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A Method for Upgrading the Performance at Track Transitions for High-Speed Service

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Next Generation High-Speed Rail Program

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High-speed trains in the speed range of 100 to 160 mph require tracks of nearly perfect geometry and mechanical uniformity, when subjected to moving wheel loads. Therefore, this report briefly describes the remedies being used by various railroads to create smoothed transition regions. Some of the associated shortcomings are pointed out. Then, the method of matched pads, as presented by Kerr and Moroney (1993) is described. The aim of this method is to eliminate the need for transition sections by "softening" the tracks on the bridge, so that dynamic loads caused by the moving trains are greatly reduced.

As part of this effort, matched pads were developed, produced, and installed on a number of open-deck bridges. Field measurements revealed that the improvements caused by the installation of the matched pads and the associated treatment of the tracks near the abutments may be very significant.

The obtained results suggest that the use of matched pads may be an effective and economical way to eliminate or greatly reduce, the effect of track transitions, and that it could be useful for adjusting existing railway tracks for high-speed train travel.

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PREFACE

Often, the most economical approach to create high-speed rail corridors in the United States is to utilize existing tracks, as was the case with the Northeast Corridor (NEC) between Boston and Washington, D.C. When crossing towns, existing tracks are often elevated over the streets, using short open-deck bridges. Since these bridges are much stiffer than the adjoining tracks, this generally causes an increase of the wheel load fluctuations in the vicinity of the bridge abutments. This, in turn, leads to track deterioration at these locations, increased track maintenance costs, and possible increased risk of derailments at high speeds. Therefore, at these locations the maximum allowable speed is often reduced. For example, the maximum allowable speed on the NEC in the vicinity of Chester, Pennsylvania, was reduced from 125 to 90 mph.

High-speed trains in the speed range of 100 to 160 mph (161 to 257 km/h) require tracks of nearly perfect geometry and mechanical uniformity when subjected to moving wheel loads. First, this paper briefly describes the remedies being used by various railroads to create smoothed transition regions. Some of the associated shortcomings are noted. Then, the method of matched pads, as presented by Kerr and Moroney (1993), is described. The aim of this method is to *eliminate* the need for transition sections by "softening" the track on the bridges, so that the dynamic load fluctuations caused by the moving trains are greatly reduced.

The recent research program at the University of Delaware has developed and implemented this matched-pad technology. As part of this effort, matched pads were developed, produced, and installed on a number of open-deck bridges. Field measurements revealed that the improvements resulting from the installation of matched pads, and the associated treatment of the tracks near the abutments, may be very significant.

It was found that the installation time needed for open-deck overpass bridges is relatively short; specifically, it takes only a few hours for a five-man crew, without any sophisticated equipment, to install the pads. The matched pads are relatively inexpensive. The results obtained to date indicate that the use of matched pads may be an effective and economical way for greatly reducing the effect of track transitions, and that it could be an important step in adjusting existing railway tracks for high-speed train travel.

METRIC/ENGLISH CONVERSION FACTORS

ENGLISH TO METRIC

LENGTH (APPROXIMATE)

1 inch (in)	=	2.5 centimeters (cm)
1 foot (ft)	=	30 centimeters (cm)
1 yard (yd)	=	0.9 meter (m)
1 mile (mi)	=	1.6 kilometers (km)

AREA (APPROXIMATE)

1 square inch (sq in, in^2)	=	6.5 square centimeters (cm^2)
1 square foot (sq ft, ft^2)	=	0.09 square meter (m^2)
1 square yard (sq yd, yd^2)	=	0.8 square meter (m^2)
1 square mile (sq mi, mi^2)	=	2.6 square kilometers (km^2)
1 acre = 0.4 hectare (he)	=	4,000 square meters (m^2)

MASS - WEIGHT (APPROXIMATE)

1 ounce (oz)	=	28 grams (gm)
1 pound (lb)	=	0.45 kilogram (kg)
1 short ton = 2,000 pounds (lb)	=	0.9 tonne (t)

VOLUME (APPROXIMATE)

1 teaspoon (tsp)	=	5 milliliters (ml)
1 tablespoon (tbsp)	=	15 milliliters (ml)
1 fluid ounce (fl oz)	=	30 milliliters (ml)
1 cup (c)	=	0.24 liter (l)
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 yard (cu yd, yd^3)	=	0.76 cubic meter (m^3)

TEMPERATURE (EXACT)

[(x-32)(5/9)] °F	=	y °C

METRIC TO ENGLISH

LENGTH (APPROXIMATE)

1 millimeter (mm)	=	0.04 inch (in)
1 centimeter (cm)	=	0.4 inch (in)
1 meter (m)	=	3.3 feet (ft)
1 meter (m)	=	1.1 yards (yd)
1 kilometer (km)	=	0.6 mile (mi)

AREA (APPROXIMATE)

1 square centimeter (cm^2)	=	0.16 square inch (sq in, in^2)
1 square meter (m^2)	=	1.2 square yards (sq yd, yd^2)
1 square kilometer (km^2)	=	0.4 square mile (sq mi, mi^2)
10,000 square meters (m^2)	=	1 hectare (ha) = 2.5 acres

MASS - WEIGHT (APPROXIMATE)

1 gram (gm)	=	0.036 ounce (oz)
1 kilogram (kg)	=	2.2 pounds (lb)
1 tonne (t)	=	1,000 kilograms (kg)
	=	1.1 short tons

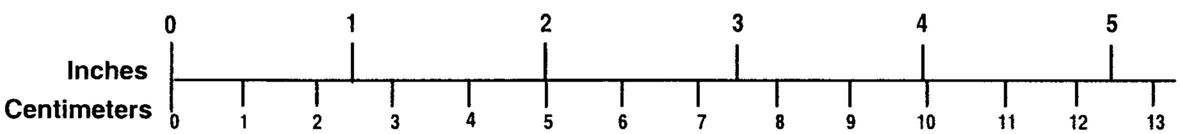
VOLUME (APPROXIMATE)

1 milliliter (ml)	=	0.03 fluid ounce (fl oz)
1 liter (l)	=	2.1 pints (pt)
1 liter (l)	=	1.06 quarts (qt)
1 liter (l)	=	0.26 gallon (gal)
1 cubic meter (m^3)	=	36 cubic feet (cu ft, ft^3)
1 cubic meter (m^3)	=	1.3 cubic yards (cu yd, yd^3)

TEMPERATURE (EXACT)

[(9/5) y + 32] °C	=	x °F

QUICK INCH - CENTIMETER LENGTH CONVERSION



QUICK FAHRENHEIT - CELSIUS TEMPERATURE CONVERSION

°F	-40°	-22°	-4°	14°	32°	50°	68°	86°	104°	122°	140°	158°	176°	194°	212°
°C	-40°	-30°	-20°	-10°	0°	10°	20°	30°	40°	50°	60°	70°	80°	90°	100°

For more exact and or other conversion factors, see NIST Miscellaneous Publication 286, Units of Weights and Measures. Price $2.50 SD Catalog No. C13 10286

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1. INTRODUCTION

During recent decades, high-speed rail service has been successfully introduced in Japan (Shinkansen lines), in France (Train à Grande Vitesse), in Germany (InterCity Express, ICE), and in the United States on the Northeast Corridor (NEC). It has been realized that additional high-speed rail corridors in North America may also relieve the congested highways and air traffic routes between closely located major urban centers.

An early U.S. high-speed line, the Metroliner service, was established on the NEC between New York and Washington, D.C. Because this line was well received by the public, it was extended to Boston and the planning of additional U.S. corridors was initiated. Examples include the South Central Corridor connecting Dallas, Houston, and San Antonio; the Chicago Hub Network consisting of routes from Chicago to Detroit, Milwaukee, and St. Louis; and the California North/South Corridor between Los Angeles and San Francisco. For details refer to the U.S. Department of Transportation (DOT) – Federal Railroad Administration (FRA) report *High-Speed Ground Transportation for America* published in 1997.

Development of additional high-speed corridors requires technology to improve, rehabilitate, and adapt existing railway lines for high-speed service. A common problem area on many existing lines are track stiffness transitions such as bridges, grade crossings, and turnouts. This report presents the development and additional tests of matched pads to minimize the vertical rail profile differential in these locations.

2. DISCUSSION OF TRANSITION PROBLEMS AND RELATED CONCEPTS

High-speed tracks, in the speed range of 90 to 160 mph (145 to 257 km/h), must be of *nearly perfect geometry* and *mechanical uniformity*, when subjected to moving trains. These requirements pose many new engineering problems that need to be addressed before high-speed trains may operate safely and economically. For example, the effect of "stiff" track locations that usually occur on open-deck bridges and at road and rail crossings, and "soft" locations that may occur at bonded insulated joints,[1] sliding expansion joints, or in turnouts, should be reduced as much as possible. This is necessary to avoid undesirable dynamic car responses that disturb passenger comfort and excessive wheel force fluctuations that may damage the track and trains, and in extreme cases, may lead to a derailment.

Often, the most economical approach to creating high-speed rail corridors is to utilize existing tracks, as was the case with the NEC between Boston and Washington, D.C. For safety reasons, and to relieve traffic congestion on the road, the tracks and intersecting roads need to be separated. As a result, when crossing urban centers, the tracks are often elevated over the streets (as well as over small rivers), using short, inexpensive, open-deck girder bridges.

These bridges are generally much stiffer vertically than the adjoining tracks, especially at the abutments, and this leads to situations as discussed by Kerr and Moroney (1993) and shown in Figure 1(*a*). It is convenient to use the track modulus k as a measure of vertical track stiffness.[2]

The distribution of k in the vicinity of a bridge abutment (or near a road crossing) is shown schematically in Figure 1(*b*). Note that the stiffer the track, the larger the corresponding k-value and the smaller the deflection under the wheel loads.

To appreciate the change in k, it should be noted that for a well-maintained track on standard wood cross ties, $k \cong 3{,}000$ lb/in². For tracks on concrete ties, $k \cong 6{,}000$ lb/in².

Upon abrupt change in the vertical track stiffness, a moving wheel experiences a *rapid* change in elevation. This causes vertical accelerations of the moving car, generating vertical wheel load fluctuations. The magnitude of these force changes depends on the elevation difference (rail deflections) between each side of the transition, on how the track modulus k varies in the transition zone, on the velocity of the moving train, and on the suspension characteristics and masses of the moving cars.

The distribution of the vertical force that a moving wheel exerts on the rail, in the vicinity of a transition point, depends also on the direction of the moving train. This is shown schematically in Figure 2.

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[1] According to Kerr and Cox, the bending stiffness of two joint bars is only about one-third of the corresponding rail (1999, p. 1255).

[2] For the definition of k and a simple method of its determination, refer to Kerr (1983, 1987, 2000).

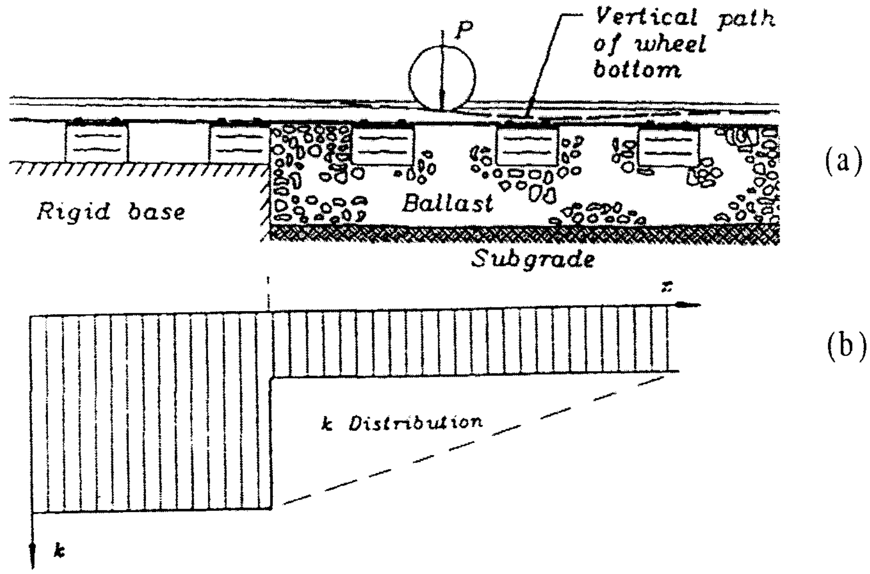


Figure 1. Track Transition at Bridge Abutment

In Case A, the train is moving toward the bridge. The increased vertical dynamic wheel force that the rail is subject to, is caused by the pushing of the wheels (and car) up onto the rigid abutment in a very short period of time (1 second or less). The region of increased dynamic wheel loads is located at the abutment, as shown. The resulting damage may include battered rails and plate-cut wood ties on top of the abutment. These impact forces may also cause damage to the abutment.

In Case B, the wheel is moving off the bridge and onto the softer ballasted track. Because the rail-tie structure in ballast deflects much more than the track on the abutment, the wheel "drops" after it leaves the abutment, imparting increased dynamic forces to the rails, as shown in Figure 2, Case B. The location of the largest force is heavily dependent on train speed; the higher the speed, the farther away this location is from the abutment. Since it takes less force to disturb the ballast and subgrade than it takes to cause rail and tie batter, these wheel loads cause fouled ballast, hanging cross ties, and permanent rail bending deformations that create the well known "dip" in the track at the end of a bridge abutment.

Similar situations may occur at the ends of a tunnel, especially where the ties in the tunnel rest on a hard base, or at both ends of a road crossing at which the track is stiffened by the road-crossing structure as well as the stiff base that has been compacted by moving auto traffic. Problems of this type may also occur where wood- and concrete-tie tracks meet, when the track moduli k differ substantially (for example, 6,000 lb/in^2 versus 3,000 lb/in^2). In this case, the dynamically generated increase of the wheel forces that move toward the concrete-tie territory, may crack the first few concrete ties that adjoin the transition point and soften its stiff side. The result is a "zipper" effect that creates its own transition to the stiffer concrete-tie section.

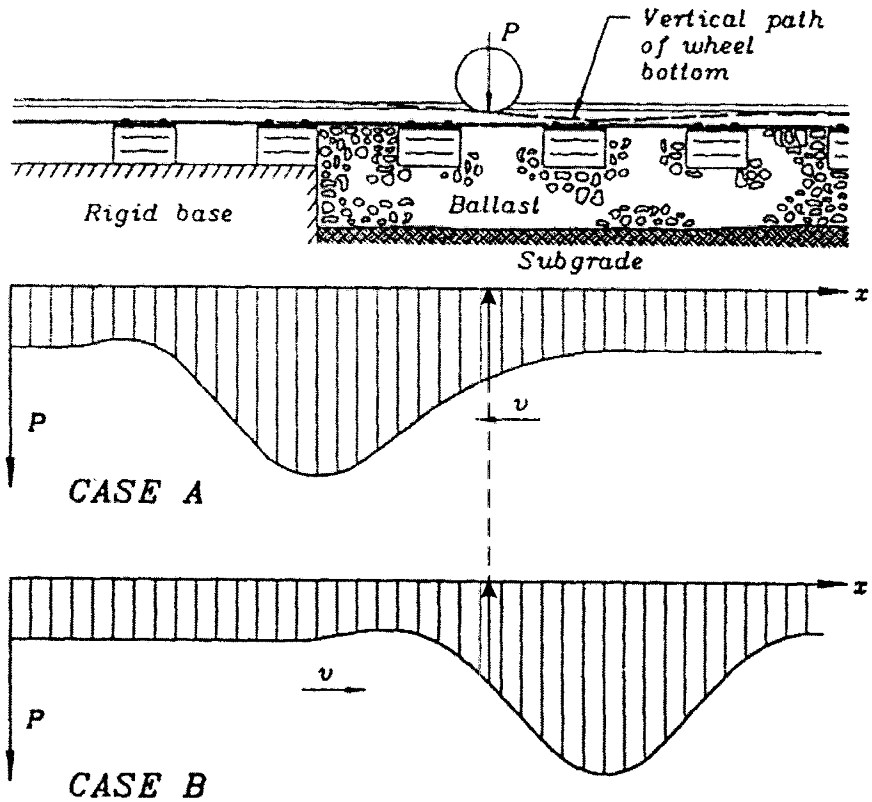


Figure 2. Schematic Dynamic Wheel Force Distribution in Transition Zones

Transition regions generally require frequent maintenance. When neglected, they deteriorate at an accelerated rate, compared to the regular track. This may lead to pumping ballast, hanging cross ties, permanent rail deformations, worn track components (ties, fasteners, etc.), increased ballast deterioration rate, and loss of surface and line. While each of the conditions is undesirable and costly for regular freight and passenger trains (50 to 80 mph), these conditions cause discomfort to passengers and may also create safety hazards that could lead to derailments for high-speed trains (90-160 mph).

The above discussion suggests that the main principle to be followed when searching for transition point remedies (Kerr and Moroney, 1993, page 271) is:

(a) *to assure that the tracks be of such design that the wheels of each truck in a moving train cause the same vertical rail deflection along the track,* or

(b) *if this is not possible, at least the vertical rail deflections should not undergo rapid changes, in order to avoid large vertical accelerations of the wheels (and hence of the cars), which cause large dynamic wheel forces.*

This report focuses on remedial method *(a)* by using matched tie pads between wood ties and tie plates to reduce the track modulus of the stiffer track support on the bridge.

3. CRITICAL SURVEY OF REMEDIES USED

For the past several decades, various measures have been developed and used by railroads to reduce the damage that occurs near transition points. All of these measures decrease the vertical accelerations of the wheels and cars as the train moves through the transition zone. According to the survey by Kerr and Moroney (1993), these remedies may be grouped into three categories:

1. Smoothing the k-distribution on the "soft" side of the transition, as indicated by the dashed line in Figure 1(a),

2. Smoothing the transition by increasing the bending stiffness of the rail-tie structure on the "soft" side, in the close vicinity of the transition point, or

3. Reducing the vertical stiffness on the "hard" side of the transition.

When considering category 1, it appears that the most well-known remedy adopted by North American railroads is the *use of oversized ties* in the track transition region, as presented in the American Railway Engineering Association (AREA) Portfolio of Trackwork Plans (1952). It utilizes a number of wood ties of progressively increasing length (usually switch ties) to create a more gradual increase in vertical track stiffness on the "soft" side of the transition. By engaging a larger section of ballast, such ties produce a stiffer track at these locations, provided they are kept fully tamped between the end of the tie and a point about 1.5 feet (50 cm) inside the running rail.

Another approach uses regular ties throughout, but *the tie spacings are decreased gradually* when approaching the "rigid" abutment. This method is applicable when the opposite joint arrangement is utilized. It was used by some European railroads for stiffening the rail support at regular expansion joints, until the introduction of mechanical tampers several decades ago that require constant tie-spacings for optimal effectiveness.

Another group of remedies that fall into category 1 are those that attempted to utilize *geotextiles*. Early designs were described by Selby (1981) and by Leubke (1982), as shown in Figure 3. The transition is formed by using 3 layers of fabric of decreasing length: 150, 100, 50 feet by Selby (1981), and 180, 120, and 60 feet by Leubke (1982). A horizontally placed fabric has to stretch substantially before it contributes to reinforcing the subgrade in the vertical plane. Therefore, it is possible that the observed increase in the vertical track stiffness near the abutment is caused mainly by the fabric's ability to facilitate drainage, resulting in an increase of the vertical track modulus, thus creating a stiffer transition track near the abutment. Another possible remedy is to add the axially stiffer geogrids, like those produced by the Tensar Corporation. Because of the varying degree of success achieved using geotextiles at transitions, tests would be needed to establish their suitability for problems of this type and, if they prove to be suitable, implementing the most beneficial arrangement of these reinforcing materials.

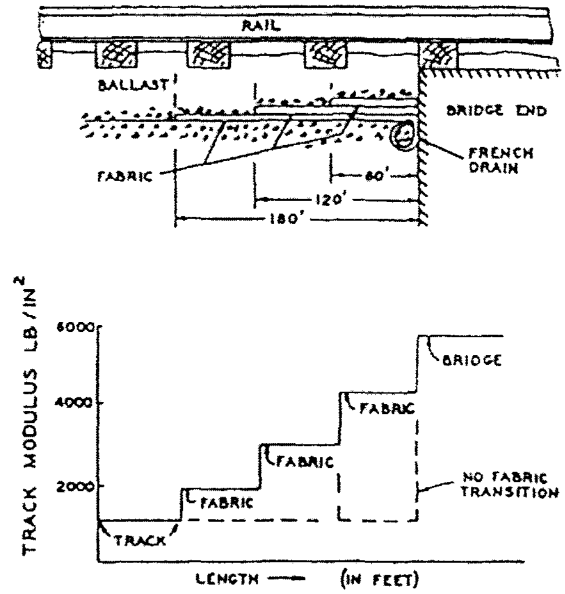


Figure 3. Vertical Stiffness Transition Using Geofabrics (Leubke 1982)

Another method that falls into category 1 uses *Hot-Mix-Asphalt* (HMA). According to Rose and Hensley (1991) and Rose (1992), a number of such underlayments were installed by North American railroads in the past. An example is shown in Figure 4. According to Rose (1992), the primary use of HMA underlayments in transition sections is for rehabilitation of those locations that have exhibited poor performance using conventional procedures.

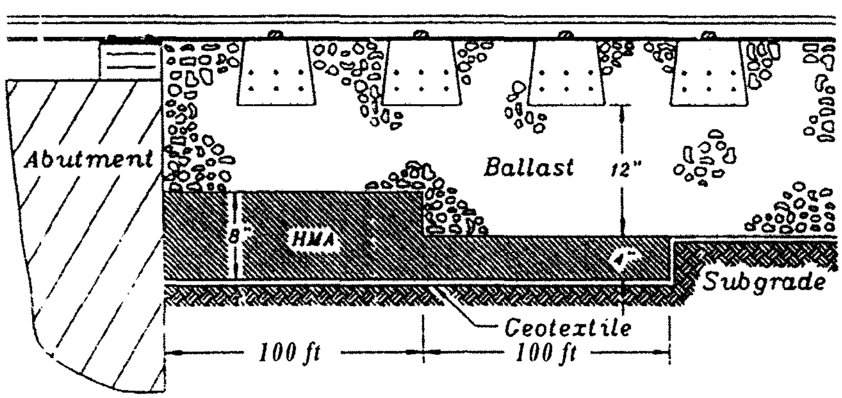


Figure 4. HMA Bridge Approach Transition

Still another method that falls into category 1, used mainly at bridge abutments or at tunnel ends (where slab track meets cross-tie track in ballast), utilizes a *cantilevered slab*, one end of which is supported by the "rigid" structure and the other is free floating, as shown in Figure 5(*a*). In addition to providing a stiffness transition, the approach slabs may be used to "square-off" bridge abutments that are located on a severe skew, as shown in Figure 5(*b*), to reduce the possibility of a derailment from rocking of rail cars. This method is similar to the one often used at highway bridge approaches.

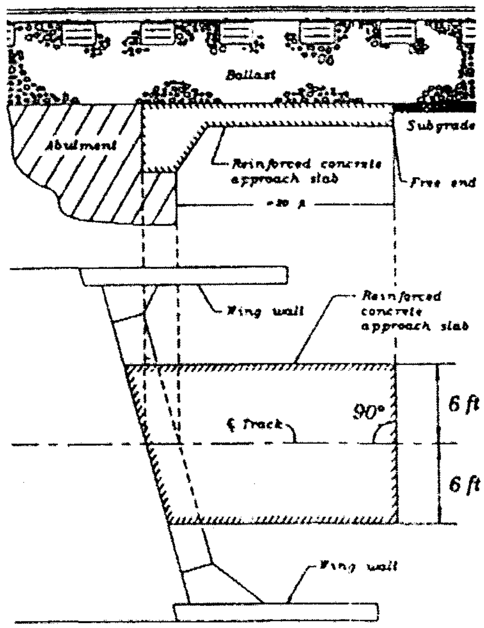


Figure 5. Transition at Bridge Abutment Utilizing a Reinforced Concrete Approach Slab

These methods – which use geotextiles, HMA, or an approach slab – require extensive track rebuilding over a section of about 200 feet (even longer for high-speed tracks) adjoining the abutment. Therefore, these methods are expensive to implement, and their installation requires substantial track time. This in turn, results in disruption of traffic.

A category 2 method creates a transition section by *increasing the bending stiffness of the rail-tie structure* in the adjoining track section, as shown in Figure 6. It was developed by the German Federal Railways (DB) for the ICE high-speed lines. The illustrated installation is at the portal of the Mühlberg tunnel (Schrewe, 1987), where the slab section inside the tunnel changes to a concrete cross-tie track in ballast. The transition section is formed by four extra rails attached to the ties; two inside and two outside the running rails. The length of this transition zone is about 100 feet (~30 m) long.

The use of *tie pads* falls into category 3. The aim of tie pads is to *reduce the stiffness* of the track on the "hard" side of the transition point. Ideally, it should match the stiffness of the track on the "soft" side of the transition point, stated previously as principle (*a*): "to assure that the tracks be of such design that the wheels of each truck in a moving train cause the same vertical rail deflection along the track."

Figure 6. Transition Section Which Utilized Additional Rails, DB (Schrewe, 1987)

The early use of the tie pads goes back to the end of the 1800s. Their purpose was mainly to prevent wood-tie abrasion and reduce rail creep in the direction of the moving trains. A survey of these developments in Europe and North America was presented by Kerr and Moroney (1993).

The notion of utilizing elastic pads under rails which are placed directly on a solid structure (like a bridge abutment), for reducing the impact forces of a moving train, and thus diminishing the damage to the "rigid" base, was described by Müller (1925, 1928, 1930). A fastener of this type, with a poplar wood pad is shown in Figure 7. Later, poplar pads were also utilized on steel bridges for force reduction and sound attenuation purposes.

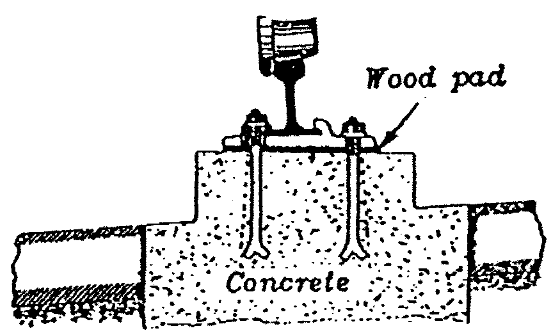


Figure 7. Early Utilization of Pads
(Müller, 1925)

Pads are currently being utilized in connection with concrete ties and spring-clip fasteners throughout the world. In addition to increasing the rail push-through resistance, and hence eliminating the need for anchors [von Schrenk (1928)], the purpose of the pads is to attenuate the impact forces of the wheels to prevent damaging the concrete ties. The necessary stiffness of these pads depends on the condition of the track and the rolling stock (Dean et al., 1983; ORE D161.1, Report 3, 1986; Reinschmidt, 1991). This problem, as yet, is not completely solved.

For studies on pads in the Russian language, refer to Shakhunyants and Demidov (1971), Shul'ga and Bolotin (1975, 1977), and Shakhunyants, Demidov, and Gassanov (1978). According to Kaess and Schultheiss (1986), the DB have been experimenting on the ICE high-speed lines with softer pads in tunnels and on bridges. The aim of these tests was to reduce the vertical track stiffness at these locations and to match it with that of the tracks outside. Since there are many tunnels and bridges on some sections of the German high-speed lines, softer pads intended to create a uniform track stiffness throughout, providing a comfortable ride, preventing the occurrence of large dynamic wheel forces, and avoiding slow orders at these locations. According to Kaess (1992, 1994) these tests revealed that the ICE tracks are generally rather stiff and that pads will have to be used throughout. (For related discussions refer to the papers by Leykauf and Mattner (1990) and Eisenmann, Leykauf and Mattner (1994).) The use of pads on bridges and in tunnels, for the purpose of reducing large variations of the vertical track stiffness, was also discussed by Bussert and Bothe (1978) for regular train traffic on the railways of the former East Germany.

In the past, pads were utilized in transition sections by a number of North American railroads. According to Worth (1992), "Tie pads are standard on wood ties on open deck bridges on Canadian National (CN) (since the early 1980s) wherever annual tonnage exceeds about 2 million gross tons (MGTs). Their primary purpose is to seal water out of the spike holes and reduce iron sickness under the plates, but they also serve to put some resilience into the system." In the United States, CSX is one example of a railroad that uses pads (Maintenance Rules and Practices, 1988, page 57). According to Hardy (1989), tie pads are being utilized by CSX on open deck bridges to reduce mechanical wear of the bridge ties. At CSX, mostly pine or softwood is used for the bridge ties "and the pad greatly reduces plate cutting. The pad material is solid neoprene, ¼-inch thick and about the size of the tie plate. It is held in place by the spikes that hold the plate and rail." CSX does not prescribe the stiffness of the pad.

In the past, tie pads were utilized in grade crossings; and an example is shown in Figure 8. According to the Hi-Rail Company rail-crossing brochure, "It is essential to use a tie pad under all tie plates in the crossing area and extended to at least three ties beyond each end." No specific pad stiffness is prescribed. The majority of crossings in North America are being constructed without tie pads.

A remedy that also falls into category 3 is based on the utilization of *mats*. According to Sato, Usami, and Satoh (1974), in order to increase the train speeds to 155 mph (250 km/h) on the Shinkansen network, rubber mats should be inserted between the ballast and the solid base of the elevated structures and in tunnels. The ballast mat was intended to reduce the dynamic wheel loads and prevent the "pulverization" of the ballast, which was occurring in the tracks of the Tokaido Shinkasen.

The ballast mat was produced from used automobile tires that had been cut into fine grains and heat-molded into a mat. This procedure produced the desired spring constant by adjusting the voids in the mat. Field and laboratory tests, initiated in 1971, were conducted on the produced mats to establish stiffness, strength, and durability. A major finding of this extensive testing program was that ballast mats were highly effective in preventing ballast pulverization. They also reduced noise and vibrations. (For details refer to Tajima and Kiura (1974), Sato and Usami (1977), and Sato, Kobayashi, Nakamura, and Kobayashi (1983).) According to Sato and Usami, ballast mats were widely used on the Japanese National Railways by 1977.

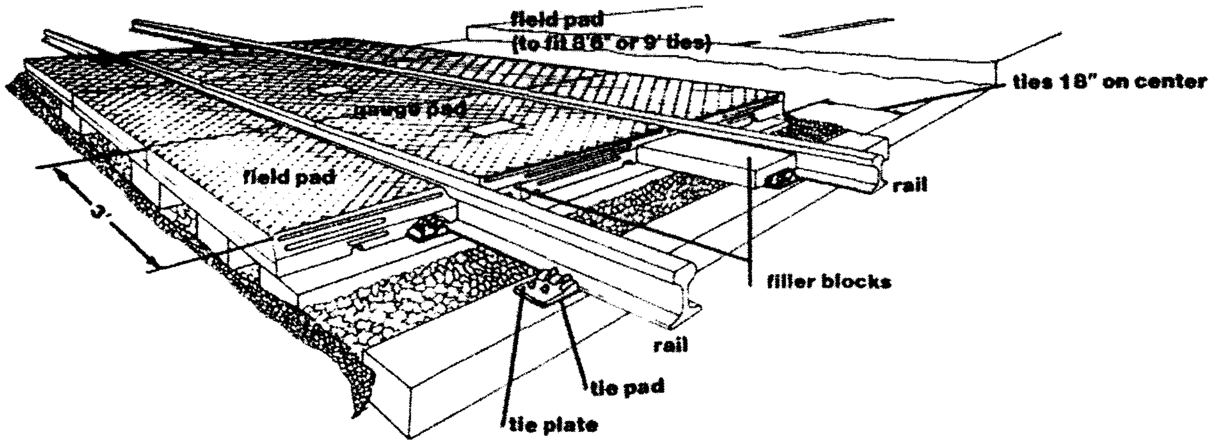


Figure 8. Hi-Rail Grade Crossing with Tie Pads (from company brochure)

In Germany, Eisenmann and Leykauf (1977) mentioned the possible use of elastic mats. Keim, Kohler, and Schober (1978) presented test results for Sylomer™ mats. Clouth[3] describes another ballast mat, Vibrax™, which consists mainly of natural rubber with fabric reinforcement (recent brochure, no date). Kaess and Stretz (1979) described an extensive study, conducted by the DB, to determine the mechanical properties of mats. (Note the recent DB (1988) specifications for ballast mats.)

Finally, it should be noted that the rubber boot and microcellular pad, surrounding each of the blocks of a two-block tie that is embedded in a very stiff base, shown in Figure 9, might be considered a mat or a pad. This tie system is being used in the recently completed Channel-Tunnel between England and France. If properly matched, it could be utilized on short "rigid" structures to avoid the occurrence of transitions.

[3] This is a recent brochure and no date is available.

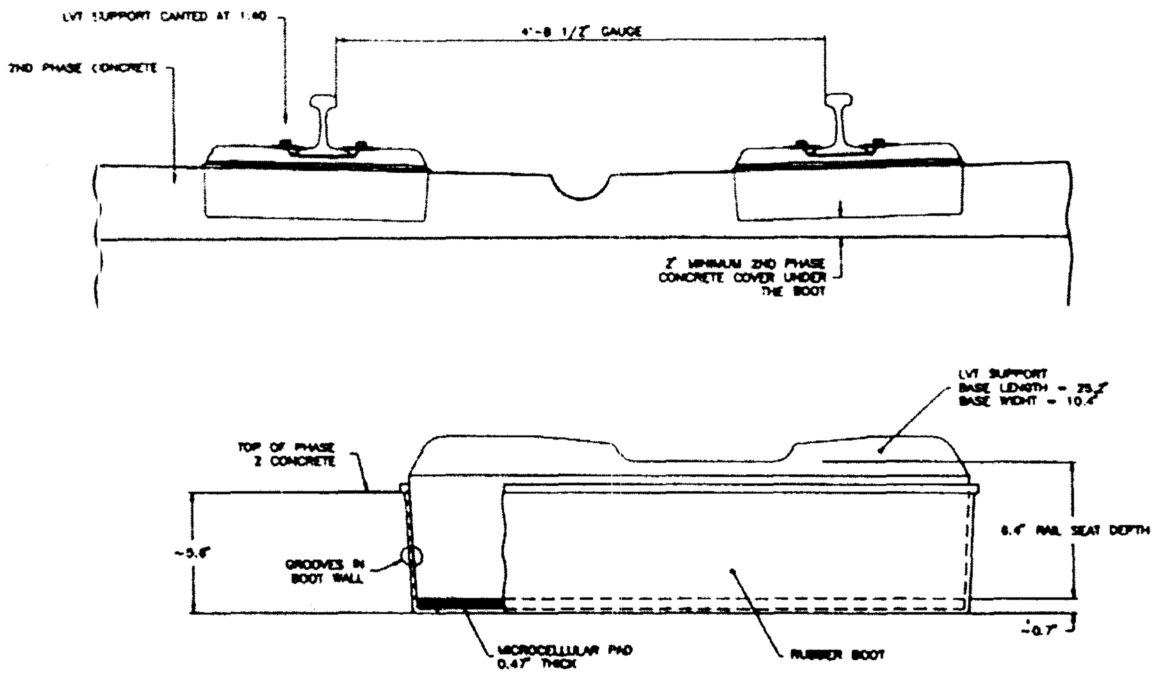


Figure 9. Two-Block Tie with Rubber Boot and Microcellular Pad (Sonneville International Corp. 1992)

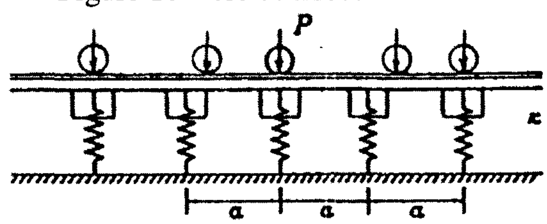
4. METHOD BASED ON USING MATCHED PADS

The above review of the various track transition remedies suggests that *for regular as well as for high-speed tracks, the use of tie pads and/or mats, is technologically and economically the most suitable approach.* This is especially true for high-speed tracks since the "smoothing" of the k-distribution (using oversized ties, asphalt, or geofabrics) requires very long, and thus costly, track transitions. Moreover, for tracks that pass through urban areas with closely spaced short bridges over streets, there is usually not enough space available for installing structural high-speed transitions.

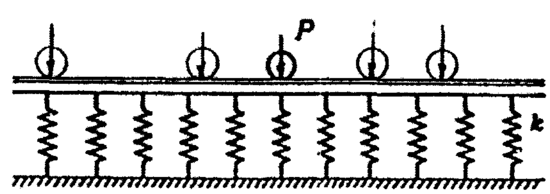
Therefore, the next step focused on improving the utilization of tie pads and mats. The aim was to eliminate the track transition sections by using *matched pads*, that would drastically reduce the "stiffer" response over a short overpass bridge or road crossing.

4.1 ANALYTICAL PROCEDURE FOR PAD MATCHING

To match the track running over the bridge with the track outside, the relation between the track modulus k and the pad-tie stiffness κ over the bridge had to be established. To achieve this aim, the analytical models shown in Figure 10 were utilized.



(I) Analytical model for track on bridge



(II) Analytical model for track adjoining bridge

Figure 10. Track Models for Bridge and Adjoining Track

The vertical equilibrium of a rail on discrete and on continuous supports is respectively,

$$\sum P = \sum_{n=-\infty}^{\infty} \kappa w_n = \kappa \sum_{n=-\infty}^{\infty} w_n \tag{1}$$

where

P	=	wheel load,
κ	=	vertical stiffness for individual tie on bridge,
w_n	=	vertical rail deflection on bridge, and

$$\sum P = \int_{-\infty}^{\infty} p(x)dx = k\int_{-\infty}^{\infty} w(x)dx\,, \tag{2}$$

where

p	=	distributed vertical pressure acting on rail away from bridge,
x	=	distance along track, and
k	=	track modulus away from bridge.

Note that in Equation (1) w_n is the vertical rail deflection at the n^{th} tie and κw_n is the corresponding vertical force the tie exerts on the rail. In Equation (2), $p(x) = kw(x)$ is the upward "distributed" pressure on the rail.

Since the sum of the vertical wheel loads on the bridge and on the track is the same in the above equations, the right-hand-sides are equated. The result is

$$\kappa\sum_{n=-\infty}^{\infty} w_n = k\int_{-\infty}^{\infty} w(x)dx\,. \tag{3}$$

The integral in the above equation represents the area formed by the deflection curve w(x). Multiplying Equation (3) by the center-to-center tie spacing a, and noting that for small a

$$\sum w_n a \cong \int_{-\infty}^{\infty} w(x)dx\,, \tag{4}$$

Equation (3) reduces to

$$\boxed{\kappa = a \bullet k}\quad . \tag{5}$$

This is the relationship between the pad-tie spring constant κ on the bridge and the rail support modulus k of the adjoining track. It will be used for matching the pads on a bridge to the adjoining tracks.

For the recorded k-range of 2,500 lb/in^2< k < 3,000 lb/in^2, and the center-to-center tie spacing on the bridge a = 16 inches, according to Equation (5) the desirable κ-range for the pads should be

$$40{,}000 \text{ lb/in} < \kappa < 48{,}000 \text{ lb/in}. \tag{6}$$

4.2 TEST PROCEDURE FOR DETERMINATION OF THE TRACK MODULUS k

Since the pads had to be matched to the tracks outside of the bridge, it was necessary to establish the vertical stiffness (i.e., the track modulus) of the adjoining track sections. This was done using the simple method proposed by Kerr (1983, 1987, 2000).

The tests were conducted on the main line of Amtrak's NEC through Chester, Pennsylvania. This area was selected due to a high number of short, open-deck bridges and the increased track geometry deterioration rate and poor track performance.

At the start of the testing program, Amtrak placed a loaded car on a scale. The determined gross weight was

$$W = 177{,}400 \text{ lb} = 88.7 \text{ tons.}$$

Because the car was supported by 2 two-axle trucks and was loaded uniformly, one wheel load was approximately

$$P = 177{,}400/\ 8 = 22{,}175 \text{ lb} = 11.09 \text{ tons.}$$

This loading is approximately the service load for this track location.

The k-tests were conducted on August 2, 1995, starting at 9:00 pm and continued until about 2:00 am on Track No. 3. Night testing was necessary, because of the lower volume of traffic during these hours. Figure 11 indicates the locations of the measurements by a circled number.

Each test began by attaching a millimeter scale (glued to a magnet) vertically to the rail web and then by backing in the test train, so that the first axle was located over Station ①, as indicated in Figure 11. The created rail deflection at ① was then recorded by two levels[4] that were mounted before the test car arrived, about 10 meters away and perpendicular to the rail. Next, the scale, as well as the levels, were moved to Station ②, and the train moved forward until the front axle came to rest over Station ②. The resulting rail deflection was then recorded. This procedure was continued until deflections at all nine stations had been recorded.

It should be noted that the rail deflections over the bridge abutments are not suitable for the determination of k. They were recorded for comparison purposes only. The recorded deflections at the various other track locations are presented below.

According to Kerr (1983), once w_m/P is calculated using $P = 11.09$ tons, the corresponding k-value is determined from the graphs shown in Figure 12.

[4] The second level was used as a check.

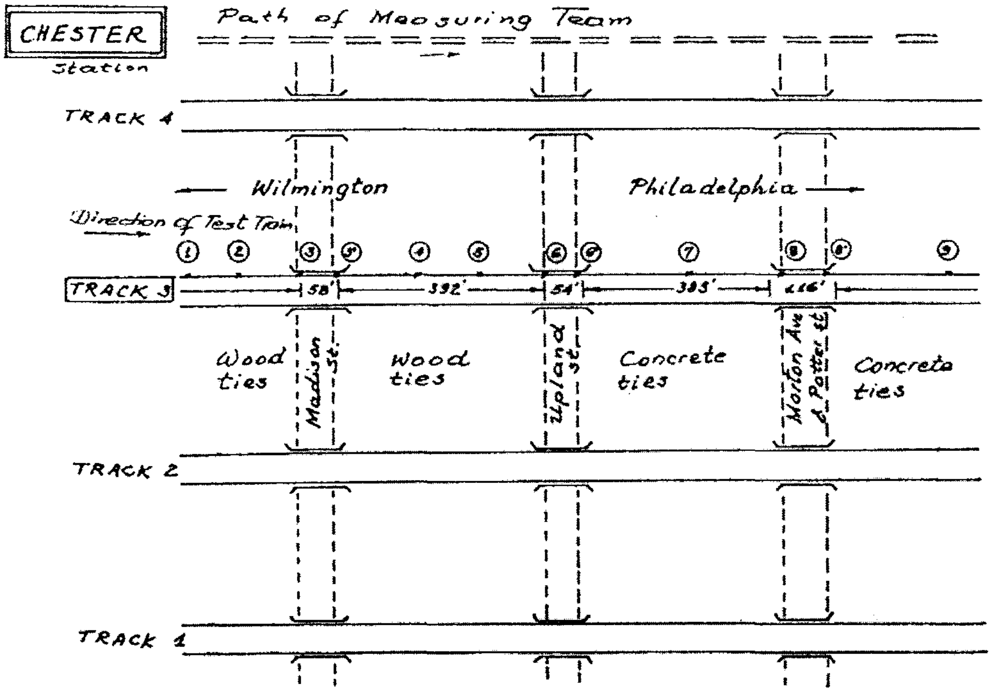


Figure 11. Locations of Deflection Measurements

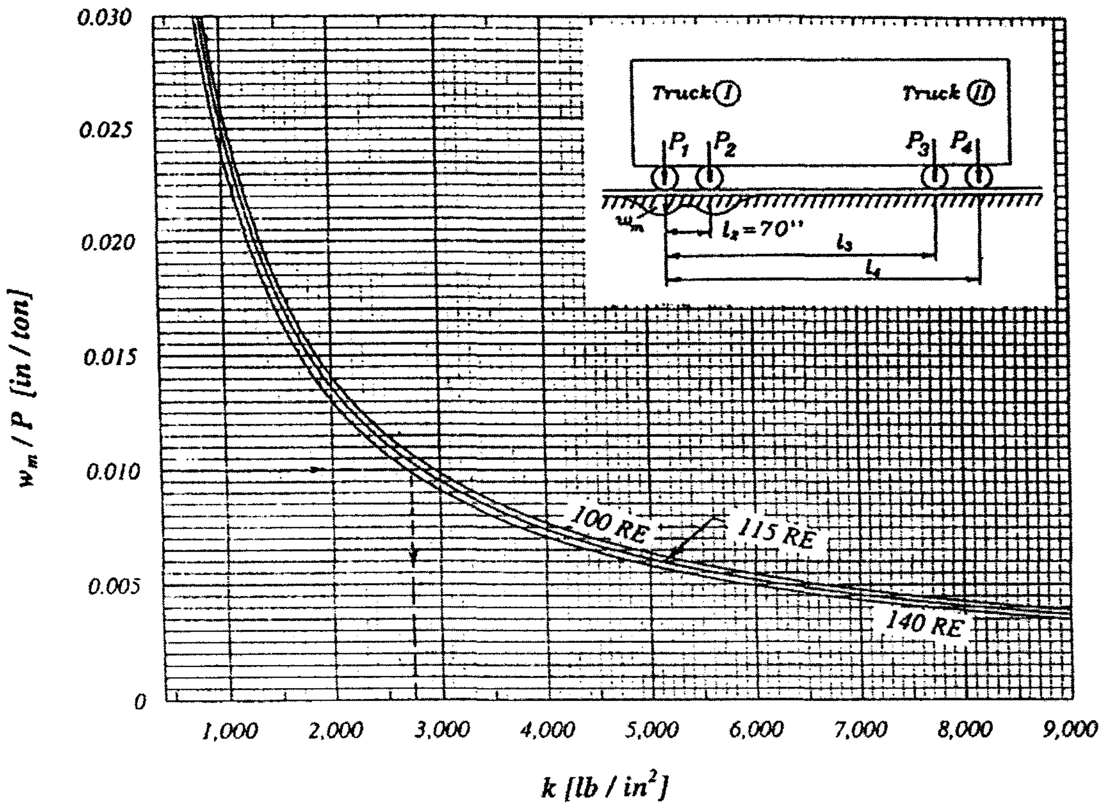


Figure 12. Graphs for the Determination of *k*

The determined values are shown below the recorded deflections. From Table 1, the average k-value for wood-tie tracks is

$$k \cong 3{,}000 \text{ lb/in}^2$$

and for the concrete-tie track it is

$$k \cong 5{,}250 \text{ lb/in}^2.$$

At the abutment points ③, ⑥, and ⑥', the response was much softer (instead of being stiffer) with an equivalent k-value of 900 and 1,150 lb/in^2. The reason for these low values is battered ties over the abutments, caused by the increased dynamic wheel forces at these locations.

4.3 TEST PROCEDURE FOR STATIC PAD MATCHING

The test set up used is shown in Figure 13. The testing machine, a 400,000 lb-capacity Gilmore, is located in the Civil Engineering Structures Laboratory of the University of Delaware. Data were collected by using the System 4000, which used linear variable differential transformers (LVDTs) to measure vertical deflections (see Table 1). The recorded load versus vertical deflection measurements were then plotted.

In the set up shown, the outer 1/3 of a wood tie is placed on the base of the testing machine. The pad is placed over the tie, and the metal tie plate over the pad. Finally, a section of rail is placed on the tie plate. The testing machine then loads the rail vertically. The applied load corresponds to the rail-seat force F. The aim of this test set up is to simulate, as closely as possible in a lab, the conditions expected on main line tracks at very slow speeds.

It was decided that the load range of interest for these tests should be about 1/3 of the expected wheel load. Therefore

$$0 < F < 10{,}000 \text{ lb} \tag{7}$$

was chosen. The reason for this choice is that, according to the standard track analysis for the wood- or concrete-tie track under consideration, the largest rail-seat force, F_{max}, is about 30 percent of the wheel load P.

Each tie pad to be used is slightly larger than the tie plate used (1/16 inch beyond the edge of the tie plate). It was stipulated that the tie-pad thickness, h, should be $h \leq ½$ inch. A certain pad thickness is necessary to soften the pad for matching purposes. On the other hand, pads that are too thick may exhibit undesirable lateral displacements of the tie plate when used in track, caused by the lateral shear deformations of the pad.

The deflections were recorded by two LVDTs, that were placed over the rail base, as shown in Figure 13, and two mechanical dial gauges as checks. For example, an early recorded load-displacement curve for a 3/8-in thick pad over a bridge tie is shown in Figure 14. The short dashed lines represent the output of the LVDTs.

Table 1. Recorded Test Deflections

1 Wood-Tie Track

Level	#1	#2
Zero	4.5	4.5
Reading	4.2	4.2
w_m	0.3	0.3
Average w_m	0.3 cm= 0.12 in.	

$w_m/P = 0.0108 \rightarrow k = 2{,}450$ lb/in^2

2 Wood-Tie Track

Level	#1	#2
Zero	4.0	4.0
Reading	3.8	3.8
w_m	0.2	0.2
Average w_m	0.2 cm = 0.08 in.	

$w_m/P = 0.0072 \rightarrow k = 3{,}900$ lb/in^2

4 Wood-Tie Track

Level	#1	#2
Zero	2.0	3.0
Reading	1.7	2.7
w_m	0.3	0.3
Average w_m	0.3 cm = 0.12 in.	

$w_m/P = 0.0108 \rightarrow k = 2{,}450$ lb/in^2

5 Wood-Tie Track

Level	#1	#2
Zero	2.0	4.0
Reading	1.74	3.75
w_m	0.26	0.25
Average w_m	0.26 cm = 0.10 in.	

$w_m/P = 0.0090 \rightarrow k = 3{,}000$ lb/in^2

7 Concrete-Tie Track

Level	#1	#2
Zero	2.00	3.00
Reading	1.85	2.83
w_m	0.15	0.17
Average w_m	0.16 cm = 0.063 in.	

$w_m/P = 0.057 \rightarrow k = 5{,}250$ lb/in^2

9 Concrete-Tie Track

Level	#1	#2
Zero	3.00	4.00
Reading	2.85	3.85
w_m	0.15	0.15
Average w_m	0.15 cm = 0.06 in.	

$w_m/P = 0.0054 \rightarrow k = 5{,}250$ lb/in^2

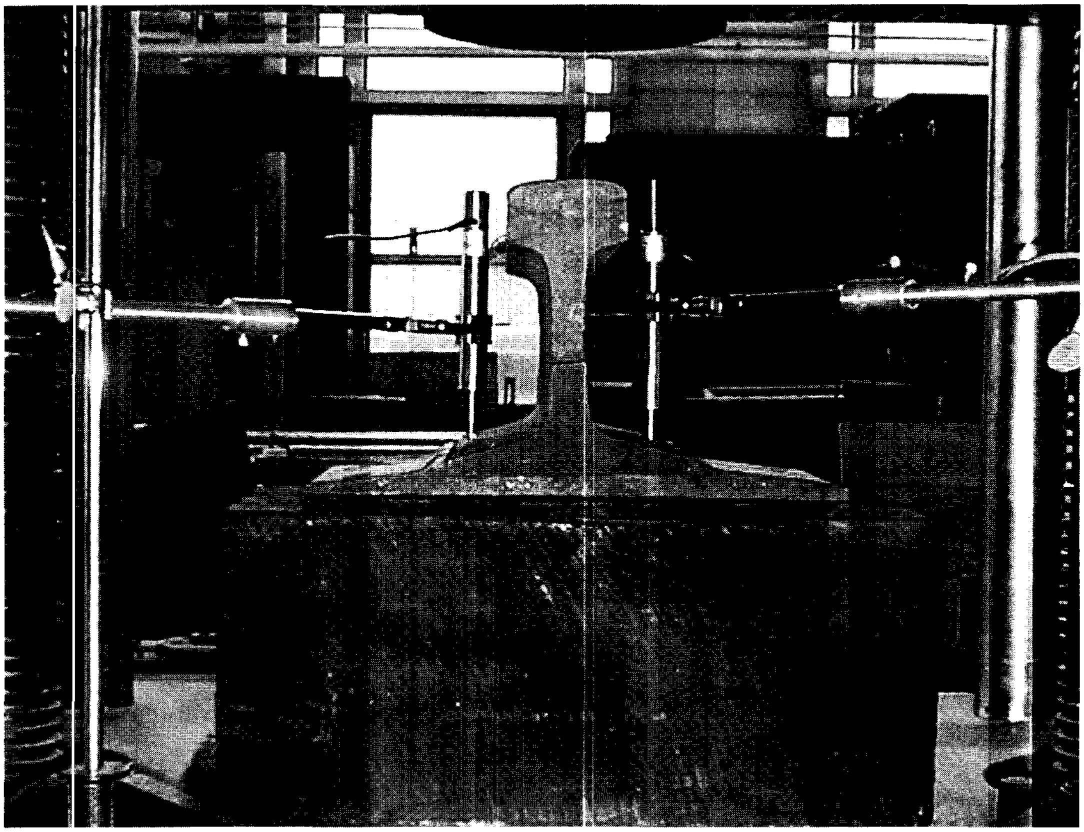


Figure 13. Test Set Up for Determining the κ-Values for Pad and/or Wood Tie.

The *spring constant* κ of the pad corresponds to an idealized linear response. Therefore, the procedure for determining the κ-value from the nonlinear test curve consists first of its intuitive linearization, based on experience, shown as a long dashed line in Figure 14, and then of the determination of the corresponding κ-value from the relation

$$\kappa = \frac{\Delta F}{\Delta w} = \frac{10{,}000}{0.05} = 200{,}000 \ \text{lb/in.} \tag{8}$$

Comparing this κ-value with the range given in Equation (6), it follows that the tested pad was too stiff. This finding and those of related tests initiated a search and testing program for softer pads.

4.4 DESIGN AND SELECTION OF MATCHED PADS

At this stage, the Alert Manufacturing & Supply Company (Alert) joined the research program by agreeing to provide pad samples for testing and produce the designed *matched pads* for three bridges; two to be installed on the Amtrak high-speed corridor between New York City and Washington, D.C. and one on a bridge of the Norfolk Southern (NS) Railroad.

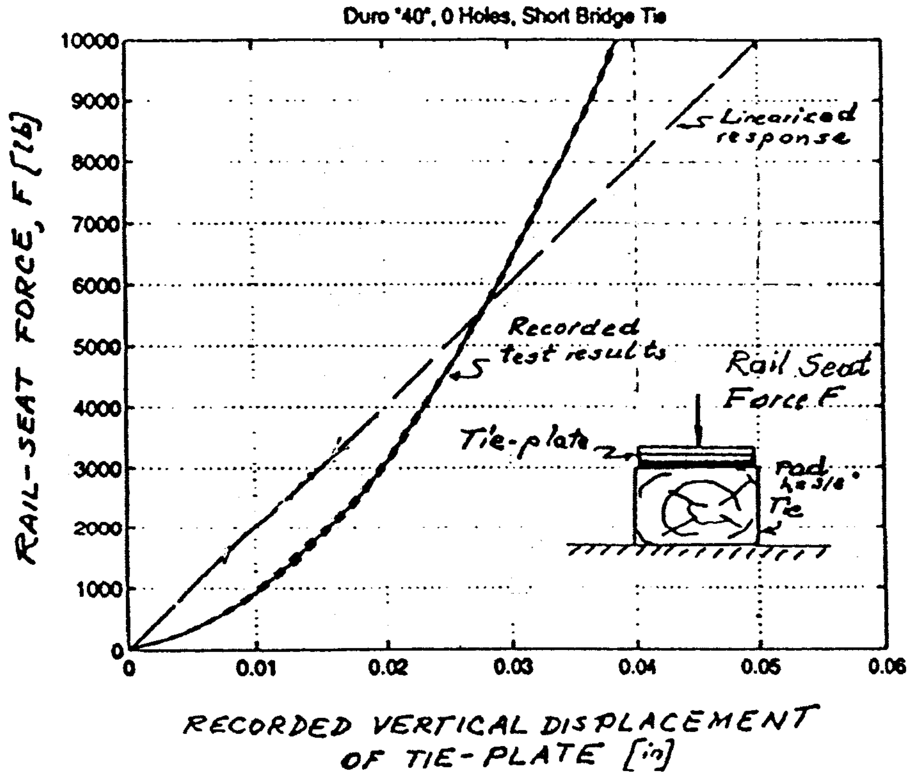


Figure 14. A Typical Test Curve for Pad and Tie

At first, Alert submitted a set of ¼-inch thick pads (mostly solid rubber of Duro 50 and 60).[5] They proved to be too stiff. Alert then submitted thicker, but softer pads of Duro 40. Tests on these 3/8-inch thick rubber pads proved that they were still too stiff. Alert next submitted a sponge-type rubber pad with visible voids. Tests on these pads yielded the desired compressibility results; however, they were considered unacceptable because of the voids and possible durability deficiencies. The next shipment to be tested was ½-inch thick solid rubber pads of Duro 50. They were still too stiff. After various attempts to reduce the stiffness, grooves were cut with a razor blade about 1 inch apart until the pad showed the desired stiffness.

Three types of static load-deflection tests were conducted on each of these pads: (1) a pad on a standard 9-inch x 7-inch wood tie, (2) a pad on a 9½-inch x 8½-inch bridge wood tie, and (3) a pad by itself, with no tie.

[5] The term "Duro" is an abbreviation for *durometer*. The durometer test is an American Society for Testing and Materials testing standard for rating the stiffness or hardness of rubber. The durometer rating scale ranges from 0 (very soft) to 100 (very hard), generally rated in multiples of ten.

It was necessary to include a wood tie in the test (as shown in Figure 11) because previous tests by Kerr and Moroney (Proceedings AREA, 1993) revealed that the compressibility of a mainline wood tie may be larger than that of some pads initially submitted by Alert. Since the tie compressibility also depends on the number of years of service and the million gross tons absorbed, it complicates the choice of proper pad stiffness. Visual inspection of the wood tie used in the above tests suggested that the tie had been in service for many years and therefore was softened. It was also concluded that a bridge tie (with a 9 ½-inch × 8 ½-inch cross-section) is a more accurate representation of the actual track condition. Therefore, a relatively new bridge wood tie (9 ½-inch × 8 ½-inch) and an Amtrak tie plate were used in the following tests.

Based on the numerous preliminary test results obtained on ³/₈-inch and ½-inch thick pads, the pad for wood-tie tracks was chosen to be ½-inch thick with square grooves ¼-inch deep, placed parallel to the long edges of the pad and in offset position, as shown in Figure 15. The pad is installed with the grooves perpendicular to the rail.

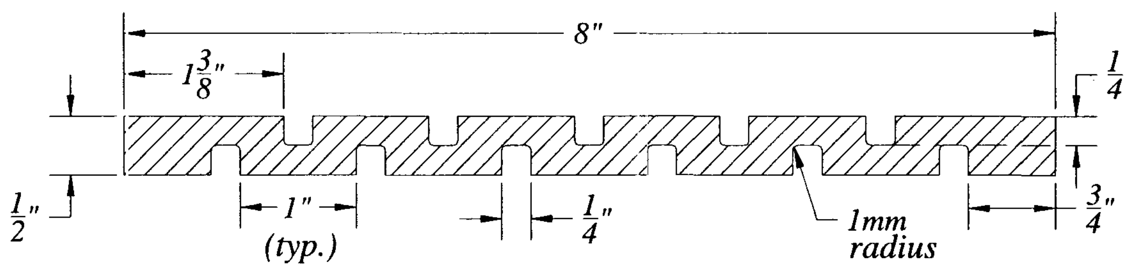


Figure 15. Chosen Pad Cross-Section for Matching Wood-Tie Tracks

This pad design was sent to Alert for reproduction. The pads received were then subjected to the compressibility test, as presented in Figure 13.

The test results are shown in Figure 16. This figure contains the results of three tests: (1) relatively new tie only, (2) pad only, and (3) pad and tie as it will be used on an actual bridge. Although for the purpose of matched-pad development, the pad-on-tie is of main interest, the other two cases were included to show the contribution of each component, especially the noticeable effect of vertical tie compressibility.

To determine the corresponding stiffnesses, proceed as in Equation (8). The corresponding results are:

$$\kappa_{tie} = \frac{10{,}000}{0.055} = 181{,}818 \text{ lb/in} \tag{9}$$

$$\kappa_{pad} = \frac{10{,}000}{0.08} = 125{,}000 \text{ lb/in} \tag{10}$$

$$\kappa_{tie+pad} = \frac{10{,}000}{0.13} = 76{,}923 \text{ lb/in.} \tag{11}$$

Since the center-to-center tie spacing on the bridge a = 16 inches, it follows, according to Equation (5), that the corresponding rail support modulus (track modulus for one rail) is

$$k = \frac{K_{tie + pad}}{a} = \frac{76{,}923}{16} = 4{,}808 \text{ lb/in}^2. \quad (12)$$

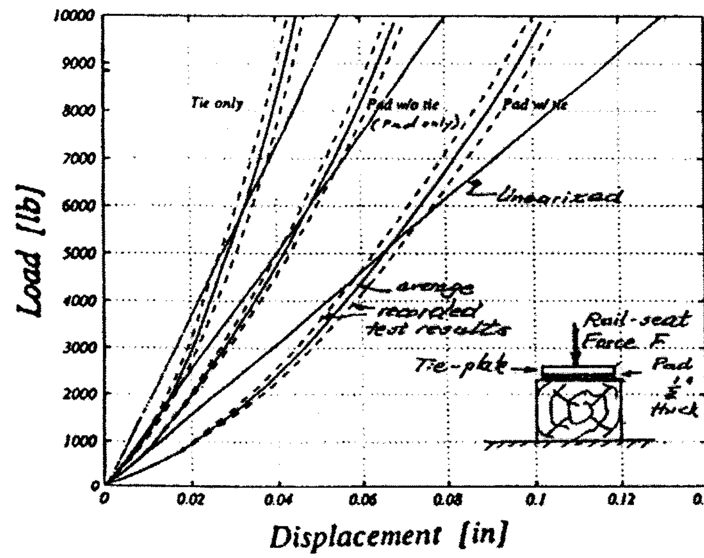


Figure 16. Test Results for Manufactured Pad and/or Tie

Wood ties on open-deck bridges are generally subjected to high tonnage and to weathering cycles. It is therefore reasonable to expect that their condition will deteriorate continuously during their service life, especially due to environmental effects and mechanical damage in the rail-tie and tie-girder contact regions. Therefore, the above k-value will decrease with time and traffic accumulation. Since the track modulus (for one rail) for well-maintained wood-tie tracks was established to be about $k = 3{,}000 \pm 250$ lb/in^2, it was decided (as a first try) to use matched pads of the tested design on the Welsh Street Bridge, in order to establish the effectiveness of this method for track transitions.

5. FIRST INSTALLATION OF MATCHED PADS ON THE NORTHEAST CORRIDOR (WOOD-TIE TERRITORY)

5.1 INSTALLATION OF PADS

The Welsh Street Bridge (near the Chester, Pennsylvania station) was selected as the first test site. Of the four tracks, Track Nos. 2 and 3 are the high-speed lines (up to 125 mph) used mainly by Amtrak. Currently, in the Chester, Pennsylvania region, the maximum speed is restricted to 90 mph due to track geometry and ride quality problems related to numerous "hard" open-deck bridges near the Chester station,. The outer tracks, Track Nos. 1 and 4, are used primarily by the slower Southeastern Pennsylvania Transportation Authority (SEPTA) trains. Track No. 3, which predominantly facilitates southbound trains (moving toward Washington, D.C.), was selected for pad installation.

Before mass-producing the matched pads for Track No. 3 of the Welsh Street Bridge, Alert sent two pads for a final test and then manufactured 98 pads and shipped them to the Wilmington, Delaware station for installation.

Due to heavy train traffic on the NEC during the day and evening hours, the track maintenance crew assigned by Amtrak began pad installation at 12:30 am on July 11, 1996. They first disconnected the west rail of Track No. 3 from the ties by pulling all spikes from the ties on the bridge and from the first 10 ties on either side beyond the abutments. The rail was jacked up to remove the existing, rather hard, tie pads and clear the top of the ties of ballast and debris. The new matched pads were placed on the ties and the tie plates placed on top of the pads. The pads and tie plates were aligned and the rail lowered back on the tie plates. Proper alignment of the rail was checked and cut spikes were then driven through the tie plates and pads, to secure the rail to the ties. A similar procedure was followed for the east rail of Track No. 3. Proper gage was assured prior to spike driving of the east rail.

After the placement of the matched pads and the respiking of the rails, the maintenance crew tamped the ballast under 10 ties adjoining the bridge abutments, using portable pneumatic tampers. The installation of the pads was completed at approximately 4:00 am, and Track No. 3 was opened to regular traffic.

It is worth noting that the total time needed for the installation of the matched pads on the Welsh Street Bridge was only 3 ½ hours. With more experience, the installation time could be reduced to about 3 hours.

To ensure proper rail-tie support adjacent to the bridge at the abutments, the ballast was retamped about a week after the pad installation, using mechanical tamping equipment.

5.2 MONITORING THE EFFECT OF THE INSTALLED PADS

To establish the effectiveness of using matched pads, the geometry car measurements recorded on July 25, 1996, (2 weeks after pad installation) were compared with measurements recorded shortly before installation. This is shown in Figures 17 and 18. Both figures show a significant improvement of the vertical rail geometry in the transition region; namely, that the installation of the matched pads and the associated tamping of the track near the abutments (bridge approaches) reduced the maximum rail profile amplitude near the abutments by almost 50 percent.

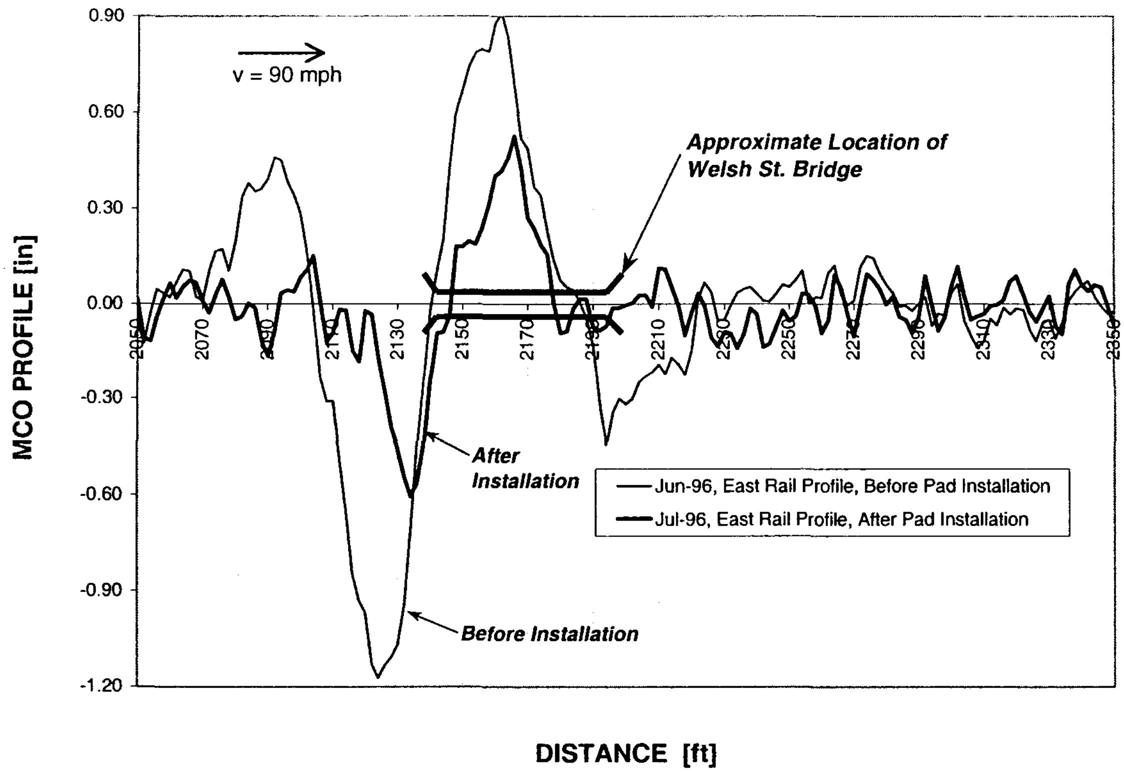


Figure 17. Effect of Matched Pads on *East* Rail Profile of Track No. 3 of Welsh Street Bridge (near Chester, Pennsylvania station)

The next step was to further reduce the remaining deflections by improving the ballast support conditions at the bridge ends. Realizing that mechanical tamping near the abutments loosened the ballast, and that, subsequently, the moving trains compacted it again leading to a sagging of the ballast around the ties, it was decided to compact the track near the abutments by ballast-packing. This was done in April 1997. Since a *stoneblower*[6] was not available at the time, it was decided to use manual *shovel packing*. In this method, the crib ballast was removed, then the rail-tie structure lifted slightly in the vicinity of each abutment, and the content of 25 bags of finer ballast (about ½ inch size) was pushed under the six ties on each side of the bridge, using shovels and forks. This packing was repeated after the passage of two trains. When shovel packing was completed, the removed crib-ballast was returned to the cribs. The shovel packing

[6] *Stoneblowing* is a new technique where a machine blows fine ballast under the existing ties, without disturbing the existing ballast base.

was performed during the day (April 16, 1997). Thus, the moving trains were compacting the approaches while the packing was progressing. No stopping or slowing down of traffic was necessary.

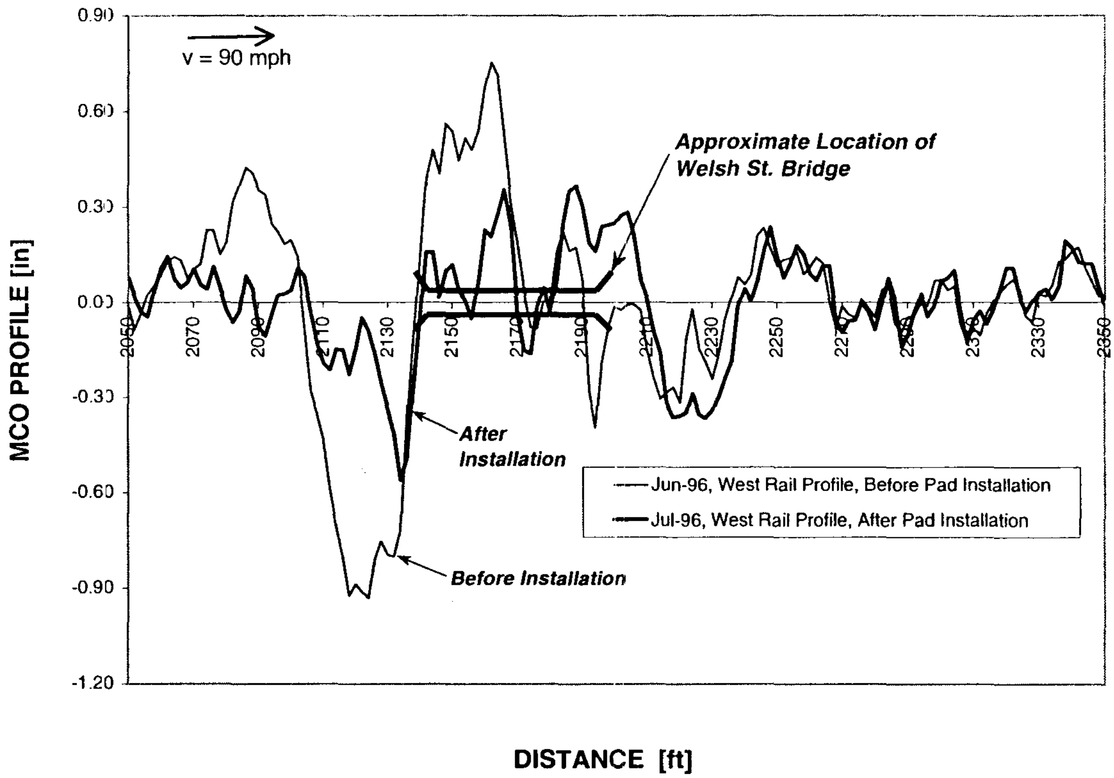


Figure 18. Effect of Matched Pads on *West* Rail Profile of Track No. 3 of Welsh Street Bridge (near Chester, Pennsylvania Station)

The effect of shovel packing was recorded by the geometry car and the resulting charts are shown in Figures 19 and 20.

The April 1997 curve, shown as a dash-dot line, was recorded before ballast-packing. This set of data resembles data recorded in July 1996 shown in Figures 17 and 18, especially at the north (left) abutment. The May 1997 data, shown in Figures 19 and 20 as short dashed lines, were recorded after ballast packing. (Note the evident additional improvement achieved by ballast packing in reducing the rail deflections near the left abutment.)

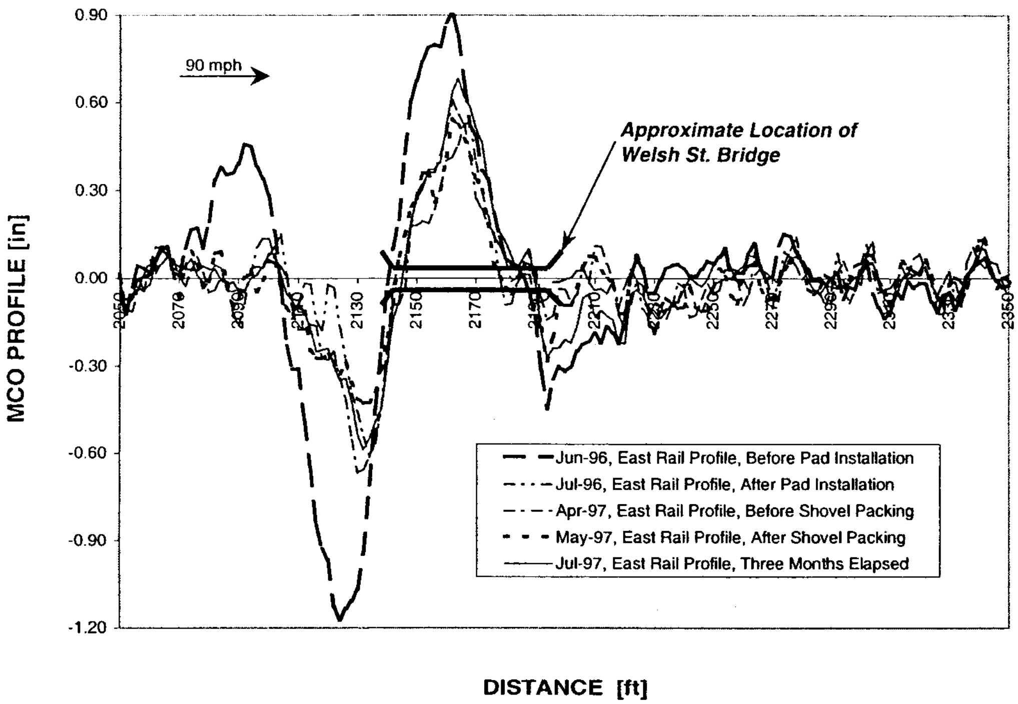


Figure 19. ***East* Rail Profile of Track No. 3 on Welsh Street Bridge, Before and After Shovel Packing**

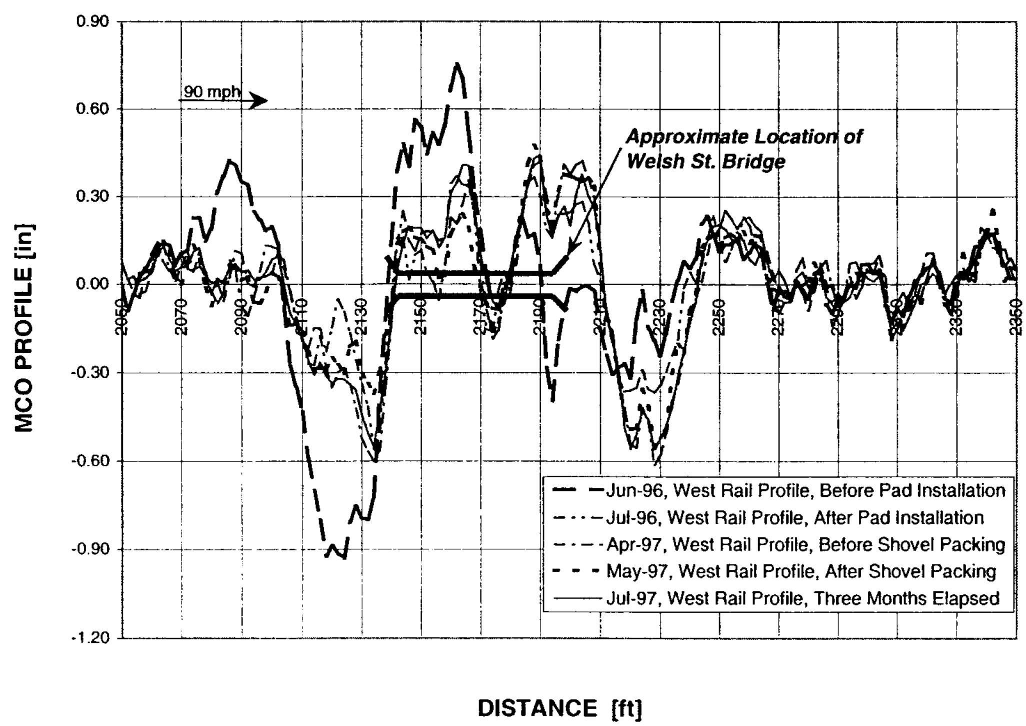


Figure 20. ***West* Rail Profile of Track No. 3 on Welsh Street Bridge Before and After Shovel Packing**

Also shown, as solid lines, are the recorded data of July 1997 (2 months later). (Note: When compared to May 1997 data, these graphs indicate a deterioration of the transition region.) This indicates that the improvement achieved by ballast packing was only temporary.

There was evidence that in the vicinity of each abutment the subgrade was wet and the ballast was fouled. This became very obvious after the crib ballast was removed during the ballast-packing operation. Therefore, it was decided to stabilize the ballast and subgrade in the immediate vicinity of the abutments by installing strip drains, as shown in Figure 21.

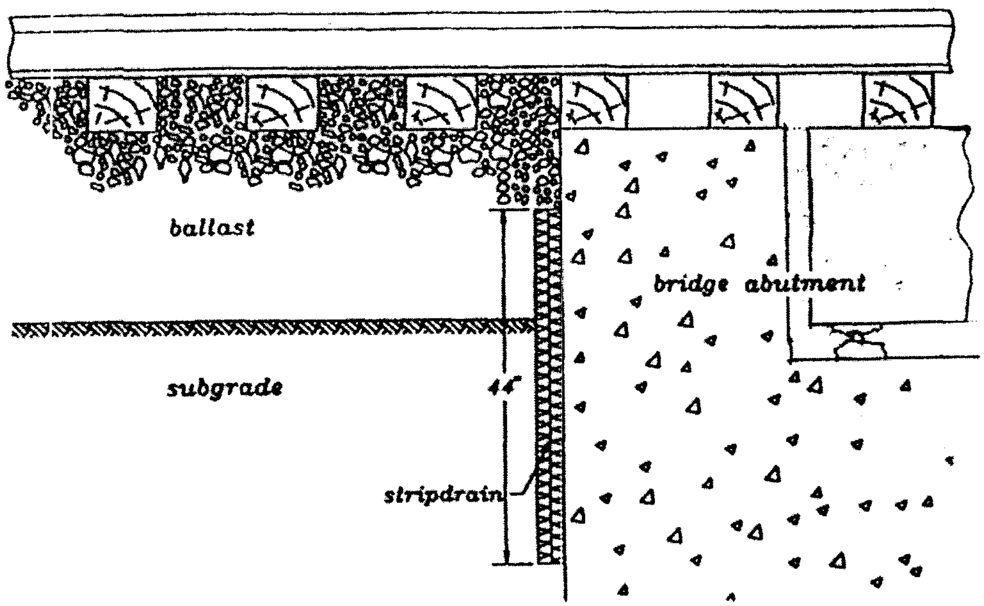


Figure 21. Schematic Diagram of Strip Drains Location

The Contech Corporation donated 125 feet of strip drains to this project, for installation at the Welsh Street Bridge. They are stored at the Wilmington, Delaware railroad station. Installation is being delayed due to the track time required and possible modifications to the bridge abutments. As yet, no date for installation has been set by Amtrak.

As another check of pad effectiveness, consider Amtrak's geometry car recordings of the vertical car *accelerations* in the vicinity of Chester Station before pad installation (February 29, 1996) and after (July 25, 1996). The results are shown in Figures 22 and 23. Note the reduction of the vertical acceleration (i.e., the "calming") of the car after pad installation in the vicinity of the Welsh Street Bridge.

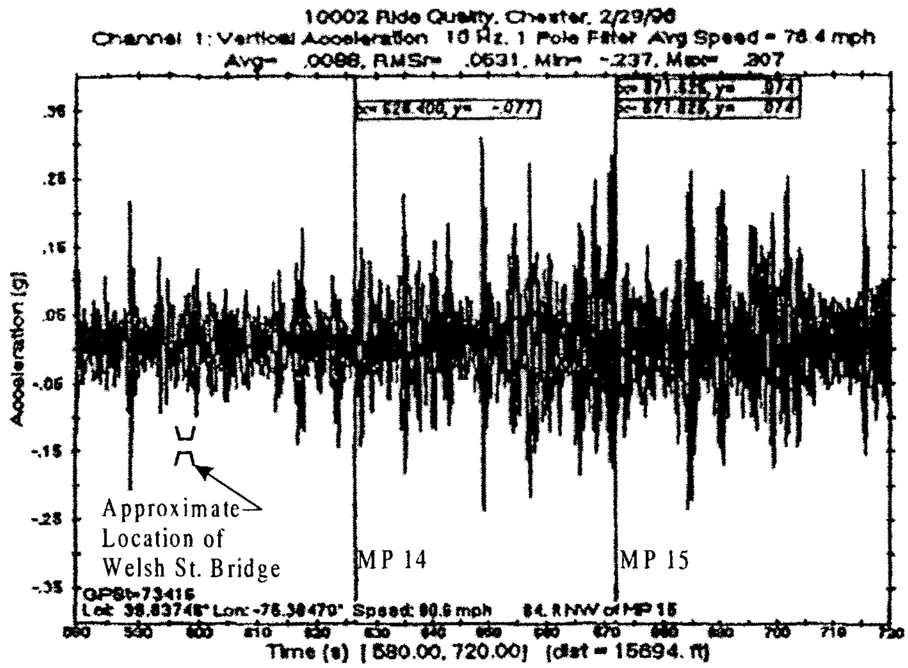


Figure 22. Vertical *Acceleration* of Geometry Car *Before* Pad Installation (February 29, 1996)

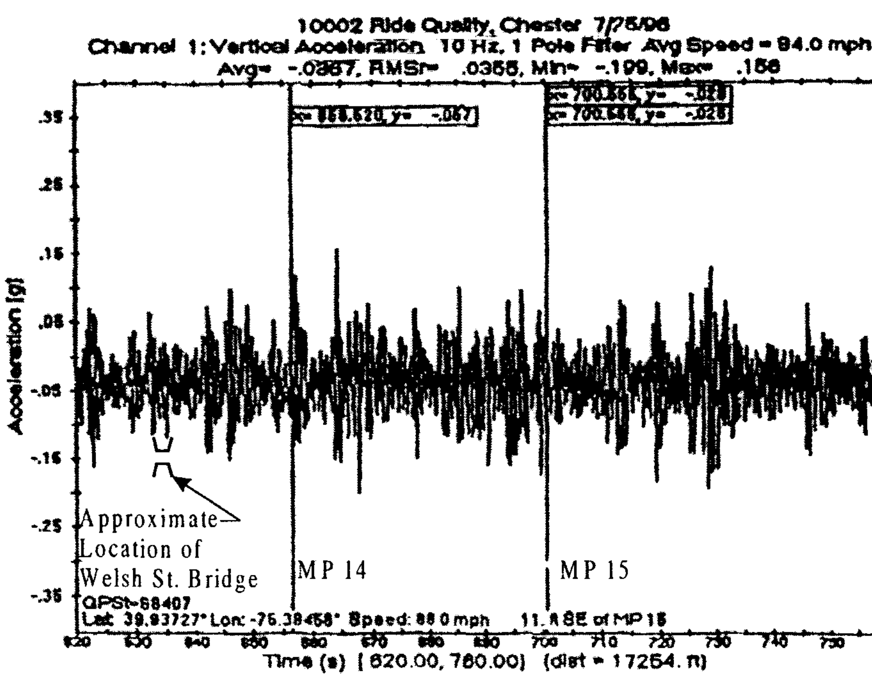


Figure 23. Vertical *Acceleration* of Geometry Car *After* Pad Installation (July 25, 1996)

Amtrak has conducted a Roughness Index (R^2) study[7] for the bridges on Track No. 3, in the vicinity of Chester Station. It covered 10 short bridges, including the Welsh Street Bridge. This is the section of the NEC that limits train speeds to 90 mph (from 125 mph). The results of this study are shown in Figures 24 and 25. Figure 24 contains the Roughness Index plots for the profile *before* installation of the matched pads. Figure 25 contains the calculated plots for the profile shortly *after* pad installation on Welsh Street Bridge. Note that, whereas before pad installation, the peak of the R^2 plot at the Welsh Street Bridge is the *largest* among the 10 Chester bridges, after installation of the matched pads the Welsh Street Bridge exhibits one of the smallest R^2 profile peaks. Thus, the installed matched pads and the track lining and surfacing significantly reduced the R^2 profile plot at the Welsh Street Bridge.

In July 1998, Amtrak decided to upgrade the Welsh Street Bridge by replacing the stringers and installing new ties and spring-clip fasteners. Therefore, this test had to be discontinued. Amtrak installed new modified matched pads, redesigned by Dr. Kerr, on the upgraded Welsh Street Bridge. Their response is being monitored. The pads removed from this bridge were not available for further testing on condition or deterioration.

5.3 CONCLUSIONS BASED ON THE RECORDED RESULTS OBTAINED TO DATE

The foregoing discussion and the graphs presented show that even without the special compaction of the track near the abutments, the improvement caused by the installation of the matched pads and track surfacing is very significant and their use should be considered when improved track longevity at transitions is required to increase the maximum allowable train speed.

It is essential to develop effective compaction methods, to achieve a long term solution in maintaining adequate track profile along the track sections that adjoin the abutments.

[7] The Roughness Index is calculated for any track geometry parameter as

$$R^2(y) = \sum_{n=-50}^{50} \frac{(y_n * 100)^2}{100^2},$$

where y is the track geometry parameter (vertical deflection in this study). It is calculated using the first 50 data points to either side of the specified location.

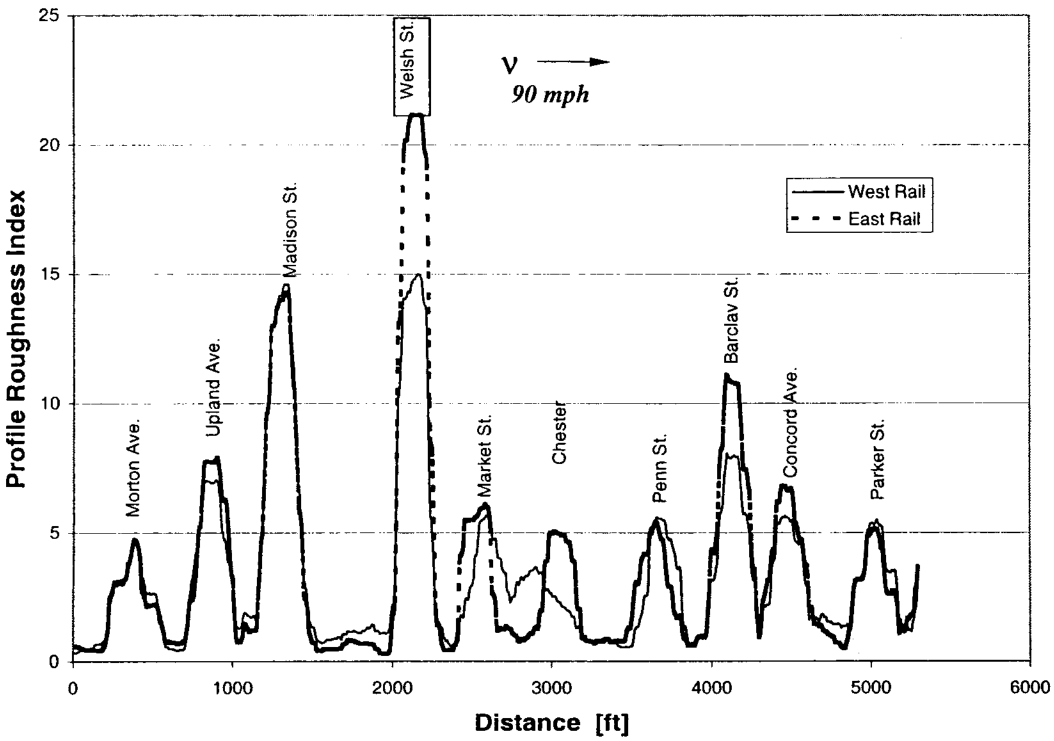


Figure 24. R^2- Profile Plots for the Chester Station Bridges *Before* Pad Installation (MP13-MP14, Track No. 3, June 1996)

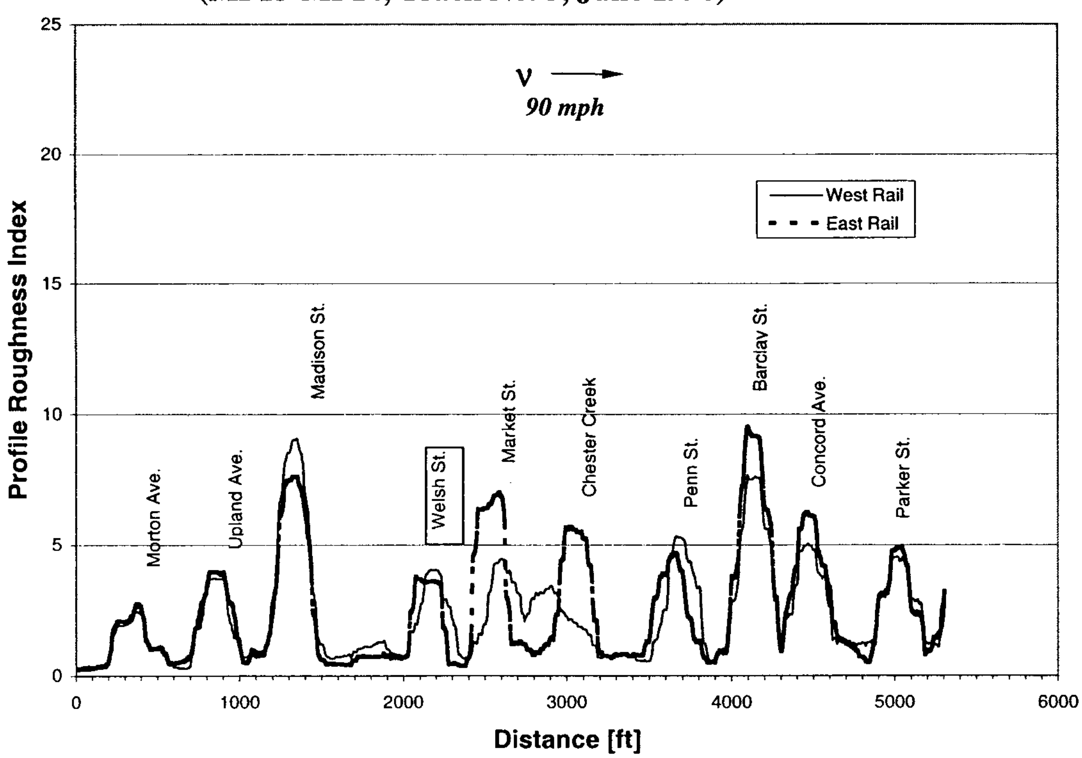


Figure 25. R^2-Profile Plots for the Chester Station Bridges *After* Pad Installation (MP13-MP14, Track No. 3, July 1996)

6. INSTALLATION OF MATCHED PADS ON A NORFOLK SOUTHERN BRIDGE

To diversify the use of matched pads on freight lines, Dr. Kerr approached NS to inquire if they would participate in the track transition study by installing matched pads on a short "rigid" bridge on one of their higher speed lines. The pads would be designed by Dr. Kerr and manufactured by Alert.

The first bridge considered was found to be unsuitable for the planned pad installation because of a nearby road crossing. A more suitable bridge was recommended, located in a tangent section with no road crossing in the vicinity, near Catlett, Virginia, about 20 miles southwest of Manassas. The allowable speed on this line is about 80 mph. The suggested open-deck bridge accommodates 27 wood ties. The center-to-center tie spaces on the bridge are 16 inches. The tie plates used by NS on this bridge are 18 inches long (compared to 14.75 inches on the Welsh Street Bridge). Because the contact area between pad and wood tie on the NS bridge was larger than on the Welsh Street Bridge, the pad cross-section had to be modified, in order to soften the rail response on the bridge and match the *k*-value of 2,750 to 3,000 lb/in^2, anticipated in the adjoining wood-tie tracks. Therefore, the groove dimensions were modified by changing the 1/4 inch × 1/4 inch square grooves to *trapezoidal* grooves, as shown in Figure 26.

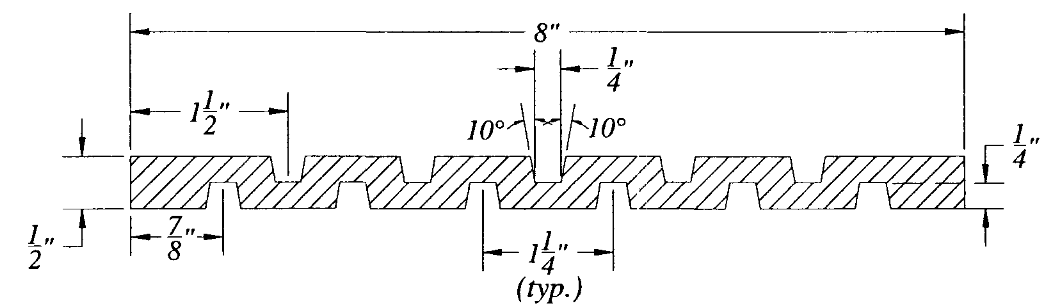


Figure 26. Modified Pad Cross-Section for Use on NS Bridge in Wood-Tie Territory

The changes in the pad size and groove shape required Alert to produce another mold. Two manufactured pads, 18 inches × 8 inches in size, were received in February 1997 and subjected to the compressibility test, as shown in Figure 14. The obtained load-deflection curves for each pad were very similar to those shown in Figure 16. They corresponded to *k*-values of 2,941 lb/in^2 and 2,778 lb/in^2, respectively; thus, the new pad satisfied the matching criterion used for wood-tie tracks. Next, Alert produced 60 tie pads and shipped them to NS for installation.

These pads were installed on the open-deck bridge near Catlett, Virginia, on March 24, 1997. Because of the relatively light traffic density on this line at the time, the installation was performed during the day by a four-man crew from the NS Bridge and Building Division. This installation lasted only a few hours and was videotaped.

Within 3 weeks, ride quality improvement over this transition area had been noticed by an NS locomotive engineer who operates on this line.

The first vertical rail profiles received from NS were recorded by the geometry car on March 18, 1997 (before pad installation). Two later recordings were made 6 months apart, on September 10, 1997 and March 16, 1998. They are shown in Figure 27.

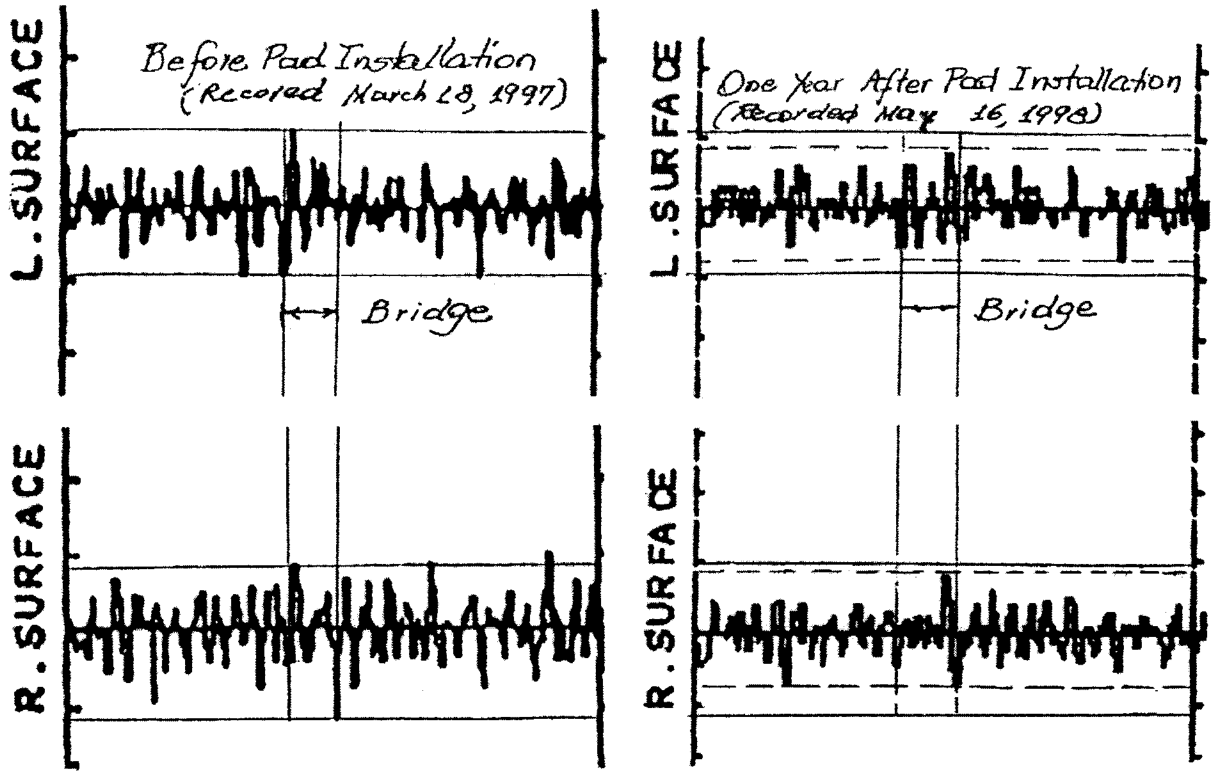


Figure 27. Recorded Vertical Profiles from NS Bridge at Catlett, VA (Matched Pads Installed March 24, 1997)

From examining these track geometry strip charts, it appears that the installation of the pads and other associated work have reduced the vertical geometry deviations in the transition zone by about 25 percent. Solid lines enclosing the extreme values of the rail profile changes recorded before pad installation on March 18, 1997 also were placed on the more recent strip chart of May 16, 1998 to facilitate a comparison. The monitoring of this bridge is continuing. More frequent monitoring is not possible, because NS runs its geometry cars over this line only twice a year.

7. SECOND INSTALLATION OF MATCHED PADS ON THE NORTHEAST CORRIDOR

Whereas on the Welsh Street Bridge the bridge pads had to match the adjoining wood-tie tracks, a decision was made to generalize this study by installing matched pads on a bridge with adjoining *concrete-tie* tracks; the usual situation on the Northeast high-speed corridor.

An inspection of the Amtrak tracks in the Wilmington, Delaware and Philadelphia, Pennsylvania regions, resulted in the selection of the Reading RR bridge over CSX at milepost 84.21 in North Philadelphia for pad installation. It is a relatively short, rigid, open-deck bridge that utilizes 41 wood ties (10 inches × 8 inches × 8 feet 6 inches) with standard AREA tie plate (14.75 inches × 8 inches) and cut spike fasteners. The adjoining tracks utilize prestressed concrete ties, up to the bridge abutments. There are numerous open-deck bridges with similar configurations on the high-speed NEC.

As suggested by the *k*-tests near the Chester, Pennsylvania railroad station described previously, the pads installed had to match the stiffer concrete-tie track with $k \cong 5{,}250$ lb/in^2. Based on preliminary tests, Dr. Kerr designed a pad 3/8 inches thick with *trapezoidal* grooves on one side only, as shown in Figure 28.

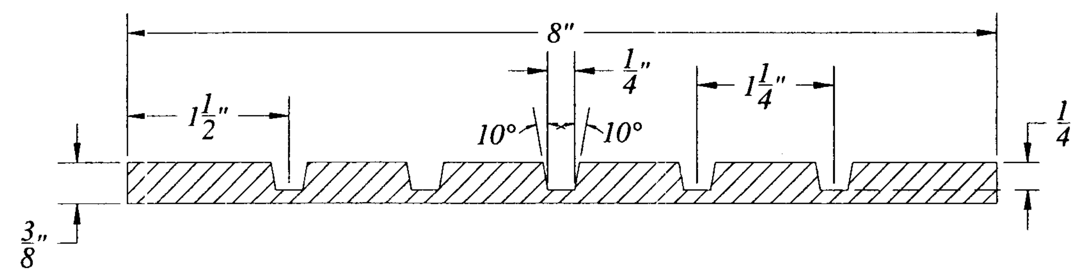


Figure 28. Matched Pad for Use on Amtrak Bridge in Concrete-Tie Territory Over the Reading Line

The necessary pad (and tie) "spring constant" is $\kappa = a \times k$ where a is the center-to-center tie spacing on the bridge. Since on this bridge $a = 16$ inches, and $k = 5{,}250$ lb/in, κ should be $16 \times 5{,}280 = 84{,}000$ lb/in. In August 2001, Alert ordered a precision mold for the casting of pads with the cross-section shown in Figure 28. The first two pads were cast and then tested in the lab for their stiffness (i.e. their κ-value). They were too stiff. Then, Alert cast two softer pads and shipped the pads to Amtrak for installation on the Reading RR bridge over CSX. Installation took place on April 7, 2000, and the performance of this bridge is being monitored.

8. A RELATED STUDY: THE DYNAMIC RESPONSE OF PAD AND WOOD TIE

In the foregoing study, the pad stiffness κ was chosen using static considerations – a procedure generally valid for relatively low train speeds, υ. But the stiffness of many materials may be affected by the rate of loading. It was, therefore, necessary to find out if, for high-speed trains with $80 < \upsilon_o < 150$ mph, the stiffness κ of the chosen pads would be affected (for example, by exhibiting a harder response). The purpose of this study was to determine if this effect is significant.

In this study, the wheel of a moving train was replaced by a moving "force train," as shown in Figure 29. Considering a fixed location on the track, A, the effect of this moving train was simulated by a dynamic actuator at point A that operates at a prescribed frequency.

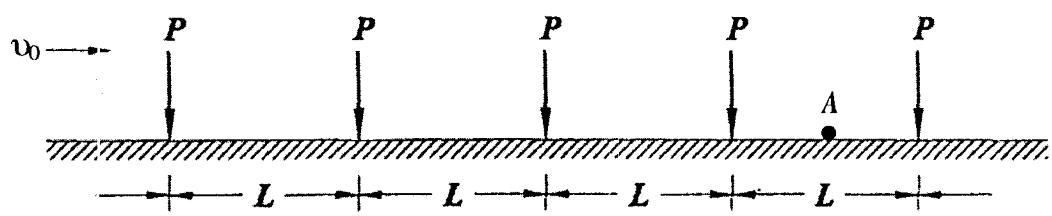


Figure 29. Substitute "Force Train" in Motion

The frequency of the loads passing A is

$$f = \frac{1}{T}, \tag{13}$$

where T is the time between two moving consecutive loads P passing point A. Noting that

$$L = \upsilon_o T \tag{14}$$

thus, that

$$T = \frac{L}{\upsilon_o}, \tag{15}$$

it follows from Equation (13) that the frequency is

$$f = \frac{\upsilon_o}{L}. \tag{16}$$

For example, the train speed

$$\upsilon_o = 120 \text{ mph} = 120\frac{5{,}280}{3{,}600} \text{ ft/sec} = 176 \text{ ft/sec} \tag{17}$$

corresponds to the actuator driving frequency of

$$f = \frac{\upsilon_o}{L} = \frac{176}{85} = 2.1\,/\text{sec} = 2.1 \text{ Hz} \tag{18}$$

where L = 85 feet is the approximate length of an Amfleet car. This is a frequency that falls within the low range of the available dynamic actuators at the Structures Laboratory. The next step was to mount the pad-tie specimen in the dynamic tester, obtain the corresponding dynamic κ-values, and then compare them with those of the corresponding static tests.

The test set up was similar to that shown in Figure 13, except that the load was generated by an actuator for the present tests. A typical load-deflection result recorded by the actuator for the first 5 cycles at 2.0 Hz, is shown in Figure 30. Also shown is the linearized response. The corresponding κ-value was determined to be 66,667 lb/in and the equivalent k = 4,167 lb/in^2.

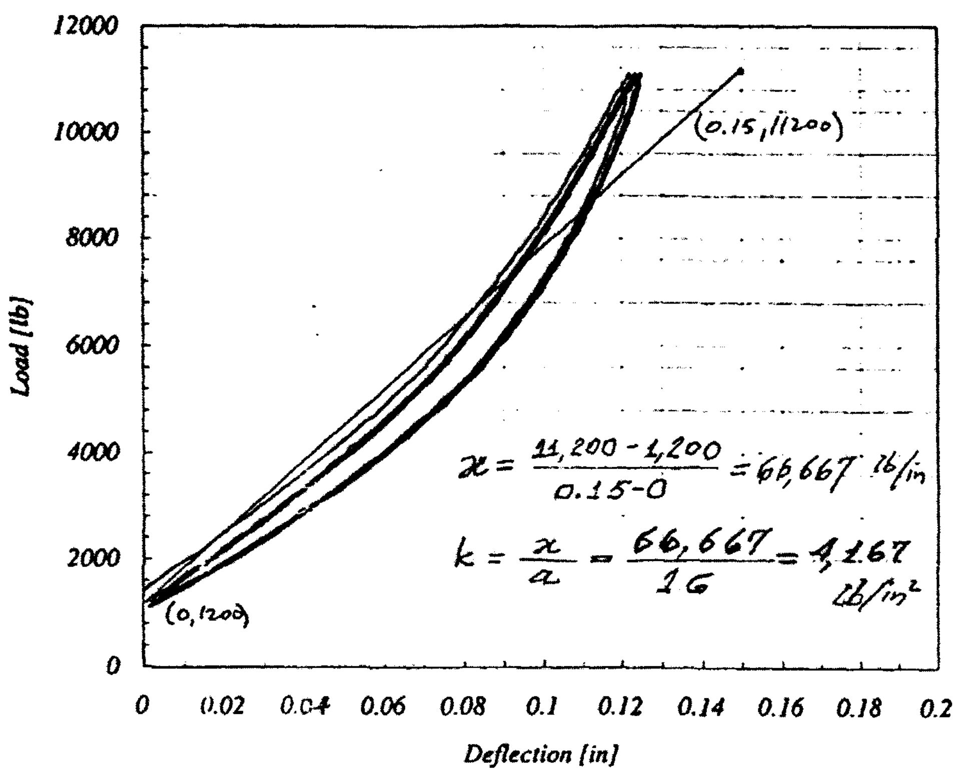


Figure 30. Dynamic Load-Deflection Data for 2.0 Hz

Dynamic tests were conducted at the frequencies presented in the following table, noting that $\upsilon_o = f \times L$:

Table 2. Dynamic Test Results

Load Frequency (Hz) in Test, f	quasi-static	0.2	0.5	1.0	1.5	2.0
Corresponding Train Speed (mph), υ_o	≈ 5	17.0	42.5	85	127.5	170.0

It was found that as the frequency (train speed) increases, the *k*-value increases slightly from the *k*-value of about 3,000 lb/in^2 determined from the quasi-static test. The *k*-values ranged from 2,910 at 0.2 Hz (12 mph) to about 3,333 lb/in^2 at 2.0 Hz (120 mph); a rather small increase.

Dynamic testing of these pads was subsequently conducted to determine the long-term effects that repetitive dynamic loads have on the stiffness of the matched pads used. Taking into consideration the dynamic magnification of a static wheel load due to train speed, the amplitude of the applied "rail-seat" force was increased from 10,000 lbs (stated in Equation (7)) to 15,000 lbs. In each test the matched pad was subjected to 40,000 cycles. *Static* compressibility tests were conducted before and after the dynamic repetitive tests and no significant difference in pad stiffness was detected.

This laboratory finding was confirmed on the matched pads removed from the Welsh Street Bridge after more than 2 years of exposure to high-speed traffic and the environment. The removed pads appeared to be in very good condition.

9. CONCLUSIONS

The field results obtained to date reveal that the *improvement* due to the installation of matched pads and the associated treatment of the tracks near the abutments *is very significant*. Also note that, according to Amtrak, the Welsh Street Bridge did not meet Amtrak's ride quality standards prior to pad installation. However, once the pads were installed and the approaches tamped, the transition complied with these standards. The design, production, and installation of the developed matched pads follow a very simple procedure. Unlike some of the methods used to smooth a transition region, that require extensive track work in the adjoining tracks, the matched pads may be installed on a short, open-deck bridge in about 3 ½ hours.

The method described, which is based on the use of matched pads, is very economical. It requires 3 to 4 hours using an installation crew of 4 to 5 workers. The current price per pad is about \$45. Thus, the cost of the pads for a bridge with 50 cross ties is $2 \times 50 \times 45 = \$4{,}500$. The pad installation, coupled with tamping of bridge approaches, is simple and does not require any sophisticated equipment.

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