Geotechnical Engineering Research Laboratory
Dept. of Civil and Environmental Engineering
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NCHRP 24-31 LRFD Design Specifications for Shallow Foundations

TRB AFS30 Committee Meeting January 26, 2011

The lecture is based on

NCHRP Report 651

LRFD DESIGN AND CONSTRUCTION OF SHALLOW FOUNDATIONS FOR HIGHWAY STRUCTURES

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OBJECTIVES
NCHRP RESEARCH PROJECT 24-31

Develop and Calibrate Procedures and Modify AASHTO's Section 10 (Foundations) Specifications for the Strength Limit State Design of Bridge Shallow Foundations.

For NCHRP Research Report 651, Google NCHRP 651

Task 1: Design & Construction Practices

- **Questionnaire - Establishing design methods, construction practices, design cases, and loads**
- **Developed and distributed to 161 State Highway Officials, TRB Representatives, and State and FHWA bridge engineers.**
- **Obtained responses from 39 states and 1 Canadian Province**
- **Previous relevant information was obtained via a questionnaire circulated in 2004 for the research project *NCHRP 12-66 AASHTO LRFD Specifications for the Serviceability in the Design of Bridge Foundation***

Design & Construction Practices - Questionnaire

Foundation Alternatives

Results on distribution of bridge foundation usage from our previous questionnaires conducted in 1999 and 2004, and the current questionnaire (over the past 3 years, 2004-2006):

	shallow foundations	*driven piles*	*drilled foundations*
1999/2004	*14%/17%*	*75%/62%*	*11%/21%*
current	*17%*	*59%*	*24%*

- **The average use changes significantly across the country. The use of shallow foundations in the Northeast exceeds by far all other regions of the USA, ranging from 40% in NY, NJ and ME, to 67% in CT. Other "heavy users" are TN (63%), WA (30%), NV (25%) and ID (20%). In contrast, out of the 39 responding states, six states do not use shallow foundations for bridges at all, and additional eight states use shallow foundations in 5% or less of the highway bridge foundations.**
- **In summary, 55.8% of the shallow foundations are built on rock (average of piers and abutments) with additional 16.8% on IGM, hence 72.6% of the foundations are build on rock or cemented soils and only 27.4% are built on soils of which 24.2% on granular soils and 3.2% on clay or silt.**

TASK 2: DATABASES

- **UML-GTR ShalFound07 Database**

  549 cases built in ACCESS platform, 415 cases are suitable for ULS.

- **UML-GTR RockFound07 Capacity Database**

  122 Cases of load tests to failure including 61 rock sockets, 33 shallow foundations on rock surface, 28 shallow foundations below surface

- **ShalFound07**

  Divided into vertical centric and eccentric, and inclined cases

- **RockFound07**

  All vertical centric, shallow and drilled shafts

Database – Overview

UML-GTR ShalFound07

Foundation type	Predominant Soil Type					Total	Country	
	Sand	Gravel	Cohesive	Mix	Others		Germany	Others
Plate load tests B ≤1m	346	46	--	2	72	**466**	253	213
Small footings 1 < B ≤ 3m	26	2	--	4	1	**33**	--	33
Large footings 3 < B ≤ 6m	30	--	--	1	--	**31**	--	31
Rafts & Mats B > 6m	13	--	--	5	1	**19**	1	18
Total	**415**	**48**	**0**	**12**	**74**	**549**	**254**	**295**

Note:

"Mixed" are cases with alternating layers of sand or gravel and clay or silt

"Others" are cases with either unknown soil types or with other granular materials like loamy Scoria

1m ≈ 3.3ft

Large foundations are often not loaded to ULS failure (SLS controls)

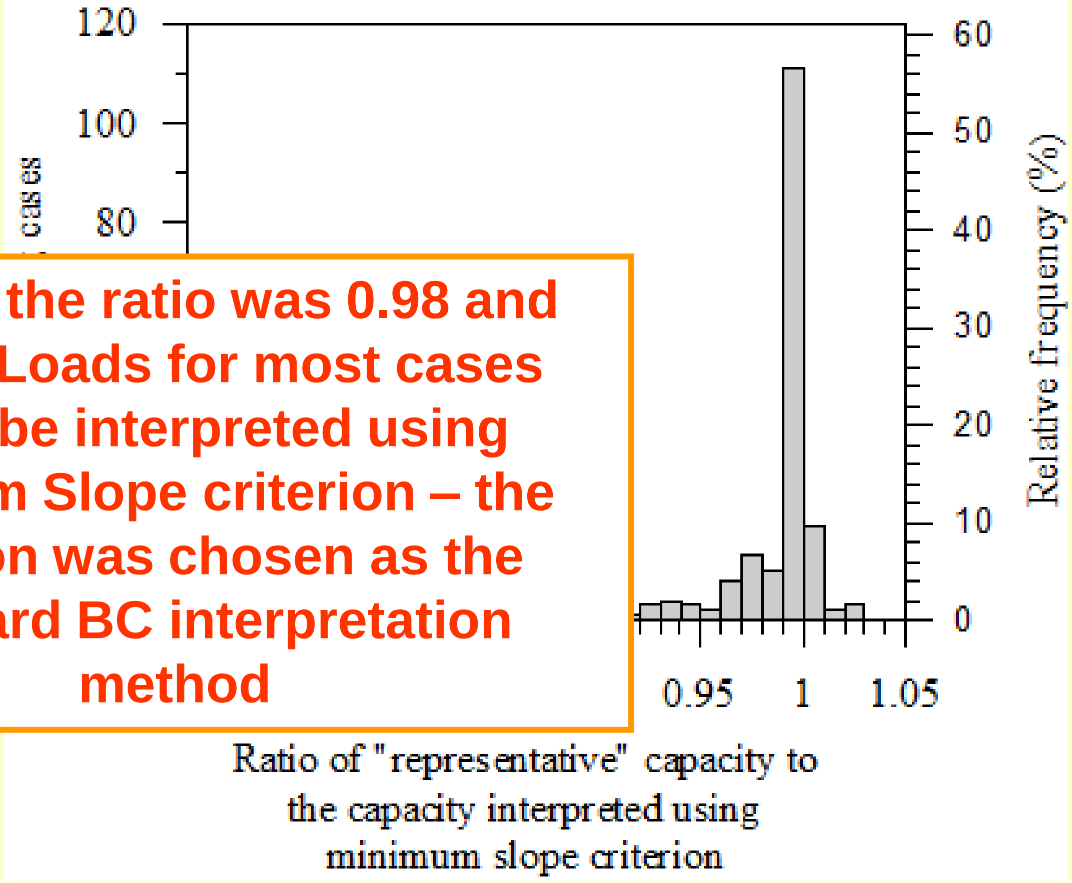
TASK 3: BC Shallow Foundations on Soil - OUTLINE

1. **BC of Shallow foundations**
   - BC Factors
   - BC modification Factors
2. **Determination of ULS from case histories**
   - ULS and Modes of Failure – Overview
   - Modes of Failure
3. **Failure (Ultimate Load) Criteria**
   - Minimum slope criteria (Vesić, 1963)
   - Limited settlement criterion of 0.1B (Vesić, 1975)
   - Log-log plot of load-settlement curve (DeBeer, 1967)
   - Two-slope criterion
   - Selection of failure criteria (representative values and minimum slope)
   - Examples in soil & rock
4. **Uncertainty Evaluation – BC of Centric Vertically Loaded Footing on Granular Soils**
   - Database overview
   - Calculated BC – missing soil parameters and equations used for BC calculations
5. **Calibration**
6. **Summary and Conclusions**

Recommended Failure Criterion

Minimum Slope Failure Load Criterion, Vesic (1963)

- **Failure load interpreted were for 196 cases using each of the proposed methods**
- **“Representative Failure Load” defined as the mean value of all the failure loads interpreted using each criterion**



Mean of the ratio was 0.98 and Failure Loads for most cases could be interpreted using Minimum Slope criterion – the criterion was chosen as the standard BC interpretation method

Bias of Estimated BC

Cases with Vertical Centric Loading

Vertical Centric Loading
n = 173; mean bias = 1.59, COV = 0.291

- Natural soil conditions (ϕ_f from SPT-N counts)
  n = 14; no. of sites = 8
  mean = 1.00
  COV = 0.329
  - *B* > 1.0m
    n = 6
    no. of sites = 3
    mean = 1.01
    COV = 0.228
  - 0.1 < *B* ≤ 1.0m
    n = 8
    no. of sites = 7
    mean = 0.99
    COV = 0.407
- Controlled soil conditions ($D_r \geq 35\%$)
  n = 159; no. of sites = 7
  mean = 1.64
  COV = 0.267
  - *B* ≤ 0.1m
    n = 138
    no. of sites = 5
    mean = 1.67
    COV = 0.245
  - 0.1 < *B* ≤ 1.0m
    n = 21
    no. of sites = 3
    mean = 1.48
    COV = 0.391

Figure 60 Summary of bias (measured over calculated BC) for vertical centric loading cases (Database I); 0.1m = 3.94in; 1m = 3.28ft.

Bias of Estimated BC

Cases with Vertical Centric Loading

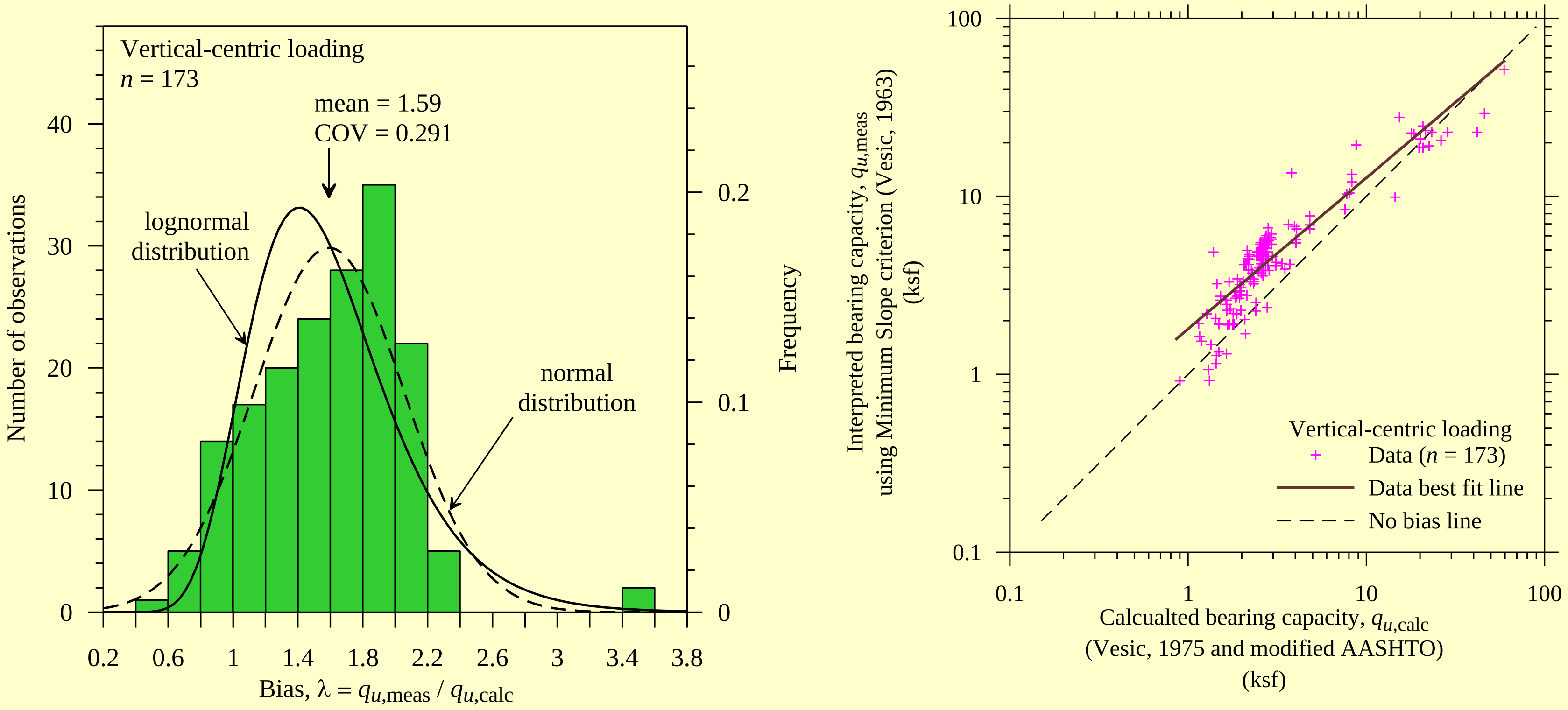


Figure 61. (a) Histogram and probability density functions of the bias and (b) relationship between measured and calculated bearing capacity for all cases of vertical centrically loaded shallow foundations.

Bias of Estimated BC

Cases with Vertical Centric Loading

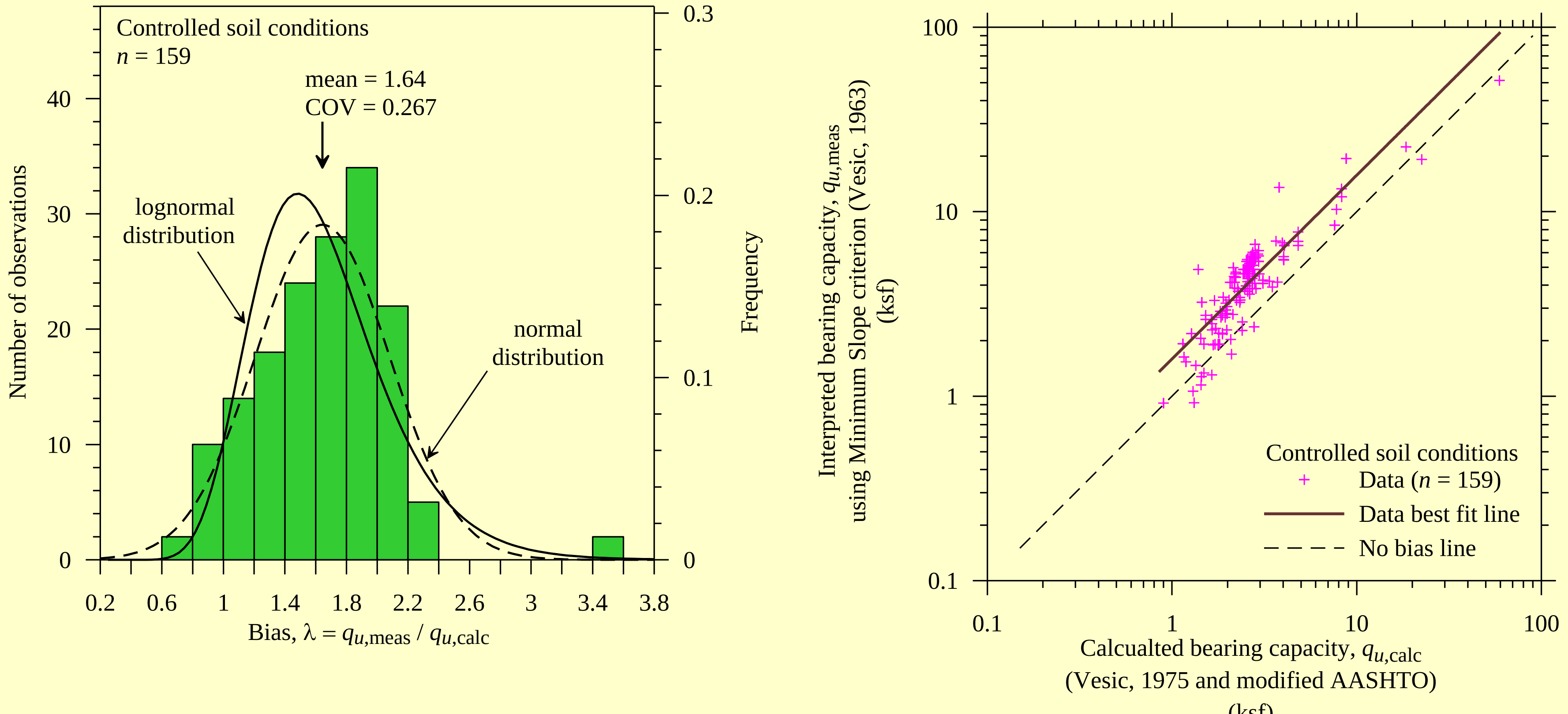


Figure 62. (a) Histogram and probability density functions of the bias and (b) relationship between measured and calculated bearing capacity for vertical centrically loaded shallow foundations on controlled soil conditions.

Bias of Estimated BC

Cases with Vertical Centric Loading

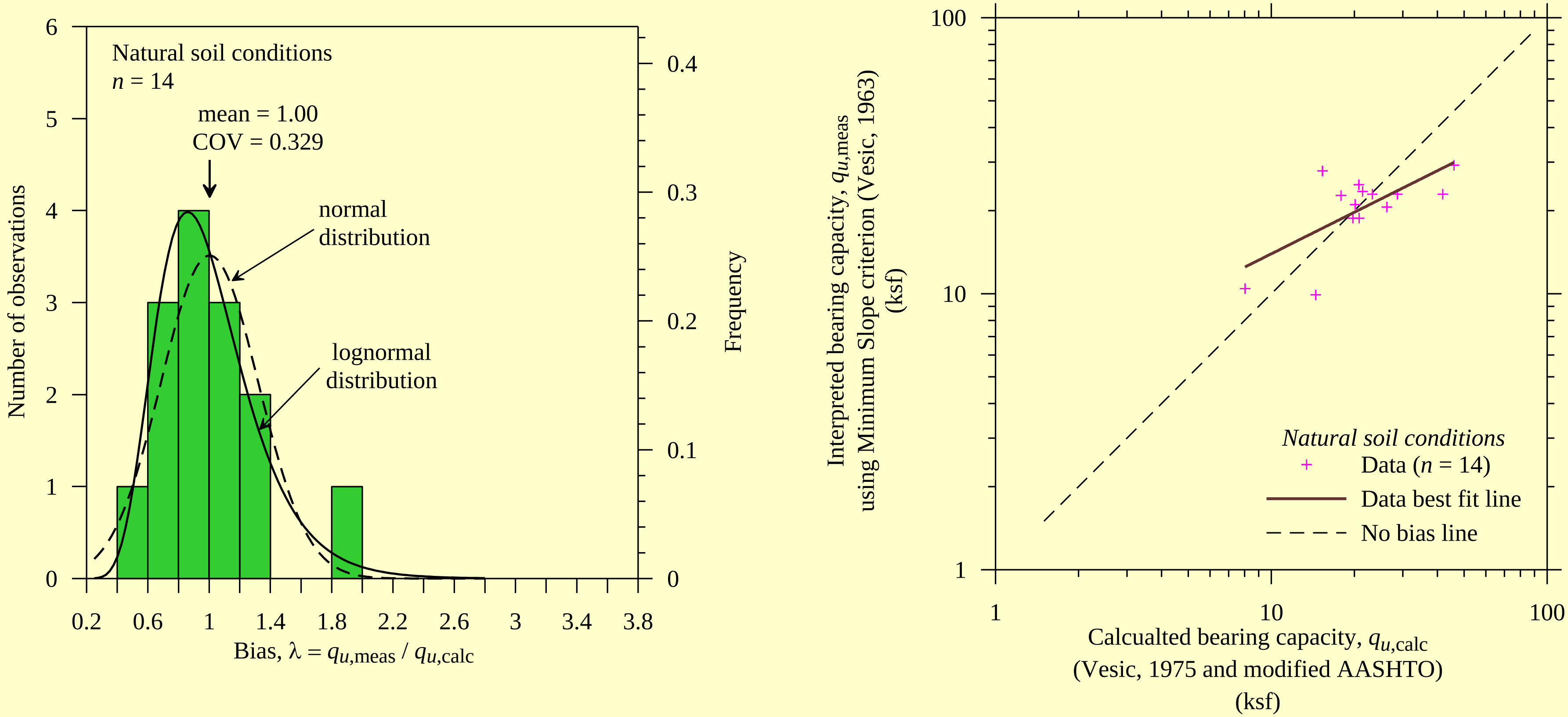


Figure 63. (a) Histogram and probability density functions of the bias and (b) relationship between measured and calculated bearing capacity for vertical centrically loaded shallow foundations on natural soil conditions.

Bias versus Footing Width

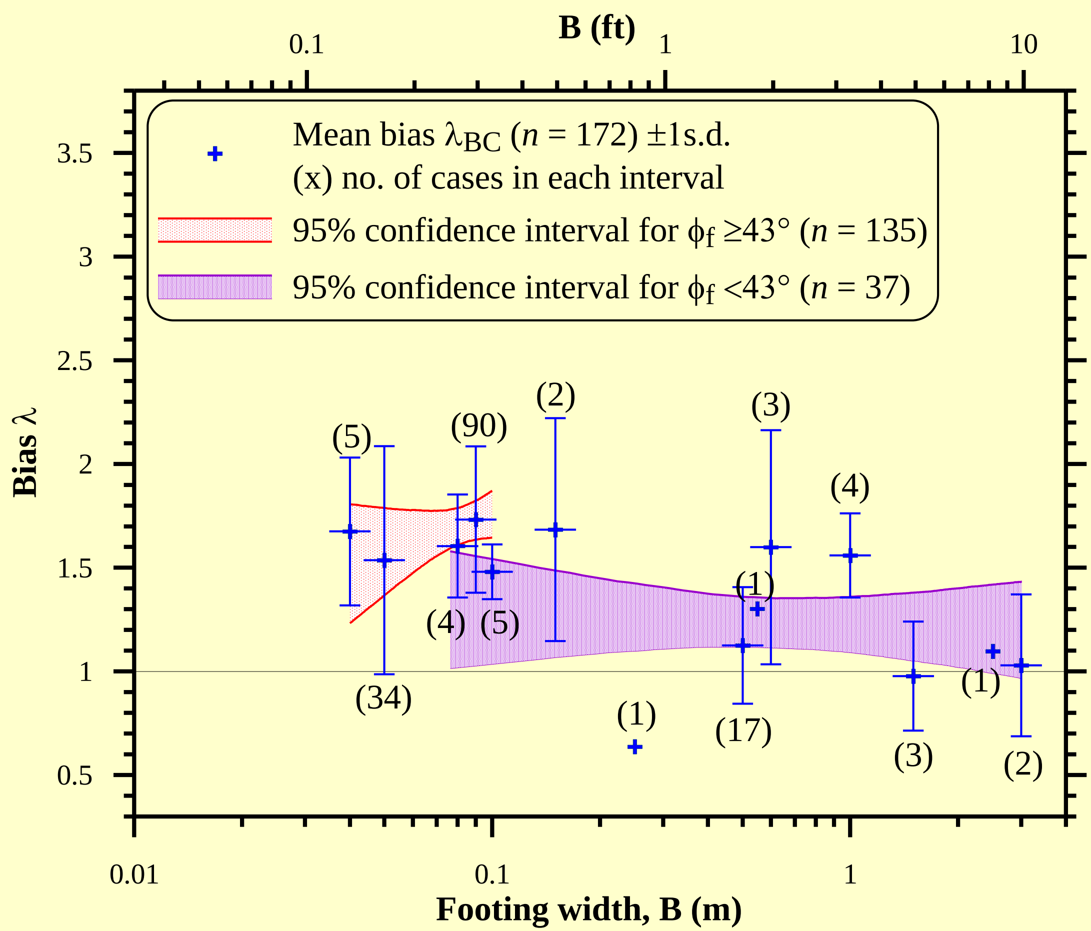
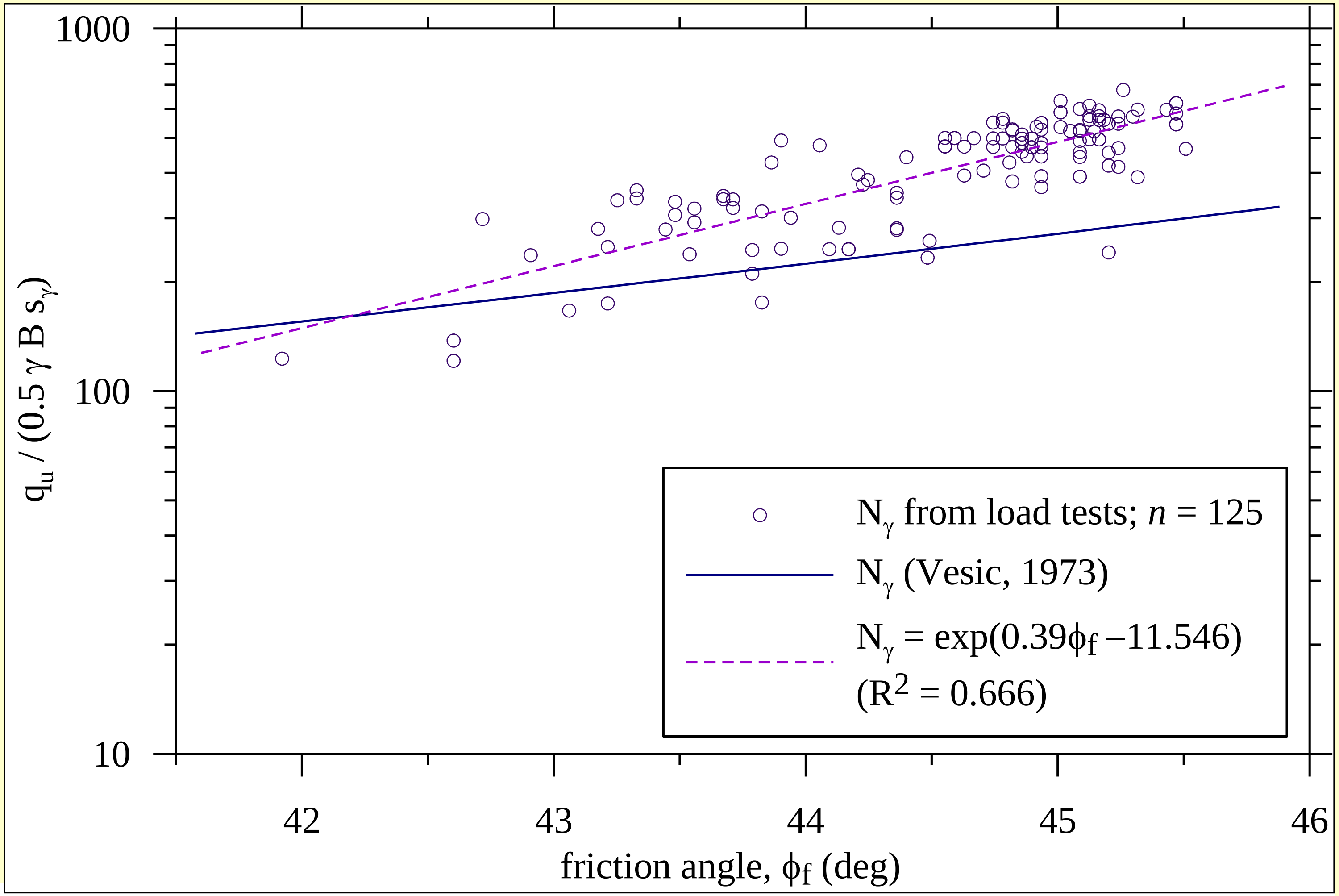


Figure 100. Variation of the bias in bearing resistance versus footing size for cases under vertical-centric loadings: $\phi_f \geq 43°$ and $\phi_f < 43°$.

Uncertainty in N_γ



Comparison of bearing capacity factor calculated based on test results; $N_\gamma = q_u / (0.5\gamma\ B\ s_\gamma)$ from 125 tests carried out in controlled soil conditions (tests by Perau, 1995) and N_γ proposed by Vesic (1973) in the range of soil friction angle of 42° and 46°

Uncertainty in N_γ

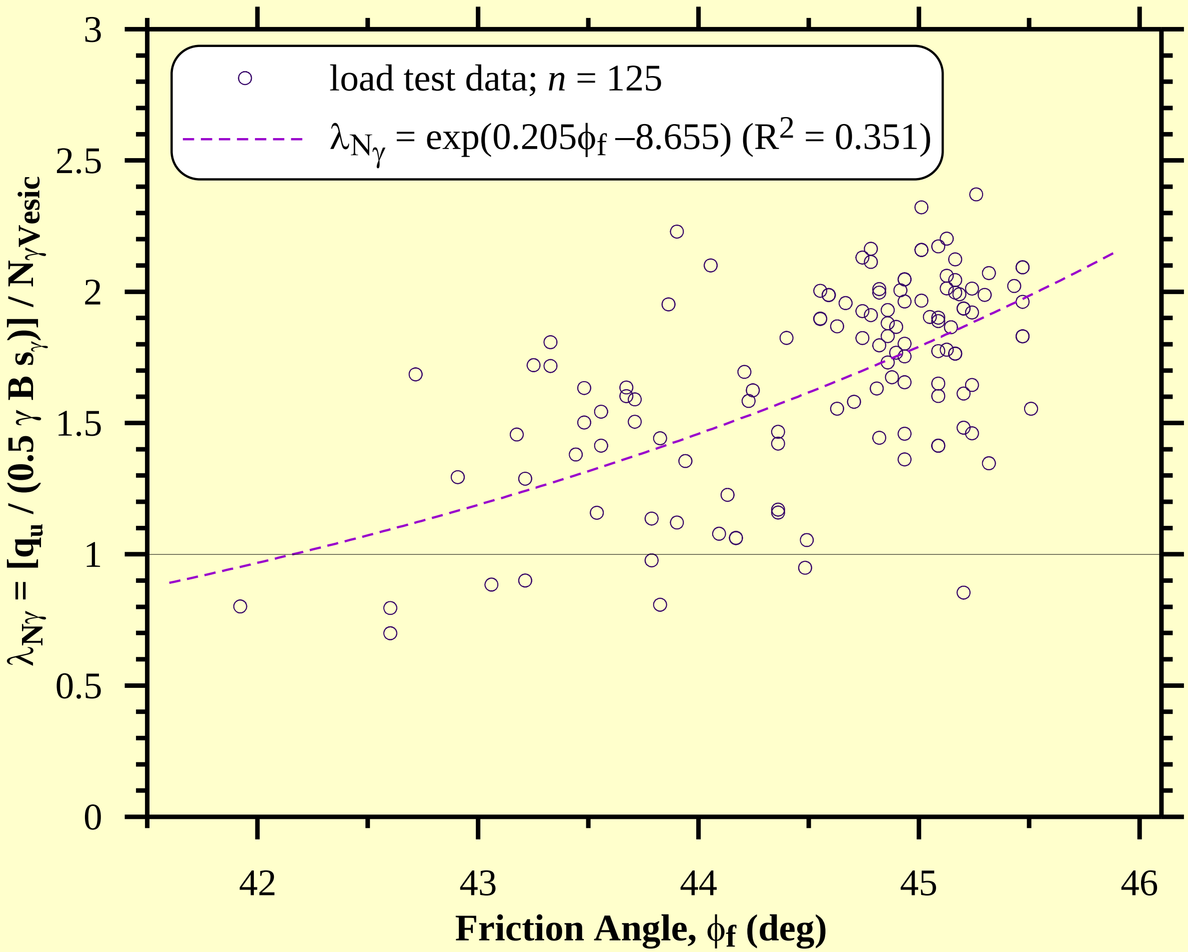


Figure 93. The ratio ($\lambda_{N\gamma}$) between the back-calculated B.C. factor N_γ based on experimental data to that proposed by Vesić versus soil friction angle.

Uncertainty in N_γ

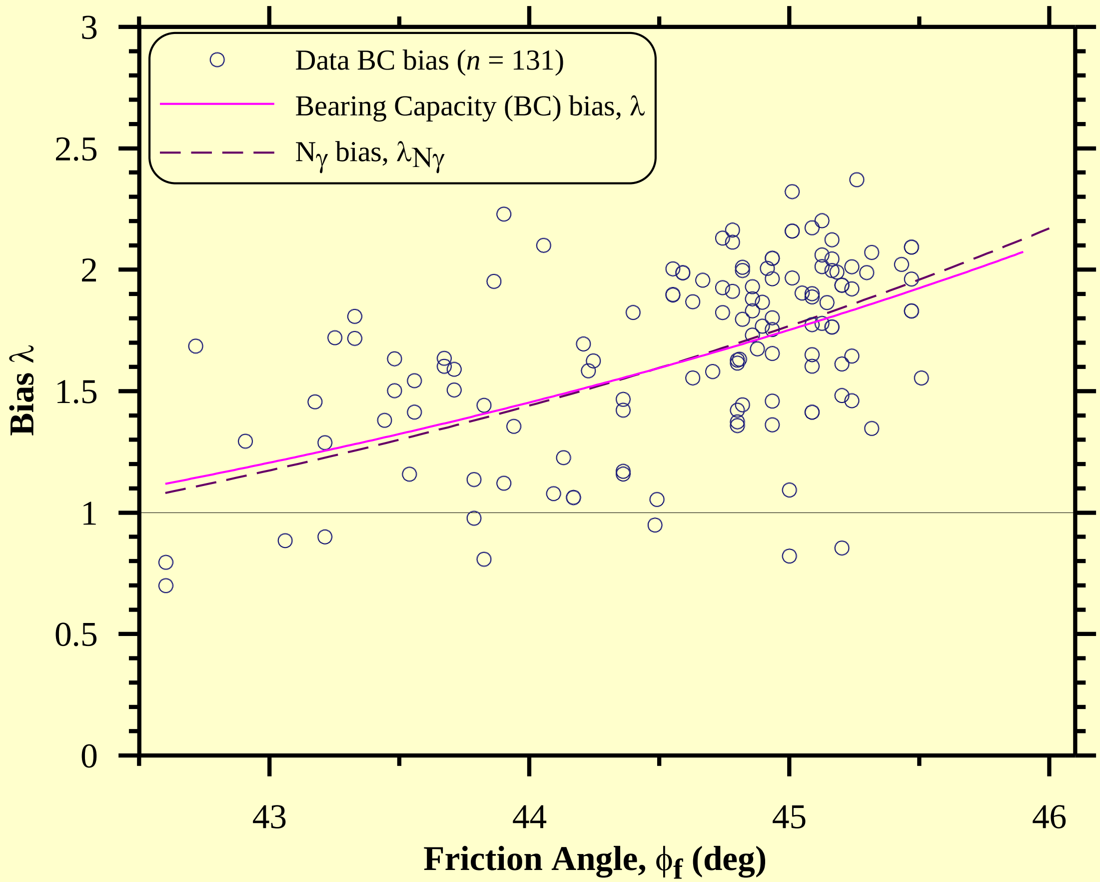


Figure 94. The ratio between measured and calculated bearing capacity (bias λ) compared to the bias in the B.C. factor N_γ ($\lambda_{N\gamma}$) versus the soil's friction angle for footings under vertical-centric loadings.

Uncertainty in B.C.

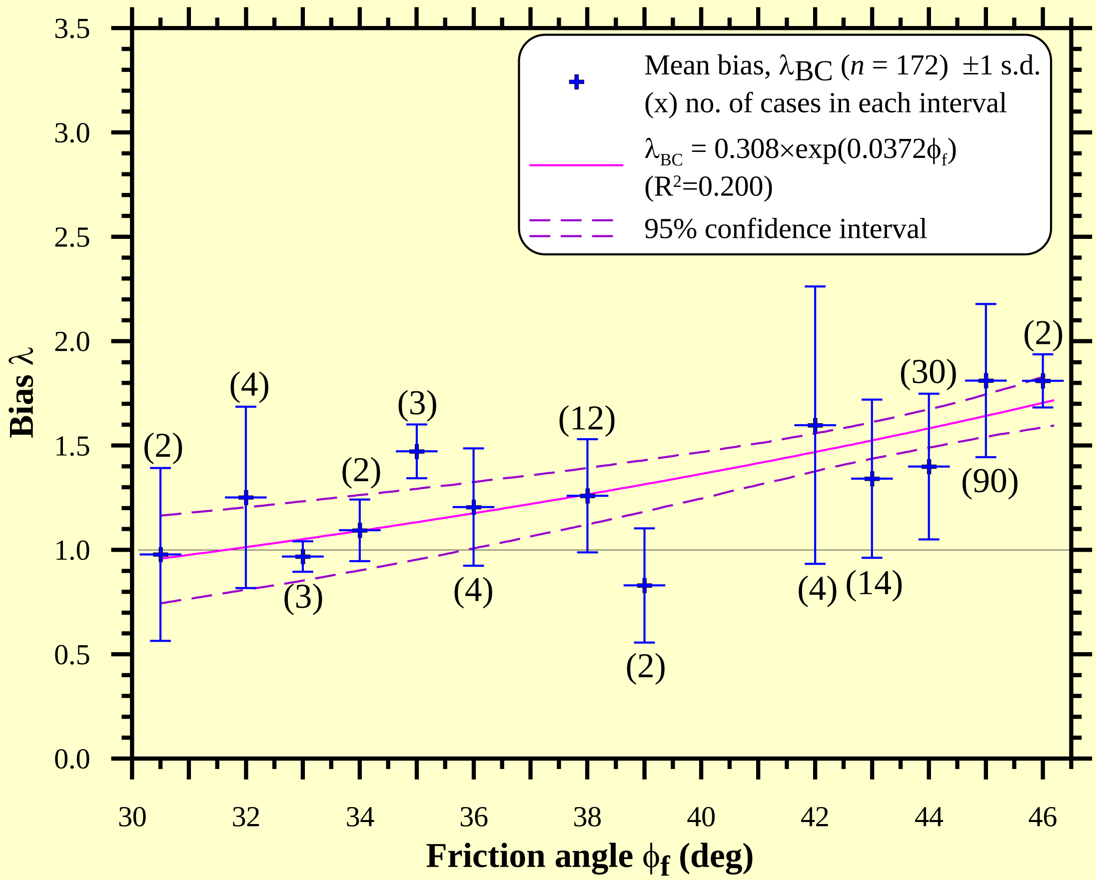


Figure 103. Bearing resistance bias vs. average soil friction angle (taken ϕ_f ±0.5°) including 95% confidence interval for all cases under vertical-centric loading.

Final Resistance Factors – Controlled Conditions

Table 66 Recommended resistance factors for shallow foundations on granular soils placed under controlled conditions

Soil friction angle ϕ_f	Loading conditions			
	Vertical-centric or -eccentric	Inclined-centric	Inclined-eccentric	
			Positive	Negative
$30^\circ - 34^\circ$	0.50	0.40	0.40	0.70
$35^\circ - 36^\circ$	0.60			
$37^\circ - 39^\circ$	0.70	0.45	0.45	0.75
$40^\circ - 44^\circ$	0.75	0.50	0.50	0.80
$\geq 45^\circ$	0.80	0.55		

Notes:

1) ϕ_f determined by laboratory testing
2) compacted controlled fill or improved ground are assumed to extend below the base of the footing to a distance to at least two (2.0) times the width of the foundation (B). If the fill is less than 2B thick, but overlays a material equal or better in strength than the fill itself, then the recommendation stands. If not, then the strength of the weaker material within a distance of 2B below the footing; prevails.
3) The resistance factors were evaluated for a target reliability $\beta_T = 3.0$.

Final Resistance Factors – Natural Conditions

Table 67 Recommended resistance factors for shallow foundations on <u>natural deposited</u> granular soil conditions

<table>
<tr><th rowspan="3">Soil friction angle ϕ_f</th><th colspan="4">Loading conditions</th></tr>
<tr><th rowspan="2">Vertical-centric or -eccentric</th><th rowspan="2">Inclined-centric</th><th colspan="2">Inclined-eccentric</th></tr>
<tr><th>Positive</th><th>Negative</th></tr>
<tr><td>$30^\circ - 34^\circ$</td><td>0.40</td><td rowspan="3">0.40</td><td rowspan="2">0.35</td><td>0.65</td></tr>
<tr><td>$35^\circ - 36^\circ$</td><td>0.45</td><td rowspan="2">0.70</td></tr>
<tr><td>$37^\circ - 39^\circ$</td><td>0.50</td><td rowspan="2">0.40</td></tr>
<tr><td>$40^\circ - 44^\circ$</td><td>0.55</td><td>0.45</td><td rowspan="2">0.75</td></tr>
<tr><td>$\geq 45^\circ$</td><td>0.65</td><td>0.50</td><td>0.45</td></tr>
</table>

Notes:

1) ϕ_f determined from Standard Penetration Test results
2) granular material is assumed to extend below the base of the footing at least two (2.0) times the width of the foundation.
3) The resistance factors were evaluated for a target reliability $\beta_T = 3.0$

Intermediate Conclusions and Summary

- **It was found that for the footings of larger sizes (B>3m (9.9ft)), the load tests were not carried out to the failure load**
- **Biases for the tests in Natural Soil Condition and Controlled Soil Conditions were analyzed separately**
- **For the footing sizes in similar ranges (0.1m < B ≤ 1.0m), the scatter of bias was larger for footings on/in natural soil conditions**
- **The majority of the relevant data refers to small size foundations (B ≤ 3.3ft (1.0m)) on controlled compacted material. Many of the highway shallow foundations on soils are built on compacted materials and hence, the statistical data of the uncertainty can be used for that purpose**
- **There appears to be a trend of increase in bias with the footing size within the range of footing sizes available for testing (which seems to conform with the observation made by Vesic (1969))**

ULS of Inclined Loading

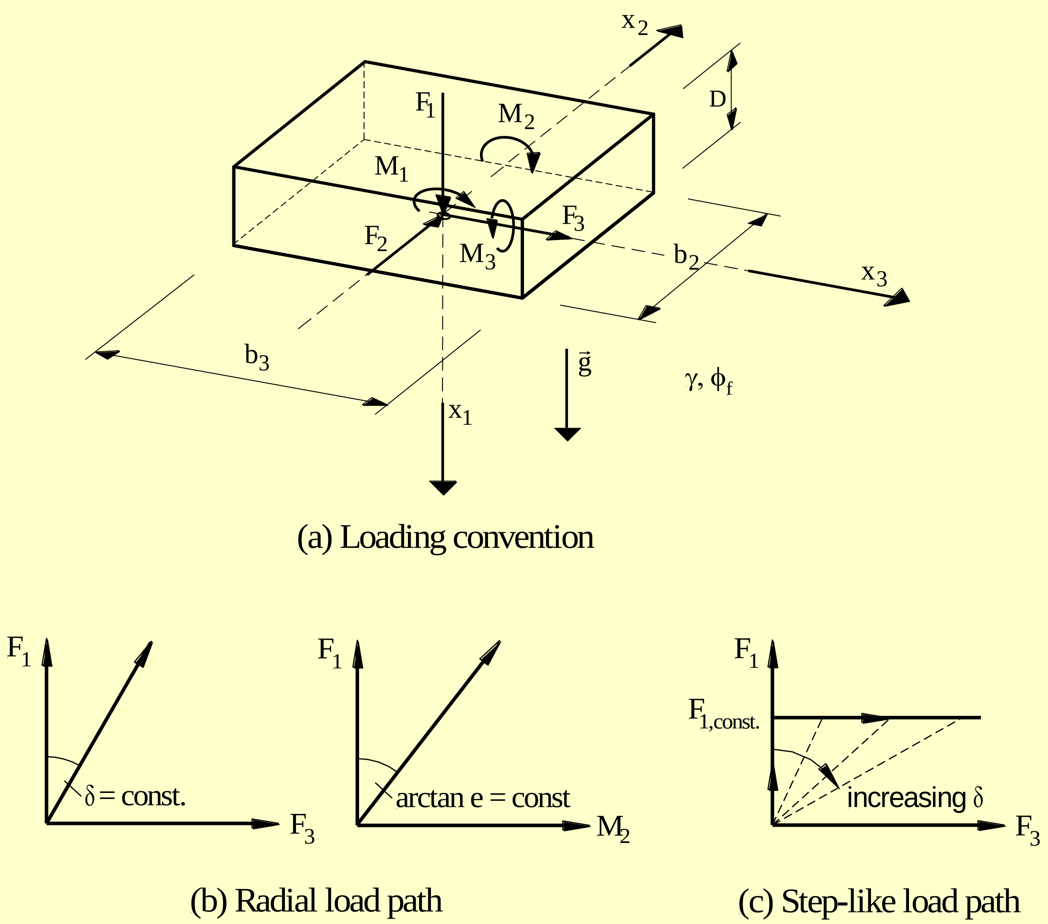


Figure 64. Loading convention and load paths used during tests.

ULS of Inclined Loading

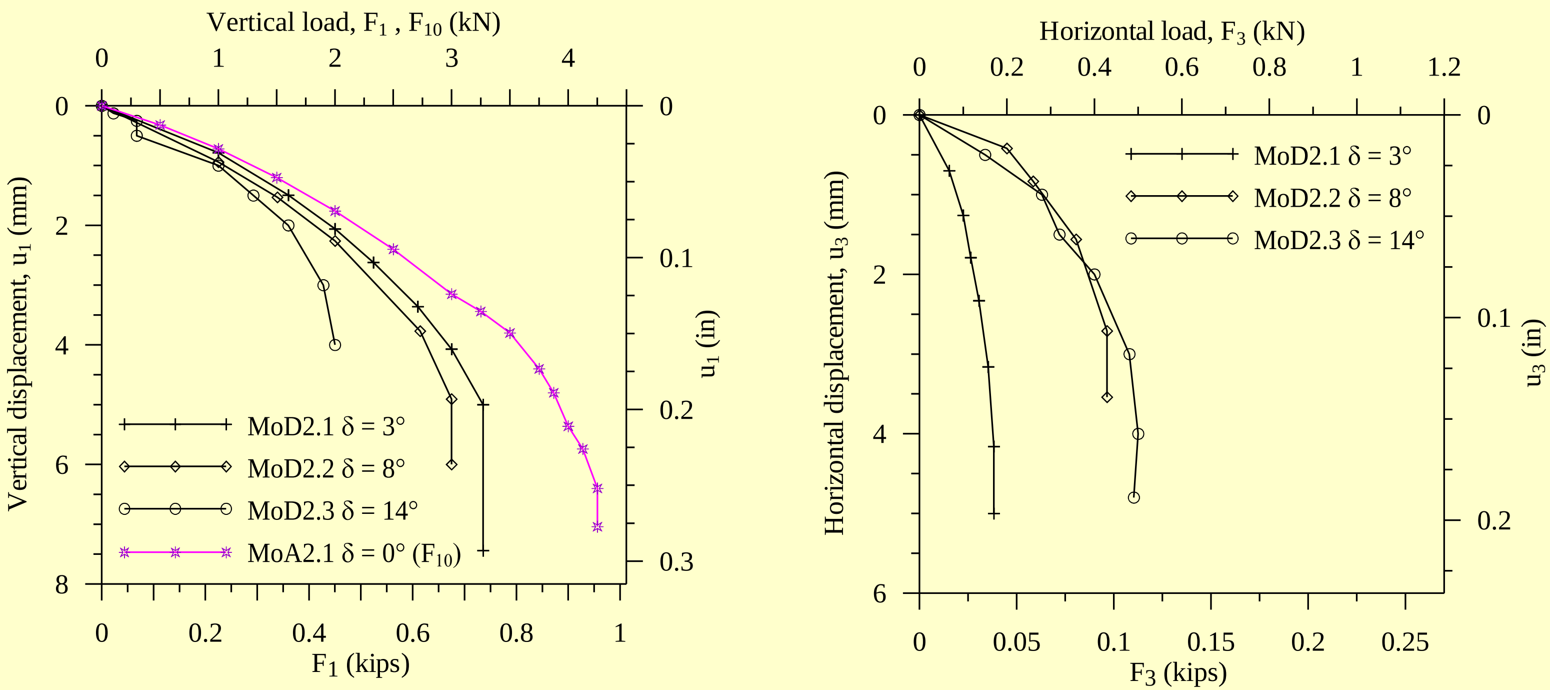


Figure 65. Load–displacements curves for model tests conducted by Montrasio (1994) with varying load inclination: (a) vertical load vs. vertical displacement and (b) horizontal load vs. horizontal displacement.

Bias of Estimated BC

Cases with Vertical-Eccentric Loading (using B′)

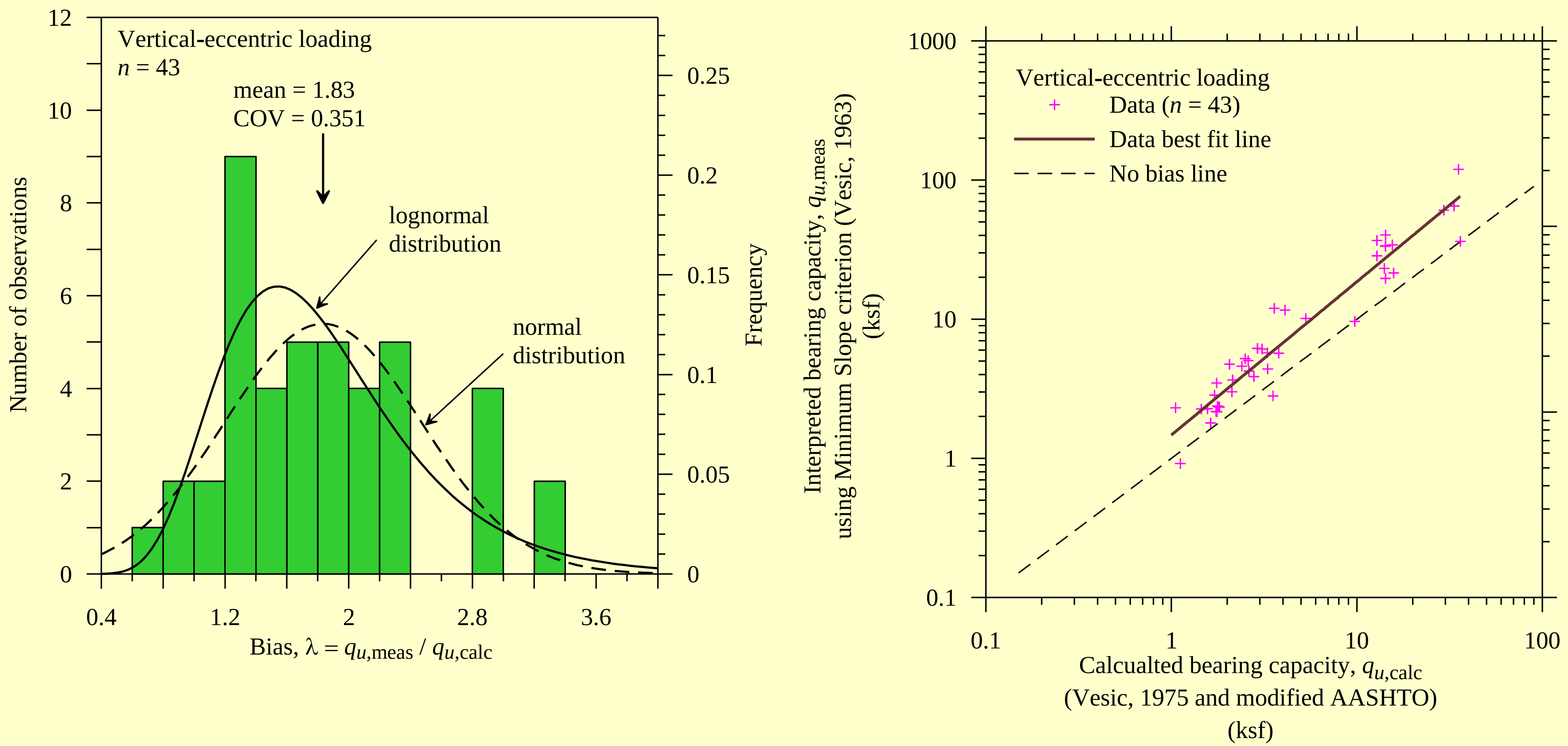


Figure 66. (a) Histogram and probability density functions of the bias and (b) relationship between measured and calculated bearing capacity for all cases of vertical eccentrically loaded shallow foundations.

Bias of Estimated BC

Cases with Vertical-Eccentric Loading

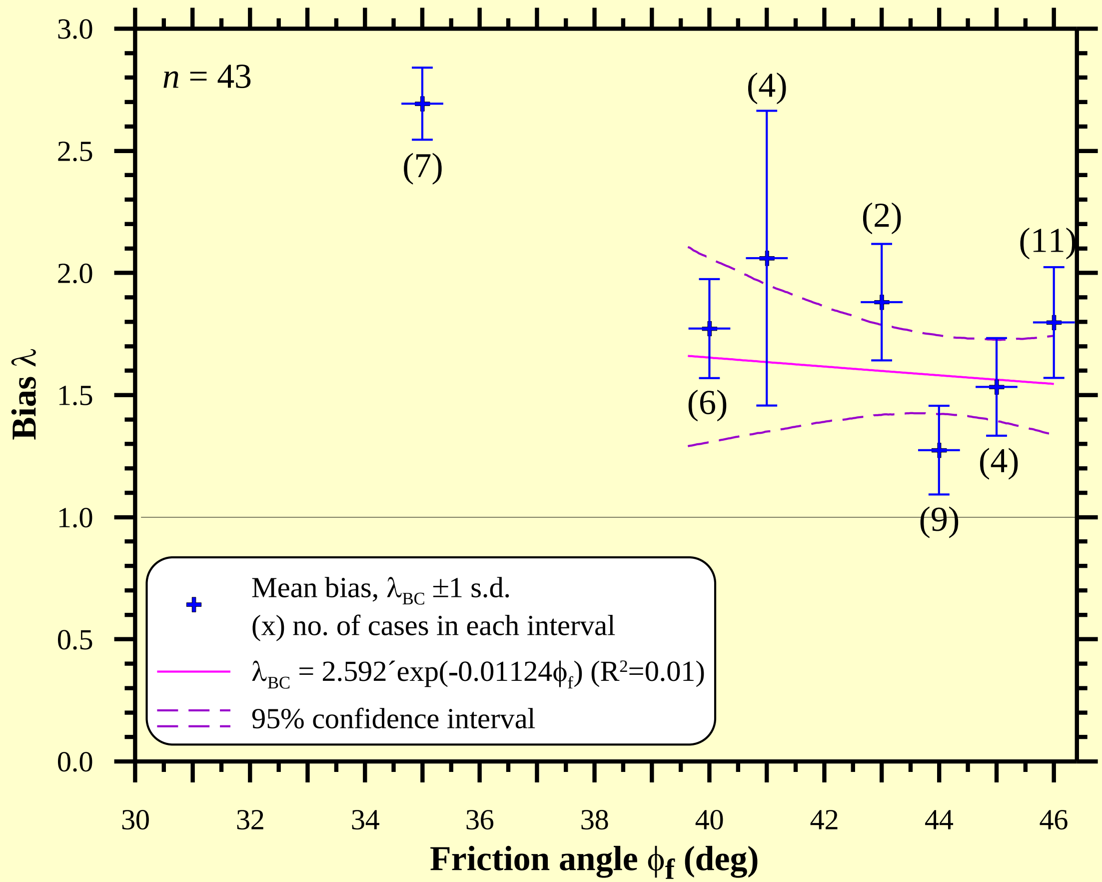
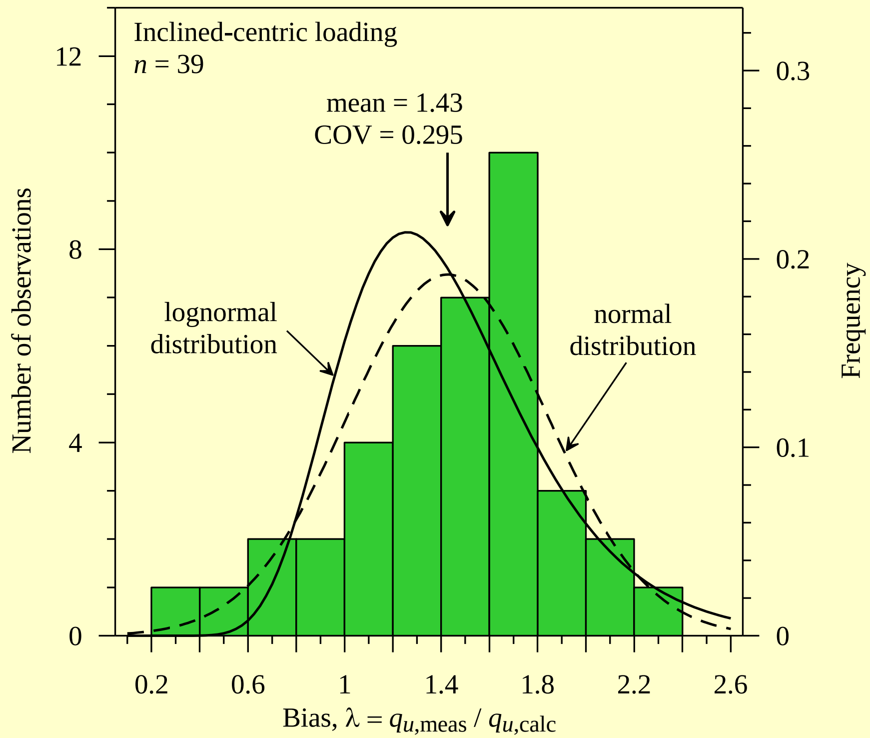


Figure 105. Bearing resistance bias versus soil friction angle for cases under vertical-eccentric loadings; seven cases for ϕ_f = 35° (all from a single site) have been ignored for obtaining the best fit line.

Bias of Estimated BC

Cases with Inclined-Centric Loading



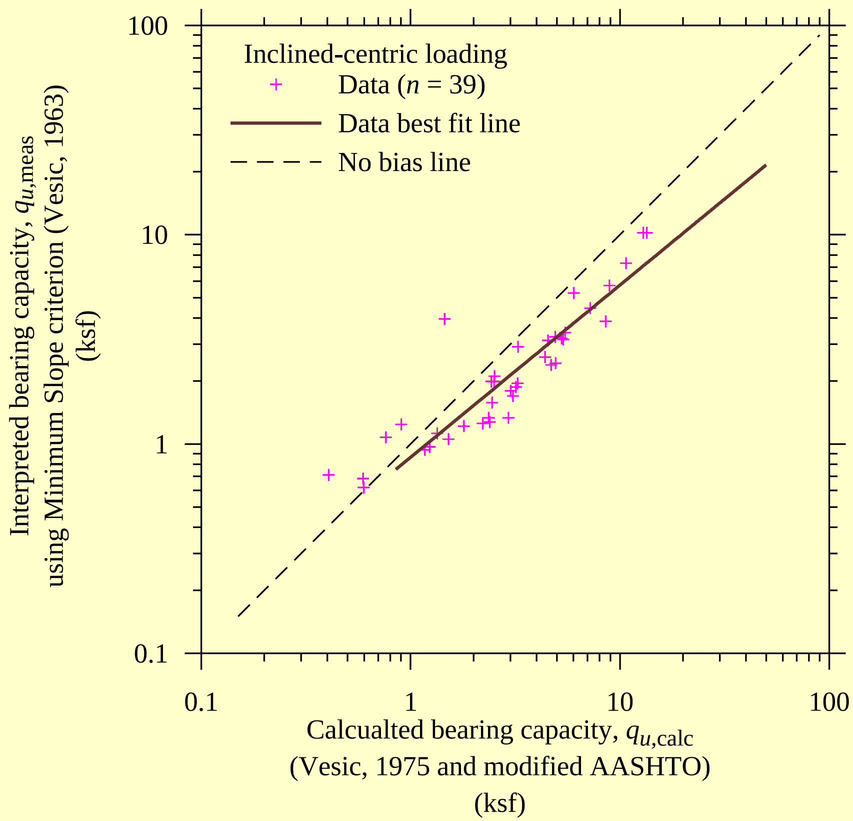


Figure 67. (a) Histogram and probability density functions of the bias and (b) relationship between measured and calculated bearing capacity for all cases of inclined centric loaded shallow foundations.

Bias of Estimated BC

Cases with Inclined-Eccentric Loading

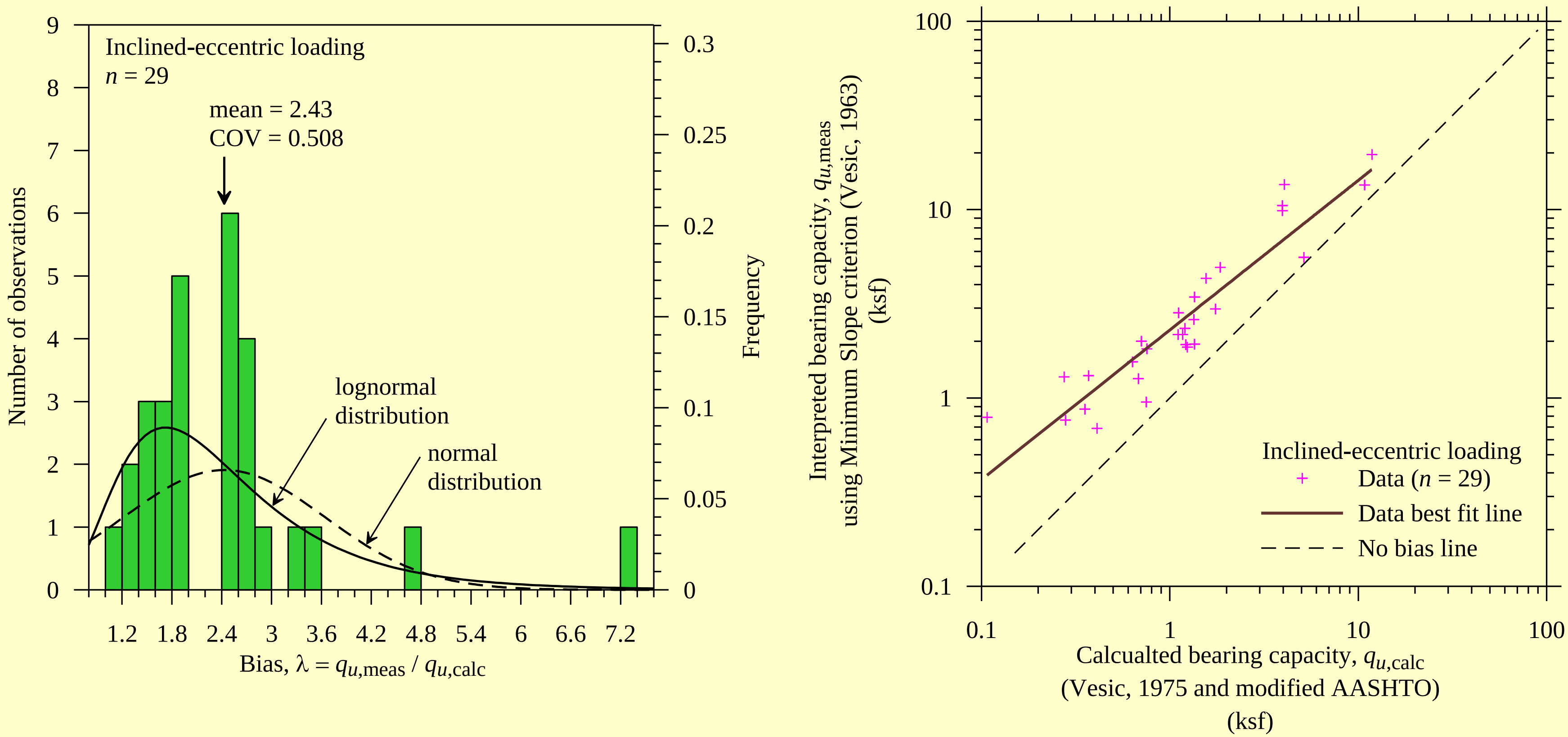
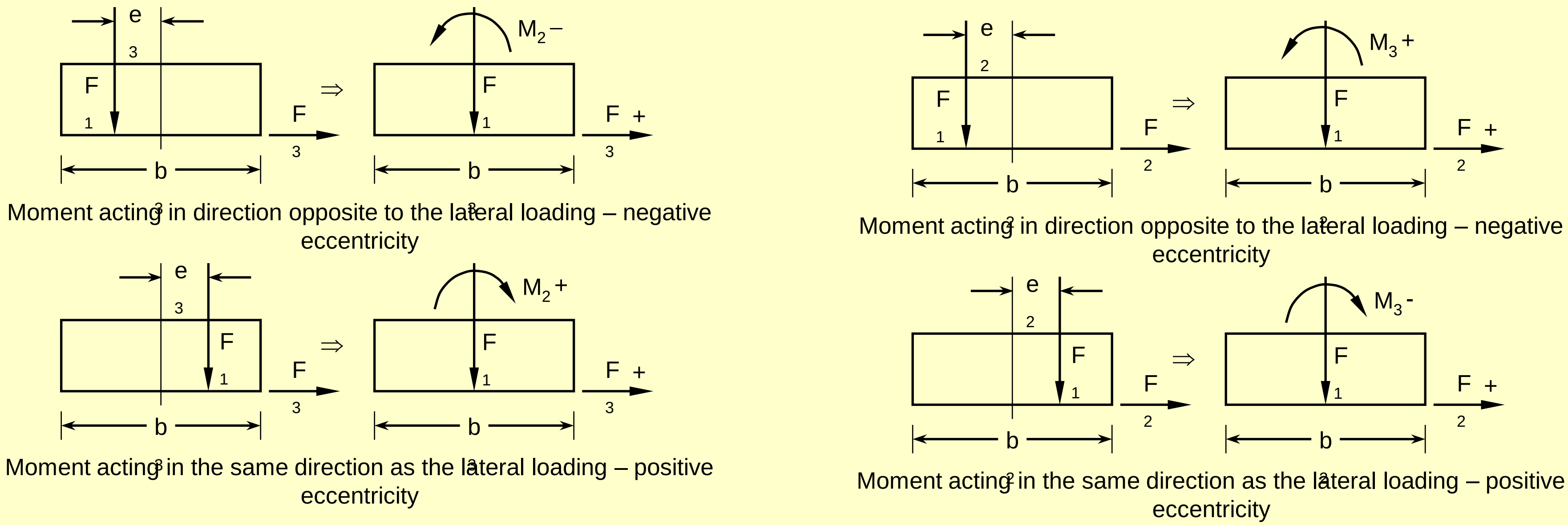


Figure 68. (a) Histogram and probability density functions of the bias and (b) relationship between measured and calculated bearing capacity for all cases of inclined eccentrically loaded shallow foundations.

Loading Directions for Inclined-Eccentric Loadings



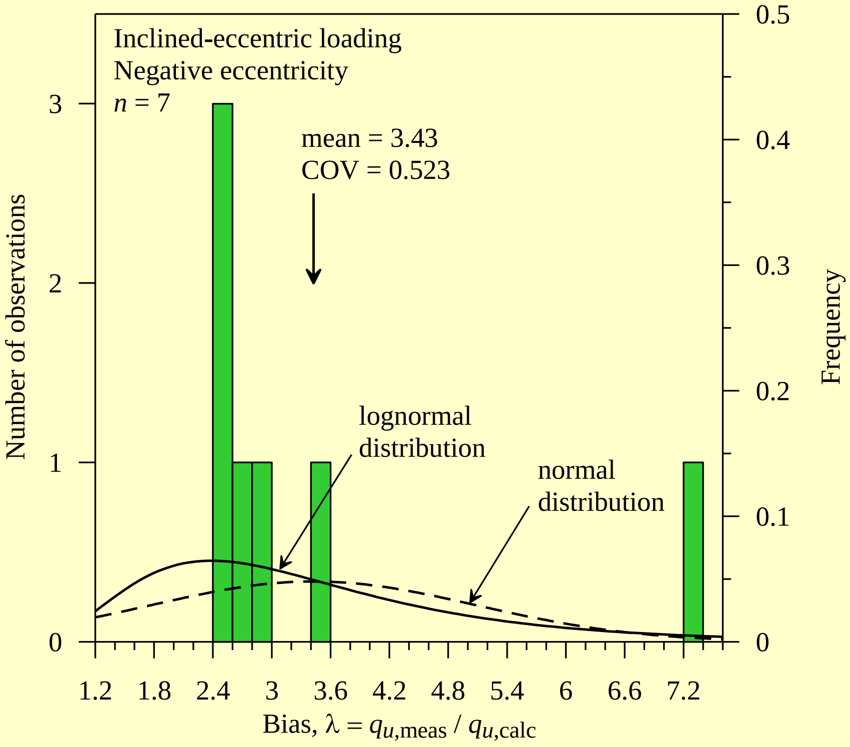
(a) along footing width

(b) along footing length

Figure 69. Loading directions for the case of inclined-eccentric loadings: (a) along footing width and (b) along footing length

Bias of Estimated BC

Cases with Inclined-Eccentric Loading



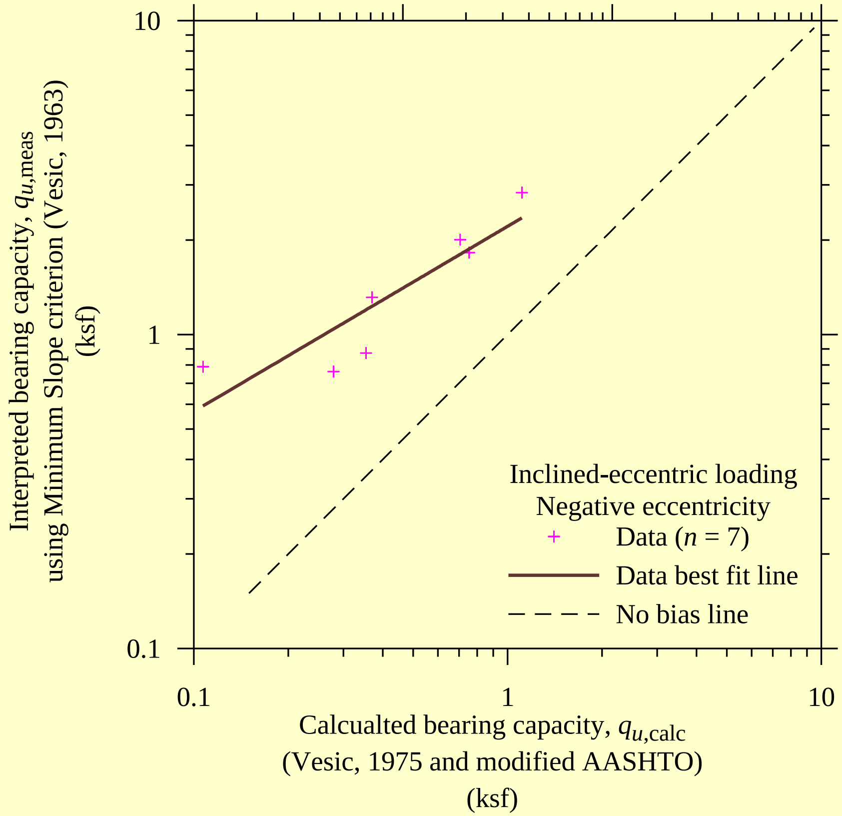


Figure 71. (a) Histogram and probability density functions of the bias and (b) relationship between measured and calculated bearing capacity for all cases of inclined eccentrically loaded shallow foundations under negative eccentricity.

Final Resistance Factors – Natural Conditions

Table 67 Recommended resistance factors for shallow foundations on natural deposited granular soil conditions

<table>
<tr><th rowspan="3">Soil friction angle ϕ_f</th><th colspan="4">Loading conditions</th></tr>
<tr><th rowspan="2">Vertical-centric or -eccentric</th><th rowspan="2">Inclined-centric</th><th colspan="2">Inclined-eccentric</th></tr>
<tr><th>Positive</th><th>Negative</th></tr>
<tr><td>30° – 34°</td><td>0.40</td><td rowspan="3">0.40</td><td rowspan="2">0.35</td><td>0.65</td></tr>
<tr><td>35° – 36°</td><td>0.45</td><td rowspan="2">0.70</td></tr>
<tr><td>37° – 39°</td><td>0.50</td><td rowspan="2">0.40</td></tr>
<tr><td>40° – 44°</td><td>0.55</td><td>0.45</td><td rowspan="2">0.75</td></tr>
<tr><td>≥ 45°</td><td>0.65</td><td>0.50</td><td>0.45</td></tr>
</table>

Notes:

1) ϕ_f determined from Standard Penetration Test results
2) granular material is assumed to extend below the base of the footing at least two (2.0) times the width of the foundation.
3) The resistance factors were evaluated for a target reliability β_T = 3.0

Conceptual Design – Influence of Serviceability

ϕ's based on Serviceability Limit States

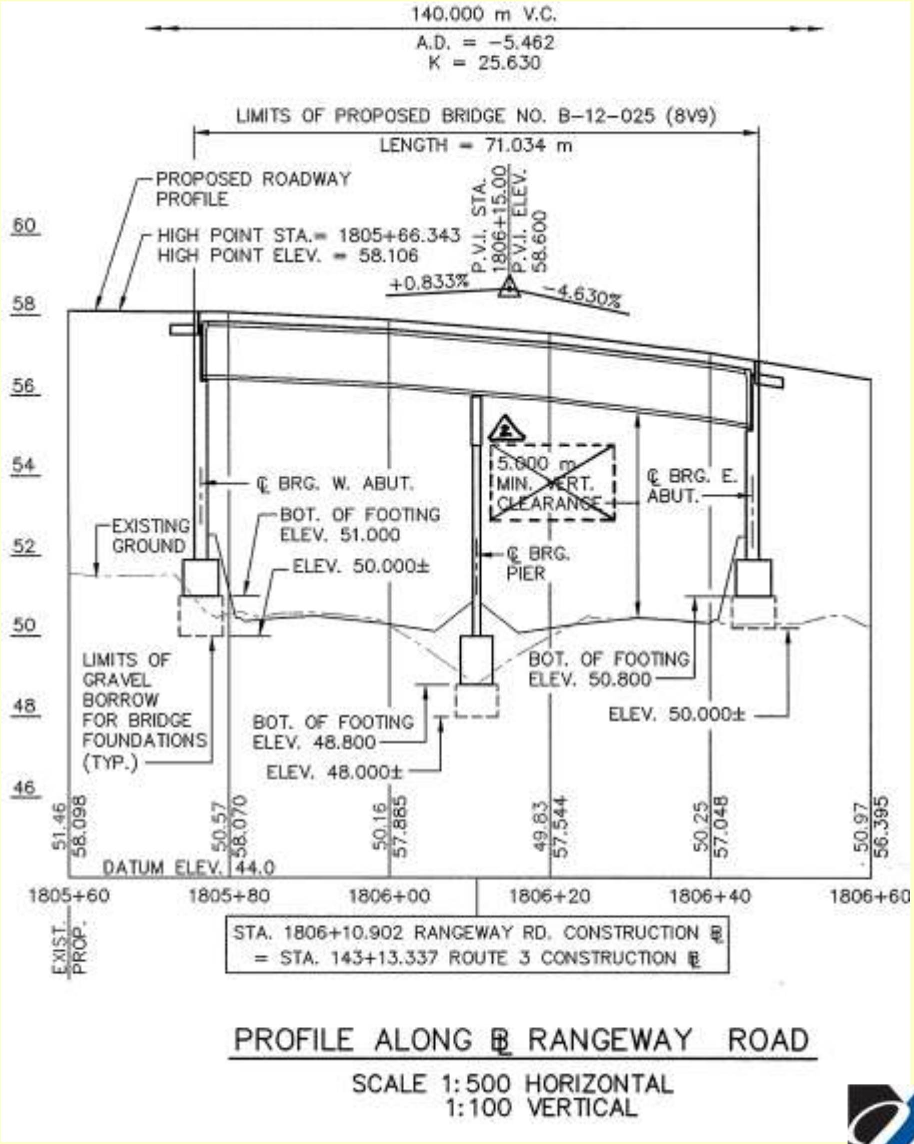
- **Developed as a part of Project NCHRP 12-66**
- **Bias = measured load / calculated load for a given settlement**
- **For reliability index = 1.28 (p_f = 10%), and load factors taken as unity**
- **Bias of LL = 1.15, COV_{QL} = 0.2**
  Bias of DL = 1.05, COV_{QD} = 0.1

Method	Range of Settlement Δ (inch)	Resistance Factor ϕ	Efficiency Factor ϕ/λ
AASHTO	$0.00 < \Delta \leq 1.00$	0.85	0.34
	$1.00 < \Delta \leq 1.50$	0.80	0.48
	$1.50 < \Delta \leq 3.00$	0.60	0.48

Conceptual Design – Granular Soils

Example 2 NCHRP Report 651: Known Load and Settlement

- **Central Pier of a bridge in Billerica (Rangeway Rd over Rte.3) (B-12-025)**
- **Design (factored) load is 3688.3kips for ultimate limit state and 2750kips for service limit state (unfactored)**
- **Allowable settlement 1.5inches**



Conceptual Design – Granular Soils

Factored Resistances (ksf)

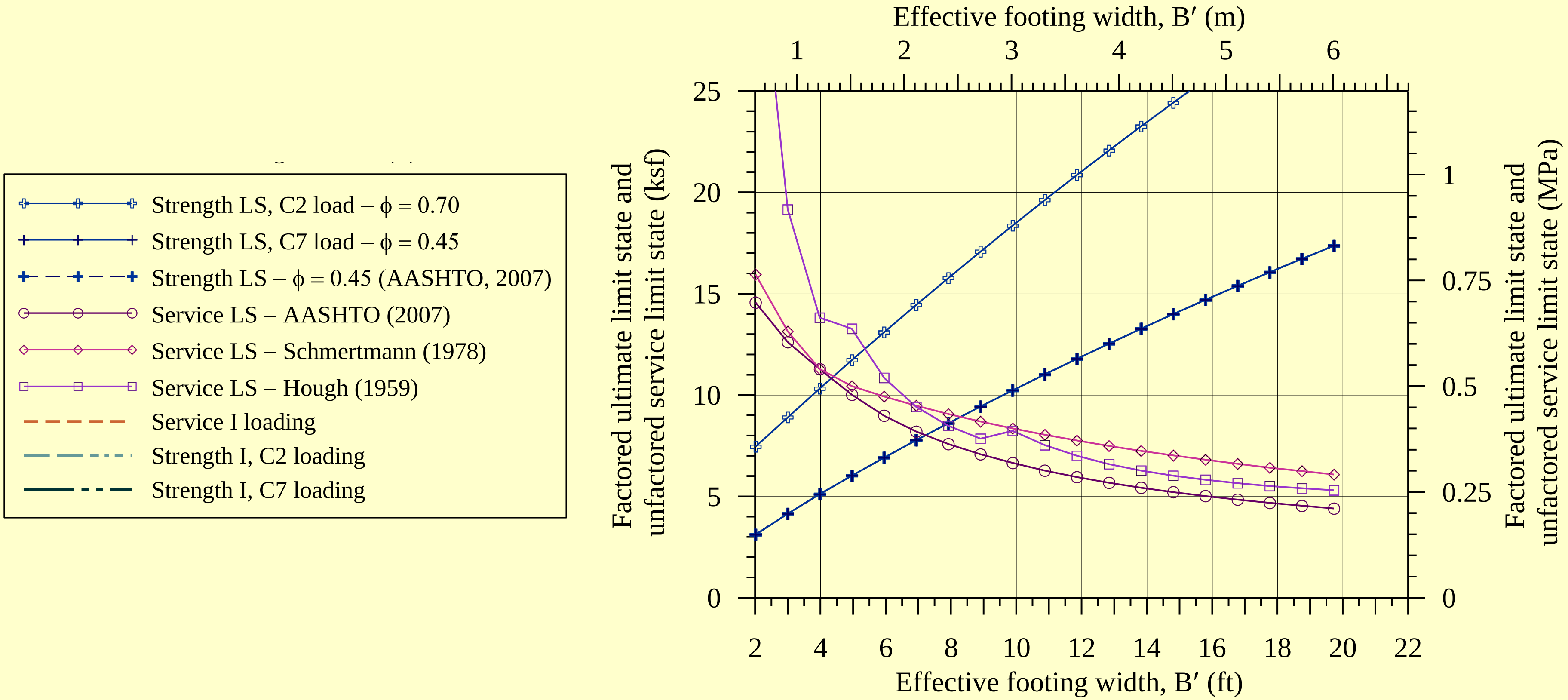
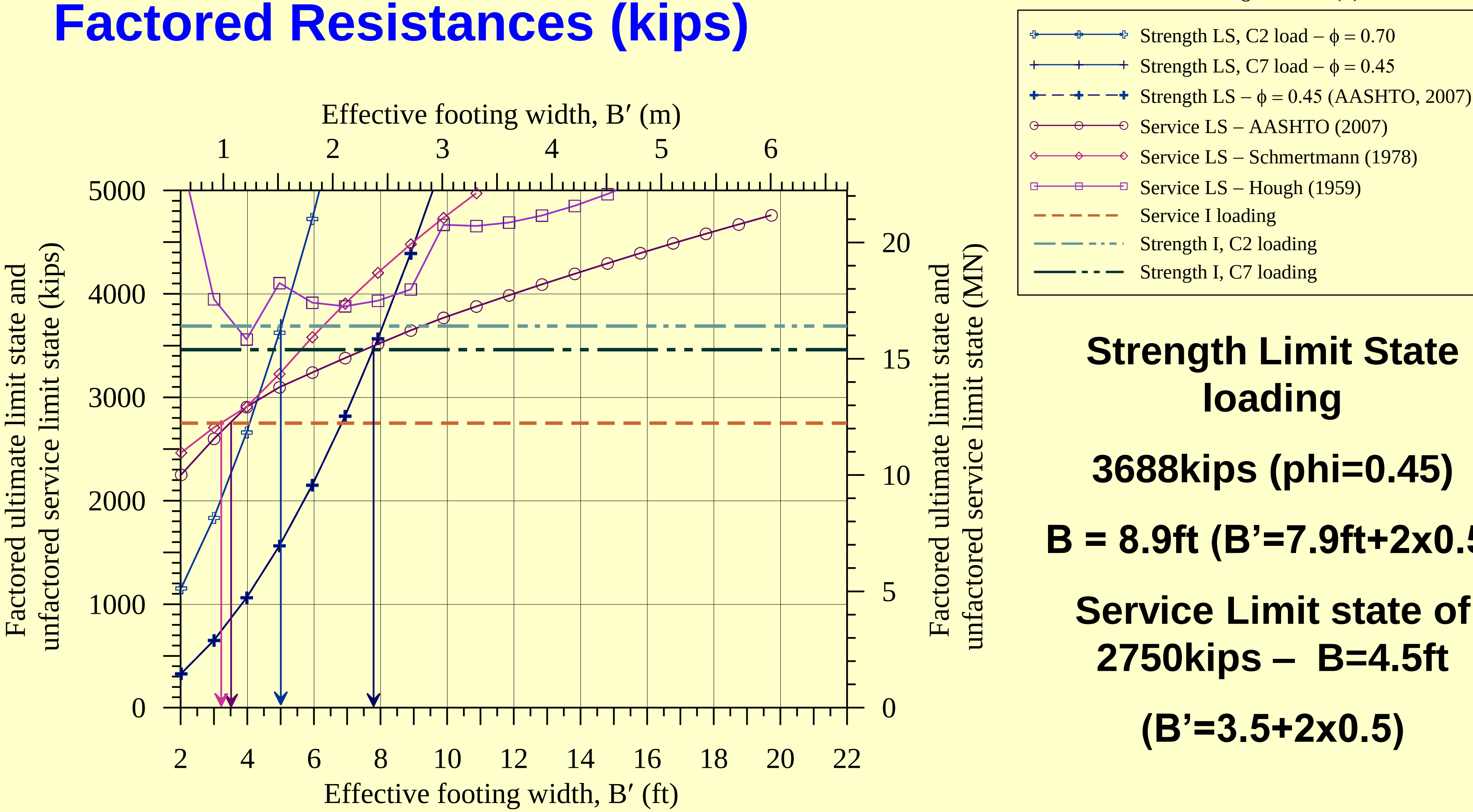


Figure H-6 Variation of factored bearing resistance for Strength-I and unfactored resistance for Service-I limit state with effective footing width for Example 2 (NCHRP Report 651)

Conceptual Design – Granular Soils

Factored Resistances (kips)



Strength Limit State loading

3688kips (phi=0.45)

B = 8.9ft (B'=7.9ft+2x0.5)

Service Limit state of 2750kips – B=4.5ft

(B'=3.5+2x0.5)

Figure H-6 cont. Variation of factored bearing resistance for Strength-I and unfactored resistance for Service-I limit state with effective footing width for Example 2 (NCHRP Report 651)

Intermediate Conclusions

- The Strength Limit State governs the footing dimensions in this design example with a requirement for B=8.9ft vs. B=4.5ft for the service limit state
- The bridge was designed with B=13.1ft most likely due to the differences in design procedures (especially settlement)

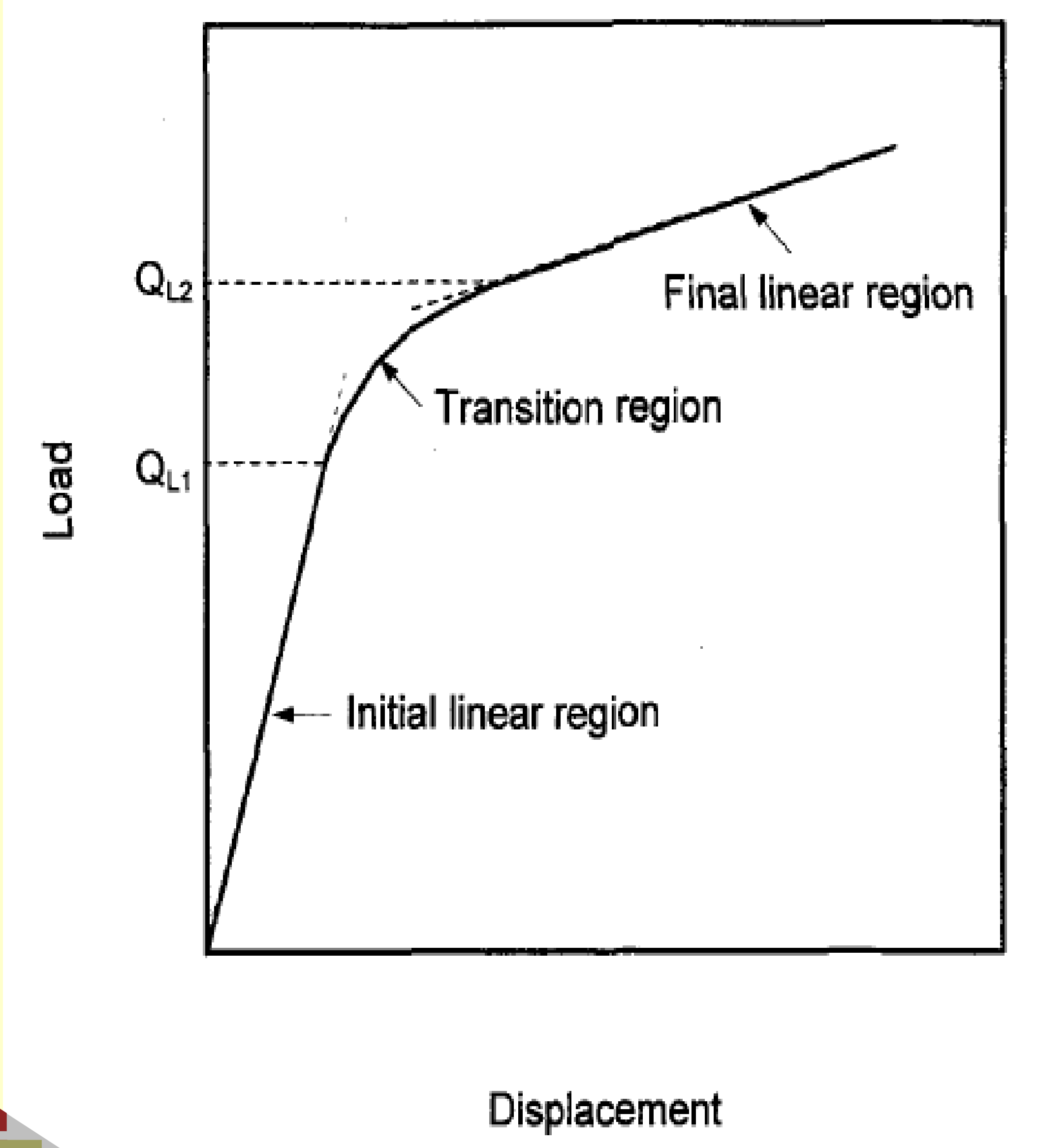
Task 3: BC Shallow Foundations on Rock - OUTLINE

1. **Broad Objectives**
2. **Database UML/GTR RockFound07**
3. **Rock Classification and Properties**
4. **Methods of Analyses Selected for Establishing the Uncertainty in B.C. of Foundations on Rock**
5. **Calibration – evaluation of resistance factors**
6. **Summary and Conclusions**

2. DATABASE UML/GTR RockFound 07

- Comprised of 122 foundation case histories of load tests in/on rock and IGM's.
- The database has 61 footings cases (28 cases D>0, 33 cases D=0) and 61 rock socket cases for which the base behavior (load and displacement) under loading was monitored.
- 89 of the 122 cases were used for the uncertainty determination of the settlement of foundations on rock.

Hirany and Kulhawy (1988) Failure criterion



Hirany and Kulhawy (1988) proposed the L1-L2 method for interpreting the "failure" load or "ultimate" capacity of foundations from load-displacement curves.

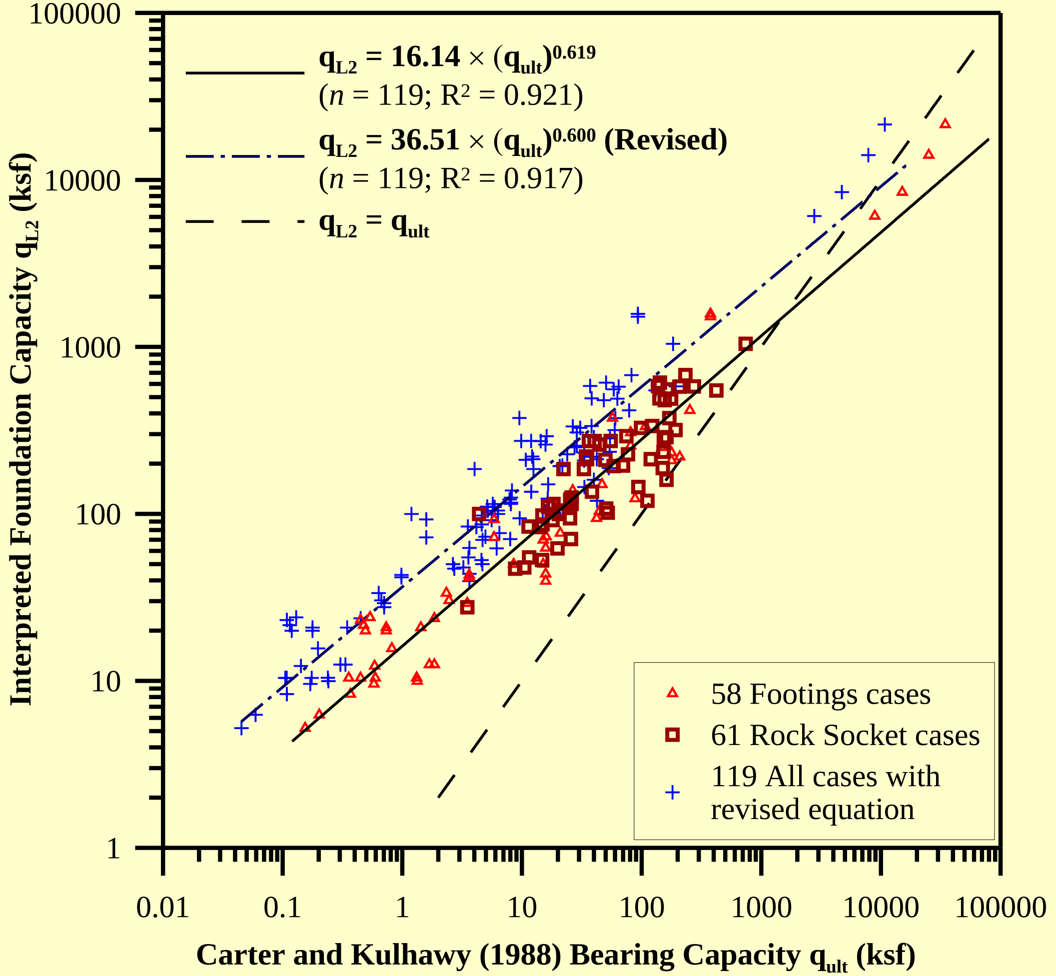
The unique peak or asymptote value in the curves is taken as the measured or interpreted capacity (QL2=qL2).

For 79 cases qL2 could be evaluated, 43 cases are based on reported failure load.

Using the limit-equilibrium approach, Carter and Kulhawy (1988) developed a lower bound to the B.C. for strip and circular footings on jointed rock masses presented below.

$$q_{ult} = \left(m + \sqrt{s}\right) q_u \qquad \textbf{(2)}$$

Carter and Kulhawy (1988) - B.C. of Foundations on Rock



Relationship between Carter and Kulhawy (1988) calculated bearing capacity (q_{ult}) using two variations (equations 82a and 82b) and the interpreted bearing capacity (q_{L2}).

Carter and Kulhawy (1988) - B.C. of Foundations on Rock

Table 69 Calibrated resistance factors for different datasets of resistance bias obtained using Carter and Kulhawy's (1988) method

Dataset	No. of cases	Bias		Resistance factor ϕ ($\beta_T = 3$)	
		Mean λ	COV_λ	MCS	Recommended
All cases	119	8.00	1.240	0.372	0.35
RMR ≥ 85	23	2.93	0.651	0.535	0.50
65 ≤ RMR < 85	57	3.78	0.463	1.149	1.00
44 ≤ RMR < 65	17	8.83	0.651	1.612	1.00
3 ≤ RMR < 44	22	23.62	0.574	5.295	1.00

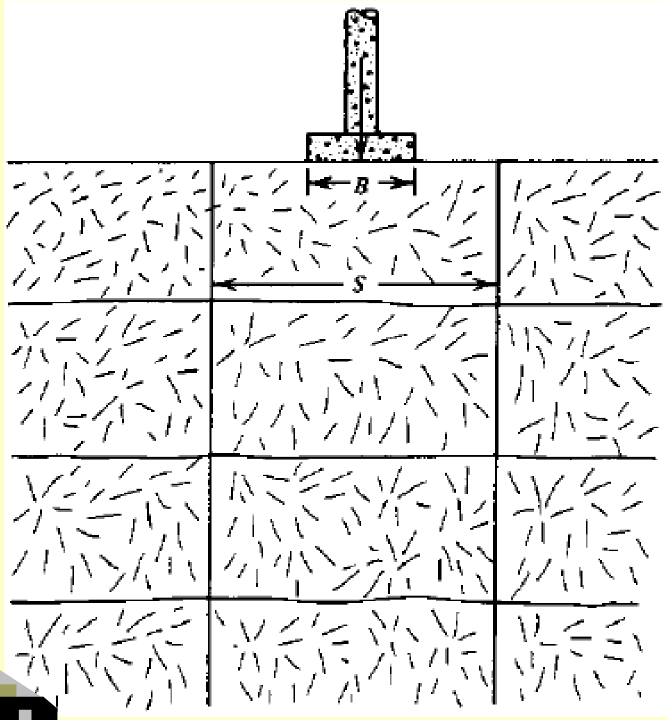
Goodman (1989) - B.C. of Foundations on Rock

The lower bound is represented by the following Equation:

$$q_{ult} = q_u\left(N_\phi + 1\right) \qquad (3)$$

in which

$$N_\phi = \tan^2\left(45 + \phi/2\right) \qquad (4)$$



Goodman (1989) developed the B.C. Equation 5 for footings resting on orthogonal vertical joints each spaced distance s in which lateral stress transfer is nil.

$$q_{ult} = q_u\left\{\frac{1}{N_\phi - 1}\left[N_\phi\left(\frac{S}{B}\right)^{\left(N_\phi - 1\right)/N_\phi} - 1\right]\right\} \qquad (5)$$

Goodman (1989) - B.C. of Foundations on Rock

Summary of the statistics for the Ratio of Measured to Calculated B.C. using Goodman's (1989) Method

Cases	n	No. of Sites	Mean of Bias m_λ	Standard Deviation σ_λ	COV_λ
All Foundations	119	78	1.35	0.72	0.535
All rock sockets	61	49	1.52	0.82	0.541
All footings	58	29	1.23	0.66	0.539

Sub-categorization suggests that if more details of rock measurements are available, the uncertainty is reduced.

1. 34 Rock Socket cases with measured discontinuity spacing had a $COV_l = 0.48$.
2. 8 Rock Socket cases with measured discontinuity spacing and friction angle had a $COV_l = 0.18$.

Goodman (1989) - B.C. of Foundations on Rock

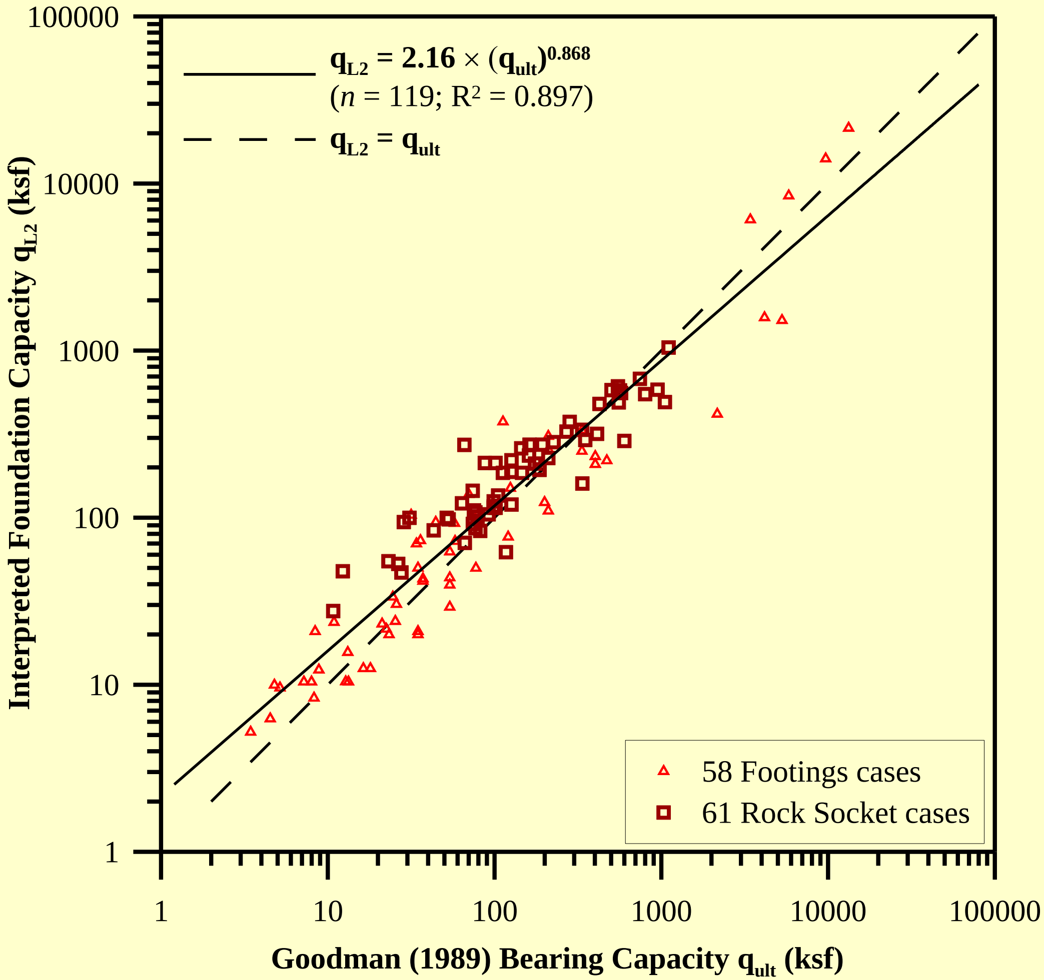


Figure 78. Relationship between Goodman's (1989) calculated bearing capacity (q_{ult}) and the interpreted bearing capacity (q_{L2}).

Goodman (1989) - B.C. of Foundations on Rock

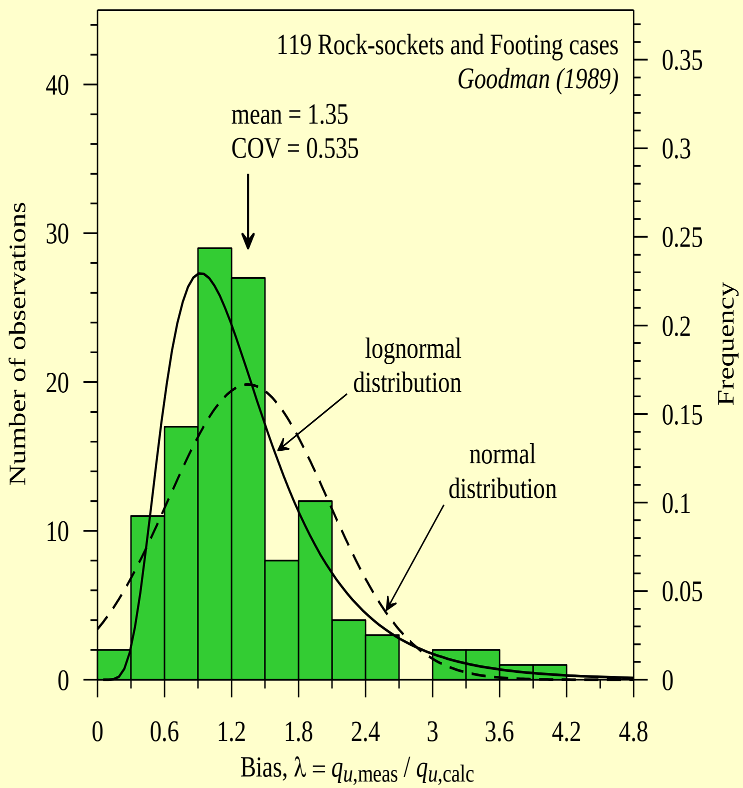


Figure 79. Distribution of the ratio of the interpreted bearing capacity (q_{L2}) to the bearing capacity (q_{ult}) calculated using Goodman's (1989) method for the rock sockets and footings in database UML-GTR RockFound07.

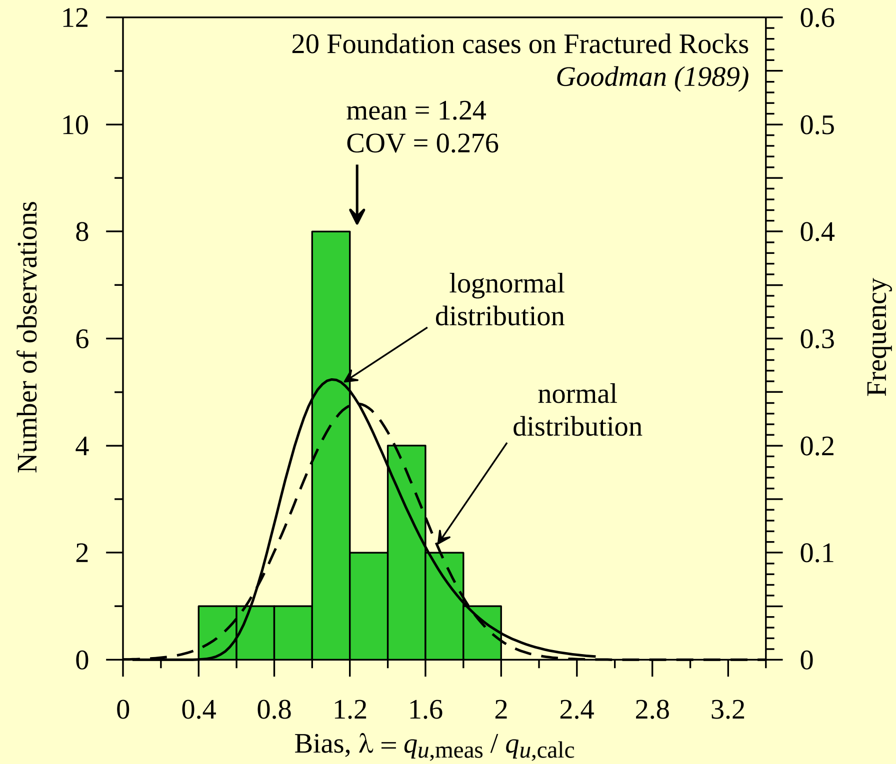


Figure 80. Distribution of the ratio of the interpreted bearing capacity (q_{L2}) to the bearing capacity (q_{ult}) calculated using Goodman's (1989) method for foundations on fractured rock in database UML-GTR RockFound07

Goodman (1989) - B.C. of Foundations on Rock

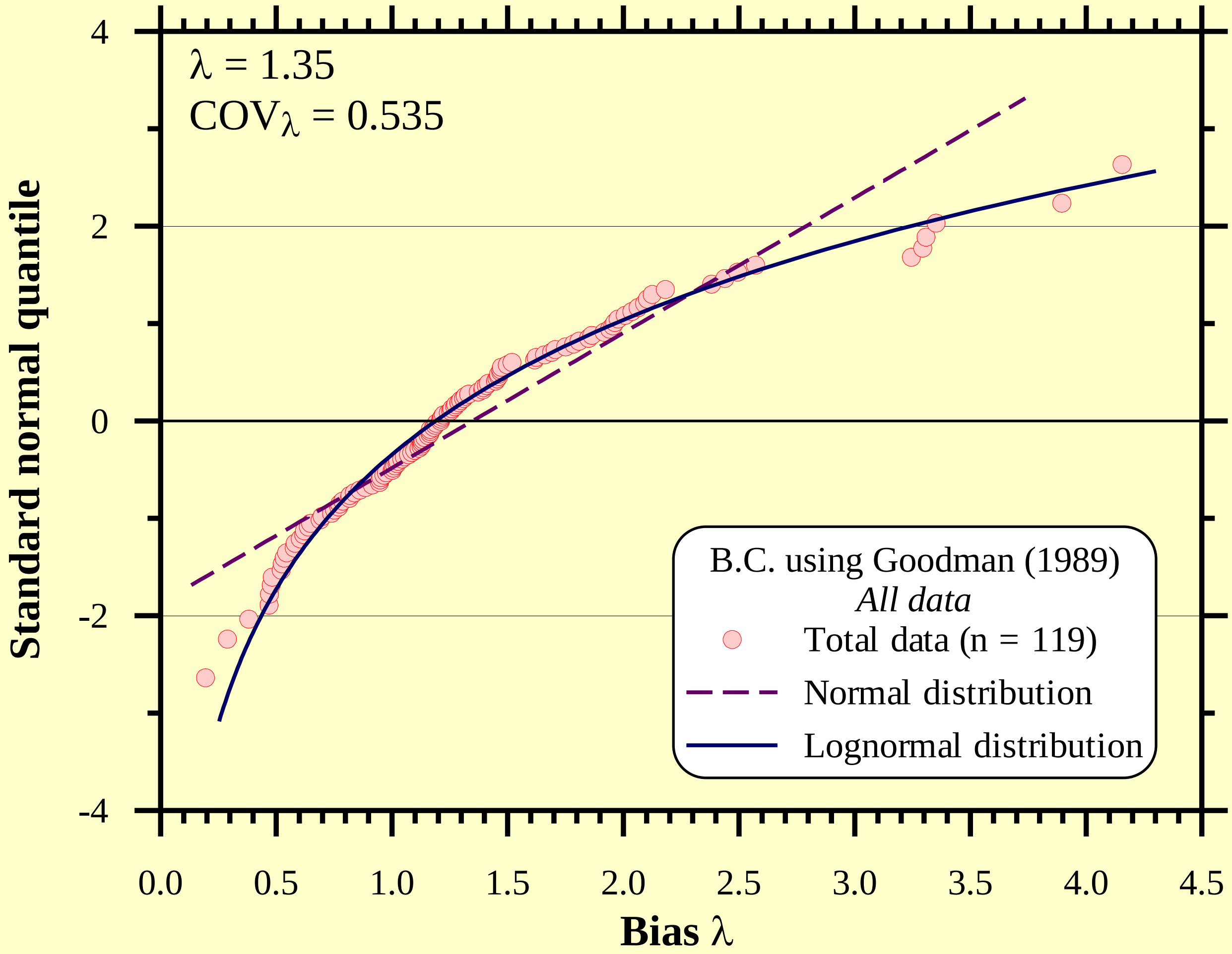


Figure 113. Comparison of the unfiltered bias for BC calculated using Goodman (1989) method for all data and the theoretical normal and lognormal distributions.

Goodman (1989) - B.C. of Foundations on Rock

Table 68 Calibrated resistance factors for different datasets of resistance bias obtained using Goodman's (1989) method

Dataset	No. of cases	Bias		Resistance factor ϕ (β_T = 3)	
		Mean λ	COV_λ	MCS	Recommended
All data	119	1.35	0.535	0.336	0.30
Measured friction angle ϕ_f	98	1.41	0.541	0.346	0.35
Measured spacing s′	83	1.43	0.461	0.437	0.40
Measured friction angle ϕ_f and s′	67	1.51	0.459	0.464	0.45

Calibration of Resistance Factors

Table 70 Recommended resistance factors for foundations in/on rock based on $\beta_T = 3.0$ ($p_f = 0.135\%$)

Method of Analysis	Equation	Application	ϕ	Efficiency Factor ϕ/λ (%)
Carter and Kulhawy (1988)	$q_{ult} = q_u\left(m + \sqrt{s}\right)$	All	0.35	4.4
		RMR $\geq$ 85	0.50	17.1
		65 $\leq$ RMR < 85	1.00	26.5
		44 $\leq$ RMR < 65		11.3
		3 $\leq$ RMR < 44		4.2
Goodman (1989)	For fractured rocks: $q_{ult} = q_u\left(N_\phi + 1\right)$	All	0.30	22.2
		Measured ϕ_f	0.35	24.8
	For non-fractured rocks: $q_{ult} = q_u\left(\frac{1}{N_\phi - 1}\left\{N_\phi\left(\frac{s'}{B}\right)^{(N_\phi - 1)/N_\phi} - 1\right\}\right)$	Measured s′	0.40	28.0
		Measured s′ and ϕ_f	0.45	29.8


THANK YOU VERY MUCH FOR YOUR ATTENTION
The End!

Uncertainty in B.C.

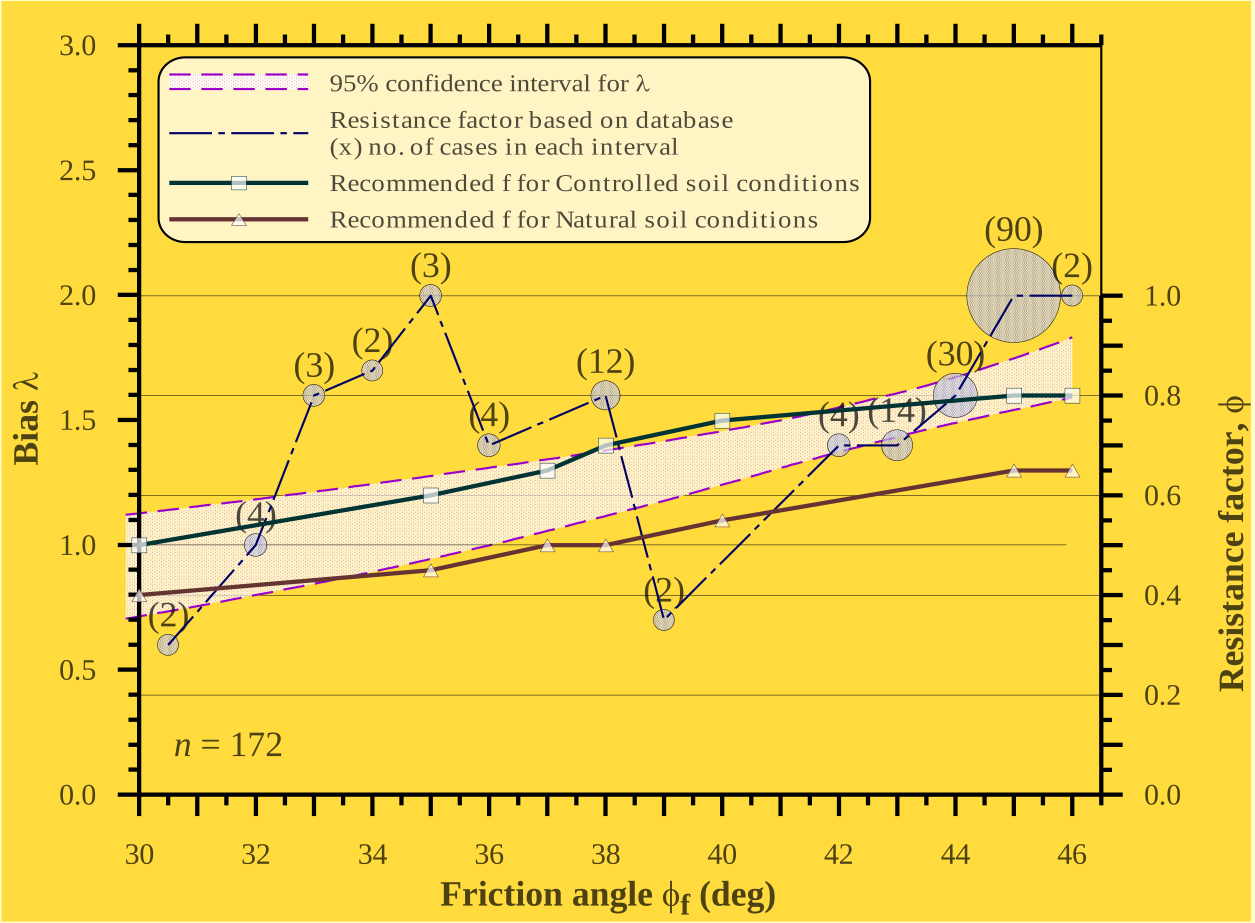


Figure 104. Recommended resistance factors for soil friction angles (taken $\phi_f \pm 0.5°$) between 30° and 46°, with comparisons to 95% confidence interval and resistance factors obtained for the cases in the database; the bubble size represents the number of data cases in each subset.

Carter and Kulhawy (1988) - B.C. of Foundations on Rock

Table 38 Summary of the statistics for the ratio of measured (q_{L2}) to calculated bearing capacity (q_{ult}) of rock sockets and footings on rock using Carter and Kulhawy (1988) method

Cases	n	No. of Sites	m_λ	σ_λ	COV
All rock sockets	61	49	4.29	3.08	0.716
All rock sockets on fractured rock	11	6	5.26	1.54	0.294
All rock sockets on non-fractured rock	50	43	4.08	3.29	0.807
Rock sockets on non-fractured rock with measured discontinuity spacing (s')	34	14	3.95	3.75	0.949
Rock sockets on non-fractured rock with s' based on AASHTO (2007)	16	13	4.36	2.09	0.480
All footings	58	29	11.90	12.794	1.075
All footings on fractured rock	9	3	2.58	2.54	0.985
All footings on non-fractured rock	49	26	13.62	13.19	0.969
Footings on non-fractured rock with measured discontinuity spacing (s')	29	11	15.55	14.08	0.905
Footings on non-fractured rock with s' based on AASHTO (2007)	20	11	10.81	11.56	1.069

n = number of case histories m_λ = mean of biases σ_λ = standard deviation
COV = coefficient of variation Calculated capacity based on equation (82a)

Goodman (1989) - B.C. of Foundations on Rock

Table 40 Summary of the statistics for the ratio of measured (q_{L2}) to calculated bearing capacity (q_{ult}) of rock sockets and footings on rock using Goodman (1989) method

Cases	n	No. of Sites	m_λ	σ_λ	COV
All	119	78	1.35	0.72	0.535
Measured discontinuity spacing (s') and friction angle (ϕ_f)	67	43	1.51	0.69	0.459
Measured discontinuity spacing (s')	83	48	1.43	0.66	0.461
Measured friction angle (ϕ_f)	98	71	1.41	0.76	0.541
Fractured	20	9	1.24	0.34	0.276
Fractured with measured friction angle (ϕ_f)	12	7	1.33	0.25	0.189
Non-fractured	99	60	1.37	0.77	0.565
Non-fractured with measured s' and measured ϕ_f	55	37	1.55	0.75	0.485
Non-fractured with measured discontinuity spacing (s')	63	39	1.49	0.72	0.485
Non-fractured with measured friction angle (ϕ_f)	86	64	1.42	0.81	0.569
Spacing s' and ϕ_f, both based on AASHTO (2007)	5	3	0.89	0.33	0.368
Discontinuity spacing (s') based on AASHTO (2007)	36	21	1.16	0.83	0.712
Friction angle (ϕ_f) based on AASHTO (2007)	21	7	1.06	0.37	0.346

n = number of case histories m_λ = mean of biases σ_λ = standard deviation COV = coefficient of variation