Strength reduction factors for foundations and earthquake load combinations including overstrength

S J Palmer
Tonkin & Taylor Ltd, Wellington, NZ
SPalmer@tonkin.co.nz (Corresponding author)

Keywords: Seismic design, earthquake engineering, strength reduction factor, capacity design

ABSTRACT

Clause B1/VM4 of the New Zealand Building Code allows a concession on strength reduction factors applied to foundation design for load cases involving earthquake load combinations with overstrength factors. It allows strength reduction factors of 0.8 to 0.9 compared to 0.45 to 0.60 typically specified for other load combinations. The Canterbury Earthquakes Royal Commission recommends that this concession be re-assessed. It recommends that strength reduction factors in B1/VM4 be revised to reflect international best practice including consideration of risk and reliability. This paper explores the debate. Opinions of New Zealand and international geotechnical and structural engineers are considered. New Zealand and international practice, and factors which may influence the selection of a strength reduction factor are discussed. Recommendations are provided for further research. Possible guidance for the selection of strength reduction factors for seismic capacity design is presented for further debate.

1 INTRODUCTION

Load and resistance factored design and capacity design are procedures which have been practiced in New Zealand since the mid 1970's. Since their introduction standards and codes have allowed a factor of safety of 1.1, or a strength reduction factor of 0.9, to be applied to foundation capacity for load combinations including overstrength factors (seismic capacity design). Applying these low factors of safety has been normal practice. Kevin McManus in his report for the Canterbury Earthquake Royal Commission "Foundation Design Reliability Issues", October 2011 has highlighted that this practice may not be appropriate. McManus suggests that the geotechnical strength reduction factors used for seismic capacity design should be the same as those used for other ultimate limit state load combinations. The purpose of this paper is to explore if there is justification for allowing higher strength reduction factors for seismic capacity design.

2 LOAD AND RESISTANCE FACTORED DESIGN

In New Zealand load and resistance factored design (LRFD) of structures is used. LRFD may be described as follows:

$$\Sigma Q_i\gamma_i \leq \Sigma R_i\phi_i$$

Where Q_i = Nominal value of load type $_i$; γ_i = Load factor to allow for uncertainties in Q_i; R_i = Nominal value of resistance of component $_i$; and ϕ_i = Strength reduction factor to allow for uncertainties in $R_{i.}$

2.1 What is considered in nominating R_i and ϕ_i?

ϕ_i allows for uncertainties in $R_{i.}$ Selection of ϕ_i for foundation design is discussed in subsequent sections of this paper.

R_i is selected allowing for the loading and environmental conditions which could influence R_i during the life of the structure. In designing foundations for earthquake loading these conditions could include: Liquefaction or cyclic softening and associated reduction in bearing capacity and shaft resistance; Cyclic loading and reversed loading (push and pull); Short duration of the loading.

These factors are complex, and there are a lot of uncertainties in evaluating them and thus uncertainty in nominating an appropriate R_i. This uncertainty and the level of conservatism in allowing for these conditions needs to be considered in nominating ϕ.

2.2 Reliability Analysis

The loads and resistances considered in design have probability distributions as indicated by (Figure 1).

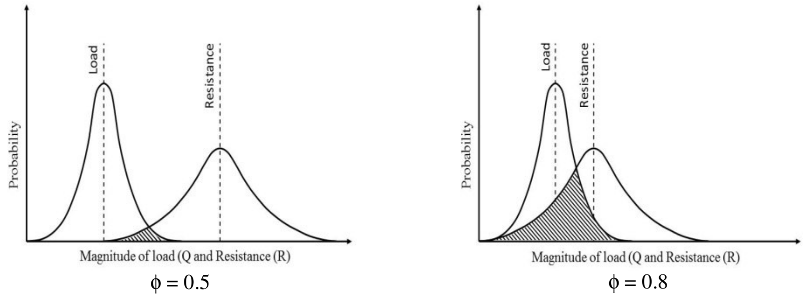

Figure 1: Idealised probability distributions (adapted from FHWA – Drilled Shafts Manual 2010)

The overlap between these probability distributions is indicative of a failure. The objective of LRFD is to limit the probability of failure (the overlap) to an acceptable level. Figure 1 illustrates an example where ϕ = 0.5 may be appropriate and ϕ = 0.8 may result in an unacceptably high probability of failure. The load factors and strength reduction factors proposed by some codes and guidelines have been developed on the basis of probabilistic analysis and the reliability theory described by Figure 1.

3 COMPARISON OF NEW ZEALAND AND INTERNATIONAL PRACTICE

New Zealand's practice of applying higher ϕ values for seismic capacity design is consistent with USA practice as indicated by (Table 1).

Table 1: Summary of specified strength reduction factors (ϕ) for foundation design.

Document	Country	Typical Specified Strength Reduction Factor		
		Seismic Capacity Design	Other Load Combinations	
			Without Pile Load Tests	With Pile Load Tests
Building Code B1/VM4	NZ	0.8-09	0.40-0.65	0.65-0.90
Piling AS2159-2009 ****	Australia NZ	Not specified	0.45-0.60	0.6-0.90
NZTA Bridge Manual 2003	NZ	0.53-0.94 *	0.45-0.55	0.5-0.80
Building Code	USA	0.8-1.0 **	0.5-0.65 **	-
AASHTO, 2002 (Bridges)	USA	0.8-1.0 ***	0.40-0.55	0.6-0.7

Notes
*Specified as for other load combinations multiplied by 1/0.85.
** Foundation design is by Allowable Stress Design. Equivalent ϕ values provided by US engineers.
*** For load case Extreme Event 1: 1000 year earthquake with similar design requirements as NZS1170.0-2002
**** Piling – design and installation AS2159-2009 provides a risk based procedure to evaluate ϕ. The factors considered in this procedure include: Complexity of ground conditions; Extent, type and quality of geotechnical investigations; Design method used and its basis; Level of conservatism in selecting the design value; Level of construction control; Level of performance monitoring; Redundancy in the foundation system; The type and number of pile load tests (if any).

4 HISTORY OF GEOTECHNIAL STRENGH REDUCTION FACTORS IN NEW ZEALAND

The history of geotechnical strength reduction factors in New Zealand is described in McManus (2011). The concession of allowing a higher strength reduction factor in the earthquake load case with overstrength factors related back to the introduction of LRFD to New Zealand by NZS4203:1976. NZS4203:1976 suggested that for certain ductile structure types where .... "design loadings in the foundation system are determined by the yield capacity of other parts of the structure, a factor of safety of 1.1 for soil pressure is suggested, pending revision of the foundation code, because at this extreme condition partial yielding of the subsoil might not be significantly damaging and it provides an additional energy dissipating mechanism" (ie a strength reduction factor ϕ = 0.9).

5 POST CANTERBURY EARTHQUAKE RECOMMENDATIONS

5.1 Canterbury Earthquakes Royal Commission recommendations

The Commission recommends that the concessional strength reduction factors in B1/VM4 for load cases involving load combinations and overstrength actions (ϕ =0.8-09) be reassessed. Further it recommends that these values be revised to reflect international best practice including considerations of risk and reliability.

5.2 SESOC Practice note, design of conventional structural systems following the Canterbury earthquakes (18 September 2012)

SESOC proposes the use of a risk based assessment such as AS2159-2009 to assess strength reduction factors. It recommends that higher values (such as those suggested by B1/VM4) not be used for seismic capacity design unless specifically instructed by the geotechnical engineer.

6 SUMMARY OF THE DEBATE

As part of research for this paper geotechnical and structural engineers in New Zealand and USA have been asked to provide their views on seismic capacity design and geotechnical strength reduction factors (refer acknowledgements).

In New Zealand those contacted were well aware of the debate with the majority considering that the use of higher strength reduction factors for seismic capacity design was justified but not as high as proposed by B1/VM4. In USA the engineers contacted considered the high strength reduction factors currently used were justified, and this was not an issue of debate in USA at this time.

The views of engineers contacted are summarised as follows:

6.1 Views promoting lower ϕ

Engineers who considered that ϕ values for design of foundations for seismic capacity design should be the same as ϕ values applied with other design methods/load combinations supported their view with the following comments:

Uncertainties: There is a high level of uncertainty in predicting the ultimate capacity of a foundation. The ϕ values proposed by AS2159-2009 (typically 0.45 to 0.6 without pile load testing) have been specifically assessed to allow for these uncertainties and to provide an acceptably low probability of foundation failure. These uncertainties can be reduced and higher ϕ values applied by undertaking pile load tests. AS2159-2009 ϕ values are appropriate.

Purpose of ϕ: ϕ is to allow for uncertainties in ground conditions and other factors influencing the capacity of a foundation. These uncertainties are not less, and probably greater for load combinations including earthquake compared to those without earthquake. Higher values for load combinations with earthquake are not justified.

Higher ϕ implies acceptance of plastic deformation: ϕ values are generally determined to provide an acceptably low probability of failure. A low probability that loads will exceed foundation capacity. By using the higher ϕ we are accepting a higher probability that foundation capacity will be exceeded resulting in plastic deformation.

Plastic deformation of soils can be unpredictable: For loads up to "failure" methods are available for predicting deformations. There are various definitions of "failure" of a foundation but for piles in compression it is commonly taken as settlement greater than 10% of the pile diameter and for piles or anchors in tension a load at which a maximum creep criteria is not met. Beyond the "failure" load deformations become unpredictable.

Large deformations: The plastic deformations are likely to be large and beyond the safe tolerance of the structure. Quoting from Mc Manus (2011) "behaviour of the structure will be unpredictable and, most likely, undesirable".

6.2 Views promoting higher ϕ

Engineers who considered higher ϕ values were justified for seismic capacity design compared to other methods/load combinations supported their view with the following comments:

Design requirements: For ULS design AS/NZS1170.0-2002 section 3.2 requires design "with an appropriate degree of reliability sustain all actions". Section 3.2 makes a special case for earthquake ULS stating a requirement to "avoid collapse". The intent with earthquake capacity design is that deformation and yielding of parts of the structure are acceptable provided the performance of the structure remains dependable and stable. Much higher deformations are acceptable for seismic capacity design than for other design cases.

Certainty of loads: Seismic capacity design constrains, to an extent, the maximum load that can be transmitted to the foundations. This increases reliability allowing some increase in ϕ to be considered.

Energy dissipation: Some yielding of foundations provides additional energy dissipation reducing loads.

Cyclic short term loading: The cyclic short term (less than a few seconds) loading results in significantly smaller foundation deformations than would be predicted by a static load assessment. For cohesive soils in bearing, short term loading may produce higher capacities than long term loading.

Precedence: Foundations have been designed with higher ϕ values in conjunction with seismic capacity design for more than three decades. Building collapse has not been attributed to foundation failures as a consequence of this approach during Christchurch or other earthquakes.

Economics: The proposed reduction in ϕ would have significant impact on construction cost without an associated significant improvement in building performance. Understanding and allowing for liquefaction and other seismic effects is likely to have a substantially higher benefit for additional construction cost. This is highlighted by the Christchurch experience where issues associated with foundation performance are likely related to liquefaction effects rather than selection of the design ϕ values.

Predicting deformations: Rather than reducing ϕ values the challenge should be put to the geotechnical fraternity to develop an understanding of deformation of foundations under seismic loading. Geotechnical and structural design should focus on ensuring foundation deformations can be safely tolerated by the structure.

7 PERFORMANCE OF FOUNDATIONS

A common thread from both sides of the debate is the predictability and performance of foundations under seismic loading and at loads exceeding the foundations assumed "failure" load. The following sub-sections consider this issue.

7.1 Performance of foundations under static loading

A history of static load testing of foundations has provided a relatively good understanding of their performance under static loads. Performance under seismic loads is poorly understood. (Figures 2 to 4) provide an indication of the performance of foundations under static loading

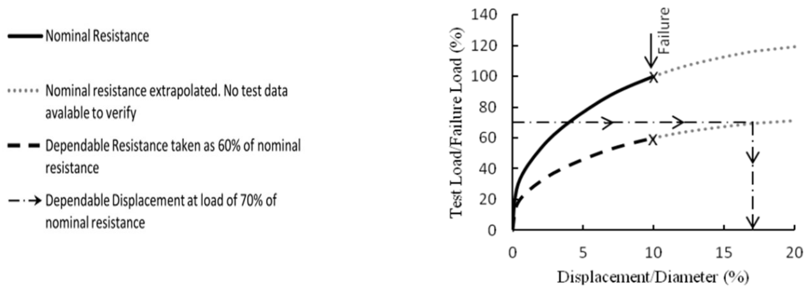

Figure 2: Load-Displacement. Axial compression. Pile in cohesionless soil (nominal resistance based Chen and Kulhawy, 2002)

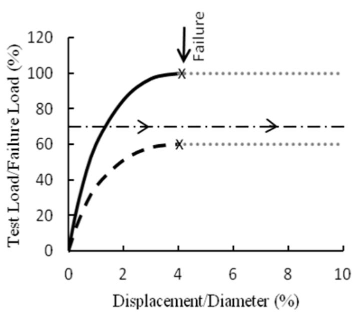

Figure 3: Load-Displacement. Axial compression Pile in cohesive soil (nominal resistance based on Chen and Kulhawy, 2002)

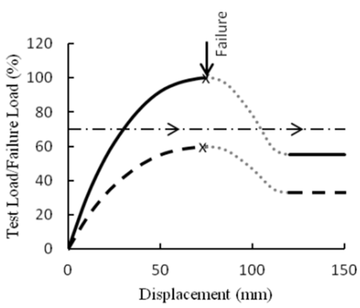

Figure 4: Load-Displacement. Axial tension. Anchor in Wellington greywacke (based on test data from Tonkin & Taylor Ltd files)

In (Figures 2 to 4) the 'nominal resistance' (R) is shown as a solid line. Because of the uncertainties in predicting the 'nominal resistance'(R), it has been reduced by a factor of 0.6 (ϕ) to give the 'dependable resistance' shown as a dashed line. This ϕ of 0.6 is that which would be used with load combinations without earthquake overstrength actions. The value of 0.6 provides an acceptably low probability of failure under static loading with the uncertainties which exist in the nominated resistance for the particular foundations referred to by Figures 2 to 4.

(Figures 2 to 4) indicate an assessment of the 'dependable displacement' if a higher ϕ of 0.7 were to be used for static loading. The conclusions drawn from Figures 2 to 4 are:

Bored piles in axial compression in cohesionless soil. (Refer Figure 2).
Up to the nominal failure load at a displacement of 10% of the pile diameter, the load – displacement relationship is well understood. Beyond this displacement further research of pile behaviour is required. However some increase in capacity with displacement can be expected as indicated on Figure 2. This is supported by the observation that driven (displacement) piles have higher capacity than bored (non-displacement) piles in cohesionless soils.

If a higher ϕ value of 0.7 rather than 0.6 were applied for this particular foundation design it is inferred that there would be an acceptably low probability of the foundation being unable to support the static design loads, but the associated displacements could be large i.e. 15% to 20% of pile diameter or possibly higher.

If the seismic capacity structural design can tolerate these large displacements this could possibly be a justification for applying a ϕ value of say up to 20% higher than that applied for other design cases, for bored piles in cohesionless soil.

Bored piles in axial compression in cohesive soil (refer Figure 3)
At displacement beyond the failure load no increase in pile capacity is expected. This indicates that for static loading use of a ϕ value in design greater than that assessed on a risk and reliability basis (e.g. AS2159-2009) could result in an unacceptably high probability of the foundation being unable to support the design loads, at any displacement. This indicates that it may not be appropriate to apply higher ϕ values for bored piles in cohesive soil. However, short term seismic loading in axial compression on cohesive soils may produce higher capacities than long term static loading. This would require specific testing and assessment to investigate.

Anchors in tension (refer Figure 4)
At displacement beyond that at which the peak test load is recorded for a ground anchor some drop off in capacity to a residual capacity can be expected. The amount of drop off in capacity from peak to residual depends on the soil/rock type and anchor construction method. Anchor tests in Wellington greywacke have recorded residual capacities of the order of 50% of the peak. Straight shafted bored piles are likely to behave in a similar manner to anchors but with the drop off in capacity from peak to residual being less pronounced. This indicates that for anchors and bored piles in tension, use of higher strength reduction factors is likely to result in an unacceptability high probability of undesirable behaviour of the foundations.

7.2 Performance of foundations under seismic loading

Performance of foundations under seismic loading is not well understood. Because of the short duration of the loading, displacements under seismic loading are likely to be less than those assessed on the basis of static loading. This is provided the nominated resistance has made appropriate allowance for factors such as liquefaction and cyclic loading (refer section 2.1)

The performance of foundations under seismic loading requires further research and could provide justification using higher ϕ values.

Geotechnical seismic design of slopes and flexible retaining structures are normally undertaken on a displacement design basis. When Newmark sliding block theory is applied, displacements are calculated as the sum of the movements which could be expected for each interval during the earthquake when the loads applied exceed the yield resistance of the soil. For displacement of 10's or 100's of mm's this approach can develop acceptable designs, while the factor of safety assessed on a load and resistance basis is substantially less than 1. Because of the seismic response of the building and high stress levels in the ground beneath foundations, this approach if applied to foundations, is not likely to provide the same level of design benefit as it does for the design of slopes and retaining structures, however further research is required. What is being proposed for consideration is similar to time history analysis of a structure but complicated by the inelastic and less predictable behaviour of soils.

8 CONCLUSIONS AND RECOMMENDATIONS

The Canterbury Earthquake Royal Commission recommends: "the strength reduction factors in B1/VM4 should be revised to reflect international best practice including considerations of risk and reliability"

The higher strength reduction factors allowed by B1/VM4 for seismic capacity design, compared to that specified for other design methods and load combinations, are consistent with the New Zealand Bridge Manual and practice in USA for the seismic design of bridges and buildings.

B1/VM4 is consistent with international practice, however review is considered appropriate. This review should include research into the seismic performance of foundations. Load-displacement relationships should be assessed, including for loads beyond what is traditionally considered to be failure. Load-displacement relationships should be assessed for short term cyclic loading as would be expected under seismic conditions. This could possibly lead to justification of higher strength reduction factors in some situations. In the longer term this research could lead to further development of displacement based design.

In the interim the following is presented for further debate:

Selection of strength reduction factors for foundations and earthquake load combinations including overstrength factors should consider:

1 Undertaking pile load tests to reduce uncertainties in predicting pile capacity and consequently allowing the use of higher strength reduction factors. These tests should, if practical, extend beyond the 'failure' load to assess pile performance at higher loads.
2 Consider the design objectives, potential deformations, risk and reliability for the structure and foundations. This requires consideration and coordination of structural and geotechnical aspects of the design to promote a consistent design approach. The geotechnical engineer needs to assess potential deformations and risks. In conjunction with the structural engineer what these deformations and risks mean for the overall structure relative to the design objectives needs to be considered.
3 Consider the relative costs and benefits in terms of building seismic performance for various strength reduction factors. The costs, benefits and minimum requirements need to be outlined to the client by the geotechnical and structural engineers to allow the client to provide informed direction.
4 Higher strength reduction factors than those used for load combinations without earthquake are not recommended for ground anchors or for straight shafted bored piles in tension.
5 Higher strength reduction factors could be considered for bored piles and shallow foundations in axial compression in cohesionless soil. This is to include careful consideration of items 1) to 3) above, and the strength reduction factor should not be more

than 20% higher than that used for other load combinations without earthquake.

6 Any consideration of higher strength reduction factors for foundations in cohesive soils should be with caution. Similarly for driven piles in cohesionless soil, or bored piles in cohesionless soils where pile capacity is dominated by shaft resistance.

9 ACKNOWLEDGEMENTS

The author acknowledges the comments and views provided by the engineers listed below. It is not implied that all of these engineers support all of the conclusions presented in this paper.

Structural Engineers
Alistair Cattanach; Rob Jury; Adam Thornton

Geotechnical Engineers
Richard Cole; Alexi Murashev

International Engineers
Geoff Martin (Los Angeles USA); Frank Rollo (San Francisco USA); Jeff Tucker (Portland USA)

REFERENCES

AS/NZS 1170.0:2002 Structural Design Actions, Part 0.General Principles. Standards Australia/Standards New Zealand

AS/NZS 2159:2009 Piling – Design and Installation. Standards Australia

Basu, D. & Salgado, R. (2012) Load and Resistance Factored Design of Drilled Shafts in Sand. Journal of Geotechnical and Geoenvironmental Engineering. 138:1455-1469

Canterbury Earthquakes Royal Commission (2012) ”Final report Volume 1: Section 1: Summary and recommendations – Volume 1-3”.

Chen, Y. & Kulhawy, F.H. 2002 Evaluation of Drained Axial Capacity for Drilled Shafts, Deep Foundations, ASCE

FHWA. 2010. Drilled Shafts Manual. USA:FHWA

McManus, K.J. 2011. Foundation Design Reliability Issues. Christchurch, New Zealand: Canterbury Earthquakes Royal Commission

NZTA. 2003. Bridge Manual. Transit (NZTA)

SESOC. 2011. Practice Note Design of Conventional Structural Systems Following the Canterbury Earthquakes, Interim Design Standards. New Zealand: SESOC