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Bridge Approaches and Track Stiffness

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13. ABSTRACT (Maximum 200 words)
Over time, it is not uncommon for a dip in the track to develop off the end of a bridge—on the bridge approach. This dip, or bump at the end of the bridge, is often a rough-riding spot and one that requires resurfacing at more frequent intervals than does the rest of the track. A commonly held belief is that this dip in the track is caused by dynamic forces resulting from wheel loads crossing an abrupt change in stiffness between the track on the bridge and the track off the bridge. A literature search found no documented field measurements to support this belief, only two modeling studies which predicted that the effect would be negligible. Five different methods were then employed to evaluate the potential for a track stiffness change to produce dynamic loads of practical significance. These methods ranged from the most technically sophisticated to the most basic. The results from all five pointed to the same conclusion—that changes in track stiffness at a bridge end have no practical effect on either track settlement or ride quality at a bridge approach.

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1 yard (yd)	=	0.9 meter (m)	1 meter (m)	=	3.3 feet (ft)
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1 pint (pt)	=	0.47 liter (l)			
1 quart (qt)	=	0.96 liter (l)			
1 gallon (gal)	=	3.8 liters (l)			
1 cubic foot (cu ft, ft^3)	=	0.03 cubic meter (m^3)	1 cubic meter (m^3)	=	36 cubic feet (cu ft, ft^3)
1 cubic yard (cu yd, yd^3)	=	0.76 cubic meter (m^3)	1 cubic meter (m^3)	=	1.3 cubic yards (cu yd, yd^3)
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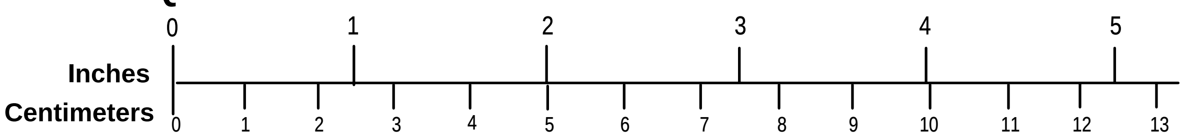

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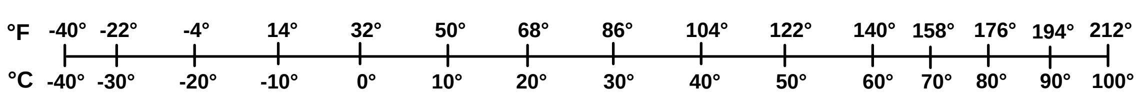

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Executive Summary

Bridge approaches are a common source of poor ride quality and often require more frequent resurfacing due to track settlement within about 50 feet from the bridge end. The source of the problem is often said to be the difference in stiffness between the stiffer track on the bridge and the less stiff track off the bridge. As the less stiff track will experience greater vertical deflection under load, this has the effect of creating a track surface deviation (a bump in the track) at the point where track stiffness changes.

As is well known and documented, track surface deviations cause dynamic loads or impacts when trains pass over them. In addition, track subjected to higher loads will settle more than track not subjected to these loads.

Numerous attempts have been made to eliminate the bridge approach track dip by reducing the track stiffness difference or by creating a more gradual stiffness transition; yet none of these have worked to any great degree. This inability to eliminate the bridge approach problem prompted a study to examine the track stiffness difference concept to determine why its past application had been repeatedly unsuccessful.

For a stiffness difference to cause track settlement or have an adverse effect on ride quality, it must produce dynamic forces large enough to have these effects. While the stiffness difference theory itself appears unquestionably correct, a literature search found no evidence documenting the magnitude of its actual effect in track. The literature search did find results of two modeling studies, which predicted that the stiffness change effect would be negligible.

The following five methods were then employed to evaluate the potential effect of track stiffness difference, ranging from the most technically sophisticated to the most basic:

1. Analysis with the NUCARS® track/train dynamics computer program
2. Examination of vertical force and carbody acceleration data collected with instrumented wheelsets
3. Examination of settlement and top-of-rail profile data at bridge approaches
4. Calculations using simple beam deflection equations and beam-on-elastic-foundation track deflection theory, with the results compared to common railroad track maintenance criteria
5. The train ride test—a simple but revealing observation that can be made aboard a moving train

The results from the literature search and the five analysis methods all pointed to the same conclusion—that changes in track stiffness have no practical effect on ride quality or track settlement at bridge approaches.

1. Introduction

1.1 Background

Over time, it is not uncommon for a dip in the track to develop off the end of a bridge—on the bridge approach, as shown in Figure 1. This dip, or bump at the end of the bridge, is often a rough-riding spot and one that requires resurfacing at more frequent intervals than does the rest of the track.

Figure 1. Track settlement off a bridge end

To estimate the extent of the occurrence of bridge approach roughness, the Transportation Technology Center, Inc. (TTCI) commissioned a survey to sample railroads from North America, Australia, and Europe on this subject (Briaud, et al., 2006). Responses from 11 railroads, including 4 major North American carriers, indicated that approximately 50 percent of bridge approaches were said to develop a low approach which adversely affected ride quality, frequently required a speed restriction, and generally required above average surfacing maintenance. Responses indicated the formation of low spots on bridge approaches ranging from 1/4 to 4 in. in depth and from 4 to 50 ft in length.

In an Association of American Railroads (AAR) project to measure freight car load environment, dynamic loads from a loaded coal gondola were recorded while traveling from the mine to a power plant—a trip of more than 1,000 miles. A description of the locations causing high vertical and lateral forces was provided. During the trip, four locations registered vertical loads above twice the static wheel load level. Of these four, three were described as bridges or culverts (Koch, 2007). The fourth was a turnout. All had in common a significant track surface deviation (at or near Federal Railroad Administration (FRA) track safety limits for the track speed operated).

A commonly held belief is that low spots in the track at bridge approaches are caused by dynamic forces resulting from wheel loads crossing an abrupt change in stiffness between the track on the bridge and the track off the bridge. Figure 2 shows one case of the measured difference in track stiffness (or track modulus) between the track on a ballast deck bridge and the track off the bridge. In this figure, data is averaged over a 30-ft moving window.

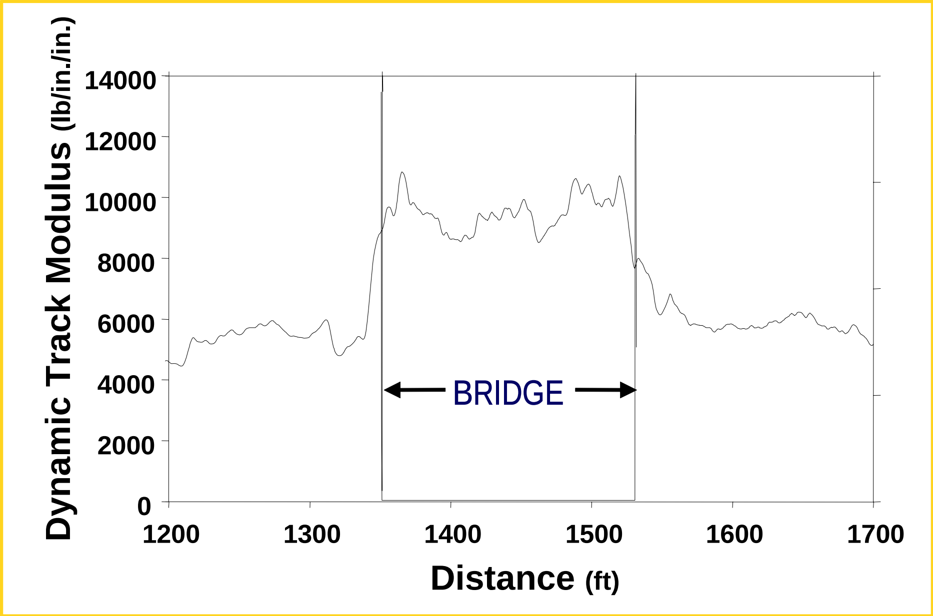

Figure 2. Measurements of track modulus at a ballast deck bridge

Many attempts have been made to remedy the bridge approach problem by trying to reduce the abrupt stiffness change. These attempts have included increasing the track stiffness on the approach by using gradually longer ties or by installing a concrete or asphalt pad below the ballast, or reducing the track stiffness on the bridge by installing rail seat pads. None of these methods have produced much success.

This inability to eliminate the bridge approach problem prompted a study to examine the track stiffness difference concept to determine why its past application had been repeatedly unsuccessful and, further, to find effective and affordable methods for remedying ride quality and track settlement problems at bridge approaches and other track transitions—locations where track construction changes, as at bridges, turnouts, road crossings, and track crossings. This report presents the first product from the study.

1.2 Objective

The objective of this first part of the bridge approach and track transition study was to quantify the effect that changes in track stiffness at bridge ends might have in causing track settlement and adversely affecting ride quality at bridge approaches.

1.3 Approach

No field tests were conducted specifically for this work. The approach was to first see if existing data and analysis methods could provide a satisfactory answer to the question being investigated. To compensate for the absence of targeted field tests, the plan was to look at the possible effect of track stiffness difference using several different methods. If the results from each were in sufficient agreement and collectively, sufficiently conclusive, then field tests would not be necessary.

1.4 Scope

This phase of the study examined only the possible effect that abrupt changes in track stiffness might have in producing dynamic loads of sufficient magnitude to cause additional track settlement or adversely affect ride quality. The study did not cover other applications of track stiffness theory or measurements.

1.5 Track Stiffness and Track Modulus

Track stiffness, its resistance to deflection when subjected to a vertical load, is typically indicated by a measure called track modulus. Track modulus is most commonly expressed in units of pounds required to cause a 1-in length of track to deflect 1 in.

Track modulus is not a fixed value. For any location, it will typically vary over the course of 1 year, often by a factor of 2 or more. In areas of the United States that have cold winters and snow, track modulus is highest when the ground is frozen and during long dry periods. In these areas, track modulus is usually lowest during the spring thaw and after long periods of heavy or steady rain when the subgrade is at or near saturation from surface water infiltration and/or a temporary rise in the water table. Table 1 shows common ranges of modulus values for well-maintained track on a primary main route and during periods when ballast and subgrade are not frozen and the subgrade is near its average moisture content.

Table 1. Typical track modulus value ranges for track on main routes

Track Type	Typical Modulus Range
Wood Tie Track	2,000 to 3,500
Concrete Tie Track	3,500 to 6,000
Track on a Concrete Ballast Deck Bridge	8,000 to 12,000

The values in Table 1 can serve as references when stiffness differences are examined in following sections.

1.6 The Track Stiffness Difference Theory

As normally built, track on a bridge is relatively stiff, being supported by a rigid bridge structure. Track off a bridge is supported by an earth subgrade, which is not so rigid as a bridge and thus permits more deflection when subjected to the same load. In level track, with no load present, the top of rail off the bridge is even with that on the bridge. As a loaded wheel rolls over the end of a bridge and passes from the stiffer track on the bridge to the less stiff track off the bridge, however, its weight causes the less stiff track to deflect further, causing a difference in vertical deflection, which creates a bump in the track—a track surface deviation. Figure 3 illustrates this concept, showing a difference in deflection as wheels of a fully loaded freight car cross a change in track modulus from 10,000 to 2,000 lbs/in/in.

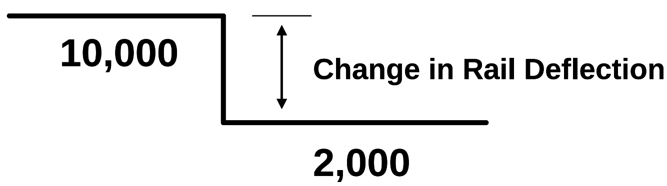

Figure 3. Stiffness difference concept, illustrating an abrupt change in track modulus from 10,000 to 2,000 lbs/in/in

Track surface deviations cause dynamic loads or impacts when trains pass over them. In addition, track subjected to higher loads will settle more than track not subjected to these loads. This is how an abrupt change in track stiffness is said to cause settlement at bridge approaches.

To show that this theory has a practical effect, it must be shown:

- How large dynamic forces must be to cause additional bridge approach settlement or affect ride quality
- That crossing a stiffness change does produce dynamic loads of this magnitude
- That the dynamic forces produced are actually responsible for the increased settlement or adverse effect on ride quality

2. Analysis and Results

To search for evidence that a stiffness difference could cause dynamic forces sufficient to affect track settlement or adversely affect ride quality at bridge approaches, a literature review was conducted, followed by analyses using five different methods.

2.1 Literature Review

A search was conducted to find sources that offered credible information quantifying the stiffness difference effect at bridge ends. Many sources either stated or referred to the theory, but none provided field measurements or other evidence to demonstrate that crossing a stiffness change produced dynamic loads sufficient to cause accelerated track settlement or to adversely affect ride quality. Two sources documented modeling studies conducted to predict the magnitude of dynamic loads that a stiffness change might produce.

In one modeling study, Lundquist and Dahlberg (2005) constructed a finite element model to analyze wheel-rail forces from a 24,000-lb wheel load crossing an abrupt change in track stiffness from approximately 2,775 to 7,000 lbs/in/in at 200 mph. When moving in the direction from the softer to the stiffer track, the model calculated a dynamic increase in wheel load of approximately 2,000 lbs, an 8.3 percent increase over the static load. When moving in the opposite direction, a dynamic wheel load increase of 1,600 lbs resulted, a 7 percent increase over the static load.

In an unpublished study (see Singh, 2003), the NUCARS® track-train dynamics model was used to simulate a loaded 100-ton hopper running at 60 mph on theoretically perfect and uniform track, which included a stiffness change from 2,000 to 30,000 lbs/in/in. In crossing the stiffness change, the model calculated that the wheel-rail force would increase from 32,964 lbs to 35,181 lbs, a 6.7 percent increase.

While these two studies did show that, under certain conditions, a stiffness change could cause some increase in dynamic load, neither indicated how this load increase might affect track settlement. As will be covered in Section 2.3, dynamic forces produced at the rail surface are not always or completely transmitted to the ballast and subgrade and thus do not necessarily have the same effect on track settlement as they do at the rail surface. Section 2.2 will further show that vertical forces on the ballast and subgrade must increase by a certain percentage to produce noticeably accelerated track settlement.

In addition to the general literature review, portions of the Talbot Reports (AREA, 1980) were re-read to determine if any of their content may have suggested the stiffness difference theory or a practical effect from it, as it was these reports that established the concept and application of track stiffness in the United States. In particular, Report 1; Report 2–Part 3; Report 5–Section 57; and Report 6–Parts 5, 6, and 8 deal with the general theory and application of track deflection and track stiffness.[1] Nothing was found in this material to suggest either the stiffness difference theory or its potential to have a practical effect in producing dynamic wheel-rail forces.

2.2 Dynamic Forces and Accelerated Track Settlement

The literature search found 2 model studies which predicted that crossing a stiffness change could cause dynamic wheel-rail forces in the range of 7 percent to 8 percent under certain circumstances, requiring either considerably higher speed or substantially greater stiffness change than would likely occur in current U.S. railroad operations.

What needs to be determined or estimated is how much dynamic force would be required to cause accelerated track settlement, that is, track settlement at a rate noticeably higher than normal or average.

If observation of the track surfacing interval is used as an indicator of track settlement rate, a judgment could be made of the relative surfacing interval that would be considered noticeably shorter than normal or average. It would appear necessary to have an interval at least 25 percent shorter than usual to be clearly noticeable. Thus, for example, in territory typically having a 4-year surfacing cycle, the surfacing requirement at a location would need to be 3 years or less to be considered abnormally frequent.

For track at a ballast deck bridge, how much additional vertical force would be needed to cause a location off the bridge to settle at least 25 percent faster than track on the bridge? Figure 4 shows data from a track settlement model being developed by TTCI (Davis et al., 2007), which is used to provide this estimate. As this graph indicates, about a 50 percent increase in load below the rail would be required to produce what is considered to be a noticeably greater differential track settlement rate.

From this determination, the first observation is that even if all the increased force predicted by the Singh (2003) and Lundquist and Dahlberg (2005) studies, with their extreme conditions, was transmitted through the rail to the ties, it would not be enough to produce accelerated track settlement. By the minimum criteria defined above, to have a noticeable effect, forces would have to be at least six times higher than these studies indicate could be produced.

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[1] Reports of the Special Committee on Stresses in Railroad Track. This work by a joint American Railway Engineering Association (AREA)-American Society of Civil Engineers (ASCE) committee was the first effort in the United States to systemically investigate the nature of various stresses in track. The results appeared in seven reports published from 1918 to 1940, subsequently reprinted as a compilation by AREA in 1980. Included in this work is the introduction of the concepts and significance of track stiffness, track deflection, and track modulus. The work related to vertical track deflection and track stiffness was published beginning with the First Progress Report in 1918 and ending with the Sixth Progress Report in 1933.

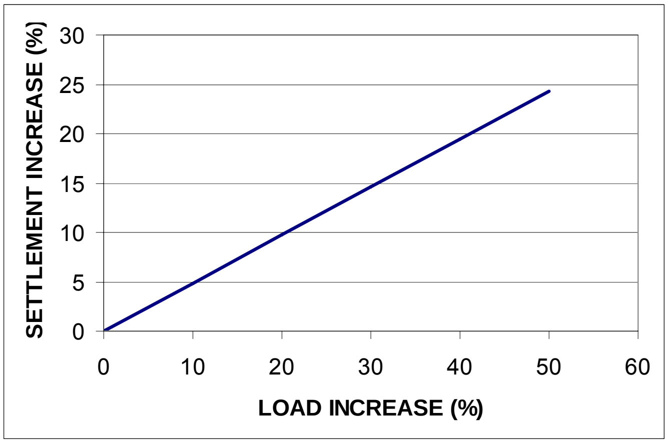

Figure 4. Estimated change in wheel load required to increase differential track settlement rate

2.3 Computer Modeling

The NUCARS® track-train dynamics computer program is designed to compute various forces and accelerations produced by a freight or passenger car moving over the track. It was used to model the effect of an abrupt track stiffness change. Figure 5 shows results of modeling the vertical forces produced below the rail (on the ties) when 286,000-lb freight cars cross a 10,000 to 2,000 stiffness change at 50 mph. In the graph, the vertical scale is a normalized load, with 1.0 equal to the typical force on 1 tie on the bridge as a loaded wheel passes directly over it (roughly equal to about 45 percent to 50 percent of the wheel load). The left half of the graph shows forces on the bridge, with the end of the bridge at tie 192. The wheel is traveling from left to right.

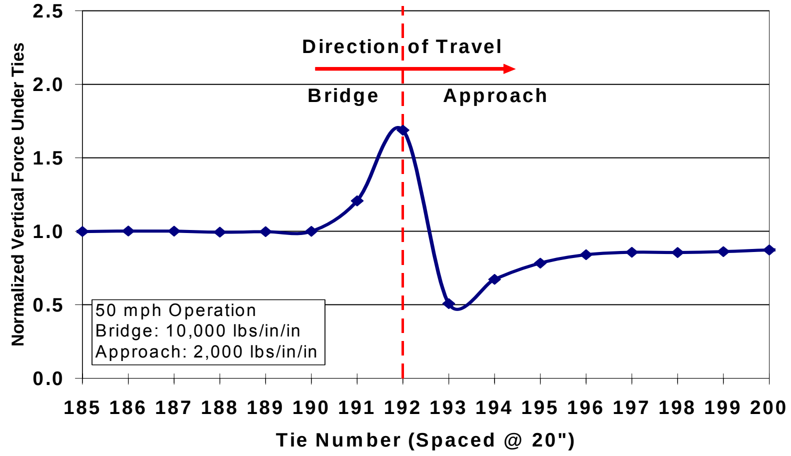

Figure 5. Vertical forces on the ties from a 286,000-lb car crossing a stiffness change

The graph shows that the load increases on the last 2 ties on the bridge, up to about 70 percent above the nominal level, and then drops to about half its nominal value on the first tie off the bridge (tie 193), returning to a steady state by about the third tie past the bridge (tie 195). The main observation from the graph is that in the approximately 13 ft off the bridge, no forces on the ties are either above the nominal level (1.0) or the lower steady-state level off the bridge.

At first, the modeling results in Figure 5 may seem to differ from those reported in Section 2.1 by Singh (2003) and Lundquist and Dahlberg (2005). The differences relate to the location within the track structure where the forces are being measured.

Singh (2003) and Lundquist and Dahlberg (2005) reported forces at the wheel-rail interface. As these will travel both upward into the car and downward into the track, they have the potential to affect ride quality and track settlement. They may, however, have only some or even no effect on either due to the filtering actions of the car's suspension and the rail. To affect ride quality, forces must create an impact large enough in magnitude and long enough in duration to get through the car's suspension. Wheel-rail forces of duration less than about 1/150 of a second (or dynamic forces with frequency above 150 Hz) are not well transmitted to the ballast and subgrade and therefore do not significantly affect track settlement. At shorter durations, forces are dissipated in the rail as vibrations or minute elastic rail deformations (see Eisenmann, 1992, and Jenkins, 1974, p. 5).

What can be concluded from the Singh (2003) and Lundquist and Dahlberg (2005) studies is that if wheel-rail forces increase by the reported 7 percent to 8 percent, forces transmitted to the roadbed cannot increase by more than this, but the amount transmitted to the roadbed may be less.

To assess the potential effect on track settlement, the NUCARS® model was run to estimate the forces transmitted below the rail, on the ties. At this level, even with no change in applied force from above, forces on one tie may differ from that at adjacent ties if support conditions are not the same, which would mean that not all ties are sharing the load equally. The data in Figure 5 should reflect the combined effect of applied loads from the passing train, as well as any unequal load sharing due to the stiffness change.

As the Singh (2003) and Lundquist and Dahlberg (2005) studies resulted in wheel-rail force increases of about 7 percent to 8 percent (but at more extreme conditions, higher speed or greater stiffness change than in this example), this indicates that at least 90 percent of the load increase seen in Figure 5 at tie 192 (edge of the bridge) results from the change in support at that location. The most important figure, though, is the total magnitude of the load on a tie—regardless of the source, as this is what could affect track settlement. As Figure 5 shows, the high load peak occurs at the last tie on the bridge, not on the approach track.

Figure 5 also shows a lower steady-state tie force off the bridge, about 85 percent of nominal (1.0) at ties 195 to 200. This is due to the softer support (2,000 modulus), which has the effect of spreading out the wheel load over a somewhat longer length of track and results in any single tie receiving and supporting a somewhat smaller percentage of the wheel load. This effect is expected and was demonstrated by the Talbot Committee through their numerous field and lab experiments related to vertical track stiffness conducted from 1914 to 1933 (see AREA, 1980).

2.4 Wheel-Rail Force and Carbody Acceleration Measurements

Data were examined from vertical wheel force measurements taken by FRA's research car T-16. This car (Figure 6) is sometimes equipped with instrumented wheelsets, which are the nearest two wheelsets in this picture. Figures 7, 8, and 9 show these wheels in more detail.

Figure 6. FRA's research car T-16

Figure 7. Back of instrumented wheelset, in the shop with cover off (note strain gauges and connecting wires)

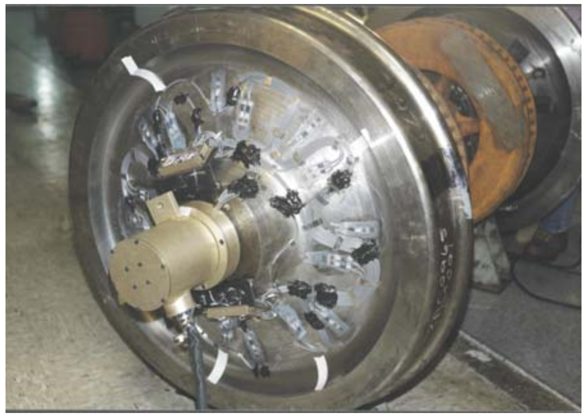

Figure 8. Outside face of instrumented wheelset, in the shop with cover off (note strain gauges)

Figure 9. Instrumented wheelset in place on T-16

Through their numerous strain gauges, as shown in Figures 7 and 8, these wheels are capable of measuring a variety of forces and accelerations, in different directions, at speeds up to 160 mph, with measurements recorded up to 300 times a second (300 Hz). Figure 10 shows the location designation of the four instrumented wheels. The data examined for this report was taken on Track 2 of the Northeast Corridor between Washington and New York on December 2, 2003, with the static wheel loads as given in Table 2.

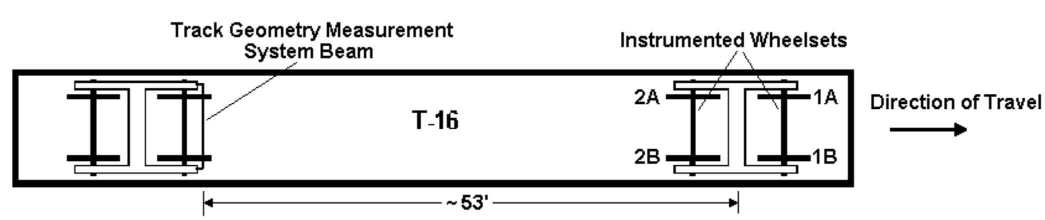

Figure 10. Instrumented wheelset configuration on T-16

Table 2. Static loads on T-16's instrumented wheelsets

Wheel	Static Load	Wheel	Static Load
2A	17,661 lbs	1A	17,298 lbs
2B	16,564 lbs	1B	16,714 lbs

Figures 11 through 18 show data for 400-foot sections of track, selected to show vertical wheel-rail forces when approaching a bridge, while on the bridge, and just past the bridge. This series includes seven ballast deck bridges and two open deck bridges.[2] Vertical carbody acceleration is also shown at two ballast deck bridges (Milepost (MP) 85.18) and one open deck bridge (MP 13.41). The force and acceleration data are filtered at 50 Hz, so any force or acceleration of 1/50 second duration or longer will appear. The system should register any force greater than about 300 lbs. The location error for the bridge ends with respect to the data traces is a maximum of 30 ft. In all graphs, the train is running left to right—northbound, with the bridge location marked below the data, as indicated in each caption. The operating speed at the time data was taken ranges from 84 to 123 mph, as marked on the graphs.

Examination of the vertical force data shows that even in the best track, a dynamic component is always added to the static load. This nominal dynamic addition would be considered the noise level. When running at these speeds, most of the track is typically subjected to this load level. Therefore, this is the reference (or zero) point for examining the significance of vertical forces. In this case, the noise level is at the 20,000-lb level. All of the vertical force graphs have a horizontal line drawn at this level for reference.

As the train is traveling from left to right in all the figures, the most important focus is the 50 feet adjacent to the right (north) end of the bridges. If crossing the bridge-to-track stiffness change does produce dynamic forces of significance on a bridge approach, they should be visible in this area.

In all cases, the forces on the north bridge approach are either at or barely above the noise level. If the bridge locations were not marked, no indication would be apparent from the force measurements that a deviation or change of any kind was present in the track. According to the railroad, no alterations were made to the track to change stiffness on the approaches or on the bridges; all is standard construction.

The 300-ft point in Figure 14 and the 40- to 60-ft range in Figure 17 are examples of an instantaneous force spike and force increases that occasionally occur even in good track and

[2] Bridges are often categorized by the manner in which track is constructed across them. The two main types are open and ballast deck. With an open deck bridge (as seen in the cover photo), the rail is supported by bridge ties (a larger version of track ties) fastened directly to the bridge structure. No ballast is used, and open spaces exist between the ties. With a ballast deck bridge (as seen in Figure 1), a solid floor or deck is used on which a conventional ballasted track section is constructed. Track on a ballast deck bridge is surfaced and lined in the same manner as track off the bridge.

would be considered inconsequential. None of the forces in the north bridge approaches in any of the figures are nearly at these levels.

Figures 11 and 16 include data for vertical carbody accelerations, to provide an indication of ride quality at two ballast deck bridges and one open bridge. While defining ride quality can become complex, as a general indication, an acceleration would need to be higher than approximately minus to plus 0.2g and sustained for at least 1 second before a noticeable sensation would be typically felt. This would require acceleration at or above the top of the vertical scale in the acceleration graphs in these 2 figures, remaining at this level for at least 160 ft in Figure 11 and 130 ft for Figure 16. As shown, the accelerations are far below this level. The acceleration data at the other five bridges was similar.

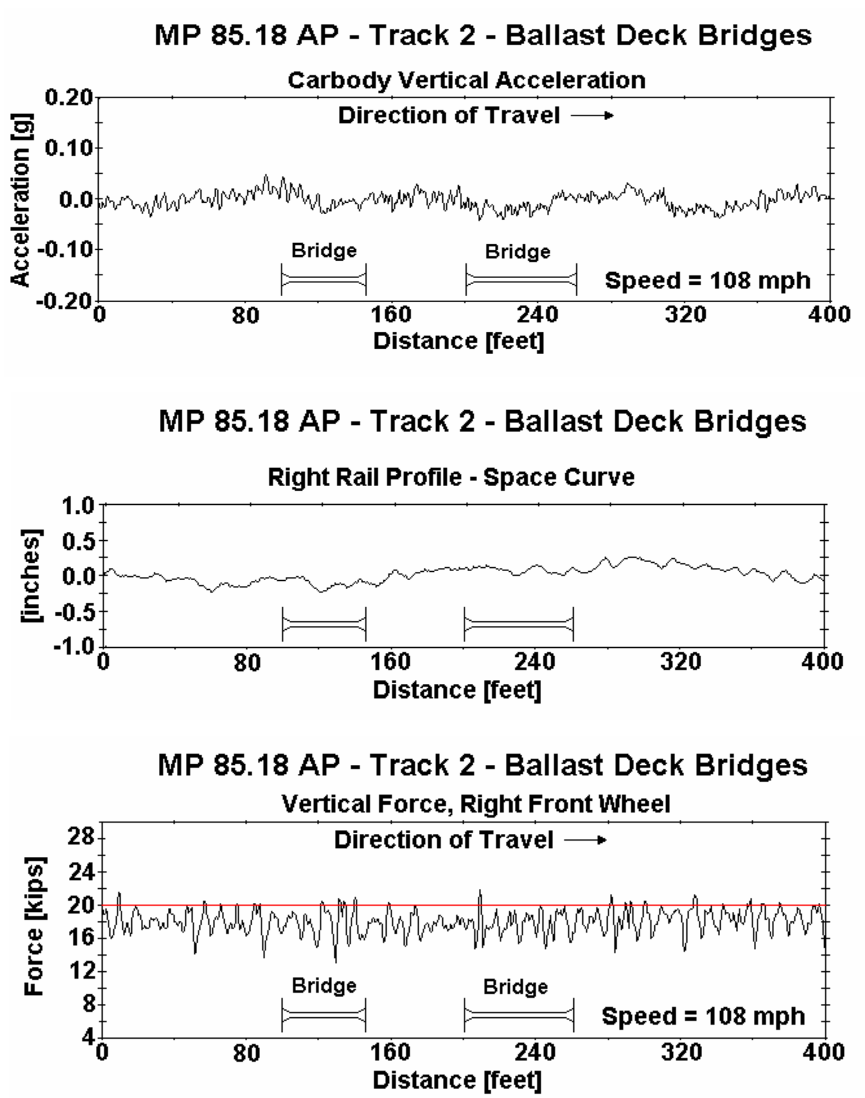

Figure 11. Vertical carbody acceleration, top-of-rail profile, and vertical wheel-rail forces at 2 ballast deck bridges near MP 85.18, about 10.4 miles north of Baltimore, MD

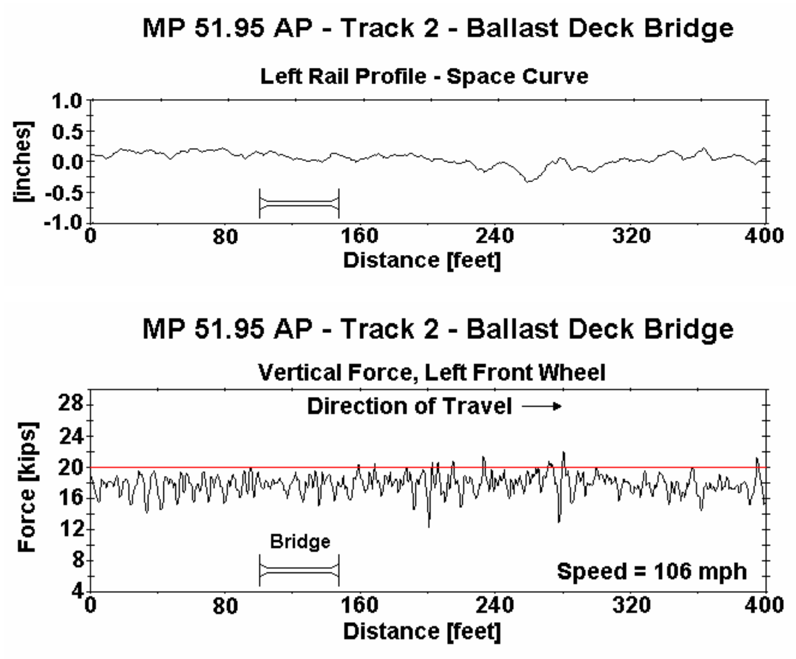

Figure 12. Top-of-rail profile and vertical wheel-rail forces at a ballast deck bridge, MP 51.95, near North East, MD

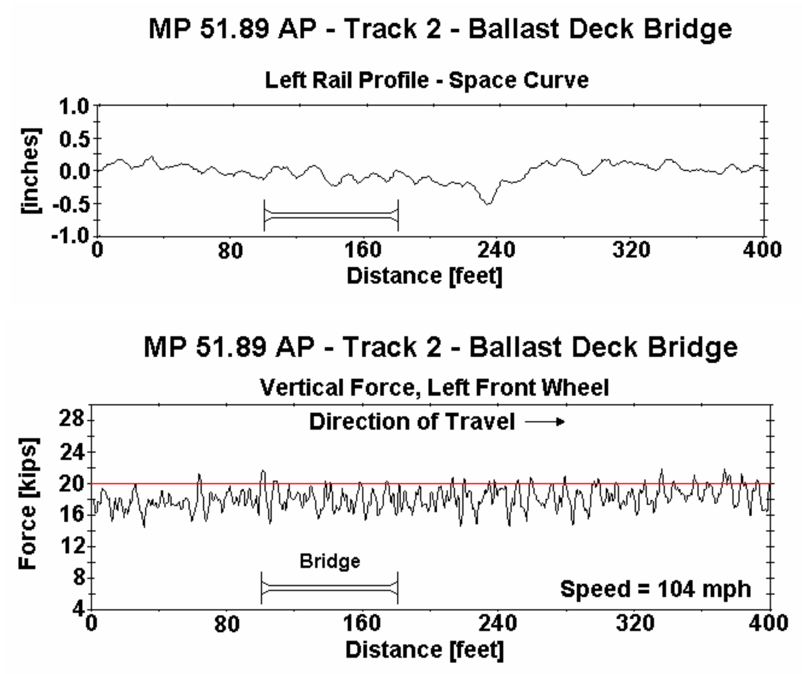

Figure 13. Top-of-rail profile and vertical wheel-rail forces at a ballast deck bridge, MP 51.89, near North East, MD

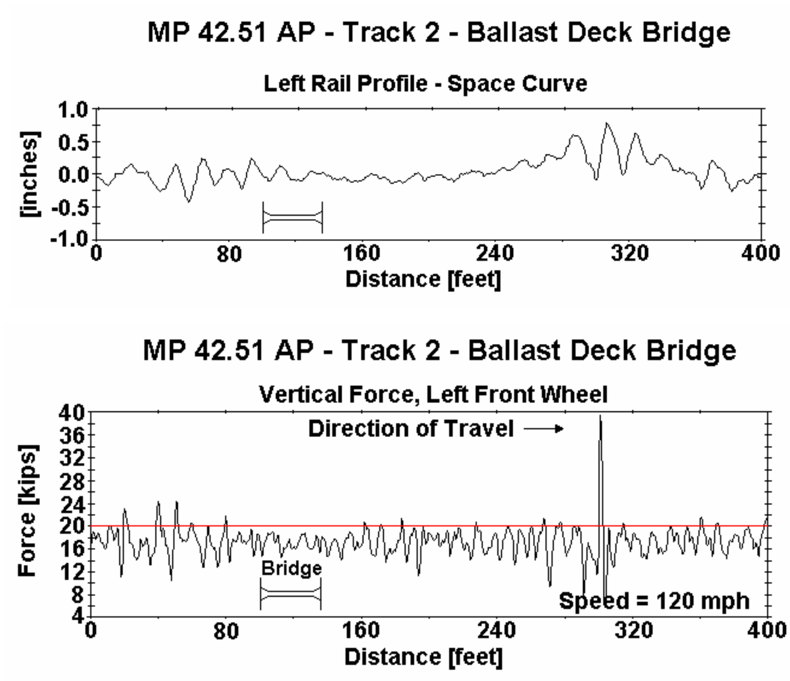

Figure 14. Top-of-rail profile and vertical wheel-rail forces at a ballast deck bridge, MP 42.51, near Elkton, MD

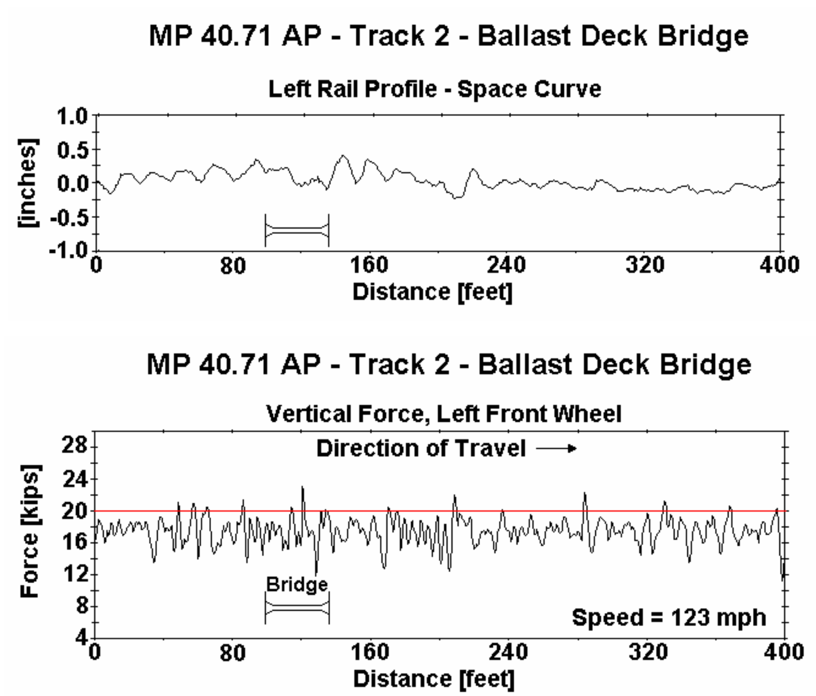

Figure 15. Top-of-rail profile and vertical wheel-rail forces at a ballast deck bridge, MP 40.71, near Newark, DE

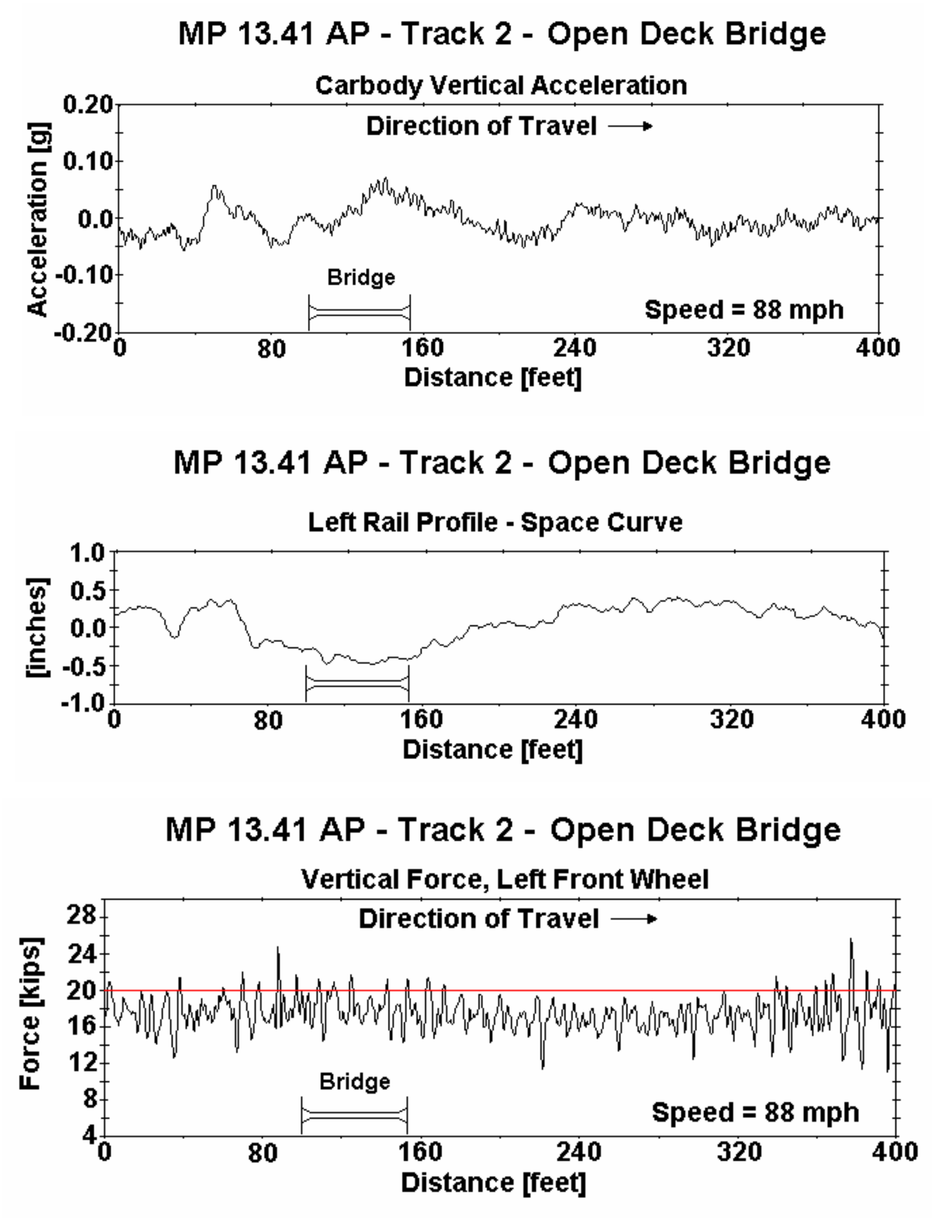

Figure 16. Vertical carbody acceleration, top-of-rail profile, and vertical wheel-rail forces at an open deck bridge, MP 13.41, Chester, PA

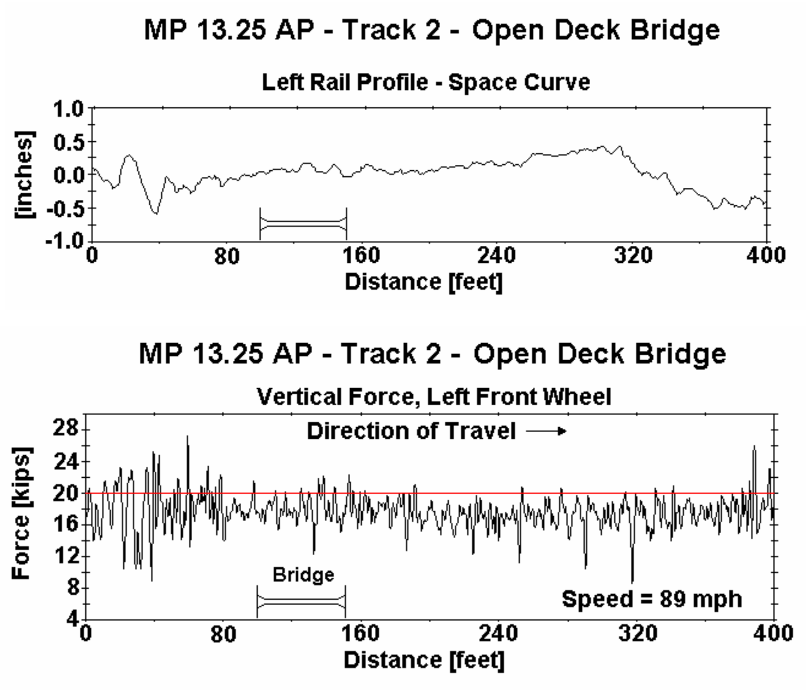

Figure 17. Top-of-rail profile and vertical wheel-rail forces at an open deck bridge, MP 13.25, Chester, PA

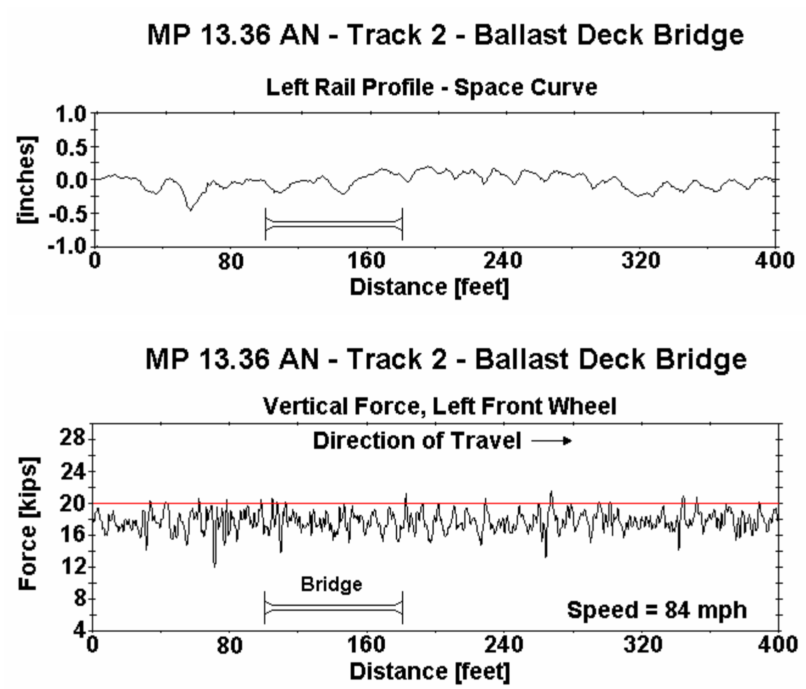

Figure 18. Top-of-rail profile and vertical wheel-rail forces at a ballast deck bridge, MP 13.36, North Elizabeth, NJ

2.5 Bridge Approach Profiles

Track settlement at a location would be greater than it is elsewhere for either or a combination of two main reasons:

- The track support is weaker (for any of numerous reasons).
- Vertical forces are higher.

Of these two, an abrupt change in track stiffness could only affect the second. Thus, if bridge approaches do not typically have stronger support than does the rest of the track, it would be expected that if abrupt changes in track stiffness did cause bridge approaches to be subjected to higher vertical forces, track at approaches would typically settle more than track farther away from the bridge.

From what is known about bridge approaches, these locations have mostly about equal or lower support strength than does track in general, with weaker support often the case where the approach is on a narrow embankment with steep side slopes or is weakened by poor drainage. If the stiffness difference theory is true, it would be expected that the majority of bridge approaches would develop low spots.

A limited examination of bridge approach settlement, as indicated by unloaded top-of-rail profile, showed variation in the approach profile patterns. Observed and measured profiles similar to that shown in Figure 19 were common, in which top-of-rail elevation within about 50 ft or so adjacent to the bridge was sometimes lower than in track farther away but often not. Even on the same route, where traffic was the same and bridge construction similar, track profiles at bridge approaches varied.

The view as seen in Figure 1 was also common. In that picture, a pronounced drop in track elevation off the bridge end is evident, but the top-of-rail elevation near the bridge is not lower than in track farther away, suggesting that track settlement on the approach was not greater than it was elsewhere. To be more definitive, information would be needed about traffic level, predominate direction of loaded cars (if any), and track surfacing history.

Even within the limitations of these observations, however, if the stiffness difference theory was correct, a more consistent appearance of low spots at bridge approaches would have been expected. These observations indicate that bridge approaches often settle at about the same rate as does the rest of the track, which suggests that vertical forces on bridge approaches are often not significantly greater than elsewhere along the track.

Figure 19. Top-of-rail profile (with average gradient removed) for the bridge with stiffness data shown in Figure 2

2.6 Simplified Analysis

Without access to such technology as instrumented wheelsets or track/train dynamics computer programs, is there a simpler way to get a general idea of how large the stiffness difference effect might be? The following method relies on readily available information to accomplish this.

This example will use a stiffness difference represented as the difference between a track modulus of 10,000 lbs/in/in on a bridge and 2,000 lbs/in/in on the adjacent approach track, estimated to be about as large a difference as would usually be found. The initial (unloaded) top-of-rail profile is level; the only changes that occur are due to the loads as the wheels roll over the example section.

Under a passing train, the sudden change in stiffness is said to produce a deflection difference, which, in concept, appears somewhat as shown in Figure 20. For this example, the load on the track will be that from fully loaded 286,000-lb freight cars traveling at 50 mph. The representative wheel load is 41,500 lbs.[3] The track has 136-lb rail.

The most important observation is that, while the support below the rail can have an abrupt step as shown in Figure 20, the rail cannot bend in that fashion. For the case illustrated in Figure 20, the rail surface will appear much closer to that shown in Figure 21; it will form a ramp between

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[3] Recent data from instrumented wheelsets and wheel load impact detectors suggest that good wheels (not out-of-round or with flat spots) on good track with welded rail and traveling near the highest speed allowable for that FRA track class, commonly produce a dynamic load about 15 percent greater than their static load. For a 286,000-lb car, the static wheel load is 36,000 lbs, and the total dynamic load would be 1.15 x 36,000, or about 41,500 lbs. Li (2006) is one example reference showing wheel load data.

the elevation difference. Still oversimplified in this figure, near the ends of the ramp, the rail will of course bend in vertical curves, but the general idea is that the actual wheel path over the stiffness change will be along a ramp and not down a step.

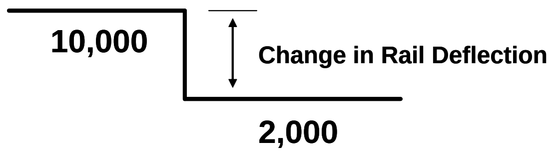

Figure 20. Change in track elevation under load, between track with a modulus of 10,000 and track with a modulus of 2,000

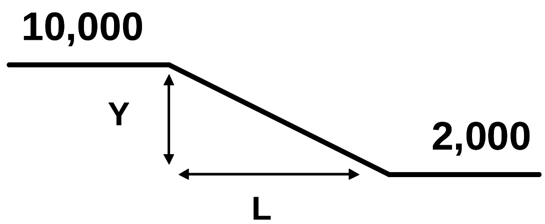

Figure 21. Rail profile ramp when track modulus abruptly changes from 10,000 to 2,000

The next steps are to determine the height of the ramp (Y) and its length (L), which will give an indication of this bump's severity in the track. The ramp height would be the difference between the deflection of the 2,000 modulus track and the 10,000 modulus track. This can be calculated by using the track deflection equation, as found in Hay (1982, p. 249):[4]

$$Y = \frac{P}{(64EIu^3)^{0.25}}$$

Y = vertical rail deflection (in)

P = wheel load (lbs)
41,500 in this example

E = Young's modulus for steel
29,600,000 psi

I = rail moment of inertia
94.2 in^4 for 136-lb rail

u = track modulus

[4] This is the well-known track deflection equation from the theory of representing the track as a beam (the rail) on an elastic foundation.

For the 2,000 modulus track, $Y = 0.213$ in

For the 10,000 modulus track, $Y = 0.064$ in

The difference $= 0.149$ in

Rounding off, the ramp height is then 0.15 in. Some initial thought might be given to how big an effect a bump of this size could possibly have and its potential capability to generate dynamic forces that would affect track settlement along a bridge approach of approximately 50 feet or so in length. One observation is that this vertical difference, even if relatively abrupt, would represent a relatively small track surface deviation.

The analysis now continues toward estimating the ramp length. This will be more difficult, as no simple equation exists for calculating the rail bending length when the strength of the support under the wheel load varies. To produce an estimate, two simplified methods are used to determine what could be called a theoretical worst case—the shortest length over which the rail would probably bend to accommodate a 0.15-in change in support elevation.

The beam-on-elastic-foundation theory once again provides something usable. Figure 22 illustrates the general arrangement for this approach, with the arrow at W representing the wheel load applied to the line representing the flexible rail and the upward arrows under the rail representing the support reaction. X_2 is the distance between the point of maximum rail deflection and the point of zero rail deflection, or about the same as the ramp length L.[5]

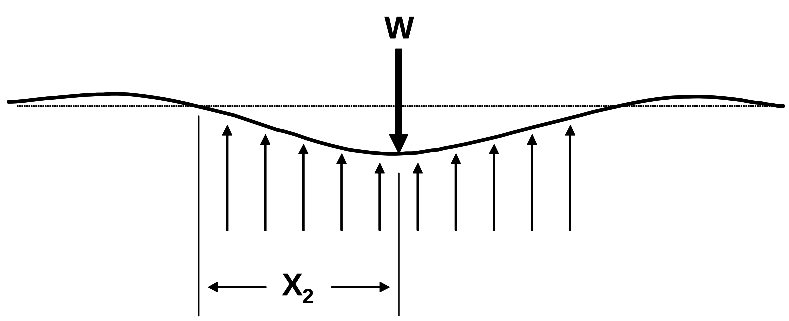

Figure 22. Rail deflection under load for beam-on-elastic-foundation theory

The following equation for X_2 can be found in Hay (1982, p. 250), but this theory does assume that the support under the rail (the track modulus) is uniform along the track segment being loaded. Generally, it should be clear that given the same vertical deflection, the stiffer the track support, the sharper the rail bending will be to return to the zero deflection point. Table 3 shows results from several trial stiffness values. As the track stiffens, X_2 approaches a value in the range of 58 to 61 in, although it takes a stiffness above 25,000 to produce that short a bending length.

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[5] As Figure 22 shows, the rail rises slightly above the unloaded elevation on the far right and left, but no load on the rail occurs at these points. The length of the stiffness change ramp being analyzed is assumed to be as short as the distance X_2.

$$X_2 = 3/4\pi\,(4EI/u)^{0.25}$$

X_2 = distance from point of maximum rail deflection to point of zero rail deflection (in)

E = Young's modulus for steel 29,600,000 psi

I = rail moment of inertia 94.2 in^4 for 136-lb rail

u = track modulus

Table 3. Distance from maximum to zero rail deflection for various track modulus values

Track Modulus	X_2 (In)
10,000	77
15,000	69
20,000	64
25,000	61
30,000	58

Another way to estimate how short the rail bending length could possibly be is to use a simple beam bending formula, with an arrangement as shown in Figure 23. This figure shows the rail as a beam with fixed ends (allowing no rotation) and no support between endpoints. Compared to the actual case in track, the fixed ends would provide more upward support than actually occurs, but the absence of support under the beam is severe in the opposite effect.

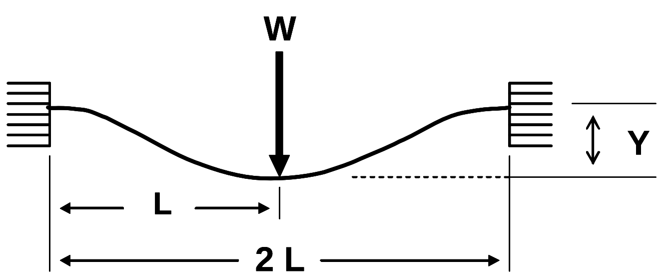

Figure 23. Deflection of beam with fixed ends

The equation for the distance L is:

$$L = (24\,EI\,Y/W)^{1/3}$$

L = distance from beam endpoint to center, the point of maximum deflection (in)

E = Young's modulus for steel 29,600,000 psi

I = rail (beam) moment of inertia 94.2 in^4 for 136-lb rail

Y = vertical rail deflection (in) 0.15 in this example

W = wheel load (lbs) 41,500 in this example

For a center deflection of 0.15 in, L is 60 in. From the 2 methods it appears that a ramp length of about 60 in is a reasonable estimate of the likely shortest length. This results in a ramp in track as shown at the top of Figure 24, with the height and length normalized in the lower illustration—a ramp of 1 in 400.

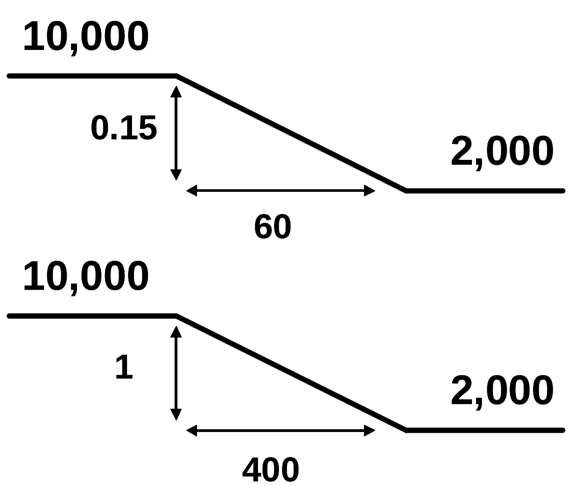

Figure 24. A 10,000 to 2,000 modulus change—a 1:400 ramp

What effect might a 1:400 ramp have on a passing train? One way to estimate this is through a comparison with track surfacing runoff requirements, the minimum length for transitioning from the end of a section of track that has been surfaced and raised to the adjacent track that was not surfaced. Table 4 shows a comparison with runoff standards from one large U. S. railroad. According to the railroad, these runoffs are said to provide a smooth ride at the given speeds.

By the criteria in Table 4, the 10,000-2,000 stiffness change ramp is about equivalent to a runoff that provides a smooth ride at approximately 70 mph—as a probable worst case. If so, what is the likelihood that this track profile would produce vertical dynamic loads sufficient to cause noticeably accelerated track settlement on a bridge approach?

Table 4. Comparing the effect of a 10,000-2,000 track stiffness change with railroad runoff standards

Runoff (or Equivalent)	Maximum Speed for Smooth Ride
1:331	60 mph
1:400	Example 10,000-2,000 Stiffness Change
1:496	80 mph

2.7 The Train Ride Test

Another simple but revealing observation is the train ride test, what can be sensed aboard a moving train while traveling over a bridge and bridge approaches. When the track at a bridge (either open deck or ballast deck) is well surfaced, no vertical roughness or movement can be felt, and only a change in sound will be apparent while traveling over the bridge. This indicates that the change in stiffness clearly has no effect on ride quality. Regarding vertical force level, the car suspension can filter out forces at frequencies that could transmit to and through the track, so no clear conclusion can be made about this aspect. It would, however, raise the question of whether forces large enough to cause abnormally greater track settlement could be generated when not even a slight vertical sensation is apparent from inside a passing train.

3. Summary and Conclusions

Table 5 summarizes the results of the literature review and analyses. The results refer to the possibility that crossing a stiffness difference at bridge ends might:

- Produce dynamic loads large enough to cause accelerated track settlement at bridge approaches
- Adversely affect ride quality

Table 5. Summary of Results

Analysis or Source	Result[6]
General Literature Search	No data showing magnitude of dynamic loads produced by crossing a track stiffness difference.
Review of Talbot Reports	No indication that crossing a track stiffness difference might produce dynamic loads.
Model Studies: • Lundquist and Dahlberg (2005) • Singh (2003) • Gurule and Davis	• A 7 percent increase in wheel-rail load when crossing a 7,000 to 2,775 stiffness change at 200 mph. • A 6.7 percent increase in wheel-rail load when crossing a 2,000 to 30,000 stiffness change at 60 mph. • Crossing a 10,000 to 2,000 stiffness change at 50 mph caused a higher reaction force below the rail on the stiff edge of the transition (the bridge end), but no reaction forces above static load for 13 feet after the transition (on the bridge approach).
Wheel-Rail Force and Carbody Acceleration Measurements	• No wheel-rail forces above the dynamic noise level at bridge ends—data from 9 bridges, at speeds from 84 to 123 mph. • Vertical carbody accelerations appear well below the sensation level.
Bridge Approach Track Profiles	Bridge approach settlement often appeared no greater than in track away from the bridge, suggesting that vertical forces on approaches are often not significantly greater than elsewhere along the track.
Simplified Analysis	The effect of a 10,000 to 2,000 stiffness change is, at worst, roughly equivalent to a 1 in 400 track runoff, said to be good for a smooth ride at 70 mph.
Train Ride Test	Crossing a well-surfaced bridge approach has no effect on ride quality.

The literature search found no sources to support the claim that crossing a change in track stiffness at bridge ends produced dynamic loads of any significance, much less of sufficient

[6] All stiffness values are given in units of lbs/in/in.

magnitude to cause accelerated track settlement at bridge approaches. This finding alone would be sufficient to dismiss the idea, as any such claim would require satisfactory supporting evidence to warrant acceptance.

Following the literature search, five different methods, ranging from the most technically sophisticated to the most basic, were employed to evaluate the possibility that a stiffness difference between the track on a bridge and off the bridge might cause accelerated track settlement or adversely affect ride quality at bridge approaches. To provide a benchmark for comparison, it was estimated that vertical forces on a bridge approach would have to be 50 percent higher than elsewhere on the track to cause noticeably accelerated track settlement (judged to be at least 25 percent greater than average).

These analyses found that any vertical dynamic forces generated by crossing an abrupt track stiffness change are not large enough to be sensed aboard a passing train or to be measured by instrumented wheelsets. Both simple analysis and computer modeling indicate that the ballast and subgrade on a bridge approach are not subjected to increased vertical dynamic forces. Observations of the track profile at bridges indicate that approach settlement is often not greater than in track farther away, which suggests that these approaches are not subjected to higher forces. The results from all five methods pointed to the same conclusion—that even the largest changes in track stiffness do not generate dynamic loads of practical significance and, otherwise, have no practical effect on track settlement or ride quality at bridge approaches.

To clarify, the conclusion is not that changes in vertical stiffness along the track cannot or do not have an effect on track settlement. The computer modeling results shown in Figure 5 clearly indicate such an effect is not only possible, but likely, through highly uneven loading. Even the initial work of the Talbot committee, first published in 1918, indicates this possible effect (see AREA, 1980). The conclusion is that a change in stiffness at one location (the bridge end) cannot and does not result in the generation of dynamic loads that cause or affect track settlement across a bridge approach. As Figure 5 indicates, a change in track stiffness affects track loading only in the vicinity (within about 4-5 ft) of the stiffness change, not on the adjacent 50 ft of track. Further, the localized variation in track loading at a bridge end actually causes a reduced loading on the track immediately adjacent to the bridge, with the offsetting increase in load occurring on the bridge structure.

4. Future Research

This report presented the first product from a study initiated by FRA to find effective and affordable methods for improving ride quality and reducing differential track settlement at bridge approaches and other track transitions.

As this initial study concluded that track stiffness differences had no practical effect on track settlement at bridge approaches, the main source of the settlement difference appears to result from changes in construction. Bridge structures are designed to keep settlement to negligible levels, with support provided by deeply driven piling or piers constructed on bedrock, while track constructed on a crushed rock ballast section over an earth subgrade will experience moderate levels of settlement. As a bridge approach represents a junction between these two construction types, over time, a distinct settlement difference can often occur.

Subsequent work will include further examination of the factors that affect track settlement at bridge approaches, which may accelerate the development of a low spot. These efforts will address both the strength of the track support and the vertical forces acting on the track. The results from this work will serve as a basis to search for effective and affordable methods for providing and maintaining smooth transitions between the track on and off a bridge.

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Abbreviations and Acronyms

AAR	Association of American Railroads
AREA	American Railway Engineering Association
ASCE	American Society of Civil Engineers
FRA	Federal Railroad Administration
ft	foot/feet
in	inch/inches
lb	pound/pounds
MP	Milepost
mph	miles per hour
TTCI	Transportation Technology Center, Inc.