Uncertainty in shallow foundations settlement analysis and its utilization in SLS design specifications

L'incertitude de l'analyse de tassement des fondations superficielles et son utilisation pour les spécifications de conception SLS

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ABSTRACT

Design of shallow foundations on soils is most often governed by settlement or Service Limit State (SLS) rather than the bearing capacity restrictions, i.e. Ultimate Limit State (ULS). In an effort to develop the SLS design specifications under the National Cooperative Highway Research Program NCHRP project 12-66 (AASHTO LRFD specifications for the serviceability in the design of bridge foundation), a large database of case histories of shallow foundation load tests has been assembled in MS ACCESS platform. Out of the 329 case histories compiled, only those related to granular materials and which include Standard Penetration Test (SPT) results have been utilized. These cases, being most relevant to bridge foundation design (soil type and site investigation), have been used for analyzing the uncertainty in the relationship between the measured to calculated loads for given displacements.

RÉSUMÉ

La conception de fondations superficielles sur les sols est le plus souvent régie par le tassement ou l'État Limite de Service (SLS), plutôt que par la capacité portante ou l'État Limite Ultime (ULS). Dans un effort visant à développer les spécifications de conception SLS, dans le cadre du National Cooperative Highway Research Program NCHRP projet12-66 (AASHTO LRFD spécifications de service pour la conception des fondations de ponts), une grande base de données d'essais-de-charge sur fondations superficielles a été assemblée en plate-forme MS ACCESS. Parmi les 329 cas compilés, seulement ceux qui sont liés à des matériaux granulaires et qui comprennent des résultats du Standard Penetration Test (SPT) ont été utilisés. Ces cas, les plus pertinents pour la conception des fondations de ponts (type de sols et d'explorations), ont été utilisés pour l'analyse de l'incertitude dans la relation entre les charges mesurées et les charges calculées pour des déplacements donnés.

Keywords : shallow foundations, settlement, service limit state, uncertainty evaluation, design specifications, LRFD

1 INTRODUCTION

Design of shallow foundation in/on soils is most often governed by a Service Limit State (SLS) expressed as a settlement or an angular distortion rather than Ultimate Limit State (ULS) i.e. strength-bearing capacity restrictions. An effort to develop the SLS AASHTO design specifications was carried out under the National Cooperative Highway Research Program (NCHRP project 12-66) entitled "AASHTO LRFD specifications for the serviceability in the design of bridge foundation".

Five methods of settlement estimation have been compared in this paper. While traditionally the performance of a method is expressed as settlement measured over settlement calculated, the uncertainty for each method in this study follows the design process, i.e. expressed in terms of a bias, defined as the ratio of the measured required load to the estimated required load to produce a given settlement magnitude. Defining so, the bias incorporates wide sources of uncertainties, such as those arising from, but not limited to, the soil parameter interpretation, site uncertainty, model formulation, etc.

2 DATABASE AND UNCERTAINTY EVALUATION OF SETTLEMENT ANALYSIS

2.1 *Database*

A large database, UML-GTR ShalFound06, of case histories of shallow foundations load tests has been assembled for the evaluation of uncertainties in the estimated loads for given displacements. A total of 329 load tests cases on various footing sizes has been compiled with a majority being plate load tests on sand. A breakdown summary of the database by foundation type, test site and soil type is presented in Table 1. The foundation type has been categorized after Lutenegger and DeGroot (1995).

Table 1. Summary of database UML-GTR ShalFound06 compiled for the evaluation of uncertainty in settlement analysis

Foundation type	Soil type					Total	Region		
	Sand	Gravel	Cohesive	Mixed	Unknown		USA	World	Unknown
Plate load tests (B ≤ 1.0m)	102	40	--	31	52	225	18	178	29
Small footings (1.0 < B ≤ 3.0m)	15	--	--	39	--	54	17	20	17
Large footings (3.0 < B ≤ 6.0m)	15	--	--	16	--	31	25	5	1
Rafts and mats (B > 6.0m)	9	--	--	10	--	19	9	9	1
Total	141	40	--	96	52	329	69	212	48

Note: "Mixed" refers to cases with alternating layers of sand and gravel or clay and silt.

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2.2 *Soil parameter estimation*

Soil parameters which are missing in the database, have been estimated from their correlations with the SPT blow counts. The correlation proposed by Paikowsky et al. (2005) has been used to estimate the soil unit weight (Equation 1).

$$\gamma = 0.138N1_{60} + 15.54 \quad (\text{kN/m}^3) \quad \text{for } \gamma \le 22.9\text{kN/m}^3 \tag{1}$$

where the corrected values of N_{60} for overburden, $N1_{60}$, have been obtained based on the proposal by Liao and Whitman (1986):

$$N1_{60} = \sqrt{\frac{p_a}{\sigma'_v}} \cdot N_{60} \tag{2}$$

where p_a is the atmospheric pressure (≈100kPa or 1tsf) and σ'_v is the effective overburden pressure in the same unit as that of the atmospheric pressure.

Further, the soil Young's modulus of elasticity has been estimated using Equations 3a and 3b (NAVFAC 1982), which are also specified in AASTHO (2007).

For fine to medium sand and slightly silty sands:

$$E_s = 7N1_{60} \times 95.76 \quad (\text{kPa}) \tag{3a}$$

For coarse sand and sands with little gravel:

$$E_s = 10N1_{60} \times 95.76 \quad (\text{kPa}) \tag{3b}$$

The corrected SPT values $N1_{60}$ have been calculated at the mid-height of each layer.

2.3 *Settlement analysis*

2.3.1 *Overview*

Five settlement analysis methods have been used to estimate the loads required to produce given settlements within the range established for service limits of bridge foundations (Paikowsky and Lu 2006). The specific settlements for which the loads were calculated for are: 0.25, 0.50, 0.75, 1.00, 1.25, 1.50, 1.75, 2.00, 2.50 and 3.00inches (about 6.5, 12.5, 19.0, 25.5, 32.0, 38.0, 44.5, 50.0, 63.5 and 76.0mm). A brief introduction of each method is provided below, while in the analyses the methods are reversed to calculate a load for a given settlement.

2.3.2 *AASHTO (2007) method*

Based on the elastic half-space method, the AASHTO (2007) specifications uses Equation 4 to estimate the settlement S_e of a footing of base area A resting on granular soil(s) with Poisson's ratio ν and modulus of elasticity E_s.

$$S_e = \frac{q(1-\nu^2)\sqrt{A}}{E_s \beta_z} \tag{4}$$

where q is the applied vertical stress on the footing and β_z is the elastic shape and rigidity factor ranging from 1.04 to 1.41 depending on the rigidity and the shape of the footing (Kulhawy et al. 1983, AASHTO 2007). The Poisson's ratio has been taken as 0.30 for granular soils, and the Young's modulus have been estimated using Equations 3a and 3b. The elastic parameter of the soil below the footing base have been taken as the weighted average of each soil layer up to the influence depth. The influence depth has been taken as 2B for the footings with L/B ratio from 1 to 5, 3B for those with 5 to 10 and 4B for those with more than 10 inclusive, where B is the footing width.

2.3.3 *Hough (1959) method*

The total elastic settlement S_e is calculated as the sum of the settlement in each soil layer below the footing base, ΔH, i.e.

$S_e = \sum_{i=1}^{n} \Delta H_i$ where n is the number of layers present to the influence depth and the settlement of each layer is given by

$$\Delta H = \frac{1}{C'} \Delta z \log\left(\frac{\sigma'_{v0} - \Delta\sigma'_v}{\sigma'_{v0}}\right) \tag{5}$$

in which C' is bearing capacity index obtained using correlations with the corrected SPT values depending on the soil type (Cheney and Chassie 2000), Δz is the initial soil layer thickness, σ'_{v0} is the initial effective stress and $\Delta\sigma'_v$ is the change in effective vertical stress, both at the mid-height of the layer. The stress below the footing due to loading has been calculated using the 2:1 method, which approximates the stress at a point z below the footing base of area $B \cdot L$ as the load applied over an increased area $(B + z)\cdot(L + z)$. The influence depth has been taken as the one defined in the AASHTO (2007) method.

2.3.4 *Schmertmann (1970* and *1978) methods*

The settlement of a footing subjected to an applied footing pressure of q is estimated as:

$$S_e = C_1 C_2 \Delta q \sum_{i=1}^{n} \left(\frac{I_z}{E_s} z\right)_i \tag{6}$$

where C_1 is the depth correction factor = 1.0 – 0.5 ($q_0/\Delta q$) and always ≥ 0.5, C_2 is the creep correction factor (taken as 1.0 in the present study), z is the layer thickness and E_s the Young's modulus of elasticity of each of a total n number of soil layers considered (here, estimated using Equations 3a and 3b), and I_z is the strain influence factor. The Schmertmann (1970) and Schmertmann et al. (1978) methods differ in the way I_z are defined. Schmertmann (1970) method assumes the maximum value of I_z is 0.6 and occurs at a depth of 0.5B below the footing base irrespective of the footing shape, while Schmertmann et al. (1978) assumes that the depth at which the maximum value of I_z occurs varies between 0.5B to 1.0B depending on the footing L/B ratio. A detail is listed in Table 2.

For computational simplicity and automation, the soil layer to the influence zone depth has been divided into six layers, three above the depth where the strain influence factor reaches the maximum value (I_{zp}) and three below the peak at the deeper soil strata. The soil strata which have the most influence are the ones immediately below the footing base. Dividing the soil strata into six layers as described above, adequately considers the variability of soil parameters (in spite of its simplicity), thereby their influences on the foundation settlement. The soil parameter at the mid-layer depth of each stratum has been taken as the weighted average of each sub-divided stratum.

Table 2. Values of strain influence factor I_z at depths below footing base

Schmertmann (1970)		Schmertmann et al. (1978)		
I_z	Depth	Footing shape	I_z	Depth
0.1	0.0	L/B = 1	0.1	0.0
0.6	0.5B		I_{zp}	0.5B
0.0	2.0B		0.0	2.0B
		L/B = 10	0.2	0.0
			I_{zp}	1.0B
			0.0	4.0B
For all footing shapes		For other footing shapes, these values are interpolated		
$I_{zp} = 0.5 + 0.1\sqrt{\frac{\Delta q}{\sigma'_{vp}}}$		where, Δq = net applied pressure = $q - q_0$; q = applied footing stress; q_0 = effective stress at footing depth; σ'_{vp} = initial effective vertical pressure at the depth where I_{zp} occurs		

2.3.5 *D'Appolonia (1970) method*

D'Appolonia et al. (1970) suggested Equation 7 to estimate the settlement based on the elastic solution.

$$S_e = \frac{qBI}{M} \tag{7}$$

where q is the applied stress on the footing of width B. The influence factor I is a product of two factors μ_0 and μ_1 (given by Christian and Carrier 1978), which are correction factors for the footing embedment depth and the depth of incompressible layer, respectively; M is the modulus of compressibility. The uncorrected SPT N_{60} required to determine M has been taken as the weighted average of SPT counts of the soil layers below the footing base up to the influence depth (taken as that defined for AASHTO (2007) method).

3 RELIABILITY ANALYSIS AND RESISTANCE FACTORS

3.1 *Uncertainty evaluation*

Out of the 141 shallow foundations tested in sand (Table 1), only those including SPT results have been analyzed. The specific number of cases analyzed was subjected to the information required for each method and available for each case study (e.g. soil information to the influence depth etc). The total number of cases which could be analyzed therefore varied from 74 to 85 for a settlement of 0.25inch per method with the number of cases decreasing with the increase in the settlement magnitude. Table 3 lists the number of cases analyzed for a settlement magnitude per method of analysis.

The statistical summary in the form of the mean bias (expressed as the ratio of measured to calculated loads for a given settlement) and coefficient of variation of the bias is presented in Figures 1 and 2, as a function of the settlement magnitude, respectively. Observing the trends of the statistical parameters in the figures suggest that the ratio of the measured load to the calculated load reduces as the settlement increases while the COV remains roughly constant (in the central zone of 1 to 2inch settlement) as a result of the fact that the standard deviation roughly follows the change in the mean. The COV mainly changes as the number of cases decreases to a small set of single digit data cases. The reduction in the mean bias is most pronounced for Hough method, then for AASHTO method, whereas, the biases for Schmertmann methods (both 1970 and 1978) are found to be the least affected by the magnitude of the settlement. Hough method is found to be the most conservative for settlements less than 1.25in (32.0mm), while D'Appolonia method over-predicts for settlements greater than 0.50in (12.7mm). Schmertmann (1970) method over-predicts while Schmertmann et al. (1978) method under-predicts the required load for all the settlements but both remain with a relatively consistent bias for the entire settlement range. The COV of the bias, shown in Figure 2, fluctuates between about 0.50 to 0.75 for settlements up to 2.0in (50.0mm). An increasing trend for D'Appolonia method can be observed.

Table 3. Number of cases evaluated for given settlements

S_e		Number of shallow foundation cases				
inch	mm	AASHTO	Hough	Schm70	Schm78	D'App
0.25	6.5	85	80	81	81	74
0.50	12.5	51	49	46	46	52
0.75	19.0	36	34	32	32	40
1.00	25.5	18	16	13	14	22
1.25	32.0	18	16	13	14	22
1.50	38.0	17	15	12	13	21
1.75	44.5	14	12	9	10	19
2.00	50.0	13	12	9	10	18
2.50	63.5	7	7	5	6	14
3.00	76.0	6	6	4	5	11

Note: Schm70 and Schm78 are Schmertmann 1970 and Schmertmann et al. 1978 methods and D'App is D'Appolonia method

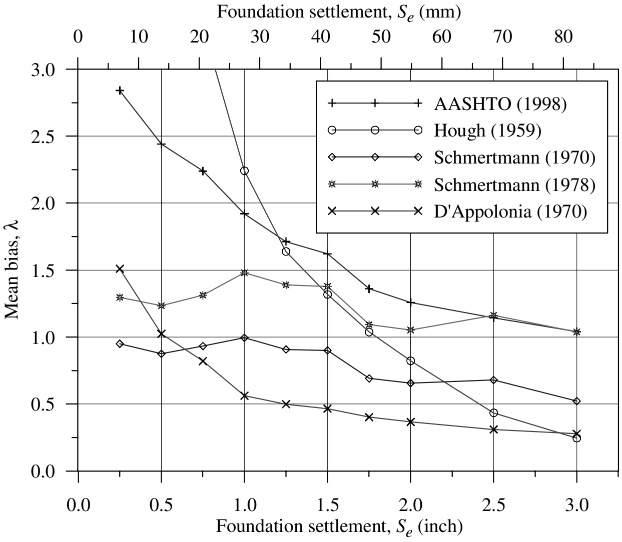

Figure 1. Mean bias versus settlement for all analyzed settlement methods

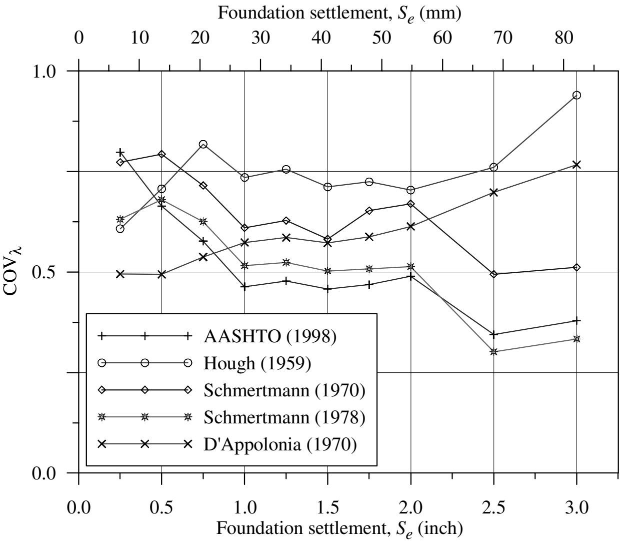

Figure 2. COV of the mean bias vs. settlement for all analyzed settlement methods

3.2 *Resistance factor calibration*

In the present study, the resistance factors have been calibrated for a target probability of exceedance p_f of 10% (reliability index of 1.28), which is inline with other SLS guidelines, using load factors of 1.0. A live load to dead load ratio of 2.0 has been considered and resistance factors are developed based on Monte Carlo simulation of 10,000 samples, assuming the distributions followed by loads and resistance are log-normal. The bias and COV, expressed as (λ, COV_λ), of the dead load and the live load have been taken as (1.05, 0.10) and (1.15, 0.20), respectively following the calibrations presented by Paikowsky et al. (2004) based on distributions suggested by Nowak (1999).

The biases obtained in this study are biases in the estimated required loads to produce a given settlement. Hence, the application of the resistance factors to the force required to

Table 4. Recommended resistance and efficiency factors for SLS of shallow foundations in/on granular soils

Method		Range of Settlement S_e (inch)	Resistance Factor ϕ	Efficiency Factor ϕ/λ	
AASHTO		$0.00 < S_e \leq 1.00$	0.85	0.34	
		$1.00 < S_e \leq 1.50$	0.80	0.48	
		$1.50 < S_e \leq 3.00$	0.60	0.48	
Hough		$0.75 < S_e \leq 3.00$	$\phi=2.5e^{(-1.2\ Se)}$	$S_e = 0.75$	0.29
				$1.0 \leq S_e \leq 2.5$	0.32
				$S_e = 3.0$	0.26
Schmertmann	1978	$0.00 < S_e \leq 3.00$	0.50	0.39	
	1970	$0.00 < S_e \leq 3.00$	0.30	0.32	
D'Appolonia		$0.25 \leq S_e \leq 3.00$	$\phi=0.25\ S_e^{(-0.85)}$ where $\phi \leq 0.7$	$0.25 < S_e < 0.50$	0.45
				$0.75 < S_e < 2.00$	0.40
				$2.00 < S_e < 3.00$	0.35

develop a given settlement, using a given estimations method, will not exceed the prescribed settlement in more than 10% of all the cases.

The recommended resistance factors ϕ for the different settlement estimation methods are given in Table 4. Two of the methods, Hough and D'Appolonia, resulted in a steep variation in the resistance factor with a change in settlement. For these methods, the resistance factors may be obtained using the equation tabulated in Table 4.

4 CONCLUSIONS

A comprehensive database has been utilized to develop and recommend the resistance factors for service limit states of shallow foundations on/in granular soils for five methods of settlement analysis. The soil parameters required have been estimated from their correlations with SPT values, and weighted average values of soil layers taken when necessary.

It has been found that the mean of the bias (measured over calculated loads required to develop a prescribed settlement) and its COV obtained for Schmertmann et al. (1978) method shows the least fluctuation, whereas, the mean bias of AASHTO (2007) and Hough (1959) methods show a drastic decreasing trend with increasing settlement magnitude. This settlement prediction variation is most likely associated with, one, the non-linearity of the soil response, hence a variation of the modulus with the displacement takes place, and two, the relative measurement errors for the smaller settlements being expectedly higher. The recommended resistance factors are given in Table 4.

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REFERENCES

AASHTO. 2007. Section 10: Foundations, *LRFD Bridge Design Specifications*, American Association of State Highway and Transportation Officials, Washington, D.C.

Cheney, R. and Chassie, R. 2000. *Soils and Foundations Workshop Reference Manual*, NHI-00-045. National Highway Institute, Federal Highway Administration, U.S. Dept. of Transportation, Washington, D.C.

Christian, J.T. and Carrier, W.D. III. 1978. Janbu, Bjerrum and Kjaernsli's Chart Reinterpreted, *Canadian Geotechnical Journal*, Vol.15, No.1, pp.123-128

D'Appolonia, D.J., D'Appolonia, E. and Brissette, R.F. 1970. Settlement of spread footings on sand, Closure, *Journal of the Soil Mechanics and Foundations Div.*, ASCE, Vol.96, No.SM2, pp.754-762

Hough, B.K. 1959. Compressibility as the basis for soil bearing value, *Journal of the Soil Mechanics and Foundations Div.*, ASCE, Vol.85, No.SM4, pp.11-39

Kulhawy, F.H., Trautmann, C.H., Beech, J.F., O'Rourke, T.D., McGuire, W., Wood, W.A., and Capano, C. 1983. *Transmission Line Structure Foundations for Uplift Compression Loading*, Report EL-2870, EPRI, Palo Alto, California

Liao, S.S.C., and Whitman, R.V. 1986. Overburden Correction Factors for SPT in Sand, *Journal of Geotechnical Engineering*, ASCE, Vol.112, No.3, pp.373-377

Lutenegger, A.J. and DeGroot, D.J. 1995. *Settlement of shallow foundations on granular soils*, Report UMTC-94-4, Univ. of Massachusetts Transportation Center, Amherst, Mass., USA (for Massachusetts Highway Department, Transportation research project Contract #6332 Task #4).

NAVFAC. 1982. DM7.1, *Soil Mechanics*, U.S. Department of the Navy, Naval facilities engineering command, Virginia.

Nowak, A. 1999. *Calibration of LRFD Bridge Design Code*, NCHRP Report 368, Transportation Research Board of the National Academies, Washington, D.C.

Paikowsky, S.G., Birgisson, B., McVa, M., Nguyen, T., Kuo, C., Baecher, G., Ayyub, B., Stenersen, K., O'Malley, K., Chernauskas, L. and O'Neill, M. 2004. *Load and Resistance Factor Design (LRFD) for Deep Foundations*, NCHRP Report 507, Transportation Research Board of the National Academies, Washington, D.C., 126p.

Paikowsky, S.G., Fu, Y. and Lu, Y. 2005. *LRFD foundation design implementation and specification development*, NCHRP Project 20-07, Task 183, National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Washington, D.C.

Paikowsky, S.G. and Lu, Y. 2006. Establishing Serviceability Limit State in the Design of Bridge Foundations, *Foundation Analysis and Design – Innovative Methods*, ASCE, GSP No.153, GeoShanghai International Conference, Shanghai, China, June 6-8, pp.49-58

Schmertmann, J.H. 1970. Static cone to compute static settlement over sand, *Journal of Soil Mechanics and Foundation Div.*, ASCE, Vol.96, No.SM3, pp.1011-1043

Schmertmann, J.H., Hartman, J.P. and Brown, P.R. 1978. Improved strain influence factor diagrams, *Journal of the Geotechnical Engineering Div.*, ASCE, Vol.104, No.GT8, pp.1131-1135