Seismic Stability Assessment of New Zealand Hill Slopes

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Summary: New methods recently proposed in California for the assessment of slope stability under earthquake loading are reviewed, and considered alongside current New Zealand code provisions, practice and within the local geological setting. The application of methods used for seismic slope stability analysis procedures, in particular pseudostatic screening methods, are considered. Several regionally typical slope stability scenarios are assessed in order to consider the state of current New Zealand practice alongside the new California recommendations.

INTRODUCTION

The approach to static slope stability assessment in New Zealand typically attempts to demonstrate a factor of safety against failure between 1.2 and 1.7 depending on the application and assumptions. Different safety factors are appropriate for different situations, including temporary or permanent works, the use of conservative or average parameters. A static factor of safety of 1.5 is often regarded as being sufficient to ensure an adequate factor of safety under more extreme conditions. As with all Geotechnical Engineering processes a 'flexible' approach is advocated, which depends on the assumptions made of significant parameters, such as groundwater levels, unit weights and effective strength parameters.

In conjunction with intrusive investigations and soil mechanics theory, the following all have a significant part to play in assessing slope stability: application of 'engineering experience', desk studies, engineering geological mapping and walkover inspections. However, when it comes to the design of slopes to adequately resist seismic loading, there would be few practicing professionals in New Zealand who could inspect a slope and reliably judge the stability of the slope during some level of earthquake shaking based on past performance. This paper investigates the requirements for seismic slope assessment in New Zealand's published codes and standards, and compares the state of practice in this country with guidelines recently published for California.

ASSESSMENT REQUIREMENTS

The following sections indicate the requirements for slope assessment or performance incorporated in various standards and codes that apply to the stability of land for building purposes in New Zealand.

Building Act 1991

Section 36 of the Building Act discusses building on land subject to erosion, etc, or slippage. It is well known to the geotechnical community, being the subject of much recent debate within the New Zealand Geotechnical Society (NZGS). In general terms section 36(1) gives the territorial authority the power to refuse a building permit on land likely to be subject to '...slippage', and section 36(2) provides an alternative where the civil liability is transferred to the landowner under certain circumstances. The likelihood of instability of slopes is discussed, and could be interpreted to include instability that might arise from seismic shaking, although this is never discussed as a separate or special case.

Building Regulations 1992 and the Building Code

The first schedule to the Building Regulations requires that buildings, along with sitework (the definition of 'sitework' includes earthworks), shall meet performance requirements for the serviceability and ultimate limit states. Account is to be taken of a variety of loading conditions, including: self weight, imposed gravity loads, earth pressure, water, and earthquake loads. Due allowance is to be made for variations in the properties of the materials and the characteristics of the site. Where there is impact on structures from slope instability, it may be appropriate to apply NZS 4203 seismic loadings, adjusted for topographic amplification effects. The main objective of the regulations is to safeguard people and property from injury, loss of amenity, and physical damage in the event of failure.

In the Approved Document for the New Zealand Building Code (BIA, 2000), the definition for good ground specifically excludes ground which could forseeably experience movements of 25mm or greater. This includes land instability, a similar definition is contained in NZS 3604 - Light Timber Framed Buildings. The section on 'Verification method for foundations' assumes general slope stability, and notes that assessment is outside the scope of the verification method. The superseded 'Verification method for foundations' (BIA, 1993) was much more specific. Under section 3.2.1 Slope stability shall be analysed using unfactored loads. Slopes include unsupported earth faces, banks and vertical ground profiles. Under Section 3.2.2, permanent slopes were to have a factor of safety against instability of no less than 1.5. Section 3.2.3 stated that the factor of safety for temporary slopes was to be evaluated for each specific case, having regard to confidence in the soil and rock data and the consequence of failure.

Draft Earthquake Loading Standard

Section 1.1.1 of the Draft ASiNZS 1170.4 (DR PPCD 8) limits the scope to exclude the effects of slope instability and/or liquefaction resulting from ground shaking. However, the draft code does contain the results of new probabilistic seismic hazard analyses for New Zealand that may be appropriate for inputs into seismic slope stability analyses. In addition, guidelines for design annual exceedance probability events for different types of structures (e.g. normal, essential) are contained in AS/NZS 1170.0, which could be applied to the seismic design of slopes.

NZ Codes of Practice for Subdivisions and Earthfills

The codes of practice for Urban Land Subdivision (SANZ, 1981) require that a soils engineer shall be appointed to carry out assessments of slope stability. The subdivision code notes that 'in most cases, it is unnecessary or impracticable to measure quantitatively the factor of safety of a slope against shear failure. Maximum slopes of cuts and fills may be determined by the soils engineer from experience and from observation of slopes in the vicinity.' And 'Where necessary or a precedent is not available, a special soils engineering investigation should be carried out...' (SANZ, 1981). The code on Earthfill for Residential Development (SANZ, 1989) contains similar clauses to the land subdivision code. However there is an additional clause that specifically relates to seismic design: 'In the assessment of slopes, account should be taken of possible future changes in subsurface seepage conditions and of the effects of earthquake'.

STATE OF PRACTICE

It is difficult to gauge the state of practice in New Zealand, as different Territorial Authorities and consultants have their own quality standards, design criteria, investigation procedures and analytical approaches. This will of course vary throughout the country, dependent on different geological materials, seismic settings, and requirements of the particular authority. An attempt is made to summarise the general approach to seismic slope assessment by reviewing the results of a questionnaire published in the New Zealand Geomechanics News (NZGN, 1994). While this questionnaire is now almost ten years old, it is likely that the responses are relevant today. The questionnaire relates to land development.

The following questions were included in the questionnaire:

1) Is a numerical slope stability analysis that produces a factor of safety always necessary?
2) A minimum factor of safety of 1.5 is acceptable for the following conditions: (conditions not repeated)
3) What factor of safety is required for earthquake loading?
4) What earthquake return period is required?

Responses received to question 1) indicate that most territorial authorities and consultants consider that numerical slope stability analysis is not always required, however some consultants point out that such an analysis is usually required to design remedial works, and a sensitivity analysis can be very useful.

The responses to 2) suggested that both territorial authorities and consultants considered the design factor of safety to be a site-specific issue. Consultants indicated that the factor of safety adopted depends on the confidence of site investigation, parameters chosen and so on.

The answers to 3) and 4) showed wide variation, amongst territorial authorities and consultants. The responses are indicated in Figure 1.

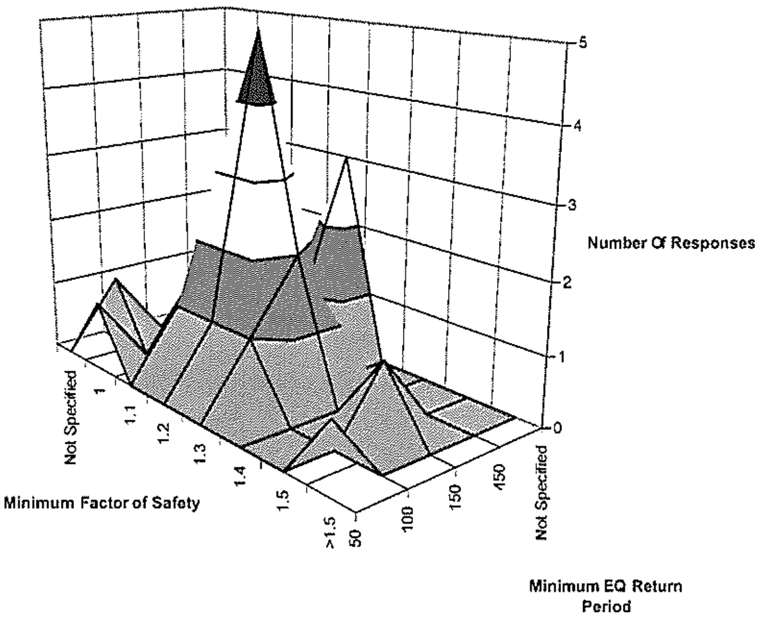

Figure 1. NZGS Slope Questionnaire Responses.

Figure 1 shows that the majority of territorial authorities and consultants consider that seismic slope design should assume a 150 year earthquake return period, and a minimum factor of safety of 1.2. What is not obvious in this figure, is that two responses indicated a minimum factor of safety of 1 for a 450 year return period, and one response indicated a minimum factor of safety of 1.2 for a 450 year return period.

The following interpretations are made from the questionnaire:

- Slopes are not routinely assessed quantitatively for earthquake effects. This therefore implies that reliance is made of past experience of performance of similar slopes under similar ground motions.
- There is a lack of consistency in approach to choosing ground motion return periods, and acceptable factors of safety.

Summary and Comments

The general state of practice for seismic slope design appears to include the following:

- Non-analytical assessment of a slope by an 'experienced' professional, or 'might be okay' approach.
- Limit equilibrium analysis. Target a static FoS $\geq$ 1.5, and assume that the seismic load case will achieve an adequate factor of safety (e.g. FoS $\geq$ 1.2), or 'hope for the best'.
- Limit equilibrium analysis. Undertake a pseudo-static assessment, using a seismic coefficient based on Seed's (1979) and others criteria for embankment dams.
- Limit equilibrium analysis. Undertake a pseudo-static assessment, using a seismic coefficient based on the design basis return period earthquake. This appears to be seldom done in practice.

The design basis earthquake (DBE) for safety (the ultimate limit state) under AS/NZS 1170.0:2002 for normal structures is an annual probability of exceedance (AEP) of 1/500. This is consistent with the 450-500 year return period event, and a factor of safety $\geq$ 1. The DBE for serviceability is considered to be an event which is likely to be experienced during the life of the structure, and so a 50-150 year return period event and a factor of safety of somewhat greater than the ultimate limit state could be justified, say 1.2 to 1.3. This increased factor of safety is to reduce deformations to that which should not cause loss of amenity. The actual design events and factor of safety obviously depend on the type of structure and risk to people and property. Crawford and Millar (1998) suggest a minimum factor of safety of 1.5 for conditions which may occur during the design life of the structure - 100 years for dwellings and 50 years for retaining structures, while a reduced minimum factor of safety of 1.2 is applicable for extreme conditions (including seismic events).

For embankment design (i.e. not slopes in the context of this paper), it is routine practice in New Zealand to follow the pseudostatic screening criteria proposed by Seed (1979) and Hynes-Griffin and Franklin (1984). If these screening analyses indicate an unacceptable factor of safety, then conventionally a Newmark type displacement analysis (eg. Kramer, 1996) is undertaken. It is likely that in the absence of a more appropriate methodology, most NZ practitioners have applied the same pseudostatic philosophy to routine slope design (not embankments), omitting the Newmark analysis. The pseudostatic design provides some rationale to the design of slope stabilization measures, for example toe berms or re-contouring.

However, the pseudostatic screening processes mentioned above were formulated for triangular embankments, and are not necessarily appropriate for application to hill slopes. Also, the definition of 'acceptable' deformations for embankments (~1000mm) may be an order of magnitude higher than what is tolerable for slope instability affecting structures (~100mm). The allowable magnitude of deformation will depend on the consequences of that deformation for each particular case.

SCEC LANDSLIDE GUIDELINES

Introduction

Under the Seismic Hazards Mapping Act (1990), the State of California requires seismic stability analysis for certain projects, and most counties and cities require static analysis for routine projects. The California Department of Conservation, Division of Mines and Geology, presented guidelines for evaluation of seismic hazards other than surface fault rupture (DMG, 1997). At the request of some of the regulating agencies, two "Implementation Committees" were formed under the banner of the Southern California Earthquake Centre (SCEC) to produce detailed guidelines for evaluation of liquefaction and slope stability. The following sections discuss the methodology and application of principles to New Zealand conditions contained in the guidelines (SCEC, 2002).

While the guidelines make clear that the function is not to provide a 'cookbook approach' to slope stability, it is still useful to compare a snapshot of the USA approach to the state of practice in New Zealand. The committee notes that it was not possible to reach consensus on seismic slope stability and acceptable seismic displacement, due to the departure from existing practice.

Background to Guidelines

The usual approach in southern California was based on the pseudostatic procedure modified from the recommendations of Seed (1979). The procedure uses a seismic coefficient k = 0.15 and FoS 21.15. The method was based on Makdisi-Seed displacement analyses (Makdisi and Seed, 1978) to produce embankment dam slope deformations of 1 m under a M 8.25 earthquake. Some agencies allowed a FoS 2 1.10.

The guidelines' approach is to undertake a pseudostatic analysis as a screening analysis for slopes within identified hazard zones, and undertake a Newmark-type deformation analysis on those slopes failing the screening criteria. The main refinement in the guidelines is the development of a rational approach to the selection of an appropriate seismic coefficient applicable to hill slopes, rather than embankments. The seismic coefficient selection approach is discussed below.

Selection of Seismic Coefficient

The selection of the seismic coefficient (k) is based on a deaggregated probabilistic seismic hazard, of the type recently undertaken for New Zealand (Stirling et. al., 2000). It is required to:

- Analyse the maximum horizontal acceleration (MHA) at the site for a rock site condition (MHA_r) and a 475 year return period ground motion.
- Analyse the mode magnitude ($\bar{M}$) and mode site-source distance ($\bar{r}$) of the earthquake sources that contribute most significantly to the 475 year MHA

The seismic coefficient is then taken as:

$$k_{eq} = f_{eq} \times \left(MHA_r / g \right) \qquad (1)$$

wheref,, is a factor related to site seismicity, which can be determined from charts. Two cases are presented in Figure 2 for threshold displacements (u = 5 cm, or u = 15 cm).

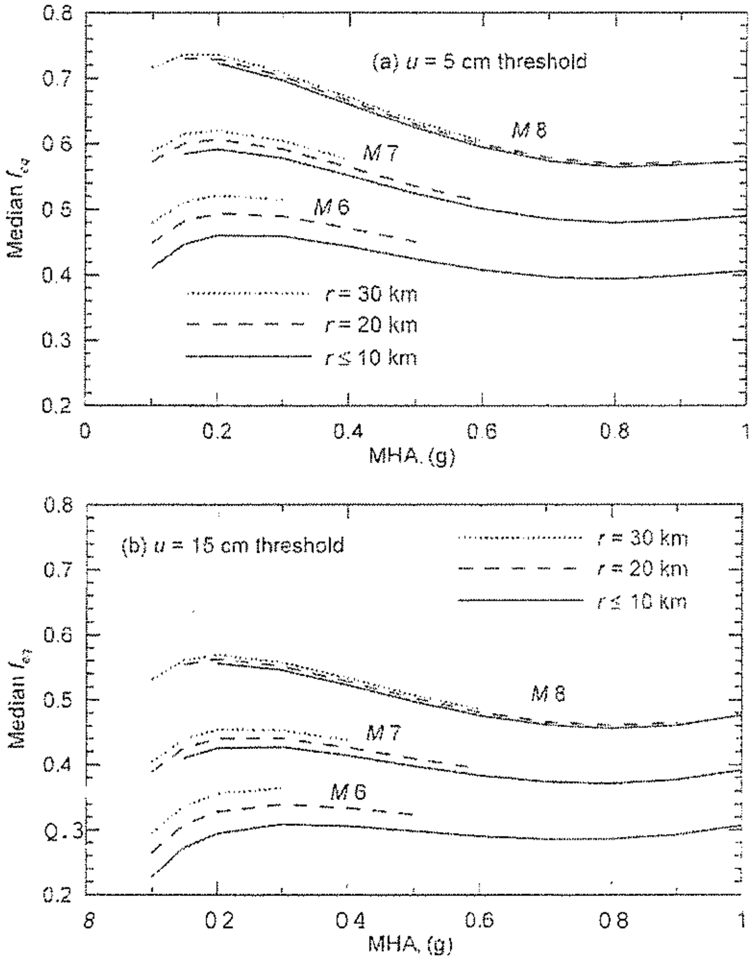

Figure 2. Required Values of f_{eq} as function of MHA_r and Seismological Condition. From SCEC (2002).

For details of the methodology used to determine the relationship for k_{eq}, the reader is referred to SCEC (2002).

Pseudostatic Analysis – Screening Criteria

The pseudostatic analysis is then undertaken in the usual way, with k_{eq} (horizontal) input into the slope model. It is considered that the screening analysis is satisfied if the pseudostatic factor of safety is greater than unity. Using the SCEC methodology, the particular displacement criteria (5 cm or 15 cm) must be considered along with the factor of safety.

If the factor of safety is not greater than unity, then a slope displacement analysis should be performed. Suggested methodologies for a displacement analysis are given in SCEC (2002), and include a Makdisi-Seed type analysis. It is recommended that account be taken of the fact that the Makdisi-Seed analysis was derived for embankments, and an appropriate procedure needs to be taken to account for the crest acceleration of an embankment versus a hillside.

SOME NEW ZEALAND EXAMPLES

Some frequent NZ slope stability situations were analysed, using the SCEC recommendations. Analyses were undertaken using limit equilibrium software. Fictional slope models were set up for Auckland and Wellington, using some typical soil parameters, an 8m high slope, and 475 year return period ground motions. Seismic parameters were derived as shown in Table 1. Shear strength parameters did not include any strain-softening effects, although it may be appropriate to reduce peak strengths for the seismic case.

Location	MHA,[1] (g)	$\bar{M}$[2]	$\bar{r}$[2] (km)	f_{eq}[3] (for u = 5cm)	k_{eq}[4] (for u = 5cm)
Auckland	0.13	6.2	62	0.50	0.07
Wellington	0.44	7.3	< 10	0.57	0.25

2

3
4

The assumptions and results of these stability assessments are shown in Table 2.

Table 2. Summary of Slope Analyses.

Material Description[1]	Slope angle (H:V)	c' (kPa)	ϕ' (°)	γ (kN/m^3)	PWP Ratio R_u	k_{eq}	FoS Static ■ Seismic	% Change
Auckland RW Waitemata Group	2.5:1	8	28	18	0.3	0.07	1.61/1.36	-16
Wellington CW Greywacke	1:1	20	29	18	0.3	0.25 0.11[2]	1.45/1.01 1.45/1.23	-30 -15
Wellington SW Greywacke	1:2	50	36	20	0.3	0.25	2.28/1.58	-31
Sand (Wellington seismicity)	2.5:1	0	35	18	0.1	0.25	1.65/0.92	-44
Clayey Sand (Wellington seismicity)	2.5:1	5	32	18	0.3	0.25	1.58/0.89	-44

CONCLUSIONS & RECOMMENDATIONS

The New Zealand state of practice lacks consistency of approach in terms of selection of design ground motion return periods, development of seismic coefficients, and acceptable factors of safety. Using the latest available seismic hazard information for the country, it is likely that many commonly designed slopes will pass the SCEC screening criteria for a 475 year return period ground motion. There would seem to be little point in routinely checking a shorter return period ground motion for a serviceability type event, with a higher acceptable factor of safety.

The results of several basic slope stability analyses suggest that slopes in Auckland (and by inspection Northland and Dunedin) that have an 'adequate' static factor of safety will seldom fail the SCEC seismic screening criteria. Often it may not be necessary to test the seismic case based on the prior experience of a practitioner with similar slopes and seismic conditions. For other areas of the country with higher seismicity, it would be prudent to undertake seismic stability screening for all slopes where a numerical analysis is warranted. Soils that have lower effective cohesion tend to be more vulnerable to loss of stability under earthquake loading.

While this paper was written in the context of residential developments, design for strategic services such as important highway routes, or essential post-disaster relief services may require increased return period ground motions. Therefore screening investigations and analyses are likely to be required for important facilities/structures in areas of low to moderate seismicity, while they may not be required for residential developments.

The recommended procedure for slopes requiring numerical analysis (see guidelines of Crawford & Millar, 1998) is as follows:

1) Consider screening out sites from seismic assessment that have a 475 year return period ground motion bedrock PGA ≤ 0.13g (see Draft AS/NZS 1170.4, 2003). Important facilities which require longer return period earthquake design should not be screened out.

2) Determine the earthquake magnitude most contributing to this ground motion for the particular city or region, and site-source distance (see deaggregated seismic hazard in Stirling et. al., 2000).
3) Determine the seismic coefficient for the appropriate displacement threshold, based on SCEC (2002) procedure.
4) Sites having a factor of safety greater than unity may be considered acceptable.
5) Sites which the fail the screening criteria (475 year motions) should have deformation analyses undertaken, or remedial design.

Any numerical assessment of slope stability under seismic conditions must develop a sound approach that considers suitable displacement criteria and evaluation of site (or region) specific seismicity appropriate to the local geology. The seismic screening methods proposed by the SCEC offer a rational and consistent approach to the determination of seismic coefficients for hill slopes in land development.

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