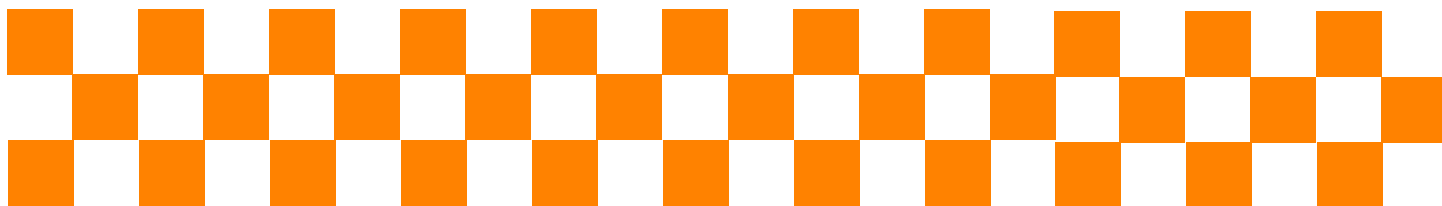
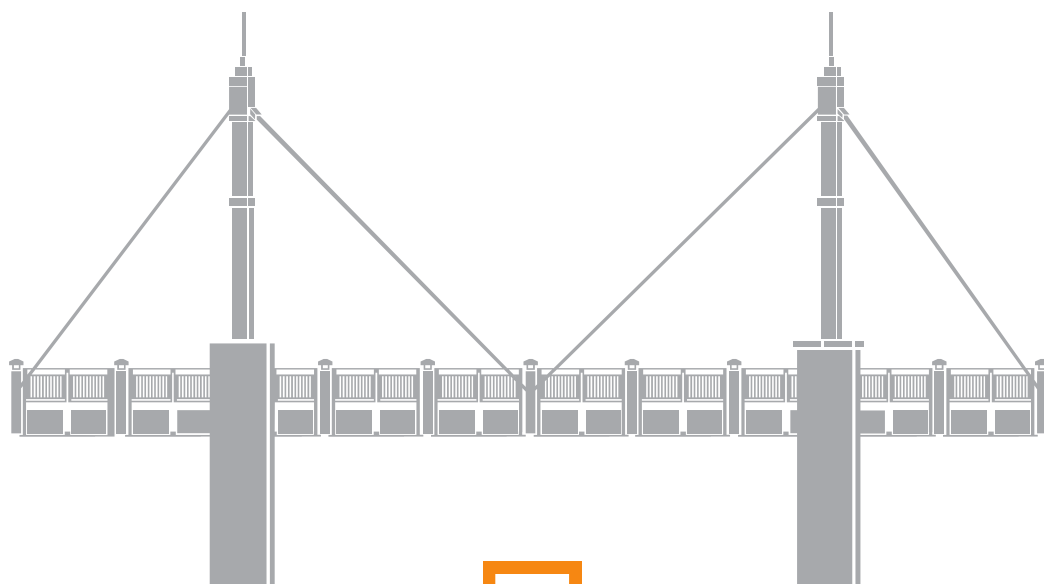


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Bump at the End of the Bridge: Review and Analysis of Rider Discomfort

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DEPARTMENT OF CIVIL &
ENVIRONMENTAL ENGINEERING

The University of Tennessee is a Public Land Grant University that was founded in 1794 as Blount College. Instruction in civil engineering began at the University of Tennessee in 1834, then East Tennessee College, after Joseph Estabrook became president. In 1993 the department's name changed to Civil and Environmental Engineering to reflect the significance of Environmental Engineering in the department's activities. In 2013, the department moved into the newly constructed John D. Tickle Engineering Building. The bridge connecting the John D. Tickle Engineering Building with Estabrook Road is featured on the cover page of this report series and serves as a symbol for the department.

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Abstract

Localized irregularities in the road profile are a well-known and persistent cause of rider discomfort when entering and exiting many bridges. This work addresses this so called “bump at the end of the bridge” problem first, through a review of relevant literature focusing on causes of the bump problem, mitigation techniques, retrofitting techniques, and special bump problems related to integral abutment bridges. Then, recognizing that approach slabs play a crucial role in the development of the bump, this problem is addressed through an investigation and comparison of approach slab designs and details utilized by various states. And, finally, the “bump at the end of the bridge” problem is addressed through dynamic analyses to ascertain the impact that various parameters of the bump geometry, road conditions, and vehicle speed have on rider discomfort. The results of the dynamic analyses indicate that the slope of the approach slab (i.e., the bump) and vehicle speed have the biggest impact on rider discomfort. Recommendations for future research are also noted.

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Chapter 1: Introduction

Among the most common complaints that the Tennessee Department of Transportation receives is of rider discomfort at the interface between their bridges and roadways. The evident cause of the discomfort, the so called “bump at the end of the bridge”, has been the subject of research and investigation across the United States since at least the 1960s.

This report documents a literature review of the previous studies as well as a comparison of design approaches and a numerical investigation into rider discomfort. The literature presented in the second chapter of this report provides a general overview of identified causes of the bump problem, mitigation and retrofitting techniques, and unique aspects relevant to integral abutment bridges, a common form for highway bridges that are constructed in Tennessee. The third chapter investigates and compares standard approach slab designs from several state transportation departments; methods for surface drainage are also compared. The fourth chapter presents a numerical evaluation of the bump at the end of the bridge in terms of rider discomfort using a dynamic car model. The fifth chapter summarizes the work completed and identifies potential future research areas.

Chapter 2: Literature Review

Several states have conducted or sponsored studies to investigate the bump at the end of the bridge problem. This chapter provides an overview of the work that has been done previously on this topic, as well as any recommendations made to address the issue. Specifically, information related to causes of the bump problem, mitigation and retrofitting techniques will be included. Also included is literature regarding integral abutment bridges and their relation to the bump problem.

Causes of the Bump Problem

The general consensus in the literature is that the bump problem is caused by the differential settlement between the approach slab and the bridge deck or abutment (Ha et al. 2002), (Cai et al. 2005), (Abu-Farsakh and Chen 2014), (Ng et al. 2014). A physical illustration of this process can be seen in Figure 1.

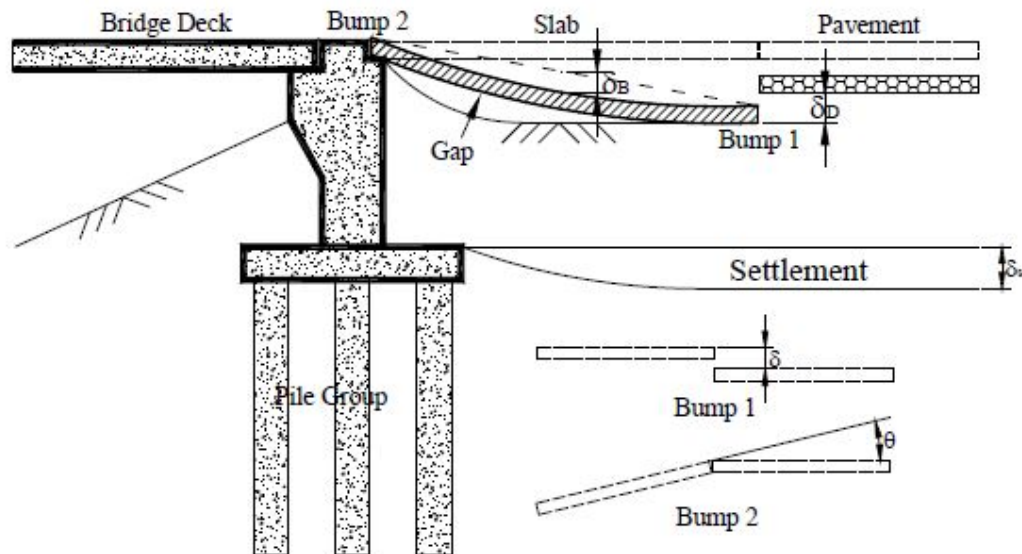


Figure 1. Illustration of bump mechanics (Abu-Farsakh and Chen 2014).

In their paper, (Cai et al. 2005) discuss how the approach slab can appear as a change in slope of grade at the approach slab-bridge deck interface (labeled as Bump 2 in Figure 1), or a

vertical fault at the approach slab-roadway pavement interface (labeled Bump 1 in Figure 1). Some potential causes of this differential settlement include: settlement of the natural soil, poor construction practices, compression of the fill material, poor fill material, high traffic loads, erosion, poor joints, and temperature cycles (Ha et al. 2002).

Mitigation Techniques

Quality of fill material. Poorly graded material is far more susceptible to compression. Some suggest minimum grading requirements include: PI (plasticity index) less than 15, less than 20% of material passing #200 sieve, and a coefficient of uniformity larger than 3 (Ha et al. 2002).

Another suggestion is to use granular or non-cohesive soils over cohesive soils (Puppala et al. 2011). Cohesive soils are more susceptible to lateral movement. Furthermore, non-cohesive soils have time-dependent settlement properties, unlike cohesive soils which typically settle immediately. The use of a granular fill will result in the differential settlement being reduced.

Compaction of fill material. Proper compaction of the fill material can reduce the severity of soil settlement. However, compaction of the material behind the abutment and beneath the approach slab is difficult (Ha et al. 2002). One suggestion made in the literature is to construct abutments with no notches or overhangs so that the fill material becomes easier to compact (Luna 2004). Another suggestion made by (Luna 2004) is that the construction of the approach slab be delayed as much as possible in order to allow the soil more time to settle.

Use of predictive techniques. Several pieces of literature indicate that predictive techniques can be useful tools in determining how to approach the fill material. (Luna 2004) stated that soil mechanics can be used to predict when problems may occur. (Cai et al. 2005) suggest that finite element programs can be used to design approach slabs for expected embankment settlements. In fact, another report recommends that all embankment design include a complete

settlement analysis (Miller et al. 2013). (Abu-Farsakh and Chen 2014) conclude that the piezocone penetrometer test provides a more accurate evaluation of the strength and consolidation properties of the soils. These properties can be used to estimate the embankment settlement, which can then be used by geotechnical engineers to determine the amount of preloading required (Abu-Farsakh and Chen 2014).

Use of geosynthetic material. Another mitigation technique suggested in the literature is the use of geosynthetic reinforced backfill. Using geosynthetic reinforcement would increase the bearing capacity and reduce the settlement of the soil (Abu-Farsakh and Chen 2014). Geotextiles have regularly been used in the state of Wyoming since the 1980's, with good results (Ng et al. 2014). (Luna 2004) concludes that geosynthetic reinforced backfill should be used for embankments greater than 10 feet in height.

A few reports suggested combining the geosynthetic reinforcement with a polyethylene sheet or filter fabric underneath the approach slab. The polyethylene sheet helps minimize the friction of the approach slab against horizontal movement (Mistry 2005).

Other techniques for foundation soils. There are a few other methods used to improve the foundation soil. One such method is the complete removal and replacement of the weak soil (Helwany et al. 2007). The removal and replacement method is likely the simplest method and can be cost-effective depending on the depth of the excavation. The proximity of the replacement soil is a key factor in deciding whether removal and replacement is the best solution. Foundation soils can also be improved through chemical means, such as deep soil mixing or grout or lime stabilization (Helwany et al. 2007).

Surcharging is another technique used to improve foundation soils; however, this method requires a lengthy construction period (Helwany et al. 2007). The final method is the use of deep

foundations. While this method performs well, it is usually the most expensive of the ground improvement options (Helwany et al. 2007).

Approach slab stiffness. The Louisiana Transportation Research Center conducted a study where they increased the flexural rigidity of the approach slab by increasing its thickness and reinforcement. They found that the approach slab with the increased stiffness performed better and reduced the severity of the bump (Abu-Farsakh and Chen 2014). A key objective would be to have more compatible stiffness between the bridge structure and the embankment (Luna 2004). Too many changes in vertical stiffness across the length of the bridge system can lead to the development of a bump (Phares et al. 2011). (Abu-Farsakh and Chen 2014) included geosynthetic reinforcement to help with the stiffness issue in addition to increasing the bearing capacity and reducing the settlement of the soil.

Drainage improvement and erosion mitigation. Poorly designed, constructed and maintained drainage systems can lead to several issues. Drainage problems can be found in both the surface and subsurface drainage systems (Miller et al. 2011). Some results of poor drainage include: surface erosion, slope stability features, increases in hydrostatic pressures, and pumping of fines (Helwany et al. 2007). Improper surface drainage can lead to water entering the joints, eventually reaching the backfill (Puppala et al. 2011). One potential result of poor drainage is pumping, where water and fine particles seep up through pavement joints and cracks. Piping, another concern related to poor drainage, is where water creates a drag force that carries small particles through the soil (Miller et al. 2011). Both of these mechanisms create voids in the soil, allowing for settlement of the soil and the approach slab.

Erosion of the side slopes can lead to soil from the embankment and backfill to fill in, creating a void underneath the approach slab (Helwany et al. 2007). Based on field observations,

another study found that the most common path of erosion is from the approach slab to underneath the abutment (Miller et al. 2013). This concept is illustrated in Figure 2. This erosion path can lead to the piles being exposed as well as an increase in the potential of backfill erosion.

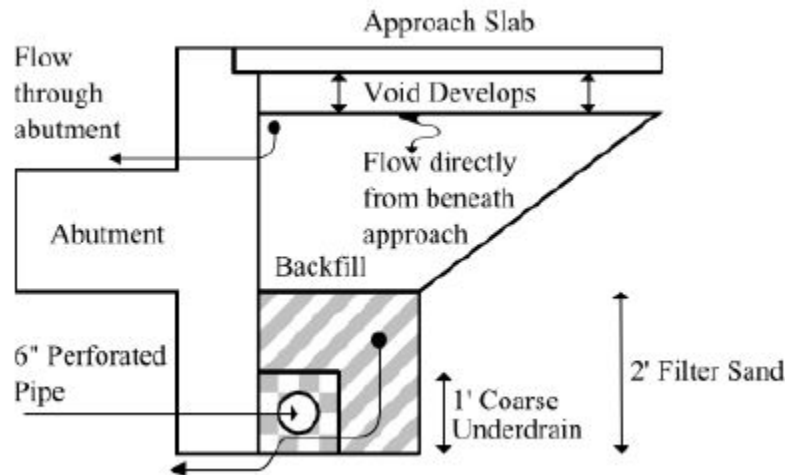


Figure 2. Erosion flow path (Miller et al. 2013).

(Helwany et al. 2007) recommended not allowing the means of drainage to stop short within or at the top of the slope. Their report also mentions that numerous states have experienced success with the use of geosynthetic reinforced backfill as an improvement for settlement and drainage. (Miller et al. 2013) suggest using a water stop to help mitigate erosion under the abutment. (Puppala et al. 2011) suggest consistent maintenance of joints and drainage systems be carried out in order to ensure proper functionality.

Retrofitting Techniques

Eventually, some bridges may reach the point where the bump problem becomes severe enough that driver safety becomes a concern; though no specific criteria for this could be found. At this point the bump may also be a structural concern, as vehicles moving over the bump (even at lower speeds) create a dynamic loading situation on the bridge (Luna 2004). In integral abutment

bridges, this dynamic loading can cause cracking in the abutments (Arsoy et al. 1999). Either way, it is at this point that a part of the bridge system will need to be retrofitted.

Lifting and realigning. Lifting and realigning, also known as mudjacking, is a retrofitting method that fills in voids and raises the approach slab back to its original elevation (Luna 2004). This procedure requires holes to be drilled in the approach slab (and possibly the sleeper slab). The materials used in this method can range from sand and clay mixtures to special foams. One limitation is that this technique is not useful in cases where large voids have developed (Ng et al. 2014). Other issues include the possibility of drainage systems becoming clogged and difficulty in placing the material (Puppala et al. 2011). Though some states have used the lifting and realigning method with success, it is still one of the lesser commonly used retrofitting techniques.

Overlaying. Overlaying is one of the faster retrofitting techniques (Helwany et al. 2007). Overlaying is simply paving over and smoothing out the bump. This method is also usually the least expensive of the retrofitting technique options. However, overlaying the approach slab with new pavement adds to its total weight, thus increasing the stress on the backfill (Ng et al. 2014). Even so, this is the most commonly used retrofitting technique.

Replacement. The final retrofitting technique is to simply replace the approach slab. This method is performed when the quality of the approach slab has deteriorated beyond the point where overlaying or jacking are effective options for retrofitting (Helwany et al. 2007). Replacement of the approach slab can be costly and requires far more time than overlaying or jacking the slab. Still, replacement is the second most common retrofitting technique after overlaying (Ng et al. 2014).

Integral Abutment Bridges

An integral abutment is a type of abutment that is rigidly connected to a bridge deck and does not require the use of expansion joints. This rigid connection allows the abutment and the deck to move as one. The abutment goes through both displacement and rotation. Integral abutment bridges have several advantages including: lower construction and maintenance costs, improved seismic response, and a fewer number of piles required (Arsoy et al. 1999). Due to their increasing popularity (Greimann et al. 2008), bump problems specifically related to integral abutments have been identified and are discussed in the subsequent paragraph.

According to (Phares et al. 2013) and as cited by (Briaud et al. 1997); despite these advantages, the bump at the end of the bridge is still a consistent problem with integral abutment bridges. Temperature changes, both daily and throughout the year, cause a cyclic thermal loading on the bridge structure (Huffaker 2013). Due to the thermally induced movements of expansion and contraction, a void can develop behind the abutment underneath the approach slab due to soil settlement. The void typically develops within a year of construction (Greimann et al. 2008). These mechanisms are illustrated in Figure 3.

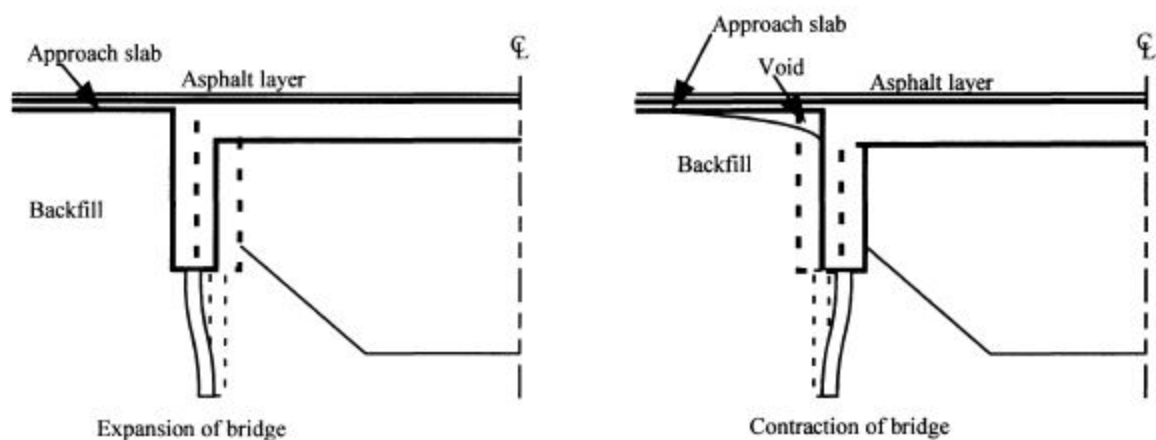


Figure 3. Thermal effects on bridge and integral abutment (Arsoy et al. 1999).

The void can lead to settlement of the approach slab, thus creating the bump. Furthermore, the rotation and lateral movement of the abutments results in lateral earth pressure on the abutment (Huffaker 2013). Settlement of approach slabs in integral abutment bridges can cause damage to the abutment backwall (Hoppe 1999). A few recommendations made in the literature to alleviate this problem have been found.

(Phares et al. 2013) suggested that one solution may be to integrally connect the approach slab to the abutment with an expansion joint at the other end of the approach slab. They developed a connection detail which can be seen in Figure 4. Their design was implemented in the construction of several bridges in the state of Iowa. After monitoring the performance of these bridges, they determined that their design performed well with no apparent damage to the bridges.

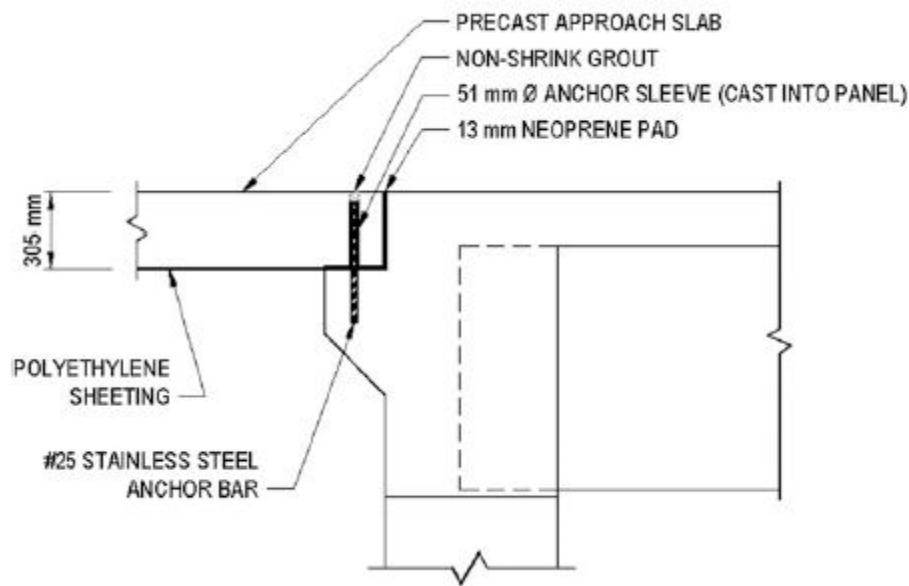


Figure 4. Integral connection detail for approach slab to abutment (Phares et al. 2013).

(Ng et al. 2014) mention the use of geosynthetic reinforcement along with a spacer which has proven to be effective in overcoming the problems associated with cyclic thermal effects. The spacer is created by a cardboard void between the backfill and the abutment. According to (Reid et al. 1999) and cited in (Phares et al. 2011), this gap reduces the passive pressures on the soil.

However, (Reid et al. 1999) also mention that the gap at the abutment can silt in (over time) and lead to the development of voids.

(Mistry 2005) suggests (as cited in (Greimann et al. 2008)) placing polyethylene sheets or filter fabric to reduce friction caused by horizontal movement. This would reduce the magnitude of the forces on the bridge structure as well as the connection between the approach slab and the bridge.

Skewed integral abutment bridges are susceptible to rotating due to cyclic changes in earth pressure (Puppala et al. 2011). To counter this, (Abendroth et al. 2007) recommends ensuring that the abutments are parallel and of equal height.

Chapter 3: Approach Slab Designs of Various States

Description of Review

Based on the literature review of the previous chapter, approach slabs (when designed properly) can help mitigate issues with the bump problem. This chapter provides a brief comparison between the different approach slab designs of various state transportation departments. The standard drawings and specifications for a handful of states were reviewed and information was compiled into Table A 1, which can be found in Appendix A. Information such as approach slab dimensions, sleeper slab dimensions (if applicable), surface type and more are included in the table. Additional relevant information not included in Table A 1 is mentioned in the following section. Furthermore, surface drainage provisions of various state departments are also presented. Proper surface drainage is needed to help prevent erosion of the soil underneath the approach slab.

State Design Details

Arkansas does not allow the backfill to be placed near abutments, retaining walls and box culverts until the concrete has cured for 14 days (AHTD 2014). The slightly different approach slab designs used by Alabama are based on the presence of an expansion joint in the abutment wall limits and whether a bituminous pavement overlay is included (ALDOT 2017). Colorado approach slab designs can include a ½” expansion joint at the approach slab-abutment interface, a 0-4” for an expansion device and drainage, or both (CDOT 2015). 7’ of the 10’ long sleeper slab from the Illinois design lie under the approach slab, while the remaining 3’ lie under the roadway (IDOT 2017). Additionally, Illinois also has guidelines for approach slabs with a skew angle greater than 30 degrees, less than 30 degrees and no skew. The expansion joint in the first approach slab type (from Illinois) is located at the approach slab-roadway interface. The strip seal and bearing pad for

the second approach slab type are at the approach slab-roadway interface and approach slab-abutment interface, respectively.

The Kentucky design includes a 1.5” diameter dowel in the integral end bent and a 1/8” neoprene pad at the vertical and horizontal interfaces between the approach slab and abutment (KYTC 2015). The type of connection used at the approach slab-abutment interface of the Ohio designs depends on: the surface type of the approach slab and the bridge deck, if the bridge utilizes precast concrete box beams, and if the approach slab is supported on an abutment backwall (ODOT 2015).

In the Texas approach slab designs, the construction joint runs along the direction of travel (TxDOT 2015). Missouri uses a 4” layer of Type 5 aggregate base under their approach slabs (MoDOT 2017). All of Missouri’s approach slab designs use a perforated drain pipe under the approach slab at the roadway end. The drain pipe is in the center of an 18”x18” “block” of Type 5 aggregate. Additionally, all of Missouri’s designs place 2 mil polyethylene sheeting between the approach slab and granular base. Finally, Missouri places 2 layers of 30-lb roofing felt between the approach slab, roadway and sleeper slab.

Surface Drainage Provisions

Arkansas. The Arkansas State Highway and Transportation Department (AHTD) has standard drawings for approach slab surface drainage for each approach slab type (AHTD 2014). Typically, surface drainage is addressed by including gutters in the approach slab at varying sizes and distances. These gutters carry water to the gutters running along the guardrail or wingwall at the edge of the approach slab. Figure 5 shows a plan view of one of AHTD’s approach gutter designs for an approach slab of 30’ in length. Figure 6 and Figure 7 both show cross section views of the edge of the approach slab where the gutters meet the guard rails.

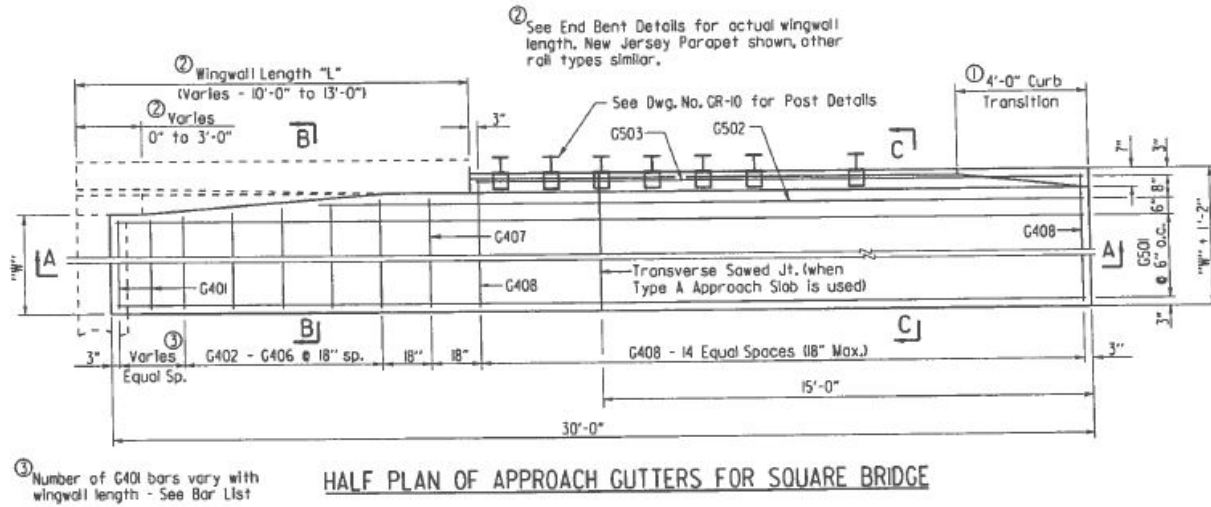


Figure 5. Approach gutter plan view (AHTD 2014).

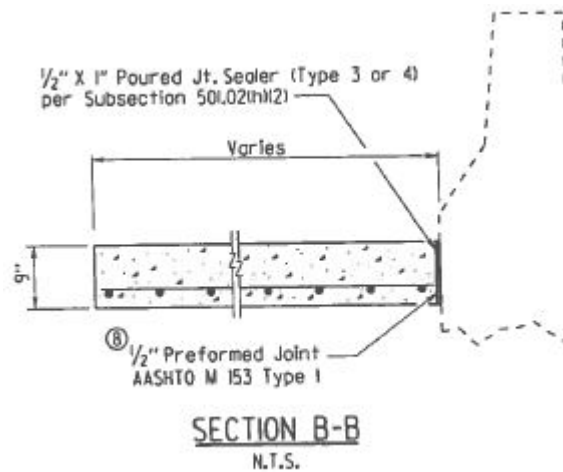


Figure 6. Approach gutter section view 1 (AHTD 2014).

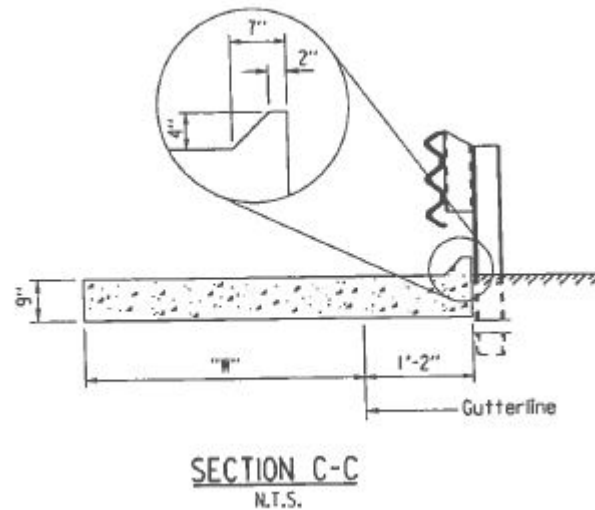


Figure 7. Approach gutter section view 2 (AHTD 2014).

Alabama. The Alabama Department of Transportation (ALDOT) designs their approach slabs and bridge decks such that they have a slight slope towards the edges where the barrier rails are located (ALDOT 2017). As can be seen in Figure 8, a 4" diameter PVC pipe is placed just before the barrier rail. A waterstop is used to seal any open joints in the barrier rail.

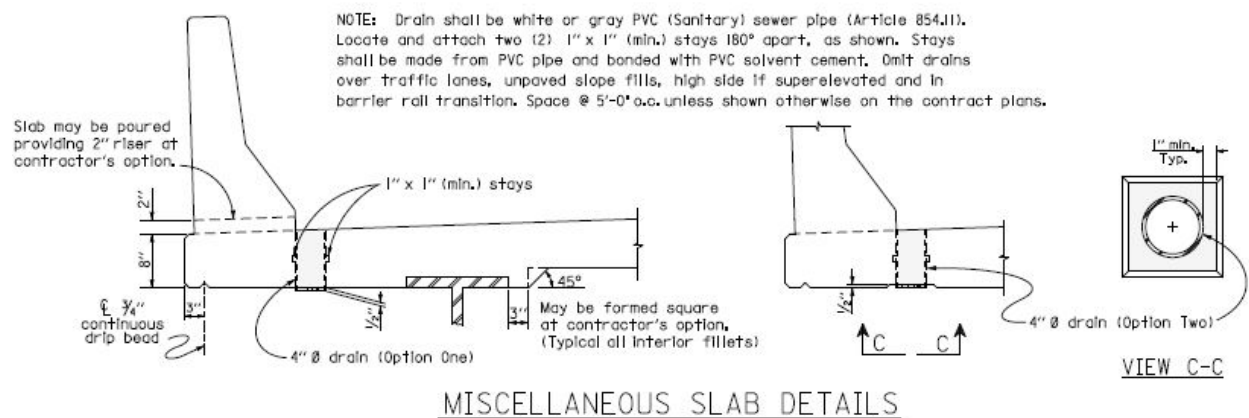


Figure 8. Alabama drainage at barrier rail (ALDOT 2017).

Colorado. The Colorado Department of Transportation (CDOT) uses deck drains at intervals designed to carry water away from expansion devices and bearings (CDOT 2017). CDOT generally follows AASHTO procedures when designing and detailing their deck drains. The

preferred drainage method is to minimize the number of deck drains required by directing water to approach drainage grates. This is done to reduce the amount of maintenance needed in the future. Finally, CDOT places approach slab drains on the high side of expansion devices. This is needed to reduce the flow of water over the joint. The difficulty with the approach slab drain is that it must be designed to withstand the movements of the bridge.

Illinois. The Illinois Department of Transportation (IDOT) typically uses 6" diameter floor drains (made of either fiberglass or aluminum) at the interface between the deck edge and concrete railing (IDOT 2012). There can be variances in the floor drains depending on the locations of the girders beneath. Figure 9 shows a section view of a typical floor drain design. When the 6" diameter drain cannot be used, a 4" by 12" aluminum drain is typically considered. IDOT also uses drainage scuppers as a means of surface drainage. IDOT has several standard scupper designs for varying sizes and situations. Figure 10 shows the drainage scupper design most commonly used by IDOT.

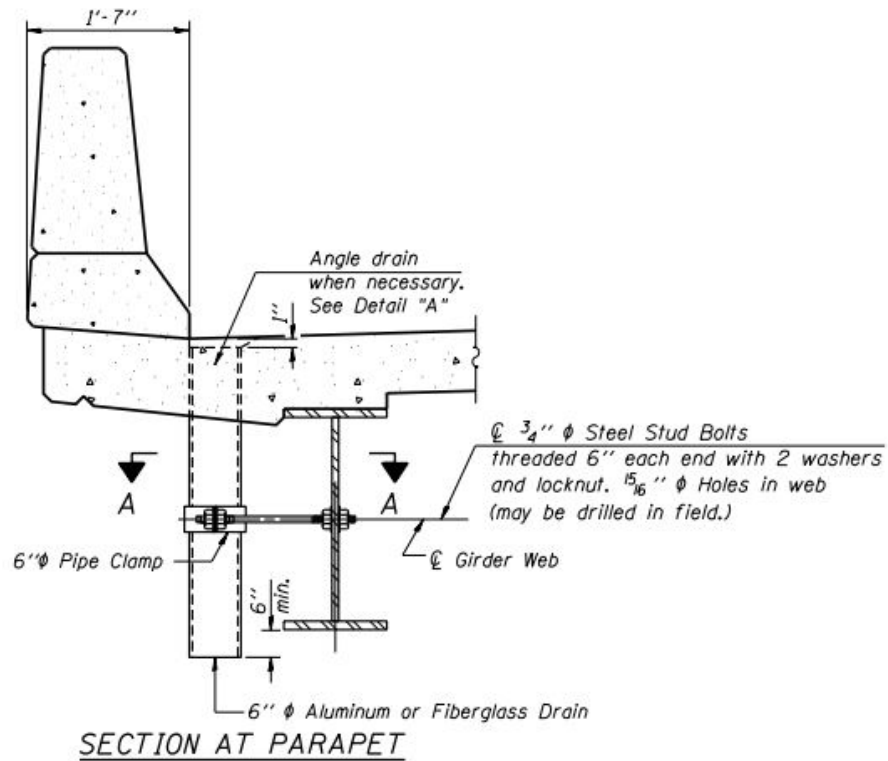


Figure 9. Illinois floor drain view at parapet (IDOT 2012).

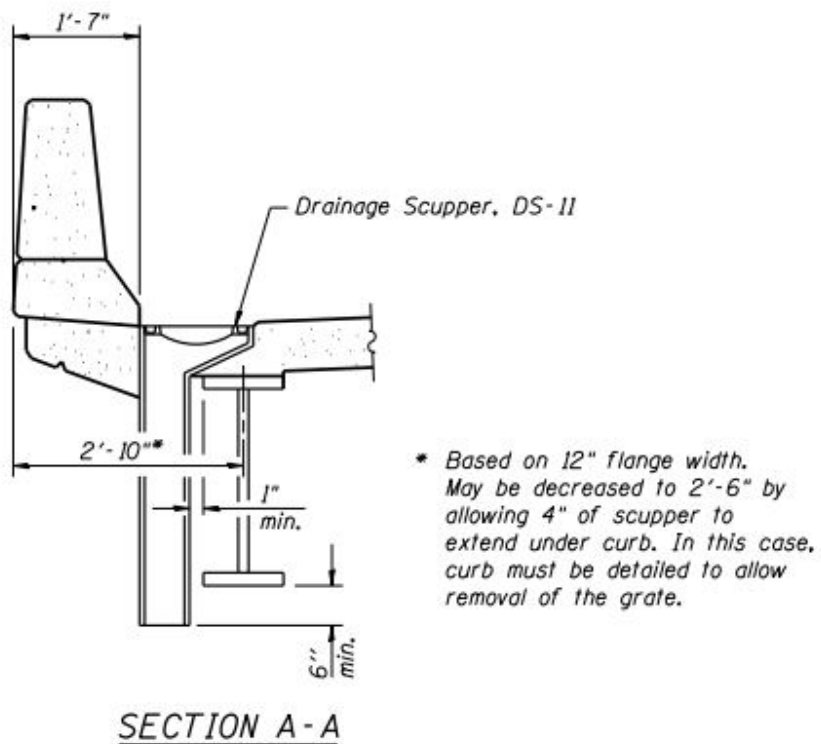


Figure 10. Illinois typical drainage scupper (IDOT 2012).

Kentucky. The Kentucky Transportation Cabinet (KYTC) designs bridge deck inlets with either barrier drains or deck drains (KYTC 2010). The drains are located such that they limit the spread of water. KYTC analyzes the barrier drains as curb inlet openings and the deck drains as grated inlets. Additionally, KYTC uses bridge end drains to collect water at the end of the bridge structure. This is somewhat similar to the end of bridge drain used by TDOT; however, KYTC locates their bridge end drain on the bridge deck rather than the approach slab (TDOT 2015).

Ohio. The Ohio Department of Transportation (ODOT) requires bridges decks to have a minimum longitudinal slope of 0.3% when concrete parapets are present (ODOT 2007). ODOT also uses scuppers as part of their deck drainage system, and prefer to minimize or eliminate the number of scuppers required. Scuppers with drainage collection systems are placed as close as possible to the connected substructure drainage unit. Uncollected or freefall scupper downspouts are placed as far away from the structure as possible. ODOT allows heavy duty iron cast deck drains to be used when uncollected scupper downspouts are not feasible.

Texas. The Texas Department of Transportation (TxDOT) has general bridge drain details for cast and welded bridge drains (TxDOT 2016). Generally, TxDOT uses sloped inlets that carry water to a drain pipe covered by a grate. TxDOT also provides details on connecting vertical pipe supports to the inlet. Figure 11 shows a plan view of the cast deck drain detail, while Figure 12 shows a section view. Similarly, Figure 13 and Figure 14 show the plan and section views of the welded deck drain details. It should be noted that TxDOT states that these design guides are not to be used as standards.

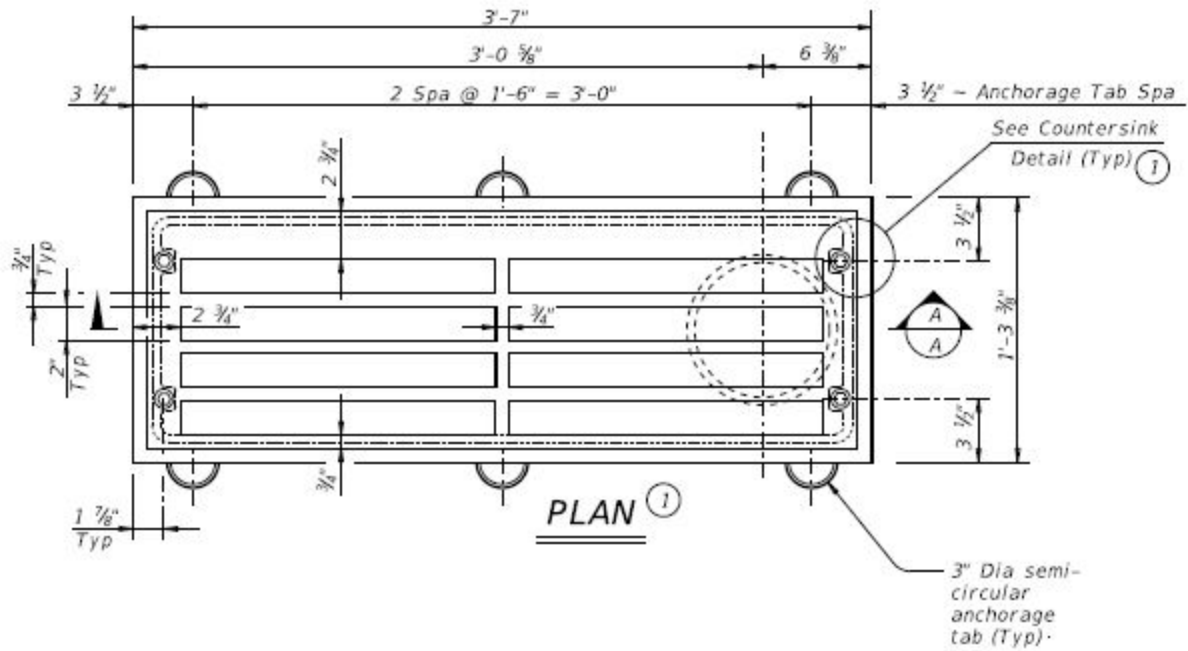


Figure 11. TxDOT plan view of cast bridge drain (TxDOT 2016).

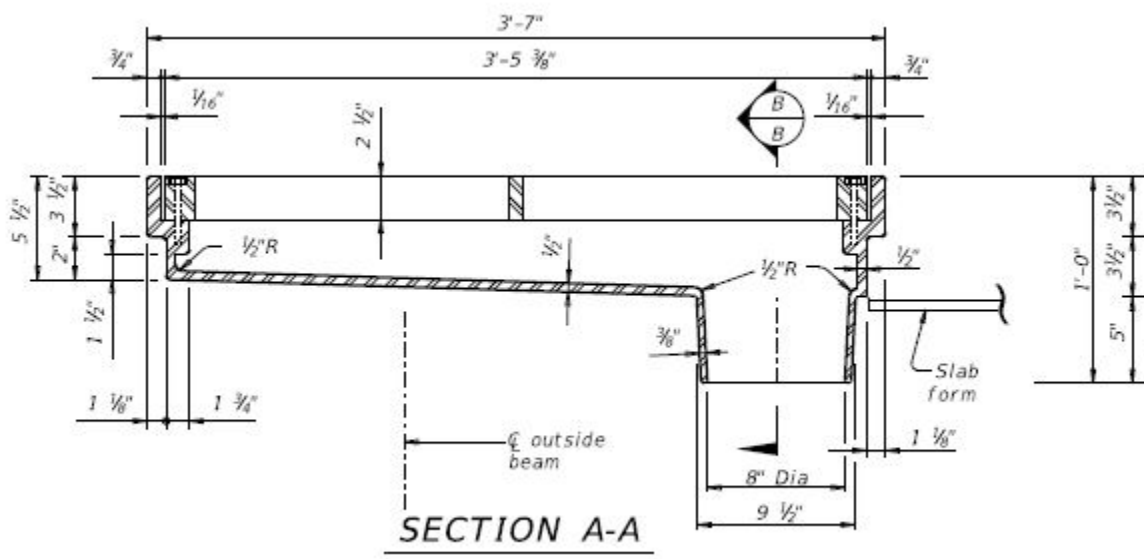


Figure 12. TxDOT section view of cast bridge drain (TxDOT 2016).

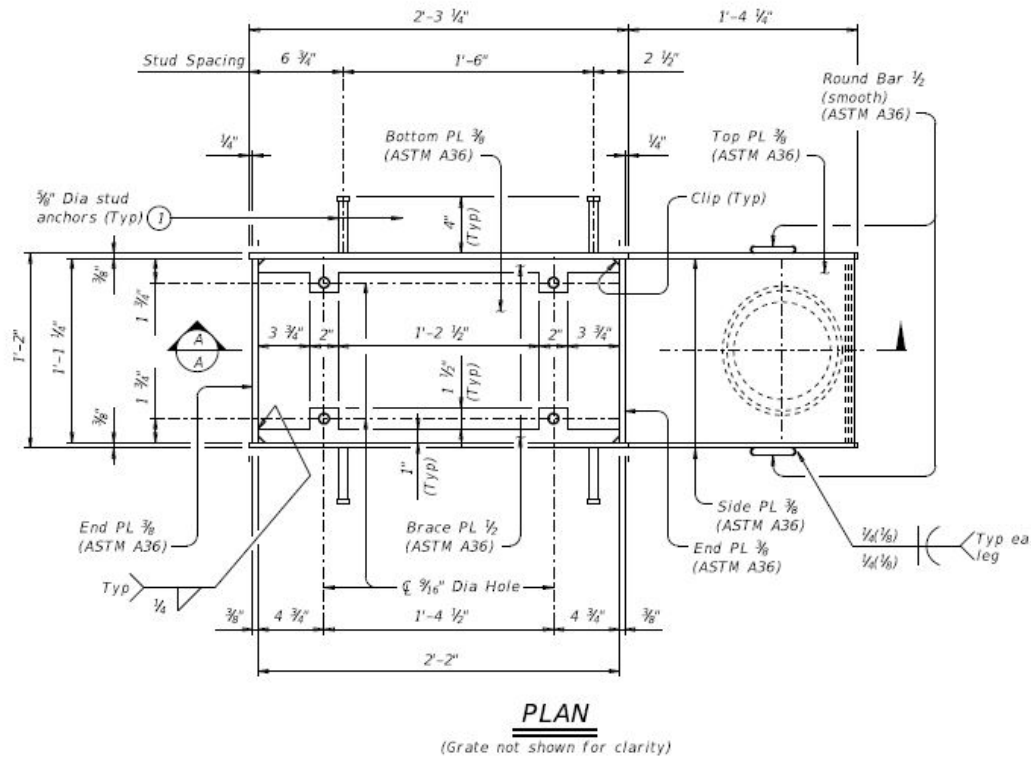


Figure 13. TxDOT plan view of welded bridge drain (TxDOT 2016).

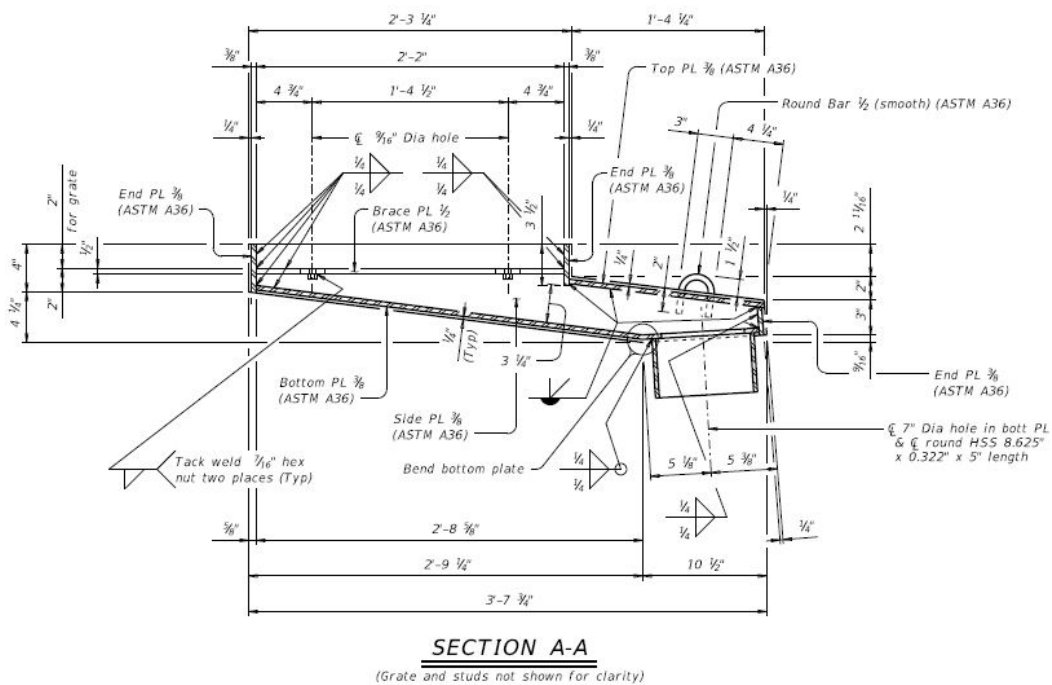


Figure 14. TxDOT section view of welded bridge drain (TxDOT 2016).

Chapter 4: Evaluation of Rider Discomfort by Rider Simulation

In this chapter, methods of evaluating smoothness and rider discomfort will be presented. A sinusoidal method of generating random road profiles is also discussed. Finally, the quarter-car model (a dynamics model) will be discussed and used to evaluate the effects different variables have on the bump and rider discomfort.

International Roughness and Profile Indices

The International Roughness Index (IRI) is a type of smoothness statistic that uses a mathematical model to represent the response of a vehicle to the road surface (FHWA 2016). The road profile can be recorded using a profilograph or an inertial road profiler, which is then used to calculate the suspension motions. These motions are summed and divided by the length of the profile to calculate the IRI. A mathematical expression for this can be seen in Equation (1) below (Sayers 1995).

$$IRI = \frac{1}{L} \int_0^{L/V} |\dot{z}_s - \dot{z}_u| dt \quad (1)$$

where L is the length of the profile, V is the horizontal speed of the car, and $\dot{z}_s$ and $\dot{z}_u$ are the vertical velocities of the sprung and unsprung masses, respectively. These values are typically given in units of inches/mile or mm/km. The mathematical model most typically used in producing the IRI is the quarter-car model. The profile used for the IRI calculation must have elevation values recorded at constant intervals (ASTM 2015). The IRI will not be calculated as part of the analysis of this report as it describes the overall ride quality and the focus of this report is the localized “bump at the end of the bridge”. However, the quarter-car model will be used for the analysis conducted later in this chapter.

The Profile Index (PI) is a smoothness statistic that is typically used to evaluate new pavements. PI values are typically calculated from profile data obtained from profilographs. However, utilizing special software allows one to calculate PI values from profile data obtained from inertial profilers. Similar to the IRI, PI values are given in units of inches/miles or mm/km. Acceptable PI values vary by agency and depend on the width of the blanking band used on the profilograph (FHWA 2016). The PI is used in several states (including Tennessee). However, there is no dynamic model associated with the PI.

Quarter-Car Analysis

The quarter-car model is illustrated below in Figure 15 (Sayers 1995). The simulation of this model is typically conducted at a speed of 80 km/h (or 50mph) and has defined values for the masses, springs and dampers. The values used for masses, springs, and the damper in this quarter-car model are typically normalized with respect to the mass of the spring (m_s). The values obtained from (Sayers 1995) are $c = 6.0$, $k_1 = 653$ (k_t/m_s), $k_2 = 63.3$ (k_s/m_s) and $\mu = 0.15$ (m_u/m_s). With these values, the model is known as the “Golden Car” model. This model represents a vehicle with average natural frequencies and a higher than typical level of damping. This higher damping broadens out the response spectrum of the vehicle, such that its response frequency is not as defined and the results of an analysis with this model are not overly tuned to one set of vehicle dynamics.

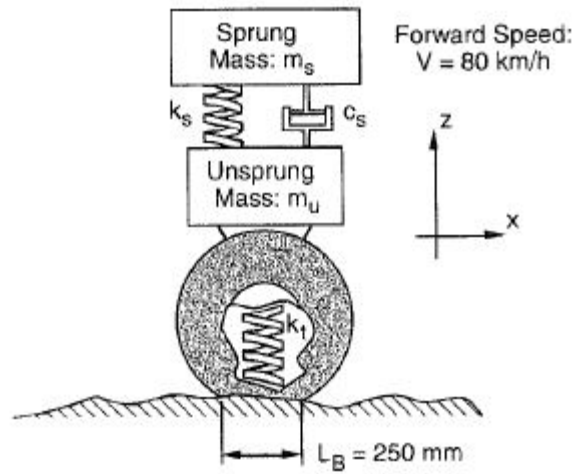


Figure 15. Quarter-car model (Sayers 1995).

The state space system based on this model is defined with matrices A, B, C and D below. The output is a matrix with the vertical displacements, velocities, and accelerations of the sprung and unsprung masses.

$$A = \begin{bmatrix} 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 1 \\ -k_2 & k_2 & -c & c \\ k_2/\mu & -(k_1+k_2)/\mu & c/\mu & -c/\mu \end{bmatrix}$$

$$B = \begin{bmatrix} 0 & 0 & 0 & k_1/\mu \end{bmatrix}$$

$$C = \begin{bmatrix} 1 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 \\ 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 1 \\ -k_2 & k_2 & -c & c \\ k_2/\mu & -(k_1+k_2)/\mu & c/\mu & -c/\mu \end{bmatrix}$$

$$D = \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & k_1/\mu \end{bmatrix}$$

A moving average filter is typically used to smooth the profile data that is used as an input to this model. This moving average filter is defined in Equations 2 and 3 (Sayers 1995).

$$h_{ps}(i) = \frac{1}{k} \sum_{j=1}^{i+k-1} h_p(j) \quad (2)$$

$$k = \max[1, \text{nint}\left(\frac{L_B}{\Delta}\right)] \quad (3)$$

In these equations, h_{ps} is the smoothed vertical profile of the roadway, h_p is the vertical profile of the roadway, Δ is the sample interval, L_B is the moving average base (250mm), and nint stands for the nearest integer.

The following figures (Figure 16 and Figure 17) are sample time history responses showing the sprung accelerations and vertical displacements of the quarter car model to three different road roughness categories (a concept that is described in a later section of this chapter). The results in each plot (for the same roughness category) are based on the same profile.

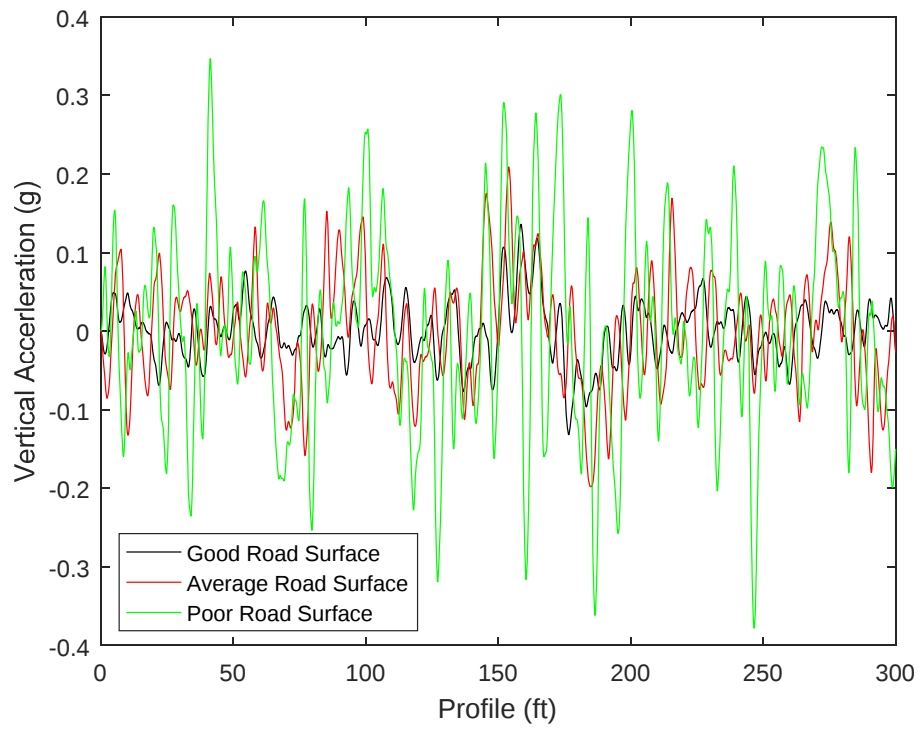


Figure 16. Vertical acceleration time history of quarter-car model.

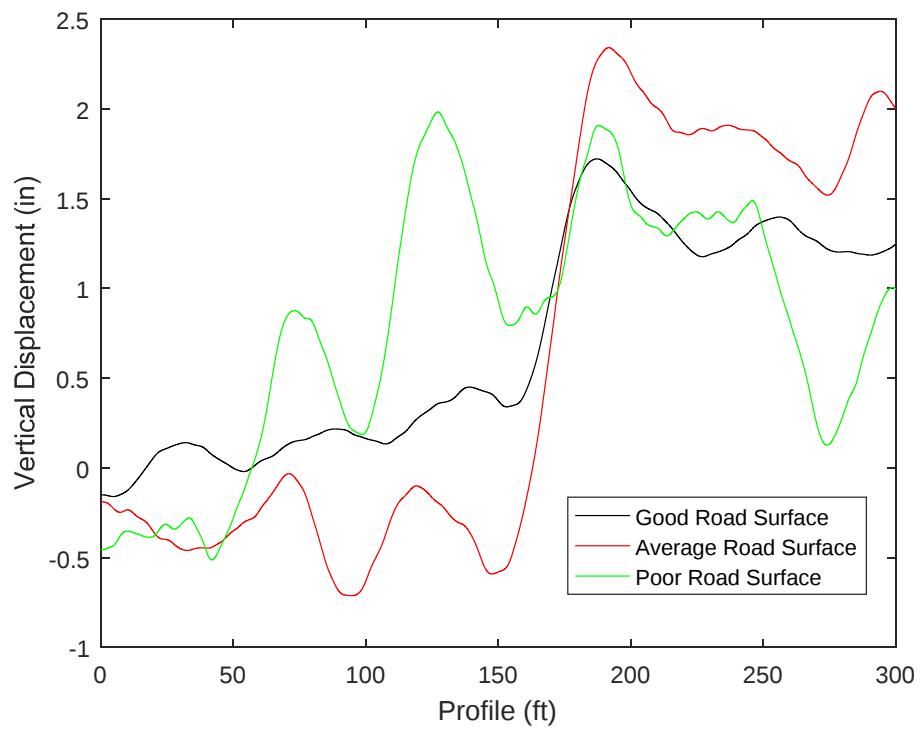


Figure 17. Vertical displacement time history of quarter-car model.

Discomfort Criteria

Discomfort criteria are needed to correlate the results of the quarter-car model and driver response. Discomfort is typically quantified by acceleration of the driver in some form. Peak vertical acceleration and root-sum-of-square (RSS) acceleration have both been used for discomfort criteria in several studies (Ekoru and Pedro 2013). The RSS combines root-mean-square (RMS) accelerations along different axes and takes duration into account (Weber and Braaksma 2000). Although no consensus was found in the literature, ISO 2361-1 states that frequency weighted RMS acceleration is the basis for evaluation of vehicle ride comfort (ISO 1997). In any case, a higher acceleration indicates a higher driver discomfort. For the purpose of this paper, the peak vertical acceleration of the sprung mass as determined from the analysis of the quarter-car model (see following section) was chosen as the first criteria for measuring driver discomfort.

Additionally, the absolute maximum displacement of the suspension was also used as a discomfort criteria. This was done to take into account the visual aspect of a driver's response to the bump. This can include the effect of the motion of a driver's own car or seeing another car respond as it drives over a bump. Furthermore, vehicles exposed to higher vertical displacement risk damage to their suspensions.

Bump Profile Generation

Three different bump types were generated for analysis. The first bump type was a sloped line made with a selected bump length and bump slope, with the resulting bump height being a function of these two parameters. In other words, this type of bump represents a linear transition

from one pavement height to another. This general sloped shape can be seen in Figure 18, and was the shape used for the majority of the analyses.

The second bump type is a smoothed version of the first type. The bump was smoothed by using a circle with a given radius (called the bump radius here) that intersected at the horizontal and sloped parts. The original part of the bump profile between the two intersection points was then removed, leaving the rounded section from the circle. This process was applied to both ends of the bump. In order to make sure the two circles do not intersect more than once, a maximum radius (R_{max}) was found using Equations 4 and 5. An illustration of a circle with a given bump radius applied to the sloped bump can be seen in Figure 18.

$$R_{max} = \frac{Bump\ Length}{2 \tan \frac{\theta}{2} \cos \theta} \quad (4)$$

$$\theta = \tan^{-1}(Bump\ Slope) \quad (5)$$

The third and final bump type is one that has an abrupt change in height as shown in Figure 19.

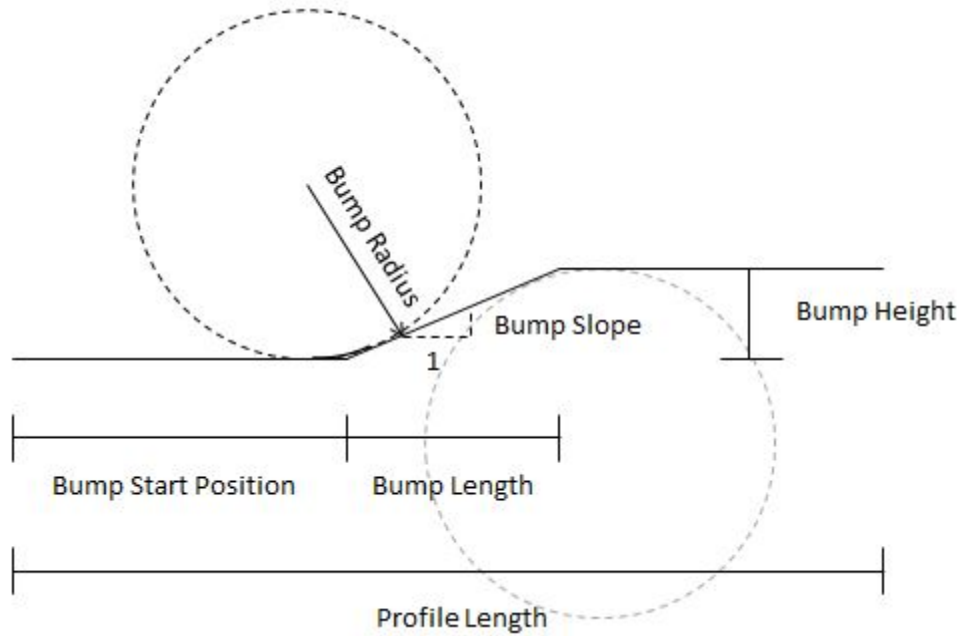


Figure 18. Bump profile diagram.

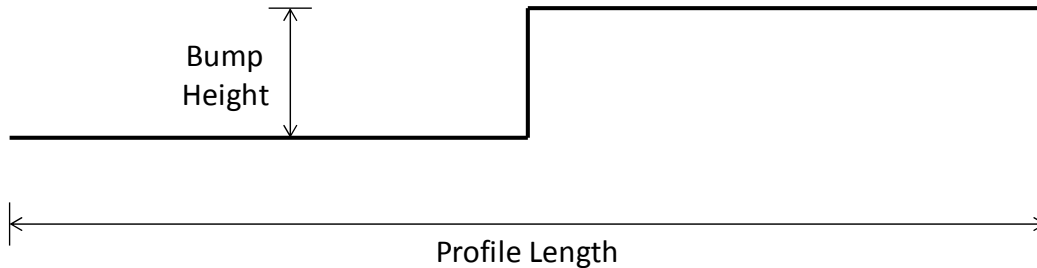


Figure 19. Abrupt bump profile diagram.

Road Profile Generation

In reality, roadways will include sources of roughness in addition to the bump at the end of the bridge focused on in this report. In order to include the effect of these other sources of roughness in the calculation of response measures evaluating the surface roughness of roads, random road profiles were generated. The generated road profiles were represented by a Power Spectral Density (PSD) function. PSD values were selected based on recommendations by the

International Standardization Organization (ISO) to classify road conditions (Goenaga et al. 2017). For the purpose of this study, road profiles were only generated for categories representing good, average, and poor. The form used for the PSD is shown in Equation 6,

$$\Phi(\Omega) = \Phi(\Omega_0) \left(\frac{\Omega_i}{\Omega_0} \right)^{(-m)} \quad (6)$$

where $\Phi(\Omega_0)$ is the pavement roughness degree based on $\Omega_0 = 1$ rad/m, Ω_i is the angular spatial frequency (rad/m), and m is the pavement waviness indicator estimated to be 2. Values for $\Phi(\Omega_0)$ for different roughness categories can be found in Table 1.

Table 1 Pavement roughness degree values for $\Omega_0 = 1$ rad/m (Tyan et al. 2009).

DEGREE OF ROUGHNESS EXPRESSED IN TERMS OF Ω			
	degree of roughness $\Phi(\Omega_0)(10^{-6}\text{m}^3)$ where $\Omega_0 = 1\text{rad/m}$		
road class	lower limit	geometric mean	upper limit
A (very good)	—	1	2
B (good)	2	4	8
C (average)	8	16	32
D (poor)	32	64	128
E (very poor)	128	256	512

The two most common techniques used to simulate a road profile are the shaping filter approach and sinusoidal approximation. In this study, the sinusoidal approximation was used to generate random profiles in MATLAB using the following equations (Goenaga et al. 2017).

$$Z_R(s) = \sum_{i=1}^N A_i \sin(\Omega_i s - \phi_i) \quad (7)$$

$$A_i = \sqrt{\Phi(\Omega_i) \frac{\Delta\Omega}{\pi}} \quad (8)$$

$$\Delta\Omega = \frac{\Omega_N - \Omega_1}{N - 1} \quad (9)$$

In these equations, Z_R is the random profile height, s is a point along the length of the profile, N is a number approaching an infinite number of sine waves and ϕ_i is a random variable on the interval $[0, 2\pi)$. For the purpose of this report the value of N was chosen as 1000. $\Delta\Omega$ is a frequency interval, where $[\Omega_1, \Omega_N]$ represents a range of spatial frequencies where the PSD form has significant values (Sun 2003). Here, the values were selected to be $\Omega_N = 1$ rad/m and $\Omega_1 = 0.01$ rad/m. Additionally, using Equation 7a from (Sun 2003), the following equation was used to find Ω_i .

$$\Omega_i = \Omega_1 + (i - 1/2)\Delta\Omega \quad (10)$$

Figure 20 and Figure 21 are sample time history responses showing the sprung accelerations and vertical displacements of the quarter car model to three different road roughness categories. The results in each plot (for the same roughness category) are based on the same profile.

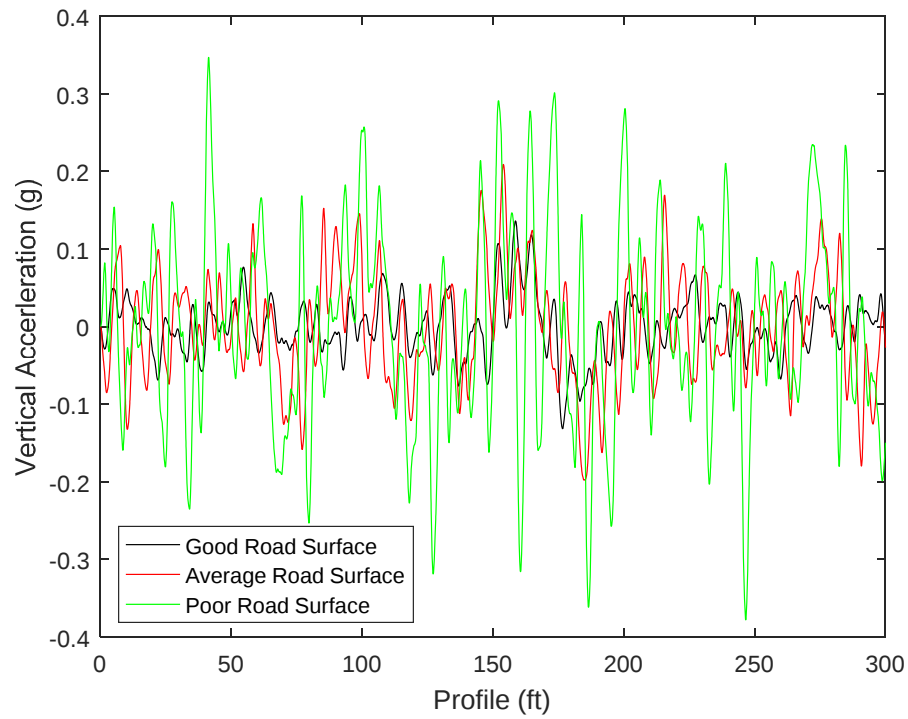


Figure 20. Vertical acceleration time history of quarter-car model.

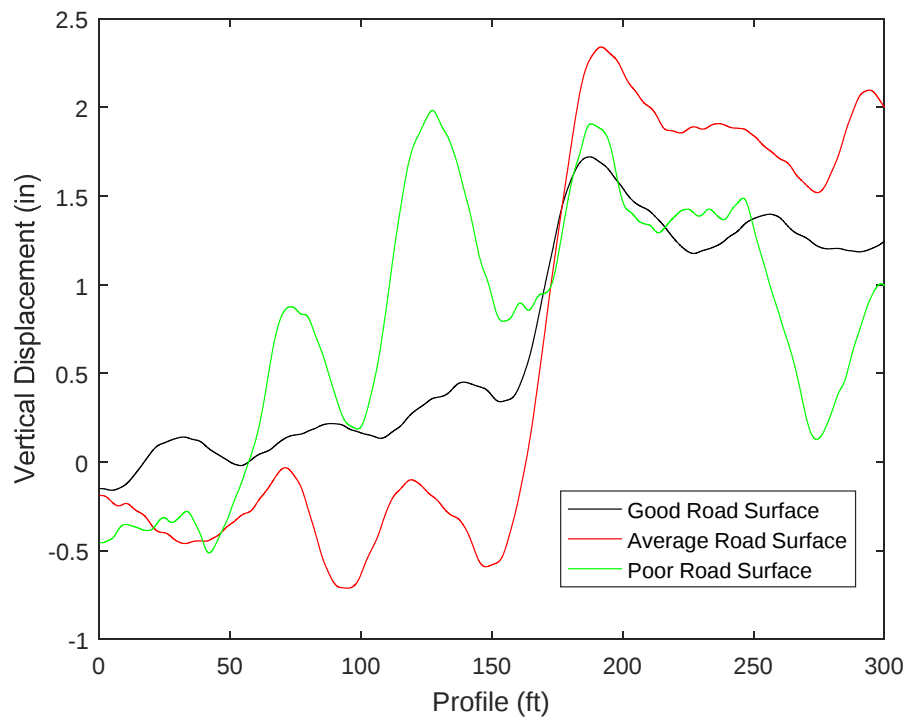


Figure 21. Vertical displacement time history of quarter-car model.

The following figures (Figure 22 through Figure 27) show three different simulations of random and total road profiles with good, average, and poor surface qualities according to Table 1. Note that the profiles are indeed random for each generation. To produce a final total profile, the random and sloped bump profiles were simply added together.

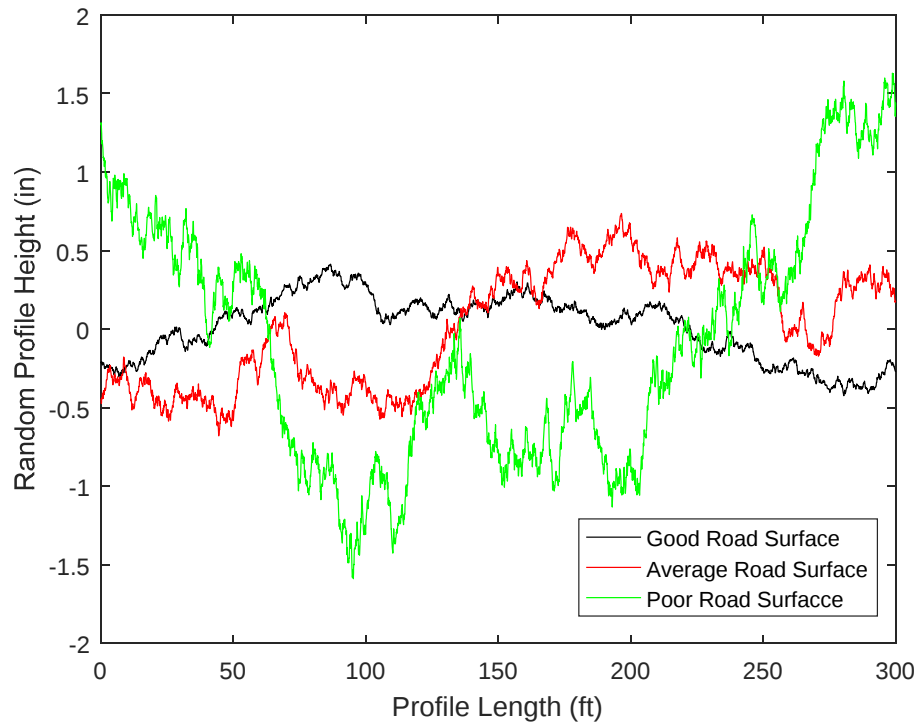


Figure 22. Random plot generation 1 not including bump.

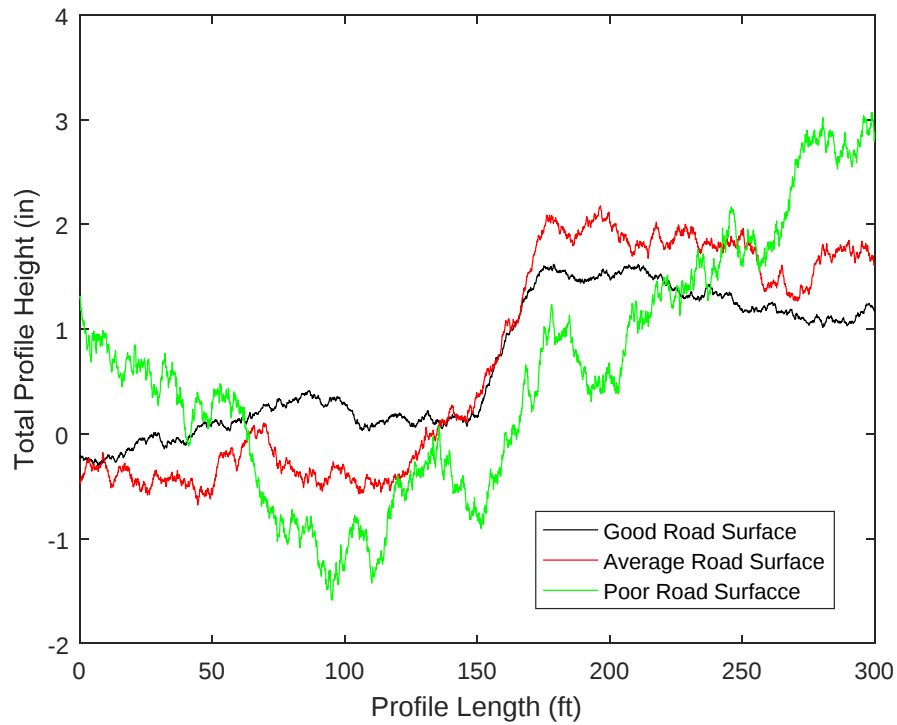


Figure 23. Random plot generation 1 with bump.

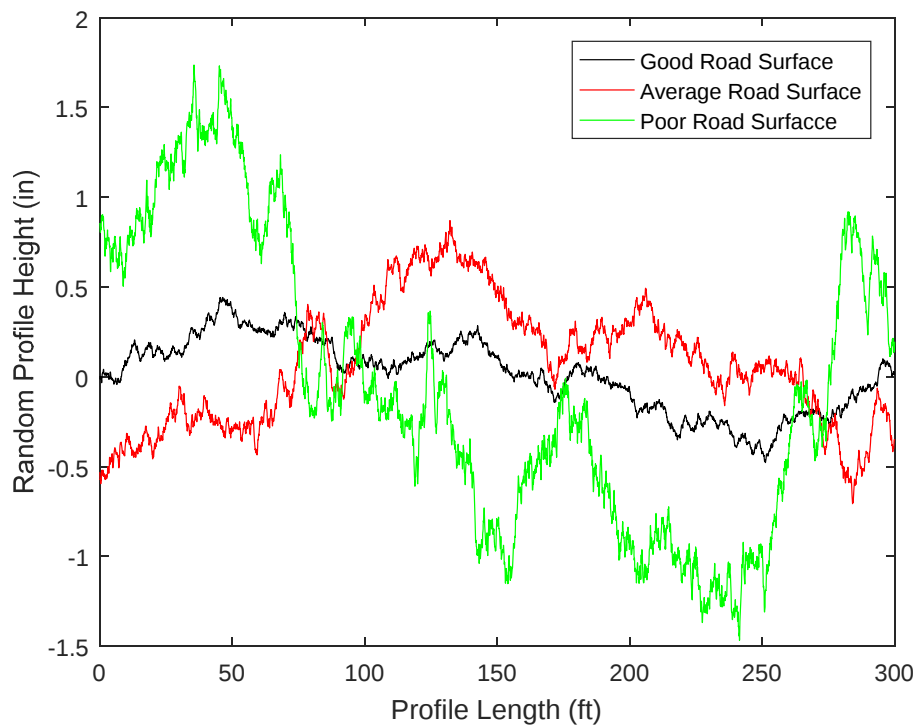


Figure 24. Random plot generation 2 not including bump.

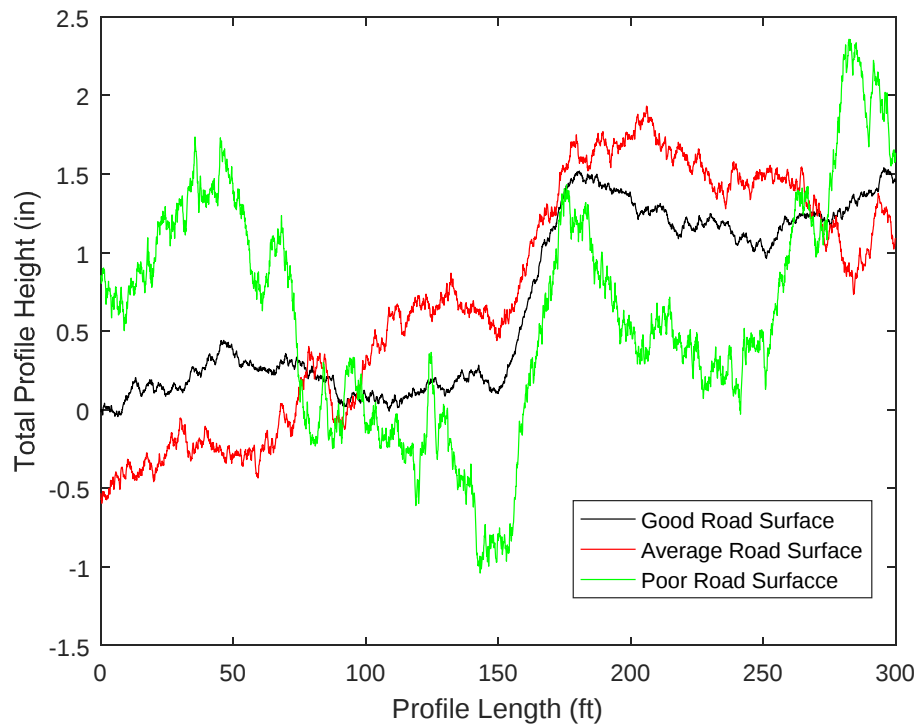


Figure 25. Random plot generation 2 with bump.

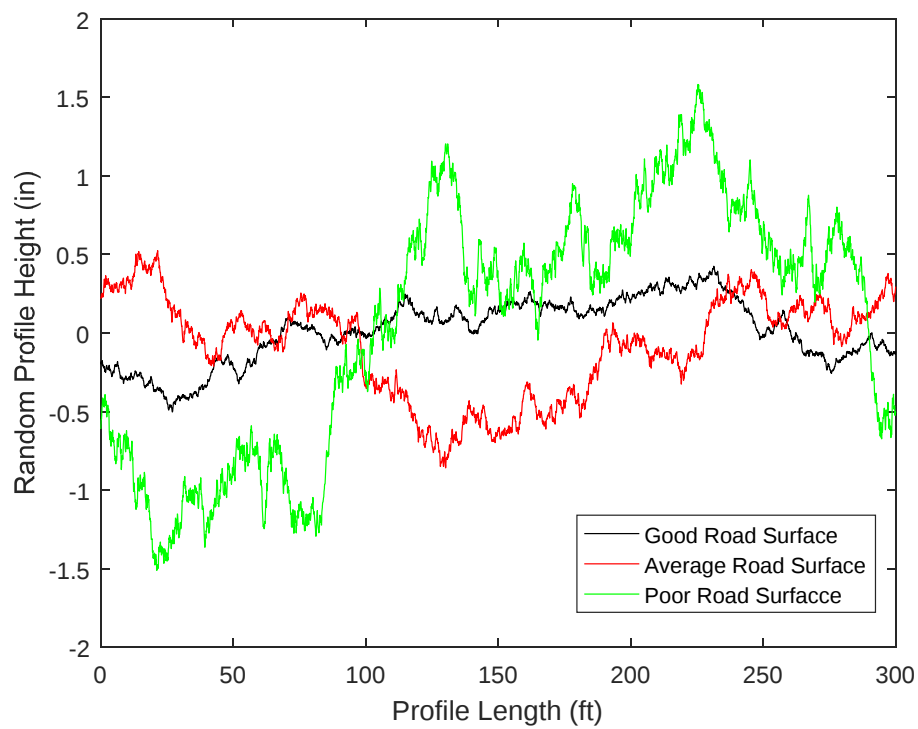


Figure 26. Random plot generation 3 not including bump.

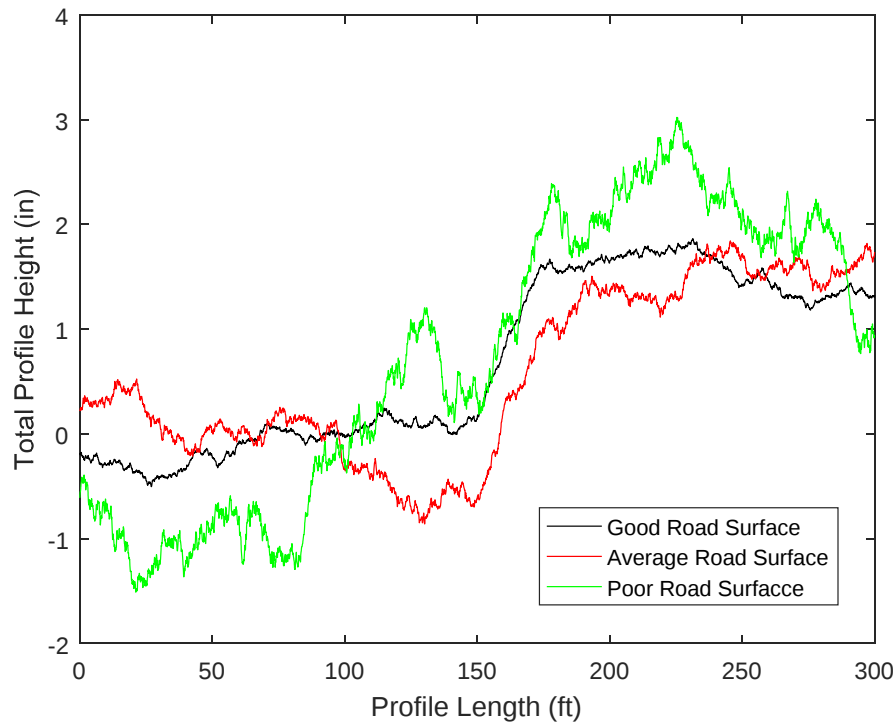


Figure 27. Random plot generation 3 with bump.

Analysis

In this analysis the values for the bump slope, bump length, profile length, and vehicle speed (all with the bump with the sloped, linear transition) were analyzed in order to assess their effects on driver discomfort. Plots were created with either peak vertical acceleration or absolute maximum displacement on the y-axis and the variable considered on the x-axis. The default variables utilized for the bump included a bump slope of 1/200, a bump length of 24ft, a profile length of 300ft, and a vehicle speed of 80 km/h (~50 mph). The speed of 50 mph was used as this is the default value for most simulations using the quarter-car model.

To account for different road surface quality, the analysis was ran with three different pavement roughness degree ($\Phi(\Omega_0)$) values for each variable. The three pavement roughness degree values were chosen based on geometric means for the good, average, and poor categories

in Table 1. Additionally, a case with no additional pavement roughness, besides the bump, was also considered.

The underlying pavement roughness for the road profiles used for this analysis are created using a random process; therefore, in order to ensure consistency in the results, a total of 500 simulations were needed for each bump parameter set and road roughness category. This number of simulations was chosen because, with this number, consistency was achieved when the maximum acceleration and absolute maximum displacement were calculated and plotted. In addition to plotting these average values, the lines representing the different roughness categories in each figure featured triangular plot markings of the 80th and 20th percentile of the values considered for the 500 simulations.

Bump slope. The results of the bump slope analysis can be seen in Figure 28. The analysis was executed for bump slope values of 0, 1/400, 2/400, 3/400, 4/400 and 5/400. The figure shows that an increasing bump slope will lead to increased peak vertical acceleration. Increasing bump slope has a lower effect on poorer quality roads. In fact, the poor category requires a much higher bump slope for there to be a noticeable increase in the peak vertical acceleration. Interestingly, the baseline average peak vertical accelerations, as measured when the bump slope is zero, for the good, average and poor conditions are nearly 0.1, 0.2, and 0.4g respectively. There also appears to be a higher variability in the results for the poor category when compared to the other two. The absolute maximum vertical displacement follows the same trends as the peak vertical acceleration for varying bump slope, as can be seen in Figure 29.

In addition to running this analysis with positive bump slopes, negative bump slope values were ran to determine the effect the direction of travel has on rider discomfort. This resulted in a

mirror image of the results from the positive bump slopes, indicating that the direction of travel does not have any effect for this type of bump. Because of this result, travel in only one direction was considered when evaluating the effect of the other bump variables on the response of the vehicle.

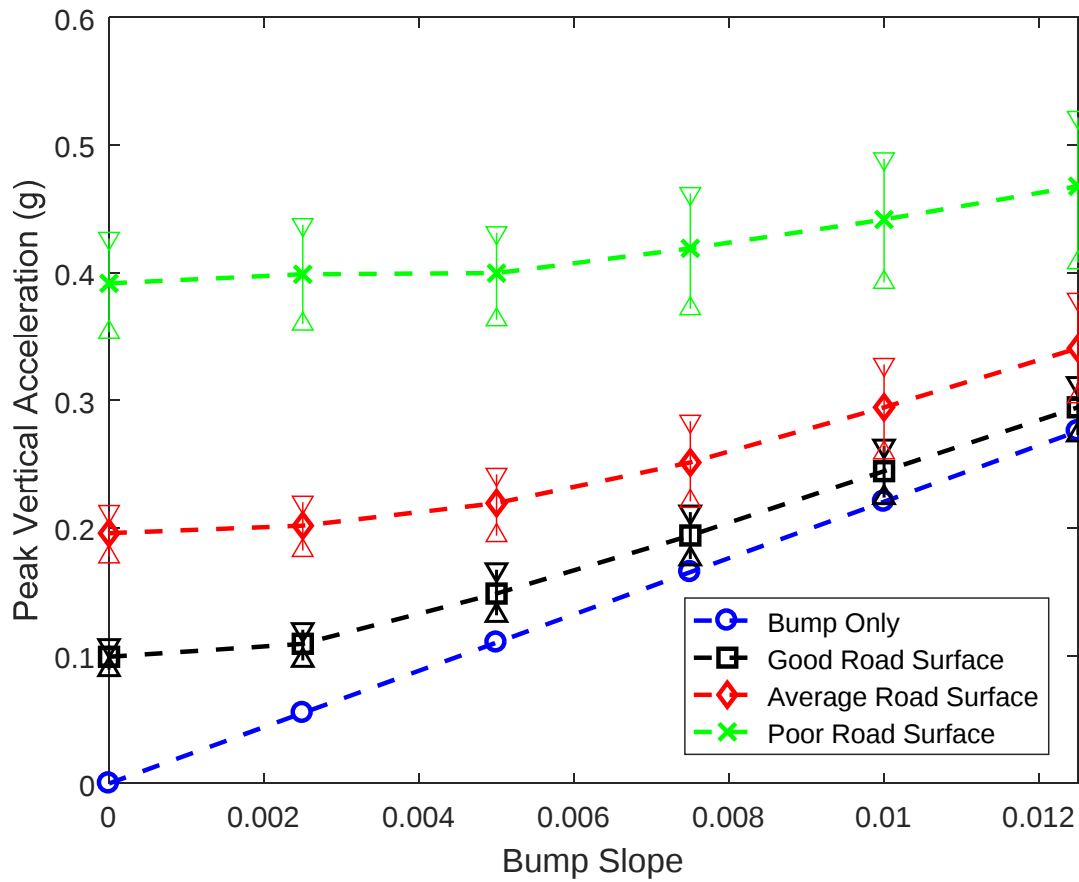


Figure 28. Peak vertical acceleration vs bump slope.

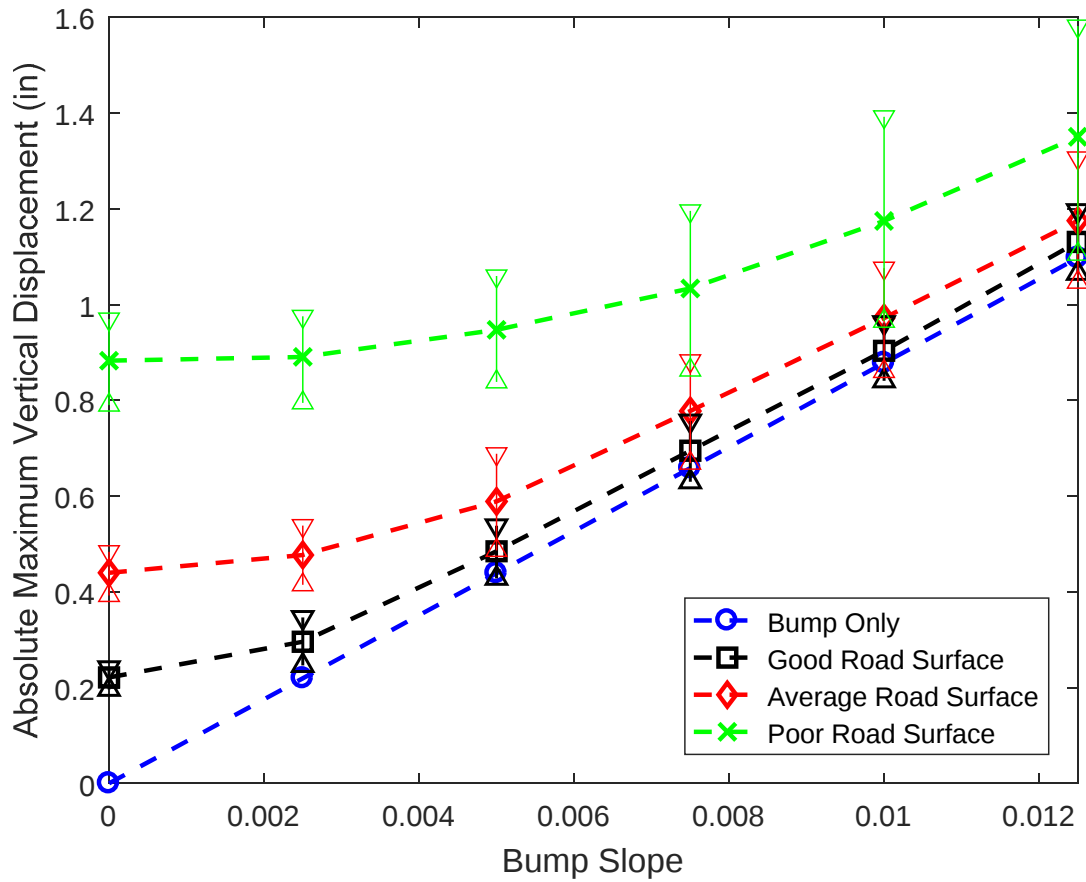


Figure 29. Absolute maximum vertical displacement vs bump slope.

Bump length. The analysis to evaluate the effect of the length of bump was performed with bump (approach slab) lengths of 16ft, 20ft, 24ft, 28ft, and 32ft. The lower and upper bounds of these values were chosen based on the approach slab lengths in Table A 1. The results of the bump length analyses can be found in Figure 30. The data in the figure show slight increases in peak accelerations with increasing bump lengths. The most pronounced effects are on the good and average quality road surfaces. The peak vertical acceleration increases very little for the poor quality road surface. Overall, the length of the bump has little effect on the peak vertical acceleration.

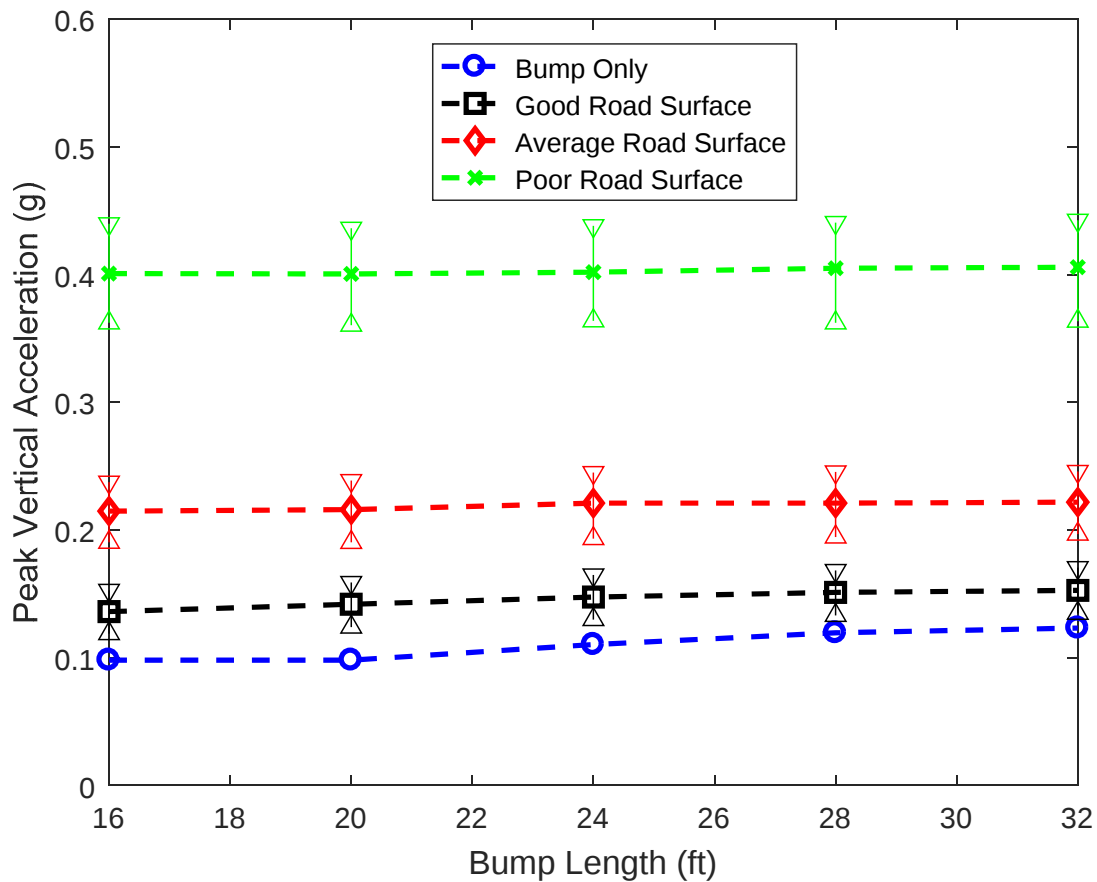


Figure 30. Peak vertical acceleration vs bump length.

Conversely, the length of the bump does appear to have more of an effect on the absolute maximum vertical displacement, as can be seen in Figure 31. The vertical displacement increases (at a decreasing rate) with increasing bump length for the bump only profile over all roughness categories. This can be explained by recognizing that by increasing the bump length, while maintaining the slope, the bump height increases.

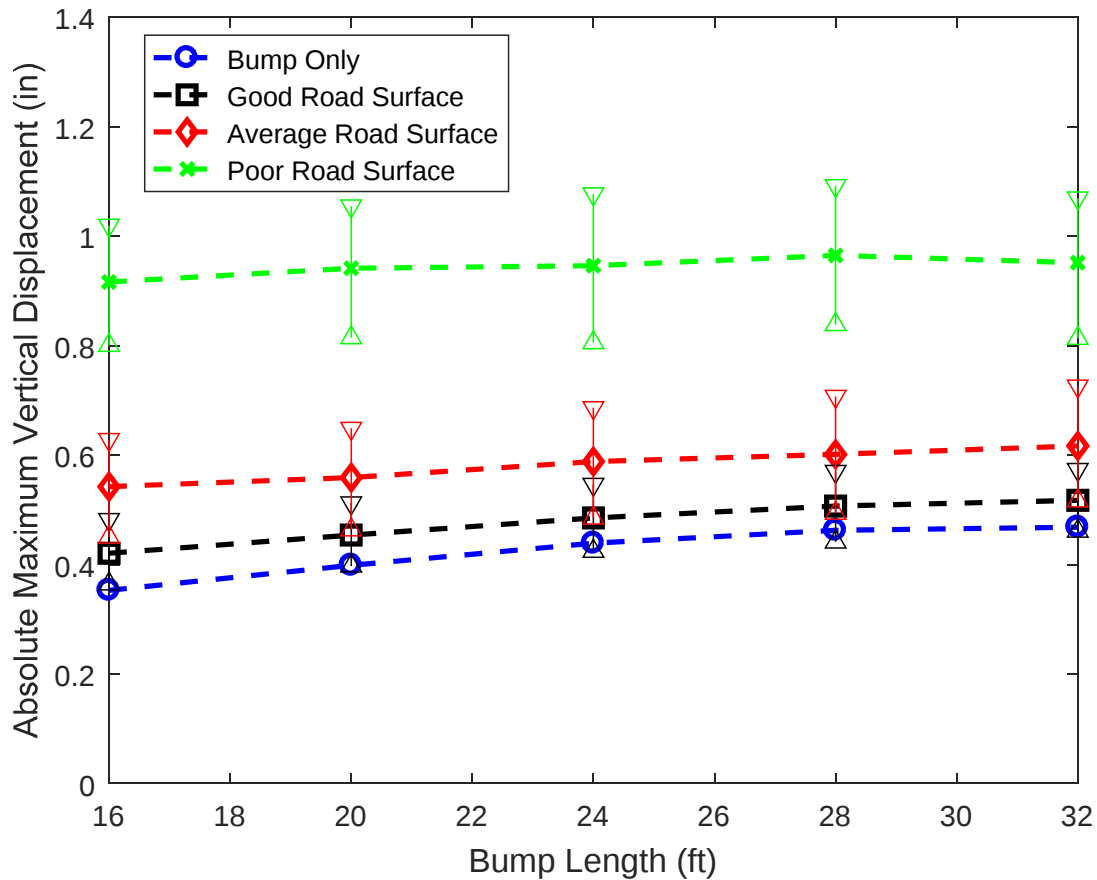


Figure 31. Absolute maximum vertical displacement vs bump length.

Profile length. Profile lengths (as defined in Figure 18) of 300ft, 400ft, 500ft, 600ft, 700ft, and 800ft were analyzed to study their potential effects on rider discomfort. Based on the results shown in Figure 32, profile length appears to have little to no effect on the vertical acceleration. The bump only case shows no change in acceleration, and the good road surface case shows only minimal change. The data are more variable for the cases with average and poor road surfaces. However, this variability may be attributed more to roughness of the profiles rather than the length of the profile. The absolute maximum vertical displacement follows the same trend as the peak vertical acceleration, which can be seen in Figure 33. Both of these results were expected, as the length of the profile should have little to do with peak vertical acceleration and displacement.

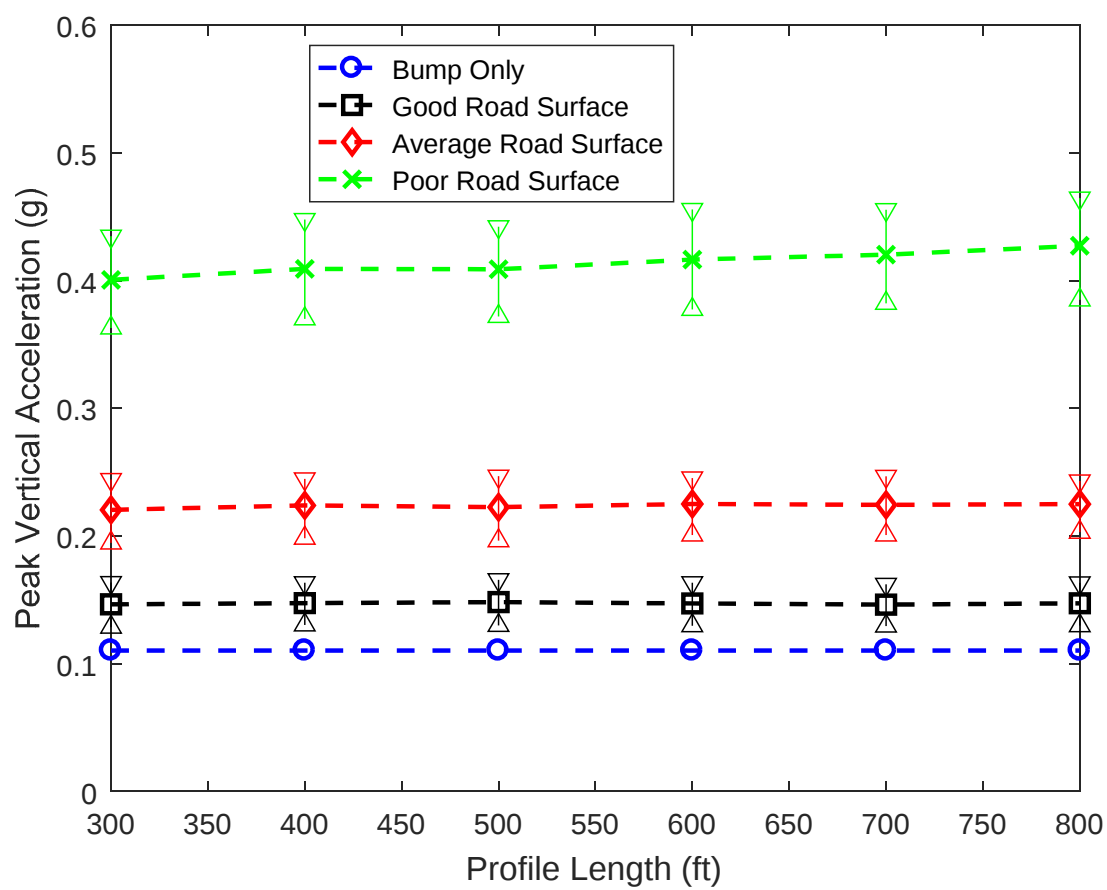


Figure 32. Peak vertical acceleration vs profile length.

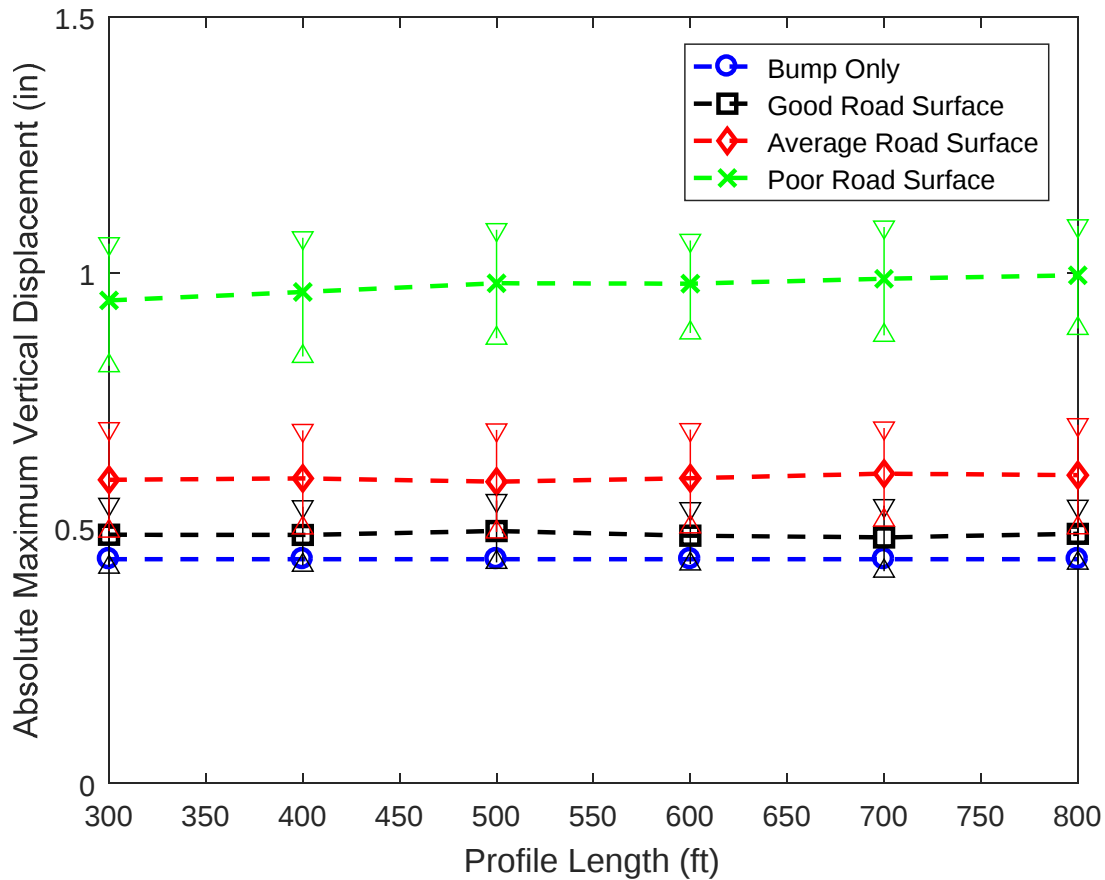


Figure 33. Absolute maximum vertical displacement vs profile length.

Vehicle speed. Vehicle speeds of 40, 47, 54, 61, 68, and 75 mph were tested. A total of 500 iterations were needed to insure consistency of the results. The mean peak vertical accelerations of the vehicle speed tests can be found in Figure 34. The total profile lines all have slight increases in acceleration with increasing speed; therefore, drivers going fast are more likely to experience discomfort from this bump, along with discomfort from the underlying road roughness. Physically speaking, this makes sense; one would expect to feel the impact of the bump more when going over it faster. As with the previous sections, the simulations with poor roughness had higher variability in their results.

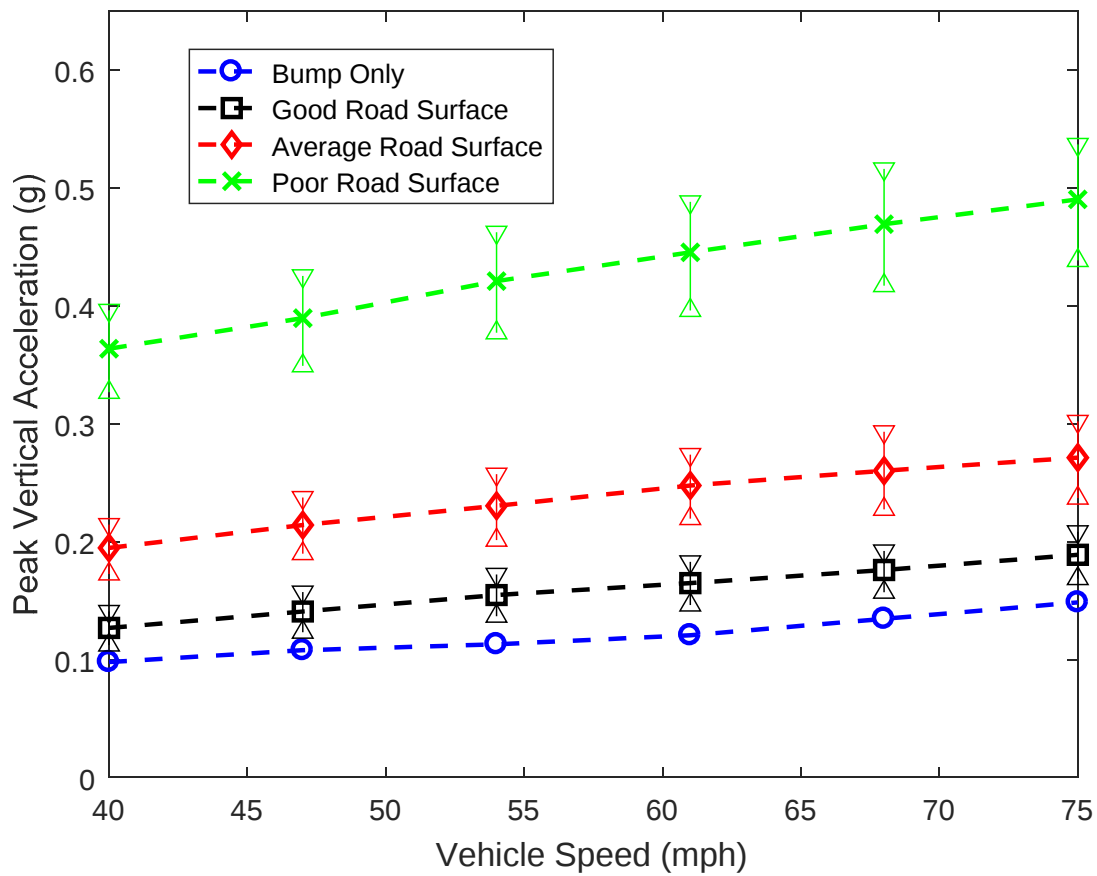


Figure 34. Peak vertical acceleration vs vehicle speed.

The vertical displacement responses of the model follow the same trends as the acceleration values, and can be seen in Figure 35. The results indicate that increasing vehicle speed results in higher absolute maximum vertical displacements.

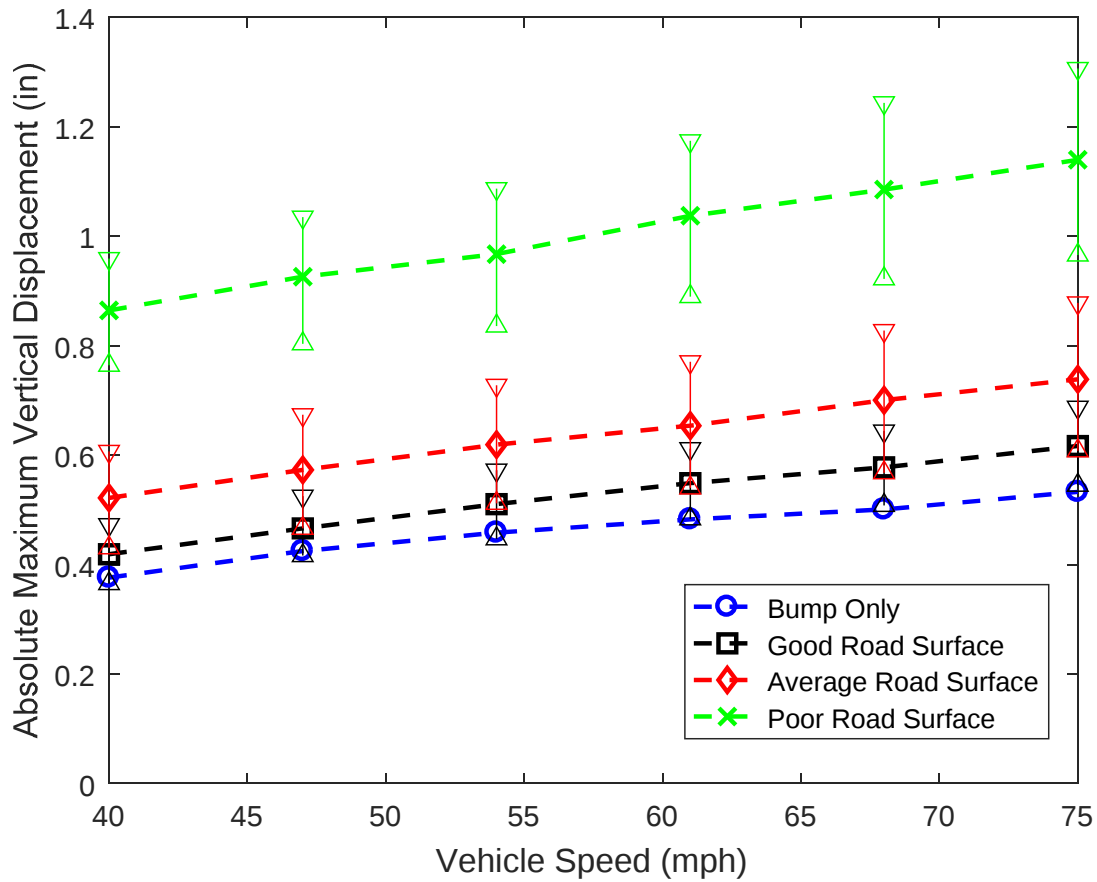


Figure 35. Absolute maximum vertical displacement vs vehicle speed.

Smoothed bump profile. Another set of simulations were ran to determine the effects of varying bump radii (or the radii of the circles used to smooth the bump profiles). The peak vertical acceleration responses can be seen in Figure 36, while the absolute maximum vertical displacement responses are shown in Figure 37. Both are plotted against the radius normalized with respect to R_{max} , the maximum possible radius. The trends for each show decreasing accelerations and displacements with increasing bump radius. This indicates that an increased bump radius results in a smoother bump profile, and thus a more comfortable ride quality.

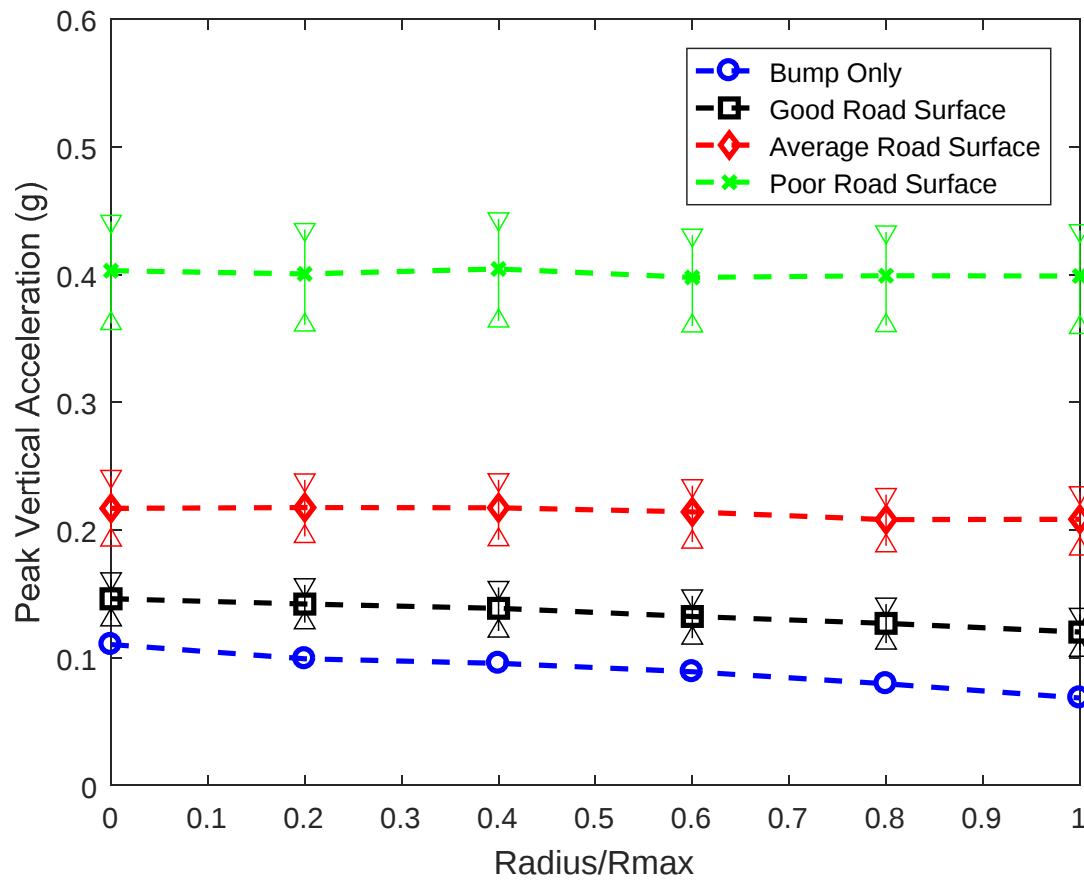


Figure 36. Peak vertical acceleration vs radius/rmax.

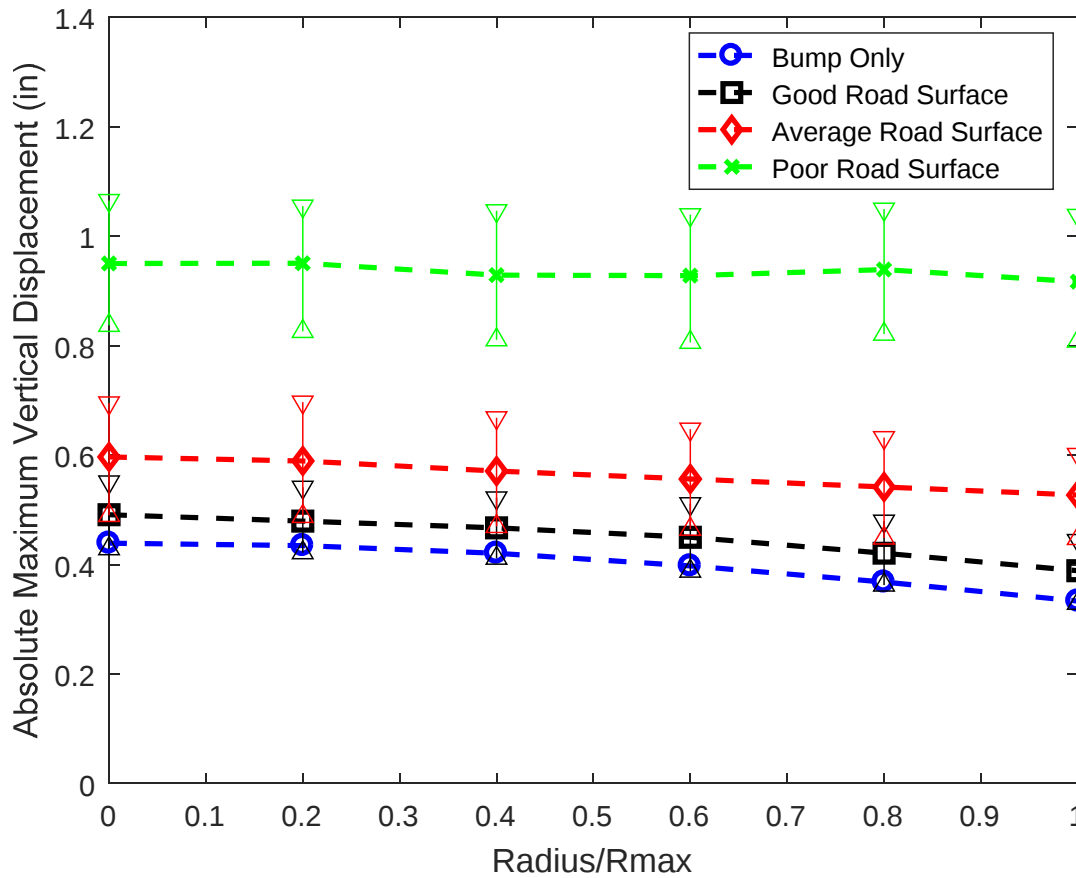


Figure 37. Absolute maximum vertical displacement vs bump radius.

Abrupt bump profile. Lastly, the effects of various bump heights ranging from 0.25” to 1” for the abrupt bump profile were analyzed. The peak vertical acceleration results are in Figure 38, while the absolute maximum vertical displacement results can be found in Figure 39. The results show that increasing bump heights lead to increasing vertical accelerations and displacements. The vertical accelerations for this situation are higher than for any of the other parameters. Meanwhile, the vertical displacements are the second-highest of the parameters tested, with the bump slope yielding the highest. The lower quality road surfaces do have an effect on the lower bump height values; however, as the bump height approaches 1” the effects of road quality

become negligible. In fact, the results for the different roughness categories converge to the point where they are almost indistinguishable.

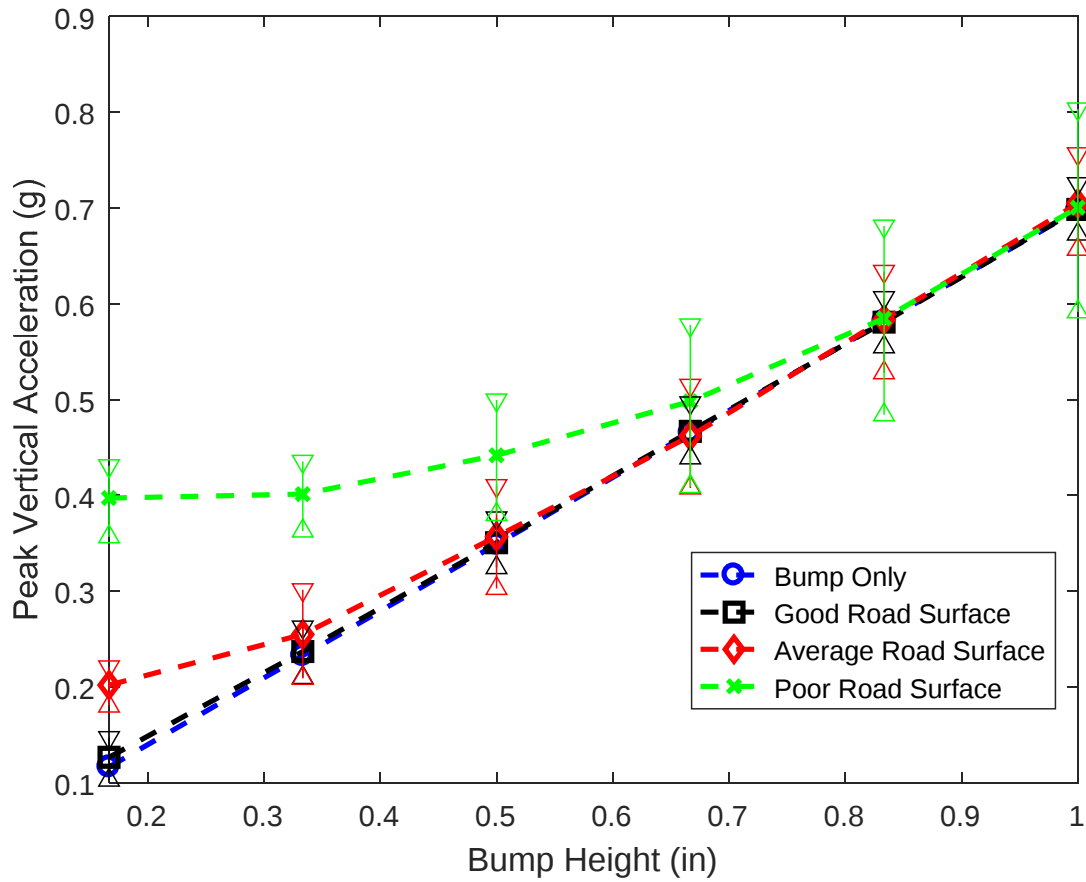


Figure 38. Peak vertical acceleration vs bump height (abrupt).

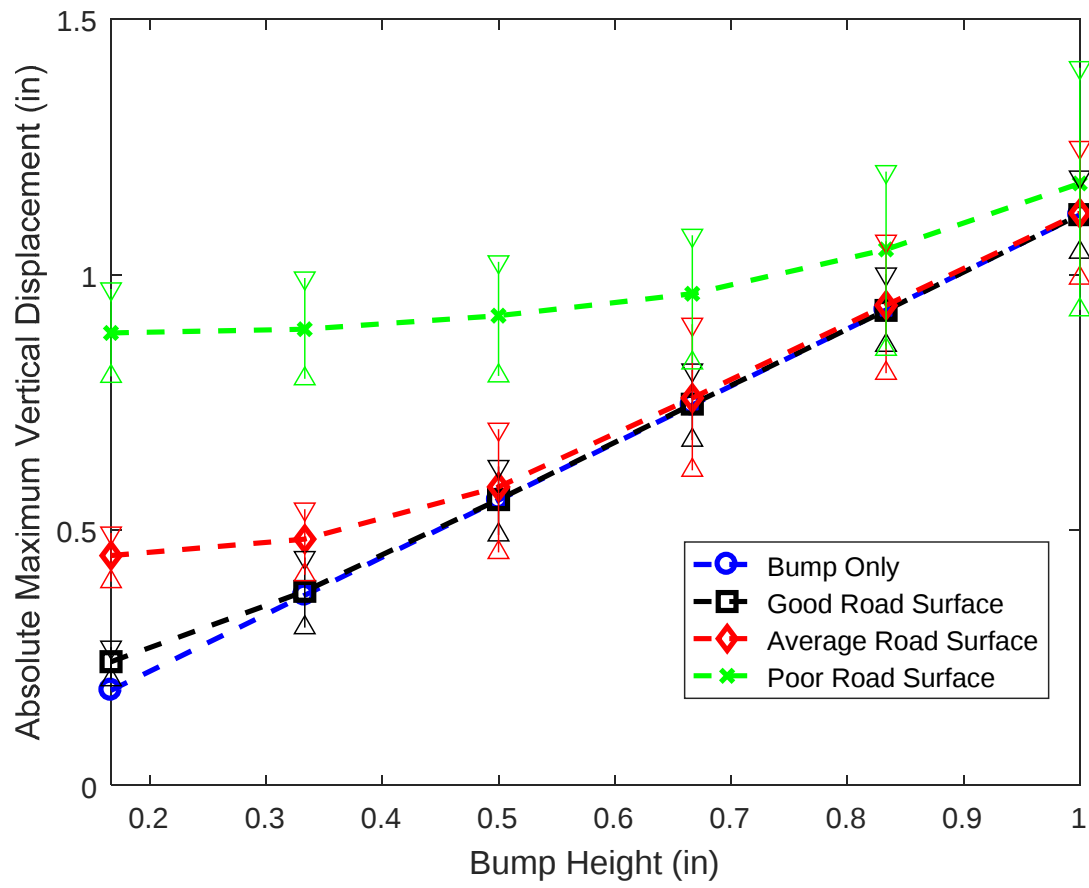


Figure 39. Absolute maximum vertical displacement vs bump height (abrupt).

Chapter 5: Conclusions and Recommendations

The bump at the end of the bridge remains a persistent problem for many state transportation departments and can result in driver complaints and costly repairs. Several mitigation techniques and retrofitting methods have been implemented with varying degrees of success. Despite the advantages of integral abutment bridges, they are still susceptible to the bump problem. Recent developments in structural connection design and the use of backfill reinforced with geosynthetic material have shown some success in alleviating the bump problem with integral abutment bridges.

The general approach slab design of several states were found and compiled in this report; however, the performance of each type could not be found. Parameters that vary between states that could affect the performance of the approach slab include slab length and thickness, placement of reinforcement, presence of a sleeper slab, sleeper slab dimensions, and details of the approach slab and bridge interface.

Based on numerical analysis of a simplified model vehicle, the bump (or approach slab) slope and vehicle speed have the biggest impacts on rider discomfort. The rider discomfort increases with increasing bump slope and vehicle speed. The sinusoidal approach to random road profile generation and the quarter-car model were successfully merged to analyze the effects of different bump parameters as well as vehicle speed. In addition to the sloped bump profile used for the majority of the analyses; both a smoothed, sloped bump profile and a profile with an abrupt vertical height difference were also analyzed. Increasing smoothness of the corners of the sloped bump profile did have slight positive effects on the response of the model. The abrupt vertical bump profile results indicate that once a certain bump height is reached, the quality of the road surface becomes negligible.

Further research on this topic could focus on a number of different objectives. Obtaining performance information for approach slabs of other state departments and analyzing that data would be useful to determine the field success of different design elements. Refinements to the dynamics model used in this work such as including three-dimensional behavior, more realistic damping, or the ability to model a wider variety of vehicles (e.g., trucks with trailers) would aid in the development of a rational basis for limitations on the acceptability of approach slab settlement and for the evaluation of different potential remediation actions. Three-dimensional measurement of road-bridge interfaces in the field would allow for a better determination of the underlying cause of the problem and improvements in the fidelity of the dynamic simulations.

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Appendix¹

Table A 1. Summary of Approach Slab Designs by Various Transportation Departments.

State	Thickness (in)	Length	Width	Construction Type(s)	Sleeper Length	Sleeper Thickness	Sleeper Connected?	Overlay(s)	Backfill Note(s)	Reinforcement	Skew	Other Notes
Tennessee	12	24'-0"	Roadway	CIP	3'-0"	1'-0"	No	Concrete		Middle, Bottom	Variable	EOB drain, Felt paper,etc.
Arkansas	9	16'-0"	Roadway	CIP	3'-0"	1'-9"	Yes	Same as bridge	6" layers compacted to 95%	Bottom	Variable	For approach slabs of 30' or greater, two sleeper slabs are used
	9	20'-0"	Roadway	CIP	3'-0"	1'-9"	Yes	Same as bridge	6"layers compacted to 95%	Bottom	Variable	
	9	30'-0"	Roadway	CIP	3'-0"	1'-9"	Yes	Same as bridge	6"layers compacted to 95%	Bottom	Variable	
	9	33'-0"	Roadway	CIP	3'-0"	1'-9"	Yes	Same as bridge	6"layers compacted to 95%	Bottom	Variable	
	14.5	36'-6"	Roadway	CIP	3'-0"	1'-9"	Yes	Same as bridge	6"layers compacted to 95%	Top & Bottom	Variable	
Alabama	10	20'-0"	24'-0"	CIP	N/A	N/A	N/A	Concrete, Bituminous Pavement		3" from Bottom	0° to 15°	Uses seating rather than sleeper. Uses 4 different approach slab designs
	10 to 11	20'-0"	24'-0"	CIP	N/A	N/A	N/A	Concrete, Bituminous Pavement		3" from Bottom	15° to 33°	
	10 to 14	20'-0"	24'-0"	CIP	N/A	N/A	N/A	Concrete, Bituminous Pavement		3" from Bottom	33° to 45°	
	10 to 14	20'-0"	24'-0"	CIP	N/A	N/A	N/A	Concrete, Bituminous Pavement		3" from Bottom	> 45°	
Colorado	12	20'-0"	Roadway	CIP	3'-0"	1'-0"	No	Asphalt		Top & Bottom	Variable	Expansion Joint
	12	20'-0"	Roadway	CIP	3'-0"	1'-0"	Yes	Concrete		Top & Bottom	Variable	Expansion Device
	12	20'-0"	Roadway	CIP	3'-0"	1'-0"	No	Asphalt		Top & Bottom	Variable	2 Expansion Joints
	12	20'-0"	Roadway	CIP	3'-0"	1'-0"	No	Concrete		Top & Bottom	Variable	2 Expansion Joints
Illinois	15	30'-0"	Roadway	CIP	10'-0"	10" minimum	No	Concrete		Top & Bottom	Variable	Expansion Joint
	16	30'-0"	Roadway	Precast	10'-0"	10" minimum	No	Concrete		Top & Bottom	Variable	Strip Seal, Bearing Pad
Kentucky	17	25'-0"	Roadway	CIP	N/A	N/A	N/A	Concrete	Geotextile Fabric	Bottom	Variable	Dowel, Neoprene Pad
Ohio	12	15'-0"	Roadway	CIP	N/A	N/A	N/A	Concrete, Asphalt		Top & Bottom	Variable	Reinforcement details on design lengths. Several connection types available at slab-abutment interface
	13	20'-0"	Roadway	CIP	N/A	N/A	N/A	Concrete, Asphalt		Top & Bottom	Variable	
	15	25'-0"	Roadway	CIP	N/A	N/A	N/A	Concrete, Asphalt		Top & Bottom	Variable	
	17	30'-0"	Roadway	CIP	N/A	N/A	N/A	Concrete, Asphalt		Top & Bottom	Variable	
Texas	13	20'-0"	Roadway	CIP	5'-0"	10"	Yes	Concrete		Top & Bottom	Variable	Construction joint in approach slab
	13	20'-0"	Roadway	CIP	N/A	N/A	N/A	Asphaltic Concrete		Top & Bottom	Variable	
Missouri	12 (min.)	20'-0"	Roadway	CIP	3'-0"	1'-6"	No	Concrete	Type 5 Aggregate Base	Top & Bottom	Variable	Perforated drain pipe, polyethylene sheeting, roofing felt, etc.
	12 (min.)	20'-0"	Roadway	CIP	N/A	N/A	N/A	Concrete, Asphalt	Type 5 Aggregate Base	Top & Bottom	Variable	
	12 (min.)	20'-0"	Roadway	Prestressed Posttensioned	3'-0"	1'-6"	No	Asphalt	Type 5 Aggregate Base		Variable	

¹ (TDOT 2011), (AHTD 2014), (ALDOT 2017), (CDOT 2015), (IDOT 2017), (KYTC 2015), (ODOT 2015), (TxDOT 2015), (MoDOT 2017)