

---

# Beyond Mononobe-Okabe: Evaluating Seismic Earth Pressures in ‘ $c-\phi$ ’ Materials under High PGA Conditions

*U. Fauzi, N. Clendon & N. Schumacher*

Torlesse Limited, Wellington, New Zealand.

## ABSTRACT

The 2022 New Zealand National Seismic Hazard Model and Draft TS 1170.5:2024 indicate substantial increases in peak ground accelerations (PGAs) across Wellington and other high-seismic regions, highlighting critical limitations in the long-used Mononobe–Okabe (M–O) method for computing seismic earth pressures. When  $k_h$  exceeds approximately 0.3–0.4 g and steep backslopes are present, M–O frequently produces mathematically unstable or “no-solution” results. The recent SESOC (2024) guide on Timber Pole Retaining Walls provides some simple approaches to managing these aspects by assuming the material will simply ‘overtop’ the wall and, therefore, minimise the loading. A similar approach can be seen in MBIE/ NZGS Module 6 (Appendix F) when  $k_h$  values are limited. Our analysis reveals that adopting capped  $k_{ae}$  values, as currently proposed in MBIE/NZGS Module 6 and SESOC (2024), may underestimate wall demands under high PGA conditions. We have looked at a range of options, including Shukla (2015), Wood (2022), simplified geometric or surcharge adjustments, Trial Wedges, and Limit Equilibrium (LE) approaches for calculating seismic earth pressures on retaining walls in ‘ $c-\phi$ ’ materials.

Comparative assessments demonstrate that methods incorporating cohesion and realistic failure geometry yield more stable, physically meaningful estimates of seismic earth pressure. The paper presents case results and provides practical recommendations for selecting suitable analysis methods based on site conditions and desired performance objectives.

## 1 INTRODUCTION

Lateral earth pressure is crucial for the design and stability of various retaining walls. It impacts the selection and properties of the retaining wall, such as type, depth, spacing, material type, etc. The static earth pressure is well-known, typically estimated using two classical soil mechanics theories: (a) Coulomb theory, formulated in 1776 based on a limit equilibrium method that considers a planar failure wedge, and (b) Rankine theory, developed in 1857 based on the stress field solution assuming a cohesionless soil, a non-battered and frictionless vertical wall, and a horizontal backfill surface, with failure occurring along a planar

surface within the soil mass. To this day, these two methods have been used in practice in designing retaining walls for static conditions without earthquake demands. These methods have been recommended in MBIE/ NZGS Module 6 (2021), while SESOC (2024) has recommended the use of a modified Coulomb theory based on Mayniel for wall friction and Muller-Breslau for non-horizontal backfill and non-vertical walls.

During earthquakes, however, an additional dynamic lateral force acts on retaining walls due to the inertial forces generated within the soil mass by ground accelerations. This additional load is known as seismic earth pressure. Seismic earth pressure has been studied extensively since its first closed-form solution, published independently by Okabe in 1924 and Mononabe and Matsuo in 1929. The solution is famously called the Mononabe-Okabe (M-O) method.

## 2 THE MONONABE-OKABE METHOD AND ITS LIMITATIONS

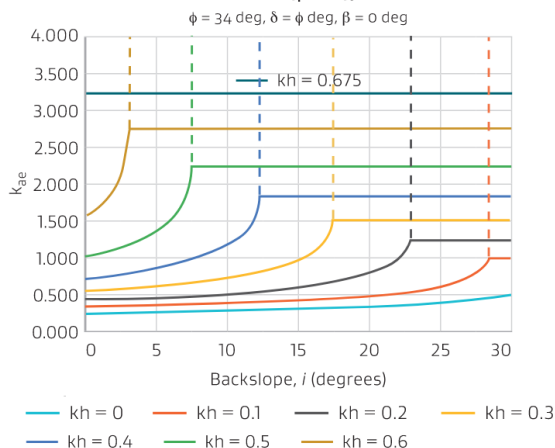
The M-O method extends Coulomb's theory by Okabe (1924) and extends Rankine's theory by Mononabe and Matsuo (1929), introducing horizontal ( $k_h$ ) and vertical ( $k_v$ ) seismic coefficients to represent the inertial forces acting on the potential failure wedge behind a retaining wall.

The M-O method assumes the following:

- The retaining wall is assumed to yield (i.e., move away from the backfill).
- The soil mass behind the wall is considered a rigid body, implying that  $k_h$  and  $k_v$  are uniform in the entire failure wedge.
- The backfill soil is assumed to be cohesionless and dry.
- The failure surface within the soil mass is assumed to be planar, extending from the base of the wall to the surface of the backfill, similar to the failure mechanism in Coulomb's theory.

The M-O method has become popular due to most of New Zealand being an earthquake-prone area and the simplicity of M-O equations. However, the assumptions above, particularly the rigid soil mass and uniform acceleration, may not always accurately represent the complex dynamic behaviour of soil and retaining structures during strong earthquakes.

**Walls with vertical back-face ( $\beta = 0$ ), full interface friction ( $\delta = \phi$ ):**



**Figure 1: Seismic active earth pressure coefficient ( $K_{ae}$ ) from M-O method: dash line – NHCPR, increases exponentially; and solid line – Module 6, limited the value. (Chart is modified from Appendix F of Module 6).**

While the M-O method is recommended in both MBIE/ NZGS Module 6 and the SESOC guideline, it exhibits a significant limitation, especially when the peak ground acceleration (PGA) is high, and the geometry has a back slope. When calculating the seismic active earth pressure coefficient ( $K_{ae}$ ), there are two schools of thought, one where the loads increase exponentially, as in NHCPR (2008). The second is that it reaches a limit in soil loading and, therefore, no longer increases as in Module 6 and the SESOC guideline. The calculated  $K_{ae}$  for the two methods is shown in Figure 1.

Authors consider limiting  $K_{ae}$  due to 'no solution' as per Appendix F of Module 6 and Appendix C of SESOC is only applicable if the soil is anticipated to fail overtop of the wall, i.e., a coarse-grained material displaying no cohesion (including apparent

cohesion such as roots and interlocking) will fail in shallow slope failures, sloughing to the bottom of the slope. This assumes the failure thickness is limited, does not impact the wall and, most importantly, a deeper

global stability issue does not occur. As the slope angle increases, the likelihood of a deep-seated failure also increases (assuming homogenous soil).

The limiting  $K_{ae}$  in Module 6 is shown in two equations below to accommodate high PGA and back slope:

$$k_h \leq \tan \phi \quad (1)$$

where  $k_h$  = horizontal seismic coefficient;  $\phi$  = friction angle.

$$\phi - \theta \leq i \quad (2)$$

where  $\theta$  = seismic inertia angle =  $\tan^{-1} \left( \frac{k_h}{1-k_v} \right)$ ;  $i$  = infinite slope angle.

### 3 PROPOSED ALTERNATIVE METHODS

Following the release of MBIE/ NZGS Module 1 Version 1 in 2021, the PGA for retaining wall design has increased significantly in the Wellington area. Based on the author's experience, it is now more common in the Wellington area that M-O method results in 'no solution'.

The recent public consultation draft of TS 1170.5:2024, released last year, will increase the PGA for the Wellington area further. Table 1 shows the evolution of PGA in the Wellington area over the last decade.

*Table 1: ULS PGA values in Wellington based on MBIE/ NZGS Module 1 Ver. 0, Ver 1, and TS 1170.5:2024*

| Case* | Module 1 Ver. 0 (2016)** | Module 1 Ver 1 (2021)**        | TS 1170.5:2024***                    |
|-------|--------------------------|--------------------------------|--------------------------------------|
| IL1   | 0.17 – 0.23g             | 0.28g<br>(22% – 65% increase)  | 0.32 – 0.39g<br>(14% – 39% increase) |
| IL2   | 0.34 – 0.45g             | 0.68g<br>(51% – 100% increase) | 0.71 – 0.91g<br>(4% – 34% increase)  |
| IL3   | 0.44 – 0.59g             | 0.91g<br>(54% – 107% increase) | 0.93 – 1.25g<br>(2% – 37% increase)  |
| IL4   | 0.61 – 0.81g             | 1.27g<br>(57% – 108% increase) | 1.2 – 1.79g<br>(-6% – 41% increase)  |

\*For design life 50 years. \*\* Subsoil Class A-E. \*\*\* Site Class I-VI

It should be noted that TS 1170:2024 classifies soil into seven classes, not five, as in Module 1. The basis for PGA in TS 1170.5:2024 is from the latest Earth Sciences NZ - National Seismic Hazard Model (NSHM, 2022). The disaggregation of PSHA in NSHM 2022 shows that the Hikurangi subduction contributes more than 50% to the updated PGA. The proposed use of deterministic seismic hazard analysis (DSHA) to limit the PGA in TS 1170.5:2024 is unlikely to be applied since Bradley (2025) recommends embracing more advanced analysis methods than simple pseudo-static approaches. Therefore, a simple method like M-O method may not be applicable when TS 1170.5:2024 PGA is enforced.

In addition, the M-O method is not recommended for  $k_h$  values greater than 0.4g (Nakamura 2006) based on centrifuge model tests of gravity walls.

It is unknown when TS 1170.5:2024 will replace the current standard. However, geotechnical engineers have been aware that the PGA for Wellington will increase significantly again. Therefore, the question arises: Is the recommendation to limit the  $K_{ae}$  realistic/ appropriate for design, especially with a high PGA? Are there

any other methods that can be used? The authors research several options that do not use a sophisticated method like a finite element or time-histories analysis, which are discussed in the following sections.

### 3.1 Option 1: Modified M-O with $c-\phi$ soil backfills

Numerous researchers (Prakash and Saran, 1966; Das and Puri, 1996; Saran and Gupta, 2003; Gosh and Sengupta, 2012; among others) have proposed a solution for seismic active earth pressure supporting  $c-\phi$  backfill extending the current M-O method. It is well known that the presence of cohesion decreases the seismic earth pressure, which is applicable for residual soil and weathered greywacke rock that is common in Wellington. However, there is no consensus or guideline in New Zealand on which  $c-\phi$  equations should be used. Wood (2022) has studied extensively on earth pressures on retaining walls, and his recommendation is to use the Shukla (2015) method. The Shukla method considers cohesion, friction angle, backfill slope angle, surcharge on the backfill, tension crack, wall-backfill friction and adhesion, inclination of the wall-soil interface, and both horizontal and vertical acceleration. In addition to the closed-form solution by the Shukla method, Wood (2022) also provides a simplified method for determining the critical angle by iteration of the maximum value of  $K_{ae}$  using a Microsoft Excel solver add-in.

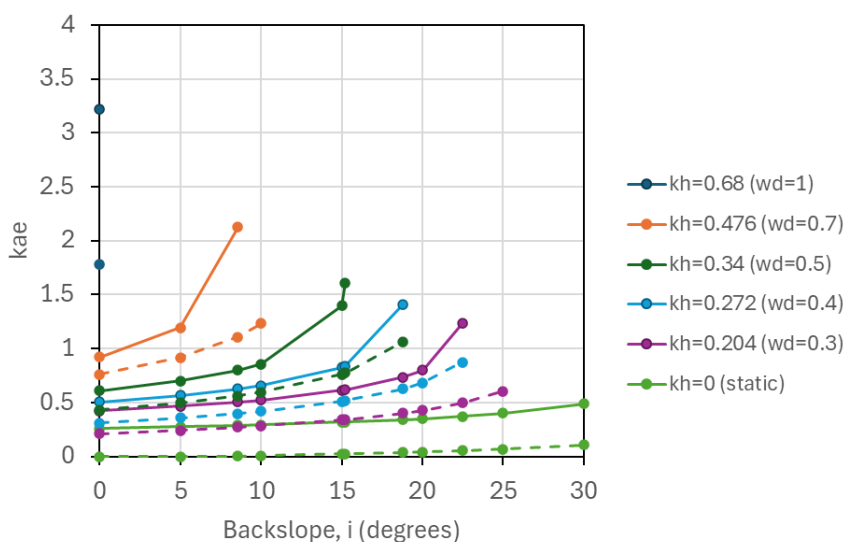


Figure 2: Shukla (including cohesion, dashed line) versus M-O (solid line) for Wellington. Note that  $K_{ae}$  of  $k_h=0.68$  is actually based on  $k_h=0.675$  due to 'no solution'.

weight ( $\gamma$ ) = 18 kN/m<sup>3</sup>, cohesion ( $c$ ) = 5 kPa, interface angle ( $\delta$ ) = friction angle ( $\phi$ ) = 34°, and the retained height = 5m. From this figure, it is clear that:

- Adding cohesion can reduce  $K_{ae}$ .
- No solution is offered for Shukla and M-O on high PGA, i.e. when  $k_h = 0.34$  and  $i = 20^\circ$ .
- When adding cohesion, the backslope angle can be slightly increased further (beyond M-O method) to result in a solution (i.e. the solid line has one less data point compared to the dashed line).

### 3.2 Option 2: Increase Design Wall Height

When there is 'no solution' and the back slope levels off quickly behind the wall, a simple approach by increasing the design height may be suitable, refer illustration in Figure 3. Figure 2 shows that for Wellington PGA with  $W_d = 0.5$  ( $k_h = 0.34$ ) and  $i = 30^\circ$ , the  $K_{ae}$  results in 'no solution', while if we assume flat ground at the back of the retaining wall and increase the retained height, the earth pressure can be calculated

The authors have tried the proposed iteration method (Wood, 2022) with MathCAD software. However, doing iterations depends on the initial value assumed, which can result in 'no solution' if the initial value is incorrect. Therefore, we used the original Shukla equation, although it is a long and complicated equation, it does not require iteration and results in an immediate solution.

The comparison between Shukla and M-O methods can be seen in Figure 2 for various wall displacement factors ( $W_d$ ) cases in Wellington for an IL2 retaining wall. It is assumed that the unit

using  $K_{ae}$  of 0.61.

### 3.3 Option 3: Apply as a Surcharge

Option 3 is similar to Option 2. Instead of increasing the retained height, the soil above the flat ground behind the retaining wall is assumed as a gravity surcharge acting on the retaining wall. A failure plane of  $45^\circ + \phi/2$  can be used to determine the surcharge area. The illustration can be seen in Figure 4. Surcharge can be modelled as triangular using geotechnical software like Wallap or manually calculated using Boussinesq Theory. A simplified method by averaging the surcharge can also be considered for use in hand calculations.

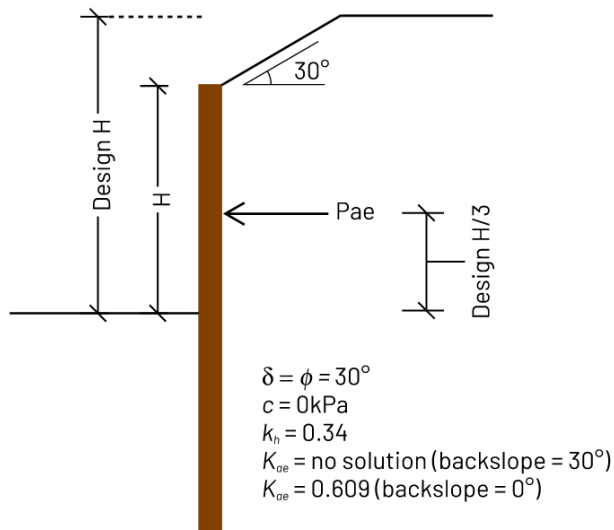


Figure 3: Option 2 – increasing retained height

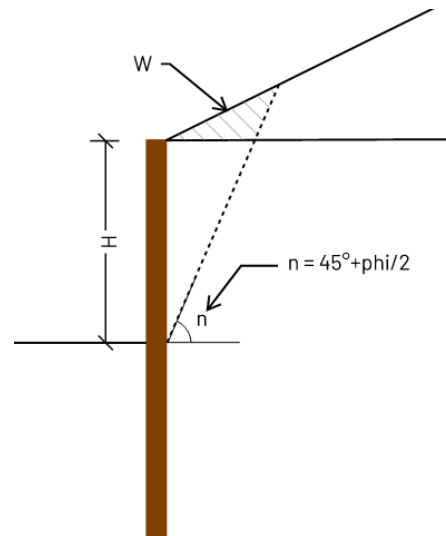


Figure 4: Option 3 – soil above acting as gravity surcharge

### 3.4 Option 4: Trial Wedge

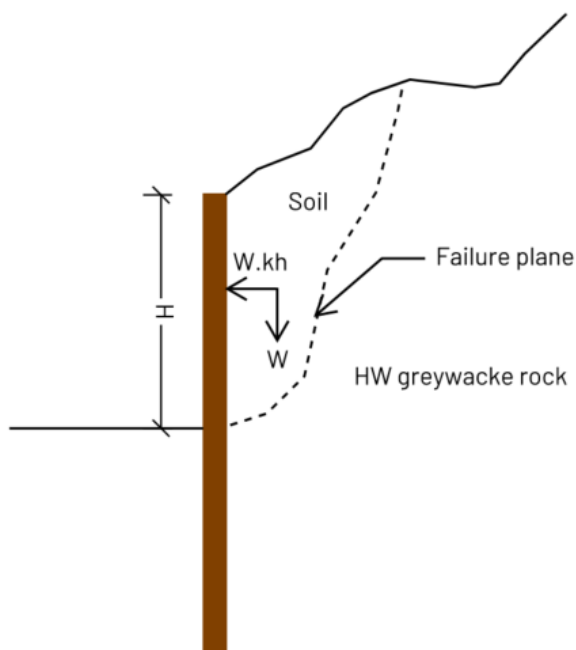


Figure 5: Option 4 – trial wedge method

The trial wedge method provides an alternative approach for calculating seismic active earth pressures when the M-O equation is not applicable. It is particularly useful in situations where the active wedge contains non-uniform soils, cohesive materials, or where the backslope angle varies (FHWA, 2011). However, from our experience with homogeneous soils, the method requires iterative adjustment of the failure angle to determine the critical wedge that yields the maximum seismic earth pressure. This involves calculating the wedge area, weight, failure surface length, and adhesion length, which can become complex and time-consuming. As a result, the method is typically reserved for cases where the failure plane can be reasonably assumed to follow a distinct contrast in strength, such as at the interface between soil and rock.  $P_{ae}$  can be calculated directly without calculating  $K_{ae}$ . Refer to FHWA 2011, Section 11.3.2 for an example calculation. The illustration can be seen in Figure 5.

### 3.5 Option 5: Limit Equilibrium (SLIDE2)

Another option for evaluating seismic earth pressures is to use the limit equilibrium (LE) method. Any standard slope stability program can be adapted for this type of analysis. The process involves applying a seismic coefficient to the model and iterating an external force at mid-height of the wall until the factor of safety equals 1, which defines the seismic active force,  $P_{ae}$ . The method is described in more detail in NCHRP Report 611 (NCHRP, 2008) and re-modified as below:

1. Set up the model geometry, groundwater profile, and design soil properties.
2. Choose GLE/Morgenstern-Price slope stability method.
3. Choose an appropriate sliding surface search scheme.
4. Apply the earth pressure as a boundary force on the face of the retained soil. The location of the force is assumed at one-third from the base ( $1/3 H$ , where  $H$  is the retained soil height) for static cases. The static case is used for the verification against Coulomb or Rankine methods. For seismic cases, the location can be reasonably assumed at mid-height ( $1/2 H$ ) of the retained soil with an angle equal to the wall/ backfill interface friction angle.
5. Complete design iteration using the support back analysis function.
6. Verify design assumptions and material properties by examining the loads on individual slices in the output.

The earth pressure coefficients,  $K_a$  and  $K_{ae}$ , can be calculated by using:

$$K_{ae} = \frac{2P_{ae}}{\gamma H^2} \quad (2)$$

We have provided several examples of using SLIDE2 in determining seismic earth pressures and coefficients, compared against the static case (Coulomb). Table 2 below presents the solution for  $K_{ae}$  without limiting its value to ‘no solution’, as recommended by Module 6 - Appendix F.

In total 20 cases have been calculated based on M-O, Shukla, and SLIDE2. The results can be seen in Table 2.

*Table 2: Outputs from SLIDE2*

| Example     | Height<br>(m) | Back<br>Slope (°) | Cohesion<br>(kPa) | Seismic<br>Load ( $k_h$ ) | $K_a$ or $K_{ae}$ value |        |        | Figure<br>No. |
|-------------|---------------|-------------------|-------------------|---------------------------|-------------------------|--------|--------|---------------|
|             |               |                   |                   |                           | M-O*                    | Shukla | SLIDE2 |               |
| Flat Ground | 2             | 0                 | 0                 | 0                         | 0.33                    | 0.33   | 0.33   | Figure 6      |
| Flat Ground | 2             | 0                 | 0                 | 0.2                       | 0.47                    | 0.47   | 0.4    |               |
| Flat Ground | 2             | 0                 | 0                 | 0.34                      | 0.62                    | 0.62   | 0.48   |               |
| Flat Ground | 2             | 0                 | 0                 | 0.6                       | n/a                     | n/a    | 2.93   | Figure 8      |
| Flat Ground | 2             | 0                 | 0                 | 0.6-<br>limited**         | n/a                     | n/a    | 1.13   | Figure 9      |
| Flat Ground | 3             | 0                 | 0                 | 0                         | 0.33                    | 0.33   | 0.2    |               |
| Flat Ground | 3             | 0                 | 0                 | 0.2                       | 0.47                    | 0.47   | 0.29   |               |
| Flat Ground | 3             | 0                 | 0                 | 0.34                      | 0.62                    | 0.62   | 0.47   |               |

| Example               | Height<br>(m) | Back<br>Slope (°) | Cohesion<br>(kPa) | Seismic<br>Load ( $k_h$ ) | $K_a$ or $K_{ae}$ value |        |        | Figure<br>No. |
|-----------------------|---------------|-------------------|-------------------|---------------------------|-------------------------|--------|--------|---------------|
|                       |               |                   |                   |                           | M-O*                    | Shukla | SLIDE2 |               |
| Flat Ground           | 3             | 0                 | 0                 | 0.6                       | n/a                     | n/a    | 2.19   |               |
| Option 1 - Cohesion   | 3             | 0                 | 3                 | 0                         | 0.33                    | 0.23   | 0.12   |               |
| Option 1 - Cohesion   | 3             | 0                 | 3                 | 0.2                       | 0.47                    | 0.36   | 0.28   |               |
| Option 1 - Cohesion   | 3             | 0                 | 3                 | 0.34                      | 0.62                    | 0.49   | 0.38   |               |
| Option 3 – Back Slope | 3             | 34                | 3                 | 0                         | n/a                     | 0.47   | 0.31   |               |
| Option 3 – Back Slope | 3             | 34                | 3                 | 0.2                       | n/a                     | n/a    | 1.37   |               |
| Option 3 – Back Slope | 3             | 34                | 3                 | 0.34                      | n/a                     | n/a    | 2.61   |               |
| Option 3 – Back Slope | 3             | 34                | 3                 | 0.34-<br>limited**        | n/a                     | n/a    | 1.02   |               |
| Option 3 – Back Slope | 3             | 34                | 0                 | 0                         | n/a                     | n/a    | 0.90   | Figure 7      |
| Option 3 – Back Slope | 3             | 34                | 0                 | 0.2                       | n/a                     | n/a    | 1.82   |               |
| Option 3 – Back Slope | 3             | 34                | 0                 | 0.34                      | n/a                     | n/a    | 3.47   |               |
| Option 3 – Back Slope | 3             | 34                | 0                 | 0.34-<br>limited**        | n/a                     | n/a    | 1.58   |               |

\*when  $k_h = 0$ , M-O method is equal to Coulomb method

\*\*Limited is referring to forcing the failure through the toe of the wall – Refer to Section 3.6 below.

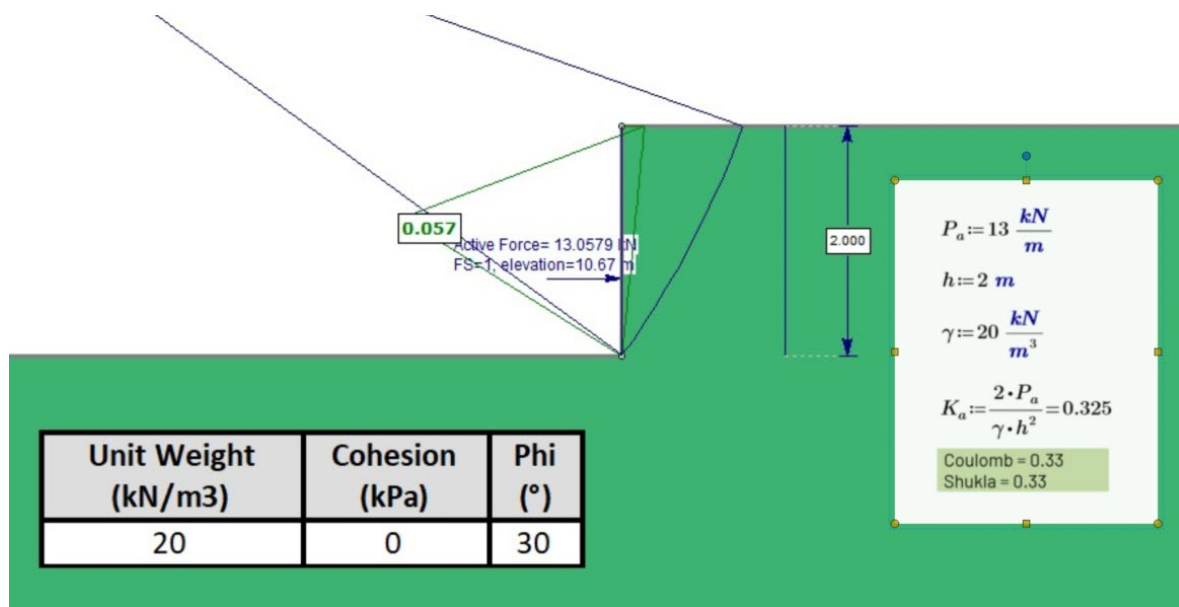


Figure 6: Example 1 –  $K_a$  (static) verification against Coulomb and Shukla

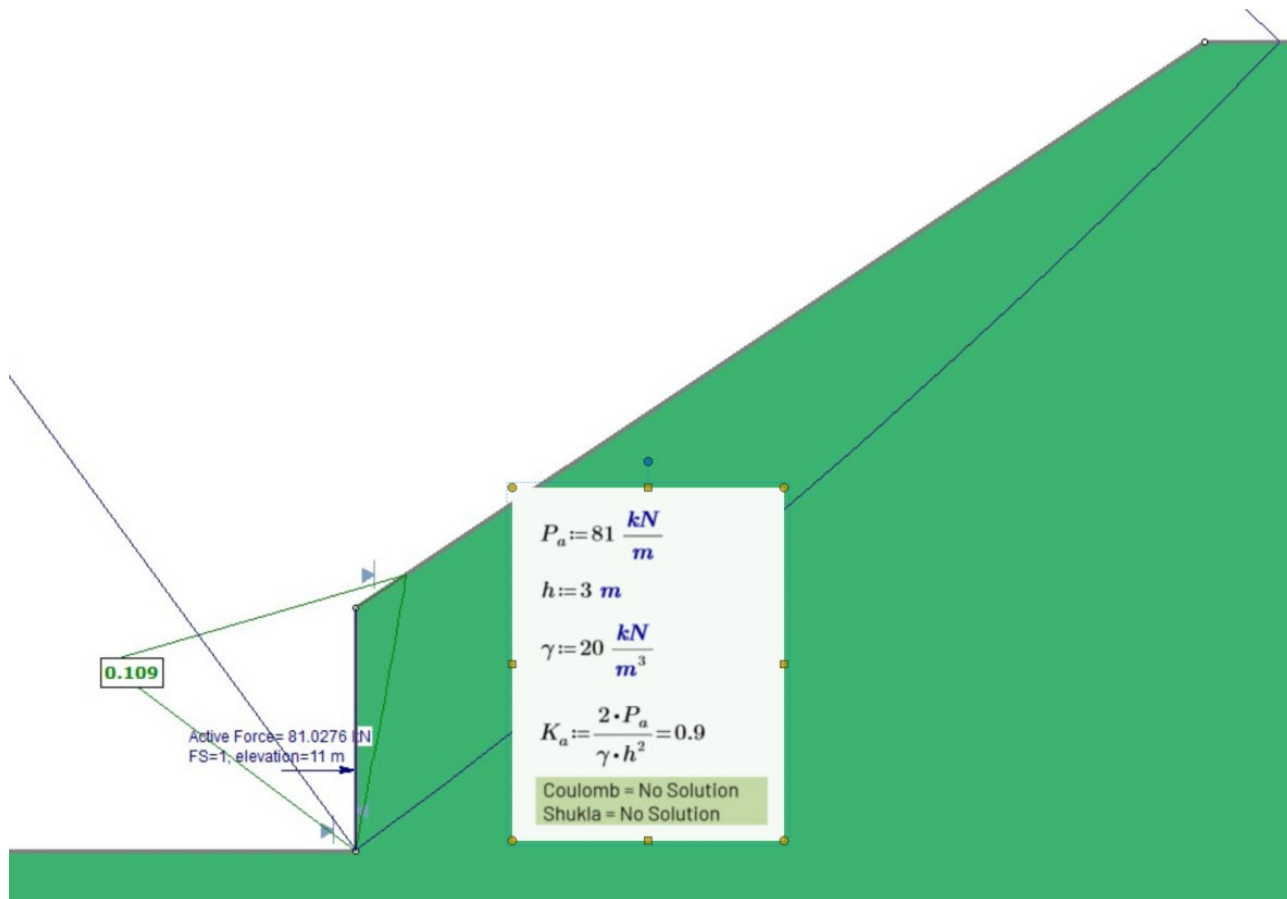


Figure 7 –  $K_{ae}$  calculation (seismic) SLIDE2 vs Coulomb and Shukla

The results demonstrate that the LE approach provides a viable and practical alternative to the M-O method, particularly when facing the ‘no solution’ issue under high PGA and steep backfill scenarios.

- In Figure 6, the static earth pressure coefficient ( $K_a$ ) computed using SLIDE2 closely matches the classical Coulomb solution, validating the model setup.
- Back slope increases  $K_a$  and  $K_{ae}$  significantly, especially under high seismic loads.
- Higher PGA increases  $K_{ae}$  rapidly.
- Cohesion has strong influence in reducing seismic earth pressure, especially in SLIDE2 and Shukla.
- The  $K_a$  obtained from SLIDE2 provides a solution even when the Coulomb, M-O and Shukla method reaches its mathematical limitation, i.e., results in a ‘no solution’, i.e. Figure 7.
- Notably with high back slopes SLIDE2 is able to calculate an active pressure, and therefore a  $K_a$  or  $K_{ae}$  value can be calculated.

These results suggest that the LE approach can offer a more flexible and reliable solution for calculating seismic earth pressures, especially in high seismicity regions such as Wellington, where the revised PGA values under TS 1170.5:2024 could render conventional methods inadequate or no longer applicable once  $k_h$  values become greater than 0.4g. However, the method requires careful setup of the model geometry, appropriate boundary conditions, and validation against known static solutions to ensure robustness.

For cases with high seismic loading ( $k_h > 0.4$ ), some restrictions have been applied to the slope stability model. See section 3.6 below for more details.

### 3.6 Seismic Loading

As the applied  $k_h$  increases ( $>0.4g$ ), the theoretical displaced soil body becomes increasingly large, see Figure 8 below. However, this outcome is not consistent with observed wall failure. Typically, the mobilised soil body or evidence of cracking is contained within a distance equal to the height of the wall. From the authors personal experience and excluding considerations of deep-seated movement such as slope failure or lateral spreading, simply applying  $k_h$  values in excess of  $0.4g$  in either M-O equations or into a slope stability program is unlikely to provide the realistic answer.

As such when establishing the loading location acting on the wall, we recommend forcing the failure through the toe of the wall as shown in Figure 9, and not allowing large circular type failures to develop (as seen in Figure 8).

Limiting the failure area does not remove the need to check for deep seated or global stability issues, such as checking pole embedment length, however, it will provide a more realistic value for seismic earth pressures acting on the wall.

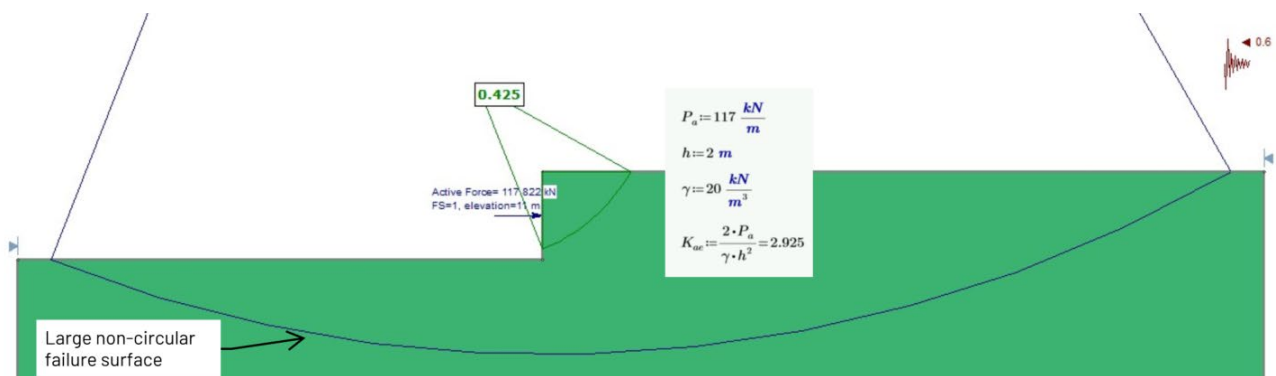


Figure 8 –  $K_{ae}$  calculation for high  $k_h = 0.6$

Where a deep failure surface is allowed to develop, very high active forces are required to achieve equilibrium and therefore high active pressure coefficients as seen above,  $K_{ae} = 2.9$  (based on Figure 8). M-O and Shukla equations do not provide a value once  $k_h$  reaches  $0.6$ . Module 6 would provide a  $K_{ae}$  of approximately  $1.4$ , where the limit of the graphs in Appendix F is reached. LE is able to calculate a lower value, as shown below in Figure 9. However, even in Figure 9 below, the soil body acting on the wall is still large. Further work is required in this area to identify soil loading extents.

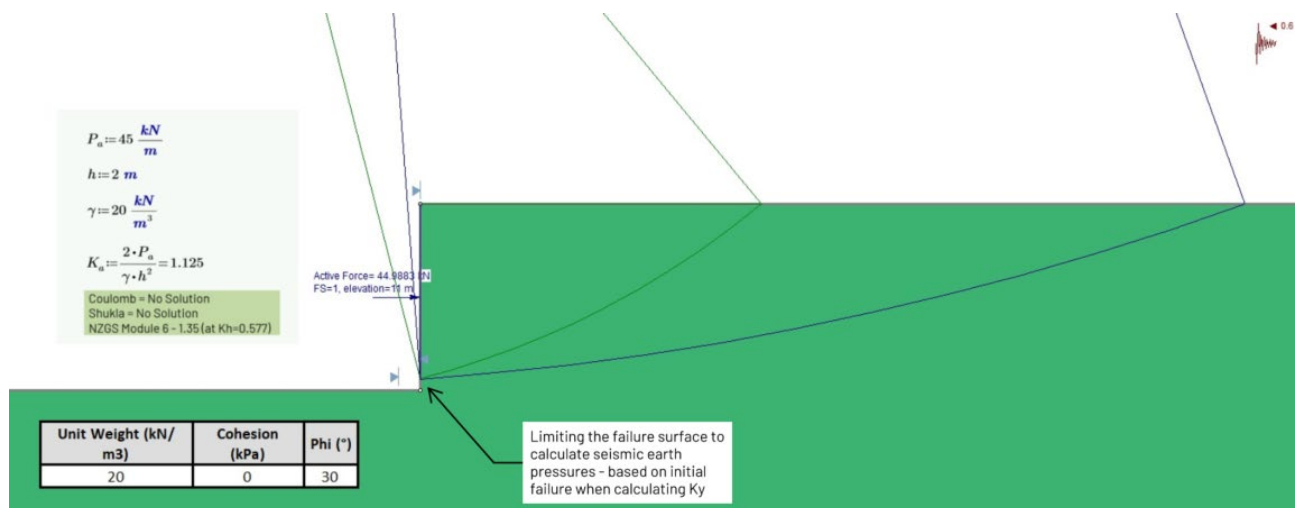


Figure 9 – Limiting failure surface to  $k_y$  for high  $k_h = 0.6$ ,  $K_{ae} = 1.125$

## 4 CONCLUSIONS

The Mononobe-Okabe (M-O) method has long been the default approach for seismic earth pressure calculations in New Zealand due to its simplicity and historical acceptance. However, as seismic hazard models have evolved, particularly with the latest updates from the New Zealand National Seismic Hazard Model (2022) and the proposed TS 1170.5:2024, the resulting increase in peak ground accelerations (PGA) has exposed critical limitations of the M-O method, especially when cohesive soils or steep backslopes are involved. M-O is not considered suitable where  $k_h$  values exceed 0.4g, or in some cases even 0.2g (high back slopes).

This study evaluated several alternative methods, including the Shukla (2015) extension of M-O for materials with cohesion (i.e.,  $c-\phi$ ) materials, design height adjustment strategies, gravity surcharge methods, the Trial Wedge method, and Limit Equilibrium (LE) analyses using slope stability software. Each method offers advantages depending on site-specific conditions and project objectives.

Notably, the LE approach through programs such as SLIDE2 provides a reliable pathway to quantify seismic earth pressures where the M-O method fails, without the need for artificially limiting  $k_h$  or  $K_{ae}$  values. Similarly, methods that consider cohesion in backfill materials, such as the Shukla method, offer more realistic assessments for Wellington conditions where residual soils and weathered greywacke are common.

We feel more work should be undertaken to assess the appropriateness of the LE approach to various cases and also assessing more realistic soil loading values when dealing with high PGA values. Limiting the ‘failure’ extent from high seismic conditions to the toe of the wall is considered a practical solution, however, a concerted effort to compare outputs with some well-established case history is recommended.

Lastly, the authors recommend caution is used when adding cohesion as it can be sensitive to site conditions, moisture, tension cracks etc. As such we would recommend limiting cohesion values as suggested in FHWA (2011).

Overall, engineers should be cautious when applying simple limiting approaches to seismic earth pressure calculations, especially in high seismic demand areas. A more nuanced, site-specific selection of methods — incorporating cohesion effects, surcharge approximations, or numerical limit equilibrium methods — should be considered to ensure safe, economical, and code-compliant designs in the evolving seismic landscape of New Zealand.

## 5 REFERENCES

- Bradley, B.A., 2025. Seismic hazard with deterministic maximum limits: considerations in a new zealand-specific context. *Bulletin of the New Zealand Society for Earthquake Engineering*, 58(1).
- Coulomb, C.A., 1776. Essai sur une application des regles de maximis et minimis a quelques problemes de statique relatifs a l'architecture . *Memoires d'Academie Roy. Pres. Diverssavants*.7
- Das, B.M. and Puri, V.K., 1996. Static and dynamic active earth pressure. *Geotechnical & Geological Engineering*, 14, pp.353-366.
- Gerstenberger, M., Bora, S., Bradley, B., DiCaprio, C., Van Dissen, R.J., Atkinson, G.M., Chamberlain, C., Christophersen, A., Clark, K.J., Coffey, G.L. and de la Torre, C., 2022. New Zealand National Seismic Hazard Model 2022 revision: model, hazard and process overview.
- Ghosh, S. and Sengupta, S., 2012. Extension of Mononobe-Okabe theory to evaluate seismic active earth pressure supporting  $c-\phi$  backfill. *Electronic Journal of Geotechnical Engineering*, 17, pp.495-504.

- FHWA. Kavazanjian, E., Wang, J.N., Martin, G., Shamsabadi, A., Lam, I., Dickenson, S.E. and Hung, C.J., 2011. LRFD seismic analysis and design of transportation geotechnical features and structural foundations-NHI course no. 130094 reference manual geotechnical engineering circular no. 3 (No. FHWA-NHI-11-032). United States. Federal Highway Administration.
- MBIE/NZGS. 2021. Module 6: Earthquake resistant retaining wall design – Version 1.
- MBIE/NZGS. 2021. Module 1: Overview of the geotechnical guidelines – Version 1.
- MBIE/NZGS. 2016. Module 1: Overview of the geotechnical guidelines – Version 0.
- Mononobe, N., 1929. On determination of earth pressure during earthquake. *In Proc. World Engrg. Congress*, Tokyo (Vol. 9, pp. 177-185).
- Nakamura, S., 2006. Reexamination of Mononobe-Okabe theory of gravity retaining walls using centrifuge model tests. *Soils and foundations*, 46(2), pp.135-146.
- NCHRP, Anderson, D.G., 2008. Seismic analysis and design of retaining walls, buried structures, slopes, and embankments (Vol. 611). Transportation Research Board.
- Okabe, S., 1924. General theory of earth pressure. *Journal of the Japanese Society of civil Engineers*. tokz, Japan, 12(1).
- Prakash, S. and Saran, S., 1966. THIRD SYMPOSIUM ON EARTHQUAKE ENGINEERING. *In Proceedings-Symposium on Earthquake Engineering* (p. 229). Sahu Cement Service.
- Rankine, W.J.M., 1857. II. On the stability of loose earth. *Philosophical transactions of the Royal Society of London*, (147), pp.9-27.
- Saran, S. and Gupta, R.P., 2003. Seismic earth pressures behind retaining walls. *Indian Geotechnical Journal*, 33(3), pp.195-213.
- SESOC. 2024. Timber Pole Retaining Walls - SESOC Design Guide.
- Shukla, S. K. 2015. Generalized Analytical Expression for Dynamic Active Thrust from  $c-\phi$  Soil Backfills. *International Journal Geotechnical Engineering*, Vol 9(4), 416-421.
- Standards New Zealand. 2024. Draft New Zealand Standard: DZ TS 1170.5:2024 – Public consultation draft.
- Wood, J. H. 2022. Earthquake design loads on retaining walls. *NZSEE 2022 Annual Conference*.