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USE OF FULLY SOFTENED VERSUS PEAK STRENGTH TO PREDICT THE CAPACITY OF FOOTINGS ON GEOSYNTHETIC REINFORCED SOIL

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ABSTRACT: A database of load tests performed on geosynthetic reinforced soil (GRS - reinforcement in this study is of the extensible variety) was developed using results of recent load tests performed on large scale GRS structures at the Federal Highway Administration's Turner Fairbank Highway Research Center as well as results from the literature. The measured capacities were compared to those predicted using the Wu and Pham [1] equation utilizing both the peak and fully softened soil shear strength parameters. It was found that the fully softened strengths yielded capacities that agreed better with the measured capacities. A rationale for this finding is that the robust reinforcement in a GRS strengthens the soil considerably causing the GRS to experience large strains prior to failure. Because the soil peak strengths are mobilized at relatively small displacements/strains even in large scale direct shear or triaxial tests compared to the GRS load tests, it is postulated that the fully softened values are more appropriate to estimate the GRS bearing capacity. A follow-on to this is that since large movements are required to fail say a GRS abutment, the design of GRS abutments will most likely be governed by the serviceability limit state rather than the ultimate limit state.

Keywords: Geosynthetic Reinforced Soil, Bearing Capacity, Fully Softened Strength and Bridge Abutments

1. INTRODUCTION

Geosynthetic Reinforced Soil (GRS) is defined as closely-spaced (≤ 0.3 m) layers of geosynthetic reinforcement and compacted granular fill material [2]. GRS has been used for a variety of geotechnical applications but has recently been promoted by the Federal Highway Administration (FHWA) for use as abutments for single span steel or concrete bridges in their Everyday Counts Initiative, which is focused on accelerating implementation of proven, market-ready technologies. GRS-IBS, where IBS stands for Integrated Bridge Systems, consists of a reinforced soil foundation (RSF), a GRS abutment and a GRS integrated approach (Fig. 1). The RSF consists of granular fill compacted and encapsulated in geotextile. The RSF provides embedment and increases the bearing width and capacity of the GRS abutment. The GRS abutment provides load-bearing support for the bridge, which is placed directly on the abutment. GRS is also used to construct the integrated approach adjacent to the superstructure. GRS-IBS has the following advantages:

1. Fast and cost-effective method of bridge support. It eliminates the need for cast-in-place reinforced concrete abutments traditionally supported on deep foundations.
2. Quality compaction control can be realized since closely-spaced geosynthetics ensure backfill is placed in thin lifts.
3. Catastrophic collapse was not observed in numerous load tests carried out to failure; GRS abutments behave in a ductile fashion.
4. Can be built in variable weather with common labor, materials and equipment, and can be easily modified in the field.
5. Alleviates the "bump at the end-of-the-bridge" problem caused by differential settlement between the bridge abutment and the approach roadway. This is made possible by eliminating deep foundations, by using GRS to construct the integrated approach and by limiting its use to short, single-span integral bridge systems.
6. Enjoys all the advantages associated with an integral abutment bridge.
7. Very flexible system that is amenable to differential settlement and seismic loading.

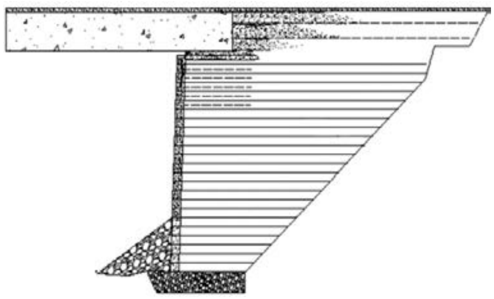

Fig. 1 Typical cross-section of a GRS-IBS [2]

Because GRS-IBS is load bearing, its capacity is an important design consideration. Many studies of the bearing capacity of strip footings supported on

RSF [3]-[11] consider the RSF to extend horizontally on both sides of the footing. In the case of a GRS-IBS, the ground is limited on one side and the bearing capacity of the bridge footing on a GRS abutment wall must be known. Work in this area is rather limited. One exception is the work by Pham [12], who performed plane strain load tests on GRS and who derived an expression for the bearing capacity of a footing on a GRS wall. The applicability of this expression is examined using results from an extensive series of load tests performed at FHWA's Turner-Fairbank Highway Research Center (TFHRC) as well as those from the literature.

1.1 Motivation for This Study

Christopher et al. [13] proposed that peak strengths of the backfill should be used when predicting bearing capacity of footings on mechanically stabilized earth (MSE) type bridge abutments reinforced using extensible elements. However, it was observed that the peak strength from large scale direct shear (LSDS) tests on granular soils in this study was mobilized at about 13 to 16 mm lateral displacements, which correspond to about 4.5 to 5.4% shear strain for a 0.3 m x 0.3 m x 0.2 m high direct shear sample. Also, the peak strength from large scale (0.15 m diameter x 0.3 m high) triaxial tests on granular soils was mobilized at about 2.3 to 5.0% axial strain [12]. As will be presented later, load tests on GRS with closely spaced (< 0.3 m) reinforcement having a wide width tensile strength of at least 70 kN/m generally fail at strains greater than 10%, implying that the soil shear strength is then past its peak.

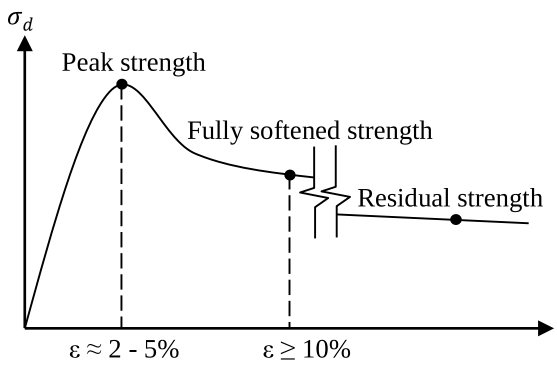

Fig. 2 Typical stress-strain curve

The notion of using fully softened strength is not new in geotechnical engineering. Duncan and Wright [14] recommend the use of fully softened strength when analyzing the stability of cuts in heavily overconsolidated soil. The rationale for this is that swelling and softening was found to have occurred along the slip surfaces during forensic studies of such slides and use of fully softened strength, or the strength if the soil was normally consolidated, provided better agreement when back-calculating the factors of safety in these failed slopes. Wu [15] indicated that the fully softened strength is typically mobilized at strains on the order of 10%. In the case of GRS, the reinforcement strengthens the soil and forces failure to occur very often at double-digit strains (see TF test series in Table 1). Therefore, in the interest of preserving strain compatibility, a study was conducted to see whether fully softened strengths will provide a better prediction of the bearing capacity of footings on GRS abutments than peak strengths.

2. BEARING CAPACITY EQUATION OF A FOOTING ON A GRS ABUTMENT WALL

Pham [12] derived the bearing capacity of a footing on a GRS abutment wall (q_{ult}) as follows:

$$q_{\text{ult}} = \left(\sigma_{\text{h}} + W\frac{T_f}{S_v}\right)K_p + 2c\sqrt{K_p} \tag{1}$$

where σ_h is the lateral stress, T_f and S_v are the reinforcement strength and spacing, respectively, c is the soil cohesion, K_p is the Rankine passive earth pressure coefficient, defined as

$$K_p = \frac{1+\sin\phi}{1-\sin\phi} \tag{2}$$

ϕ is the soil friction angle. W is a dimensionless factor that amplifies the contribution of S_v to the GRS capacity, and was semi-empirically derived as

$$W = 0.7^{\frac{S_v}{6d_{max}}} \tag{3}$$

where d_{max} is the maximum particle size of the GRS backfill. Note that the 0.7 factor was theoretically derived using the concept of "average stresses" proposed by Ketchart and Wu [16] while the exponent was empirically derived. For details on this derivation, refer to [12].

For a GRS wall with dry stacked modular block facing, σ_h = lateral stress exerted by the facing on the GRS mass, defined by Pham [12] as

$$\sigma_h = \gamma_{bl} D \tan\delta \tag{4}$$

where γ_{bl} = bulk unit weight of facing block = weight of block/volume of block assuming it is not hollow, D = depth of facing block perpendicular to the wall face and δ = friction angle between geosynthetic reinforcement and the top and bottom surface of the facing block.

2.1 GRS versus MSE

Mechanically stabilized earth (MSE) differs from GRS in many respects. The most significant difference involves the maximum reinforcement spacing (0.3 m in GRS versus 0.8 m in MSE). The basic MSE design premise is that each reinforcement layer is responsible for equilibrium within the reinforcement tributary area; i.e.; $\sigma_h = T_f/S_v$. Implied in this equation is that a GRS with reinforcement strength T_f at spacing S_v will behave the same as a GRS with reinforcement strength $2T_f$ at spacing $2S_v$, which has been shown to be untrue by Adams et al. [17] and Pham [12]. Instead, S_v has a bigger influence on the bearing capacity than T_f. This led Pham to propose the addition of the W term in Eq. 1, without which, the equation is the expression for the bearing capacity of a footing on a MSE abutment wall.

3. APPLICABILITY OF BEARING CAPACITY EQUATION

Considering that most of the load tests in the database were performed on GRS columns (mostly square with some circular in plan) while a bridge footing resting on an abutment more resembles a plane strain (PS) condition, the relationship between the column tests and that of a strip footing loading the top of a GRS wall is of interest. Assume that the strength of a GRS column can be represented by the Mohr-Coulomb equation as follows:

$$\tau = c_{GRS} + \sigma tan\phi_{GRS} \tag{5}$$

where τ = shear strength, σ = applied normal stress, c_{GRS} and ϕ_{GRS} = cohesion and friction angle of the GRS composite, respectively. In an unconfined compression load or Performance Test (PT), where the facing has been removed, the ultimate capacity of the GRS column ($q_{ult,PT}$) can be expressed as

$$q_{ult,PT} = 2c_{GRS} \tag{6}$$

For the PS condition, the bearing capacity of a footing supporting the bridge superstructure can be estimated using Meyerhof's [19] solution for a rough strip bearing on top of a slope

$$q_{ult,PS} = c_{GRS}N_{cq} + 0.5\gamma_{GRS}bN_{\gamma q} \tag{7}$$

where $q_{ult,PS}$ = ultimate capacity of strip footing under PS conditions, γ_{GRS} = unit weight of the GRS backfill, b = footing width, and N_{cq} and $N_{\gamma q}$ = Meyerhof's [19] bearing capacity factors for a strip footing with a rough base. $N_{\gamma q}$ approaches zero when the slope angle is 90° for a GRS abutment wall; thus Eq. (7) reduces to

$$q_{ult,PS} = c_{GRS}N_{cq} \tag{8}$$

Dividing Eq. (8) by (6), the ratio of the bearing capacity of a strip footing on top of a GRS abutment to that of a GRS column can be estimated as

$$\frac{q_{ult,PS}}{q_{ult,PT}} = \frac{N_{cq}}{2} \tag{9}$$

For a surface footing on top of a vertical GRS abutment, the value of N_{cq} varies with the footing offset from the edge of the wall face, a, wall height, H, footing width, b, and stability factor, $N_s = \frac{\gamma_{GRS}H}{c_{GRS}}$, as shown in Fig. 3. c_{GRS} can be obtained from laboratory or numerical experiments.

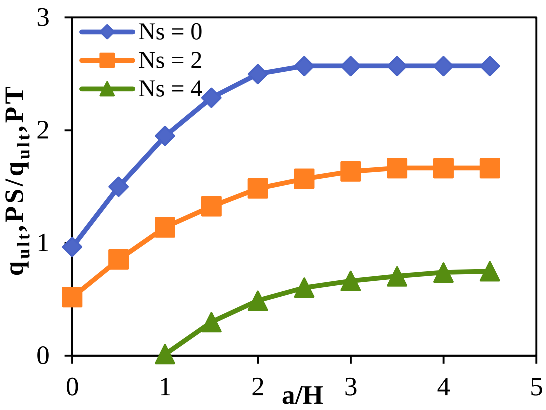

Fig. 3 Variation of N_{cq} with footing geometry and the stability factor.

For example, Pham [12] conducted a series of plane strain load tests on 1.94-m-high GRS that can be used to derive a cohesion value for the GRS. Two of the tests (GSGC2 and 5 in Table 1) were identical in every respect (Tensile strength of reinforcement = 70 kN/m, Reinforcement spacing = 0.2 m, Backfill c = 70 kPa and ϕ = 50°) except for the confining stresses (0 in GSGC5 and 34 kPa in GSGC2). The corresponding failure stresses were 2032 and 3396 kPa for the 0 and 34 kPa confining stresses, respectively. The resulting shear strength parameters for the GRS are c_{GRS} = 160 kPa and ϕ_{GRS} = 72°. The corresponding stability factor $\gamma H/c \approx 0.29$. Based on this stability factor, the ratio of plane strain capacity for a typical GRS abutment with a typical set-back a = 0.2 m and H varying from 3 m to 10 m (i.e. a/H = 0.02 to 0.07) to column (PT) capacity is close to unity. Therefore, the column PT is fairly representative of an in-service PS condition for well-graded gravels in this case.

4. LOAD TESTS

Eleven GRS load tests performed at FHWA's TFHRC (designated as "TF") with and without cast masonry unit or CMU facing that was frictionally connected to the geosynthetic are reported in Table 1.

Table 1 Test parameters of GRS performance tests selected from literature and available studies

Test	d_{max} m	USCS Symbol	ϕ_{peak} °	c_{peak} kPa	ϕ_{fs} °	c_{fs} kPa	Strength Test Type	T_f kN/m	S_v m	Facing Type	σ_h kPa	Boundary Conditions	$q_{ult,emp}$ kPa	ε_f[1] %	Reference
GSGC2[2]	0.0330	GS-GM	50	70	41	118	TX[3]	70	0.2	[4]	34	Plane Strain	3396	6.5	[12]
GSGC3[2]	0.0330	GS-GM	50	70	41	118	TX[3]	140	0.4	[4]	34	Plane Strain	2038	6.1	[12]
GSGC4[2]	0.0330	GS-GM	50	70	41	118	TX[3]	70	0.4	[4]	34	Plane Strain	1783	4.0	[12]
GSGC5[2]	0.0330	GS-GM	50	70	41	118	TX[3]	70	0.2	None	0	Plane Strain	2032	6.0	[12]
Elton1	0.0127	SP	40	28	41	27	DS[5]	9	0.15	None	0	Cylindrical Column	230	1.7	[18]
Elton2	0.0127	SP	40	28	41	27	DS[5]	9	0.3	None	0	Cylindrical Column	129	3.1	[18]
Elton3	0.0127	SP	40	28	41	27	DS[5]	14	0.2	None	0	Cylindrical Column	306	3.9	[18]
Elton4	0.0127	SP	40	28	41	27	DS[5]	15	0.2	None	0	Cylindrical Column	292	4.5	[18]
Elton5	0.0127	SP	40	28	41	27	DS[5]	19	0.2	None	0	Cylindrical Column	402	4.7	[18]
Elton6	0.0127	SP	40	28	41	27	DS[5]	20	0.2	None	0	Cylindrical Column	397	7.7	[18]
Elton7	0.0127	SP	40	28	41	27	DS[5]	25	0.2	None	0	Cylindrical Column	459	8.5	[18]
VS-1[6]	0.0127	GP	54	23	51	0	LSDS[7]	70	0.2	CMU	1.82	Square Column	1116	8.0	[20]
VS-2[6]	0.0191	GP	46	19	45	0	LSDS[7]	70	0.2	CMU	1.82	Square Column	1087	7.1	[20]
VS-5[6]	0.0127	GP	51	0	51	0	LSDS[7]	70	0.2	CMU	1.82	Square Column	1031	10.4	[20]
MPA[8]	0.0254	GW-GM	54	75	53	0	LSDS[7]	70	0.6	None	0	Square Column	225	1.9	[17]
MPB[8]	0.0254	GW-GM	54	75	53	0	LSDS[7]	70	0.4	None	0	Square Column	170	2.2	[17]
MPC[8]	0.0254	GW-GM	54	75	53	0	LSDS[7]	20	0.2	None	0	Square Column	460	6.4	[17]
TF-1	0.0127	GP	53	69	55	0	LSDS[7]	35	0.2	CMU	1.82	Square Column	981	10.9	[20]
TF-2	0.0254	GW-GM	54	75[9]	53	0	LSDS[7]	35	0.2	CMU	1.82	Square Column	1209	11.5	[20]
TF-3	0.0254	GW-GM	54	75[9]	53	0	LSDS[7]	35	0.2	None	0	Square Column	837	13.8	[20]
TF-6	0.0254	GW-GM	54	75[9]	53	0	LSDS[7]	70	0.2	CMU	1.82	Square Column	2095	15.7	[20]
TF-7	0.0254	GW-GM	54	75[9]	53	0	LSDS[7]	70	0.2	None	0	Square Column	1271	12.5	[20]
TF-9	0.0254	GW-GM	54	75[9]	53	0	LSDS[7]	70	0.4	CMU	0.91	Square Column	1068	15.6	[20]
TF-10	0.0254	GW-GM	54	75[9]	53	0	LSDS[7]	70	0.4	None	0	Square Column	494	14.3	[20]
TF-11	0.0254	GW-GM	54	75[9]	53	0	LSDS[7]	20	0.1	None	0	Square Column	1113	12.8	[20]
TF-12	0.0254	GW-GM	54	75[9]	53	0	LSDS[7]	20	0.1	CMU	1.82	Square Column	1390	13.4	[20]
TF-13	0.0254	GW-GM	54	75[9]	53	0	LSDS[7]	53	0.3	None	0	Square Column	620	12.3	[20]
TF-14	0.0254	GW-GM	54	75[9]	53	0	LSDS[7]	53	0.3	CMU	2.73	Square Column	1128	12.7	[20]

Notes:
1. ε_f = Strain of GRS load tests at failure
2. GSGC = Generic Soil-Geosynthetic Composite
3. TX = Consolidated drained triaxial compression tests on 0.15-m-diameter and 0.3-m-high samples
4. No facing was used. Instead, a confining pressure = 34 kPa was applied using a rubber membrane wrapped all around the GRS
5. DS = direct shear test. Because soil was SP, direct shear sample was 0.063 m diameter performed in accordance with ASTM D3080
6. VS = Performance tests conducted in Defiance County, OH as part of the FHWA's Every Day Counts GRS Validation Sessions
7. LSDS = Large Scale Direct Shear tests on 0.3 m by 0.3 m by 0.2 m high specimen
8. MP = Mini pier tests or more widely referred to herein as performance tests
9. Best fit linear Mohr-Coulomb envelopes for soil used in TF-2 through TF-14 yielded values of cohesion of 75 kPa and 6 kPa, respectively for the partially saturated and saturated samples. A cohesion of 75 kPa was used when estimating the GRS capacity since soil was partially saturated during load testing in FHWA's TFHRC laboratory.

Each CMU block was 0.194-m-high x 0.397-m-long x 0.194-m-wide (Fig. 4a).

A schematic of tests TF-1, 2, 6, 9 and 12, each 10 blocks high, is shown in Figs. 4b, 4c, 5a, and 5b. Tests TF-4, 7, 10 and 11 have a similar set-up except the blocks were removed prior to testing. TF-14 (Fig. 5c) has the same area in plan but was slightly taller (H = 2.0 m) with 7 pairs of full- and half-height blocks. TF-13 is identical to 14 except the blocks were removed. Details of the geotextile and soil utilized are summarized in Table 1 along with other tests collected from the literature giving a total of 28 load tests in this database.

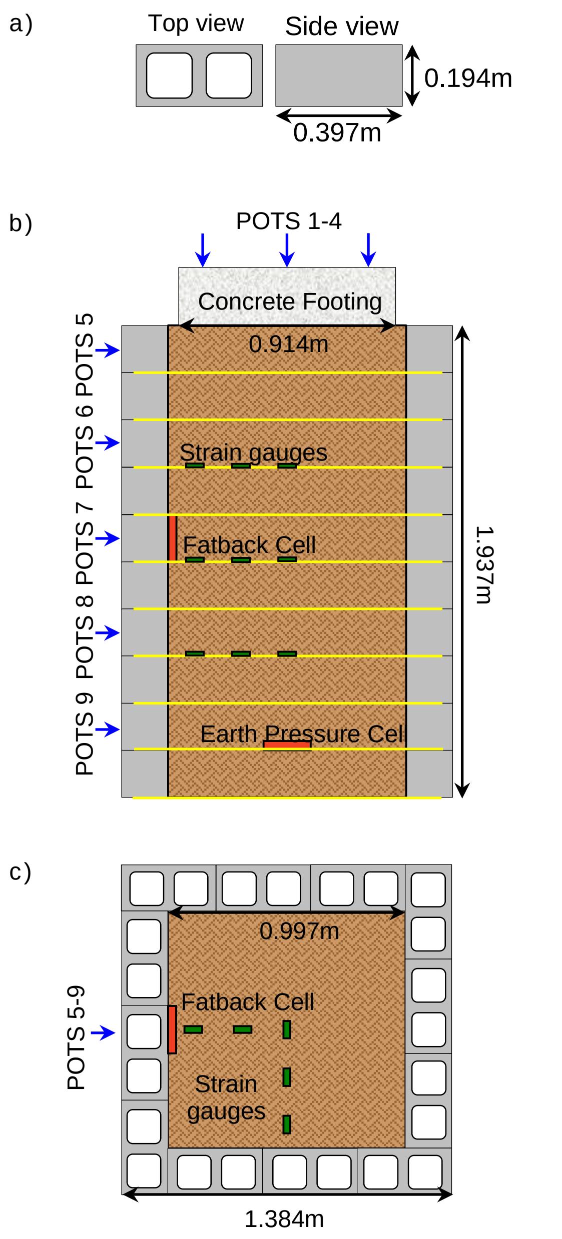

Fig. 4 (a) CMU dimensions; (b) and (c) schematic of TF 1, 2, 6, 9, and 12

Fig. 5 (a) Photo of mini-pier with CMU; (b) photo of TF-14; (c) photo of mini-pier without CMU.

5. RESULTS

The ultimate bearing capacities of the 28 load tests were predicted using Eq. 1 and both the backfill's peak and fully softened shear strength parameters.

Table 2 contains the predicted capacities ($q_{ult,peak}$ and $q_{ult,fs}$) along with the measured capacities ($q_{ult,emp}$). Also shown in Table 2 are the bias, λ, defined as the ratio of the measured to predicted capacities. The mean, standard deviation and coefficient of variation (COV) of the bias are shown at the bottom of Table 2.

The mean bias using peak strengths was 0.79 with a COV of 36%. Fig. 6a contains the corresponding plot of the histogram and the probability density function (PDF) of the normal distribution of the bias while Fig. 6b shows a plot of the predicted versus measured capacities. Clearly, Eq. 1 over-predicts the GRS capacity when using peak strengths.

Table 2 Predicted ultimate bearing capacities of footings on GRS using peak and fully softened strengths

Test	$q_{ult,emp}$ kPa	$q_{ult,peak}$ kPa	$q_{ult,fs}$ kPa	Bias Peak	Bias FS
GSGC2	3396	2511	1885	1.35	1.80
GSGC3	2038	1957	1529	1.04	1.33
GSGC4	1783	1294	1102	1.38	1.62
GSGC5	2032	2260	1723	0.90	1.18
Elton1	230	250	252	0.92	0.91
Elton2	129	151	150	0.86	0.86
Elton3	306	325	329	0.94	0.93
Elton4	292	333	337	0.88	0.87
Elton5	402	393	398	1.02	1.01
Elton6	397	416	422	0.95	0.94
Elton7	459	486	494	0.94	0.93
VS-1	1116	1587	1159	0.70	0.96
VS-2	1087	1329	1155	0.82	0.94
VS-5	1031	1159	1159	0.89	0.89
MPA	225	717	259	0.31	0.87
MPB	170	1085	620	0.16	0.27
MPC	460	1070	606	0.43	0.76
TF-1	981	1080	734	0.91	1.34
TF-2	1209	1528	1055	0.79	1.15
TF-3	837	1511	1038	0.55	0.81
TF-6	2095	2585	2093	0.81	1.00
TF-7	1271	2568	2077	0.49	0.61
TF-9	1068	1134	668	0.94	1.60
TF-10	494	1126	660	0.44	0.75
TF-11	1113	2001	1520	0.56	0.73
TF-12	1390	2018	1536	0.69	0.90
TF-13	620	1296	828	0.48	0.75
TF-14	1128	1322	852	0.85	1.32
			Mean Bias	0.79	1.00
			Standard Deviation	0.28	0.32
			Coefficient of Variance (%)	0.36	0.32

In contrast, the mean bias using fully softened strengths was 1.00 with a coefficient of variation (COV) of 32%. Fig. 7a contains the corresponding plot of the histogram and the probability density function (PDF) of the normal distribution of the bias while Fig. 7b shows a plot of the predicted versus measured capacities where the data points are more centered around the line of equality. Based on these results, Eq. 1 along with the use of fully softened strengths yield a bias that is close to unity with a slightly smaller COV compared to the use of peak strengths for this dataset.

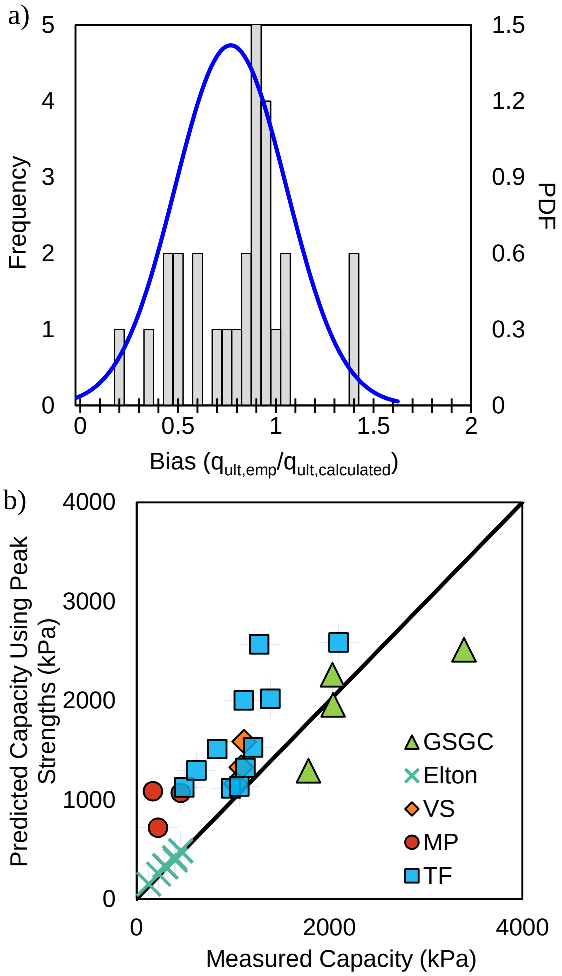

Fig. 6 (a) Histogram and normal distribution of bias using peak strengths; (b) predicted versus measured capacities using peak strength

Hypothesis testing on the normal distribution of the bias indicate that using Eq. 1 and fully softened strengths will result in a mean bias of 1.00, with a 90% confidence that the bias will be within 3 standard deviations of the mean.

6. SUMMARY AND CONCLUSIONS

The current state of practice is to use the peak strength of the backfill material, as suggested by Christopher et al. [13]. However, the results of this study provide evidence that the fully softened strengths are more appropriate for estimating footing capacities on GRS. Prediction of GRS bearing capacity improved by using fully softened versus peak strengths obtained from large scale direct shear and triaxial tests. Mean bias between measured and predicted values increased from 0.79 to 1.00 and the COV improved from 36% to 32%.

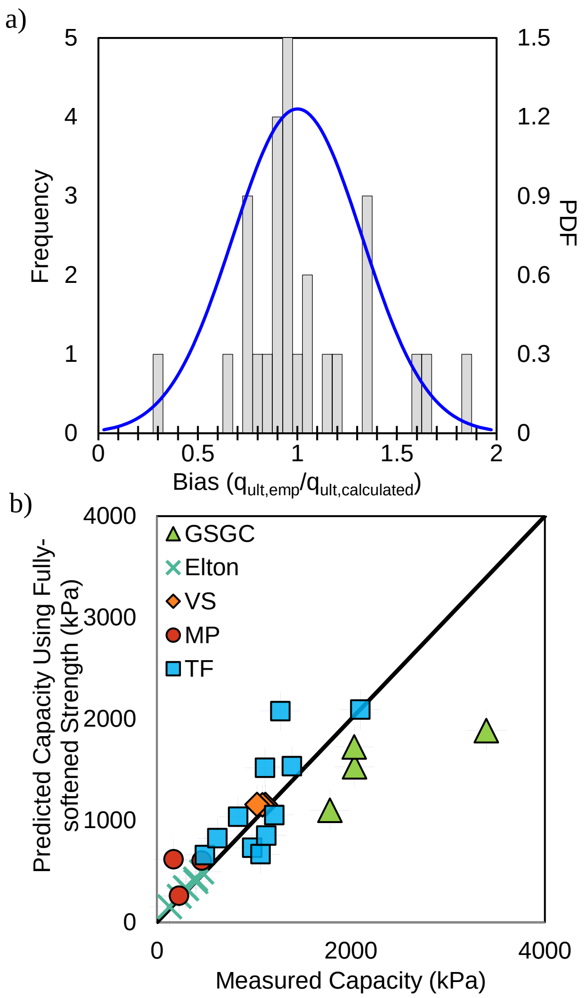

Fig. 7 (a) Histogram and normal distribution of bias using fully softened strength; (b) predicted versus measured capacities using fully softened strength

Fully softened strengths are more suitable for bearing capacity predictions because GRS with closely spaced reinforcement generally fail at large strains past the backfill's peak strengths. A follow-on to this is that since large movements are required to fail say a GRS abutment, the design of GRS abutments will most likely be governed by the serviceability limit state rather than the ultimate limit state.

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