

Co-Seismic Slope Displacements; A Comparative Case Study

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ABSTRACT

Estimation of co-seismic slope displacements remains an area of discussion and uncertainty in geotechnical design. While there are several available methods, with varying applicability to site conditions and design scenarios, it is relatively commonplace within New Zealand engineering practice for the same three methods to be used for legacy reasons, namely the rigid block methods described by Ambraseys & Srbulov (1995), Jibson (2007), and Bray & Travasarou (2007). The objective of this paper is to discuss the importance of applying the right analysis method for the site conditions. An assessment of the most appropriate co-seismic displacement method at three bridge sites was carried out, with a different method ultimately used for each. At Site 1 a simplified Newmark-type displacement method was deemed appropriate. For Site 2, a coupled analysis using selected time history records was used due to the slope height and depth of predicted failure surfaces from pseudo-static analyses. For Site 3, Newmark displacement methods were not considered appropriate due to the presence of deep liquefiable deposits. As a result, finite element analyses were used to better estimate potential seismic displacements.

Keywords: seismic, displacements, slope stability

1 INTRODUCTION

Estimation of co-seismic slope displacements for geotechnical design presents a challenge, with upwards of ten available methods of calculation. These methods have varying degrees of conservatism and applicability to site conditions. Careful consideration is needed to determine the most appropriate method to provide realistic results. It is commonplace in New Zealand engineering practice for simplified Newmark sliding block methods to be used, and this often consists of the same three. These are the rigid block methods described by Ambraseys & Srbulov (1995), Jibson (2007), and Bray & Travasarou (2007). These methods have a narrow range of applicability, typically limited to shallow surficial slides along a well defined failure plane. Discussion of the applicability of these methods has been presented in previous literature (Bray, 2007) and this case study is intended to serve as a comparative discussion only. The cut-off values that were considered for rigid methods are therefore summarised as follows:

Rathje & Bray (1999;2000) present the ratio of T_s/T_m as an indicator of whether a rigid block approach is applicable, where T_s is the fundamental site period, calculated according to Eq. 1 for a one dimensional failure, and T_m is the average period of the ground motion.

$$T_s = 4H/V_s \quad (1)$$

where H = the average height of the sliding mass and V_s = the weighted average shear wave velocity of the sliding mass.

Rigid block methods are indicated to be applicable where $T_s/T_m \leq 0.1$.

The Bray & Travasarou (2007) method presents a modification for a rigid sliding mass which is applicable for $T_s \leq 0.05s$.

It should also be noted that more recent methods have been published by Bray & Macedo (2018) and Bray et al. (2019), which provide less conservative results than the Bray & Travasarou (2007) equations.

As this case study is intended to provide a comparative assessment, the examples provided will focus on the Damage Control Limit State (DCLS) design scenario only. Project details are subject to confidentiality, therefore a simplified description of the site setting and geology is utilised.

2 SITE SETTING

2.1 General Geological Setting

All three sites are located in close proximity to each other and are within a volcanic region of the North Island of New Zealand, with bedrock deposits comprising Quaternary ignimbrite associated with the Taupo Volcanic Zone. Within lower lying areas, the volcanic deposits are overlain by a highly variable unit of interbedded weathered ignimbrite, tephra and alluvial sediment. This interbedded material is typically capped by a thick layer of tephra and recent surficial ash deposits at higher elevations, and Holocene alluvial deposits in the low-lying valleys.

Design parameters were derived on a site-by-site basis, and the range of values considered for each stratum is summarised in Table 1 along with the stratigraphic sequence. All modelling was undertaken using undrained parameters where appropriate.

Table 1: Summary of design parameters

Layer Name	Layer ID	ϕ' (°)	c' (kPa)	S_u (kPa)	Section Colour
Recent Alluvium (Cohesive)	RAc	25	1	10 - 27	Light Green
Recent Alluvium (Granular)	RAg	33	0	n/a	Dark Green
Quaternary Airfall	QA	30 - 32	5 - 8	50 - 81	Yellow
Tephra	Tep.	33	5	82	Orange
Interbedded Material	Int.	33 - 34	3 - 5	42 - 43	Pink
Ignimbrite	Ign.	38 - 40	0	n/a	Blue

2.2 Seismic Setting

The seismic design parameters used for analysis were based on a site-specific seismic hazard assessment, and are summarised in Table 2.

Table 2: Summary of DCLS seismic design parameters

Design Scenario	Mw	PGA
DCLS	5.9	0.27

3 SITE 1

3.1 Overview of Bridge and Geomorphological Setting

The first site provides an example for which simplified Newmark sliding block methods were deemed appropriate for calculating co-seismic displacements. The surficial site geology comprises QA at elevations above approximately RL+10m (NZVD 2016), and RAc below this level. These units are underlain by the Int. and Ign. The site geology is indicated in Figure 1.

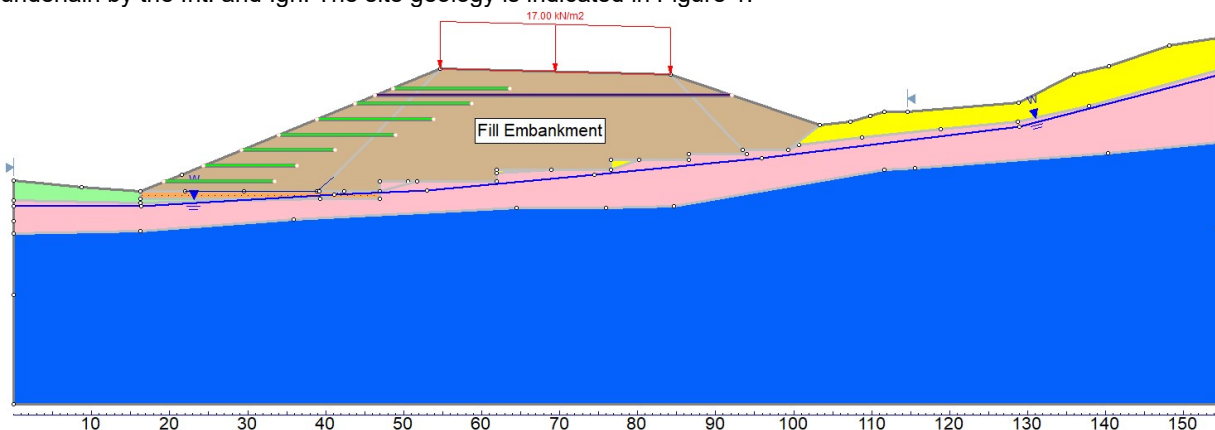


Figure 1: Transverse section through Site 1 western approach embankment

The bridge abutments and approach embankments are to be constructed on the northern side of a shallow alluvial valley with the road mainline roughly parallel to the valley floor. The approach embankment is to be benched into the existing hillslope as shown in Figure 1.

3.2 Calculation of Seismic Displacements

A pseudo-static slope analysis was carried out for four transverse slope sections using Rocscience's *SLIDE2* software. A section was modelled through each MSE abutment, and each approach embankment. The MSE abutments were found to have a DCLS factor of safety (FoS) >1.0 so no calculation of seismic displacements was required. The approach embankments both indicated a FoS of <0.7 at the DCLS event, with minimum failure surfaces through the interbedded deposits.

The critical seismic coefficient (K_y) for Embankment A was calculated as 0.12g. This value was used to undertake DCLS displacement calculations using the methods noted above. The T_s value was calculated to be 0.29s which suggested a flexible method is more appropriate than a rigid method. Both variations of the Bray & Travasarou equations were therefore checked as well as the more recent Bray & Macedo, and Bray et al. methods. The calculated 50th percentile values are shown in Figure 2.

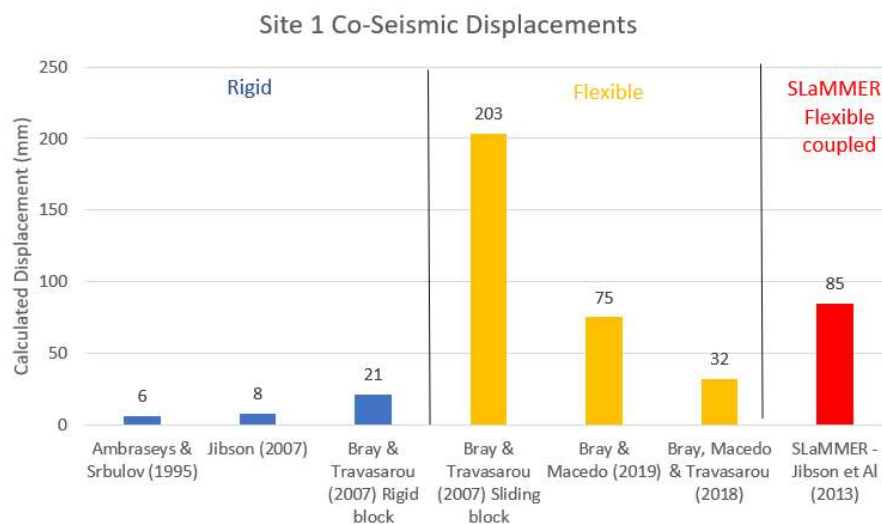


Figure 2: Summary of Site 1 co-seismic displacements to various methods

With the exception of the Bray & Travasarou method, all of the above displacement values were found to be within the DCLS performance criteria for the bridge and did not pose a design risk to the structure.

As can be seen in Figure 2, the Bray & Travasarou (2007) sliding block method predicts significantly higher displacements than the more recent publications so was eliminated from consideration.

Based on the above assessment, a conservative design value of 75mm displacement was adopted recommended using the Bray & Macedo (2019) method.

For the purpose of comparison, analysis was also undertaken using the SLAMMER module (Jibson et al., 2013) inbuilt in *SLIDE2* using ten time history records which were scaled to the site PGA.

The SLAMMER analysis indicated an average displacement of 85mm across the ten seismic records, and is also presented in Figure 2 for comparison. This value was still within the design criteria and further confirms the design value as being a reasonable estimate for this site.

From the assessment of Site 1, it is evident that use of the rigid block methods were not appropriate would significantly underestimate the likely seismic displacements of the structure, in this case by roughly a factor of 3. The impact in this particular case of an approach embankment is not significant, as the worst-case outcome would likely be a relevel of the road and surficial work to the embankment. However, once these values became critical to design of the structural elements, method selection became a key design choice as discussed for Bridge 2.

4 SITE 2

4.1 Overview of Bridge and Geomorphological Setting

The second site is characterised by a steep slope of up to 30m in height. The bridge pier columns are to be founded on piled foundations at the base of the slope. Transverse sections were assessed at the upper abutment, and through each of the pier locations. The critical section was found to be at Pier B, which will form the basis of this example. A cross section indicating the slope geometry, geology, and location of the pier column is shown in Figure 3.

The surficial geology of the slope comprises QA, underlain by Tep. and Ign. at depth.

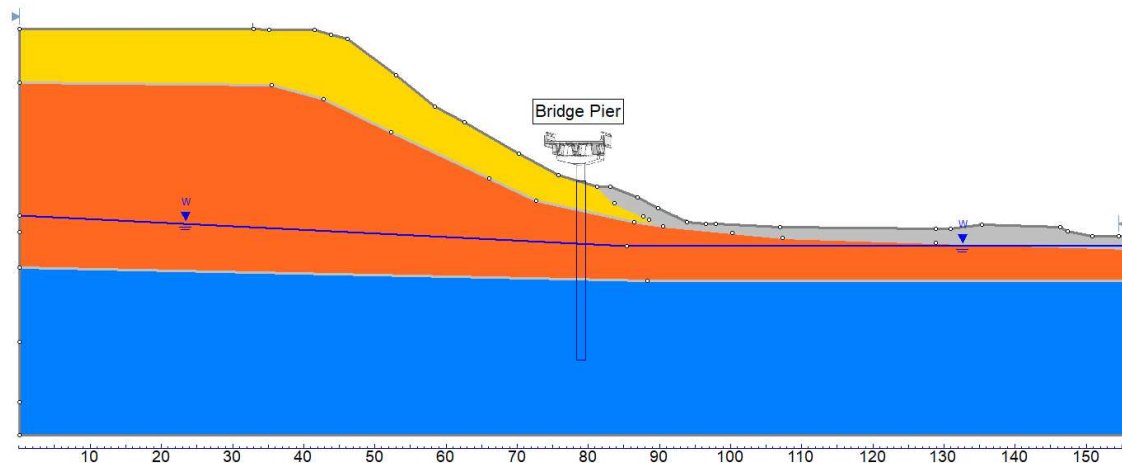


Figure 3: Transverse section through Site 2

4.2 Calculation of Seismic Displacements

Pseudo-static analysis of the slope indicated FoS <0.7 at the DCLS design event. The minimum failure surfaces indicated were global in nature. The sliding mass was assessed to have a T_s value of 0.6s and a T_s/T_m ratio of 1.2 which precluded use of rigid block methods. The calculated displacements for the flexible methods are presented in Figure 4, alongside the rigid method displacements for comparison.

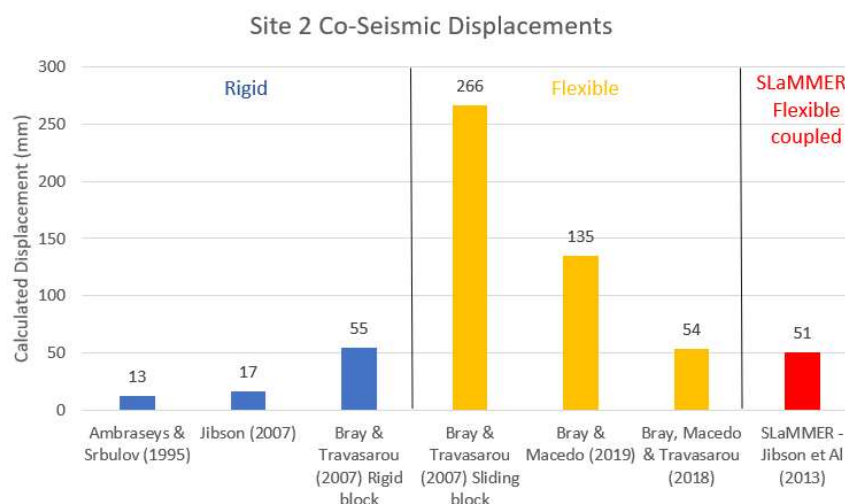


Figure 4: Summary of Site 2 co-seismic displacements to various methods

As noted for Bridge 1, the Bray & Travasarou (2007) sliding block method calculates high displacement values even where K_y approaches PGA, and is considered to be highly conservative. The Bray & Macedo (2019) method also calculated displacements in excess of 130mm which were anticipated to be intolerable by the bridge structure. As a result, a more advanced coupled analysis using selected time history records was undertaken to try and further constrain the likely seismic displacement.

Ten ground motion records were selected from the Pacific Earthquake Engineering Research Centre (PEER) ground motion database and scaled to the site PGA. The coupled analysis was undertaken using the SLaMMER module (Jibson et. al. 2013) inbuilt into Rocscience's Slide software.

The SLaMMER analysis calculated displacement values ranging between 19mm and 161mm, with an average value of 51mm. The average value was adopted as the design value as per AASHTO guidelines (2010b).

The resultant reduction in the design displacement facilitated successful structural design of the piles.

5 SITE 3

5.1 Overview of Bridge and Geomorphological Setting

The third bridge is a ten-span piled structure with the abutments constructed on approximately 8m high approach embankments.

The surficial geology of the slope comprises deep RA to around 20m BGL, underlain by Int. and Ign. at depth. A typical longitudinal section through the western approach and the site geology are shown in Figure 5.

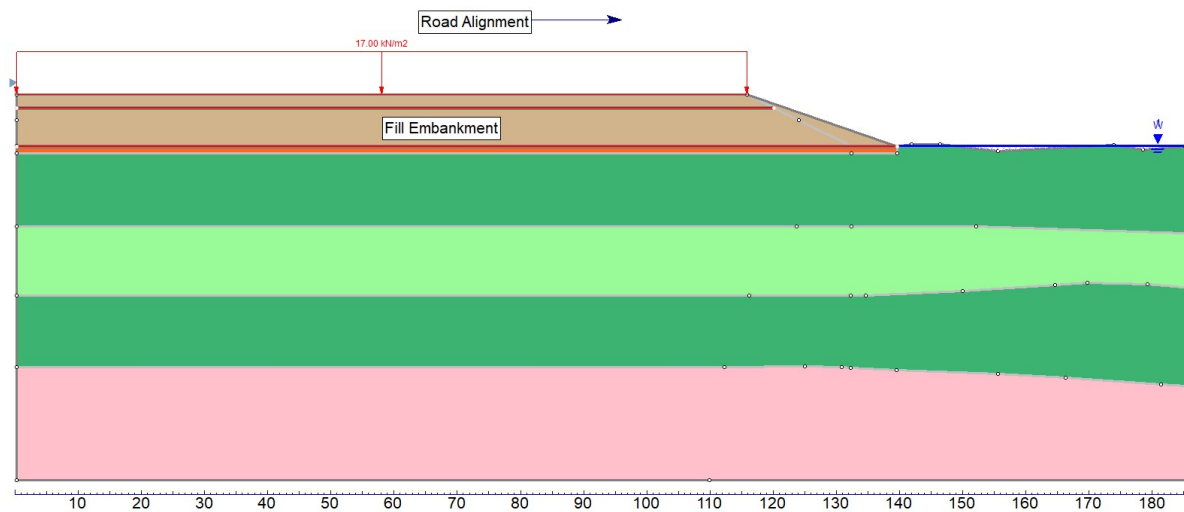


Figure 5: Longitudinal section through Site 3 western approach embankment

As shown in Table 1, the RAc consists of very soft material which was found to be subject to cyclic softening. The RA_g and areas of the Int were also found to be susceptible to liquefaction. Given this significant depth of soft/liquefiable soils, failure surfaces from pseudo-static analysis were found to extend to depths of around 20m BGL, with a $K_y = 0.06g$. When dealing with a highly complex site such as this, calculation of seismic displacements can no longer be characterised as a simple rigid or flexible block, particularly when the failure mode is more of a bearing/lateral spread type failure than slope failure.

5.2 Calculation of Seismic Displacements

Due to the high complexity of the site, it was deemed necessary to undertake a dynamic finite element analysis to accurately characterise the seismic response of the system as a whole. This analysis was undertaken using seven of the time history records used for Site 2.

The range of maximum horizontal displacements across the 10 seismic records was found to be 155mm – 225mm, with an average displacement of 200mm.

For comparison, the seismic displacements calculated using Newmark methods are summarised in Figure 6.

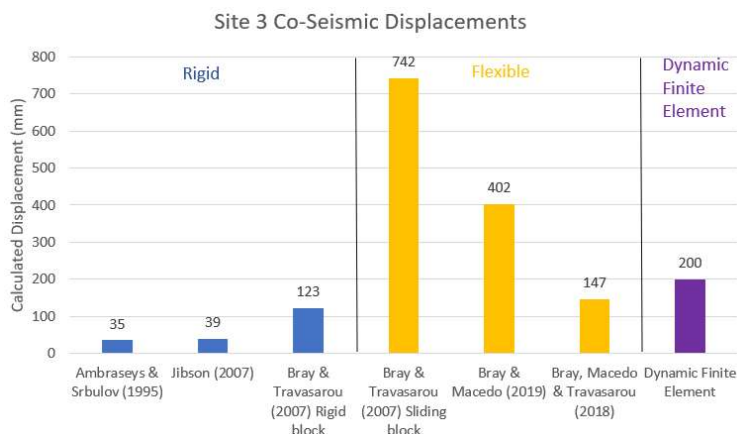


Figure 6: Summary of Site 3 co-seismic displacements to various methods

As with the previous two sites, the rigid methods are shown to significantly underestimate the likely displacement and were discarded as not being relevant to the site. The flexible methods, if considering the two worst case values of 742mm and 402mm, would render the preferred solution unworkable and were deemed inappropriate given the site complexities. The use of dynamic finite element modelling facilitated successful structural design of the bridge where simplified methods would have either drastically underestimated displacements, or over-estimated to the point of requiring a far costlier solution.

6 CONCLUSION

The case study provides an overview of three very different problems and the challenges faced in accurately characterising seismic displacements for design. Were simplified rigid methods to be used for all three as is not uncommon in geotechnical design practice, the risk of significantly underestimating the potential for seismic displacement is shown to be very apparent. Sites 2 and 3 also provide examples of where more advance analysis beyond simplified Newmark type calculations facilitate structural design of elements which would otherwise require significant additional engineering to meet performance criteria. The key message is to avoid complacency and use the right tool for the job, with careful consideration of the limitations and benefits of each.

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