Aged embankment imaging and assessment using surface waves

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Rapid, non-intrusive surface wave surveys provide depth profiles from which ground models can be generated for use in earthwork condition assessment. Stiffness throughout earthworks controls the behaviour under static and dynamic loads, and characterising heterogeneity is of interest in relation to the stability of engineered backfill and life-cycle deterioration in aged utility and transportation infrastructure. Continuous surface wave methods were used to identify interfaces between fine- and coarse-grained fill in an end-tipped embankment along the Great Central Railway in Nottinghamshire, UK. Multichannel analysis of surface wave (MASW) methods were used to characterise subsurface voiding in a canal embankment along the Knottingley and Goole canal near Eggborough, Yorkshire. MASW methods are currently being used to study extreme weather impacts on the stability of a high-plasticity clay embankment along the Gloucestershire–Warwickshire railway near Laverton. Optimal results were obtained using equipment capable of generating and detecting over wide frequency ranges.

Notation

c'	drained cohesion measured at effective stress: kPa
f	frequency: Hz
f_s	sleeve friction: kPa
G	small strain stiffness or shear modulus: Pa
L	spacing: m
L_L	moisture content value of the liquid limit: mass ratio convention % g/g
L_P	moisture content value of the plastic limit: mass ratio convention % g/g
q_c	cone penetration resistance: MPa
V_R	Rayleigh wave velocity: m/s
V_S	shear wave velocity: m/s
$\Delta\phi$	phase differences
λ_R	wavelength of Rayleigh wave: m
ν	Poisson's ratio (dimensionless)
ρ_b	bulk density: kg/m ³
ϕ'	angle of friction at effective stress: °

1. Introduction

1.1 Construction and heterogeneity of aged UK earthworks

Much of the construction of the UK canal network started in the mid-eighteenth century at the beginning of the Industrial Revolution (Stevenson, 1886). Much railway construction followed in the nineteenth century, during the formative years of the Industrial Revolution (Skempton, 1996). The largest concentration of UK canals, roads and railways is in the South-East and around the

major cities of Birmingham, Leeds, Liverpool and Manchester. Excavation of aged canal and railway cuttings commonly employed large teams of labourers using driven wedges, horse-pulled ploughs, hand tools and, on the later railways such as the Great Central, steam-powered excavators (Brees, 1841; Skempton, 1996; Stevenson, 1886). While the construction materials were influenced by the underlying geological formations, the engineering characteristics of fine-grained overconsolidated clay or weak mudstone formations favoured relatively easy excavation using these tools; hence, many aged canal and railway earthworks comprise London Clay, Oxford Clay, Gault Clay, Mercia Mudstone and Lias Clay (Reeves *et al.*, 2006). The absence of any established practice resulted in embankment construction methods varying considerably between networks, often based on the experiences of the chief engineer. Side tipping was commonly employed in the construction of canal embankments, and railway embankments were often end tipped, both using local materials (Skempton, 1996). While modern embankments tend to be structured into well-compacted layers, aged embankments often have poor levels of compaction, a greater variability of fill material grades and usually exhibit highly unique heterogeneity (Selig and Waters, 1994; Skempton, 1996). Earthwork assessment requires the determination of conditions important for the evaluation of performance. Soil type, moisture, stress levels and strength control problems such as plastic deformation, heave, shear failure and mud pumping, which lead to a loss of embankment level and support (Perry *et al.*, 2003). Surface wave surveys provide a non-invasive means of assessing density and stiffness, which themselves are dependent on these soil properties and, thus, provide key earthwork condition indicators.

1.2 Embankment characterisation approaches and methods

Repeated visual inspections are mostly used to identify embankment problems, essentially looking for morphological features that confirm movement and anomalous groundwater conditions (Perry *et al.*, 2003). This approach is limited – for example because vegetation can often obscure signs of ground movement, subsurface ground and water conditions are not accessible and slopes often fail ‘rapidly’ without displaying visible signs of distress. Most common geotechnical monitoring approaches involve displacement measurements of embankments, often following observations of morphological features associated with instability (Dunnicliff, 2012). Surface and downhole tilt meters or extensometers are deployed to assess the displacement profile with depth. Such approaches require boreholes and are often accompanied by groundwater level measurements using piezometers. These data inform stability analyses, aid risk assessments and may contribute to remedial design. These approaches include the expense of intrusive works and implicitly accept the potential for failure, which does not honour the strict terms of ‘early warning’. Indeed, a recent Department for Transport review on infrastructure resilience (DfT, 2014) highlighted the importance of maintaining climate-resilient infrastructure and specifically recommended that geotechnical asset owners should ‘maintain a strong focus on trialling newly available condition monitoring technologies ... [and] improve [their] ability to identify and anticipate slopes that will fail and target remedial work as efficiently as possible’ (DfT, 2014: p. 102). Consequently, remotely sensed and non-invasive surface approaches are being explored by many asset owners. Such approaches are better suited for more rapid, cost-effective network coverage designed to detect smaller-scale changes considered to be the ‘precursors’ to the morphological features currently used to define marginal condition. Satellite or ground-based radar (lidar), robotic total stations and photogrammetry provide high-resolution ground displacement information (Mazzanti, 2012), but still essentially confirm the morphological response to underlying subsurface property (condition) changes that form earlier phases of asset deterioration. With no standard practice and no, or very poor, ‘as-built’ documentation, development of conceptual models for predicting the failure modes of aged infrastructure is particularly challenging. Geophysical imaging approaches can monitor internal property (condition) changes. These are the precursors connecting surface morphological response to subsurface processes driven by climate and ageing stresses (Gunn *et al.*, 2015a, 2016). These property change signatures offer a potential baseline against which internal condition thresholds can be identified and used as early warning of future instability.

1.3 Embankment characterisation and imaging using surface waves

Penetration into engineered pavements, sub-base and heterogeneous earthworks is particularly challenging; thus, invasive methods can become prohibitively time consuming and costly. Also, while useful for ground truthing, the limited sampling of invasive site

investigation (SI) methods makes them unsuitable for imaging irregular heterogeneous structures. Non-invasive geophysical techniques are cost-effective and rapid, and may provide two-dimensional (2D) and three-dimensional (3D) information, which makes them ideal for studying the spatial and temporal variations within assets that cannot be readily captured using discrete boreholes or other forms of geotechnical investigation alone (Bergamo *et al.*, 2016; Chambers *et al.*, 2014; Gunn *et al.*, 2015a, 2016).

Surface wave surveys can be mobilised to image the volumetric distribution of shear wave velocity and, thus, map the small strain stiffness (shear modulus) throughout earthworks, from which heterogeneity can be assessed (Bergamo *et al.*, 2016; Donohue *et al.*, 2011; Gunn *et al.*, 2013, 2015b; Zagyapan and Fairfield, 2000). These images can be used to assess the unique internal heterogeneity- and strength-related characteristics arising from the original tipping construction methods, or due to subgrade problems resulting from progressive deterioration. They can inform further SI design in relation to poor ground conditions and aid location of more challenging invasive methods such as pitting, cone penetration resistance testing and coring (Gunn *et al.*, 2012; Raines *et al.*, 2011). Rayleigh wave analysis, in particular, has been established as a reliable tool for the characterisation of the small strain stiffness (shear modulus G) of the near surface at the engineering scale (Donohue *et al.*, 2011, 2013a; Foti, 2003). Rayleigh waves are dispersive; their phase velocity varies with wavelength and, hence, depth of investigation (Socco and Strobbia, 2004). Parametric studies (Xia *et al.*, 1999) have shown the prevailing sensitivity of Rayleigh wave phase velocity to the shear wave velocity structure of the subsurface, so that information on shear modulus distribution can be assessed (Rucker, 2003). Also, reasonably short-term monitoring programmes by Bergamo *et al.* (2016) indicated distributed variation in Rayleigh wave phase velocity throughout a clay embankment to be sensitive to internal seasonal moisture variations. Such observations indicate the potential for repeat surface wave surveys to be used as a basis for long-term asset condition and deterioration monitoring – for example raising the prospects for condition matrices based directly on stiffness or its use as a proxy for consolidation, density or saturation. To this end, this paper presents three case histories to demonstrate the application, benefits and limitations of continuous surface wave (CSW) and multichannel analysis of surface wave (MASW) methods in relation to the assessment of the internal condition of aged embankments. Figure 1 shows the location of the three study sites, which include (a) an end-tipped rail embankment in Nottinghamshire, (b) a side-tipped canal embankment in North Yorkshire and (c) a clay rail embankment in Worcestershire.

2. Non-invasive surface wave surveys

2.1 Relevance of surface waves and stiffness to engineering characterisation

Rayleigh waves are often observed as the surface roll resulting from vertical impact on the ground. In fact, two-thirds of the total

Figure 1. Map of England and Wales showing location of case study sites

seismic wave energy generated by such impact propagates as Rayleigh waves (Gunn *et al.*, 2012; Richart *et al.*, 1970). Rayleigh wave velocity can be derived as a fraction of the shear wave velocity for rocks and soils (Woodward *et al.*, 2011), using

$$1. \quad V_R = \frac{(0.87 + 1.12 \nu)}{(1 + \nu)} V_S$$

where ν is Poisson's ratio. Poisson's ratios for rocks are commonly within the range of 0.2–0.3. Soils tend to have higher Poisson's ratios around 0.3–0.4, which can be even higher in very soft, fully saturated fine-grained materials, resulting in Rayleigh wave velocities between 90 and 95% of shear wave velocities. Shear wave velocity V_S is controlled by the stiffness, or the small strain shear modulus, G (Abbiss, 1981; Hight *et al.*, 1997), and the bulk density ρ_b of the medium, given as

$$2. \quad V_S = \sqrt{\frac{G}{\rho_b}}$$

The bulk density of the soil is the volumetric sum of the densities of the solid soil particles, porewater and the unsaturated air voids. The contribution of the air component is negligible because of its

very low density, but porosity and saturation exert very significant control over both bulk density and stiffness of soils and engineered fill (Donohue *et al.*, 2013b; Gunn *et al.*, 2003). Stiffness is defined as the ratio of stress to strain in material undergoing deformation. Many soils exhibit viscoelastic behaviour leading to strain softening (Bardet, 1992), which results in the absolute values of stiffness at small strains ($<10^{-3}$) generally being far greater than at larger strains. For fully saturated soils, porosity, bulk density and interparticle friction are interrelated with the way in which a soil consolidates and thus have a very significant influence on its stiffness and shear wave velocity (Gunn *et al.*, 2003). Hardin and Richart (1963) and Richart *et al.* (1970) showed stiffness to be linearly dependent on void ratio for uncemented sands, and Viggiani and Atkinson (1995) showed broad linear dependence of clay stiffness on the overconsolidation ratio. In partially saturated soils, there is an additional contribution to stiffness from suctions, which Whalley *et al.* (2012) related to net stress and the matric potential, and Cosentini and Foti (2014) related to soil porosity and saturation by using the van Genuchten equation.

2.2 Development of practice and modern surface wave survey field methods

Early procedures using surface Rayleigh wave techniques for soil property testing were suggested in the 1950s – for example Jones (1958) – but were limited by the technology then available. The procedure, called the steady-state Rayleigh method (Jones, 1958; Richart *et al.*, 1970; Rix, 1988) used a vertically acting sinusoidal vibrator, working at a frequency (f), placed on the ground surface and vertical receivers also on the ground surface, as shown in Figure 2(a). Receiver pairs were moved to locations away from the vibrator until the signals measured by them were in phase. At these positions, the distance between the receiver pairs was equivalent to the Rayleigh wavelength λ_R at that particular frequency. The phase velocity of the Rayleigh surface waves was determined using the relationship

$$3. \quad V_R = f \lambda_R$$

By changing the frequency, it is possible to construct a phase velocity–wavelength curve, also known as a Rayleigh wave dispersion curve (V_R plotted against λ_R ; or V_S plotted against λ_R by way of Equation 1). This study's surveys included CSW methods, which are a variant of the steady-state Rayleigh method. CSW also utilises a controlled frequency, a vertical oscillator and a small number of low-frequency geophones (Figure 2(b)). But in this case, the receiver positions remain fixed and the wavelength at each frequency was calculated from the phase differences $\Delta\phi$ between the ground motions on successive geophones and the spacing L between them (Joh, 1996; Menzies, 2001) using

$$4. \quad \lambda_R = L\Delta\phi/2\pi$$

Figure 2. Field source and receiver configurations used in surface wave surveys. (a) Field curves from two receivers used to calculate phase velocity in the steady-state Rayleigh method (modified from Rix (1988)). (b) Velocity from phase measurements at fixed

frequencies on small number of receivers in the CSW method. (c) Velocities across geophone arrays over a range of frequencies calculated in the MASW method. (d) Hammer and plate source and towed streamer use for MASW surveys

Joh (1996) suggested wavelengths as short as one-third of the shortest receiver spacing and as long as three times the largest receiver spacing could be measured with CSW. The range of wavelengths propagated (V_R/f from Equation 3) is anticipated for many sites to be from 0.38 m (minimum velocity of 75 m/s at maximum frequency of 200 Hz) to 60 m (maximum velocity of 300 m/s at minimum frequency of 5 Hz).

MASW can be performed on the data gathered using the same receiver array configuration adopted in shallow seismic refraction and reflection surveying (Park *et al.*, 1999) (Figure 2(c)). This study's MASW surveys employed impulsive sources capable of providing a broad range of frequencies. The source energy was provided by hammer and plate, where higher-frequency ranges are produced by lighter sources. Again, velocity dispersion curves are extracted from the phase delays of the different frequency

components of the source signal recorded by the receiver array (Park *et al.*, 1999; Rucker, 2003). MASW surveys were undertaken along profiles running parallel to the axis of embankments. Where static arrays were used, they comprised 24–48 vertically polarised geophones, spaced at 0.5 or 1 m. Longer section coverage was achieved by pulling along and relocating each successive geophone array to overlap with the previous array that was recorded. Where land streamers were used (Figure 2(d)), these comprised a 24-channel array of vertically polarised geophones that were mounted on small steel plates, which were towed between intervals equivalent to the shot spacing.

2.3 Shear wave velocity and stiffness profiles and sections

In both the MASW and CSW methods, the field dispersion curves were inverted to produce a velocity–depth profile for the shallow

subsurface (Foti, 2003; Joh, 1996). The profile was referenced to the centre of the geophone array subgroup, whose records are used to construct the dispersion curve, which could include around four geophones for CSW and from around 8 to 12 or even 24 geophones for MASW (Figures 3(a) and 3(b)). The dispersion curve was interactively forward modelled, which involves ground model inversion and generation of an associated modelled dispersion curve with the best fit to the field curve to determine the subsurface shear wave velocity profile (Foti, 2003; Joh, 1996; Raines *et al.*, 2011). The simplest method is attribution of a factored shear wave velocity (usually 1.1 times Rayleigh wave velocity) to a depth equivalent to a fraction of the Rayleigh wavelength λ_R (Foti, 2003; Joh, 1996) (Figures 3(c) and 3(d)). A

factor of $\lambda_R/3$ is most commonly used because a significant proportion of the particle motion in the ground associated with Rayleigh wave propagation is approximately at this depth (Gunn *et al.*, 2006; Joh, 1996; Richart *et al.*, 1970). The small strain stiffness is the product of the square of the shear wave velocity and the bulk density. The mean (and standard deviation) of 19 bulk density determinations on core from depths of 0.8 to 5.2 m within the embankment fill were 2.01 Mg/m^3 (0.33 Mg/m^3) (Gunn *et al.*, 2011), whereas velocity variation ranged from approximately 150 to 250 m/s, over a 60% variation. Because the stiffness is sensitive to the square of velocity, the bulk density at all depths can be represented by 2.0 Mg/m^3 , with little effect on the stiffness distribution. On this basis, the stiffness–depth profile

Figure 3. Overview of surface wave processing scheme from field coverage to the generation ground model images. (a) Relative positions of one-dimensional profiles by way of roll-along or towed procedures used to generate 2D sections. (b) Field record showing refracted and Rayleigh wave: eight-channel group selected from Rayleigh wave used to construct dispersion curve

and velocity profile. (c) Phase velocity–frequency transform for the eight-channel group: Rayleigh wave picked from high-intensity low-velocity feature. (d) Factored S-wave phase velocity–depth profile plotted at group midpoint: depth equivalent to one-third wavelength

was estimated from the shear wave velocity–depth profile. Two-dimensional sections were constructed by contour infilling using anisotropic inverse distance weighting over a grid between each of the shear wave velocity–depth profiles (Gunn *et al.*, 2011, 2013). Also, where MASW surveys were undertaken along a series of parallel arrays, the same weighting method was used to infill grids between the parallel sections to produce a pseudo-3D model of the embankment. (The models were constructed using array parallel surface wave propagation only. More complete 3D models would require surveys across further orthogonally orientated arrays, which are often impractical due to limited crest width across most embankments.)

3. Case histories

3.1 Great Central Railway embankment, East Leake, Nottinghamshire

CSW and MASW surveys were undertaken along a 140 m section of an embankment on the Great Central Railway (GCR) that runs

south of the overbridge at East Leake, Nottinghamshire, to the East Leake tunnel that passes under the A6006. The whole embankment extends 800 m and was constructed in 1897 by using end-tipped materials excavated from adjacent cuttings to the south-west (SW) and the north-east (NE) (Bidder, 1900). The railway is operated as a branch from the main line, taking heavy freight class 66 locomotives pulling wagons loaded with supplies for the British Gypsum works at East Leake. Track geometry is very poor due to dipping points and pronounced lateral and vertical warping. Figure 4(a) shows the layout CSW survey stations relative to the line of MASW surveys that were undertaken using a series of static arrays with a 1 m geophone spacing. Stiffness–depth logs were generated at a series of locations using both methods, and a ground model was created by way of grid infill between each depth log (Figure 4(b)).

Interpretation of the 2D section through the embankment (Figure 4(b)) includes a basal, high-stiffness zone below 5 m depth associated with the underlying Branscombe Mudstone

Figure 4. Ground model constructed from CSW and MASW surveys at embankment along GCR. (a) Field map of embankment showing location of MASW survey lines and CSW profile points

(coordinates in bold). (b) Stiffness section showing interface between fine and coarse fill grades

Formation bedrock and an extensive low stiffness zone in the upper 5 m interval associated with the engineered fill of the embankment. Within this upper zone, a prominent high-stiffness feature appears between chainage 60–100 m. While in the shallow subsurface between 60 and 80 m, this feature is related to the ballast; from 80 to 100 m, it is associated with a deeper lens extending to the top of the Branscombe bedrock. This lens appears to be associated with infillings of high-stiffness materials, including locally sourced sand, gravel and siltstone. Visualisation of the morphology of the high-stiffness feature across the embankment in three dimensions enables a fuller understanding of the spatial context of the structure in relation to the end-tipping construction technique (Figure 5). Core samples from boreholes through the embankment proved fill source material from local cuttings immediately SW and NE of the embankment. Fill materials to the SW of the small stream include Westbury Mudstone that has degraded to soft clay, siltstone of the Blue Anchor Formation (also degraded to clay in places) from the cutting to the SW and glaciofluvial sand and gravel, thought to be a later infill from local quarries (Bidder, 1900). Clays from the Westbury Formation have stiffnesses ranging from 40 to 80 MPa, whereas the Blue Anchor siltstone and the glaciofluvial sands and gravels tend to be associated with stiffnesses greater than 80 MPa (Gunn *et al.*, 2013). Figure 5(b) shows stiffnesses from 80 to 120 MPa – that is, effectively removing the clays of the Westbury Mudstone and also revealing the original ground surface on which the embankment was tipped. Limiting the stiffness ranges enables a domain or anatomical visualisation of morphology, such as the high-stiffness structure (from 60 to 100 m) dominated by Blue Anchor siltstone and glaciofluvial sand and gravel fill. The

authors believe that this feature resulted from end tipping as larger boulders ran downslope to the toe of the advancing embankment to be later infilled by finer materials. The engineer in charge of works, Bidder (1900), did not explicitly describe the East Leake embankment construction, but photographs by S. W. A. Newton show end-tipping wagons of the contractor, Henry Lovatt, at the East Leake tunnel cutting, which is consistent with the ‘cut-and-fill’ methods used at the time (LCC, 2016).

3.2 Knottingley and Goole canal embankment, Eggborough, North Yorkshire

The study site was a section of embankment, approximately 450 m west-north-west of Whitley Bridge near Eggborough, in the Selby district of North Yorkshire. The embankment, constructed in 1821, forms the retaining barrier on the southern side of the canal, which is raised above the surrounding land by approximately 1.75 m. The site is within a section of the embankment, which is lined on the waterside by a vertical masonry wall made of large gritstone ashlar. Parts of the masonry-lined section of the embankment have shown a history of void formation and leakage, resulting in localised partial collapse of the embankment. A series of surface openings were observed during visual inspections in September 2008. One of the prominent features was a collapsed cavity showing significant water seepage at its base. The surface opening appeared to be approximately oval, 1 m by 0.7 m. The subsurface cavity appeared to be funnel shaped, decreasing in diameter with depth, but extending to 1 m or deeper. It is suspected that the masonry wall was originally lined with puddle clay and the embankment to have been side tipped against the wall. The water appears to have

Figure 5. Anatomical visualisation of embankment end-tip structure by way of deconstruction of a 3D ground stiffness model. (a) Stiffness range: 40 MPa < stiffness < 140 MPa. (b) Stiffness range: 80 MPa < stiffness < 140 MPa

breached the puddle clay seal, entered the north side of the cavity at a depth of around 0.75 m and to have flowed in a south-westerly direction deeper into the embankment.

A complete MASW survey was undertaken along four transects, each running parallel to the line of the canal, but offset from the masonry wall by 2–4 m; see Figure 6. Each line was 17.5 m long, comprising 36 channels of vertically polarised geophones spaced at 0.5 m. Embankment geomorphology and survey lines were positioned within local and British National Grid coordinate grids using a Leica SmartRover Global Positioning System (GPS). GPS points were at 0.5 m along the axis of the embankment and 1 m across the flank. Source locations included an end line offset of 4 m, an end line location and locations within the line spaced at 4 m intervals to enable clearly defined pulses with high signal–noise ratios to be identified on the field traces of the nearby subgroups of geophones. This was done to guard against the loss of high-frequency energy, which is highly attenuated in heterogeneous or disturbed ground.

A 3D ground model was constructed by way of weighted infill of a grid between each of the shear wave velocity profiles created along a series of overlapping eight-channel geophone groups

along each survey line (Figure 6). The aerial extent of the model extends 16 m along the axis and 2 m across the transect of the embankment, approximately centred on the surface void; the depth interval is from 0.5 to 3 m below the embankment crest. The model includes velocities in the range of 85–165 m/s. The broad characteristics of soils associated with velocities in the range of 85–100 m/s can be considered as loose (low packing density) coarse-grained or poorly compacted, very soft or soft fine-grained soils (possibly of strengths <40 kPa) (Gunn *et al.*, 2003; Hardin and Richart, 1963; Richart *et al.*, 1970). In contrast, the broad characteristics of soils associated with velocities in the range of 140–165 m/s can be considered as medium-dense, coarse-grained or firm, fine-grained soils (possibly of strengths >75 kPa). Thus, the contours within this soil classification indicate an overall wedge of lower-velocity, more poorly compacted, lower-strength material thickening towards the east (lower *x*-axis values) – shown as the darker zones in Figure 6. Within this wedge, two near-surface very low-velocity (85–90 m/s) zones occur, where one coincides with the open cavity. A higher-velocity (>130 m/s), higher-strength, stiff basal zone occurs deeper into the section, which the authors suspect would coincide with the base of the embankment or the top of the underlying superficial geology.

Figure 6. Surface wave surveys at embankment along Knottingley and Goole canal near Eggborough. (a) Location of the model section relative to cavity, embankment axis and positions of geophone line arrays. (b) Subsurface structure visualised by way of 3D shear wave velocity model

The boundary between the stiffer base (i.e. shear wave velocity >130 m/s) and the overlying weaker soils undulates with a series of marked peaks and troughs, which the authors suspect provide a subsurface channel for groundwater escaping from the canal to drain into the embankment and formation. Progressively removing ground with velocities below 120 m/s from the 3D model reveals a trough that appears spatially coincident with the flow of water into a subsurface pipe observed from a view into the void (Figure 7). (This observation was made from a position equivalent to $x = 8$ m, $y = 1$ m on the local survey grid.) The authors suspect that this trough is formed in stiff sandy, silty clay, where overlying, looser, less cohesive materials were removed by water flowing into this zone by way of a breach in the canal wall and puddle clay seal. At site, water was observed in video recordings to enter the subsurface pipe from the north, at approximately 0.75 m deep and at $x \cong 8.5$ m along the section. The authors suspect that the wall and embankment were breached further to the east and that the water followed within the low-velocity interval (<90 m/s) between 0.5 and 1 m deep and between $x = 7$ m and $x = 8$ m seen in sections $y = 0$ –0.5 m in Figure 7.

3.3 Laverton railway embankment, Worcestershire

The Laverton embankment marks the point of a halt in the trains on the current heritage Gloucester and Warwickshire railway. The line was originally part of the Great Western Railway's Cheltenham–Stratford-upon-Avon–Birmingham line, known as the Honeybourne line, built between 1900 and 1906. The Laverton embankment is around 6 m high and is believed to have been constructed by way of end tipping of local Charmouth Mudstone. Currently, the embankment comprises a 0.9 m thick upper layer of ballast fouled with fines, ash and soil (rich in humus), generally

Figure 7. Void associated with very low velocity zones in 3D ground shear wave velocity model. (a) Extent of void highlighted using selected iso-volume velocity ranges. (b) Fence panels show

small trough structure through which water flowed from canal into void

underlain by clay fill of reworked Charmouth Mudstone of about 5 m high. The pits at CPT 3 and CPT 4 (see Figure 8) revealed a layer of cobbles (usually of 100–250 mm dimensions) at the base of the ballast that would have been laid as part of the original engineered subgrade, as shown in Figure 9. Figure 8(a) shows the location of repeat MASW surveys that were undertaken at 2-month intervals by using a 24-channel land streamer with 4.5-Hz geophones spaced at 1 m. Each time, a 300 m long survey line was completed as a series of 50 increments each 6 m, where in each case Rayleigh waves were generated by a hammer/plate source offset 2 m from the nearest geophone (Figure 2(d)). A dispersion curve was generated for each source-line location, and an associated shear wave velocity profile plotted at the midpoint of the 24-geophone array. A 2D section was generated from each survey by infilling a grid between each velocity profile (Figure 8(b)).

The south-south-west (SSW) half of the section of the embankment may have been constructed in two lifts, possibly with the boundary at about 3 m depth. Generally, the upper interval exhibits very low penetration resistances of around 1 MPa, while the fill between 3 and 5 m exhibits slight greater

penetration resistances (1.3 MPa). Sampling of the clay immediately beneath the ballast around CPT 3 indicated high levels of moisture and weathering. Overall, the corresponding shear wave velocity ranges in the SSW half (0 to CPT 6 at station 180 m) of the section are 80–90 m/s around 1 m depths; 100–120 m/s around 1.5 m depths; 140–150 m/s around 3 m depths; and up to 180 m/s around 5 m depths. The shear wave velocity section indicates an apparent stiffening of the mid and basal zones of the north-north-east (NNE) half of the embankment. The CPT 9 log indicates a thinning of the low penetration resistance interval beneath 1 m interval of ballast – for example to around 0.5 m thick. Beneath this, penetration resistances within the interval from 1.5 to 4 m are above 2 MPa, where the associated shear wave velocities are around 140–160 m/s. This change in the geotechnical properties may relate to increased trafficking or use of different materials in the NNE half of the section approaching the drainage culvert.

Figure 10(a) shows a section of the cone penetration resistance q_c across the embankment located close to the location of CPT 3 in Figure 8(a). The embankment and the weathered zone of the

Figure 8. MASW survey at the Laverton embankment along the Gloucestershire–Warwickshire railway. (a) Layout of survey line relative to CPT profile locations. (b) 2D section of shear wave velocity profile along the axis of embankment

underlying formation can readily be distinguished from the unweathered formation. Values up to 1 MPa are present in the upper interval to around 2.5 m deep below the crest and in the upper 1 m over the east flank. Values within the lower interval (3–5 m) of the embankment range from 1 to 2 MPa. Values in the formation increase from around 2 MPa in the upper 1 m of the

weathered material to above 4 MPa in the unweathered Charmouth. Core logs from a borehole at CPT 3 indicate relatively high moisture content levels and low densities throughout the embankment fill (Figure 10(b)). The density log shows a slight increase in density with depth with a mean bulk density through the embankment of 1.92 Mg/m^3 , which is below

Figure 9. Photograph of wall and floor of pit at CPT 3

Figure 10. Geotechnical properties through Laverton embankment section near CPT 3. (a) 2D cross-section showing CPT penetration resistance. (b) Moisture content and bulk density profiles from core taken beneath embankment crest

the median value for bulk density of samples gathered from in situ outcrops in the Worcester basin reported by Hobbs *et al.* (2012) (see Table 1). Relatively low densities are consistent with fill of reworked material – for example a consequence of the cut-fill construction method and subsequent ageing processes such as shrink–swell and fissure development associated with seasonal moisture cycling. Testing on unweathered in situ Charmouth Mudstone sampled from outcrops in the Worcester basin around Laverton by Hobbs *et al.* (2012) provided values for drained cohesion c' and friction angle ϕ' of 19 kPa and 26°; see Table 1. Equivalent test values on recompacted auger samples recovered from shallow depths were 11.3 kPa and 19.3° from the east flank, where values of 7.3 kPa and 16.2° from the west flank appear to be very close to residual. Thus, it is likely that fill close to residual strength occurs within a shallow interval (upper 0.5–1 m) across the flanks and also the upper interval just underlying ash beneath the rails. Atterberg index tests on the fill materials

augered from the upper 0.5 m on the east and west flanks indicated plastic (L_P) and liquid (L_L) limits within the ranges of 27–37 and 61–77%. Moisture levels over several intervals through the fill are within this L_P range, which is considerably greater than the natural in situ L_P of 23% (Figure 10(b)). Because of its high moisture state, the fill consistency here is low. It is likely to be low throughout the zones of low shear wave velocity from CPT 1 to CPT 5 and also around CPT 6, showing how the surface wave image can potentially identify parts of the embankment that are more susceptible to plastic deformation and, potentially, shear failure. Around CPT 3, morphological features including a midslope berm with lower slope bulging provide evidence of progressive deformation. Backscarp, accumulation and run-out zones from a former slip occur above a widened embankment between CPT 5 and 6, which is either the original slipped material or added toe support. Combined moisture content logging and repeat MASW surveys by Bergamo *et al.* (2016)

Reworked Charmouth Mudstone from embankment samples (6 number off)		
Mean bulk density: 1.92 Mg/m ³	Plastic limit range	Liquid limit range
Moisture content: range: 23–33%	27–37%	61–77%
Mean: 26%		
Triaxial tests at effective stress	Drained cohesion c' : kPa	Drained friction angle ϕ' : °
East flank (upper 0.5 m)	11.3	19.3
West flank (upper 0.5 m)	7.3	16.2
Median values for Charmouth Mudstone in Worcester basin from Hobbs <i>et al.</i> (2012) (number off samples stated)		
Index test values (>100 samples)	BS EN 1377-1:1990 (BSI, 1990a)	
Bulk density: 2.03 Mg/m ³	Plastic limit	Liquid limit
Natural in situ moisture content: 22%	23%	53%
Triaxial tests at effective stress	BS EN 1377-8 :1990 (BSI, 1990b)	
>10 samples)	Drained cohesion c'	Drained friction angle ϕ'
	19 kPa	26°
Residual values	Drained cohesion, c'_r	Drained friction angle ϕ'_r
	9 kPa	14°

Table 1. Geotechnical properties of Charmouth Mudstone measured on samples taken from the Laverton embankment and outcrops within the Worcester basin

showed a dependence of V_s on the moisture content within the embankment core. In Figure 11, the sequence of sections between CPT 2 and CPT 5 captured in 2014 indicate velocity changes respond very dynamically to weather events. The lowest velocities (<110 m/s) occur in the upper 3 m over the SSW half of the section between CPT 2 and CPT 3, where very soft clay fill was observed. While velocities are greater (up to 130 m/s) in the NNE half between CPT 4 and CPT 5, larger velocity changes also occur here. Notable are the surveys on 14 January within a period of heavy rain and 14 September within a relatively dry spell. These sections provide some insight into the possible end-member conditions – for example those that could be potential trigger points to inspect the embankment for evidence of either very wet or very dry fissured near surface fill. Currently, progressive deterioration in shear strength can only be inferred from repeat V_s surveys. However, quantitative condition assessment is a real possibility in the near term provided robust relationships between shear strength, stiffness and shear wave velocity can be established.

4. Conclusions

CSW and MASW surface wave surveying provide rapid, portable and non-intrusive tools to assess the small strain stiffness characteristics of the railway subgrade and embankment fill. Information is provided in the form of a stiffness profile that provides a useful input into models to characterise the subgrade response to static and dynamic loads. Survey measurements are

repeatable, making these methods very suitable to assess the efficacy of retrospective stiffening of the subgrade such as by vibro-tamping, soil nailing or chemical treatment, such as shown in the case histories by Moxhay *et al.* (2001, 2008). In the case histories presented in this paper, signal frequencies up to 200 Hz were measureable with the CSW, whereas the MASW was often limited to frequencies below 100 Hz. Signal frequency limitations can occur in the MASW method when using heavier source hammers or long offset distances, but these can be readily overcome by using lighter sources at closer offsets or using high frequencies with a CSW method. Impulsive sources are more convenient and combined with spectral processing yield results far more rapidly than continuous frequency approaches such as used in CSW surveys. Thus, provided a source or sources can be used to generate a relatively wide frequency band (e.g. 5–100 Hz), an MASW survey will provide more effective ground coverage. Simple MASW survey field procedures to guard against ‘far offset’ effects include the use of a constant source–nearest receiver distance and roll-along receiver array relocation to extend lateral coverage.

Like most non-invasive geophysical imaging methods, MASW arrays can be scaled to capture complex structures within aged geotechnical assets. The case histories demonstrate that heterogeneity of aged earthwork structures is not consistent with uniform, laterally extending horizontal layers. Two-dimensional sections can be built up from a series of inline velocity–depth profiles spaced at intervals suitable for capturing the heterogeneity

Figure 11. Embankment velocities relative to 2014 temperature and rainfall sequence (after Bergamo *et al.*, 2016)

even on a submetric level. Pseudo-3D sections can be constructed using a series of CSW surveys or using an MASW approach with overlapping gathers of selected channels in subgroups of geophones from larger geophone arrays. Heterogeneous structure related to original construction or the effect on strength-related properties of subsequent localised deterioration within the asset can be located on high-resolution V_S or stiffness images. Anatomical imaging is possible, including the location of stiffness contrasts between different engineering materials, understanding dynamic load performance and early identification of progressive subgrade deterioration that precedes track problems or slope failure.

5. Practical relevance and potential applications

Avoidance of the costs associated with service and business disruptions provide compelling cases for early interventions; stated simply by Glendinning *et al.* (2015) as ‘emergency repairs cost 10 times that of planned works’. ‘Early’ is about when to intervene, or when to act, which depends on the definition of earthworks condition and the subsequent problems and solutions. The metrics used to define condition are very dependent on the monitoring approaches and associated technologies. Table 2 maps typical monitoring methods onto the asset management spectrum showing how a progressive transition from responsive to preventative practice involves a shift from the surface to the subsurface. Current practice, such as the example literature in Table 2, largely

involves monitoring changes in conditions or properties that precede the morphological signs that generally trigger decisions to monitor movement or deformation. Technologies to monitor condition cover a range of scales, from a sensor detecting a property time series variation at a point (or at least small volume) to non-invasive geophysical imaging methods that can provide whole asset property variations, in both space and time, potentially enabling holistic (or anatomical) condition assessment. Currently, such images inform further invasive investigation, but with increased acceptance, they could also inform design and monitor efficacy of more sophisticated, customised interventions. Improved understanding and quantification of the relationship of seismic properties (G , V_S , ρ) to engineering properties would increase acceptability and use of surface wave surveys. Early focus should include the control of moisture content on both matrix- and clast-supported fill – for example mapping seismic properties onto consistency and identifying threshold values of G and V_S associated with critical shrinkage, plastic and liquid limits of fine-grained materials (of various plasticities). The contribution of suction to undrained shear strength also requires further study, in particular to quantify its relationship to G and V_S and their sensitivity to saturation, such as from seasonal moisture cycling. Convincing and timely delivery of this information from the research community to the geotechnical asset owners would stimulate the take-up of surface wave surveys as part of routine monitoring and management practice.

Condition timeline		As built	Serviceable	Marginal	Poor	Failure
Management strategy		Preventive		Responsive		
Actions/relative costs		Intervention		Refurbish	Renewal	
Monitoring approaches		Subsurface		Surface		
Monitoring parameters		Thresholds	Precursors	Visual signs	Movement	
		——— Ground conditions ———		——— Surface deformation ———		
		——— Subsurface deformation ———				
Monitoring methods	Ground conditions	Point sensors: moisture; ^{a,b} suction; ^{c,d} pore pressure; ^{e,f} wireless sensor networks ^g				
		Non-invasive geophysical imaging GPR; ^h resistivity; ⁱ seismic ^{j,k} (including surface waves)				
	Subsurface deformation	Small strain (precursors): acoustic emission (Alarms) ^{l,m}				
		Large strain: inclinometers; shape arrays ⁿ				
	Surface deformation	Remote: satellite (InSAR); ^{o,p} terrestrial laser; robotic total station; ^p photogrammetry ^q (UAV) ^r				
		Walk-over: GPS; dGPS ^s				
		Ground installation: tilt; extensometer ^t				

Alarms, assessment of landslides using acoustic real-time monitoring systems; dGPS, differential global positioning system; GPR, ground penetrating radar; InSAR, interferometric synthetic aperture radar; UAV, unmanned aerial vehicle

^aTopp *et al.* (1980)

^bKane *et al.* (2001)

^cRidley *et al.* (2003)

^dToll *et al.* (2013)

^eSmethurst *et al.* (2012)

^fLehmann *et al.* (2013)

^gGong *et al.* (2013)

^hSteelman *et al.* (2012)

ⁱChambers *et al.* (2014)

^jDonohue *et al.* (2013a)

^kBergamo *et al.* (2016)

^lDixon *et al.* (2015)

^mSmith and Dixon (2015)

ⁿAbdoun *et al.* (2013)

^oScaioni *et al.* (2014)

^pMazzani (2012)

^qBemis *et al.* (2014)

^rHugenholtz *et al.* (2015)

^sMillis *et al.* (2008)

^tWang *et al.* (2008)

Table 2. Common asset monitoring approaches: surface and subsurface methods mapped onto preventative–responsive spectrum

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