NUMERICAL STUDY AND LOAD AND RESISTANCE FACTOR DESIGN (LRFD) CALIBRATION FOR REINFORCED SOIL RETAINING WALLS

By

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Abstract

Load and resistance factor design (LRFD) (often called limit states design (LSD)) has been mandated in the AASHTO Bridge Design Specifications and will be adopted in future editions of Canadian Highway Bridge Design Code for all transportation-related structures including reinforced soil retaining walls. The ultimate objective of this thesis work was to carry out reliability-based analysis for load and resistance factor design calibration for rupture and pullout limit states for steel and geosynthetic reinforced soil walls under self-weight and permanent surcharge loading conditions. In order to meet this objective it was necessary to generate large databases of measured load and resistance data from many sources and in some cases to propose new design models that improve the accuracy of underlying deterministic load and resistance models. Numerical models were also developed to model reinforced soil wall performance. These models were used to investigate load prediction accuracy of current analytical reinforcement load models. An important feature of the calibration method adopted in this study is the use of bias statistics to account for prediction accuracy of the underlying deterministic models for load and resistance calculations, random variability in input parameter values, spatial variation and quality of data. In this thesis, bias is defined as the ratio of measured to predicted value. The most important end product of the work described in this thesis is tabulated resistance factors for rupture and pullout limit states for the internal stability of steel and geosynthetic reinforced soil walls. These factors are developed for geosynthetic reinforced soil wall design using the current AASHTO Simplified Method, a new modified Simplified Method, and the recently proposed K-Stiffness Method. Useful quantitative comparisons are made between these three methods by introducing the concept of computed operational factors of safety. This allows designers to quantify the actual margin of safety using different design approaches.
The thesis format is paper-based. Ten of the chapters are comprised of journal papers that have been published (2), are in press (2), in review (3) and the remaining (3) to be submitted once the earlier background papers are accepted.
Co-authorship

Chapter 2

Bing Huang conducted the numerical study using an updated version of the FLAC code reported by Hatami and Bathurst (2005, 2006). The revisions to these earlier codes were extensive. The two full-scale test walls (Walls 1 and 6) that were numerically simulated were constructed by previous RMC graduate students under the supervision of Dr. Bathurst. Bing Huang coded the modified Duncan-Chang hyperbolic model and Lade’s single hardening model in C++ and implemented them using the FLAC UDM option. Bing Huang also created an Excel macro (VBA) that was used to automatically extract data and create plots from FLAC output files. The chapter was published as a full paper in the ASCE Journal of Geotechnical and Geoenvironmental Engineering in October 2009 (Huang et al. 2009).

Chapter 3

The parametric study was carried out by Bing Huang using the FLAC code and other tools reported in Chapter 2. Full-scale shear tests for the interface between modular block and concrete or crushed stone levelling pads were conducted by personnel at a local testing company (BCGT). The use of the test results was approved by Dr. Bathurst who is President of BCGT. Test interpretation was carried out by Bing Huang. The primary parameters investigated were selected in consultation with Mr. Allen and Dr. Bathurst to fill in gaps in the database of RMC test walls and other monitored walls and to assist with the development of the K-Stiffness Method. The chapter is in press as a full paper in the Canadian Geotechnical Journal (Huang et al. 2010).

Chapter 4

LRFD calibration in this chapter was carried out by Bing Huang using the methodology proposed by Allen et al. (2005) and Bathurst et al. (2008a). Steel grid pullout test results from BCGT
were provided by Dr. Bathurst to increase the pullout test database. A new design chart for the
calculation of pullout capacity was proposed by Bing Huang under the supervision of Dr.
Bathurst. The Chapter was submitted for publication as a full paper in the journal GeoRisk in
September 2009 (Bathurst et al. 2010a).

Chapter 5

Load data used for LRFD calibration were initially collected by Dr. Bathurst and Mr. Allen and
results of these analyses reported by Bathurst et al. (2008b, 2009). The pullout resistance data
were collected by Bing Huang from the open literature. Revisions to the current load and
resistance calculation charts were made by Bing Huang under the supervision of Dr. Bathurst.
This chapter was submitted for publication as a full paper in the ASCE Journal of Geotechnical
and Geoenvironmental Engineering in August 2009 (Huang et al. 2010a).

Chapter 6

Geosynthetic pullout test results were collected by Bing Huang and Dr. Bathurst. Bing Huang
performed the statistical analysis and proposed one modified and two new pullout models. The
chapter was published as a full paper in the ASTM Geotechnical Testing Journal in October 2009
(Huang and Bathurst 2009).

Chapter 7

Field installation damage trial results were collected and compiled by Bing Huang, Dr. Bathurst
and Mr. Allen. A new four-category criterion for the backfill soils according to the Dso particle
size was proposed by Dr. Bathurst and Mr. Allen. The statistical analyses were carried out by
Bing Huang under the supervision of Dr. Bathurst. This chapter has been submitted for
publication as a full paper in the journal Geotextiles and Geomembranes in December 2009
(Bathurst et al. 2010b).
Chapter 8

Geosynthetic creep test results were collected by Bing Huang from the open literature and additional data supplied by Mr. Allen and Dr. Bathurst. Bing Huang performed the statistical analyses under the supervision of Dr. Bathurst. This chapter has been submitted for publication as a full paper in the journal Geosynthetics International in December 2009 (Huang et al. 2010b).

Chapter 9

Geosynthetic load data used in this LRFD calibration analysis were collected by Dr. Bathurst and co-workers over the last decade and prior to this thesis work. All statistical analyses and modifications to the current AASHTO Simplified Method were made by Bing Huang under the supervision of Dr. Bathurst. Resistance models were selected from the pullout models reported in Chapter 6. This chapter will be submitted for publication as a full paper in the ASCE Journal of Geotechnical and Geoenvironmental Engineering.

Chapter 10

Geosynthetic load data used in this LRFD calibration analysis were collected by Dr. Bathurst and co-workers and made available to Bing Huang. The modified Simplified Method was taken from Chapter 9. The resistance bias statistics were based on the results reported in Chapters 7 and 8. This chapter will be submitted for publication as a full paper in the ASCE Journal of Geotechnical and Geoenvironmental Engineering once the background papers (Chapters 7 and 8) are accepted.

Chapter 11

Geosynthetic load data used in this LRFD calibration analysis were collected by Dr. Bathurst and co-workers and made available to Bing Huang. These load data were used to develop the K-
Stiffness Method and have been reported in a number of papers (e.g. Allen et al. 2002, 2003; Bathurst et al. 2008c; Miyata and Bathurst 2007). The K-Stiffness Method was used for load calculations. The resistance bias statistics were based on the results reported in Chapters 6, 7 and 8. All of the statistical analyses were performed by Bing Huang and the proposed resistance factors are a direct result of these calculations. This chapter will be submitted for publication as a full paper in the journal Geosynthetics International once the background papers (Chapters 7 and 8) are accepted.

Collaboration with Mr. Tony Allen

The work reported in this thesis was funded in part by the National Mechanically Stabilized Earth (MSE) Pooled Fund in the USA. This is a consortium of 12 USA Departments of Transportation: Alaska, Arizona, California, Colorado, Idaho, Minnesota, New York, North Dakota, Oregon, Utah, Washington and Wyoming. Mr. Allen is the State Geotechnical Engineer, Washington State Department of Transportation (WSDOT). WSDOT is the lead state DOT for the MSE Pooled Fund. Mr. Allen is the vice-chair of the T-15 Foundation and Walls Technical Committee of the AASHTO Bridge Subcommittee in the USA. This committee is tasked with maintaining and updating the AASHTO bridge design specifications referred to in this thesis work (e.g. AASHTO 2007, 2009) and identifying research needs. As the lead technical monitor of the national pooled fund research project, Mr. Allen played a major role in identifying the research objectives for this thesis work. He also supplied the writer and thesis supervisor with data and other privileged information without which much of this thesis work would not have been possible. Most importantly, Mr. Allen provided detailed opinion and scrutiny to ensure that the papers that comprise this thesis work are relevant to national (USA) design practice, easily understood by practicing engineers and of practical value to future updates of the AASHTO bridge design specifications.
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Bathurst, R.J., Huang, B. and Allen, T.M. 2010b. Interpretation of installation damage testing for reliability-based analysis and LRFD calibration. In review with Geotextiles and Geomembranes.


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The work presented in this thesis was carried out at the Department of Civil Engineering at the Royal Military College of Canada (RMC) under the supervision of Dr. Richard J. Bathurst. I am grateful to Dr. Bathurst for his invaluable suggestions, encouragement and patience during this journey.

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I would like to express my gratitude to my family, without whom my dream of completing a PhD program would not have come true. Unconditional support from my parents has never stopped. I thank my parents-in-law for taking care of my youngest son since he was born in 2007. My lovely sons, Huan-Chang and Jin-Chang, have provided me with the motivation to succeed. Finally, particular thanks are due to my wife, Chao Zhu, who put up with me throughout the course of this work. I am so proud of her.
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Chapter 1
General Introduction

1.1 General

Load and resistance factor design (LRFD) (often called limit states design) has been used in structural engineering for decades. It is now mandated in American Association of State Highway and Transportation Officials design specifications (AASHTO 2007, 2009) and the Canadian Highway Bridge Design Code (CHBDC 2006) for the design of foundations and earth retaining structures in transportation related structures. Included in the category of earth retaining structures are steel and geosynthetic reinforced soil wall systems.

A fundamental distinction between LRFD and conventional allowable stress design (ASD) is that LRFD explicitly considers the probability of failure of the structure while ASD does not. Most civil engineers are familiar with LRFD in the context of prescribed limit state equations and load and resistance factors taken from tables in design codes. However, as shown in Appendix A in this thesis, current LRFD approaches (AASHTO 2007, 2009) for the design of reinforced soil retaining walls are based on back-fitting to ASD past practice which uses factors of safety. In other words, these design methods are essentially ASD packaged to appear in an LRFD format.

At present, properly calibrated LRFD for the internal stability of reinforced soil retaining walls has not been undertaken in North America or elsewhere. The major focus of this thesis work has been the development of new LRFD equations for the internal stability of reinforced soil walls that have been calibrated using reliability-based analysis.

There are a number of important challenges to LRFD calibration for geotechnical engineering applications compared to structural engineering. First, structural engineering involves manufactured materials, such as concrete and steel which have properties that are well defined and vary within narrow ranges. This is not the case for geotechnical soil structures which are constructed with soils that are complex natural materials with potentially large range in mechanical properties. Second, the underlying deterministic models in structural engineering are most often analytical models developed from basic fundamental mechanics with proven good accuracy (i.e. low model error). However, this is not the case for many geotechnical soil-structure engineering applications. A large amount of geotechnical design practice is empirical or semi-empirical based and the models for load and resistance have much more prediction uncertainty. Recasting ASD past-practice into a properly calibrated LRFD framework is challenging.
necessary requirement to quantify uncertainty for the load side in limit state equations is the collection of a sufficient quantity of high-quality physical measurements from full-scale monitored structures. Similarly, good quality data from standard laboratory or performance tests are necessary to provide the statistical data for the resistance side in limit states design equations. A unique feature of this thesis is that LRFD calibrations were carried out using large databases of actual load and resistance data from many sources.

The determination of reinforcement loads for the internal stability design of reinforced soil retaining walls is particularly challenging. This is due to the complex mechanics of these composite systems that is the result of the mechanical behaviour of the constituent materials (e.g. polymeric reinforcement and soil) but also the interactions between the backfill soil, reinforcement and facing column. In North America, the current approach for calculating reinforcement loads in reinforced soil retaining walls is the AASHTO (2002, 2007) Simplified Method. Previous studies of walls under operational conditions have shown that the Simplified Method provides reasonable predictions for steel reinforced soil walls but does very poorly for geosynthetic reinforced soil walls (Allen et al. 2002, 2003; Bathurst et al. 2005, 2009a). The reason is that the Simplified Method for steel reinforced soil walls was originally adjusted by calibration against measured reinforcement loads. This was not the case for geosynthetic reinforced soil walls. Consequently, the Simplified Method over-predicts geosynthetic reinforcement loads by a factor of almost three on average and was judged to be excessively conservative (e.g. Bathurst et al. 2008a). Furthermore, the observed distribution of maximum reinforcement loads over wall height is fundamentally different from computed results. The poor prediction accuracy of geosynthetic reinforcement loads using the Simplified Method for operational conditions adds another challenge to LRFD calibration.

As noted earlier, physical data can be used to modify load models to improve prediction accuracy. Nevertheless, physical data that can be used to verify the accuracy of underlying deterministic models for reinforcement load are not always available or not in sufficient quantity to establish statistical measurements for calibration purposes. A strategy adopted in this thesis work has been the development of numerical simulation programs that have been verified against full-scale wall tests and then used to extend the database of physical test measurements and to fill in gaps. It should be noted that all predicted load data used in the LRFD calibrations reported in this thesis were calculated using analytical design equations rather than numerical simulations. The terms “predicted”, “calculated” and “computed” are used interchangeably in this thesis.
1.2 Prior related work

This thesis work is a continuation of a larger research program carried out by the GeoEngineering Centre at Queen’s-RMC since the 1990s under the supervision of Dr. R.J. Bathurst and in collaboration with other researchers. The larger research program is focused on the analysis, performance, prediction and design of reinforced soil retaining walls. Four prior milestones have provided the starting point for the current research program; they are:

(1) The collection of physical data from a series of 11 full-scale reinforced soil walls constructed at the Royal Military College of Canada (e.g. Bathurst et al. 2000; Bathurst et al. 2006; Bathurst et al. 2009b).

(2) Compilation of large load databases for steel and geosynthetic reinforced soil walls collected from carefully instrumented and monitored full-scale laboratory and field walls from around the world (e.g. Allen et al. 2002; Miyata et al. 2007a,b; Bathurst et al. 2008a).

(3) Development of an empirical design approach (K-Stiffness Method) to calculate reinforcement loads in geosynthetic reinforced soil retaining walls under operational conditions. This method has been proven to give much more accurate predictions of the magnitude and distribution of reinforcement loads compared to the current AASHTO Simplified Method (e.g. Bathurst et al. 2008a).


1.3 Objectives of the research program

The three major research objectives of this thesis work are:

(1) Develop and verify a numerical simulation program(s) that can be used to investigate the mechanics of reinforced soil wall behaviour and to extend the database of physical test measurements from previous related work and to fill in any gaps in the database.

(2) Carry out reliability-based LRFD calibration for the internal limit states of tensile rupture and pullout for steel and geosynthetic reinforced soil walls using different load and resistance models for each limit state.
(3) Identify further research needs.

To meet the first two principle objectives a number of secondary tasks were completed:

- Results of full-scale interface sliding tests for the concrete block-levelling pad interface were interpreted and used in numerical models.

- Verification of the numerical program (*Itasca 2005*) against results of full-scale reinforced soil walls constructed at the Royal Military College of Canada.

- Development of a modified hyperbolic constitutive model that can be used in numerical simulations to model the soil in earth structures under working stress conditions.

- The utility of the verified numerical code was used to explore the role of the restrained toe and facing column on the lateral load carrying capacity of geosynthetic reinforced soil walls.

- The results of the numerical simulations were compared to predicted geosynthetic reinforcement loads using the current AASHTO Simplified Method and K-Stiffness Method to independently demonstrate the improved accuracy of the K-Stiffness Method.

- A new modified AASHTO Simplified Method was developed to improve load prediction accuracy while maintaining the current AASHTO model structure.

- The utility of existing databases for reinforced soil wall loads was expanded by writing software that greatly improved examination of the data for back-analysis of parameters used in the modified AASHTO Simplified Method, K-Stiffness Method and for calculation of statistics for reliability-based LRFD calibration.

- New pullout models were developed and the prediction accuracy of current and new models for geogrid pullout assessed using a statistical approach.

- Databases and statistical tools were developed for the geosynthetic reinforcement rupture limit state. The installation damage data were collected from many sources. Recommendations for selection of soil gradations to be used in the interpretation of installation damage trials were proposed.
A database of geosynthetic laboratory creep tests was compiled from many sources and statistical tools developed to analyze test results for input in LRFD calibration.

1.4 Scope of the research program

Steel and geosynthetic reinforced soil retaining walls have been part of geotechnical design practice for about four decades. However, mechanical understanding of these systems is still subject to debate and actual operational margins of safety for internal stability design were speculative prior to this research work. The major motivation for this work is the need to migrate from ASD past practice to LRFD in order to bring the design of reinforced soil walls in line with current LRFD structural engineering practice. Nevertheless the scope of this research is limited, as discussed next.

For example, only two limit states are considered in this research work: tensile rupture of the steel and geosynthetic reinforcing elements due to over-stressing and pullout of the reinforcing layers from the soil anchorage zone. In design, there are also external limit states for base sliding, overturning about the toe, foundation bearing capacity and global instability modes of failure. Failure limit states can also be identified for reinforcement connections, block interface shear and the like (Bathurst and Simac 1994). However, the two limits states examined in this thesis work are very important and are useful candidates to carry out the first LRFD calibration for this class of retaining wall structures. The choice of limit states for LRFD calibration was also based in part on availability of measured load and resistance data. Nevertheless, as additional measured data are compiled in the future, load and resistance factors for other limit states can be developed. Primary candidates in this regard are connection and block-block interface shear failure since test protocols to quantify capacity have been in place for a number of years.

Another limitation of the scope of this thesis work is the restriction of LRFD calibration of geosynthetic reinforced soil walls for the pullout limit state to structures reinforced with geogrids. This is simply because there was a paucity of pullout data for geotextiles. Nevertheless, in North American practice the predominant geosynthetic materials for wall reinforcement applications are geogrids.

A distinguishing feature of the methodology adopted in this thesis work for LRFD calibration is the use of bias statistics to improve the prediction accuracy of the underlying deterministic load and resistance models. A consequence of this approach is that the load and resistance models proposed here for future design must be restricted to the envelope of material properties, static
load and boundary conditions matching the range of physical data in the case studies. As one example, caution must be exercised in extrapolating the load models used in this thesis to walls with weaker and (or) more compressible foundations which could generate other deformation mechanisms and load response which is not the same for walls constructed on competent foundations and good toe support (i.e. the walls in the current reinforcement load database).

In North America (Canada and the USA), LRFD is based on a factored resistance approach. Uncertainty in the calculation of resistance capacity is captured by a single resistance factor while uncertainties associated with the load side are treated by applying different load factors to each contributing load or load effect. In this thesis work, only one load effect due to soil self-weight plus permanent uniform surcharge applied at backfill surface is investigated (i.e. only one load term in each limit state function).

Finally, and consistent with the strategy to carry out LRFD calibration for simple and most manageable scenarios first, only static load conditions are considered. In fact, seismic loads are an important consideration and may control design using ASD past practice.

Clearly, the points raised here suggest a research-rich environment to extend the methodologies adopted in this thesis work to carry out LRFD calibration to a much broader range of wall scenarios including live loads due to traffic and earthquake-induced dynamic loads. Some of these opportunities are identified in the final chapter of this thesis.

1.5 Outline of the thesis

This thesis consists of twelve chapters including Chapter 1 (Introduction) and Chapter 12 (Conclusions and Recommendations). Chapters 2 to 11 are written in manuscript form for separate publication. Each is self-contained and independent of the other chapters with its own introduction, results, conclusions and references. Each chapter focuses on a specific issue of the entire research program.

In Chapter 1 (this chapter), an outline of the research program and main objectives are presented.

Chapter 2 presents the results of numerical modelling of two RMC full-scale test walls constructed with geosynthetic and steel reinforcement. The chapter focuses on the influence of constitutive soil model on prediction accuracy using linear elastic-plastic Mohr-Coulomb model, modified Duncan-Chang hyperbolic model, and Lade’s single hardening model. Numerical
results are compared to measured toe footing loads, foundation pressures, facing displacements, connection loads and reinforcement strains.

In Chapter 3, the numerical code verified in Chapter 2 is used to investigate the influence of magnitude of horizontal toe stiffness on the performance of reinforced soil segmental retaining walls under operational conditions. Results of full-scale interface shear tests were used to back-calculate toe stiffness values. This chapter provides insight into the contribution of footing restraint and structural column capacity on wall lateral earth force capacity. The prediction accuracy of the AASHTO Simplified Method and the K-Stiffness Method for reinforcement loads is discussed.

Chapter 4 reports the results of LRFD calibration for pullout and rupture limit states for steel grid reinforced soil walls. This chapter proposes a new design chart to improve the accuracy of the current pullout model and to remove hidden dependency with calculated pullout capacity values. This chapter also compares the practical consequences of using the proposed LRFD approach and ASD past practice based on computed actual (operational) factors of safety.

Chapter 5 reports the results of LRFD calibration for pullout and rupture limit states for steel strip reinforced soil walls. In order to ensure sensible values for the magnitude of load and resistance factors a small adjustment to the calculation of reinforcement loads and two new design charts for the estimation of the pullout resistance coefficient for smooth and ribbed steel strips are proposed. Practical consequences of using the proposed LRFD approach and ASD past practice are presented.

In Chapter 6, the accuracy of the current AASHTO/FHWA (2001) in-soil geogrid pullout model is examined using a statistical approach applied to a large database of laboratory pullout test results. The interpretation of pullout tests is examined using five different approaches. Two new pullout models using presumptive values for model parameters are proposed for the case when project-specific pullout data are not available. Practical recommendations are made to select reinforcement lengths during the experimental design for pullout testing to increase the likelihood of a pullout rather than rupture mode of failure in the laboratory.

In Chapter 7, interpretation of geosynthetic installation damage trial results for use in reliability-based design for the reinforcement rupture limit state is described. Installation damage bias statistics are calculated for six different categories of geosynthetic and four categories of backfill soils classified according to the $D_{50}$ particle size. The sensitivity of probability of failure to
magnitude of installation damage bias statistics is discussed. The data and interpretation of test results presented here are required to carry out LRFD calibration for rupture in Chapters 9, 10 and 11.

In Chapter 8, interpretation of laboratory creep test results for use in reliability-based design for the reinforcement rupture limit state is described. Bias statistics associated with the determination of creep strength reduction at the end of service lifetime are computed using a database of tests from a large number of sources. The variability of original tensile strength of geosynthetic products is also determined. The impact of long-term creep strength on practical reliability-based LRFD calibration is discussed.

Chapter 9 reports the results of LRFD calibration for the pullout limit state for geogrid reinforced soil walls using the AASHTO Simplified Method for load calculations. The current Simplified Method is modified to improve prediction accuracy. The resistance (pullout) models used in the calibration are taken from Chapter 6. A series of computed resistance factor values is presented that matches the current AASHTO-prescribed load factor value of 1.35 and a probability of failure of 1%. The actual (operational) factors of safety based on ASD past practice and using the current and proposed modified Simplified Method within an LRFD framework are compared.

Chapter 10 presents the results of LRFD calibration for the limit state of reinforcement rupture for geosynthetic reinforced soil walls. Load predictions are made using the original and modified AASHTO Simplified Method described in Chapter 9. Resistance (long-term tensile strength) bias statistics are based on results reported in Chapters 7 and 8. A series of computed resistance factor values is presented.

Chapter 11 presents the results of LRFD calibration for the limit states of reinforcement pullout and rupture for geosynthetic reinforced soil walls. In this chapter, the K-Stiffness Method is used for the first time as the reinforcement load model in LRFD calibration. Long-term tensile strength bias statistics are taken from Chapters 7 and 8 and two pullout resistance models from Chapter 6. A series of computed resistance factor values is recommended. The advantages of using the K-Stiffness Method for load calculations are outlined.

In Chapter 12, the main findings of this thesis are summarized and recommendations are given for further research.
REFERENCES


Chapter 2
Numerical Study of Reinforced Soil Segmental Walls Using Three Different Constitutive Soil Models

2.1 Introduction

Geosynthetic reinforced soil walls are now used routinely as retaining wall structures since their introduction in the 1970s (Allen et al. 2002). Current design methods in North America for internal stability are based on limit-equilibrium tie-back wedge methods of analysis that were adapted from steel reinforced soil retaining wall design (AASHTO 2002). Statistical analyses of instrumented full-scale field and laboratory geosynthetic reinforced soil walls has revealed that tie-back wedge methods are generally excessively conservative, and predicted reinforcement loads under operational conditions do not correlate with measured values (Allen et al. 2003; Bathurst et al. 2005, 2008; Miyata and Bathurst 2007a,b). Nevertheless, the number of instrumented wall structures with sufficient quality and range of quantitative performance data to investigate the accuracy of existing and proposed new analysis methods is limited. A strategy to extend the database of physical wall tests is to carry out numerical analyses using finite element method (FEM) or finite difference method (FDM) codes. Examples of numerical simulations for reinforced soil retaining walls are reported by Karpurapu and Bathurst (1995), Rowe and Ho (1997), Ling and Leshchinsky (2003), Leshchinsky and Vulova (2001) and Yoo and Song (2006) amongst others. However, very few cases exist in the literature in which an attempt has been made to verify the results of numerical models against carefully constructed, instrumented and monitored full-scale walls. Examples where this has been done are Ling (2003), Hatami and Bathurst (2005, 2006) and Guler et al. (2007). Useful reviews of geosynthetic reinforced soil wall numerical modelling efforts can be found in the papers by Bathurst and Hatami (2001) and Ling (2003).

An important question that arises when assessing the accuracy of numerical modelling results is the influence of the constitutive soil model on the accuracy of numerical predictions. In the numerical modelling studies cited above, the behaviour of the backfill soil was most often simulated using the Duncan-Chang hyperbolic model (Duncan et al. 1980) or the simple linear elastic-plastic Mohr-Coulomb model. It is reasonable to assume that these models may give different predictions for the same nominally identical reinforced soil wall. Furthermore, neither
model can simulate the strain-softening behaviour of granular soils which may be a concern if walls are of sufficient height and (or) subjected to large surcharge loads.

Ling (2003) used a non-linear elastic (hyperbolic) model and a generalized plasticity model to simulate the construction behaviour of a full-scale instrumented modular block (segmental) wall reported by Tajiri et al. (1996). He found that both models gave similar results for facing displacements, earth pressures acting against the facing, vertical earth pressures at the base of the backfill soil and reinforcement strains. Furthermore, Ling concluded that there was acceptable agreement between the measured and predicted results. Nevertheless, the modelling effort was restricted to a single structure with one reinforcement type.

Hatami and Bathurst (2005) compared the results of FDM simulations for three reinforced soil segmental retaining walls with measured results from physical tests. The walls varied only with respect to type and number of the polymeric reinforcement layers. They used the Duncan-Chang model combined with a Mohr-Coulomb failure criterion and the linear elastic-plastic Mohr-Coulomb model for the soil in the simulations. Numerical results were shown to be in good agreement with measured toe boundary forces, vertical foundation pressures, facing displacements, connection loads, and reinforcement strains. Numerical results using the linear elastic-plastic Mohr-Coulomb model for the soil also gave good agreement with measured wall displacements and boundary toe forces but gave a poorer prediction of the distribution of strain in the reinforcement layers.

The good agreement between predicted and measured results in the two cited studies is encouraging. However, further investigation is warranted to explore the influence of different constitutive soil models on predictions of wall performance features for walls with other reinforcement materials and to compare these predictions to measured results.

In the current study, three well-known constitutive soil models are used to simulate the construction and surcharge loading response of two carefully constructed, instrumented and monitored reinforced soil segmental walls reported by Hatami and Bathurst (2005, 2006). One wall was constructed with extensible geogrid reinforcement and the other with a relatively stiff welded wire mesh. The three soil models in order of increasing complexity are: a) linear elastic-plastic Mohr-Coulomb; b) Duncan-Chang hyperbolic model (Duncan et al. 1980) with a modification by Boscardin et al. (1990), and; c) Lade’s single hardening constitutive model for frictional soils (Kim and Lade 1988; Lade and Kim 1988a,b). An excellent review of the
capabilities and shortcomings of different soil constitutive models can be found in the paper by Lade (2005). All numerical simulations in the current study were carried out using the program FLAC (Itasca Consulting Group 2005).

2.2 RMC full-scale physical model tests

A series of 11 full-scale reinforced soil retaining walls has been tested using an indoor test facility at the Royal Military College (RMC) of Canada. Some of the initial test results have been reported by Bathurst et al. (2000, 2001, 2006). These test walls were carefully constructed, instrumented and monitored. For example, more than 300 instrument measurement points were placed within the wall and at the boundaries of these structures to measure boundary displacements, boundary loads and pressures, connection loads and internal reinforcement strains. Two of these walls are considered in this chapter (Wall 1 and Wall 6). The test walls were seated on a rigid foundation and were 3.6 m high, 3.3 m wide and retained a sand backfill to a distance of about 6 m from the wall face. The wall facing was built with commercially available solid masonry concrete blocks placed at a target batter of 8 degrees from the vertical using an integral cast concrete shear key. Each block was 300 mm in width (toe to heel), 150 mm in height and 200 mm in length. The backfill soil was a clean washed sand with D50 = 0.34 mm, coefficient of curvature Cc = 2.25, and coefficient of uniformity Cu = 1.09. The walls were backfilled in 150 mm lifts matching the height of the solid block units. The backfill in the first four walls was compacted with a light-weight gasoline driven vibratory plate compactor. The backfill in subsequent walls was compacted to the same density using a heavier electro-mechanical vibrating rammer compactor to prevent fumes from accumulating in the indoor test facility. Following construction, a series of stepped surcharge loads were applied uniformly across the entire soil surface using an airbag system. The surcharge loads were typically applied in 10 kPa increments. Additional details of these two RMC full-scale test walls are reported by Hatami and Bathurst (2005, 2006).

Wall 1 and Wall 6 were selected because they have very different reinforcement stiffness. Wall 1 was constructed with six layers of a weak biaxial punched and drawn polypropylene (PP) geogrid. Wall 6 was nominally identical to Wall 1 but was constructed with six layers of steel welded wire mesh (WWM). The vertical reinforcement spacing was 0.6 m and the reinforcement length was 0.7 times wall height in both structures. Table 2.1 shows that the WWM for Wall 6 is about 25 times stiffer than the PP geogrid for Wall 1.

Table 2.1
2.3 Numerical models

2.3.1 General
In the current study, an updated version of the FLAC code reported by Hatami and Bathurst (2005, 2006) was used to simulate the plane strain response of Wall 1 and Wall 6. Figure 2.1 shows the FLAC grid used in the numerical simulations. The FLAC library Mohr-Coulomb model was used for the linear elastic-plastic Mohr-Coulomb case in this investigation. The modified Duncan-Chang model and Lade’s model were coded in C++ and implemented using the FLAC UDM (User Defined Model) option.

2.3.2 Construction and surcharge load simulation
In this study, wall performance during construction and subsequent stepped uniform surcharge loading was simulated. The bottom-up construction process in the physical RMC model tests was modelled using sequential 0.15-m thick layers. The moving local datum in the physical tests was captured as each row of facing units and soil layer was placed over the previous lift in the numerical model. A transient uniform pressure (8 kPa for Wall 1 and 16 kPa for Wall 6) was applied to each soil lift to account for the influence of different dynamic compaction used in the construction of the two walls (Hatami and Bathurst 2005, 2006). The influence of type of compaction equipment on the performance of the RMC test walls is reported by Bathurst et al. (2009). Following construction of each wall model, a uniform surcharge load was applied to the backfill surface in 10 kPa increments matching the typical load steps in the physical test walls. The model was solved to equilibrium for each construction lift and surcharge load increment.

2.3.3 Reinforcement constitutive model
The soil reinforcement layers in this study were simulated using FLAC cable elements. The strength and stiffness of the polypropylene geogrid is load, time and temperature dependent. The load-strain-time properties for the polymeric reinforcement were determined from constant-load creep tests carried out at RMC (Walters et al. 2002). Because the reinforcement material tests and RMC physical wall tests were carried out at the same in-door ambient temperature of 20 ± 1 °C, temperature effects were not an issue in the numerical model. The load-strain-time characteristics of the PP reinforcement can be expressed as a hyperbolic function of the secant axial stiffness of the reinforcement (cable) elements. The cable elements were continuously updated in FLAC using the equivalent tangent stiffness function \( J_t(\varepsilon, t) \) proposed by Hatami and Bathurst (2006):
\[ J_1(\varepsilon, t) = \frac{1}{J_o(t)} \left( \frac{1}{J_o(t)} + \frac{\eta(t)}{T_f(t)} \varepsilon \right)^2 \]

where: \( J_o(t) \) is the initial tangent stiffness, \( \eta(t) \) is a scaling function, \( T_f(t) \) is the stress-rupture function for the reinforcement, \( \varepsilon \) is the reinforcement strain and \( t \) is time (i.e. duration of loading).

The properties of PP reinforcement used in this study correspond to the duration of loading matching the actual construction period of the RMC test walls. The WWM reinforcement was modelled as linear elastic-plastic material since it does not exhibit time-dependent (creep) behaviour at the load levels investigated here. The constant stiffness of the WWM reinforcement was expressed using **Equation 2.1** with \( \eta(t) = 0 \). A limiting yield strain of 0.2% was used to match results of tensile tests performed on WWM longitudinal members. For simplicity in the numerical code, the maximum tensile capacity of the WWM was set to the corresponding yield strength \( T_y \). The measured strains in the WWM reinforcement at the 80 kPa surcharge load level were observed to reach yield as demonstrated later in this chapter. The reinforcement material properties from laboratory testing at RMC are given in **Table 2.1** and are the same values used by **Hatami and Bathurst (2006)**.

### 2.3.4 Backfill soil

The three constitutive soil models are briefly described in this section in order of relative complexity.

**Mohr-Coulomb model**

The soil is modelled as a linear elastic-plastic material using the Mohr-Coulomb model in the FLAC library. The elastic behaviour is expressed by the generalized Hooke’s law with constant Young’s modulus (E) and constant Poisson’s ratio (\( \nu \)) or equivalent (i.e. using shear modulus G). The yield (failure) criterion is taken as the Coulomb failure criterion for frictional materials. A non-associated flow rule is adopted in the FLAC Mohr-Coulomb model and the shear potential function defined using dilation angle (\( \psi \)). A small cohesion value was assumed for the RMC soil to ensure numerical stability for soil zones with very low confining pressure (**Table 2.2**).
Limitations of the Mohr-Coulomb model used in this study are: (i) confining pressure and strain-dependent soil stiffness behaviour are not captured; and (ii) shear-softening behaviour is not included.

In this study the magnitude of $E$ was taken as 40 MPa for Wall 1 and 80 MPa for Wall 6 deduced from vertical compression measurements of the column of soil located equidistant from the wall face and the back of the test facility during surcharging. The relatively higher stiffness of the soil for Wall 6 is consistent with Filtz et al. (2000) who compared the influence of a rammer-type compactor with a vibrating plate-type compactor on granular soil compacted to the same dry density. The elastic modulus value used for Wall 1 was observed to fall within the range of secant modulus values taken at 0.2% to 0.7% strain in Figure 2.2a. However, a well-known disadvantage of linear elastic theory in geotechnical modelling of frictional soils is that selection of a single value for $E$ is problematic, particularly at low confining pressures. Hence, no recommendations are offered here regarding the selection of a suitable $E$ value for other soils and for walls of greater height.

The FLAC Mohr-Coulomb model is capable of modelling shear dilatancy by using a non-zero $\psi$ value. In fact, non-linear elasticity can be readily implemented in this model. However, the influence of non-linear elasticity on simulation results is not investigated in the current study. Since only the major and minor principal stresses are included in the failure criterion, effects of intermediate principal stress are excluded. Hence the plane strain conditions that are applicable to the RMC test walls are not taken into account in the model. However, the soil friction angle from plane strain testing was used as described below.

Modified Duncan-Chang hyperbolic model

The soil is modelled as a non-linear elastic material with a hyperbolic stress-strain function. This model is based on the generalized Hooke’s law. The stress-dependent elastic tangent modulus is expressed as:

$$E_t = \left[ 1 - \frac{R_f (1 - \sin \phi)(\sigma_1 - \sigma_3)}{2c \cos \phi + 2\sigma_3 \sin \phi} \right]^2 K_c \frac{\sigma_3}{p_a}^n$$

[2.2]
where \( \sigma_1 \) = major principle stress, \( \sigma_3 \) = minor principle stress, \( p_a \) = atmospheric pressure and the other parameters are defined in Table 2.2. The unloading and reloading behaviour using the Duncan-Chang model is simulated as:

\[
[2.3] \quad E_{ur} = K_{ur} p_a \left( \frac{\sigma_1}{p_a} \right)^n
\]

where, \( K_{ur} = 1.2K_e \) (Duncan et al. 1980).

In previous studies by Hatami and Bathurst (2005, 2006) a variable Poisson’s ratio value was adopted by taking the formulation for bulk modulus (B) as:

\[
[2.4] \quad B = K_b p_a \left( \frac{\sigma_3}{p_a} \right)^m
\]

In this original Duncan-Chang formulation, B is a function only of \( \sigma_3 \), and \( K_b \) and m are bulk modulus number and bulk modulus exponent, respectively. The B value is restricted to the following range in the user-defined model in the current investigation and past studies:

\[
[2.5] \quad \frac{E_i}{(1-2\nu_{t_{\text{min}}})} \leq B \leq \frac{E_i}{(1-2\nu_{t_{\text{max}}})}
\]

where \( \nu_{t_{\text{max}}} = 0.49 \) and \( \nu_{t_{\text{min}}} = 0 \).

The Duncan-Chang model (Duncan et al. 1980) was developed for axi-symmetric (triaxial) loading conditions and cannot explicitly account for the plane strain boundary conditions that are applicable to the reinforced soil walls in this chapter. Under plane strain conditions, \( \sigma_2 \geq \sigma_3 \) and hence confining pressure is under-estimated. Hatami and Bathurst (2005) showed that the Duncan-Chang parameters back-fitted from triaxial tests on the RMC sand under-estimated the stiffness and strength of the same soil when tested in a plane strain test apparatus. To account for this effect, they increased the elastic modulus number by a factor of 2.25 to simulate the plane strain test results. To overcome this deficiency in the original Duncan-Chang formulation, a
unique set of input parameters that applies to both triaxial and the plane strain loading conditions is used in the current study by adopting the bulk modulus formulation proposed by Boscardin et al. (1990):

\[
[2.6] \quad B = B_i \left[ 1 + \frac{\sigma_m}{B_i \varepsilon_u} \right]^2
\]

where: \( \sigma_m = \text{mean pressure} = (\sigma_1+\sigma_2+\sigma_3)/3; B_i \) and \( \varepsilon_u \) are material properties that are determined as the intercept and the inverse of slope from a plot of \( \sigma_m/\varepsilon_{\text{vol}} \) versus \( \sigma_m \), respectively, in an isotropic compression test and \( \varepsilon_{\text{vol}} \) is volumetric strain.

It is known that granular soils exhibit greater friction angle values under plane strain conditions than under axi-symmetric loading conditions (e.g. Hanna 2001; Rowe 1969; Lade 2003). The peak friction angle from plane strain testing of the RMC soil was used in the numerical models. In practice, plane strain test results are seldom available for design engineers. However, empirical relationships are available to increase the peak friction angle from triaxial test results to the peak plane strain friction angle (e.g. Lade and Lee 1976; Hanna 2001) and to use this adjusted value in the modified Duncan-Chang model proposed here. This approach demonstrates how conventional triaxial compression test results in combination with the modified Duncan-Chang model can be used for plane strain numerical modelling of earth structures with frictional fills.

In order to account for the influence of heavier compaction equipment on Wall 6, Hatami and Bathurst (2006) increased the stiffness by a factor of 2.45 from Wall 1. This value is greater than the factor of 2 used in the Mohr-Coulomb model in this chapter. This is a reasonable assumption because the elastic component of the Mohr-Coulomb model is linear while the Duncan-Chang model is a non-linear elastic model. The same factor of 2.45 was used to increase the values of \( K_e \) and \( B_i \). The value of \( \varepsilon_u \) was assumed to be constant because the soil in these two walls was compacted to approximately 95% of standard Proctor maximum dry density.

Limitations of the Duncan-Chang model used in the current study are: (i) post-peak shear strength strain softening cannot be captured; (ii) shear dilatancy is not taken into account; and (iii) failure of the soil is not modelled realistically.

*Lade’s model*
Lade’s model is an elastic-plastic, work-hardening and -softening constitutive model with a single yield surface developed for frictional geo-materials (Kim and Lade 1988; Lade and Kim 1988a,b). The implementation of this model within an FEM program has been carried out by Lade and Jakobsen (2002) and Jakobsen and Lade (2002). In the current study, Lade’s model is implemented within a FLAC code for the first time to the best knowledge of the writer. For brevity, the results of an independent verification of the FLAC code that was carried out by the writer using triaxial and plane strain compression tests are not reported here. Similarly, space permits only a brief description of the basic components of this model.

The elastic soil behaviour was simulated by the generalized Hooke’s law using an elastic model developed by Lade and Nelson (1987). Poisson’s ratio is assumed to be constant and the Young’s modulus (E) is calculated from:

\[ E = M p_s \left[ \frac{I_1}{p_s} \right]^2 + 6 \left( \frac{1 + v}{1 - 2v} \right) \frac{J_2}{p_s^2} \]

The elastic properties of the Lade’s model are related to the Duncan-Chang parameters by:

\[ K_{ur} = 3^{2\kappa} M \]

and

\[ n = 2\kappa. \]

The elastic component of Lade’s model reduces to Equation 2.3 for hydrostatic pressure conditions. A symmetric bullet-shape failure surface in principal stress space is defined by the failure criterion, specifically:

\[ \left( \frac{I_1^3}{I_3} - 27 \right) \left( \frac{I_1}{p_s} \right)^m = \eta_i \]
Once the stress path reaches the failure surface strain-softening may take place. In plasticity theory, the flow rule can be expressed as:

\[ 2.11 \quad d\varepsilon^p = d\lambda_p \frac{\partial g_p}{\partial \sigma} \]

Lade’s model adopts a non-associated flow rule and the plastic potential function \( g_p \) is given by:

\[ 2.12 \quad g_p = \left( \psi_1 \frac{I_1}{I_3} - \frac{I_1^2}{I_2} + \psi_2 \left( \frac{I_1}{p_a} \right) \right)^\mu \]

The yield surfaces are defined as contours of constant total plastic work. The yield criterion is a function of the stress tensor and plastic work and can be expressed as:

\[ 2.13 \quad f'_p (\sigma) = f''_p (W_p) \]

where

\[ 2.14 \quad f'_p (\sigma) = \left( \psi_1 \frac{I_1}{I_3} - \frac{I_1^2}{I_2} + \psi_2 \left( \frac{I_1}{p_a} \right) \right)^h e^q \]

and

\[ 2.15a \quad f''_p (W_p) = \left( \frac{1}{D} \right)^{1/p} \left( \frac{W_p}{p_a} \right)^{1/p} \quad \text{(for hardening)} \]

\[ 2.15b \quad f''_p (W_p) = Ae^{-BW_p/p_a} \quad \text{(for softening)} \]

In the formulation of Lade’s model, \( I_1, I_2, I_3 \) and \( J'_2 \) are invariants of the stress tensor and the other parameters are related to the model parameters summarized in Table 2.2. Effects of cohesion, pre-shearing and over-consolidation are included in Lade’s model. Details can be found in the cited papers by Lade and co-workers.
The larger compaction effort applied to Wall 6 was assumed to only influence the hardening parameter \( C \) in Lade’s model. Under isotropic compression conditions, the plastic work \( W_p \) is related to parameter \( C \) in Lade’s model by:

\[
W_p = C_p \left( \frac{I_1}{p_a} \right)^p
\]

Here, \( I_1 \) is three times the isotropic pressure and other parameters are defined in Table 2.2. Parameters \( C \) and \( p \) can be determined in theory from isotropic compression tests. However, the plastic strain increments at the low confining pressures that exist in the RMC walls are so small that they could not be measured with sufficient resolution or repeatability using conventional triaxial testing equipment. Instead, \( C \) and \( p \) values were determined from a best fit of predicted stress-strain curves to the independent sets of measured triaxial and plane strain test results. A factor of 0.5 applied to parameter \( C \) was used to increase the stiffness of the backfill in Wall 6.

A disadvantage of Lade’s model is that many model parameters lack physical meaning for geotechnical engineers who are familiar with conventional soil properties such as friction angle and cohesion. The advantage of the model is that the effects of stress-dependent stiffness, shear dilatancy and strain-softening are included. Moreover, Lade’s model explicitly accounts for the effects of plane strain conditions and no empirical adjustment is required to increase elastic modulus values from triaxial test results as described by Hatami and Bathurst (2005) for the Duncan-Chang model. However, greater computing resources and execution time are required.

**Comparison of predicted and measured response of triaxial and plane strain tests**

*Figure 2.2* shows measured and predicted stress-strain behaviour under triaxial and plane strain loading conditions using the three soil models. The plots show that the modified Duncan-Chang hyperbolic model and Lade’s model are reasonably accurate for the triaxial tests (*Figure 2.2a*) for axial strains up to (say) 4%. Lade’s model is judged to be most accurate for prediction of volumetric strain response. The Duncan-Chang model cannot predict dilatancy and this explains the compression-only response in *Figure 2.2b*. The Mohr-Coulomb model gives the poorest fit. Nevertheless it captures the trend in volumetric response but over-estimates the magnitude of dilatancy. The Duncan-Chang model captures the stress-strain response of the plane strain tests...
over a range 1% to 3% depending on stress level (Figure 2.2c). However, only Lade’s model is able to predict post-peak strain softening.

2.3.5 Interfaces and boundary conditions

The interfaces at the facing column-backfill, block-block, foundation-backfill, and reinforcement-backfill were modelled as linear spring-slider systems with interface shear strength defined by the Mohr-Coulomb failure criterion (Itasca 2005). The interface shear stiffness between modular blocks was set to 40 MN/m/m based on results of direct shear tests carried out on the solid masonry blocks used in the reference RMC test walls. The mechanical behaviour of the interface with a shear key is assumed to be the same with and without the presence of the geogrid in order to simplify the numerical model (Hatami and Bathurst 2005, 2006).

A fixed boundary condition in the horizontal direction was assumed at the numerical grid points on the backfill far-end boundary, representing the bulkheads that were used to contain the soil at the back of the RMC test facility. A fixed boundary condition in both horizontal and vertical directions was used at the foundation level matching the test facility concrete strong floor. The toe of the facing column was restrained horizontally by a very stiff spring element with $K_{\text{toe}} = 4$ MN/m/m matching the measurement at this boundary in the RMC physical tests. Interface properties are summarized in Table 2.3. The reinforcement (cable) elements were assumed to be bonded to the backfill using the FLAC grout utility. A large bond strength along the reinforcement-backfill interface was selected to prevent slip and to simplify the model. A perfect bond has been assumed in previous numerical modelling of geosynthetic reinforced soil walls by previous researchers cited in the introduction. Measured reinforcement displacements showed that this is a reasonable assumption under working load conditions for the combination of reinforcement products and compacted sand used in the RMC physical tests. It should be noted that this assumption may not be true when the wall is subjected to larger surcharge loads. Nevertheless this no-slip interface model assists to keep the interpretation of numerical results as simple as possible.

2.4 Simulation results

2.4.1 General

A single numerical FLAC grid was used to simulate Wall 1 and Wall 6 (Figure 2.1). Each wall was simulated using the three constitutive soil models described in the previous section while the properties of the other materials were kept the same. For brevity, only selected physical and
numerical results are presented. The focus in this section is on comparisons of predicted and measured wall performance at the end of construction and during surcharge loading up to 80 kPa.

Measured strains reported later in this chapter show that the 80 kPa surcharge level was the maximum surcharge load that could be applied to keep the strains in the polymeric reinforcement layers within about 3%. The results of analysis of a large number of full-scale monitored field walls constructed with granular backfill and geosynthetic reinforcement materials have shown that walls with strains below this value exhibit good performance (i.e. satisfy serviceability criteria) (Allen et al. 2003; Bathurst et al. 2005). At higher strains, contiguous shear zones can develop well within the reinforced soil zone and the performance of the wall approaches a collapse limit state. In this chapter we are focused on numerical simulations that can be used to predict operational (working stress) conditions rather than incipient wall collapse. Wall behaviour under operational conditions is of interest to engineers as geotechnical design for reinforced soil walls moves toward performance-based design (Allen et al. 2005).

2.4.2 Toe reactions

Experimental and numerical results have shown that a stiff horizontal toe at the base of a wall can provide a significant contribution to horizontal equilibrium of a hard facing in geosynthetic reinforced soil walls (e.g. Bathurst 1993; Rowe and Ho 1997; Huang et al. 2007; Bathurst et al. 2006, 2007). Analysis of a database of full-scale monitored field and laboratory walls has shown that geosynthetic reinforcement loads are typically attenuated at the base of hard-faced walls as a result of the stiffer toe restraint (Allen et al. 2003). Figure 2.3 shows the calculated and measured values of vertical and horizontal toe loads for Wall 1 and Wall 6 during construction and subsequent surcharging. Numerical results using all three soil models are similar and in good agreement with measured values particularly during construction. However, the discrepancy between the predicted and measured data becomes more apparent in Wall 1 when the surcharge pressure is greater than 50 kPa. This suggests that the numerical model predictions may become less accurate if the deformation of a wall is relatively large. Also, the vertical toe loads tend to be over-predicted by the Mohr-Coulomb and Lade model at this stage. The numerical models are able to capture the trend towards slightly lower horizontal toe loads during surcharging that may be expected for the wall constructed with a stiffer metallic reinforcement material. Finally, predicted vertical toe loads are shown to be greater than the facing column self-weight. This is due to down-drag forces that are generated at the connections as the soil is placed and compacted against the relatively vertically rigid facing column and during subsequent outward wall rotations of the facing during surcharging.
2.4.3 Vertical foundation pressures

Figure 2.4 shows calculated and measured vertical foundation earth pressures for both walls normalized against soil column pressure ($\gamma H + q$). The range bars on measured data points are estimates of the range of pressures recorded over the period of time that the corresponding surcharge load was applied. For each wall the three numerical models give very similar results. In addition, all three models capture the trend towards a reduction in vertical earth pressure immediately behind the facing column due to down-drag forces on the connections. At many locations, the predicted and measured values are visually indistinguishable. Physical and numerical results show that in the retained soil zone the vertical earth pressures acting on the rigid foundation are close to or equal to $\gamma H + q$. The consistency between numerical results is largely ascribed to computed gravity loads and surcharge pressure effects which are independent of the constitutive soil model employed.

2.4.4 Wall facing displacements

Figure 2.5 shows the calculated and measured relative facing displacements at the end of construction. In this plot the displacements are with respect to datum readings taken at the time each reinforcement layer was placed and the overlying soil layer compacted in the wall. Hence, these plots have a moving datum. An advantage of this approach compared to simply plotting wall profiles is that it eliminates the effect of unquantifiable manual adjustments required to set the facing units during construction. Hence, moving datum wall profiles are easier to predict numerically. Error bars on the measurement data points capture the range of movements that occurred across the middle 1-m wide instrumented section of the physical models as a result of the discrete modular block construction. The range of measured displacement values is very small for both walls but there is a detectable predicted smaller movement for the wall with the stiffer WWM reinforcement. Numerical results are considered by the writer to be reasonably accurate given the small displacements involved. There is no consistent trend in displacement profiles that can be ascribed to soil model type. From a practical point of view, the differences are insignificant and all predictions are reasonably close to the experimental data.

Figure 2.6 shows the numerically predicted and experimental post-construction facing displacements during surcharge loading. In all cases the calculated profiles show increasing wall deformation profiles with surcharge level consistent with the trend in measured profiles. For clarity, only the range bars for maximum and minimum wall displacements for the 80 kPa surcharge level are shown. For Wall 1 constructed with extensible PP geogrid reinforcement, the
Mohr-Coulomb and Lade’s model appear to give better agreement with measured data points than the Duncan-Chang model (Figure 2.6a). However, predictions using the Duncan-Chang model reasonably match the measured data up to the surcharge load of 60 kPa. There is not a clear best fit based on soil model type for the measured and calculated profiles for the stiffer metallic reinforced soil wall (Wall 6) (Figure 2.6b). It can be argued that for this wall, the Mohr-Coulomb and Lade models give better estimates of the maximum post-construction wall deformation at 20 kPa surcharge level than the modified Duncan-Chang model.

2.4.5 Reinforcement-facing connection loads

An important feature of reinforced soil walls constructed with a hard facing is the design of the connections between the wall and the reinforcement layers. As noted earlier, additional down-drag forces can develop on the connections due to relative downward movement of the soil behind the facing. Figure 2.7 shows measured and calculated connection loads at end of construction and during the 80 kPa surcharge load increment for both walls.

Predicted and calculated values show that connection loads are larger (as may be expected) for the stiffer reinforcement case. At the end of construction the predicted values are very similar using the three constitutive soil models (Figure 2.7a). The distribution of measured connection loads is less uniform than the predicted values particularly at the bottom of the wall. This may be the result of manual block seating efforts during initial construction, the effect of proximity of compaction equipment to the face, or possibly the influence of proximity of the bottom-most reinforcement layers to the rigid foundation base and restrained toe boundary. Nevertheless, there is very good agreement with respect to the range of connection loads for both walls at end of construction. While not shown here, numerical predictions for the 50 kPa surcharge level also fell within the range of measured connection loads for both walls.

The range of measured connection loads during the 80 kPa surcharge increment is very much larger for the relatively flexible PP geogrid reinforced case (Wall 1) than for the WWM structure (Wall 6) (Figure 2.7b). The Duncan-Chang model under-predicts Wall 1 connection loads. With the exception of the top of the wall, the Mohr-Coulomb and Lade’s model predicted loads that fell within measurement ranges. For Wall 6, all three models gave similar results. There is a consistent over-prediction of connection loads but, in the opinion of the writer, the values are still acceptably close from a practical point of view considering the complexity of the wall systems investigated. It is possible that there was some slip between the metallic WWM and the soil in
Wall 6 which resulted in some load relaxation which propagated to the connections at the front of each reinforcement layer.

2.4.6 Reinforcement strains

Figure 2.8 shows the simulated and measured reinforcement strain distributions at the end of construction and at the end of the 80 kPa surcharge level for both walls. The measured data for Wall 1 were taken from strain gauges and pairs of extensometer points. The strain gauge readings represent the mean of multiple readings taken at the same nominal location from the back of the facing. Only strain gauges were used to measure the very much smaller strains in the longitudinal members of the WWM structure (Wall 6). The range bars represent ±1 standard deviation based on statistical analysis of measurement redundancy in the RMC tests (Bathurst et al. 2002).

As a general observation, the predicted strains using all three soil constitutive models are judged by the writer to be in good agreement with the measured results. In many cases the numerical results pass within the plotted measurement limits. However, there are some notable discrepancies. For example, predicted strains for layer 5 in Wall 6 at end of construction and during surcharging fall below measured values (Figure 2.8b). However, the peak measured and predicted strains in any layer are most often in agreement. This is a desirable outcome if the objective of these simulations was the selection of a suitably stiff reinforcement material for design. Where there are discrepancies they may be due to the effects of local over-compaction leading to pre-stressing of the reinforcement layer or other construction variables. Differences in predicted strains for Wall 1 are greater than those for Wall 6. This may be due to the very low strain levels for the metallic reinforced soil wall (i.e. predominantly elastic response for all three model types).

An interesting pattern in predicted strains can be seen in Figure 2.8a for the PP geogrid reinforced soil wall at the end of construction. There is a local peak in strain values that propagates further along the length of the reinforcement layer with height above the base for the Mohr-Coulomb simulation results and to a lesser extent for Lade’s model. This peak does not appear in the Duncan-Chang model which may be because this model cannot generate soil dilatancy. The peak reinforcement strains at the locations mentioned are consistent with conventional notions of the onset of a contiguous zone of plasticity (internal failure surface) propagating from the heel of the wall facing. The same computed local peak at end of construction is detectable at the 80 kPa surcharge load level but is exceeded by the locally higher
strains developed close to the back of the facing column due to down-drag forces acting at the connections. However, the locally higher strains at distances away from the facings were not detected in the strain measurement data at the same locations nor was there any indication of a shear band during careful excavation of the wall. The Mohr-Coulomb results are believed to be due to the simple model employed and the particular choice of elastic modulus to represent the non-linear stress-strain response of the RMC sand. The delayed onset of internal failure in these walls may also be due to the fact that the surcharge load was applied uniformly over the backfill surface and the facing column was constructed with a stiff column of masonry blocks. This configuration encourages the development of the highest loads (or strains) at the connections rather than at locations deeper within the reinforced soil mass that may be expected based on conventional wedge-type failure mechanisms for flexible-face walls (e.g. geosynthetic wrapped-face walls) (AASHTO 2002).

2.5 Discussion and conclusions

Three different constitutive soil models were used in numerical models of reinforced soil walls: elastic-plastic Mohr-Coulomb, a modified Duncan-Chang model, and Lade’s model. The model formulations predict different soil response at any stress level under plane strain conditions. The results of simulations using these constitutive models showed that there were detectable differences in numerical results but predictions were typically within the range of measured values for the loading conditions investigated – i.e. end of construction and maximum post-construction surcharge pressures equal to the soil unit weight multiplied by about 1.3 times the height of the walls. This range of loading is considered to be consistent with working stress (operational) conditions characterized by small deformation and strain levels and hence of practical interest to design engineers.

As a reasonable estimate, maximum polymeric reinforcement strains at interior locations in the reinforced soil zone should be no greater than about 3% to ensure that plasticity (shear banding) is not well-developed. From the numerical and physical data presented in this chapter, the maximum value of reinforcement strains at the end of construction for Wall 1 (extensible polymeric reinforcement) was less than 1% which is well within the 3% threshold strain level. Bathurst et al. (2007) have shown that maximum strains in monitored full-scale reinforced soil walls reported in the literature that have performed well are typically 1% or less at end of construction. The reinforcement strains for the WWM wall in this chapter were well within the elastic range of the reinforcement (0.2%) at end of construction and only reached yield during
surcharging to the 80 kPa level. The predominantly small (elastic) strain levels explain the generally consistent results for the WWM wall using the three soil constitutive models investigated.

It could be argued that for hard-faced walls on rigid foundations a simple elastic-plastic Mohr-Coulomb soil model is sufficiently accurate if the objective is to simulate these walls under operational conditions. However, accurate predictions depend on a fortuitous choice of elastic modulus for frictional soils. The Mohr-Coulomb elastic-plastic soil model is best suited for the analysis of reinforced soil walls that are at incipient collapse (failure) than for the working stress conditions that are the focus of the current investigation.

The Lade model is attractive because it can predict a wide range of soil response. However, it requires nine plasticity-related parameters, most of which lack obvious physical meaning. From a practical point of view some parameters could not be measured with conventional laboratory testing equipment at the low confining pressures typical for the walls in this investigation and likely for walls of greater height.

The results of this study are consistent with Ling (2003) and Hatami and Bathurst (2005, 2006) who concluded that complicated constitutive models for the component materials are not warranted if the accuracy of numerical predictions using simpler models is within the range of physical data measurements. The modified Duncan-Chang model which uses parameters from conventional triaxial compression testing to simulate plane strain conditions is a good compromise between prediction accuracy and availability of parameters from conventional laboratory testing.

This model can be used in commercial FEM and FDM computer packages that permit user-defined soil models to investigate a wider range of wall geometry, loading condition, soil type and reinforcement properties for walls seated on competent foundations than is possible using full-scale physical tests. The modified Duncan-Chang model described here can be used in the modelling of other plane strain earthwork problems where frictional fills and low confining pressures are involved.

However, regardless of the soil model adopted, accurate simulation results also require proper modelling of all interfaces and the time-dependent non-linear load-strain behaviour of the polymeric reinforcement. The prediction accuracy of numerical codes with different soil
constitutive models for reinforced soil walls constructed over less competent foundation support cannot be appraised until more high quality physical data are available.

REFERENCES


### Table 2.1 Reinforcement properties.

<table>
<thead>
<tr>
<th>Wall</th>
<th>Reinforcement type</th>
<th>( J_o(t) ) (kN/m)</th>
<th>( \eta(t) )</th>
<th>( T_f(t) ) (kN/m)</th>
<th>( T_y(1) ) (kN/m)</th>
<th>Ultimate (index) strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>PP</td>
<td>115</td>
<td>0.85</td>
<td>7.7</td>
<td>14</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>WWM</td>
<td>3100</td>
<td>0</td>
<td>7</td>
<td>7</td>
<td></td>
</tr>
</tbody>
</table>

Notes: PP = polypropylene biaxial geogrid, WWM = welded wire mesh. \(^{(1)}\) Based on peak strength measured during 10% strain/minute constant-rate-of-strain (CRS) test.
Table 2.2 Backfill soil properties.

<table>
<thead>
<tr>
<th>Model and properties</th>
<th>Wall 1</th>
<th>Wall 6</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Duncan-Chang model</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$K_e$ (elastic modulus number)</td>
<td>800</td>
<td>1968</td>
</tr>
<tr>
<td>$K_{ur}$ (unloading-reloading modulus number)</td>
<td>960</td>
<td>2362</td>
</tr>
<tr>
<td>$n$ (elastic modulus exponent)</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>$R_f$ (failure ratio)</td>
<td>0.86</td>
<td></td>
</tr>
<tr>
<td>$\nu_t$ (tangent Poisson's ratio)</td>
<td>0-0.49</td>
<td></td>
</tr>
<tr>
<td>$\phi$ (friction angle) (degrees)</td>
<td>44</td>
<td></td>
</tr>
<tr>
<td>$c$ (cohesion) (kPa)</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>$B/p_a$ (initial bulk modulus number)</td>
<td>110</td>
<td>270</td>
</tr>
<tr>
<td>$\varepsilon_u$ (asymptotic volumetric strain value)</td>
<td>0.012</td>
<td></td>
</tr>
<tr>
<td>$p_a$ (atmospheric pressure) (kPa)</td>
<td>101</td>
<td></td>
</tr>
<tr>
<td><strong>Mohr-Coulomb model</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$E$ (Young's modulus) (MPa)</td>
<td>40</td>
<td>80</td>
</tr>
<tr>
<td>$\nu$ (Poisson's ratio)</td>
<td>0.3</td>
<td></td>
</tr>
<tr>
<td>$\phi$ (friction angle) (degrees)</td>
<td>44</td>
<td></td>
</tr>
<tr>
<td>$\psi$ (dilation angle) (degrees)</td>
<td>11</td>
<td></td>
</tr>
<tr>
<td>$c$ (cohesion) (kPa)</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td><strong>Lade's model</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$M, \lambda, \nu$ (elastic properties)</td>
<td>550, 0.25, 0.3</td>
<td>1360, 0.25, 0.3</td>
</tr>
<tr>
<td>$m, \eta_1, a$ (failure criterion)</td>
<td>0.107, 36.0, 0.2/p_a</td>
<td></td>
</tr>
<tr>
<td>$\Psi_2, \mu$ (plastic potential)</td>
<td>-3.65, 2.425</td>
<td></td>
</tr>
<tr>
<td>$h, \alpha$ (yield criterion)</td>
<td>0.432, 0.34</td>
<td></td>
</tr>
<tr>
<td>$C, p$ (hardening/softening law)</td>
<td>$0.145 \times 10^{-3}, 1.22$</td>
<td>$0.073 \times 10^{-3}, 1.22$</td>
</tr>
<tr>
<td>$\gamma$ (unit weight) (kN/m$^3$)</td>
<td>16.8</td>
<td></td>
</tr>
</tbody>
</table>
Table 2.3 Interface properties from Hatami and Bathurst (2005, 2006).

<table>
<thead>
<tr>
<th>Interface</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Soil-Block</strong></td>
<td></td>
</tr>
<tr>
<td>(\delta_b) (friction angle) (degrees)</td>
<td>44</td>
</tr>
<tr>
<td>(\psi_{lb}) (dilation angle) (degrees)</td>
<td>11</td>
</tr>
<tr>
<td>(K_{nsb}) (normal stiffness) (MN/m/m)</td>
<td>100</td>
</tr>
<tr>
<td>(K_{ssb}) (shear stiffness) (MN/m/m)</td>
<td>1</td>
</tr>
<tr>
<td><strong>Block-Block</strong></td>
<td></td>
</tr>
<tr>
<td>(\delta_{bb}) (friction angle) (degrees)</td>
<td>57</td>
</tr>
<tr>
<td>(c_{bb}) (cohesion) (kPa)</td>
<td>46</td>
</tr>
<tr>
<td>(K_{nbb}) (normal stiffness) (MN/m/m)</td>
<td>1000</td>
</tr>
<tr>
<td>(K_{sbb}) (shear stiffness) (MN/m/m)</td>
<td>40</td>
</tr>
<tr>
<td><strong>Backfill-Reinforcement</strong></td>
<td></td>
</tr>
<tr>
<td>(\phi_b) (friction angle) (degrees)</td>
<td>44</td>
</tr>
<tr>
<td>(s_b) (adhesive strength) (kPa)</td>
<td>1000</td>
</tr>
<tr>
<td>(K_b) (shear stiffness) (kN/m/m)</td>
<td>1000</td>
</tr>
</tbody>
</table>
Figure 2.1 FLAC numerical grid.
Figure 2.2 Measured and predicted stress-strain response of RMC sand from (a, b) triaxial tests and (c) plane strain tests.

Note: No data was taken for volumetric strain response in plane strain tests.
Figure 2.2 (continued) Measured and predicted stress-strain response of RMC sand from (a, b) triaxial tests and (c) plane strain tests.

Note: No data was taken for volumetric strain response in plane strain tests.
Figure 2.3 Measured and calculated vertical and horizontal toe loads for (a) Wall 1 (PP reinforcement) and (b) Wall 6 (WWM reinforcement).

(a) Wall 1 (PP reinforcement)
Figure 2.3 (continued) Measured and calculated vertical and horizontal toe loads for (a) Wall 1 (PP reinforcement) and (b) Wall 6 (WWM reinforcement).

(b) Wall 6 (WWM reinforcement)
Figure 2.4 Normalized measured and calculated vertical foundation pressures for Wall 1 (PP reinforcement) and Wall 6 (WWM reinforcement) at end of construction and surcharge $q = 50$ kPa.
Figure 2.5 Measured and calculated relative facing displacements located at reinforcement elevations for (a) Wall 1 (PP reinforcement) and (b) Wall 6 (WWM reinforcement) at the end of construction.
Figure 2.6 Measured and calculated post-construction facing displacement profiles for (a) Wall 1 (PP reinforcement) and (b) Wall 6 (WWM reinforcement).

Notes: Datum taken at end of construction; range bars represent maximum and minimum displacements recorded across middle 1-m width of wall facing at 80 kPa surcharge load level.

(a) Wall 1 (PP reinforcement)
Figure 2.6 (continued) Measured and calculated post-construction facing displacement profiles for (a) Wall 1 (PP reinforcement) and (b) Wall 6 (WWM reinforcement).

Notes: Datum taken at end of construction; range bars represent maximum and minimum displacements recorded across middle 1-m width of wall facing at 80 kPa surcharge load level.

(b) Wall 6 (WWM reinforcement)
Figure 2.7 Measured and calculated connection loads at (a) end of construction and (b) surcharge $q = 80$ kPa.

Note: horizontal range bars show the range of connection loads measured across 1 m of wall face at each reinforcement elevation.
Figure 2.7 (continued) Measured and calculated connection loads at (a) end of construction and (b) surcharge $q = 80$ kPa.

Note: horizontal range bars show the range of connection loads measured across 1 m of wall face at each reinforcement elevation.
Figure 2.8 Measured and calculated reinforcement strain distributions for (a, c) Wall 1 (PP reinforcement) and (b, d) Wall 6 (WWM reinforcement) at end of construction and surcharge $q = 80$ kPa.

(a) Wall 1 (PP reinforcement) at end of construction
Figure 2.8 (continued) Measured and calculated reinforcement strain distributions for (a, c) Wall 1 (PP reinforcement) and (b, d) Wall 6 (WWM reinforcement) at end of construction and surcharge $q = 80$ kPa.

(b) Wall 6 (WWM reinforcement) at end of construction
Figure 2.8 (continued) Measured and calculated reinforcement strain distributions for (a, c) Wall 1 (PP reinforcement) and (b, d) Wall 6 (WWM reinforcement) at end of construction and surcharge $q = 80$ kPa.

(c) Wall 1 (PP reinforcement) at surcharge $q = 80$ kPa
Figure 2.8 (continued) Measured and calculated reinforcement strain distributions for (a, c) Wall 1 (PP reinforcement) and (b, d) Wall 6 (WWM reinforcement) at end of construction and surcharge $q = 80$ kPa.

(d) Wall 6 (WWM reinforcement) at surcharge $q = 80$ kPa
Chapter 3
Influence of Toe Restraint on Reinforced Soil Segmental Walls

3.1 Introduction

Current design for the internal stability of geosynthetic reinforced soil walls is based on the “tie back wedge method” or variants (AASHTO 2002; BS8006 1995; CFEM 2006; NCMA 2009). These approaches assign load to each reinforcement layer based on the total load developed by a classical active failure wedge located directly behind the wall facing within the reinforced soil zone. The total active load (or pressure) is partitioned to each layer based on the tributary area of each reinforcement layer. For non-surcharged or uniformly surcharged walls with reinforcement layers placed at uniform spacing, the tensile load in each reinforcement layer increases linearly with depth below the top of the wall.

Back-analysis of more than 30 full-scale instrumented field and laboratory geosynthetic reinforced soil walls has demonstrated that load predictions at end of construction (working stress condition) using the AASHTO (2002) approach (called the Simplified Method) are very conservative for design (i.e. excessively safe). Specifically, predicted loads are (on average) three times greater than “measured” loads (Allen et al. 2003; Miyata and Bathurst 2007a,b; Bathurst et al. 2008b). Typically the measured loads have been computed from best estimates of geosynthetic stiffness and strains recorded by strain gauges or extensometer points affixed to the reinforcement layers.

The reasons for this discrepancy are the following:

a) The underlying limit equilibrium-based deterministic model is not appropriate for walls under operational (working stress) conditions (i.e. an ultimate limit state is not present in walls with good performance at end of construction);

b) Soil shear strength is typically underestimated; and

c) Contribution of the structural facing to carry earth loads in combination with a restrained toe is ignored.

A series of 11 instrumented full-scale reinforced soil walls have been constructed at the Royal Military College of Canada (Bathurst et al. 2000, 2006; Hatami and Bathurst 2005, 2006), some details of which appear later in this chapter. Bathurst et al. (2006) quantified the
contribution of a structural facing to carry earth loads by comparing the performance of two of these walls that were nominally the same with the exception of the facing. The sand backfill walls were 3.6 m high and reinforced with a polypropylene geogrid. One wall was constructed with a dry-stacked column of modular blocks and the companion wall was constructed with a very flexible wrapped-face that provided little or no horizontal load carrying capacity. They showed that the Simplified Method was able to make reasonably accurate estimates of the maximum load in the critical reinforcement layer of the wrap-faced wall if the peak plane strain friction angle was used in computations. However, the same method over-predicted the maximum reinforcement load by about a factor of three for the hard-faced wall which is consistent with back-analyses of other full-scale walls in the database mentioned above. The paper by Bathurst et al. (2006) prompted a discussion (Leshchinsky 2007) and response (Bathurst et al. 2007) regarding the influence of the restrained toe boundary condition on the performance of the hard-faced wall. Leshchinsky (2007) opined that the toe in the RMC wall was an unusually severe constraint that resulted in under-mobilization of reinforcement load capacity. Furthermore, the rigid foundation and low height of the RMC walls could be expected to further amplify the contribution of the toe to load capacity beyond what may be expected in actual field walls of greater height and with less rigid foundation support. Bathurst et al. (2007) responded that the toe constraint used in the RMC walls with a structural facing is reasonable since most field walls are embedded at the toe, mechanically attached to a footing and (or) develop significant frictional resistance at the base of the wall facing. However, they acknowledged that the relative contribution of the toe to load capacity can be expected to decrease as wall height increases and all other parameters remain the same. Some evidence in support of this hypothesis was presented using preliminary numerical modelling work by the writer (Huang et al. 2007). However, the results of a systematic quantitative investigation of this important issue were not available at that time.

This chapter first presents measurements taken from full-scale reinforced soil walls that directly or indirectly show the influence of the toe boundary condition on wall performance. However, the bulk of this chapter reports results from a numerical investigation focused largely on the influence of magnitude of toe restraint stiffness on the performance of segmental (modular block) geosynthetic reinforced soil walls. The numerical simulations were carried out using a previously verified numerical code for the RMC test walls (Hatami and Bathurst 2005, 2006; Bathurst et al. 2009a; Huang et al. 2009). Verification was carried out using model material properties deduced from independent laboratory testing of the wall components and quantitative comparison
of wall performance predictions with measurements from several of the walls in the RMC test program. The current study includes equivalent toe stiffness values back-calculated from direct shear tests on concrete blocks placed over crushed stone and concrete levelling pads. This numerical study allows the effect of toe restraint on wall behaviour to be investigated more fully than is possible using the results from full-scale tests alone. The other parameters investigated are wall height, interface shear stiffness between blocks, wall facing batter, reinforcement stiffness and reinforcement spacing.

3.2 Results from full-scale wall testing

A cross-section view of the control structure (Wall 1) in the series of 11 structures built in the RMC retaining wall test facility is shown in Figure 3.1. The solid segmental blocks used in these tests were 0.3 m wide (toe to heel). The base of the wall facing column was placed on steel plates and rollers, and supported by load cells. This arrangement allowed the measured horizontal and vertical toe loads to be decoupled. The horizontal toe was restrained laterally by a series of load rings and a steel reaction frame bolted to the laboratory structural floor. The horizontal stiffness of the reaction system was 4 MN/m/m computed from measured loads and horizontal displacements (Hatami and Bathurst 2005). Load rings were used to record the (connection) load transferred from each layer of reinforcement to the structural facing column. Details of the instrumentation deployed in this wall can be found in earlier related papers (e.g. Bathurst et al. 2000, 2006) and are not repeated here.

The wall was constructed from the bottom-up at a target batter of $\omega = 8$ degrees from the vertical. After each row of facing blocks was seated, a 150-mm thick lift of sand was placed and compacted. Following construction, uniform surcharge loading was applied in stages across the entire backfill surface using a system of airbags. Surcharge loading was continued until large facing deformations were recorded and an internal soil failure mechanism was detected in the reinforced soil zone. Following surcharge unloading, the horizontal structural support at the toe was removed.

The evolution of horizontal toe load and sum of connection loads during construction and during staged uniform surcharge loading is shown in Figure 3.2. The data show that horizontal toe load and connection loads were mobilized progressively with height of wall during construction and during subsequent surcharging. There were a few initial small local deviations from the general trends in the plots that were likely the result of construction activities. At the end of construction approximately 80% of the total horizontal load acting against the facing column was carried by
the restrained toe. At the end of surcharging about 60% of total load was carried by the reinforcement and remainder by the toe. The initial greater load carried by the toe is attributed to larger toe stiffness compared to the reinforcement layers. However, as the wall moved outward the reinforcement layers strained axially thus mobilizing tensile load capacity and reducing the amount of load that must be carried by the wall toe to maintain horizontal equilibrium. The distribution of load between the reinforcement layers and the toe can be expected to be influenced by the interface stiffness between facing column blocks. The influence of block interface shear stiffness is investigated numerically later in this chapter.

Post-construction wall profiles are shown in Figure 3.3 up to the maximum surcharge load applied. At the end of the test, the horizontal toe support was removed. The resulting outward toe displacement was about 20 mm. Figure 3.4 shows connection loads and horizontal toe load at end of surcharge unloading and at subsequent toe release. The plots show that a portion of the load taken by the toe was redistributed to the lowest layer, causing the load in this reinforcement layer to increase. Interestingly, the total load on the back of the facing column computed as the sum of the connection loads plus the toe load decreased. This is attributed to the further mobilization of soil shear strength consistent with the notion of a stress state in the soil that is closer to an active earth pressure condition.

Clearly, a significant amount of horizontal load was carried by the toe during this test. This was true for all RMC walls constructed with a modular block facing in this test series, although there were some quantitative differences depending on wall batter angle, reinforcement type (stiffness), number of reinforcement layers and type of compaction equipment used to prepare the sand backfill. However, it should be noted that toe release occurred after the maximum surcharge load had been applied to the structure and an internal soil failure mechanism was fully developed. Internal soil failure was detected based on local strain peaks measured in the reinforcement layers and large increases in wall deformation (Bathurst et al. 2009c). Had toe release been carried out at the end of construction, while the wall was under working stress conditions, it is reasonable to expect that the magnitude of toe movement would have been less than the measured value after the wall had been heavily surcharged.

Evidence of the combined effect of toe fixity and facing type (stiffness) on load distribution to the wall toe (or foundation) and reinforcement layers can be seen in Figure 3.5. All the walls in this figure were vertical-faced and were constructed with geosynthetic reinforcement and sand backfill. Furthermore, the wall had uniform reinforcement spacing but only selected layers were
instrumented. The strains ($\varepsilon_{\text{max}}$) are the maximum strains recorded in the instrumented layers at end of construction. These values do not include strain measurements that may be influenced by proximity to the facing connections. Locally high strains may be anticipated at these locations due to soil settlement relative to the structural facing. The reader is directed to the papers by Allen et al. (2003), Bathurst et al. (2008b) and Miyata and Bathurst (2007a) for detailed descriptions of the case studies identified in Figure 3.5.

Figure 3.5a presents data for instrumented field walls. Walls GW5, GW8 and GW10 show that the bottommost monitored layer (where the layer is within 10% of the height of the wall from the base) has the locally lowest strain value. Wall GW26C shows a reduction in the rate of strain accumulation with depth at the wall base. However, the strain in the lowermost layer may be less due to the higher stiffness of the reinforcement at this location compared to layers located higher in the wall. For walls GW5, GW8 and GW10 the maximum strains in the reinforcement increase with decreasing facing stiffness (i.e. in the order of GW5, GW8 and GW10). There is an exception to this trend for Wall GW26C but this may be because this wall was designed using the original K-Stiffness Method (Allen and Bathurst 2006; Allen et al. 2003) and hence higher strains were anticipated compared to values using the more conservative Simplified Method (AASHTO 2002). Data for carefully instrumented field walls are limited compared to full-scale laboratory walls, and deviation from the trends noted here may be expected for walls with other foundation conditions, facing type and quality of construction.

Figure 3.5b shows similar strain data for a series of walls built at the Public Works Research Institute (PWRI) in Japan. In these tests the bottom of the wall was horizontally restrained with the exception of GW21. For these walls there is visually apparent local reduction in strain for layers located within the bottom 20% of the wall height for those structures with a restrained toe. There is a visually apparent reduction in rate of strain accumulation with depth below the wall crest for the wall with the least restrained toe condition (GW21). For the walls with the same toe fixity but varying facing type, there is a trend towards decreasing strains with increasing wall structural stiffness (wall structural stiffness increasing in the order of GW22 to GW24).

Taken together, the data in Figures 3.3, 3.4 and 3.5 show that strain (or load) in reinforcement layers is often attenuated with increasing proximity to the wall toe (foundation) and that strain (or load) is further attenuated with increasing stiffness of the wall facing. An important implication to wall performance is that load that is not being resisted by the bottom layer(s) must be carried by the foundation (including the toe) to satisfy horizontal equilibrium. Horizontal toe load capacity is
considered by the writer to be typical for a wall with a structural facing due to friction between
the base of the facing column and a concrete or gravel levelling pad, embedment and (or)
structural attachment to a concrete footing. However, there is no data from physical full-scale
testing that can be used to quantitatively isolate the contribution of horizontal toe stiffness to wall
capacity. The only practical methodology is numerical modelling using computer codes
previously verified against carefully instrumented and monitored physical tests. This is the
strategy adopted in this chapter.

3.3 Numerical model

3.3.1 General approach
Numerical 2-D finite difference (FLAC) codes (Itasca 2005) have been developed to predict
performance features of several RMC test walls including the control wall described earlier
(Hatami and Bathurst 2005, 2006; Bathurst et al. 2009a; Huang et al. 2009). The models
have used strain- and time-dependent constitutive models to simulate the behaviour of the
polymeric geogrid reinforcement materials used in the RMC walls and three different constitutive
models for the sand backfill. In order of increasing complexity the soil models were: a) linear
elastic-plastic Mohr-Coulomb; b) modified Duncan-Chang hyperbolic model, and; c) Lade’s
single hardening model. Calculated results were compared against measured toe footing loads,
foundation pressures, facing displacements, connection loads and reinforcement strains. In
general, all three approaches gave similar results for wall performance under operational
(working stress) conditions prior to development of contiguous shear failure zones in the
reinforced soil zone. Furthermore, the predictions were typically within measurement accuracy
for the end-of-construction and surcharge load levels below pressures needed to generate soil
failure. However, numerical results using the linear elastic-plastic Mohr-Coulomb model were
sensitive to the choice of a single-valued elastic modulus which is problematic when this value is
deducted from conventional triaxial compression tests. The results of these investigations show
that numerical models that incorporate the hyperbolic soil model are adequate to predict the
performance of reinforced soil walls under typical operational conditions provided that the
reinforcement, interfaces, construction sequence, soil and soil compaction are modelled correctly.
The writers concluded that further improvement of predictions using more sophisticated soil
models is not guaranteed. A similar conclusion has been reached by Ling (2003) who compared
results of numerical simulations using a hyperbolic soil model with measured results from an
instrumented full-scale reinforced soil wall constructed in Japan.
In the current investigation a version of the numerical code described by Bathurst et al. (2009a) and Huang et al. (2009) was used. The construction process was modelled by sequential bottom-up placement of the blocks and matching soil layer placement and compaction. The thickness of each block-soil layer was taken as an individual block height (0.15 m). Compaction effects were simulated as a transient uniform surcharge pressure of 8 kPa applied to the top surface of each soil layer following the procedure described by Hatami and Bathurst (2005). Computations were carried out in large-strain mode to ensure sufficient accuracy in the event of large wall deformations or reinforcement strains. The numerical mesh was updated to simulate the moving local datum as each row of facing units and soil layer was placed during construction. The same reinforcement length to wall height ratio of 0.7 matching the original RMC test walls was used in the numerical models. This value is also recommended by AASHTO (2002). The ratio of wall height (H) to length of soil mass (B) was kept the same at 0.65 in the wall models of different height. Figure 3.6 shows a typical numerical FLAC grid.

3.3.2 Material Properties

Soil

The compacted backfill sand was assumed as an isotropic, homogeneous, nonlinear elastic material using the Duncan-Chang hyperbolic model. The elastic tangent modulus is expressed as:

\[
E_t = \left[1 - \frac{R_f (1 - \sin \phi)(\sigma_1 - \sigma_3)}{2c \cos \phi + 2\sigma_3 \sin \phi}\right]^2 K_c p_a \left(\frac{\sigma_3}{p_a}\right)^n
\]

where: \(\sigma_1\) = major principle stress, \(\sigma_3\) = minor principle stress, \(p_a\) = atmospheric pressure and other parameters are defined in Table 3.1. The original Duncan-Chang model was developed for axi-symmetric (triaxial) loading conditions and therefore the bulk modulus is a function only of \(\sigma_3\); specifically:

\[
B = K_b p_a \left(\frac{\sigma_3}{p_a}\right)^m
\]
However, under plane strain conditions, $\sigma_2 > \sigma_3$ and the confining pressure is hence underestimated. Hatami and Bathurst (2005) showed that the Duncan-Chang parameters back-fitted from triaxial tests on the RMC sand under-estimated the stiffness of the same soil when tested in a plane strain test apparatus. In the current study and in previous related modelling (Bathurst et al. 2009a; Huang et al. 2009), the bulk modulus formulation proposed by Boscardin et al. (1990) was used to replace the original formulation. The bulk modulus is expressed as:

$$[3.3] \quad B_i = B_i \left[ 1 + \frac{\sigma_m}{B_i \varepsilon_u} \right]^2$$

where: $\sigma_m =$ mean pressure $= \frac{\sigma_1 + \sigma_2 + \sigma_3}{3}$; $B_i$ and $\varepsilon_u$ are material properties that are determined as the intercept and the inverse of slope from a plot of $\sigma_m/\varepsilon_{vol}$ versus $\sigma_m$ in an isotropic compression test. Table 3.1 summarizes the properties of the backfill soil. The Duncan-Chang parameters were taken from Boscardin et al. (1990) with some adjustments to represent a high quality sand material prepared to 95% standard Proctor density and designated SW using the Unified Soil Classification System.

Reinforcement

Soil reinforcement layers were simulated using FLAC cable elements. Two reinforcement materials were used to represent polymeric geogrid and steel strip reinforcement products having very different stiffness properties (values). To simplify parametric analyses and to focus the results on the influence of toe stiffness, both materials were assigned linear load-strain properties. The stiffness of polyester (PET) geogrid materials has been shown to be sensibly time- and strain-independent over the range of strains anticipated at end-of-construction in typical reinforced soil walls (Walters et al. 2002; Bathurst et al. 2009c). Reinforcement stiffness values are given in Table 3.1. The stiffness value for the PET material is typical for geosynthetic reinforcement materials. The steel material is approximately 100 times stiffer than the polymeric material but only about 50% of the stiffness of typical steel strip reinforcement (Bathurst et al. 2009b). Nevertheless, the focus of this chapter is on geosynthetic reinforced soil walls. A very stiff reinforcement material (steel strip) was used in numerical simulations to generate large detectable qualitative and quantitative differences between walls that can be ascribed to the influence of reinforcement stiffness. The index global reinforcement stiffness of the walls with the PET
reinforcement material fall within the range of values determined from instrumented field and full-scale laboratory walls (Allen et al. 2003, 2004). The index global stiffness value is calculated as:

\[
S_{\text{global}} = \frac{\sum_{i=1}^{n} J_i}{H} = \frac{J_{\text{ave}}}{H/n}
\]

where: \( J_i \) is the tensile stiffness of an individual polymeric reinforcement layer at 2% strain as measured in a wide-width strip tensile test (e.g. ASTM D4595 1996), and at the elastic limit for steel strip reinforcement, and; \( J_{\text{ave}} \) is the average tensile stiffness of all \( n \) reinforcement layers in a wall with height \( H \). Using Equation 3.4, the global reinforcement stiffness value for walls with spacing \( S_v = 0.6 \) m is 475 kN/m² and 50 MN/m² for the PET and steel reinforced walls, respectively.

**Interfaces and Boundary Conditions**

The interfaces between facing-backfill, block-block, foundation-backfill, and reinforcement-backfill were modelled as linear spring-slider systems with interface shear strength defined by the Mohr-Coulomb failure criterion. Details can be found in the FLAC manual (Itasca 2005).

In this study, the facing column stiffness was assumed to be controlled by the interface shear stiffness between modular blocks. The value for the solid masonry blocks used in the original RMC test walls was back-calculated to be 40 MN/m/m from direct shear tests (Hatami and Bathurst 2005).

All RMC test walls were built on a rigid concrete floor (Figure 3.1). In the numerical models, a fixed boundary condition in both horizontal and vertical directions was assumed at the foundation level representing a similar rigid foundation condition. A fixed boundary condition in the horizontal direction was assumed at the backfill far-end boundary. The toe of the facing column was restrained horizontally by a spring element whose stiffness was varied in the current study to represent different toe restraint conditions. The control stiffness value was taken as 4 MN/m/m matching measurements recorded at this boundary in the RMC physical tests. The bottom of the facing column was fixed in the vertical direction but free to translate in the horizontal direction and rotate. The interface between the bottom block and the foundation was unrestrained (as in the
RMC physical tests) so that the axial force recorded in the spring element could be interpreted as the toe load. In some simulations a perfectly rigid horizontal toe condition was applied by fixing the toe of the facing column in the horizontal direction. In this case the horizontal toe force was computed from the horizontal reaction force at the corresponding grid point.

Direct interface shear testing was carried out using a large dry-cast masonry concrete block seated on three different base materials. The bottom of the block was flat. The bases were a poured-in-place unreinforced wet-cast concrete mat and two different crushed stone materials. Crushed stone 1 was well-graded with medium particle size $D_{50} = 12$ mm and crushed stone 2 was uniformly graded with $D_{50} = 22$ mm. The tests were carried out under a range of normal loads corresponding to wall heights up to about 12 m. The shear load per unit length ($V$) to generate 2 mm and 6 mm of shear displacement ($\Delta$) was recorded and the interface shear stiffness computed as $V/\Delta$. The results of these tests are plotted in Figure 3.7. The value of 2 mm corresponds to the displacement estimated for the RMC walls at end of construction. The value of 6 mm is a typical maximum displacement value predicted for the PET wall simulations presented later using $H = 6$ m and a toe stiffness value 4 MN/m/m. It is also a maximum value from the results of steel wall simulations of the same height. The shear test results show that the concrete mat provided the greatest shear stiffness using the same displacement criterion. For the same configuration, the computed stiffness values decreased with increasing displacement. The data show that the toe stiffness computed for the RMC walls falls within the range of values plotted in the figure and hence a value of 4 MN/m/m is considered reasonable for these structures. It is possible that greater interface shear stiffness values would result with a block with a rougher base or a shear key that projects below the base of the block into a granular base material. However, later in this chapter it is demonstrated that equivalent toe stiffness values orders of magnitude lower than those shown in this figure are required to generate qualitatively different wall response from walls with a reference toe stiffness of 4 MN/m/m.

The reinforcement was bonded to the backfill using the FLAC built-in grout algorithm. A large bond strength was assigned to the reinforcement-backfill interface to prevent slip. A perfect bond assumption has been adopted in other numerical studies to simplify modelling and the interpretation of results (Rowe and Ho 1997; Leshchinsky and Vulova 2001; Hatami and Bathurst 2005, 2006). Experience with the high quality sand used in RMC tests and measured reinforcement displacements suggest that this is a reasonable assumption for soil reinforcement layers under working stress conditions. In order to keep the interpretation of results as simple as possible, the same no-slip interface was assumed in all the simulations in this numerical study.
The influence of reinforcement-sand interface stiffness and strength was not investigated in this study. Leshchinsky and Vulova (2001) carried out similar numerical modelling and reported that reducing the interface friction angle did not significantly change simulation results. Interface properties used in the current study are summarized in Table 3.2.

3.4 Variables in parametric study

A total of 42 wall cases were simulated in this study although not all results are presented. The model parameters that were investigated are summarized in Table 3.3. The simulation matrix was carried out with the objective to investigate the influence of parameter values in the first four rows of Table 3.3 in combination with the range of toe horizontal stiffness values shown in the bottom row. It should be recalled that the focus of this chapter is on wall behaviour under operational conditions (i.e. working stress conditions). Allen et al. (2003) reviewed a large number of monitored geosynthetic reinforced soil walls with granular backfills and concluded that contiguous failure zones from heel of the wall facing to the backfill surface and other obvious signs of poor wall performance did not occur if polymeric reinforcement strain levels were kept to less than about 3%. Using this value as an indicator of working stress conditions, it was found that all numerical results for PET wall models satisfied this constraint. The yield strain limit of steel reinforcement was assumed to be equal to 0.2% (Hatami and Bathurst 2006). Numerical results from the current study confirmed that strains in the steel reinforcement layers were less than this limit.

3.5 Numerical results

3.5.1 Facing displacements

Normalized relative facing displacement profiles for a 6-m high wall with PET and steel reinforcement layers at spacing $S_v = 0.6$ m are plotted in Figure 3.8. The target facing batter was $\omega = 8$ degrees from the vertical. The displacements were computed with respect to the time that the layer was placed. Hence the displacement profiles represent a moving datum and should not be confused with wall profiles at the end of construction. The normalized facing displacements in Figures 3.8a ii and 3.8b ii are taken with respect to the toe stiffness value of the RMC control wall (4 MN/m/m).

Facing displacements at each elevation decrease non-linearly with increasing toe stiffness (Figure 3.8a i and 3.8b i). The difference in deformations for walls with a fixed toe and a toe with stiffness of 40 MN/m/m is negligible. Not unexpectedly, displacements are larger for the
walls with lower reinforcement stiffness. The ratio of maximum wall displacements ranges from about two to three between the PET and steel reinforcement cases. The influence of relative toe stiffness value with respect to the control case can be seen to diminish with height above the toe and with increasing reinforcement stiffness. For toe stiffness values $\geq 4$ MN/m/m the influence of magnitude of toe stiffness on wall deformations can be argued to be limited to about 0.25H above the base of the wall (Figures 3.8a ii and 3.8b ii). The data in Figure 3.8 plots demonstrate that it is the combination of toe stiffness and global reinforcement stiffness that influence end-of-construction wall deformations.

Figure 3.9a shows the influence of toe release on wall displacements for the same wall in Figure 3.8. The toe of the wall can be seen to move almost 20 mm and additional outward movements are largely restricted to the bottom half of the wall. These observations are similar to those for Wall 1 in the RMC physical test program (Figure 3.3) even though the RMC wall was shorter, less stiff reinforcement material was used and the wall was surcharge loaded well beyond working stress levels. The redistribution of load from the toe to the two bottom-most reinforcement layers computed at the connections can be seen in Figure 3.9b. Qualitative features are similar to those shown in Figure 3.4. For example, the total load on the back of wall becomes less after toe release. However, individual connection loads do not become less than the end-of-construction connection loads as shown in Figure 3.4. This is because the numerical wall was not heavily surcharged at end of construction and possibly because reinforcement stress relaxation was not simulated in the numerical model.

### 3.5.2 Toe and connection loads

The contribution of the toe to wall capacity can be quantified by considering the wall facing column as a free body. In this simple approach the sum of connection loads and toe loads must be equal to the horizontal component of total earth force acting against the back of the facing to satisfy horizontal equilibrium. Figure 3.10 shows plots of toe load, total load and relative contribution of toe and connection loads to resist the total horizontal load acting against the wall facing. The load carried by the toe increases roughly log-linearly with increasing toe stiffness over much of the range of stiffness values investigated (Figures 3.10a i and 3.10b i). Furthermore, there is a log-linear trend when the fraction of total horizontal earth load carried by the toe and connections is plotted against toe stiffness (Figures 3.10a ii and 3.10b ii). However, the magnitude of loads and fraction of total load carried by the toe and connections is sensitive to the stiffness of the reinforcement. As reinforcement stiffness increases the contribution of the toe decreases. This can be understood if the free-body analogue for the wall facing is further
developed by considering the wall facing as a continuously supported beam with the toe and connections acting as spring reactions and the earth pressure as the distributed load (Bathurst et al. 2007). This analogue then leads to the expectation that as the toe stiffness increases more load is carried by the stiffest spring (toe). For the reinforcement layers to carry the total earth load acting against the back of a wall with a structural facing, the toe must be unrestrained which is, in the opinion of the writer, an unlikely boundary condition. Finally, the data show that the reference toe stiffness value (4 MN/m/m) used earlier gives results that fall roughly in the middle of the range of dependent values on the vertical axes in these plots. This gives some support to the argument that the RMC walls have a horizontal toe stiffness compliance which falls in the middle of walls with idealized unrestrained and fixed horizontal toe conditions.

### 3.5.3 Reinforcement strains

The magnitude and distribution of reinforcement strains at end of construction for the 6-m high wall used in the parametric study are plotted in Figure 3.11 for selected layers. The results for the fixed toe case are not visually distinguishable from the case with toe stiffness of 40 MN/m/m and hence are not presented here. In general, the strains increase with decreasing toe stiffness consistent with observations for connection loads discussed in the previous section. However, with the exception of layer 3 in the wall with the least stiff toe, the strains at the connections are the largest along the reinforcement length. This is attributed to the effect of relative soil settlement behind the facing column as the soil is compacted, compresses under self-weight and the wall facing rotates outward during construction. The observation that the highest strains in a reinforcement layer occur close to the connections has been made for the RMC walls and other instrumented field and full-scale laboratory walls where strain monitoring was carried out in close proximity to the back of structural facings. As toe stiffness decreases there is a detectable increase in local strain at locations well beyond the back of the facing. This is attributed to the onset of internal soil shear failure mechanisms in the reinforced soil zone. The development of soil shear zones can be seen in Figure 3.12. The contours are shown with two intervals (< 2% and 2% to 5%). For the two least stiff toe cases there are local high strain levels at the heel of the wall that were developed during initial bottom-up construction. However, these zones are not contiguous through the height of the backfill and reinforcement strains are less than 3% (Figure 3.11). Hence these walls are assumed to be under working stress conditions according to criteria introduced earlier in this chapter.
3.5.4 Influence of wall height

The influence of wall height on toe and connection loads is shown in Figure 3.13. The reference toe stiffness value of 4 MN/m/m and a fixed toe condition were used in these simulations. The magnitude of toe load can be seen to increase with wall height for both reinforcement stiffness cases (Figure 3.13a). For the same height and reinforcement type the magnitude of toe load is less for the compliant toe restraint case. However, for the same wall height and toe condition the load is generally higher for the PET reinforcement case which is attributed to the relatively higher toe reaction stiffness with respect to the reinforcement layers as discussed earlier in the context of the continuously supported beam analogue. The relative contributions of the toe reaction and reinforcement layers are plotted in Figure 3.13b. For both reinforcement stiffness cases, the relative contribution of the toe to carry horizontal earth loads decreases with wall height but at a diminishing rate. However, the contribution of the toe for the PET reinforcement model is greater than for the steel reinforcement case for walls of the same height and toe boundary condition. For the highest wall considered (12 m) and a toe stiffness value of 4 MN/m/m, 70% of the load is carried by the PET reinforcement layers while for the matching steel reinforcement case the reinforcement layers carry about 90% of the total load. A practical implication of this observation is that for high steel reinforced soil walls the assumption in current AASHTO (2002) design practice to assign all earth loads to the reinforcement layers may be reasonable. For walls with more extensible reinforcement materials (i.e. geosynthetics) this assumption leads to very conservative (i.e. excessively safe) internal stability design.

3.5.5 Influence of wall batter

The influence of wall batter on toe and connection loads for PET reinforced soil walls is shown in Figure 3.14. A facing batter angle of $\omega = 0$ degrees corresponds to a vertical face. Most reinforced segmental retaining walls are constructed with $1^\circ < \omega < 15^\circ$ (NCMA 1997). As expected, the total load acting on the back of the wall facing decreases with increasing wall batter (Figure 3.14a). However, in these simulations the magnitude of toe load remained reasonably constant for the same toe boundary condition while the fraction of total load carried by the toe increased with increasing wall batter (Figure 3.14b).

3.5.6 Influence of reinforcement spacing

It can be expected that as the reinforcement spacing increases the magnitude of reinforcement (or connection) loads and toe load will increase when all other parameters are the same. In order to remove the influence of reinforcement stiffness as a variable when investigating the effect of
spacing, simulations were carried out with different reinforcement spacing but with the same global reinforcement stiffness value \( S_{\text{global}} = 475 \text{kN/m}^2 \) computed using \textbf{Equation 3.4}. Using a common global stiffness value, the plots in \textbf{Figure 3.15} show that the magnitude of loads and the distribution of total load to the connections and toe are sensibly independent of spacing for the same toe boundary condition. This observation is consistent with load predictions using the working stress method (K-Stiffness Method) originally proposed by \textit{Allen et al. (2003)}. This method was calibrated by fitting to loads deduced from instrumented full-scale field and laboratory walls. The influence of reinforcement stiffness is not accounted for in current limit-equilibrium-based (tie back wedge methods) such as the \textit{AASHTO (2002)} Simplified Method. For example, for a set of nominally identical walls varying only with respect to stiffness of the polymeric reinforcement, the loads in the reinforcement layers are always the same.

\textit{3.5.7 Influence of block-block interface stiffness}

Segmental retaining wall units transmit shear through interface friction, shear keys, pins and various types of connectors \textit{(NCMA 1997)}. The shear capacity and magnitude of interface stiffness may vary widely between different facing systems \textit{(Bathurst et al. 2008a)}. It may be expected that the interface shear stiffness will influence wall deformations and the distribution of total load to the toe and connections. \textbf{Figure 3.16} shows the influence of block-block interface stiffness on wall facing loads. The total load and load carried by the toe increases as interface shear stiffness increases but at a diminishing rate \textit{(Figure 3.16b)}. The distribution of load to the toe and reinforcement layers (connections) becomes sensibly constant beyond (say) \( K_{sbb} = 20 \text{MN/m/m} \) which is five times the reference toe stiffness value of 4 MN/m/m that is judged to be a reasonable value for these systems when seated on a rigid foundation.

\textit{3.6 Comparison of reinforcement loads with predicted values using current design methods}

Maximum reinforcement loads computed in numerical simulations can be compared to predicted loads using the \textit{AASHTO (2002)} Simplified Method (tie back wedge method) and the most recent version of the K-Stiffness Method \textit{(Bathurst et al. 2008b)}. According to the AASHTO approach the maximum reinforcement load \( (T_{\text{max}}) \) for non-surcharged walls can be calculated as:

\[ \text{3.5} \quad T_{\text{max}} = K \gamma z S_v \]
Here, \( z \) is the depth of the reinforcement layer below the crest of the wall and \( K \) is calculated as:

\[
K = \frac{\cos^2 (\phi + \omega)}{\cos^2 \omega \left(1 + \frac{\sin \phi}{\cos \omega}\right)^2}
\]

All other parameters have been defined previously. The maximum reinforcement load using the K-Stiffness Method and non-surcharged walls is:

\[
T_{\text{max}} = \frac{1}{2} K \gamma HS_y D_{\text{max}} \Phi_g \Phi_{\text{local}} \Phi_{fs} \Phi_{fb} \Phi_c
\]

Here: \( D_{\text{max}} \) = load distribution factor that modifies the reinforcement load based on layer location. The remaining terms, \( \Phi_g, \Phi_{\text{local}}, \Phi_{fs}, \Phi_{fb} \) and \( \Phi_c \) are influence factors that account for the effects of global and local reinforcement stiffness, facing stiffness, face batter and soil cohesion, respectively. The coefficient of lateral earth pressure is calculated as \( K = 1 - \sin \phi \) with \( \phi \) = secant peak plane strain friction angle of the soil. It is important to note that parameter \( K \) is used as an index value and does not imply that at-rest soil conditions exist in the reinforced soil backfill according to classical earth pressure theory. In the current parametric study with PET reinforcement, spacing \( S_v \) = 0.6 m and facing batter \( \omega \) = 8 degrees, the influence factors are: \( \Phi_g = 0.37 \); \( \Phi_{\text{local}} = 1 \); \( \Phi_{fs} = 0.51, 0.63, 0.76 \) and 0.86 for 3.6-, 6-, 9- and 12-m high walls, respectively; \( \Phi_{fb} = 0.84 \) and; \( \Phi_c = 1 \). Details to calculate these values for the wall used as an example here can be found in the paper by Bathurst et al. (2008b). To remove the choice of friction angle as a variable between calculation methods, the same value of peak friction angle is used in all calculations (i.e. \( \phi = \text{secant peak plane strain friction angle of the soil} = 48 \) degrees – Table 3.1).

The reinforcement loads \( (T_{\text{max}}) \) plotted in Figure 3.17 are the maximum load in each reinforcement layer at the end of construction excluding connection loads (which are higher in some cases as illustrated by the strain plots in Figure 3.11). The plots show that as toe stiffness decreases the magnitude of reinforcement load at each elevation increases and the load distributions become more triangular in shape. Nevertheless, the AASHTO Simplified Method over-predicts \( T_{\text{max}} \) values regardless of toe stiffness magnitude and the over-prediction increases.
with increasing toe stiffness. In practice, this over-prediction would be greater since lower friction angles based on direct shear tests and triaxial compression testing rather than higher plane strain test values are used in computations. For the fixed toe and toe stiffness case with 40 MN/m/m, the K-Stiffness Method is conservative. However, for the reference case corresponding to simulations matching the toe stiffness of the RMC walls, the K-Stiffness Method is very close but slightly conservative for design. For less stiff toe cases ($\leq 0.4$ MN/m/m) the K-Stiffness Method is non-conservative for design and simulations results can be seen to fall between predictions using the two methods. However, the database of wall case studies used to develop the K-Stiffness Method shows that the distribution of reinforcement loads deduced from measured strain values is typically trapezoidal with depth (Allen and Bathurst 2002; Allen et al. 2003; Bathurst et al. 2007, 2008b) lending support to the argument that the distribution of $T_{\text{max}}$ for numerical results with toe stiffness $\geq 4$ MN/m/m represents typical field walls. Furthermore, the results of direct shear testing presented earlier in this chapter (Figure 3.7) show that this value is reasonable for modular block units seated on a concrete or granular levelling pad.

As a further check on the influence of toe stiffness on reinforcement loads an additional set of simulations were carried out using normal load (height) dependent toe stiffness values (see Figure 3.7) and a range of wall heights. The stiffness values correspond closely to shear stiffness values computed for a concrete levelling pad and 6 mm displacement criterion. The numerical results in Figure 3.18 show that the influence of toe restraint on the distribution of reinforcement loads with height becomes trapezoidal in shape as wall height increases. Predicted loads using the K-Stiffness Method are compared to numerical results in the same figure. The trapezoidal shape, that is a distinguishing feature of the K-Stiffness Method, is judged to capture the trend in numerical results particularly for the higher walls while being conservatively safe (for design). This independent check of the accuracy of the K-Stiffness Method shows that this empirical-based design method is a promising approach for the internal stability design of these systems for working stress conditions.

3.7 Conclusions and implications to design and construction

Geosynthetic reinforced soil segmental retaining walls are mechanically complex systems. The magnitude of reinforcement loads under working stress conditions (i.e. operational conditions) is influenced by wall geometry, properties of the material components, boundary conditions and construction method. The number of carefully instrumented and monitored walls (other than the 11 walls in the recent RMC test program) is limited. For example, there are 13 case studies for
walls with granular backfills and another 18 case studies for walls constructed with c-ϕ soils (Bathurst et al. 2008b). Nevertheless, there is strong evidence that the current AASHTO (2002) Simplified Method (or variants) is very conservative for walls (i.e. excessively safe) when predicting maximum reinforcement loads under typical operational conditions in internal stability design. Furthermore, the distribution of reinforcement loads using the Simplified Method does not match loads deduced from measured strains. The K-Stiffness Method, which is an empirical-based working stress method calibrated against measured reinforcement loads, has been demonstrated to give better predictions of reinforcement loads (e.g. Bathurst et al. 2008b). However, because the database of physical measurements used to develop the K-Stiffness Method is limited, numerical modelling is required to systematically investigate the sensitivity of material properties and boundary conditions on wall performance. The parametric study reported here is a small subset of a much wider range of parameters that could be investigated. An important advantage of the numerical model adopted in this investigation is that the accuracy of the model has been previously verified against a wide range of measured performance responses from a series of carefully constructed, instrumented and monitored RMC physical test walls (Hatami and Bathurst 2005, 2006; Bathurst et al. 2009a; Huang et al. 2009). The major conclusions from the current investigation are:

1. A toe stiffness value of 4 MN/m/m is a reasonable value to simulate the interface shear stiffness between the bottom of a 0.3-m wide concrete block and a concrete or crushed stone base.

2. For numerical simulations with toe stiffness \( \geq 4 \) MN/m/m, the influence of magnitude of toe stiffness on wall facing displacements was limited to about 25\% of the height of the wall above the base.

3. The magnitude of toe stiffness has a potentially significant effect on the magnitude and distribution of reinforcement loads. In general, as toe stiffness decreases, the magnitude of reinforcement loads increases and the distribution of loads becomes more triangular with depth for walls with uniform spacing and the same reinforcement material in each layer. However, to generate a triangular load distribution for the reinforced soil walls with extensible geosynthetic reinforcement in this study, it was necessary to reduce the magnitude of toe stiffness to values that are orders of magnitude lower than those deduced from full-scale interface shear tests.
4. The influence of magnitude of toe stiffness on reinforcement loads diminishes with height of reinforcement layer above the toe. For a 6-m high wall and the range of toe stiffness values investigated, the toe effect was limited to the bottom half of the wall.

5. The fraction of total load carried by the toe increases with increasing horizontal toe stiffness whilst the fraction of total load carried by the reinforcement layers decreases.

6. The magnitude of load carried by the toe and reinforcement layers is influenced by the stiffness of the reinforcement layers. As reinforcement stiffness increases, reinforcement loads increase and the fraction of total earth load carried by the reinforcement layers increases.

7. The maximum strains typically occurred at the connections. In the example 6-m high wall, the reinforcement strains at the connections increased with decreasing toe stiffness over the bottom half of the wall.

8. The fraction of load carried by the reinforcement layers increased with increasing wall height. For the same wall height, the fraction of load carried by the reinforcement layers increased with increasing reinforcement stiffness. However, for walls with $H \leq 12$ m, a typical geosynthetic reinforcement stiffness and toe stiffness value of 4 MN/m/m, the fraction of load carried by the reinforcement layers was not greater than about 70%.

9. As facing batter increased from 4 to 13 degrees in this study, the magnitude of toe load remained reasonably constant but the fraction of total earth load carried by the toe increased.

10. For the walls in this numerical investigation with the same global reinforcement stiffness value there was no practical influence of reinforcement spacing on reinforcement and toe loads.

11. There was a diminishing influence of increasing magnitude of block-block interface stiffness on magnitude of toe and reinforcement loads for the range of block-block stiffness values investigated.

12. For all toe stiffness cases investigated, the current AASHTO (2002) Simplified Method over-predicted the reinforcement loads. The K-Stiffness Method (Bathurst et al. 2008b) provided a very accurate estimate of end-of-construction reinforcement loads when a
single-value toe stiffness of 4 MN/m/m was used in computations for a 6-m high segmental wall seated on a rigid foundation and reinforced with a typical PET reinforcement material. The K-Stiffness Method was shown to capture the trend towards a trapezoidal distribution of reinforcement loads with increasing wall height while being slightly conservative (i.e. safer for design).

An important implication of the results of this numerical investigation to wall performance and design is that the horizontal toe of a reinforced soil segmental retaining wall can develop a significant contribution to the resistance against horizontal earth loads developed behind a structural facing under operational conditions (i.e. working stress conditions). The required toe resistance is available from interface shear capacity developed between the base of the modular block facing and a concrete or crushed stone levelling pad. This contribution is ignored in current design methods (e.g. AASHTO 2002) and partially explains the over-estimation of reinforcement loads and the more uniform distribution of load observed in instrumented walls. This constraint can also be considered to be available at collapse of the structure if sufficient surcharge loading can be applied. However, the tests at RMC and the numerical modelling described herein demonstrate that an ultimate limit state defined by a contiguous failure zone through the reinforced soil will occur well before these walls collapse due to reinforcement pullout or rupture. This internal ultimate (failure) limit state has been introduced in the K-Stiffness Method (e.g. Allen et al. 2003; Bathurst et al. 2008).

The focus of this investigation has been on walls under working stress conditions. This emphasis has been prompted in part because loads due to working stress (operational) conditions are assumed in current reliability-based design (load and resistance factor design - LRFD) in North America (e.g. AASHTO 2009; CSA 2006). Hence analytical methods and numerical models that can accurately predict reinforcement loads under operational conditions are of great interest to develop data for reliability-based LRFD calibration.

The empirical-based K-Stiffness Method was developed by fitting to measured reinforcement loads in instrumented full-scale walls and is demonstrated to better capture the qualitative trends in the numerical results and the magnitude of predicted loads. It should be noted that the database of full-scale walls that was used to calibrate the K-Stiffness Method did not include any of the RMC walls that were used to verify the numerical model in this investigation. Hence, this study is an independent verification of the K-Stiffness Method.
An important implication of this investigation to good construction practice is that the wall toe should be embedded (this is typical design practice), good contact is developed at the base of the wall facing column and the concrete or granular levelling pad, or a mechanical shear key is in place between the bottom block and a concrete footing.

Direct interface shear testing has shown that adequate shear resistance is available to develop significant toe resistance provided the levelling pad or footing is seated on a rigid or very stiff foundation. In practice, the bottom of retaining walls is typically embedded and can provide passive earth resistance according to classical notions of earth pressure theory. This potential additional toe capacity under working stress conditions has not been investigated in this study, particularly if the mobilized passive resistance is removed by excavation over the life of the structure. However, current study shows that significant toe restraint can be generated by friction alone between the bottom of the concrete wall and the concrete toe or levelling pad. Hence, the presence of passive fill in front of the wall may not be an issue. Furthermore, the lateral deformations required to fully mobilize passive resistance in front of the embedded toe are likely larger than the deformation required to fully mobilize shear resistance. In our physical direct shear tests the deformation required to fully mobilize shear resistance was only 2 mm. Nevertheless, this issue requires further investigation.

It is possible that if poorer quality aggregate is used for the levelling pad and (or) more compliant foundation conditions are present, the effective horizontal toe stiffness available at the base of the wall may be less than that determined from the laboratory shear tests reported in this chapter. The influence of foundation compressibility on wall performance is currently under investigation.

REFERENCES


Table 3.1 Material properties.

<table>
<thead>
<tr>
<th>Property</th>
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</tr>
<tr>
<td>$\phi$ (peak plane strain friction angle) (degrees)</td>
<td>48</td>
</tr>
<tr>
<td>$c$ (cohesion) (kPa)</td>
<td>0.2</td>
</tr>
<tr>
<td>$B_i/p_a$ (initial bulk modulus number)</td>
<td>74.8</td>
</tr>
<tr>
<td>$\varepsilon_a$ (asymptotic volumetric strain value)</td>
<td>0.02</td>
</tr>
<tr>
<td>$\rho$ (density) (kg/m$^3$)</td>
<td>2250</td>
</tr>
<tr>
<td><strong>Reinforcement</strong></td>
<td></td>
</tr>
<tr>
<td>$J_{PET}$ (stiffness) (kN/m)*</td>
<td>285</td>
</tr>
<tr>
<td>$J_{steel}$ (stiffness) (kN/m)**</td>
<td>30000</td>
</tr>
</tbody>
</table>

Note:
* Ultimate (index) strength: 80 kN/m
** Yield strain: 0.2%
Table 3.2 Interface properties.

<table>
<thead>
<tr>
<th>Interface</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Backfill soil-Block</strong></td>
<td></td>
</tr>
<tr>
<td>$\delta_{sb}$ (friction angle) (degrees)</td>
<td>48</td>
</tr>
<tr>
<td>$\psi_{sb}$ (dilation angle) (degrees)</td>
<td>6</td>
</tr>
<tr>
<td>$K_{nsb}$ (normal stiffness) (MN/m/m)</td>
<td>100</td>
</tr>
<tr>
<td>$K_{ssb}$ (shear stiffness) (MN/m/m)</td>
<td>1</td>
</tr>
<tr>
<td><strong>Block-to-Block</strong></td>
<td></td>
</tr>
<tr>
<td>$\delta_{bb}$ (friction angle) (degrees)</td>
<td>57</td>
</tr>
<tr>
<td>$c_{bb}$ (cohesion) (kPa)</td>
<td>46</td>
</tr>
<tr>
<td>$K_{nbb}$ (normal stiffness) (MN/m/m)</td>
<td>1000</td>
</tr>
<tr>
<td>$K_{sbb}$ (shear stiffness) (MN/m/m)</td>
<td>40</td>
</tr>
<tr>
<td><strong>Backfill-Reinforcement</strong></td>
<td></td>
</tr>
<tr>
<td>$\phi_{b}$ (friction angle) (degrees)</td>
<td>48</td>
</tr>
<tr>
<td>$s_{b}$ (adhesive strength) (kPa)</td>
<td>1000</td>
</tr>
<tr>
<td>$K_{b}$ (shear stiffness) (kN/m/m)</td>
<td>1000</td>
</tr>
</tbody>
</table>
Table 3.3 Values used in parametric study.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Values*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall height (m)</td>
<td>(3.6), 6, 9, 12</td>
</tr>
<tr>
<td>Facing column stiffness (MN/m/m)**</td>
<td>0.4, 4, 20, (40), 80</td>
</tr>
<tr>
<td>Facing batter from vertical (degrees)</td>
<td>0, 4, (8), 13</td>
</tr>
<tr>
<td>Spacing (m)</td>
<td>0.3, (0.6), 0.9</td>
</tr>
<tr>
<td>Toe stiffness (kN/m)</td>
<td>0.04, 0.4, (4), 40, fixed</td>
</tr>
<tr>
<td></td>
<td>Normal load dependent (Figure 3.7)</td>
</tr>
</tbody>
</table>

Note:
* Values in parentheses match RMC control test (Wall 1)
** Block-to-block interface shear stiffness $K_{ubb}$
Figure 3.1 Cross-section view of Wall 1.
Figure 3.2 Evolution of horizontal toe load and sum of connection loads in RMC Wall 1. (h = height of wall above toe, q = post-construction uniform surcharge pressure, \( \gamma \) = bulk unit weight of sand backfill).
Figure 3.3 Post-construction wall deformation profiles for RMC Wall 1.
Figure 3.4 Connection and toe loads at end of surcharge unloading and toe release for RMC Wall 1.
Figure 3.5 Maximum reinforcement strains from instrumented full-scale vertical walls with granular backfill. Notes: values in parentheses are wall height (H); z = depth below crest of wall (data from Allen et al. 2003; Bathurst et al. 2008b; Miyata and Bathurst 2007a).

Field walls
- GW5 - full-height precast concrete panel (4.9 m)
- GW8 - incremental precast concrete panel (6.1 m)
- GW10 - wrapped-face (6.1 m)
- GW18 - full-height precast concrete panel (6.1 m)
- GW26C - granular infilled hollow masonry concrete block (10.7 m)
- GW26D - granular infilled hollow masonry concrete block (5.5 m)

maximum strain before gauge failure

(a) Field walls
Figure 3.5 (continued) Maximum reinforcement strains from instrumented full-scale vertical walls with granular backfill. Notes: values in parentheses are wall height (H); z = depth below crest of wall (data from Bathurst et al. 2008b; Miyata and Bathurst 2007a).

(b) PWRI test walls
**Figure 3.6** Example numerical FLAC grid.
Figure 3.7 Toe interface shear stiffness values computed from block-foundation interface shear testing.

Values used in numerical simulations with normal load dependent toe stiffness

2 mm shear displacement
- Block on concrete levelling pad
- Block on crushed stone 1 ($D_{50} = 12$ mm)
- Block on crushed stone 2 ($D_{50} = 22$ mm)

6 mm shear displacement
- Block on concrete levelling pad
- Block on crushed stone 1 ($D_{50} = 12$ mm)
- Block on crushed stone 2 ($D_{50} = 22$ mm)

Back-calculated from RMC test walls
($W = 0.3$ m, $H = 3.6$ m)
Figure 3.8 Influence of toe stiffness and reinforcement stiffness on facing displacements (H = 6 m, S_v = 0.6 m, \( \omega = 8 \) degrees from vertical).

(a) PET reinforcement

(b) Steel reinforcement
Figure 3.9 Influence of toe release following end of construction (EOC) for wall with PET reinforcement. (H = 6 m, Sv = 0.6 m, ω = 8 degrees from vertical).

(a) Wall displacements

(b) Connection and toe loads
Figure 3.10 Influence of toe stiffness and reinforcement stiffness on magnitude and distribution of toe and connection loads. (H = 6 m, S_v = 0.6 m, ω = 8 degrees from vertical).

(a) PET reinforcement
Figure 3.10 (continued) Influence of toe stiffness and reinforcement stiffness on magnitude and distribution of toe and connection loads. (H = 6 m, S_v = 0.6 m, ω = 8 degrees from vertical).

(b) Steel reinforcement
Figure 3.11 Influence of toe stiffness on reinforcement strains for wall with PET reinforcement layers. (H = 6 m, S_v = 0.6 m, \(\omega = 8\) degrees from vertical).
Figure 3.12 End-of-construction soil shear strain contours for walls with PET reinforcement and different toe stiffness. a) 40 MN/m/m; b) 4 MN/m/m; c) 0.4 MN/m/m; and d) 0.04 MN/m/m. (H = 6 m, S_v = 0.6 m, ω = 8 degrees from vertical).
Figure 3.13 Influence of wall height, reinforcement stiffness and toe stiffness on loads carried by the toe and connections. ($S_v = 0.6$ m, $\omega = 8$ degrees from vertical).

(a) Toe load

(b) Relative contribution of total load carried by toe and connections
Figure 3.14 Influence of facing batter (ω) and toe stiffness on loads carried by the toe and connections for walls with PET reinforcement. (H = 6 m, Sv = 0.6 m).

(a) Toe and connection loads

(b) Relative contribution of total load carried by toe and connections
Figure 3.15 Influence of reinforcement spacing and toe stiffness on loads carried by the toe and connections for walls with PET reinforcement. ($S_{\text{global}} = 475 \text{ kN/m}^2$, $H = 6 \text{ m}$, $\omega = 8$ degrees).

(a) Toe and connection loads

(b) Relative contribution of total load carried by toe and connections
**Figure 3.16** Influence of block-block interface stiffness and toe stiffness on loads carried by the toe and connections for walls with PET reinforcement. \( H = 6 \text{ m}, S_v = 0.6 \text{ m}, \phi = 8 \text{ degrees} \).
Figure 3.17 Influence of (constant) toe stiffness on maximum reinforcement loads and comparison with predictions using AASHTO (2002) Simplified Method and K-Stiffness Method (Bathurst et al. 2008b) for wall with PET reinforcement. (H = 6 m, Sv = 0.6 m, ω = 8 degrees).
Figure 3.18 Influence of normal load-dependent toe stiffness and wall height on maximum reinforcement loads and comparison with predictions using K-Stiffness Method (Bathurst et al. 2008b) for wall with PET reinforcement. ($S_v = 0.6$ m, $\omega = 8$ degrees, toe stiffness values from Figure 3.7).
Chapter 4
Load and Resistance Factor Design (LRFD) Calibration for Steel Grid Reinforced Walls

4.1 Introduction

Limit states design, called load and resistance factor design (LRFD) in the USA, is now mandated in AASHTO (2007) for the design of foundations and earth retaining structures. Reinforced soil walls can be broadly classified into two categories: polymeric reinforced soil walls using relatively extensible geosynthetic products, and steel reinforced soil walls. In the latter category AASHTO (2007) and FHWA (2001) distinguish between walls constructed with steel strips and walls constructed with steel grids (bar mats and welded wire mesh). In the current AASHTO (2007) design guidance document, LRFD calibration for reinforced soil walls has been carried out by fitting to allowable stress design (ASD) together with accepted factors of safety for each limit state. This practice is undesirable since there is no guarantee that an acceptable target probability of failure is achieved and (or) load and resistance factors result in similar probabilities of failure for a suite of limit states. This chapter is focused on LRFD calibration of reinforced steel grid walls using measured loads from instrumented field walls and resistance data for tensile rupture and pullout from independent laboratory tests.

The general approach used in this chapter follows that described by Allen et al. (2005) and Bathurst et al. (2008a, 2009a). These papers also used data for steel grid reinforced soil walls as an example to demonstrate LRFD calibration concepts in the context of geotechnical earth structures. This chapter extends this earlier work by using a larger database of measured loads and a very much larger database of pullout data from laboratory tests. In addition, the current study investigates LRFD calibration for the rupture (yield) limit state of the steel reinforcement. A novel approach illustrated in this chapter is a technique to adjust the underlying deterministic model for pullout to simultaneously improve the prediction accuracy of the model and remove hidden dependency between resistance bias values and calculated (predicted) pullout capacity. The overall approach has application to the internal stability limit states design of a wide range of reinforced soil wall technologies provided that sufficient measured load data for walls under operational conditions is available together with resistance capacity data from physical testing.
4.2 Wall database

Tables 4.1 and 4.2 summarize the geometry and material properties for the steel grid walls used in this study. Five walls were built using bar mat and two walls using welded wire mesh. The walls are 4 to 18 m high with uniform surcharge loads less than 1.5 m equivalent soil height. The granular soil friction angle is between 35 and 43 degrees. Reinforcement stiffness per unit width of wall is typically between 30 and 100 MN/m and global stiffness (Equation 3.4) ranges from about 50 to 150 MN/m². The latter is a useful parameter to compare overall stiffness between walls that use different reinforcement types (e.g. steel and polymeric reinforced soil walls) and number of layers over the wall height (Allen et al. 2004; Bathurst et al. 2008b, 2009b). The vertical reinforcement spacing is 0.45 to 0.75 m. The reinforcement comes in prefabricated units (elements) with width between 0.15 and 1.2 m but typically 0.4 to 0.6 m. The transverse steel bar spacing is between 0.23 and 0.61 m. Bathurst et al. (2008a, 2009a) have used the same case study data with the exception of case study WWM2 in their example to illustrate the basic concepts of LRFD calibration. Using case study WWM2 in this chapter increases the number of load data points available from 34 to 42.

4.3 Pullout and yield limit states

This chapter is restricted to calibration of pullout and yield limit states for steel grid walls subjected to soil self-weight loading only. Hence, factored limit state functions have the form

\[ \phi R_n - \gamma Q_n \geq 0 \]

Here, \( Q_n \) = nominal (specified) load; \( R_n \) = nominal (characteristic) resistance; \( \gamma \) = load factor; and \( \phi \) = resistance factor. Correct load and resistance factor calibration of a selected limit state must include bias statistics where bias is defined as the ratio of measured value to predicted (nominal) value. Bias statistics are influenced by model bias (i.e. intrinsic accuracy of the deterministic model representing the physics of the limit state under investigation), random variation in input parameter values, spatial variation in input values, quality of data and, consistency in interpretation of data when data are gathered from multiple sources (the typical case) (Allen et al. 2005). If the underlying deterministic model used to predict load or resistance capacity is accurate and other sources of randomness are small, then bias statistics have a mean value that is close to one and a small coefficient of variation. If the underlying deterministic models are poor, then adjustments to these models may be required in order to achieve sensible
values for load and resistance factors (i.e. load factor values equal to or greater than one and resistance factor values equal to or less than one). Incorporating bias statistics into LRFD calibration and assuming only one load type, Equation 4.1 can be expressed as

$$[4.2] \gamma Q X_R \geq \phi X_Q$$

Here, $X_R = \text{resistance bias computed as the ratio of measured resistance (R_m) to calculated (predicted) nominal resistance (R_n)}$, and $X_Q = \text{load bias computed as the ratio of measured load (Q_m) to the calculated (predicted) nominal load (Q_n)}$. The derivation of Equation 4.2 is presented in Appendix B in this thesis. In this chapter the predicted values for reinforcement load and pullout capacity are computed using the deterministic models (equations) recommended by AASHTO (2007) and FHWA (2001) and the specified yield tensile strength of the steel.

### 4.4 Load bias statistics

The maximum tensile load $T_{max}$ in a reinforcement layer using the AASHTO Simplified Method (AASHTO 2002, 2007) is calculated using the following expression

$$[4.3] T_{max} = S_v \sigma_v K_r$$

Here, $S_v = \text{vertical spacing of the reinforcement layer}$; $\sigma_v = \text{vertical earth pressure at the reinforcement depth}$; and $K_r = \text{lateral earth pressure coefficient}$. For steel grid walls, $K_r$ varies from $2.5 K_a$ to $1.2 K_a$ at the top of the wall to a depth of 6 m and remains constant thereafter (Figure 4.1). Here, $K_a$ is the active lateral earth pressure coefficient, which is a function of the peak soil friction angle $\phi$. According to AASHTO, the friction angle is capped at $\phi \leq 40^\circ$.

AASHTO (2007) makes no distinction between bar mat and welded wire walls. Figure 4.2a shows a plot of measured versus calculated (predicted) loads. The visual impression is that measured loads generally increase with calculated loads but there appear to be more data points below the 1:1 correspondence line than above. This is confirmed quantitatively by computing the mean of load bias values where bias value is the ratio of measured to calculated load. This computation gives $\mu_Q = 0.89$ meaning that measured loads are about 10% lower than the calculated values on average. This is a conservative (safe) result for conventional ASD.
Another visual observation is that the data for bar mat and welded wire walls are equally distributed. As a further check, the mean and COV of the bias values for each population was computed and shown to be close to the bias values for the entire dataset. Hence welded wire and bar mat bias data were are not treated separately in this investigation.

It may be tempting to correct the load model by decreasing predicted loads by 10%. This can be done by shifting the design curve to the left in Figure 4.1 (i.e. multiplying by a correction factor of 0.90). However, because this correction would be applied to the entire range of calculated load values $T_{\text{max}}$, it is more convenient to leave the load model as is and effectively include the correction in the final choice of load factor as described in the next section.

Figure 4.2b shows bias values plotted against calculated loads. The linear regression line fitted to all data is visually very close to the horizontal line (mean of the entire data set) suggesting that there is no hidden dependency of bias values on calculated loads. As a quantitative check the 95% confidence interval on the slope of the regressed line (Draper and Smith 1981) was computed and shown to bracket zero. This confirms that at a level of significance $\alpha = 5\%$, bias values are independent of calculated loads and hence no adjustment to the load model is required or, alternatively, the data does not need to be parsed into subsets based on calculated load ranges and different mean and COV values assigned to each group. Bathurst et al. (2008a) came to the same conclusion regarding non-dependency of load bias values using the Spearman’s correlation coefficient test on essentially the same data set. The cumulative distribution function (CDF) plot for the load data is presented in Figure 4.2c. The predicted log-normal distribution fits most of the data reasonably well. The data for bar mat and welded wire walls are visually equally distributed. There is a visually poor fit at the lower tail that is exaggerated using log-linear scales but this is not a practical concern since there is a good match at the upper tail. It is the fit to the upper tail of load bias values that is important for LRFD calibration as noted by Allen et al. (2005) and Bathurst et al. (2008a).

4.5 Load factor for steel grid walls

Figure 4.3 shows that 30% of the load bias values are greater than one (see cumulative distribution plot for unfactored load data). In LRFD calibration it is desirable to select a load factor that when multiplied against calculated (nominal) load increases the design load to an acceptable value (i.e. shifts the entire distribution to the left by a satisfactory amount). In USA design codes the load factor for soil self-weight is called the vertical earth load factor. Bathurst et al. (2008a) proposed a load factor of $\gamma_Q = 1.75$ for steel grid walls based on $n = 34$ (original)
data points. In the current investigation, this value of the load factor applied to \( n = 42 \) data points gives a probability of exceedance of 4\% (Figure 4.3). This value is close to the value of 3\% that was assumed as a starting point during the development of the Canadian Highway Bridge Design Code (CSA 2006) and AASHTO (2007) for limit states design of superstructure elements in bridge structures (Nowak 1999; Nowak and Collins 2000). The AASHTO (2007) design code recommends a load factor of \( \gamma_Q = 1.35 \) which, if adopted here, results in 16\% of measured loads being higher than calculated (predicted) values. However, the value of 1.35 is attractive because it is a value that is already in the AASHTO design code. One objective of LRFD calibration is to achieve a uniform target probability of failure for a set of limit states. The results of calculations presented later show that this objective can be met using \( \gamma_Q = 1.35 \). The results of using different load factors on probability of failure and calculation of resistance factors for pullout and yield limit states are also examined later (Table 4.3).

4.6 Resistance bias statistics

4.6.1 Pullout capacity

According to AASHTO (2002, 2007) and FHWA (2001) the pullout capacity of a steel grid reinforcement layer can be calculated as

\[
P_c = 2F^*L_c R_c \sigma_v
\]

where \( P_c = \) pullout capacity (kN/m); \( L_c = \) anchorage length (m); \( R_c = \) coverage ratio (b/S\( h \)); \( \sigma_v = \) vertical stress (kPa); and \( F^* = \) dimensionless pullout resistance factor. The reinforcement may be discontinuous in the horizontal direction with elements (panels) of width b placed at center to center spacing \( S_h \). The calculation of pullout capacity is on a unit running length of wall basis consistent with reinforcement load (\( T_{max} \)) dimensions (e.g. kN/m).

The pullout resistance bias values were calculated using the default \( F^* \) values specified in current AASHTO and FHWA design codes using the design chart reproduced in Figure 4.4. The default \( F^* \) value is a function of the transverse member thickness (diameter) \( t \) and transverse spacing \( S_t \). The maximum \( F^* \) value (= 20t/S\( t \)) occurs at the backfill surface and decreases linearly to 10t/S\( t \) at a depth of 6 m, remaining constant thereafter.
The steel grid pullout data were taken from a database reported by Allen et al. (2001) (total of 41 tests) and data provided by an independent testing laboratory (an additional 29 tests). The specimens were manufactured with transverse bars of thickness (diameter) \( t = 6 \) to 13 mm and transverse spacing \( S_t = 0.23 \) to 0.61 m. Tests were carried out in laboratory pullout boxes together with granular soils.

**Figure 4.5a** shows measured versus predicted (calculated) pullout capacity values for steel grid pullout data. Measured pullout capacity generally increases with increasing magnitude of predicted values indicating that the underlying deterministic model does well capturing the trend in the data. However, the visual distribution suggests that the majority of the data points fall above the 1:1 correspondence line. This is confirmed by the average of bias values which is computed as \( \mu_R = 1.20 \) and used to plot the solid diagonal line in the figure. This means that the pullout model is conservatively safe on average for conventional factor of safety design. However, an over-estimation of pullout capacity by 20% is not desirable for LRFD calibration. **Figure 4.5b** shows resistance bias data plotted against calculated (predicted) values. The horizontal line corresponds to the mean bias value for the entire data set (\( \mu_R = 1.20 \)). The data show what appears to be a visual dependency of resistance bias values with calculated pullout capacity values. A zero-slope test on all data confirms quantitatively that this is the case: specifically, the 95% confidence limits on the slope of the regressed linear line (red dashed line in the figure) are -0.009 and -0.001 which do not contain a slope of zero. The strategy adopted here is to improve the pullout model with respect to the mean of bias values and dependency by parsing the data into two groups. The breakpoint was determined using the SOLVER optimization utility in Excel with the objective function (sum of the squares of the differences) minimized and constrained to a stepped function as illustrated in **Figure 4.5b**. The solution was adjusted slightly to give convenient constant values of 1.25 and 1.00 at breakpoint \( P_c = 80 \) kN/m. A practical implication of this observation is that the underlying default model is accurate *on average* for pullout capacity values greater than 80 kN/m using **Equation 4.4** but is conservative (safe) for pullout capacity predictions less than this value (most of the data set). This deficiency in the pullout model can be corrected by multiplying calculated pullout values (**Equation 4.4**) by a factor of 1.25 when \( P_c \leq 80 \) kN/m. Corrected bias values are re-plotted in **Figure 4.5c**. The zero-slope test applied to the fitted linear line in the plot has a zero slope at a level of significance of 5%. A practical method for the design engineer to perform the same correction is to use the proposed F* design chart in **Figure 4.4**. Here the current AASHTO design curve has been multiplied by a factor of 1.25. However, a second correction is required for \( P_c > 100 \) kN/m. These values must be multiplied by
a factor of 0.8 to match the corrected bias values corresponding to \( P_c > 80 \text{ kN/m} \) plotted in Figure 4.5c. Fortunately this correction is required for only about 17% of the pullout database and in practice would apply to reinforcement layers at depths where pullout capacity does not control design.

The CDF for the corrected bias data is plotted in Figure 4.5d. The mean and coefficient of variation of the corrected bias values are now 0.99 and 0.40, respectively. The log-normal approximation to the corrected bias data is a good fit over the entire data set and there is no need to perform a separate best fit to the lower tail. Furthermore, there is no longer the requirement to assign different mean and COV values to different ranges of calculated pullout load as was done by Bathurst et al. (2008a). This greatly simplifies LRFD calibration and the implementation of the method for the design engineer.

4.6.2 Rupture (yield) strength

A total of 22 tensile test results for rupture of bar mat materials were provided by one manufacturer. AASHTO (2007) uses the yield of the steel reinforcement \( f_y = 450 \text{ MPa} \) as the nominal resistance value but these data were not available. Nevertheless, similar test data for steel strip reinforcement showed that the bias statistics are practically identical using rupture or yield strength measurements. Bias values were calculated as the ratio of measured to nominal (rupture) strength. The data are plotted as a CDF in Figure 4.6. The mean of the bias values is \( \mu_R = 1.13 \) based on all data. As may be expected for this manufactured material, the spread in bias values is very small \( \text{COV}_R = 0.08 \) and very much less than for pullout bias statistics. Bias statistics for steel rupture reported here are very similar to yield bias values for concrete reinforcing steel reported by Nowak and Szerszen (2003).

The predicted normal and log-normal distributions for the entire bias data are very close and a normal distribution can be assumed to be satisfactory. For such a narrow spread in the data, potential dependency of the type investigated earlier is not a concern.

It can be noted that there are no data points with bias values less than one. This is because the data are from production control records. Consequently, batches were rejected that had specimen strength values less than the nominal specified rupture value. Similar truncated CDF plots for tensile yield and break of steel bars or strands used to reinforce concrete have been reported by Nowak and Szerszen (2003). Nevertheless, a best-fit to lower tail is shown in Figure 4.6 since it is the overlap of the lower tail of the resistance bias data with the upper tail of the load bias data.
that is most important for the calculation of probability of failure. The influence of using approximations to the entire data set and the fitted tail is demonstrated later in this chapter.

4.7 Calibration results

In the LRFD calibrations to follow, the target probability of failure is taken as 1 in 100 for reinforced soil walls as recommended by Allen et al. (2005) and Bathurst et al. (2008a). This corresponds to a reliability index value $\beta = 2.33$.

Computed resistance factors using three assumed load factors and bias statistics for load and resistance data are summarized in Table 4.3. Monte Carlo simulations using an Excel spreadsheet (Allen et al. 2005; Bathurst et al. 2008a) were used to find the resistance factor in Equation 4.2 for each prescribed load factor and target $\beta = 2.33$. Closed-form solutions reported by Bathurst et al. (2008a) also gave the same values at accuracy of ± 0.01.

The computed resistance factors for pullout range from 0.40 to 0.52. A value of $\varphi = 0.40$ is selected matching the load factor $\gamma_Q = 1.35$ recommended by AASHTO (2007). The same codes recommend $\varphi = 0.90$ which is much higher. However, recall that the current pullout model underestimates pullout capacity and the new recommended value here is developed from bias statistics using a more accurate pullout model. Yield limit state resistance factors vary with choice of resistance bias statistics from 0.64 to 0.89. A value of $\varphi = 0.65$ using best-fit-to-tail is judged to be a reasonable choice matching $\gamma_Q = 1.35$ and is also the value currently recommended in the AASHTO (2007) design code. The recommended load and resistance factors give the same target probability of failure (Pf = 1%) for both limit states. Recall however, that to achieve the target probability of failure of 1% for pullout, Equation 4.4 and $F^*$ using the proposed new design curve in Figure 4.4 are required.

4.8 Comparison with ASD past practice

It is useful to compare factors of safety used in ASD to equivalent values using the results of LRFD calibration. For the case of a single load factor the predicted (design) factor of safety can be estimated as

$$\text{[4.5]} \quad \frac{R_n}{Q_n} = \frac{\gamma_Q}{\varphi}$$
Column 7 in Table 4.4 shows current recommended factors of safety of 1.5 and 2.08 for yield and pullout, respectively, based on ASD past-practice (AASHTO 2002, FHWA 2001). To examine the influence of the proposed load and resistance factors on performance, the actual or operational factor of safety can be computed for each limit state using ASD past practice and LRFD. The operational factor of safety (OFS) is defined as the ratio of the measured resistance \( R_m \) to measured load \( Q_m \) and is calculated as follows

\[
[4.6] \quad \text{OFS} = \frac{R_m}{Q_m} = \frac{R_n \mu_R}{Q_n \mu_Q} = \frac{H_R}{\mu_Q} = \gamma_Q \cdot \frac{H_R}{\mu_Q}
\]

The results of these calculations are summarized in Table 4.4. The table shows that operational factors of safety for pullout using ASD (column 8) are larger than design values (column 7). This is consistent with the opinion of experienced design engineers that current practice is conservatively safe (e.g. loads in steel reinforced soil walls are lower than predicted values and pullout capacities are greater than predicted). This conservativeness has also been shown quantitatively in Figures 4.2a and Figures 4.5a,b.

Table 4.4 shows that the actual operational factor of safety for pullout for the walls in the database is greater (column 2) using the proposed LRFD load and resistance factors and the revised pullout model than the operational factor of safety (column 8) using ASD. Not unexpectedly, the corresponding probability of failure using ASD is also larger (\( P_f = 11\% \) versus 1%).

For the yield limit state, the operational factor of safety is 2.64 using LRFD and ASD (column 2 and column 8) but larger than the specified value of 2.08 (column 7) in current design codes. Furthermore, regardless of which method is used the probability of failure remains 1%. Hence, there are no practical benefits of LRFD over ASD past practice for this limit state if both methods give the same target probability of failure. This also demonstrates that the result of LRFD calibration for this limit state is consistent with ASD past practice.

An alternative appreciation of the link between current practice and selection of resistance factor can be referenced to the data in the middle of Table 4.4. The values of resistance factor shown in Column 4 are computed based on the current estimated operational factor of safety (Column 5 = Column 8) and the prescribed load factor of 1.35. In other words, if the objective is to select
resistance factors to match current operational factors of safety with \( \gamma_0 \) fixed at 1.35, then the values shown in Column 5 are required. However, the penalty is that there is a large difference in the probability of failure for each limit state.

Finally it should be noted that loss of section due to corrosion has not been included in the calibrations reported here. Sacrificial thicknesses of steel reinforcement can be computed using recommendations reported by AASHTO (2007) and FHWA (2001), and pullout and yield capacities adjusted as recommended in the same guidance documents.

4.9 Conclusions and practical implications

This chapter reports the results of LRFD calibration for pullout and yield limit states for reinforced soil walls that use steel grids (welded wire and bar mat). Calibration is limited to the case of a single load contribution due to soil self-weight. The approach uses a database of measured reinforcement loads from field-instrumented walls and laboratory pullout tests. An important feature of the calibration method is the use of bias statistics to account for prediction accuracy of the underlying deterministic models for reinforcement load, pullout capacity and yield strength of the steel grids, and random variability in input parameters. In order to ensure sensible values for resistance factors (i.e. less than or equal to one) while keeping the same load factor recommended in the current AASHTO (2007) design code, modification of the current deterministic model for pullout is unavoidable. This chapter proposes a new revised chart for the estimation of the pullout model resistance factor (F*) for steel grid reinforcement.

The advantages of the proposed LRFD approach described here are:

1) Hidden dependencies between bias values and calculated capacity for the pullout limit state are reduced.

2) Comparison with allowable stress design (ASD) past practice shows that the operational factor of safety for pullout using the new LRFD-based approach gives a higher factor of safety and a lower probability of failure.

3) Compared to ASD past practice and current LRFD codes there is less chance of a reinforcement layer being under-designed or over-designed with respect to pullout since the prediction accuracy dependency has been removed by using the modified pullout resistance design chart.
4) Recommended load and resistance factors correspond to a uniform probability of failure for both pullout and yield limit states which is a desirable criterion for any set of limit states.

5) The load factor for soil self-weight \( (\gamma_Q = 1.35) \) is the same value recommended in the current AASHTO (2007) design code.

The probability of failure of 1% and 11% for yield and pullout, respectively, based on ASD past practice appears large (Column 8 in Table 4.4). There is no evidence that 1 in 11 or even 1 in 100 of reinforcement elements failed in the case study database due to internal stability modes of failure. In fact all of the walls in the database performed well. Furthermore, there is a long history of excellent performance of these systems in the field. Explanations for this apparent contradiction are as follows:

a) Current design codes require that the reinforcement length be at least 70% of the height of the wall. This typically results in additional pullout load capacity since the actual anchorage length \( (L_e) \) is greater than the value computed using pullout design equations alone.

b) Design engineers typically select reinforcement with constant cross-sectional area and grid unit width from stock materials that must be greater than that required for the most critically loaded element.

c) Reinforcement elements have additional post-yield tensile capacity.

d) Steel grid reinforced soil walls are highly strength-redundant systems.

If the load and resistance factors proposed in this study are used for design, the engineer of record should ensure that the structure being designed falls within the envelope of material property parameters, wall geometry, foundation conditions, and loading matching the database from which measured load and resistance statistics have been developed.

Finally, this chapter has focused on LRFD calibration for internal stability of steel grid reinforced soil walls. Nevertheless, the general approach is applicable to other reinforced soil wall technologies provided that sufficient measured load data for walls under operational conditions is available together with resistance capacity data from physical testing.
REFERENCES


### Table 4.1 Summary of steel grid wall cases [updated from Allen et al. (2001, 2004) and Bathurst et al. (2008a)]

<table>
<thead>
<tr>
<th>Wall designation</th>
<th>Project date</th>
<th>Wall name</th>
<th>Reinforcement type</th>
<th>Number of data points(^a)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>BM1A,B</td>
<td>1981</td>
<td>Hayward wall, Section 1</td>
<td>Bar mat</td>
<td>8</td>
<td>Neely (1993)</td>
</tr>
<tr>
<td>BM2A,B</td>
<td>1981</td>
<td>Hayward wall, Section 2</td>
<td>Bar mat</td>
<td>6</td>
<td>Neely (1993)</td>
</tr>
<tr>
<td>BM3</td>
<td>1988</td>
<td>Algonquin wall (sand)</td>
<td>Bar mat</td>
<td>5</td>
<td>Christopher (1993)</td>
</tr>
<tr>
<td>BM4</td>
<td>1988</td>
<td>Algonquin wall (silt)</td>
<td>Bar mat</td>
<td>3</td>
<td>Christopher (1993)</td>
</tr>
<tr>
<td>WW1</td>
<td>1985</td>
<td>Rainier Avenue wall</td>
<td>Welded wire</td>
<td>7</td>
<td>Anderson et al. (1987)</td>
</tr>
</tbody>
</table>

Notes: \(^a\) total number of data points (instrumented layers) \(n = 42\)
<table>
<thead>
<tr>
<th>Wall designation</th>
<th>Wall height (m)</th>
<th>Surcharge condition</th>
<th>Soil friction angle&lt;sup&gt;a&lt;/sup&gt; (°)</th>
<th>Vertical spacing, $S_v$ (m)</th>
<th>Horizontal spacing of element&lt;sup&gt;b&lt;/sup&gt;, $S_h$ (m)</th>
<th>Element width, $b$ (m)</th>
<th>Coverage ratio, $R_c = b/S_h$</th>
<th>Transverse member spacing, $S_t$ (m)</th>
<th>Reinforcement stiffness&lt;sup&gt;c&lt;/sup&gt;, $J$ (MN/m)</th>
<th>Global stiffness&lt;sup&gt;d&lt;/sup&gt; $S_{global}$ (MN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BM1A,B</td>
<td>6.1</td>
<td>2:1 slope, 25 kPa</td>
<td>41</td>
<td>0.60</td>
<td>1.05</td>
<td>0.60</td>
<td>0.57</td>
<td>0.60</td>
<td>66</td>
<td>109</td>
</tr>
<tr>
<td>BM2A,B</td>
<td>4.3</td>
<td>2:1 slope, 22 kPa</td>
<td>41</td>
<td>0.60</td>
<td>1.05</td>
<td>0.60</td>
<td>0.57</td>
<td>0.60</td>
<td>66</td>
<td>108</td>
</tr>
<tr>
<td>BM3</td>
<td>6.1</td>
<td>None</td>
<td>40</td>
<td>0.75</td>
<td>1.50</td>
<td>0.46</td>
<td>3.3</td>
<td>0.60</td>
<td>38</td>
<td>48</td>
</tr>
<tr>
<td>BM4</td>
<td>6.1</td>
<td>None</td>
<td>35</td>
<td>0.75</td>
<td>1.50</td>
<td>0.46</td>
<td>3.3</td>
<td>0.60</td>
<td>39</td>
<td>48</td>
</tr>
<tr>
<td>BM5</td>
<td>18.2</td>
<td>None</td>
<td>40</td>
<td>0.75</td>
<td>1.25</td>
<td>0.45-1.05</td>
<td>2.8-0.84</td>
<td>0.35-0.61</td>
<td>57-166</td>
<td>126</td>
</tr>
<tr>
<td>WW1</td>
<td>16.8</td>
<td>0.3 m soil surcharge</td>
<td>43</td>
<td>0.45</td>
<td>0.15</td>
<td>0.15</td>
<td>1.0</td>
<td>0.23</td>
<td>39-103</td>
<td>147</td>
</tr>
<tr>
<td>WW2</td>
<td>10.1</td>
<td>None</td>
<td>38</td>
<td>0.75</td>
<td>2.00</td>
<td>1.20</td>
<td>0.60</td>
<td>0.60</td>
<td>27-85</td>
<td>85</td>
</tr>
</tbody>
</table>

Notes:  
<sup>a</sup> Measured back-fill peak triaxial or direct shear friction angle.  
<sup>b</sup> Center-to-center spacing of prefabricated group of longitudinal elements.  
<sup>c</sup> Stiffness $J$ calculated assuming a steel modulus $E = 200$ GPa. The stiffness was computed as force per unit width of wall, based on geometry and horizontal spacing of the reinforcement (i.e. $J = EA/S_h$ where $A$ is cross-sectional area of the reinforcement grid elements and $S_h$ is center-to-center horizontal spacing).  
<sup>d</sup> Global wall stiffness $S_{global} = \Sigma J/H$.  

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Table 4.3 Computed pullout and yield resistance factors ($\varphi$) for steel grid soil reinforced soil walls ($\beta = 2.33$, $P_f = 1\%$).

<table>
<thead>
<tr>
<th>Limit state</th>
<th>Load factor, $\gamma_0$</th>
<th>Load bias values</th>
<th>Resistance bias values *</th>
<th>Resistance factor, $\varphi$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Fit to all data</td>
<td>Fit to all data</td>
<td>Fit to lower tail</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Mean $\mu_Q$</td>
<td>COV$_Q$</td>
<td>Mean $\mu_R$ COV$_R$</td>
</tr>
<tr>
<td>Pullout</td>
<td>1.35</td>
<td>0.89 0.44</td>
<td>0.99 0.40</td>
<td>0.99 0.40</td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.75</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Yield</td>
<td>1.35</td>
<td>0.89 0.44</td>
<td>1.13 0.08</td>
<td>1.01 0.01</td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.75</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes: * corrected resistance values using modified model for pullout capacity (**Figure 4.4**); ** use 0.40 for design; *** use 0.65 for design.
Table 4.4 Comparison of operational factor of safety (OFS) and probability of failure ($P_f$) using proposed LRFD approach and ASD past practice.

<table>
<thead>
<tr>
<th>Limit state</th>
<th>Proposed LRFD using $\gamma_Q = 1.35$</th>
<th>LRFD using $\gamma_Q = 1.35$ and $\phi$ required to match AASHTO (2007) ASD OFS</th>
<th>ASD using AASHTO (2007)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\varphi$</td>
<td>OFS&lt;sup&gt;a&lt;/sup&gt;</td>
<td>$P_f$ (%)</td>
</tr>
<tr>
<td>Column →</td>
<td>1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Pullout</td>
<td>0.40</td>
<td>3.75</td>
<td>1</td>
</tr>
<tr>
<td>Yield</td>
<td>0.65</td>
<td>2.64</td>
<td>1</td>
</tr>
</tbody>
</table>

Notes: <sup>a</sup> OFS = $\frac{\gamma_Q}{\phi} \cdot \frac{\mu_R}{\mu_Q}$ and bias statistics for entire data sets; <sup>b</sup> $\phi = \frac{\gamma_Q}{\text{OFS}} \cdot \frac{\mu_R}{\mu_Q}$ and bias statistics for entire data sets; <sup>c</sup> OFS = FS $\cdot \frac{\mu_R}{\mu_Q}$ and bias statistics for entire data sets; * FS = $\gamma/\phi = 1.35/0.9 = 1.5$; ** FS = $\gamma/\phi = 1.35/0.65 = 2.08$; *** using unmodified load and resistance models and ignoring hidden dependencies.
Figure 4.1 Coefficient of earth pressure for calculating maximum reinforcement loads in steel grid reinforced soil walls using Simplified Method (AASHTO 2002, 2007).
Figure 4.2 Steel grid walls: (a) measured versus predicted loads; (b) load bias versus calculated load; (c) CDF plot of load bias values
Figure 4.3 Influence of load factor on probability of exceeding measured loads for steel grid walls.
**Figure 4.4** Current AASHTO (2007) and proposed default pullout resistance factor values for steel grid reinforcement materials.

Note: For proposed new model, if $P_c > 100$ kN/m then multiple $P_c$ by 0.8.
Figure 4.5 Pullout resistance data: (a) measured versus calculated pullout capacity; (b) resistance bias values versus calculated pullout capacity; (c) resistance bias values versus corrected calculated pullout capacity; (d) CDF plot for corrected bias data.

![Figure 4.5](image-url)
Figure 4.5 (continued) Pullout resistance data: (a) measured versus calculated pullout capacity; (b) resistance bias values versus calculated pullout capacity; (c) resistance bias values versus corrected calculated pullout capacity; (d) CDF plot for corrected bias data.
Figure 4.6 CDF plots for steel rupture strength bias values.
Chapter 5
Load and Resistance Factor Design (LRFD) Calibration for Steel Strip Reinforced Walls

5.1 Introduction

Load and resistance factor design (LRFD) in the USA is now mandated in AASHTO (2007) for the design of foundations and earth retaining structures. LRFD calibration uses reliability analysis to compute the values of the load and resistance factors based on available statistical data for load and resistance quantities in otherwise conventional design equations. A review of the development of reliability-based design in geotechnical applications can be found in publications by Kulhawy and Phoon (2002), Goble (1999) and Becker (1996a,b). Current load and resistance factor design (LRFD) in the USA for reinforced soil walls is based on fitting to allowable stress design (ASD) with conventional factors of safety (D’Appolonia Engineering 1999; Allen et al. 2005; Bathurst et al. 2008a). The disadvantage of calibration by fitting is that a target probability of failure for a selected limit state or set of related limit states is not assured. A strategy to overcome this problem is to carry out LRFD calibration based on measured loads from instrumented field walls and load capacity (resistance) from in-situ or standard laboratory tests on component materials.

This chapter is focused on LRFD calibration for the internal stability of steel strip reinforced soil walls using data collected by the writer and analyzed using the general approach by Allen et al. (2005) and Bathurst et al. (2008a). Data collected from a total of 14 instrumented steel strip walls are used to develop load statistics, and the results of 137 in-situ pullout tests and 65 tensile tests on steel are used to develop resistance statistics. These data are used to compute bias statistics which are used in turn to compute load factors and resistance factors for reinforcement rupture and pullout limit states. In order to avoid undesirable dependencies in the bias statistics and to improve the accuracy of the underlying deterministic models some improvements to current design equations for reinforcement load and pullout capacity are required.

5.2 General approach

The general approach adopted in this chapter follows that used for LRFD calibration for superstructure design in highway bridge design codes in the North America (Nowak 1999; Nowak and Collins 2000). The same methodology has been extended to reinforced soil walls using load and pullout data for steel grid reinforced soil walls as an example by Allen et al. (2005)
and Bathurst et al. (2008a). For completeness, basic concepts are reviewed here. In North America, the fundamental LRFD concept can be expressed as

\[ \varphi R_n \geq \sum \gamma_i Q_{ni} \]

Here, \( Q_{ni} \) = nominal (specified) load; \( R_n \) = nominal (characteristic) resistance; \( \gamma_i \) = load factor; and \( \varphi \) = resistance factor. In LRFD codes, load factor values are typically greater than or equal to one and resistance factor values less than or equal to one. Correct load and resistance factor calibration of a selected limit state must include bias statistics where bias is defined as the ratio of measured value to predicted (nominal) value. Bias statistics are influenced by model bias (i.e. intrinsic accuracy of the deterministic model representing the mechanics of the limit state under investigation), random variation in input parameter values, spatial variation, quality of data and, consistency in interpretation of data when data are gathered from multiple sources (the typical case) (Allen et al. 2005). If the underlying deterministic model used to predict load or resistance capacity is accurate and variations in model parameter values are small, then bias statistics have a mean value that is close to one and a small coefficient of variation.

Incorporating bias statistics into LRFD calibration and assuming only one load type, Equation 5.1 can be expressed as

\[ \gamma X_R \geq \varphi X_Q \]

Here, \( X_R \) = resistance bias computed as the ratio of measured resistance to calculated (predicted) resistance, and \( X_Q \) = load bias computed as the ratio of measured load to the calculated (predicted) load. For example, in this chapter the predicted values for reinforcement load and pullout capacity are computed using the deterministic models (equations) recommended by AASHTO (2007) and FHWA (2001).

If the load and resistance bias values follow log-normal distributions, the resistance factor in Equation 5.2 can be computed using the following closed-form solution
where $\mu_R$, $\mu_Q$ = mean of resistance bias value and load bias value; COV$_R$, COV$_Q$ = coefficient of variation of resistance and load bias value; and $\beta$ = reliability index. Alternatively the reliability index can be calculated using

$$\beta = \frac{\gamma \frac{\mu_R}{\mu_Q} \sqrt{\left(1+\text{COV}_Q^2\right)/\left(1+\text{COV}_R^2\right)}}{\exp \left\{ \beta \sqrt{\ln \left[ \left(1+\text{COV}_Q^2\right)/\left(1+\text{COV}_R^2\right) \right]} \right\}}$$

The reliability index is a function of probability of failure. An alternative approach that is applicable to any statistical distributions is Monte Carlo simulation to compute the reliability index (or equivalently the probability of failure). A simple procedure to carry out Monte Carlo simulation using an Excel spreadsheet is described by Allen et al. (2005). For the case of a normal distribution of resistance bias values and a log-normal distribution of load bias values, or vice versa, then closed-form solutions (e.g. Equations 5.3, 5.4) may be inaccurate, and the Monte Carlo simulation approach should be used to compute $\beta$.

The validity of Equation 5.2 and results of calculations using the methods described here are assured only if bias values are random and uncorrelated with predicted loads or resistance values. Bathurst et al. (2008a) showed that hidden dependencies exist between bias values and predicted pullout capacity values for the case of steel grid reinforced soil walls. These dependencies can be detected using statistical tests. Spearman’s rank correlation method as used by Bathurst et al. (2008a) and a zero-slope test (e.g. Draper and Smith 1981) were both used in the current study to detect undesirable dependencies. A practical outcome of the dependency analyses carried out here is that the zero-slope test was simpler to implement and the results easier to interpret. Hence, for brevity Spearman’s rank correlation results are not reported here. Where dependencies exist they can be reduced by assigning different load or resistance factors to different ranges of the independent parameter. However, this adds additional complexity to limit states design by requiring the designer to select from a range of load and (or) resistance factors for a particular
limit state design. Furthermore, this may not reduce the computed probability of failure or the magnitude of resistance factor because smaller bias value subsets can lead to higher COV values. An alternative strategy that is adopted in this chapter is to remove or reduce hidden dependencies to acceptable levels by adjusting the underlying deterministic model to improve bias statistics across the entire range of independent parameter (i.e. predicted load or resistance). If this is done, a single load or resistance factor can be used for the case of a single load type in the limit state design equations examined in the current study.

5.3 Wall database

The database of steel strip reinforced soil walls is comprised of wall sections from 14 instrumented field structures from around the world. Details of these structures have been previously reported by Allen et al. (2001, 2004) and Bathurst et al. (2008b, 2009). However, this chapter includes two new walls with data that was not available in previous studies. Measured reinforcement load data (e.g. using strain gauges) were collected from 22 different wall sections or the same wall section at different surcharge levels. The total number of data points used here is \( n = 85 \) versus the 77 data points reported by Bathurst et al. (2008a, 2009). Therefore, load bias statistics reported in this chapter are slightly different from values reported in earlier papers.

Table 5.1 summarizes the geometry and material properties for steel strip walls. The lowest wall is 3.7 m high and the tallest wall is 16.9 m high. Most of the wall heights are between 6 and 12 m. Five wall sections were surcharged with an equivalent soil height typically less than 3 m. The reinforcement loads for the other wall sections are due to self-weight of the backfill alone. All walls were constructed using granular soils with peak friction angle \( \phi > 35^\circ \). The soil friction angle for six walls was greater than 40°. However, 40° is the maximum value for the calculation of reinforcement load specified in the current AASHTO (2007) design code. Reinforcement stiffness, calculated as the stiffness of one strip element times the number of strips per unit width of wall, ranges from about 20 to 120 MN/m. Reinforcement global stiffness (Christopher et al. 1990), defined as the average reinforcement stiffness over the wall height, ranges from about 30 to 260 MN/m² but typically is between 50 and 150 MN/m². The vertical spacing of reinforcement strips is 0.75 m with the exception of two walls, where a spacing of 0.60 and 0.30 m was used. The horizontal spacing of strips ranges from 0.4 to 1.0 m. The steel strip reinforcement is typically 3 to 5 mm thick and 60 to 100 mm wide.
5.4 Load bias statistics for steel strip walls

According to the AASHTO Simplified Method (AASHTO 2002, 2007), the maximum tensile load in a reinforcement layer is calculated as:

\[ T_{\text{max}} = S_v \sigma_v K_r \]  

Here, \( T_{\text{max}} \) = maximum tensile load in the reinforcement; \( S_v \) = tributary spacing of the reinforcement layer; \( \sigma_v \) = vertical earth pressure at the reinforcement depth; and \( K_r \) = lateral earth pressure coefficient. For steel strip walls, \( K_r \) varies from \( 1.7K_a \) to \( 1.2K_a \) at the top of the wall to a depth of 6 m and remains constant thereafter (Figure 5.1). Parameter \( K_a \) is the active lateral earth pressure coefficient, which is calculated using the peak soil friction angle \( \phi \) capped at 40°. Bathurst et al. (2009) showed that there was a slight improvement in load bias statistics if the friction angle was capped at 45°. However, the improvement is very small and in this chapter the peak friction angle is restricted to \( \phi \leq 40° \).

Figure 5.2 presents the results of load data analysis for steel strip walls. Walls with smooth and ribbed steel strip reinforcement are included together. Figure 5.2a shows a plot of measured to predicted (calculated) load values using Equation 5.5. Logarithmic scales are used for clarity. The \( n = 85 \) data points fall about the 1:1 correspondence line, which suggests that the AASHTO Simplified Method is reasonably accurate. However, the mean of load bias values is \( \mu_Q = 1.06 \) with a coefficient of variation \( \text{COV}_Q = 0.48 \). The value of \( \mu_Q > 1 \) means that on average the Simplified Method slightly under-estimates the measured load values in instrumented field walls. Closer scrutiny of Figure 5.2a reveals most of the data points corresponding to calculated loads \( T_{\text{max}} \leq 10 \text{ kN/m} \) are above the 1:1 line. It appears that the Simplified Method is more likely to under-estimate reinforcement loads when the calculated load values are low.

The influence of calculated load on bias values can be seen in Figure 5.2b. The plot shows a pronounced visual dependency of bias values on calculated load across the entire range of load values. Linear regression applied to the entire data set gives a best-fit line with a slope value equal to \(-0.0129\). The 95% confidence interval for the slope of this line is bounded by values of \(-0.021\) and \(-0.005\). This range does not include zero meaning that there is an undesirable dependency between bias values and calculated load values at a level of significance of \( \alpha = 5\% \). The data points that contribute strongly to this dependency are located at \( T_{\text{max}} \leq 10 \text{ kN/m} \). There
was no evidence that these 17 data points should be excluded from the population used in this calibration exercise based on friction angle, reinforcement type or belonging to one case study wall, etc. Four of the data points at \( z \leq 1 \) m have bias values greater than two. Close inspection of the database showed that these values correspond to four different walls but all constructed with high friction angle granular soils (\( \phi > 43^\circ \)). Bathurst et al. (2009) detected the same poor load prediction at shallow depths using back-calculated values of \( K_r/K_a \) for wall cases with \( \phi > 45^\circ \).

Assuming that the source of dependency is depth, a strategy to remove the unacceptable bias dependency at lower loads is to modify the underlying deterministic model by changing the steel strip \( K_r/K_a \) design curve in Figure 5.1. Back-calculated values of \( K_r/K_a \) are plotted against depth in Figure 5.2c. The SOLVER optimization utility in Excel was used to find a third line-segment approximation to the distribution of data points at shallow depth. The endpoints from the optimization solution were adjusted to convenient values as shown in Figure 5.2c and the modified design curve is now plotted in Figure 5.1.

Using the modified \( K_r/K_a \) design curve the load bias values were recalculated and plotted against calculated \( T_{\max} \) values in Figure 5.2d. The linear regressed curve through the entire data set is visually shallower compared to the previously fitted line in Figure 5.2b. The zero-slope test at a level of significance of \( \alpha = 5\% \) gives slope values of -0.0114 and +0.0002 which bound zero; thus depth dependency is now judged to be insignificant.

The cumulative distribution function (CDF) plot for the corrected data is shown in Figure 5.2e. A log-normal distribution fitted to the entire data gives \( \mu_Q = 0.93 \) and \( \text{COV}_Q = 0.38 \). Allen et al. (2005) and Bathurst et al. (2008a) have pointed out that the CDF approximation to load bias values should be fitted to the data points located at the upper tail, as it is the overlap between the upper tail of the load data and the lower tail of the resistance data that determine the probability of failure in reliability-based design. A best fit to the upper tail is shown in Figure 5.2e. The corresponding load bias statistics are computed as \( \mu_Q = 1.00 \) and \( \text{COV}_Q = 0.20 \).

### 5.5 Load factor for steel strip walls

AASHTO (2007) recommends a load factor \( \gamma_Q = 1.35 \) due to soil self-weight (called vertical earth load factor) for internal stability of reinforced soil walls. Ideally, the same load factor should apply to all internal strength limit states. Furthermore, the combination of load factor and resistance factor for each strength limit state should give the same acceptable probability of failure. A third criterion is to select a value that results in the factored loads satisfying an
acceptable level of exceedance. However, satisfying both a target probability of failure and a target load exceedance value may not be possible particularly if the choice of load factor is also influenced by past practice. In this case, meeting a uniform target probability of failure for all related limit states in a calculation set (e.g. internal stability design) is more important than satisfying a target load exceedance value.

Load exceedance levels will vary depending on the magnitude of load factor applied to calculated loads. Nowak (1999) and Nowak and Collins (2000) reported that a load exceedance value of 3% was used as a starting point in the development of the Canadian Highway Bridge design code (CSA 2006) and AASHTO (2007) for bridge superstructures. CDF plots for load bias values are presented in Figure 5.3 using four trial load factors $\gamma_Q = 1.00, 1.35, 1.50$ and 1.65 applied to calculated loads values using Equation 5.5 and the modified $K_r/K_a$ design curve in Figure 5.1. When the load factor is taken as one (i.e. loads are unfactored) the probability of the measured load being greater than the factored calculated load is about 50%. A value of $\gamma_Q = 1.65$ gives an exceedance value of 3%. However, a value of $\gamma_Q = 1.65$ or higher results in a resistance factor for yield greater than one which is undesirable (Table 5.2). Using the AASHTO recommended value of 1.35 gives a probability of exceedance of 12%. Based on prior practice for other limit states in LRFD calibration (Nowak 1999; CSA 2006; AASHTO 2007) this load factor may be judged to be too low. However, this value is attractive because it is currently in the AASHTO code and as noted above, the most important criterion to meet is the target probability of failure which is possible with $\gamma_Q = 1.35$ as demonstrated later (Table 5.2).

5.6 Resistance bias statistics

5.6.1 Pullout capacity

According to AASHTO (2002, 2007) and FHWA (2001) the pullout capacity of an individual steel strip reinforcement can be calculated as

\[ T_p = 2F^*L_w W_c \sigma_v \]  

where $T_p$ = pullout capacity (kN); $L_w$ = anchorage length (m); $W_c$ = strip width (m); $\sigma_v$ = vertical stress (kPa); and $F^*$ = dimensionless pullout resistance factor. In the AASHTO design specifications, a coverage ratio ($R_c$) is used for the calculation of pullout capacity per unit length.
of wall (kN/m). Smooth steel strips are no longer used in North America. However, they are still included in the AASHTO (2002, 2007) and FHWA (2001) design guidance documents. Hence, the pullout limit state for walls with smooth steel strip reinforcement materials is also examined in this chapter.

The pullout resistance bias values were calculated using the default $F^*$ values specified in current design codes using the design chart reproduced in Figure 5.4. For smooth strips, $F^* = 0.4$ is specified. For ribbed strips the default $F^*$ value is a function of the peak soil friction angle ($\phi$) and coefficient of curvature ($C_u = D_{60} / D_{10}$). $F^*$ is taken as $1.2 + \log C_u$ at the wall top and varies linearly with depth to $\tan \phi$ at 6 m, remaining constant thereafter. Regardless, the value of $F^*$ is capped at 2. Default $F^*$ values apply only to granular soils meeting AASHTO requirements on particle gradation: (i) fines content ($\leq 75 \mu m$) $\leq 15\%$; (ii) plasticity index $\leq 6$; and (iii) $C_u \geq 4$.

However, a research program carried out by the Florida Department of Transportation (RECO 1995) recommended that the coefficient of curvature criterion be relaxed to $C_u \geq 2$. This criterion was adopted for ribbed strips in the current study in order to maximize the size of the pullout database.

The steel strip pullout data from in-situ tests were taken from Schlosser and Elias (1978), a technical report by RECO (1995), and Allen et al. (2001). Only bias values are available. The measured pullout loads were not reported in these sources and are no longer available. Therefore, a check on hidden dependency of bias values with calculated pullout capacity values such as that carried out on load bias statistics was not possible. However, the pullout mechanism of steel strips is relatively simple compared to extensible polymeric materials and steel grid reinforcement materials. Due to the much larger stiffness of the steel strip elements with respect to the soil, it is assumed that the shear resistance mobilized in the anchorage zone is uniform along the length of the strip (Mitchell and Schlosser 1979). An important factor influencing pullout capacity of steel strip materials is dilation of the adjacent granular soil during interface shearing (Juran et al. 1988). Since soil dilation is dominated by the magnitude of vertical stress (e.g. fill height above the strip) and the backfill is restricted to frictional materials, a test on the possible dependency of pullout bias values on height of soil above the strip (instead of calculated pullout capacity) is judged to be a satisfactory approach. This is also consistent with default pullout models that relate $F^*$ to soil depth (Figure 5.4).
5.6.2 Smooth strip reinforcement

Figure 5.5 presents pullout resistance data for smooth strips. Figure 5.5a shows the CDF plot for bias statistics. Normal and log-normal approximations are fitted to the data. A normal distribution appears to give the best visual fit over the entire data set. Figure 5.5b shows resistance bias data plotted against fill height. The mean bias $\mu_r = 2.73$ for the entire data set demonstrates that the default value $F^* = 0.4$ results in very conservative estimates of pullout capacity on average. There are very few data points less than one. Furthermore, the bias values appear to decrease with increasing fill height, at least to a fill depth of 4 to 5 m based on moving average of data points. The regressed (sloped) line for all data in Figure 5.5b confirms an undesirable dependency between resistance bias values and fill height. This is not unexpected when it is recalled that the original design curve for $F^*$ was selected for allowable stress (factor of safety) design. This classical engineering approach encourages geotechnical engineers to select design envelopes that are at or close to the lower bound of resistance data (e.g. $F^* = 0.4$ in Figure 5.5c). Unfortunately this results in poor bias statistics when calibration for LRFD is attempted.

The strategy adopted here to remove resistance bias dependency is to correct the $F^*$ design chart as illustrated in Figure 5.5c. Back-calculated $F^*$ values are plotted against fill height. The SOLVER optimization utility in Excel was used to find a bi-linear best fit to the data. The optimization solution is shown by the heavy dashed lines in the figure. A value of $F^* = 2$ at $z = 0$ and a breakpoint at $F^* = 0.85$, $z = 4$ m were judged to be sufficiently accurate and convenient. Recomputed bias values are plotted in Figure 5.5d and the dependency observed earlier (Figure 5.5b) appears to be greatly reduced. The zero-slope test shows that a zero slope falls within the 95% confidence interval (i.e. -0.077 and +0.342) for the estimate of the slope of the regressed line plotted in the figure and thus quantitatively satisfies the dependency test. The corrected pullout resistance design chart for $F^*$ is shown in Figure 5.4. The CDF plot for the corrected resistance bias values is plotted in Figure 5.5e together with a log-normal approximation fitted to the entire data set. A log-normal distribution is also fitted to the lower tail of the corrected data set. This fit is done for the same reason given in the previous section on load bias statistics. It is the overlap of the lower tail of the resistance frequency distribution function that plays a major role in the estimation of the probability of failure, and hence it is this tail region where the approximation to the CDF data should be fitted (Allen et al. 2005).
5.6.3 Ribbed strip reinforcement

Figure 5.6a presents pullout resistance data for ribbed strips. Similar to the smooth strip data, the mean of the bias values is very large ($\mu_R = 2.5$) indicating that the model is very conservative. Visual dependency of bias data with fill height can also be seen in Figure 5.6a. A tri-linear fit to the bias data can be made as illustrated in the same figure using SOLVER and imposing the constraint $F^* = 1.8$ for $z \geq 6$ m. Slight adjustments to the optimized solution were made to give convenient $F^*$ values at $z = 0$ and at the first breakpoint. The adjusted (proposed) tri-linear solution is shown in Figure 5.6a. The same quantitative improvement in bias statistics is achieved using the proposed new design resistance factor $F^*$ plot shown in Figure 5.4b. To use this chart, corrected $F^*$ values are computed using current values multiplied by the correction factor $k$ shown in the figure. The corrected bias data is plotted in Figure 5.6b. The 95% confidence interval on the slope of the regressed line (-0.043 to +0.080) now includes the zero slope indicating that the original dependency with fill height has been removed. The CDF plot for the corrected bias data is plotted in Figure 5.6c. In this case the log-normal approximation for the entire data set is judged to also provide a satisfactory fit to the lower tail.

5.6.4 Rupture (yield) strength

A total of 65 tensile test results for steel strip rupture (yield) were supplied by the manufacturer. Bias values were calculated as the ratio of measured to nominal (yield) strength ($f_y = 450$ MPa). The data are plotted as a CDF in Figure 5.7. The mean of the bias values is $\mu_R = 1.09$ based on all data. As may be expected for this manufactured steel material, the spread in bias values is very small ($\text{COV}_R = 0.06$) compared to previous pullout bias statistics. Bias statistics reported here are very similar to values for reinforcing steel reported by Nowak and Szerszen (2003). The predicted normal and log-normal distributions for the entire data set are very close and a normal distribution can be assumed to be satisfactory. For such a narrow spread in the data, potential dependency of the type investigated earlier is not a concern.

It can be noted that there are no data points with bias values less than one. This is expected since the data are from production quality control records in which steel batches not meeting the nominal specified value for tensile strength are rejected. Truncated CDF plots for tensile yield of steel bars used to reinforce concrete have been reported by Nowak and Szerszen (2003). A best fit to tail is also shown in the figure. However, the mean and COV values for both the normal and log-normal approximations are close.
5.7 Calibration results

5.7.1 Selection of probability of failure

The probability of failure for structural engineering design is generally targeted at about 1 in 5000 (Nowak and Collins 2000; Nowak 1999). The factor-of-safety approach used in foundation engineering design in the past yields a probability of failure about 1 in 1000 in general, and 1 in 100 for highly redundant systems with many elements (Withiam et al. 1998; Barker et al. 1991; Allen 2005). Reinforced soil walls are highly strength-redundant systems due to the use of multiple layers of reinforcement. If a single reinforcement layer (or strip) fails or is over-stressed, the load will be redistributed to other layers and catastrophic failure of the wall avoided. Piled foundations are similar systems where failure of one pile in a group leads to redistribution of load to the other piles. For this reason, a target probability of failure of about 1 in 100 has been recommended by Paikowsky (2004) for pile groups, and the same value has been recommended by D’Appolonia Engineering (1999), Allen et al. (2005) and Bathurst et al. (2008a) for reinforced soil walls. In this study the target probability of failure is taken as 1 in 100, which corresponds to a reliability index value $\beta = 2.33$.

5.7.2 Calculated resistance factors and probability of failure

Computed resistance factors are summarized in Table 5.2. All calculations are based on best fit to tail bias statistics using the modified (new) models for load and resistance introduced earlier. Both closed-form solutions (Equation 5.3) and Monte Carlo simulations were conducted to estimate the resistance factors needed to achieve the target probability of failure. There was negligible difference in computed values using Equation 5.3 or Monte Carlo simulation. For a load factor of $\gamma_Q = 1.35$ and pullout of ribbed steel reinforcement, a resistance factor of $\phi = 0.35$ is judged to be a reasonable choice. This value is slightly high for the case of smooth steel reinforcement but is judged to be a practical choice to keep the load and resistance factors the same for both types of materials.

In the AASHTO (2007) design code, the recommended resistance factor for pullout of steel strip reinforcement is $\phi = 0.75$, which is greater than 0.35 recommended here. The larger value was derived from fitting to ASD using $FS = 1.82$ (FHWA 2001) (i.e. $FS = \gamma_Q / \phi = 1.35/0.75 = 1.8 \approx 1.82$), using the current AASHTO load and pullout design models. However, as shown later, $\phi = 0.75$ does not correspond to a target probability of failure of 1%. Recall also that the underlying deterministic models for load and pullout resistance have been improved. In particular the under-
estimation of pullout capacity by factors of 2.7 and 2.5 on average has been corrected to give mean of bias statistics closer to one.

For the yield limit state, a resistance factor of 0.90 is selected (i.e. within ±0.05 of computed value of 0.89 in Table 5.2 according to AASHTO (2007) practice). The load factor $\gamma_Q = 1.35$ and resistance factor $\phi = 0.90$ have the added benefit of being the same values currently recommended in the AASHTO (2007) design code and correspond to a factor of safety consistent with ASD past practice (i.e. $FS = \gamma_Q / \phi = 1.35/0.9 = 1.5$). However, it is important to note that this outcome is only possible using the corrected load model for $T_{max}$ (i.e. Equation 5.5 and $K_r$ using new design curve for $K_r/K_a$ in Figure 5.1).

The computed probability of failure using the load and resistance factors recommended here and the best fit to tail bias statistics reported earlier give a consistent reliability index value of $\beta = 2.33$ for all limit states (using Equation 5.4 or Monte Carlo simulation); this value matches the target probability of failure $P_f = 1\%$ (Column 3 in Table 5.3).

To examine the influence of the proposed load and resistance factors on performance, the operational factor of safety (OFS) can be computed for each limit state using ASD past practice and the LRFD calibrations provided herein. The OFS is defined as the ratio of the measured resistance ($R_m$) to measured load ($Q_m$) and is related to the predicted (design) factor of safety ($FS = R_n/Q_n = \gamma_Q/\phi$) as follows:

$$[5.7] \quad OFS = \frac{R_m}{Q_m} = \frac{R_n}{Q_n} \frac{\mu_R}{\mu_Q} = FS \frac{\mu_R}{\mu_Q} = \frac{\gamma_Q}{\phi} \frac{\mu_R}{\mu_Q}$$

Column 7 in Table 5.3 shows factors of safety used in ASD past practice. The results of calculations using Equation 5.7 are summarized in Table 5.3.

The table shows that operational (actual) factors of safety for pullout using ASD (column 8) are larger than design values (column 7). This is consistence with the opinion of experienced design engineers that current practice is conservatively safe (e.g. pullout capacity is under-estimated and reinforcement load is over-estimated).

The table shows that the actual in-service pullout factors of safety for the walls in the database are greater using the proposed LRFD load and resistance factors and the revised load and pullout
models than operational factors of safety using ASD (compare column 2 to column 8 for pullout). However, for the yield limit state all calculated operational factors of safety are very much closer to the specified FS = 1.82 used in ASD. Nevertheless, the ratio of OFS for ASD (1.87) and proposed LRFD method (1.76) can be interpreted to mean that ASD practice requires 6% more strip reinforcement cross-section than using the LRFD approach proposed here (i.e. (1.87-1.76)/1.87 = 0.06).

Column 9 in Table 5.3 shows that the actual probabilities of failure using ASD past practice are greater than the recommended target value of 1% and vary from 2% to 5%.

An alternative appreciation of the link between past practice and selection of resistance factor can be referenced to the data in the middle of Table 5.3. The values of resistance factor shown in Column 4 are computed based on the estimated operational factor of safety for past practice (Column 5 = Column 8) and the prescribed load factor of 1.35. In other words, if the objective is to select resistance factors to match operational factors of safety for past practice with $\gamma_Q$ fixed at 1.35, then the resistance factor values shown in Column 4 are required. However, the penalty is that there are small differences in the probability of failure for each limit state.

In contrast, for the yield limit state the ASD factor of safety for design and actual factors of safety using ASD and LRFD are much closer. An explanation for this is that the resistance yield model for steel is accurate with only a small spread in bias values, and this leads to more consistent estimates for factor of safety using ASD and LRFD. This also demonstrates that the result of LRFD calibration for this limit state is consistent with ASD past practice.

Finally it should be noted that loss of section due to corrosion has not been included in the calibrations reported here. Sacrificial thicknesses of steel reinforcement can be computed using recommendations reported in AASHTO (2007) and FHWA (2001).

5.8 Conclusions and practical implications

This chapter reports the results of LRFD calibration for pullout and yield limit states for reinforced soil walls that use steel strips. Calibration is limited to the case of a single load contribution due to soil self-weight. The approach uses a database of measured reinforcement loads from field-instrumented walls and in-situ pullout tests. An important feature of the calibration method is the use of bias statistics to account for prediction accuracy of the underlying deterministic models for reinforcement load, pullout capacity and yield strength of the steel strips and variability in input parameter values. In order to ensure sensible values for the magnitude of
load factor (i.e. greater to or equal to one) and resistance factors (i.e. less than or equal to one) adjustments to current deterministic models for reinforcement load and pullout are unavoidable. This chapter proposes a small adjustment to the calculation of reinforcement loads and two new revised charts for the estimation of the pullout resistance factor \( F^* \) for smooth and ribbed steel strips. The results of calibration lead to load and resistance factors that give a probability of failure not exceeding 1% for the three limit states cases considered. Furthermore, comparison with allowable stress design (ASD) past practice shows that the operational factors of safety using the new LRFD-based approach give the same or higher factors of safety and lower probabilities of failure.

From a practical point of view, it can be argued that design practice using ASD, current LRFD and LRFD with the proposed new load and resistance models, all give similar probabilities of failure. The range of computed probability of failures in Table 5.3 varies only from 1% to 5%. However there are advantages to using the new LRFD approach developed in this chapter, for example:

a) Hidden load and resistance capacity dependencies for the pullout limit state are reduced. The practical result is that there is less chance of a reinforcement layer being under-designed or over-designed.

b) The amount of cross-section steel required to satisfy the yield limit state is about 6% less than that used currently.

As a further check on the practical implications of using the proposed new LRFD approach versus ASD past practice, the lengths of reinforcement actually required using both design methods were compared for each of the walls in the database. The results of these calculations showed that when the calculated pullout length using LRFD was greater than the length using ASD, the required length was controlled by the current AASHTO (2007) criterion that reinforcement length must be at least 70% of the wall height (i.e. \( L/H \geq 0.7 \)). At depths \( z \leq 3 \text{ m} \), where experience has shown that pullout will typically control reinforcement length, there was less than 3% difference in total reinforcement length using both methods. An explanation for this close agreement is that while the new load model results in greater reinforcement load at shallow depths (see Figure 5.1), there is a compensating increase in pullout capacity using the new pullout capacity models (see Figures 5.4a,b).
The operational probabilities of failure reported as 1% to 5% appear large. This means that 1 in 20 to 1 in 25 reinforcement strips can be expected to fail depending on the limit state and design method. However, there is no evidence of over-stressing or pullout failure in the database of case studies used in this investigation. In fact, all walls performed well. Furthermore, there is a long history of excellent performance of these systems in the field. The apparent contradiction can be explained by the following observations: (1) the as-built pullout lengths of almost all reinforcement layers in the wall database were greater than lengths required to satisfy ASD past practice with a factor of safety of 1.5, and; (2) all calculated tensile loads were less than the design tensile capacity using ASD with a factor of safety of 1.82.

If the new LRFD approach is used in practice there may be additional sources of safety that are not accounted for quantitatively in the limit states investigated. For example, design engineers typically select a uniform reinforcement strip cross-sectional area from stock materials that must be greater than the most critically loaded reinforcement layer. Secondly, the reinforcement elements have additional post-yield tensile capacity and finally, the systems are highly strength redundant.

One final discussion point for the design of steel strip reinforced soil walls follows from the observation by Bathurst et al. (2009) that elevated tensile loads may develop in layers of steel strip reinforcement within (say) 1 m of the top of the wall when granular soils with \( \phi \geq 45^\circ \) are used in construction. They argued that these additional loads are compaction-induced and not expected to propagate to the anchorage. Thus these loads should not be used in pullout capacity calculations. However, a distinction between loads used for tensile capacity and pullout capacity design is not necessary using the LRFD approach reported here; while predicted loads are larger at the near-surface than current practice (about a factor of two – Figure 5.1), the pullout capacity at the near-surface is about 2 to 7 times greater (Figure 5.4). Hence, potentially excessive lengths of reinforcement that may result for shallow reinforcement depths for high friction angle soils using back-calculated values of \( K_r \) for \( \phi \geq 45^\circ \) reported by Bathurst et al. (2009) will not occur using the new LRFD method together with new load and pullout models.

REFERENCES


Table 5.1 Summary of wall geometry and material properties for steel strip walls [updated from Allen et al. (2001, 2004) and Bathurst et al. (2009)].

<table>
<thead>
<tr>
<th>Wall designation</th>
<th>Wall height, H (m)</th>
<th>Reinforcement type</th>
<th># of data points</th>
<th>Soil friction anglea (deg.)</th>
<th>Vertical spacing, Sv (m)</th>
<th>Horizontal spacing, Sh (m)</th>
<th>Strip thickness (mm)</th>
<th>Strip width (mm)</th>
<th>Reinforcement stiffnessb, J (MN/m)</th>
<th>Global stiffnessc, Sglobal (MN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS1</td>
<td>6.0</td>
<td>Smooth</td>
<td>6</td>
<td></td>
<td>44</td>
<td>0.75</td>
<td>0.50</td>
<td>15.0</td>
<td>80</td>
<td>48</td>
</tr>
<tr>
<td>SS2</td>
<td>6.1</td>
<td>Smooth</td>
<td>5</td>
<td></td>
<td>38</td>
<td>0.75</td>
<td>0.75</td>
<td>3.0</td>
<td>80</td>
<td>63</td>
</tr>
<tr>
<td>SS3A,B,C,D</td>
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<td>Smooth</td>
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<td></td>
<td>36</td>
<td>0.60</td>
<td>0.75</td>
<td>0.8</td>
<td>102</td>
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<td></td>
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<td>0.75</td>
<td>5.1</td>
<td>60</td>
<td>79</td>
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<td>SS5</td>
<td>8.2</td>
<td>Ribbed</td>
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<td>56</td>
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<td>53-105</td>
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<td>SS6A,B</td>
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<td>Smooth</td>
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<td></td>
<td>48</td>
<td>0.30</td>
<td>0.90</td>
<td>5.1</td>
<td>75</td>
<td>83</td>
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<tr>
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<td>12.0</td>
<td>Smooth</td>
<td>7</td>
<td></td>
<td>36</td>
<td>0.75</td>
<td>0.75</td>
<td>3.3</td>
<td>100</td>
<td>80-121</td>
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<tr>
<td>SS10</td>
<td>12.6</td>
<td>Ribbed</td>
<td>5</td>
<td></td>
<td>50</td>
<td>0.75</td>
<td>0.50-0.75</td>
<td>5.1</td>
<td>60</td>
<td>79-118</td>
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<tr>
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<td>Ribbed</td>
<td>4</td>
<td></td>
<td>40</td>
<td>0.75</td>
<td>0.75</td>
<td>4.1</td>
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<td>55</td>
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<td>0.75</td>
<td>0.75</td>
<td>5.1</td>
<td>40</td>
<td>53</td>
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<tr>
<td>SS13</td>
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<td>0.38-0.75</td>
<td>5.1</td>
<td>60</td>
<td>79-118</td>
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<td>4</td>
<td></td>
<td>37</td>
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<td>0.38-0.75</td>
<td>5.1</td>
<td>60</td>
<td>79-118</td>
</tr>
<tr>
<td>SS15</td>
<td>16.9</td>
<td>Ribbed</td>
<td>5</td>
<td></td>
<td>38</td>
<td>0.75</td>
<td>0.50-1.00</td>
<td>4.1</td>
<td>50</td>
<td>38-105</td>
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<td>SS17e</td>
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<td>Smooth</td>
<td>3</td>
<td></td>
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<td>0.75</td>
<td>1.5</td>
<td>80</td>
<td>31</td>
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<tr>
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<td>Smooth</td>
<td>5</td>
<td></td>
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<td>1.00</td>
<td>3.0</td>
<td>60-80</td>
<td>35-47</td>
</tr>
</tbody>
</table>
Table 5.1 (continued) Summary of wall geometry and material properties for steel strip walls [updated from Allen et al. (2001, 2004) and Bathurst et al. (2009)].

Notes:  

- a measured back-fill peak triaxial or direct shear friction angle.  
- b Stiffness J calculated assuming a steel modulus E = 200 GPa, with the exception of SS3 which used the value reported in the original reference. The stiffness was computed as force per unit width of wall, based on geometry and horizontal spacing of the reinforcement (i.e. J = EA/S_h where A is cross-sectional area of the reinforcement strips and S_h is center-to-center horizontal spacing).  
- c Global wall stiffness S_{global} = \sum J/H.  
Table 5.2 Computed pullout and yield resistance factors ($\varphi$) for steel strip soil reinforced soil walls ($\beta = 2.33$, $P_r = 1\%$).

<table>
<thead>
<tr>
<th>Limit state</th>
<th>Load factor, $Q_O$</th>
<th>Load bias values *</th>
<th>Resistance bias values *</th>
<th>Resistance factor, $\varphi$ using best fit to tails</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Fit to all data</td>
<td>Fit to upper tail</td>
<td>Fit to all data</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Mean $\mu_Q$</td>
<td>COV$_Q$</td>
<td>Mean $\mu_Q$</td>
</tr>
<tr>
<td>Pullout smooth strips</td>
<td>1.35</td>
<td>1.00</td>
<td>0.20</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td>0.93</td>
<td>0.38</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>1.65</td>
<td>1.00</td>
<td>0.20</td>
<td>1.09</td>
</tr>
<tr>
<td>Pullout ribbed strips</td>
<td>1.35</td>
<td>1.00</td>
<td>0.20</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td>0.93</td>
<td>0.38</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>1.65</td>
<td>1.00</td>
<td>0.20</td>
<td>1.09</td>
</tr>
</tbody>
</table>

Notes: * using modified (new) models for load and pullout capacity; ** use 0.35 for design; *** use 0.90 for design.
Table 5.3 Comparison of operational factor of safety (OFS) and probability of failure ($P_f$) using LRFD approach and ASD past practice.

<table>
<thead>
<tr>
<th>Limit state</th>
<th>Proposed LRFD using $\gamma_Q = 1.35$</th>
<th>LRFD using $\gamma_Q = 1.35$ and $\phi$ required to match AASHTO (2007)</th>
<th>ASD OFS using AASHTO (2007)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\phi$</td>
<td>OFS</td>
<td>$P_f$ (%)</td>
</tr>
<tr>
<td>Column →</td>
<td>1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Pullout smooth strips</td>
<td>0.35</td>
<td>4.15</td>
<td>1</td>
</tr>
<tr>
<td>Pullout ribbed strips</td>
<td>0.35</td>
<td>4.06</td>
<td>1</td>
</tr>
<tr>
<td>Yield</td>
<td>0.90</td>
<td>1.76</td>
<td>1</td>
</tr>
</tbody>
</table>

Notes: Columns 1 through 6 represent new LRFD approach; $^a$ OFS = $\frac{\gamma_Q \cdot \mu_R}{\phi \cdot \mu_Q}$ and bias statistics for entire data sets; $^b$ OFS = $\frac{\gamma_Q \cdot \mu_R}{\mu_Q}$ and bias statistics for entire data sets; $^c$ OFS = FS, $\frac{\mu_R}{\mu_Q}$ and bias statistics for entire data sets; $^*$ FS = $\gamma/\phi = 1.35/0.9 = 1.5$; $^**$ FS = $\gamma/\phi = 1.35/0.75 = 1.80 \approx 1.82$; $^***$ using current load and resistance models (AASHTO 2007) and ignoring hidden dependencies.
Figure 5.1 Current AASHTO (2002, 2007) and Modified coefficient of earth pressure for calculating maximum reinforcement loads in steel strip reinforced soil walls using the Simplified Method.
Figure 5.2 Steel strip walls: (a) measured versus predicted loads; (b) load bias versus calculated load; (c) Measured $K_r/K_a$ versus fill height (depth); (d) corrected load bias versus calculated load; (e) CDF plots of corrected load bias values.
Figure 5.3 Influence of load factor on probability of exceeding measured loads for steel strip walls.
Figure 5.4 Current and proposed default pullout resistance factor values for: (a) smooth steel strips; (b) ribbed steel strips

(a) Smooth steel strips
Figure 5.4 (continued) Current and proposed default pullout resistance factor values for:
(a) smooth steel strips; (b) ribbed steel strips

(b) Ribbed steel strips
Figure 5.5 Pullout resistance of smooth steel strips: (a) CDF plot for uncorrected bias values; (b) uncorrected bias values versus fill height (depth); (c) corrected $F^*$ versus fill height (depth); (d) corrected bias values versus fill height (depth); (e) CDF plots for corrected bias data.
Figure 5.5 (continued) Pullout resistance of smooth steel strips: (a) CDF plot for uncorrected bias values; (b) uncorrected bias values versus fill height (depth); (c) corrected $F^*$ versus fill height (depth); (d) corrected bias values versus fill height (depth); (e) CDF plots for corrected bias data.
Figure 5.5 (continued) Pullout resistance of smooth steel strips: (a) CDF plot for uncorrected bias values; (b) uncorrected bias values versus fill height (depth); (c) corrected F* versus fill height (depth); (d) corrected bias values versus fill height (depth); (e) CDF plots for corrected bias data
Figure 5.6 Pullout resistance of ribbed steel strips: (a) uncorrected resistance bias values versus fill height (depth); (b) corrected bias values versus fill height (depth); (c) CDF plots for corrected bias values

\[ \mu_R = 2.50 \]

\[ \text{bias} = 0.0184 z + 0.8808 \]
Figure 5.6 (continued) Pullout resistance of ribbed steel strips: (a) uncorrected resistance bias values versus fill height (depth); (b) corrected bias values versus fill height (depth); (c) CDF plots for corrected bias values
Figure 5.7 CDF plots for steel strip yield strength bias values.
Chapter 6
Evaluation of Soil-geogrid Pullout Models Using a Statistical Approach

6.1 Introduction

Pullout capacity of reinforcement layers in the anchorage zone of geosynthetic reinforced soil walls, slopes and embankments is required by design codes for stability analyses (e.g. AASHTO 2007; BS8006 1995; CFEM 2006; NCMA 1997; PWRC 2000; FHWA 2001; Geoguide 6 2002). In the USA, ASTM D 6706-01 (2001) is used to quantify pullout capacity in the laboratory. Similar test methods are described in standards in Europe (EN 13738 2004) and Asia (e.g. JGS 2009). Juran et al. (1988) reported a synthesis of experimental pullout data available at that time. Since then a large body of literature on the influence of test methodology on the results and interpretation of pullout testing is available including important contributions by Farrag et al. (1993), Palmeira and Milligan (1989), Wilson-Fahmy et al. (1994), Palmeira (2004), Alfaro and Pathak (2005), Moraci and Gioffre (2006), Moraci and Recalcati (2006) and Sieira et al. (2009). In all of these previous investigations the focus has been on understanding mechanisms of soil-structure interaction, development of pullout models of varying complexity and refinement of test methodology to ensure a consistent and reliable estimate of pullout capacity. Typically the number of tests performed has been limited and often restricted to one class of geosynthetic product or one manufacturer product line. To the best of the writer’s knowledge there has not been a systematic investigation of the accuracy of the most commonly used pullout capacity models (FHWA (2001) and variants) against results of tests encompassing a wide range of products tested in general conformity with the ASTM D 6706-01 test protocol. The writer collected 478 individual results from pullout tests from many sources. These data were first used to investigate the accuracy of the FHWA pullout model based on two conventional interpretations of test data from individual test series. Next, predicted pullout capacities using three different deterministic models with presumptive coefficient values were compared to measured pullout capacity values in the database. The first of these models expresses pullout capacity as a linear function of normal stress together with default coefficient values recommended in FHWA (2001), the second is a bi-linear stress dependent model, and the third equates pullout capacity to a power function of the product of normal stress, anchorage length and peak soil friction coefficient. The accuracy of the methods was quantified using statistics for the ratio of measured to predicted loads (bias). The linear, bi-linear and non-linear models are shown
to give statistically more accurate estimates of pullout capacity in this order. An additional benefit of the large database in this chapter is that recommendations can be made to estimate the reinforcement length for a particular soil-geogrid combination to increase the likelihood that a pullout mode of failure will occur in a test carried out in conformity with ASTM D 6706-01.

### 6.2 Current simple pullout capacity models

Current design codes (AASHTO 2007; BS8006 1995; CFEM 2006; NCMA 1997; FHWA 2001; Geoguide 6 2002) recommend a pullout model for sheet geosynthetics (geotextiles and geogrids) that has the general form:

\[
P_c = 2\sigma_v L_e \Psi \tan \phi_s
\]

where \( P_c \) = pullout capacity (ultimate limit state), \( \sigma_v \) = normal stress at elevation of the reinforcement layer, \( L_e \) = anchorage length, \( \phi_s \) = peak friction angle of the soil, and \( \Psi \) = dimensionless soil-geosynthetic interaction coefficient (efficiency factor). In the simplest models, \( \Psi \) is expressed as:

\[
\Psi = \frac{\tan \phi_{sg}}{\tan \phi_s}
\]

where \( \phi_{sg} \) is interpreted as the peak soil-geosynthetic interface friction angle. This model assumes that there is a linear relationship between pullout capacity and normal stress. However, experimental studies have demonstrated that the influence of normal stress on pullout capacity is non-linear (Juran et al. 1988; Farrag et al. 1993; Oostveen et al. 1994; Lopes and Ladeira 1996). For example, Juran et al. (1988) showed that the efficiency factor (\( \Psi \)) increased with decreasing normal stress level and exceeded one at low confining pressures. This leads to the counter-intuitive result of the soil-geosynthetic interface friction angle \( \phi_{sg} \) (Equation 6.2) being greater than the peak friction angle of soil. Juran et al. (1988) explained that this phenomenon is related to restrained dilation of dense granular soils during shear. They further pointed out that this dilation was highly dependent on confining pressure. For example, the dilation effect tends to diminish with increasing confining pressure. This may be accepted as the mechanistic reason why
the current FHWA default pullout model gives better predictions at normal stress levels greater than (say) 40 kPa, as shown later in this chapter.

In the FHWA (2001) guidance document Equation 6.1 is expressed as:

\[ P_c = 2F^*\alpha \sigma \nu L_c \]  

Here, independent values for dimensionless parameters \( F^* \) and \( \alpha \) are back-fitted to pullout test results to account for interface shear strength and geosynthetic extensibility, respectively. The general approach can be traced back to the work of Christopher (1993). However, in practice back-fitting is difficult to do and there was no evidence in any test reports available to the writer that back-fitting was attempted to isolate the values of \( F^* \) and \( \alpha \). Rather, \( F^*\alpha \) (or \( \Psi \tan \phi_s \)) was assumed as a single constant value for each test or as a mean value from a set of tests for one soil-geogrid combination, with constant specimen length and carried out under different confining pressures. In the FHWA document, the following default values are recommended: \( \alpha = 0.8 \) for geogrids and \( \alpha = 0.6 \) for geotextiles, and:

\[ 2F^* = \frac{2}{3} \tan \phi_s \]

### 6.3 Database of pullout tests

A summary of the pullout test database is given in Table 6.1. The data were found in published conference and journal papers (10), publically available technical data reports from manufacturers (3) and, unpublished reports (5) from 18 different testing laboratories. The data points are categorized according to material type: high density polyethylene (HDPE) punched and drawn uniaxial geogrid (192), polypropylene (PP) punched and drawn biaxial geogrid (60), and woven (or knitted) polyester (PET) geogrid (212). The total number of tests reviewed for this investigation was 478. This includes a total of 14 high quality geotextile tests. However, 14 data points was judged to be too small to draw statistically significant conclusions; hence, this chapter is restricted to analysis of geogrid pullout tests. The ranges of property values in each geogrid category are summarized in Table 6.2. Soil properties are listed in Table 6.3. The majority of
tests were performed with granular soils. However, approximately 25% of tests used silty sand and 2% of tests used sandy silt.

The data used in pullout calculations were restricted to sources that included load-displacement plots recorded by extensometer points (tell-tales) mounted on the test specimens. This allowed the writer to discriminate between tests in which pullout was detected (movement of the free end of the specimen at peak load) from tests in which peak load capacity was due to reinforcement rupture close to the front inside of the test box or along the in-air length between the loading clamp and test box.

Most of the data correspond to tests in which the pullout boxes complied with recommendations found in the ASTM D 6706-01 test protocol. For example the dimensions of the box, side clearance, sleeve length, soil coverage depth, surcharge loading system and rate of loading were judged to be consistent with or very close to ASTM recommendations. Where there were exceptions (a total of 48 tests) these tests were carefully reviewed to assess if the pullout data were statistically different from other tests in the same category. If not, they were included in the database to maximize the sample size for statistical analyses. For example, a statistically significant effect due to rate of loading (i.e. deviation from the 1 mm/min value recommended by ASTM), or tests with or without a sleeve could not be detected. This does not mean that these effects were not detected by the authors of published research papers based on the data in their papers; rather, it means that for a data subset taken from multiple sources with the same test conditions there was no statistically significant difference from the entire data in the same category when the measured pullout loads are compared against predicted results using the pullout models under investigation in this chapter. Tests carried out with a rigid surcharging system were disregarded because this loading arrangement is not recommended in the ASTM test standard. Palmeira and Milligan (1989) explained that rigid vertical loading platens introduce a boundary condition which further restrains soil dilatancy and thus leads to an over-estimation of pullout capacity. Furthermore, the writer found large inconsistencies in pullout capacity values reported between laboratories that used a rigid platen while other test conditions were nominally similar. Not included in the pullout database are 18 geogrid specimens that were modified to examine the influence of removing selected transverse members on pullout capacity. In summary, a total of $n = 318$ geogrid tests were identified by the writer to have failed in pullout using test equipment and methodologies that were similar enough not to distort the results of statistical analyses.
Figure 6.1 shows the cumulative distributions for normal stress and pullout load for all pullout tests in the database regardless of geosynthetic type. Figure 6.1a shows that 65% of the tests were carried out under a normal stress of 40 kPa or less, and 97% of tests were carried out at 100 kPa or less. The highest confining pressure was 192 kPa. The cumulative distribution plot in Figure 6.1b shows that about 50% of the measured pullout loads were less than 30 kN/m and almost all less than 100 kN/m.

### 6.4 Description of models and evaluation

#### 6.4.1 General

A total of five models were examined in this investigation (Table 6.4). Models 1, 2 and 3 are based on Equation 6.1 with coefficient values back-calculated using measured test data from individual test series (Models 1 and 2) or default coefficient values prescribed by the FHWA (Model 3). Models 4 and 5 are new models that express pullout capacity as bi-linear and non-linear functions of normal stress, anchorage length and soil friction coefficient, respectively. The first two models require pullout tests to be performed. Models 3, 4 and 5 are based on presumptive values for coefficient terms. The presumptive values used in Models 4 and 5 (introduced below) are back-fitted from the large database of pullout tests collected by the writer.

Model bias statistics are used to quantify the accuracy of the five models. Bias values are computed as the ratio of measured to predicted pullout capacity ($P_m/P_c$). A good prediction model yields a bias mean close to one and a small coefficient of variation (COV) value, although the choice of acceptable COV value is subjective. In the limit of a perfect model, the mean of bias values is one and the COV value is equal to zero. The latter is an exceptional case in geotechnical engineering practice. Also used in the quantification of model utility in this chapter is calculation of the Spearman rank correlation coefficient $\rho$. This parameter is a quantitative measure of the strength of a monotonic relationship between two data sets regardless of whether or not the relationship is linear (Bathurst et al. 2008a). In this chapter it is used to investigate the strength of hidden dependencies between $F*/\alpha$ and normal stress ratio $\sigma_v/\sigma_n$ (where $\sigma_n$ is the average normal stress in a test series), and bias values computed as $P_m/P_c$ and $P_c$. Spearman $\rho$ has the range $-1$ to $+1$. A value of zero represents no linear dependency between data sets which is the ideal best case. Conversely, a large $\rho$ value means that the model formulation does not uniquely capture the influence of the independent parameter(s) used in the dependency calculation. The magnitude of Spearman $\rho$ is used in this chapter as a relative measure of model reliability for the
five pullout models investigated. When comparing any two models, the model with the lower magnitude $\rho$ is better.

6.4.2 Model 1: Using average measured $F^*\alpha$ (or $\Psi \tan \phi_s$) value from all tests in a series

Common practice is to back-calculate a single value for $F^*\alpha$ based on the average of multiple tests carried out on the same soil with a single geogrid product and reinforcement length under different normal stress (one test series). The range of normal stress is typically restricted to values that allow the pullout failure mechanism to develop. In the database there were typically 3 to 4 tests in a series and only those tests that resulted in pullout were used. Figure 6.2a shows measured ($P_m$) versus calculated ($P_c$) pullout load values using Model 1 (Equations 6.1 and 6.2) for all data, and Figures 6.2b,c,d for each of the three categories of geogrid. For clarity, logarithmic axes have been used. In each plot, the data fall closely on the 1:1 correspondence line and with little spread (compared to similar plots presented later). The percentage values in the plot show that the data are sensibly evenly split above and below the 1:1 correspondence line. The mean of the bias values is computed as 1.00 and the COV value is 0.21 (Table 6.4). COV values for parsed data sets based on geosynthetic type are similar. However, the good fit is misleading because there is a hidden dependency between bias values and normal stress for all data sets, as shown in Figure 6.3. Here, normal (vertical) stresses ($\sigma_v$) on the horizontal axis have been normalized using the average normal stress ($\bar{\sigma}_v$) for a series of tests. The vertical axis is the back-calculated value of $F^*\alpha$ from an individual test divided by the mean value from all tests in the series. The percentage values in each figure correspond to the fraction of data set values falling in each quadrant. For clarity the axes have been terminated at 2.5 to focus attention on about 80% of the data. The data show a clear visual dependency between back-calculated values of $F^*\alpha$ and stress level. A regressed linear curve has been superimposed on the data as a trend line. The Spearman rank correlation coefficient is large (i.e. $\rho = -0.58$). Hence, on a test-by-test basis the model can over-predict or under-predict pullout capacity depending on whether or not the confining pressure is greater or less than the average normal stress used in the test series. This observation is not surprising since other researchers have noted that parameter $\Psi$ is dependent on normal stress magnitude (e.g. Juran et al. 1988). However, in previous work this observation has been made with respect to a small number of tests. The plots in Figure 6.3 demonstrate that coefficient term $F^*\alpha$ (or $\Psi \tan \phi_s$) is stress-level dependent and this dependency is applicable to all three geogrid categories. Unfortunately, possible dependencies with $\tan \phi_s$ or
L are not possible to determine because the magnitude of these parameters was kept constant in each pullout test series.

6.4.3 Model 2: Using first-order approximation to measured $F^\alpha$ (or $\Psi \tan \phi_s$) values from all tests in a series

An alternative approach to Model 1 is to fit a first-order approximation to data plotted as $F^\alpha$ versus normal stress ($\sigma_v$) data. This may be expected to better capture stress-level dependency of pullout capacity. Model 2 reduces to Model 1 if the fitted straight line is horizontal. This is only possible if the range of normal stress is narrow and (or) the magnitude of normal stress is very high. In general, these plots should show a negative slope, i.e. decreasing $F^\alpha$ with increasing normal stress. Figure 6.4 shows measured versus predicted pullout loads using the Model 2 approach. The bias statistics are improved and the visual impression is that the data points fall closer to the 1:1 correspondence line. Quantitatively, the mean of bias values remains one but the COV value is lower in every case (for example 0.13 for all data versus 0.21 for Model 1). Figure 6.5 shows $F^\alpha$ bias values plotted against normalized stress. The data fall evenly about the horizontal line drawn at $F^\alpha$ bias equal to one indicating that the previous undesirable dependency has been removed. The visual impression that the model bias is normal stress independent is confirmed by computed Spearman $\rho = 0.02$, which is very close to zero (Figure 6.5 and Table 6.4).

6.4.4 Model 3: FHWA (AASHTO) approach with default coefficients

Figure 6.6 shows plots of measured versus predicted pullout load using the FHWA method (Equation 6.3) with default values for $F^*$ and $\alpha$. For uniaxial HDPE, biaxial PP and woven PET geogrids, the plots show that 86%, 100% and 93% of the measured values are greater than the predicted values, respectively (i.e. bias values are greater than one). The corresponding calculated mean and COV of the bias values are shown on the figures. The overall bias mean is about 2.23 and the COV value is 0.55. Hence, on average the FHWA pullout model using default values under-predicts measured values by a factor greater than two. The bias mean becomes 1.20 by setting $F^* = \tan \phi_s$ and $\alpha = 1$. These are the maximum possible default values to preserve the original framework of the FHWA method. Similar plots to those shown in Figure 6.6 can be presented using these revised default coefficient terms; for brevity this is not done. However, while there is an improvement in the mean bias value, the recomputed COV of bias values remains 0.55 (Table 6.4).
In conventional engineering practice using a global factor of safety approach, under-estimation of pullout capacity is interpreted as conservative (safe for design). However, if the objective is to develop an accurate deterministic model for design and analysis (using analytical methods) this is a poor result. Furthermore, the use of this pullout model for load and resistance factor (limit states) design calibration (e.g. Allen et al. 2005) introduces complications if there are hidden dependencies between model bias and predicted pullout load data (e.g. Bathurst et al. 2008a). To illustrate this point see the straight lines superimposed on Figure 6.6 plots. Each line is the same power function fitted to the data in Figure 6.6a and having the general form:

\[ P_m = \beta (P_c)^{1+\kappa} \]  

This regressed function with \( \beta = 5.51 \) and \( 1+\kappa = 0.629 \) gives a visually better fit to the plotted data than the 1:1 correspondence lines. The constant coefficients have values that will change depending on the units used to perform the regression. If \( P_m \) and \( P_c \) are in units of lb/ft the coefficient value \( \beta = 26.47 \) and \( 1+\kappa \) remains unchanged. Alternatively, the regression can be carried out using normalized values for \( P_m \) and \( P_c \) (e.g. multiplying each term by \( \gamma_w/p_a^2 \) where \( \gamma_w \) is the unit weight of water and \( p_a \) is atmospheric pressure). While this method leads to a dimensionless formulation, it introduces unnecessary complexity for the purposes of this chapter. Regardless, the choice of function follows from analysis of bias value dependency described next.

Figure 6.7 shows bias values computed as the ratio of measured load to predicted (calculated) load \((P_m/P_c)\) plotted against predicted values. The data show that for values of \( P_c \) less than (say) 40 kN/m, there is a pronounced predicted load dependency consistent with the excessive conservatism noted above. The Spearman rank correlation coefficient is very large (\( \rho = -0.64 \)). At greater predicted load levels the bias value becomes essentially one indicating that dependency disappears at higher load levels. An explanation for this phenomenon has been given earlier in this chapter. However, in pullout stability analysis of geosynthetic reinforced soil walls it is typically the top layers of the wall that have the least capacity and it is in this region that the current FHWA model with default values performs poorly. Superimposed on the data in Figure 6.7a is a power function having the general form:

\[ \frac{P_m}{P_c} = \beta (P_c)^k \]
The solid curve is fitted to all the data (Figure 6.7a) using $\beta = 5.51$ and $\kappa = -0.371$ computed using the data in Figure 6.6a. The dashed line in each of Figure 6.7b,c,d has been plotted using slightly modified values of $\beta$ and $\kappa$. The approximation using all data can be seen to give a visually good fit to all data sets. In this investigation, Equation 6.5 was found first and then the constant coefficients used to fit Equation 6.6 to the data shown in Figure 6.7. Hence, Equations 6.5 and 6.6 illustrate how the general FHWA approach can be improved as demonstrated later (Model 5).

6.4.5 Model 4: Bi-linear pullout model (general model)

The distribution of bias values in Figure 6.7 can be improved by assuming a bi-linear distribution of the efficiency factor ($\Psi$) with normal stress. Bi-linear distributions are used in current design codes to estimate the coefficient of earth pressure used in the calculation of reinforcement loads for the internal stability design of steel reinforced soil walls (e.g. AASHTO 2007; BS8006 1995; Bathurst et al. 2008b, 2009) and to select pullout interaction coefficients for steel strip reinforcement (AASHTO 2007; BS8006 1995; FHWA 2001). Figure 6.8 shows back-calculated values of efficiency factor ($\Psi$) using Equation 6.1 plotted against normal stress. A bi-linear curve is superimposed on each plot. The breakpoint is set at a normal stress of 40 kPa based on the visual observation that this is a convenient value that divides the data into stress-dependent and constant value groups. Using the optimization solver bundled with Microsoft Excel and all data points, the intercept on the vertical axis was determined to be 1.92 and horizontal line as 0.81. For convenience, these values are taken as 2 and 0.8, respectively, as shown in Figure 6.8. An alternative interpretation of the bi-linear model introduced here is the approximation shown in Figure 6.9 assuming that granular soil has a compacted unit weight of 17.5 kPa (median value shown in Table 6.3) and using depth below the top of the wall as the independent parameter. The bi-linear model can then be interpreted to show that the apparent soil-geosynthetic interface friction coefficient ($\tan \phi_{sg}$) varies linearly with depth below the top of the wall from 2 to 0.8 times the soil friction coefficient ($\tan \phi_s$) at an equivalent depth of 2.3 m, and remains constant thereafter.

Figures 6.10,11 illustrate the accuracy of the simplified bi-linear model. In Figure 6.10, the number of data points falling above and below the 1:1 correspondence line is about the same. The bias mean of all data is close to one and the COV is 0.43. The COV of the biaxial PP geogrid data is the lowest of the three product categories. This may be attributed to the smaller number of data points.
points. Figure 6.11 shows that about 86% of the bias values are greater than 0.58 (mean minus one standard deviation). In other words, if 58% of the predicted pullout capacity value using the bi-linear model is taken as the design value, the probability of under-estimating the design pullout capacity is about 14%. If the mean minus two standard deviations is used, then probability of under-estimating the design pullout capacity is zero.

Compared to Figure 6.7, the visual impression in Figure 6.11 is that the dependency between model bias and predicted load is much less but nevertheless detectable as shown by the straight line fitted to the data points using linear regression. The relative dependency is confirmed by the Spearman rank correlation coefficient $\rho = -0.24$ which has a much lower magnitude than the value for Model 3. The bias statistics summarized in Table 6.4 show that the bi-linear model is an improvement over a linear model with current FHWA (2001) default values or recalibrated with $F^* = \tan \phi$.

6.4.6 Model 5: Non-linear pullout model (general model)

The power function (Equation 6.5) fitted to the data in Figure 6.7a can be used to adjust the current linear model approach used in most design codes (e.g. AASHTO 2007) to remove non-linear influences. Letting the corrected pullout load ($P_{corr}$) equal the measured value ($P_m$) leads to the following general form of the new corrected model:

$$P_{corr} = \beta (P_c)^{1+\kappa} \tag{6.7}$$

Substituting Equation 6.3 (default FHWA pullout equation) gives:

$$P_{corr} = \beta (2\sigma_s L F^* \alpha)^{1+\kappa} \tag{6.8}$$

The coefficient values are $\beta = 5.51$ and $1+\kappa = 0.629$. As noted earlier these values result in a good fit to bias data dependency (Figure 6.7) regardless of geogrid category.

Implementation of Model 5 is a two step process. First calculate the pullout capacity using the current FHWA design equation (Equation 6.3); then, correct this value using the power function expression in Equation 6.7 using dimension-dependent coefficients. Alternatively, Equation 6.3
with $F^\alpha = \tan \beta$, could be used together with recomputed $\beta$ and $\kappa$ values. However, this does not improve the accuracy of the non-linear model proposed here.

**Figure 6.12** shows measured versus predicted pullout load values using Equation 6.8. All data fall close to the 1:1 correspondence line. The fraction of data points for all tests in Figure 6.12a falling above and below the 1:1 line is 56% and 44%, respectively. This distribution corresponds to a mean of bias values that is slightly greater than 1, which is desirable for design (i.e. pullout resistance capacity is slightly under-estimated on average). Predicted values for the woven PET geogrids are slightly more conservative with a mean bias of 1.13 which is the result of using model parameters for the entire database. Clearly this result can be improved by fitting the power function to category-specific data. Nevertheless, the coefficient values used here result in bias COV values for the entire data set and individual product type that are all less than 0.40. This is marked improvement over the current FHWA default model (Table 6.4). Bias values versus predicted load values computed using Model 5 are presented in Figure 6.13. The Spearman rank correlation coefficient computed using all data is $\rho = -0.03$, which is very low. More than 83% of the bias values are greater than 0.71 (mean minus one standard deviation) and all bias values are greater than 0.31.

### 6.5 Additional observations

The maximum tensile load applied to a geosynthetic reinforcement specimen in a conventional pullout test will be limited by the in-air tensile (rupture) strength of the material. If the embedded reinforcement length is too long, then the rupture strength of the material will control. Hence, it is of interest to examine the database used in this investigation for guidance on the maximum length of reinforcement embedment that will still ensure a pullout failure mode. Recommendations for the maximum specimen length in laboratory pullout tests are possible because the database includes 318 tests that ended in pullout and 64 tests that ended in rupture. **Figure 6.14** summarizes the fraction of tests in each normalized vertical load interval that failed in rupture and pullout. The independent parameter is the normalized load acting over the length of the geogrid specimen ($\sigma L$) and $T_u$ is the in-air tensile (rupture) strength of the reinforcement reported in the source documents. The data show that when the ratio of $\sigma L$ to $T_u$ exceeds 1.5 there is a 90% to 100% probability that the test will end in rupture. Model 5 can also be used to estimate the maximum length of reinforcement for a series of pullout tests knowing the maximum normal stress level and soil type (Figure 6.15). For a trial reinforcement length $L$, Equation 6.8 is used to compute $P_{corr}$. If the value of $P_{corr}/T_u$ is greater than two, then there is a 100% probability that
the test will end in rupture based on the database used here. For values between 1.5 and 2, the probability of rupture is 65%, and so on. However, recall that the data used to generate these figures may not be valid for soil-geogrid combinations that fall beyond the envelope of physical parameters in the study database. As more data becomes available the recommendations made here may be refined.

6.6 Conclusions and discussion

This chapter examines the accuracy of the current FHWA (2001) pullout capacity formulation based on project-specific testing or using default values. The analyses are limited to geogrid products. A unique feature of this chapter is that a large database of pullout test results has been collected by the writer from multiple sources.

No attempt is made in this chapter to gain insight into the actual mechanics of geogrid-soil load transfer. Rather, the approach is phenomenological (observational) and preserves at its core the underlying deterministic pullout capacity model found in the FHWA (2001) guidance document. The accuracy of the FHWA method is assessed using statistical quantities for the model bias values computed as measured to predicted pullout capacity. Quantitative measures of the accuracy of the FHWA pullout model using two different interpretations of project-specific data or using default coefficient values are now available. If project-specific pullout test data are used, the spread in model accuracy varies from a COV value of 0.21 to 0.13 based on a single average value for $F^*\alpha$ and a linear approximation to $F^*\alpha$, respectively. Clearly the latter is more accurate. Furthermore, the computed model bias values are practically stress-level independent for the latter.

In practice, actual test data may not be available for project design. Hence, presumptive values for $F^*\alpha$ are required with consequential loss in model accuracy using the current FHWA approach (Model 3). Analyses show that using current recommended default values for $F^*$ and $\alpha$ leads to an under-estimation of pullout load that is on average a factor of 2.3 less than the measured value. Furthermore, the COV for bias values is very high at 55%. Adjusting the default $F^*$ and $\alpha$ values to the maximum possible values within the FHWA model framework leads to an improvement in predicted pullout load capacity (i.e. over-estimation is reduced to a factor of 1.20). However, the spread in bias data remains unchanged.
The bi-linear and non-linear models proposed in this chapter lead to greater model accuracy in that order. The advantage of the proposed non-linear model over the bi-linear model is that it is a smoothly continuous function and is more accurate with no hidden dependencies.

An additional practical benefit of the data collected by the writer and statistical interpretation is that procedures to estimate reinforcement lengths to increase the likelihood of a pullout mode of failure in the laboratory for a particular soil-geogrid combination are now possible. Furthermore, the data and models reported here can be used as a reference to assess the accuracy of pullout test methodology and test repeatability at commercial and research laboratories and for round robin testing. Finally, the results of this chapter have important implications to design of reinforced soil walls, slopes and embankments. In design, the pullout capacity of a reinforcement layer must be sufficient to ensure that the required tensile strength of the reinforcement can be mobilized within a prescribed strain level. However, the influence of the choice of pullout model on internal stability of reinforced soil structures is presented in Chapters 9 and 11.

REFERENCES


Table 6.1 Summary of pullout test database.

<p>| Source identification number | Number of data sets | Number of tests (a) (data points) | Source identification number | Number of data sets | Number of tests (a) (data points) | Source identification number | Number of data sets | Number of tests (a) (data points) | Source identification number | Number of data sets | Number of tests (a) (data points) | Source identification number | Number of data sets | Number of tests (a) (data points) | Source identification number | Number of data sets | Number of tests (a) (data points) | Source identification number | Number of data sets | Number of tests (a) (data points) | Reference |
|-------------------------------|---------------------|----------------------------------|-------------------------------|---------------------|----------------------------------|-------------------------------|---------------------|----------------------------------|-------------------------------|---------------------|----------------------------------|-------------------------------|---------------------|----------------------------------|-----------------------------|---------------------|----------------------------------|-----------------------------|---------------------|----------------------------------|-----------------------------|---------------------|----------------------------------|-----------------------------|
| 1                            | 5                   | 15                               | 3                             | 2.1×0.9×0.5         | Y                                | 0                            | 1                           | Unpublished report (Laboratory 1) |
| 2                            | 8                   | 66                               | 3                             | 1.3×0.7×0.3         | Y                                | 0.15                         | 0.5                         | Unpublished report (Laboratory 2) |
| 3                            | 12                  | 41                               | 3, 4                          | 1.5×0.6×0.3         | Y                                | 0                            | 1                           | Cowell and Sprague (1993) and unpublished report (Laboratory 3) |
| 4                            | 5                   | 26                               | 1                             | 2.1×0.6×0.3         | Y                                | 0                            | 1                           | Unpublished report (Laboratory 4) |
| 5                            | 13                  | 46                               | 1, 3                          | 1.5×0.9×0.6         | Y                                | 0.3                          | 2-20                        | Farrag et al. (1993) and unpublished report (Laboratory 5) |
| 6                            | 24                  | 94                               | 1, 3, 4                       | 2.3×0.8×0.6         | Y                                | 0.15                         | 1                           | Unpublished reports (Laboratory 6) |
| 7                            | 9                   | 36                               | 1                             | 1.7×0.7×0.6         | Y                                | Clamp inside box             | 1                           | Moraci and Recalcati (2006) |
| 8                            | 2                   | 8                                | 1                             | 1.5×1.0×0.8         | N                                | 0.2                          | 1.8-22                      | Lopes and Ladeira (1996) |
| 9                            | 5                   | 18                               | 1, 3                          | 1.3×0.6×0.6         | Y                                | 0                            | 0.25-1                      | Raju (1993) |
| 10                           | 1                   | 9                                | 2                             | 1.6×0.6×0.4         | Y                                | 0.2                          | 2                           | Bolt and Duszynska (2000) |
| 11                           | 5                   | 18                               | 2                             | 0.67×0.45×0.23      | N                                | 0.1                          | 1                           | (Manufacturer 1 - technical data reports) |</p>
<table>
<thead>
<tr>
<th>Manufacturer</th>
<th>Tests</th>
<th>Samples</th>
<th>Geosynthetic Type</th>
<th>Length</th>
<th>Width</th>
<th>Depth</th>
<th>Data</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manufacturer 1</td>
<td>2</td>
<td>6</td>
<td>2</td>
<td>1.5×0.6×0.3</td>
<td>Y</td>
<td>0</td>
<td>1</td>
<td>(Manufacturer 1 - technical data reports)</td>
</tr>
<tr>
<td>Unpublished reports from Manufacture 2</td>
<td>50</td>
<td>1, 2</td>
<td>0.6×0.4×0.4</td>
<td>Y</td>
<td>Not reported</td>
<td>1</td>
<td>Unpublished reports from Manufacture 2 (courtesy of J. Hironaka)</td>
<td></td>
</tr>
<tr>
<td>Alagiyawanna et al. (2001)</td>
<td>2</td>
<td>4</td>
<td>2</td>
<td>0.7×0.3×0.6</td>
<td>Y</td>
<td>0</td>
<td>1</td>
<td>Alagiyawanna et al. (2001)</td>
</tr>
<tr>
<td>Abdel-Rahman et al. (2007)</td>
<td>3</td>
<td>12</td>
<td>3</td>
<td>1.2×0.7×1.2</td>
<td>N</td>
<td>0</td>
<td>NA</td>
<td>Abdel-Rahman et al. (2007)</td>
</tr>
<tr>
<td>Ingold (1983)</td>
<td>2</td>
<td>16</td>
<td>1, 2</td>
<td>0.5×0.3×0.3</td>
<td>Y</td>
<td>0</td>
<td>1</td>
<td>Ingold (1983)</td>
</tr>
<tr>
<td>Wilson-Fahmy et al. (1994, 1995)</td>
<td>5</td>
<td>13</td>
<td>2, 3</td>
<td>1.9×0.9×1.1</td>
<td>Y</td>
<td>0.1</td>
<td>1.5</td>
<td>Wilson-Fahmy et al. (1994, 1995)</td>
</tr>
</tbody>
</table>

Total 117 478

Notes: (a) some data sets include tests that ended in rupture. B = length of pullout box in direction of pull; W = width; D = depth. Geosynthetic type: 1 = HDPE punched and drawn uniaxial geogrid (192); 2 = PP punched and drawn biaxial geogrid (60); 3 = woven or knitted polyester geogrid (212); 4 = geotextile (14)
### Table 6.2 Summary of geogrid reinforcement properties.

<table>
<thead>
<tr>
<th>Material type</th>
<th>In-isolation secant stiffness at 5% strain $^{(a)}$ (kN/m)</th>
<th>Ultimate strength $^{(a)}$ $T_u$ (kN/m)</th>
<th>Spacing in transverse direction $^{(b)}$ (mm)</th>
<th>Spacing in longitudinal direction $^{(b)}$ (mm)</th>
<th>Aperture transverse to longitudinal aspect ratio</th>
<th>Specimen width (m)</th>
<th>Specimen length L (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniaxial HDPE</td>
<td>max              2200                                      175                                   24                                             450                                   0.19                     0.76                     2.15</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>median           740                                       80                                    22                                             160                                   0.13                     0.4                      1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>min              440                                       35                                    12                                             60                                    0.05                     0.15                     0.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Biaxial PP</td>
<td>max              1220                                      102                                   60                                             155                                   1.43                     0.5                      1.8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>median           270                                       20                                    37                                             33                                    1.18                     0.4                      0.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>min              170                                       12                                    15                                             25                                    0.1                      0.24                     0.33</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Woven or knitted PET</td>
<td>max              2220                                      370                                   38                                             80                                    1.5                      0.77                     1.8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>median           680                                       100                                   13                                             80                                    0.61                     0.3                      1.15</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>min              205                                       35                                    5                                               20                                    0.06                     0.3                      0.41</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes: (a) in-isolation constant rate-of-strain varies between test methods in different countries; (b) inside edge-to-edge distance
Table 6.3 Summary of soil properties.

<table>
<thead>
<tr>
<th></th>
<th>D$_{10}$ (mm)</th>
<th>D$_{60}$ (mm)</th>
<th>Cu</th>
<th>Bulk unit weight, $\gamma$ (kN/m$^3$)</th>
<th>Peak friction angle, $\phi$ (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>max</td>
<td>5.0</td>
<td>14.0</td>
<td>1000</td>
<td>22</td>
<td>49</td>
</tr>
<tr>
<td>median</td>
<td>0.16</td>
<td>0.7</td>
<td>6.9</td>
<td>17.5</td>
<td>35</td>
</tr>
<tr>
<td>min</td>
<td>0.00132</td>
<td>0.075</td>
<td>1.3</td>
<td>15</td>
<td>24</td>
</tr>
</tbody>
</table>

Notes: D$_{10}$, D$_{60}$ = particle size for which 10% and 60% of soil sample is less than; Cu = coefficient of curvature.
Table 6.4 Bias statistics for different pullout capacity models.

<table>
<thead>
<tr>
<th>Model</th>
<th>Description</th>
<th>Bias statistics</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>All data</td>
</tr>
<tr>
<td></td>
<td></td>
<td>n = 318</td>
</tr>
<tr>
<td></td>
<td>Mean</td>
<td>COV</td>
</tr>
<tr>
<td>1</td>
<td>1.00</td>
<td>0.21</td>
</tr>
<tr>
<td>2</td>
<td>1.00</td>
<td>0.13</td>
</tr>
<tr>
<td>3</td>
<td>2.23</td>
<td>0.55</td>
</tr>
<tr>
<td>4</td>
<td>1.20</td>
<td>0.55</td>
</tr>
<tr>
<td>5</td>
<td>1.02</td>
<td>0.43</td>
</tr>
</tbody>
</table>

Notes: Models 1 & 2 require project-specific test data; Models 3, 4 and 5 require input parameters: $\sigma_v =$ normal stress, $L_c =$ anchorage length, $\phi_s =$ peak friction angle of the soil; COV = coefficient of variation = standard deviation/mean; <sup>(a)</sup> $F^*\alpha$ versus normal stress ratio $\sigma_v/\bar{\sigma}_v$ (where $\bar{\sigma}_v$ is the average normal stress in a test series); <sup>(b)</sup> $P_m/P_c$ versus $P_c$. 
Figure 6.1 Cumulative distribution plots for normal stress and measured pullout loads recorded in geogrid pullout test database: (a) normal stress; (b) pullout load.
**Figure 6.2** Measured versus predicted pullout load. Model 1 using average measured $F^*\alpha$ value in a test series to predict individual pullout test result: (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.

Notes: n = number of data points; mean and COV refer to mean and coefficient of variation of bias values, respectively.
Figure 6.2 (continued) Measured versus predicted pullout load. Model 1 using average measured F*α value in a test series to predict individual pullout test result: (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.

Notes: n = number of data points; mean and COV refer to mean and coefficient of variation of bias values, respectively.
Figure 6.3 Dependency between $F^*$ bias values and normal stress ratio. Model 1 using average measured $F^*$ value in a test series to predict individual pullout test result: (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.
Figure 6.3 (continued) Dependency between $F^*\alpha$ bias values and normal stress ratio. Model 1 using average measured $F^*\alpha$ value in a test series to predict individual pullout test result: (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.
Figure 6.4 Measured versus predicted pullout load. Model 2 using first-order approximation to measured $F^*\alpha$ values versus normal stress in a test series to predict individual pullout test result: (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.

Notes: $n =$ number of data points; mean and COV refer to mean and coefficient of variation of bias values, respectively.
Figure 6.4 (continued) Measured versus predicted pullout load. Model 2 using first-order approximation to measured F*α values versus normal stress in a test series to predict individual pullout test result: (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.

Notes: n = number of data points; mean and COV refer to mean and coefficient of variation of bias values, respectively.
Figure 6.5 Dependency between $F^*\alpha$ bias values and normal stress ratio. Model 2 using first-order approximation to measured $F^*\alpha$ values versus normal stress in a test series to predict individual pullout test result: (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.
Figure 6.5 (continued) Dependency between $F^*\alpha$ bias values and normal stress ratio. Model 2 using first-order approximation to measured $F^*\alpha$ values versus normal stress in a test series to predict individual pullout test result: (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.
Figure 6.6 Measured versus predicted pullout load using Model 3 (current FHWA default pullout model): (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.

Notes: n = number of data points; mean and COV refer to mean and coefficient of variation of bias values, respectively. Solid line in each figure is fitted approximation to all data.
Figure 6.6 (continued) Measured versus predicted pullout load using Model 3 (current FHWA default pullout model): (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.

Notes: $n = \text{number of data points}; \text{mean and COV refer to mean and coefficient of variation of bias values, respectively. Solid line in each figure is fitted approximation to all data.}$
Figure 6.7 Pullout load bias versus predicted load using Model 3 (current FHWA default pullout model): (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.

Notes: Solid curve is approximation to all data. Dashed curves are approximations fitted to $P_m$ versus $P_c$ data in each category.
Figure 6.7 (continued) Pullout load bias versus predicted load using Model 3 (current FHWA default pullout model): (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.

Notes: Solid curve is approximation to all data. Dashed curves are approximations fitted to $P_m$ versus $P_c$ data in each category.
Figure 6.8 Pullout efficiency factor ($\Psi$) versus normal stress using Model 4 (bi-linear pullout model): (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.
Figure 6.8 (continued) Pullout efficiency factor (Ψ) versus normal stress using Model 4 (bi-linear pullout model): (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.
Figure 6.9 Model 4 (bi-linear pullout model) efficiency factor versus depth assuming $\gamma = 17.5 \text{ kN/m}^3$. 
Figure 6.10 Measured versus predicted pullout load using Model 4 (bi-linear pullout model): (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.

Notes: n = number of data points; mean and COV refer to mean and coefficient of variation of bias values, respectively.
Figure 6.10 (continued) Measured versus predicted pullout load using Model 4 (bi-linear pullout model): (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.

Notes: n = number of data points; mean and COV refer to mean and coefficient of variation of bias values, respectively.
Figure 6.11 Pullout load bias versus predicted pullout load using Model 4 (bi-linear pullout model): (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.
Figure 6.11 (continued) Pullout load bias versus predicted pullout load using Model 4 (bi-linear pullout model): (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.
Figure 6.12 Measured versus predicted pullout load using Model 5 (non-linear pullout model): (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.

Notes: n = number of data points; mean and COV refer to mean and coefficient of variation of bias values, respectively.
Figure 6.12 (continued) Measured versus predicted pullout load using Model 5 (non-linear pullout model): (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.

Notes: n = number of data points; mean and COV refer to mean and coefficient of variation of bias values, respectively.
Figure 6.13 Pullout load bias versus predicted pullout load using Model 5 (non-linear pullout model): (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.
Figure 6.13 (continued) Pullout load bias versus predicted pullout load using Model 5 (non-linear pullout model): (a) All data; (b) Uniaxial HDPE; (c) Biaxial PP; (d) Woven PET.
Figure 6.14 Fraction of tests ending in rupture or pullout in each normalized vertical load interval ($\sigma_v L / T_u$). Note: $\sigma_v =$ normal stress, $L =$ specimen length, and $T_u =$ in-air ultimate tensile strength of the reinforcement.
Figure 6.15 Probability of test ending in rupture or pullout using predicted pullout capacity (Model 5) normalised against ultimate strength of reinforcement as an index parameter.
Chapter 7  
Interpretation of Installation Damage Testing for Reliability-based Analysis and Load and Resistance Factor Design (LRFD) Calibration

7.1 Introduction

The available long-term tensile strength ($T_{al}$) of a geosynthetic reinforcing layer is an important calculation for the internal stability design of geosynthetic reinforced soil walls, slopes and embankments. North American practice using allowable stress design (ASD) is to compute $T_{al}$ as follows (Allen 1991; FHWA – Elias et al. 2001; AASHTO 2002; NCMA 2009; CFEM 2006):

$$[7.1] \quad T_{al} = \frac{T_{ult}}{RF} = \frac{T_{ult}}{RF_{ID} \times RF_{CR} \times RF_{D}}$$

where, $T_{ult}$ = ultimate tensile strength of the reinforcement and RF = product of reduction factors to account for potential long-term strength loss due to installation damage ($RF_{ID}$), creep ($RF_{CR}$) and degradation due to chemical/biological processes ($RF_{D}$).

Figure 7.1 illustrates the interpretation of Equation 7.1 with respect to the design life of a geosynthetic reinforcement layer in a soil structure. In current deterministic ASD practice the (available) long-term tensile strength ($T_{al}$) must be at least equal to the maximum predicted tensile load ($T_{max}$) in a reinforcement layer. Typically an overall factor of safety (FS) is used in allowable stress design (ASD) to account for other project uncertainties so that $T_{al}/T_{max} \geq FS = 1.5$ (AASHTO 2002; NCMA 2009; CFEM 2006).

The ultimate tensile strength ($T_{ult}$) used for design in Equation 7.1 is not a mean value, but is typically a conservative value taken as the Minimum Average Roll Value (MARV) or minimum value based on the manufacturer’s quality control testing. Hence, this is a conservative estimate of the undamaged strength of the geosynthetic reinforcement using results of wide-width tensile tests (ASTM D4595 2009; ASTM D6637 2001). The selection of the ultimate tensile strength for design in the context of this investigation is described later in this chapter.
In North American practice, the value of $RF_{ID}$ is determined using field installation damage tests, where reinforcement installation method, backfill type and compaction method are the same as, or similar to, project conditions using a common test protocol (e.g. ASTM D5818 2000; WSDOT T925 2005). While it is desirable to calculate $RF_{ID}$ from project-specific installation damage testing, in practice, it is more common to base $RF_{ID}$ on non-project-specific testing available at the time of project design.

While the topic of this chapter is related to strength losses due to installation damage, the focus is not on the calculation of $RF_{ID}$ values for design; rather, this study focuses on calculation of bias statistics for installation damage, assuming that the backfill soil used to evaluate $RF_{ID}$ is very similar to the backfill soil used for the geosynthetic structure under consideration. These statistics are required for reliability-based analysis and LRFD calibration for the geosynthetic rupture limit state that is a necessary part of internal stability design of reinforced soil walls.

### 7.2 Reliability-based analysis and LRFD calibration for rupture limit state

The AASHTO (2009) and CSA (2006) design guidance documents are now committed to a load and resistance factor design (LRFD) approach for mechanically stabilized earth wall (MSEW) structures; this includes geosynthetic reinforced soil walls.

The general approach for reliability-based analysis and design can be found in text books by Harr (1987), Ang and Tang (1975) and Nowak and Collin (2000). Load and resistance factors used in LRFD codes can be computed by back-fitting to ASD past practice. However, this is not satisfactory if the objective is to select load and resistance factors that meet a prescribed probability of failure for each limit state and to ensure that the same probability of failure applies to all limit states for a set of failure modes (e.g. internal stability). Furthermore, there are uncertainties associated with each of the strength-reduction mechanisms identified above and these are illustrated by the range bars in Figure 7.1. In ASD these uncertainties are ignored or, at best, are accounted for in a subjective manner. Reliability-based design and reliability analyses for LRFD calibration require statistical estimates of the variables that appear in the underlying deterministic models for load and resistance in limit state equations.

The focus of this chapter is on the limit state function for internal rupture (or over-stressing) in geosynthetic reinforced soil walls which can be expressed as

\[ \phi T_{ai} - \gamma Q T_{max} \geq 0 \]
where, $T_{\text{max}}$ is the nominal maximum design tensile load in the reinforcement layer under operational conditions over the life of the structure, $\varphi$ is the resistance factor for the rupture limit state and $\gamma_Q$ is the load factor. For example, in the AASHTO design specifications, $\gamma_Q = 1.35$ is the load factor due to soil self weight plus permanent uniform surcharge pressure and the corresponding resistance factor is $\varphi = 0.90$. The $T_{\text{max}}$ value is computed using deterministic models such as the AASHTO Simplified Method (AASHTO 2009) or the K-Stiffness Method (e.g. Allen et al. 2003; Bathurst et al. 2008).

However, the deterministic models used in soil-structure design problems have varying accuracy when compared to measured load and resistance values where these comparisons can be made. The general approach to correct deterministic model predictions is to introduce bias values where the bias value for a particular random variable is expressed generically as $X = \text{measured value}/\text{predicted value}$ (Nowak and Collin 2000; Withiam et al. 1998). The general approach has been demonstrated by Allen et al. (2005) and Bathurst et al. (2008) for LRFD calibration of the pullout limit state for steel grid reinforced soil walls.

Bias statistics are influenced by model bias (i.e. accuracy of the deterministic model representing the mechanics of the load or resistance side of the limit state equation), random variation in input parameter values, spatial variation in input values (if applicable), quality of data, and consistency in interpretation of data when data are gathered from multiple sources (the typical case) (Allen et al. 2005). Incorporating bias statistics into LRFD calibration and assuming only one load type, Equation 7.2 can be expressed as:

\[
\gamma_Q X_R - \varphi X_Q \geq 0
\]

[7.3]

Here, $X_R = \text{resistance bias computed as the ratio of measured resistance to calculated (predicted) nominal resistance}$ and $X_Q = \text{load bias computed as the ratio of measured load to the calculated (predicted) nominal load}$. For the load side:

\[
X_Q = \frac{T_{\text{max,meas}}}{T_{\text{max}}}
\]

[7.4]
where, $T_{\text{max,meas}}$ is the measured maximum tensile load in a reinforcement layer. For the resistance side, the available strength bias value is expressed as:

$$X_R = \frac{T_{\text{al,meas}}}{T_{\text{al}}} \tag{7.5}$$

where, $T_{\text{al,meas}}$ is the measured long-term tensile strength in a reinforcement layer. The predicted (nominal) long-term design strength ($T_{\text{al}}$), is calculated using Equation 7.1. The measured long-term tensile strength can be expressed as:

$$T_{\text{al,meas}} = T_{\text{al}} \cdot X_R = \frac{T_{\text{ult}}}{\text{RF}_{\text{ID}} \times \text{RF}_{\text{CR}} \times \text{RF}_{\text{D}}} \cdot X_R = \left( \frac{T_{\text{ult}}}{\text{RF}_{\text{ID}}} \right) \cdot X_{\text{ID}} \cdot \left( \frac{1}{\text{RF}_{\text{CR}}} \right) \cdot X_{\text{CR}} \cdot \left( \frac{1}{\text{RF}_{\text{D}}} \right) \cdot X_{\text{D}} \tag{7.6}$$

The bias of the long-term tensile strength of the reinforcement, $X_R$, is expressed as a product of three bias values:

$$X_R = X_{\text{ID}} \cdot X_{\text{CR}} \cdot X_{\text{D}} \tag{7.7}$$

where, $X_{\text{ID}}$ = bias of tensile strength after installation; and $X_{\text{CR}}$ and $X_{\text{D}}$ = bias of values for reduction factors $\text{RF}_{\text{CR}}$ and $\text{RF}_{\text{D}}$, respectively. Assuming that $X_{\text{ID}}$, $X_{\text{CR}}$, and $X_{\text{D}}$ are uncorrelated, the mean and coefficient of variation (COV) of $X_R$ values are given by (Ang and Tang 1975):

$$\mu_{X_R} = \mu_{X_{\text{ID}}} \cdot \mu_{X_{\text{CR}}} \cdot \mu_{X_{\text{D}}} \tag{7.8}$$

and

$$\text{COV}_{X_R} = \sqrt{\text{COV}^2_{X_{\text{ID}}} + \text{COV}^2_{X_{\text{CR}}} + \text{COV}^2_{X_{\text{D}}}} \tag{7.9}$$

Here, the subscripts for each of the mean and COV terms can be matched to installation damage, creep and degradation reduction factors identified earlier. In theory, Equations 7.8 and 7.9 are
only valid for uncorrelated log-normal distributions of the three random variables. However, for uncorrelated normal distributions with small coefficients of variation these expressions are sufficiently accurate.

Once frequency distributions for load \((X_Q)\) and resistance \((X_R)\) are known, bias statistics for both quantities can be approximated (i.e. normal or log-normal mean and COV of each distribution). Monte Carlo simulations can then be carried out to find the magnitude of resistance factor \((\phi)\) to match a prescribed load factor \((\gamma_Q)\) and a target probability of failure. These calculations can be carried out using an Excel spreadsheet (Allen et al. 2005).

At the time of this study, load bias statistics using the AASHTO Simplified Method and the K-Stiffness Method have been published (e.g. Bathurst et al. 2008). However, bias statistics for installation damage, creep and chemical degradation have not been determined. The objective of this study was to determine bias statistics for the installation damage reduction factor \(\text{RF}_{\text{ID}}\). These statistics are required for reliability-based design using Monte Carlo simulation and for LRFD calibration using reliability analysis. Reliability-based design involves using prescribed load and resistance factors (e.g. \(\gamma_Q\) and \(\phi\)) recommended in design codes together with computed characteristic load value(s) (e.g. \(T_{\text{max}}\)) to select a material with adequate resistance (i.e. strength \(T_{\text{al}}\)) that satisfies a limit state function (e.g. Equation 7.2). The objective of LRFD calibration is to select load and resistance factor values found in design codes so that the probability of failure associated with each limit state function is greater than an acceptable minimum value.

The strategy to meet this objective for the case of the geosynthetic rupture limit state for reinforced soil structures was to first collect installation test data from many sources. Bias statistics were then computed for different categories of geosynthetic (types) and for four different categories of soil. The determination of similar bias statistics for creep and durability reduction factors and the results of LRFD calibration for the long-term rupture limit state are reserved for future publications.

### 7.3 Previous installation damage testing studies

**Allen and Bathurst (1994)** analysed the undamaged and damaged load-strain curves from more than 3500 tensile tests on 55 different geosynthetic reinforcement products taken from 12 different controlled site installation damage trials. The focus of their study was on the quantification of changes in modulus (stiffness) and peak strain as determined from index (in-air) tensile tests. These data were computed for seven different geosynthetic reinforcement product
categories and plotted against installation damage measured as $1/RF_{ID}$. They concluded that relative losses in geosynthetic modulus (stiffness) in typical wall applications were less than relative losses in index strength losses. The implication of this finding is that installation damage reduction factors based on strength loss may be conservative (safe) for design using strain-limited strengths or stiffness (e.g. K-Stiffness Method).

**Allen and Bathurst (1996)** investigated the combined effect of polymeric creep and installation damage using a database of constant load (creep) data for both undamaged and installation-damaged geosynthetic specimens. Based on the data available they concluded that multiplication of creep reduction ($RF_{CR}$) and installation damage factors ($RF_{ID}$) may be conservative and hence results in errors on the safe side for current ASD practice. **Greenwood (2002)** came to the same conclusion based on stepped isothermal creep-rupture tests performed on a polyester geosynthetic in undamaged and damaged states.

**Elias (2000)** identified the main factors that contribute to reductions in reinforcement strength as: (i) gradation and angularity of backfill, (ii) mass per unit area, manufacturing method and type of geosynthetic, (iii) lift thickness, and (iv) compaction effort applied. They concluded that installation damage is typically controlled by variables (i) and (ii). To the list of factors that will influence potential for loss of strength due to installation identified by **Elias (2000)** are the type and thickness of the coating material used in the manufacture of woven and knitted PET geogrids (e.g. PVC, acrylic or polypropylene coating materials).

**Hufenus et al. (2005)** analyzed tensile test data for 38 different geotextiles and geogrids from full-scale field installation damage trials. They used the data to investigate the influence of polymer type, product type, aggregate type, compaction method and depth of burial on computed installation damage reduction factors. They also reported installation damage reduction factors determined from laboratory installation tests. Agreement between computed mean and COV values from field and laboratory tests varied depending on the geosynthetic type and compaction method in the laboratory.

Details of individual field installation damage trials in published and unpublished reports and papers can be found in the references cited by **Allen and Bathurst (1994, 1996)** and **Hufenus et al. (2005)**; for brevity this large body of prior work is not reviewed here.
7.4 Installation damage database

The writer collected installation damage data from publically available technical data reports by manufacturers and unpublished reports from testing laboratories. A total of 20 different document sources were reviewed during the compilation of the database summarized in Table 7.1. Source materials used in the current study satisfied the following criteria:

a) Field installation damage trials were carried out in general conformity with recommendations in the WSDOT T925 (2005) guidance document and ASTM D5818 (2000).

b) Individual laboratory tensile strength test results for undamaged and damaged multiple rib (wide width) specimens were available.

Many of the references identified in Table 7.1 refer to journal and conference papers that describe installation damage trials. In these cases the writer was given access to the source data referenced in these papers in order to carry out the statistical analyses reported below. Source data for the paper by Hufenus et al. (2005) and some other studies reported in the literature were not available and thus are not included in statistical analyses of bias values. Excluded from the current study are data from laboratory installation damage testing. While, the use of laboratory installation damage testing is attractive (e.g. Huang 2006; Huang and Chiou 2006; Paula et al. 2004), individual tensile test results were not available. Furthermore, it is likely that bias statistics are different for geosynthetics tested in the field and those tested in the laboratory using synthetic aggregates (prEN ISO 10722-1 2004).

The data in Table 7.1 are grouped according to geosynthetic type (categories 1 through 8) and four soil categories. The geosynthetic product categories include groups identified in FHWA publications (Elias 2000; Elias et al. 2001). However, only two soil types based on gradation are identified in these earlier publications. One is a relatively coarse gradation with maximum particle size $D_{\text{max}} = 100$ mm and $D_{50} \approx 30$ mm and the second is for materials meeting a finer gradation with maximum particle size $D_{\text{max}} = 20$ mm and $D_{50} \approx 0.7$ mm. Unfortunately many of the installation trials in the database were carried out with soils that fall between these two gradations. Hence in the current study, four soil types were identified based on the $D_{50}$ of the soils. These materials were selected to broadly represent: a) very course gravel (Type 1); b) fine gravel (Type 2); c) coarse to medium sand (Type 3); and d) fine sand or smaller particle sizes (Type 4). The WSDOT T925 (2005) protocol for installation damage testing for product approval requires
that (as a minimum) a geosynthetic should be tested using a granular soil with \( D_{50} \geq 4.75 \text{ mm} \) (i.e. Type 2 or Type 1 in this investigation) and this influenced the choice of gradation categories in this investigation. The \( D_{50} \) ranges for the four soil types are plotted in Figure 7.2. The heavy black lines and shaded regions represent mean and range of particle size distributions from the installation tests in the database falling into each soil category. Most of the data are from North American installation damage trials; hence, the results of analysis reported in this chapter are most applicable to North American experience. Nevertheless, Types 2, 3 and 4 capture the three separate gradations for the crushed stone, rounded gravel and sand soil used by Hufenus et al. (2005) although the soils in their study were more broadly graded than most of the particle size gradations in the database collected by the writer.

As noted earlier, the field installation trials were carried out in general conformity with North American standards and practice. However, there were unavoidable variations between trials. For example, the base layer lift of aggregate below the geosynthetic samples was placed directly over ground in some cases and on steel plates in other cases, compaction equipment and number of passes was not standardized and, there were variations in lift thickness. However, based on the information available in source documents it was not possible to resolve the installation damage trial conditions between trials into finer categories. Furthermore, categorization of soils based on angularity, thickness of soils and compaction effort was not attempted in this study because the focus of the study is on the quantification of bias statistics (test variability) rather than the selection of the nominal representative value of installation damage reduction factor for design. Quantification of relationships between \( R_{FD} \) for different categories of geosynthetics or products within a product line, and soil parameters are reserved for a future publication.

The WSDOT T925 (2005) installation damage test protocol calls for a minimum of five undamaged specimens and nine or more damaged specimens depending on the COV of strength values for the exhumed (damaged) specimens. In some cases, as few as four undamaged specimens and four damaged specimens were available. However, these data were carefully reviewed and judged not to skew statistical outcomes reported in this study.

A total of 68 different geosynthetic products appear in the database (Table 7.1). All geosynthetic materials were commercially available products at the time the installation trials were carried out. A total of 799 and 2248 in-air tensile test results were reviewed for undamaged and damaged geosynthetic specimens, respectively. A total of 221 \( R_{FD} \) values were computed using the data sources summarized in Table 7.1. Silt-film woven PP geotextiles are identified as a geosynthetics
category in FHWA documents but do not appear in the analysis of bias statistics because there was judged to be insufficient data to carry out statistical analysis. Glass geogrids and yarn reinforced nonwoven geotextiles were not included because data were not available.

### 7.5 Calculation of installation damage reduction factor (RF\textsubscript{ID}) and installation damage statistics

The quantities of interest and analysis of the undamaged and damaged test data from a typical field installation damage trial are shown in Table 7.2. In this example there are eight undamaged specimen strengths and eleven damaged strength values recorded in Columns 2 and 3, respectively. The measured retained strength of an exhumed specimen is denoted as T\textsubscript{ID,meas}. RF\textsubscript{ID} is taken as the inverse of the mean value of the retained fraction of original strength as shown in Equation 7.10.

\[ RF_{\text{ID}} = \frac{\bar{T}}{T_{\text{ID,meas}}} \]  

Here $\bar{T}_{\text{ult,meas}}$ is the mean measured strength (baseline) value for the undamaged specimen tensile test results.

The design ultimate tensile strength ($T_{\text{ult}}$) used in the calculation of $T_{\text{ult}}$ (Equation 7.1) is typically set equal to the manufacturer’s minimum certified tensile strength, which is determined from manufacturer quality control (QC) data. This is typically determined as the mean minus two standard deviations of the sample population, or simply as the minimum value observed from all QC data over a given period of time or for a given number of product rolls. A useful description of the minimum certified value and its application to design and QC and quality assurance (QA) is provided in WSDOT T925 (2005) and NTPEP (2007).

However, manufacturer minimum certified tensile strengths were not available for all cases in this study. Furthermore, a manufacturer may sometimes report a lower ultimate tensile strength value than that computed from actual testing in order to position the product in the market, to consider variation in reported results between laboratories, or to be cautious. To avoid these unquantifiable complications, the design ultimate tensile strength was computed using the statistics of the tensile strength test results (e.g. ASTM D4595 2009; ASTM D6637 2001) from undamaged and damaged specimens originating from the same geosynthetic sample material; hence:
Here, COV\textsubscript{ult,meas} is the coefficient of variation of the measured strength test results for the undamaged specimens. The variability of reinforcement strength immediately after installation can be quantified by the bias value X\textsubscript{ID} introduced previously and computed as:

\[ X_{ID} = \frac{T_{ID,meas}}{T_{ID}} = \frac{T_{ID,meas}}{\left(\frac{1}{2} COV_{ult,meas} \right)_{RF}} \]

where, \( T_{ID} \) = predicted strength after installation damage, \( T_{ult,meas} \) = sample mean tensile strength for the undamaged material, and \( RF_{ID} \) is the installation damage reduction factor (Equation 7.10) based on project-specific data. The methodology adopted here to compute \( X_{ID} \) statistics represents ideal conditions because of the following assumptions:

1. The installation damage reduction factor is determined for the actual (or at least very similar) soil to be used for the geosynthetic structure under consideration, and interpolation to estimate \( RF_{ID} \) between the soil types used in the installation damage test sections is not required; and

2. The sample statistics obtained from tensile strength testing of the undamaged specimens used in the determination of the design value of \( T_{ult} \) are representative of the minimum average roll value (MARV) or value reported by the geosynthetic manufacturer as the minimum certified tensile strength for geosynthetic reinforcement design.

While it is desirable to calculate \( RF_{ID} \) from project-specific installation damage testing (assumption 1 above), in practice, it is more common to base \( RF_{ID} \) on non-project-specific testing available at design time. If installation damage data for the specific geosynthetic to be used in design are not available, installation damage data for other products within the same product line may be used. Recommendations on how to interpolate between installation tests with the same aggregate and a family of products are described in detail in WSDOT T925 (2005). If data from non-aggregate-specific installation damage tests are available, interpolation of \( RF_{ID} \) values...
between test section soil gradations is sometimes used to estimate $RF_{ID}$ for the actual project backfill material. This interpolation is most commonly carried out using the $D_{50}$ size of the backfill gradation (Elias 2000; WSDOT T925 2005). If interpolation for an intermediate soil gradation is required there is additional uncertainty in the estimation of $RF_{ID}$. Development of bias statistics to characterize the uncertainty caused by interpolating for values of $RF_{ID}$ between soil gradations is beyond the scope of this chapter and is reserved for a future paper on this subject.

Regarding the second assumption; it is recognized that the sample statistics for tensile strength may differ from the statistics for the full population that is typically used to compute the manufacturer’s minimum certified value. The “actual” tensile strength, $T_{ult}$ (see Equation 7.1) determined from the undamaged sample tensile testing will typically be slightly higher than the manufacturer’s minimum certified tensile strength (i.e. “design” tensile strength used in practice), resulting in a lower bias value that in turn will result in a more conservative resistance factor using reliability-based calibration.

To investigate the effect of using undamaged sample tensile strength statistics rather than the manufacturer’s minimum certified tensile strength, cumulative distribution function plots were generated for PVC-coated PET geogrid installation damage bias values using both approaches (i.e. manufacturer’s minimum certified tensile strength, and using the tensile strength test results for the undamaged sample from the case study sources to establish the strength that is two standard deviations below the mean for the sample). The mean and COV generated for $X_{ID}$ using the latter were 5% to 7% lower and 10% to 12% lower, respectively, than the mean and COV generated for $x_{ID}$ using the manufacturer’s minimum certified tensile strength values. However, the combination of a slightly greater (more conservative) mean and a slightly smaller (less conservative) COV using case study sample statistics rather than the manufacturer’s minimum value (presumably based on population statistics) resulted in no significant difference in the resistance factor for PVC coated PET geogrids determined using reliability theory.

It is understood from Figure 7.1 that the inherent variability of geosynthetic strength ($T_{ult,\text{mean}}$) has been included in the statistics for $X_{ID}$. The mean, standard deviation and COV of each column of data are also shown in Table 7.2. Values of $X_{ID}$ calculated using Equation 7.12 are shown in Column 4. Equation 7.12 shows that the mean of $X_{ID}$ is always greater than one. This is expected because the reference product tensile strength is two standard deviations below the mean value.
(Equation 7.11), while $RF_{ID}$ is computed using the mean of measured strength values (i.e. $\overline{\text{ult,meas}}$).

### 7.6 Results of analyses

Results of analyses are summarized in Table 7.3. In general, the upper bounds on computed $RF_{ID}$ values are consistent with the values specified by the FHWA (Elias et al. 2001). It can be noted that in cases with finer (less aggressive) backfill the lower bound on computed $RF_{ID}$ values is often less than one which is counter intuitive (i.e. it is expected that the strength of an exhumed (damaged) specimen should be less than the original nominal identical undamaged specimen). However, when damage is small, as in the case of less aggressive soils, the means of the undamaged and damaged tensile strength populations will be very close ($\overline{\text{ult,meas ID,measT}} \approx \overline{\text{ult,meas}}$); hence, the computed $RF_{ID}$ may be less than one. Allen and Bathurst (1994) and Hufenus et al. (2005) also noted instances where computed installation damage factors were less than one. Allen and Bathurst (1994) pointed out there may be mechanical reasons for this result. For woven geotextiles the initial crimp in the material may be removed during installation and compaction, and accumulation of fine particles in the fibre matrix of woven and nonwoven geotextiles may increase the strength of the materials at macro scale. Drawn polyolefin geogrids may undergo further strain hardening due to compaction stresses resulting in detectable increased strength after exhumation. In design codes, however, the minimum allowable $RF_{ID}$ value is 1.10. This code-specified design limit is not imposed on statistical analyses presented later.

The exception to reasonable agreement between upper bound $RF_{ID}$ values in the current study and upper bound values reported by Elias et al. (2001) occurs for the woven and nonwoven geotextiles. However, the FHWA-specified $RF_{ID}$ ranges for geotextiles are only applicable to products with mass per unit area ($M_a$) greater than 270 g/m$^2$. In the current study, geotextile products with lower mass per unit area are included in each sample population to increase the number of data points. However, the $M_a$-based criterion is not used in the current study to compute bias statistics since it is shown later that these materials can be better differentiated based on $RF_{ID}$ values less than or greater than 1.7. The practical implications of this criterion are described below.

Computed $RF_{ID}$ values for nonwoven geotextiles using the database in the current study are plotted in Figure 7.3a. The figure shows that the most aggressive soil (Type 1) corresponds to $RF_{ID}$ values greater than 1.7. The largest strength losses are for nonwoven geotextiles with mass...
per unit area less than 270 g/m². For less aggressive soil types the RF_ID values in the current database and values reported by Hufenus et al. (2005) are all less than 1.7.

**Figure 7.3b** presents X_ID data versus computed RF_ID for nonwoven geotextiles. The plot shows that there is a wide range of RF_ID values from about one to as high as five for the coarsest (Type 1) fill. For less aggressive fills, computed RF_ID values are less than 1.7 as noted above. Similarly, the spread in bias values (i.e. mean ± one standard deviation) is greater for the Type 1 fill than for the Type 4 fill. Unfortunately, X_ID data is not available for soil Types 2 and 3 in the current study database. Based on RF_ID values reported by Hufenus et al. (2005) and plotted in **Figure 7.3a**, it is reasonable to assume that bias statistics for nonwoven geotextiles with RF_ID < 1.7 are similar to those matching the Type 4 soil in this study.

**Figure 7.4a** shows installation reduction factors versus M_A for woven geotextiles. The data in Figures 7.4a and 7.4b suggest that quantification of the performance of these materials can be based on RF_ID greater than or less than 1.7 consistent with nonwoven geotextiles, e.g. values of RF_ID > 1.7 are typical for woven geotextiles placed in coarse gravel (Type 1) fills and there is more scatter in bias values.

For finer soils (Type 2, 3 and 4) and geotextiles falling within the property envelopes of the materials identified in the current investigation, a reasonable upper-bound value is RF_ID = 1.7 to 2.

**Figures 7.3c** and **7.4c** show cumulative distribution function (CDF) plots for nonwoven and woven geotextiles. Only the data from sources summarized in **Table 7.1** are used, since these data plots require results of individual tensile tests on undamaged and damaged geosynthetic specimens; hence, Hufenus et al. (2005) data are excluded. The two CDF plots are for data points in each geotextile category delineated by RF_ID = 1.7. The number of data points and the mean and COV values of the distributions are also presented in the figures. The data in the CDF plots are reasonably well approximated using normal distributions. The two approximations for each data set are computed using: (1) all the data; and (2) using a best fit to lower tail. In some cases the resulting approximations were the same. Allen et al. (2005) and Bathurst et al. (2008) have pointed out that it is the approximation to the lower tail of the resistance bias data that is important, since it is the overlap between the upper tail of the load bias data and lower tail of the resistance bias data that strongly influences the probability of failure in reliability-based design. The methodology to select the best fit to tail was carried out the same way for all data sets in this study as follows. First, the mean value for the entire data fit was used and only the slope of the
approximation (i.e. the COV value) was adjusted to better visually capture the lower tail data points. If the resulting fit to tail was judged to be poor, then both the mean and COV were adjusted. This second step was not required for the geotextile data discussed here.

Figures 7.5 through 7.7 present X_ID versus RF_ID data and bias value CDF plots for the remaining geosynthetic product categories in this study (geogrids). The bias statistics used to fit the CDF plots are summarized in Table 7.3. For all geogrid categories there is more spread in bias values for the coarse gravel soil (Type 1) compared to finer particle size gradations, but the COV values for Type 1 soils are lower than for geotextiles in combination with Type 1 soils. It is interesting to note that the COV values for PP biaxial geogrids and the PET products in combination with Type 2, 3 and 4 soils vary over a narrow range of 0.08 to 0.11 which is essentially the same range for the geotextile products (i.e. 0.08 to 0.12). The lowest values for mean and COV of bias values are for the HDPE geogrids. From a practical point of view the bias statistics for HDPE geogrids can be assumed to be independent of soil type. This is likely due to the fact that the data are for similar integral drawn geogrids with thick longitudinal and transverse members produced by a single manufacturer (see Table 7.1).

The woven PET geogrids and woven geotextiles have very similar mean and COV values for Type 2, 3 and 4 soil types. This may be because both materials have similar structures. The smaller the COV value the steeper the slopes of the CDF plots. The coated PET and PP geogrids have higher COVs for bias values than the HDPE geogrids.

The frequency distributions for all data sets are reasonably well approximated by normal distributions (i.e. approximations present as a straight line on each CDF plot). However, there is a visually detectable sigmoidal shape to the CDF plots for the PVC-coated PET geogrids (Figure 7.5c). Careful examination of the data could not explain this trend. However, there was no obvious reason to remove any data set contributions to these plots. A better fit to lower tail is possible in Figure 7.5c by adjusting the mean of the approximation up and increasing the COV value. However, this increase in both mean and COV of bias values was shown to not change the computed resistance factors for this product classification. This is possible because shifting the mean of the frequency distribution for resistance bias to a larger value can offset the increase in overlap with the load bias distribution due to more spread (larger COV).
7.7 Example computation of probability of failure for rupture limit state

The limit state function for rupture (over-stressing) of geosynthetic reinforcement is expressed by Equation 7.3 for the purpose of reliability analysis. To carry out an estimate of probability of failure using values of load and resistance factor and Monte Carlo simulation, it is necessary to have statistics for load and resistance bias values (X_Q and X_R, respectively). Resistance bias values are a function of the product of the bias values for the three contributing strength loss processes defined as installation damage, creep and biological/chemical degradation (Equation 7.7). This chapter has focused on the generation of bias statistics for installation damage. Nevertheless it is possible to demonstrate the influence of the range of installation damage bias statistics on probability of failure using assumed distributions for bias values for creep and durability on the resistance side and assumed values for the load side of the limit state function. In the examples to follow, closed-form solutions (Allen et al. 2005; Bathurst et al. 2008) that are a close approximation to the results of Monte Carlo simulation are used in order to generate smooth plots that highlight trends. To focus on the influence of installation damage, reduction factors for creep and durability are assumed to be unbiased with only small error; hence μ_{X_{cr}} = μ_{X_{cr}} = 1 and COV_{X_{cr}} = COV_{X_{cr}} = 0.10.

On the load side, two cases are considered: (a) load bias statistics values reported by Bathurst et al. (2008) using the current AASHTO Simplified Method and walls with cohesionless backfill (μ_{x_q} = 0.30, COV_{x_q} = 0.50) (Figure 7.8a); and (b) a perfect load model (μ_{x_q} = 1, COV_{x_q} = 0) (Figure 7.8b). The mean of installation damage values is taken as μ_{x_{id}} = 1.05 which is in the middle of the range of values for geogrids (Table 7.3). The influence of the spread in installation damage bias statistics for a range of γ_Q/φ is examined by assuming COV_{x_{id}} = 0.05, 0.10, 0.20, 0.30 and 0.40. As a reference, the current AASHTO (2009) design manual recommends γ_Q = 1.35 and φ = 0.90 for limit state design against reinforcement rupture; hence γ_Q/φ = 1.5. The latter value corresponds to the recommended minimum overall factor of safety in ASD practice; i.e. FS = T_{as}/T_{max} = γ_Q/φ = 1.5. In fact, φ = 0.90 in the current AASHTO design guidelines was selected by back-fitting to ASD past practice using the prescribed value of γ_Q = 1.35 for loads due to soil self weight plus permanent uniform surcharge pressure.

Figure 7.8a shows the results of calculations. As expected, for a constant γ_Q/φ value, the probability of failure diminishes as variability in damage bias statistics decreases. As the target “design factor of safety” (FS = γ_Q/φ) increases, the probability of failure decreases for the same
geosynthetic reinforcement-backfill combination. In this example, the probability of failure at FS = 1.5 is less than 1%. This is judged to be an acceptable outcome for reinforced earth structures that have redundant reinforcement elements (Allen et al. 2005). However, this outcome is largely due to the very conservative load model. A value of $\mu_{x_i} = 0.30$ means that actual reinforcement loads are only about one third of predicted values. It can be argued that for these designs about three times as much reinforcement is used as is actually required. In fact, Allen et al. (2002) demonstrated that the ratio of total tensile load capacity of walls to total load demand under operational conditions using the current Simplified Method is often much higher. Hence, this excess capacity means that variability in the prediction of reinforcement capacity due to installation damage is not a concern if the designer is willing to overdesign the wall (e.g. use the current AASHTO Simplified Method). However, for the case of a more accurate load model the influence of variability in installation damage bias statistics becomes much more important. For example, to simultaneously meet a target design factor of safety of 1.5 and a probability of failure of 1% using a perfect load model means that only combinations of geosynthetic type and soil that have $COV_{s_{ID} < 0.12}$ are permissible (Figure 7.8b). Based on the data in Table 7.3, this means that reinforced soil walls constructed with geotextiles may be problematic if a very good load model is used unless the designer is willing to increase the design factor of safety (or equivalently, increase the load factor).

In this chapter the focus has been on the generation of installation damage bias statistics. However, the data is also useful to identify trends between computed installation damage factor and scatter in bias statistics. Figure 7.9 shows such a plot. The general trend is increasing $COV_{s_{ID}}$ values with increasing installation damage factor. The highest installation damage values and the largest variation in $X_{ID}$ values occur for geotextiles based on the data available. Nevertheless, it is possible to get higher $RF_{ID}$ values for geogrids by selecting a more aggressive soil than was used in the database studies available to the writer. However, the important lesson from this plot is that candidate geosynthetics must be matched to the project aggregate so that $RF_{ID}$ can be kept to a minimum. In practice, this typically means using a heavier product with a more aggressive aggregate; if this is done, $COV_{s_{ID}}$ will be small, and the probability of failure due to tensile rupture will be reduced.
7.8 Conclusions and implications to design

Internal stability design of geosynthetic reinforced soil walls using a LRFD approach requires calibrated load and resistance factors to ensure that design outcomes satisfy an acceptable probability of failure. To select the resistance factor used in the limit state calculation for reinforcement rupture, bias statistics for installation damage, creep and durability are required or at least must be estimated with reasonable confidence. This chapter has focused on the generation of bias statistics for installation damage using a database of field installation damage trials from many sources. Bias statistics have been tabulated based on six geosynthetic reinforcement categories and four soil categories based on the D_{50} particle size.

In general, the mean and spread of values varied over narrow ranges for geogrid and geotextile products placed in soils with D_{50} particle size less than 19 mm (Type 2, 3 and 4 soils). For coarser granular soils (Type 1), mean and bias values were much larger for geotextiles. While the focus of the investigation is on installation damage bias statistics, computed reduction factors for installation damage (RF_{ID}) confirmed earlier recommendations by Elias (2000) that woven and nonwoven geotextiles with mass per unit area less than 270 g/m² should not be used in combination with Type 1 soils (i.e. D_{50} > 19 mm). For a particular soil type there were detectable differences in calculated RF_{ID} values depending on the geosynthetic type. However, for each geosynthetic type and any of three soil type categories with D_{50} < 19 mm, the computed bias values were judged to be the same. Hence for analysis purposes, bias statistics can be grouped into two ranges for each geosynthetic type based on D_{50} of the soil greater than or less than 19 mm.

This chapter also shows how bias statistics together with load and resistance factors for the geosynthetic rupture limit state function recommended by AASHTO (2009) can be used to calculate probability of failure using Monte Carlo simulation. At the time of writing, bias statistics for strength loss due to creep and durability are not available. However, using assumed values and published load bias statistics for the AASHTO Simplified Method and a hypothetical perfect load model, the influence of installation bias statistics on probability of failure was explored. These analyses demonstrate that the spread in installation damage bias statistics is likely not a concern if reinforcement loads are computed using a very conservative load model (e.g. the AASHTO Simplified Method). In other words, the excessive strength (or number) reinforcement layers that is the consequence of a very conservative (safe) load model compensates for lack of accuracy in the prediction of strength loss due to installation. However,
as the accuracy of the load model improves and less reinforcement load capacity is required, then corresponding improvements in prediction of loss of strength due to installation damage are required in order to take advantage of the more accurate load prediction.

An important lesson that can be gained from this study is the need to match a candidate geosynthetic to the project soil so that the applicable installation damage factor ($RF_{id}$) is kept to a minimum. This will lead to smaller scatter in installation damage bias values (lower $COV_{id}$). The result is a lower probability of failure due to tensile rupture.

Finally, a study is underway by the writer using large database of creep test results and a methodology to extract creep bias statistics. Once this data is available, LRFD calibration for the rupture limit state for the internal stability of geosynthetic reinforced soil walls will be attempted and resistance factors proposed (as required) for different geosynthetics in a range of installation environments.

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FHWA HI-98-032. Federal Highway Administration, Washington, DC, USA.
Table 7.1 Summary of the installation damage database.

<table>
<thead>
<tr>
<th>Geosynthetic type</th>
<th>Fill types or RFID</th>
<th>Number of products</th>
<th>Number of product lines</th>
<th>Number of test conditions (a)</th>
<th>Number of specimens</th>
<th>Data sources (b)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Undamaged</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Damaged</td>
<td></td>
</tr>
<tr>
<td>HDPE uniaxial geogrids</td>
<td>Type 1</td>
<td>6</td>
<td>2</td>
<td>3</td>
<td>52</td>
<td>92</td>
</tr>
<tr>
<td></td>
<td>Type 2, 3 and 4</td>
<td>5</td>
<td>2</td>
<td>5</td>
<td>46</td>
<td>168</td>
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<tr>
<td>PP biaxial geogrids</td>
<td>Type 1</td>
<td>4</td>
<td>1</td>
<td>2</td>
<td>40</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>Type 2, 3 and 4</td>
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<td>4</td>
<td>40</td>
<td>150</td>
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<tr>
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<td>Type 2, 3 and 4</td>
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<td>979</td>
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<td>Type 1</td>
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<td>1</td>
<td>2</td>
<td>40</td>
<td>36</td>
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<td></td>
<td>Type 2, 3 and 4</td>
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<td>2</td>
<td>6</td>
<td>38</td>
<td>152</td>
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<td>Woven geotextiles</td>
<td>Type 1</td>
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<td>2</td>
<td>35</td>
<td>55</td>
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<td></td>
<td>Type 2, 3 and 4</td>
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<td>4</td>
<td>9</td>
<td>133</td>
<td>188</td>
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<td>Nonwoven geotextiles</td>
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<td>2</td>
<td>1</td>
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<td>70</td>
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<td></td>
<td>Type 2, 3 and 4</td>
<td>4</td>
<td>2</td>
<td>2</td>
<td>52</td>
<td>80</td>
</tr>
<tr>
<td>Woven geotextiles</td>
<td>RFID ≥ 1.7</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>20</td>
<td>40</td>
</tr>
<tr>
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<td>RFID &lt; 1.7</td>
<td>9</td>
<td>4</td>
<td>9</td>
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<td>208</td>
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<td>Nonwoven geotextiles</td>
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<td>2</td>
<td>1</td>
<td>42</td>
<td>70</td>
</tr>
<tr>
<td></td>
<td>RFID &lt; 1.7</td>
<td>4</td>
<td>2</td>
<td>2</td>
<td>52</td>
<td>80</td>
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</tbody>
</table>
Table 7.1 (continued) Summary of the installation damage database.

Notes:
(a) Test condition is defined as a unique combination of cover thickness, soil type and compaction equipment
Table 7.2 Example calculation sheet for computed installation damage reduction factor $RF_{ID}$ and bias statistics for $x_{ID}$ values (source: Laboratory 2 unpublished data using Type 2 soil and woven polyester geogrid).

<table>
<thead>
<tr>
<th>Specimen number</th>
<th>Undamaged strength, $T_{ult, meas}$ (kN/m)</th>
<th>Damaged strength, $T_{ID, meas}$ (kN/m)</th>
<th>$x_{ID}$ (Equation 12)</th>
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<tbody>
<tr>
<td>[1]</td>
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<td></td>
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</tr>
<tr>
<td>1</td>
<td>36.9</td>
<td>29.5</td>
<td>1.06</td>
</tr>
<tr>
<td>2</td>
<td>35.0</td>
<td>30.4</td>
<td>1.10</td>
</tr>
<tr>
<td>3</td>
<td>35.8</td>
<td>31.0</td>
<td>1.12</td>
</tr>
<tr>
<td>4</td>
<td>36.6</td>
<td>27.0</td>
<td>0.97</td>
</tr>
<tr>
<td>5</td>
<td>34.9</td>
<td>30.6</td>
<td>1.10</td>
</tr>
<tr>
<td>6</td>
<td>34.8</td>
<td>30.7</td>
<td>1.11</td>
</tr>
<tr>
<td>7</td>
<td>37.0</td>
<td>27.1</td>
<td>0.98</td>
</tr>
<tr>
<td>8</td>
<td>33.9</td>
<td>30.4</td>
<td>1.10</td>
</tr>
<tr>
<td>9</td>
<td>29.5</td>
<td>1.06</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>30.3</td>
<td>1.09</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>29.6</td>
<td>1.07</td>
<td></td>
</tr>
</tbody>
</table>

Mean 35.6 29.6 1.07

Standard Deviation 1.14 1.38 0.05

COV 0.032 0.046 0.046

$T_{ult, MARV} = 35.6 - 2 \times 1.14 = 33.3$ (kN/m)

$RF_{ID} = 35.6 / 29.6 = 1.20$

$T_{ID} = T_{ult, MARV} / RF_{ID} = 33.3 / 1.20 = 27.8$ (kN/m)
Table 7.3 Summary of the installation damage database statistics.

<table>
<thead>
<tr>
<th>Geosynthetic type</th>
<th>Fill types or RF_{ID}</th>
<th>Number of RF_{ID} values computed</th>
<th>Computed range of RF_{ID}</th>
<th>Range of RF_{ID} reported by FHWA (Elias et al. 2001)</th>
<th>Bias statistics X_{ID}^{(g)}</th>
<th>Range</th>
<th>Mean μ_{x_{ID}}^{(g)}</th>
<th>COV_{x_{ID}}^{(g)}</th>
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</thead>
<tbody>
<tr>
<td>Column →</td>
<td>Type 1</td>
<td>10</td>
<td>1.05 – 1.43</td>
<td>1.20 – 1.45</td>
<td>0.88 – 1.19</td>
<td>1.03</td>
<td>0.06</td>
<td></td>
</tr>
<tr>
<td>HDPE uniaxial geogrids</td>
<td>Type 2, 3 and 4</td>
<td>17</td>
<td>0.99 – 1.17</td>
<td>1.10 – 1.20</td>
<td>0.87 – 1.16</td>
<td>1.02</td>
<td>0.04</td>
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</tr>
<tr>
<td>PP biaxial geogrids</td>
<td>Type 1</td>
<td>8</td>
<td>0.97 – 1.45</td>
<td>1.20 – 1.45</td>
<td>0.54 – 1.32</td>
<td>1.05</td>
<td>0.13 (0.20)</td>
<td></td>
</tr>
<tr>
<td>PVC-coated PET geogrids</td>
<td>Type 2, 3 and 4</td>
<td>15</td>
<td>0.94 – 1.11</td>
<td>1.10 – 1.20</td>
<td>0.76 – 1.21</td>
<td>1.05</td>
<td>0.08 (0.11)</td>
<td></td>
</tr>
<tr>
<td>Acrylic- and PP-coated PET geogrids</td>
<td>Type 1</td>
<td>20</td>
<td>1.07 – 1.85</td>
<td>1.30 – 1.85</td>
<td>0.78 – 1.71</td>
<td>1.10</td>
<td>0.14</td>
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</tr>
<tr>
<td></td>
<td>Type 2, 3 and 4</td>
<td>98</td>
<td>0.95 – 1.39</td>
<td>1.10 – 1.30</td>
<td>0.63 – 1.64</td>
<td>1.08</td>
<td>0.10</td>
<td></td>
</tr>
<tr>
<td>Woven geotextiles</td>
<td>Type 1</td>
<td>4</td>
<td>1.48 – 2.02</td>
<td>1.30 – 2.05</td>
<td>0.61 – 1.54</td>
<td>1.08</td>
<td>0.20 (0.23)</td>
<td></td>
</tr>
<tr>
<td>Nonwoven geotextiles</td>
<td>Type 2, 3 and 4</td>
<td>13</td>
<td>1.05 – 1.37</td>
<td>1.20 – 1.40</td>
<td>0.82 – 1.41</td>
<td>1.05</td>
<td>0.09</td>
<td></td>
</tr>
<tr>
<td>Type 1</td>
<td>6</td>
<td>1.20 – 4.93</td>
<td>1.40 – 2.20</td>
<td>0.29 – 2.07</td>
<td>1.06</td>
<td>0.32</td>
<td></td>
<td></td>
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<tr>
<td>Type 2, 3 and 4</td>
<td>18</td>
<td>0.89 – 1.66</td>
<td>1.10 – 1.40</td>
<td>0.78 – 1.29</td>
<td>1.07</td>
<td>0.08</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type 1</td>
<td>6</td>
<td>1.74 – 4.96</td>
<td>1.40 – 2.50</td>
<td>0.53 – 1.80</td>
<td>1.14</td>
<td>0.24</td>
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</tr>
<tr>
<td>Type 2, 3 and 4</td>
<td>6</td>
<td>1.06 – 1.46</td>
<td>1.10 – 1.40</td>
<td>0.82 – 1.51</td>
<td>1.11</td>
<td>0.12</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Woven geotextiles</td>
<td>RF_{ID} ≥ 1.7</td>
<td>4</td>
<td>3.07 – 4.93</td>
<td>NA</td>
<td>0.29 – 2.07</td>
<td>1.05</td>
<td>0.38</td>
<td></td>
</tr>
<tr>
<td>Nonwoven geotextiles</td>
<td>RF_{ID} &lt; 1.7</td>
<td>20</td>
<td>0.89 – 1.68</td>
<td>NA</td>
<td>0.78 – 1.29</td>
<td>1.07</td>
<td>0.08 (0.11)</td>
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</tr>
<tr>
<td>Type 1</td>
<td>7</td>
<td>1.74 – 4.96</td>
<td>NA</td>
<td>0.53 – 1.80</td>
<td>1.14</td>
<td>0.24 (0.26)</td>
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<td></td>
</tr>
<tr>
<td>Type 2, 3 and 4</td>
<td>8</td>
<td>1.05 – 1.46</td>
<td>NA</td>
<td>0.82 – 1.51</td>
<td>1.11</td>
<td>0.12</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 7.3 (continued) Summary of the installation damage database statistics.

Notes:
(a) Including test data for products with mass per unit area $M_A < 270 \text{ g/m}^2$
(b) Minimum allowable $RF_{ID}$ value for design is 1.10
(c) Coarse backfill: Maximum particle size = 100 mm, $D_{50} \approx 30$ mm
(d) Finer backfill: Maximum particle size = 20 mm, $D_{50} \approx 0.7$ mm
(e) Applicable only to products with mass per unit area $M_A \geq 270 \text{ g/m}^2$
(f) All bias data are normally distributed
(g) COV for best fit to lower tail in parentheses; $COV_{X_{id}} = \sigma_{X_{id}} / \mu_{X_{id}}$
**Figure 7.1** Degradation processes for reinforcement strength from as-received (undamaged) to the end of design life. Note: Range bars and installation time are exaggerated for visual presentation.
Figure 7.2 $D_{50}$ particle size range classifications for installation damage soils. Notes: Heavy black lines and shaded regions represent mean and range of distributions from database of installation tests matching soil types (Table 7.2).

Type 1: $D_{50} \geq 19$ mm

Type 2: $19$ mm > $D_{50} \geq 4.75$ mm

Type 3: $4.75$ mm > $D_{50} \geq 0.425$ mm

Type 4: $D_{50} < 0.425$ mm

<table>
<thead>
<tr>
<th>COBBLES</th>
<th>GRAVEL</th>
<th>SAND</th>
<th>SILT or CLAY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse</td>
<td>Fine</td>
<td>Course</td>
<td>Medium</td>
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</tbody>
</table>
Figure 7.3 Installation damage data for nonwoven geotextiles: (a) RF_{ID} versus M_A; (b) x_{ID} versus RF_{ID}; and (c) CDF plots for x_{ID} bias values. Notes: COV for best fit to lower tail statistics shown in parentheses; n = number of data points.
**Figure 7.3 (continued)** Installation damage data for nonwoven geotextiles: (a) $RF_{ID}$ versus $M_A$; (b) $x_{ID}$ versus $RF_{ID}$; and (c) CDF plots for $x_{ID}$ bias values. Notes: COV for best fit to lower tail statistics shown in parentheses; $n =$ number of data points.

(c)
Figure 7.4 Installation damage data for woven geotextiles: (a) $RF_{ID}$ versus $M_A$; (b) $x_{ID}$ versus $RF_{ID}$; and (c) CDF plots for $x_{ID}$ bias values. Note: COV for best fit to lower tail statistics shown in parentheses; $n =$ number of data points.
Figure 7.4 (continued) Installation damage data for woven geotextiles: (a) $R_{ID}$ versus $M_A$; (b) $x_{ID}$ versus $R_{ID}$; and (c) CDF plots for $x_{ID}$ bias values. Note: COV for best fit to lower tail statistics shown in parentheses; $n =$ number of data points.
Figure 7.5 Installation damage data for coated PET geogrids: (a) $x_{ID}$ versus $RF_{ID}$ for PVC-coated geogrids; (b) $x_{ID}$ versus $RF_{ID}$ for Acrylic- and PP-coated geogrids; (c) CDF plots for $x_{ID}$ bias values for PVC-coated geogrids; and (d) CDF plots for $x_{ID}$ bias values for Acrylic- and PP-coated geogrids. Note: COV for best fit to lower tail statistics shown in parentheses and $n = number$ of data points in (c) and (d).
Figure 7.5 (continued) Installation damage data for coated PET geogrids: (a) $x_{ID}$ versus $RF_{ID}$ for PVC-coated geogrids; (b) $x_{ID}$ versus $RF_{ID}$ for Acrylic- and PP-coated geogrids; (c) CDF plots for $x_{ID}$ bias values for PVC-coated geogrids; and (d) CDF plots for $x_{ID}$ bias values for Acrylic- and PP-coated geogrids. Note: COV for best fit to lower tail statistics shown in parentheses and $n =$ number of data points in (c) and (d).
Figure 7.6 Installation damage data for biaxial PP geogrids: (a) $x_{ID}$ versus $RF_{ID}$; and (b) CDF plots for $x_{ID}$ bias values. Note: COV for best fit to lower tail statistics shown in parentheses; $n =$ number of data points.
Figure 7.7 Installation damage data for HDPE uniaxial geogrids: (a) $x_{ID}$ versus $RF_{ID}$; and (b) CDF plots for $x_{ID}$ bias values. Note: COV for best fit to lower tail statistics shown in parentheses; $n =$ number of data points.
Figure 7.8 Influence of installation bias variability (COVx0) on probability of failure due to reinforcement rupture: (a) reinforcement loads calculated using AASHTO Simplified Method (μx0 = 0.30 and COVx0 = 0.50); and (b) reinforcement loads calculated using perfect load model (μx0 = 1 and COVx0 = 0). Notes: μxcr = μx0 = 1; COVxcr = COVx0 = 0.10.
Figure 7.9 Spread in computed installation damage bias values (COV\textsubscript{\textit{id}}) versus computed installation damage factor (RF\textsubscript{id}).
Chapter 8
Interpretation of Laboratory Creep Testing for Reliability-based Analysis and Load and Resistance Factor Design (LRFD) Calibration

8.1 Introduction

Stability design of geosynthetic reinforced soil walls, slopes and embankments requires that an estimate of the available long-term available tensile strength (T_{al}) of each geosynthetic reinforcing layer be made. In North American practice using allowable stress design (ASD), T_{al} is computed as follows (FHWA – Elias et al. 2001; AASHTO 2002; NCMA 2009; CFEM 2006):

\[ T_{al} = \frac{T_{ult}}{RF} = \frac{T_{ult}}{RF_{ID} \times RF_{CR} \times RF_{D}} \]

where, T_{ult} = ultimate tensile strength of the reinforcement and RF = product of reduction factors to account for potential long-term strength loss due to installation damage (RF_{ID}), creep (RF_{CR}) and degradation due to chemical/biological processes (RF_{D}). Typically an overall factor of safety (FS) is used in allowable stress design (ASD) to account for other project uncertainties so that \( T_{al}/T_{max} \geq FS = 1.5 \) (AASHTO 2002; NCMA 2009; CFEM 2006).

Figure 8.1 illustrates the interpretation of Equation 8.1 with respect to the design life of a geosynthetic reinforcement layer in a soil structure. In current deterministic ASD practice the available tensile strength (T_{al}) must be at least equal to the maximum predicted tensile load (T_{max}) in a reinforcement layer. Typically the value of T_{ult} used in Equation 8.1 is a certified minimum value or Minimum Average Roll Value (MARV) based on ASTM D4595 or ASTM D6637 test methods and supplied by the product vendor. The interpretation of design T_{ult} strength values in the context of this investigation is described later in this chapter.

Strength reductions due to installation damage are assumed to occur at construction, and protocols to carry out these installation damage trials are easily available (e.g. WSDOT T925 2005; ASTM D5818). The creep strength reduction factor RF_{CR} is taken as the ratio of the original strength to retained (creep-rupture) strength at the target design life of the reinforced soil structure. In North America and worldwide the laboratory methodologies to determine the creep-
rupture strength of reinforcement geosynthetics are well-established (ASTM D5262; ASTM D6992; EN ISO 13431 1999; EN ISO/TR 20432 2007; WSDOT T925 2005). The general approach is to generate a composite creep rupture envelope from conventional and accelerated creep testing that extends to at least the design life of the reinforcement (i.e. reinforced soil structure). In practice, a simple log-linear equation for the rupture envelope is typically used, assuming that time-dependent transitions/changes in creep rupture behaviour do not occur:

\[ t = P_0 + b \cdot \log t \]  

where, \( P_t \) is the (predicted) fraction of original strength retained relative to the mean ultimate tensile strength measured for the sample used for the creep testing, \( P_{\text{alt,meas}} \); \( t \) is the time in hours; \( P_0 \) and \( b \) are constants determined from conventional least square linear regression analysis. Example composite creep rupture envelopes and log-linear approximations to the data are shown in Figure 8.2. The creep reduction factor, \( RF_{CR} \) at time \( t \) is computed using the inverse of the fraction of strength retained using Equation 8.2, hence:

\[ RF_{CR} = \frac{1}{P_t} \]

The calculation is demonstrated in Figure 8.2. Close inspection of the HDPE data in Figure 8.2 shows that there is a visually relatively small under-prediction of creep-reduced strength values at short times (say \( t < 1 \) hour) and at long times (\( t > 75 \) years). Hence, a linear regression using log-log data may be slightly more accurate. However, North American practice is to fit a first order approximation to retained strength values plotted against log of time. This approach is likely slightly conservative (safe for design) for HDPE materials but not expected to influence the calculation of creep reduction factor or creep bias statistics in any practical way. A detailed description of issues related to extrapolation of creep data to design lives beyond measured data can be found in the WSDOT T925 (2005) guidance document.

The topic of this chapter is related to strength losses due to creep of polymeric geosynthetic products. However, the focus is not on the calculation of \( RF_{CR} \) values for design; rather this study focuses on calculation of bias statistics for creep. These statistics are required for reliability-based
analysis and LRFD calibration for the geosynthetic rupture limit state that is a necessary part of
stability design of reinforced soil walls, slopes and embankments.

8.2 Reliability-based analysis and LRFD calibration for rupture limit state

The AASHTO (2009) and CSA (2006) design guidance documents are now committed to a load
and resistance factor design (LRFD) approach for mechanically stabilized earth wall (MSEW)
structures; this includes geosynthetic reinforced soil walls.

The general approach for reliability-based analysis and design can be found in text books by
Harr (1987), Ang and Tang (1975) and Nowak and Collins (2000). Load and resistance factors
used in LRFD codes can be computed by back-fitting to ASD past practice. However, this is not
satisfactory if the objective is to select load and resistance factors that meet a prescribed
probability of failure for each limit state and to ensure that the same probability of failure applies
to all limit states for a set of failure modes (e.g. internal stability of reinforced soil walls).
Furthermore, there are uncertainties associated with each of the strength-reduction mechanisms
identified above and these are illustrated by the range bars in Figure 8.1. In ASD these
uncertainties are accounted for in a subjective manner by using an overall factor of safety equal to
(say) 1.5. Reliability-based design and LRFD calibration require statistical estimates of the
variables that appear in the underlying deterministic models for load and resistance in limit state
equations.

The focus of this chapter is related to the limit state function for tensile rupture (over-stressing) of
geosynthetic layers in reinforced soil walls which can be expressed as:

\[ 8.4 \quad \phi T_{al} - \gamma Q T_{max} \geq 0 \]

Here, \( T_{max} \) is the nominal maximum design tensile load in the reinforcement layer under
operational conditions over the life of the structure, \( \phi \) is the resistance factor for the rupture limit
state and \( \gamma Q \) is the load factor. For example, in the AASHTO design specifications, \( \gamma Q = 1.35 \) is
the load factor due to soil self-weight plus permanent uniform surcharge pressure and the
corresponding resistance factor is \( \phi = 0.90 \). The \( T_{max} \) value can be computed using deterministic
models such as the AASHTO Simplified Method (AASHTO 2009) or the K-Stiffness Method
(Bathurst et al. 2008).
However, the deterministic models used in soil-structure design problems have varying accuracy when compared to measured load and resistance values where these comparisons can be made. The general approach to correct deterministic model predictions is to introduce bias values where the bias value for a particular random variable is expressed generically as $X = \frac{\text{measured value}}{\text{predicted value}}$ (Nowak and Collins 2000; Withiam et al. 1998). The general approach has been demonstrated by Allen et al. (2005) and Bathurst et al. (2008) for LRFD calibration of the pullout limit state for steel grid reinforced soil walls.

Bias statistics are influenced by model bias (i.e. accuracy of the deterministic model representing the mechanics of the load or resistance side of the limit state equation), random variation in input parameter values, spatial variation in input values (if applicable), quality of data, and consistency in interpretation of data when data are gathered from multiple sources (the typical case) (Allen et al. 2005). Incorporating bias statistics into LRFD calibration and assuming only one load type, Equation 8.4 can be expressed as:

$$[8.5] \quad \gamma Q x_R - \phi x_Q \geq 0$$

Here, $x_R = $ resistance bias computed as the ratio of measured resistance to calculated (predicted) nominal resistance and $x_Q = $ load bias computed as the ratio of measured load to the calculated (predicted) nominal load. For the load side:

$$[8.6] \quad x_Q = \frac{T_{\text{max,meas}}}{T_{\text{max}}}$$

where, $T_{\text{max,meas}}$ is the measured maximum tensile load in a reinforcement layer. For example, measured maximum tensile loads under operational conditions have been calculated to be on average about one third of predicted values using the AASHTO Simplified Method (e.g. Allen et al. 2003; Bathurst et al. 2008). The ratio of total strength capacity from all reinforcement layers to load demand has been estimated to be even higher for individual structures (Allen et al. 2002). For the resistance side, the strength bias value is expressed as:

$$[8.7] \quad x_R = \frac{T_{\text{al,meas}}}{T_{\text{al}}}$$
where, $T_{al, meas}$ is the measured long-term tensile strength in a reinforcement layer. The approach adopted here is to equate this value to the predicted long-term allowable strength by introducing bias statistics for each of the three strength loss terms in Equation 8.1. Hence:

$$[8.8] \quad T_{al, meas} = T_{al} \cdot X_R = \frac{T_{ult}}{RF_{ID} \times RF_{CR} \times RF_D} \cdot X_R = \left(\frac{T_{ult}}{RF_{ID}}\right) \cdot X_{ID} \cdot \left(\frac{1}{RF_{CR}}\right) \cdot X_{CR} \cdot \left(\frac{1}{RF_D}\right) \cdot X_D$$

In practice, the $T_{ult}$ value used for design is the minimum certified value provided by the reinforcement manufacturer for the product based on the manufacturer’s quality control testing. Typically, this minimum certified value is set at two standard deviations below the mean value (defined as a MARV = Minimum Average Roll Value) to ensure that in most cases, the actual product tensile strength is greater than the specified value. This practice results in a bias for the tensile strength that is greater than 1.0 (conservative for design). A useful description of the minimum certified value and its application to design and quality control/quality assurance (QC/QA) is provided in WSDOT T925 (2005) and NTPEP (2007) documents. In this investigation, the minimum certified value for each product was not available for all source datasets. Furthermore, a manufacturer may sometimes report a lower ultimate tensile strength value than that computed from actual testing in order to position the product in the market, to consider variation in reported results between laboratories, or to be cautious.

The bias of the allowable long-term tensile strength of the reinforcement, $X_R$, is expressed as a product of three bias values:

$$[8.9] \quad X_R = X_{ID} \cdot X_{CR} \cdot X_D$$

where, $X_{ID}$, $X_{CR}$ and $X_D$ are bias values for tensile strength after installation damage, after creep and after degradation due to chemical or environmental factors, respectively. Variability in undamaged specimen strength is captured by the variability in installation damage bias values as shown in Equation 8.8. Assuming that $X_{ID}$, $X_{CR}$, and $X_D$ are uncorrelated, the mean and coefficient of variation (COV) of $X_R$ values are given by (Ang and Tang 1975):

$$[8.10] \quad \mu_{X_R} = \mu_{X_{ID}} \cdot \mu_{X_{CR}} \cdot \mu_{X_D}$$
Here, the subscripts for each of the mean and COV terms can be matched to installation damage, creep and degradation reduction factors identified earlier. In theory, Equations 8.10 and 8.11 are only valid for uncorrelated log-normal distributions of the three random variables. However, for uncorrelated normal distributions with small coefficients of variation these expressions are sufficiently accurate. For a perfect deterministic model in which there is no error in the estimation of each of the strength reduction terms in Equation 8.8, then Equation 8.8 is equivalent to the deterministic model used in ASD past practice (i.e. Equation 8.1). For this case, the mean value of each set of bias values is one \( (X_{ID} = X_{CR} = X_{D} = 1) \) and the corresponding COV values are zero. This can be understood to be an unlikely situation for any geotechnical limit state.

Once frequency distributions for load bias \( (X_{Q}) \) and resistance bias \( (X_{R}) \) are known, bias statistics for both quantities can be approximated (i.e. normal or log-normal mean and COV of each distribution). Monte Carlo simulations can then be carried out to find the magnitude of resistance factor \( (\phi) \) to match a prescribed load factor \( (\gamma_Q) \) and a target probability of failure. These calculations can be done using an Excel spreadsheet (Allen et al. 2005).

At the time of this study, load bias statistics using the AASHTO Simplified Method and the K-Stiffness Method have been published (e.g. Bathurst et al. 2008). However, bias statistics for installation damage, creep and chemical degradation have not been determined. This chapter is focused on the determination of the bias statistics for the creep-reduced strength. These statistics are required for reliability-based design and LRFD calibration. Reliability-based design involves using prescribed load and resistance factors (e.g. \( \gamma_Q \) and \( \phi \)) recommended in design codes together with computed characteristic load value(s) (e.g. \( T_{max} \)) to select a material with adequate resistance (i.e. strength \( T_{al} \)) that satisfies a limit state function (e.g. Equation 8.4). The objective of LRFD calibration is to determine load and resistance factor values found in design codes so that the probability of failure associated with each limit state function is greater than an acceptable minimum value.
The strategy to meet the objectives of this study was to first collect laboratory creep test data from available sources. Bias statistics were then computed for different categories of geosynthetic (types). The determination of similar bias statistics for installation damage and durability reduction factors and the results of LRFD calibration for the long-term rupture limit state are underway at the time of writing and the results reserved for future publications.

8.3 Background to creep behaviour and testing

The analysis of creep as it relates to geosynthetic (polymeric) reinforcement products has been a major line of investigation dating back to the 1980s (e.g. Allen et al. 1982; Andrawes et al. 1984; Yeo 1985; Allen 1991). Geosynthetic reinforcement products are visco-elastic-plastic materials that can creep to rupture under constant load provided that the load is above a threshold value (e.g. McGown et al. 1984; Allen 1991). Major variables influencing the creep response of geosynthetics are polymer type and grade, manufacturing method, load level, and temperature. In current design practice, the in-air load-strain-time behaviour of geosynthetics is assumed to apply to in-soil behaviour.

The long-term creep data needed to assess bias statistics for creep can be obtained from stepped-isothermal method (SIM) testing (ASTM D6992) or from block shifting of single-temperature creep tests obtained at multiple temperatures. These data are then used to construct a composite creep rupture envelope (ASTM D5262 and WSDOT T925). Today, the SIM is used to predict lifetime rupture strengths out to 100 years or more (Equation 8.2). The details of this method were first described by Thornton et al. (1998a,b). The SIM greatly reduces test time compared to conventional time-temperature superposition methods that use time-shifting of creep-rupture data from multiple specimens at different temperatures (Koerner et al. 1992). In addition, the problems of specimen-to-specimen variation are avoided. Greenwood et al. (2004) demonstrated that creep-rupture curves for polyester yarns using the SIM and conventional creep testing were in satisfactory agreement.

Allen and Bathurst (1996) investigated the combined effect of polymeric creep and installation damage using a database of constant load (creep) data for both undamaged and installation-damaged geosynthetic specimens. Based on the data available they concluded that multiplication of creep reduction (RF<sub>CR</sub>) and installation damage factors (RF<sub>ID</sub>) may be conservative and hence results in errors on the safe side for current ASD practice. Greenwood (2002) came to the same conclusion based on stepped isothermal creep-rupture tests performed on a polyester geosynthetic in undamaged and damaged states.
Today, the National Transportation Product Evaluation Program (NTPEP) of the American Association of State Highway and Transportation Officials (AASHTO) is collecting test data for commercial reinforcement products including creep data using accepted test standards as described in **WSDOT T925 (2005)**. This publicly available database plus data collected by the writer from other sources can now be used to generate statistics that are necessary to perform LRFD calibration of the rupture limit state for stability design of reinforced soil structures.

### 8.4 Database of laboratory creep test data

**Table 8.1** gives a summary of the database of test results used to calculate creep bias statistics. The database consists of 18 groups containing a total of 395 data points (i.e. creep rupture values). The products tested include geogrids, geotextiles and yarns. The constituent polymers were polyester (PET), polypropylene (PP) and high density polyethylene (HDPE). A group of data refers to data points used to construct a single creep rupture strength envelope. These data may include test results for test specimens from one product or several similar products in a manufacturers’ product line. For example, data group 1 is comprised of five different products in one product line that have mean measured ultimate tensile strengths ($T_{\text{ult,meas}}$) of 46.0, 62.7, 102.1, 138.6 and 370.3 kN/m (**ASTM D6637**). In this chapter, $T_{\text{ult,meas}}$ is the mean of multiple measured tensile strength values from original specimens. The creep rupture envelope for this group of data is reproduced in **Figure 8.2**. The creep-rupture curve is assumed to be representative of other products in the same product line but having different baseline strengths. Conventional creep testing was carried out in accordance with **ASTM D5262** and accelerated creep testing (SIM) was done in accordance with **ASTM D6992**. In many cases, elevated temperature testing of multiple specimens (block) of the same product sample was used to create the creep rupture envelope. In all but two cases the creep rupture data was taken beyond 75 years (the recommended minimum time by **AASHTO (2009)** for permanent structures). Hence, the issue of accuracy of time extrapolation of data was avoided and possible breaks (knees) in the log-linear trend of the data were not encountered which may occur for some polyolefins (**Popelar et al. 1991**). Finally, all of the data used in this study refer to materials at 20ºC. Methodologies to correct creep-rupture data to other temperatures and to account for daily and seasonal changes in ground temperature for design can be found in **WSDOT T925 (2005)**.
8.5 Calculation of creep and ultimate strength bias values

Bias value $X_{CR}$ is the ratio of the measured strength ($T'_i$) at time $t$ to the predicted value ($T_t = P_t \cdot \bar{T}_{ult,meas}$) using Equations 8.2 and 8.3:

$$X_{CR} = \frac{T'_t}{T_t} = \frac{T'_t}{T_{ult,meas} / RF_{CR}} = \frac{T'_t}{T_{ult,meas} (P_0 + b \cdot \log t)} = \frac{P'_t}{P_t}$$

Here, $P'_t$ is the measured fraction of ultimate strength retained at time. An example calculation sheet to compute creep bias statistics is shown in Table 8.2. The variation of $RF_{CR}$ (i.e. $X_{CR}$ values not equal to one) may be due to the inherent variability between material specimens, testing procedure or possibly the analytical approach (i.e. Equation 8.2).

Inherent variability in original reinforcement strength can be quantified using the bias of measured tensile strength, $X_{ult}$, computed as:

$$X_{ult} = \frac{T_{ult,meas}}{\bar{T}_{ult,meas}}$$

where, $T_{ult,meas}$ is the original (as-received) tensile strength of a specimen. The calculation is demonstrated in Table 8.3. Similar data from a large number of sources is used later to compare variability of original strength (COV of $X_{ult}$) with variability of creep-reduced strength (COV of $X_{CR}$). Table 8.4 provides a summary of the database and source materials collected by the writer to compute original strength bias statistics.

8.6 Results of analyses

Statistics of bias values are summarized in Table 8.5. Variability in original tensile strength ($T_{ult,meas}$) values is addressed first. The COV of $X_{ult}$ values is 0.047 for geotextiles, and 0.036 and 0.20 for PET and polyolefin geogrids, respectively. The lower COV value (less spread in data) for the polyolefin geogrids is visibly detectable in Figure 8.3a where $X_{ult}$ datasets are plotted against $T_{ult,meas}$. The trend in magnitude of COV values is consistent with the relative complexity of the geosynthetic structures in each of the three categories shown. Nevertheless, from a practical point of view the COV of $T_{ult,meas}$ bias values are small and vary over a small range (i.e. 2% to 5%).

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Figure 8.3b shows $X_{CR}$ data versus logarithm of time in hours. Since the variation in $X_{CR}$ with respect to product type is very small, product groups are not identified. As noted earlier, reinforced soil walls are typically designed for a minimum 75-year service. The visual impression from the figure is that the simple regression model (Equation 8.2) does not result in time-dependent $X_{CR}$ values within design lifetimes up to approximately 120 years. There is evidence of possible time dependency at longer times since there are some relatively large $X_{CR}$ values beyond $t = 120$ years. This may indicate that the simple log-linear model is not applicable beyond 120 years and a more complex model such as the double logarithmic regression method is appropriate (Small and Greenwood 1992). However, the focus of the current investigation is on shorter design lifetimes and hence the observations noted here are not of practical concern. A practical outcome of the observed small $X_{CR}$ values in Figure 8.3b is that creep testing beyond (say) 1000 hours may not be necessary if the creep rupture mechanism is log-linear for all times of interest. In every case examined in the investigation, extrapolation of data over the first 1000 hours gave the same predicted RF$_{CR}$ value at 75 years with the same accuracy using Equation 8.2 fitted to the entire data set. However, this technique is only valid if the mechanism of creep rupture and the creep rupture envelope function do not change during the targeted design lifetime for the material.

CDF plots for $X_{CR}$ and $X_{ult}$ bias values are presented in Figure 8.3c. The individual $X_{ult}$ plots are visually very close to the CDF curve computed for all $X_{CR}$ data. The COV of $X_{ult}$ values for all original strength measurements is 0.038 versus 0.044 for all creep bias values ($X_{CR}$). It can be argued that the difference between original and reduced-strength bias values is a result of creep testing and interpretation. However, at two significant figures the values for both data are 4%. Hence, it can be assumed that current practice for the determination of RF$_{CR}$ does not introduce additional uncertainty to the calculation of long-term allowable reinforcement strength. This validates the assumption in WSDOT T925 (2005) that creep strength is proportional to the short-term tensile strength. Furthermore, the variability of $X_{CR}$ appears to be fully the result of the variability in the tensile strength of each product which is taken care of in the statistics for installation damage bias ($X_{ID}$ in Equation 8.8). In practical terms the mean of $X_{CR}$ can be taken as one and the COV as zero in Equations 8.10 and 8.11; thus the magnitude of RF$_{CR}$ for the geosynthetic rupture limit state in LRFD can be treated as deterministic.

These conclusions do not apply to installation damage ($X_{ID}$) bias values and the reduction factor for installation damage (RF$_{ID}$). Preliminary investigation by the writer shows that the COV for $X_{ID}$ values can be as high as 0.30. This is not unexpected since the magnitude of installation
damage-induced strength reduction for nominally identical tests can vary over a wide range depending on the geosynthetic type, backfill soil and compaction method. For the load side of the rupture limit state function, both mean and COV for $X_Q$ values have been calculated to be about 0.30 and 0.50, respectively for cohesionless backfill soils using the AASHTO Simplified Method (Allen et al. 2003; Bathurst et al. 2008). Hence, the results of future LRFD calibration for the geosynthetic rupture limit state in reinforced soil structures can be expected to be most sensitive to the accuracy of the underlying load model to compute $T_{\text{max}}$.

8.7 Conclusions and implications to design

Stability design of geosynthetic reinforced soil walls, slopes and embankments using an LRFD approach requires calibrated load and resistance factors to ensure that design outcomes satisfy an acceptable probability of failure. To select the resistance factor used in the limit state calculation for reinforcement rupture, bias statistics for installation damage, creep and durability are required or at least must be estimated with reasonable confidence. This chapter has focused on the generation of bias statistics for creep-reduced strength using a database of creep tests from many sources and the implications of these data to LRFD and LRFD calibration for the geosynthetic rupture limit state.

The analysis results show that variability in the prediction of creep-reduced strength is very low and is largely captured by the magnitude of variance in the original strength ($T_{\text{ult}}$) of the test specimens. For LRFD calibration purposes, the variability in accuracy of creep predictions is effectively contained in the bias statistics for installation damage. This greatly simplifies future LRFD calibration using reliability-based analysis.

An important implication of the analyses reported here is that the value of the strength reduction factor for creep ($R_{F_{CR}}$) used in LRFD can be taken as deterministic. Nevertheless, this general recommendation applies only to geosynthetic reinforcement products that exhibit a composite log-linear creep-rupture curve obeying Equation 8.2 over the design lifetime of interest. A practical observation in this investigation is that in every available case, creep testing up to 1000 hours would have been sufficient to estimate the 75-year design creep reduction factor value using the log-linear fit to the first 1000 hours with the same accuracy using the regressed model fitted to all the data. This is particularly true for data using the SIM protocol (ASTM D6992) which removes specimen variability as a factor in the interpretation of creep test results.
Finally, the values of $RF_{CR}$ for a design lifetime of 75 years are summarized in Table 8.1. These values provide a useful benchmark for future product approval programs and for preliminary design purposes. However, creep reduction factors can be expected to decrease over time as improvements in manufacturing and polymer formulation occur.

REFERENCES


### Table 8.1 Summary of the creep database.

<table>
<thead>
<tr>
<th>Data group</th>
<th>Reinforcement material</th>
<th>Number of different products</th>
<th>Range of $\overline{\mathbf{T}}_{ult,meas}$ (kN/m)</th>
<th>Number of data points</th>
<th>Test Method</th>
<th>Range of creep rupture time (log-hours)</th>
<th>$RF_{CR}$ (75 years)</th>
<th>COV$_{XCR}$</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>PET geogrids</td>
<td>5</td>
<td>52.9 – 436</td>
<td>39</td>
<td>Block (c)</td>
<td>0.30 – 6.88</td>
<td>1.57</td>
<td>0.02</td>
<td>NTPEP (2008a)</td>
</tr>
<tr>
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<td>PET geogrids</td>
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<td>0.03</td>
<td>NTPEP (2009a)</td>
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<td>Block SIM</td>
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<td>0.03</td>
<td>NTPEP (2009b)</td>
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<td>Wrigley et al. (1999)</td>
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\[ \sum = 395 \]

Notes:
(a) Unprotected PET yarns stitched to a backing sheet of nonwoven polypropylene
(b) Reported in source data
(c) A “block” of specimens taken from a sample and each specimen tested at a difference temperature
(d) A single specimen tested using the Stepped Isothermal Method (SIM)
(e) Times to rupture correspond to temperatures at 20°C
(f) COV = coefficient of variation = standard deviation/mean
(g) NA = not available. Baseline tensile strengths not reported
Table 8.2 Example calculation sheet for $X_{CR}$ using a composite creep rupture envelope constructed from SIM and conventional creep test results for three woven polyester geogrid products in one product line (source: NTPEP 2008b).

<table>
<thead>
<tr>
<th>Test Method</th>
<th>Product designation</th>
<th>Log-time to creep rupture, log t (hours)</th>
<th>Measured retained strength, $P'_t$ (%)</th>
<th>Predicted retained strength, $P_t$ (%) (Equation 2)</th>
<th>Creep bias value, $X_{CR}$</th>
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<td>4.72</td>
<td>75.00</td>
<td>74.41</td>
<td>1.008</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>6.32</td>
<td>69.99</td>
<td>69.16</td>
<td>1.012</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>6.64</td>
<td>65.01</td>
<td>68.11</td>
<td>0.954</td>
</tr>
<tr>
<td>Conventional</td>
<td>C</td>
<td>0.84</td>
<td>88.00</td>
<td>87.09</td>
<td>1.010</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>2.16</td>
<td>82.00</td>
<td>82.79</td>
<td>0.990</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>2.43</td>
<td>80.00</td>
<td>81.91</td>
<td>0.977</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>3.57</td>
<td>78.00</td>
<td>78.18</td>
<td>0.998</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>3.86</td>
<td>76.00</td>
<td>77.23</td>
<td>0.984</td>
</tr>
<tr>
<td></td>
<td>C *</td>
<td>1.27</td>
<td>84.90</td>
<td>85.70</td>
<td>0.991</td>
</tr>
<tr>
<td></td>
<td>C *</td>
<td>3.19</td>
<td>81.67</td>
<td>79.40</td>
<td>1.029</td>
</tr>
<tr>
<td></td>
<td>C *</td>
<td>3.76</td>
<td>80.00</td>
<td>77.55</td>
<td>1.032</td>
</tr>
</tbody>
</table>

| Mean         | 1.000                |
| Standard deviation | 0.030               |
| $\text{COV}_{X_{CR}}$ | 0.030               |

Regressed log-linear function of data in Column 3 and Column 4: $P_t$ (%) $= -3.2738 \log t + 89.851$

Note: * multi-rib rather than single rib tests
Table 8.3 Example calculation sheet for computation of bias values ($X_{ult}$) for tensile strength of original (as-received) geosynthetic specimens (source: Laboratory 2 unpublished data for woven polyester geogrid).

<table>
<thead>
<tr>
<th>Specimen number</th>
<th>Original strength, $T_{ult,meas}$ (kN/m)</th>
<th>$X_{ult}$ (Equation 13)</th>
</tr>
</thead>
<tbody>
<tr>
<td>[1]</td>
<td>[2]</td>
<td>[3] = $[2]/\bar{T}_{ult,meas}$</td>
</tr>
<tr>
<td>1</td>
<td>36.9</td>
<td>1.037</td>
</tr>
<tr>
<td>2</td>
<td>35.0</td>
<td>0.983</td>
</tr>
<tr>
<td>3</td>
<td>35.8</td>
<td>1.006</td>
</tr>
<tr>
<td>4</td>
<td>36.6</td>
<td>1.028</td>
</tr>
<tr>
<td>5</td>
<td>34.9</td>
<td>0.980</td>
</tr>
<tr>
<td>6</td>
<td>34.8</td>
<td>0.978</td>
</tr>
<tr>
<td>7</td>
<td>37.0</td>
<td>1.039</td>
</tr>
<tr>
<td>n = 8</td>
<td>33.9</td>
<td>0.952</td>
</tr>
</tbody>
</table>

Mean

\[
\bar{T}_{ult,meas} = \frac{\sum_{i=1}^{n} T_{ult,meas}}{n} = 35.6 \\
\mu_{X_{ult}} = 1.00
\]

Standard deviation

\[
\sigma_{X_{ult}} = 0.032
\]

COV

\[
\text{COV}_{X_{ult}} = 0.032
\]
Table 8.4 Summary of original (as-received) specimen database.

<table>
<thead>
<tr>
<th>Geosynthetic type</th>
<th>Number of products</th>
<th>Number of product lines</th>
<th>Number of specimens</th>
<th>Data sources (^{(a)})</th>
</tr>
</thead>
<tbody>
<tr>
<td>HDPE uniaxial geogrids</td>
<td>7</td>
<td>3</td>
<td>58</td>
<td>4, 6, 12</td>
</tr>
<tr>
<td>PP biaxial geogrids</td>
<td>4</td>
<td>1</td>
<td>40</td>
<td>12</td>
</tr>
<tr>
<td>PVC-coated PET geogrids</td>
<td>39</td>
<td>14</td>
<td>438</td>
<td>8, 9, 10, 11, 12, 14, 15, 16, 17, 18, 19, 20</td>
</tr>
<tr>
<td>Acrylic-coated and PP-coated PET geogrids</td>
<td>5</td>
<td>2</td>
<td>58</td>
<td>12, 13</td>
</tr>
<tr>
<td>Woven geotextiles</td>
<td>10</td>
<td>4</td>
<td>113</td>
<td>2, 3, 5, 7, 11</td>
</tr>
<tr>
<td>Nonwoven geotextiles</td>
<td>4</td>
<td>3</td>
<td>92</td>
<td>1, 2</td>
</tr>
</tbody>
</table>

\[ \sum = 69 \quad \sum = 27 \quad \sum = 799 \quad \sum = 20 \]

Notes:
Table 8.5 Summary of creep-reduced and original strength bias statistics.

<table>
<thead>
<tr>
<th>Number of data points, n</th>
<th>Mean</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bias of original ultimate strength, $X_{ult}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>All data</td>
<td>799</td>
<td>1.00</td>
</tr>
<tr>
<td>Geotextiles</td>
<td>205</td>
<td>1.00</td>
</tr>
<tr>
<td>Geogrids (PET)</td>
<td>496</td>
<td>1.00</td>
</tr>
<tr>
<td>Geogrids (PP, HDPE)</td>
<td>98</td>
<td>1.00</td>
</tr>
<tr>
<td>Bias of creep-reduced strength, $X_{CR}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>All data</td>
<td>395</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Note: statistics are taken from all data points in each population.
Figure 8.1 Degradation processes for reinforcement strength from as-received (undamaged) to the end of design life.

Note: Vertical scale, range bars and installation time are exaggerated for visual presentation.
Figure 8.2 Example composite creep rupture envelopes.
Figure 8.3 (a) Bias of original strength versus mean of strength from multiple specimens; (b) Bias of creep strength versus logarithm of time; and (c) CDF plots for the bias values of original strength and creep-reduced strength respectively.

Note: $X_{ult}$ bias data computed from data sources reported in Table 8.4; bias statistics summarized in Table 8.5.
Chapter 9

Load and Resistance Factor Design (LRFD) Calibration for Geogrid Pullout Limit State Using the AASHTO Simplified Method

9.1 Introduction

Limit states design, which is called load and resistance factor design (LRFD) in the United States, has been used for structural engineering design for decades. To ensure a common approach for the design of both steel/reinforced concrete structures and soil structures, the American Association of State Highway and Transportation Officials (e.g. AASHTO 2007) and the Canadian Highway Bridge Design Code (e.g. CSA 2006) now recommend LRFD for all structures, including reinforced soil retaining walls (Mechanically Stabilized Earth Walls (MSEWs) in USA terminology). To date, the strategy to migrate from allowable stress design (ASD) past practice (factor of safety approach) has been to back-calculate resistance factors for each limit state based on recommended factors of safety in ASD past practice (e.g. AASHTO 2002) and prescribed load factors. This approach has the disadvantage that there is no guarantee that an acceptable target probability of failure is achieved for the limit state considered or that the corresponding probability of failure is consistent with values for other limit states in a set of calculations (e.g. internal stability limit states for reinforced soil walls).

This chapter reports reliability-based LRFD calibration for the geogrid pullout limit state in geosynthetic reinforced soil walls subjected to soil self-weight plus permanent uniform surcharge loading. Calibration can now be attempted because a large database of reinforcement loads from full-scale instrumented walls (e.g. Bathurst et al. 2008b) and geogrid pullout capacity statistics from a large database of laboratory tests (Huang and Bathurst 2009) are now available.

This chapter first demonstrates that the current resistance factor $\phi = 0.90$ recommended by AASHTO (2007, 2009) for reinforcement pullout for MSEW structures cannot be justified. Next, this chapter proposes modifications to the current AASHTO Simplified Method and a new default pullout model that can be used to improve the prediction accuracy of load and resistance capacity for the pullout limit state. Finally, this chapter investigates the influence on pullout design of current empirical criteria that restrict the minimum reinforcement length regardless of pullout capacity.
9.2 Load and resistance factor design (LRFD) concepts

In load and resistance factor design, engineers use prescribed limit state equations and load and resistance factors specified in design codes to ensure that a target probability of failure for each load carrying member in a structure is not exceeded. The objective of LRFD calibration in the context of this chapter is to compute load and resistance factor values to meet this same objective using measured load and resistance data rather than fitting to ASD past practice. The fundamental limit state expression used in LRFD is:

\[ \varphi R_n \geq \sum \gamma_i Q_{ni} \]

Here, \( Q_{ni} \) = nominal (specified) load; \( R_n \) = nominal (characteristic) resistance; \( \gamma_i \) = load factor; and \( \varphi \) = resistance factor. In design codes, load factor values are typically greater than or equal to one and resistance factor values are always less than or equal to one.

It is important to emphasize that for bridge design, a nominal load is not a failure load but rather a value that is a best estimate of the load under operational conditions (Harr 1987). For example, this nominal load may be due to structure dead loads plus a representative vehicle load based on statistical treatment of bridge traffic. Conceptually, the margin of safety is largely provided by the resistance side of the equation where the resistance value is calculated based on the failure capacity (ultimate limit state) or a deformation criterion (serviceability limit state) for each element analyzed. For the case of a steel member, the ultimate resistance of the member is based (typically) on flexure or shear capacity, and serviceability on a prescribed allowable deformation.

The same concepts described above must apply to strength limit states for internal stability design of geosynthetic reinforced soil walls using LRFD. The reinforcement loads due to soil self-weight can be estimated using the current AASHTO (2007, 2009) Simplified Method. A common source of confusion and conflict with LRFD using the Simplified Method is that the underlying deterministic model to calculate reinforcement loads is based on active earth pressure theory or Coulomb wedge analysis and hence the soil and critical reinforcement layers are assumed to be simultaneously at incipient failure. Even if this unlikely coincidence was accepted \( a \ priori \) at an ultimate limit state, it is reasonable to expect that the operational loads in the very large number of successful walls in place today are very much lower than loads predicted using methods extrapolated from classical active earth pressure theory (i.e. Simplified Method and variants).
reinforcement strains in monitored field walls that have behaved well under operational conditions have stayed the same or strain rates have decreased with time. Equivalently, there is evidence in some monitored walls of reinforcement load relaxation with time following construction (Allen and Bathurst 2002b; Bathurst et al. 2005; Tatsuoka et al. 2004; Kongkitkul et al. 2010). Hence, tensile reinforcement loads at the end-of-construction condition are the maximum loads for LRFD provided original site and boundary conditions for which the wall was designed do not change.

Despite the shortcomings of limit equilibrium-based methods for the design of geosynthetic reinforced soil walls noted above in the context of observed behaviour and LRFD calibration, the Simplified Method is the only method currently available in AASHTO (2007, 2009) and FHWA (2001) guidance documents to estimate tensile loads in geosynthetic reinforcement layers. Variations of the Simplified Method for steel reinforced soil walls are also found in the AASHTO, FHWA and BS8006 (1995) design codes. In general, the AASHTO Simplified Method for steel reinforced soil walls does well in predicting reinforcement loads under operational conditions due to soil self-weight (Allen et al. 2004; Bathurst et al. 2009). This is because empirical adjustments were made to the coefficient of active earth pressure to match measured reinforcement loads in steel reinforced soil walls under operational conditions. Hence, a strategy to preserve the AASHTO Simplified Method for internal stability LRFD of geosynthetic reinforced soil walls is to use an empirically adjusted model to compute reinforcement loads under end-of-construction (operational) conditions. To the best of the writer’s knowledge, no attempt has been made to adjust the AASHTO Simplified Method to match operational tensile loads in geosynthetic reinforced soil walls. This was due initially to lack of measured strain (or load) data and the challenge of estimating the load in reinforcement layers from strain measurements. The writer and co-workers have overcome these obstacles by: (a) collecting data from a large number of carefully instrumented full-scale walls (e.g. Allen et al. 2002; Miyata and Bathurst 2007a,b); (b) developing a methodology to estimate reinforcement stiffness values for geosynthetic products based on load-time response of laboratory creep data (Walters et al. 2002); and (c) using a suitably selected stiffness value and strain measurements to estimate reinforcement loads in full-scale structures (e.g. Bathurst et al. 2005, 2008b).
9.3 Pullout limit state

This chapter is restricted to LRFD calibration of the strength limit state for geogrid pullout under soil self-weight and permanent uniform surcharge loading. The maximum reinforcement load \( T_{\text{max}} \) using the AASHTO Simplified Method is computed as

\[
\text{[9.2]} \quad T_{\text{max}} = \lambda S_v K_r \sigma_v = \lambda S_v K_r \gamma_b (z + S)
\]

where, \( S_v \) = vertical spacing of the reinforcement layer, \( K_r \) = dimensionless lateral earth pressure coefficient which is calculated as a function of peak soil friction angle (\( \phi \)) and facing batter, \( \sigma_v \) = normal stress due to the self-weight of backfill (\( \gamma_b \times z \)) and equivalent height of uniform surcharge pressure (\( S = q/\gamma_b \)), \( \gamma_b \) = bulk unit weight of soil, \( z \) = depth below crest of the wall, and \( q \) = uniform distributed surcharge pressure. The constant coefficient \( \lambda \) is introduced in this equation for convenience. Hence, \( \lambda = 1 \) when the current AASHTO Simplified Method is used and \( \lambda = 0.3 \) or 0.25 apply to the modified AASHTO Simplified Method introduced later in this chapter.

According to AASHTO (2007, 2009) and FHWA (2001) the ultimate pullout capacity for sheet geosynthetics (geotextiles and geogrids) is estimated using

\[
\text{[9.3]} \quad P_v = 2(F^* \alpha)\sigma_v L_v = 2(\Psi \tan \phi)\sigma_v L_v
\]

Here, \( L_v \) = anchorage length, \( F^* \) and \( \alpha \) = dimensionless parameters, \( \Psi = \tan \phi_{sg}/\tan \phi \) = dimensionless efficiency factor where \( \phi_{sg} \) = peak geosynthetic-soil interface friction angle. In the FHWA document, the following default values are recommended: \( \alpha = 0.8 \) for geogrids and \( \alpha = 0.6 \) for geotextiles, and \( F^* \) is calculated as

\[
\text{[9.4]} \quad F^* = \frac{2}{3} \tan \phi
\]

In the absence of project-specific data, AASHTO and FHWA guidance documents recommend that the soil peak friction angle be capped at \( \phi = 34 \) degrees.
From Equation 9.1, the limit state function for geosynthetic pullout due to soil-self weight plus permanent uniform surcharge pressure is

\[ 9.5 \quad \phi P_c - \gamma_0 T_{\text{max}} \geq 0 \]

Here, \( \gamma_0 \) denotes the corresponding load factor applicable to internal MSEW stability, assuming that no live load is present (called vertical earth pressure load factor in AASHTO and FHWA design codes).

### 9.4 Load and resistance factor design (LRFD) calibration approach

LRFD is based on reliability theory (e.g. Nowak and Collins 2000; Ang and Tang 1975, 1984; Benjamin and Cornell 1970). An overview and history of LRFD for geotechnical engineering has been reported by Becker (1996a,b) and Kulhawy and Phoon (2002).

The calibration approach in this chapter follows that used for LRFD calibration for superstructure design in highway bridge design codes in North America (Nowak 1999; Nowak and Collins 2000) and the NCHRP standard for LRFD calibration for both geotechnical and structural design adopted by the AASHTO Bridge Subcommittee (Allen et al. 2005). Examples of the general approach for LRFD calibration of steel grid reinforced soil walls have also been reported by Bathurst et al. (2008a). An important feature of the general approach is the use of bias statistics where bias is defined as the ratio of measured value to predicted (nominal) value. Bias statistics are influenced by model bias (i.e. accuracy of the deterministic model representing the mechanics of the load or resistance side of the limit state equation under investigation), random variation in input parameter values, spatial variation in input values, quality of data, and consistency in interpretation of data when data are gathered from multiple sources (the typical case) (Allen et al. 2005). If the underlying deterministic model used to predict load or resistance capacity is accurate and other sources of randomness are small, then bias statistics have a mean value that is close to one and a small coefficient of variation. If the underlying deterministic models are poor, then adjustments to these models may be required in order to achieve sensible values for load and resistance factors (i.e. load factor values equal to or greater than one and resistance factor values equal to or less than one).

Incorporating bias statistics into LRFD calibration for geogrid pullout and assuming only one load type, Equation 9.1 can be expressed as
\[ \gamma_0 X_R \geq \phi X_Q \]

Here, \( X_R \) = resistance bias computed as the ratio of measured resistance (\( R_m \)) to calculated (predicted) nominal resistance (\( R_n = P_c \)), and \( X_Q \) = load bias computed as the ratio of measured load (\( Q_m \)) to the calculated (predicted) nominal load (\( Q_n = T_{max} \)). In this chapter the predicted values for reinforcement load and pullout capacity are computed using the deterministic models (equations) recommended by AASHTO (2007, 2009), FHWA (2001) and variants proposed by the writer. Bathurst et al. (2008a) have pointed out that in order for Equation 9.6 to be valid, bias values must be uncorrelated with calculated values (i.e. no hidden dependencies). Phoon and Kulhawy (2003) gave an example of hidden dependency between shaft capacity values and calculated capacities taken from a database of tests on laterally-loaded rigid drilled shafts.

In reliability-based design terminology the peak friction angle and cohesion term are characteristic values that are assumed to be best estimates of these peak soil strength input parameters (i.e. unfactored). This convention is adopted for both load and pullout resistance calculations where soil shear strength values are used in computations.

### 9.5 Reinforcement load data

The reinforcement load data used in analyses were collected from full-scale laboratory test walls and field case studies reported by Allen et al. (2003), Miyata and Bathurst (2007a,b) and Bathurst et al. (2008b). A summary of key information is provided here for completeness.

The database is comprised of 31 different wall sections ranging in height from 3 to 12.6 m. All were constructed on competent foundations. Hence, the performance of these structures was not influenced by excessive settlements or failure of the foundation or wall toe. A total of 21 wall sections were constructed with a vertical face; the remaining walls were constructed with facing batter from 3° to 27°. Most walls were constructed with a hard structural facing. A total of 58 data points were collected from 13 wall sections built with frictional soils (\( \phi > 0, c = 0 \)) and 79 data points for sections built with cohesive-frictional soils (\( \phi > 0, c > 0 \)). Current AASHTO design codes allow only frictional (\( c = 0 \)) soils to be used in the reinforced soil zone. The cohesive component of shear strength, if present, is conservatively ignored. However, in order to increase the database and to broaden the application of the modified load model (presented later) to a wider range of backfill soils, c-\( \phi \) soil case studies are included. The cohesion strength
component of the soil was only considered in calculations using the modified load model. However, even for these calculations the cohesive component of soil shear strength was included in an equivalent peak secant plane strain friction angle ($\phi_{sec}$) using the conversion procedure reported by Miyata and Bathurst (2007b) and Bathurst et al. (2008b). The peak secant friction angle was used in all load computations in this chapter in order to capture the influence of plane-strain loading conditions that are typical for these structures in the field and to make fair comparisons between loads computed using the current and modified load model.

All data correspond to walls at the end of construction and only walls that were judged to have performed well were considered. Good performance for walls with frictional backfills was judged to occur if creep strains and strain rates decreased with time and there was no evidence of failure in the reinforced soil zone such as cracking or slumping. Most walls had measured maximum strains in the backfill that were less than 1% strain. Based on a review of available performance data for full-scale walls, Allen and Bathurst (2002b) recommended that geosynthetic reinforced soil walls be designed to keep reinforcement strains to less than 3%. Above 3% strain, contiguous shear zones were detected in the backfill consistent with strain softening of the soil leading to an internal failure state. Furthermore, Allen and Bathurst (2002b) concluded that by keeping the reinforced soil from failing, creep strains and strain rates may be expected to decrease with time leading to good long-term wall performance.

For cohesive-frictional soils, strain hardening behaviour can be expected to lead to larger strains at peak shear capacity. Miyata and Bathurst (2007) reviewed the performance of monitored reinforced soil walls with c-\(\phi\) backfills and noted that good performance was observed when post-construction wall deformations were less than 0.03H or 300 mm (whichever is less) as mandated in current Japanese design codes. The maximum strain recorded in the database of c-\(\phi\) soil walls did not exceed 3% strain when these criteria were adopted (Bathurst et al. 2008b). Hence, the empirical-based criterion that reinforcement strains not exceed 3% for all backfill soil types is adopted in the current study to distinguish between reinforcement loads that are judged to be typical of operational conditions from those that are consistent with failure of the reinforced soil.

A total of 137 geosynthetic reinforcement load values were used to generate load bias statistics. Load values were computed from the maximum strain measured in each layer of reinforcement using the approach recommended by Walters et al. (2002). They showed that there was good agreement between measured loads (i.e. computed using in-situ measured reinforcement strains
and stiffness values from laboratory constant (creep) load tests) and load cell readings where these comparisons were possible.

9.6 Load bias statistics

9.6.1 Current Simplified Method

Figure 9.1a shows measured versus calculated $T_{\text{max}}$ values using the current AASHTO Simplified Method for all wall cases in the database with cohesionless soil ($c = 0$) backfills. None of the data points fall above the 1:1 correspondence line. In some cases, the calculated load values are an order of magnitude higher than the measured value. Bias statistics are summarized in Table 9.1. The mean of load bias values is $\mu_q = 0.30$ which means that “measured” load values are 30% of the calculated values on average. Recall that estimates of $T_{\text{max}}$ are based on peak secant friction angles that are greater than (uncorrected) peak friction angles using triaxial compression or direct shear tests. Hence, the over-prediction of reinforcement loads is greater if the conversion to peak plane-strain friction angle is not made.

Bias data are plotted against the calculated load values in Figure 9.1b. The linear regressed line confirms the visual weak trend of decreasing magnitude of bias values with increasing calculated $T_{\text{max}}$ values. As a quantitative check, the 95% confidence interval on the slope of the regressed line (Draper and Smith 1981) was computed and shown to be equal to -0.013 and 0.007; these limits contain zero. This confirms that at a level of significance of 5%, the adopted criterion that bias values be independent of calculated loads is not violated.

Figure 9.1c shows all bias data plotted as a cumulative distribution function (CDF). A log-normal fit to all data using bias statistics ($\mu_q = 0.30$, $\text{COV}_q = 0.54$) is judged to be a satisfactory fit to the entire data with the possible exception of the end of the upper tail.

Figure 9.2 shows normalized load data for measured and predicted values at end of construction using the current Simplified Method and cohesionless soils. The maximum load $T_{\text{max}}$ in a reinforcement layer is normalized with $T_{\text{mmx}}$ which is the maximum reinforcement load in the wall, and the reinforcement depth ($z + S$) is normalized with equivalent height of wall ($H + S$) where $H$ is the height of the reinforced soil zone. The deviation of the predicted load values from a straight line is because some walls did not have uniform reinforcement spacing at all layers. It is clear that the general trend towards increasing $T_{\text{max}}$ with depth using Equation 9.2 falls well within the envelope that captures 98% of the measured data. The large spread in the load bias data
(i.e. large COV\textsubscript{o} values) is due to the wide range of over-estimated and under-estimated measured loads with respect to predicted values. The underlying deterministic model (Equation 9.2) does poorly to capture both the trend in the distribution of load data with depth and the magnitude of load in each reinforcement layer for walls under operational conditions.

### 9.6.2 Modified Simplified Method

It is clear that the current Simplified Model for calculation of reinforcement loads for operational conditions is very poor for frictional backfill soil cases because the model over-estimates loads on average by a factor of three. This deficiency can be corrected empirically by multiplying Equation 9.2 by a factor $\lambda = 0.30$.

In order to extend the utility of the modified Simplified Method, frictional ($c = 0$) and cohesive-frictional ($c - \phi$) soils have been included in its development. For the case of $c - \phi$ soils the constant coefficient value is $\lambda = 0.25$. The $\lambda$ values were computed for each data set using the SOLVER optimization utility in EXCEL with the objective function equal to a mean bias value of one. An explanation for the slightly lower $\lambda$ factor from back-calibration using all soils in the database is that this dataset includes soils with a cohesive shear strength component. Based on classical notions of earth pressure theory, the higher the cohesive shear strength component of a backfill soil the lower the active earth pressure. This trend is captured when measured load data for frictional soil cases are separated from the entire load database.

A complication that does arise when all $c = 0$ and $c - \phi$ soils are considered is that there is an undesirable hidden dependency between load bias values $X_Q$ and calculated load $T_{\text{max}}$ based on the zero slope test at a level of significance of 5%. This deficiency can be corrected by parsing the load data based on calculated $T_{\text{max}}$ into two or more groups or adjusting the model. An example of the former approach for steel grid walls has been demonstrated by Bathurst et al. (2008a). However, this will result in different resistance factors for different load ranges and thus complicates design. The strategy adopted here to correct load dependency for all wall cases is to adjust the load model so that the minimum computed reinforcement load is $T_{\text{max}} = 0.5$ kN/m. This is a subjective choice which nevertheless leads to slightly conservative (i.e. safer) design outcomes.

Reinforcement loads and load bias statistics for the modified Simplified Method are plotted in Figure 9.3a for all data ($c \geq 0$) and statistics are summarized in Table 9.1. The grey symbols in the figure are data points that have been shifted to the right to satisfy the new model criterion that
calculated $T_{\text{max}} \geq 0.5$ kN/m. The data in Figure 9.3a show that the mean bias value is now one which means that on average the new model is much more accurate and over- and under-estimates of reinforcement load are equi-probable. Hidden load dependency is also removed at a level of significance of 5% using the zero-slope test applied to the data in Figure 9.3b (i.e. the 95% confidence limits on the slope of the regressed line are -0.182 and 0.016 and thus bracket a slope of zero). Figure 9.3c shows that the load bias data are log-normal distributed and that the fit to the entire data is also a good fit to the upper tail of the data set. Allen et al. (2005) and Bathurst et al. (2008a) have pointed out that it is the approximation to the upper tail of the load bias data that is important since it is the overlap between the upper tail of the load bias data and lower tail of the resistance (pullout) bias data that strongly influences the probability of failure in reliability-based design.

The results of similar analyses are presented in Figure 9.4 for frictional backfill soil cases only. The same model adjustments improve the mean of load bias statistics for these wall cases; but the spread of load bias values remains the same (Table 9.1). In fact, calculations showed that load bias values are independent of calculated load for all $T_{\text{max}}$ values for $c = 0$ soils using the zero slope test at a level of significance of 5%. Hence, restricting calculated loads to $T_{\text{max}} \geq 0.5$ kN/m using the new model and cohesionless backfill soils is not required. However, this adjustment is adopted for frictional soil cases in this chapter to be consistent for both soil categories. Figure 9.4c shows normal and log-normal fits to the entire data set and a log-normal fit to the upper-tail. In the computations to follow the log-normal fits to the entire data set and to the upper tail were used.

9.7 Pullout resistance bias statistics

9.7.1 Pullout database

Huang and Bathurst (2009) collected data for 478 pullout box tests from multiple sources. The tests were carried out in general conformity with ASTM D6706 (2001). A total of 318 geogrid tests were identified as having failed in pullout and these test results are used here. The reinforcement types were typical commercial materials and for analysis purposes were classified as high density polyethylene (HDPE) uniaxial geogrids, polypropylene (PP) biaxial geogrids and woven polyester (PET) geogrids. There was insufficient number of geotextile pullout tests to develop statistics for this class of products and it is for this reason that this chapter is restricted to geogrid reinforced soil walls. The majority of tests were performed with granular soils. However, approximately 25% of tests used silty sand and 2% of tests used sandy silt.
9.7.2 Pullout models and resistance bias statistics

Huang and Bathurst (2009) investigated the accuracy of the current FHWA geogrid pullout model (Equation 9.3) using two different interpretations of project-specific laboratory testing (Models 1 and 2) and using the current FHWA default model parameters for \( F^* \alpha \) (Model 3) including the default peak friction angle \( \phi = 34 \) degrees. The default model is used when project-specific pullout data are not available. As demonstrated later, the accuracy of this model is very poor even when project-specific friction angles are used in calculations (i.e. project soil friction angle in Equation 9.4). To overcome this deficiency Huang and Bathurst proposed two new models: a bi-linear model (Model 4) and a non-linear model (Model 5). Prediction accuracy is greatly improved, particularly for the non-linear model. In the current study, three of the five pullout models investigated by Huang and Bathurst (Models 2, 3 and 5) are used to generate pullout (resistance) bias statistics and to carry out LRFD calibration. It should be noted that Models 2, 3 and 5 in the current study are used with peak friction angles reported in the source materials rather than the more conservative interpretation using the default (capped) value of \( \phi = 34 \) degrees (Huang and Bathurst 2009). The capped value is recommended in AASHTO and FHWA guidance documents for design when there is no project-specific pullout data and project soil strength data is not available. Model types and related resistance bias statistics used in calibrations are summarized in Table 9.2.

Model 2 captures stress-level dependency in pullout tests by fitting a first-order (linear) approximation to \( F^* \alpha \) versus normal stress data. Figure 9.5a shows that measured \((P_m)\) versus predicted \((P_c)\) resistance values plot tightly around the 1:1 correspondence line. The quantitative accuracy of the model is confirmed by the bias statistics which have a mean and COV value of 1.00 and 0.13, respectively. A zero-slope test on resistance bias values versus predicted (calculated) capacity showed no hidden dependency at a level of significance of 5% and for brevity these data are not presented. Figure 9.5b shows the CDF plot and the normal and log-normal approximations to the bias data. For the resistance data in LRFD calibration, it is the lower tail of the distribution that makes the major contribution to the computed probability of failure value. Neither the normal nor log-normal approximation to the entire data set is a good fit to the lower tail. Two examples for best fit to lower tail based on visual fitting are plotted on the figure. The choice for LRFD calibrations is subjective. In this chapter, the best fit to lower tail (2) distribution was used in the calibrations to follow since this distribution captured a larger portion of the lower tail. It can be noted that the CDF plot for the physical data has a distinctive sigmoidal shape. This shape also appeared when each of the three sets of pullout data was plotted separately.
There was no obvious reason based on review of the original data sources to exclude data over any range of the CDF plot in Figure 9.5b. Interestingly, the results of a companion study (Chapter 7) on installation damage revealed a similar sigmoidal shape for CDF plots of the installation damage bias statistics for woven PET geotextiles.

Model 3 corresponds to the current FHWA (2001) geogrid pullout model with default values for $F^*\alpha$ and $\phi$ reported in source materials. Figure 9.6a shows measured versus predicted pullout resistance values. Most (about 90%) of the data fall above the 1:1 correspondence line and the bias mean is $\mu_r = 1.99$. Hence, Model 3 under-estimates the pullout capacity by a factor of about two on average. This result is not unexpected when it is recalled that in conventional ASD (factor of safety) past practice the tendency has been to under-estimate resistance capacity in order to ensure conservative (i.e. safe) design outcomes. However, excessively conservative design models can lead to unacceptable outcomes when accepted a priori in LRFD calibration as demonstrated later. In addition to excessive conservatism, there is a strong hidden dependency with calculated resistance capacity which is visually apparent in Figure 9.6b. The non-linear trend in dependency is reasonably well represented by a power function fitted to the data and which is analytically related to the function plotted in Figure 9.6a (see Huang and Bathurst 2009). The fitted power function in Figure 9.6a is used below to improve the prediction accuracy of the FHWA geogrid pullout model (Model 5).

Model 5 is a pullout model which accounts for non-linear influences that cannot be captured in the original FHWA model approach (Equation 9.3) and removes hidden dependency. The power function fitted to the data in Figure 9.6a is used to adjust the current AASHTO/FHWA default pullout model (Model 3). The general form of the non-linear pullout model proposed by Huang and Bathurst (2009) is

$$P_{corr} = \chi(P_c)^{1+\kappa} = \chi(2\sigma_c L_c F^* \alpha)^{1+\kappa}$$

Here, dimension-dependent terms $\chi$ and $1+\kappa$ are equal to 5.70 and 0.586, respectively when pullout capacity is computed in units of kN/m. Implementation of Model 5 is a two-step process. First calculate the pullout capacity ($P_c$) using Equation 9.3 with default coefficients for $F^*$ and $\alpha$; then, correct this value ($P_{corr}$) using the power function expression in Equation 9.7. Figure 9.7a illustrates the prediction accuracy of Model 5. All measured versus predicted pullout load data
fall close to the 1:1 correspondence line with a bias mean of 1.08 and COV = 0.37. The zero-
slope test applied to the data plotted in Figure 9.7b confirms that dependency between resistance
bias values and calculated resistance values is negligible. These are all marked improvements
over Model 3 that is used in current AASHTO and FHWA design codes. Figure 9.7c shows that
a log-normal fit to the lower tail of the resistance bias data CDF plot is visually better able to
capture the distribution at low bias values.

Pullout (resistance) bias statistics for fits to all data and fits to lower tail of CDF plots
corresponding to the three different pullout models are summarized in Table 9.2.

9.8 Selection of target probability of failure $P_f$

The objective of LRFD calibration is to select values of resistance factor and load factor(s) such
that a target probability of failure is achieved for the limit state function. In this chapter the target
probability of failure is taken as 1 in 100 ($P_f = 0.01$) which corresponds to a reliability index
value $\beta = 2.33$. This target $P_f$ value has been recommended for reinforced soil wall structures
because they are redundant load capacity systems (Allen et al. 2005). If one layer fails in pullout,
load is shed to the neighboring reinforcement layers. Pile groups are another example of a
redundant load capacity system; failure of one pile does not lead to failure of the group because
of load shedding to the remaining piles. Pile groups are designed to a target reliability index value
of $\beta = 2.0$ to 2.5 (Paikowsky 2004) which contains $\beta = 2.33$.

9.9 Selection of load factor $\gamma_Q$

There are an infinite number of load and resistance factors that can satisfy a target $P_f$ value.
However, as noted earlier it is desirable that resistance factors are equal to or less than one and
load factors are equal to or greater than one. The choice of load factor is influenced by current
codes, type of structure and back-fitting to ASD past practice (factor of safety approach). For
example, a load factor of 1.35 is currently recommended by AASHTO (2007, 2009) for
reinforcement loads in reinforced soil walls due to soil self-weight and permanent uniform
surcharge pressures. During the development of the AASHTO and Canadian bridge design codes,
trial load factors were initially selected to ensure that factored dead loads would not be exceeded
by actual (measured) loads in more than 3% of cases (Nowak 1999; Nowak and Collins 2000).
Ideally, a consistent maximum level of load exceedance is desirable for a set of related limit
states in bridge and bridge foundation design, but this is not a necessary condition. Figure 9.1a
shows that using the current AASHTO Simplified Method and frictional soil cases, all measured
values are less than predicted values. Hence, a load factor of $\gamma_Q = 1.00$ is satisfactory based on past LRFD calibration of bridge superstructure elements. In fact, to achieve the 3% exceedance level a load factor of 0.65 is required for walls with frictional backfill. This unacceptable outcome reveals one of a number of unfortunate consequences of reliability-based LRFD calibration for a limit state function that contains an excessively conservative load model.

Figure 9.8 shows cumulative distribution plots for the ratio of measured to factored and unfactored reinforcement loads using the modified AASHTO Simplified Method (Figures 9.3 and 9.4). For all data the load exceedance level varies from about 50% for $\gamma_Q = 1.00$ (unfactored) to 3% for $\gamma_Q = 2.50$. For frictional backfill case studies only, a trial load factor $\gamma_Q = 2.00$ corresponds to an exceedence level of about 5%. Bathurst et al. (2008a) carried out a similar analysis of measured and calculated design loads in steel grid reinforced soil walls using the current load model in AASHTO design codes. They showed that a load factor of 1.75 corresponds to an exceedance value of about 3% for these structures. Figures 9.8a and 9.8b show that the exceedance level is approximately 25% using load factor $\gamma_Q = 1.35$ specified in AASHTO (2007, 2009) codes. To calculate the resistance factor for the pullout limit state a load factor and target probability of failure must be assumed a priori. In this study the results of calculations using a range of load factors are presented.

9.10 Calibration results

Monte Carlo simulation was used to compute the resistance factor required to meet a target probability of 1% for a prescribed load factor and using the bias statistics reported in Tables 9.1 and 9.2. The simulations were carried out using an Excel spreadsheet as described by Allen et al. (2005) and Bathurst et al. (2008a). Table 9.3 presents the results of calibration. The calculations are carried out using different sets of bias statistics for different load and resistance model combinations. The load side includes current and modified AASHTO methods applied to frictional backfill soil ($c = 0$) cases only and modified AASHTO method for all soil ($c \geq 0$) cases. The pullout resistance models are restricted to Models 2, 3 and 5. Model 3 is the current AASHTO/FHWA pullout model with default values and $\phi$ taken as the peak friction angle in the pullout reports used to generate pullout capacity statistics.

For a given limit state and set of statistics for random variables involved, there are many combinations of load and resistance values that can satisfy a target probability of failure. Data in Table 9.3 show that the computed resistance factor value ($\phi$) increases with increasing value of
load factor \((\gamma_0)\) selected for each model/bias statistics combination. Calibration using the current AASHTO Simplified Method (Column 1) for load (with \(c = 0\) soils only) and AASHTO/FHWA Models 2 and 3 for the resistance side gives \(\phi > 1\) for all \(\gamma_0 > 1\). These values are greater than 0.90 which is the current recommended resistance factor in AASHTO (2007, 2009) design guides for geosynthetic MSEW structures under self-weight loading plus permanent uniform surcharges. More importantly, this outcome violates the North American limit states design convention that resistance factors should be less than one regardless of choice of load factor.

If the current AASHTO Simplified Method for load is used together with a load factor of \(\gamma_0 = 1.35\) then pullout Model 5 with fit-to-tail statistics is the best choice for LRFD (e.g. \(\phi = 1.02 \approx 1\)) based on recommendations summarized by Allen et al. (2005). If the Modified AASHTO Simplified Method is used (i.e. \(\lambda = 0.3\) (Equation 9.2) and \(T_{\text{max}} \geq 0.5\) kN/m) then all back-calculated resistance factor values are less than one and thus consistent with North American LRFD convention.

Table 9.4 summarizes recommended resistance factor values (to an accuracy of \(\pm 0.05\)) matching the load factor of 1.35 recommended in AASHTO (2007, 2009) practice for MSEW structures using the current and modified AASHTO Simplified Method. Resistance factors are given for different combinations of load and resistance models. It should be recalled that no restriction on load exceedance level has been applied to estimate the load factor for LRFD in this chapter. Rather, the current AASHTO load factor value has been accepted \textit{a priori}. If the exceedance of measured to factored loads is kept to 3% to 5%, consistent with values assumed for LRFD of bridge superstructure elements (Nowak 1999), then a load factor between 2.0 and 2.5 is appropriate (Figure 9.8). Computations of the type reported here using these larger load factors result in larger resistance factor values as demonstrated in Table 9.3.

An important benefit of using project-specific pullout testing is that pullout capacity can be computed using Model 2. Model 2 is more accurate than the non-linear default Model 5. The practical consequence is that the resistance factor is higher, resulting in less reinforcement anchorage length based on this limit state calculation.
9.11 Comparison of ASD and LRFD for pullout using computed operational factor of safety (OFS)

It is useful to compare the actual or operational factors of safety (OFS) using different load and/or resistance calculation models for LRFD and to compare these outcomes with allowable stress design (ASD) past practice.

For the case of a single load term (self-weight plus permanent uniform surcharge) the predicted (design) factor of safety can be estimated as

\[ [9.8] \quad FS = \frac{R_n}{Q_n} = \frac{\gamma Q}{\phi} \]

In AASHTO (2007), the recommended load and resistance factor values for the limit state of geogrid pullout were derived from fitting to FS = 1.50 used in ASD past practice (Column 4 in Table 9.5). The operational factor of safety is defined as the ratio of the measured resistance (R_m) to measured load (Q_m) and can be expressed as

\[ [9.9] \quad OFS = \frac{R_m}{Q_m} = \frac{R_n \mu_R}{Q_n \mu_Q} = \frac{FS \mu_R}{Q_m} \cdot \frac{\mu_R}{\mu_Q} \]

The OFS (Equation 9.9) represents the true factor of safety for pullout by correcting the underlying deterministic models for pullout capacity and load using mean bias values \( \mu_R \) and \( \mu_Q \). If the models are unbiased (i.e. \( \mu_R = \mu_Q = 1 \)), OFS will be equal to FS.

The OFS values in Columns 2 and 5 in Table 9.5 indicate that ASD using the current AASHTO Simplified Method for load calculations is much safer (more conservative for design) than the proposed LRFD approach regardless of resistance model adopted in computations. It is interesting to note that ASD standard practice, which is a combination of the AASHTO Simplified Method for load and Model 3 for resistance, corresponds to OFS = 9.95 which is very much larger than the specified (design) FS = 1.5. This is not a surprise to experienced reinforced soil wall designers who understand that pullout typically does not control design and documented cases of geogrid pullout failure of reinforced soil walls are rare. However, this chapter is the first
attempt to quantify the actual average factor of safety using the ASD approach, which despite the AASHTO mandate to standardize design calculations within a LRFD framework, remains common practice. Alternatively, the magnitude of conservativeness in ASD practice can be understood by noting that \( P_\text{f} = 0.1\% \) (bottom of column 6) which is an order of magnitude lower than that recommended for LRFD of other redundant soil-structure systems (e.g. pile groups). The resistance factors in Column 1 of Table 9.5 are similar. However, recall that estimated reinforcement pullout capacity values are larger using the proposed pullout Models 2 and 5. The net result is that using the modified AASHTO Simplified Method and either of the two proposed pullout models the probabilities of failure will be closer to the 1% value recommended for internal stability. Alternatively stated, the potentially large conservativeness in pullout capacity that occurs in ASD practice and significant over- or under-design depending on the calculated pullout capacity (e.g. Figure 9.6) can be reduced in principle.

9.12 Additional design considerations

According to AASHTO (2007, 2009) and FHWA (2001) design codes, the entire length of reinforcement (L) must not be less than 70% of the wall height and the anchorage length (L_e) must be at least 1 m. In some cases the length of the reinforcement may exceed 0.7H as a result of external stability requirements. It is interesting to investigate the influence of the first two empirical criteria on pullout design using the load and pullout resistance models described above.

The ratio of available pullout capacity (P_c) to design pullout load (T_{des}) was computed for all of the reinforcement layers in the database used to compute load statistics. A value of \( P_c/T_{des} = 1 \) means that pullout capacity and thus anchorage length is controlled by the pullout model adopted for design; values greater than one mean that one of the two empirical criteria control pullout design.

The available pullout capacity (P_c) based on ASD practice was calculated using the maximum value of L_e computed using pullout Model 3 (current default AASHTO approach), L = 0.7H and L_e = 1 m. The pullout tensile design load (T_{des}) was computed using the current AASHTO Simplified Method (Equation 9.2 with \( \lambda = 1 \) and \( c = 0 \) soils only) and FS = 1.5. Hence, for ASD practice the ratio of interest is:

\[
[9.10a] \quad \frac{P_c}{T_{des}} = \frac{P_c}{(FS \cdot T_{max})} \quad (\text{for ASD})
\]
For the equivalent LRFD-based ratio, the maximum value of $L_e$ was computed using the non-linear pullout Model 5, $L = 0.7H$ and $L_e = 1$ m. The pullout design load was computed using the proposed Modified AASHTO Simplified Method (Equation 9.2 with $\lambda = 0.30$ and 0.25 and $T_{\text{max}} \geq 0.5 \text{kN/m}$) together with $\gamma_Q = 1.35$ and $\phi = 0.30$ (for all soils) or 0.40 (for frictional soils only). Hence, for LRFD practice the ratio of interest is:

$$\frac{P_c}{T_{\text{des}}} = \frac{P_c}{\left(\frac{\gamma_Q}{\phi} \cdot T_{\text{max}}\right)} \quad \text{(for LRFD)}$$

Note that to carry out calculations for cohesive-frictional soils an equivalent peak secant plane strain friction angle ($\phi_{\sec}$) was computed using the conversion procedure reported by Miyata and Bathurst (2007b) and Bathurst et al. (2008b) and noted earlier in the chapter.

Computed ratios are plotted against normalized depth below the wall crest in Figure 9.9. The data in Figure 9.9a are all greater than one. If frictional soils are considered only, the ratio of pullout capacity to pullout design load is generally in the range of 10 to 100 (Figure 9.9b) using both the current and modified Simplified Method. This means that the two empirical criteria identified above will control pullout capacity and thus the minimum anchorage length to satisfy the pullout limit state. The range of computed ratios (i.e. range of conservatism) is visually greater for ASD data (Figure 9.9b). Finally, there is no pronounced visual trend of increasing design conservatism with depth.

### 9.13 Conclusions

This chapter reports for the first time reliability-based LRFD calibration for the geogrid pullout limit state in geosynthetic reinforced soil walls due to soil self-weight plus permanent uniform surcharge loading. The general approach follows the LRFD calibration methodology described by Allen et al. (2005) and Bathurst et al. (2008a) which is consistent with the methods used to develop load and resistance factors for bridge superstructures (Nowak and Collins 2000).

The results of calibration show that while current LRFD practice is safe, the current resistance factor $\phi = 0.90$ recommended by AASHTO (2007, 2009) for the reinforcement pullout limit state in MSE structures under soil self-weight cannot be justified for the case of geosynthetic
reinforced soil walls. This is due largely to the poor prediction accuracy of the underlying deterministic tensile load model (Simplified Method) in current AASHTO codes. The combination of AASHTO-recommended load factor $\gamma_Q = 1.35$ and the poor models for tensile load and default pullout capacity make it impossible to generate resistance factors less than one consistent with LRFD practice. The explanation for this unfortunate outcome is that before the Simplified Method was first adopted (unlike the steel reinforcement models recommended in AASHTO codes) no attempt was made to empirically adjust the model to match estimates of reinforcement loads under operational conditions.

To overcome these problems, modifications to the current AASHTO Simplified Method are proposed and a new default pullout model by Huang and Bathurst (2009) is used to improve the prediction accuracy of load and resistance capacity for the pullout limit state. These new models result in reasonable resistance factors values (i.e. less than one) and match a target probability of failure of 1% when the current AASHTO-recommended load factor of $\gamma_Q = 1.35$ is used. Depending on the reinforced soil type (frictional or cohesive-frictional) and the pullout model adopted, the resistance factor varies in the range of $\varphi = 0.30$ to 0.50. While these values are lower than $\varphi = 0.90$ recommended by AASHTO, load values are on average 30% of the predicted values using the current AASHTO Simplified Method and pullout capacity predictions using the AASHTO default model are a factor of two higher on average. The predicted load and pullout capacity values using the new models described in this chapter are statistically much more accurate based on measured values (i.e. bias mean is closer to one and COV of bias values are lower).

Finally this chapter demonstrates that regardless of which design method is adopted (ASD past practice with a factor of safety of 1.5 or the proposed LRFD approach with new models for load and pullout capacity) the length of reinforcement will likely be controlled by current empirical criteria that restrict the minimum reinforcement length to not less than 70% of the wall height and the anchorage length to not less than 1 m. A practical outcome is that for the 31 wall sections used to generate load statistics, these empirical criteria result in walls with probabilities of pullout failure that are likely orders of magnitude less than 1%.

This chapter has been restricted to LRFD calibration of the pullout limit state for geosynthetic reinforced soil walls under soil self-weight and permanent uniform surcharge loading. A similar approach can be used for other internal-facing stability limit states including reinforcement tensile rupture and reinforcement-facing connection. These calibration exercises are currently underway.
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Table 9.1 Summary of load bias statistics ($X_Q$) for $T_{max}$ using current and modified AASHTO Simplified Method

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Soil type</th>
<th>Soil type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Frictional ($c = 0, \phi &gt; 0$)</td>
<td>Cohesive-frictional (All data) ($c \geq 0, \phi &gt; 0$)</td>
</tr>
<tr>
<td>Current model</td>
<td>Modified model $\lambda = 0.30$ $T_{max} \geq 0.5 \text{ kN/m}$</td>
<td>Modified model $\lambda = 0.25$ $T_{max} \geq 0.5 \text{ kN/m}$</td>
</tr>
<tr>
<td></td>
<td>Fit to all data</td>
<td>Fit to upper tail</td>
</tr>
<tr>
<td>n (number of data points)</td>
<td>58</td>
<td>137</td>
</tr>
<tr>
<td>$\mu_Q$ (mean)</td>
<td>0.30 1.00 1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>COV$_Q$ (coefficient of variation)</td>
<td>0.54 0.72 0.53</td>
<td>0.72</td>
</tr>
</tbody>
</table>

Note: bold face values are used later based on recommendations by Allen et al. (2005) to use best fit-to-tail statistics if applicable.
Table 9.2 Bias statistics for different pullout capacity model types (see Huang and Bathurst (2009) for description of pullout models)

<table>
<thead>
<tr>
<th>Model</th>
<th>Description</th>
<th>Approximation to CDF plot</th>
<th>Bias statistics</th>
<th>P_m/P_c versus P_c dependency (^{(a)})</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Mean (\mu_R)</td>
<td>COV(_R)</td>
</tr>
<tr>
<td>2</td>
<td>First-order approximation to measured (F*\alpha)</td>
<td>Normal fit to all data</td>
<td>1.00</td>
<td>0.13</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Log-normal fit to all data</td>
<td>1.00</td>
<td>0.13</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Normal fit to lower tail (1)</td>
<td>1.00</td>
<td>0.21</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Normal fit to lower tail (2)</td>
<td>1.17</td>
<td>0.27</td>
</tr>
<tr>
<td>3</td>
<td>FHWA method with default values ((F*\alpha = 0.8 \times (2/3) \tan \phi))</td>
<td>Log-normal</td>
<td>1.99</td>
<td>0.59</td>
</tr>
<tr>
<td>5</td>
<td>Non-linear model</td>
<td>Log-normal fit to all data</td>
<td>1.08</td>
<td>0.37</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Log-normal fit to lower tail</td>
<td>1.12</td>
<td>0.50</td>
</tr>
</tbody>
</table>

Notes: Model 2 requires project-specific test data; Models 3 and 5 require input parameters: \(\sigma_v\) = normal stress, \(L_e\) = anchorage length, \(\phi\) = peak friction angle of the soil; \(^{(a)}\) based on zero-slope test at a level of significance of 5%.
Table 9.3 Computed resistance factor $\varphi$ for $P_t = 0.01$ ($\beta = 2.33$) and selected load factors $\gamma_Q$

<table>
<thead>
<tr>
<th>Resistance (pullout) model</th>
<th>Load factor $\gamma_Q$</th>
<th>Resistance factor $\varphi$</th>
<th>Current AASHTO $c = 0$ (Fit to all data)</th>
<th>Modified AASHTO $c = 0$ (Fit to all data)</th>
<th>$c = 0$ (Fit to tail)</th>
<th>$c \geq 0$ (Fit to all data)</th>
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<tr>
<td></td>
<td></td>
<td>Column $\rightarrow$ 1 2 3 4</td>
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<tr>
<td></td>
<td></td>
<td>1.00</td>
<td>1.11</td>
<td>0.34</td>
<td>0.38</td>
<td>0.26</td>
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<td></td>
<td></td>
<td>1.35</td>
<td>1.50</td>
<td>0.46</td>
<td>0.51</td>
<td>0.36</td>
</tr>
<tr>
<td>Model 2</td>
<td></td>
<td>1.75</td>
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<td>0.59</td>
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Table 9.4 Summary of recommended resistance factor values for $\beta = 2.33$ and $\gamma_Q = 1.35$ using current and modified AASHTO Simplified Method for load calculations and different pullout models for resistance capacity

<table>
<thead>
<tr>
<th>Resistance (pullout) model</th>
<th>Resistance factor $\varphi$</th>
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</thead>
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<tr>
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<td>Current AASHTO $c = 0$</td>
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<tr>
<td>Column $\rightarrow$</td>
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</tr>
<tr>
<td>Model 2</td>
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</tr>
<tr>
<td>Model 5</td>
<td>1.00**</td>
</tr>
<tr>
<td>Model 3 (Current AASHTO model)*</td>
<td>1.00**</td>
</tr>
</tbody>
</table>

Notes:
* Ignoring dependency;
** Calculated resistance factor values (Table 9.3) are greater than one but resistance factors for design should be capped at one.
Table 9.5 Comparison of operational factor of safety (OFS) and probability of failure ($P_f$) using proposed LRFD approach and ASD past practice for $c = 0$ backfill soils

<table>
<thead>
<tr>
<th>Resistance (pullout) model</th>
<th>Proposed LRFD using $\gamma_0 = 1.35$ and Modified AASHTO Simplified Method</th>
<th>ASD using Current AASHTO Simplified Method</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\varphi$</td>
<td>OFS ($^a$)</td>
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<td>Column $\rightarrow$ 1</td>
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<td>Model 5</td>
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<td>3.65</td>
</tr>
<tr>
<td>Model 3 (Current AASHTO model) $^{**}$</td>
<td>0.45</td>
<td>5.97</td>
</tr>
</tbody>
</table>

Notes:

$^a$ OFS = $\frac{\gamma_0 \cdot \mu_R}{\varphi \mu_Q}$ and bias statistics for entire data sets; $^b$ OFS = $FS \cdot \frac{\mu_R}{\mu_Q}$ and bias statistics for entire data sets;

$^* FS = \gamma_0 / \varphi = 1.35 / 0.9 = 1.5$;

$^{**}$ Ignoring hidden dependencies.
Figure 9.1 Measured and predicted load data using current AASHTO Simplified Method for $c = 0$ soils: (a) measured versus calculated load values; (b) load bias versus calculated load values, and (c) CDF plots for load bias data.
Figure 9.2 Normalized measured and predicted loads using current AASHTO Simplified Method and cohesionless backfill soils \((c = 0)\)
Figure 9.3 Measured and predicted load data using modified AASHTO Simplified Method for all backfill soil cases: (a) measured versus calculated load values; (b) load bias versus calculated load values, and (c) CDF plots for load bias data.
Figure 9.4 Measured and predicted load data using modified AASHTO Simplified Method for frictional soils \((c = 0, \phi > 0)\): (a) measured versus calculated load values; (b) load bias versus calculated load values \((T_{\text{max}} \geq 0.5 \text{kN/m})\); and (c) CDF plots for load bias data.
Figure 9.5 Measured and predicted resistance data using current FHWA geogrid pullout model for project-specific testing (Model 2): (a) measured versus calculated pullout resistance values; and (b) CDF plots for resistance bias data.
Figure 9.6 Measured and predicted resistance data using current default FHWA geogrid pullout model (Model 3): (a) measured versus calculated pullout resistance values; (b) resistance bias versus calculated resistance values, and (c) CDF plots for resistance bias data.
Figure 9.7 Non-linear geogrid pullout model (Model 5): (a) measured versus calculated pullout resistance values; (b) bias values versus calculated pullout resistance values; and (c) CDF plots for resistance bias data

(a) Calculated resistance, $P_c$ (kN/m) vs. Measured resistance (kN/m)

(b) Resistance bias, $X_R$ vs. Calculated resistance, $P_c$ (kN/m)

(c) Normal fit to all data vs. Log-normal fit to all data vs. Log-normal fit to tail

Regression line: $X_R = 0.0006P_c + 1.0633$

- $n = 318$
- $\mu_Q = 1.08$ (mean)
- $COV_Q = 0.37$

Fit to tail:
- $\mu_Q = 1.12$ (mean)
- $COV_Q = 0.50$
Figure 9.8 Cumulative fraction plots for the ratio of measured to factored and unfactored calculated load values using modified AASHTO Simplified Method: (a) all soil backfill cases; and (b) frictional backfill soils
Figure 9.9 Ratio of pullout capacity to pullout design load using ASD and proposed LRFD approach versus normalized depth: (a) all soil wall cases; and (b) frictional soil wall cases
Chapter 10
Load and Resistance Factor Design (LRFD) Calibration for Rupture Limit State Using the AASHTO Simplified Method

10.1 Introduction

Limit states design, which is called load and resistance factor design (LRFD) in the United States, has been used for structural engineering design for decades. To ensure a common approach for the design of both steel/reinforced concrete structures and soil structures, the American Association of State Highway and Transportation Officials (e.g. AASHTO 2009) and the Canadian Highway Bridge Design Code (e.g. CSA 2006) now recommend LRFD for all structures, including reinforced soil retaining walls (Mechanically Stabilized Earth Walls (MSEWs) in USA terminology). To date, the strategy to migrate from allowable stress design (ASD) past practice (factor of safety approach) has been to back-calculate resistance factors for each limit state based on recommended factors of safety in ASD past practice (e.g. AASHTO 2002) and prescribed load factors. This approach has the disadvantage that there is no guarantee that an acceptable target probability of failure is achieved for the limit state considered or that the corresponding probability of failure is consistent with values for other limit states in a set of calculations (e.g. internal stability limit states for reinforced soil walls).

This chapter reports reliability-based LRFD calibration for the strength limit state in geosynthetic reinforced soil walls subjected to soil self-weight plus permanent uniform surcharge loading. Calibration can now be attempted because large databases for reinforcement loads (e.g. Bathurst et al. 2008b), creep laboratory tests (Huang et al. 2010b) and installation damage testing Chapter 8 are now available.

The chapter first demonstrates that while current ASD design practice is safe, the current resistance factor $\varphi = 0.90$ recommended by AASHTO (2009) for reinforcement rupture for MSEW structures cannot be justified. Next, an empirically-modified AASHTO Simplified Method for the calculation of reinforcement load (Huang et al. 2010a) is used to improve the accuracy of load predictions for walls under operational conditions. The modified load model leads to reasonable resistance factor values matching a load factor of 1.35 and an acceptable probability of failure of 1% due to long-term tensile rupture. This target probability of failure is also consistent with the 1% value for the geogrid pullout limit state using the modified AASHTO Simplified Method for load (Huang et al. 2010a).
10.2 Reinforcement load

The nominal maximum load \( T_{\text{max}} \) in a reinforcement layer using the AASHTO (2002, 2009) Simplified Method is computed as

\[
T_{\text{max}} = \lambda S_v K_r \sigma_v = \lambda S_v K_r \gamma_b (z + S)
\]

where, \( S_v \) = vertical spacing of the reinforcement layer, \( K_r \) = dimensionless lateral earth pressure coefficient which is calculated as a function of peak soil friction angle (\( \phi \)) and facing batter, \( \sigma_v \) = normal stress due to the self-weight of backfill (\( \gamma_b \times z \)) and equivalent height of uniform surcharge pressure (\( S = q/\gamma_b \)), \( \gamma_b \) = bulk unit weight of soil, \( z \) = depth below crest of the wall, and \( q \) = uniform distributed surcharge pressure. The constant coefficient \( \lambda \) is introduced in this equation for convenience to distinguish between the current AASHTO Simplified Method (\( \lambda = 1 \)) and the modified AASHTO Simplified Method (\( \lambda = 0.3 \) or \( 0.25 \)) described by Huang et al. (2010a). The same formulation with \( \lambda = 1 \) is found in other design guidance documents (e.g. NCMA 2009; CFEM 2006).

The AASHTO Simplified Method and variants are extensions of classical active earth pressure theory and assume that the reinforcement and soil are simultaneously at an ultimate collapse (failure) limit state. However, operational loads in monitored walls that have behaved well are very much lower. A summary of these walls from many sources can be found in papers by Allen et al. (2002), Allen and Bathurst (2002a) and Bathurst et al. (2008b). The estimated loads computed from measured strains and suitably selected reinforcement stiffness values from more than 30 full-scale walls are about one third of the predicted values on average.

Furthermore, reinforcement strains in monitored field walls that have behaved well under operational conditions have stayed the same or decreased (i.e. strain relaxed) with time following construction (Allen and Bathurst 2002b; Tatsuoka et al. 2004; Kongkitkul et al. 2010). Hence, tensile reinforcement loads at the end-of-construction condition are the maximum loads for LRFD provided original site, soil and boundary conditions for which the wall was designed do not change.

Despite the shortcomings of limit equilibrium-based methods for the design of geosynthetic reinforced soil walls noted above in the context of observed behaviour and LRFD calibration, the
Simplified Method is the only method currently available in AASHTO (2009) and FHWA (Elias et al. 2001) guidance documents to estimate tensile loads in geosynthetic reinforcement layers.

Variations of the Simplified Method for steel reinforced soil walls are also found in the AASHTO, FHWA and BS8006 (1995) design codes. In general, the AASHTO Simplified Method for steel reinforced soil walls does well in predicting reinforcement loads under operational conditions due to soil self-weight (Allen et al. 2004; Bathurst et al. 2009). This is because empirical adjustments were made to the coefficient of active earth pressure to match measured reinforcement loads in steel reinforced soil walls under operational conditions. In the previous chapter, the writer made similar empirical adjustments to the AASHTO Simplified Method for geosynthetic reinforced soil walls to give a better match with measured operational tensile loads at end of construction while preserving the structure of the original formulation (Huang et al. 2010a). The modified AASHTO Simplified Method for reinforcement loads (Equation 10.1) includes an empirical factor which is $\lambda = 0.30$ for purely frictional ($c = 0$) soils and $\lambda = 0.25$ for cohesive-frictional backfill soils ($c \geq 0$). A second modification was to limit the minimum calculated tensile load to $T_{\text{max}} = 0.5$ kN/m. This is a conservative criterion (i.e. safer for design) but was actually adopted to remove undesirable hidden dependency between load bias statistics and calculated loads.

10.3 Long-term tensile strength

In North America, the (nominal) available long-term tensile strength ($T_{al}$) of each geosynthetic reinforcement layer is computed as follows (FHWA – Elias et al. 2001; AASHTO 2002; NCMA 2009; CFEM 2006):

$$T_{al} = \frac{T_{\text{ult}}}{RF} = \frac{T_{\text{ult}}}{RF_{ID} \times RF_{CR} \times RF_{D}}$$

where, $T_{\text{ult}}$ = ultimate tensile strength of the reinforcement and $RF = product of reduction factors to account for potential long-term strength loss due to installation damage ($RF_{ID}$), creep ($RF_{CR}$) and degradation due to chemical/biological processes ($RF_{D}$).

**Figure 10.1** illustrates the interpretation of **Equation 10.1** with respect to the design life of a geosynthetic reinforcement layer in a soil structure. In current deterministic ASD practice the available tensile strength ($T_{al}$) must be at least equal to the maximum predicted tensile load ($T_{\text{max}}$)
in a reinforcement layer. The ultimate tensile strength is a conservative estimate of the expected tensile strength of the reinforcement using standard laboratory tests (ASTM D4595 2009; ASTM D6637 2001). Typically this is a certified minimum value supplied by the vendor or the Minimum Average Roll Value (MARV) of the material based on production quality control. Nevertheless, a manufacturer may sometimes report a lower ultimate tensile strength value than that computed from actual testing in order to position the product in the market, to consider variation in reported results between laboratories, or to be cautious.

Strength reductions due to installation damage are assumed to occur at construction and protocols to carry out these installation damage trials are available (Elias 2000; WSDOT T925 2005; ASTM D5818 2000). The creep strength reduction factor $R_{CR}$ is taken as the ratio of the original strength to retained (creep-rupture) strength at the target design life of the reinforced soil structure. In North America and worldwide the laboratory methodologies to determine the creep-rupture strength of geosynthetic reinforcement are well-established (ASTM D5262 2004; ASTM D6992 2009; EN ISO 13431 1999; EN ISO/TR 20432 2007; WSDOT T925 2005).

Typically an overall factor of safety (FS) is used in allowable stress design (ASD) to account for other project uncertainties so that $T_{al}/T_{max} \geq FS = 1.5$ (AASHTO 2002; NCMA 2009; CFEM 2006).

10.4 Load and resistance factor design (LRFD) concepts for tensile rupture limit state

In load and resistance factor design, engineers use prescribed limit state equations and load and resistance factors specified in design codes to ensure that a target probability of failure for each load carrying member in a structure is not exceeded. The objective of LRFD calibration in the context of the current chapter is to compute load and resistance factor values to meet this same objective using measured load and resistance data rather than fitting to ASD past practice. The rupture limit state expression for LRFD assuming only one load term can be expressed as:

$$\phi T_{al} - \gamma_Q T_{max} \geq 0$$  \hspace{1cm} [10.3]

Here, $T_{max} = $ nominal (specified) load using Equation 10.1; $T_{al} = $ nominal (characteristic) resistance using Equation 10.2; $\gamma_Q = $ load factor for soil self-weight plus permanent uniform distributed surcharge pressure (no live load present); and $\phi = $ resistance factor. In design codes,
load factor values are typically greater than or equal to one and resistance factor values typically less than or equal to one.

It is important to emphasize that for bridge design, a nominal load is not a failure load but rather a value that is a best estimate of the load under operational conditions (Harr 1987). For example, this nominal load may be due to structure dead load effects plus a representative vehicle load based on statistical treatment of bridge traffic. Conceptually, the margin of safety is largely provided by the resistance side of the equation where the resistance value is calculated based on the failure capacity (ultimate limit state) or a deformation criterion (serviceability limit state) for each element analyzed. For the case of a steel member, the ultimate resistance of the member is based (typically) on flexure or shear capacity, and serviceability on a prescribed allowable deformation.

The same concepts described above have been adopted by AASHTO for LRFD calibration of transportation-related structures which include the strength limit states for internal stability design of geosynthetic reinforced soil walls (Allen et al. 2005). The reinforcement loads due to soil self-weight can be estimated using the current AASHTO (2009) Simplified Method. A common source of confusion and conflict with LRFD using the Simplified Method is that the underlying deterministic model to calculate reinforcement loads is based on active earth pressure theory or Coulomb wedge analysis and hence the soil and critical reinforcement layers are assumed to be simultaneously at incipient failure. This is not a problem if it is understood that the Simplified Method is a conservative (safe) model to estimate reinforcement loads and can be empirically adjusted to improve load predictions for the operational conditions assumed in LRFD. Suitable adjustments have been demonstrated by Huang et al. (2010a) for geosynthetic reinforced soil walls and during the original development of the Simplified Method for the internal stability design of steel reinforced soil walls.

10.5 Load and resistance factor calibration using bias statistics

Deterministic models used in soil-structure design problems have varying accuracy when compared to measured load and resistance values where these comparisons can be made. The general approach to correct deterministic model predictions is to introduce bias values where the bias value for a particular random variable is expressed generically as $X = \frac{\text{measured value}}{\text{predicted value}}$ (Nowak and Collins 2000; Withiam et al. 1998). The general approach has been demonstrated by Allen et al. (2005) and Bathurst et al. (2008a) for LRFD calibration of the pullout limit state for steel grid reinforced soil walls.
Bias statistics are influenced by model bias (i.e. accuracy of the deterministic model representing the mechanics of the load or resistance side of the limit state equation), random variation in input parameter values, spatial variation in input values (if applicable), quality of data, and consistency in interpretation of data when data are gathered from multiple sources (the typical case) (Allen et al. 2005). Incorporating bias statistics into LRFD calibration, Equation 10.3 can be expressed as:

\[ \gamma_Q X_R - \phi X_Q \geq 0 \]  

Here, \( X_R \) = resistance bias computed as the ratio of measured resistance to calculated (predicted) nominal resistance and \( X_Q \) = load bias computed as the ratio of measured load to the calculated (predicted) nominal load. For the load side:

\[ X_Q = \frac{T_{\text{max,meas}}}{T_{\text{max}}} \]

where, \( T_{\text{max,meas}} \) is the measured maximum tensile load in a reinforcement layer. As noted in the previous section, measured maximum tensile loads under operational conditions have been calculated to be on average about one third of predicted values using the AASHTO Simplified Method (e.g. Allen et al. 2003; Bathurst et al. 2008b). For the resistance side, the strength bias value is expressed as:

\[ X_R = \frac{T_{\text{al,meas}}}{T_{\text{al}}} \]

where, \( T_{\text{al,meas}} \) is the measured long-term tensile strength in a reinforcement layer. The approach adopted here is to equate this value to the predicted long-term strength by introducing bias statistics for each of the three strength loss terms in Equation 10.2. Hence:

\[ T_{\text{al,meas}} = T_{\text{al}} \cdot X_R = \frac{T_{\text{ult}}}{RF_{ID} \times RF_{CR} \times RF_{D}} \cdot X_R = \left( \frac{T_{\text{ult}}}{RF_{ID}} \right) \cdot X_{ID} \cdot \left( \frac{1}{RF_{CR}} \right) \cdot X_{CR} \cdot \left( \frac{1}{RF_{D}} \right) \cdot X_{D} \]

As noted earlier, the value of \( T_{\text{ult}} \) used for design is a manufacturer-certified value that can be expected to be lower than the MARV value computed based on the mean and standard deviation
of ultimate strength values for original test specimens taken from project rolls (Huang et al. 2010b). Hence, using a manufacturer-certified minimum \( T_{ul} \) value in Equation 10.2 will be safer for design. A useful description of the differences noted above can be found in WSDOT T925 (2005) and in the NTPEP (2007).

The bias of the long-term tensile strength of the reinforcement, \( X_R \), is expressed as a product of three bias values:

\[
[10.8] \quad X_R = X_{ID} \cdot X_{CR} \cdot X_D
\]

where, \( X_{ID} \), \( X_{CR} \) and \( X_D \) are bias values for tensile strength after installation damage, after creep and after degradation due to chemical or environmental factors, respectively. Variability in undamaged specimen strength is captured by the variability in installation damage bias values (Bathurst et al. 2010). Assuming that \( X_{ID} \), \( X_{CR} \), and \( X_D \) are uncorrelated, the mean and coefficient of variation (COV) of \( X_{al} \) values are given by (Ang and Tang 1975):

\[
[10.9] \quad \mu_{X_R} = \mu_{X_{ID}} \cdot \mu_{X_{CR}} \cdot \mu_{X_D}
\]

and

\[
[10.10] \quad COV_{X_R} = \sqrt{COV_{X_{ID}}^2 + COV_{X_{CR}}^2 + COV_{X_D}^2}
\]

Here, the subscripts for each of the mean and COV terms can be matched to installation damage, creep and degradation reduction factors identified earlier. In theory, Equations 10.9 and 10.10 are only valid for uncorrelated log-normal distributions of the three random variables. However, for uncorrelated normal distributions with small coefficients of variation these expressions are sufficiently accurate. For a perfect deterministic model in which there is no error in the estimation of each of the strength reduction terms in Equation 10.7, then Equation 10.7 is equivalent to the deterministic model used in ASD past practice (i.e. Equation 10.2). For this case, the mean value of each set of bias values is one \( (X_{ID} = X_{CR} = X_D = 1) \) and the corresponding COV values are zero. This can be understood to be an unlikely situation for any geotechnical limit state.
Once frequency distributions for load ($X_Q$) and resistance ($X_R$) are known, bias statistics for both quantities can be approximated (i.e. normal or log-normal mean and COV of each distribution). Monte Carlo simulations can then be carried out to find the magnitude of resistance factor ($\phi$) to match a prescribed load factor ($\gamma_Q$) and a target probability of failure. These calculations can be done using an Excel spreadsheet (Allen et al. 2005).

Load bias statistics using the AASHTO Simplified Method and modified Simplified Method have been reported by Bathurst et al. (2008b) and Huang et al. (2010a), respectively. Bias statistics for installation damage and creep have been reported by Bathurst et al. (2010) and Huang et al. (2010a), respectively.

10.6 Load bias statistics

Table 10.1 reproduces load bias statistics reported by Bathurst et al. (2008b) using the current AASHTO Simplified Method for frictional backfill soil cases only and Huang et al. (2010a) using the modified Simplified Method for both frictional and cohesive-frictional backfill soil cases. The data for these calculations were collected from 31 instrumented and monitored full-scale laboratory test walls and field case studies reported by Allen et al. (2003), Miyata and Bathurst (2007a,b) and Bathurst et al. (2008b).

A total of 137 geosynthetic reinforcement load values were used to generate load bias statistics. Load values were computed from the maximum strain measured in each layer of reinforcement using the approach recommended by Walters et al. (2002). They showed that there was good agreement between measured loads (i.e. computed using in-situ measured reinforcement strains and stiffness values from laboratory constant (creep) load tests) and load cell readings where these comparisons were possible.

Current AASHTO design codes allow only frictional ($c = 0$) soils to be used in the reinforced soil zone. The cohesive component of shear strength, if present, is conservatively ignored. The cohesion strength component of the soil was only considered in calculations using the modified load model by conversion to an equivalent peak secant plane strain friction angle ($\phi_{sec}$) using the procedure reported by Miyata and Bathurst (2007a,b) and Bathurst et al. (2008b). The peak secant friction angle was used in all load computations in this chapter in order to capture the influence of plane-strain loading conditions that are typical for these structures in the field and to make fair comparisons between loads computed using the current and modified load model.
Figure 10.3 shows cumulative frequency distribution (CDF) plots of load bias values using the current and modified Simplified Methods. The plots show the best fit to upper tail approximations that were used to generate the mean and COV load bias statistics ($\mu_{x_o}$, $\text{COV}_{x_o}$) for the modified Simplified Method (Table 10.1).

10.7 Resistance bias statistics

10.7.1 Installation damage

Table 10.2 reproduces installation damage bias statistics reported by Bathurst et al. (2010). Installation damage for a particular geosynthetic product can be understood to vary with soil type, gradation, compaction method and lift thickness. Bathurst et al. (2010) analysed strength data collected from installation damage trials that reported individual ultimate strength values for geosynthetic specimens in original (as-received) and damaged (exhumed) conditions. The database includes 103 different geosynthetic products from 20 different publication sources. The results of 799 and 2248 in-air tensile tests on damaged and undamaged specimens, respectively, were used to compute bias statistics. The bias statistics for geotextiles were divided into two categories: materials with high nominal installation damage factors $RF_{ID} \geq 1.7$ and those with $RF_{ID} < 1.7$. The former were typically the result of geotextiles with mass per unit area less than 270 g/m$^2$ in combination with coarse gravel fills with $D_{50}$ particle size $> 19$ mm. These aggregate materials were classified as Type 1 (Figure 10.2). It was recommended that these low weight geotextiles in combination with Type 1 soils should be avoided for reinforced soil wall design. Three other soil categories were selected in this study based on the $D_{50}$ particle size in an attempt to match $RF_{ID}$ ranges and installation damage bias statistics to less aggressive soil gradations. Although four categories of soil were considered in the analysis of installation damage data, the practical outcome was that bias values can be grouped into two ranges for each geogrid type based on $D_{50}$ of the soil greater than or less than 19 mm.

In practice, the $RF_{ID}$ factor for design is chosen based on field installation damage trials where reinforcement installation method, backfill type and compaction method are the same as, or similar to, project conditions (e.g. ASTM D5818 2000; WSDOT T925 2005). Typically, $RF_{ID}$ values from non-project-specific testing are used at design time by making a conservative but close match to installation damage trial test results for the same product or similar product in a product line and a range of soil gradations. Assuming that a suitable $RF_{ID}$ value is available, a matching set of mean and COV of bias statistics can be taken from Table 10.2.
10.7.2 Creep

Huang et al. (2010b) analyzed a database of creep tests on 42 different geosynthetic products collected from 18 different sources. A total of 799 in-air tensile test results and 395 creep-rupture data points were examined. This database was used to compute creep strength reduction bias statistics for three different geosynthetic product categories. A single bias mean value of $X_{CR} = 1.00$ and $COV_{X_{cr}} = 0.044$ was computed by treating all geosynthetics as a single population. In the current study, a value of $COV_{X_{cr}} = 0.05$ is judged to be a reasonable and convenient value for LRFD calibration. This variability in the prediction of creep-reduced strength is very low and is largely captured by the magnitude of variance in the original strength ($T_{ult}$) of the test specimens estimated to be about 0.04 by treating all geosynthetics as a single population (Huang et al. 2010b). Hence, it can be assumed that current practice for the determination of RF$_{CR}$ does not introduce additional uncertainty to the calculation of long-term reinforcement strength. This validates the assumption in WSDOT T925 (2005) that creep strength is proportional to the short-term tensile strength and the variability of $X_{CR}$ is fully covered by variability in the installation damage bias values ($X_{ID}$) where inherent strength variability is included. In practical terms this means that the mean of $X_{CR}$ can be taken as one and the COV as zero in Equations 10.9 and 10.10; thus the magnitude of RF$_{CR}$ for the geosynthetic rupture limit state in LRFD can be treated as deterministic. Nevertheless, in the sensitivity analyses to follow the influence of varying the magnitude of creep strength variability on resistance factor is explored quantitatively.

10.7.3 Durability

There is no single test protocol or representative test environment that can be recommended to generate durability bias statistics in the same manner described above for installation and creep strength reduction mechanisms. The selection of RF$_D$ for design is based on a minimum prescribed value (RF$_D > 1.10$). Project-specific testing is required if a lower value for RF$_D$ is used in design. Recommendations for project-specific durability testing can be found in WSDSOT T925 (2005). In the analyses to follow it is assumed that chemical and biological degradation is deterministic. A mean bias value of 1.00 is used and $COV_{X_{0}}$ is taken as zero. However, the influence of the magnitude of $COV_{X_{0}}$ is explored in the sensitivity analysis presented later.

10.8 Results of LRFD calibration

Monte Carlo simulation using Equation 10.4 was carried out using trial values of resistance factor $\varphi$ and $\gamma_Q = 1.35$ until a probability of failure of 1% was satisfied (reliability index value $\beta =$
This target $P_t$ value has been recommended for reinforced soil wall structures because they are redundant load capacity systems (Allen et al. 2005). If one layer fails in rupture, load is shed to the neighboring reinforcement layers. Pile groups are another example of a redundant load capacity system and are designed to a target reliability index value of $\beta = 2.0$ to 2.5 (Paikowsky 2004) which contains $\beta = 2.33$.

The results of LRFD calibration are summarized in Table 10.3. For the current AASHTO Simplified Method and $c = 0$ backfill soils, the recommended resistance factor is typically one. In fact, for all but one case the actual computed resistance factors were greater than one and in the range of 1.40 to 1.74. This is an unacceptable outcome since the expectation in LRFD codes is that resistance factors are typically less than or equal to one. In the current AASHTO (2009) LRFD specifications the resistance factor for reinforcement long-term tensile rupture is $\varphi = 0.90$. Hence, actual computed values have been replaced by one in the table. For the one case with computed resistance factor less than one, this exception applies to woven geotextiles in a coarse gravel fill (or larger particle sizes) where large strength losses have been recorded in installation damage trials. As noted earlier, this combination of materials should be avoided for reinforced soil wall structures. The large computed values for resistance factor (i.e. greater than one) are the result of the very conservative underlying deterministic load model (Simplified Method) as discussed earlier in this chapter. It can be expected that reducing computed resistance factors that were greater than one to one will decrease the probability of failure to values less than the target value of 1% recommended by Allen et al. (2005).

10.9 Sensitivity analysis

Figure 10.4 shows the results of sensitivity analysis to investigate the quantitative influence of changes in resistance bias statistics on computed probability of failure. A closed-form solution for log-normal distributions that is an approximation to Monte Carlo simulation results was used to create the plots (e.g. Allen et al. 2005). A small quantitative difference of no practical consequence may occur between Monte Carlo simulation results and the closed-form solution. However, the closed-form solution has the advantage of generating smooth plots.

In each plot the baseline for comparison (Case 1) corresponds to mean and bias statistics for installation damage ($\mu_{x_{io}} = 1.05$ and $\text{COV}_{x_{io}} = 0.05$) which were used in the Monte Carlo simulations to arrive at resistance factors matching the modified Simplified Method for cohesionless and cohesive-frictional soil cases. Figure 10.4a demonstrates that the current
AASHTO Simplified Model gives probabilities of failure between 0.1% and 0.01% if there is a perfect match between nominal values of $T_{\text{max}}$ and $T_{\text{sl}}$ during design. Figures 10.4b and 10.4c apply to the modified Simplified model. The curves are all close to the target value of $P_f = 1\%$. The probability curves in every case are similar and in practical terms the differences can be considered to be negligible; this is because the magnitude of probability of failure is largely controlled by the variability in load bias statistics which are much larger than the bias statistics for strength reduction mechanisms.

10.10 Comparison of ASD and LRFD for long-term tensile rupture using computed operational factor of safety (OFS)

It is useful to compare the actual or operational factors of safety (OFS) using different load calculation models and to compare these outcomes with allowable stress design (ASD) past practice.

For the case of a single load term (self-weight plus permanent uniform surcharge) the predicted (design) factor of safety can be estimated as

\[10.11\] \[FS = \frac{T_{\text{sl}}}{T_{\text{max}}} = \frac{\gamma_Q}{\phi}\]

In AASHTO (2009), the recommended load and resistance factor values for the limit state of tensile rupture were derived from fitting to $FS = 1.50$ used in ASD past practice (Column 4 in Table 10.4). The operational factor of safety is defined as the ratio of the measured resistance to measured load and for the limit state under investigation can be expressed as

\[10.12\] \[OFS = \frac{T_{\text{sl}}}{T_{\text{max}}} = \frac{T_{\text{sl}} \mu_{X_R}}{T_{\text{max}} \mu_{X_Q}} = FS \cdot \frac{\mu_{X_R}}{\mu_{X_Q}} \cdot \frac{\gamma_Q}{\phi} \cdot \frac{\mu_{X_Q}}{\mu_{X_Q}} \]

The OFS (Equation 10.12) represents the true factor of safety for tensile rupture by correcting the underlying deterministic models for tensile rupture and load using mean bias values $\mu_{X_R}$ and $\mu_{X_Q}$. If the current models were unbiased (i.e. $\mu_{X_R} = \mu_{X_Q} = 1$), then OFS would be equal to FS.
The OFS values in Columns 2 and 5 in Table 10.4 indicate that ASD using the current AASHTO Simplified Method for load calculations is much safer (more conservative for design) than the proposed LRFD approach in all but one case. It is also interesting to note that ASD standard practice, results in operational factors of safety (Column 5) that are much larger than the specified (design) FS = 1.5. This is not a surprise to experienced reinforced soil wall designers who understand that load calculations are conservative (i.e. safe) and rupture strength is underestimated since documented cases of reinforcement tensile failure in reinforced soil walls are rare. However, this chapter is the first attempt to quantify the actual average factor of safety for long term tensile rupture limit state using the ASD approach which is still used in some design codes (e.g. NCMA 2009; CFEM 2006). Alternatively, the magnitude of conservativeness in ASD practice can be understood by noting that for geogrid reinforced walls and geotextile reinforced soil walls with RFID < 1.7, Pf values are in the range 0.03% to 0.13%. This is one to two orders of magnitude lower than that recommended for LRFD of other redundant soil-structure systems (e.g. pile groups).

10.11 Conclusions

This chapter reports the results of reliability-based LRFD calibration for the long-term rupture limit state in geosynthetic reinforced soil walls due to soil self-weight plus permanent uniform surcharge loading. The general approach follows the LRFD calibration methodology described by Allen et al. (2005) and Bathurst et al. (2008a) which is consistent with the methods used to develop load and resistance factors for bridge superstructures (Nowak and Collins 2000).

The results of calibration show that while current LRFD practice is safe, the current resistance factor \( \varphi = 0.90 \) recommended by AASHTO (2009) for the reinforcement rupture limit state in MSE structures under soil self-weight cannot be justified for the case of geosynthetic reinforced soil walls. This is due largely to the poor prediction accuracy of the underlying deterministic tensile load model (Simplified Method) in current AASHTO design documents. The combination of AASHTO-recommended load factor \( \gamma_Q = 1.35 \) and the poor model for tensile load make it impossible to generate resistance factors less than one consistent with LRFD practice.

To overcome these problems, modifications to the current AASHTO Simplified Method proposed by Huang et al. (2010a) are used to improve the prediction accuracy of the nominal load term used in the tensile rupture limit state function. This new model results in reasonable resistance factor values (i.e. less than one) that match a target minimum probability of failure of 1% when the current AASHTO-recommended load factor of \( \gamma_Q = 1.35 \) is used. For reinforcement
geosynthetics in non-aggressive frictional soils a resistance factor of 0.55 is recommended. For cohesive-frictional soils a value of 0.40 is recommended. These values are lower than $\varphi = 0.90$ recommended in AASHTO design specifications. However, recall that the new modified Simplified Method for reinforcement load reduces nominal reinforcement load estimates by about one third for cohesionless soils. The predicted load values using the new model are statistically more accurate based on measured values (i.e. bias mean is closer to one) than the current AASHTO Simplified Method that is currently restricted to cohesionless soils. Furthermore, the utility of the Simplified Method has been improved by using the method proposed by Miyata and Bathurst (2007a) to convert the strength of cohesive-frictional soils to an equivalent secant friction angle.

An important benefit of the modified Simplified Method used to compute loads is that the corresponding proposed resistance factors give the same 1% probability of failure as the pullout geogrid pullout failure limit state reported in the previous chapter.

REFERENCES


Huang, B., Bathurst, R.J. and Allen, T.M. 2010a. Load and resistance factor design (LRFD) calibration for geogrid pullout limit state using the AASHTO Simplified Method. In review with ASCE JGGE.


Table 10.1 Summary of load bias statistics ($X_Q$) for $T_{\text{max}}$ using current and modified AASHTO Simplified Method (after Huang et al. 2010a).

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Frictional ($c = 0$, $\phi &gt; 0$)</th>
<th>Cohesive-frictional ($c \geq 0$, $\phi &gt; 0$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter</td>
<td>Current model</td>
<td>Modified model</td>
</tr>
<tr>
<td></td>
<td>$\lambda = 0.30$</td>
<td>$\lambda = 0.25$</td>
</tr>
<tr>
<td></td>
<td>$T_{\text{max}} \geq 0.5$ kN/m</td>
<td>$T_{\text{max}} \geq 0.5$ kN/m</td>
</tr>
<tr>
<td>n (number of data points)</td>
<td>58</td>
<td>58</td>
</tr>
<tr>
<td>$\mu_Q$ (mean)</td>
<td>0.30</td>
<td>1.20</td>
</tr>
<tr>
<td>COV$_Q$ (coefficient of variation)</td>
<td>0.54</td>
<td>0.35</td>
</tr>
</tbody>
</table>
Table 10.2 Summary of installation damage database statistics (after Bathurst et al. 2010).

<table>
<thead>
<tr>
<th>Geosynthetic type</th>
<th>Fill types or RFID</th>
<th>Number of RFID values computed</th>
<th>Computed range of RFID (a)</th>
<th>Bias statistics $X_{\text{ID}}$ (b)</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Range</td>
<td>Mean $\mu_{\text{ID}}$</td>
<td>COV $X_{\text{ID}}$ (c)</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>HDPE uniaxial geogrids</td>
<td>Type 1</td>
<td>10</td>
<td>1.05 – 1.43</td>
<td>0.88 – 1.19</td>
<td>1.03</td>
<td>0.06</td>
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Type 2, 3 and 4</td>
<td>17</td>
<td>0.99 – 1.17</td>
<td>0.87 – 1.16</td>
<td>1.02</td>
<td>0.04</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PP biaxial geogrids</td>
<td>Type 1</td>
<td>8</td>
<td>0.97 – 1.45</td>
<td>0.54 – 1.32</td>
<td>1.05</td>
<td>0.20</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Type 2, 3 and 4</td>
<td>15</td>
<td>0.94 – 1.11</td>
<td>0.76 – 1.21</td>
<td>1.05</td>
<td>0.11</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PVC-coated PET geogrids</td>
<td>Type 1</td>
<td>20</td>
<td>1.07 – 1.85</td>
<td>0.78 – 1.71</td>
<td>1.10</td>
<td>0.14</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Type 2, 3 and 4</td>
<td>98</td>
<td>0.95 – 1.39</td>
<td>0.63 – 1.64</td>
<td>1.08</td>
<td>0.10</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Acrylic- and PP-coated PET geogrids</td>
<td>Type 1</td>
<td>4</td>
<td>1.48 – 2.02</td>
<td>0.61 – 1.54</td>
<td>1.08</td>
<td>0.23</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Type 2, 3 and 4</td>
<td>13</td>
<td>1.05 – 1.37</td>
<td>0.82 – 1.41</td>
<td>1.05</td>
<td>0.09</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Woven geotextiles</td>
<td>RFID $\geq$ 1.7</td>
<td>4</td>
<td>3.07 – 4.93</td>
<td>0.29 – 2.07</td>
<td>1.05</td>
<td>0.38</td>
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>RFID $&lt;$ 1.7</td>
<td>20</td>
<td>0.89 – 1.68</td>
<td>0.78 – 1.29</td>
<td>1.07</td>
<td>0.11</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nonwoven geotextiles</td>
<td>RFID $\geq$ 1.7</td>
<td>7</td>
<td>1.74 – 4.96</td>
<td>0.53 – 1.80</td>
<td>1.14</td>
<td>0.26</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>RFID $&lt;$ 1.7</td>
<td>8</td>
<td>1.05 – 1.46</td>
<td>0.82 – 1.51</td>
<td>1.11</td>
<td>0.12</td>
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</tr>
</tbody>
</table>

Notes:
(a) Minimum allowable RFID value for design is 1.10
(b) All bias data are normally distributed
(c) $\text{COV}_{X_{\text{ID}}}$ = standard deviation of bias/mean of bias
Table 10.3 Recommended resistance factor $\phi$ for long-term tensile rupture due to loading from soil self-weight and permanent uniform surcharge (load factor $\gamma_Q = 1.35$).

<table>
<thead>
<tr>
<th>Geosynthetic reinforcement material</th>
<th>Load model (^{(c)(d)})</th>
<th>Original AASHTO (^{(a)(b)})</th>
<th>Modified AASHTO (^{(b)})</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Soil type or RFID value</td>
<td>$c = 0$</td>
<td>$c = 0$ (a)</td>
</tr>
<tr>
<td>Nonwoven geotextiles</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\text{RF}_{ID} \geq 1.7$</td>
<td>1.00</td>
<td>0.40</td>
</tr>
<tr>
<td></td>
<td>$\text{RF}_{ID} &lt; 1.7$</td>
<td>1.00</td>
<td>0.55</td>
</tr>
<tr>
<td>Woven geotextiles</td>
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\text{RF}_{ID} \geq 1.7$</td>
<td>0.55</td>
<td>0.15</td>
</tr>
<tr>
<td></td>
<td>$\text{RF}_{ID} &lt; 1.7$</td>
<td>1.00</td>
<td>0.55</td>
</tr>
<tr>
<td>HDPE uniaxial geogrids</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>All soil types</td>
<td>1.00</td>
<td>0.55</td>
</tr>
<tr>
<td>PP biaxial geogrids</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Type 1</td>
<td>1.00</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>Type 2, 3 and 4</td>
<td>1.00</td>
<td>0.55</td>
</tr>
<tr>
<td>PVC-coated PET geogrids</td>
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<td>All soil types</td>
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<td>0.55</td>
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<tr>
<td>Acrylic-and PP-coated PET geogrids</td>
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<tr>
<td></td>
<td>Type 2, 3 and 4</td>
<td>1.00</td>
<td>0.55</td>
</tr>
</tbody>
</table>

Notes:

\(^{(a)}\) Cohesionless soils only; \(^{(b)}\) With exception of woven geotextile and $\text{RF}_{ID} > 1.7$ probabilities of failure are all greater than 1%; \(^{(c)}\) Soil friction angle computed using peak plane strain friction angle. Using peak friction angles from direct shear and triaxial compression testing will result in the same or smaller probability of failure; \(^{(d)}\) Resistance factors computed to meet probability of failure of 1% using $\gamma_Q = 1.35$ and then round off to nearest 0.05.
Table 10.4 Comparison of operational factor of safety (OFS) and probability of failure ($P_f$) using proposed LRFD and ASD past practice for $c = 0$ backfill soils.

<table>
<thead>
<tr>
<th>Geosynthetic reinforcement material</th>
<th>LRFD using modified AASHTO Simplified Method ($\gamma_Q = 1.35$)</th>
<th>ASD using current AASHTO Simplified Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil type or RFID value</td>
<td>$\phi$</td>
<td>OFS&lt;sup&gt;a&lt;/sup&gt;</td>
</tr>
<tr>
<td>Nonwoven geotextiles</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RFID $\geq 1.7$</td>
<td>0.40</td>
<td>3.85</td>
</tr>
<tr>
<td>RFID $&lt; 1.7$</td>
<td>0.55</td>
<td>2.72</td>
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<tr>
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<td></td>
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<td>9.45</td>
</tr>
<tr>
<td>RFID $&lt; 1.7$</td>
<td>0.55</td>
<td>2.63</td>
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<tr>
<td>HDPE uniaxial geogrids</td>
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<tr>
<td>All soil types</td>
<td>0.55</td>
<td>2.50</td>
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<td>PP biaxial geogrids</td>
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</tr>
<tr>
<td>Type 1</td>
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<td>2.84</td>
</tr>
<tr>
<td>Type 2, 3 and 4</td>
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<td>2.58</td>
</tr>
<tr>
<td>PVC-coated PET geogrids</td>
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<td></td>
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<tr>
<td>Type 1</td>
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<td>2.70</td>
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<tr>
<td>Type 2, 3 and 4</td>
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<td>2.65</td>
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<tr>
<td>Acrylic- and PP-coated PET geogrids</td>
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<td></td>
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<tr>
<td>Type 1</td>
<td>0.45</td>
<td>3.24</td>
</tr>
<tr>
<td>Type 2, 3 and 4</td>
<td>0.55</td>
<td>2.58</td>
</tr>
</tbody>
</table>

Notes:  
<sup>a</sup> $OFS = \frac{\gamma_Q \cdot \mu_{X_P}}{\phi \cdot \mu_{X_o}}$ and bias statistics for entire data sets;  
<sup>b</sup> $OFS = \frac{\mu_{X_P}}{\mu_{X_o}}$ and bias statistics for entire data sets;  
* $FS = \gamma_Q/\phi = 1.35/0.90 = 1.5$
Figure 10.1 Degradation processes for reinforcement strength from original (as-received) condition to the end of design life (Huang et al. 2010b). Range bars and installation time are exaggerated for visual presentation.
Figure 10.2 $D_{50}$ particle size range classifications for installation damage soils. Notes: Heavy black lines and shaded regions represent mean and range of distributions from database of installation tests matching soil types (Table 10.2) (Bathurst et al. 2010).
Figure 10.3 CDF plots for load bias values using original Simplified Method and modified Simplified Method: (a) $c = 0$ backfill soil walls only; (b) all backfill cases ($c \geq 0$).
**Figure 10.4** Sensitivity of probability of failure to magnitude of bias statistics for installation damage, creep and durability ($\gamma_Q = 1.35$): (a) current Simplified Method with cohesionless ($c = 0$) backfill soil walls only ($\phi = 0.90$); (b) modified Simplified Method with cohesionless ($c = 0$) backfill soil walls only ($\phi = 0.55$); (c) modified Simplified Method with cohesive-frictional ($c \geq 0$) backfill soil walls ($\phi = 0.40$).

<table>
<thead>
<tr>
<th>Legend</th>
<th>Mean $\mu_X$</th>
<th>COV$_X$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1: Baseline</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Installation damage</td>
<td>1.05</td>
<td>0.05</td>
</tr>
<tr>
<td>Creep</td>
<td>1.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Durability</td>
<td>1.00</td>
<td>0.00</td>
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<tr>
<td>Case 2:</td>
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<td>Installation damage</td>
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<td>Creep</td>
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<td>Durability</td>
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Figure 10.4 (continued) Sensitivity of probability of failure to magnitude of bias statistics for installation damage, creep and durability ($\gamma_Q = 1.35$): (a) current Simplified Method with cohesionless ($c = 0$) backfill soil walls only ($\phi = 0.90$); (b) modified Simplified Method with cohesionless ($c = 0$) backfill soil walls only ($\phi = 0.55$); (c) modified Simplified Method with cohesive-frictional ($c \geq 0$) backfill soil walls ($\phi = 0.40$).
Chapter 11
Load and Resistance Factor Design (LRFD) Calibration for Geosynthetic Rupture and Pullout Limit States Using the K-Stiffness Method

11.1 Introduction

Geosynthetic reinforcement materials used in reinforced soil walls must be selected to have adequate tensile strength and pullout capacity over the design lifetime of the structure. Deterministic models used in allowable stress design (ASD) to compute reinforcement loads and pullout capacity can be found in design codes (AASHTO 2002, FHWA 2001, CFEM 2006, NCMA 2009, amongst others). Modern design codes are committed to a load and resistance factor design (LRFD) approach to bring the design of reinforced soil retaining walls in line with reliability-based design codes that have been used for decades for buildings and bridges (e.g. AASHTO 2007, 2009; CSA 2006).

Current load and resistance factors that appear in AASHTO (2007, 2009) LRFD codes for internal stability limit states for geosynthetic reinforcement have been based on fitting to allowable stress design (ASD) past practice that uses a factor of safety approach. This approach has the following disadvantages: (a) the probability of failure for a selected limit state or suite of limit states is not controlled, and (b) the underlying deterministic models for prediction of nominal load and resistance values have varying accuracy. In particular it has been demonstrated that the prediction of reinforcement loads under operational conditions is very conservative using the current AASHTO Simplified Method (Allen et al. 2003; Bathurst et al. 2008a). An unfortunate outcome of the poor prediction accuracy of the AASHTO Simplified Method is that LRFD calibration using reliability-based analysis cannot generate resistance factors for limit states with simple permanent load conditions that have magnitudes less than or equal to one as typically expected in LRFD codes (Bathurst et al. 2010b; Huang et al. 2010b). To overcome the poor prediction accuracy of reinforcement loads for internal stability design of reinforced soil walls the empirical-based K-Stiffness Method has been proposed (e.g. Allen et al. 2003; Bathurst et al. 2008a).

This chapter describes the methodology and outcomes for LRFD calibration using reliability-based analysis for the limit states of geosynthetic reinforcement long term rupture (over-stressing) and pullout using the K-Stiffness Method to compute load. The results of analyses are a suite of
resistance factors that can be used for internal stability design of reinforced soil walls using the currently prescribed AASHTO load factor of 1.35 for loads due to soil self-weight plus permanent uniform surcharge loading (AASHTO 2007, 2009) and a target probability of failure of 1%.

11.2 General approach

AASHTO has adopted the reliability-based methodology described by Allen et al. (2005) for LRFD calibration for geotechnical and structural design. Additional details of this approach have been reported by Bathurst et al. (2008a). These two references provide the background to the reliability-based analysis approach used in the current study and in previous related studies for LRFD calibration of steel-reinforced soil walls (Bathurst et al. 2010c; Huang et al. 2010c) and geosynthetic reinforced soil walls (Bathurst et al. 2010a,b; Huang et al. 2010a,b).

A distinguishing feature of the general approach is the use of load and resistance bias statistics to express each limit state function. Bias is defined as the ratio of measured value to predicted value. Bias statistics include the influence of model bias (i.e. intrinsic accuracy of the deterministic model representing the physics of the limit state under investigation), random variation in input parameter values, spatial variation in input values, quality of data and, consistency in interpretation of data when data are gathered from multiple sources (the typical case) (Allen et al. 2005). In this calibration study a single load term is assumed and the fundamental limit state function can be expressed as:

\[ \gamma_Q X_R \geq \phi X_Q \]  

Here, \( X_R \) = resistance bias computed as the ratio of measured resistance \( (R_m) \) to calculated (predicted) nominal resistance \( (R_n) \); and \( X_Q \) = load bias computed as the ratio of measured load \( (Q_m) \) to calculated (predicted) nominal load \( (Q_n) \).

A general condition to carry out calibration using bias statistics (e.g. Equation 11.1) is that there must not be significant hidden dependencies between the bias values and predicted values (Phoon and Kulhawy 2003). Possible dependency can be examined using the Spearman correlation coefficient (e.g. Iman and Conover 1989) or zero-slope test (e.g. Draper and Smith 1981). The zero-slope test is easier to interpret and is used in this study. In this approach, if a zero slope
value is included in the 5% significance interval from linear regression of the bias values against nominal predicted values, then influence of hidden dependency is judged to be negligible.

The load bias statistics in this chapter were computed using the most current version of the K-Stiffness Method (Bathurst et al. 2008b) but with four more load data points added using additional data gathered by the writer since this earlier study. However, these new data points do not detectably change load prediction statistics. The K-Stiffness Method is an empirical design approach for calculating loads in reinforced soil walls under operational conditions and is applicable to both cohesionless ($\phi > 0, c = 0$) and cohesive-frictional ($\phi > 0, c > 0$) backfill soils. Operational conditions are defined as load conditions at the end of construction in which the soil and reinforcement are under working stress conditions and thus at pre-failure conditions. Operational conditions are a necessary condition for the load side in reliability-based design in civil engineering practice (Harr 1987). In fact, the K-Stiffness Method includes an internal soil failure limit state that must not be exceeded in order to ensure that working stress conditions apply. The K-Stiffness Method is now recommended as an alternative design method by the Washington State Department of Transportation (WSDOT 2006).

Two internal limit states are considered in this study: geosynthetic reinforcement rupture (over-stressing) and pullout. For the geosynthetic rupture limit, the resistance term is the available long-term strength of geosynthetic reinforcement. According to North American design codes (FHWA 2001; CFEM 2006; AASHTO 2009; NCMA 2009), the available long-term strength of geosynthetic reinforcement is estimated using an index tensile test value and considering strength reduction due to installation damage, creep and chemical/biological durability. The bias statistics for long-term strength used in this study are taken from statistical data for installation damage reported by Bathurst et al. (2010a) and laboratory creep data reported by Huang et al. (2010a). Uncertainty associated with strength reduction due to chemical and (or) biological processes is treated as deterministic since bias statistics for this degradation process can only be considered on a project-specific basis (WSDOT T925 2005).

Two different resistance models were considered for the pullout limit state and these models have been described in previous related papers (Huang and Bathurst 2009; Huang et al. 2010a). The first (called Model 2) requires project-specific pullout tests to be performed. The second (called Model 5) is a new default (general) model which is a modified version of the current FHWA (2001) default model.
11.3 Reinforcement loads under operational conditions

11.3.1 K-Stiffness Method

The nominal predicted reinforcement loads in this study were computed using the K-Stiffness Method. The method is an empirical method with model parameters back-fitted to loads estimated from measured reinforcement strains converted to loads using suitably selected reinforcement stiffness values. In the initial development of the K-Stiffness Method, calibration was carried out using wall case studies with cohesionless soils (Allen et al. 2003). The method was extended to cohesive-frictional backfill soils by Miyata and Bathurst (2007) and then refined further using more case studies to its current form at the time of writing (Bathurst et al. 2008b). The key features of the method formulation are described here for completion.

The predicted nominal maximum load \( T_{\text{max}} \) in a reinforcement layer using the K-Stiffness Method is expressed as:

\[
T_{\text{max}} = \frac{1}{2} K \sigma_v S_v D_{\text{max}} \Phi
\]

where, \( K \) = lateral earth pressure coefficient; \( \sigma_v \) = vertical stress at the reinforcement elevation due to soil self-weight plus permanent surcharge; \( S_v \) = tributary area (vertical spacing) of the reinforcement layer; \( D_{\text{max}} \) = empirical load distribution parameter with a range from zero to one; and \( \Phi \) = influence factor. Parameter \( K \) is an index parameter and calculated using the Jaky equation for “at rest” earth pressure, hence \( K = 1 – \sin \phi \). However, this does not imply that at-rest conditions apply. This equation is used simply as a familiar equation in geotechnical engineering which captures the empirically-observed effect that reinforcement load increases with decreasing soil friction angle when all other conditions remain the same. Parameter \( \Phi \) is the product of five influence factors that account for the effect of global reinforcement stiffness (\( \Phi_g \)), local reinforcement stiffness (\( \Phi_{\text{local}} \)), facing stiffness (\( \Phi_f \)), facing batter (\( \Phi_b \)), and soil cohesion (\( \Phi_c \)), respectively, on reinforcement load. The maximum value of reinforcement load \( T_{\text{max}} \) in a wall occurs when \( D_{\text{max}} = 1 \). In this chapter this value is denoted as \( T_{\text{mxmx}} \).

The global stiffness factor \( \Phi_g \) is computed as:
\[
\Phi_g = \alpha \left( \frac{S_{\text{global}}}{P_a} \right)^\beta
\]

where, \(\alpha, \beta = \text{constant coefficients (0.25, 0.25)}\); and \(S_{\text{global}} = \text{global reinforcement stiffness of the wall (Christopher et al. 1990)}\) computed as:

\[
S_{\text{global}} = \sum_{i=1}^{n} \frac{J_i}{H}
\]

where, \(J_i = \text{tensile stiffness of an individual reinforcement layer; } n = \text{total number of reinforcement layers in the wall; and } H = \text{height of the wall.}\)

The local stiffness factor \(\Phi_{\text{local}}\) is used to account for the local influence of changes in relative stiffness of a reinforcement layer with respect to the average reinforcement stiffness of the wall and is calculated as:

\[
\Phi_{\text{local}} = \left( \frac{S_{\text{local}}}{S_{\text{global}}} \right)^a
\]

where, \(a = \text{constant coefficient taken as one for geosynthetic reinforced walls and zero for steel reinforced walls (Allen et al. 2004)}\); and \(S_{\text{local}} = \text{local stiffness of reinforcement layer } i \) calculated as:

\[
S_{\text{local}} = \left( \frac{J}{S_v} \right)_i
\]

Here, \(J\) is the tensile stiffness of reinforcement layer \(i\) where the tributary area is \(S_v\).

An important source of conservatism in current design practice for the computation of maximum reinforcement loads in geosynthetic reinforced soil walls is the contribution of a structural facing
to carry earth loads and to transmit them to the footing at the base of the footing. Bathurst et al. (2006) demonstrated that a structural facing can reduce reinforcement loads by a factor of three for low-height walls constructed on a competent foundation and with a horizontally restrained toe. The quantitative comparison was made using two nominal identical full-scale structures: one with a modular block facing and the other with a very flexible wrapped-face construction. In the K-Stiffness Method, the influence of facing stiffness is accounted for using influence factor $\Phi_{fs}$:

$$[11.7] \quad \Phi_{fs} = \eta (F_r)^\kappa$$

where, $\eta$, $\kappa$ = constant coefficients (0.69, 0.11); and $F_r$ = normalized facing column stiffness parameter. In the current version of the K-Stiffness Method, $F_r$ is computed as:

$$[11.8] \quad F_r = \frac{1.5H^3 p_a}{Eb^3 \left( \frac{h_{\text{eff}}}{H} \right)}$$

where, $h_{\text{eff}}$ = equivalent height of an unjointed facing column that is 100% efficient in transmitting moment through the height of facing column; $b$ = thickness of the facing column; $E$ = elastic modulus of the facing column; and $p_a$ = atmospheric pressure. Some subjective rules are required to select the value of $h_{\text{eff}}$. For modular block walls, $h_{\text{eff}}$ is taken as two times the toe-to-heel dimension of the facing units; for continuous and incremental panel walls, $h_{\text{eff}}$ is taken as the wall height $H$ and panel height, respectively; for flexible sand-bag faced walls, $h_{\text{eff}}$ is equal to the primary reinforcement spacing; and for wrapped-face walls, $h_{\text{eff}}$ is equal to the wall height $H$. Parameter $b$ is taken as the length of the horizontal leg of the welded wire facing panel in welded wire walls and the approximate width of the facing wrap for wrapped-face geosynthetic walls. The elastic modulus of the facing column in both cases is taken as the elastic modulus of backfill.

The effect of wall batter on reinforcement loads has been demonstrated in full-scale reinforced soil wall tests on modular block walls (Nernheim et al. 2010) and in numerical modelling (Huang et al. 2010d). These studies confirmed the conventional notion based on classical earth pressure theory that reinforcement loads will decrease with increasing wall batter. Here, wall face
batter refers to the inclination of the facing from the vertical. The wall facing batter influence factor $\Phi_{fb}$ in the K-Stiffness Method is calculated as:

$$\Phi_{fb} = \left( \frac{K_{abh}}{K_{avh}} \right)^d$$

where, $K_{abh}$, $K_{avh}$ = horizontal components of active earth pressure coefficients considering a sloped wall face and a vertical wall face, respectively; and $d =$ constant coefficient (0.5). As wall batter increases, $\Phi_{fb}$ decreases and reinforcement load decreases when all other parameters are kept constant (Equation 11.2).

The influence of soil cohesion on maximum reinforcement load is taken into account by $\Phi_c$:

$$\Phi_c = 1 - \lambda \frac{c}{\gamma H}$$

where, $\lambda =$ constant coefficient (6.5). Parameter $\Phi_c$ is constrained to $0 \leq \Phi_c \leq 1$ and hence $c/\gamma H \leq 0.153$ is necessary before reinforcement load can be generated.

Because the K-Stiffness Method is an empirical-based method with influence factors back-fitted to loads deduced from strain measurements, the method applies only to the envelope of input parameters and wall conditions that were used to perform the calibration. The range of conditions that are applicable to the K-Stiffness Method is reviewed in the next section.

11.3.2 Bias statistics for load

The load bias statistics used in this chapter are taken from the same database used to calibrate the current version of the K-Stiffness Method (Bathurst et al. 2008b) with the exception of four more load data points. The database is composed of strain measurements from 31 different full-scale wall sections taken at the end of construction or shortly thereafter when reinforcement strains were judged to have reached a constant value or were decreasing with time. The reinforcement loads were calculated using a suitably selected stiffness value from in-air constant load creep tests performed on project-specific reinforcement materials or materials in the same product line (Walters et al. 2002).
All walls were judged to have behaved well based on criteria identified by (Allen et al. 2003) and (Bathurst et al. 2008b). The wall heights vary from 3 to 12.6 m. A total of 22 walls were built in the field on natural soils or on a depth of foundation soil in the laboratory. The remaining nine were built on rigid foundations. Hence, the K-Stiffness Method is applicable to walls built on typical competent foundations where the performance of the structures is not influenced by excessive settlements or failure of the foundation or wall toe. Similar foundation criteria apply to the tie-back wedge approach (i.e. Simplified Method) for the calculation of internal reinforcement loads (e.g. AASHTO 2002; FHWA 2001; CFEM 2006; NCMA 2009).

A total of 21 wall sections were constructed with a vertical face; the remaining walls were constructed with facing batter from 3° to 27°. Most of the walls were constructed with a hard structural facing. A total of 58 data points were collected from 13 wall sections built with cohesionless soils and 79 data points from sections built with cohesive-frictional soils. A summary of the K-Stiffness model parameters (Equation 11.2) and their values in the database are given in Table 11.1.

Figures 11.1 and 11.2 show the bias data for $T_{\text{max}}$ and $T_{\text{mxmx}}$ using the K-Stiffness Method. Here, $T_{\text{mxmx}}$ refers to the maximum load from all reinforcement layers in the wall. Hence, the bias value for $T_{\text{mxmx}}$ is the ratio of the largest measured reinforcement load in the wall divided by the largest predicted reinforcement load in the wall. The mean ($\mu_{X_0}$) and coefficient of variation (COV$_{X_0}$) values of all $T_{\text{mxmx}}$ bias data (Figure 11.2) are 0.97 and 0.24, respectively, while corresponding values ($\mu_{X_0}$, COV$_{X_0}$) using all $T_{\text{max}}$ bias data are 0.82 and 0.44, respectively (Figure 11.1). The mean for $T_{\text{mxmx}}$ bias values is close to one because back-fitting of influence factors was based on optimization with the objective function equal to the mean of $T_{\text{mxmx}}$ bias values set to one (Bathurst et al. 2008b).

The mean of $T_{\text{max}}$ bias values is less than one. However, a bias mean less than one for load calculations is desirable because measured loads are close to but less than the predicted value on average which is a conservative (i.e. safe) outcome for design. Figures 11.1b and 11.2b show bias values versus the calculated load values. There is no visual dependency observed based on the linear regressed lines fitted to both $T_{\text{max}}$ and $T_{\text{mxmx}}$ data sets. A zero slope test confirmed no hidden dependency at the 5% significance level. In other words, at this level of significance a zero slope for the regressed line cannot be rejected.
Cumulative distribution function (CDF) plots for load bias data are plotted in Figure 11.1c. A log-normal distribution for \( T_{\text{max}} \) bias data and a normal distribution for the \( T_{\text{maxmx}} \) bias data are fitted to the data points. These approximations were judged to fit the upper tail of the distributions reasonably well. Allen et al. (2005) and Bathurst et al. (2008a) have pointed out that it is the approximation to the upper tail of the load bias data that is important since it is the overlap between the upper tail of the load bias data and lower tail of the resistance bias data that strongly influences the probability of failure in reliability-based design.

A summary of the load bias statistics using the K-Stiffness Method is given Table 11.2. Current design specifications (FHWA 2001; CFEM 2006; AASHTO 2009; NCMA 2009) recommend that frictional (\( c = 0 \)) backfills be used in the reinforced zone of reinforced soil walls. Table 11.2 shows load bias statistics for case study data sets with frictional soils and cohesive frictional soils. Using the K-Stiffness Method, there is no significant difference in the bias statistics for each soil category. In some cases there are small differences in bias values from the earlier study by Bathurst et al. (2008b) but these differences are judged to be insignificant. LRFD calibration results presented later in this chapter are based on the bias statistics for \( T_{\text{max}} \) and \( T_{\text{maxmx}} \) for the larger (current) database.

### 11.4 Bias statistics for pullout resistance \( (P_c) \)

According to AASHTO (2007, 2009) and FHWA (2001) the ultimate pullout capacity for sheet geosynthetics (geotextiles and geogrids) is estimated using:

\[
[11.11] \quad P_c = 2(F^* \alpha)\sigma_v L_e = 2(\psi \tan \phi)\sigma_v L_e
\]

Here, \( L_e = \) anchorage length, \( \sigma_v = \) vertical stress, \( F^* \) and \( \alpha = \) dimensionless parameters, \( \Psi = \tan \phi_{sg}/\tan \phi = \) dimensionless efficiency factor where \( \phi_{sg} = \) peak geosynthetic-soil interface friction angle. In AASHTO/FHWA design codes, the following default values are recommended: \( \alpha = 0.8 \) for geogrids and \( \alpha = 0.6 \) for geotextiles, and \( F^* \) is calculated as:

\[
[11.12] \quad F^* = \frac{2}{3}\tan \phi
\]
In the absence of project-specific data, AASHTO and FHWA guidance documents recommend that the soil peak friction angle be capped at $\phi = 34$ degrees.

**Huang and Bathurst (2009)** compiled a total of 318 geogrid pullout test results from 17 sources. These tests were carried out using flexible vertical loading arrangements such as airbags and pullout boxes meeting the minimum dimensional requirements of *ASTM D6706 (2001)*. The reinforcement types include all three typical commercial geogrid products (uniaxial, biaxial and woven polyester (PET)). This database was used to quantify the prediction accuracy of five pullout models. In their study, no significant statistical differences related to product type were found and hence different pullout models for different product types were not needed. Because there was insufficient number of geotextile pullout tests to develop statistics for this class of products, the pullout limit state calibration in this chapter is restricted to geogrid reinforced soil walls.

In this chapter, two of the five pullout models described by *Huang and Bathurst (2009)* (Models 2 and 5) were used to generate pullout (resistance) bias statistics. Model 2 can be used when project-specific laboratory pullout test results are available. Model 5 is a general non-linear model with default coefficient values that does not require project-specific pullout tests to be performed. Pullout capacity is expressed as:

$$[11.13] \quad P_{\text{corr}} = \chi (P_c)^{1+\kappa} = \chi (2\sigma_sL_cF*\alpha)^{1+\kappa}$$

Here, dimension-dependent terms $\chi$ and $1+\kappa$ are equal to 5.70 and 0.586 when pullout capacity is computed in units of kN/m *(Huang et al. 2010b)*. Implementation of Model 5 is a two-step process. First calculate the pullout capacity ($P_c$) using Equation 11.11 with the default value for coefficient $\alpha$; then, correct this value ($P_{\text{corr}}$) using the power function expression in Equation 11.13. It should be noted that Model 5 in the current study is used with peak friction angles reported in the source materials (see *Huang et al. 2010b*) rather than the more conservative interpretation using the default (capped) value of $\phi = 34$ degrees *(Huang and Bathurst 2009)*. The capped value is recommended in AASHTO and FHWA guidance documents for design when there is no project-specific pullout data and project soil strength data is not available.

Model types for pullout resistance and corresponding bias statistics used in the LRFD calibrations to follow are summarized in Table 11.3. Best fit to lower tail bias statistics are used in the LRFD
calibrations for the reasons noted earlier. There were no hidden dependencies between pullout capacity bias values and predicted pullout capacity for either model at a 5% significance level using the zero slope test.

11.5 Bias statistics for long-term tensile strength (T_{al})

11.5.1 General

Current North American practice \textit{(FHWA 2001; CFEM 2006; AASHTO 2009; NCMA 2009)} is to calculate the (nominal) available long-term tensile strength of a reinforcement layer (T_{al}) as follows:

\begin{equation}
T_{al} = \frac{T_{ult}}{RF} = \frac{T_{ult}}{RF_{ID} \times RF_{CR} \times RF_{D}}
\end{equation}

where, T_{ult} = is the design ultimate tensile strength of the reinforcement; RF = product of reduction factors to account for potential long-term strength loss due to installation damage (RF_{ID}), creep (RF_{CR}) and degradation due to chemical/biological processes (RF_{D}). Typically T_{ult} is a certified minimum value supplied by the vendor or the Minimum Average Roll Value (MARV) of the material based on production quality control. MARV is defined as the strength value computed at two standard deviations below the mean value from multiple tests. Nevertheless, a manufacturer may sometimes report a lower ultimate tensile strength value than that computed from actual testing in order to position the product in the market, to consider variation in reported results between laboratories, or to be cautious. An important consequence of typical strength reporting practice is that a manufacturer-certified minimum T_{ult} value for design can be expected to be lower than the MARV value computed from the mean and standard deviation of ultimate strength values for original test specimens taken from project rolls (e.g. the strength values used in this study). Hence, using a manufacturer-certified minimum T_{ult} value in \textbf{Equation 11.14} will be safer for design. A useful description of the differences noted above can be found in \textit{WSDOT T925 (2005)} and \textit{NTPEP (2007)} guidance documents.

The bias of available long-term tensile strength, X_{al}, is expressed as:
where, \( T_{al,meas} \) = measured available long-term tensile strength of the reinforcement. According to Equations 11.14 and 11.15, \( T_{al,meas} \) can be related to the predicted long-term tensile strength (\( T_{al} \)) through bias statistics for each of the three strength loss terms as follows:

\[
T_{al,meas} = T_{al} \cdot X_{al} = \frac{T_{ult}}{RF_{ID} \times RF_{CR} \times RF_{D}} \cdot X_{al} = \left( \frac{T_{ult}}{RF_{ID}} \right) \cdot X_{ID} \cdot \left( \frac{1}{RF_{CR}} \right) \cdot X_{CR} \cdot \left( \frac{1}{RF_{D}} \right) \cdot X_{D}
\]

Here, \( X_{al} \) is expressed as a product of three bias values:

\[
X_{al} = X_{ID} \cdot X_{CR} \cdot X_{D}
\]

where, \( X_{ID} \) = bias of tensile strength after installation, and \( X_{CR} \) and \( X_{D} \) = bias of values for strength reduction factors \( RF_{CR} \) and \( RF_{D} \), respectively. Assuming that \( X_{ID}, X_{CR}, \) and \( X_{D} \) are uncorrelated, the mean and coefficient of variation (COV) of \( X_{al} \) values are given by (Ang and Tang 1975):

\[
\mu_{X_{al}} = \mu_{X_{ID}} \cdot \mu_{X_{CR}} \cdot \mu_{X_{D}}
\]

and

\[
COV_{X_{al}} = \sqrt{COV_{X_{ID}}^2 + COV_{X_{CR}}^2 + COV_{X_{D}}^2}
\]

Here, the subscripts for each of the mean and COV terms can be matched to installation damage, creep and degradation reduction factors identified earlier. It is noted that Equations 11.18 and 11.19 are only valid for uncorrelated log-normal distributions of the three random variables. However, for uncorrelated normal distributions with small coefficients of variation these
expressions are sufficiently accurate. If a strength reduction process is treated as deterministic, the related bias data will have a mean value of one and coefficient of variance of zero. This is the assumption for creep and durability as discussed later. Hence, only the statistics for $X_{ID}$ are used for the Monte Carlo simulation.

11.5.2 Installation damage

Bathurst et al. (2010a) analyzed a database of field installation damage trials comprised of 69 different geosynthetic products collected from 20 different sources. The database includes a total of 3047 in-air tensile test results (799 for original and 2248 for damaged geosynthetic specimens).

The original ultimate tensile strength for each installation damage trial was computed using the statistics of the tensile strength test results (e.g. ASTM D4595 2009; ASTM D6637 2001) from undamaged and damaged specimens originating from the same geosynthetic sample material; hence:

$$[11.20] \ T_{ult} = \overline{T}_{ult, meas} \left(1 - 2 \cdot \text{COV}_{ult, meas}\right)$$

Here, $\text{COV}_{ult, meas}$ is the coefficient of variation of the measured strength test results for the undamaged specimens. The variability of reinforcement strength immediately after installation can be quantified by the bias value $X_{ID}$ introduced previously and computed as:

$$[11.21] \ X_{ID} = \frac{T_{ID, meas}}{T_{ID}} = \frac{T_{ID, meas}}{\overline{T}_{ult, meas} \left(1 - 2 \cdot \text{COV}_{ult, meas}\right)} \left(\frac{\text{RF}_{ID}}{\overline{T}_{ult, meas} \left(1 - 2 \cdot \text{COV}_{ult, meas}\right)}\right)$$

where, $T_{ID}$ = predicted strength after installation damage, $\overline{T}_{ult, meas}$ = sample mean (baseline) tensile strength for the undamaged material, and $\text{RF}_{ID}$ is the installation damage reduction factor (Equation 11.14) based on project-specific data. Variability in undamaged specimen strength is captured by the variability in installation damage bias values as shown in Equation 11.21.

Based on installation damage trial data available, Bathurst et al. (2010a) proposed the following four soil categories: a) very course gravel with $D_{50} > 19$ mm (Type 1); b) fine gravel with 19 mm
≥ D_{50} > 4.75 \text{ mm} \ (\text{Type 2}); \ c) \ coarse \ to \ medium \ sand \ with \ 4.75 \text{ mm} ≥ D_{50} > 0.425 \text{ mm} \ (\text{Type 3}); \ \text{and d) fine sand or smaller particle sizes with} \ D_{50} ≤ 4.75 \text{ mm} \ (\text{Type 4}). \ Installation \ damage \ performance \ and \ bias \ statistics \ for \ geotextiles \ were \ based \ on \ RF_{ID} \ greater \ than \ or \ less \ than \ 1.7. \ These \ classification \ criteria \ were \ adopted \ in \ this \ chapter. \ For \ geogrid \ products, \ however, \ Bathurst \ et \ al. \ (2010a) \ showed \ that \ bias \ values \ could \ be \ grouped \ into \ two \ ranges \ based \ on \ D_{50} \ of \ the \ soil \ greater \ than \ 19 \text{ mm} \ (i.e. \ Type 1 \ or \ coarse) \ or \ less \ than \ 19 \text{ mm} \ (i.e. \ Types 2, 3 \ and \ 4). \ Table 11.4 \ summarizes \ the \ computed \ X_{ID} \ statistics \ for \ the \ six \ categories \ of \ geosynthetics \ matching \ each \ soil \ category \ (for \ geogrids) \ and \ RF_{ID} \ ranges \ (for \ geotextiles). \ The \ bias \ statistics \ are \ based \ on \ best \ fit \ to \ lower \ tail \ and \ all \ CDF \ plots \ of \ bias \ data \ were \ approximated \ using \ normal \ distributions.

11.5.3 Creep-reduced strength

Tensile strength degradation of geosynthetics due to creep is taken into account by RF_{CR} in current design codes (FHWA 2001; AASHTO 2009; NCMA 2009; CFEM 2006). RF_{CR} is computed as the inverse of the fraction of retained baseline strength that is predicted using a composite creep-rupture envelope. Data points used to construct a composite creep-rupture envelope are based on block shifting of conventional creep tests carried out at different temperatures or using the stepped isothermal method (SIM). Details of the SIM approach can be found in the papers by Thornton et al. (1998a, b) and ASTM D6992 (2009). Conventional practice in North America is to fit a linear line to creep-rupture data points computed as fraction of baseline tensile strength retained and plotted against the logarithm of time to rupture.

Huang et al. (2010a) collected a total of 395 creep strength data points from 18 composite creep-rupture envelopes reported from 16 different sources. The constituent polymer types were PET, polypropylene (PP) and HDPE. The creep tests were conducted on geotextiles, geogrids and yarns. Creep strength bias values (X_{CR}) were computed as follows:

\[
[11.22] \ X_{CR} = \frac{P_{t}'}{P_{t}} = \frac{P_{t}'}{\bar{F}_{ult,meas}/RF_{CR}}
\]

Here, P_{t}' is the measured fraction of tensile strength retained at design life, and P_{t} is the predicted value using the regressed line fitted to all the data on the composite creep-rupture envelope. Huang et al. (2010a) reported the mean and COV values of X_{CR} data to be 1.0 and 0.045,
respectively, when all geosynthetic creep data were treated as a single population. The spread in bias values is very small and there was no practical dependency between $X_{CR}$ bias values for creep durations up to 120 years, which is well in excess of the 75-year design life recommended in current AASHTO specifications.

### 11.5.4 Durability

There is no single test protocol or representative test environment that can be recommended to generate durability bias statistics. The selection of $RF_D$ for design is based on a minimum prescribed value ($RF_D > 1.10$). Project-specific testing is required if a lower value for $RF_D$ is used in design. Recommendations for project-specific durability testing can be found in *WSDSOT T925 (2005)*. For LRFD calibration in this chapter, chemical and biological degradation is assumed to be deterministic. Hence, the mean bias value is taken as $\mu_{X_{ID}} = 1.00$ and $COV_{X_{ID}} = 0$. However, the influence of the magnitude of $COV_{X_{ID}}$ is explored in the sensitivity analysis presented later.

### 11.5.5 Bias statistics for available long-term tensile strength

As mentioned earlier in this chapter, random variables must be uncorrelated (independent) in order for Equations 11.18 and 11.19 to be valid. Also, it has been noted that the inherent variability of material tensile strength ($T_{ult,meas}$) is included in the $X_{ID}$ data and hence should not be repeated in the $X_{CR}$ statistics. *Huang et al. (2010a)* compared bias statistics for original strength and creep-reduced strength data and concluded that the difference in COV values for the two datasets was negligible from a practical point of view. Hence, variability in the prediction of creep-reduced strength is largely captured by the magnitude of variance in the original strength ($T_{ult}$) of the test specimens which is estimated to be about 0.04 by treating all geosynthetic creep data as a single population (*Huang et al. 2010a*). Therefore, it can be assumed that current practice for the determination of $RF_{CR}$ does not introduce additional uncertainty to the calculation of available long-term reinforcement strength. This is consistent with the assumption in *WSDSOT T925 (2005)* that creep strength is proportional to the short-term tensile strength and the variability of $X_{CR}$ is fully covered by variability in the installation damage bias values ($X_{ID}$). Since the mean of creep bias values is by definition equal to one, and assuming that durability mean and COV bias statistics are one and zero, respectively, then for LRFD calibration computations:
[11.23] \( \mu_{x_{a}} = \mu_{x_{d}} \)

and

[11.24] \( \text{COV}_{x_{a}} = \text{COV}_{x_{d}} \)

11.6 Selection of target probability of failure \( P_f \)

The objective of LRFD calibration is to select values of resistance factor and load factor(s) such that a target probability of failure is achieved for the limit state function. In this chapter the target probability of failure is taken as 1 in 100 \( (P_f = 0.01) \) which corresponds to a reliability index value \( \beta = 2.33 \). This target \( P_f \) value has been recommended for reinforced soil wall structures because they are redundant load capacity systems (Allen et al. 2005). If one layer fails in pullout or ruptures due to over-stressing, load is shed to the neighboring reinforcement layers. Piled groups are another example of a redundant load capacity system; failure of one pile does not lead to failure of the group because of load shedding to the remaining piles. Pile groups are designed to a target reliability index value of \( \beta = 2.0 \) to 2.5 (Paikowsky 2004) which contains \( \beta = 2.33 \).

11.7 Selection of load factor \( \gamma_Q \)

There are an infinite number of load and resistance factors that can satisfy a target \( P_f \) value. To calculate the resistance factor, load factors must be selected \emph{a priori}. A load factor of 1.35 is currently recommended by AASHTO (2007, 2009) for reinforcement loads in reinforced soil walls due to soil self-weight and permanent uniform surcharge pressures. This load factor value was adopted in the analysis. Ideally, the choice of load factor will keep the probability of load exceedance at a consistent maximum level for a set of related limit states in bridge and bridge foundation design. During the development of the AASHTO and Canadian bridge design codes, trial load factors were initially selected to ensure that factored dead loads would not be exceeded by actual (measured) loads in more than 3% of cases (Nowak 1999; Nowak and Collins 2000). It is useful to examine the level of load exceedance in the current study.

\textbf{Figure 11.3} shows cumulative distribution plots for the ratio of measured to factored and unfactored reinforcement loads using the K-Stiffness Method. For all data (\textit{Figure 11.3a}), the load exceedance level varies from about 30% for \( \gamma_Q = 1.00 \) (unfactored) to 8% for \( \gamma_Q = 1.35 \); a
load exceedance level of about 3% requires $\gamma_Q = 1.75$. For cohesionless backfill case studies only (Figure 11.3b), similar exceedance levels corresponding to $\gamma_Q$ from 1.00 to 1.75 were found. Bathurst et al. (2008a) carried out a similar analysis of measured and calculated design loads in steel grid reinforced soil walls using the current load model in AASHTO design specifications. They showed that a load factor of 1.75 also corresponds to an exceedance value of about 3% for these structures.

11.8 Calibration results

Monte Carlo simulation was used to compute the resistance factor required to meet a target probability of 1% for a prescribed load factor of 1.35 and using the bias statistics given in Tables 11.2, 11.3 and 11.4. The calculations were carried out using different sets of bias statistics for load and resistance combinations. Two sets of load bias statistics, one for $T_{\text{max}}$ data and the other for $T_{\text{mxmx}}$ data, were used in the calculations. The bias statistics for available long-term tensile strength were selected based on product type (four types of geogrid and two types of geotextile) and backfill type (Types 1, 2, 3 and 4 for geogrids) and $\text{RFID}$ value ($\text{RFID} \geq 1.7$ or $\text{RFID} < 1.7$ for geotextiles). Since the bias statistics for HDPE geogrids installed in different backfill types are very close, one set of bias statistics was used in the calculations for this product class. The pullout resistance models are restricted to Models 2 and 5. Computed resistance factors rounded to the nearest 0.05 are summarized in Table 11.5.

The recommended resistance factors can be seen to be less than one for all cases. This is a desirable outcome for LRFD calibration but is not guaranteed if the underlying deterministic model for load is excessively conservative.

11.9 Sensitivity analysis

Figure 11.4 shows the results of sensitivity analysis to investigate the quantitative influence of changes in resistance bias statistics on computed probability of failure for the rupture limit state using $T_{\text{mxmx}}$ for predicted load. A closed-form solution for log-normal distributions that is an approximation to Monte Carlo simulation results was used to create the plots (e.g. Allen et al. 2005). A small quantitative difference of no practical consequence may occur between Monte Carlo simulation results and the closed-form solution for $\text{COV}_R$ values in excess of (say) 0.30. However, the closed-form solution has the advantage of generating smooth plots. The baseline for comparison (Case 1) corresponds to mean and bias statistics for installation damage taken as $\mu_{\text{Xid}} = 1.05$ and $\text{COV}_{\text{Xid}} = 0.05$. The bias statistics for $T_{\text{mxmx}}$ were kept constant at $\mu_Q = 1.00$ and
$COV_Q = 0.25$. The mean of resistance bias values $\mu_R$ (Equation 11.18) is 1.05 and the range of $COV_R = 0.05$ to 0.35 (Equation 11.19).

The figures show that increasing the COV of bias values for contributing strength reduction processes leads to visually detectable increases in the probability of failure for a selected $\gamma_Q/\phi$ value. However, in the vicinity of the baseline case and a probability of failure of 1% the deviation from the 1% target value is judged to be of no practical significance (a maximum of 3%). The computed curves at $P_f = 1\%$ all fall within the range of $\gamma_Q/\phi$ values corresponding to the recommended resistance factors listed in Table 11.5. As the level of installation damage goes up the recommended resistance factor will decrease. Hence, the ratio of design $\gamma_Q/\phi$ will increase to the right in the order of Figure 11.4a, 11.4b and 11.4c. A practical outcome of this analysis is that for all installation damage levels up to a $COV_{X_{cr}}$ of 0.30 plus additional uncertainty due to creep up to $COV_{X_{cr}}$ of 0.05 and durability up to $COV_{X_0}$ of 0.10, the recommended resistance factors result in a probability of failure in the vicinity of 1% to 3%.

11.10 Implications to design

Table 5 shows that for most cases, resistance factor values for the tensile rupture limit state computed using the load bias statistics for $T_{mxmx}$ are greater than those using the load bias statistics for $T_{max}$. The underlying reason for the higher resistance factor for design using the most heavily loaded reinforcement layer is that the COV of the bias statistics for $T_{mxmx}$ is smaller than for $T_{max}$. Designing for the critical reinforcement load layer in a reinforced soil wall is common practice for some designers because it simplifies design and construction. The relative margin of safety using both approaches can be quantified using computed operational factors of safety discussed later in this section.

There are a number of other observations in this table that provide important insight into the relationship between resistance factor and project materials. For example, in general, as the soil environment becomes more aggressive for a candidate geosynthetic (e.g. Type 1 soil versus the finer particle size gradations) the resistance factor is less. This is the result of more variability (scatter) in the prediction of strength loss due to installation damage in more aggressive soil environments. For woven geotextiles, the scatter in installation bias values is very large for cases with $RF_{ID} \geq 1.7$. In practical terms, the large nominal strength losses for these geosynthetics may make them a poor choice in very coarse aggregate soils. Clearly, the best strategy to maximize the
available tensile strength for design is to match the mass per unit area of the geotextile to the soil so that the computed installation damage factor (RF_{ID}) is less than 1.7.

An important benefit of using project-specific pullout testing is that pullout capacity can be computed using Model 2. Model 2 is more accurate than the default Model 5 and allows a larger resistance factor value to be used. However, if minimum reinforcement lengths due to other limit state calculations or a minimum anchorage length criterion (e.g. 1 m in AASHTO design guidelines) are imposed on internal stability design, then the choice of pullout model may not be a factor in final design outcomes as demonstrated by Huang et al. (2010b).

Also presented in Table 11.5 are the actual or operational factors of safety (OFS) for different load and resistance combinations and design based on T_{max} or T_{maxx}. However, it is important to distinguish between OFS and the factor of safety (FS) used in conventional ASD past practice. For the case of a single load term (the case here) the (design) factor of safety, FS, can be expressed as:

\[ \text{[11.25]} \quad FS = \frac{R_n}{Q_n} = \frac{\gamma_Q}{\phi} \]

where, \( R_n \) and \( Q_n \) were introduced at the beginning of the chapter as the nominal predicted resistance and load values, respectively. The operational factor of safety, OFS, is defined as the ratio of the measured resistance \( (R_m) \) to measured load \( (Q_m) \). OFS can be expressed as:

\[ \text{[11.26]} \quad \text{OFS} = \frac{R_m}{Q_m} = \frac{R_n \mu_R}{Q_n \mu_Q} = \text{FS} \cdot \frac{\mu_R}{\mu_Q} = \frac{\gamma_Q}{\phi} \cdot \frac{\mu_R}{\mu_Q} \]

This formulation demonstrates that the mean bias values \( (\mu_R \text{ and } \mu_Q) \) for the underlying deterministic resistance and load models (with one load term) can be used to correct the target design factor of safety to the actual operational factor of safety. If the models to predict load and resistance were unbiased (i.e. \( \mu_R = \mu_Q = 1 \)), OFS will be equal to FS. Otherwise, the parameters cannot be directly compared.
All OFS values are much larger than the minimum factor of safety of 1.5 specified for both pullout and rupture modes of failure in ASD past practice. Designing for the maximum reinforcement load in a layer will result in greater operational margins of safety. Nevertheless, the K-Stiffness Method results in operational factors of safety for the two limits states investigated that are judged to be adequate according to conventional notions of safety in reinforced soil wall design.

11.11 Conclusions
This chapter describes LRFD calibration for the limit states of reinforcement tensile rupture and pullout in geosynthetic reinforced soil walls due to soil self-weight plus permanent uniform surcharge loading. This is the first time that the current K-Stiffness Method has been used for reinforcement load prediction in LRFD calibrations. The general calibration approach follows the methodology described by Allen et al. (2005) and Bathurst et al. (2008a). This approach is consistent with the methods used to develop load and resistance factors for bridge superstructures (Nowak and Collins 2000).

The results of calibration for the rupture limit state are presented as a table of resistance factors matching different combinations of geosynthetic type and soil (or installation damage factor) and reinforcement load type (i.e. maximum load in a reinforcement layer or maximum reinforcement load in the wall). Resistance factors for the pullout limit state are also summarized. All resistance factors were computed using the current AASHTO prescribed load factor of 1.35 and a probability of failure of 1%.

An important benefit of the LRFD approach used here is that the suite of proposed resistance factors gives the same 1% probability of failure for both reinforcement rupture and pullout limit states. An additional benefit is that the K-Stiffness Method for load calculations is applicable to both cohesionless and cohesive-frictional backfill soils which is not the case for the current AASHTO Simplified Method and variants that appear in current design guidelines. Finally, the accuracy of load predictions using the K-Stiffness Method for operational conditions is much greater than current ASD practice using the AASHTO Simplified Method and the operational factors of safety remain greater than the (nominal) factor of safety of 1.50 recommended in ASD past practice (AASHTO 2002).
REFERENCES

Bathurst, R.J., Huang, B. and Allen, T.M. 2010b. Load and resistance factor design (LRFD) calibration for rupture limit state using the AASHTO Simplified Method. In review with ASCE JGGE.


Huang, B. and Bathurst, R.J. 2009. Evaluation of soil-geogrid pullout models using a statistical approach. ASTM Geotechnical Testing Journal, 32(6), (online)


### Table 11.1 Summary of the model parameter values of the K-Stiffness Method.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Mean</th>
<th>COV&lt;sup&gt;(c)&lt;/sup&gt;</th>
<th>Min</th>
<th>Max</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma$ (kN/m$^3$)</td>
<td>17.9</td>
<td>0.11</td>
<td>14.7</td>
<td>22.0</td>
</tr>
<tr>
<td>$\phi_{\text{peak}}$&lt;sup&gt;(a)&lt;/sup&gt; (deg)</td>
<td>43.5</td>
<td>0.20</td>
<td>25.0</td>
<td>55.0</td>
</tr>
<tr>
<td>$\phi_{\text{sec}}$&lt;sup&gt;(b)&lt;/sup&gt; (deg)</td>
<td>44.8</td>
<td>0.18</td>
<td>27.0</td>
<td>57.0</td>
</tr>
<tr>
<td>$S_v$ (m)</td>
<td>0.80</td>
<td>0.39</td>
<td>0.25</td>
<td>2.40</td>
</tr>
<tr>
<td>$\sigma_v$ (kPa)</td>
<td>62</td>
<td>0.70</td>
<td>5</td>
<td>243</td>
</tr>
<tr>
<td>$K$</td>
<td>0.32</td>
<td>0.35</td>
<td>0.18</td>
<td>0.58</td>
</tr>
</tbody>
</table>

Influence factors: in Equation 11.2

$$\Phi = \Phi_g \Phi_{\text{local}} \Phi_{\text{fb}} \Phi_{\text{fs}} \Phi_c$$

<table>
<thead>
<tr>
<th>Influence factor</th>
<th>Mean</th>
<th>COV&lt;sup&gt;(c)&lt;/sup&gt;</th>
<th>Min</th>
<th>Max</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Phi_g$</td>
<td>0.36</td>
<td>0.13</td>
<td>0.24</td>
<td>0.49</td>
</tr>
<tr>
<td>$\Phi_{\text{local}}$</td>
<td>1.02</td>
<td>0.32</td>
<td>0.30</td>
<td>2.68</td>
</tr>
<tr>
<td>$\Phi_{\text{fb}}$</td>
<td>0.94</td>
<td>0.15</td>
<td>0.36</td>
<td>1.00</td>
</tr>
<tr>
<td>$\Phi_{\text{fs}}$</td>
<td>0.74</td>
<td>0.34</td>
<td>0.42</td>
<td>1.58</td>
</tr>
<tr>
<td>$\Phi_c$</td>
<td>0.76</td>
<td>0.38</td>
<td>0.20</td>
<td>1.00</td>
</tr>
</tbody>
</table>

**Notes:**

(a) Peak friction angle from triaxial compression or direct shear tests
(b) Peak secant plane strain friction angle computed from reported peak shear strength parameters
(c-\(\phi\)) using the method reported by Miyata and Bathurst (2007).
(c) COV = coefficient of variation = standard deviation/mean
Table 11.2 Summary of load bias statistics.

<table>
<thead>
<tr>
<th></th>
<th>Backfill: $c = 0$ only</th>
<th>Backfill: $c \geq 0$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$T_{\text{max}}$</td>
<td>$T_{\text{mxmx}}$</td>
</tr>
<tr>
<td>Number of data points, $n$</td>
<td>58</td>
<td>13</td>
</tr>
<tr>
<td>$\mu_{X_q}$</td>
<td>0.83</td>
<td>0.94 (0.98)</td>
</tr>
<tr>
<td>$\text{COV}_{X_q}$</td>
<td>0.40</td>
<td>0.22 (0.21)</td>
</tr>
<tr>
<td>Min</td>
<td>0.25</td>
<td>0.57</td>
</tr>
<tr>
<td>Max</td>
<td>2.03</td>
<td>1.31</td>
</tr>
</tbody>
</table>

Notes: Values in parentheses are from Bathurst et al. (2008b) where numbers are different from current study. Where there are differences these are due to the smaller number of load data points (133 in the earlier study versus 137 in this study).
Table 11.3 Bias statistics reported by Huang et al. (2010b) for pullout capacity Model 2 and 5 reported by Huang and Bathurst (2009).

<table>
<thead>
<tr>
<th>Model</th>
<th>Description</th>
<th>Bias statistics</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mean $\mu_R$</td>
</tr>
<tr>
<td>2</td>
<td>First-order approximation to measured $F^*\alpha$</td>
<td>1.17</td>
</tr>
<tr>
<td>5</td>
<td>Non-linear model</td>
<td>1.12</td>
</tr>
</tbody>
</table>

Notes: Model 2 (Equations 11.11 and 11.12) requires project-specific pullout test data; Model 5 (Equation 11.13) requires input parameters $\sigma_v$, $L_c$, and $\phi$ matching project design.
Table 11.4 Summary of the installation damage database statistics (after Bathurst et al. 2010a).

<table>
<thead>
<tr>
<th>Geosynthetic type</th>
<th>Fill types or RF&lt;sub&gt;TD&lt;/sub&gt;</th>
<th>Number specimens</th>
<th>Bias statistics X&lt;sub&gt;TD&lt;/sub&gt;</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Undamaged</td>
<td>Damaged</td>
<td>Mean µ&lt;sub&gt;x&lt;/sub&gt;</td>
</tr>
<tr>
<td><strong>HDPE uniaxial geogrids</strong></td>
<td>Type 1</td>
<td>52</td>
<td>92</td>
<td>1.03</td>
</tr>
<tr>
<td></td>
<td>Types 2, 3 and 4</td>
<td>46</td>
<td>168</td>
<td>1.02</td>
</tr>
<tr>
<td><strong>PP biaxial geogrids</strong></td>
<td>Type 1</td>
<td>40</td>
<td>80</td>
<td>1.05</td>
</tr>
<tr>
<td></td>
<td>Types 2, 3 and 4</td>
<td>40</td>
<td>150</td>
<td>1.05</td>
</tr>
<tr>
<td><strong>PVC-coated PET geogrids</strong></td>
<td>Type 1</td>
<td>131</td>
<td>264</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td>Types 2, 3 and 4</td>
<td>222</td>
<td>605</td>
<td>1.08</td>
</tr>
<tr>
<td><strong>Acrylic- and PP-coated PET geogrids</strong></td>
<td>Type 1</td>
<td>40</td>
<td>36</td>
<td>1.08</td>
</tr>
<tr>
<td></td>
<td>Types 2, 3 and 4</td>
<td>20</td>
<td>70</td>
<td>1.05</td>
</tr>
<tr>
<td><strong>Woven geotextiles</strong></td>
<td>RF&lt;sub&gt;TD&lt;/sub&gt; ≥ 1.7</td>
<td>20</td>
<td>40</td>
<td>1.05</td>
</tr>
<tr>
<td></td>
<td>RF&lt;sub&gt;TD&lt;/sub&gt; &lt; 1.7</td>
<td>133</td>
<td>208</td>
<td>1.07</td>
</tr>
<tr>
<td><strong>Nonwoven geotextiles</strong></td>
<td>RF&lt;sub&gt;TD&lt;/sub&gt; ≥ 1.7</td>
<td>36</td>
<td>60</td>
<td>1.14</td>
</tr>
<tr>
<td></td>
<td>RF&lt;sub&gt;TD&lt;/sub&gt; &lt; 1.7</td>
<td>58</td>
<td>90</td>
<td>1.11</td>
</tr>
</tbody>
</table>
Table 11.5 Summary of recommended resistance factor values for $\beta = 2.33$ ($P_f = 1\%$) and $\gamma_Q = 1.35$ using the K-Stiffness Method for load calculations.

<table>
<thead>
<tr>
<th>Limit state</th>
<th>Geosynthetic reinforcement material</th>
<th>Resistance factor $^{(a)}$</th>
<th>OFS $^{(b)}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Soil type or RFID value</td>
<td>$\varphi$ Using $T_{max}$</td>
<td>Using $T_{max}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$T_{\text{mx}}$</td>
<td>$T_{\text{mx}}$</td>
</tr>
<tr>
<td>Rupture</td>
<td>HDPE uniaxial geogrids</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>All soils types</td>
<td>0.70</td>
<td>2.40</td>
</tr>
<tr>
<td></td>
<td>PP biaxial geogrids</td>
<td>0.60</td>
<td>2.88</td>
</tr>
<tr>
<td></td>
<td>Type 1</td>
<td>0.70</td>
<td>2.47</td>
</tr>
<tr>
<td></td>
<td>Types 2, 3 and 4</td>
<td>0.70</td>
<td>2.47</td>
</tr>
<tr>
<td></td>
<td>PVC-coated PET geogrids</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Type 1</td>
<td>0.70</td>
<td>2.47</td>
</tr>
<tr>
<td></td>
<td>Types 2, 3 and 4</td>
<td>0.70</td>
<td>2.47</td>
</tr>
<tr>
<td></td>
<td>Acrylic- and PP-coated PET geogrids</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Type 1</td>
<td>0.60</td>
<td>2.96</td>
</tr>
<tr>
<td></td>
<td>Types 2, 3 and 4</td>
<td>0.70</td>
<td>2.47</td>
</tr>
<tr>
<td></td>
<td>Woven geotextiles</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\text{RFID} \geq 1.7$</td>
<td>0.20</td>
<td>8.64</td>
</tr>
<tr>
<td></td>
<td>$\text{RFID} &lt; 1.7$</td>
<td>0.70</td>
<td>2.52</td>
</tr>
<tr>
<td></td>
<td>Nonwoven geotextiles</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\text{RFID} \geq 1.7$</td>
<td>0.55</td>
<td>3.41</td>
</tr>
<tr>
<td></td>
<td>$\text{RFID} &lt; 1.7$</td>
<td>0.70</td>
<td>2.61</td>
</tr>
<tr>
<td></td>
<td>Pullout</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Pullout (Resistance) Model 2</td>
<td>0.55</td>
<td>2.99</td>
</tr>
<tr>
<td></td>
<td>Pullout (Resistance) Model 5</td>
<td>0.40</td>
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354
Table 11.5 (continued) Summary of recommended resistance factor values for $\beta = 2.33$ ($P_f = 1\%$) and $\gamma_Q = 1.35$ using the K-Stiffness Method for load calculations.

Notes: (a) Soil friction angle computed using peak plane strain friction angle. Using peak friction angles from direct shear and triaxial compression testing will result in the same or smaller probability of failure. Resistance factors computed to meet probability of failure of 1% using $\gamma_Q = 1.35$ and then round off to nearest 0.05. (b) $OFS = \frac{\gamma_Q}{\phi} \cdot \frac{\mu_R}{\mu_Q}$ and bias statistics for entire data sets
Figure 11.1 Measured and predicted load data for $T_{\text{max}}$ using the K-Stiffness Method: (a) measured versus calculated loads; (b) load bias versus calculated loads; and (c) CDF plots for load bias data.

(a) Measured load (kN/m) vs. calculated load (kN/m)

(b) Load bias, $X_Q$, vs. calculated load, $T_{\text{max}}$ (kN/m)

(c) CDF plots for load bias data

Mathematical equations and data:

- Load bias, $X_Q = 0.0064 T_{\text{max}} + 0.8104$
- All data
- $n = 137$
- $\mu_Q = 0.82$
- COV$Q = 0.44$
- $\mu_Q$ (all data)
- $c > 0$
- $c = 0$

Graphical representations:
- Scatter plots with data points
- Line graphs
- CDF curves

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Figure 11.2 Measured and predicted load data for $T_{mxmx}$ using the K-Stiffness Method: (a) measured versus calculated loads; (b) load bias versus calculated loads; and (c) CDF plots for load bias data.

(a) Calculated load (kN/m)

Measured load (kN/m)

(b) Load bias, $X_Q$

(c) Standard normal variable

$X_Q = -0.0122 T_{mxmx} + 0.9975$

$T_{mxmx}$ data only

$n = 30$

$\mu_{Q_{T_{mxmx}}}$

$\text{COV}_{Q_{T_{mxmx}}} = 0.24$

$\mu_{Q_{T_{mxmx}}}$ (data only)

$c > 0$

$c = 0$

$\mu_{Q_{T_{mxmx}}}$ (data only)

$\mu_{Q_{T_{mxmx}}}$ (Tmxmx data only)

$X_Q = -0.0122 T_{mxmx} + 0.9975$

$
\begin{align*}
\mu_{Q_{T_{mxmx}}} & = 0.97 \\
\text{COV}_{Q_{T_{mxmx}}} & = 0.24
\end{align*}
$
Figure 11.3 Cumulative fraction plots for the ratio of measured to factored and unfactored calculated load values using the K-Stiffness Method: (a) all soil backfill cases; and (b) frictional backfill soils.
Figure 11.4 Sensitivity of probability of failure to magnitude of bias statistics for installation damage, creep and durability ($\gamma_Q = 1.35$): (a) low installation damage (COV$_{ID} = 0.05$); (b) medium installation damage (COV$_{ID} = 0.20$); (c) high installation damage (COV$_{ID} = 0.30$).

Notes: Load bias statistics for $T_{\text{max}}$ taken as $\mu_Q = 1.00$ and COV$_Q = 0.25$. The range of $\gamma_Q / \phi$ between 1.59 and 2.45 was computed using $\gamma_Q = 1.35$ and $\phi = 0.55$ and 0.85 (Table 11.5)

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<tr>
<td>Creep</td>
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<tr>
<td>Durability</td>
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Notes: Load bias statistics for $T_{\text{max}}$ taken as $\mu_Q = 1.00$ and COV$_Q = 0.25$. The range of $\gamma_Q / \phi$ between 1.59 and 2.45 was computed using $\gamma_Q = 1.35$ and $\phi = 0.55$ and 0.85 (Table 11.5)
**Figure 11.4 (continued)** Sensitivity of probability of failure to magnitude of bias statistics for installation damage, creep and durability ($\gamma_Q = 1.35$): (a) low installation damage (COV ID = 0.05); (b) medium installation damage (COV ID = 0.20); (c) high installation damage (COV ID = 0.30).
Chapter 12

General Discussion

The focus of this thesis work has been on the internal stability analysis, performance, prediction and load and resistance factor design (LRFD) calibration of reinforced soil retaining walls constructed with geosynthetic and steel reinforcement materials. The results reported in Chapters 3 to 11 provide supporting materials to update the reinforced soil wall design content in the next versions of the American Association of State Highway and Transportation Officials (AASHTO) and Canadian Highway Bridge Design Codes. Nevertheless, the approaches adopted and conclusions drawn in individual chapters also provide valuable guidance for LRFD calibration of other soil-structure interaction problems.

Three well-known constitutive soil models were implemented in a numerical code for reinforced soil walls in Chapter 2. In the order of increasing complexity they were: (i) a simple elastic-plastic Mohr-Coulomb model; (ii) a modified Duncan-Chang model; and (iii) Lade’s model. The selection of constitutive soil model in numerical modelling of geotechnical engineering problems should be a balance between prediction accuracy, load (stress) level, availability of parameters from laboratory testing, and computational effort. The modified Duncan-Chang model is recommended to predict reinforcement loads in reinforced soil walls under operational (working stress) conditions. In this thesis, operational conditions are defined as load conditions in which there is no component over-stressing including contiguous failure mechanisms through the reinforced soil mass. The simple elastic-plastic Mohr-Coulomb model is not recommended because accurate predictions depend on a fortuitous choice of elastic modulus for the soil.

Another major shortcoming of the simple elastic-plastic Mohr-Coulomb model is that soil stress-dependent elastic response cannot be captured using a single elastic modulus value. The Mohr-Coulomb model is best suited for the analysis of structural response at incipient collapse (failure). To overcome this problem, stress-dependent elastic models, such as the Lade-Nelson model described in Chapter 2, can be used as the elastic component of the constitutive soil model. Theoretically, Lade’s model is capable of simulating the entire range of soil response including soil plasticity and dilatancy. However, Lade’s model requires nine plasticity-related parameters, most of which lack obvious physical meaning. Some parameters are very difficult to determine using conventional laboratory testing equipment. In addition, Lade’s model requires considerably larger computing effort than the other two simple models. Therefore, Lade’s model is attractive theoretically but may not be practical for most geotechnical engineering problems. The modified
Duncan-Chang model developed as part of this thesis work has great potential for a wide range of engineered earth fill problems where the structure is designed for working stress conditions (i.e. small soil strains).

As geotechnical engineering design moves towards performance-based design, structural response under operational conditions is of particular interest to design engineers and owners. In fact, loads due to working stress (operational) conditions rather than at incipient failure are assumed in current LRFD codes in North America. This concept has been used in the LRFD calibrations in this thesis. The empirical criterion used to identify operational conditions in this thesis work is that maximum strain in any reinforcement layer must not exceed 3%. Observations from a large number of reinforced soil field walls have shown that contiguous failure zones through the soil mass and other obvious signs of poor wall performance did not occur if this criterion was satisfied.

If the reinforcement loads are restricted to operational conditions and the original site and boundary conditions for which the wall is designed do not change, an ultimate limit state (collapse of the wall due to internal stability modes of failure) based on limit equilibrium conditions will not occur.

Ideally, LRFD calibration should be carried out based on the failure capacity (ultimate limit state) or a deformation criterion (serviceability limit state) for each element analyzed. For the case of a structural steel member, the ultimate resistance of the member is based (typically) on flexure, shear or tensile capacity, and serviceability defined by a prescribed allowable deformation. However, an important feature of the LRFD calibration philosophy adopted in this thesis is that the computed resistance factor value can satisfy the requirement of both failure capacity and a deformation criterion simultaneously. The same concept can be applied to other geotechnical applications such as pipes and other buried infrastructure. A common characteristic of the working stress conditions in these soil-structure interaction problems, including reinforced soil retaining walls, is that the magnitude of load effects is dependent upon the deformation of the structure under investigation.

The LRFD calibration approach used in this thesis is based on bias statistics. Bias is defined as the ratio of measured value to predicted value. An important feature of this approach is that the influence of model error (i.e. intrinsic accuracy of the deterministic model representing the physics of the limit state under investigation) is included in the bias statistics. As shown in the
previous chapters in this thesis, significant prediction error can result from the design model alone. This is often an important challenge in recasting allowable stress design (ASD) past practice into a properly calibrated LRFD framework, because many load and resistance design models in geotechnical design practice are empirical or semi-empirical based. Practitioners in the past have tended to develop design models based on lower bound resistance capacity data and upper bound load data in order to ensure “safe” designs. These design models often lead to: (i) a bias mean that is seldom close to one; (ii) a large bias coefficient of variation (COV) value; and (iii) significant hidden dependency between the bias and calculated values. In design codes, load factor values are typically greater than or equal to one and resistance factor values are always less than or equal to one. As shown in this thesis, an obvious consequence of using these models for LRFD calibration is that unreasonable load and/or resistance factor values may result. An underlying design model with large bias COV value will lead to a lower required resistance factor value which means less efficient use of the resistance capacity (i.e. higher cost). Significant hidden dependency between the bias and calculated values will increase the chance of over-design or under-design because the prediction accuracy of the design model is not constant but varies depending on the magnitude of the calculated value. Therefore, an important step in LRFD calibration is to evaluate the prediction accuracy of the underlying deterministic models and make corrections if necessary. This thesis provides several successful examples in this regard. In particular, the use of a power function to improve the prediction accuracy of the current AASHTO default pullout model (Chapter 6) is applicable to many other empirical and semi-empirical geotechnical design models.

Closed-form solutions and the Monte Carlo method were used to find the probability of failure for reinforcement pullout and over-stressing limit states in this thesis. The simple Monte Carlo method used in this thesis work was judged to be sufficiently accurate because the limit state functions analyzed involve only two random variables (load bias and resistance bias) and the target probability of failure is quite high (1%) compared with typical reliability analysis for structural engineering problems. If the limit state function involves many (say, more than four) random variables and/or the probability of failure is very small (say, commonly 1 in 5000 in structural engineering), advanced Monte Carlo methods, such as Latin Hypercube sampling technique or low discrepancy sequences, may be needed.

Since it is the overlap between the upper tail of the load bias data and lower tail of the resistance bias data that strongly influences the probability of failure in reliability-based design, proper quantification of the distribution of data in the tail of bias cumulative distribution function plots is
important for the success of LRFD or LRFD calibration. In practice, care must be exercised to evaluate the computed results using different fit-to-tail trials to give a reasonable estimate of the probability of failure of the structure regardless of the type of application and computing method used.

Influence of hidden prediction accuracy dependency in an underlying deterministic model must be evaluated in LRFD calibration work. Among several evaluation approaches, the zero-slope test adopted in this thesis work is recommended for geotechnical problems because it is simple to implement and the results easy to interpret. However, the significance level (e.g. 5% in this thesis) used in the zero-slope test should be selected based on the quantitative influence of any dependency on the end result for design. In other words, actual over-design or under-design effects (e.g. safety level and cost) must be assessed on a case-to-case basis for a target level of significance used in the zero slope test.

Practical consequences of using ASD past practice or a LRFD approach are often a concern to geotechnical engineers. The concept of operational factor of safety (OFS) is proposed for the first time in this thesis. The calculation of OFS can be applied to any soil-structure interaction problem for which sufficient load and resistance bias data are available. The calculation of OFS allows the geotechnical engineer to use a common quantitative measure of the relative margin of safety using the two fundamentally different design approaches (i.e. ASD and LRFD). This comparison will provide additional confidence for geotechnical engineers to accept an LRFD approach for soil-structure interaction problems where ASD is currently common practice.
Chapter 13

Summary, Conclusions and Recommendations

13.1 General

The work presented in this thesis is focused on numerical investigations and load and resistance factor design (LRFD) calibration of reinforced soil retaining walls under simple static loading conditions. The findings of this work are expected to be of value to the analysis, design and construction of these structures. This chapter summarizes the major results and conclusions drawn from individual chapters of the thesis. Recommendations for future related research work are also provided. The most valuable outcome of this thesis work is a set of recommended resistance factor values for the pullout and rupture limit states for reinforced soil retaining walls that are tabulated in Appendix C.

13.2 Summary and conclusions

The results of numerical investigations presented in Chapter 2 demonstrate that simple soil constitutive models are adequate to predict the performance of reinforced soil walls under typical operational conditions provided that the soil reinforcement, interfaces, construction sequence and soil compaction are modelled correctly. Further improvement of predictions using more sophisticated soil models is not guaranteed. Based on a balance between accuracy, simplicity and computing intensity, the modified Duncan-Chang hyperbolic model is recommended as the best choice for soil model of the three soils models investigated. This soil model has applications to numerical modelling of other reinforced soil earth structures.

Numerical results in Chapter 3 show that a restrained wall toe can generate a significant portion of the resistance to horizontal earth pressures in a wall. This partially explains why reinforcement loads under operational (working) stress conditions are typically over-estimated using current limit equilibrium-based design methods (e.g. the AASHTO Simplified Method). The K-Stiffness Method is shown to better capture the qualitative trends in reinforcement loads and is also quantitatively more accurate compared to the AASHTO Simplified Method. An important practical implication of this investigation to wall design and construction is that: (a) the wall toe should be embedded (this is typical design practice); (b) good frictional contact is developed at the base of the wall facing column and the concrete or granular levelling pad; or (c) a mechanical shear key is placed between the bottom block and a concrete footing.
In Chapter 4, a new LRFD approach is proposed for the limit states of pullout and rupture for steel grid reinforced soil walls. Resistance factor values of 0.40 and 0.65 are recommended for a target probability of failure of 1% and a prescribed load factor of 1.35 for pullout and rupture, respectively. Furthermore, comparison with ASD past practice shows that the operational factor of safety for pullout using the new LRFD-based approach gives a higher factor of safety and a lower probability of failure. For the limit state of rupture, the new LRFD and ASD past practice approaches give similar operational factors of safety and probability of failure. It is noted that the observed satisfactory performance of field walls results from additional safety from other sources.

In Chapter 5, a new LRFD approach is proposed for the limit states of pullout and rupture for steel strip reinforced soil walls. For a target probability of failure of 1% and a prescribed load factor of 1.35, the resistance factor for pullout is 0.35 and the resistance factor for rupture is 0.90. Furthermore, comparison with ASD past practice shows that the new LRFD approach gives similar or slightly higher operational factors of safety and lower probabilities of failure for the two limit states. An important finding for designers is that the new LRFD approach will likely result in about 6% less steel than using ASD past practice or current AASHTO LRFD approach.

Pullout test data analyses in Chapter 6 suggest that good practice for the determination of pullout capacity is to fit a linear approximation to dimensionless interaction coefficients when project-specific pullout test results are available. However, in many cases project-specific pullout data are not available. If the current AASHTO/FHWA model with default values is used, the prediction accuracy is shown to be very poor with mean of bias values = 2.23 and coefficient of variation (COV) = 0.55. In addition, the current model has unacceptable hidden dependency between bias values and predicted pullout capacity. The linear relationship between pullout capacity and normal stress adopted in the current pullout equation was identified as the main source of the observed poor accuracy. Two new models are proposed to overcome this deficiency. One model is bi-linear and the other non-linear. The non-linear model is shown to be more accurate with a mean bias close to one and COV = 0.36. The non-linear model has the advantage of being smoothly continuous with practically no detectable hidden dependencies.

Chapter 7 is focused on the generation of bias statistics for installation damage. Four backfill soil categories based on the D_{50} particle size are proposed. It was found that the bias mean and COV values vary over narrow ranges for geogrid and geotextile products placed in soils with D_{50} particle size less than 19 mm (Type 2, 3 and 4 soils). Hence for LRFD calibration purposes, bias statistics for installation damage can be grouped into two ranges for each geosynthetic type based
on $D_{50}$ of the soil greater than or less than 19 mm. The bias mean values range from 1.05 to 1.15 and COV values typically are less than 0.20 with the exception of light-weight geotextile products installed in coarse granular soils (Type 1). Data in this chapter also demonstrates that the spread in predicted installation damage strength increases with installation damage factor. This chapter highlights the need to match a candidate geosynthetic to the project soil so that the applicable installation damage reduction is kept to a minimum. This will lead to smaller scatter in installation damage bias values and hence a lower probability of failure due to tensile rupture.

In Chapter 8, bias statistics due to creep strength reduction are assessed. The results of analysis showed that variability in the prediction of creep-reduced strength is very low (COV = 0.044) and is largely captured by the magnitude of variance in the original strength of the test specimens (COV = 0.038). From a practical point of view, the difference is negligible and hence it was concluded that current practice for the determination of creep strength reduction does not introduce additional uncertainty to the calculation of available long-term reinforcement strength. This greatly simplifies future LRFD calibration for the geosynthetic rupture limit state since creep strength reduction factors can be taken as deterministic in LRFD calibration. A practical observation in this investigation is that in every available case, creep testing up to 1000 hours would have been sufficient to estimate the 75-year design creep reduction factor value using the log-linear fit to the first 1000 hours with the same accuracy using the regressed model fitted to all the data. However, this observation needs to be examined further using a wider range of data before it can be applied in practice.

Chapter 9 shows that LRFD calibration using the current AASHTO Simplified Method to calculate reinforcement loads in geosynthetic reinforced soil walls under operational conditions is not possible if the objective is to compute sensible load and resistance factor values. This is due to the poor prediction accuracy of the model. Refinements to the Simplified Method are proposed. Reasonable resistance factors are generated using a load factor of 1.35 and a target probability of pullout failure of 1%. The proposed LRFD approach gives recommended resistance factors of 0.50 using the proposed project-specific pullout model (Model 2) and 0.40 using the proposed pullout model with presumptive constant parameter values (Model 5). The operational factors of safety using the new LRFD approach are greater than 1.5 which is the required minimum factor of safety in ASD past practice. Regardless of design approach (ASD or LRFD), analysis results demonstrate that the current empirical minimum reinforcement length criterion (0.7 times wall height and anchorage length of at least 1 m) will likely control design for pullout.
Chapter 10 again demonstrates that the current AASHTO Simplified Method is not a suitable load model for the design of geosynthetic walls under operation conditions within the LRFD framework. This is because almost all of the computed resistance factors are greater than one using a load factor of 1.35 and a target probability of rupture failure of 1%. Assuming a resistance factor of one for these cases, ASD past practice using the current AASHTO Simplified Method is shown to give operational factors of safety between 5 and 6 compared to a specified minimum factor of safety of 1.5. Using the new LRFD approach, the recommended resistance factors are between 0.40 and 0.55. However, a practical outcome for design is that combinations of lightweight geotextile products with aggressive backfill soils (Type 1) where large installation damage strength losses can occur should be avoided.

In Chapter 11 a new LRFD approach is proposed for the limit states of pullout and rupture for geosynthetic reinforced soil walls using the K-Stiffness Method for load calculations. All computed resistance factors are less than one (i.e. sensible) using a load factor of 1.35 and a target probability of failure of 1%. For the limit state of rupture, a single resistance factor of 0.70 is recommended for non-aggressive backfills (Type 2, 3 and 4 soils). If aggressive Type 1 soils are used, recommended resistance factor values are from 0.55 to 0.70 with the exception of lightweight geotextile products. For the limit state of pullout, the proposed LRFD approach should be used with a resistance factor of 0.55 using the project-specific pullout model (Model 2) and resistance factor of 0.40 using the general pullout model (Model 5) with presumptive constant parameter values. Chapter 11 also proposes an alternative LRFD approach based on the bias statistics of the maximum reinforcement loads in a wall. This alternative approach simplifies design and uses larger resistance factors but may lead to structures with greater global stiffness ($S_g$) (more costly structures).

### 13.3 Recommendations for future work

- In the numerical wall models created for the current study, reinforcement (cable) elements were assumed to be bonded to backfill using the FLAC grout utility. Large bond strength along the reinforcement-backfill interface was selected to prevent slip and to simplify the model. Measured reinforcement displacements show that this is a reasonable assumption under working load conditions for the combination of reinforcement products and compacted sand used in the RMC physical tests. However, it is understood that this assumption may not be true when the wall is subjected to large surcharge loads and finer backfill soils are used. In order to simulate the soil-reinforcement interaction more
realistically, advanced soil-reinforcement interface models that allow for relative slip may be needed. Pullout tests should be performed to back-calculate key parameters for interface constitutive models. A novel pullout test apparatus using transparent artificial soils and particle image velocimetry (PIV) technique has been developed at RMC. These technologies hold promise to provide mechanical understanding and the data necessary to develop advanced geosynthetic-soil interface models.

• In the numerical studies in this thesis, all walls were assumed to be constructed on a rigid foundation and hence are expected to represent the behaviour of field walls that are typically built on competent foundations. However, it is valuable to investigate the influence of a deformable foundation on wall response. For example, it may be expected that additional settlement of the backfill relative to the facing may create larger facing-reinforcement connection loads. A series of small-scale model walls with independent controlled compressibility below the facing column and backfill soil have been tested at RMC. The results of these tests hold promise to investigate the influence of foundation and vertical toe support compressibility on reinforcement loads and to adjust the K-Stiffness Method if required. The numerical model developed as part of the current research program could be verified against these test results and the database of numerical wall simulations extended to the case of non-rigid foundation conditions.

• Prior to this thesis, there have been a total of 14 (including the three wall tests completed most recently) full-scale reinforced soil walls tested using the RMC indoor Retaining Wall Testing Facility. The most recent three walls were constructed with “non-select” sandy-silt backfill soil. However, further investigation of the effects of soil suction, using soil suction measurement and further refinement of the soil-water characteristic curve should be considered. Triaxial testing is also required to characterize the properties of the sandy-silt backfill under both saturated and unsaturated conditions. The data collected from these walls provide the possibility to extend the numerical model used in the current study to simulate walls with “non-select” backfill soils. The reinforcement loads back-calculated from measured strain values in these walls can be added to the database used to calibrate the K-Stiffness Method for cohesive-frictional backfills.

• The numerical model developed in this thesis can be further extended to incorporate spatial variability of soil properties and three-dimensional effects. Redistribution of reinforcement loads due to failure of one or more reinforcement layers is a useful line of
investigation to examine the hypothesis that these structures are highly load capacity redundant systems.

- The LRFD approach proposed in this thesis for the tensile rupture limit state of the reinforcement in geosynthetic walls requires project-specific installation damage test results to determine the value of RF_{ID}. An empirical equation that can be used to estimate RF_{ID} based on backfill gradation parameters such as D_{10}, D_{50}, D_{90} and coefficient of curvature C_u and/or other variables would be useful to properly match candidate geosynthetic reinforcing materials with project soils. The installation damage database compiled as part of the research work is a valuable first step in this direction. Once a general RF_{ID} equation for combinations of geosynthetic type soil properties is developed, the LRFD approach proposed in this thesis can be updated accordingly.

- The durability of reinforcement materials (corrosion of steel and chemical degradation of geosynthetic) has been treated as deterministic in this thesis. Uncertainty associated with material durability needs to be taken into account. There are two major obstacles at this stage. First, current design guidance documents do not explicitly specify the methodology to calculate the loss of capacity associated with durability in design. Second, there is lack of suitable data for performing statistical analysis on durability.

- Locally high reinforcement loads almost always develop at the connection between the reinforcement and the facing for walls constructed with a structural facing (e.g. modular block or concrete panel walls). This is a complicated down-drag mechanism involving response to settlement during compaction and settlement that occurs as the wall face moves outward and (or) rotates about the toe. These down-drag forces can generate additional tensile loads in reinforcement layers particularly if the structural facing and footing are vertically stiffer than the backfill soil (which is the usual case in the field for walls with a structural facing). However, in current design codes, there is no explicit design equation that can be used to estimate connection loads. A calculation method should be developed and statistically verified using measured connection load data in future work. Concurrently, a database of laboratory connection test results can be used to generate resistance-side bias statistics. Once the load and resistance bias statistics are generated, LRFD calibration for the limit state of connection failure can be carried out.
• For modular block reinforced soil walls, block-block interface shear failure must be considered in design. However, current design codes do not provide an explicit design method for this limit state. After a deterministic model is developed to calculate connection loads between reinforcement layers and facing column, a design method can be developed to predict interface shear forces in modular block walls. Statistical data for shear resistance can be obtained based on a large volume of block-block interface shear test results that is available to researchers at the GeoEngineering Centre at Queen’s-RMC. An LRFD-based approach can then be developed for the limit state of interface shear failure for modular block walls.

• In this thesis only one load term (reinforcement loads due to the self-weight of backfill plus uniform permanent surcharge applied at the top of the wall) is considered for LRFD calibration. Recently, reinforced soil wall structures have been proposed to carry vertical loads at the bridge abutments (rather than using pile support). The load model used in LRFD calibration should be updated to the combined effects of vertical and horizontal loads that will develop for this application.

• The LRFD model for load investigated in this thesis work should be developed for multiple dead load sources, live loads (e.g. due to traffic) and other extreme events.

• The LRFD approach should be extended to external stability design (i.e. bearing capacity, overturning, base sliding and overall stability) of reinforced soil walls. A consistent reliability level (i.e. probability of failure) between internal and external stability should be an objective of this work.

• Generally good agreement between numerical results and measured wall response shows that the verified numerical code developed as part of this thesis work is a powerful tool to carry out parametric studies and to fill in the database gaps from full-scale physical instrumented laboratory and field walls. An initial example of how this data can be used for this purpose has been demonstrated in a paper by Nernheim et al. (2008).

REFERENCE

Appendix A - Comparison of current AASHTO LRFD approaches for reinforced soil retaining walls to ASD past practice

Table A.1 Summary of load and resistance factor values in current LRFD code and the values of factor of safety in ASD past design codes.

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<th>ASD</th>
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<td>$\gamma_Q$</td>
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<tr>
<td>Yield, steel strips</td>
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<td>0.75</td>
</tr>
<tr>
<td>Pullout, steel grids</td>
<td>1.35</td>
<td>0.90</td>
</tr>
<tr>
<td>Yield, steel grids</td>
<td>1.35</td>
<td>0.65</td>
</tr>
<tr>
<td>Pullout, geosynthetics</td>
<td>1.35</td>
<td>0.90</td>
</tr>
<tr>
<td>Rupture, geosynthetics</td>
<td>1.35</td>
<td>0.90</td>
</tr>
</tbody>
</table>

Notes:
* $\gamma_Q / \phi = \text{or} \approx \text{FS}$

$\gamma_Q$ denotes the prescribed load factor; $\phi$ denotes the recommended resistance factor in current AASHTO design codes; and FS denotes the recommended factor of safety in ASD past design codes.
Appendix B - Derivation of Equation 4.2

A limit state function with one resistance term and one load term, which is the case considered in this thesis, can be expressed as:

\[ A1 \quad g = R_m - Q_m \geq 0 \]

Where: \( g \) = limit state function; \( R_m \) = measured resistance; and \( Q_m \) = measured load.

In this thesis, bias is defined as the ratio of measured to calculated value. Hence, the measured value can be expressed as the product of bias times calculated value, specifically:

\[ A2 \quad R_m = X_R R_n \]
\[ A3 \quad Q_m = X_Q Q_n \]

Where: \( X_R \) = resistance bias; \( X_Q \) = load bias; \( R_n \) = calculated nominal resistance; and \( Q_n \) = calculated nominal load.

From Equations A1, A2 and A3, the limit state function can be expressed as:

\[ A4 \quad X_R R_n - X_Q Q_n \geq 0 \]

For the case of one resistance term and one load term, the LRFD design equation in North America can be expressed as:

\[ A5 \quad \varphi R_n - \gamma_Q Q_n = 0 \]

Where: \( \gamma_Q \) = load factor; and \( \varphi \) = resistance factor. Equation A5 can be rewritten as:
\[ R_n = \frac{\gamma Q_n}{Q} \]

Substituting Equation A6 into Equation A4 yields:

\[ X_R \frac{\gamma Q_n}{Q} - X_Q Q_n \geq 0 \]

Equation A7 can be further rearranged as:

\[ \gamma Q X_R \geq \varphi X_Q \quad \text{(Equation 4.2)} \]
Appendix C - Recommended resistance factor values for the limit states of pullout and rupture for reinforced soil retaining walls in this thesis

Table C.1 Summary of recommended resistance factor values/ranges for $P_f = 0.01$ ($\beta = 2.33$) and a prescribed load factor of 1.35.

<table>
<thead>
<tr>
<th>Reinforcement type</th>
<th>Limit states</th>
<th>Load calculation model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pullout</td>
<td>Rupture</td>
</tr>
<tr>
<td>Steel grid</td>
<td>0.40$^{(1)}$</td>
<td>0.65</td>
</tr>
<tr>
<td>Steel strip</td>
<td>0.35$^{(1)}$</td>
<td>0.90</td>
</tr>
<tr>
<td>Geosynthetic</td>
<td>0.30 to 0.50$^{(1)(2)}$</td>
<td>0.30 to 0.50</td>
</tr>
<tr>
<td>Geosynthetic</td>
<td>0.40 to 0.55$^{(1)(2)}$</td>
<td>0.55 to 0.85</td>
</tr>
</tbody>
</table>

Notes:
(1) Requires modified or new pullout models
(2) Corresponds to different pullout models
Appendix D - List of publications by Bing Q. Huang


