# Applications in Vehicle-Bridge Interaction

The simulation tools demonstrated in Part 1 and developed in Part 2 are leveraged in this part to address some of the challenges facing bridges associated with VBI.

Parts 1 and 2 both demonstrated the influence of roadway profile roughness on bridge response and dynamic amplification. In Part 2 it was shown that IRI has a strong (but inconsistent) correlation with dynamic amplification. Chapter 1 once again addresses profile roughness with examination of rolling straightedge criteria. Various straightedge limits are used to modify a real (measured) profile and the effect of these modifications on dynamic amplification is quantified.

Chapter 2 extends vehicle-bridge interaction to include multiple vehicles. While a single vehicle (per lane) is often the load case analyzed for design and evaluation, real world structures are routinely subjected to traffic conditions in which the bridge is subjected to multiple vehicles. The additional response due to multiple vehicles is traditionally ignored in design and evaluation methodologies because of the inherent conservatism associated with the live-load model (i.e. it is unlikely that every lane will be simultaneously occupied with a HL-93 truck). However, the effect of multiple vehicles on bridge dynamics and dynamic amplification has not been addressed.

Therefore, chapter 2 seeks to identify the effect of multiple vehicles on dynamic amplification and provide guidance on how the effect of multiple vehicles should be considered when it is impractical to perform VBI simulation (i.e. static analysis). This is accomplished by first examining the effects of a few traffic patterns and then extended to truck platoons.

## Remediation and Smoothness Criteria

If a bridge is suspected to exhibit large dynamic amplification as a result of a rough roadway, the bridge owner may wish to grind the roadway smooth. Furthermore, to reduce dynamic amplification in new construction, deck profile specifications should provide smoothness targets. Currently, smoothness criteria are prescribed differently for different locations.

The IRI, which is widely used as a smoothness criterion by providing upper limits, has already been shown to influence dynamic amplification. The IRI is a measure of vehicle response and can therefore only be implemented as a performance metric. As such it provides no methods for specifying or monitoring the smoothness during construction or as a result of grinding and will not be presented in this section. However, if a deck profile is shown to have a high IRI, intervention should be considered.

Rolling straightedge requirements are widely used for specifying localized roughness criteria and are expressed in terms of deviation over a certain length. Parameters commonly range from 1/8 to ¼ inch deviation over a 10 to 16-foot distance. A computer algorithm was developed that effectively smooths the profile according the straightedge requirements.

This algorithm steps through the profile, incrementally advancing the reference point and assessing if any point, between the reference point and a point at a distance equal to the straightedge length, exceeds a straight-line fit between the two points by more than the specified deviation. If a point falls outside of the bounds specified by the deviation value, the elevation of the point is reduced such that the deviation equals the specified deviation. This is accomplished in the following steps.

* Linearly interpolate every point within the straightedge window based on the elevation of the first and last point (i.e. create a straight line between the first and last point).
* Compute the vertical deviation that results in a perpendicular (to the interpolated road surface) deviation equal to the specified deviation.
* Find indices of points that exceed the interpolated surface (line) plus the vertical deviation and set equal to the interpolated surface plus the vertical deviation.

The effect of this smoothing is illustrated in the following plots whereby a measured profile (measured during Part 1, Phase 3 testing) is compared to the same profile after applying the smoothing algorithm.



Figure 3.17.1: Effect of Smoothing (1/8" over 10ft.) on Real Profile



Figure 3.17.2: Effect of Smoothing (1/4" over 30ft.) on Real Profile

The ability of straightedge criteria to limit dynamic amplification is assessed by simulating vehicle-bridge interaction with a problematic profile, and again with the profile smoothed according to specified criteria. The FE model of a single 140 ft. span with the 2.5 Hz vehicle model as described in Part 2 was utilized for this study. Simulation parameters are summarized in the following table.

Table 3.17.1: Simulation Parameters for Smoothing Studies

|  |  |  |
| --- | --- | --- |
| Number of modes included | 15 |  |
| Incremental distance along load-path | 6 | inches |
| Structural damping | 1% |  |
| Vehicle speed | 720 | in/sec |
| Solution time-step | 0.0015 | sec |

The following plot compares bridge midspan displacement amplification for the previously plotted profile with smoothing applied according to 1/8th inch over 10 feet. Amplification was computed according to equation (89, for which the static displacement was taken as the maximum quasi-static (i.e. 5 in/sec) displacement.

Figure 3.17.3: Comparison of Bridge Dynamic Amplification with Rough Profile and with Smoothed Profile

The above plot clearly shows the 1/8th inch over 10 feet criterion is ineffective at limiting dynamic amplification.

The straightedge requirements target features with lengths less than the straightedge length. At normal traffic speeds profile features with lengths of 5-50 feet result in forcing frequencies within the range of natural frequencies commonly exhibited by bridges. Therefore, the straightedge length should be specified long enough to avoid/remove features in this range. For a speed of 720 in/sec, a feature wavelength of 30 feet results in a forcing frequency equal to 2 Hz.

A variety of straightedge criterion were assessed with the 2-DOF state-space model of the 140ft single-span bridge (as described in Part 2, Chapter 4). Straightedge smoothing of the same profile was performed with maximum specified deviation ranging from 1/8th inch to ½ inch and straightedge length ranging from 10 to 50 feet. The results are plotted below.

Figure 3.17.4: Effect of Smoothing Parameters on Dynamic Amplification

The preceding plot demonstrates the ability of straightedge requirements to limit dynamic amplification. Criteria that specify deviation limits greater than ¼” are ineffective at limiting amplification. Even criteria with ¼” deviation limits were only successful at reducing the amplification to 1.33 when the straightedge length was greater than 27 feet. Based on these findings it is recommended that straightedge criteria specify a straightedge length of at least 16 feet for a deviation of 1/8th inch, and a straightedge length of at least 30 feet for a deviation of ¼ inch.

### Conclusions

Rolling straightedge criteria with common limits (i.e. 1/8 to ¼ inch deviation over 10 to 16 feet) are not effective at reducing dynamic amplification. This is because (1) deviations even as small as ¼ of an inch (0.635 cm) can significantly influence bridge response, and (2) short straightedge lengths fail to remove features with large wavelengths that still have appreciable effect on bridge response. The shorter the straightedge length, the more stringent the deviation limit must be. Based on these simulations, if this criterion is to continue being used, the straightedge length should be at least 16 feet (4.88 m) for a deviation limit of 1/8th inch (0.318 cm), and at least 30 feet (9.14 m) for a deviation limit of ¼ inch (0.635 cm).

## Multiple Vehicles

The many simulations that have thus far been reported, have considered only a single vehicle traversing a bridge. In reality, bridges experience a large variety of different live-load configurations and many times are subjected to multiple vehicles at the same time. This is investigated in the following sections.

### Traffic

The scenario of multiple-vehicle loading is accounted for in most design methodologies through the use of multi-presence factors. These factors serve to reduce the load presented by vehicles in adjacent lanes based on the assumption that vehicles with legal-limit weights are unlikely to occupy adjacent lanes at the same time. While this assumption may be valid for static bridge responses, other vehicles may contribute to the dynamic response. The effect of other vehicles (traffic) on a bridge’s dynamic response and the dynamic amplification of a major load event are investigated.

#### Methods

Several traffic patterns were created with randomly distributed vehicles and varying density (i.e. vehicle spacing). Each pattern sampled 8 different vehicles that included HS20 trucks, a dump truck, a tractor-trailer, and small passenger vehicles. The vehicles, their axle configurations and suspension parameters are provided below.

Table 3.18.1: Vehicle Parameters for Traffic Simulations

|  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- |
| Vehicle # | Axle # | Name | Distance from rear axle (ft) | Weight (kip) | Spring Stiffness (kip/in) | Damping Coefficient (lb-s/in) | Damping Ratio | Natural Frequency |
| 1 | 1 | HS20-32\_1 | 0 | 32 | 20 | 257.529 | 0.1 | 2.472 |
| 1 | HS20-32\_1 | 14 | 32 | 20 | 257.529 | 0.1 | 2.472 |
| 2 | HS20-8\_1 | 28 | 8 | 15 | 111.513 | 0.1 | 4.282 |
| 2 | 3 | HS20-32\_2 | 0 | 32 | 16 | 460.682 | 0.2 | 2.211 |
| 3 | HS20-32\_2 | 30 | 32 | 16 | 460.682 | 0.2 | 2.211 |
| 4 | HS20-8\_2 | 44 | 8 | 8 | 407.189 | 0.5 | 3.127 |
| 3 | 5 | HS20-32\_3 | 0 | 32 | 13 | 415.253 | 0.2 | 1.993 |
| 5 | HS20-32\_3 | 22 | 32 | 13 | 415.253 | 0.2 | 1.993 |
| 6 | HS20-8\_3 | 36 | 8 | 10 | 455.251 | 0.5 | 3.496 |
| 4 | 7 | tst-tand | 0 | 17 | 15 | 162.557 | 0.1 | 2.937 |
| 7 | tst-tand | 6 | 17 | 15 | 162.557 | 0.1 | 2.937 |
| 8 | tst-drive | 29 | 17 | 15 | 162.557 | 0.1 | 2.937 |
| 8 | tst-drive | 35 | 17 | 15 | 162.557 | 0.1 | 2.937 |
| 9 | tst-front | 51 | 12 | 2.2 | 261.522 | 0.5 | 1.339 |
| 5 | 10 | dump-rear | 0 | 25 | 12 | 176.318 | 0.1 | 2.166 |
| 10 | dump-rear | 5 | 25 | 12 | 176.318 | 0.1 | 2.166 |
| 11 | dump-front | 20 | 20 | 12 | 315.407 | 0.2 | 2.422 |
| 6 | 12 | car1 | 0 | 1.5 | 0.2 | 27.878 | 0.5 | 1.142 |
| 12 | car1 | 8 | 1.5 | 0.2 | 27.878 | 0.5 | 1.142 |
| 7 | 13 | car2 | 0 | 2 | 0.4 | 45.525 | 0.5 | 1.398 |
| 13 | car2 | 10 | 2 | 0.4 | 45.525 | 0.5 | 1.398 |
| 8 | 14 | car3 | 0 | 3 | 2 | 124.676 | 0.5 | 2.553 |
| 14 | car3 | 14 | 3 | 2 | 124.676 | 0.5 | 2.553 |

Six traffic patterns were created with the following parameters. Each traffic pattern concluded with the dump truck (vehicle 5). Every other vehicle in a traffic pattern was randomly selected from the vehicle list and the spacing was randomly (uniform) sampled between the bounds specified in the table.

Table 3.18.2: Traffic Pattern Parameters

|  |  |  |  |
| --- | --- | --- | --- |
|  | Num. of Vehicles | Min. Spacing | Max. Spacing |
| 1 | 36 | 20 ft. | 100 ft. |
| 2 | 36 | 40 ft. | 200 ft. |
| 3 | 36 | 60 ft. | 300 ft. |
| 4 | 36 | 80 ft. | 400 ft. |
| 5 | 36 | 100 ft. | 500 ft. |
| 6 | 36 | 280 ft. | 1400 ft. |

The 3D FE model of the 2-span bridge with span lengths of 140 ft (Part 2, Chapter 4[\_140ft.\_Bridge](#_140ft._Bridge)) was used for the simulations. Each traffic pattern was simulated at a speed of 960 in/sec (≈55 mph) as well as at 5 in/sec to provide the quasi-static response. A real profile (as measured in Part 1) was included in the simulations. The first 15 modes of vibration were included in the simulation. Simulation time-steps of 0.002 seconds and 0.5 seconds were used for the 960 in/sec and quasi-static simulations, respectively.

#### Results

The bridge responses acquired from the simulations of the traffic patterns are summarized in the following table.

Table 3.18.3: Maximum Responses from Traffic Simulations

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
|  | Max. Static Disp. (in.) | | Max. Dynamic Disp. (in.) | | Max. Amplification | |
|  | Span 1 | Span 2 | Span 1 | Span 2 | Span 1 | Span 2 |
| Traffic1 | **-0.41113** | **-0.41906** | **-0.588347** | -0.48383 | 1.431056 | 1.154555 |
| Traffic2 | -0.32891 | -0.32505 | -0.369981 | **-0.54378** | 1.12486 | **1.672918** |
| Traffic3 | -0.26793 | -0.26538 | -0.407771 | -0.38494 | 1.521942 | 1.450507 |
| Traffic4 | -0.26009 | -0.25963 | -0.438268 | -0.40802 | **1.685082** | 1.571517 |
| Traffic5 | -0.26012 | -0.25962 | -0.380595 | -0.40768 | 1.463129 | 1.570283 |
| Traffic6 | -0.25986 | -0.25962 | -0.356183 | -0.37011 | 1.370688 | 1.425589 |

Except in a few cases, large dynamic amplifications were observed for the traffic patterns. The time history corresponding to the events that produced the largest dynamic response are plotted below along with the quasi-static response.

Figure 3.18.1: Span-1 response to traffic pattern no. 1

Figure3. 18.2: Span-2 response to traffic pattern no. 2

The vehicles that are inducing the response shown in the above plots are vehicle numbers 3 and 5 (HS-20 and dump truck), respectively. The responses that these vehicles alone produce are provided below for comparison.

Table 3.18.4: Single Vehicle Responses for Vehicles 3 and 5

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
|  | Max. Static Disp. (in.) | | Max. Dynamic Disp. (in.) | | Max. Amplification | |
|  | Span 1 | Span 2 | Span 1 | Span 2 | Span 1 | Span 2 |
| Vehicle3 | -0.251 | -0.252 | -0.304 | -0.320 | 1.21 | 1.27 |
| Vehicle5 | -0.245 | -0.245 | -0.336 | -0.337 | 1.37 | 1.38 |

All but two of the amplifications induced by the traffic patterns presented in Table 3.18.3meet or exceed the amplification produced by a single vehicle. It can therefore be concluded that multiple vehicle loading serves to not only increase the static bridge response but may also result in dynamic amplification even greater than that which would be observed for a single vehicle.

The response of span 1 to the final vehicle (dump truck) is compared in the following plot and serves to illustrate the effect that bridge initial conditions (preexisting motion) has on bridge response to a vehicle crossing.

Figure 3.18.3: Time History of Bridge Displacement for Last Vehicle in Traffic Patterns

The (dynamic) bridge response over the duration for which the last vehicle was on the bridge was a maximum for the first span under traffic pattern number 1 for which the final vehicle was preceded by an HS-20 (vehicle 3) with a headway of 20 ft. That for span 2 was a maximum under traffic pattern number 2, for which the final vehicle was preceded by a tractor-trailer (vehicle 4) with a headway of 40 ft. Those responses are plotted below.

Figure 3.18.4: Span-1 Response to final vehicle (#5) in traffic pattern 1

Figure 3.18.5: Span-2 Response to final vehicle (#5) in traffic pattern 2

The remaining traffic patterns failed to produce greater responses than if the final vehicle had no preceding vehicles. Incidentally, for all these remaining traffic patterns, the final vehicle was preceded by either a dump truck (vehicle 5) or HS-20 (vehicle 3) and at a distance not less than 60ft. The maximum responses for this final loading event for all traffic patterns are summarized in the following table.

Table 3.18.5: Maximum Responses for Final Vehicle Loading Event

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
|  | Max. Dynamic Disp. (in.) | | Dynamic Amplification | | Preceding Vehicle | Distance (ft.) |
|  | Span 1 | Span 2 | Span 1 | Span 2 |
| Traffic1 | **-0.438** | -0.484 | 1.31 | 1.15 | 3 | 20 |
| Traffic2 | -0.370 | **-0.544** | 1.12 | **1.67** | 4 | 40 |
| Traffic3 | -0.348 | -0.350 | 1.30 | 1.32 | 3 | 60 |
| Traffic4 | -0.341 | -0.301 | 1.39 | 1.23 | 5 | 80 |
| Traffic5 | -0.360 | -0.326 | **1.77** | 1.33 | 5 | 100 |
| Traffic6 | -0.351 | -0.344 | 1.43 | 1.41 | 3 | 280 |
| Vehicle5 | -0.336 | -0.337 | 1.37 | 1.38 | - | - |

The results provided in the previous table further demonstrate that traffic serves to increase structural response due to both increased static effects as well as increased dynamic amplification. However, small passenger vehicles have insufficient mass to appreciably excite the structure and thus will not have significant impact on bridge response to truck loads. Therefore, it is repeated truck loading that results in the greatest bridge response. The following section explores this further.

### Platooning

Platooning is when several trucks follow closely with reduced headway supported by control and communications technologies, resulting in a “train” of trucks. Therefore, a truck-train may be described by the following parameters:

1. Number of vehicles
2. Individual vehicle dynamics (mass and suspension properties)
3. Vehicle spacing
4. Speed

To reduce simulation computing requirements, the number of vehicles will first be investigated. We may first bound this parameter by considering that the upper limit for number of vehicles is controlled by the time it takes for bridge motion induced by past trucks to damp out. Therefore, this parameter is dependent on vehicle spacing and speed and bridge length. Vehicle spacing is conservatively bounded at 15 feet for the closest spacing, and vehicle speed is not likely to exceed 1200 in/sec (68 mph).

Therefore, the number of vehicles in a platoon was investigated while holding vehicle spacing to the conservative minimum of 15 feet and platoon speed to the conservative maximum of 1200 in/sec (68 mph), thereby maximizing the energy input to the bridge. Platoons were therefore composed of dump trucks (vehicle 5) spaced at 15 feet. The number of vehicles in each platoon was varied: {1, 2, 4, 6, 8, 10}. Simulations were repeated for a vehicle headway spacing of 150 feet.

Simulations were performed using the 3D FE model of the 2-span bridge with span lengths of 140 ft (Part 2, Chapter 4) and a real profile (measured in Part 1). The first 15 modes of vibration were included in the simulation. A time-step of 0.002 seconds was used.

The following plot displays the effect the number of vehicles has on dynamic bridge response.

Figure 3.18.6: Effect of Number of Sequential Vehicles on Bridge Response

As can be seen in the above plot, for both large and small vehicle spacing, the maximum response is captured with just 4 vehicles and any additional vehicles fail to increase the bridge response or appreciably change the nature of its motion.

The vehicle spacing was further investigated with the single-span 100-ft. FE model and with the single-span 140-ft. FE model; the same profile was used. Four dump trucks (vehicle 5) were arranged with a constant headway spacing. That spacing was varied from 15 feet to 100 feet for the 120 ft. model and from 15 feet to 75 feet for the 100 ft model. Platoons were assigned a speed of 1200 in/sec (110 kmh). The results are summarized in the following plots which illustrate the effect of vehicle spacing on bridge response: amplification and displacement.

Figure 3.18.7: Response summary for 140ft single-span model

Figure 3.18.8: Response summary for 100ft single-span model.

Amplification in the above plots was computed by dividing the maximum dynamic displacement by the maximum quasi-static displacement (5 in/sec), thereby accounting for the static effect of multiple vehicles.

As vehicle spacing decreases, more vehicles are able to fit on the bridge, thus increasing the total load experienced by the bridge and increasing the bridge response (both static and dynamic). However, the dynamic amplification associated with this increased load tends to decrease as the vehicle spacing decreases (number of vehicles present on the bridge increases).

The dynamic amplification reaches the level corresponding to just a single vehicle when the spacing is greater than 75% of the bridge length. The dynamic amplification is a maximum when the vehicle spacing is roughly 60% of the bridge length and is greater than the dynamic amplification experienced for a single vehicle. For the single-span simulations, the maximum dynamic amplification was 12% greater (than the dynamic amplification for a single vehicle) for a headway spacing of 60 feet for the 100-foot span length and 18% greater for a headway spacing of 80 feet for the 140-foot span length.

If a bridge is to be designed for closer vehicle spacing (less than 35% of bridge length) and the additional load is considered in static analyses, the dynamic amplification factor computed for a single vehicle will suffice.

The simulations were repeated with the two-span continuous model with 140-foot span lengths (Part 2, Chapter 4). The headway spacing was varied from 20 feet to 500 feet. Platoons were assigned a speed of 1200 in/sec. The results are summarized in the following plots which illustrate the effect of vehicle spacing on bridge response: amplification and displacement.

Figure 3.18.9: Response Summary for 140ft 2-Span Model (Span 1)

Figure 3.18.10: Response Summary for 140ft 2-Span Model (Span 2)

The results presented in the previous two plots illustrate the wide variation in dynamic response and dynamic amplification. Depending on the headway spacing, the contact force from the multiple vehicles may result in constructive interference, increasing the dynamic response, or destructive interference, reducing dynamic response. Furthermore, when headway spacing increases to the point that only a single vehicle is ever present on the bridge at one time, the same constructive or destructive interference may occur depending on the phase of bridge oscillation when the vehicle enters the bridge, which is dependent on vehicle spacing and the profile. The interference is demonstrated in the following plot which compares the responses of the second span for 140 ft. and 160 ft. headway spacing.

Figure 3.18.11: Span 2 Displacement Time History for Varied Platoon Headway Spacing

The inconsistent sensitivity of the 2-span model to platoon headway spacing prevents a clear relationship between headway spacing and dynamic amplification from being established. However, the simulation results still demonstrate that as headway spacing decreases to the point that the static load effects exceed those from a single vehicle (i.e. multiple vehicles are simultaneously present), the dynamic amplification associated with this increased load tends to decrease. The results also indicate that the bridge motion induced by a previous vehicle may significantly increase the dynamic amplification. For the two-span simulations, the maximum dynamic amplification was 34% greater (than the dynamic amplification for a single vehicle) for a headway spacing of 300 feet for span 1 and 18% greater for a headway spacing of 160 feet for span 2.

### Conclusions

Additional vehicles present additional loads to the bridge and ultimately increase bridge response. The response will exhibit dynamic amplification which may exceed that experienced for a single vehicle when a vehicle enters the bridge while it is still in motion from the previous vehicle crossing. But vehicles also present damping sources and will generally serve to reduce bridge vibration and thus dynamic amplification as they become more numerous on the bridge (simultaneously).

This is echoed by simulations of truck platoons, which suggest that platooned vehicles present no greater risk to increased bridge dynamics than posed by other traffic conditions because it only takes a single previous truck to induce the bridge conditions that result in maximum dynamic amplification.

Similarly, the amplification experienced when multiple lanes are occupied would be less than if only a single vehicle is present. Therefore, the conservatism introduced by considering loading in multiple lanes is further increased if dynamic amplification for a single vehicle is implemented. It should be stressed that even though multiple vehicles, when present on the bridge simultaneously, serve to reduce the level of dynamic amplification, it is still present and should be considered in analysis.

If the total response of the bridge is to be minimized, spacing between sequential trucks should be such that the bridge has time to settle down. When performing analysis for platoons with very small headway spacing, the effect of the additional simultaneous vehicles should be included in a static analysis with an amplification factor as predicted for a single vehicle.

Estimations of dynamic amplification for a single vehicle event should consider the possibility that the bridge may already have been excited by a previous vehicle. However, the additional amplification experienced by the bridge in this scenario is not likely to be more than 35% greater than that predicted for a single vehicle.

# Part 3 Conclusions and Future Work

### Conclusions

A case structure was extensively investigated through field testing and simulation. The results from field testing were used to validate FE models that were subsequently used to simulate VBI. These investigations yielded the following conclusions:

* A validated 3D FE model is capable of simulating bridge-vehicle interaction if roadway profile is included and accurately positioned on the model.
* The observed vibrations may be principally attributed to the bridge deck roadway profile. The vehicle is excited by the profile, which in-turn excites the bridge.
* The high dynamic amplification does not seem to pose a risk to the structure’s performance (at strength or service limit states).
* The bridge exhibits dynamic amplification in excess of 1.75 even under heavy vehicles (compared to AASHTO recommended 1.33).

The VBI simulation methods validated in Part 1 were used to develop and validate a simplified VBI model that reduced the bridge to a single degree of freedom. This model was leveraged to investigate the role of influential parameters on VBI from which the following conclusions may be gleaned:

* Amplification factors determined from displacement measurements will be greater (more conservative) than those determined from strain measurements.
* Ignoring the interaction of vehicle and bridge (i.e. computing contact force without consideration of the bridge) results in conservative estimates of bridge response.
* Bridge responses are greatest when the profile induces oscillation in the vehicle close to the bridge’s natural frequency and when the vehicle’s natural frequency is 10-20% greater than that of the bridge.
* Position of the profile and phase angle distribution of harmonic components have significant effect on bridge response.
* Current roughness metrics (e.g. IRI and ISO 8608) cannot reliably predict dynamic amplification.

The simulation tools developed in Part 2 were further leveraged through a series of parametric studies to generate recommendations related to roadway smoothness criteria and repeated vehicle loading (truck platooning). Key findings include:

* Rolling straightedge criteria with common limits (i.e. 1/8 to ¼ inch deviation over 10 to 16 feet) are not effective at reducing dynamic amplification. Rolling-straightedge length should be no less than 16 feet for a specified deviation of 1/8th inch, and no less than 30 feet for a specified deviation of ¼ inch.
* Traffic and truck platoons can result in increased dynamic amplification because even a single previous truck can induce the bridge conditions (motion) that result in increased dynamic response (≈20%).
* As spacing between vehicles decreases and more vehicles are present on the bridge, the static load effect increases, but the dynamic amplification will likely be less than what would occur for a single vehicle.
* When designing for truck platoons, if static analysis accounts for multiple simultaneous vehicles, the dynamic amplification factor for a single vehicle can be used to conservatively account for the total dynamic response.
* Dynamic amplification for simultaneous loading of multiple lanes can be conservatively estimated as that for a single vehicle.

### Future Work

The studies presented in this part are, by no means, exhaustive. Bridge type and geometry, vehicle characteristics and speed, and profile were selected to provide a reasonable upper bound for responses and dynamic amplification. Therefore, these results should be corroborated by larger scale studies that include a greater range of bridge types and parameter values. Furthermore, this thesis considered only harmonic profiles and a real profile from a single bridge and the bridge responses at midspan. Therefore, the following are suggestions for the continuation of this work.

* Perform simulations with a greater sample of real profiles as well as profiles with a bump at the beginning of the bridge and profiles with ramped approaches.
* Identify construction practices that produce harmonic profile content (e.g. slipform paving machines).
* Evaluate the influence of VBI parameters on the dynamic amplification of responses at other locations and the dynamic amplification of shear.
* Perform simulations with a greater variety of vehicle parameters, preferably based on vehicle suspension population statistics.

The investigation of smoothness considered only rolling-straightedge criteria. Additional criteria should be developed and assessed that are appropriate for actual mitigation strategies (i.e. grinding).

Finally, the simplified models presented in Part 2 included only a single mode of vibration/deformation. The accuracy of these models, especially the 2-span model, could be improved by including more modes.