TECHNOTE

Synthesis—The Reduction and Analysis of Pavement Profiler Data to Quantify the Bump at the End of the Bridge

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INTRODUCTION AND BACKGROUND

Relative settlement between a bridge deck and approach pavement results in an elevation difference at the joint between the two. This difference in elevation is generally known as the bump at the end of the bridge (BEB) and has been recognized as a common problem throughout the Nation. Depending on its severity, a BEB can not only cause discomfort to the driving public but can also damage vehicles or lead to a driver's loss of steering control. As a result, a BEB reduces the serviceability of approach pavements and can often lead to liability concerns for transportation agencies. Moreover, it leads to decreased service life of bridges and increased maintenance requirements for both automobiles and bridge infrastructure. The annual maintenance cost associated with this problem was reported at approximately $100 million in 1997 (Briaud, James, and Hoffman 1997). Accounting for inflation, this cost is approximately $160 million as of 2019 (In2013dollars.com 2019).

The occurrence of a BEB is usually detected qualitatively based on road-user feedback, and maintenance strategies are implemented accordingly to improve ride quality. Although bridge approaches and departures often represent locations with rough ride quality, it is not common practice for transportation agencies to measure this roughness quantitatively. Past research has indicated that smooth pavements correspond to the following benefits over rough pavements (National Highway Institute (NHI) 2001):

- Reduced fuel consumption and vehicle maintenance costs.
- Longer bridge service lives.
- Lower dynamic loads on pavements.

A research study conducted at the WesTrack pavement testing facility near Reno, NV, observed that a rehabilitated pavement section with a smoother surface corresponded to significant reductions in fuel consumption and truck spring failures (Sime, Ashmore, and Alavi 2000). The reduction in fuel consumption corresponded to 46,660 L (12,326 gal) of fuel savings for a trucking company with a fleet operation of 1.6 million km (1 million mi). Similarly, smoother bridge approaches can lead to significant benefits in terms of reduced bridge and vehicle damage (Wahls 1990). Factors contributing to the BEB problem have been identified through an evaluation of the structural and geotechnical aspects involved in bridge design and construction (Schaefer et al. 2013). Transportation agencies use standardized equipment, such as the inertial profiler, to measure pavement roughness but typically not to evaluate bridge approaches for BEBs. Data collected using inertial profilers are commonly expressed in terms of standardized surface roughness indices, such as the International Roughness Index (IRI), to indicate ride quality on a particular roadway.

To facilitate efficient pavement and bridge management programs, it is important to implement practices that measure and quantify roughness at bridge approaches. It is vital to establish standardized quality indices based on quantitative approaches. Engineers can use these indices to determine the condition of a particular bridge approach and subsequently evaluate the need for maintenance and rehabilitation efforts. This research study was initiated to explore potential methods to quantify the surface roughness at bridge approaches and analyze the advantages and disadvantages of each method. This TechNote presents findings from an extensive literature review carried out under the scope of this project.

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OBJECTIVE AND SCOPE

The primary objective of this synthesis was to identify different available methods that use inertial profilers to quantify ride quality at bridge approaches. To understand current quantification techniques, this TechNote includes information and research found on the current status of the BEB issue. Information collected through this synthesis effort will facilitate successful completion of future data analyses of numerous bridges.

TECHNOTE OUTLINE

This TechNote reports findings from an extensive literature review carried out related to the BEB problem and potential methods to quantify this problem. Initially, a brief discussion is presented on different factors contributing to the development of BEBs. In an effort to identify potential methods to quantify the bump magnitude at bridge approaches, this TechNote provides background information on the concept of surface profile measurements and methods to quantify the roughness of a particular surface profile. This is followed by information on different profile measurement methods used for transportation infrastructure, such as roadways, railroads, and airfield pavements. Next, different proposed methods focused on quantifying the roughness at bridge approaches are considered, with particular attention paid to reduction and analysis of profile data collected using inertial profilers. Finally, this TechNote describes different remedial measures adopted in practice to mitigate BEBs.

BEB CAUSES

The BEB is a complex issue caused by multiple interactive factors, with differential settlement being one of the primary factors. Settlement occurs at the interface between the bridge abutment and approach embankment. This can be caused by multiple factors, including seasonal temperature changes causing lateral movement of the superstructure, erosion of backfill materials, poor construction of joint and drainage systems, poor soil compaction of the approach way near the bridge interface, and soil settlement under the embankment (Seo, Ha, and Briaud 2002).

Several research efforts have focused on identifying different factors contributing to BEBs and suitable remedial measures. High-traffic loads and the breakdown of bridge surface materials are among some of the secondary factors causing surface roughness (Phares et al. 2011; Lu et al. 2018). However, the primary factors contributing to the differential settlement resulting in a BEB have been identified as follows (Briaud, James, and Hoffman 1997; Abu-Hejleh et al. 2000; Briaud et al. 2002; Phares et al. 2011; Nicks and Adams 2017; Short et al. 2018; Lu et al. 2018; Hassona et al. 2018):

- Poor design and construction practices.
- Soil erosion and void development because of inadequate drainage systems.
- Thermal expansion and contraction from daily and seasonal temperature variation.
- Varying characteristics of different geographical regions.
- Inadequate drainage.

Differential settlement can occur in the embankment or foundation soils between the approach road and the bridge structure, particularly those bridges supported by deep foundations. An illustration by Briaud, James, and Hoffman (1997) shows different factors contributing to the development of a BEB at bridge approaches (figure 1).

Compaction issues are a common occurrence and another leading cause of the development of the BEB, as shown in figure 1. These issues are found at bridge sites where compressible and cohesive soils exist in foundations or are used to construct embankments. Achieving a specific level of compaction is difficult at the embankment, allowing for unwanted settlement during the service life of the structure. Some States have increased compaction requirements, especially near the abutment, to reduce excessive settlement (Short et al. 2018).

Thermal cycling is another prominent issue leading to void development and changes in road surface elevation. Expansion and contraction of materials and components of a bridge structure, caused by daily and annual temperature cycles, can have an adverse impact on the integrity of a bridge approach. Thermal effects are most prominent in structures using integral abutments (Phares et al. 2011). While integral abutment bridges were believed to be a solution to many BEB issues, "the bump at the end of the bridge is still a consistent problem with integral abutment bridges" (Short et al. 2018, p. 8). As integral abutments do not include expansion joints, expansion and contraction occur in the approach backfill, creating a gap that increases in size with each expansion/contraction cycle (Phares et al. 2011). Void development leads to increased settlement, water seepage into the foundation soil, and soil erosion. Temperature fluctuations can also cause cracking in the surface layer of the pavement, leaving foundation soils susceptible to water drainage and erosion. Increased roughness of bridges and pavements may also occur as different materials and components undergo volume changes because of temperature fluctuations.

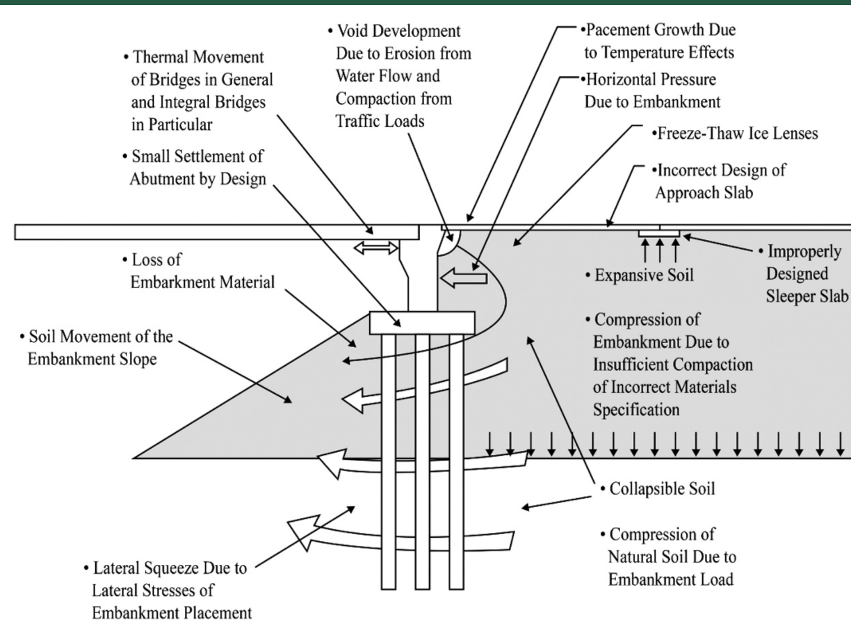
Structural design components of bridges can also impact the development of BEBs. According to Briaud, James, and Hoffman (1997) and Nicks and Adams (2017), skew angles and approach slabs contribute to the development of BEBs. Based on their analyses of eight bridges in St. Lawrence County, NY, Nicks and Adams (2017) reported that, in two bridges with similar ages and abutment heights, there were significant differences in the extent of the BEB problem; this was because of variations in the abutment orientation. One bridge was built perpendicular to the roadway, while the other had a 20-degree skew. The skewed structure "produced almost double the amount of maximum surface deviation at the bridge approach" (Nicks and Adams 2017, p. 12). The study also observed additional bumps at the approach/departure slab locations as well as at the start and end of the bridge. While approach slabs were installed to improve the BEB issue, they contributed to the problem for this subset of bridges.

NEED TO QUANTIFY BEB MAGNITUDES

Although several studies in the past have investigated BEBs, most focused primarily on qualitative evaluation of the BEB and did not attempt to quantify its magnitude (Briaud, James, and Hoffman 1997; Mishra 2006). While performance issues related to BEBs are relatively obvious, quantification of these issues is difficult. BEBs can progressively deteriorate through a negative feedback loop. In other words, once developed, a BEB will become worse with time. A similar phenomenon is observed when analyzing pavement surface roughness. For example, Smith et al. (1997) analyzed the effect of initial pavement smoothness on future pavement smoothness for 200 pavement projects and observed that initial smoothness was a significant factor governing future smoothness for 80 percent of portland cement concrete and asphalt concrete pavements. Smith et al. (1997) also analyzed pavement smoothness data for the American Association of State Highway Officials (AASHO) Road Test and observed that smoother sections tended to stay smoother over time (Highway Research Board 1962).

Methods to quantify the surface roughness at bridge approaches are being developed using pavement profile data. Surface profile indices, established to quantify the ride quality or pavement roughness at bridge approaches, can significantly aid transportation agencies in developing effective bridge management and maintenance programs. To facilitate the development of effective bump quantification methods at bridge approaches, an indepth understanding of surface profiles and their attributes is imperative. The following sections briefly analyze different methods available to quantify surface roughness for pavements as well as railroad tracks, which experience similar issues related to BEBs.

Figure 1. Illustration. Problems contributing to the development of a BEB (Briaud, James, and Hoffman 1997).



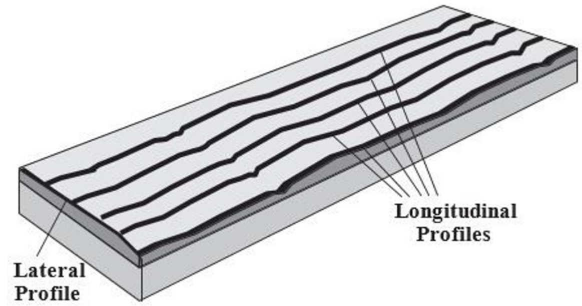
SURFACE PROFILE AND QUANTIFICATION METHODS

Surface profiles, defined as two-dimensional sections of a surface, are useful representations of the smoothness along roadways and railroad tracks. Depending on the direction of measurement, the surface profile can either represent the longitudinal or lateral profile of a roadway. Figure 2 shows a schematic of a roadway with the longitudinal and lateral profiles highlighted. Generally, a longitudinal profile is used to measure the design grade, roughness, and texture on any road, pavement, or ground surface. Lateral profiles indicate superelevation and distresses along the surface. These profile measurements can also be used for railroad track geometry and airfield pavement roughness (Sayers and Karamihas 1998).

Surface profiles are measured using various methods and devices, such as rods and levels, rolling straightedge (RSE), profilograph, response-type instruments, walking profilers, and inertial profilers (Sayers and Karamihas 1998; Zang et al. 2018). When measuring a surface profile using any data collection method, it is necessary to understand and interpret the results based on the exact type of information being gathered by that particular method. Not all measurement approaches are capable of establishing the true profile of the roadway surface. In other words, the measurements obtained using these devices are dependent on device-specific dimensions and attributes. Depending on its dimensions, a particular measurement device may not capture certain roughness features of the surface being monitored. The RSE, response-type road roughness measurement system (RTRRMS), and profilograph are equipment that can be used to measure the smoothness of a roadway surface, but they are not capable of measuring the true profile of the pavement (NHI 2001). An RSE, for example, can report different numbers based on the wavelengths present in the surface roughness; certain wavelengths are amplified and others are attenuated (Sayers and Karamihas 1998). A good example of this dependence on wavelength can be observed from practices within the railroad industry, where track geometry defects are often measured using the concept of mid-chord offset (MCO).

The MCO approach is based on measuring the "deviation from a measure of uniformity. The uniform reference is commonly a straight line, or chord where error is measured at some point along it, typically at the mid-point" (Li et al. 2015). In the United States, commonly used chord lengths are 9.45, 18.90, and 38.40 m (31, 62, and 124 ft, respectively). It

Figure 2. Illustration. Longitudinal and lateral profiles of a roadway segment (Sayers and Karamihas 1998).

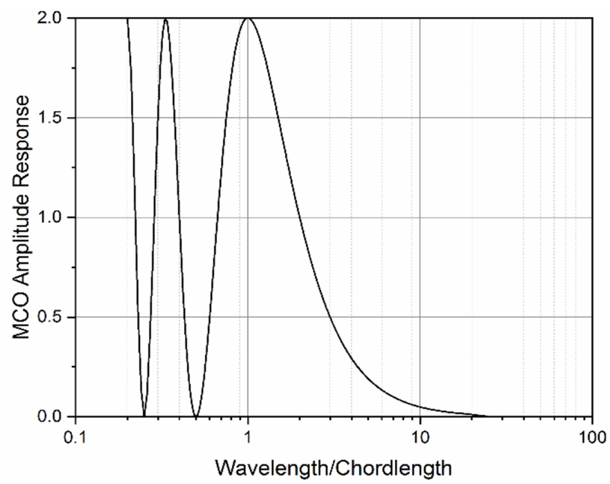


is important to note that MCO values reported for a particular track geometry can change significantly based on the relative magnitude of the chord length compared to the wavelength associated with the track defect. This phenomenon is easily illustrated by plotting the MCO values against the wavelength-to-chord-length ratio. As seen in figure 3-A, when this ratio equals unity, the MCO approaches a value of 2.0. This means the MCO value will equal twice the amplitude of the waveform inherent in the track geometry. "For example, a 62-ft [18.90-m] MCO measurement of an error wavelength of 62 ft [18.90 m] will have twice (two times) the actual wavelength amplitude" (Li et al. 2015). Similarly, measuring a 9.45-m (31-ft) wavelength using an 18.90-m (62-ft) chord (wavelength-to-chord-length ratio = 0.5) will result in an MCO value of 0. Such scenarios can also arise for composite waveforms. Figure 3-B presents a scenario where the composite waveform was constructed using three waveforms with wavelengths equaling 3, 6, and 12 m (9.84, 19.69, and 39.37 ft, respectively). The MCO measurements were carried out using a chord length of 24 m (78.7 ft). As shown in the figure, the measurement resulted in an MCO value of 0, which is not an accurate representation of the geometry error.

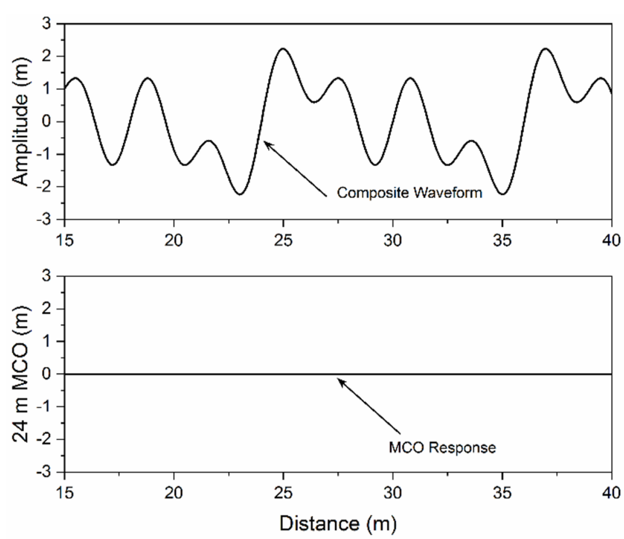
Just like the MCO measurements, surface deviation measurements using an RSE for pavements can lead to erroneous values depending on the relative length of the RSE with respect to the wavelength of the surface roughness. It is therefore of utmost importance to ensure that the dimension of the straightedge being used is adequate to capture the roughness wavelengths pertinent to road-user comfort. Unfortunately, no standard specification exists with respect to typical wavelengths present in a pavement surface profile. Detailed information about the surface profile can be extracted only by using multiple devices capable of capturing different characteristic wavelengths.

Just like the RSE, roadway smoothness measurements using RTRRMS or profilographs have inherent disadvantages. For example, measurements using RTRRMS units are affected by characteristics of the mechanical system and by speed of travel; they are not transportable across devices or comparable across units, and they lack a standard roughness scale (NHI 2001; Rose et al. 2009). Similarly, measurements using profilographs can vary significantly depending on the wavelength of surface roughness. Moreover, there are concerns about how well a profilograph measures wavelengths related to ride quality (NHI 2001).

Figure 3. Graphs. Representation of MCO-based measurements and geometry error (Hyslip 2002; Li et al. 2015).



A. Effect of the ratio of error wavelength to measuring chord length on MCO value.



Source FHWA; based on concepts from Li et al. 2015.
1 m = 3.28 ft.

B. Errors associated with the MCO approach where a composite waveform is reported with an MCO value of 0.

In general, a suitable profile measurement device should measure components of the true surface profile to provide an acceptable representation and deliver relevant information for data reduction and analysis (Sayers and Karamihas 1998). The following subsections present information about commonly available equipment used to establish the true profile of a pavement surface. Using any of this equipment at a bridge approach will help establish the true approach profile, which can subsequently be analyzed to calculate desired ride quality indices or BEB magnitudes, as specified by a transportation agency. These desired/acceptable bump levels would ideally be established based on factors such as bridge type, speed of operation of vehicles, and/or local construction practices.

Equipment to Measure True Surface Profile of a Pavement Surface

Different alternatives available to establish the true profile of a pavement system can be used as baseline references during pavement surface profile measurements. Surface roughness data collected using other commonly used equipment, such as inertial profilers, can be compared against these baseline data to evaluate their accuracy. Detailed discussions on inertial profilers are presented later in the Inertial Profiler subsection of this TechNote.

Rods and Levels

Using a rod and level is a common measurement technique in land surveying and for bridge construction. However, this method can also be used to establish the roughness of a given surface by taking elevation readings at small intervals to create a profile of surface elevations. Figure 4 illustrates the common setup for rod and level measurements. Measuring surface roughness using a rod and level is a tedious process compared to normal uses for these surveying tools. Elevation measurements must be taken at intervals of 305 mm (12 inches) or less. The accuracy of each height measurement must be within 0.5 mm (0.02 inch), as indicated by Sayers and Karamihas (1998). Requirements for measuring surface profiles using a rod and level can be found in ASTM E1364 (ASTM International 1996).

The Dipstick®

Another common device used to measure road profiles is The Dipstick®, as shown in figure 5. While this method is more efficient than a rod and level, it still requires manually guiding the device slowly along the profile being measured.

The Dipstick® "contains a precision inclinometer that measures the difference in height between the two supports, normally spaced 305 mm [12 inches] apart" (Sayers and Karamihas 1998, p. 5). The apparatus is guided along the profile path, pivoting 180 degrees about the leading foot. The device's computer constantly monitors the placement of each step, recording the elevation changes along the path. Analyses of profiles recorded from The Dipstick® and rod and level data produce similar results when the initial elevation value is matched (Sayers and Karamihas 1998).

Walking Profiler

A walking profiler (figure 6) is a multiwheeled, inclinometer-based system pushed by an operator at walking speed. Typical operational speed for this instrument is 0.8 km/h (0.5 mi/h). The device records the relative elevations of successive points at 241-mm (9.5-inch) intervals, and the incremental changes are summed to obtain the height of each measured point with respect to the starting point (NHI 2001).

Inertial Profiler

Different studies have established that inertial profilers represent one of the fastest and most advanced alternatives for profile measurement (Sayers and Karamihas 1998; McGhee 2002; Olmedo et al. 2015; Zang et al. 2018). According to Sayers and Karamihas (1998), the high speed profiling system was originally developed in the 1960s and is a vehicle-mounted laser cross section measuring system with three main components: height sensors, an accelerometer, and a distance-recording device (figure 7). Longitudinal profile data are recorded by the laser scanner and accelerometer. Inertial profilers typically come in two varieties: high speed profilers mounted on vans and lightweight profilers mounted on utility vehicles. The high speed profilers are commonly used by transportation

Figure 4. Illustration. Rod and level (Sayers and Karamihas 1998).

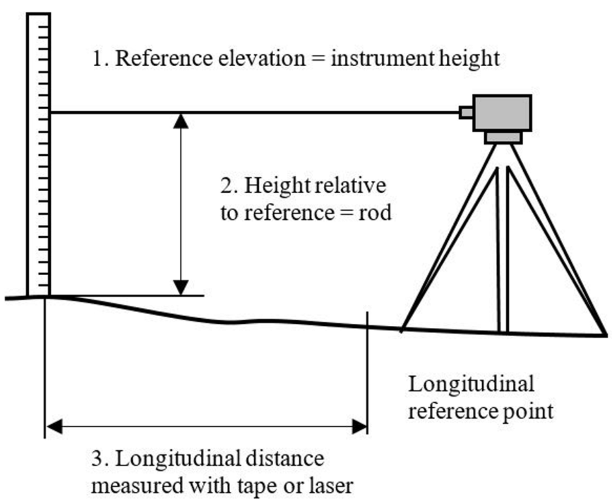
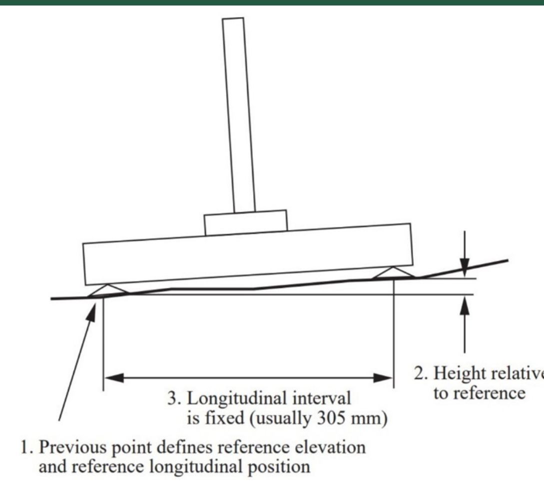


Figure 5. Illustration. The Dipstick® road surface profiler (Sayers and Karamihas 1998).



1 mm = 0.039 inches.

Figure 6. Photo. Walking profiler (FHWA Office of Technical Services Resource Center P&M TST 2015).

Source: FHWA.

Figure 7. Photos. Components of a typical inertial profiler system (Nicks and Adams 2017).

Source: FHWA.

agencies for network-level pavement management practices, whereas the lightweight versions are commonly used for construction quality control purposes and to test on top of freshly constructed concrete pavements. The principles of operation for high-speed and lightweight profilers are identical.

The inertial profiler measures the road profile and can produce various roughness statistics through reduction and analysis of the collected data (Zang et al. 2018). However, the profile obtained from inertial profiler measurements is not necessarily equal to the true elevation of the pavement. Certain features of the true profile, such as the grade of the pavement, are not reported by inertial profiler measurements (NHI 2001). This is because "the profile obtained from an inertial profiler is filtered to show profile features that are relevant to ride quality" (NHI 2001, p. 18). The primary difference between using an inertial profiler and a profilograph (commonly used in the past) is that data from the inertial profiler can be analyzed to evaluate ride quality on the pavement surface. The data from the profilograph, on the other hand, represent how the profilograph responded to the surface roughness and are not necessarily related to the ride quality experienced by typical automobile users. Inertial profilers can be useful tools to collect profile data at bridge transitions and more accurately detect BEBs.

The filters and accelerometers on inertial profilers have a significant effect on the length of roadway required for the measurements to stabilize. Swanlund and Law (2001) performed an experiment using inertial profilers with different filters to collect pavement roughness data and compare those data against the true profile collected using a walking profiler. They examined the convergence length, defined as the length of measurement using the inertial profiler after which the data matched with that from the walking profiler, and observed that it changed significantly with filter type. Table 1 lists the convergence lengths for different filter types, as reported by Sawnlund and Law (2001).

Table 1. Effect of filter type on convergence length (Swanlund and Law 2001).

Profiler	Filter Type	Convergence Length (ft)
1	100-ft high pass	50
2	300-ft high-pass	150
3	314-ft high-pass third order Butterworth	120
4	300-ft high-pass third order Butterworth	300
5	200-ft high-pass	150

1 ft = 0.305 m.

Comparing Salient Features of Different Profiling Devices

Table 2 lists pros and cons associated with different profiling devices. From table 2, it is apparent that inertial profilers present the best approach to measure pavement surface profiles. A unique benefit of the inertial profiler is its ability to operate at normal traffic speeds and therefore to measure profiles quickly. This allows measurements at multiple sites per day. It is important to understand that, because high wavelengths are filtered out, collected data needs to be analyzed using software programs to determine profile components as they relate to the true profile.

Table 2. Comparison of pavement profile testing devices (modified from Henderson et al. 2016)

Device	Pros	Cons
Rod and level	Common equipment in bridge construction and land surveying.	• Slow pace of measurement. • Tedious process.
The Dipstick®	True profile.	• Lane closure. • Site preparation necessary. • Slow pace of measurement (approximately one site per day). • A 0.305-m (1-ft) sample rate.
Walking profiler	• A 25.4-mm (1-inch) or less sample rate. • True profile.	• Lane closure. • Site preparation necessary. • Slow pace of measurement (approximately one site per day).
Inertial profiler	• A 25.4-mm (1-inch) or less sample rate. • Quick (multiple sites per day). • No lane closure required. • No site preparation.	High wavelengths filtered out.

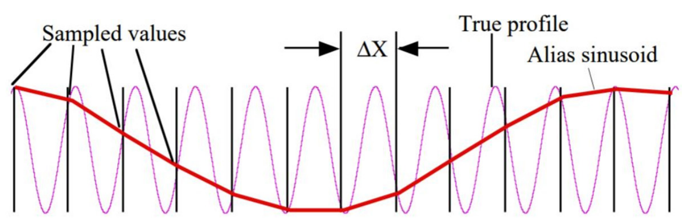
With technological advancements in profiler equipment, different positive and negative components have been discovered. One issue common to all measurement devices is the negative effect of texture. Textured surfaces, such as chip sealed pavements, can cause alias errors (Sayers and Karamihas 1998). Figure 8 shows an example of aliasing, where the sample interval is larger than the wavelength of the true profile. Aliasing shows the importance of measuring surfaces at the right interval length. Sayers and Karamihas (1998, p. 90) state that "in order to see a sinusoid, the sample interval must be half of the wavelength or smaller."

Other Profile Measurement Methods Used by Transportation Agencies

Besides the previously listed commonly used profile measurement methods, several State transportation agencies use agency-specific methods to assess the quality of surface profiles, specifically at bridge approaches. Lu et al. (2018) reported that the Florida Department of Transportation (FDOT) used the FDOT video log method to collect and analyze data of 1,155 Florida interstate highway bridges. The FDOT video log records images of roadways, which are then visually evaluated to determine bridge approach/departure asphalt pavement condition and performance.

Phares et al. (2011) reported results from profile measurements along nine in-service bridges in Ohio. Figure 9-A shows the monowheel cart used in this study. This cart must be driven across the bridge at a slow speed. The total station shown in figure 9-B remains stationary and records the distance, elevation, and azimuth information of the monowheel cart across the structure. This measurement approach, although slow, can give an accurate representation of the true surface profile for the bridge approach as well as the bridge deck.

Figure 8. Illustration. Aliasing sinusoid example (Sayers and Karamihas 1998).



ΔX = arbitrary distance in the horizontal direction greater than the wavelength.

Figure 9. Photos. Bridge global geometric evaluation system (Phares et al. 2011).

A. Monowheel cart and 360-degree prism.

B. Laser-guided total station.

Researchers have also focused on measuring the surface roughness of roads for nonmotorized traffic, such as pedestrian and bicycle lanes. Zang et al. (2018) developed a mobile phone application, RoadSR, to measure pedestrian and bicycle lane roughness. RoadSR is capable of tracking location through Global Positioning Systems and computing surface roughness through double integration of accelerometer data. IRI values from RoadSR showed a strong and positive correlation with values produced by other widely used profiling devices. However, any device relying primarily on accelerometer-based measurements will be significantly affected by the response of the measurement vehicle to the surface profile and will have the same limitations as an RTRRMS.

Data collected using inertial profilers are commonly filtered to extract information relevant to the ride quality experienced by road users. Filtering the collected profile data is an important task before analysis can begin. Figure 10 shows sample data collected from three different profiler devices before filtering was applied. Figure 11 shows the same set of sample data after applying filters.

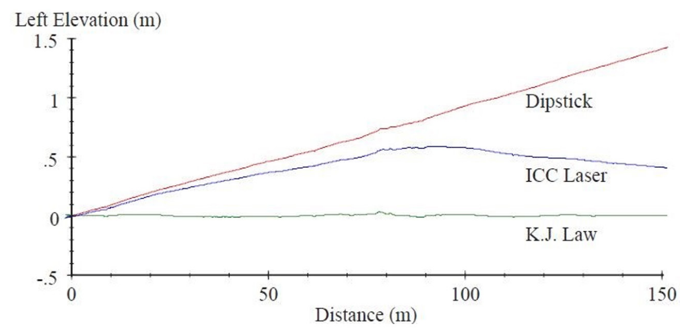
The changes in elevation are more clearly defined with the filters applied in figure 11. By applying the filters, "the grade and long undulations are removed mathematically" to allow a better interpretation of surface profile roughness (Sayers and Karamihas 1998, p. 7).

Surface profile data comprise a series of numbers that represent the elevations of the roadway at different points. It is often difficult to extract meaningful engineering inferences from such data. Different summary statistics are commonly established through reduction and analysis of surface profile data. These summary statistics can be classified under the generic name of a profile index. According to Sayers and Karamihas (1998, p. 43), "a profile index is a summary number calculated from the many numbers that make up a profile." This index is only valid if the data collected from the profiler are comparable to the true profile. Over time, several profile indices have been proposed by different transportation agencies, including IRI, present serviceability rating, root mean square vertical acceleration, mean panel rating, ride number, slope variance, profile index, and the bridge approach performance index (BI) (Sayers and Karamihas 1998; Das et al. 1999; McGhee 2002; Zhang and Hu 2007; Nam et al. 2016; Henderson et al. 2016; Lu et al. 2018). Among these, IRI is the most commonly used index by highway agencies to quantify the ride quality along roadways. The brief examination of IRI in the following subsection is important to assess how useful IRI can be in quantifying the surface profile at bridge approaches.

IRI

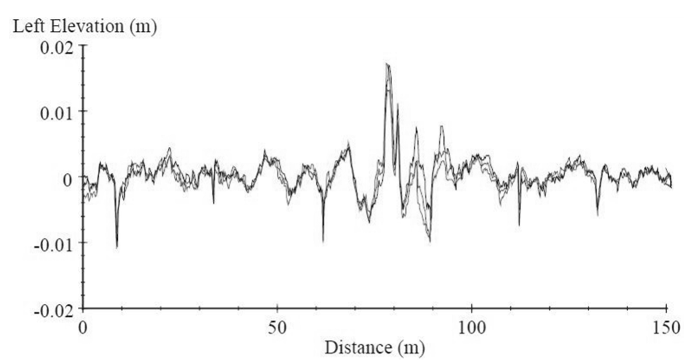
The concept of IRI was developed in the early 1980s under the scope of a study sponsored by the World Bank (Sayers and Karamihas 1998). IRI is a mathematical model of the amount of roughness measured along the longitudinal plane using a quarter-car model (Olmedo et al. 2015). The quarter-car model "is meant to be a theoretical representation of the response-type systems in use at the time the IRI was developed" and was intended to ensure a standard was available to compare future collected data (Sayers and Karamihas 1998, p. 48). An illustration of the quarter-car model can be seen in figure 12 (Sayers and Karamihas 1998).

Figure 10. Graph. Three raw profile measures (Sayers and Karamihas 1998).



1 m = 3.28 ft.
ICC Laser = laser profiler manufactured by International Cybernatics; K.J. Law = profiler manufactured by K.J. Law Engineers.

Figure 11. Graph. Three filtered profile measures (modified from Sayers and Karamihas 1998).



1 m = 3.28 ft.

Although different countries and State departments of transportation (DOTs) use different methods to determine road roughness, the results can be directly compared using IRI (Sayers and Karamihas 1998). Pavement profile data collected using inertial profilers can be analyzed to compute a corresponding IRI value. This analysis is usually performed using ProVAL, a profile viewing and analysis software developed by the Federal Highway Administration (FHWA) (Henderson et al. 2016; ProVAL 2015; Transtec Group 2015).

Standard procedures established by most State transportation agencies for reduction of profiler data require two profile readings, one along each wheel path. Regulation also suggests a minimum of five passes in each direction to ensure reliable information is captured (Perera and Elkins 2013). The profile readings from all passes are then interpreted specific to each longitudinal position (left wheel, center, or right wheel), and the values are averaged to determine the roughness for each section. IRI values are usually calculated for roadway segments at fixed length intervals, usually 160.9 m (528 ft) long. These values are subsequently combined to establish a representative value for the entire roadway section under evaluation. IRI values can be compared against pre-established thresholds to assess whether the ride quality along the section is acceptable. Roadways of different functional classifications and at different stages of their service lives can be categorized by typical representative IRI ranges (figure 13) (Sayers and Karamihas 1998; Olmedo et al. 2015).

Profile data collected during multiple runs of the profiler should be compared to assess the repeatability, reproducibility, and accuracy of the measurements. The cross-correlation method proposed by Perera and Kohn (2005) is one effective approach to compare data from multiple runs. Cross correlation is performed by taking two or more profile measurements and plotting them on the same chart to see similarities and differences in the recorded data. The method produces a rating (ranging from 0 to 1) of how similar the profiles are. The greater the rating, the greater the similarities between multiple runs. Different combinations of measurement components can be used to determine different ratings: "repeatability when it is applied to two measurements of the same profile by the same device, reproducibility when it is applied to two measurements of the same profile by different devices, and accuracy when a measurement from one of the devices is deemed to be correct" (Perera and Kohn 2005, p. 28).

Examples of cross-correlation analysis can be seen in figure 14. Figure 14-A shows a high correlation value between the three profiles (>0.995). The value is determined by comparing two of the three profiles in every possible combination and then finding the average of the cross-correlation values. The same process was used to find the cross-correlation value in figure 14-B where some areas have similar profile readings, but other sections show three distinct profiles. This explains why the average cross-correlation value was found to be only 0.84. During cross-correlation analysis, it is important to ensure that the profiles are properly

Figure 12. Illustration. Theoretical quarter-car model (Sayers and Karamihas 1998).

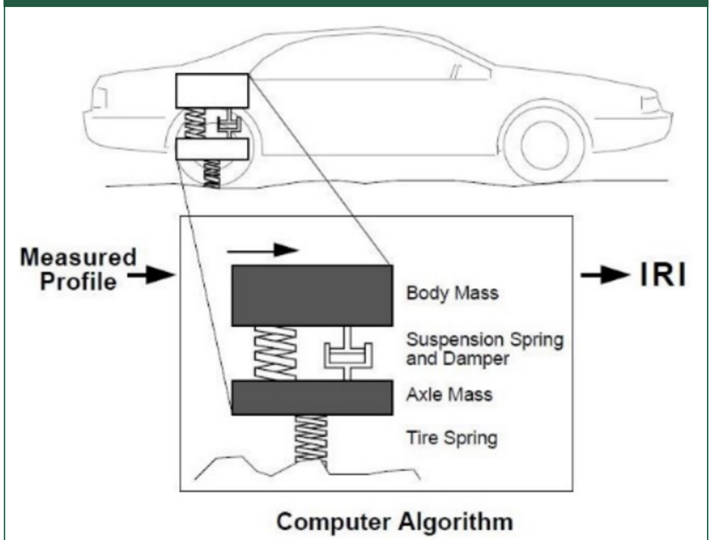
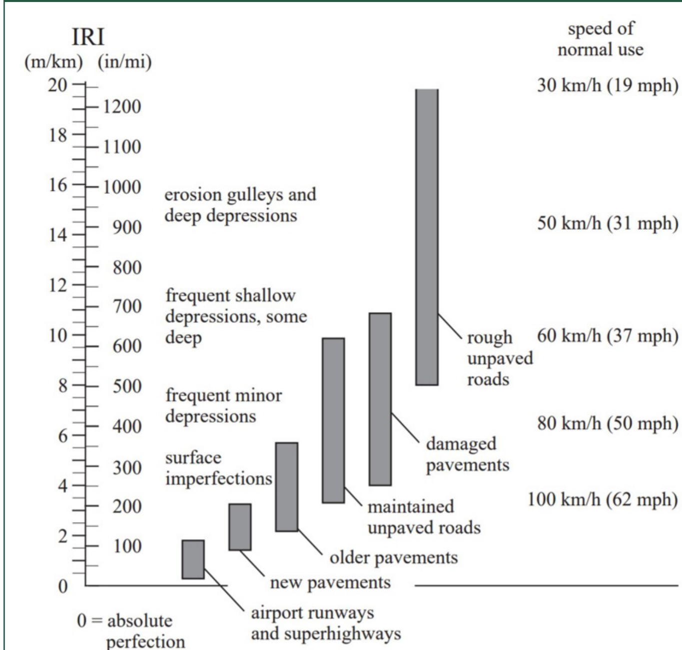
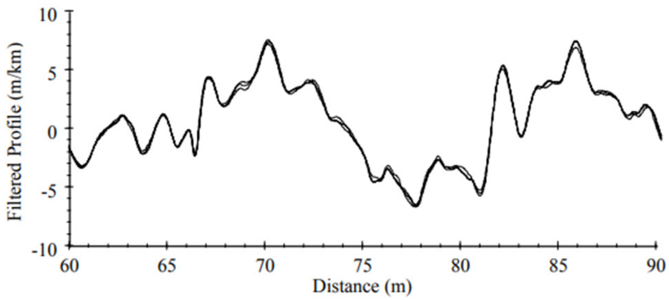


Figure 13. Chart. Road classification by IRI (Sayers and Karamihas 1998).



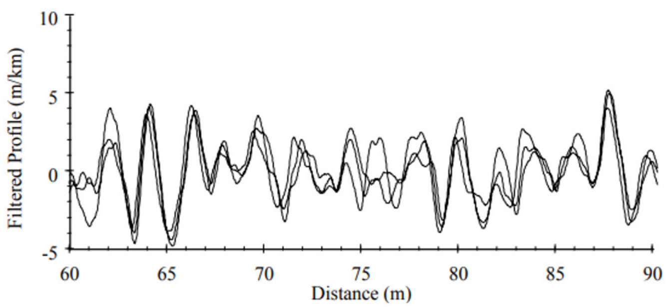
1 m = 3.28 ft; 1 km = 0.621 mi.

Figure 14. Graphs. Cross-correlation plots for three IRI filtered profiles (Karamihas 2004).



© 2004 Karamihas.
1 m = 3.28 ft; 1 m/km = 5.28 ft/mi.

A. Three IRI filtered profiles with an average correlation greater than 0.995.



© 2004 Karamihas.
1 m = 3.28 ft; 1 m/km = 5.28 ft/mi.

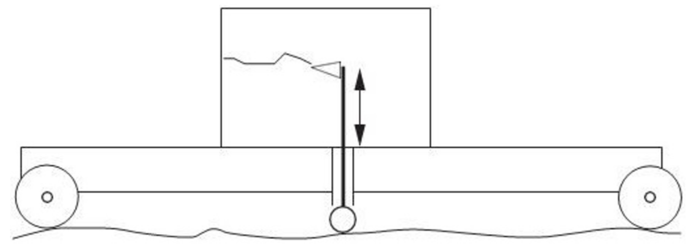
B. Three IRI filtered profiles with an average correlation of 0.84.

aligned at the same starting location. Even a small error in the starting position, for example, could result in false readings (Perera and Kohn 2005). Although there is no standard value required for cross-correlation ratings, it is important that the data being analyzed are correlated based on instrumentation and user precision. This is particularly critical when attempting to develop reliable profile quantification methods for bridge approaches. Multiple measurements should be carried out using profilers, and the cross correlation must be established before the ride quality at a particular bridge approach is quantified.

RSE Simulations

As mentioned in the IRI section, the IRI value is established by running a quarter-car simulation through the surface profile measured using inertial profilers. In addition to IRI, surface roughness along any roadway (particularly for bridge approaches) can also be determined using RSE simulations. Inertial profiler data can be used to simulate the RSE, as is done for IRI values. Figure 15 provides an illustration of the RSE apparatus.

Figure 15. Illustration. RSE device (Sayers and Karamihas 1998).



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The RSE measures the vertical deviation between the center of the straightedge and the profile for each interval in the profile data. The data collected from the RSE apparatus can be analyzed with software, such as ProVAL, to generate a representation of the elevation changes along the measured section (ProVAL 2015). Similarly, data collected using inertial profilers can be analyzed to simulate RSE runs along the roadway profile. During analysis, the RSE profile data require filter applications for the straightedge length and the deviation threshold to determine out-of-range locations. The plot created from RSE data collection shows the location of significant deviations in elevation; these indicate road roughness. Data from RSE measurements can be reduced using the equation in figure 16.

Figure 16. Equation. RSE output equation (Sayers and Karamihas 1998).

$$SE_i = \frac{1}{2}\left(Y_i - 2Y_{i+k} + Y_{i+2k}\right)$$

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Where:

SE_i = deviation of the profile from the straightedge at a particular point.

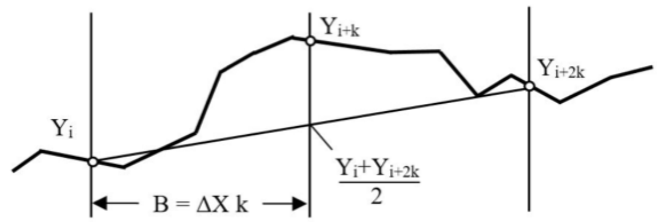
Y_i = *i*th profile elevation.

k = parameter for the filter.

The geometric model of the equation can be seen in figure 17.

Graphically modelled RSE data can indicate the general positions of different locations along the bridge, including the approach slab, bridge deck, and departure slab. A similar measurement can be taken from the quarter-car model used for IRI measurements, which is based on the acceleration collected by inertial profilers. This alternative measure is a single value called the root mean square (Sayers and Karamihas 1998).

Figure 17. Illustration. Geometric view of an RSE output (Sayers and Karamihas 1998).



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As stated in Sayers and Karamihas (1998, p. 9), "measuring a profile is half the job. The other half is running the profile through a computer program to get a roughness index." Various forms of roughness analysis can be performed using ProVAL to provide information regarding the roughness of the road and identify the magnitude or impact of BEBs (ProVAL 2015). A similar program, known as ProFAA, is used by the Federal Aviation Administration (FAA) to quantify the roughness along airfield pavements (FAA n.d.). Detailed analyses concerning the differences between the two programs are beyond the scope of this literature review.

Surface Profile Quantification at Bridge Approaches

To ensure uniform standards across transportation agencies in different parts of the country, threshold values, similar to those in figure 13, have been proposed by different researchers and practitioners to identify tolerable magnitudes of BEBs and road roughness. For pavements, FHWA requires annual IRI reports of roadways in the National Highway System. To establish acceptable thresholds, FHWA developed a rating system to categorize the condition of interstate highways. These ratings are listed in table 3.

These values are widely accepted as the standard to determine when maintenance is required for pavements. However, these values exclude measurements of bridge approaches and bridge decks. Das et al. (1999) proposed a rating system specific to bridge approaches using IRI measurements in a parametric study evaluating approach slabs in southeastern Louisiana (table 4).

Henderson et al. (2016) suggests that IRI values calculated for bridges immediately after construction should be lower than 1.263 m/km (80 inch/mi). Additionally, 2.68 m/km (170 inch/mi) would be the threshold between smooth and rough bridge locations. However, no justifications behind the selection of these particular values could be found during this literature review effort. Although IRI is the most common roughness index used by most State DOTs to determine BEBs and road roughness, other roughness metrics can also be used to compare the roughness at bridge approaches (Henderson et al. 2016). McGhee (2002) completed a field survey to identify characteristics that influence the ride quality of bridges and to compare IRI and RSE measurement methods. The analysis found that designed-in girder camber bridges contributed as much as 1.0 m/km (64 inch/mi) of roughness to IRI measurements. This discourages the use of IRI as the sole measurement to rank the ride quality on bridges. The reliability of the IRI measurement is uncertain because of the impact of measurement length on the collected data. This does not mean IRI measurements are incorrect, but factors of specific bridge lengths make them unreliable as the only form of measurement.

The use of RSE analyses of bridge structures provides an important model representation of elevation changes across road sections and bridge structures. This information indicates locations of BEBs and excessive roughness along the section; however, at this time, there is no standard threshold or index that transportation agencies apply to the data. Thresholds vary between agencies, making it difficult to apply RSE data to determine required maintenance (Olmedo et al. 2015; Nicks and Adams 2017).

The profiles of 38 bridge approaches and decks were measured in Iowa to examine differential settlement and BEB development (White et al. 2005). To assess the change from the original elevation, the BI was

Table 3. FHWA condition ratings and IRI ranges (modified from Olmedo et al. 2015).

Condition Rating	IRI Thresholds (m/km)
Very good	IRI < 0.95
Good	0.95 < IRI < 1.50
Fair	1.50 < IRI < 2.68
Poor	2.68 < IRI < 3.47
Very poor	IRI > 3.47

1 m = 3.28 ft; 1 km = 0.621 mi.

Table 4. IRI ratings for bridge approaches (modified from Das et al. 1999).

Condition Rating	IRI Thresholds (m/km)
Very good	IRI < 3.9
Good	4.0 < IRI < 7.9
Fair	8.0 < IRI < 9.9
Poor	10.0 < IRI < 11.9
Very poor	IRI > 12.0

1 m = 3.28 ft; 1 km = 0.621 mi.

developed to quantify the condition of approach slabs. “The BI is calculated as the area between the original profile and the existing profile of the approach slab divided by its length. The area is determined by subtracting the integration of the original profile...and the existing profile over the length of the approach slab” (White et al. 2005, p. 220). It is assumed that the original profile is a straight line from the bridge deck to the road; however, this is not always the case depending on the construction. Figure 18 shows a plot of one of the bridges from the study with the assumed original profile and measured profile traces clearly annotated.

The shaded area between the curves represents the BI values of the approach and departure slabs. By combining BI and IRI measurements, White et al. (2005) developed a rating system to categorize bridge approach performance. The rating system, shown in figure 19, includes acceptable IRI values much greater than those produced in more recent publications. Henderson et al. (2016), for example, proposed a value of 2.68 m/km (170 inches/mi) as a good IRI rating.

A similar rating system was proposed by Lu et al. (2018) using IRI and rut depth values measured at the bridge approach. Pavement rut depths were measured along the approaches of several bridges in Florida and were correlated with IRI values. A threshold value of 12.7 mm (0.5 inch) for the average rut depth is widely used as a pavement failure indicator.

Figure 20 shows the proposed rut depth rating system, which categorizes the bridge approach and departure. Additional rating systems were proposed to rate the severity of rutting and cracking in bridge pavements (Lu et al. 2018). These rating systems can be applied to the data collected at bridge sites to determine when maintenance is necessary. Depending on the determined cause, several possible mitigation techniques can be used to improve the structure and prevent future BEBs and roughness occurrences.

Figure 18. Graph. BI plot (White et al. 2005).

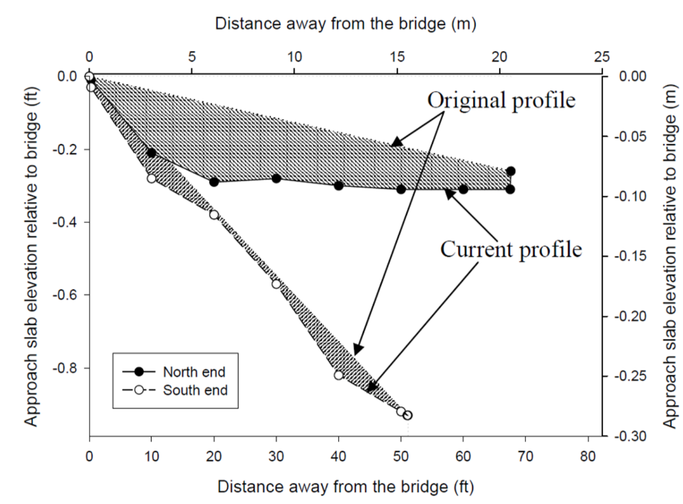
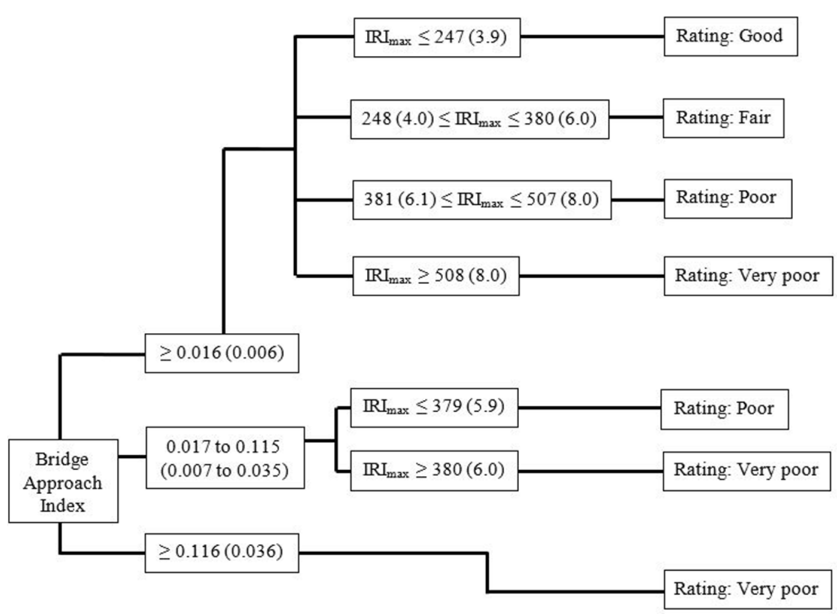
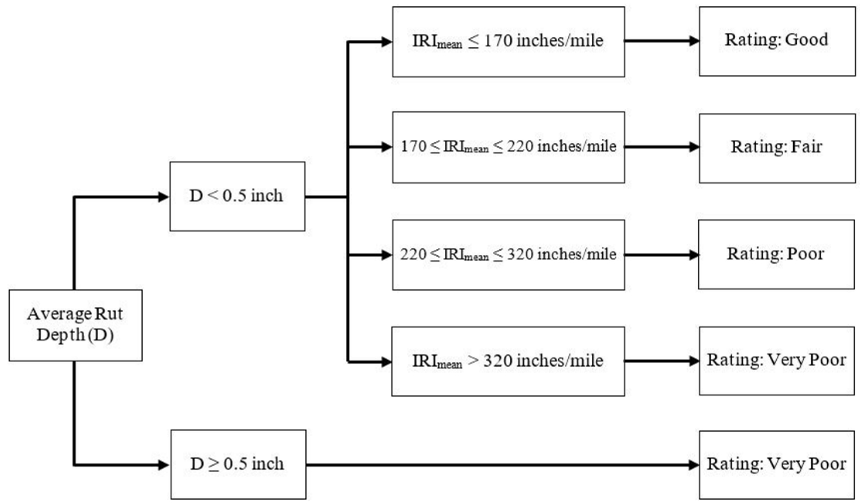


Figure 19. Flowchart. Bridge approach performance rating system developed in Iowa (White et al. 2005).



Note: BI measurements are shown in feet followed by the equivalent measurement in meters in parentheses. IRI measurements are shown in inches per mile followed by the metric equivalent in meters per kilometer in parentheses.

Figure 20. Flowchart. Rut depth rating system for bridge approach/departure asphalt pavements (Lu et al. 2018).



1 inch = 25.4 mm; 1 mi = 1.61 km.

Nicks and Adams (2017) considered the findings from a pilot study to quantify the BEB at eight bridge approaches in New York State and compared the bump magnitudes for bridges constructed over deep and shallow foundations. They concluded that inertial profilers can serve as a good tool for bridge approach evaluation as most State DOTs already own this equipment. They also observed that total surface deviation (TSD) values calculated from RSE simulations on inertial profiler data yielded more information about the bridge approach profile compared to IRI values.

COMMONLY USED MITIGATION TECHNIQUES

The scope of the current research project involves analyzing pavement profile data collected at 66 different bridge approaches to evaluate what site-specific conditions may significantly affect the BEB problem. That effort required a thorough understanding of different pavement profile measurement approaches and indices available to quantify pavement profile roughness, detailed in this TechNote. It is therefore important to identify different construction and rehabilitation approaches commonly used by highway agencies to mitigate the BEB problem. The following paragraphs briefly discuss several of these mitigation techniques. This information will be particularly helpful later in the study during statistical analyses of different contributing factors on the extent of BEBs. Detailed evaluations on different mitigation techniques can be found elsewhere (Puppala, Chittoori, and Saride 2014).

The BEB problem affects an estimated 25 percent of the bridges in the United States (Briaud, James, and Hoffman 1997; Phares et al. 2011). According to Lu et al. (2018, p. vi), “most states do not have special maintenance and rehabilitation criteria and guidelines for bridge approach/departure asphalt pavements.” Resolution for the BEB issue has primarily been performed in two phases: repairing structures that are already in use and finding a way to prevent BEB formation from happening in future bridge structures.

The main causes of BEB formation are related to structural and material inadequacy; thus, the usual repair method is just a temporary fix. According to a survey compiled by Briaud, James, and Hoffman (1997), the most common methods to repair bridge approaches experiencing the BEB problem in decreasing order are as follows:

1. Leveling with asphalt cement concrete.
2. Mudjacking.
3. Removing and replacing the approach slab.
4. Improving drainage.
5. Retrofitting the approach slab.
6. Changing joint(s).
7. Improving the backfill.
8. Improving the natural soil.

Some methods are used in conjunction with others to try to extend the service life of the bridge before reconstruction is necessary. One common tactic is to use an approach slab: “the use of approach slabs is relatively recent; therefore, older bridges tend to not have them…old bridges are sometimes retrofitted with approach slabs" (Briaud, James, and Hoffman 1997, p. 26). Approach slabs were installed at several sites to remove the BEB; however, the BEB has sometimes appeared at the interface of the approach slab and the approach pavement (essentially shifting it from the bridge–pavement interface to the approach slab–pavement interface) (Briaud, James, and Hoffman 1997).

Several State transportation agencies have recommended altering current bridge designs to improve surface roughness quality. Briaud, James, and Hoffman (1997) categorized proposed modifications as those concerning foundation soil, backfill material, bridge foundation, approach slab, or drainage. Design alternatives proposed by different agencies include improvement of the foundation soil; use of well-graded backfill material; use of abutments supported on shallow foundations; use of elastic, collapsible inclusion, or expandable material behind the abutments; installation of more effective drainage systems; use of filter wrap to prevent soil erosion; construction of approach slabs with a vertical angle (precambering); and reinforcement of the backfill material using geosynthetics. The following subsections present a basic review of some of these remedial measures.

Improvement of Foundation Soil

To ensure successful performance at bridge approach locations, analysis of the natural soil is important. Understanding the properties of the natural soil will help in the design phase to anticipate soil settlements (Wahls 1990). Several different design alterations can be implemented; one option involves reducing the embankment and abutment loads to transfer the loads to stronger soil layers. Improvements can also be made to the foundation soil by replacing the natural soil with more suitable materials. Densification via consolidation, dynamic compaction and compaction piles, and reinforcement are some of the other alternatives (Short et al. 2018).

Use of Well-Graded Backfill Materials

An ideal backfill material should have the following characteristics: easily compacted, erosion resistant, elastic, and having no time-dependent properties. Many State DOTs recommend compacting the backfill material to 95–100 percent of the maximum dry density established in the laboratory using the standard compactive effort (Wahls 1990). It is important to note that compaction of the backfill material immediately adjacent to the bridge abutment can be challenging because of space constraints (White et al. 2005). To accommodate the resulting inconsistency in compaction, well-graded backfill materials are often recommended. Material with less than 5 percent by mass that is finer than 0.075 mm (0.003 inch) (passing the No. 200 sieve) has been recommended by several studies, including Wahls (1990) and Briaud et al. (2002). Transportation agencies, however, specify typical aggregates that exceed this limit for percent fines, primarily because of availability.

Use of Abutments—Shallow Foundations and Alternative Materials

The success of abutments in bridges has been a debated topic in several publications found in the literature. There are five abutment designs that are used based on the applied loads. Conventional abutments are designed with expansion joints to allow lateral movement during thermal cycling (figure 21).

Abutment designs that are considered conventional are closed (i.e., high) abutments, stub (i.e., perched) abutments, and pedestal (i.e., spill-through) abutments. Two other abutment design types are integral abutments and mechanically stabilized abutments. Integral abutments (figure 22) are similar to pedestal abutment designs, with the elimination of the expansion joint. This design alteration was attempted "to fully transfer the stress caused by thermal effect to the abutment" (Seo, Ha, and Briaud 2002, p. 27). The use of the integrated abutment is among the more popular designs found in reviewed literature, but problems arose from the use of all abutment designs, including the integral abutment (Wahls 1990; Seo, Ha, and Briaud 2002).

A recommended improvement suggested by Wahls (1990, p. 23) was "the development of new compressible elastic materials that could be installed easily between the abutment and the backfill." This improvement was used by the Wyoming and South Dakota DOTs by incorporating a small gap between the abutment and the backfill. Geosynthetic-reinforced fill was placed to form "an air gap between the abutment and retained fill" (Abu Hejleh et al. 2000, p. 6). Lateral movement of the abutment, caused by thermal cycling, occurred without impacting the backfill soil. To maintain the space between the abutment and the backfill, panels made from collapsible cardboard or compressible expanded polystyrene are placed in the air gap.

Installation of Effective Drainage Systems

Effective drainage is a key component of well-designed bridge structures. Improvements to drainage systems are consistently encouraged in literature related to BEBs and road roughness mitigation. To ensure effective drainage

Figure 21. Illustration. Components of a conventional bridge approach (Mishra 2006).

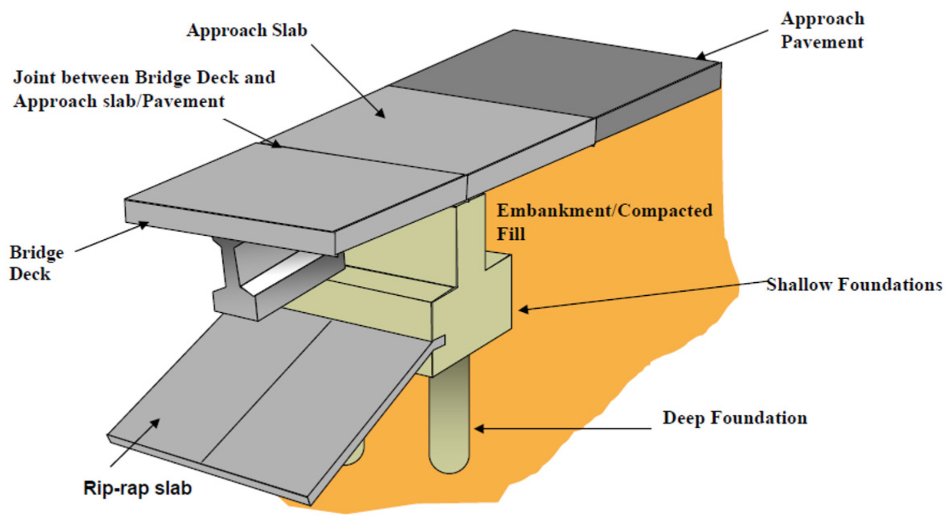
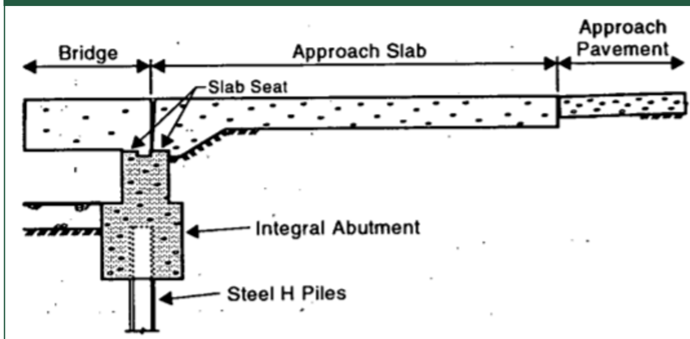


Figure 22. Illustration. Typical integral abutment (Briaud, James, and Hoffman 1997).



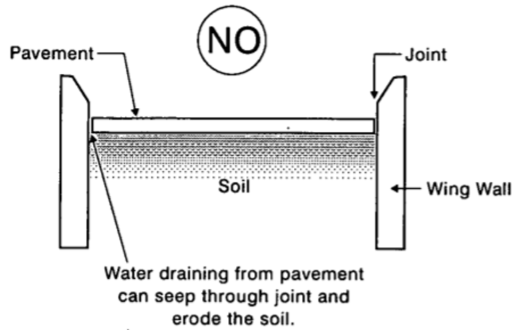
on bridges, both surface and subsurface drainage must be addressed (Briaud, James, and Hoffman 1997; Short et al. 2018). To avoid erosion, multiple designs are used to direct surface runoff away from the bridge approach and joints. Recommended drainage systems include wingwall structures, gutters and ditches, geosynthetic reinforced backfill, geocomposite drains, porous backfill around perforated drain pipes, and geotextiles wrapped around porous backfill (Wahls 1990; Briaud, James, and Hoffman 1997; White et al. 2005; Short et al. 2018). According to White et al. (2005), the three main variations used by State DOTs include applying porous backfill designs, wrapping a geotextile around the porous fill, and using a geocomposite drainage system. Structural techniques, as shown in figure 23, are helpful to prevent runoff from infiltrating the approach and departure embankments. This structural design is considered a wingwall assembly.

Reinforcement of Backfill Material Using Geosynthetics

The use of reinforcements in bridge soils is important because compaction is inconsistent during construction. Techniques for structural reinforcement proposed by Briaud, James, and Hoffman (1997); White et al. (2005); and Short et al. (2018) include stone columns, deep soil mixing, lime columns, and embankment piles. Each technique is used in different natural and backfill materials depending on soil properties. Two primary modern methods of reinforcing soil include mechanically stabilized earth (MSE) and geosynthetic-reinforced soil (GRS). MSE has also been categorized in two different groups: "proprietary structures built with metallic (inextensible) reinforcements and proprietary structures built with geosynthetic (extensible)reinforcements" (Adams et al. 2011, p. 3). Wahls (1990) examined four major types of geosynthetics along with their functions, as seen in table 5.

These examples of geosynthetics refer to multiple aspects of bridge designs that can utilize the benefits of the reinforced material. The Geosynthetic Reinforced Soil–Integrated Bridge System (GRS-IBS) is a bridge design gaining popularity among DOTs since its original use by the U.S. Forest Service in the 1970s for logging roads. This design method incorporates three main components: reinforced soil foundation, abutments, and integrated approaches (Adams et al. 2011). An example illustration of this design can be seen in figure 24. IBS utilizes aspects of the integral abutment design; however, there are additional design factors for GRS-IBS structures compared to conventional bridge designs

Figure 23. Illustrations. Cross section showing a wingwall assembly (Briaud, James, and Hoffman 1997).



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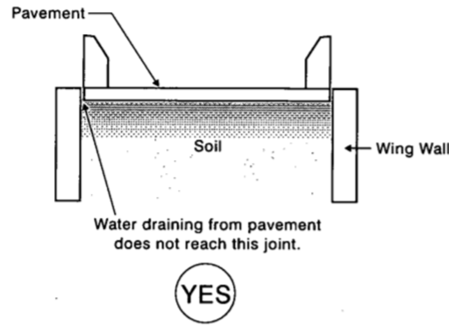
A. Improper wingwall assembly.



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B. Recommended wingwall assembly.

Table 5. Types and functions of geosynthetics (Wahls 1990).

Type	Function
Geogrids	Reinforcement
Geotextiles	• Reinforcement • Separation • Filtration • Drainage
Geocomposites, Geonets	Drainage
Geomembranes	Isolation

using integral approaches. GRS is often used for both bridge foundations and approaches (Nicks and Adams 2017).

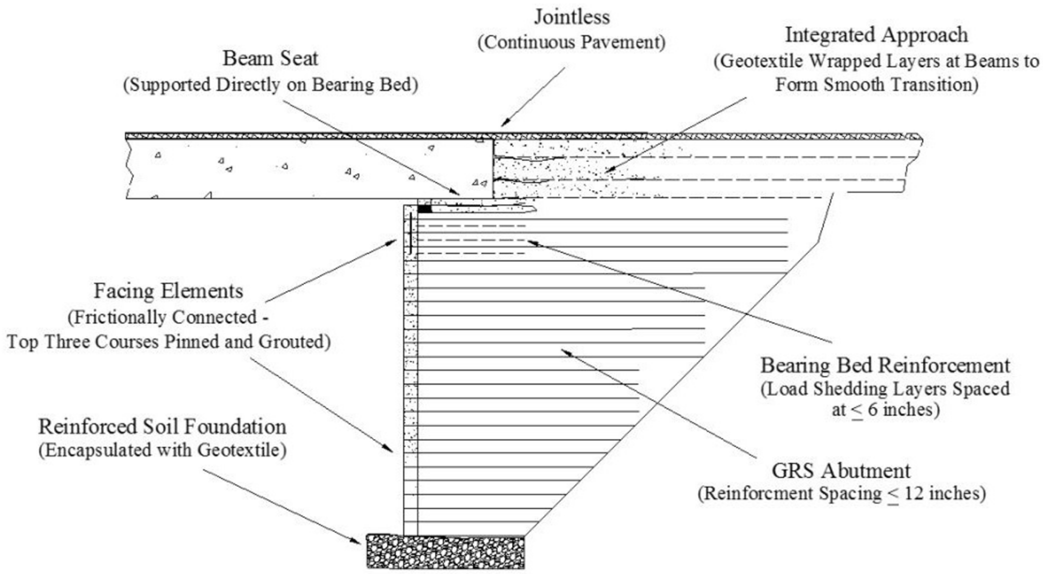
Helwany, Wu, and Kitsabunnarat (2007) defined four parameters important in analyzing the performance of GRS designs: (1) vertical displacement at the abutment seat, (2) horizontal displacement at the abutment seat, (3) maximum displacement of the segmental facing, and (4) sill distortion. These performance parameters can be measured with the use of surface profiling, along with some additional analysis of the foundation performance. Adams, Schlatter, and Stabile (2008) and Adams et al. (2011) indicated two basic rules to ensure GRS designs are successful: good compaction and closely spaced reinforcement. The use of these rules ensures adequate internal stability of GRS structures.

GRS-IBS

Many State DOTs use GRS to alleviate the BEB issue and improve ride quality at bridge approaches. The Wyoming DOT has been using geosynthetic materials in embankment structures regularly since the 1980s with great success (Nicks and Adams 2017). The Colorado DOT used GRS as bridge and approach support in the retaining walls of the new Founders/Meadows Bridge near Denver, CO. The structure was completed and opened to traffic in summer 1999 (Abu-Hejleh et al. 2000). When the Founders/Meadows Bridge was evaluated 18 mo later, minimal settlement was found. Lee and Wu (2004, p. 191) observed: "the GRS bridge abutment shows no sign of the 'bridge bump' problem… [and] has exhibited excellent short- and long-term performance characteristics." The Founders/Meadows Bridge project was completed as an experiment, utilizing a pilot instrumentation program to analyze the performance of in situ GRS-IBS structures. This is just one of many examples of successful use of GRS-IBS structures. Additional cases of the use of GRS designs are discussed by Lee and Wu (2004).

With the increasing demand for improvements on the Nation's bridge infrastructures, time and money are key factors to successful delivery. Two primary benefits of the GRS-IBS design are decreased cost and construction time, making it a "sound economical alternative to current bridge design" (Adams, Schlatter, and Stabile 2008, p. 1). The use of GRS-IBS designs has also reduced the required length for a bridge structure, as seen for the Bowman Road Bridge in Defiance County, OH. The use of GRS abutments reduced the beam length from 40.8 m (133.86 ft) down to 25 m (82.02 ft). The Bowman Road Bridge design, which used GRS abutments, cost 25 percent less than conventional designs. This was both because the abutments are less expensive than conventional designs and the bridge length was shorter (Adams et al. 2011). The Bowman Road Bridge was also built in a very short period of time compared to conventional designs; the process took approximately 6 weeks. It was the first production bridge to be built with GRS design. The process has since been reduced to approximately 2 weeks. This is a significant improvement compared to the several months required to build conventional bridges. Additionally, factors like unexpected weather or site conditions are less impactful on the construction of GRS-IBS structures in the field; the

Figure 24. Illustration. Section view of GRS-IBS (Adams and Nicks 2018).



Source: FWHA.
1 inch = 25.4 mm.

design can be modified in the field as necessary (NCMA 2016). Cost savings associated with GRS-IBS can also be close to 30 percent depending on bridge length and abutment height (Adams, Schlatter, and Stabile 2008). Besides cost savings, the GRS IBS construction alternative can also lead to significant performance-related benefits. From the time the Bowman Road Bridge was open to traffic in 2005 until their analysis in 2011, Adams et al. (2011) reported no indications of cracking in the approach caused by settlement. This was hypothesized to be an indication of uniform settlement between the approach slab, abutment, and bridge deck. The design of the GRS-IBS combines these factors as if they were a single structure, allowing all components to settle together and avoid differential settlement between systems (NCMA 2016).

SUMMARY

This document summarized findings from an extensive review of published literature related to BEBs and different methods to quantify the surface roughness at bridge approaches. Initially, different factors contributing to the development of BEBs were listed. Background discussion was provided on the concept of surface profile measurement and methods to quantify roughness of a particular surface profile. Subsequently, different profile measurement methods were discussed, and different methods proposed by researchers to quantify the roughness at bridge approaches were also presented. IRI and TSD values calculated using RSE simulations on inertial profiler data were identified as primary indices to quantify pavement roughness at bridge approaches; however, both of these indices have some limitations, requiring multiple methods to be used during roadway profile analysis. Finally, different remedial measures adopted in practice to mitigate the BEBs problem were listed. While the performance of the GRS-IBS design has been found to improve the ride quality at bridge approaches, most bridges constructed using this design are still in the early stages of their lifecycle; long-term effects of this design alternative on the BEB problem are yet to be established.

The current research study involves extensive analysis of pavement profile data collected at 66 different bridge locations to assess the effects of different bridge features on approach/departure roughness. Differences in the surface profiles of conventional and GRS-IBS bridges will be considered when analyzing this dataset. The research team will also continue exploring alternative methods to quantify the surface profile roughness at bridge approaches. Identification and implementation of improved methods to quantify BEBs will significantly benefit bridge monitoring and management programs adopted by State and local highway agencies.

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