# A Guide to Track Stiffness

## AUGUST 2016

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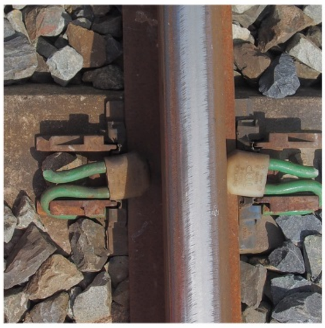
*the Cross Industry Track Stiffness Working Group*





# Acknowledgements

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ISBN: 9780854329946

Published by the University of Southampton
University Road, Southampton, SO17 1BJ, UK

## Contents

# 1. INTRODUCTION

## 1.1 *How to use this Guide*

Track stiffness has a major influence on the performance of a railway track as trains traverse along it. It must lie within certain limits not only to control deflections, but also to maintain track geometry and ensure the longevity of track components. A track support that is too soft, or varies too widely over a short distance, may lead to excessive deformation and a rapid loss of geometry. A track support that is too stiff may result in damage to components such as rails and clips.

Problems associated with track stiffness manifest themselves in a variety of ways, for example, by the presence of voids below sleepers (also referred to as *voided* or *hanging* sleepers), the repeated recurrence of track faults after remediation, and rapid rates of track geometry deterioration. This Guide is intended to help practising engineers identify and understand faults associated with track stiffness problems, enabling them to work with track bed specialists to select appropriate repair and remediation techniques to get the best out of the existing ballasted track network.

Sections 1 to 5

- summarise the theoretical background
- show how the various track system components combine to provide an overall or global track stiffness
- summarise common methods of measuring track stiffness
- give typical values of track stiffness and deflection on both acceptably and poorly performing sections of track, and
- illustrate how to diagnose track stiffness problems.

These sections are intended mainly for background and context; they may be read and referred back to as needed. References are included in section 6. Practical implementation of the Guide is expected to be primarily through Appendices A, B and C.

Appendix A contains a number of case studies, each describing a problem and a remediation technique. It is not necessarily the best or the only technique for the problem described, but practising engineers may find within these case studies situations matching experience on their own routes. Over time, evidence as to the most appropriate remediation technique will accrue and the repository of case studies will be updated to reflect this.

Track stiffness on an existing network varies substantially. Achieving a single value of track stiffness is unlikely to be a realistic option, as opportunities to completely re-engineer the trackbed are rare. Thus the Guide does not set out to provide a single target value of track stiffness, but rather to provide the means to manage stiffness effectively. An exception to this is newly constructed track, for which target values of track stiffness are specified within various railway authority standards. These may vary depending on the specific details of the route, the trackform (e.g. slab track) and the type and speed of the trains that will operate on it.

Appendix B describes methods for measuring track stiffness and the associated level of intrusion on normal route operations.

Appendix C is a flowchart illustrating how track stiffness problems may be diagnosed by local maintenance engineers.

Appendix D lists the standard symbols and defines key terms and acronyms used in the Guide.

This Guide has been written for a wide audience of engineers operating in maintenance, renewal, engineering and research environments. As a result, not all sections of the Guide will be relevant to all roles.

### 1.2 *What is track stiffness?*

In a typical railway track system, the rails are supported by a number of elements in series – in descending order, the railpads, sleepers, ballast, sub-ballast and subgrade or formation, as illustrated schematically in Figure 1. In a well-constructed and well-maintained track, these elements all deform essentially elastically (in the sense that the majority of their deflection is not permanent) when loaded and unloaded during train passage.

In its most general form, the term “track stiffness” may be taken to mean the (point) load required to produce a unit deflection of the rail at the location where the load is applied. This is a simple and intuitive definition, which has units of force divided by deflection (e.g. kN/mm). It is what the train “sees”, and its value depends on the effective stiffnesses of all of the individual elements of the track system combined, including the flexural rigidity (bending stiffness, *EI*) of the rails. It may therefore be considered a *composite* or global stiffness.

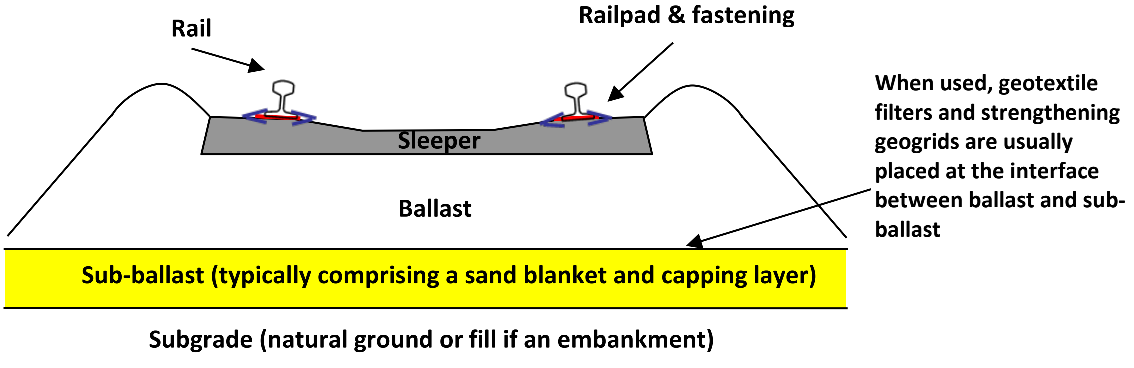



**Figure 1: Schematic cross section of a ballasted track system**

Many analyses model the track as a beam (the rails) on an elastic foundation (the support) – generally abbreviated to “BOEF”. It is therefore convenient to separate the flexural rigidity of the rails (which for a standard rail type is well defined) from the direct stiffness of the support system (railpads, ballast, sub-ballast and subgrade), which can vary widely. This concept is illustrated in Figure 2, which also shows a series of moving loads.

The rail support stiffness may be further separated into that above the sleepers (principally arising from the railpads, which is relatively easy to control) and that below the sleepers (arising from the ballast, sub-ballast and subgrade – together termed the trackbed). Their individual contributions may be represented by springs in series within the BOEF model.

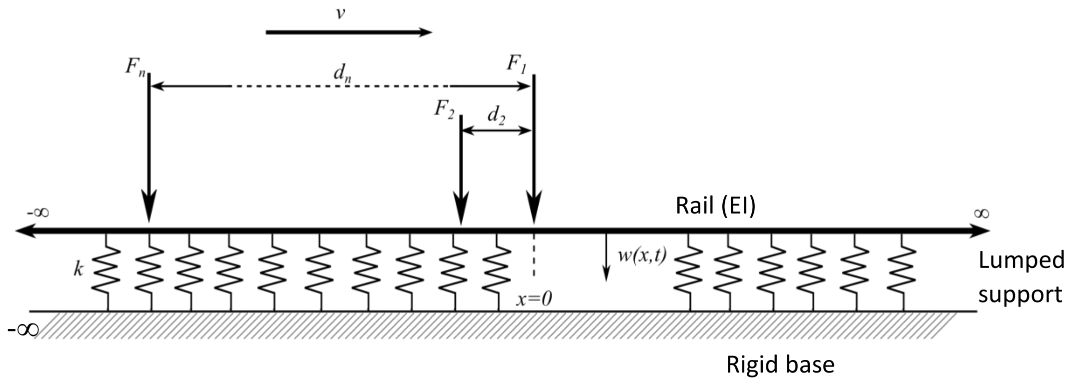



**Figure 2: Infinite beam on elastic foundation quasi static model**

The effective or combined stiffness of the system supporting the rails may be characterised by means of a *modulus*. This is defined as the stiffness of the equivalent springs supporting the rails in terms of the load per unit length needed to produce a unit displacement at the point of measurement on the rail. This type of modulus has units of force per unit length divided by displacement, for example, $MN/m^2$. The modulus at the level of the rails can be further divided into the contributions from the different components, such as the trackbed and the railpads. A recent innovation has been the use of under sleeper pads (USPs) to modify the support stiffness below the sleeper. This can be considered within the mathematical framework for trackbed support presented in this Guide, as shown later.

Models and analyses generally focus on vertical track deflections and stiffness, but the same principles also apply to lateral deflections and stiffness.

### 1.3 *Definitions used in this guide*

In this Guide the term *spring stiffness*, denoted *S*, is adopted to indicate a stiffness in terms of the point load needed to cause a unit deflection. *S* has units of force divided by deflection, for example MN/m (which is numerically the same as kN/mm). The term *modulus*, denoted *k*, indicates the line load needed to cause a unit deflection. The line load is quantified in units of force divided by length, for example MN/m, giving *k* in units of force divided by length squared, e.g. $MN/m^2$.

The spring stiffness or the modulus may relate to the trackbed (i.e. below the sleeper and including any USPs), the railpad, or the complete support system. These are denoted by the subscripts *trackbed*, *railpad* and *system* respectively.

The *composite* or *global* track stiffness depends on both the rail and its support system. It is defined solely as a spring stiffness, and is denoted by the subscript *composite*.

Stiffness values are commonly given per sleeper end or per rail. This means that when calculating associated deflections, wheel loads (rather than axle loads) and the bending stiffness of a single rail (rather than both rails acting together) are used.

A glossary of standard symbols and definitions of key terms is given in Appendix D.

## 2. THEORY

### 2.1 *Beam on Elastic Foundation model (BOEF)*

Ignoring dynamic effects, the beam on an elastic foundation model (Figure 2) gives the quasi-static rail displacement *w(0,t)* at a time *t* and at a reference point *x = 0*, resulting from a series of *n* loads $F_n$ each at a distance of $d_n$ from the reference point, all moving with the same velocity *v*, as

$$w(0,t) = \sum_{n=1}^{N} \frac{F_n}{2k_{system}L} e^{-\frac{vt-d_n}{L}} \left(\cos\left(\frac{vt-d_n}{L}\right) + \sin\left(\frac{vt-d_n}{L}\right)\right) \tag{1}$$

where *w(x,t)* is vertical displacement (m), $k_{system}$ is the support system modulus (N/m/m), *EI* is the bending stiffness of the rail (Nm$^2$) and

$$L = \sqrt[4]{\frac{4EI}{k_{system}}} \tag{2}$$

*L* is known as the characteristic length. *L* is a measure of how far away from the point load along the rails the deflection bowl extends; i.e. as the support system modulus reduces *L* increases. Equation (1) includes all of the wheel loads present, so that any overlapping of deflection bowls from adjacent wheels is accounted for. However, in most cases such overlapping is minimal and setting *v* = *d* = 0 and *N* = 1 in Equation (1), we obtain for a single point load

$$w(x) = \frac{F}{2k_{system}L} e^{-\frac{x}{L}} \left(\cos\left(\frac{x}{L}\right) + \sin\left(\frac{x}{L}\right)\right) \tag{3}$$

For x = 0, Equation (3) simplifies to

$$w(0) = \frac{F}{2k_{system}L} \tag{4}$$

Recalling our definition of composite or global track system stiffness $S_{composite}$ as the (point) load required to produce a unit deflection of the rail at the location where the load is applied, Equation (4) may be rearranged to give

$$S_{composite} = \frac{F}{w(0)} = 2.k_{system}.L \tag{5}$$

Equation (5) relates the composite track stiffness, $S_{composite}$, to the support system modulus $k_{system}$; although it must be borne in mind that the characteristic length *L* is itself a function of $k_{system}$ and the flexural rigidity of the rails, *EI* (see Equation 2).

$k_{system}$ arises from the railpad modulus $k_{railpad}$ and the trackbed modulus $k_{trackbed}$ acting in combination. These are related by the well-known formula for springs in series:

$$\frac{1}{k_{system}} = \frac{1}{k_{trackbed}} + \frac{1}{k_{railpad}} \tag{6}$$

For a quasi-static analysis[1], it does not matter that the trackbed and the railpad are separated physically by the rigid sleeper.

The railpad stiffness is in principle related to its fundamental properties (Young's modulus *E*, area *A* and thickness *t*) by

$$S_{railpad} = \frac{E.A}{t} \tag{7}$$

However the Young's modulus is rarely linear and many railpads are textured, making direct measurement of *A* and *t* problematic. Thus $S_{railpad}$ is usually determined experimentally from load-deflection tests over a load range representative of any pre-stress (clamping force) and train loading.

The error associated with neglecting the railpad is illustrated in Figure 3, for a railpad having a spring stiffness $S_{railpad}$ = 60 MN/m. This equates to a railpad modulus $k_{railpad}$ of 92.3 MN/m$^2$ for sleepers spaced at intervals $s$ = 0.65 m ($k_{railpad} = S_{railpad}/s$) and represents, approximately, a type of railpad in common use on the UK railway network. Figure 3 shows that as the trackbed modulus increases, the error associated with neglecting the railpad (i.e. assuming $k_{system} = k_{trackbed}$, indicated by the 1:1 line) becomes more significant. This is because the deflection associated with the railpad as a proportion of the whole increases as the trackbed deflection reduces. In any case the combined system modulus cannot be greater than that of the least stiff component.

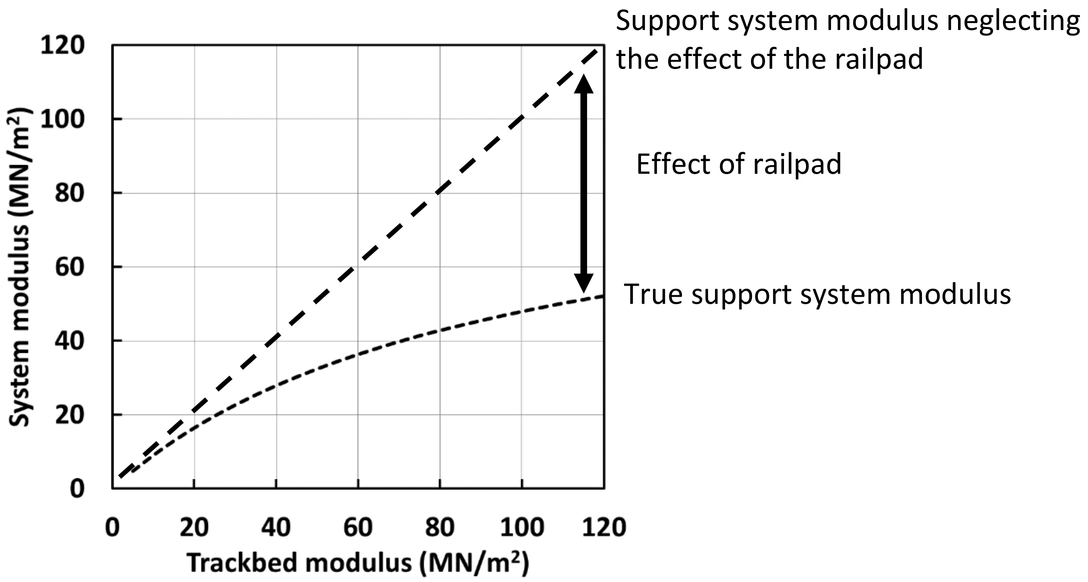



**Figure 3: Effect of railpad compliance on the overall support system modulus (railpad stiffness $S_{railpad}$ = 60 MN/m; sleeper spacing $s$ = 0.65 m; railpad modulus $k_{railpad} = S_{railpad}/s$ = 92.3 MN/m$^2$; CEN 60 E1 rail)**

## 2.2 *Shortcomings of the elastic track stiffness approach and the BOEF model*

The concept of a simple elastic track or component spring stiffness assumes that deformations caused on loading are fully recovered when the load is removed. For well-

[1] A quasi-static analysis does not automatically calculate loads that may arise from vehicle dynamic effects, and assumes that the accelerations of the track structure and the ground are negligible.

constructed and well-maintained track on a reasonably firm subgrade, this is approximately true: displacements are generally in the order of 1 – 2 mm ($10^{-3}$ m) during a typical loading cycle, of which all except for perhaps 1 – 20 nm (nanometre, $10^{-9}$ m) is recovered on unloading. Thus the ratio of elastic (recoverable) to plastic (irrecoverable) displacements is $10^5$ or $10^6$, and it is reasonable to regard the behaviour over a single loading cycle as reversibly elastic. (An exception to this is immediately following tamping or renewal, where initial bedding-in can be significant).

Selection of a single value for a modulus or a spring stiffness implies further that the material behaviour is linear. This is less likely to be true; some polymers used in railpads have hysteretic, nonlinear and possibly frequency-dependent stress-strain relationships, while soils and ballast are likely to have strain- as well as stress- dependent stiffness properties. Sleepers with voids below them will exhibit a dramatically non-linear response. Reasonable estimates of displacements of a non-linear elastic system can be made by adopting a secant (chord) stiffness value appropriate to the expected load. However, for exploring perturbations about a loaded state, the tangent stiffness at the load in question should be used.

Strictly, the springs shown schematically in Figure 2 do not act independently because the transmission of shear stresses in a continuum effectively causes them to interact. This is not usually a problem as long as measurement, interpretation and analysis are carried out according to the same behavioural model. Also, ballast and soils cannot carry tension. This is ignored in the BOEF approach, in which a slight upward movement is calculated beyond a certain distance from a single point load as the rail curves back to its undeflected position. However, for multiple loads representing whole trains, the error is negligible. Alternative approaches, in which the ground is represented as an elastic continuum of constant stiffness or stiffness increasing linearly with depth, or as a series of distinct elastic layers, are generally at least as problematic or become too site-specific to be useful as a general guide.

The sleeper supports are in reality discrete, whereas the BOEF model idealises the support offered to the rails as continuous. This convenient simplification is justifiable; conventional methods of structural analysis show that when a wheel load is mid-span there is usually no significant additional increment of deflection of the rail relative to the adjacent sleeper supports. However, there are exceptions to this; notably, corrugations on some types of slab track offering a very stiff support have been found to be related to the wavelength of the rail supports (Grassie, 2009).

The BOEF model essentially idealises the track system as a beam element with significant bending stiffness (the rails) on a quasi-elastic support. Where a stiffer soil layer or a stiff embankment overlies a softer soil, it might be more appropriate conceptually to consider the stiff soil layer or embankment as part of the “beam”. In these circumstances, a bespoke numerical model able to take into account the different subgrade stiffnesses at different depths might be more appropriate.

Finally, the BOEF model takes train loads as an input. It does not automatically account for dynamic effects such as the increases in dynamic loading likely to be associated with increases in local track deflection and train speed, although these can be estimated in a separate calculation and input manually. Also, the model is quasi static in assuming that accelerations of the track structure and the ground are not significant.

### 2.3 *Summary*

The various stiffness measures are summarised in Table 1, which gives some typical values and indicates how the different definitions are related using an illustrative example.

| Name | Symbol | Definition | Illustrative example value | Notes |
|---|---|---|---|---|
| Trackbed stiffness | $S_{trackbed}$ | Equivalent spring stiffness (load per unit displacement), measured on an unclipped sleeper | 50 MN/m per sleeper end (input) | Can be measured directly using a falling weight deflectometer (FWD) |
| Sleeper spacing | $s$ | Horizontal spacing between centres of adjacent sleepers | 0.65 m (input) | Typical value for UK track |
| Trackbed modulus | $k_{trackbed}$ | The load per unit length causing a unit displacement of the sleeper | 76.9 MN/m$^2$ per rail | = trackbed stiffness divided by the sleeper spacing, $k_{trackbed} = S_{trackbed} \div s$ |
| Railpad stiffness | $S_{railpad}$ | Equivalent spring stiffness (load per unit displacement), of the railpad in isolation | 60 MN/m per railpad (input) | Found from load deflection tests |
| Railpad modulus | $k_{railpad}$ | The load per unit length causing a unit displacement of the railpad | 92.3 MN/m$^2$ per rail | = railpad stiffness divided by the sleeper spacing, $k_{railpad} = S_{railpad} \div s$ |
| Rail flexural rigidity (per rail) | $EI$ | Bending stiffness of each rail | 6.23 MN.m$^2$ (input) | Value for E1 60 rail |
| Support system modulus | $k_{system}$ | The load per unit length causing a unit displacement at the rail | 41.95 MN/m$^2$ per rail | Calculated from Equation (6) |
| Characteristic length | $L$ | Arises from the BOEF analysis, see Equations (1) and (2) | 0.878 m | Calculated from Equation (2) |
| Composite track stiffness | $S_{composite}$ | The point load required to produce a unit deflection of the rail at the location where the load is applied | 73.65 MN/m | Calculated from Equation (5); depends on both $k_{system}$ (support) and $EI$ (rail) |

**Table 1: Summary of relationships between stiffness parameters and example values (shaded cells denote assumed typical input values used in calculating the derived parameters)**

## 3. IMPORTANCE OF TRACK STIFFNESS

In theory, a uniformly-loaded wheel will sit within the deflected profile of the rail and the train will be unaware of the support stiffness except when either it or the load changes, causing variations in rail level and associated vehicle accelerations and forces. However, the support stiffness affects the extent to which the wheel load is spread along the length of the track. A lower support stiffness results in a wider deflection bowl, spreading the load and reducing the contact stresses between the sleepers and the ballast and hence stresses throughout the substructure. However, this is at the expense of increased rail bending. This is illustrated in Figure 4, which shows (a) rail deflections for a stationary 10 tonne wheel load on UIC 60 rail, with different support system moduli $k_{system}$; and (b) the percentage of the wheel load transferred away from the point of application in each case.

Figure 4 shows that, as the support system modulus is reduced, the maximum rail deflection increases but the maximum load reduces and the load is spread over a greater length of track. This is because, as the support system modulus is reduced, a greater length of rail is subjected to bending. Thus a reduced support system modulus may be beneficial for the trackbed, in spreading the load more effectively away from the point of application. However, the trackbed is only one part of the track system and the increased deflections and rail bending tend to increase the stresses in, and reduce the life expectancy of, other track system components such as sleepers, rails and rail to sleeper fastenings. Overall, a soft support is generally associated with poor track system performance and an increased maintenance requirement (Meissonnier & Cleon, 2000; Hunt, 2000; Sussman et al., 2001).

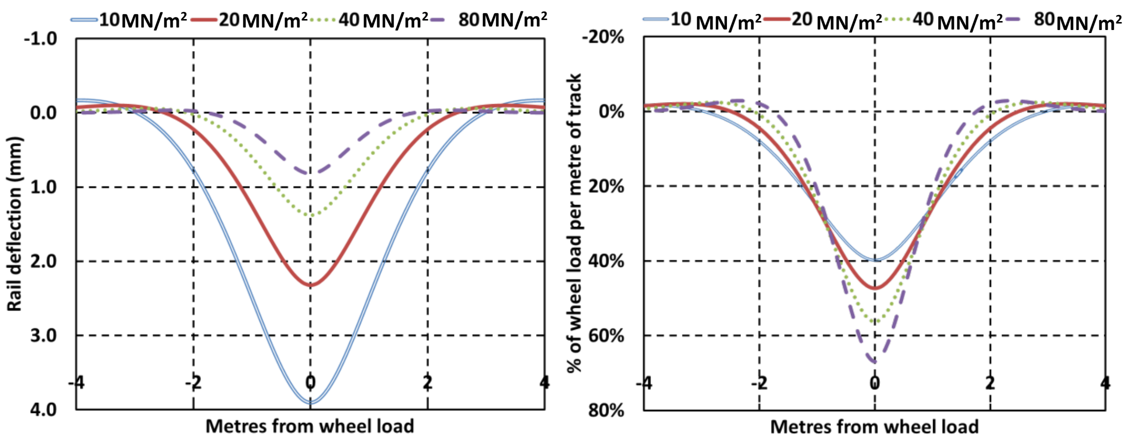



**Figure 4: BOEF model calculations showing (a) rail deflection for a 10 tonne wheel load on UIC 60 rail, with different support system moduli $k_{system}$; and (b) the redistribution of the wheel load about the point of application in each case.**

As a further and more detailed illustration, Figure 5 shows graphs of displacement against time for an individual sleeper calculated using the BOEF model for the passage of the first two vehicles of a class 390 (Pendolino) trainset travelling at 50 m/s, for different support system moduli and the input data on static wheel loads and spacings given in Table 2.

Figure 5 shows that the greatest deflections occur beneath the wheels, and that deflections increase as the support system modulus is reduced. Not only do the displacements increase with reducing support system moduli, but also the relative ratios of the peak-to-trough movements associated with bogie and axle passages change. At lower support system moduli, individual axles appear as minor undulations on the bogie passing deflection bowl while between adjacent bogies on adjacent cars the rail does not return to its original level. At higher support system moduli, individual axles are more distinct and the rail returns almost to its original level between axles on the same bogie. In all cases, but especially when the support system modulus is low, uplift of the rail occurs just prior to and after bogie/axle passage; this is a consequence of the flexural rigidity of the rail, which means that some length is required for the track to curve back to a level (un-deflected) position.

| Parameter | Symbol | Value | Units |
|---|---|---|---|
| Average Pendolino wheel load at 180% tare (Le Pen, 2008) | $F_n$ | 72.56 | kN |
| Typical value for E1 60 rail | $EI$ | 6.23 | $MNm^2$ |
| Axle spacing relative to axle 1 on first car | $d_n$ | 0, 2.7, 17.0, 19.7 | m |
| Car length when coupled | $d_c$ | 23.9 | m |
| Train speed | $v$ | 50 | m/s |

**Table 2: Train/track data used for BOEF Pendolino simulation in Figure 5**

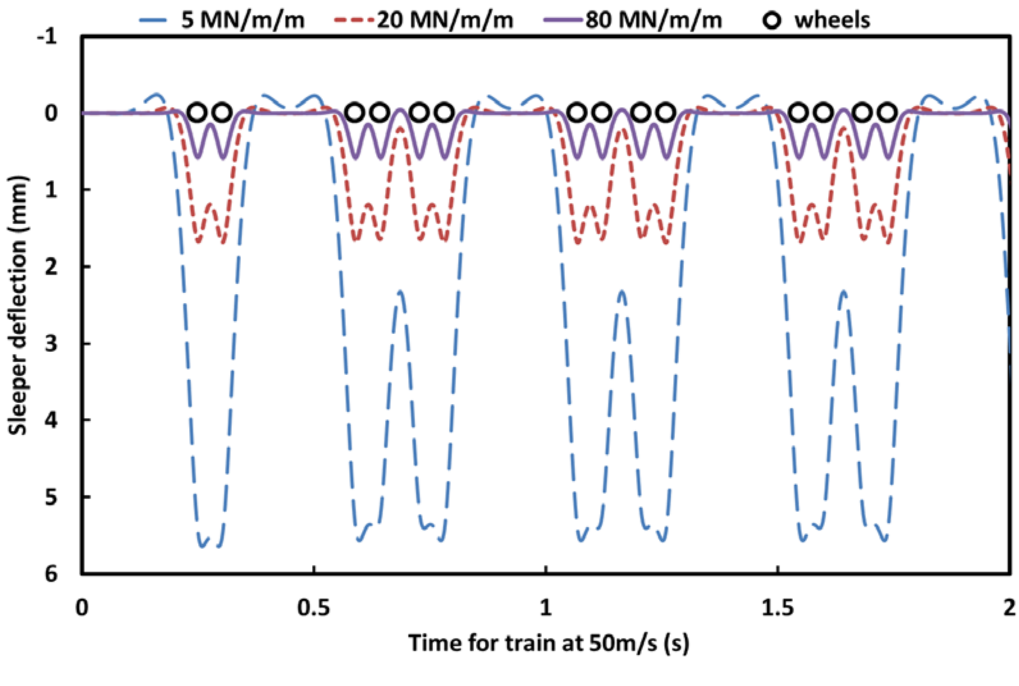



**Figure 5: Track deflections for different values of support system modulus: BOEF simulation for first two vehicles of a Pendolino train travelling at 50 m/s**

In reality, even on well-constructed and well-maintained track, the support system moduli, component spring stiffnesses and composite stiffnesses will vary from sleeper to sleeper. Dynamic effects (for example associated with vertical and horizontal accelerations including curving) will likewise cause the actual wheel or axle loads to vary from the static case. If the speed of the train approaches the Rayleigh Wave velocity of the ground on which the track is built, displacements may be amplified in what is generally termed a *critical velocity effect*.

Increased variability in the support stiffness at sleeper positions along the track leads to more onerous dynamic loads, which increase with train speed. Like a support that is too soft, an erratic or rapidly varying track stiffness leads to accelerated track degradation (e.g. Hunt, 2000; Sussman et al., 2001). Differential track stiffness often becomes reflected in the development of differential permanent track settlements along the affected section. This is most obvious on transitions from soft to hard support or vice versa such as the approaches/exits from bridges (e.g. Coelho et al., 2011; Paixão et al., 2015; Varandas et al., 2014; Milne et al., 2014), and at switches and crossings.

In short, the track stiffness needs to be reasonably, but not excessively, stiff. A stiffer support reduces rail bending and tends to minimise the general variability in track stiffness along the length of the track. However, some "give" (flexibility) in the support is needed to spread loads, because the vertical contact between the wheel and the rail is itself extremely stiff. In general terms, the track stiffness can be too high, too low, or too variable. However, there is a range in between "too high" and "too low", within which track performance has been found to be acceptable for existing track. Numerical values of stiffness for well and poorly performing track are indicated in sections 4.2 and 4.3, and in the case studies in Appendix A.

The above has been written in the context of the vertical track stiffness. Similar comments apply in principle to the lateral track stiffness (although the primary wheel-rail contact in this case is rather less stiff than in the vertical direction, so that the need for compliance laterally is perhaps reduced); and to the torsional track stiffness, which may relate to the resistance to twist of an individual rail or to the resistance of the track to asymmetric loading, for example associated with trains curving at a high cant deficiency.

## 4. MEASUREMENT OF TRACK STIFFNESS AND DEFLECTIONS

### 4.1 *Categories of measurement method*

There are three broad categories of method for determining some form of track stiffness parameter from field measurements. These are:

(1) by measuring the deflection (of the sleeper or the rail) in response to a train passing (or more rarely, a specifically applied load), and calculating a stiffness from the load and the measured deflection. The principal methods of deflection measurement are by:
- geophones, integrating the signal once,
- accelerometers, integrating the signal twice,
- video camera followed by digital analysis of the images captured (digital image correlation (DIC),
- laser systems, and
- multi-depth-deflectometers (MDD).

These are summarised in Table 3 and in Appendix B.

The load may be:

- assumed on the basis of the train type (which is disadvantageous in that the weight of passengers and additional loads due to vehicle dynamic effects must be estimated),
- estimated on the basis of data from load gauges for the same train at a different location (where vehicle dynamic loads may be different, even if the actual gross static loads are captured more closely); or
- determined from strain gauges applied to the rail at the point of measurement (for which permission from the infrastructure owner / operator may be difficult to obtain, and which still have to be calibrated by the application of a series of known loads).

(2) using a falling weight deflectometer (FWD) on a sleeper unclipped from the track (which therefore measures the sleeper support stiffness directly), or some other track-mounted device. Some such methods are summarised in Table 4 and Appendix B.

(3) using signal processing techniques to infer a stiffness from the ratios of principal harmonics in the signal obtained from any of the instruments listed in (1) above (Le Pen et al, 2016), without the need to know the axle load. This is a highly significant recent advance.

In each case, data processing and analysis requires an assumed model of track system behaviour, which is usually taken as the beam on an elastic foundation (BOEF). In most cases, measurements are made with the track system in place and the parameter back-calculated after adjustment for the effect of the rails is a linear modulus (of either the trackbed or the support system, depending on where deflections are measured) in $MN/m^2$ or compatible units. The FWD differs in that it directly determines a trackbed spring stiffness per sleeper end (in MN/m). This may be converted to a modulus by dividing by the sleeper spacing *s*. For comparability with the other methods, and to obtain the support system modulus as seen by a train, a further adjustment would be needed to take account of the railpad stiffness as illustrated in Table 1.

A summary of each of the main measurement techniques listed in Tables 3 and 4 is given in Appendix B.

| **Instrument and parameter measured** | **Attributes, advantages (+) and disadvantages (-)** | **References for further reading** |
|---|---|---|
| Geophone: measures velocity | **+**Simple to deploy<br>**+**Requires only one stage of integration and filtering<br>**-**Train principal vehicle passing frequency must be above geophone natural frequency (typically 1 Hz)<br>**-**Signal processing requires skill and care | Bowness et al., 2007<br>Priest et al., 2013<br>Le Pen et al., 2014 |
| Accelerometer: measures acceleration | **+**Micro Electrical Mechanical devices (MEMs) are low cost<br>**+**Simple to deploy<br>**-**Requires two stages of integration and filtering<br>**-**Less reliable at lower frequencies (typically < 3 Hz)<br>**-**Signal processing requires skill and care | Lamas-Lopez et al., 2014 |
| Digital Image Correlation (DIC) of high speed filming: measures deflection | **+**Can be used for any realistic speed of train<br>**+**Accurate at lower speeds where accelerometers and geophones tend to be less reliable<br>**-**Susceptible to vibration at the camera location (groundborne and wind), although methods to correct for this are available<br>**-**Line of sight may be problematic | Bowness et al., 2007<br>Le Pen et al., 2014<br>Murray et al., 2014 |
| Laser based systems: measure deflection | As for DIC although differing processing methods may result in differences in accuracy | Paixão et al., 2014<br>Kim et al., 2014 |
| Multi depth deflectometer: measures deflection | **+**Will give an absolute measure with no zero shift and will in principle measure permanent settlements<br>**-**Requires fixed datum at depth<br>**-**Can be problematic to install | Gräbe & Shaw, 2010<br>Priest et al., 2010<br>Mishra et al., 2014 |

**Table 3: Summary of methods for measuring dynamic track displacement**

| Instrument | Measures | Attributes, advantages (+) and disadvantages (-) | References and further reading |
|---|---|---|---|
| Falling Weight Deflectometer (FWD) | Deflection of unclipped sleeper and nearby track in response to a known falling mass | +Isolates the support from the trackbed<br>-Does not show the effect of any voiding | URS, 2014<br>(URS is now known as AECOM)<br>Brough et al., 2013<br>Sharpe & Govan 2014 |
| Lightweight Drop Test (LWD) | Measures the modulus of stiffness of subgrade and formation | +Simple to deploy<br>-Can only measure surface stiffness<br>-Cannot be used on gravelly surfaces | |
| Vehicle based systems | Frequency/track/ load response | +Continuous measurement<br>+Can cover long lengths of track<br>-In reality a rolling average evaluated over a length related to the vehicle speed<br>-Currently experimental and rarely used | Hosseingholian et al., 2011 (Portancemeter)<br><br>Roghani, et al, 2015. (Mrail) |

**Table 4: Summary of methods of track stiffness measurement that do not rely on the passage of a service train to provide the load**

## 4.2 *Typical measured values*

Measurements have demonstrated that the inferred trackbed modulus may vary significantly from one sleeper to another, even over short lengths of track with nominally similar support conditions (e.g. Oscarsson, 2002; Bowness et al., 2007; Le Pen et al., 2014, Murray et al., 2014). Le Pen at al. (2016) carried out geophone measurements at a number of sites in the UK (summarised in Table 5), all on apparently well performing sections of track. Measurements were taken on up to 10 adjacent or alternate sleepers. Figure 6 illustrates the measured sleeper movements and inferred support system moduli (taking into account the effects of the railpad).

| Site | Notes | Type of train (axle weight tonnes) | Speed | Radius of track (m) | Cant (mm) | Date of measurements |
|---|---|---|---|---|---|---|
| 1 | Stoneblown 2013 | Class 390 (12.9)<br>Class 221 (14.1) | 125 mph (200 kph) | 2777 | 72 | Aug 14 |
| 2 | Renewed 2013 | Class 171 (11.0) | 70 mph (112 kph) | 2777 | 72 | May 14 |
| 3 | Opened 2003 | Class 395 (10.9) | 140 mph (225 kph) | Straight | 0 | May 14 |
| 4 | Renewed 2012 | Class 377 (11.0) | 75 mph (120 kph) | Straight | 0 | Feb 13 |

**Table 5: Study site and train data**

Sleepers at sites 1, 2 and 4 were spaced at 0.65 m centres and fitted with Pandrol type 6650 pads. At Site 3, the sleepers were spaced at 0.6 m with Vossloh ZW900 pads. The sleepers at Sites 1, 2, 3 and 4 were respectively monoblock (G44), monoblock (EG47), twin block and monoblock (EG47). Rails at sites 2 and 4 were CEN 56 E, and at sites 1 and 3 CEN 60 E1.

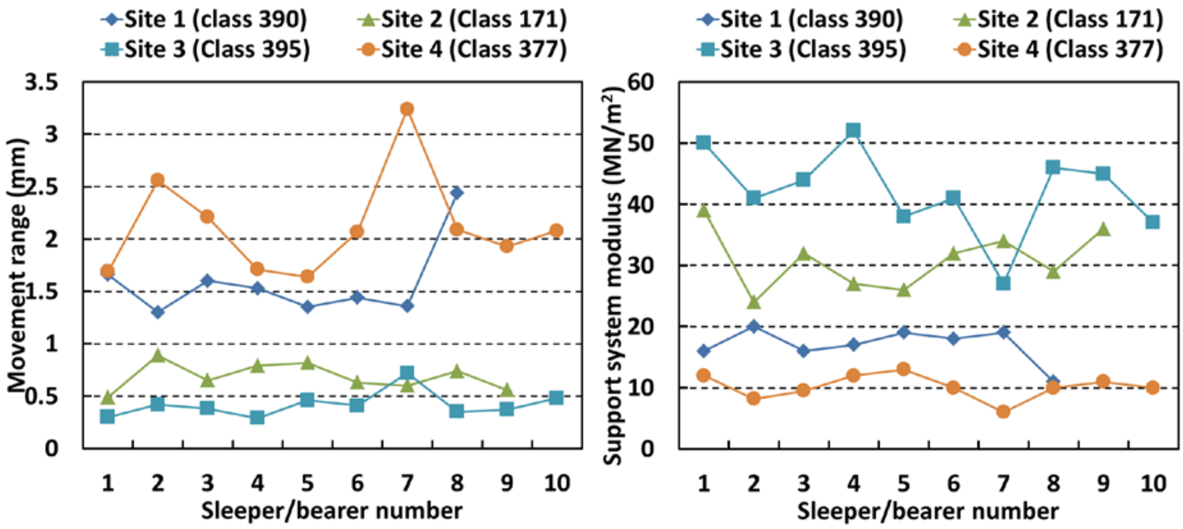



**Figure 6: Data from well performing track: (a) measured sleeper movements (b) inferred support system modulus seen by the rail**

Figure 6 shows that for apparently well-performing track, the range of sleeper movements and implied support system moduli at a given site is reasonably consistent, although there is some variation about the average. However, Figure 6 also shows that the average support system modulus varies considerably from site to site, perhaps as a consequence of differing support geologies.

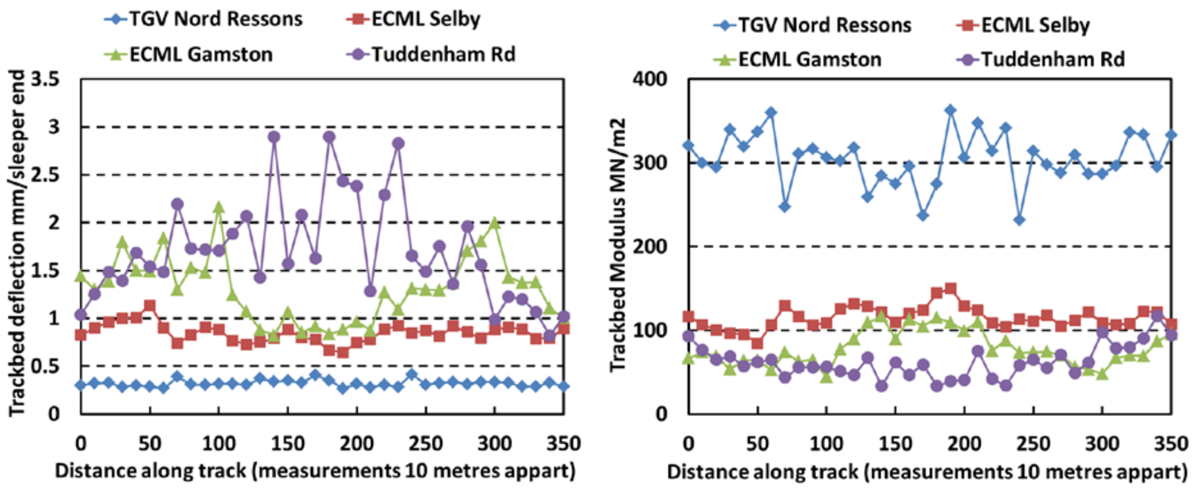



**Figure 7: Examples of FWD surveys (a) trackbed deflection (b) inferred trackbed modulus (sleeper spacing 0.65 m)**

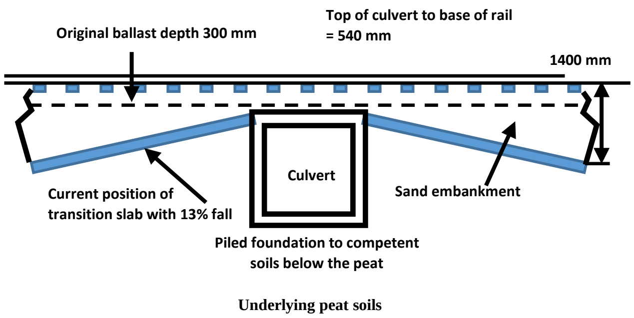
The measurements shown in Figure 6 were made at individual sleepers, closely spaced over short lengths of track. Longer lengths of track may be characterised using the falling weight deflectometer test (FWD) during a full possession. Figure 7 shows a range of trackbed deflections, corresponding to a range of trackbed stiffnesses, determined using the FWD test with measurements at approximately 10 m intervals. Applying a simple load transfer model (see Appendix B) and an estimate of the layer thicknesses, FWD data can also be used to estimate the contributions to deflection from the sub-ballast and the subgrade.

The trackbed modulus shown in Figure 7(b) for the line labelled TGV Nord Ressons is particularly high. However, this is the trackbed modulus; the rail support system modulus will be substantially lower owing to the effect of the railpads. For example, if the railpad modulus were 100 MN/m$^2$, the combined support system modulus could not be higher than 100 MN/m$^2$, whatever the trackbed modulus. Extrapolation of Figure 3 suggests it is unlikely to be greater than about 70 MN/m$^2$.

Figures 6 and 7 show that well performing track is characterised by a relatively consistent support modulus; and that the acceptable range of support system moduli is quite wide (between 10 and 50 MN/m$^2$ according to Figure 6b). Poorly performing track is characterised by a greater variation in local support modulus (and hence deflections); this is apparent in some of the case studies in Appendix A. Deflection measurements at poorly performing transition sites are considered in section 4.3.

### 4.3 *Measured track deflections at poorly performing transitions*

Large changes in track stiffness support often occur at transitions from or onto hard structures such as bridges or culverts. If the transition design is ineffective major geometry faults develop, which require regular reactive maintenance to manage. Two examples are given in Figures 8 and 9.



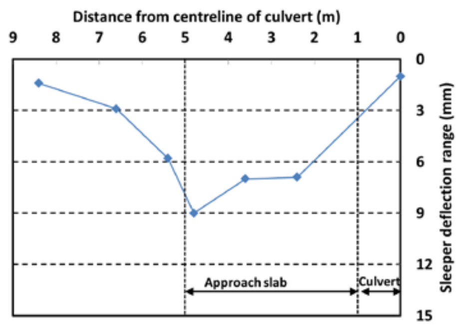




**Figure 8: (a) General arrangement and (b) dynamic displacement range (from geophone measurements) for a typical train passage over a piled culvert within an embankment founded on soft ground (from Coelho *et al.*, 2011)**

Figure 8 shows data from of an ineffective transition (Coelho *et al.*, 2011). At this location, the track on the embankment (which was founded on soft soils including peat) settled relative to the track over the culvert (which was founded on piles driven into deeper, much stiffer ground). Conventional survey measurements showed that over an 8 month period the approach track settled by 20 mm while the level of the track over the culvert remained virtually unchanged. The resulting differential settlement led to large sleeper deflections on the approach to the culvert of up to 9 mm as trains passed. On the culvert itself deflections remained small (less than 1 mm). Regular maintenance, packing and lifting of the track was required to maintain safe train operation but did not address the underlying cause.

Figure 9 shows a transition from an embankment onto a bridge. At this location, the problems arose from the rapid change in support stiffness between a relatively soft embankment suffering drainage problems and the relatively stiff bridge. Measurements do not show the rail movements on the bridge, but these would have been significantly less than the 5 mm or so on the approach. Repeat wet beds and voids (up to 20 mm) below sleepers developed as the embankment settled relative to the bridge. Continuing differential settlement required weekly maintenance interventions (lifting and packing) to enable safe operation, again without treating the underlying cause.

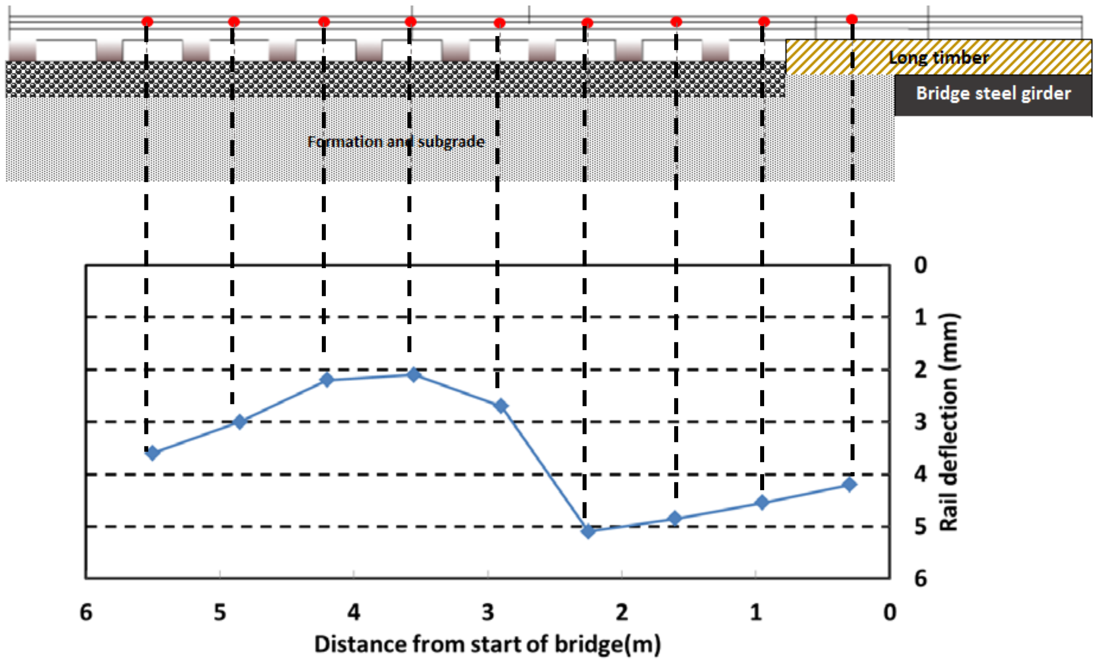



**Figure 9: General arrangement and maximum dynamic displacement at the approach to an overbridge (measurements by high speed filming and DIC)**

## 5. IDENTIFYING AND REMEDYING TRACK STIFFNESS PROBLEMS

The characteristics of track whose support is too stiff, too flexible or too variable (including at a transition onto or off a hard substructure) are summarised in Table 6. In addition to those listed in Table 6, symptoms nearly always include a rapid or persistent loss of track geometry that is difficult to address using normal maintenance methods. Mitigation may always include temporary or permanent speed restrictions. References in Table 6 are to case studies in Appendix A and are identified in square brackets [A1 to A10].

| Problem / cause | Typical symptom(s) | Example conditions and case studies [sections in Appendix A] | Potential remedies |
|---|---|---|---|
| Support stiffness generally too high: limited spread of load leads to excessive localised stresses. | Damage to components e.g. rail clip breakage.<br>Ballast attrition.<br>Rail surface damage.<br>Ground vibrations. | Ballast onto a rigid structure such as a concrete bridge deck [A10], concrete tunnel invert or directly onto bedrock. Also, modern earthwork fills can be very stiff [A10]. | Reduce support stiffness, e.g. by installing softer rail pads, under-sleeper pads, ballast mats over structures, sand blanket and/or increased depth of ballast and sub-ballast. |
| Support stiffness generally too low: increased variability in stiffness from sleeper to sleeper leads to increased variability in dynamic loads and generally increased rates of plastic settlement and increased rail bending. | Relatively rapid loss of track geometry.<br>Rail bending fatigue (may not be visible).<br>Strained clips and clips falling out of housings.<br>Broken rails.<br>Ground vibrations. | Poor quality fill materials creating a historically soft earthwork [A8] or presence of soft natural geology, e.g. alluvial clays/peat [A2, A5, A9].<br><br>High water table and/or poor drainage [A4]. | In severe conditions, where a retro fitted solution is required, use micro-piles and other reinforcement techniques (geowebs and geogrids) to increase track stiffness.<br><br>Improve drainage (e.g. through the use of counterfort drains). |
| Inadequate transition of support stiffness e.g. onto or off a substructure or from slab onto ballasted track.<br><br>Stiffness locally too variable. | Damage to rails and components.<br>Ballast attrition.<br>Excessive voiding below sleeper.<br>Loss of ballast shoulder. | Running onto/off of a hard substructure e.g. earthwork onto/off of a concrete bridge [A4, A7, A10], or a direct fastening bridge. | Manage / control the stiffness on the hard structure through the selective application of softer railpads, under sleeper pads and ballast mats (to reduce stiffness).<br><br>Manage / control the stiffness on the approach/exit to the structure e.g. (1) for new build install designed earthworks transition with appropriate wedges of stiffer fill material.<br>(2) in severe conditions use micro-piles and other reinforcement techniques (geowebs and geogrids) to balance track stiffness.<br><br>Consider providing lateral restraint (ballast containment) to approach/ exit earthworks to reduce potential for differential settlement. |
| Discontinuity in support stiffness - a localised hard spot. | Ground vibration.<br>Pad wear.<br>Ballast and sleeper attrition (white spots/surface dust visible in dry conditions).<br>Ballast migration on canted track where locally increased dynamic loading leads to excessive track deflection. | An insufficiently buried under-track crossing (UTX) or culvert [A3].<br><br>Cast manganese crossings.<br><br>Variable subgrade conditions. | Equalise stiffness through the selective application of softer railpads, under sleeper pads and/or ballast mats (to reduce the support stiffness locally as required).<br><br>In the short term apply targeted packing extending either side of the obvious fault location.<br><br>Increase ballast and sub-ballast depth to reduce support system stiffness. |
| Discontinuity in support stiffness – a localised soft spot. | Damage to track fastenings.<br>Ballast voiding.<br>Sleeper and ballast attrition.<br>Ballast migration on canted track where locally inadequate support stiffness leads to excessive track deflection. | Wet beds [A1].<br><br>Different sleeper types such as at S&C [A6]. | Excavate and reconstruct the track bed formation layer with use of geosynthetic filter and/or sand-blanket to prevent ballast fouling from below. Improve drainage (ensure drains are clear).<br><br>Local specification of additional compliant layer e.g. under sleeper pads or under ballast mats. |

**Table 6: Problems potentially associated with inadequate, excessive or discontinuous track system support stiffness**

In many cases, inappropriate track stiffness will be the primary cause of faster track geometry degradation rates but different initiation mechanisms are also possible. For example, a wet bed may initiate as a result of a “knocking” (dipped or slightly peaked) weld or a local hard spot, either of which could give rise to additional dynamic loads (potentially both vertical and horizontal) that damage the trackbed. A further major contributor to poor ride quality in certain locations, particularly embankments constructed of plastic clay, is the cyclic shrinkage and swelling associated with seasonal changes in weather and vegetation.

Once a local trackbed stiffness fault has developed, it must be remedied and any remaining trigger removed. For example, remediation of a dipped weld would not be sufficient on its own because once a susceptible track bed soil has been damaged, it too will need to be repaired. Further discussion is given in a number of illustrative case studies in Appendix A.

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## APPENDIX A: CASE STUDIES ILLUSTRATING PROBLEMS ARISING FROM INADEQUATE, EXCESSIVE OR TOO VARIABLE TRACK STIFFNESS AND POSSIBLE REMEDIATION METHODS

Appendix A contains a number of case studies illustrating some of the effects of a track stiffness that is too high, too low or too variable. In each case, remedial measures are described; these are not intended as exemplars of good or standard practice, but as examples of remedial measures that have been tried, usually with some degree of success. In some cases they represent part of a research investigation.

Appendix A is intended to be "live", with the facility to add further case studies in the standard format as they become available. The case studies selected are characterized by good measurement data, so that an evidence base of potential remedial solutions that have (or have not) worked can be built up. The current case studies, and the reason(s) for their inclusion, are given in Table A1. In some cases a reference to a more detailed publication/report is given for further details.

| Section | Nature of the problem | Case study name | Remediation implemented | Contributing authors |
|---|---|---|---|---|
| **A1** | Locally too soft | Monk's Lane | Renewal | G. Watson, L. Le Pen, A. Hudson, W. Powrie |
| **A2** | Too soft | Gravel Hole | Micro piles | P. Musgrave, P. Sharpe, L. Le Pen, G. Watson, W. Powrie |
| **A3** | Locally too hard | North downs tunnel | Local packing | M. Hayward, S. Morley , D. Milne, L. Le Pen, W. Powrie |
| **A4** | Locally too variable | Bishops Stortford | Counterfort drains | G. Barnard |
| **A5** | Too soft | Grays and Purfleet | Geoweb | G. Barnard, P. Sharpe |
| **A6** | Locally too variable | Reading | Under sleeper pads | G Watson, L. Le Pen W. Powrie |
| **A7** | Locally too variable | Fishbourne | Tamping (ineffective) | L. Le Pen, G. Watson, W. Powrie |
| **A8** | Too soft (lateral and vertical) | Marlborough | Micropiles | P. Musgrave, M. Wehbi |
| **A9** | Too soft | Woodacre | Micro piles | P. Musgrave, M. Wehbi |
| **A10** | Too hard | Öbisfelde DB/TTCi | Under Ballast Mats and Under sleeper pads | P. Sharpe |

**Table A1: List of case study sites, problems encountered and remedial actions taken**

## A1. Monk's Lane (locally too soft): Remediation of mud pumping

*Site overview*

The site lies in a cutting at a high point in the local topography which, immediately adjacent to the study zone, reaches 4 m above track level. The track has a 1:100 gradient and is on a transition between a curve (2510 m radius, 40 mm cant) and a preceding straight section. The geology is alluvial clay, silt, sand and gravel overlying mudstone. Service trains are Class 171 diesel multiple units travelling at up to 70 mph. The site suffered from mud pumping. It is possible that water runoff from the surrounding higher ground ponding on top of the underlying clay was a contributing factor; however mud pumping was apparent on only one side (Figure A1.1a: the localised wet bed on the left hand track is lighter in colour than the surrounding ballast, as a result of the drying of clay pumped to the surface). In Figure A1.1b, ponded water can be seen within the crib (i.e. the space between adjacent sleepers). Previous remediation attempts, involving the local digging out of the ballast and repacking beneath the voided sleepers with chippings, had not resolved the problem on a long term basis.

**Figure A1.1: Photos prior to remediation (a) Site is circled (b) Close-up of worst region.**

Prior to the current remediation this stretch of track was described as "very poor" with a mean standard deviation of 4.9 mm for the 35 m wavelength vertical top. The rate of deterioration of 1 mm/year was noted as 'high - indicative of potential formation problems'.

*Remediation*

The remediation was a renewal involving replacement of the track, with the ballast dug out to a depth of 200 mm below sleeper base level over several hundred metres of track on either side of the study site. A micro-porous filter sandwiched between geotextile layers was used to separate the subgrade and fouled sub-ballast from the new ballast.

*Measurements*

Trackside measurements were made using geophones prior to, immediately after and 5 months after remediation (May 2013). The instrument array is shown in Figure A1.2.

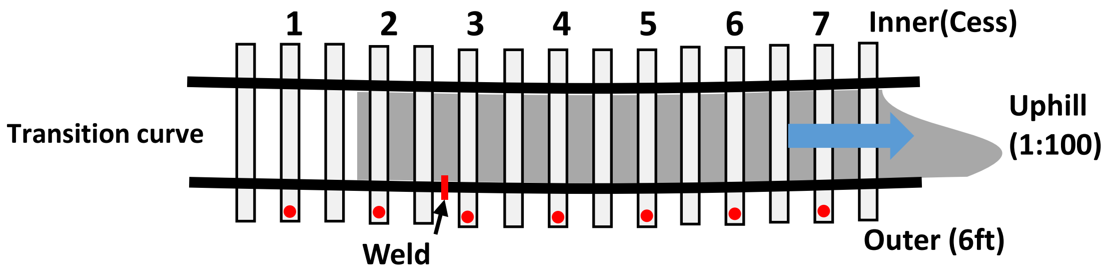


**Figure A1.2: Site layout prior to renewal showing the approximate extent of the wet bed (in grey) and the vertical geophone locations (circles). The arrow indicates the direction of train travel.**

Figure A1.2 indicates a weld next to sleeper 3. This could have been a potential trigger for the development of the mud pumping by causing increased dynamic loads. However, welds were also present on the adjacent track, where no mud pumping occurred. It may be significant that the mud pumping site is on a transition curve potentially more prone to bogie hunting and the associated additional erratic horizontal loads.

*Results*

Figure A1.3 shows the ranges of movement of the instrumented sleepers. The largest movements (at sleepers 2, 3 and 4 prior to renewal) were between 2.5 and 4.0 mm, indicative of soft support conditions. After renewal the movements reduced to between 0.5 and 1.0 mm, indicative of well supported track. There was also much less variation in movement between sleepers.

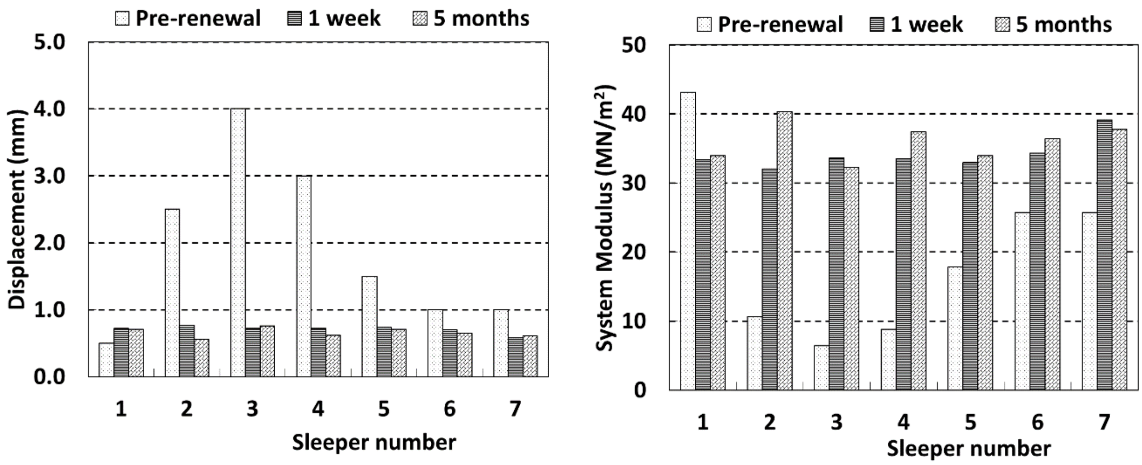


**Figure A1.3: Class 171 (a) Average sleeper vertical deflections (b) Support system modulus**

*Observations*

Measurements show that while repeated localised digging out and re-packing with chippings pea/gravel was ineffective, the renewal appears to have resolved the problem at this site, at least over the period of monitoring. The support system modulus following renewal is consistent and relatively high, indicative of well performing track.

*Further information*

Hudson, A., Watson, G. V. R., Le Pen, L. & Powrie, W. (2016). Remediation of mud pumping on a ballasted railway track. *Proc 3rd International Conference on Transportation Geotechnics (ICTG)* 4-7 September 2016. Guimarães, Portugal.

## A2. Gravel Hole (too soft): Subgrade stiffening using micropiles

*Site overview*

Large track displacements began to occur at this site following an increase in line speed from 160 kph to 200 kph in 2006. The track is located in a cutting with a high water table; and is underlain by a horizon of peat, over layers of stiffer sand, clay and gravel. The peat layer varies from 1.4 m to 4 m in thickness, and its low stiffness was considered to be the main cause of the large track movements. Following a recent renewal there was no obvious indication at the track surface (Figure A2.1) of anything untoward.

**Figure A2.1: (a) Prior to remediation, Sept 2013 (b) Train approaching zone**

*Remediation*

Micro piles with enlarged caps were placed in the cribs between alternate sleeper pairs to transfer loads down to competent soils below the peat (Figure A2.2). The remediation was carried out in or around January 2014, followed by stoneblowing in March 2015.

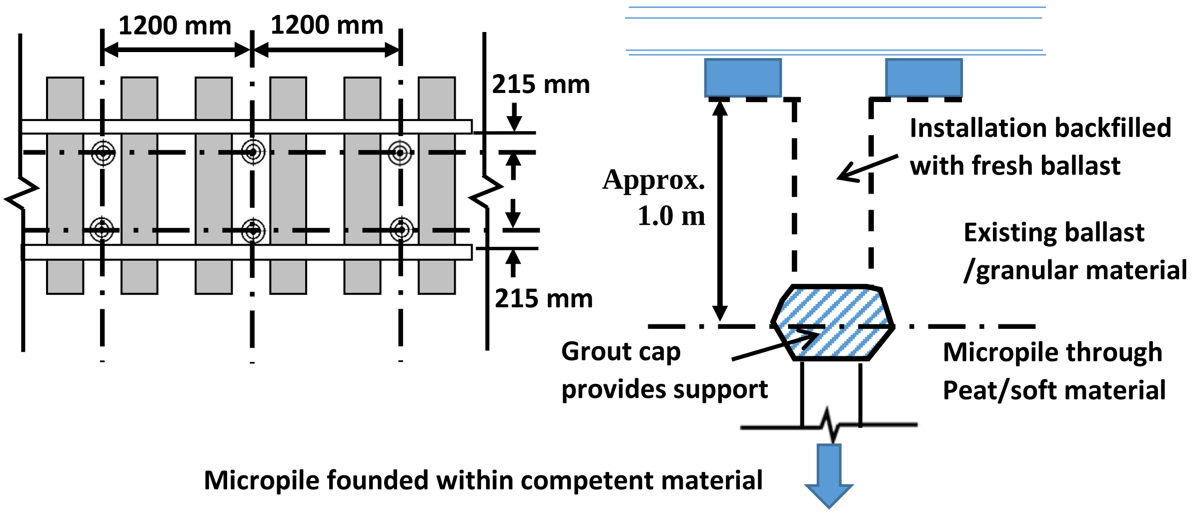


**Figure A2.2: Remediation design (not to scale); (a) plan; (b) illustrative elevation of individual pile**

*Measurements*

Nine geophones were mounted on every other cess side sleeper end to quantify track movements before (September 2013) and after (November 2014) remediation. During an interim visit in June 2014, 13 broken rail clips over a length of 26 sleepers through the middle of the site were noted. These were replaced prior to the November 2014 visit, during which only one broken clip was observed. Track geometry data from measurement trains were also available for evaluation.

*Results*

For simplicity, geophone data from only two trains are compared in Figure A2.3. These were nine car class 390 Pendolinos travelling at 52m/s and 57m/s during the September 2013 and November 2014 visits respectively. Figure 4 shows track geometry data from 2006 to 2016.

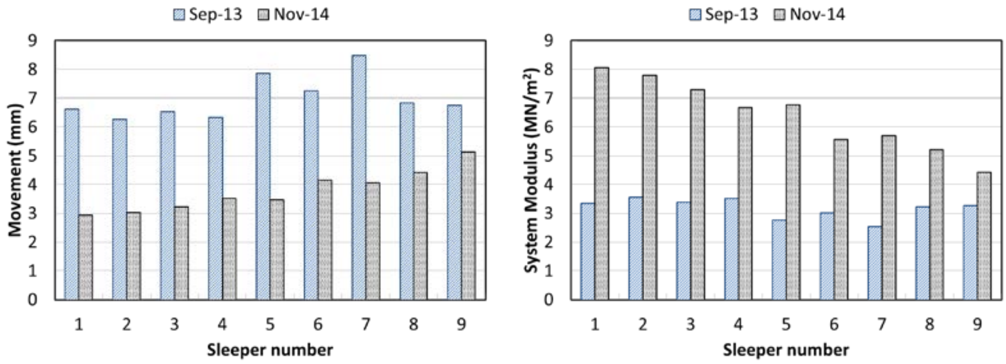


**Figure A2.3: (a) Average sleeper vertical deflections (b) Support system modulus**

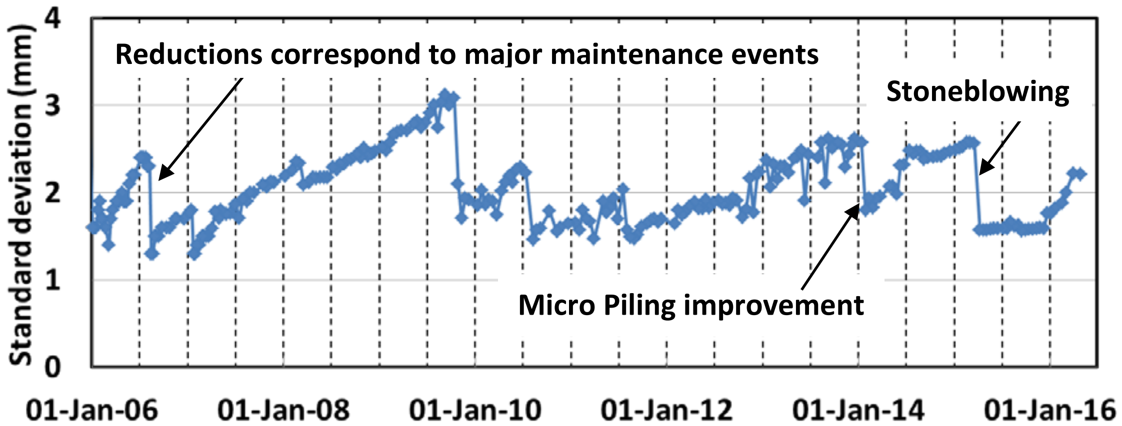


**Figure A2.4: Vertical (worst top) standard deviation over a 35 m wavelength for a 220 yard length of track including the study site**

*Observations*

The placement of the micro-piles significantly reduced deflection magnitudes (Figure A2.3). However, the site continues to exhibit signs of soft support conditions, for example the broken railclips observed during the June 2014 visit. The track geometry data showed a step improvement following micropiling (January 2014) and again following stoneblowing (March 2015). In any future remediation of this type it may be preferable to place micro piles in all (rather than alternate) cribs.

*Further information*

Duley, A., Le Pen, L., Thompson, D., Powrie, W., Watson, G.V.R., Musgrave, P., Cornish, A. (2014). Modelling and measurements of critical train speed effects and associated track movements. *Proc International Conference on High Speed Rail*, 8th-10th December 2014 Birmingham, UK. Available online: http://eprints.soton.ac.uk/381370/

## A3. North Downs Tunnel (locally too hard): Shallow under track crossing

*Site overview*

The site is located just beyond two concrete under track crossings (UTX) spaced at 6 m centres (Figure A3.1). These UTXs are robust concrete structures with a rectangular profile 2.4 m wide and a level top 0.7 m below sleeper base. They provide a comparatively rigid and more settlement resistant region below the track. A twist fault had developed starting from the trailing edge of the second UTX. There was evidence of ballast attrition and voiding extending over seven sleeper bays. The track has a gradient of 1 %, a radius of 4900 m and a design cant of 130 mm. Traffic consisted of high speed trains operating at 300 kph and 225 kph. Data from track recording vehicles indicated that both UTX locations were associated with relative high spots with a low spot between them, and a geometry irregularity having a wavelength similar to the UTX spacing.

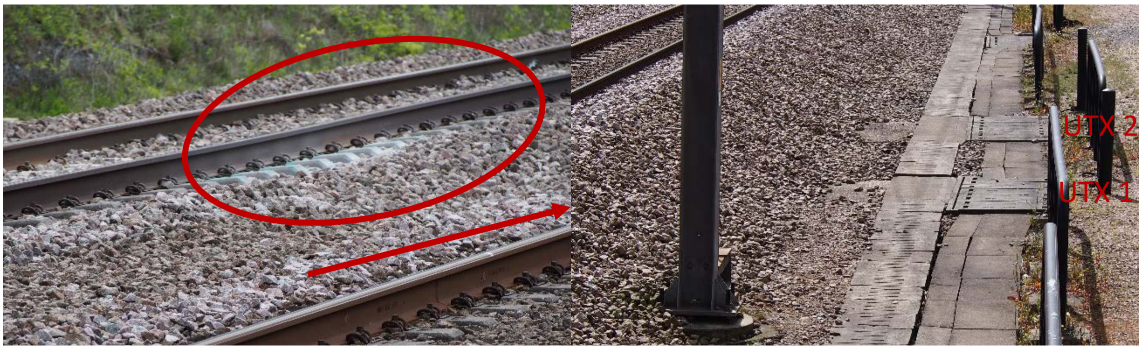


**Figure A3.1: (a) View of visible defect, (b) View of UTX inspection chambers adjacent to the line.**

*Remediation*

Manual hand packing was used to remediate the defect at this site on two occasions. The first attempt involved an over-lift wholly within the defect zone and repacking of the ballast beneath the sleepers to remove voids. This appeared to restore performance within the defect zone at least temporarily. However, the over-lift affected the geometry above the second UTX, resulting in increased dynamic deflections. This required a second intervention, in which repacking was extended beyond the visible extent of the fault to encompass six additional sleepers at either end. The second intervention was implemented three months after the original repair.

*Measurements*

Trackside measurements from MEMs accelerometers were recorded continuously over a period of five months, starting two weeks prior to the intervention (Figure A3.2).

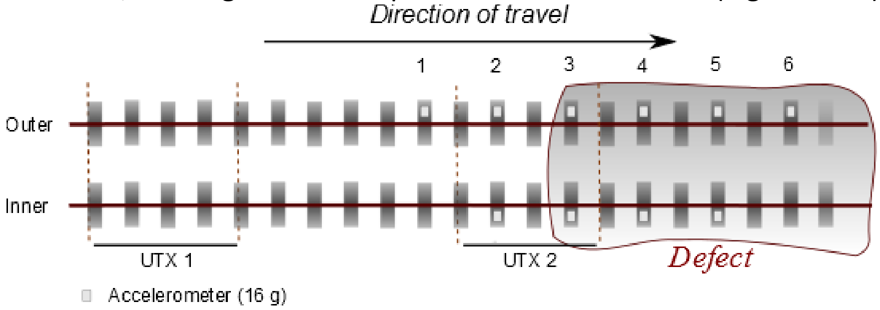


**Figure A3.2: Site layout showing the extent of the defect and transducer locations (grey boxes).**

*Results*

Figure A3.3(a) shows the recorded sleeper movements on key dates during passage of a Class 395 train. Before the first intervention, downward sleeper deflections in the defect zone (sleepers 3 to 6) were large (up to nearly 10 mm). Deflections at the outer rail were greater than at the inner rail. Deflections outside the defect zone but above UTX 2 were < 1 mm (sleepers 1 and 2). After the first intervention, deflections reduced within the defect zone to <2 mm. However, above UTX 2 the deflections at sleepers 1 and 2 increased to >2 mm. This was considered to have been a consequence of the over-lift within the defect zone. The effect of the second intervention, encompassing sleepers on either side of the original fault, is indicated by the final bar for each instrument location in Figure A3.3. The second intervention had the effect of reducing movements at sleepers 1 and 2 induced by the first intervention. Figure A3.3(b) shows the corresponding track moduli calculated directly for an axle load of 57.5 kN, a sleeper spacing of 0.6 m and a railpad stiffness of 100 MN/m. After the final intervention the support system moduli within the defect zone increased to more acceptable values in the range 15 to 30 MN/$m^2$.

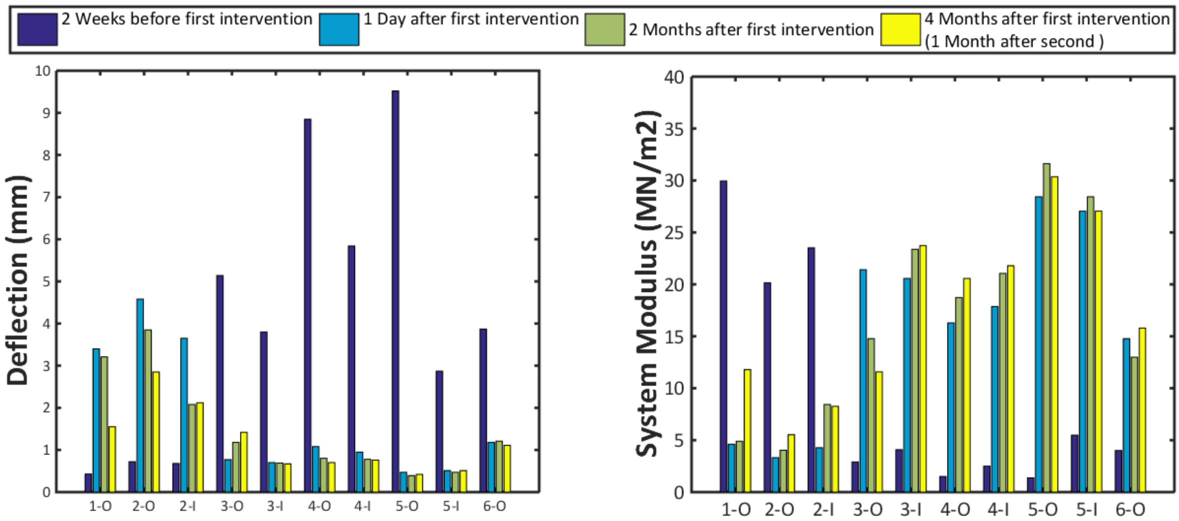


**Figure A3.3: Key data for class 395 train a) Average downward vertical sleeper deflections. (b) Support system modulus**

*Observations*

The data suggest that it may be more effective to repack over an extended region beyond the length of the visibly affected sleepers, to ensure continuity of support and geometry. However the underlying hard spots caused by the shallow UTXs remain a long term problem.

**A4. Bishops Stortford (locally too variable): Reduction of seasonal shrink/swell cycles through ballast drainage at the top of a clay embankment**

*Site overview*

The track at this site is on an ash and clay embankment adjacent to a road under-bridge. The site suffered from persistent cyclic top faults, with voiding below sleepers at the interface with the under-bridge (Figure A4.1). The embankment is partly retained at the bridge interface by wing-walls, to a depth of approximately 4 m below rail level. Track-bed investigation by means of cross-trenches showed a lens-shaped depression in the subgrade, filled as track ballast had been added during successive tamping operations. The track has no significant gradient and is straight. Service trains are electric multiple units travelling at up to 135 kph. Previous attempts at remediation by regular packing had not resolved the problem on a long-term basis.

*Remediation*

The remediation implemented was de-watering through a series of counterfort drains, 0.5 m wide and 0.2 m deep filled with granite chippings, installed to drain the ballast filled depression in the sub-base. The drains were dug, filled, and spoil removed using road/rail plant and works trains.

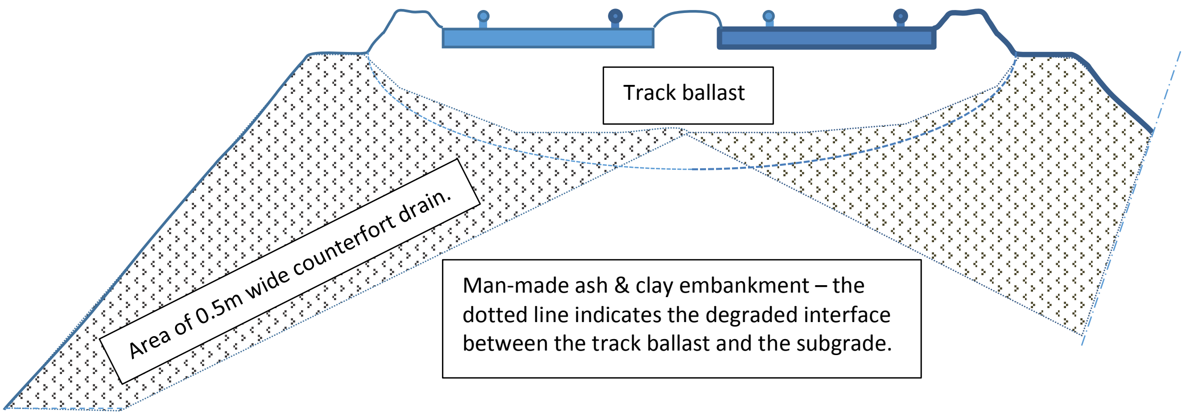


**Figure A4.1: Schematic cross-section of track with counterfort drains.**

*Measurements and Results*

Prior to the remediation, successive Track Recording Car traces had shown seasonal movement of the track as the clay swelled and shrank during wet and dry periods, resulting in a loss of cross level as the inner rails sank.

The remediation resulted in a stable track-bed, reducing both the shrink/swell movement of the clay and excessive movement under traffic.

*Observations*

Clay embankments often deform in this way and remediation of this sort can be very effective.

## A5. Grays and Purfleet (too soft): Subgrade stiffening using geoweb

*Site overview*

Grays Mainline and Purfleet Long Siding lie on the north side of the Thames Estuary, between the towns of Purfleet and Grays, approximately 400 m from the river. The track is just above high tide level.

On the mainline at Grays, the severity of the problem was such that line-side equipment and troughing were observed to move in response to trains passing and the site suffered from cyclic top faults. The underlying geology is soft-firm organic clay, to a depth of >4 m below rail level. There is no significant gradient and the track is virtually straight for approximately three miles. Service trains (125/day) are electric multiple units travelling at up to 90 kph, and heavy freight trains to and from Tilbury Docks and Ripple Lane Oil Terminal.

Ground conditions at Purfleet Long Siding, which serves the Foster Yeoman stone depot on the Thames, are similar. The siding is subjected to particularly heavy freight train loading at one end as stone trains enter the siding from the main line, travel part way along the siding and then reverse into the depot. The heavily loaded end of the siding suffered the greatest deterioration in track geometry (Figure A5.1).

Previous attempts to remediate the problems at both sites by conventional ballasted track renewal had not provided a long term solution.

*Remediation*

At Grays a new type of renewal was tried in 1984. This involved replacement of the track, with the ballast and sub-base dug out to a depth of 500 mm below sleeper base level and the placement of a honeycomb geosynthetic (Geoweb) layer 200 mm deep (Figure A5.2a) filled with stone chippings, underlain by an impermeable geocomposite (Filtram 1BZ). The same technique was adopted on a section of the most heavily loaded 80 m of track at Purfleet Long Siding (Figure A5.1) in 1998, with the remainder of the siding being reballasted (Figure A5.2b).

**Figure A5.1: (a) Subgrade surface prior to installation of geoweb at Purfleet – note shallow unstable formation of adjacent track (b) Very poor track quality at the start of Purfleet Long Siding close to the main line**

*Measurements*
Trackside measurements were taken prior to and after remediation of the Grays site in 1984, and track geometry measurements showed considerable improvement. At Purfleet the stiffness of the trackbed was measured using Falling Weight Deflectometer (FWD) tests carried out at 5 m spacing.

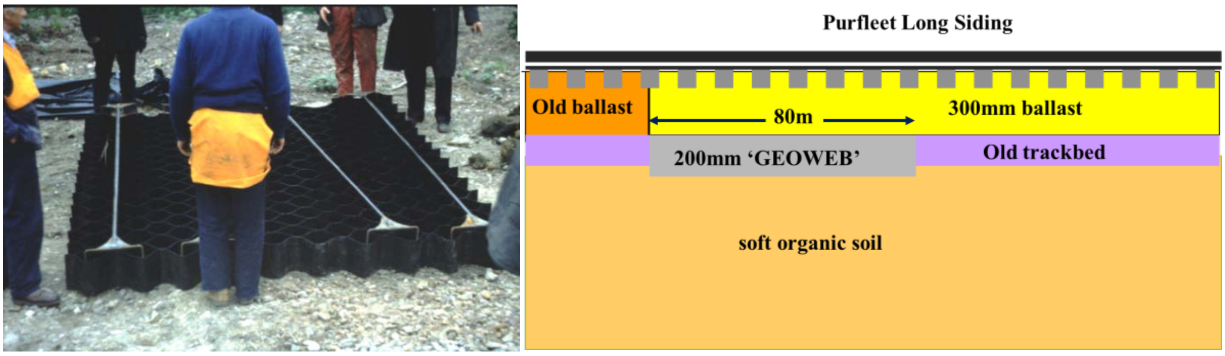


**Figure A5.2: (a) Preparation of Geoweb section prior to installation (b) Locations of geoweb and reballasting treatments**

*Results*
Figure A5.3 shows the FWD test data from Purfleet. Surface deflections measured on each sleeper and at distances relative to the FWD of 0.3 m and 1.0 m along the track provide an estimate of the deflection attributable to (1) all the material below the level of the sleeper (the ballast), (2) material below the bottom of the ballast bed (sub-ballast) and (3) the subgrade (see Figure B6.1).

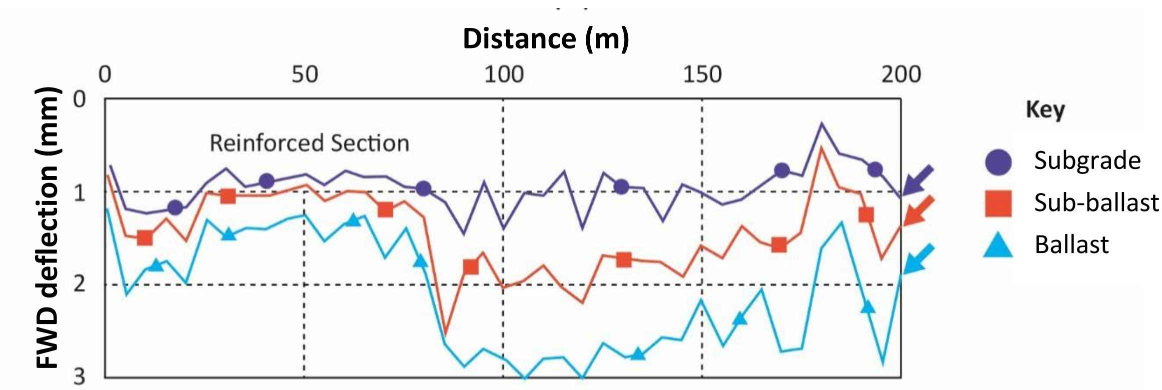


**Figure A5.3: Effect of installing a reinforced layer (geoweb) on FWD data (soft subgrade)**

*Observations*
The geoweb treatment appears to have reduced the variation of the subgrade deflection along the reinforced section, although the overall magnitude has remained similar. Deflections within the sub-ballast and ballast layers have been reduced considerably, suggesting that strain levels within the ballast are now lower making it better able to resist long term deformation.

## A6. Reading (locally too variable): Behaviour at a switch

*Site overview*

The study site (Figure A6.1) is a series of switches that had recently been renewed. At some of these renewed switches, fortnightly packing with Kango hammers was found to be necessary to maintain alignment. The problem was thought to have been due to the 20 mm difference in height between the standard sleepers and the hollow bearers used in the renewal.

**Figure A6.1: Renewal of switches, arrows point to the hollow bearers**

*Remediation*

At selected locations, products usually used as under ballast mats were placed as under sleeper pads (USPs) below the hollow bearers during renewal. The trial initially included five switches. One was used as a control and had no USP; two were fitted with Brand A USPs and two with Brand B USPs, the properties of which are shown in Table A6.1. Because the Brand A and Brand B products were manufactured for use as under ballast mats, they had a greater thickness and compliance than typical under sleeper pads. In each case where a USP was installed, it was glued to the underside of the hollow bearer. The USPs were intended to reduce or eliminate the gap and increase friction between the hollow steel bearer and the levelled ballast bed.

| Mat fitted | Nature of mat | $C_{stat}$ ($Nmm^{-3}$) | Thickness (mm) | Mass ($kgm^{-2}$) |
|---|---|---|---|---|
| None (control) | None | - | - | - |
| Brand A | Sub-ballast mat sylomer D1519 | 0.15 | 19 | 10.5 |
| Brand B | TED – 6 | 0.1 | 14 | 5.8 |

**Table A6.1: Ballast mats used in the trial**

*Measurements*

Digital image correlation (DIC) of video images was used to determine the vertical movements of the hollow bearers as well as one or two nearby sleepers (Figure A6.2). Site access restrictions meant that it was possible to monitor only two of the five sites within the trial; these were the control bearer (no USP) and one of those fitted with a Brand A USP.

*Results*
Trackside measurements of dynamic track movements were made using high speed filiming with DIC. At each location, two freight train passages were recorded and subsequently analysed (two Class 66 at the “no USP” site and two Class 70 at the Brand A site, with locomotive axle loads of 21.6 t and 21.5 t respectively). The results are shown in Figure A6.3.

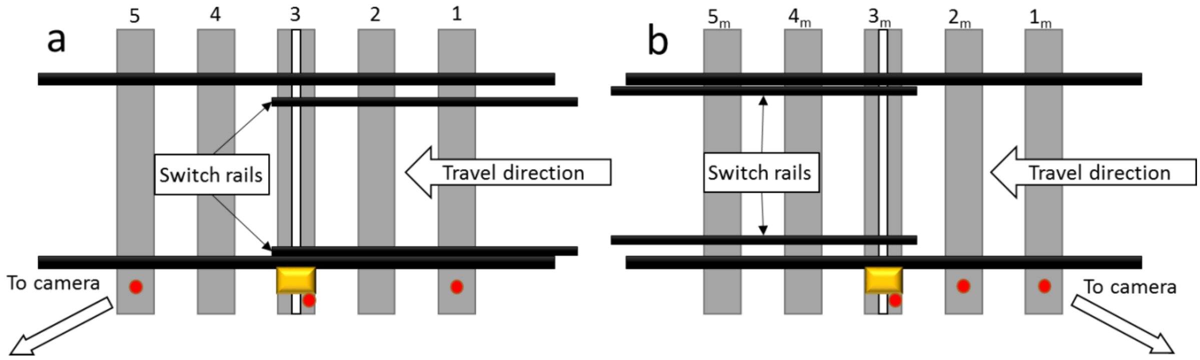


**Figure A6.2: Track diagrams with sleeper reference numbers (a) no mat (b) Brand A mat. Circles mark measurement locations.**

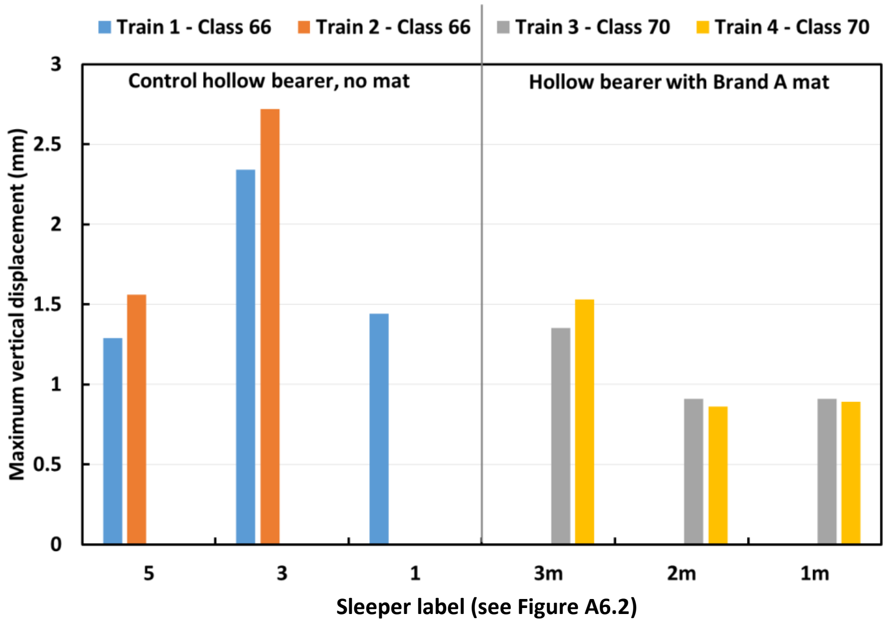


**Figure A6.3: Key data: average downward vertical sleeper deflections**

*Observations*
The hollow bearer fitted with a ballast mat suffered lower deflections, both on the bearer itself and at the adjacent sleepers, and smaller differential deflections than the unpadded bearer. It is not currently known how the ballast mat USPs will perform in the longer term.

*Further information*
Watson, G. V. R., Le Pen, L. & Powrie, W. (2015). *Reading West ballast mat monitoring*. Report by the University of Southampton to Network Rail, 10 September 2015.

## A7. Fishbourne (locally too variable): Behaviour at a level crossing

*Site overview*

The study site (Figure A7.1) is the approach to a level crossing on track that was due for a final maintenance tamp before a planned renewal. The site is underlain by clay, silt and sand. Maintenance at level crossings is problematic; level crossings are a fixed point in the profile of the track (because the level of the road cannot readily be changed), hence the track cannot be lifted or re-canted. The track at this site was known to perform badly and it was suspected that voids were present below some sleepers close to the crossing.

**Figure A7.1: (a) Level crossing looking in the direction of the train movement on the near track (b) From opposite side of crossing**

*Remediation*

The performance of the approach to the level crossing was assessed before and after a maintenance tamp that took place on 4 June 2013.

*Measurements*

Geophones and Digital Image Correlation (DIC) of high speed video images were used to determine track movements during train passage along a run of seven alternate sleepers on the approach to the crossing, as shown in Figure A7.2. A brick culvert carrying a stream passes below the track on the side of the level crossing and represents a potential hard spot. A plastic cable conduit is also present (Figure A7.1(a)) between sleepers 2 and 3.

*Results*

Trackside measurements of dynamic sleeper displacements were made using geophones in October 2012, and using geophones and high speed filming with DIC in July 2013 and March 2014. On each visit, data for a number of Class 313 and 377 trains in either a 3 car or a 4 car configuration were captured. Both classes of train showed similar behaviour. Characteristic deflection data for a Class 377 travelling at approximately 110 kph are shown in Figure A7.3, together with the implied support system modulus based on an estimated static wheel load of 5.425 tonnes, a sleeper spacing of 0.65 m, CEN 56 E rail properties and a railpad stiffness of 60 MN/m. During the first visit, sleepers 2 and 3 moved so much that the geophones used to measure their movements went offscale; consequently the exact magnitude of these movements is not known (although it is likely to have been greater than 6 mm). During the later two visits, high speed filiming was used to capture these larger movements.

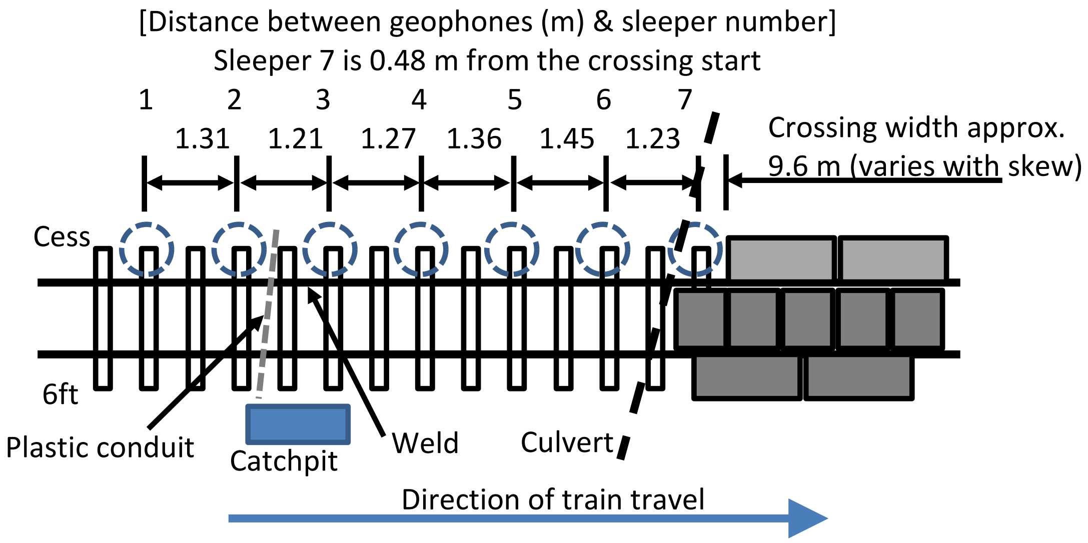


**Figure A7.2: Schematic plan of the area of interest, dashed circles show measurement locations**

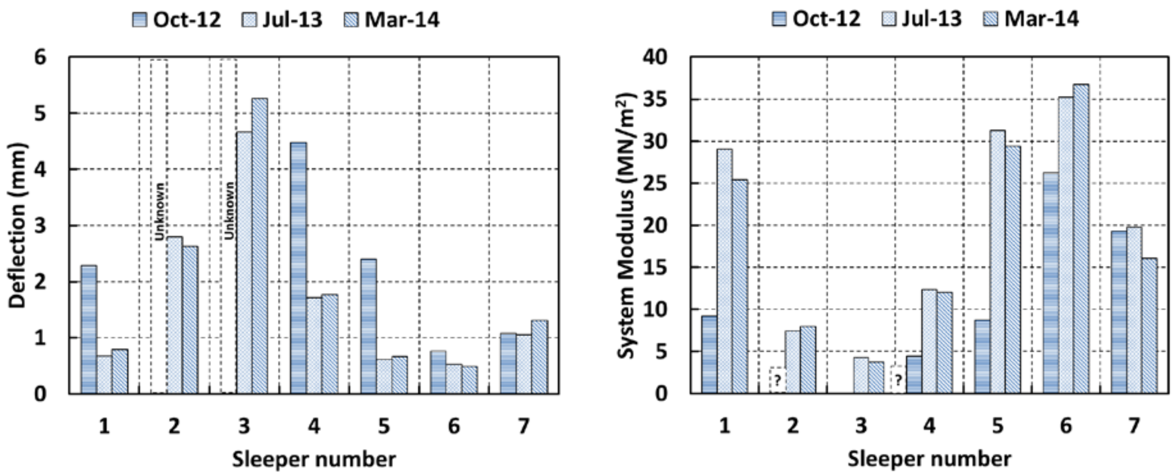


**Figure A7.3: Class 377 train travelling at 65 mph, (a) Sleeper deflections (b) Support system modulus**

*Observations*

Prior to tamping sleepers 1 to 5 showed large movements (>2 mm), indicative of voids beneath them. Sleepers 6 and 7 moved less, perhaps as a consequence of their proximity to the crossing and the possible hardspot provided by the culvert. Tamping reduced the movement at most locations, although it did not fully remove the voiding beneath sleepers 2 and 3. The voided or poorly supported sleepers remained, even after tamping. This may have been a result of the difficulty of tamping right up to and over the crossing, and of localised features such as the culvert, the catchpit, the weld and the plastic conduit at this particular site.

*Further information*

Le Pen, L., Watson, G. V. R., Powrie, W., Yeo, G., Weston, P. & Roberts, C. (2014). The behaviour of railway level crossings: insights through field monitoring. *Transportation Geotechnics*, 1, 201-213.

**A8. Marlborough (too soft): Micropiling for embankment core stabilisation**

*Site overview*

The down main line at this site is located on an embankment 3 to 6 metres high with a slope angle of approximately 40°. The track falls towards the centre of the site at gradients of 1:115 and 1:415 in the down and up directions respectively. The track comprises BS113A rail on F27 sleepers installed in 1972. The ballast is assumed to be of a similar age to the track components and appeared to be in a serviceable condition. Problems of persistent poor vertical / horizontal track alignment and loss of cross level were attributed to the embankment being unable to support the track adequately at full line speed. A speed restriction of 45 kph was imposed and either weekly manual lifting and packing or periodic stoneblowing carried out to maintain an operational railway at this reduced speed. The rate of track quality deterioration was very high, with the vertical standard deviation (worst top) over a 35 m wavelength increasing by up to 2.5mm/year. There were also signs of seasonal variation due to embankment shrinkage and swelling, with the track quality deteriorating more rapidly in dry seasons than in wet (Figure A8.1).

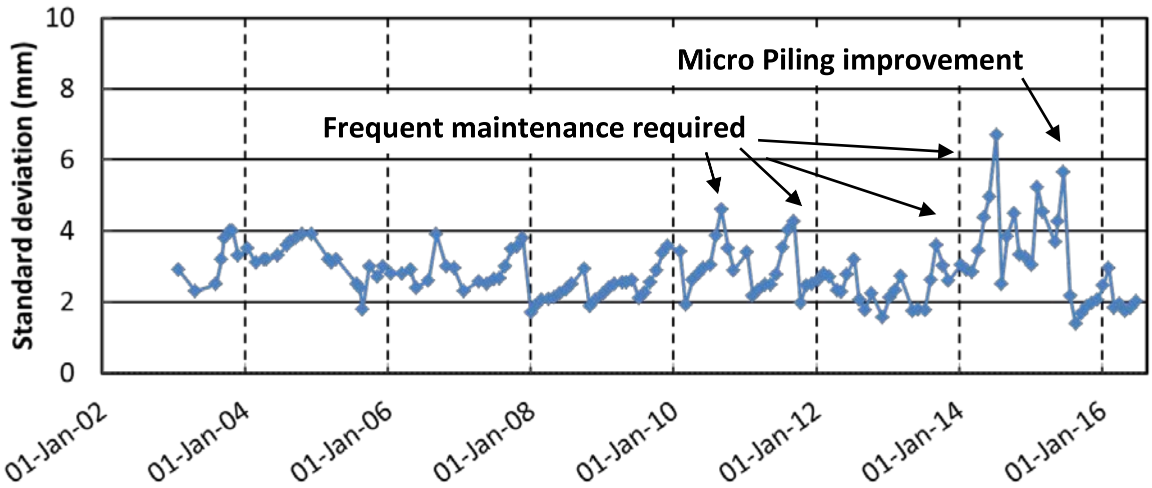


**Figure A8.1: Vertical standard deviation over a 35 m length vs time, at Marlborough**

*Remediation*

Micropiles were installed with the existing track and trackbed remaining in place, to reinforce the upper layers of the embankment and reduce stresses within the embankment core by transferring the load to more competent deeper soils. In total 164 micropiles (Figure A8.2) were installed over a length of approximately 80 sleeper bays. In most sleeper bays two micropiles were installed, outside each of the rails. However, in the central worst performing seven sleeper bays, four micropiles were installed per bay, two on each side of both rails. On completion of the piling, tamping was undertaken to restore the track to design level.

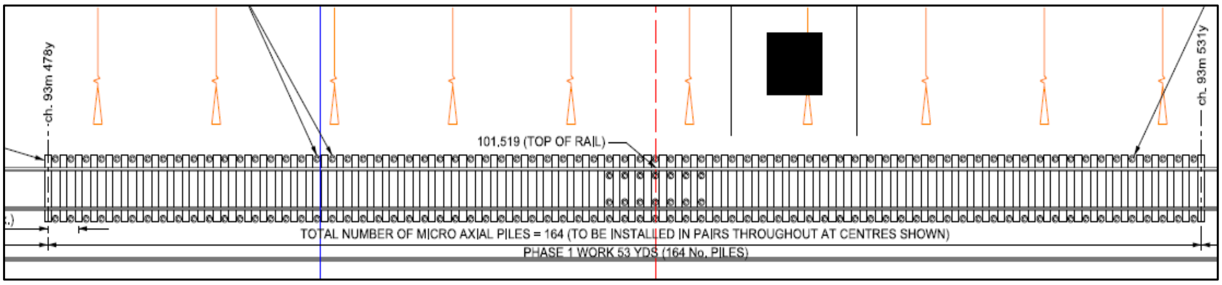


**Figure A8.2: Piling layout showing piles installed at sleeper ends**

*Measurements*

25 cross-level sensors were installed on alternate sleepers to monitor the inclination of the track (cant) over time along the section of track remediated.

*Results*

Figure A8.3 shows average daily cant measurements from sensors placed on the two worst performing sleepers before the installation of the micro-piles (March 2015). Prior to remediation the track was losing cant on a daily basis, in some locations at a rate of 10 mm per month. Major changes in the cant (Figure A8.3) indicate when corrective maintenance took place. Following the micropile remediation cant measurements showed little or no further changes. The worst top standard deviation measure of track geometry quality also shows a marked improvement, as indicated in Figure A8.1.

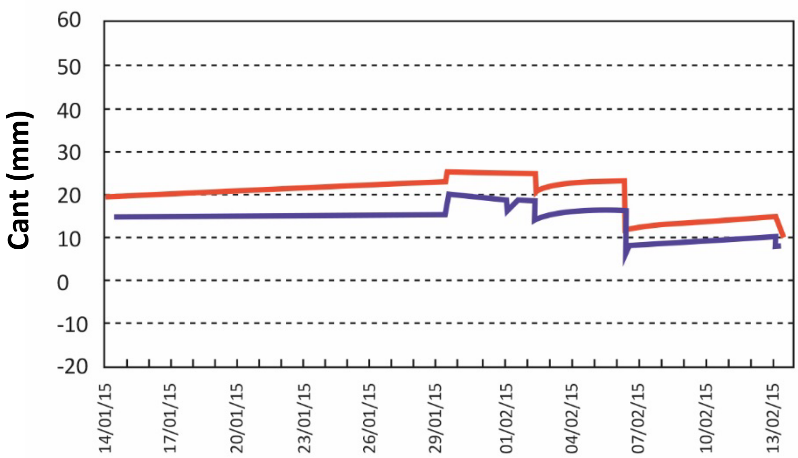


**Figure A8.3: Cant measurements before micropiling**

*Observations*

The cant measurements showed that micropiling stabilised the site from a track settlement point of view. The technique offers cost savings in the order of 50% over some other methods, with significantly less disruption to the operational railway: no track removal or signal disconnection was required during the works. Long-term monitoring is ongoing.

## A9. Woodacre (too soft): Micropiling

*Site overview*

The down main line at Woodacre on the West Coast Main Line (WCML) was experiencing poor track quality and high rates of deterioration, resulting in the imposition of a temporary speed restriction (TSR) of 127 kph compared with the normal line speed of 190 kph. Site investigation using Automatic Ballast Sampling (ABS), trial pits and Ground Probing Radar (GPR) found no indication of formation problems. However, a supplementary site investigation using dynamic probing and deep window sampling indicated a 2 m layer of soft peat under the track bed (Figure A9.1). Dynamic track deflections (up to 4.2 mm) and rates of track top geometry deterioration at the site were high.

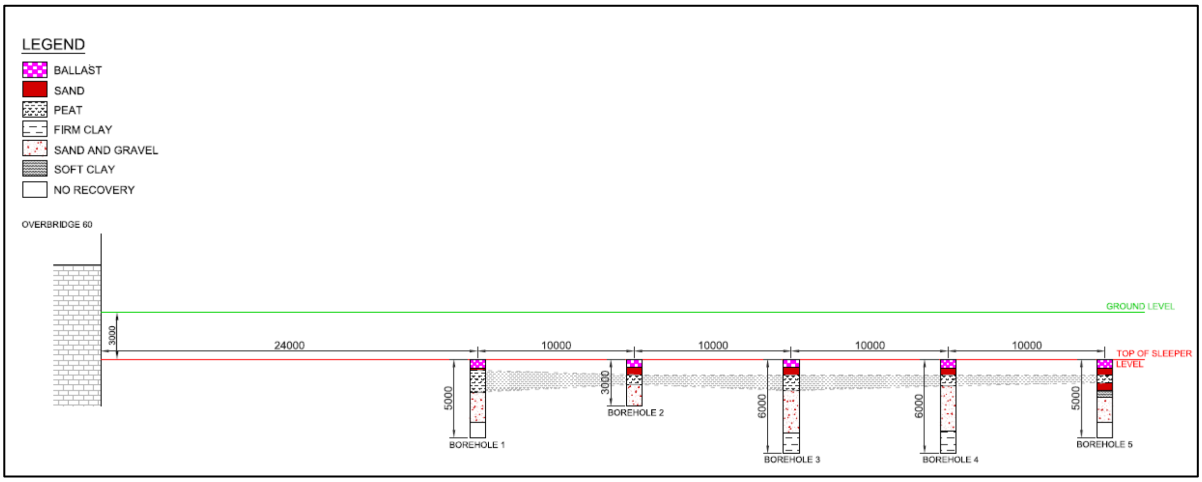


**Figure A9.1: Dynamic probing & deep sampling at Woodacre (the shaded horizon indicates peat)**

*Remediation*

Micropiles were installed between alternate sleepers at the end of 2013 (Figure A9.2), to transfer the vertical stresses through the peat to a deeper, more competent layer of sand and gravel and reduce track deflections. The existing track and trackbed remained in place during the works, and the full line speed limit was restored on completion.

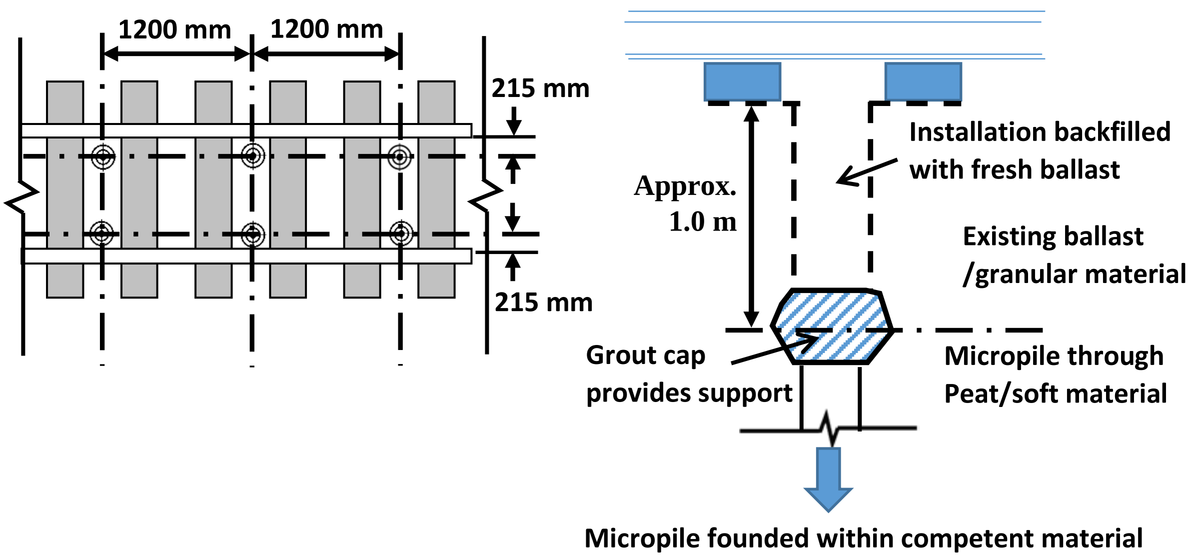


**Figure A9.2: Remediation design (not to scale); (a) plan; (b) illustrative elevation of individual pile**

*Measurements and results*

Figure A9.3 shows the time history of track quality (35 m worst top) over the past 10 years. Installation of the micropiles at the end of 2013 appears to have reduced the rate of track top deterioration from ~ 2 mm/year to ~0.5 mm/year (a four-fold reduction), providing a maintainable railway at the design line speed.

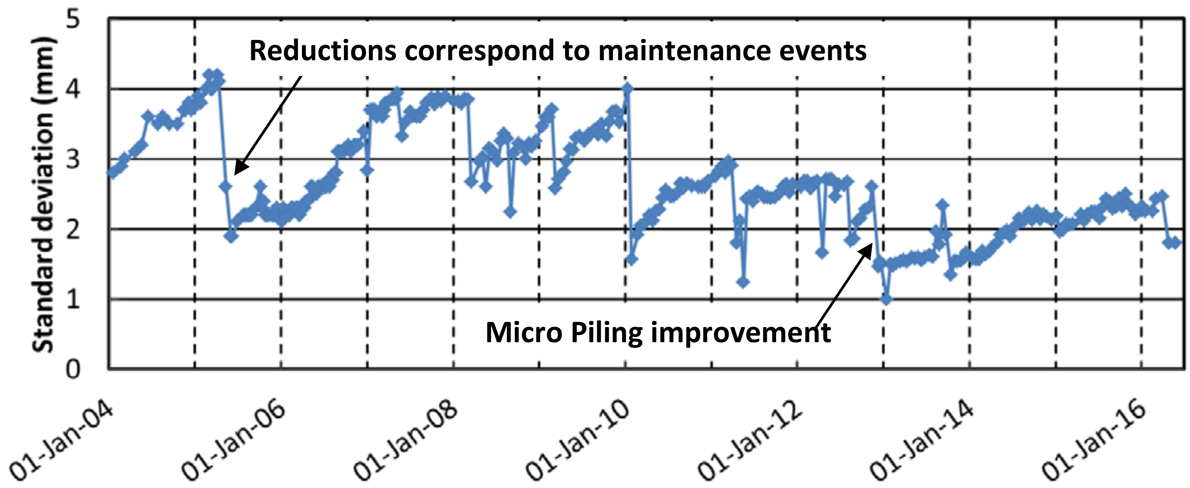


**Figure A9.3: Vertical (worst top) standard deviation over a 35 m wavelength for a 220 yard length of track including the study site**

*Observations*

The technique is minimally disruptive, and has so far been successful at this site. Further improvement could probably be achieved by installing micro-piles in every sleeper bay.

## A10. Öbisfelde DB/TTCi (too hard): Use of elastomer layers to soften trackbed on stiff formation

*Site overview*

It is generally accepted that stiff track is undesirable because it is associated with high dynamic loads from wheel or rail irregularities and excessive vibration effects. Two sites are described here: (1) Öbisfelde, a section of new high speed line in Germany with a very stiff trackbed on new earthworks built to a modern specification; and (2) Pueblo, a concrete bridge deck on the high tonnage loop of the US test track at the Transportation Technology Centre in Colorado. The bridge is located on a section of low embankment, and prior to treatment had suffered from transition problems as well as accelerated deterioration of ballast due to dynamic loading.

*Remediation*

On a stiff formation, softer railpads might be used to provide additional resilience; however this reduces the lateral rail stiffness, which can cause other problems such as accelerated wheel and rail wear. USPs (Under Sleeper Pads) have therefore become more popular as a means of softening the track, as they do not affect lateral rail stiffness and stability. Modern earthworks are also generally relatively stiff, so in the late 1990s Deutsche Bahn trialled a short section of under sleeper pads near Öbisfelde for comparison with conventional sleepers.

At Pueblo the installation of Under Ballast Mats (UBMs) was trialled, to soften the ballast support (hence reducing dynamic loads) and also to minimise the difference in stiffness between the embankment and the bridge.

*Measurements*

The Falling Weight Deflectometer (FWD) was used at both sites on unclipped sleepers. Only a single test was undertaken at Pueblo.

*Results*

At Öbisfelde, tests were carried out on 42 consecutive monobloc sleepers, 21 without USPs and 21 with USPs. Peak displacements are given in Figure A10.1.

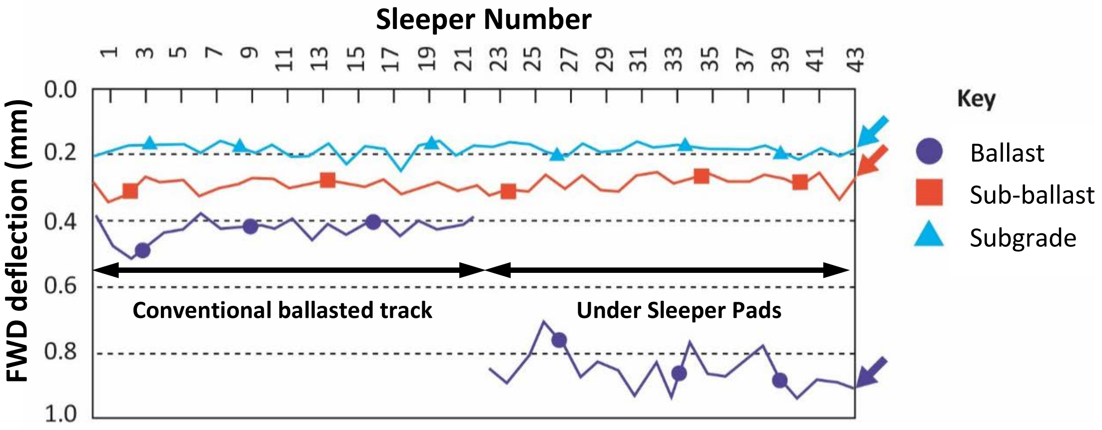


**Figure A10.1: Softening effect of under sleeper pads**

Figure A10.2 compares the deflection vs time traces of selected FWD tests at Öbisfelde for sleepers with and without USPs, and with the deflection vs time traces of tests carried out on a sleeper above the bridge deck at Pueblo where UBMs were present. The three deflection values at each sleeper indicate, in order of lowest to greatest deflection, the movements at the top of the subgrade, the top of the sub-ballast and the top of the ballast (see Figure B6.1b).

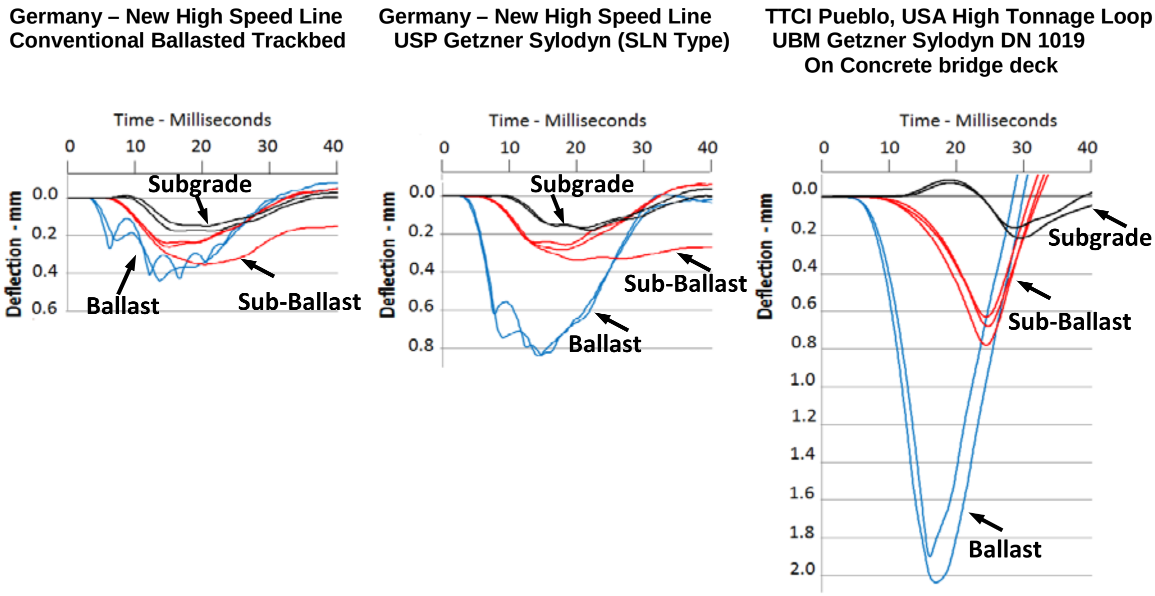


**Figure A10.2: Comparison of FWD data for sleepers with USPs and a UBM**

*Observations*

The USPs at Obisfelde increased the ballast (sleeper) deflections by a factor of 2, (equivalent to a reduction in the sleeper end stiffness from about 140 to 70 MN/m). This would give a reduction in dynamic load without increasing strains in the ballast and sub-ballast. Inspection of the ballast bottom deflection on the bridge deck at Pueblo indicates that ballast support conditions were broadly similar to those at Obisfelde. However, all other aspects of the FWD response at Pueblo are different. The ballast (sleeper) displacement was 2 mm, indicating low stiffness (~30 MN/m per sleeper end).

## APPENDIX B: INSTRUMENTATION/MEASUREMENT SYSTEMS THAT MAY BE USED TO QUANTIFY PARTICULAR DEFINITIONS OF TRACK STIFFNESS

Appendix B gives an overview of the instrumentation/measurement systems listed in Table 3 and indicates their level of intrusiveness on normal route operations. The descriptions provided are not exhaustive; further information can be obtained from the references listed in Table 3.

Appendix B contains the following sections:

**B1. Geophones**
**B2. Accelerometers**
**B3. Video camera / digital image correlation**
**B4. Laser systems**
**B5. Multi-depth deflectometers**
**B6. Falling Weight Deflectometer (FWD)**
**B7. Lightweight Drop Test (LWD)**

## B1. Geophones

A geophone (Figure B1.1) is essentially a mass on a spring. As the mass moves, a voltage related to its velocity is generated. By recording this voltage (typically at 500Hz), the data can be processed to obtain velocities, accelerations, deflections and frequency content. Geophones are generally suitable for train speeds greater than 45 kph.

**Figure B1.1: Geophones mounted on brackets glued to sleepers**

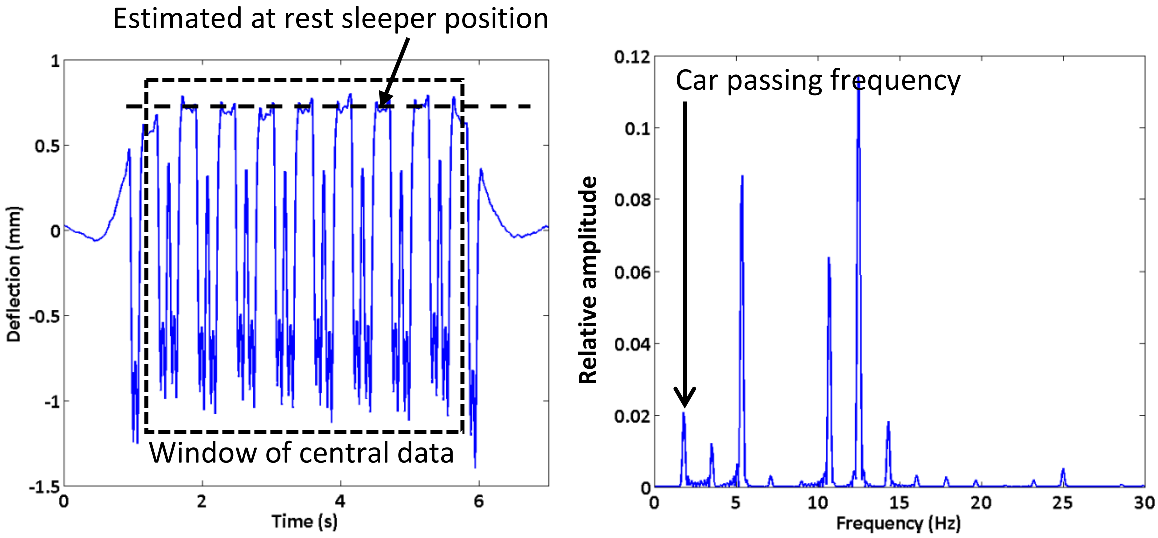


**Figure B1.2: Example geophone data: (a) track deflections (b) Frequency spectrum**

Figure B1.2 shows (a) the deflection against time and (b) the frequency spectrum for the passage of a 9 car Class 390 Pendolino. The cars, bogies and axles can clearly be identified in Figure B1.2(a). The first major spike in the frequency spectrum for well performing track is the car passing frequency; other major spikes occur at multiples of the car passing frequency. Figure B1.2(a) was obtained by integrating and filtering to remove low frequency data below the threshold of linearity for the geophone used (the natural frequency) and high frequency data not significant to the major displacements. The filtering means that the trace in Figure B1.2(a) has a zero mean, with the relative levels of the first and last axles most affected. The dashed lines in Figure B1.2(a) indicate the estimated at-rest sleeper position and the window of central data over which the relative levels are least affected by signal processing.

Geophone monitoring usually involves placement of some tens of sensors mounted on brackets glued to sleeper ends. The sensors are wired into a locally deployed data logger housed in protective enclosure. Deployment can usually be completed within one to two hours. Depending on the characteristics of a site, deployment may be during normal train operation (using lookouts and an appropriate warning system) or in full track possessions. Geophone systems can be triggered to record data automatically by approaching trains. Recorded data may then be downloaded manually or transmitted wirelessly for later processing.

**B2. Accelerometers**

An accelerometer is a motion sensor that produces a signal proportional to acceleration. The signal can be calibrated and processed to obtain acceleration, velocity and displacement vs time histories and associated frequency spectra. Two types of accelerometer are widely available: piezoelectric and micro electro mechanical systems (MEMS). MEMS accelerometers are significantly less expensive but are generally noisier and less precise. However, recent improvements in the fidelity of MEMS devices may be beginning to change this. Piezoelectric sensors have traditionally offered superior high frequency performance for vibration studies.

**Figure B2.1: MEMS accelerometer in a protective enclosure deployed with geophones and DIC targets**

Displacements can be obtained from accelerometer data by filtering then integrating twice. Filters are used to remove low frequency data that could cause low frequency drift and high frequency data less significant for track deflection. The increased processing requirements mean that displacements obtained from acceleration data are likely to be noisier than from velocity data. As with geophones, the necessary filtering means that the relative at rest position of the sleeper is lost but may be estimated as shown in Figure B2.2.

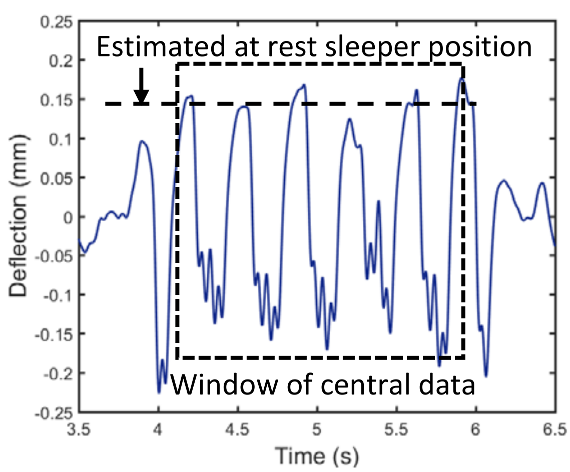


**Figure B2.2: Example displacement vs time history obtained using a low cost MEMS accelerometer**

Installation of an accelerometer track monitoring system has similar track access requirements to that for geophones.

**B3. Digital Image Correlation (DIC) of high speed video recordings**

The technique involves tracking the relative movement of corresponding patterns in a series of images. Filming is usually carried out using small targets as shown in Figure B3.1, although in some cases the texture of the rail may provide a pattern sufficient for recognition. Frame rates are appropriate to the train speed (usually at least 100 per second).

**Figure B3.1: (a) targets on sleepers and (b) camera set up**

The filming set up at any given location will depend on the local conditions. Camera movement, which would influence the results, should be minimised (although techniques are being developed to compensate for this).

Example data from DIC and comparable calculations using a beam on an elastic foundation analysis are shown in Figure B3.2 (a) and (b). The absolute position of the track and overall shape of the data from the DIC are in approximate agreement with the calculation. Small variations between theory and measurement are a result of non-homogeneous loading (dynamic effects) and varying support conditions on the real track, and the omission of the railpad movement from the measurements. There is also some extraneous noise in the DIC results which manifests itself as undulations of the trace after the train has passed (see Figure B3.2(b)). High speed filming and DIC are particularly effective when trains are moving too slowly for geophones and/or accelerometers, at sites where ground vibration is minimal and movements are relatively large.

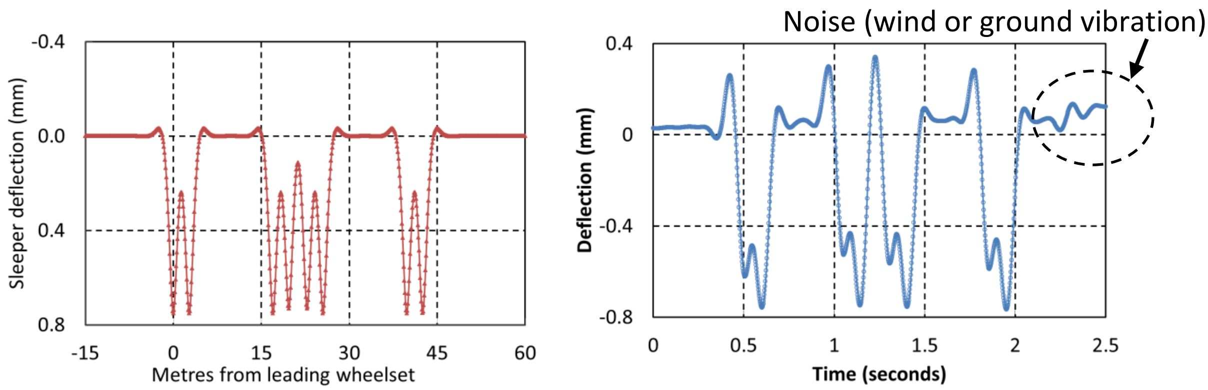


**Figure B3.2: (a) Calculated deflections for 2 cars of a Pendolino using a Beam on an Elastic Foundation approach (system modulus = 50MN/m$^2$) (b) Sleeper deflections measured using high speed filming and DIC for a 2 car Class 171.**

High speed filming can, in principle, be carried out remotely without the need for track access. However, the use of filming targets to aid digital image correlation requires track access for target placement on the sleeper ends and/or rail webs. Filming targets can be placed during normal train operation using lookouts and appropriate warning systems or during full possession. Filming is usually carried out with the camera operator present, but an automatic system could be devised.

**B4. Laser systems**

The advantages/disadvantages of laser based systems are generally similar to those of high speed filming with DIC. Both systems require line of sight and are susceptible to vibration at the point of filming or laser projection. However, the level of accuracy and number of measurement points possible can differ depending on the characteristics of the particular high speed filming or laser system used.

Laser based systems may use an infrared fan laser as the laser source, with Position Sensitive Devices (PSD), mounted along a run of sleeper ends or rails. The PSDs detect the position of the laser on their respective surfaces, hence the deflection of the object on which they are mounted.

Installation access requirements for PSDs are similar to those for geophones, accelerometers and DIC filming targets.

**B5. Multi-depth deflectometers**

A multi-depth deflectometer (MDD) typically consists of a number of metal rods installed into the ground, through a common hole. Each rod is housed within a protective sleeve, allowing the rod to move relative to the surrounding soil. At the level from which the relative movement is to be measured, the protective sleeve is discontinued and the rod is anchored into the ground, usually by grouting. The top end of each rod is then connected to a displacement measuring device (e.g. a Linear Variable Differential Transformer: LVDT), within a common top cap. By recording the electrical signals (e.g. the voltage of the LVDTs) and applying the appropriate calibration, the relative deflection between the anchor level and the top cap may be obtained for each rod. An alternative name for this type of instrument is a rod extensometer.

MDDs have been used in North America and South Africa to gain insights into trackbed and subgrade behaviour. When the top cap is connected to the sleeper base and one of the LVDTs is connected to a rod anchored at a sufficient depth to be isolated from the ground deformation, the system will indicate absolute sleeper deflections as trains pass. By anchoring rods at different depths, relative movements of different soil layers may be measured and their stiffness evaluated. Disadvantages of the system are the intrusive nature of the installation and its cost. There may also be some difficulty or uncertainty in ensuring that at least one of the rods is anchored at a depth below that affected by train-induced deflections.

Placement of a multi-depth deflectometer requires full track possession and the use of installation plant.

### B6. Falling Weight Deflectometer (FWD)

The Falling Weight Deflectometer (FWD) generates a pulse-load by dropping a weight onto a customised loading beam placed on an unclipped sleeper. Geophones are positioned as shown in Figure B6.1(a), and their outputs integrated to give displacements. Peak loads and displacements are measured and recorded automatically. Typically 30 locations can be tested in an hour. The spacing of FWD tests depends on the length of the site and the purpose of the testing. For a detailed assessment of dynamic sleeper support stiffness a maximum test spacing of 10 m is normally used. For assessing the impact of localised variation in track support conditions, such as at a bridge transition, testing at every sleeper is recommended.

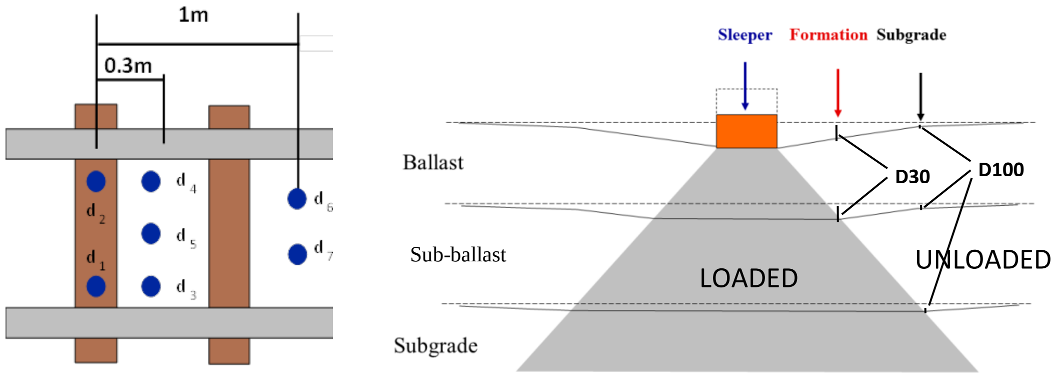


**Figure B6.1: (a) Geophone layout; (b) Interpretation of FWD test – indicative deflections**

The FWD gives a direct indication of the trackbed spring stiffness, $S_{trackbed}$ in Table 1. Ballasted track consists essentially of unbound material, and the load distribution through the upper parts of the trackbed is generally consistent from site to site. Surface deflections outside the loading "cone" at a given depth are therefore considered to be indicative of deflections of the lower layers, as indicated in Figure B6.1.

In addition to peak deflections it is possible to calculate the surface wave velocity by measuring the time taken for the wave generated by the FWD to travel a known distance (typically 2 m). This can be useful in the assessment of the potential for critical velocity effects.

FWD measurements require sleepers to be unclipped and must therefore be carried out during a full track possession.

**B7. Lightweight Drop Test (LWD)**

The Lightweight Drop Test (LWD) is similar in principle to the FWD, but is carried out using a portable dynamic testing tool (light weight hammer device). It can be used to estimate the equivalent spring stiffness of a subgrade before construction, or a placed layer immediately following construction, for site investigation and quality control purposes.

**Figure B7.1: Lightweight Drop Test (LWD) equipment**

Depending on the site and application, LWD tests may be carried out during normal train operations or with full track possessions.

**APPENDIX C: IDENTIFICATION OF STIFFNESS PROBLEMS**

It can be difficult to diagnose stiffness problems as the underlying cause of poor track geometry, except where the likelihood is obvious, e.g. at discontinuities such as underbridges or where the underlying ground is clearly very soft.

Appendix C presents a flowchart in the form of a series of questions as an aid to the identification of stiffness related track problems.

Where poor stiffness characteristics are thought to be a significant underlying cause of poor track geometry quality and where repeated conventional remediation works have been unsatisfactory, it would be appropriate to seek the advice of an experienced trackbed specialist. Where the problem merits it, and / or where major track works are planned anyway (e.g. as part of a renewal), the trackbed specialist would typically carry out a desk study drawing on a range of generally available information and then go on to specify a track bed investigation utilising standard geotechnical investigation procedures to confirm the underlying causes of any poor performance.

Based on the results of the desk study and the geotechnical investigation an appropriate remediation may be developed to address explicitly the underlying cause of any track stiffness related problems. Some of the remediation solutions available for track stiffness problems are shown in Table 6 of the main report and in the case studies shown in Appendix A.

Desk study and geotechnical investigation methods are not discussed in this document, although they may be topics for future Cross Industry Track Stiffness Working Group guidance.

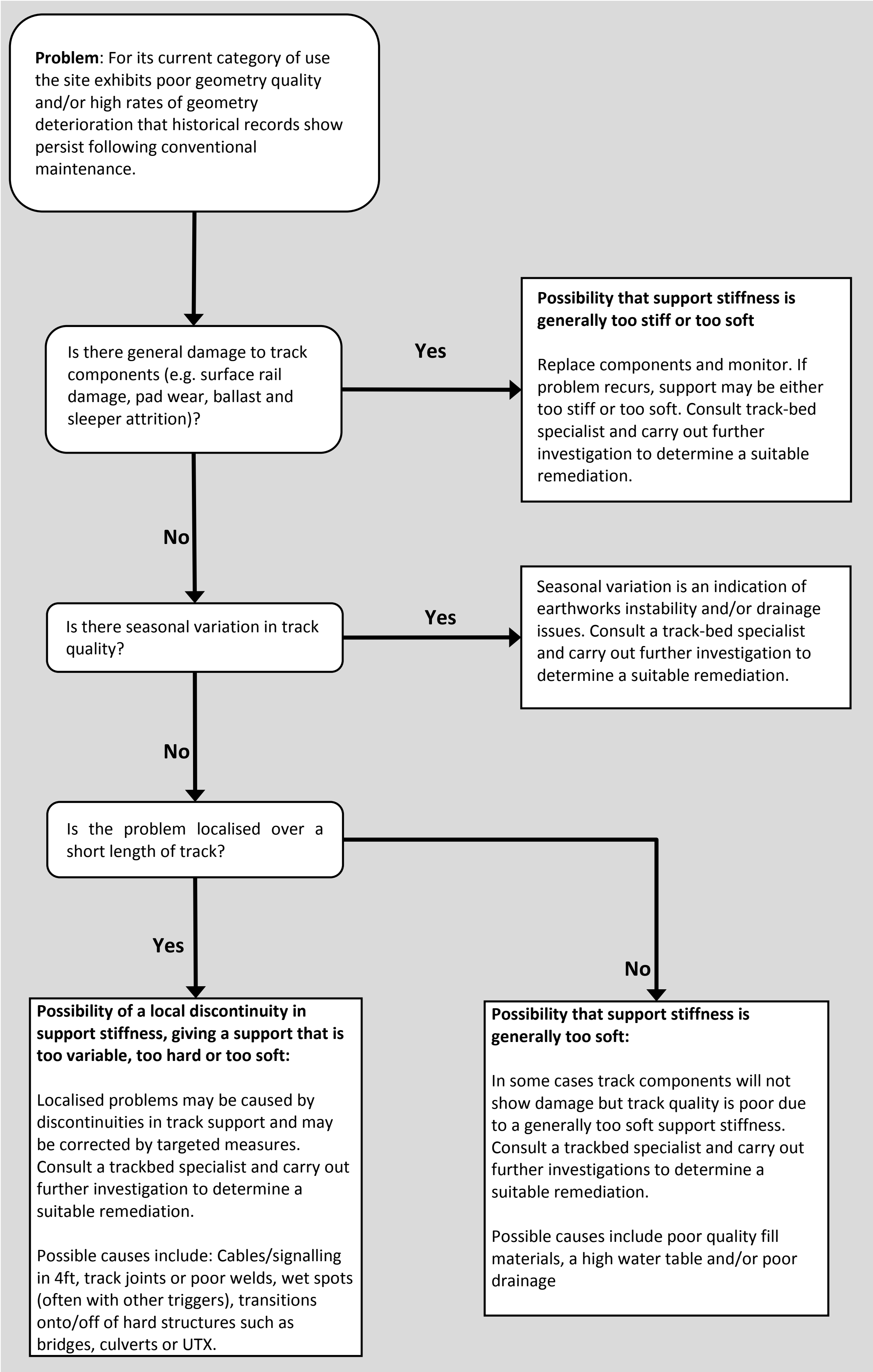


**Figure C1: Track stiffness diagnosis tool**

## APPENDIX D: GLOSSARY OF TERMS AND ABBREVIATIONS

| Acronym/symbol | Typical units | Description |
|---|---|---|
| BOEF | - | **B**eam **O**n an **E**lastic **F**oundation |
| FWD | - | **F**alling **W**eight **D**eflectometer |
| LWD | - | **L**ight **W**eight **D**eflectometer |
| LVDT | - | **L**inear **V**ariable **D**ifferential **T**ransformer |
| USP | - | **U**nder **S**leeper **P**ad |
| UBM | - | **U**nder **B**allast **M**at |
| UTX | - | **U**nder **T**rack **X**ing (crossing) |
| $d_n$ | m | Distance from a reference point e.g. Axle spacing relative to axle 1 on first car |
| $d_c$ | m | Car length |
| $E$ | $MN/m^2$ | Young's modulus |
| $EI$ | $MNm^2$ | Bending stiffness of each rail |
| $F$ | MN | Force (wheel load) |
| $I$ | $m^4$ | Second moment of area |
| $k$ | $MN/m^2$ | Modulus: the load per unit length needed to cause a unit deflection |
| $k_{railpad}$ | $MN/m^2$ | Modulus: The load per unit length (adjusted for sleeper spacing) causing a unit displacement of the railpad |
| $k_{system}$ | $MN/m^2$ | Modulus: The load per unit length causing a unit displacement at the rail |
| $k_{trackbed}$ | $MN/m^2$ | Modulus: The load per unit length causing a unit displacement of the sleeper |
| $L$ | m | Characteristic length (Arises from the BOEF analysis) |
| $n$ | | Number of loading events (axles) |
| $s$ | m | Horizontal spacing between centres of adjacent sleepers |
| $S$ | MN/m | Spring stiffness: the load needed to cause a unit deflection |
| $S_{composite}$ | MN/m | Spring stiffness: The point load required to produce a unit deflection of the rail at the location where the load is applied |
| $S_{railpad}$ | MN/m | Spring stiffness: the load per unit displacement of the railpad in isolation |
| $t$ | seconds | Time |
| $v$ | m/s | Train speed |
| $w$ | m | Vertical deflection |
| $x$ | m | distance |

**Table D1: Glossary of Terms and Abbreviations**