

MONITORING OF SOIL STIFFNESS DURING GROUND IMPROVEMENT USING SEISMIC SURFACE WAVES

October 2000

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ABSTRACT

The Continuous Surface Wave (CSW) technique relies on the propagation properties of vertically polarised seismic surface waves, or Rayleigh waves. Surface waves have depths of penetration depending on their wavelength and hence frequency. Surface wave velocity is determined over a range of frequencies and used to calculate the variation in soil stiffness with depth. The nature of the propagation of seismic waves dictates that the stiffness calculated is the maximum value occurring at very small strain (<0.001%). At such strain levels most soils are believed to behave elastically, as a result of which stiffness is independent of strain (Matthews et al., 1996).

The introduction of ground improvement techniques such as vibro and dynamic compaction has led to different methods of monitoring and quality control being used with varying degrees of success. Despite the wide range of testing available, none is ideally suited to the huge variation in grain size found in the semi-cohesive superficial deposits that predominate as overburden throughout the U.K. The coarse nature of such material often militates against invasive testing methods such as continuous cone penetration or pressuremeter testing.

Surface wave testing, being non-intrusive, does not suffer the same disadvantages. Following extensive trials carried out on numerous sites, clear evidence of stiffness improvement after treatment is beginning to emerge as illustrated in this paper by case studies.

INTRODUCTION

For many years, partial-depth treatment and the creation of stiffened soil rafts over weak deposits has tempted ground improvement design engineers. However, a lack of understanding of the true beneficial effects of compactive improvement techniques and the inability to demonstrate this enhancement because of inapplicable testing methods has generally deterred their use. CSW testing has now shown, for the first time, the actual worth of such treatments and how they can be used with confidence in lieu of long piles or large-scale earth moving.

The CSW method (Matthews et al., 1996; Menzies and Matthews, 1996) presents a number of advantages compared with other testing methods. There is no ground penetration involved and measurements are made in situ, so stiffness values are not affected by sampling issues. Soils of all types can be tested, including residual soils and glacial tills which may exhibit marked heterogeneity. Furthermore, the equipment is portable and the data acquisition rapid, making the method relatively low-cost.

Since late 1997, the effectiveness of CSW monitoring has been investigated on a number of ground-improvement sites involving a variety of different soil types and treatment methods. This will be illustrated by four particular case histories following a brief description of the method itself.

THE CONTINUOUS SURFACE WAVE (CSW) METHOD

The Continuous Surface Wave System (CSWS) used in this testing is completely personal computer based and controlled. The surface wave source used is an electromagnetic vibrator capable of exerting a peak sine force of 498N. The surface waves generated are detected by low natural frequency (2Hz) geophones, the outputs of which are passed through signal conditioning amplifiers and then to a high speed 16-bit data acquisition unit.

To acquire data the vibrator is placed on a level ground surface and a row of between three and six 2Hz geophones is inserted co-linearly with it. The vibrator is energised at discrete frequencies between 5 and 600 Hz at operator specified intervals.

The signals received at the geophones are recorded digitally in the time domain and subjected to a Fast Fourier Transform to convert the signals into the frequency domain (i.e. spectral amplitude v. frequency). The frequency domain data is used to determine the phase of the generated signal at each geophone location. The geophones are positioned at known distances apart, d , and so the phase difference between the geophones, ϕ , can be used to calculate the wavelength, λ , for each discrete Rayleigh wave frequency.

$$\lambda = \frac{360.d}{\phi} \quad \text{Eq. 1}$$

The phase velocity of the Rayleigh wave, V_R , is determined from the wavelength and the frequency, f .

$$V_R = f.\lambda \quad \text{Eq. 2}$$

It can be shown from the theory of elasticity (Richart et al., 1970) that the relationship between the velocity of shear waves, V_S , and Rayleigh waves, V_R , in an elastic medium is given by:

$$V_R = C.V_S \quad \text{Eq. 3}$$

The constant C is dependent on Poisson's ratio and varies from 0.911 to 0.955 for the range of Poisson's ratio associated with most soils and rocks if anisotropy is ignored.

The maximum shear modulus or stiffness, G_{max} , is then determined using elastic theory as outlined in Matthews et al. (1996). If ρ is the bulk density of the medium, G_{max} is given by:

$$G_{max} = \rho.V_S^2 \quad \text{Eq. 4}$$

The depth assigned to each stiffness measurement is derived using the factored wavelength method described by Gazetas (1982) and Matthews et al. (1996). In this method the depth, z , at which the calculated shear wave velocity is considered representative of the propagation properties of the ground, is taken to be a fraction of the wavelength. In other words, λ/z is a constant. Gazetas (1982) recommended a ratio of 4 at sites where the stiffness increases rapidly with depth and a ratio of 2 where the stiffness remains reasonably constant. He suggested that a ratio of 3 was a reasonable compromise, hence depths are normally assigned using wavelength/3 in the processing of CSW data.

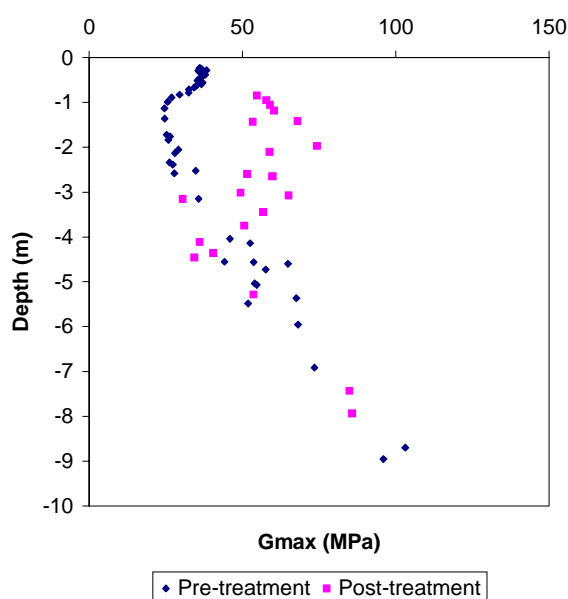
CASE HISTORIES

Each of the following examples relates to a development where the creation of soil stiffness has been imperative to the performance of the structure being built. The use of ground improvement over other foundation options, such as piling or excavation end replacement, has provided sufficient economic advantage to make the schemes desirable. Techniques involving stone columns and dynamic compaction to create stiffened soil rafts are somewhat radical and their technical feasibility has been questioned on occasions, but in each case illustrated the results achieved and monitored by CSW testing have justified application of the ground improvement concept.

Prospective Power Station, East Anglia

The geomaterial profiled was approximately 2m of sandy fill overlying essentially sandy soil becoming more dense with depth. Ground improvement was to be made by the insertion of vibro stone columns. Test columns some 6m long were installed to determine the optimum column configuration. During these tests CSW measurements were made to see if an increase of stiffness in the ground between the columns could be detected. The results from surveys prior to and post column installation are shown in Fig 1. The increase in stiffness in the first 4m of soil is evident in the post-treatment survey. Below this, the apparent lack of improvement is likely to be temporary and due to the presence of groundwater. These data are discussed in greater detail by Sutton and Snelling (1998).

Fig.1 East Anglia site pre- and post-treatment



Industrial Development, Erith, Kent

It was proposed to construct a steel-framed industrial unit on an infill site in an area widely known as having very poor soils. Whilst it was agreed that the main building frame had to be supported on piles, the warehouse floor (which had to withstand an imposed load of 37.5kN/m^2) required an alternative solution due to the prohibitive cost of a fully piled and suspended slab. The typical soils profile revealed by boreholes and dynamic cone penetrometer probes is illustrated in Figs 2(a) and 2(b). A medium-firm clay capping overlies successive layers of soft clays, silts and peat. For drainage reasons, it was necessary to raise the overall site level by 600mm following a 150mm topsoil strip using imported graded granular fill. Grids of stone columns at 2m centres were then installed under the entire future floor area together with a surrounding 3m safety curledge. The columns consisted of 75-40mm graded crushed concrete and were inserted to depths of between 3 and 4m. CSW testing was carried out both before and after treatment and typical stiffness profiles are shown in Fig 2(c). As the pre-treatment testing was carried out before the site level was raised, the depth values allocated to the pre-treatment stiffness data have been increased by 0.45m to compensate. Ground level on the borehole and dynamic probe records therefore corresponds to a depth of 0.45m on the stiffness profiles.

Fig 2(a) Erith site borehole record

Strata			
SPT N	Description	Depth	Legend
	TOPSOIL.	0.40	
5	Soft light brown silty clay with some peat and occasional pockets of fine sand.	1.70	
6	Soft grey brown organic silty clay with root traces and bands of fibrous peat.	2.70	
1	Very soft to soft brown and grey organic silty clay with occasional traces of organic fibrous material.	4.00	
*	Very loose grey silty clayey fine sandy silt.	6.00	
*			

Fig 2(b) Erith site dynamic probe record

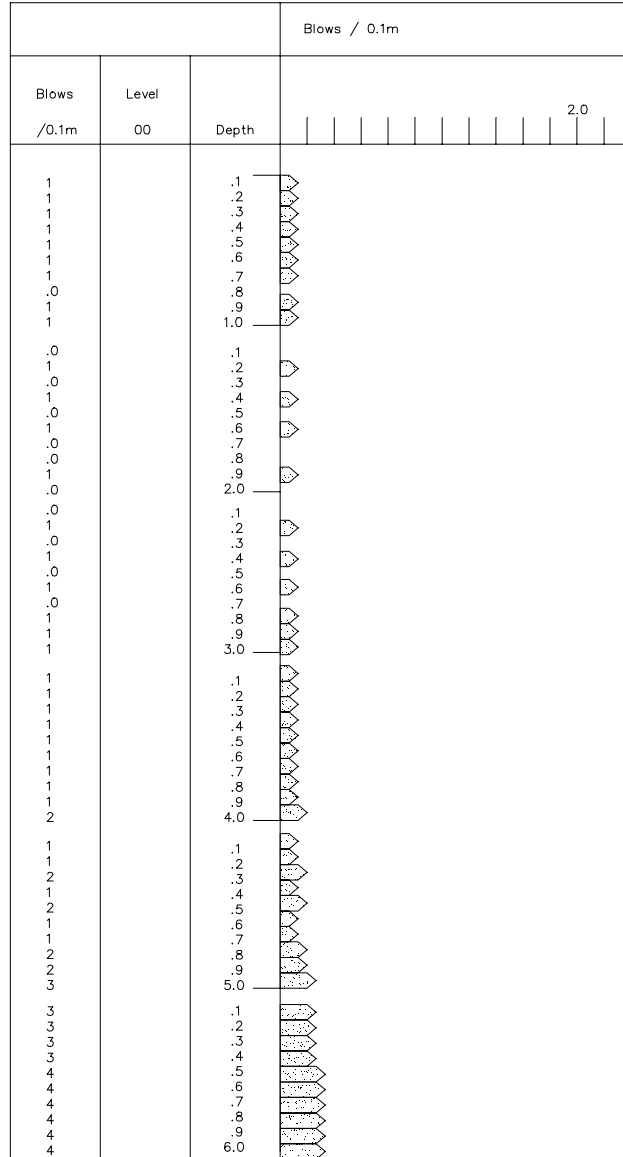
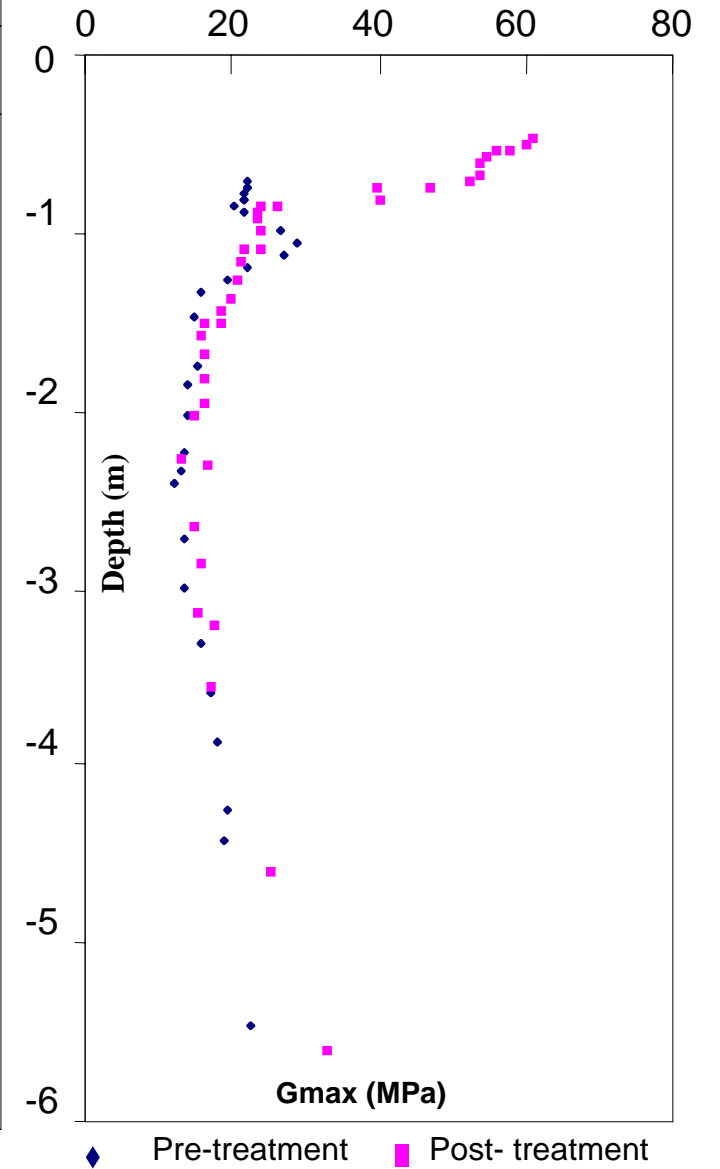


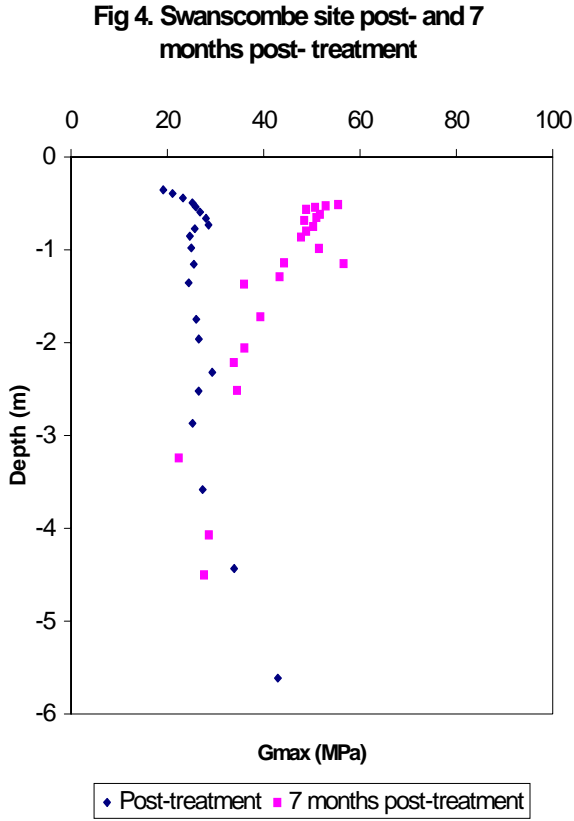
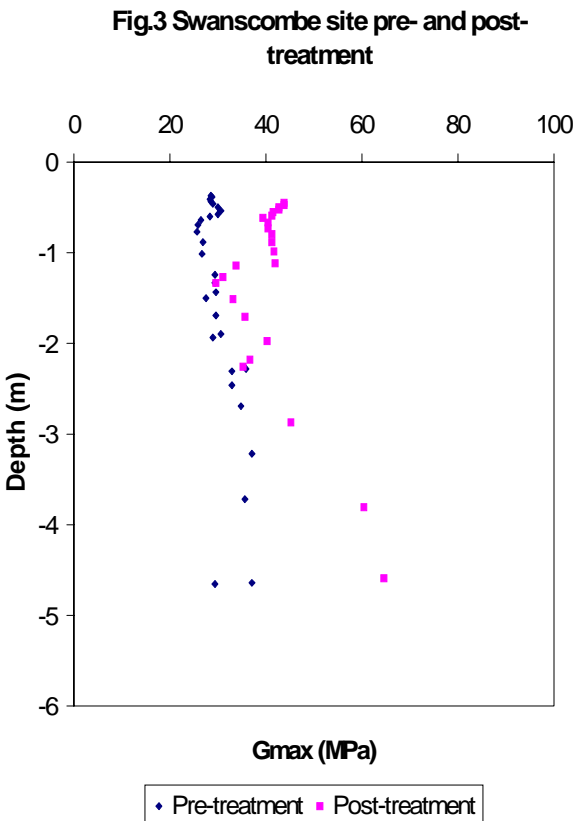
Fig 2(c) Erith site pre and post treatment



The pre-treatment surface wave data indicate a thin zone in the region of 1m depth which is noticeably stiffer than the surrounding soils. This feature is not clearly defined by either the standard penetration test or dynamic probe results. However, its existence was proven when it was found necessary to pre-bore holes through it to enable the insertion of the stone columns. The post-treatment surface wave results show a much harder layer has been created at the surface following the deposition of the layer of stone and subsequent ground treatment. They also indicate a small but significant stiffness increase below 2.5m depth. At the time of writing, the floor slab of the warehouse has been completed and loaded for approximately 6 months with no noticeable settlement recorded.

Quarry Rehabilitation, Swanscombe, Kent

The site was formerly a chalk quarry and had been filled during the 1950s with up to 32m of Greensand, known locally as Thanet sand. The proposal was to build 173 houses together with associated structures on the site. Thanet sand usually shows high variation in grain size and silt content. Particle size distribution tests indicated that in some places around 75% of the sample fell outside the grading normally accepted as suitable for improvement by mechanical methods (Baumann and Bauer, 1974; Hughes and Withers, 1974). A blanket treatment was proposed, using stone columns to stiffen the upper layers of the sand fill. Thousands of columns were installed in a regular grid pattern to a depth of between 4 and 4.5m and the ground in between tested by both static and dynamic cone penetrometer. The penetration tests failed to show any consistency in improvement, even in areas with closer column centres, while load tests on the columns themselves and on zoned areas showed excellent results in terms of settlement. CSW testing was introduced and the anticipated improvement to a depth in the region of 5m could be observed (see Fig 3). Furthermore, an additional visit to another part of the site 7 months after stone column installation showed that improvement had actually continued and that the stiffness up to around 2m depth had increased significantly whilst the site remained untouched (see Fig 4). The fact that the post-treatment stiffness at depth in Fig 4 is similar to the pre-treatment stiffness in Fig 3 illustrates the variable nature of the Thanet sand across the site as a whole.



Industrial Development, Basildon, Essex

A 4 500m² site was undertaken for an industrial development. The site had been filled some years previously with clay and small quantities of hardcore and rubble. The underlying natural material below the fill at between 2.5 and 3.5m depth was made up of bands of sand and gravel overlying firm to stiff London clay. Groundwater was not found in the upper soil and fill, but the high density of the fill in places meant that to install stone columns holes would have to be pre-bored, adding expense. The highly variable densities meant in situ ground treatment was essential to control future settlements. Dynamic compaction incorporating stone pillars was chosen as the most technically viable and cost-effective solution to support both ground-bearing slab loadings up to 75kN/m² and main building foundations with loads of 150kN/m². Excavations on site following initial weight dropping indicated the pillars were around 1.75m deep. A programme of CSW testing prior to treatment helped identify areas of the site with less stiff soil profiles and hence in need of the greatest improvement effort. A second set of tests after treatment showed a significant overall increase in ground stiffness as well as demonstrating that all areas of the site, including those known to be less stiff, were satisfactorily improved to a uniform standard. Typical results from an initially stiffer area of the site are illustrated in Fig 5 and from an initially softer area in Fig 6. The difference in stiffness between the two pre-treatment profiles and similarity between the two post-treatment ones is notable. It is also evident that the stiffened soil raft that has been created extends to a greater depth than the rock pillars themselves.

Fig.5 Basildon site west side pre- and post-treatment

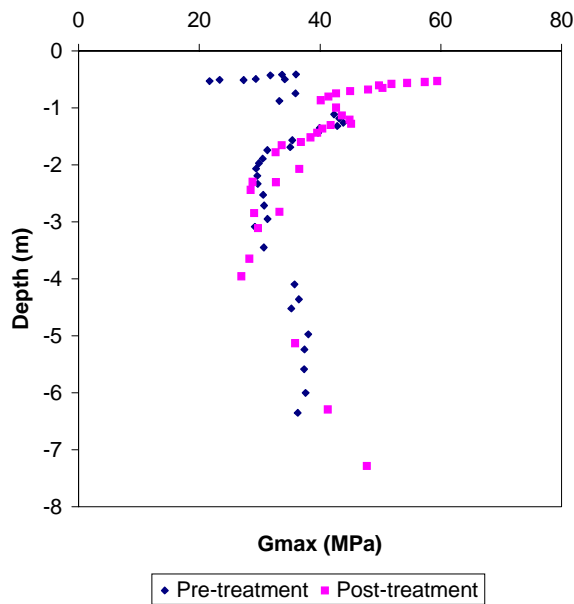
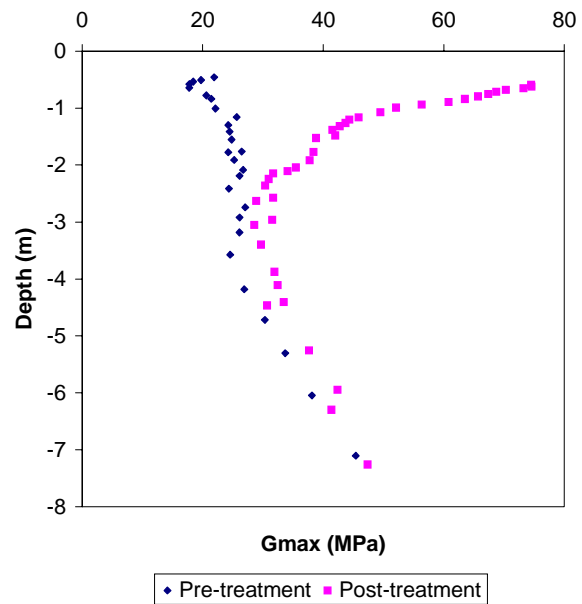


Fig.6 Basildon site east side pre- and post-treatment



A cold food processing unit has now been constructed on the site and is in use.

DISCUSSION

To date the use of ground improvement as a means of permanent structural support has been restricted, despite its considerable commercial advantages. This restriction has been in part due to the lack of a reliable method of demonstrating its effectiveness.

The non-invasive nature of surface wave testing for the measurement of stiffness means that the area under test is not disturbed and that the stiffened formation created by the ground improvement is not destructured by the measuring technique. Destructuring of the ground under test is thought to be a contributing factor where traditional measurement techniques do not indicate an increase in stiffness post-treatment. The case histories discussed indicate that continuous surface wave (CSW) testing is a reliable means of monitoring soil stiffness on ground improvement sites and can produce positive results when other methods fail to do so.

The data from Erith and Swanscombe (Fig 2(c) and Fig 3) indicate that immediately after treatment a minimum stiffness increase occurs halfway between the surface and the base of the stone columns and that the stiffness increase continues to become more pronounced below the column bases. This is in part due to destructuring of the material into which the columns have been inserted, but also a result of stiffness improvement taking place from the lowest point of influence upwards (Ménard and Broise, 1975). The destructuring is only temporary and the stiffness increases naturally with time (see Fig 4). It is, as yet, uncertain as to what mechanism this stiffness increase may be attributed, but self-weight compaction is thought to be the most likely cause. West (1976) suggests that this improvement may well continue for a considerable time.

The very fine grading of the Thanet sand at Swanscombe makes it behave more like a very silty, sandy clay when any form of mechanical improvement is attempted. Like the clay fill at Basildon, it is essentially cohesive in nature. The CSW results obtained at these sites therefore indicate that an immediate stiffness increase in cohesive soils after ground treatment can be observed, which contradicts popular thinking on this issue.

CONCLUSIONS

It has been shown that for a range of geomaterials surface wave testing is a viable and economic technique for monitoring the effectiveness of ground improvement work. For some natural materials and fills where standard penetrative methods are non-viable, this may be the only method by which stiffness increase can be successfully measured. Although CSW testing is best suited to granular soils, it has been shown that the method can be applied to cohesive soils as well.

It is felt that further development and wider use of CSW techniques will open new horizons for ground improvement systems and commercial advantages to those using them.

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