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THE APPLICABILITY OF 3D EFFECTS TO PSEUDO-STATIC DESIGN ACCEPTANCE CRITERIA: THE RECOVERY DESIGN FOR WALL 375

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ABSTRACT

The legislated use of pseudo-static analysis in many jurisdictions mandates its continued application by practitioners. This paper offers guidance to slope stability practitioners on the use the method's three-dimensional formulation to meet Design Acceptance Criteria. The paper demonstrates that three-dimensional seismic k-coefficients cannot be used as 1:1 substitutes to meet historically established DAC and presents a case for developing the correct equivalencies. The case study of the recovery design along Wall 375 along New Zealand's State Highway No. 1 in the aftermath of the Kaikoura earthquake is used for illustrative purposes.

DEDICATION

In memory of Dr. John Read, co-author of this paper, who passed away on April 18, 2024, after this manuscript was completed but prior to its publication. Dr. Read's dedication and contributions to his profession and to upholding high professional standards in New Zealand and elsewhere left a lasting mark.

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INTRODUCTION

With recent technological advancements, three-dimensional slope stability analysis is now accessible to geotechnical practitioners, and the routine use of the less computationally demanding methods (e.g., the limit equilibrium methods) is feasible at operational level. In consequence, a growing number of practitioners are supporting the expanded use of three-dimensional slope stability analyses, as these are viewed as potentially more accurate than their two-dimensional counterparts owing to their ability to capture the three-dimensional features of the geological model as well as the stresses acting along the slip surface that contribute to resistance against sliding.

While two- and three-dimensional slope stability analysis outputs diverge, published Design Acceptance Criteria (DAC) for these do not differentiate between the two. For example, the Canadian Dam Association specifies for dams a long-term minimum factor of safety of 1.5 without elaborating whether this DAC must be met using two- or three-dimensional analysis [1]. Similarly, Chilean legislation prescribes values for the horizontal seismic k-coefficient and factor of safety in the analysis of seismic hazards using pseudo-static analysis, yet makes no mention of analysis dimensionality [2]. The authors did not identify any guiding or legislative documents that make such distinction.

While in older literature, published when three-dimensional analysis was either unavailable or very uncommon (e.g., [3]), the use of two-dimensional analysis can be inferred, the same cannot be said about newer geotechnical guidance. This causes a division of opinion amongst practitioners, some of whom interpret the DACs as values that can be met using either two- or three-dimensional analysis, while others maintain that these values have been derived largely from two-dimensional analysis and must only be used in conjunction with it.

It is therefore evident to the authors that the transition to a broader use of three-dimensional analysis in geotechnical practice necessitates offering proper guidance regarding the interpretation of three-dimensional slope stability effects and the relationship between these and pseudo-static DAC.

Pseudo-Static analysis

This paper addresses the question posed to the practice of geotechnical engineering by Brown [4] on whether three-dimensional stability effects can be considered to meet the seismic k-coefficients used in pseudo-static analysis to evaluate a slope's resilience against seismic hazards. While more advanced tools for seismic analysis have been proposed since the introduction of this method in mid-20th Century, the authors recognize that its legislated use in many jurisdictions around the world (e.g., Chile, Brazil, Italy) mandates its continued application by practitioners, regardless of the objections voiced by researchers regarding its reliability and accuracy [2,5].

Therefore, it is the authors' view that geotechnical researchers bear a responsibility to the profession to address the question of correct interpretation of three-dimensional pseudo-static analysis outputs. The authors are especially motivated to address the issues raised by Brown [4] because of the recognition that the incorrect interpretation of this matter may lead to nonconservative design choices.

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Case Study Background

On November 14, 2016, a seismic event with a 7.8M_w magnitude occurred in and offshore of the South Island of New Zealand [6]. This event, known as "the Kaikōura earthquake" after the district at its epicentre, caused two fatalities and 57 injuries; triggered between 80,000 and 100,000 landslides; and resulted in widespread damage to the transportation infrastructure networks in the region [6,7].

New Zealand's State Highway (SH) No. 1 is the country's main transportation corridor, carrying substantial traffic volumes along the Island's eastern coastline, including the Kaikōura district. Owing to its proximity to the epicentre, SH No. 1 was among the most severely affected roadways, requiring numerous emergency repairs. One such repair is documented by Kendal-Riches [8]. The repaired section, identified as "Wall 375", was a part of the North Canterbury Transport Infrastructure Recovery effort. Completed in 2017, the design was evaluated against existing recommendations.

In the aftermath of the earthquake, New Zealand's Institute of Geological and Nuclear Sciences Ltd. (GNS) initiated a review of the seismic risks across the country. Its new PGA maps delineate the increased probability of shaking events with a higher magnitude, especially along the boundary of the Australian and Pacific plates, following the Alpine Fault to the south and the Hikurangi Trench to the north. These PGA values increased by an average of 50% or more across the country, and in places by 100%.

The National Seismic Hazard Model (NSHM) is an instrument used by the industry to evaluate the seismic risks of engineering designs. It does not mandate specific DAC; instead, practitioners are encouraged to use site- and project-specific information to produce risk-informed designs. For slope stability practitioners, the 2022 changes mean that their seismic stability design criteria, such as k-coefficients in pseudo-static analysis, should be re-evaluated upward. Brown [4] uses the example of Wall 375 to demonstrate that, if three-dimensional effects are considered, the seismic k-coefficient of slopes can be considerably higher than if the traditional two-dimensional analysis is used. He then poses the question whether these effects could be used to accommodate the higher new seismic design criteria.

THEORY AND METHOD

The subject of this paper concerns a subclass of the limit equilibrium methods (LEM) called "pseudo-static analysis" that seeks to evaluate the resilience of slopes against seismic loading. This approach shares in the limitations of the LEM; like the parent analysis, its use in design is heavily predicated on the accumulated practical experience related to the selection of the appropriate design criteria.

The theoretical portion of this paper leaves out the theory of LEM as its underpinnings and limitations are well-documented elsewhere. Instead, the authors seek to highlight a lesser-known aspect of slope stability, namely the correct interpretation of the factor of safety, its role in slope stability and how the latter was historically established. These issues have profound implications on the manner in which design factor of safety values and seismic k-coefficients are derived in our practice.

Interpretation of the Factor of Safety in Slope Stability

In slope stability analysis, while the definition of the factor of safety is reminiscent of equivalent ones applied, for example, in structural engineering, the complexities of mechanical behaviours in soil and rock impose some important limitations, complicating its implementation in analysis and the interpretation of the results. A literature review of the subject reveals some paucity of discussion on the subject, albeit some researchers have been working to clarify the limitations of the factor of safety imposed by the specifics of soil and rock slope problems, and to offer meaningful ways to interpret the results [9-11]. This section seeks to further clarify these matters.

Traditionally in civil engineering, the factor of safety (in reference to the mechanical strength of a structure) is a measure of its capacity to support loads over and above the current ones and can be more formally defined as "the largest factor by which the working load can be scaled without failure". Note that this definition can be formulated in terms of either forces or stresses; here, we favour the latter to simplify the comparison with geotechnical applications. When the load is maximized and cannot be increased any further, the system is said to be at its limiting equilibrium, with the working load equalling the shear strength and the factor of safety reducing to unity.

Consider the example of a bag hanging on a hook. If the bag exerts a shear stress of 5 kPa owing to its weight and the hook has a shear strength of 10 kPa, it can be said that the bag-hook system has a factor of safety equal to 2. We often interpret this to mean that we can double the load without causing shearing failure. Conversely, we may say that the hook-bag system is "twice as strong as it needs to be".

A similar definition was adopted for slope stability problems by the early proponents of the limit equilibrium method. For example, in the Swedish Circle Method, Fellenius defines the factor of safety of a rotational slide as the ratio of shear strength to driving shear stress along the slip line [12]. Expressed in this form and using constant undrained shear strength s_u, the factor of safety of a slope parallels the one quoted earlier. Fellenius went on to enhance the Swedish Circle Method by formulating the solution known as "the Ordinary Method of Slices", involving the subdivision of the slide into slices, with individual factors of safety per the above definition calculated for each slice's base and then rolled into a weighted calculation of the overall factor of safety. This enhancement allowed for the application of a more realistic variable shear strength along the slip line, for example by using the Mohr-Coulomb strength model. An unintended consequence of this approach was to create the so-called "zones of overstress", where the factors of safety at the base of individual slices may be below unity in violation of the second Newtonian law under static equilibrium conditions and suggesting an inadmissible stress state plotting above the strength envelope. This limitation was recognized early on, with the subsequent proponents of limit equilibrium solutions, starting with Bishop [13], specifying the condition of a uniform factor of safety along the slip line to be satisfied along with static equilibria for the solution to be deemed valid. Bishop [13] proposes the following definition of the factor of safety: "the ratio of available shear strength of the soil to that required to maintain equilibrium". Using the Mohr-Coulomb failure criterion for "strength", this definition is as follows:

$$FoS = \frac{c' + \sigma_n' \tan \varphi'}{s} \quad (1)$$

where s is the mobilized shear stress along the slip line. Bishop further rearranges this expression:

$$s = \frac{1}{FoS}(c' + \sigma_n' \tan \varphi') \quad (2)$$

The definition by Bishop [13] is known as "the shear strength definition of the safety factor". One of its limitations is explored here, clarifying the significance of Eq. 2.

Bishop's definition invokes "the available shear strength of the soil". The issue is that this quantity is not uniquely defined [14]. Bishop [13] defines it using the current stress σ'_n. A graphic representation of this is seen in Figure 1; the factor of safety per it would be represented by the ratio A/s. However, this "available shear strength" is arguably no more justifiable than strength B or C. The figure also elucidates that this definition

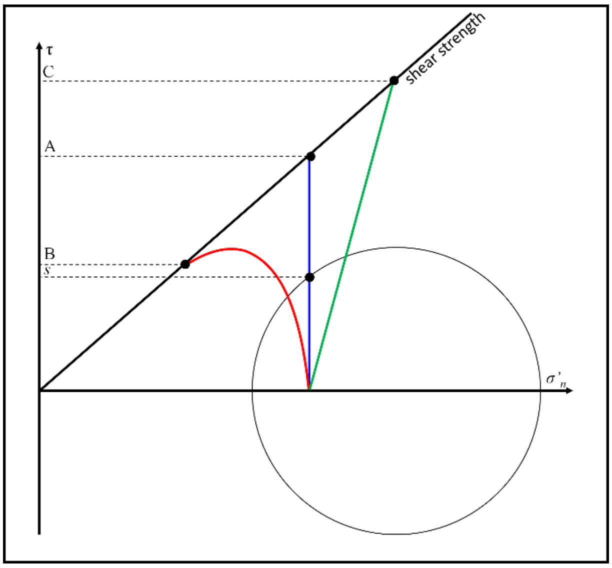


Figure 1: An illustration of the stress path to failure using Bishop's [13] definition of the factor of safety; "s" is the shear stress acting along the slip plane. Adapted with modifications from Barron [14].

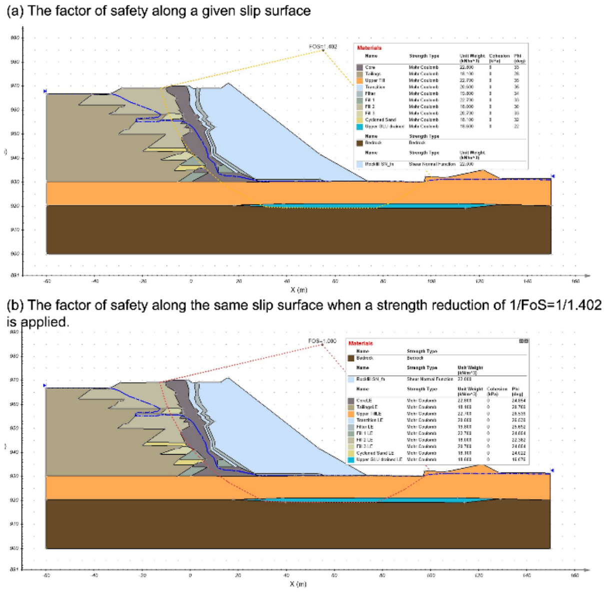


Figure 2: An illustration of the definition of the factor of safety by Morgenstern and Price [15]. Analysis of the Mt. Polley TSF adapted for this example from Zabolotnii [17]. In this example, the factor of safety is calculated along a given slip line (a). Then, this factor of safety is used reduce the shear strength parameters c' and tan φ' in each soil along the slip line. With these reduced strength parameters, the factor of safety of the slope along the same slip line reduces to unity. While the mathematical definition is precise, the interpretation of these manipulations in real life terms is illogical.

of the factor of safety is a particular stress path (indicated by the blue line) and no more special than the stress paths to failure at points B, C or indeed any of the points along the strength envelope; the ultimate strength of the soil element will be defined by the stress path that it follows (for simplicity, complicating factors such as strain-weakening and stress history effects are left out; their presence would not negate this point, quite the contrary).

The non-uniqueness of "the available shear strength" calls into question the exactness of the factor of safety definition cited by Bishop [13] and is the reason why an alternative formulation was soon proposed as follows: "*The factor of safety is the factor by which the shear strength parameters may be reduced in order to bring the slope into a state of limiting equilibrium along a given slip surface*" [15].

In the context of this definition, the term 1/FoS in the earlier given Eq. 2 becomes the shear strength reduction factor. It should be noted that the definition by Morgenstern and Price [15] is algebraically equivalent to that used by Bishop [13]. Eq. 1 and thus shares its limitations; the difference lies with its interpretation: it offers mathematically exact definition as opposed to the more ambiguous earlier version. However, its interpretation remains unclear, as illustrated by the example in Figure 2. The Alameda dam case study by Quinn et al [16] further illustrates this issue.

This series of arguments allows us to reach the conclusion that the traditional interpretation of the factor of safety stated at the start of this section is not directly applicable to slope stability problems. This definition, adapted by Bishop [13] and reframed by Morgenstern and Price [15] is restrictive because of the assumption of a specific stress path to failure [9]. The authors caution against the interpretation of this quantity as a "measure of excess strength" or a "factor by which we can scale the working load," as these are ambiguous as well as potentially inaccurate. Instead, they encourage the recognition that the factor of safety definition is a stress path.

A Historic Perspective on the Development of DAC

The limitations of the definition of the factor of safety discussed above do not negate its important role in slope stability. The extensive application of the factor of safety in slope stability established its use as an empirical measure of (a) uncertainty and (b) acceptable performance. The use of the factor of safety to mitigate uncertainty is a well-recognized strategy that will not be additionally discussed here.

The other function of the factor of safety in slope stability is that of a performance indicator. Geotechnical literature expounds on the issue as follows [9,10]:

"An additional major role of the Factor of Safety is that it constitutes an empirical tool whereby deformations are limited to tolerable amounts within economic restraints. In this way, the choice of the Factor of Safety is greatly influenced by the accumulated experience (…)."

This quote speaks to the process of selecting appropriate factor of safety values for specific applications, and, by extension, the determination of the DAC. For example, over the last decades, we have come to think of a FoS=1.5 as being appropriate for dams; this DAC value is recommended by a variety of best practices documents such as the CDA Guidelines [1]. In pit slopes, an FoS=1.2 and FoS=1.3 are usually accepted for inter-ramps scale failures and overall failures, respectively. How did we arrive at these values and why do they differ in different applications?

The answer is related to the target performance that we seek to achieve. Dams must be safeguarded against excessive deformations to avoid core cracking and other damage with potentially catastrophic consequences; our cumulative experience with many dam designs over the last decades has taught us that a FoS=1.5 generally ensures this desired performance. In pit slopes, excessive deformation is not usually a critical issue – instead, the goal is to protect life and equipment while maximizing ore recovery. Hence, the acceptable FoS values are lower as higher deformation levels, and even occasional failure, can be tolerated. In this sense, the target (or design) factor of safety is a surrogate indicator that we have come to associate with the target performance. Simply put, we choose what worked in the past and only deviate from it with extreme caution.

Three-Dimensional Slope Stability Effects and DAC

Starting in the late 1970s, the limit equilibrium method was extended to three dimensions [18-20]. Geotechnical publications of that period abound with benchmarking studies comparing the two- and three-dimensional limit equilibrium solutions [21]. The consensus has been that three-dimensional factors of safety are greater than their two-dimensional equivalents, commonly by 20% to 40% and potentially more for concave or convex surfaces or other complex 3D features. This difference arises from (a) the so-called "sidewall effects" whereby there is more shear resistance (relative to driving shear stresses) along the shallower sides of a slide than along its deepest, critical section; and (b) three-dimensional features such as concavities, convexities, and asymmetry of strength or supports. The difference between the two- and three-dimensional factors of safety (taken as a percentage difference above the former) is formally defined by Zabolotnii [17] as "three-dimensional slope stability effects". For simplicity, let us assume that an average slope's three-dimensional stability effects are a nominal 30%, a commonly quoted value whose actual magnitude is immaterial for the argument here.

Bearing in mind that every slope has some three-dimensional slope stability effects, one must consider the implications on the selection of the appropriate design FoS (i.e., DAC). Recall that the design FoS is a surrogate indicator of the target performance. The appropriate design factors of safety values have been determined from decades of empirical experience by noting designs that performed well and associating that desirable performance with their factors of safety. However, those factors of safety were calculated using two-dimensional analyses, and mostly limit equilibrium methods. We know this because three-dimensional analyses were relatively rare until recently and because the limit equilibrium methods were the standard tool. Therefore, it is reasonable to infer that the DAC, such as FoS=1.5 for dams and FoS=1.2 for pit slopes, are associated with two-dimensional analysis. Is it then reasonable to use three-dimensional FoS when evaluating a slope design against these criteria?

To answer this question, consider the hypothetical example of a generic dam. This dam, designed to a FoS_{2DLEM}=1.5, is observed to perform well. This good performance is then associated with its two-dimensional factor of safety, contributing to the body of growing empirical knowledge that a FoS=1.5 "works" for dams, a value eventually enshrined in the best practice guidelines as a DAC. This same slope would have a FoS_{3DLEM}=FoS_{2DLEM}*130%=1.95. If we assume that three-dimensional FoS values can be used to meet DAC, then that means that this slope's design can be changed; for example, we can steepen the outer slope to achieve a FoS_{3DLEM}=1.5. This design alteration would also reduce the FoS_{2DLEM} by about 30% to 1.15. The question is, would the altered slope exhibit the same good performance as the original one? Will it deform the same or more? Will its probability of failure remain the same or increase? The answer is obvious: the altered slope's performance will be diminished; one can expect greater deformations and a heightened risk of instability compared to the original analysis as well as to the target performance standard.

One must conclude that three-dimensional factors of safety should not be used at face value to meet design factors of safety that have been determined substantively from two-dimensional evaluations. Doing so may lead to non-conservative design and poor performance and would elevate the risk of failure.

Slopes with Pronounced Three-Dimensional Features

The conclusion reached in the previous section is a generalization that has some important caveats. While all slopes have some three-dimensional slope stability effects accounting for sidewall resistance, the slopes with distinct three-dimensional features may have more pronounced three-dimensional slope stability effects; their three-dimensional factors of safety could be significantly higher than the nominal 30% average. The authors' opinion is that these cases might warrant special consideration when determining the appropriate FoS design value. However, establishing these would require a body of research and calibration.

Pseudo-Static Analysis

Pseudo-static analysis is an adaptation of the limit equilibrium method to evaluate a slope's resilience against seismic events. One of the simplest methods used in earthquake engineering, this approach applies a seismic coefficient k (sometimes specified as its vertical and horizontal components, k_v and k_h) to calculate the vertical and lateral forces imposed by seismic loading as a fraction of gravity loads (Melo and Sarma, 2004):

$$F_h = k_h W \tag{3}$$

$$F_v = k_v W \tag{4}$$

In other words, in pseudo-static analysis, the destabilizing driving the slide is Fdr= W+F (as opposed to Fdr=W in LEM), and the factor of safety calculated using a specified earthquake-induced load F, should be interpreted along the lines of the arguments presented earlier herein (i.e., not as a ratio of available strength to driving stresses but as a proxy indicator whose use is established from experience). The same reasoning applies to the critical or ULS k-coefficients.

Pseudo-Static DAC

A slope's seismic resilience is evaluated by increasing the k-coefficient until its factor of safety is reduced to some minimum value, commonly 1.1-1.2, with the reasoning that under short term (e.g., dynamic) loading conditions, slopes with this factor of safety will remain stable. The k-coefficient that reduces the slope to its limiting equilibrium (e.g., FoS=1.0) is "the ultimate limit state" (ULS) k-coefficient, also sometimes called "the critical k-coefficient"; this value is used to estimate the critical seismic conditions that would cause the slope to fail.

Minimum and/or ULS k-coefficients are commonly legislated, recommended by regulatory agencies and in geotechnical publications, or derived using prescribed PGA or displacement-based methods; these values are the DAC for this method. Terzaghi [3] recommends values between 0.1 and 0.5 for seismic events ranging from "severe" to "catastrophic"; whereas the US Corps of Engineers [22] suggests values between 0.1 and 0.2. The Chilean legislation establishes k_h-values of 0.095 and 0.18 for earthquakes with respective magnitudes of Mw=7.5 and Mw=8.0 [2]. Eurocode 8 recommends pseudo-static analysis where inertia forces are related to PGA [23] and the Italian code inspired by it recommends correlating the design k-value to displacements using Newmark's [24] general approach and its more modern modifications [25,26].

Melo and Sharma [27] point out that while the selection of the design k-coefficient is critical for this type of analysis, the choice is often "subjective and lacks in a clear rationale". A review of the literature indicates some efforts to calibrate the design seismic k-coefficients against field experience, much like the design factors of safety for dams or pits slopes. As with all empirical values, the choices tend to be conservative at the start owing to a lack of data, and a downward corrective may be introduced as knowledge accumulates. This might explain the decrease in the recommended range of k-values in the 1983 US Corps of Engineers guidelines [22] compared to Terzaghi's 1950 works [3].

Despite these efforts, the validity of the recommended seismic k-coefficients is not clearly demonstrated. For example, Newmark [24] discusses cases of earthquake-induced failures in dams where the back-analysed k-coefficients would suggest that the structures should have remained stable; and Melo and Sharma [27] use dynamic analysis to demonstrate that literature-recommended design k-coefficients may be underestimates.

Seismic k-Coefficients in Three-Dimensional Analysis

Like two- and three-dimensional factors of safety, the k-coefficients derived from two- and three-dimensional pseudo-static analyses are different: the three-dimensional values reported in literature tend to be higher. For example, using the data from McPherson [28] and Brown [4], this difference can be estimated at 45-61%.

The following example seeks to demonstrate that these differences are not independent, but that the former drives the latter. Consider the earlier discussed dam slope with $FoS_{2D}=1.5$. This slope would require some seismic force $F_{2D}=k_{2D}W_{2D}$ to reduce it to the minimum acceptable factor of safety. Subdividing the three-dimensional soil mass of this slide into parallel cross-sections one unit thick each and aligned with the slip direction, the critical section, corresponding to the one used in 2D, would require a higher seismic force $F_{3D}=k_{cr}W_{2D}$ owing to a higher FoS=1.95, and with the weight being the same, a higher scaling coefficient k_{cr}. The other sections, being shallower and hence with lower weights Wi and the same $FoS_{3D}=1.95$, would require yet greater destabilizing F_i and k_i. Three-dimensional limit equilibrium calculations would iteratively seek the solution that would yield an average $k=k_{3D}$. While it is not easy to follow them cerebrally, in view of the arguments above, the k_{3D} would be expected to be greater than its two-dimensional equivalent where the $FoS_{3D}>FoS_{2D}$. If the above reasoning stands, then the two issues with the three-dimensional factors of safety highlighted earlier (namely, their interpretation in terms of "available strength" and, in consequence, their non-applicability as DAC where the latter have been established from a body of two-dimensional analyses) also apply to three-dimensional k-coefficients. This argument is even more compelling with the recognition that with earthquake loading, the stress path to failure is likely "undrained" owing to pore pressure generation.

An overall conclusion that can be drawn is that, as it is the case with the 2D/3D factors of safety, 3D k-coefficients cannot be used as 1:1 substitutes for their 2D equivalents. The idea that such substitutions might be acceptable arises from the erroneous interpretation of a slope's factor of safety (and by extension its k-coefficient) as being a measure of "reserve strength" that is captured only partly by 2D analyses but more fully by 3D analyses. The earlier arguments and example demonstrate the precarity of such an interpretation and clarify how it could lead to nonconservative design decisions, elevating the risks of failure from natural disasters.

RESULTS AND DISCUSSION

The North Canterbury Transport Infrastructure Recovery Design for Wall 375, New Zealand

SH No. 1 is located on a narrow coastal platform between the steep slopes of the Seaward Kaikōura mountain range and the Pacific Ocean. The basement rocks at Wall 375 are 160 to 100 million years old interbeds of weak siltstone and mudstone belonging to the locally named Pahau Terrane. The upper bed of mudstone is overlain by a colluvial loess containing traces of gravel that in turn is covered by superficial deposits of gravel. Figure 3 captures a section of this coastline in the aftermath of the Kaikōura earthquake. Numerous emergency repair works were completed along SH No. 1, including at the location in North Canterbury identified as Wall 375. The recovery design, seen in Figure 4, included a steep gabion basket wall with a height of ~3m providing lateral support for backfill material. It was placed on a ~13m bench excavated in foundation soils. The foundation profile consists of loess (1-3m thick) overlying sloped mudstone (≤6m thick) which in turn rests on top of siltstone bedrock. Soil nails were driven through the loess and anchored in the mudstone.

As the Wall 375 repairs were completed prior to 2022, they would have been designed using the 2010 NSHM. Kendal-Riches [8] does not cite selected k-coefficients for the location but rather demonstrates that k=0.4 reduces the slope's factor of safety to ~1.2 when using two-dimensional analysis. This compares reasonably well with the two-dimensional analyses conducted for this study, as well as with those reported by Brown [4] (Table 1 & Figure 4).

Table 1: Summary of pseudo-static analyses of Wall 375 produced for this study and by others.

Analysis		Software	FoS for k=0.4	ULS k
Kendal-Riches [7]	2D	GeoStudio	1.2	n/a
Brown [4]	2D	TAGA	1.17	0.504
Our analysis #1	2D	Plaxis LE	1.25	0.577
Our analysis #2	2D	TAGA	1.17	0.504
Our analysis #3	2D	GeoStudio	1.21	0.516
Brown [4]	3D	TAGA	2.00	0.811
Our analysis #4	3D	Plaxis LE	2.02	0.888
Our analysis #5	3D	TAGA	2.00	0.811

Using the 2010 NSHM maps, the PGA value with a 10% probability of exceedance in 50 years is estimated at 0.4-0.42; in this context, the choice of k=0.4 for design purposes appears reasonable and conservative. From the 2022 maps, the PGA value at the same location is 0.7-0.72, a 70-75% increase over the previous value [29]. This determination suggests that, if two-dimensional analysis is used, the 2017 design of Wall 375 might not meet the increased 2022 recommendations should the same selection criteria for the design k-coefficient be used as in the original design.

Using three-dimensional pseudo-static analysis of an extrusion of the same slope, Brown [4] calculates a ULS k_{3D}-coefficient of 0.811. This study replicates this result when using a three-dimensional slip surface shaped as an ellipse with a ratio of 0.5:1, same as reported by the original study (Table 1 and Figure 5); some variation of the ULS k_{3D}-coefficient (<5%) is noted when varying the discretization level of the model. In other words, if three-dimensional slope stability effects are considered, the old design appears better aligned with the 2022 NHSM guidelines [29]. Brown [4] poses the question whether it is appropriate to consider such effects when assessing the seismic resilience of slopes, i.e. whether it is appropriate to consider k_{3D}-coefficients instead of k_{2D} values to meet the design requirements.

Per the arguments presented in the Theory section, doing so would likely be unsafe. Historically slopes designed using two-dimensional pseudo-static analysis would have all exhibited some three-dimensional slope stability effects and their three-dimensional k-coefficients would have been higher than their two-dimensional equivalents. McPherson [28] and Brown [4] place this difference at 45-61% for an average of 53%. Considering that on average, the PGA values increased by ~50%, this would mean that many or even most of the historical slopes designed to the old standard would also meet the new one without improvement whatsoever, if k_{3D}-coefficients are to be taken as fully equivalent to their two-dimensional counterparts. Following this line of reasoning, one might be led to conclude that natural disasters like the Kaikōura earthquake did not raise concerns about the seismic performance of existing infrastructure or the design standards used to build it.

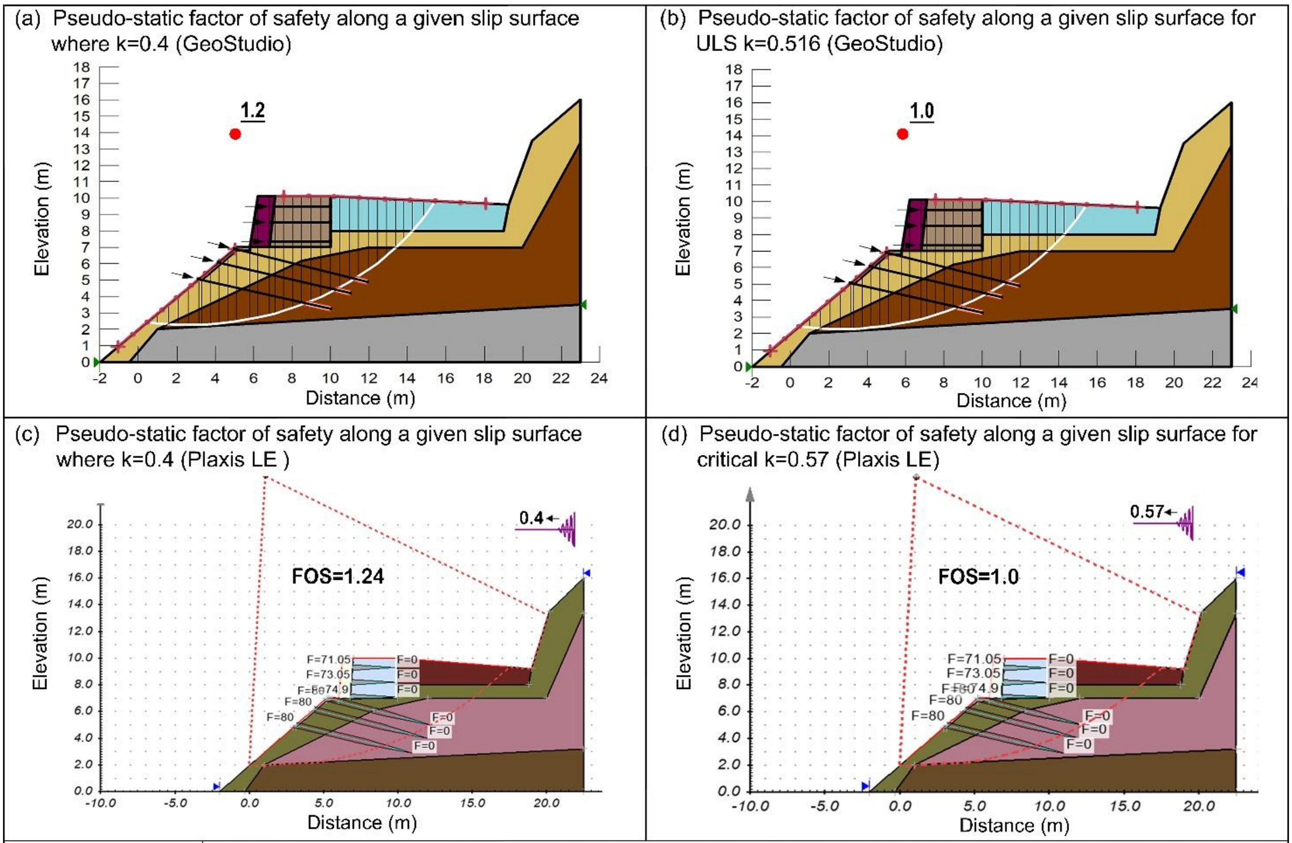
In fact, accommodating three-dimensional stability effects in pseudo-static analyses does not address the GNS' fundamental concern reflected in its 2022 NSHM, that seismic hazards have increased and that, going, forth, designs must be more resilient. The widespread damage to the infrastructure resulting from the Kaikōura earthquake is the empirical feedback that our historical design practices do not meet the changed performance expectations. It is the authors' position that the geotechnical community must respond to this by designing slopes that are better than their historical equivalents when compared using the same tools used to assess the originals, rather than contemplate if old designs were strong enough by today's standards owing to some previously overlooked effects.

Replicability of Three-Dimensional Pseudo-Static Analyses

While this study was able to replicate Brown's [4] three-dimensional pseudo-static analysis of Wall 375 when using the reported ellipsoid ratios, the rationale for selecting this particular three-dimensional shape to represent the slip surface is not compelling. Historic case studies reveal a variety of three-dimensional slip surface shapes, and narrow slip surfaces such as the one used by Brown are infrequent [16]. As the magnitude of the three-dimensional effects is inversely correlated with a slide's width, wider slides would have lower FoS_{3D} and k_{3D} [17,30]. The effect of the aspect ratio on the ULS k_{3D}-coefficient is demonstrated in Figure 6 visualizing the Plaxis LE analyses; a 3D9% drop of the ULS k_{3D} value from 0.888 to 0.813 is noted when the ellipse ratio is doubled from of 0.5:1 to 1:1, resulting in a slide width-to-depth ratio W:D increase from 2.5:1 to 4.5:1. When the ellipse ratio is further increased to 1.5:1, the difference becomes even greater at 13%, with a ULS k_{3D}-coefficient of 0.787 and a W:D ratio of 5.5:1.

As in designs and forecasts, the shape of the slip surface (and therefore the aspect ratio) is unknown, this limit equilibrium input is left to the engineer's judgment. This has been known to lead to inconsistent predictions for the same geometry of the problem [31]. If the industry were to make a case for the use of k_{3D}-coefficients, it would also mean providing a rational basis for selecting the shape of the slip surface, and to demonstrate the consistency of the predictions.

Figure 3: A view of the South Island's east coastline in the aftermath of the Kaikōura earthquake. Reproduced with permission from: Kaikoura Earthquake Slope Hazards - Risk Mitigation and Network Resilience by Justice, Saul and Mason, NZ Geomechanics News, December 2018.



LEGEND

Name	Plaxis LE	GeoStudio	Strength Type	Unit Weight (kN/m^3)	Cohesion (kPa)	Friction angle (deg)
Loess			Mohr Coulomb	18	5	30
Gabion wall			Mohr Coulomb	18	300	0
Fill			Mohr Coulomb	20	5	32
Mudstone			Mohr Coulomb	19	3	40
AP65 Fill			Mohr Coulomb	20	0	37
Concrete			Mohr Coulomb	24	500	0
Siltstone			Bedrock			

Figure 4: Two-dimensional pseudo-static analyses of Wall 375 using Bentley's (top) GeoStudio® and (bottom) Plaxis LE®. Left column: FoS values for k=0.4. Right column: ULS k values.

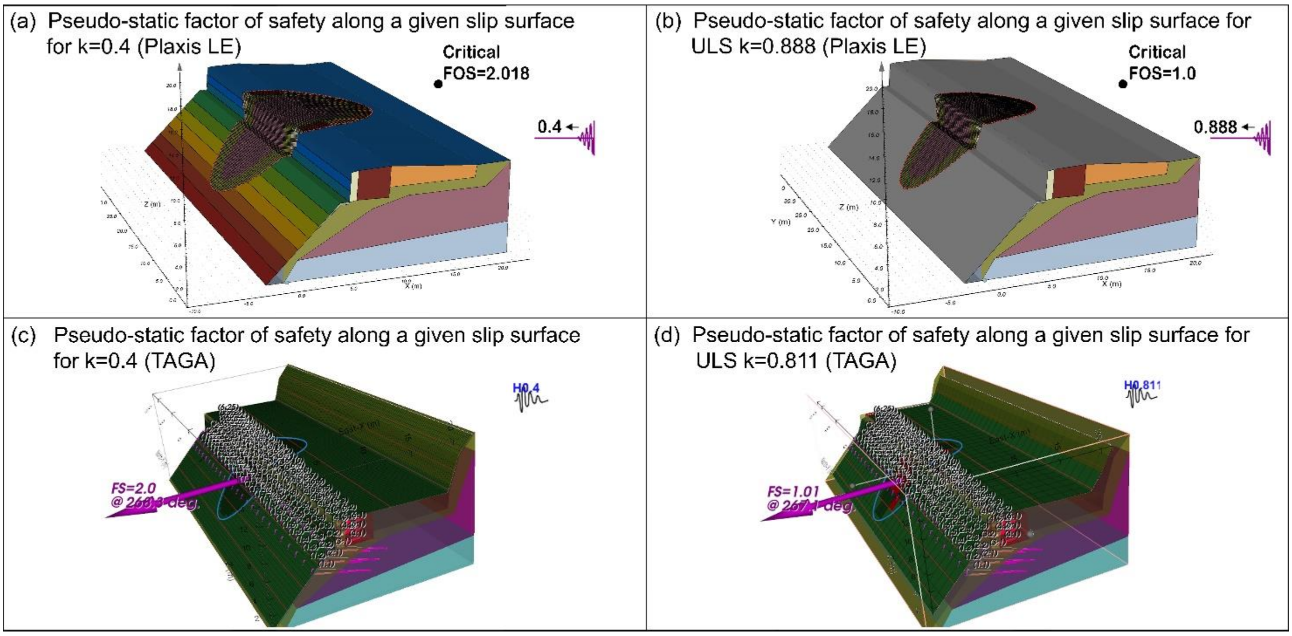


Figure 5: Three-dimensional pseudo-static analyses of Wall 375 using the ellipse ratio reported by Brown [4].

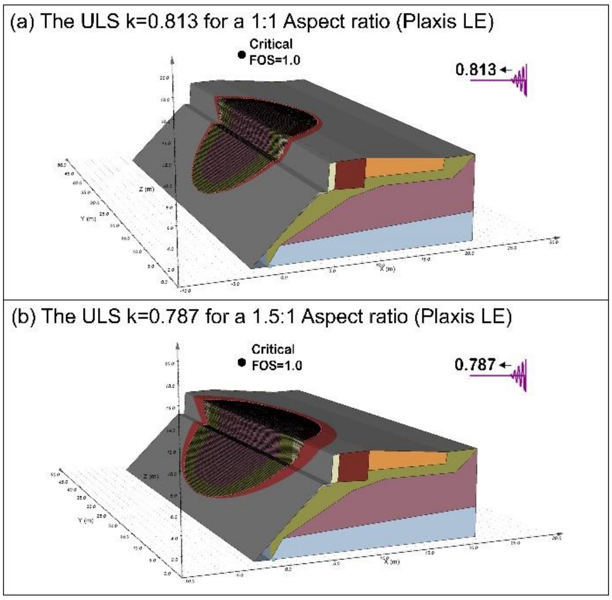


Figure 6: Three-dimensional pseudo-static analyses of Wall 375 using higher aspect ratios than reported by Brown [4].

Software-Related Differentiation of Pseudo-Static Analysis Results

A differentiation of pseudo-static analysis results was noted between different software when using the same or equivalent modelling inputs (e.g., geometry, strength models and slip surface definition). So, the 3D analyses when using an ellipse ratio of 0.5:1 yield a USL k-coefficient of 0.811 and 0.888 in TAGA and Plaxis LE respectively, a difference of 9.5%.

Possible Routes for Establishing 2D/3D Equivalencies for Pseudo-Static and Other Seismic Analyses

The performance (i.e., deformation levels) of slopes subjected to seismic excitation is a fundamental consideration in the seismic design of slopes. For example, NZGS guidance [32] discusses deformation limits as a starting point for such evaluations, referencing NZTA/Waka Kotahi Bridge Manual [33] as one resource for such values. A logical conclusion that can be drawn from this is that 2D/3D equivalencies for seismic analysis inputs could be selected on the basis of comparable deformations.

Pseudo-static analysis does not predict deformations; rather, acceptable displacement levels can be used to select the input k-coefficient (e.g., using acceleration-time plots per Newmark's [24] or equivalent methods). One potential approach for establishing 2D/3D equivalencies would be to perform dynamic analyses in two and three dimensions in order to establish if a differentiation of displacement response exists (indicating the presence of 3D stability effects). If such can be established, one might use scaling techniques to determine the equivalent acceleration-time records that elicit comparable deformations in 2D and 3D so that 2D/3D k-coefficient equivalence can be quantified using Newmark's approach. One potential complication is that in 2D dynamic analysis, seismic excitations in the direction parallel to the slope alignment cannot be applied (and their effect would be unclear considering the plane-strain assumption that applies to 2D analysis); that would mean that the effect of full acceleration-time records cannot be compared side-by-side in 2D and 3D analyses.

One limitation of this study is with its scope, restricted to a single case study. Should an in-depth examination of three-dimensional effects in seismic slope stability and associated issues be undertaken, a larger sample of case studies would be advisable. As well, in order to help establish model equivalence to in-situ conditions (an essential component of validation), case studies of slope that failed marginally or were on the verge of failure would be of special interest. This is because, unlike with traditional slope failures, considered to take place at conditions corresponding to FoS≈1, seismicity-induced slope failures may occur under conditions of FoS≤1; in consequence, slopes that "just failed" or "almost failed" offer a better indication of the in-situ states.

Finally, where rock mass is involved, such studies should consider the 2D/3D equivalence of rock mass parameters. Fractures, joints and other damage are traditionally quantified in LEM studies using GSI or similar indices, whereby laboratory-derived intact strengths are downward-adjusted based on the quality of the rock mass. Geotechnical researchers have pointed out that these downward correction factors were historically derived using two-dimensional back-analyses and thus it is unclear whether they can be applied to three-dimensional analysis without a corrective.

CONCLUSIONS

Applicability of Three-Dimensional Effects in the Selection of the k-Coefficients

It has been reasoned here that limit equilibrium DAC, including seismic k-coefficients, have been historically calibrated against empirical experience with two-dimensional analysis to ensure some target performance. k-coefficients produced by three-dimensional pseudo-static analysis are generally higher and should not be used as 1:1 substitutes in these evaluations; they have not been calibrated against field data and we do not know the values that correspond to acceptable performance. Using k_{3D}-coefficients in this manner would likely produce non-conservative designs. A case for developing three-dimensional DAC can be made for slopes where three-dimensional stability effects over and above baseline levels exist. The authors pose the question of validation of the seismic $k_{2D,3D}$-coefficients hoping to generate discussion.

With little research interest in the pseudo-static method amongst academics, the authors wish to highlight its legislated and recommended use in many jurisdictions, and by implication its continued use by practitioners. Without proper guidance, there is a risk that the method might be improperly applied, leading to non-conservative slope designs and elevated risks of seismic failures. It is the authors' position that geotechnical researchers have a responsibility to offer appropriate guidance to the practice on the issues raised by this paper.

Consistency and Replicability

The study questions the consistency of three-dimensional limit equilibrium predictions of the k_{3D} coefficient, owing to different slip surface shapes. If the industry were to make a case for the use of k_{3D}-coefficients, it would necessarily mean providing a rational basis for selecting the shape of the slip surface, and to demonstrate the consistency of the predictions.

Likewise, the differentiation of pseudo-static analysis results among different software must be addressed.

Increase in Hazard vs. Impact

GNS points out that the increase of impact is not linearly correlated with the increase in PGA [29]. For example, a doubling of PGA values at the location of Wall 375 may produce disproportionate damage. From a soil and rock mechanics perspective, this fits well with our understanding of peak strengths, plastic processes below peak and tolerable deformations. In view of this we are unclear about the reasoning behind the linear relationship that is suggested by some between forecast PGAs and selected k-coefficients. The authors pose this question to the geotechnical community hoping to generate discussion.

Path Forward

A path forward is proposed herein to establish 2D/3D equivalencies in seismic slope analysis, including pseudo-static analysis. It involves using 2D and 3D dynamic modelling for benchmarking, as this is the most advanced approach currently available to the geotechnical practice, and should necessarily include the back-analysis of known slope instabilities following earthquakes, especially cases of "just failed" slopes or "near-failures". Where rock mass is involved, one should additionally consider the 2D/3D equivalencies for rock mass quality.

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DATA AVAILABILITY

Some or all data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request. Some or all data, models, or code used during the study were provided by a third party. Direct requests for these materials may be made to the provider as indicated in the Acknowledgements.

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