (20 000 to 36 000 lbf)] on the same tie, the peak pressures move toward the rail-seat region. This load-dependent behavior indicates that the high ballast pressures from heavy cars are causing voids in the rail seat region under the ties.

Recent results from repeated-load laboratory tests at the PCA and Queen's University in Canada have confirmed this load-dependent behavior for different size concrete ties. Reducing the variation in pressure distribution on the ballast and subgrade under ties may be a key factor in improving track performance. This is particularly important for track that has poor drainage or very moisture-sensitive subgrades. Under these conditions, depressions or ruts in the subgrade in the rail-seat region will retain moisture and the rate of track settlement will increase greatly. Changes in tie design, reduced tie spacing, and increased ballast depth are possible ways to reduce this mode of degradation.

Effect of Tie Spacing

The data given above for the maximum (0.1 percent exceedance) loads measured at each test site showed that, in most cases, the maximum tie loads and bending moments measured at site two, which has 0.51-m tie spacing, were not significantly lower than those measured at site one, which has 0.61-m spacing. Reducing the tie spacing from 0.61 to 0.51 m, a 16 percent reduction, is normally expected to reduce the ver-

Figure 9. Effect of train speed on average vertical and lateral wheel-rail loads: all traffic at site 1.

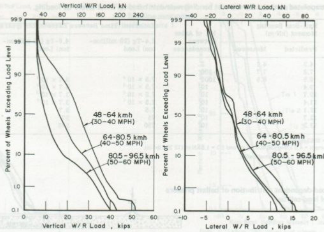
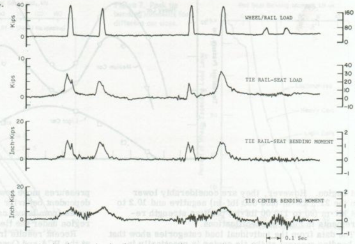


Figure 10. Time history of track loads for four-axle locomotive.



tical rail-seat loads and tie bending moments by about 16 percent. However, the large tie-to-tie variation in support conditions makes it difficult to compare results for different track designs by using single-tie measurements. It is more appropriate to average the data for identical measurements at several different locations to include these typical spatial variations.

The percentage changes in the average mean and 0.1 percent load levels caused by reducing the tie spacing from 0.61 to 0.51 m are given below.

Item	Change in Avg Mean			Change in Avg 0.1 Percent Load		
	All Cars	Locomotives	Heavy Cars	All Cars	Locomotives	Heavy Cars
Rail-seat vertical load	8.9	18.5	16.9	8.8	30.2	20.6
Tie-rail-seat bending moment	36.4	24.8	36.5	12.0	19.0	33.2

These data demonstrate the difficulties in reaching definitive conclusions by using track-response measurements. Reducing tie spacing by 16 percent reduces average and maximum vertical rail-seat loads by about 9 percent for all traffic. Average tie bending moments at the rail seat were reduced more than were rail-seat loads. This indicates a nonlinear support condition in which the reduced tie loading provides a substantially greater reduction in both average mean and average 0.1 percent bending moments; the maximum bending moments are reduced by 12 percent and the average mean is reduced by 36 percent for all traffic. It should be noted, however, that there is no difference in the maximum rail seat loads and tie bending moments for the most severely loaded tie at the different tie spacing locations although there should be fewer ties subjected to these maximum loads in the section that has 0.51-m spacing.

Many of the measured data indicate that nonlinear support conditions have a very significant effect on

Figure 11. Time history of track loads for lightly loaded freight car that has wheel flats.

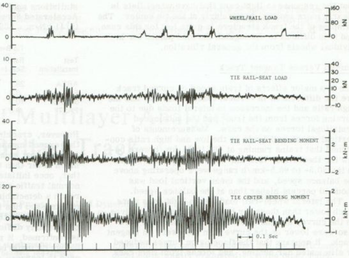


Table 2. Average track component loads: all traffic on track having 0.61-m (24-in) tie spacing.

Item	Tangent Track	Curved Track (high rail)
Vertical wheel-rail load (P), kN		
Avg mean	74.7	65.4
Avg SD	35.6	36.5
Avg 0.1 percent load	185	178.4
Rail-seat vertical load (Q)		
Avg mean, kN	29.4	40.4[a]
Avg SD, kN	18.2	26.2
Mean ratio, Q/P	0.39	0.62
Avg 0.1 percent load, kN	85.8	121.4
Rail-seat moment, kN·m		
Avg mean	0.06	0.29[a]
Avg SD	0.44	0.67
Avg 0.1 percent load	1.4, 1.3	2.4, 1.8
Tie-rail-seat bending moment (M_{rs})		
Avg mean, kN·m	1.7	1.9
Avg SD, kN·m	1.0	1.2
Mean ratio, M_{rs}/P	0.923	1.16
Avg 0.1 percent load, kN·m	4.5	5.9
Tie-center bending moment (M_c), kN·m		
Avg mean	1.0	1.1
Avg SD	0.72	1.3
Avg 0.1 percent load	3.3, 1.2	4.8, 2.7

Notes: 1 kN = 225 lbf and 1 kN·m = 8852 lbf·in.
Average 0.1 percent load levels predicted from average mean and SD by assuming normal probability distribution; i.e., 0.1 percent load = mean ± 3.1 (SD).

[a] Average based on data for only two instrumented tie plates.

track loads. The results suggest that if the population of heavy cars becomes a greater portion of revenue service, i.e., if there are more unit trains of 63.50 and 91.0-Mg (70- and 100-ton) hopper cars, changes in tie spacing might have a much greater effect on tie moments than would be normally expected by using conventional track design estimates. Therefore, although a reduction in tie spacing might provide a large benefit, an increase might cause an unexpectedly large increase in tie bending moments. This suggestion requires additional evaluation because the effect of these variations in tie support conditions cannot be predicted for an increase in average wheel load.

Effect of Train Speed

A review of the mean values of vertical wheel-rail loads in the different speed categories showed the somewhat unexpected result that the average loads in the 48- to 64-km/h (30- to 40-mph) low-speed range were as much as 50 percent higher than the all-traffic average. Further investigation showed that this was caused by the fact that trains that have very heavily loaded cars operate at lower speeds past the test site than do trains that have a higher percentage of lightly loaded or empty cars. This type of speed effect reflects railroad operations rather than vehicle dynamics. It is not known whether this is typical of operations at other track sites on the FEC or on other railroads.

Speed effects related to vehicle dynamics can be evaluated only by using data for a common type of vehicle. Measured variations in mean vertical loads for identical locomotives operating at different speeds were less than 5 percent from the mean for all speeds. It was concluded from this that the effect of operating speed on vertical track loads from vehicle-dynamic effects was negligible on the FEC tangent track test sites.

Figure 9 shows the effect of train operating speed on the vertical and lateral wheel-rail loads. It is evident that the vertical-load bias in the 48- to 64-km/h range was responsible for the fact that that speed also caused the highest lateral loads on an all-car basis. This is true also for the heavy-car category alone. However, data for light cars, where the load bias versus speed was small, showed that the highest lateral loads occurred above 80.5 km/h (50 mph) and the lowest lateral loads occurred at 48 km/h. This is indicative of hunting cars. Other investigators (4) have confirmed that lightly loaded and empty freight cars have a lower hunting critical speed than have heavy cars.

Effect of Wheel-Flat Impact Loads

Recordings of track-load time histories showed considerable vibration, especially from the impacts of wheel flats. Data from FEC indicate that about 10 percent of the car wheels have flats of sufficient size to excite noticeable vibration but that a much smaller portion of these cause loads that exceed the normal load for a heavily loaded car. Figure 10 shows load data for a locomotive and demonstrates track response to heavy cars that have no apparent wheel flats. Figure 11 shows that the response to light cars that have wheel flats is clearly more severe, particularly at the tie center. The damping of the track structure is quite low for this case, and it is difficult to distinguish the load pulses for individual wheels from the general vibration.

Curved Versus Tangent Track

The two major effects of train speed on curved track are the differences in vertical loads on the low and high rails and the increases in lateral loads due to the curving forces from the truck and the unbalanced centrifugal forces on the cars. Measurements of vertical wheel-rail loads on the low and high rails confirmed that trains running at 48- to 64-km/h were below the theoretical 72.4-km/h balance speed. Trains in the 80.5- to 96.5-km/h range were operating above the balance speed, and the mean vertical load was about 10 percent higher than at the balance speed.

The lateral wheel-rail loads from light cars were much lower than those for heavy cars and locomotives on the curve, and the lateral loads for the light cars also were lower on the curve than they were on tangent track. It appears that the flanging on curves reduced or eliminated car hunting, and forces from light cars due to truck curving were much lower than those from hunting.

Table 2 summarizes the overall statistics for all traffic (all cars, all speeds) at the curve site and compares these to the same data for the tangent site (site one) that has the same 0.61-m tie spacing. The major differences between the two sites are that the average tie bending moments at the 0.1 percent exceedance level are 25 percent higher at the rail seat and 50 percent higher at the tie center than they were on tangent track even though the mean bending moments were nearly identical. This is a result of the increase in load variation (SD) that occurs in the curve from trains operating both below and above the balance speed. The significance of the higher variability of loads in the curve is that the low-probability high loads will exceed those on tangent track even though the mean loads will be quite similar.

SUMMARY AND CONCLUSIONS

Data from measurements of rail and tie loads on concrete-tie track were used to develop a statistical description of track loading for typical railroad service. This description can be used to evaluate performance specifications for concrete ties and fasteners and to validate track analysis models for predicting the effects of tie spacing, ballast depth, and tie size on track loads.

Typical mean rail-seat loads were on the order of 40 to 60 percent of the mean vertical wheel loads, depending on tie spacing and whether the track was tangent or curved. Data from adjacent ties showed considerable tie-to-tie variations in support condition.

Data on tie bending moments and tie-ballast-interface pressure distributions indicated a strong load-dependent response. There was a noticeable center-bound support condition for light wheel loads, but the support shifted toward the rail-seat region for heavy wheel loads. The high ballast and subgrade pressures from heavy cars evidently cause voids or depressions in the roadbed under the rail-seat region of the ties.

Tie moments from revenue traffic on the FEC were considerably lower than the current flexural strength requirements, even for a probabilistic estimate of maximum loads for a 50-year life. Similar conclusions can be made based on tie-load data from other test installations such as at Streator and the Facility for Accelerated Service Testing (FAST) at Pueblo, Colorado (5) (1 kN·m = 8852 lbf·in).

Test Installation	Tie Bending Moment (kN·m) Rail Seat (+)	Center (-)	Center (+)
AREA	28.2	22.5	10.1
FEC	8.8	6.3	7.5
Streator	10.9	8.1	
FAST	9.0	14.6	

However, cracking of ties having static flexural strengths that exceed measured loads has persisted. It is conjectured that small cracks may be initiated at loads much lower than the static strength requirements and that, once initiated, the repeated fatigue loading of normal traffic will cause the cracks to grow until they reach a detectable size. Locating small cracks in prestressed ties is practically impossible, and this makes the investigation of the crack-initiation mechanism particularly difficult. However, if a fatigue mechanism is confirmed, it may be possible to improve tie life by design or material changes that are different from those used to increase ultimate strength.

The necessity for eliminating tie cracking has not been verified by service experience, and preliminary results of tests at FAST that used precracked ties indicate no major structural failure after 6 Tg (50 million tons) of traffic. The reason cited most frequently for the elimination of cracking is that a crack that reaches the prestress tendons will eventually cause bond failure from the cyclic loading of normal traffic. Other problems that could result from cracking are corrosion of the metal tendons and concrete damage from freeze-thaw cycles.

ACKNOWLEDGMENT

This paper is based on the results of a research project conducted by Battelle Columbus Laboratories and Bechtel. The research was sponsored by the U.S. Department of Transportation, Federal Railroad Administration, through the Transportation Systems Center. We gratefully acknowledge the assistance and enthusiastic cooperation of the staffs of Battelle Columbus Laboratories, Bechtel, and FEC. This paper presents our views and positions and does not necessarily reflect those of the U.S. Department of Transportation.

REFERENCES

1. Specifications for Concrete Tie and Fastening. Proc., AREA, 655, Nov.-Dec. 1975, pp. 193-236, and Bull. 660, Nov.-Dec. 1976, pp. 133-137.
2. R. H. Prause, H. C. Harrison, and R. C. Arnlund. Measurement Plan for the Characterization of the Load Environment for Crossties and Fasteners. Battelle Columbus Laboratories, Columbus, OH; Federal Railroad Administration, Rept. FRA/ORD-77/03, April 1977.
3. R. H. Prause, H. D. Harrison, J. C. Kennedy, and R. C. Arnlund. An Analytical and Experimental Evaluation of Concrete Crosstie and Fastener Loads. Battelle Columbus Laboratories, Columbus, OH; Bechtel, San Francisco; Federal Railroad Administration, Rept. FRA/ORD-77/71, Dec. 1977.
4. D. R. Ahlbeck, H. D. Harrison, and S. L. Noble. An Investigation of Factors Contributing to Wide Gauge on Tangent Railroad Track. Journal of Engineering for Industry, Trans., ASME, Vol. 99,

Series B, No. 1, Feb. 1977, pp. 1-9.
5. A. Kish, D. P. McConnell, R. M. McCafferty, H. Moody, and A. Sluz. Track Structures Performance: Comparative Analysis of Specific Systems and Component Performance. Federal Railroad Administration, Rept. FRA/ORD-77/29, June 1977.

Development of Multilayer Analysis Model for Tie-Ballast Track Structures

James C. Kennedy, Jr., and Robert H. Prause, Applied Dynamics and Acoustics Section, Battelle Columbus Laboratories, Columbus, Ohio

A multilayer analysis model for tie-ballast track structures has been developed. The model includes the effects of rail bending, rail-fastener stiffness, tie bending, variable ballast and subgrade material types, and variable tie spacing and ballast depth. The results predicted by using the model are compared with experimental results and excellent agreement is shown. The model offers the advantages of simplicity of use and reduced computer run time when compared with the finite-element codes currently used.

The evaluation of track performance and track design for vertical loads requires the ability to predict realistic pressure distributions at the interfaces between the tie and the ballast and between the ballast and the subgrade. This requires a model that includes the effects of tie bending; rail-fastener stiffness; and changes in ballast depth, roadbed material properties, and tie spacing in a unified manner. In such a model, changes in roadbed configuration that affect track moduli and the distribution of loads from the rails to individual ties are apparent.

A track model and computer code that incorporates the above features has been developed. This paper compares its ease of use, computer time required per run, and accuracy of results with those of other existing analysis codes. Analytical validation and a comparison of computer predictions and experimental results are also presented.

The Multi Layer Track Analysis (MULTA) computer routine discussed here is a two-stage numerical procedure for determining the three-dimensional load and stress distribution in a railroad track system subjected to static loads.

MULTA can be used to evaluate new or existing track-system configurations for various combinations of concentrated vertical loads or moments exerted on either or both rails.

TYPICAL METHODS OF ANALYSIS OF TRACK STRUCTURES

Currently, the analysis of track structures usually follows one of two paths: (a) the track structure is represented very simply (e.g., a beam on an elastic foundation wherein the substructure is represented as a series of discrete springs) or (b) the track structure is modeled in great detail by using a finite-element representation. In the first case, the system is represented so simply that individual contributions (such as ballast material type and depth, subgrade material type, and tie bending) are not sufficiently detailed or easily evaluated. On the other hand, the detail characteristic of most finite-element codes requires preparation of input data and running time for computer analysis of such magnitude that extensive analyses are quite often prohibitive.

A finite-element code was selected that could simulate variable ballast depth and material type and subgrade depth and material type so that the results obtained by using it could be compared with those obtained by using MULTA. MULTA is not a finite-element code as such; the differences between it and a typical finite-element code will be pointed out below. The finite-element code used for this comparison was the prismatic solid analysis (PSA) code originally developed at the University of California, Berkeley, and modified by the Association of American Railroads (AAR). The comparison between the results obtained by using the two codes showed negligible differences in predicted stresses and displacements. [A complete description of the PSA code and the comparison have been given by Prause and others (1)].

Typically, the preparation of input data for use in MULTA requires considerably less time than do seemingly equivalent finite-element codes. In the results that are discussed below, 11 ties are used in the simulation of the track structure. Preparation of input data for MULTA, including punched data cards, required about 3 person-h. Running time required about 400 computer s. On the other hand, the preparation of input data for the analysis that used the PSA finite-element code required about 8 person-h preparation time and about 750 s computer run time. Thus, the MULTA program has the advantage of being able to simulate and evaluate the effects of parameters such as ballast depth and material type, subgrade material type, tie bending, and rail-fastener stiffness where similar analysis codes (such as the beam-on-elastic-foundation formulation) do not. On the other hand, its relative ease of input-data preparation and considerably smaller amount of computer run time offer definite advantages over the more detailed finite-element codes without compromising the results for a vertical linear-elastic track-analysis tool.

The results predicted by using the MULTA code have also been compared with those predicted by using the ILLI-TRACK structures code. This is a two-dimensional finite-element code developed at the University of Illinois (2). The comparison shows that ballast pres-