Seismic performance of integral abutment bridges in liquefiable soils

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ABSTRACT

Integral abutment bridges are cost-effective solutions for short to medium span bridges because they are less expensive to construct and maintain, resulting in sustainable bridges. However, during seismic events, the integral abutments and foundations must accommodate the large horizontal movements caused by ground motions through complex soil-structural interactions. Past earthquakes have indicated that integral abutment bridges are more resilient to large earthquakes than traditional abutment bridges. Despite the favorable seismic performance of integral abutment bridges observed during past earthquakes, its application in liquefiable soils is still restricted by bridge design codes, such as the Canadian Highway Bridge Design Code, CSA-S6-14, as its seismic performance is presumed to be adversely affected by the ground deformation induced by liquefaction. This paper will review the performance of integral abutment bridges in the past earthquakes and its seismic design requirements of various international bridge design codes, as well as performing case studies of a typical integral abutment bridge subjected to various magnitudes of lateral spreading induced by earthquakes. In the case studies, seismic performance of steel piles employed in the integral abutment bridge is assessed using pushover analyses, which utilize a set of non-linear soil springs to capture the soil-structural interactions between piles/abutment diaphragm and surrounding soils. Some practical limits on the extent/magnitude of ground deformation caused by liquefaction are discussed in the context of integral abutment bridges.

Keywords: Integral abutment bridges, liquefaction, seismic designs, earthquakes, soil-structure interactions.

INTRODUCTION

Integral abutment bridges are widely used in North America and around the world. They are considered cost-effective structures for short to medium span bridges as their construction and maintenance costs are normally less than non-integral abutment bridges. A survey conducted in the US concluded that use of integral abutments almost always results in lower bridge maintenance cost. In most of states of the US, construction cost of integral abutment bridges is also lower than that of conventional bridges [1]. Due to these reasons, integral abutment bridges became a very common type of bridges. In eight states of the US, the number of integral abutment bridges in service in each state exceeds 1000.

Design of integral abutment bridges requires consideration of complex soil-structural interactions between abutment backfills and diaphragm. For example, the abutment backfills of an integral abutment bridge are in direct contact with the abutment diaphragm. As such, soil pressures acting on the abutment diaphragm will inevitably restrain any thermal expansion movement of the bridge, causing additional axial loading for the superstructure. This adverse effect would have to be considered in the design. However, for earthquake loadings, the interaction between backfills and abutment diaphragm would be beneficial to the design as it would provide significant resistance to seismic loads [2]. Recent analytical investigations performed by FHWA [3, 4] provided further evidence to support the field observations that integral abutment bridges perform better than non-integral abutment bridges in terms of overall displacements and column forces. These investigations have identified some critical geotechnical and pile parameters that may influence the performance of integral abutment bridges. For example, densely-compacted abutment backfills tend to reduce the pile deflection, the abutment displacement, and pile bending demands. However, most of these studies have focused on the abutment-soil interactions under inertia loads but not specifically on lateral spreading loads that could impose large kinematic loads on the piles. In such situations, piles can become vulnerable to lateral spreading loads as relatively slender piles are usually used in integral abutment bridges to provide flexibility to accommodate thermal movements. Lateral spreading mostly occurs when liquefiable soils are present. However, lateral deformations could also occur in non-liquefiable sites due to the inertial seismic loads acting on sloping ground. This study investigates the use of integral abutment bridges in areas that are subjected to small to moderate lateral spreading and discusses key design paymasters that could restrict the applicability of integral bridges under the liquefiable condition.

SEISMIC PERFORMANCE OF INTEGRAL ABUTMENT BRIDGES

Based on the past bridge performance during earthquakes, integral abutment bridges are considered among the most resilient bridges. For example, in the 1989 Loma Prieta and 1994 Northridge earthquakes, Wood [5] reported that there was less damage to bridges with integral abutments than for bridges with structural separations at the abutments. In New Zealand, during several earthquakes including 2010 Darfield, 2011 Christchurch, 2013 Cook Strait and 2013 Lake Grassmere earthquakes, it was found that integral abutment bridges were only damaged slightly [5]. The observed damages included cracking near the top of concrete piles, cracking in the abutment walls, and flexural failure in abutment walls due to lateral spreading triggered by soil liquefaction [5]. The main cause of bridge damage in the 2010 and 2011 Christchurch earthquakes was the liquefaction-induced lateral spreading of the approaches adjacent to abutments. For instance, integral abutment bridges at Gayhurst Road and Swanns Road were extensively damaged by lateral spreading [6]. The bridges were constructed in 1952 and consisted of reinforced concrete portals with integral abutments founded on driven concrete piles. The abutments experienced significant rotations, the piles formed plastic hinges and the superstructure to abutment knee joints cracked significantly. It was believed that aftershocks occurred concurrently with lateral spreading at the bridge sites. However, investigators concluded that integral abutments have helped to limit rotation due to lateral spreading as well as ground settlement.

Ní Choineet et al. [7] conducted a probabilistic fragility analysis to compare the seismic performance of integral abutment bridges and non-integral abutment bridges. It was found that integral abutment bridges performed consistently better. It was also found that seismic vulnerability increases with an increase in bridge length for integral abutment bridges. Itani and Pekcan [8] performed case studies of steel plate girder bridges with integral abutments. It was suggested that the piles can sustain high ductility demand as long as the piles have sufficient embedment length into the abutment. This is mainly due to the fact that the piles are confined by the soils. Based on the poor performance observed in closely spaced slender piles, Waldinet et al. [6] recommended using discrete, stiff and robust piles in bridges located in areas susceptible to liquefaction. They also recommended spill-through abutments with shallow walls to limit the kinematic demands from lateral spreading. However, the latter contradicts the recommendation by Wood [5] that high abutment diaphragms are considered advantageous due to the passive resistance provided by the backfills. This observation may be accurate for inertial loading. However, it is unclear if high abutment diaphragms would be beneficial for lateral spreading (in this case soils would act as a load rather as a resistance). Nonetheless, the recent research has also shown that magnitude of the passive resistance provided by the backfills is greatly reduced as the skew angle of the abutment is increased, with damage rates as high as two times of that for a non-skewed bridge [9]. Although the main objective of this paper is not to address this issue, it should be recognized that the abutment diaphragm height plays a significant role in resisting lateral spreading and influencing the pile-soil interactions. In addition, Carvajal [10] studied the embankment-abutment-structure interaction of integral abutment bridges and concluded that far-field embankment can affect the response of integral abutment bridges in some conditions.

DESIGN OF INTEGRAL ABUTMENT BRIDGES

Design guidelines and restrictions of integral abutment bridges vary from jurisdictions to jurisdictions. In New Zealand, its Agency [2] stipulates that the span length of integral concrete bridges be less than 70 m and the span length of integral steel bridges be limited to 55 m. Similarly, Finland limits the integral abutment bridge length to 70 m [11]. In the US, the State DOTs had reported that up to 300 m long integral bridges using steel girders and 200 m long integral bridges using concrete girders were designed and constructed. Several US DOTs have limited the horizontal movement ranges of integral abutment [12-14]. In Japan, it is suggested that the maximum height of abutment diaphragm of the integral bridges be approximately 10 m [15]. White [11] mentioned that the UK and Ireland require all bridges less than 60 m in length with a skew angle less than 30° be constructed as integral abutment bridges.

The seismic design of integral abutment bridges requires careful considerations of interactions between soils and substructure element such as abutment diaphragm and piles. The design shall account for inertia loads from bridge superstructure and from soil retained by bridge abutments lateral spreading from soils moving towards low-points such as river channels. Some jurisdictions require the combination of inertial demands and kinematic demands [16, 17]. For example, in the province of British Columbia in Canada, the load combinations include [i] 100% kinematic demands; [ii] 100% inertial demands and [iii] 50% inertial demands + 100% kinematic demands.

Seismic detailing of integral abutment bridges has been extensively researched in the State of Indiana. As per the Indiana DOT abutment details, a minimum abutment pile embedment length of 0.61 m is required [18,19]. A survey taken in 1983 in the US showed that the State DOTs differ in opinion with regard to integral abutment pile orientations. Some States orient the pile weak direction in the bridge longitudinal direction and some States do the opposite. Both methods have proven to be satisfactory [20]. In the Recommended LRFD for the seismic design of highway bridges [20], an example of integral abutment bridge design for liquefaction is presented. It was suggested that allowing some hinging in the pile during relatively large lateral spreading events is necessary. In addition, since spreading deformation are displacement-controlled, instability of the system is unlikely [21]. In most of the design codes, damping is typically assumed to be 5%. However, based on testing of integral abutment bridges, structural damping values much higher than 5% have been reported for densely compacted sand, fine gravel and coarse gravel, and such higher values are attributed to the contribution from the soils to dampen the seismic energy [5, 22]. Consequently, the seismic inertial loads considered for the pile design should decrease considerably in comparison to a non-integral bridge.

CASE STUDY

A case study was performed to investigate the magnitude of soil movements (triggered by liquefaction) that typical integral abutments can accommodate. This section is only focused on the kinematic demand, thus the combined kinematic and inertia demands are not considered. In this case study, the bridge is 87.3 m long and 16.13 m wide with two unequal spans. The superstructure is composed of steel box girders and a cast-in-place concrete deck. The substructure consists of two integral abutments supported on H piles and a pier bent on 1.5 m diameter caissons. 14 HP 310 x 125 piles are used to support each abutment. The pier bent is constructed of 3 concrete columns of 1.2 m diameter. Expansion bearings are used to release superstructure longitudinal movement from the pier bent. Shear keys are used at the pier location to provide transverse seismic load path. An elevation view of the bridge is shown in Figure 1. The cross-section of the superstructure is shown in Figure 2. The abutment pile arrangement is shown in Figure 3. The bridge has a skew angle of 4 degrees. The different orientations of H piles with respect to the bridge longitudinal direction were investigated (i.e., weak and strong axis of H piles).

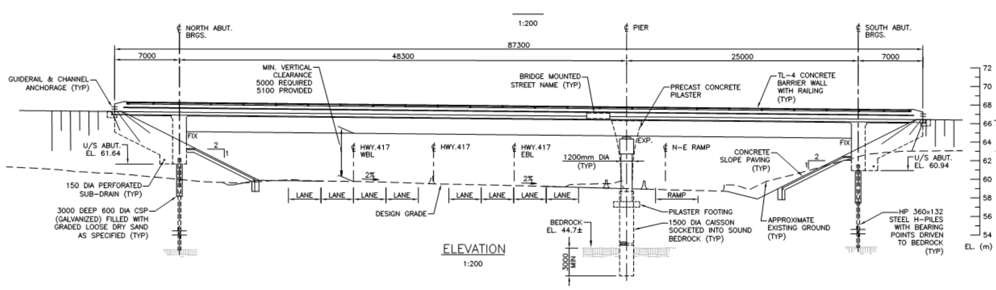
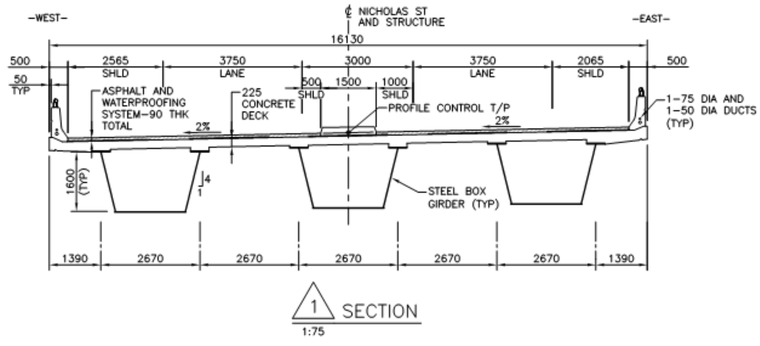


Figure 1. Bridge elevation view

The structural analyses were performed for four different depths of ground deformation (H_1) and two different abutment diaphragm heights (h) as shown in Figure 4 considering different pile orientations. Conservatively, a rectangular soil deformation profile (sliding mass) was considered with a thickness H_1 which is similar to assuming failure occurring at the base of the backfill layer in a weaker native soil layer. Typically, the backfill encountered behind the abutment diaphragm will be well-compacted granular material to modern design standards, therefore abutment springs recommended by Caltrans were used to simulate the soil loads from the abutment diaphragms and soil p-y curves were used to model soil reactions on the H piles. The lateral soil springs were derived using the LPILE [23] program assuming backfill material has a unit weight of 20 kN/m^3 and a friction angle of 38 degrees. The soil unit weight of 19 kN/m^3 and a friction angle of 34 degrees was considered for the soil underlying the backfill. For both soils, the p-y curves were derived using the recommendations given in the American Petroleum Institute (API) for sand. The groundwater level was assumed to be at the base of the backfill layer. If liquefaction occurs, the residual shear strength of liquefiable soils was obtained using the p-multiplier approach as outlined by Ashford et al. [24]. The lateral pile group effects were ignored for simplicity.



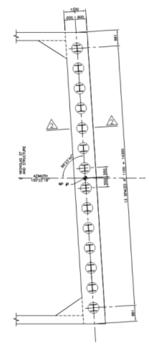


Figure 2. Superstructure cross section

Figure 3. Abutment Pile arrangement

In the structural analysis, nonlinear two-node link elements were used in CSiBridge to represent the soil springs at 1.0 m spacing. Ground deformation was applied to the far side node of the link elements (Figure 5). In the absence of liquefaction, the granular backfill typically reaches the peak resistance/load at about 3 mm to 4 mm of deformation. A screenshot of the bridge model is shown in Figure 6, where the soil movement is toward the positive direction of the x-axis. Lateral spreading load was applied at north abutment (left-hand side, shown in Figure 1), no lateral spreading load was applied to the south abutment (right-hand side).

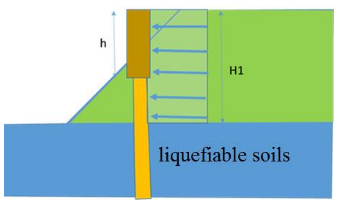


Figure 4. Soil block movement

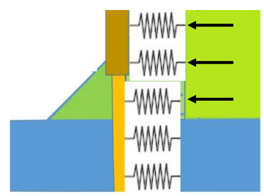


Figure 5. Imposed displacement on soil springs

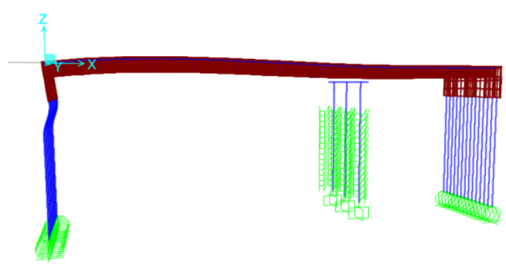


Figure 6. Deformed shape of the bridge and pile foundations under lateral spreading (exaggerated)

A summary of the parameters and analysis results are presented in Tables 1 and 2 for the cases where weak and strong axes of H piles are aligned with the bridge longitudinal direction, respectively. Each case is assigned by a name indicating the relevant parameters. For example, 6H4h represents the case with backfill depth of 6 m and an abutment diaphragm height of 4 m. In all cases, the soil displacements corresponding to different damage states are reported. The damage states are based on the recommendations given in ATC 49 [21] for steel piles. The plastic hinge rotation for the "Life Safety" performance level is limited to 0.035 radians. The plastic hinge rotation for "In-Ground Hinge" performance level is limited to 0.01 radians and for "Immediate Use" is limited to 0.005 radians. A typical bending moment distribution along a pile of integral abutments is shown in Figure 7. With the increase in depth of ground deformation, the pile plastic hinge zone occurs in the location away from the top of the pile.

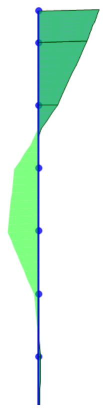


Figure 7. Typical bending moment distribution

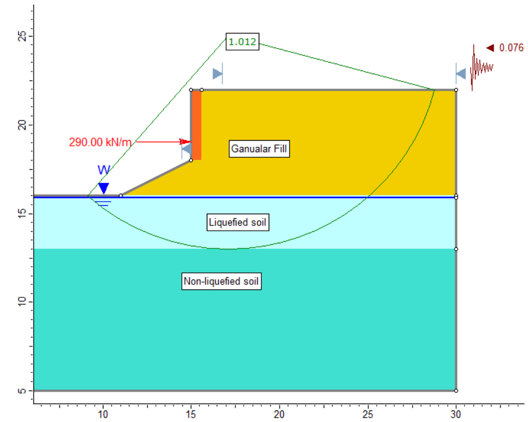


Figure 8. Yield acceleration estimated for the 6H4A

Table 1. Tolerable ground deformation (mm) for pile weak axis aligned in bridge longitudinal direction

Case Name	As per ATC 49	4H4h	6H4h	8H5.5h	10H5.5h
Abutment diaphragm height h, m		4	4	5.5	5.5
Depth of ground deformation H1, m		4	6	8	10
Performance level	Immediate Use	50	60	30	20
	In-Ground Hinge	60	70	40	30
	Life Safety	150	110	80	50

Table 2. Tolerable ground deformation (mm) for pile strong axis aligned in bridge longitudinal direction

Case Name	As per ATC 49	4H4h	6H4h	8H5.5h	10H5.5h
Abutment diaphragm height h, m		4	4	5.5	5.5
Depth of ground deformation H1, m		4	6	8	10
Performance level	Immediate Use	80	80	70	40
	In-Ground Hinge	100	150	80	50
	Life Safety	>>300 (1)	300	110	60

Note (1): Retained soil at the back of abutment diaphragm reaches yielding before the pile reaches the plastic hinge rotation limit for Life Safety performance level.

As one would generally expect, Table 1 and Table 2 show that bridge displacement capacity is higher when the H Pile strong axis is aligned in the bridge longitudinal direction. For the Life Safety performance level, the bridge displacement capacity decreases with the increase of ground deformation depths. Not surprisingly, the total kinematic load exerted on the piles would increase as the depth of ground deformation increases. In the case of 4H4h with strong axis aligned in the bridge longitudinal direction, the backfill material reaches its yield strength before the pile plastic hinge rotation reaches its limit of 0.035 radians. In other words, no additional kinematic load to the piles would be expected as soon as the backfill material reaches its yield strength, i.e., backfill material would flow freely around the piles.

The results shown in Table 1 & 2 indicate that piles can accommodate only a small magnitude of ground deformation before exceeding the prescribed plastic hinge rotation limits for Immediate Use and In-Ground Hinge (in some cases, e.g. 8H5.5h & 10H5.5h) performance levels. In general, any ground deformation of less than about 50 mm could not be reliably estimated using current geotechnical assessment approaches. In other words, integral abutment bridges with more stringent seismic performance requirements (e.g., Immediate Use) may not be feasible in areas where finite yet minor ground deformation expected due to liquefaction or cyclic softening of the foundation soil. However, bridges with less stringent performance requirements (such as “Other” bridges as per the CSA-S6-14 [6]) may be considered feasible, provided that detailed assessments are performed to confirm the viability of the bridge structure. Clearly, generalization of these results is not possible due to differences in soil types, abutment configurations, liquefiable layer thicknesses, pile types and sizes. Despite, this limitation, an attempt was made to back-calculate the seismic hazard levels (represented by PGA) that corresponds to the ground deformation levels given in Tables 1 and 2 for different seismic performance levels and substructure configurations. For this purpose, the ground deformation listed in Table 1 and 2 were considered as the average or design ground deformation, although we recognize that the actual ground deformation may vary between 0.5 to 2 times the design ground deformation due to the inherent uncertainties associated with ground deformation prediction methods. The results are expected to be somewhat conservative due to the block failure mechanism considered for the ground deformation profile compared to a triangular or trapezoidal ground deformation profile.

The magnitude of design lateral spreading displacement and pile demand can also be reduced by considering the pile pinning and abutment/deck restraining effects [25]. Conservatively, the restraining force from pile pinning was not considered in this exercise - the contribution may also be small due to the relatively smaller displacement. The passive earth pressure contribution from the bridge deck/abutment was considered, for which the mobilized passive resistance was determined by interpolating between at-rest conditions at zero abutment diaphragm displacement and full passive resistance mobilized at a displacement of 0.1 times height of the abutment wall [26]. For simplicity, the passive resistance was determined using the Rankine's approach, although the limitations of this approach have been highlighted by others (e.g., [27]).

The ground displacement prediction equation proposed by Jibson [28] was considered. This is a slightly modified version of the relationship originally proposed by Ambraseys and Menu [29] based on Newmark [30] sliding block approach. In this method, the ground displacement is directly related to a seismic hazard parameter (i.e., PGA) that can be readily obtained for a site. However, designers shall consider the epistemic uncertainty associated with ground displacement prediction equations, especially since other ground displacement prediction parameters (e.g., Peak Ground Velocity, Arias Intensity, Spectral Acceleration at a degraded period) have been identified as better indicators of ground displacement than using only PGA [31]. Using this method, the ground displacement (D) in centimeters is estimated using the following:

$$\log D = -2.71 + \log\left[\left(1 - \frac{k_y}{k_{max}}\right)^{2.335}\left(\frac{k_y}{k_{max}}\right)^{-1.478}\right] + 0.424\,\boldsymbol{M} \mp 0.454 \quad (\text{for } 5.3 \le \boldsymbol{M} \le 7.6) \qquad (1)$$

where k_{max} is the peak acceleration coefficient, generally equal to the ground amplification factor times PGA. $\boldsymbol{M}$ is the moment magnitude. For this study, a ground amplification factor of 1.2 and $\boldsymbol{M}$ of 7.0 were considered. The yield acceleration coefficient (k_y) was estimated from slope stability analyses (Figure 8). A fill slope angle of 2H:1V was considered. For this screening-level assessment, 3 m thick liquefiable soil layer below the backfill layer was considered with a residual shear strength ratio of 0.1 (subject to a minimum 15 kPa residual shear strength at shallow depths). Using the above assumptions and methods, PGA levels that satisfy the seismic performance requirements are summarized in Table 3.

In Table 3, certain abutment configurations and backfill thicknesses were not found to be feasible if potentially liquefiable layers exist. The back-calculated PGA values were low (less than 0.1g) and such values are generally not sufficient to trigger liquefaction (although seismic inertial forces are still sufficient to cause ground displacements). If the thickness of the sliding mass is less, substructure is likely to be capable of supporting the lateral spreading loads.

Table 3. Maximum Peak Ground Acceleration values that satisfy the seismic performance requirement

H-pile Orientation	Seismic Performance Level	4H4h	6H4h	8H5.5h	10H5.5h
Along weak axis	Immediate Use	Not feasible			
	In-Ground Hinge	Not feasible			
	Life Safety	0.28	Not feasible		
Along strong axis	Immediate Use	0.21	Not feasible		
	In-Ground Hinge	0.23	Not feasible		
	Life Safety	0.35>	0.25	Not feasible	

In Table 3, certain abutment configurations and backfill thicknesses were not found to be feasible if potentially liquefiable layers exist. The back-calculated PGA values were low (less than 0.1g) and such values are generally not sufficient to trigger liquefaction (although seismic inertial forces are still sufficient to cause ground displacements). If the thickness of the sliding mass is less, substructure is likely to be capable of supporting the lateral spreading loads.

Provided that general bridge configuration is similar to the case study and soil conditions considered in this paper, PGA values given in Table 3 can be considered for a high-level screening assessment to decide the feasibility of the integral abutment bridge option. As highlighted earlier, several assumptions were made in the above computation in relation to liquefaction, residual shear strength, contribution from piles and bridge deck/abutments, soil layering and properties, thus clearly limiting the direct use of these results to all integral abutment bridges in liquefiable soils. Nonetheless, the results demonstrate that certain integral abutment bridge configurations are feasible even in liquefiable soils.

The maximum allowed lateral spreading displacement will depend on the seismic performance category (Immediate Use, In-Ground Hinge, Life Safety). After a plastic hinge is formed, the surrounding soil provides confinement, inhibits P-Δ effects, and allows the piles to continue carrying axial load. As pointed out by Arduinoet al. [32] who cited a few examples, piles can even form two plastic hinges and both hinges can be degraded to pins and still support the bridge. Thus, H piles with adequate ductility would be a feasible solution for integral abutment bridges in liquefiable soils. In AASHTO Guide Specifications for LRFD Seismic Bridge Design [33], steel structural members are classified to essentially Elastic Components and Ductile Members based on the width-thickness ratios. Based on the AASHTO criteria, the slenderness limit (λ) is 13.4 for essentially elastic components and 10.8 for ductile members. For the pile studied in this paper, HP 310 x 125 has slenderness of 8.9, which would be considered a ductile member. This is consistent with the CSA-S6-14 [26] classification.

CONCLUSIONS

Although seismic design of integral abutment bridges is currently restricted to non-liquefiable sites as per CSA-S6-14, the limited case study presented in this paper demonstrates that integral abutment bridges if designed properly (using appropriate diaphragm configuration, adequate pile size and proper pile orientation) can achieve the prescribed performance criteria. A ground displacement in the order of 50 mm or less cannot be reliably estimated from a geotechnical analysis. With certain slope and abutment configurations requiring such low displacement values to meet more stringent seismic performance requirements, integral abutment bridges are deemed not feasible. On the contrary, if the depth of sliding soil mass is shallow, integral abutments may be feasible in moderate seismic areas with liquefiable soils and PGA values greater than 0.2g. The feasibility also depends on the required seismic performance, particularly the Life Safety requirements can be satisfied with relative ease. Although the results presented in this paper cannot be generalized due to number of assumptions made in the analysis, the results demonstrate that existence of potential minor liquefiable soils may not preclude the use of integral abutment bridges depending on the intended seismic performance requirements and thickness of the sliding mass. Therefore, a refined structural and cost analysis may prove that the integral abutment bridge may still the preferred option in these situations.

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