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The prediction of ground settlement from continuous surface wave data

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Abstract

The continuous surface wave (CSW) technique (Matthews et al., 1996, Menzies and Matthews, 1996) relies on the propagation properties of vertically polarised seismic surface waves, or Rayleigh waves. It measures seismic shear wave velocity and hence minimum-strain stiffness (G_{max}) as a function of depth. The method has been used to monitor the effectiveness of ground improvement on construction sites for the past eight years **(Figure 1)**.

Surface wave testing is non-intrusive and therefore better able than invasive methods to cope with the huge variation in grain size which occurs in overburden throughout the UK (Moxhay et al., 2001).

The need to develop a method of predicting foundation settlement from CSW stiffness / depth profiles is two-fold. First, as the CSW stiffness is minimum-strain (<0.001%) it can sometimes be difficult to relate to other measurements made at operational strain levels of between 0.01% and 0.1%. Second, predictions of settlement following ground improvement by mechanical means have, to date, been notoriously inaccurate and generally much higher than those observed to occur in practice. Over-estimation by an order of magnitude is quite common. So a more accurate method of predicting settlement is urgently needed to avoid unnecessary cost in the design and installation of foundations.

Engineers such as Burmister (1938) working in the middle of the last century recognised the advantages of basing settlement prediction on soil stiffness but had no means of taking measurements over the wide areas of ground required. This can now be accomplished by CSW surveying and a method using it has been fully developed and tested. This paper describes the method and discusses results obtained with particular reference to two sites at Dagenham (Essex) and Jarrow (Tyne and Wear). The conclusions drawn are that it is robust and reliable and produces settlements much closer numerically to the ones actually observed than those obtained by conventional calculation.

Figure 1: Continuous surface wave survey operation

Introduction

For some years, the idea of using ground improvement to produce a stiffened crust of soil capable of providing safe and continuous support for surface structures has interested both engineers and contractors. Generations of road and rail builders and housing developers in areas of deep, saturated and compressible soils like the Somerset Levels have long been using ground improvement techniques without realising it.

The cost of constructing potentially very long piles through good, responsive near-surface soils and soft, compressible material below has always been prohibitively high. Clients who point to old Victorian and Edwardian foundations, seemingly supported on nothing and yet still capable of sustaining serviceable buildings themselves, are frequently curious and frustrated when told that there is no alternative to very deep and expensive piling solutions.

Many engineers involved in ground improvement felt that it should be possible to provide the required degree of structural support by concentrating on the stiffening and verification of responsive near-surface soils. However, apart from carrying out static load tests on stanchion bases there has been no suitable and reliable means of measuring the actual stiffness generated.

In the absence of real long-term testing for the area of major benefit, the ground bearing slab, only the very confident would proceed and then at the cost of battling with established opinion.

Criticism against the creation and use of soil rafts is mainly directed towards settlement. How could a 3m to 4m layer of improved ground overlying 5m or more of soft, saturated, compressible soil ever hope to stay in one place? If immediate or consolidation loads produced excessive settlement, then the sustained application of load seemed certain to instigate an ongoing settlement condition which would lead to either a continuous pattern of building maintenance or ultimate catastrophe in terms of structural failure.

Such criticism is based on the ever relied-upon settlement calculation methods of Boussinesq (ca. 1885), Fadum (1948) and others which provide an estimated net stress increase to apply to previous case history figures for the coefficient of volume compressibility (mv) or SPT 'N' value.

CSW surveying was first introduced in 1997 to measure soil stiffness resulting from the installation of vibro stone columns and dynamic compaction by surface tamping. At that time, the standard method of using the results was simply to visually compare the post-treatment data with that obtained beforehand. It soon became clear that stiffness generation was not only achievable in a wide range of soil profiles, but also improvement was progressive over time (Moxhay et al., 2001). The time taken is a function of energy applied, drainage paths available and soil matrix constituents.

Data has now been collected from over fifty studies of actual building and construction works, many of which involved soil raft creation. The question of how to make best use of this data has now become a priority.

It was obvious that attempts to assimilate stiffness generation measured in terms of G_{max} into existing soil mechanics doctrine for ground improvement and settlement analysis would not work. This is because measured settlements of treated ground compared with those from various forms of calculation carried out were significantly different.

Both short-term and long-term settlements observed were invariably far less than those predicted by traditional analysis. The usual reaction to this has been to assume that floor loadings are smaller in reality than those specified. However, this is a risky and unfair view as floors can just as easily be overloaded as under-loaded and frequently are. The reasons why the current methods of soil mechanics do not compare favourably with the results from accurate loads imposed in static tests are that soil sampling methods tend to be unreliable and the calculations used are often being incorrectly applied.

As a consequence of this it was thought that CSW testing and analysis should be developed to produce its own settlement relationships which would assist in the design confirmation required in relation to soil rafts.

Settlement methodology

CSW field measurements provide values of stiffness, G_{max}, which are related to very small strains well below those experienced in typical engineering situations. A computerised method has been developed which uses these values to calculate Young's Modulus, E, at the practical strain levels experienced in actual foundation conditions and so enables ground settlements to be predicted. Data required is:

(a) Values of the minimum-strain stiffness (or shear modulus) G_{max} meas ured in MPa (MN/m^2) versus depth (in m) from a CSW survey.
(b) Foundation size, shape and depth below the surface. The shape must be square, rectangular or circular.
(c) The applied load, q, in kPa (kN/m^2).

The calculation then proceeds as:

1. Average values of G_{max} in layers of a chosen thickness (usually taken to be 1m) starting from the base of the foundation, and continuing to a maximum depth of ideally twice the foundation width depending on the quality of the data, are calculated from the CSW results. Each value is allocated to a depth (z) which is the distance from the foundation base to the centre of the layer.

2. The initial value of E for each layer is taken to be $2.5G_{max}$ (average).

As $E = 2(1+\mu)G$ where μ is Poisson's Ratio (Craig, 1993) this means Poisson's Ratio is assumed to be 0.25.

3. The vertical stress at the centre of each layer, σ_z, is then calculated from the Boussinesq formula appropriate to the plate shape **(Appendix 1)**. Note that for a square or rectangular plate the dimensionless numbers m and n must first be calculated where m = long side of plate/z and n = short side of plate/z. This enables calculation of the influence factor $I_{\sigma z}$ which represents the fraction of the surface load produced at depth z.
   (a) For the corner of a square or rectangular plate, $\sigma_z = qI_{\sigma z}$.
   (b) For the centre of a square or rectangular plate the plate dimensions must be halved before calculating $I_{\sigma z}$ and $\sigma_z = 4qI_{\sigma z}$.
   (c) For a circular plate, σ_z is calculated directly as shown in **Appendix 1** (Craig, 1993).

In each case σ_z is then divided by 1000 to change units from kN/m^2 to N/mm^2.

4. An initial value for the strain in each layer is found by dividing the vertical stress by the initial value of Young's Modulus, E. This is multiplied by 100 to convert it to a percentage.

5. The strains just calculated will be too high to relate to the G_{max} values from the CSW results. The values of E must therefore be revised using a standard curve of stiffness against strain **(Appendix 2)**. A reduction factor of G/G_{max} (between 0 and 1) is read off for each strain value.
   It should be noted that the shape of the standard curve varies slightly with soil type. The one illustrated in **Appendix 2** is taken from Matthews et al. (1996) and has been tested on a number of sites involving different landfill materials, producing calculated settlements very close to those actually observed.

6. The initial values of Young's Modulus are then revised using these factors and the calculations repeated to produce new strains. If they are repeated several times the new values of E converge to the previous ones. This can typically be achieved in three to five iterations.

7. The final values of Young's Modulus and strain in each layer are recorded. The settlement in each layer can then be calculated by multiplying the final strain by the layer thickness. As the strain is a percentage and the layer thickness is in metres, a factor of 10 is introduced to give settlements in millimetres. Finally, the settlements in each layer are added to give the total settlement prediction.

An example of the final output is given in **Table 1**.

It should be noted that the theory of stress distribution which has been used assumes the soils under load are homogeneous and isotropic. This is not the case, in general, for the landfill materials which are often worked on, particularly if vibro stone columns or rock pillars have been inserted.

No account has been taken of the effects of load dispersal through stiffer, inhomogeneous materials and it is therefore accepted that a certain amount of inaccuracy is inherent in the method. The key issue is that G_{max} values are being used as the starting point for calculations rather than mv or SPT 'N' values. The settlement values resulting for sites that have been investigated tend to be between 100% and 150% of those actually observed, which is a much better agreement than can usually be achieved by conventional means.

A concern with iterative methods of calculation such as this is that there may be more than one route of convergence and hence a risk of non-exclusive results being arrived at. To investigate this possibility, back-calculations were

Z	E	Strain	Settlement
0.5	9.251075	0.427373	4.273725
1.5	19.19105	0.193017	1.930174
2.5	10.00865	0.312208	3.122075
3.5	9.868539	0.250448	2.504483
4.5	15.15825	0.127443	1.274432
5.5	24.70538	0.06107	0.610702
6.5	40.91931	0.029306	0.293059
			14.00865

Table 1: Example output from the settlement program

Z	E	Strain	Settlement
0.5	79.66467	0.049986	0.499863
1.5	74.28192	0.050017	0.500167
2.5	13.20374	0.236046	2.360455
3.5	49.0658	0.050616	0.506157
4.5	19.85208	0.097296	0.972964
5.5	31.9958	0.047149	0.471491
6.5	53.6301	0.022366	0.223661
			5.534758

Table 2: Results of convergence tests

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Figure 2: Ground improvement operation, Dagenham site

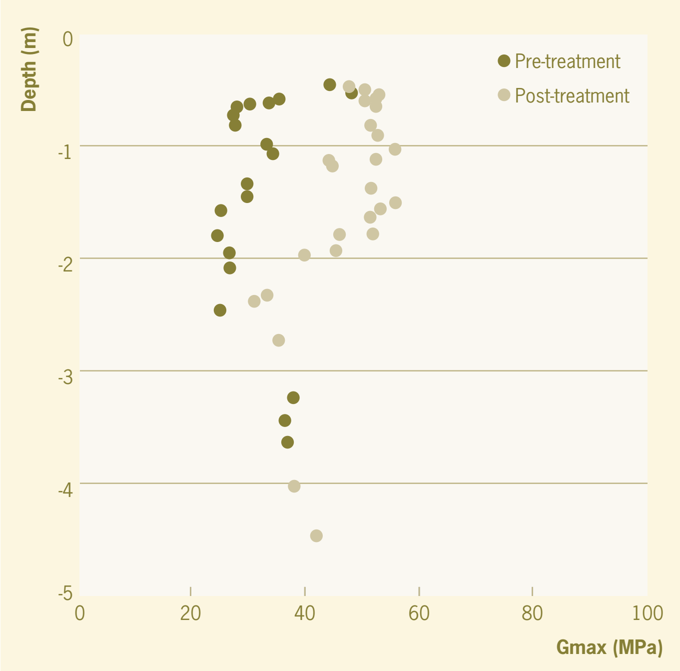


Figure 3: Typical CSW survey results, Dagenham site

carried out. This involved assuming a strain of 0.05%, calculating back to the value of E which would produce this strain and then inputting this value to the program to see if convergence did actually take place at 0.05%.

It was decided to test the first, second and fourth layers for a particular load situation. In each case the vertical stress at the centre of the layer was calculated manually from the surface stress using the Boussinesq formula. This value was divided by the strain to give the reduced value of E for the layer. This was then multiplied by G_{max}/G calculated from the standard curve (and found to be 1/0.2983) to give the initial value of E and finally divided by 2.5 to give the value of G_{max} for input to the program.

The percentage strain values resulting are shown in the first, second and fourth rows of **Table 2**. The first and second layers yield almost perfect results and the error for the fourth layer is only just over 1%. This indicates that an improper route of convergence is not an issue of major concern.

Of the construction schemes previously mentioned involving CSW testing, some are now well advanced in years and two of these have been selected as detailed examples.

Case histories

One: Factory refurbishment, Dagenham, Essex

In 2001 a factory was under refurbishment in Dagenham, Essex, south east England. Soil borings showed very deep deposits of weak, saturated silt with peat inclusions beneath a covering of clinker and ballast made ground.

Whereas the building's structure was in good order, the ground bearing floor slab of 2000m^2 had failed dramatically and could not even be walked on. Excavation to remove the old floor revealed a second buried slab that had failed in a similar catastrophic fashion. All of the old concrete waste was removed, crushed on site and replaced. Grids of vibro compacted stone columns were then installed in the upper silt deposits. Due to the limited headroom available under the existing steelwork of the building's roof (under 6m in places) a vibro lance had to be specially adapted and suspended from a mini crawler crane **(Figure 2)**.

CSW testing was carried out before and after the ground improvement and the results clearly illustrate the additional stiffness generated **(Figure 3)**.

Following completion of the ground improvement work, a relatively thin and lightly reinforced ground bearing slab was laid and the owner moved in with machinery.

The average observed settlement of the new floor slab after four years when measured against the main building's steelwork was 10mm compared with an originally calculated 60mm. New calculations were carried out using the algorithm for 6m x 6m sections of floor slab at six locations with the design load of 40kN/m^2. The slab centre result for the data illustrated in **Figure 3** is shown below:

The centre settlements predicted from the CSW data varied from 6mm to 15mm at the six locations with an average of 11mm.

Two: Warehouse construction, Jarrow, Tyne and Wear

Early in 2002 a scheme was developed to construct a steel framed high bay warehouse on a site in Jarrow, Tyne and Wear, north east England **(Figure 4)**.

The footprint area of the new building was just over 6500m^2. The principal difficulty with the foundations stemmed from the fact that part of the building was located over an old railway cutting where the existing fill material was

Figure 4: North side of warehouse, Jarrow site

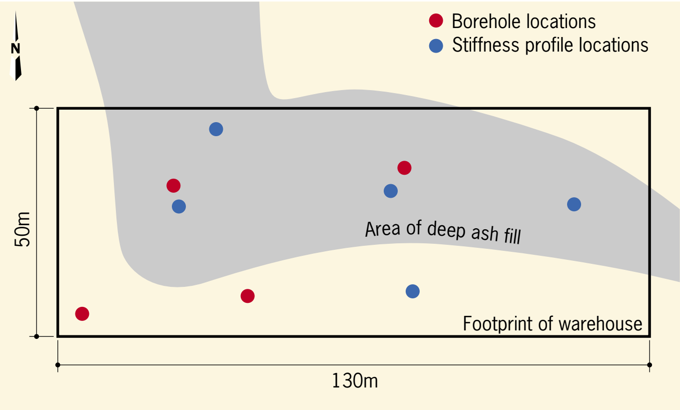


Figure 5: Warehouse footprint, Jarrow site

thought to be up to 12m thick in places. The approximate extent of this deep fill is shown in **Figure 5**.

The remainder of the building was to stand on natural, firm to stiff glacial clays having only a thin layer of fill covering. The thin surface fill, which was up to 3m thick and covered the entire site, was comprised of coarse, essentially granular material with brick, cobbles, pottery, sand and ash in a silty clay matrix.

Below this in the cutting, the deep fill was loose and ash-based with a moisture content varying from 17% to 30%. Although it had been in place for a number of years, trial pits dug as part of a site investigation had shown it to be in a generally poor state of compaction. As part of this investigation, four boreholes were drilled and their locations are shown in **Figure 5**.

The fill thicknesses at each of the four locations are summarised in **Table 3**.

In addition, some heavy dynamic probing was carried out in the region between boreholes 1 and 4. The results of this suggested that the ash fill may extend to as much as 12m below the surface in places, although this could not be substantiated by recovery of soil samples.

If the site was to be successfully prepared for construction using ground improvement, it was essential to have a testing method available which would ensure that comparable levels of stiffness had been achieved over both the deep ash fill and the natural ground so that significant differential settlement could be avoided. The variable nature, thickness and extent of the ash fill rendered predictions of likely settlement from established theory unreliable due to their dependence, in part, on accurate depth and density information. Static cone probing could possibly have been used, but accurate interpretation of the results would have been difficult because of all the site variations which would need to be taken into account. CSW testing before and after ground treatment was considered to be a much better option, as this would indicate whether or not a similarity of stiffness had been achieved regardless of the properties of the made ground.

Before the ground improvement contractor started work, further trial pits and trenches were dug to better establish the positions of the edges of the cutting. Vibro stone columns were then inserted in a regular grid across the entire building footprint to support the floor slab, with predetermined clusters beneath the differently sized foundation bases being used for the main structure.

Finally, the area of deep fill was given additional treatment in the form of controlled surface tamping. CSW profiles were acquired at the five locations shown in **Figure 6**, both before and after treatment. To achieve the required uniformity of stiffness, significant improvement of the deep fill was needed to make its stiffness comparable to that of the natural ground. The results from location 5 close to borehole 4 illustrate how this was successfully accomplished **(Figure 6)**.

Following completion of the warehouse in autumn 2002, it has been in continuous use and no settlement problems concerning either the structure or the floor slab have been reported. Settlement calculations have now been carried out using the new algorithm for 6m x 6m sections of the floor slab at each of the five CSW survey locations with the design load of $50kN/m^2$. The slab centre result for the data illustrated in **Figure 6** is shown below:

A summary of the results for the centres of all five sections is shown in **Table 6**.

Due to the essentially granular nature of the fill material, it is likely that the bulk of these relatively small settlements would have taken place during the construction phase of the warehouse, which is why no differential settlement has been observed since the warehouse has been in use.

Location No.	Centre Settlement (mm)
1	12
2	6
3	9
4	19
5	11

Table 6: Predicted settlements at CSW survey locations, Jarrow site

Conclusions

The lack of a reliable means of predicting ground settlement has historically led to the design and installation of unnecessarily sophisticated foundations. Engineers have been forced to err widely on the side of caution due to the constraints imposed by indemnities and insurance backed warranties. This has led to excessive cost and use of resources. While predictions of settlement grossly in excess of actual building

Z	E	Strain	Settlement
0.5	27.5889	0.144048	1.440479
1.5	26.85222	0.138098	1.380982
2.5	18.53303	0.168462	1.684624
3.5	22.26731	0.111213	1.112125
4.5	29.84521	0.06478	0.647796
			6.266007

Table 3: Example settlement calculation, Dagenham site

Fill description	Borehole 1 (m)	Borehole 2 (m)	Borehole 3 (m)	Borehole 4 (m)
Coarse surface	1.8	2.8	2.2	1.1
Deep ash	2.6	-	-	5.7
Total	4.4	2.8	2.2	6.8

Table 4: Fill thickness at borehole locations, Jarrow site

Z	E	Strain	Settlement
0.5	42.65469	0.116662	1.166617
1.5	24.17479	0.191539	1.915385
2.5	15.07862	0.258184	2.581837
3.5	14.13413	0.218797	2.187972
4.5	15.99433	0.150947	1.509471
5.5	19.72421	0.095531	0.955306
6.5	26.76954	0.055917	0.559165
			10.87575

Table 5: Example settlement calculation, Jarrow site

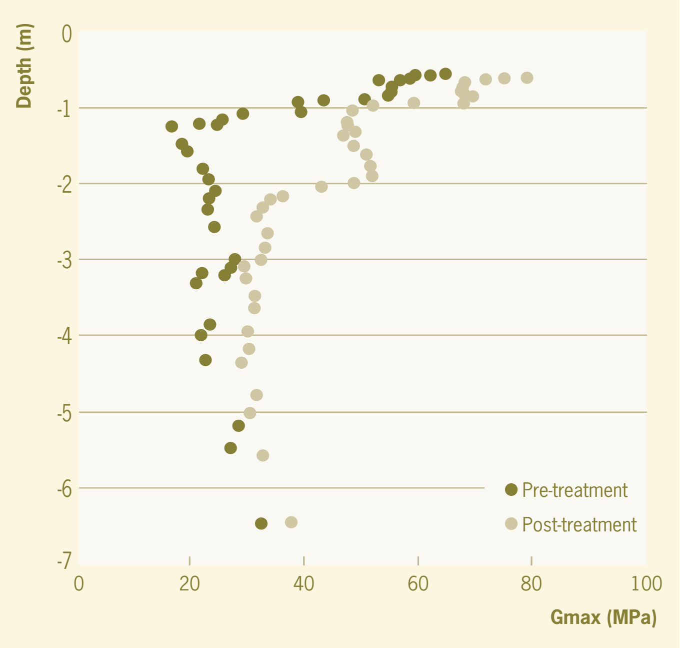


Figure 6: CSW survey results on deep fill, Jarrow site

performance might in some ways satisfy the requirements of our modern litigious society, it is surely the responsibility of the engineer to pursue the path closest to accuracy and reality.

To assist in addressing this problem, a computerised method has been developed for predicting settlement from the minimum-strain stiffness data obtained in continuous surface wave surveys. This method has been described along with tests to show that the results from it converge correctly.

Finally, its usefulness has been illustrated with two case studies. The results obtained have been shown to be much closer to the settlements which actually occur than those from traditional calculations and would have been very helpful to the design engineers had they been available at the times of construction.

As the method uses measurements dependent on the elastic properties of the ground, it predicts immediate settlement only. To alleviate any fears that margins of error could result that are too small for safety, it should be pointed out that provided CSW stiffness data is available to a satisfactory depth, there still remains a tendency to overestimate the settlement. There are two reasons for this:

1. The soil is assumed to be elastic and isotropic and the starting point for E in each layer is $2.5G_{max}$, which corresponds to a Poisson's Ratio of 0.25. In reality it is likely to be larger than this, corresponding to higher initial values for E and smaller settlements.

2. It is known that soil stiffness after ground improvement tends to increase naturally over a period of time (Moxhay et al., 2001). The G_{max} values for the soil once the building is constructed are therefore likely to be higher than those used in the calculations, again leading to smaller settlements.

Although there is no allowance for the longer-term secondary settlement or creep, the accuracy of the predictions achieved suggests that the contribution of the secondary component to the overall settlement is perhaps smaller than generally recognised and that this element is at least partially compensated for by the over-estimation of the immediate settlement just described.

CSW testing has reliably and successfully assisted ground improvement contractors and engineers in recent years, but its use has been restricted due to the stiffness data relating to minimum rather than operational strain levels. The settlement prediction method developed not only produces accurate results, but does so by converting the data to the higher strain levels with which engineers, quite understandably, feel more comfortable.

It is therefore felt that the value of the tool has been considerably enhanced, presenting real opportunities for future cost-saving in foundation design and installation.

Acknowledgements

The authors would like to dedicate this paper to the memory of the late Alf Campion, former Principal of Campion and Partners Consulting Engineers, who originally formulated the settlement algorithm and but for whose illness this work would have been completed significantly earlier. We got there in the end, Alf.

We would also like to thank Jerry Sutton of GDS Instruments, for his helpful advice and the ground improvement contractors involved, Roger Bullivant and Piling Solutions.

Appendix 1

Boussinesq formulae

1. For a rectangular area under load, the influence factor $I_{\sigma z}$ which represents the fraction of the surface load produced at depth z is given by:

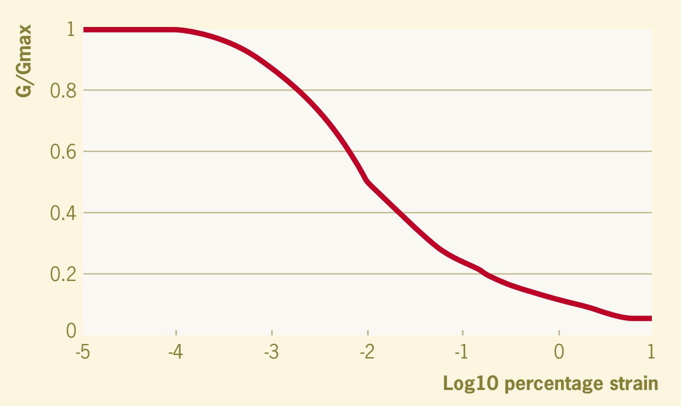
$$I_{\sigma z} = [\tan^{-1}\{mn/(m^2 + n^2 + 1)^{0.5}\} + mn\{1/(1 + m^2) + 1/(1 + n^2)\} /(m^2 + n^2 + 1)^{0.5}]/2\pi$$

where m = long side of area/z and n = short side of area/z.

2. For a circular area under load, the vertical stress at depth z on the axis of the area is given by:

$$\sigma_z = q[1 - 1/\{1 + (a/z)^2\}^{3/2}\}]$$

where q is the surface load and a the radius of the area.



Appendix 2

Appendix 2

Standard curve of stiffness against strain taken from Matthews et al. (1996)

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