Liquefaction-induced parabolic subsidence method for analysis of shallow foundations

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ABSTRACT

In New Zealand, the document "Repairing and Rebuilding Houses Affected by the Canterbury Earthquakes" issued by the MBIE, has been widely embraced by consulting engineers and local authorities as a reference for building solutions on liquefaction prone sites. However, the fundamental engineering principles underpinning these recommendations originating from the observations of the 2010 and 2011 Canterbury earthquake sequences have remained elusive, prompting consulting engineers to develop their own calculation methods over the years.

Among these independently devised calculation methods, some have become prevalent within the engineering community. Nevertheless, their efficacy in aligning with the objectives set forth in the MBIE document has been questionable, leading to concerns about the reliability and resilience of structures constructed based on these approaches.

Recognizing this conundrum and the apparent divergence between locally developed methods and the recommendations of the MBIE document, our study leverages New Zealand's rich dataset and experience. We introduce a rational method aimed at resolving these discrepancies. This approach enables the development of specific design of shallow foundations on liquefiable soils while addressing the critical issue of aligning with the objectives of the MBIE document.

Our research strives to provide engineers with a more comprehensive grasp of Soil-Structure Interaction (SSI) on sites prone to liquefaction. The proposed rational method equips engineers with the essential tools needed to fashion bespoke and dependable designs for shallow foundations, effectively mitigating the risks associated with soil liquefaction, all while ensuring compliance with the objectives of the MBIE Guidance document.

1.1 The genesis and application of the foundation options detailed in the MBIE Guidance

In New Zealand, the "Repairing and Rebuilding Houses Affected by the Canterbury Earthquakes" document, issued by MBIE (hereinafter referred to as MBIE Guidance), is widely adopted by engineers and local authorities for building solutions on liquefaction-prone sites. Section 5 outlines four options for reinforced concrete foundations and performance objectives for engineer-designed solutions. To develop these solutions, New Zealand authorities opted for an empirical approach based on observed resilience to Canterbury seismic events. While MBIE emphasizes that its guidance is only a guide, these options are frequently regarded as deemed-to-comply solutions, partly because Engineers encounter a lack of essential information to formulate specific foundation designs that diverged from the design options provided within the MBIE Guidance. Consequently, when there was a desire to construct using more efficient systems than those described in the MBIE Guidance, or when boundary conditions such as structural loads or geotechnical conditions fell outside the applicability limits of the predetermined solutions, designers often turned to various (sometimes debatable) design methods.

1.2 MBIE Guidance performance criteria

Clause 5.4 of the MBIE Guidance outlines criteria for designed solutions for reinforced concrete slab foundations, including:

- analysing the foundation for loss of support beneath sections of the floor to 4 m and 2 m at the ends and outer corners.
- Restricting floor hogging or sagging to 1 in 400 for lengths over 4 m and 1 in 200 for 2 m cantilevers at the extremes.
- Providing flexible services entry to accommodate potential foundation settlement indicated in the geotechnical report.
- Designing for settlements indicated in MBIE Guidance Table 5.3 for sites classified as Technical Category 2 (TC2).

It is interesting to note that MBIE recommends parameters based on direct observations (e.g. settlements magnitude) rather than indirect geotechnical data (e.g. soil stiffness) , guiding designers towards seismic soil-structure interaction (SSI) analysis similar to methods proposed by Mitchell, Walsh, Lytton, and Wray.

For brevity, this analysis focuses on loss of support at the floor extremes for Option 4 on TC2 sites. These considerations can be extrapolated to various foundation solutions, sites with different liquefaction potentials, and alternative loss of support scenarios.

1.3 Option 4: Understanding its applicability thresholds

MBIE Guidance Option 4 (cl. 5.3.1) consists in a 385 mm deep stiffened waffle raft with ribs at 1.2 m.

The materials (with reference to NZS3101 terminology) are concrete f'_c = 20 MPa grade and F_y = 500 MPa and 300 MPa grades for reinforcing bars and mesh respectively.

MBIE Guidance specifies that Option 4 is applicable with the following limitations:

- Soil categorized under Technical Category 2 (TC2) performance (as per MBIE Guidance Table 5.3), with total land settlements reaching up to 50 mm for Serviceability Limit States and up to 100 mm for Ultimate Limit States.
- Soil Ultimate Bearing Capacity of at least 200 kPa (MBIE Guidance cl. 5.3.1).

- Loads not exceeding those induced by a double storey timber-framed residential building with medium-weight wall cladding and light-weight roof cladding (MBIE Guidance Table 7.2).

1.4 Critical analysis of commonly applied design methods

A critical examination of a calculation method must consider its rationality and alignment with well-established theories. To assess its compatibility with the MBIE Guidance objectives, the following criteria apply:

1. The method should consistently represent scenarios involving loss of support scenarios and magnitude of settlements as specified in the MBIE Guidance.
2. It should only use calculation parameters outlined or directly derivable from the MBIE Guidance.
3. Results should align with MBIE Guidance design solutions, particularly for Option 4 with minimum Factor of Safety (FoS=1) for maximum loads and settlements.
4. Results must adhere to physically meaningful reference values, such as ground pressures not exceeding tolerable limits derived from the MBIE Guidance (i.e. DBC < 100 kPa = ϕ x UBC where ϕ = 0.5 from NZBC Clause B1 Table 1, paragraph 3.5.1).

SSI analysis should follow the G+0.3Q+L load combination (where G represents permanent loads, Q signifies imposed loads and L is the soil distortion that is induced by liquefaction). For low-rise buildings, seismic actions need not be considered in liquefaction SSI analysis due to seismic forces dissipating post-earthquake and liquefaction settlements occurring only after the seismic event (N. Abraseys and S. Sarma, 1969).

The calculations presented in this document involve a factored point load of 15.1 kN/rib and a uniform load of 4.6 kN/m/rib, consistently with MBIE Guidance table 7.2 specifications. Structural section stiffness may be assumed as 60% of the stiffness at rest for deformation checks, following NZS3101.

Maximum differential settlements are determined based on total free field settlements specified in MBIE Guidance Table 5.3 for TC2 land. The method for deriving differential settlements from total settlements is supported by significant studies by Anderson et al. (2007) and Martin et al. (1999), recommending half of the total settlement for differential calculations, assuming a uniform soil profile across the site.

1.4.1 Fixed end cantilever method

In the aftermath of the release of the MBIE Guidance, seminars and meetings were held in Christchurch to provide training for engineers, enabling them to design according to the criteria established by MBIE. In these sessions, of which the author has direct experience, the empirical nature of the presented options was clarified. However, the message conveyed was that for soils not exceeding the criteria of TC2, the adequacy of specific engineered solutions derived from the proposed solutions (such as, for examples, waffle raft foundations with a thickness different from 385 mm) could be assessed purely from a structural perspective. In other words, it was commonly understood and accepted that the structural capacity of a foundation to overhang by 2 m and to remain suspended with a span of 4 m were sufficient conditions to meet compliance. It is not surprising, therefore, that for a long time and sometimes even today, specific engineering designs for foundations on TC2 are often based on fixed end cantilever and simply supported free body diagrams rather than soil-structure interaction (SSI).

The simplified fixed-end cantilever method generates a deflected shape that is unlikely to accurately represent a foundation on soils prone to liquefaction. In fact, in this schematization, rotations are impeded at the section where the loss of support begins, and this is not plausible for a foundation structure due to the soil's deformability and non-linear behaviour.

Therefore this method appears to be flawed in evaluating the deformation of a foundation structure.

Moreover, the simplified fixed-end cantilever method fails to provide information about the actions affecting the portion of the structure beyond the point where support is lost. In the pursuit of structural optimization, some designers may choose to incorporate localized flexural and shear reinforcement, overlooking the fact that the actions extend (and, for bending moment, actually increase) beyond the analysed portion of structure.

Table 1: Fixed end cantilever method's consistency with MBIE Guidance objectives.

Criteria	**Consistency**	**Notes**
Ability to represent a 2m and 4m loss of support scenarios at floor extremities and at floor centre	Partially satisfied	This method allows modelling loss of support scenarios, but in a manner that is somehow unrealistic for the type of structure under consideration.
Ability to analyse the effects of 25mm(50mm) over 2m differential settlements at SLS(ULS)	**NOT** satisfied	Since this method does not involve soil modelling, it cannot allow for the analysis of effects due to specific soil settlement values.
Exclusively employ calculation parameters that are either specified or directly derivable from parameters outlined in the MBIE Guidance	Satisfied	
Calibration against MBIE Option 4: Flexural	**NOT** satisfied	$M_{,target} \simeq 33.7$ kNm/rib $= \phi M_{,option\ 4}$ $M_{,calculated} = 39.4$ kNm/rib Calibration error = 17%
Calibration against MBIE Option 4: Shear	Satisfied*	$V_{,target} > 8.0$ kN/rib $= \phi V_{,option\ 4\ (without\ shear\ reinforcement)}$ $V_{,target} < 101.4$ kN/rib $= \phi V_{,option\ 4}$
Calibration against MBIE Option 4: Deflection	**NOT** satisfied	$\triangle/L_{,target} \simeq$ 1 in 200 = Deflection limit $\triangle/L_{,calculated} =$ 1 in 67 Calibration error = 198%
Calibration against MBIE Option 4: Soil pressure	**NOT** satisfied	Since this method does not involve soil modelling, it cannot provide information about the pressure exerted by the foundation onto soil.

* For this particular case, it is more pertinent to evaluate whether the demand calculated using the selected method would require shear reinforcement. In fact, checking the alignment between the calculated action and the capacity of Option 4 is not relevant, as the capacity progression for an unreinforced section and a reinforced section is non-linear.

Based on the results presented in Table 1, we conclude that the “Fixed end cantilever method” does not meet the objectives set out in the MBIE Guidance.

1.4.2 SSI analysis with removed supports method

As inconsistencies arose between results from free body diagrams and the performance of options in the MBIE Guidance, designers pondered if the analysis could proceed without SSI. The author, in discussions with engineers contributing to the MBIE Guidance, confirmed the necessity of including SSI analysis.

The simplest and most simplistic way to simulate a loss of support in SSI modelling is to eliminate some of the foundation structure's support constraints. For instance, assuming a beam model on elastic Winkler soil, this is achieved by deleting (or equivalently imposing zero stiffness) on the elastic support constraints present in the portion of the beam extending over the 2m unsupported length.

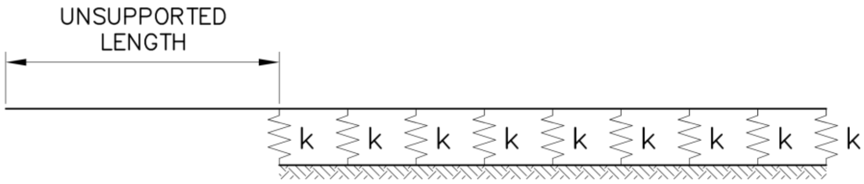


Figure 1: Diagram representing SSI analysis with removed supports. The upper line represents the foundation, while the lower one represents the soil. Nodal supports (where provided) are non-linear (compression only) Winkler springs with stiffness k.

Although this method represents an improvement over the one described in the previous paragraph, it has notable limitations. Indeed, it does not allow for modelling scenarios characterized by different values of differential settlements, thus limiting the ability to modulate the effects for cases with varying degrees of subsidence. In this context, different magnitudes of differential settlements (e.g. those for sites classified within TC2 and TC3), result in the same distortions to the foundation, which is inherently implausible.

This method, although not codified in regulations for SSI, can be employed for structural pre-design. In fact, all else being equal, it yields more conservative results compared to more sophisticated methods involving the modelling of pre-distorted soil. However, due to the highlighted limitations and approximations, it yields inconsistent results with those presented in the MBIE Guidance. In fact, if assessed using this method, some of the options outlined in the MBIE Guidance (e.g., Option 4) would not appear suitable for the specified loads and support loss scenarios. Even from a geotechnical perspective, there are inconsistencies. Indeed, the pressures exerted by the foundations on the building platform would be so high as to be incompatible with typical soil bearing capacity values.

The above considerations remain valid when conducting a sensitivity analysis on the modulus of subgrade reaction (k). In fact, varying k between 10,000 kN/m^3 and 50,000 kN/m^3 does not yield solutions that meet the strength and deformability requirements of the structure concurrently with sensible geotechnical results.

In particular, assuming k=10,000 kN/m^3, the soil pressures are higher (but still somewhat acceptable) that what allowed ($P_{,calculated}$ = 158 kPa > 100 kPa = DBC) and the strength (M* = 44.7 kNm/rib > 33.7 kNm/rib = $\phi M_{,option\ 4}$) and deformability ($\triangle/L_{,calculated}$ = 1 in 115 >> 1 in 200 = Deflection limit) requirements are significantly unmet. By increasing the modulus of subgrade reaction (k=50,000 kN/m^3), the deformability criterion is satisfied, but it results in unacceptable soil pressure values ($P_{,calculated}$ = 250 kPa >> 100 kPa = DBC). In any case, even in this second scenario, the strength requirement (M* = 39.1 kNm/rib > 33.7 kNm/rib = $\phi M_{,option\ 4}$) remains unfulfilled.

Table 2: SSI analysis with removed supports method's consistency with MBIE Guidance objectives.

Criteria	Consistency	Notes
Ability to represent a 2m and 4m loss of support scenarios at floor extremities and at floor centre	Satisfied	

Ability to analyse the effects of 25mm(50mm) over 2m differential settlements at SLS(ULS)	**NOT** satisfied	This method does not allow for the analysis of effects due to specific soil settlement values.
Exclusively employ calculation parameters that are either specified or directly derivable from parameters outlined in the MBIE Guidance	Satisfied	
Calibration against MBIE Option 4: Flexural	**NOT** satisfied	$M_{,target} \simeq 33.7$ kNm/rib = $\phi M_{,option\ 4}$ $M_{,calculated} = 39.1 – 44.7$ kNm/rib Calibration error = 16% – 33%
Calibration against MBIE Option 4: Shear	Satisfied*	$V_{,target} > 8.0$ kN/rib = $\phi V_{,option\ 4\ (without\ shear\ reinforcement)}$ $V_{,target} < 101.4$ kN/rib = $\phi V_{,option\ 4}$
Calibration against MBIE Option 4: Deflection	**NOT** satisfied	$\triangle/L_{,target} \simeq$ 1 in 200 = Deflection limit $\triangle/L_{,calculated} =$ 1 in 189 – 1 in 115 Calibration error = 6% – 42%
Calibration against MBIE Option 4: Soil pressure	**NOT** satisfied	$P_{,target} \simeq 100$ kPa = 1/2 x UBC $P_{,calculated} = 158$ kPa – 250 kPa Calibration error = 58% – 150%

* For this particular case, it is more pertinent to evaluate whether the demand calculated using the selected method would require shear reinforcement. In fact, checking the alignment between the calculated action and the capacity of Option 4 is not relevant, as the capacity progression for an unreinforced section and a reinforced section is non-linear.

Based on the results presented in Table 2, we conclude that the “SSI analysis with removed supports method” does not meet the objectives set out in the MBIE Guidance.

The method described above is sometimes adopted due to the incorrect interpretation of the term "unsupported length" in the context of SSI analysis.

It's worth emphasizing, in fact, that the term "unsupported length" (or similar terms) does not imply a portion of the foundation overhanging in the void. The concept of loss of support extends beyond the issue of liquefaction and is to be understood as a localized depression in the ground where the ground profile can be represented by different curves depending on various parameters, such as the type of soil under consideration (Franza et all, 2019).

Following the MBIE Guidance, the maximum ground surface deformation for TC2 sites, can be described through the values of differential settlements provided at table 5.3.

1.4.3 SSI analysis using variable stiffness supports method

An iteration of the method presented in the previous paragraph involves adopting a beam-on-Winkler soil model, where the stiffness of the modal spring supports is subject to adjustments, being lower where support loss is expected.

In this approach, there is a primary inconsistency. The MBIE Guidance specifies that in the portion of soil susceptible to liquefaction, the foundation loses support from the ground (meaning the soil settles relative to the foundation support level, regardless of the induced pressure). However, in this model, it is assumed that there is still support from the soil in that portion, albeit less rigid (meaning that locally the soil settles more under the same pressures).

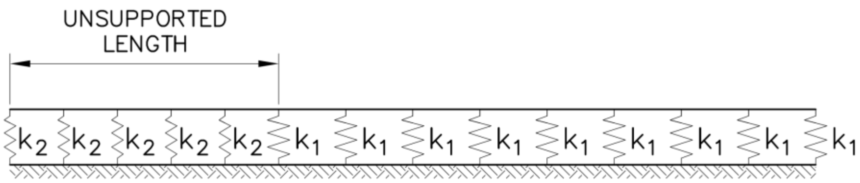


Figure 2: Diagram representing SSI analysis using variable stiffness supports. The upper line represents the foundation, while the lower one represents the soil. Nodal supports are non-linear (compression only) Winkler springs with variable stiffness ($k_1 > k_2$).

Another, and more substantial, drawback of this model stems from the fact that it prompts speculations when determining the stiffness values of the Winkler springs (in particular k_2, being very difficult to estimate the value for the modulus of subgrade reaction for soil undergoing liquefaction).

The main risk in approaching the problem in this way is to end up, even unintentionally, searching each time for a combination of values for k_1 and k_2 under which the structure is verified, effectively losing sight of the underlying issue.

Finally, this method also fails to incorporate the magnitude of the differential settlements specified in the MBIE Guidance into the analysis, thus forfeiting the capability to differentiate the effects caused by varying levels of subsidence.

Table 3: SSI analysis using variable stiffness supports method's consistency with MBIE Guidance objectives.

Criteria	**Consistency**	**Notes**
Ability to represent a 2m and 4m loss of support scenarios at floor extremities and at floor centre	Satisfied	Despite a logical inconsistency, this method can simulate the scenarios from the MBIE Guidance by using a correct set of springs.
Ability to analyse the effects of 25mm(50mm) over 2m differential settlements at SLS(ULS)	**NOT** satisfied	This method does not allow for the analysis of effects due to specific soil settlement values.
Exclusively employ calculation parameters that are either specified or directly derivable from parameters outlined in the MBIE Guidance	**NOT** satisfied	This model prompts speculations when determining the stiffness values of the Winkler springs.
Calibration against MBIE Option 4: Flexural	N/A	Calibration assessment requires establishing a reference value for both k_1 and k_2.
Calibration against MBIE Option 4: Shear	N/A	Calibration assessment requires establishing a reference value for both k_1 and k_2.

Calibration against MBIE Option 4: Deflection	N/A	Calibration assessment requires establishing a reference value for both k_1 and k_2.
Calibration against MBIE Option 4: Soil pressure	N/A	Calibration assessment requires establishing a reference value for both k_1 and k_2.

Based on the results presented in Table 3, we conclude that the "SSI analysis using variable stiffness supports method" does not meet the objectives set out in the MBIE Guidance.

1.5 The L.I.P.S. method

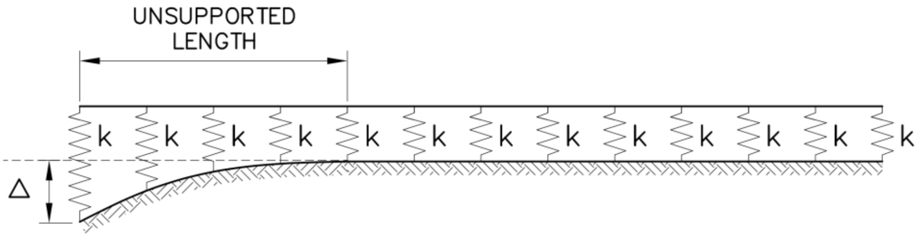


Figure 3: Diagram representing the L.I.P.S. Method. The upper line represents the foundation, while the lower one represents the soil. Nodal supports are non-linear (compression only) Winkler springs with stiffness k. △ is the differential settlement of soil caused by liquefaction.

The Liquefaction Induced Parabolic Subsidence Method (L.I.P.S.) requires the execution of a SSI analysis, wherein a foundation possessing a specific stiffness and subjected to the design loads, is superimposed with elastic Winkler interaction onto a pre-distorted soil conforming to the profiles as described in this paragraph.

The soil profile considered in the L.I.P.S method is defined with Equation 1 with reference to Figure 4:

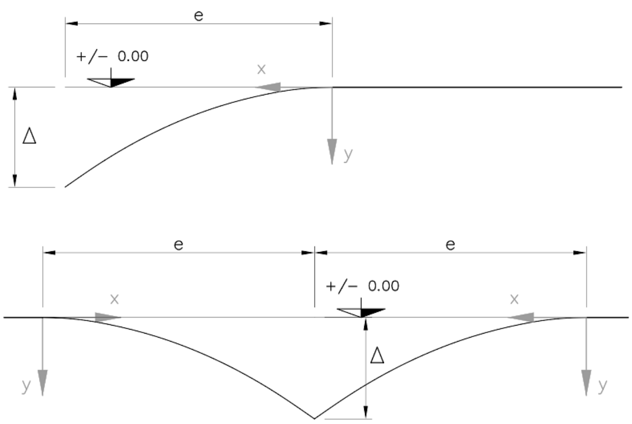


Figure 4: Loss of support: cantilever at the extremes of the floor and sag at centre of the floor

$$y = \frac{\Delta}{e^2}x^2 \quad (1)$$

Where,

y is the elevation of the soil at the *x* location

△ is the differential settlement of the soil across the unsupported length *e*

In order to place the proposed method in the context to the loss of support scenarios and design settlement limits set out in the MBIE guidance, one must substitute e = 2 m and Δ_{SLS} = 1/2 x 50 mm, Δ_{ULS}, = 1/2 x 100 mm and equation (1) then becomes:

$$y = \frac{25}{4}x^2 \text{ (SLS)} \tag{2}$$

$$y = \frac{50}{4}x^2 \text{ (ULS)} \tag{3}$$

We notice that parabolic equations as well the Winkler mathematical model have been widely used for SSI analysis of foundations on movement-prone soils (e.g. Walsh Method and Lytton Method for expansive soils). This type of analysis can be carried out with standard Finite Element Analysis software equipped with non-linear capabilities.

The method has been developed through a kind of reverse engineering process of MBIE Guidance "Option 4" deemed-to-comply solution. Specifically, when applying the method to an Option 4 foundation subjected to maximum loads and maximum settlements permissible under the MBIE Guidance, it was observed that the demand-to-capacity ratio approached unity.

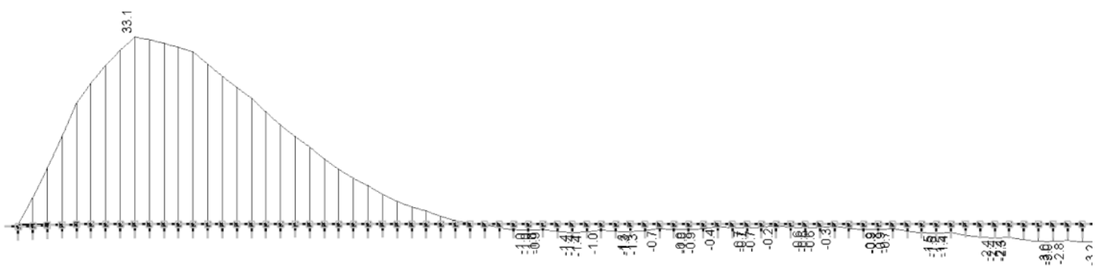
The profile of the soil is described with equations equation (2) and (3) with reference to the most relevant loss of support scenario (2 m loss of support at the perimeter of the foundation).

Summarizing the following *calibration set* of parameters has been selected:

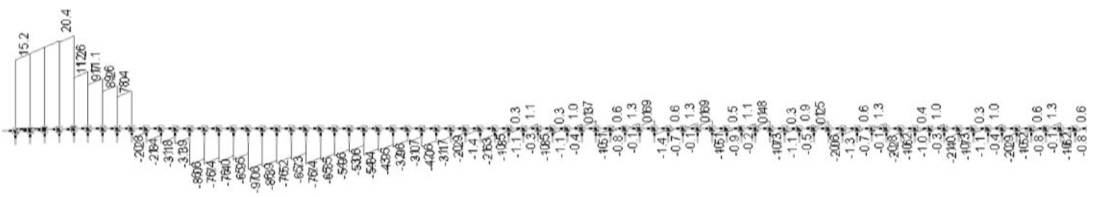
△ = Differential Settlements = 25 mm (SLS) and 50 mm (ULS)

e = Loss of Support = 2 m cantilever (similarly, diagrams for 4 m internal loss of support may be obtained)

k = Modulus of Subgrade Reaction = 10,000 kN/m^3



(a)



(b)

(c)

(d)

Figure 5: (a) Bending Moment [kNm/rib], (b) Shear [kN/rib] (c) Deflection [mm] (d) Reactions [kN]

Commercial non-linear finite element analysis software (Wafflesuite and AXIS VM) have been used to produce and validate the results presented in Figure 5.

These software tools enable the analysis of more intricate configurations, such as grids of beams and plates, in contrast to solely analysing individual beams. This capability is particularly beneficial when dealing with multiple loads. However, it is important to note that for the simplest loads cases, being the loss of support scenario cylindrical, the results obtained from analysing models involving beams on the Winkler soil are sufficiently accurate.

Figure 5(a) shows the bending moment calculated for a rib of the waffle slab using equation (3). The maximum value for the bending moment is to be checked against a flexural strength of 33.7 kN/m/rib which equates to the capacity of MBIE Guidance Option 4.

The shear force diagram in Figure 5(a) was also derived from the soil profile described by equation (3). The peak value for shear (V= 20.4 kN/rib) confirms the requirement for specific shear reinforcement, as prescribed in MBIE Guidance Option 4.

Figure 5(a) depicts the deformed profile of the analysed rib due to induced loads and soil distortions. The maximum curvature of the structure must be assessed with respect to the value of 1/200 as specified in the MBIE Guidance. For this analysis, equation (2) was employed to define the soil profile.

The maximum value for soil pressure P_{max} may be derived from Figure 5(a) and it is checked against DBC with equation:

$$P_{max} = max\ (\frac{R_{z,i}}{A_{,i}}) < \text{DBC} \qquad (4)$$

Where,

$R_{z,i}$ is the vertical reaction (compression) at node i

A_i is the contact area of node i

DBC is taken as 100 kPa

At any point the foundation does not exert an excessive pressure onto the soil, being P_{max} = 100 kPa.

It is essential to understand that while the LIPS method is valid for SSI analysis (similar to other internationally recognized methods), its accuracy relies on the selection of the appropriate parameters; in other cases than TC2 and when the structural loads are incompatible with MBIE Guidance table 7.2, the designer shall seek advice from a geotechnical engineer for more precise information about the parameters △, e and k.

Table 4: L.I.P.S. method's consistency with MBIE Guidance objectives.

Criteria	**Consistency**	**Notes**

Ability to represent a 2m and 4m loss of support scenarios at floor extremities and at floor centre	Satisfied	
Ability to analyse the effects of 25mm(50mm) over 2m differential settlements at SLS(ULS)	Satisfied	
Exclusively employ calculation parameters that are either specified or directly derivable from parameters outlined in the MBIE Guidance	Satisfied	
Calibration against MBIE Option 4: Flexural	Satisfied	$M_{,target} \simeq 33.7$ kNm/rib = $\phi M_{,option\ 4}$ $M_{,calculated} = 33.1$ kNm/rib Calibration error = 2%
Calibration against MBIE Option 4: Shear	Satisfied*	$V_{,target} > 8.0$ kN/rib = $\phi V_{,option\ 4\ (without\ shear\ reinforcement)}$ $V_{,target} < 101.4$ kN/rib = $\phi V_{,option\ 4}$
Calibration against MBIE Option 4: Deflection	Satisfied	$\triangle/L_{,target} \simeq$ 1 in 200 = Deflection limit $\triangle/L_{,calculated}$ = 1 in 200 Calibration error = 0%
Calibration against MBIE Option 4: Soil pressure	Satisfied	$P_{,target} \simeq 100$ kPa = 1/2 x UBC $P_{,calculated} = 100$ kPa (=2.9 kN / (0.1 m x 0.3 m)) Calibration error = 0%

* For this particular case, it is more pertinent to evaluate whether the demand calculated using the selected method would require shear reinforcement. In fact, checking the alignment between the calculated action and the capacity of Option 4 is not relevant, as the capacity progression for an unreinforced section and a reinforced section is non-linear.

Based on the results presented in Table 4, we conclude that the "L.I.P.S method" meets the objectives set out in the MBIE Guidance.

1.6 Conclusions

The key objective of this study is to address the discrepancies inherent in design solutions locally developed in New Zealand and the objectives specified in the MBIE Guidance document.

Leveraging the country's extensive dataset and expertise, we present a rational method known as the L.I.P.S. method, standing for Liquefaction Induced Parabolic Subsidence Method. This method is designed for the purpose of engineering shallow foundations on liquefiable soils.

The proposed rational method equips engineers with the essential tools needed to fashion bespoke and dependable designs for shallow foundations, effectively mitigating the risks associated with soil liquefaction, all while ensuring compliance with the objectives of the MBIE Guidance document.

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