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Railway track substructure: Recent research and future directions

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ABSTRACT

Rail has a vital role to play in the decarbonisation of transportation, both now and in the future. It has the potential to mitigate, but also suffers from the effects of climate change, with the ageing infrastructure on historic networks increasingly vulnerable to more frequent extremes of temperature, rainfall and storminess. If rail is to remain affordable, it is essential that we target resources where they are most needed, avoiding unnecessary maintenance and over-engineered reconstruction or renewal. This requires an improved understanding of the engineering behaviour of railway infrastructure, especially the track and its supporting substructures, so that we have the confidence to adopt new approaches to construction, maintenance, refurbishment and repair. This paper is a written version of the 3rd International Society for Soil Mechanics and Geotechnical Engineering Ralph Roscoe Proctor Lecture, delivered at the 4th International Conference on Transportation Geotechnics on 25 May 2021. It summarises some of the research carried out at the University of Southampton (UK) over the past decade or so to develop a better understanding of the loading applied to, and the response of, the railway track and substructures; and more cost- and carbon-effective ways of preventing deterioration or restoring performance. The importance of trackbed or rail support system stiffness, track vertical alignment and train speed are demonstrated and discussed, and practical ways of reducing the ongoing settlement of railway track are presented. It is argued that, to facilitate future development and improvement, more rigorous but practically usable models for the evolution of track settlement are needed, validated with reference to high quality field data; together with a better understanding of material and system damping.

Introduction: role and challenge for rail in a net zero transport world

Climate change is already happening, with parts of the world facing existential environmental threats associated with increasing extremes and intensity of heat, rainfall, wind, storminess and sea level rise. It is therefore imperative that society both improves its resilience to the impacts of climate change through adaptation, and rapidly reduces greenhouse gas emissions (principally methane and carbon dioxide, CO_2) to lessen (mitigate) the magnitude of the change in climate we face. Decarbonising transport, which is currently responsible for about $^1/_3$ of global CO_2 emissions, is essential to the latter.

Railways have a key role in the decarbonisation of transport. They are easily electrified, giving zero CO_2 emissions at the point of use and, if the electricity is renewably sourced, zero emissions in any terms. While this could potentially also be achieved by road vehicles through battery or hydrogen operation, or even as is being mooted fixed electrical supply infrastructure for freight (lorries or trucks), the low resistance to rolling of a steel wheel on a steel rail gives railways an inherent advantage. Indeed, this – manifest as the ability to haul trains of wagons and tonnes of payload using a single horse – was the reason railways first came into being. It is a law of physics that cannot be circumvented. Data from Pritchard [62] indicate that the average energy consumed by a UK 11-car Class 390 Pendolino train travelling at a speed of up to 200 km/hour is 25 Wh (Watt-hours) per seat km. This is about half that (48 Wh/seat km) of an average electric car travelling at a maximum speed of 112 km/h [22]. A fuller analysis is given by Pritchard et al. [63], from which it is clear that modal shift from road to rail must be a major part of any net zero transport strategy. Furthermore, the emissions of small particles potentially injurious to health associated with wear on tyres/wheels, roads/rails and brake pads and discs are far smaller for rail than for road vehicles.

The above data relate to operational energy use and potential greenhouse gas emissions or, to use the vernacular, "carbon". However, carbon is also spent or invested in building transport infrastructure and in maintaining it. The way an infrastructure has been built might also commit the expenditure of operational carbon for its entire lifetime. The carbon associated with the infrastructure was traditionally unaccounted for in engineering projects, but is becoming an increasingly important consideration. Calculating embodied, operational and committed carbon is in principle straightforward. Whole life financial cost modelling is well-established, and similar principles can be applied to carbon. However in reality, defining the system boundaries sufficiently rigorously and obtaining realistic quantitative data are challenging, and discrepancies in these are responsible for potentially widely differing outcomes in calculations of whole-life infrastructure and other carbon costs.

Traditionally, there would be expected to be a trade-off between the

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up-front (capital) financial or carbon cost of building an infrastructure asset and the ongoing (recurrent or operational) cost associated with maintaining it. Some would argue that the 19th century engineers built their earthworks with the aim of minimising the capital cost, passing a legacy of ongoing maintenance need to their present-day counterparts. Modern construction takes perhaps the opposite approach, with heavy initial engineering including high-quality embankment fill materials and relatively shallow slopes being deployed in an attempt to almost eliminate major maintenance over a 120-year design life. Given the urgency of avoiding CO_2 emissions today, this up-front expenditure of large amounts of carbon is questionable in sustainability terms; and quantifying its pay-back period is key. The challenges of carrying out this calculation realistically and reliably are illustrated in the assumptions that need to be made in the example given in Table 1.

In addition to the assumptions about construction, maintenance and use, the carbon payback time for the modern embankment depends very significantly on the mode from which it is assumed journeys are displaced and the renewable/non-renewable mix in the electricity used in each case. The 21st century embankment is based on an actual design, in which piles and a concrete raft were incorporated to reduce calculated long-term settlements to within allowable limits. In the event, sound engineering prevailed and the piles and raft were not installed. The calculated saving in CO_2(e) emissions is 6 tonnes per linear metre, or about 13.5 %. This reduces the payback period to 27 years for modal shift from battery electric road vehicles and 2 years for modal shift from air.

Maintenance of a traditionally-constructed embankment is not obviously a huge carbon burden, but an over-engineered refurbishment or new-build to avoid it is. However, maintenance does incur a financial cost, not just for the embankment but also for the track. Network Rail, the UK railway infrastructure owner and operator, currently (in 2022) spends ~£100 M each year on tamping to restore the line and level of its track, and ~£1Bn on track renewals, of a total annual operating budget of ~£10Bn. This Paper describes some recent research in railway trackbed engineering aimed at better understanding and reducing maintenance needs, with the overall objective of reducing the carbon footprint of the rail infrastructure.

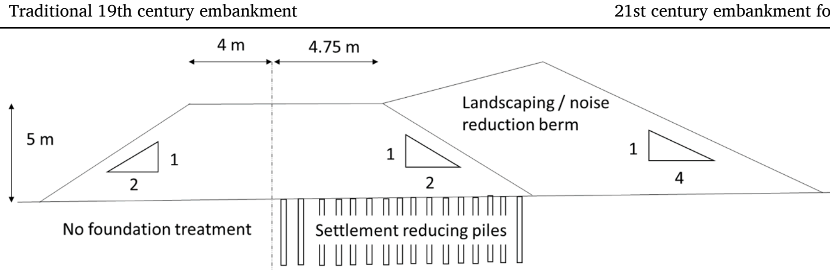
Key performance indicators for railway track: level and stiffness

Two key performance indicators for railway track are its vertical alignment or level, and the support stiffness offered by the trackbed. The horizontal alignment and the lateral restraint provided by the ballast are also important, but the discussion here will focus on the parameters in the vertical direction.

The quality of the vertical alignment is traditionally measured by the standard deviation (in metres) from level (after allowing for any intended gradient) over a 30 m or 60 m wavelength; on high-speed railways, wavelengths up to 150 m must also be considered (Liu et al. 2017) [38]. The track level must be as smooth as possible, with no or minimal bumps or dips. With trafficking, the track tends to settle. Settlements originate in the ballast owing to compaction, lateral spread and possibly grain breakage; and settlement or heave might be caused by seasonal cyclic changes in the moisture content of the underlying ground or an earthwork, owing to the effects of weather and vegetation. Uniform settlement of the track would not adversely affect the quality of the geometry as measured by the standard deviation. However, track settlement is almost never uniform, owing to minor differences in track, ballast and subgrade condition; the presence of essentially fixed points such as underbridges; and variations in other factors such as drainage and the degree or type of vegetation along the route.

Hence the vertical geometry tends to deteriorate and must from time to time be restored to an acceptable quality. This is almost always achieved by a process of tamping, in which the track is lifted by a machine which then drives vibrating tines into the ballast on either side of each sleeper. These tines compact the ballast horizontally along the direction of the track, raising the level of the ballast surface before the track is lowered back into contact. Tamping is a slightly hit-and-miss operation, because the amount by which the track will settle during the passage of subsequent trains as the ballast recompacts must be

Table 1
Estimates of the carbon pay-back period of typical 19th and 21st century railway embankments, assuming modal shift not new trip generation. Calculations by Tracey Najafpour Navaei using the RSSB Rail Carbon Tool [65]. [1] Construction and maintenance CO_2(e) emissions exclude transport of staff and plant to site, embodied carbon in plant and removal of waste; [2] Assumes current UK National grid mix of renewable/non-renewable sources of electricity; [3] Includes radiation forcing (RF).

	Traditional 19th century embankment	21st century embankment for a new high-speed line			
Schematic half-section					
Geometry	5 m high; 8 m crest width; 1:2 slopes; no foundation treatment	5 m high; 9.5 m crest width; 1:2 m slopes for the core; landscaping berms with 1:4 slopes; settlement reducing piles below core			
Summary description of construction	Fill as available from nearest cutting (minimal transport distance), placed and compacted in layers	High quality fill in core (treated or greater transport distance) placed and compacted in layers; landscaping berms on either side; concrete settlement-reducing piles below core			
CO_2 (equivalent) emissions: construction	0.86 tonnes per linear metre [1]	44.53 tonnes per linear metre [1]			
CO_2 (equivalent) emissions: major maintenance	2× major maintenance interventions @ 40 years and @ 80 years: 0.26 tonnes per linear metre [1]	Nil			
Assumed train service pattern	72×400 seat trains/day in each direction	144×600 seat trains/day in each direction			
Mode from which journeys are transferred	Petrol/diesel car	Car			Air [3]
		Petrol/ diesel	Plug-in hybrid	Battery electric	
Carbon payback period [2]	4 months	4.2 years	9.6 years	31.3 years	2.7 years

estimated and allowed for. Also, there is evidence that tamping damages the ballast grains by breakage [14,72,16], increasing the rate of ballast settlement and bringing forward the next required tamping intervention (Selig and Waters 1994). This happens at each successive tamp, so that after about 20 tamps the ballast is conventionally considered to be "life expired" and is replaced.

Ongoing monotonic settlement of a modern earthwork that has been properly placed and compacted in layers with good drainage, or of an historic earthwork that has already undergone consolidation, should not be an issue. However, seasonal shrink-swell cycles associated with different rates of net evapotranspiration in winter and summer are problematic on high-plasticity clay embankments without proactive vegetation management [69].

The rail support system stiffness gives an indication of the deflection of the rail under load. It is commonly defined as the load per sleeper end or per unit length along the track that causes a unit deflection, measured in MN/m or MN/m^2 respectively. It includes the contribution of every element in the load path from the rail into the earth – the rail pads, under-sleeper pads, ballast, subgrade and the underlying ground. Interaction between the track and the ground under the influence of a train represented by a moving series of loads is often analysed as a beam on an elastic foundation. This is a very useful conceptual model, although the limitations of the classical form in not representing dynamic, non-linear, non-uniform or inelastic response of the ground (including voiding below sleepers) must be borne in mind. The rails are represented as a beam and the support system, including the rail pads and any under-sleeper pads, by the elastic foundation.

A beam on an elastic foundation analysis quickly demonstrates that, with increasing rail support system stiffness k (MN/m^2).

- the maximum deflection below an axle or a bogie reduces.
- the deflection bowl associated with an axle or a bogie narrows.
- the recovery between axles becomes relatively more pronounced.
- the local rail stresses, indicated by the curvature of the rail deflection, increase (Fig. 1).

It is immediately apparent that the track support stiffness should be

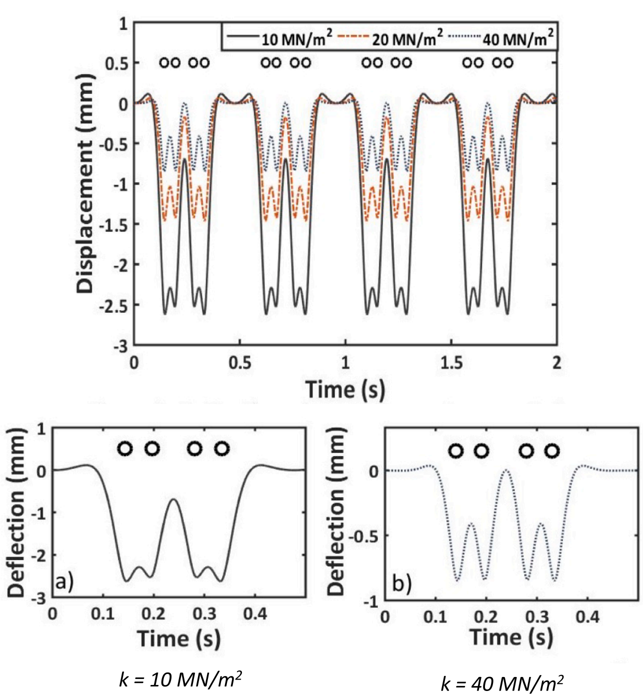


Fig. 1. Effect of rail support system stiffness on deflections during train passage (quasi-static beam on an elastic foundation analysis). Figure: Dr L M Le Pen.

neither too high (because it can lead to potentially damaging stresses within the rail and other track components), nor too low (because it can lead to excessive vertical movements), but somewhere in between. The former UK Railway Group Standard GC/RT 5014 for trackbed and track drainage [64] required a minimum dynamic sleeper support stiffness of 60 MN/m per sleeper end for new track, and 30 MN/m per sleeper end for trackbed renewals in areas with a history of poor track geometry, or where the vertical or horizontal alignment is to be changed significantly. For a sleeper spacing of 0.65 m and a typical railpad stiffness of 60 MN/m, these correspond to rail support system moduli of about 45 MN/m^2 and 30 MN/m^2 respectively. Field measurements on what is considered to be well-performing track at several sites reported by Powrie et al. [57] show that in practice, the acceptable range of rail support system stiffness is quite wide and in three of the four cases shown at or below the notional lower limit of 30 MN/m^2, although it must be reasonably uniform (Fig. 2).

Measurement of track level and support stiffness

There is a saying attributed to Lord Kelvin that "if you can't measure it, you can't improve it". Track geometry is routinely recorded by means of high-speed measurement trains such as UK Network Rail's New Measurement Train (200 km/h) and Japanese Railways' Dr Yellow (up to 270 km/h). These traverse the network frequently. While some railway administrations have developed algorithms to use the data to determine the track support system stiffness (for example, [6]), there is uncertainty as to what exactly is being measured in terms of loaded as opposed to unloaded geometry and it is difficult to match on-train with trackside measurements in terms of magnitude and exact location (for example, [35]). Measurement train data are invaluable as a diagnostic comparative tool, but difficult to interpret in absolute terms. For this reason, there is also a role for track-side measurements of track deflections as trains pass. This is easier said than done; direct measurement of displacement requires a reliable datum unaffected by train passage, while inferring displacement by measuring velocity or acceleration using geophones or accelerometers requires careful processing of the signal, which almost invariably introduces artifacts into the processed data.

Trackside methods of measuring track deflection – usually vertical, but the methods can also be used to assess horizontal (lateral or longitudinal) movements – are;

- multi-depth deflectometers (MDDs) to measure deflection directly (see, for example, [27,60,48]). These require a datum, usually at a depth sufficient for the displacement there to be negligible.
- video camera recording of a target or the texture on the rail, followed by digital image analysis to determine deflection directly [9,35]. This requires the camera to be placed at a location unaffected by ground vibration or turbulence from the passing train, or a second camera at the same location backsighting onto a remote fixed point [49].
- laser based systems to measure deflection directly [53,33]. These have many of the same issues as video recording.
- geophones to measure velocity [9,61,35]. The data are filtered and integrated once to obtain displacement, and the effect of transients at start and end of the trace needs to be eliminated.
- accelerometers to measure acceleration [34,43]. The data are filtered and integrated twice, and similar issues as with geophones apply.

More detailed descriptions, and a summary of the advantages and disadvantages of each approach, are given by Powrie et al. [57]. 15 years ago, direct video image capture was only feasible for relatively slow trains (up to 100 km/h), owing to the limited framerate of affordable video cameras [9]. This is not now an issue; reasonably inexpensive video cameras with frame-rates up to 500/s suitable for train speeds up to 400 km/h are now readily available. Finding a suitable camera

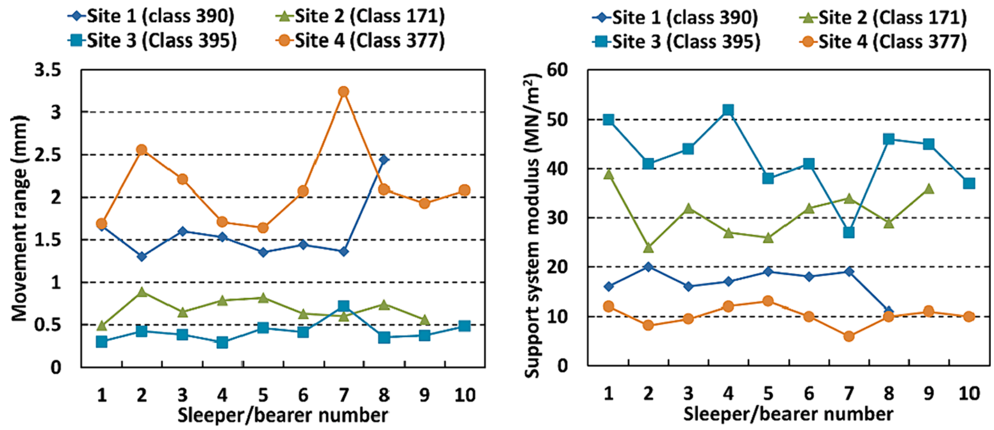


Fig. 2. (a) Measured deflections and (b) inferred rail support system stiffnesses on well-performing track (Powrie, [36].

location and ensuring uninterrupted lines of sight to the on-track targets are the factors most likely to militate against the approach. A further development in recent years has been the improvement in the quality of low-cost MEMS accelerometers, which a decade ago would only have been suitable for high-speed trains (>200 km/h) owing to the high noise to signal ratio at lower speeds. This was a limitation even with high quality accelerometers; some affordable accelerometers are now able to give reliable and representative data at train speeds of 10 km/h and above [43]. Low frequency (in 2007 3 Hz, but now through improved electronics 0.3 Hz) geophones have been and remain suitable for recording vibrations from trains at speeds more than about 20 km/h.

Processing velocity or acceleration data to assess deflections leads to an apparent ramping up of the displacement at the start of the trace and a corresponding ramping down at the end, as indicated in the processed measured data shown in Fig. 3a. The representative displacement during the passage of this train is given by the peak-to-trough amplitude over the "stationary region" indicated. It can be extracted manually from each trace, but when there are many traces to analyse this becomes tedious and time-consuming. Milne et al. [45] developed a method of processing the data automatically, based on the true zero being the displacement with a 70 % probability of being exceeded (Fig. 3c). The benefit of this approach is that the typical track displacement during a particular train passage can be rapidly extracted and used to assess the performance of the track, and whether it is deteriorating or remaining stable over time. The method was used in processing much of the data reported later in this paper.

A second major development in the use of trackside measurements of track deflection during train passage has been the use of analysis in the frequency domain (Fig. 4) to determine the track support system stiffness without needing to know the train weight or axle load [36]. This is useful because unless there is a nearby weighing-in-motion (WIM) device, it is difficult to know the train or axle load to within 20 % or so. The method works because, as can be shown by a beam on an elastic foundation analysis, the ratios of harmonic peaks in the frequency domain are independent of the applied load. For a given train type, a relationship between the track support system modulus and the ratio of the amplitudes of distinctive harmonic peaks (in this case, the 7th and the 3rd peaks) in the velocity (acceleration or displacement) spectrum may be calculated (Fig. 5).

Effects of track level and support stiffness

We will now consider the effects and relative influence of track level and support stiffness on train passage, through

- deflection and stiffness measurements based on MEMS accelerometer data interpreted using the methods summarised above.
- precise surveying to determine absolute track levels, and
- vehicle-track interaction modelling.

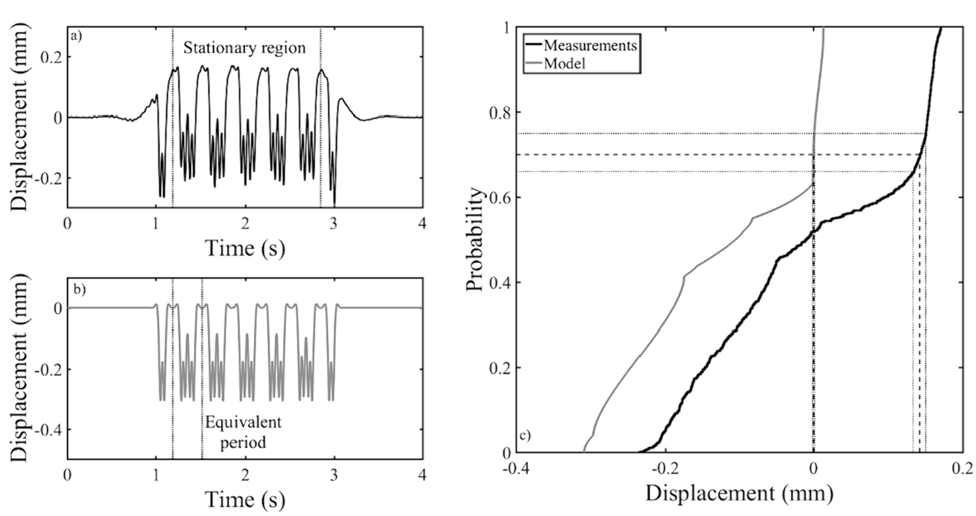


Fig. 3. (a) track vertical displacements obtained from measured accelerometer data during the passage of a Class 395 Javelin train; (b) modelled displacements calculated using a beam on an elastic foundation analysis; (c) measured and modelled displacements plotted as probability distribution curves (i.e., the probability that the downward displacement is greater (i.e., more negative) than value indicated on the x-axis). [45].

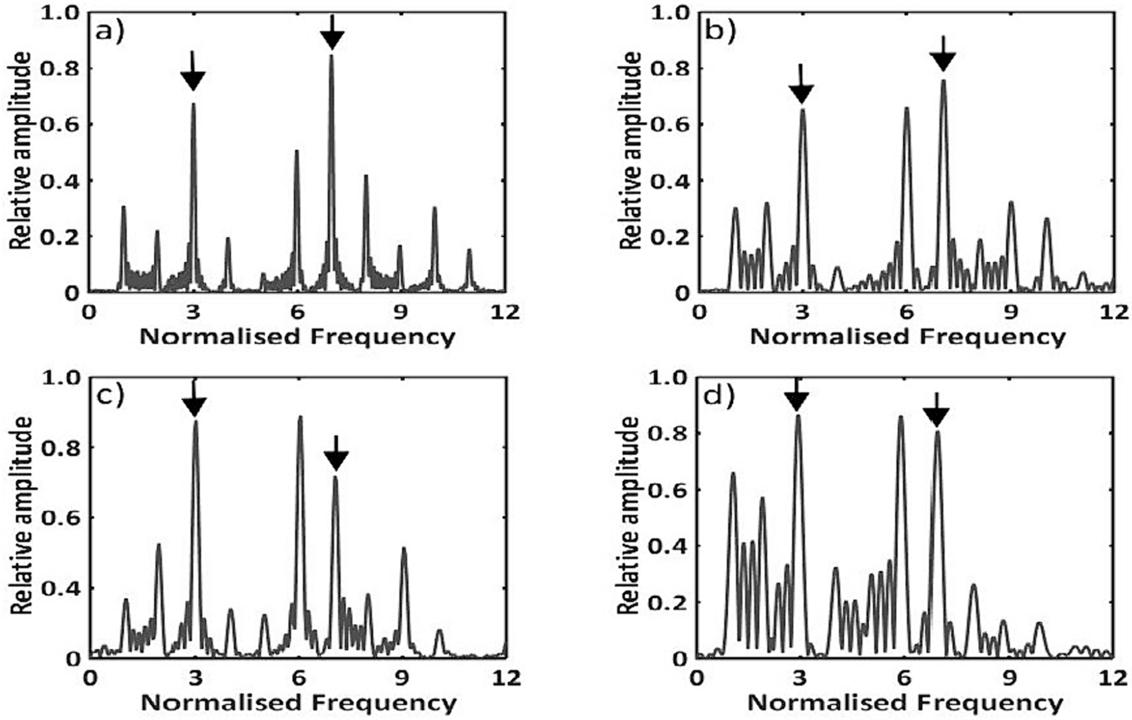


Fig. 4. Frequency spectra for measured velocity data from 4 different trains: (a) 11 car Class 390 Pendolino; (b) 5 car Class 221 Supervoyager; (c) 6 car Class 171 Turbostar; (d) 4 car Class 377 Electrostar. Frequency normalised by car passing frequency [36].

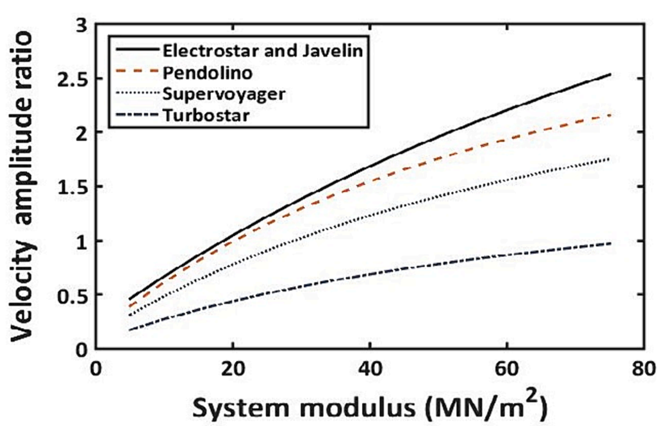


Fig. 5. Velocity amplitude ratio (7th to 3rd peaks) against rail support system modulus for different train types [36].

over a 350-sleeper length of track on a high-speed railway line in the UK.

The measured static track vertical geometry is shown in Fig. 6a, the measured deflections for a typical passage of a Class 395 Javelin train in Fig. 6b, and the inferred track support system moduli in Fig. 6c.

Four simulations were carried out using a two-dimensional dynamic finite element vehicle-track interaction model programmed in Matlab and solved in the time domain. The track model comprised a single rail formed from Timoshenko beam elements, supported by rail pads represented by springs and dampers on sleepers modelled as discrete masses. Each sleeper was connected to a ballast mass through a spring and a damper in parallel. Each ballast mass was connected vertically to a rigid foundation and in the along-track direction to the adjacent ballast masses, via springs and dampers in parallel. Each ballast mass was connected in the along-track direction to the adjacent ballast masses, and supported vertically by springs and dampers onto a foundation. The vehicle was modelled by masses representing the car body, bogie and wheels, connected through springs and dampers in parallel representing the primary and secondary suspension systems. Vehicle-track interaction was simulated using a linear contact spring. An irregular track profile could be introduced between the wheel and the rail, but the model did not include any short wavelength (<20 mm) roughness. Voids were modelled using bi-linear springs to represent non-linear behaviour arising from a gap between the sleeper and the ballast. These springs had negligible stiffness (5 kN/m) relative to the train loads until the specified initial gap had closed. Full details are given in Milne et al. [47]. The four simulations represented various combinations of ideal and measured initial track vertical geometry (absolute level) and rail support system stiffness, as summarised in Table 2.

The calculated sleeper deflections are compared with those measured in Fig. 7, and the calculated wheel-rail contact forces are shown in Fig. 8.

The results in Figs. 7 and 8 shows that, for the ranges of variation in rail support system stiffness and initial vertical level considered,

- sleeper deflections depend much more on the rail support system stiffness than on the track level, and modelling voids is locally important.
- wheel/rail contact forces depend more on the track level than on the rail support system stiffness, although voiding is again locally important.

However, the study does not explore any linkage between geometry change and the least certain component of the rail support system stiffness, provided by the ballast and the ground. For example, compliance (the inverse of stiffness) is often either explicitly or implicitly taken as a proxy for the propensity for the development of plastic strain [54,66]. Permanent settlements will then become progressively more significant where the stiffness of the underlying subgrade is lower, even if the initial geometry profile is smooth. As differential settlement develops, the track profile will become increasingly uneven as higher wheel-rail contact forces in vulnerable locations accelerate the local rate

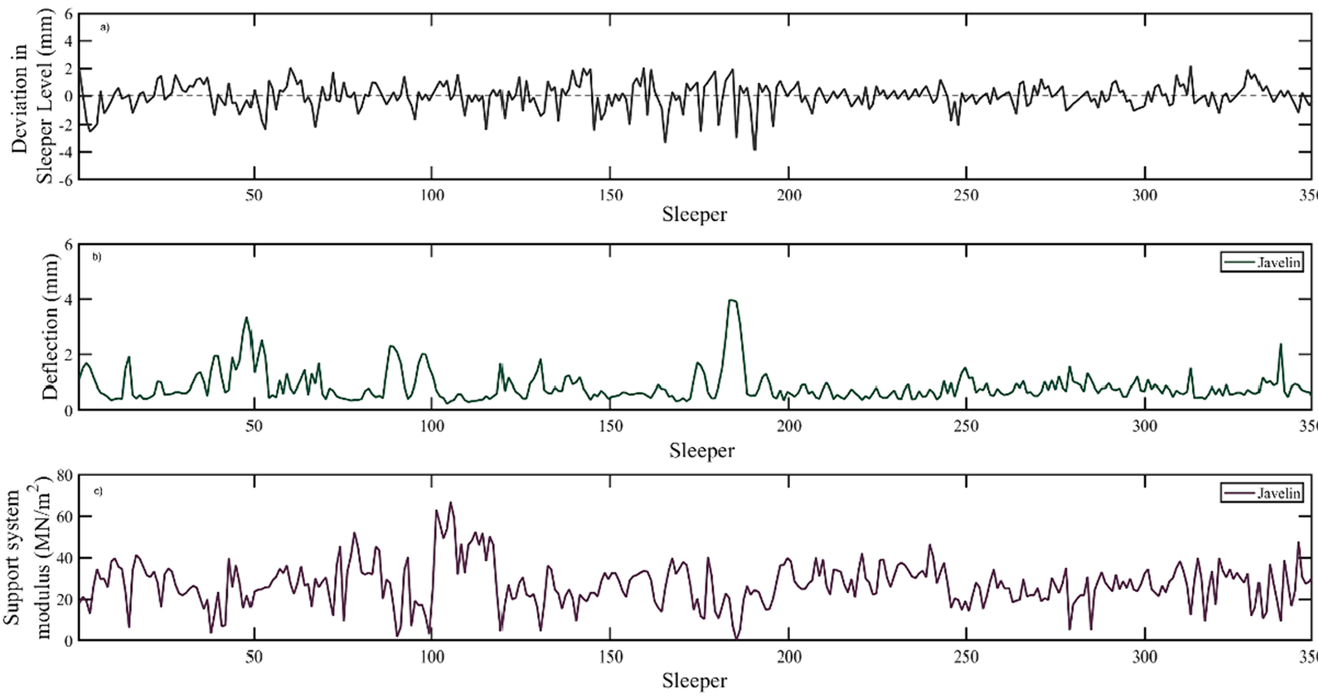


Fig. 6. (a) Static track levels; (b) typical measured deflections; and (c) inferred rail support system moduli during passage of a Class 375 Javelin train over a 350-sleeper length of track at a study site.
Adapted from [47]

Table 2
Combinations of track absolute initial level and rail support system stiffness modelled in simulations [47].

Simulation #	Track initial level	Rail support system modulus	Under sleeper voiding
1	Measured	Uniform 30 MPa	None
2	Level and smooth	Measured	None
3	Measured	Measured	None
4	Measured	Measured	Included based on measurements

Note: Measured track initial level from survey data; Measured rail support system modulus from geophone data; Under-sleeper voiding based on measured movements of large deflections at individual sleepers.

of geometry deterioration. Modelling this type of ground behaviour is complex and is a topic for future research to which we shall return, later in this paper.

Reducing ballast settlement

Track settlement may originate in the underlying earthwork or subgrade, and in the ballast. Abadi et al. [1], Abadi et al. [2] report the results of experiments carried out in the Southampton Railway Testing Facility (SRTF) to investigate the evolution of ballast settlement and interventions to the ballast or the sleeper-ballast interface that might be deployed to mitigate it. The testing facility is described in detail by Le Pen and Powrie (2011) [76]. In summary, it models a single sleeper bay of a full-size ballast bed 300 mm deep, 5 m long and 650 mm wide. A single sleeper is typically subjected to sinusoidal loading of 100 kN (that is, half a typical 20-tonne axle load) at a frequency of 3 Hz. This is not intended to mimic any actual train speed; as discussed by Powrie et al. (2019b) [59], train loading is properly represented by a series of sinusoidal loads (some negative) at frequencies up to about the 10th harmonic of the car passing frequency. Nor is the test designed to do anything other than elicit a quasi-static response from the sleeper-ballast system under investigation.

In most tests, the ballast bed sloped downward at an angle of about 45° (1v:1h) from a distance of 0.375 m from both ends of the sleeper. This is quite steep, but typical of railway track throughout the world (Fig. 9), which only ever tends to be lifted during tamping maintenance operations.

A typical vertical cyclic loading test in the SRTF involves monitoring the resilient (elastic) deflection during each load cycle and the residual (plastic) settlement at the end of each load cycle, throughout the test. Data are typically visualised as a graph of the plastic settlement against the logarithm of the number of loading cycles (Fig. 10).

Tests were carried out to investigate a number of potential improvements to the standard system of a G44 reinforced concrete monoblock sleeper, a standard-graded ballast bed 300 mm thick, no under-sleeper pad and a ballast shoulder slope of 1v:1h (the "baseline" data in Fig. 10). These included interventions to both the sleeper:

- *twin-block* and *timber*, rather than reinforced concrete monoblock sleepers.
- hard (*USP1*) and soft (*USP2*) under-sleeper pads, used with concrete monoblack sleepers.

and to the ballast:

- changing the ballast grading (*Variants 1*, *2* and *3* with increasing ranges of particle size).
- random fibre ballast reinforcement (*fibre*).
- reducing the shoulder ballast slope from 1v:1h to 1v:2h (*RPS*).
- replacement of the uppermost 50 mm of the ballast bed by a 50 mm thickness of 10/20 mm aggregate, as used for stone blowing, to form a two-layered ballast system (*TLB*).

Other approaches not considered here include reinforcement of the ballast using geogrids (for example, [13]), or of the subgrade using geosynthetics (for example, [31]).

Grading data for the various ballast types used are summarised in

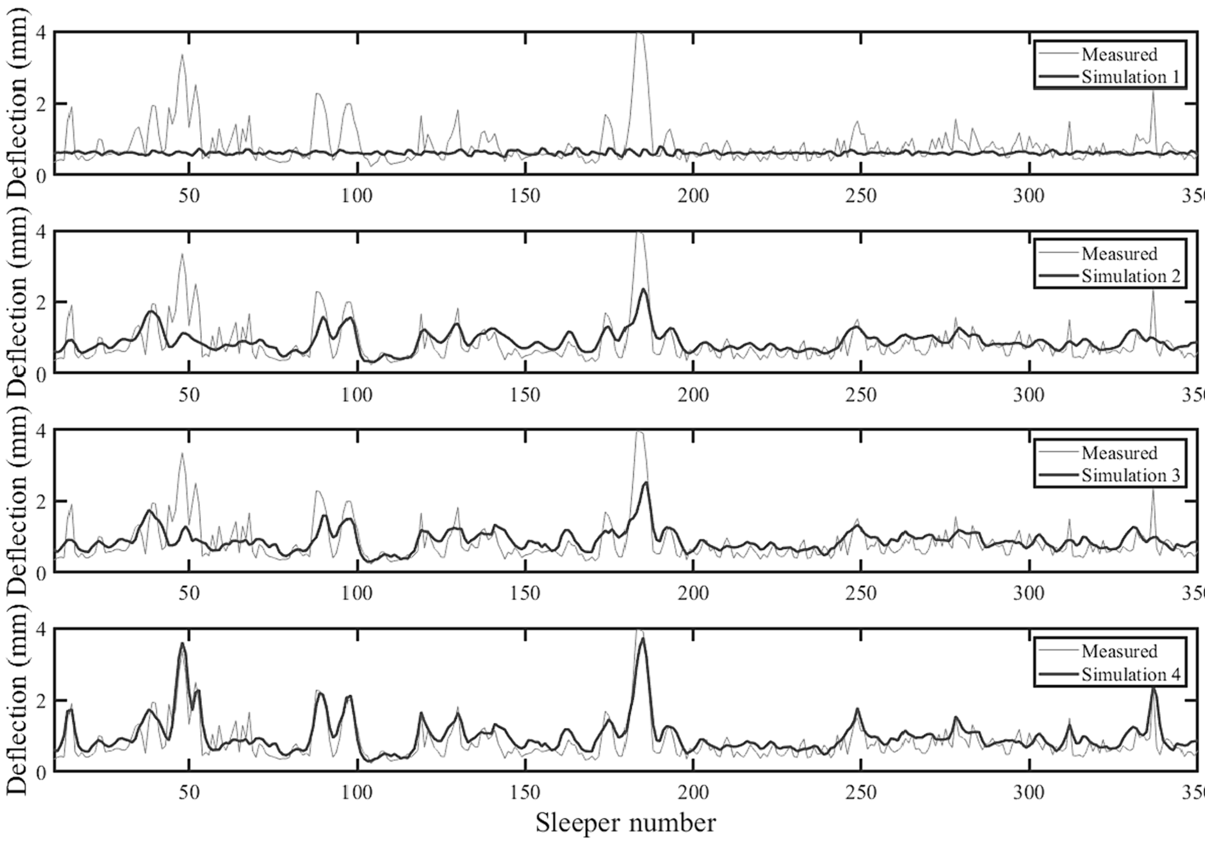


Fig. 7. Calculated and measured sleeper deflections during train passage [47].

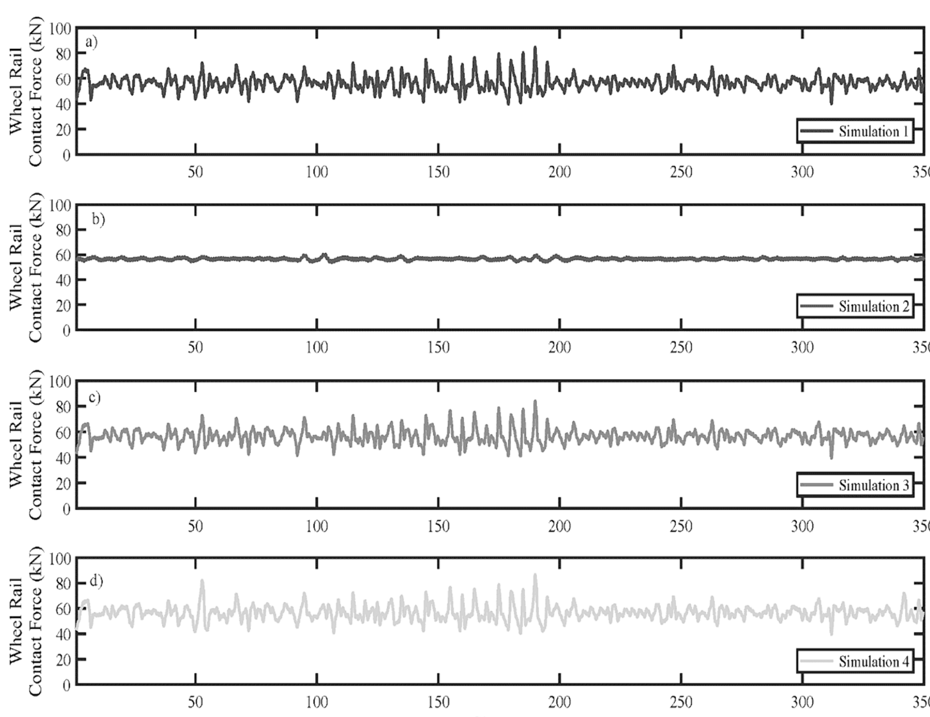


Fig. 8. Calculated wheel-rail contact forces during train passage [47].

Table 3. Results from the tests are shown in Fig. 10a for sleeper-based, and Fig. 10b for ballast-based interventions.

The results show that any of the interventions investigated could potentially reduce the rate of settlement compared with the standard case. Supporting measurements and data obtained in the original experiments by Abadi et al. [1], Abadi et al. [2] gave reasoning and

Fig. 9. Typical ballast shoulder edge slope; railway line between Bad Nieweschans, Groningen, Netherlands and Bunde, Lower Saxony, Germany. The track in the foreground is heading due west (towards the right-hand side of the picture). Date of photograph: 7 September 2023.

provisos as follows.

- Under-sleeper pads reduce plastic settlement of the ballast primarily by increasing the actual area of contact and number of grain contacts between the sleeper and the ballast, creating a more stable interface. This was demonstrated using pressure-sensitive paper at the sleeper-ballast interface to record contacts during the tests. The under-sleeper pads also reduce ballast attrition, which though not excessive with a high-quality granite ballast did occur to a limited extent at the interface with a hard concrete sleeper.
- Changing the ballast grading to increase the proportion of smaller grains can reduce plastic settlement. However, the proportion of smaller grains must be sufficient to improve the interlocking and mechanical stability of the ballast. Variants 1 and 2 in Fig. 10 did not have a sufficient proportion of smaller grains to achieve this, whereas Variant 3 did.
- the addition of random fibre reinforcement was shown in supporting triaxial tests to improve ballast ductility [4–5]. However, attention must be paid to the proportion and shape of the fibres; work by Ferro et al. [23] has shown that round filaments are preferable to flat in terms of not disrupting the initial packing/density of the ballast.
- Timber and twin-block sleepers both gave lower cumulative plastic settlements than the standard G44 monoblock sleeper, but reducing the shoulder ballast slope from 1v:1h to 1v:2h gave the greatest beneficial effect. Image analysis showed substantially reduced ravelling of grains down the shallower slope, indicating much less lateral spread. This is consistent with other tests that have shown huge benefits in terms of reducing plastic settlements of confining the ballast laterally. It also suggests that lateral spreading is a major contributor to ongoing ballast settlement in the railway context.

The observations in relation to under sleeper pads and ballast grading are consistent with findings from other researchers around the world. For example, under sleeper pads were adopted as standard in Austria in 2006 [42]and have been specified for use at certain locations on the UK rail network; while ballast grading has been changed in Australia following work at the University of Wollongong and latterly University Technologie Sydney [32]. The most effective remedy, reducing the shoulder slope or confining the ballast laterally, has not really found traction; possibly because at many locations there is not enough space or it is just too difficult (Fig. 11).

The lack of space to widen to reduce the steepness of the ballast shoulder slope down from the sleeper end may arise for at least one of several reasons. Ballast might have been added to compensate for historic settlement of the earthwork, although in adjusting the level of track it seems that often the only way is up. Embankment crest widths may have been limited by a desire to reduce construction costs – historically in terms of money, but there would also be a cost in terms of carbon dioxide emissions. In this regard, the calculations presented earlier would indicate an additional carbon cost of about 70 kg of CO_2(e) per linear metre for widening the crest of the traditional earthwork indicated in Table 1 by 0.75 m on each side. This is about 12 times the carbon cost of a tamping maintenance intervention (estimated at about 6 kg CO_2(e) per linear metre). Hence the "right" decision is on these numbers not obvious, and other factors such as the relative value of carbon savings in time and secondary carbon costs, for example incurred if the line is closed for maintenance, would need to be considered quantitatively and in greater detail.

Localised anomalies and defects

Insights into the effects of a localised variation in track support system stiffness and voiding can be gained with reference to the behaviour of under-track crossings (UTX). These are ducts used to enable signalling cables, other services or water courses to cross beneath the track from one side to the other. Depending on the depth and method of installation (including any backfilling) and the material from which the duct is made, under-track crossings may offer a stiffer or less stiff support to the track than the surrounding ground.

The effect on sleeper vertical deflections of a 376 mm square, 9-channel HDPE cable duct at a depth of about 600 mm below sleeper base (soffit) level in plain line at Oddingley, near Gloucester, was monitored using geophones and modelled numerically using elastic finite element analysis. The analysis was carried out using the program Abaqus [18] in 2D, representing conditions on the track centreline, over a length of 165 sleepers (approximately 107 m). A generic two-carriage train with primary and secondary suspension was run across the model domain at the site line speed of 40 m/s. The ballast, firm clay embankment and natural ground were modelled as three separate elastic layers, with a further zone representing the UTX bedding material. The model is shown schematically in Fig. 12, and the geotechnical materials properties used are given in Table 4. Track component parameters are given in [58].

Fig. 13 compares the sleeper deflections measured at the site (a) with those calculated by the finite element model (b) assuming perfect initial contact with the underlying ground and (c) after the introduction of under sleeper gaps representing the measured sleeper deflections. The sleeper base pressures assuming perfect initial contact with the underlying ground (d) and after the introduction of gaps (e) are also shown.

Fig. 13 shows that the difference in support stiffness at the UTX has apparently relatively little effect on the calculated deflections and stresses if the sleepers are initially in perfect contact with the ballast. Immediately above the UTX (Sleeper 0), the vertical stress is slightly increased and the displacement slightly reduced compared with the trackbed immediately adjacent. This is a result of the relatively stiff modelled bedding and backfill material surrounding the plastic duct; although the duct is relatively compliant, the combined effect of the duct and the bedding and backfill is a UTX that is slightly stiffer than the surrounding embankment. This mirrors the practical difficulty of matching the stiffness of the surrounding ground exactly.

Although the direct impact is small, it is possible that these slightly increased stresses could result in an infinitesimal increase in the residual plastic strain during each load cycle, which over hundreds of thousands of load cycles could accumulate to a significant differential settlement and potentially recursive (positive feedback) behaviour. This was not modelled in the analyses, which assumed purely elastic material behaviour. Some modern soil mechanics models are able to represent this effect (for example [75,74,67] but are unproven over hundreds of load cycles, let alone the millions of cycles to which railway track subgrade is subjected. Further, there is a concern that the output of a numerical model seeking to simulate such a complex process simply reflects the characteristics of the constitutive law(s) used. This is a topic requiring further research and validation against high-quality field data.

To replicate the measured deflections in the model, it was necessary

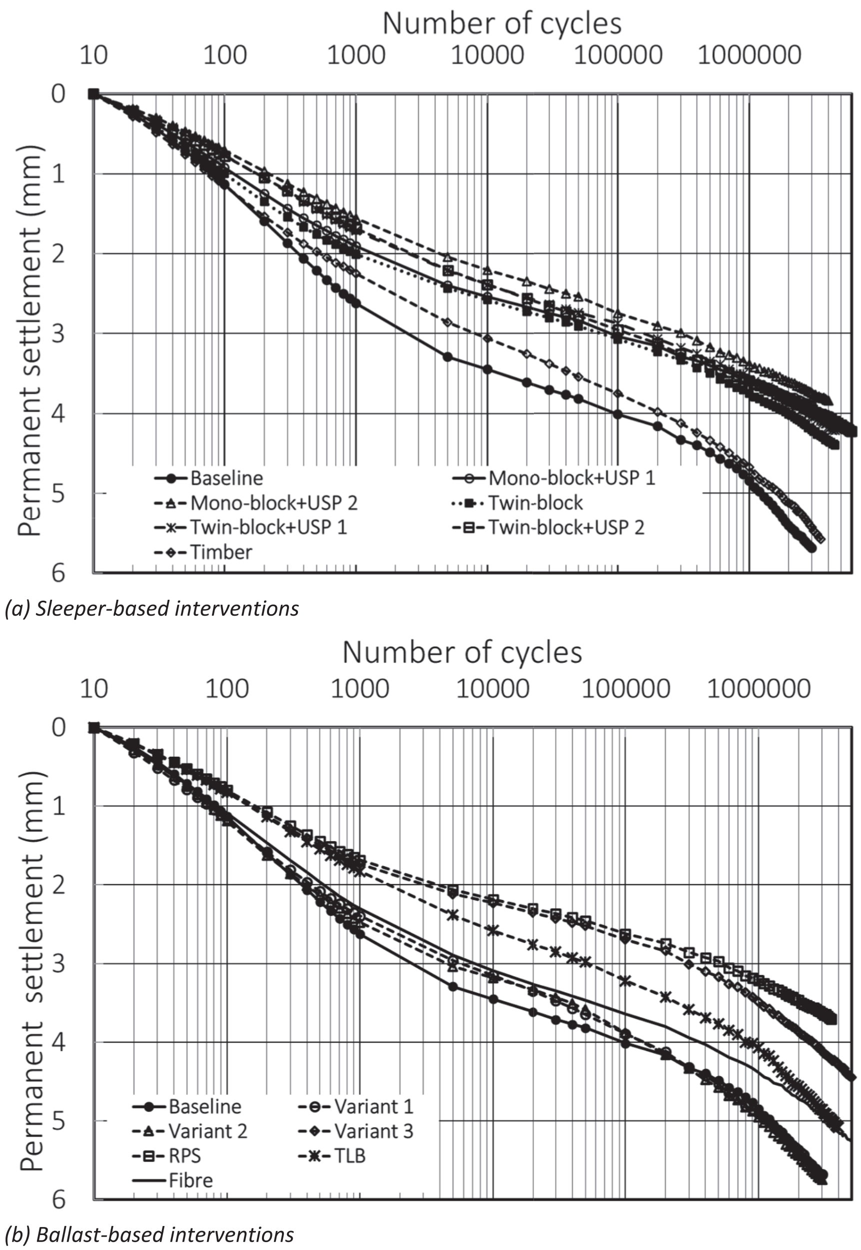


Fig. 10. Permanent settlement vs the logarithm of the number of loading cycles for different (a) sleeper-based and (b) ballast-based interventions in the Southampton Railway Test Facility [1].

Table 3
Grading (particle size distribution) data for the ballast types used in sleeper vertical load tests reported by Abadi et al. [1].

Grading	Particle size (mm)						Bulk density (kg/m^3)	
	D_{10}	D_{50}	D_{70}	D_{90}	D_{100}	C_u	Loose	Dense
NR	28	38	43	49	63	1.45	1418	1625
Variant 1	20	34	42	48	63	1.93	1453	1672
Variant 2	15	33	41	49	63	2.52	1517	1681
Variant 3	15	27	24	48	63	1.94	1512	1744
10/20	12	16	18	21	23	1.38	1432	1608

to introduce gaps below the sleepers into the analysis. These gaps (geometry imperfections) did then result in significant in-cycle variations in sleeper deflections and stresses transmitted to the subgrade (Fig. 13c and e).

Tamping by large, powerful track-mounted machines has long been established as the default method for restoring railway track geometry. However, as already mentioned it is not an ideal approach. It has been known for at least half a century that tamping damages the individual grains, causing breakage and abrasion. It destroys the structure that has developed over hundreds of thousands of cycles to effectively support vertical loads, to such an extent that it is not readily regained on subsequent trafficking [3]. An element of judgement is required in terms of the degree of overlift needed such that compaction under subsequent train passages will bring the track to the desired level. Furthermore, as demonstrated by Milne et al. [46] using measurement and analysis techniques developed at Southampton and described in this paper, tamping may be ineffective at repairing specific defects covering only a few sleepers.

This is illustrated by the example in Fig. 14, which shows a defect manifest by excessive vertical movement of individual sleepers under train loading, at a transition from twin block to monoblock sleepers associated with a series of switches and crossings (S&C; Fig. 14a). The

Fig. 11. Nowhere to go – the difficulty of reducing or eliminating the ballast shoulder slope; railway line near Kincraig, Badenoch and Strathspey, Scotland. The left-hand photograph is looking approximately south-west and the right-hand photograph north-east. Date: 25 June 2018.

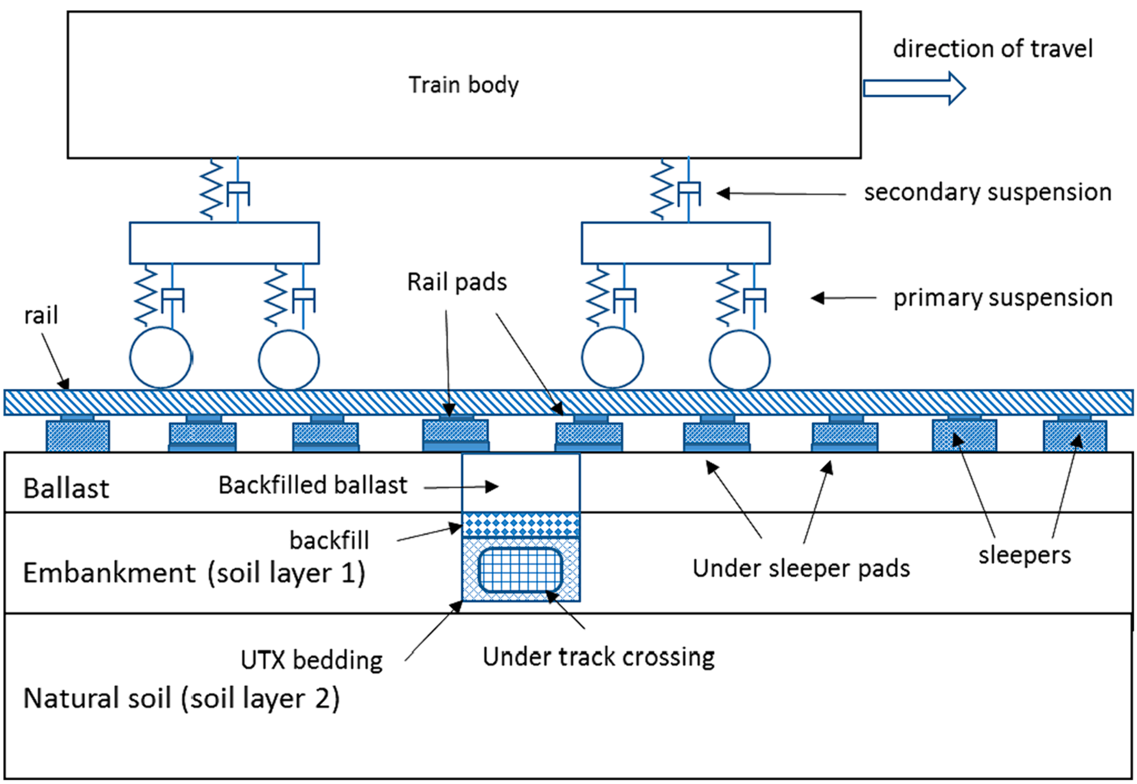


Fig. 12. Schematic of 2D elastic finite element model of flexible under-track crossing (UTX) installation at Oddingley [58].

Table 4
Geotechnical materials properties used in finite element model of Oddingley UTX [58]. Track component parameters are given in Powrie et al. [58].

Material	Layer thickness (m)	Young's Modulus E' (MPa)
Ballast	0.3	150
UTX bedding	0.2	40
Embankment (firm clay)	1.7	24
Natural ground	14	30 + (7× depth)

sleepers suffering excessive movement were probably voided or hanging (unsupported). MEMS accelerometers were used to determine vertical sleeper displacements during train passage, before and after a conventional tamping operation (indicated by the left hand vertical dashed lines in Fig. 14b). As shown in Fig. 14b, conventional tamping at best made no difference and at two of the three measurement locations increased the severity of the defect. The measured displacements were therefore used to inform a targeted repair, in which under-sleeper pads were retrofitted to the twin block sleepers. This was wholly successful in eliminating the excessive movement, both initially and for the subsequent period of five months during which measurement using the MEMS accelerometers continued.

Predicting track settlement

The ability to model and predict the evolution of track settlement

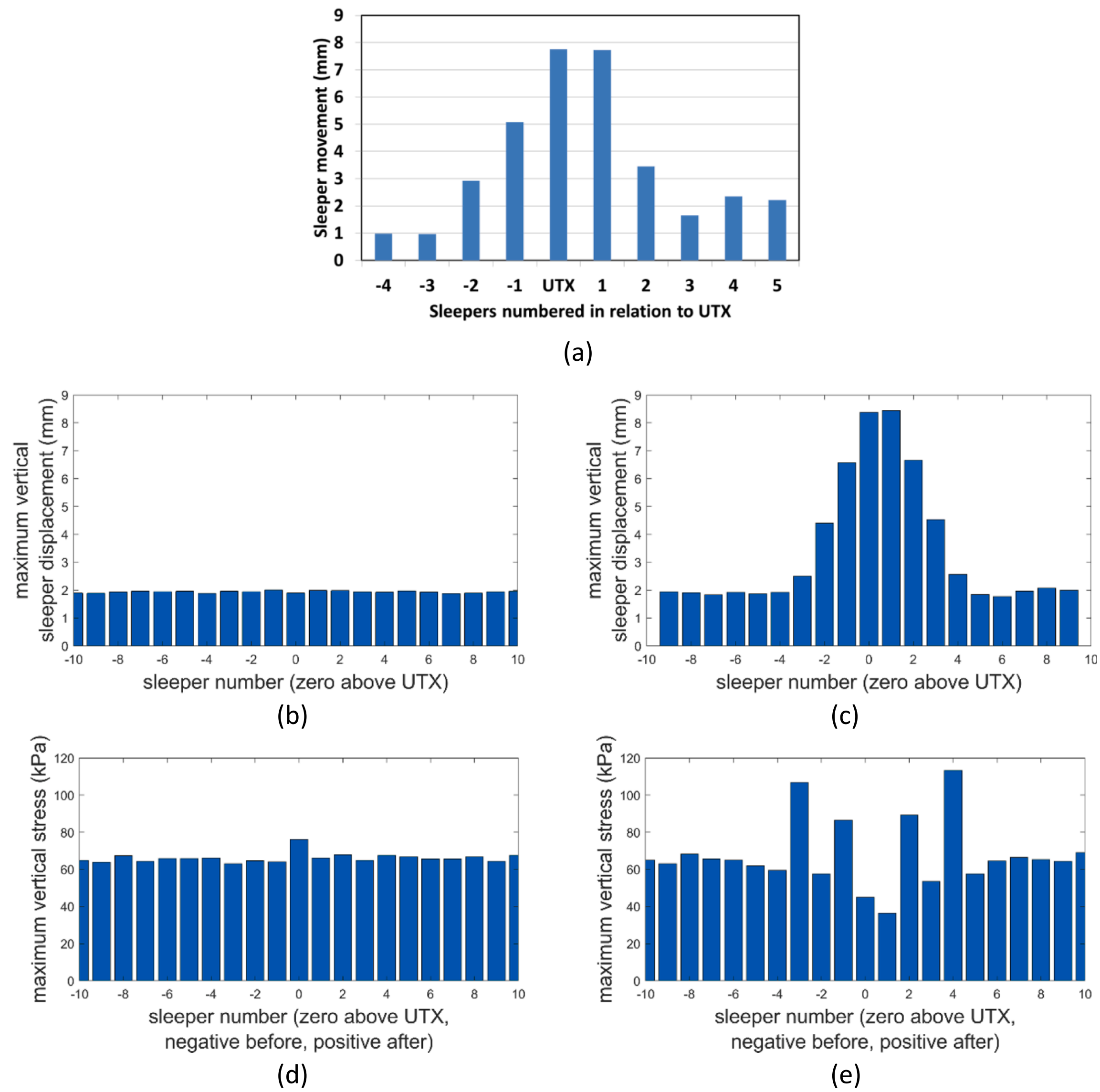


Fig. 13. Sleeper deflections (a) measured at the site; calculated by the finite element model assuming perfect initial contact with the underlying ground (b) and after the introduction of under sleeper gaps representing the measured sleeper deflections (c); modelled sleeper base pressures calculated (d) assuming perfect initial contact with the underlying ground and (e) after the introduction of under sleeper gaps [58].

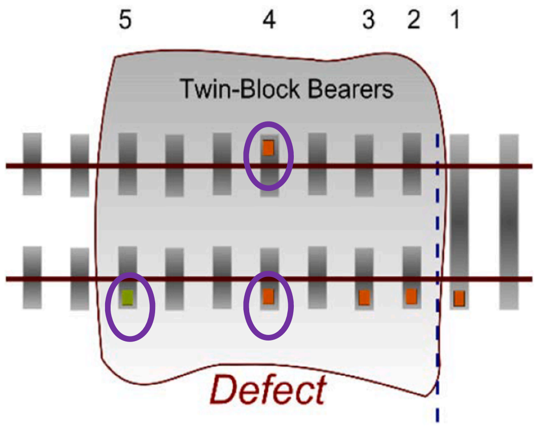
with trafficking and over time would be useful for maintenance planning and understanding the whole life financial and carbon cost/benefit ratio of alternative track system and trackbed designs.

Many empirical equations for estimating the rate of development of plastic settlement of railway track with train passage have been proposed. Reviews such as those by Dahlberg [17] and Grossoni et al. [28] show that these equations (i) do not reproduce the relationships of settlement vs number of load cycles seen in the field, (ii) do not reflect current knowledge of the behaviour of soil subgrades in cyclic loading, and (iii) are often critically dependent on the curve fitting parameters used, which in turn depend on the circumstances in which the calibration data were obtained. A further challenge is that differential settlements, which lead to "long" (in the order of metres) wavelength rail unevenness, are of much greater concern than modest uniform settlements and cannot be predicted without some form of along-track variation. A stochastic approach to quantifying the along-track variation in rail support system stiffness is proposed by Le Pen et al. [37], but is not yet routinely adopted in practice.

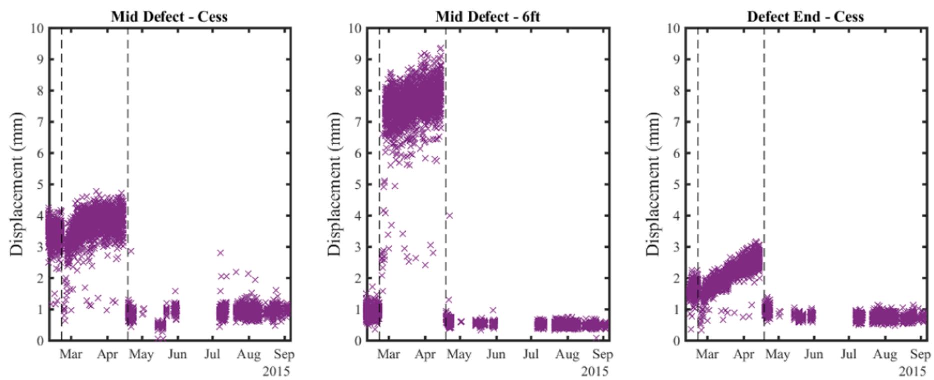
Track engineers often associate the accelerated development of track defects with an inadequate support stiffness. It is likely that the experiential perception is based on low stiffness being a proxy for plastic settlement. Work reported by Milne et al. [47] and Powrie et al. [55] has shown that variations in track level have a far greater effect on in-cycle vehicle dynamic loads than realistic variations in support stiffness (as long as a sleeper is hanging or voided). However, as already noted, it remains to be determined whether even small variations in dynamic load associated with small variations in support stiffness, coupled with small differences in the propensity for the development of plastic settlement, lead through positive feedback to the development of irregular vertical geometry.

The Hatfield rail disaster in October 2000[1] triggered a huge programme of research to understand and develop models for wheel-rail interaction with a focus on short wavelength roughness, crack growth, and fatigue of the wheel and the rail. There has been no corresponding effort on the effect of track support conditions, which are generally still modelled as elastic (as above), even in otherwise sophisticated vehicle-

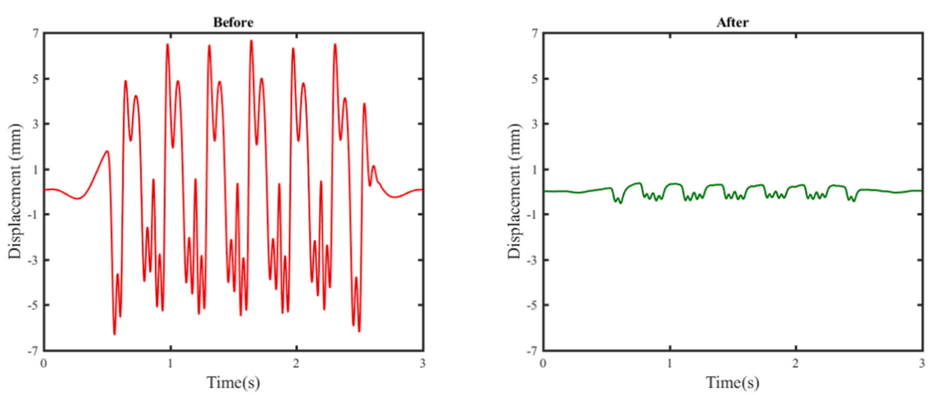
[1] https://webarchive.nationalarchives.gov.uk/ukgwa/2013https://doi.org/1001175041/http://www.rail-reg.gov.uk/upload/pdf/297.pdf.



(a): Schematic plan of defect zone



(b): Evolution of representative deflections of selected sleepers during train passage over time: each cross represents an individual train. The left hand vertical dashed line indicates a conventional tamping intervention and the second vertical dashed line a targeted spot repair based on the measured sleeper deflection.



(c): Displacement of sleeper 4 (6ft rail) during passage of a Javelin Class 395 train passage (i) before and (ii) after the targeted repair

Fig. 14. (a) Plan view of defect, (b) representative sleeper deflections before and following a conventional tamping intervention and targeted repair, (c) typical sleeper displacements during train passage (i) initially and (ii) after the targeted repair. [46].

track interaction (VTI) analyses. However, empirical (for example, [50]) and semi-analytical (for example, [28]) settlement equations have been incorporated by some authors.

The semi-analytical approach proposed and demonstrated by Grossoni et al. [28] models the ballast and subgrade as having an elasto-plastic response, in which plastic strains occur above a certain threshold stress. If the initial threshold stress is exceeded, the threshold stress is increased to the maximum previous stress for subsequent load cycles. The threshold stress is greater for materials of higher stiffness; otherwise, increasing the subgrade stiffness increases the rate of plastic settlement, which is counter to general practical experience. (This perhaps suggests that the threshold is better represented as a strain than as a stress, as noted by [40,41]. Calculated plastic strains are used to update the variation in track level along a length of track every 10,000 cycles or so in a vehicle dynamics-based vehicle-track interaction analysis. Starting from an initial variation in vertical track geometry based on actual measurements, and using the subgrade and vehicle parameters indicated in Table 5, the approach is shown to give an evolution of mean vertical settlement with number of loading cycles that is within the range calculated using other popular empirical formulae proposed by Guerin [29], Frohling [25] and Sato [68] (Fig. 15).

However, the approach is still unable fundamentally to account directly or transparently for the effects of embankment geometry or changes in earthwork or subgrade moisture content and effective stress state associated with vegetation, climate and drainage effects; hence its value as a predictive, rather than an exploratory or explanatory tool is limited. The effects of principal stress rotation on the stress-strain response of the subgrade at shallow depths may also need to be considered (for example [26,12,40,41,7,8]. A fundamental soil mechanics model that does reproduce known soil behaviour under conditions of dynamic cyclic loading [74] has been implemented into a finite element code and used in conjunction with new understandings of the loading actually applied by trains [44] to investigate stresses and resilient/permanent settlements in an embankment as a train passes to demonstrate the feasibility of the approach [67]. However, it is some way from being capable of general use and the computational demand is high. Hence other methods of calculation for routine use, for example a high-cycle accumulation model of the type proposed by Niemunis et al. [51], will be needed. High-quality field data for validating the models, whose absence is a weakness of nearly every current railway track settlement model, are also essential. These are important areas of future research.

Effects of train speed

Dynamic components of train loading, over and above the static train weight, may result from lateral accelerations (including during curving) and vertical accelerations of unsprung mass due to variations in track level or support stiffness. These are routinely calculated and accounted for in vehicle-track interaction analyses such as that illustrated in Fig. 11, in which the train is represented by a multi-body mass-spring-dashpot system and the track and its support as a beam on an elastic half-space or a Winkler-type elastic foundation. In this approach, the response of the track and its support system is usually taken to be quasi-static – that is, the accelerations of the track and the ground are assumed to be negligible. This is usually an acceptable approximation, with modest increases in track vertical deflection with train speed, as reported in numerical analyses and field measurements by Powrie et al. [56], attributable largely to an increase in the dynamic component of the moving vertical load.

At higher train speeds on soft ground, a disproportionate increase in vertical displacement with increasing train speed can occur. The effect was reported by Madshus and Kaynia [39] with reference to the observed behaviour of a high speed (200 km/hour) railway constructed on a shallow (~1.5 m high) embankment on ground comprising a firm crust about 1.5 m thick overlying a 6 m depth of soft organic clay at

Table 5
Vehicle and track support parameters used in semi-analytical approach reported by Grossoni et al. [28].

Parameter	Value
Vehicle axle load	22.5 tonnes
Vehicle unsprung mass	1350 kg
Trains speed	80 km/hour
Number of sleepers	80
Mean trackbed modulus	169 MN/m^2
Trackbed modulus standard deviation	23 MN/m^2
Mean rail support system stiffness (modulus) assuming a railpad stiffness of 60 MN/m per railpad	60 MN/m^2

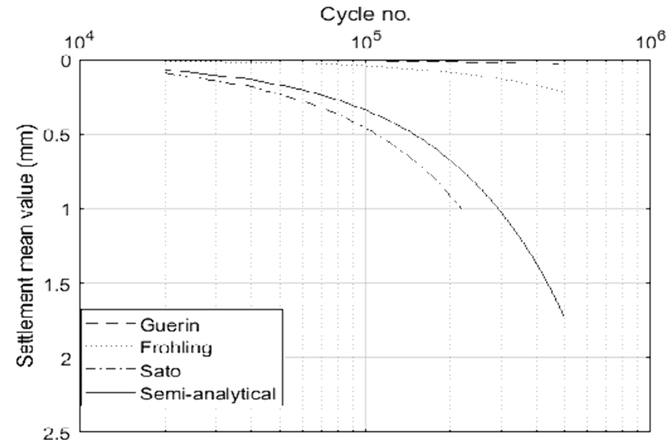


Fig. 15. Evolution of plastic settlement calculated in vehicle-track interaction analysis using elasto-plastic combined trackbed response [28].

Ledsgård, Sweden. Other instances are discussed by Woldringh and New [71] and by Duley et al. [21], among others. The train speed at which track deflections during train passage peak is known as the critical velocity. It corresponds approximately to the velocity of Rayleigh waves in the underlying ground. For a uniform elastic layer ("half-space"), the Rayleigh wave velocity v_R is given approximately by

$$v_R = v_S \times \left(\frac{0.862 + 1.14\upsilon}{1+\upsilon}\right) \tag{1}$$

(see, for example, [24]), where ν is the Poisson's ratio and v_S is the shear wave velocity, which is related to the soil shear modulus G and the bulk density ρ by the expression

$$v_S = \sqrt{\frac{G}{\rho}} \tag{2}$$

The shear modulus G is related to the effective stress Young's modulus E and the Poisson's ratio ν by

$$G = \frac{E}{2(1+\upsilon)} \tag{3}$$

For Poisson's ratios in the range 0 to 0.5 (the latter representing an undrained soil response), Equation (1) suggests

$$0.876 < \frac{v_R}{v_S} < 0.954 \tag{4}$$

or

$$\frac{v_R}{v_S} \approx 0.9 \tag{5}$$

However, soil is not really an elastic material; and the nature of the track system (for example, ballast or slab) together with the presence of soil layers of different stiffness and the tendency for soil stiffness to increase with depth make the critical velocity a system parameter rather than a property associated with an individual soil. This is recognised by Madshus and Kaynia [39], Woldringh and New [71], Duley et al. [20], Duley [19] and others, whose analyses are generally based on a layered soil system. Duley [19] matches the strains calculated by models with various sets of trial parameters with the soil stiffnesses appropriate to those strains, determined from resonant column, bender element and advanced triaxial tests.

Critical velocity effects are conventionally illustrated on a normalised plot of vertical deflection divided by static deflection against train speed divided by Rayleigh wave speed, as shown in Fig. 16.

The normalised plot masks the fact that in absolute terms the

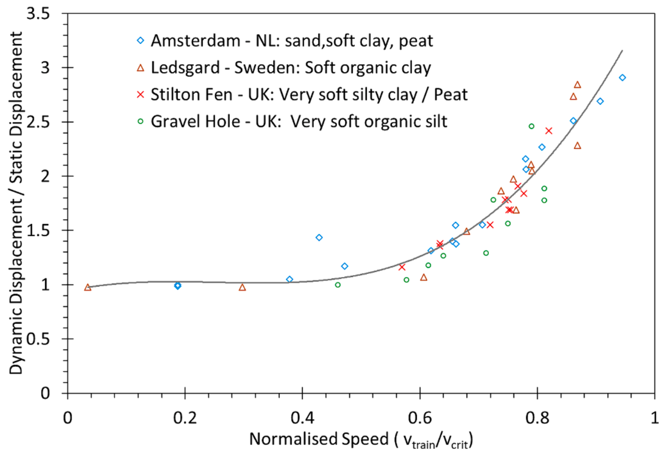


Fig. 16. Typical plot of dynamic displacement ÷ static displacement against train speed ÷ Rayleigh wave speed [19].

displacements decrease with increasing stiffness and critical speed. For soft ground, the static displacement is large and could easily be amplified to an alarming if not dangerous degree at a relatively modest train speed. This was the case at Ledsgård. High speed trains may travel at 60 % or more of the critical velocity even for stiffer ground – however the static displacement is much reduced, so amplification is less of an issue. This is illustrated in Table 6, which summarises the results of calculations carried out for a representative train on a typical ballasted track supported on a homogeneous elastic half-space representing various ground types. The calculations were carried out using the program MOTIV (Modelling Of Train Induced Vibration), by Vangelis Ntotsios following the approach of Ntotsios et al. [52], with track parameters and subgrade Poisson's ratio from their Case B1.

Concern has also been expressed about the effect of loading frequency on components of the track support system, notably the ballast. The first point that needs to be considered is the appropriate frequency of loading in laboratory or element tests. This is not, as is sometimes suggested, the frequency associated with the passage of four successive axles on adjacent bogies at the ends of adjoining carriages (vehicles). It is shown by Milne et al. [44] that for a long train (>about 3 vehicles), a spectrum of loading is applied to each sleeper that may be idealised as a complex Fourier series, with loading coefficients at multiples of the car passing frequency (Fig. 17). To replicate this loading in the laboratory or a test rig, it would be necessary to apply this spectrum of loading rather than a single sinusoidal load at a single frequency [59].

Secondly, real systems are damped, both through the properties of the materials (although friction might be a more appropriate conceptual model for soils than viscosity) and through radiation damping, which results in the more rapid attenuation of higher frequencies of loading with depth, as demonstrated in field measurements by Priest et al. [60] (Fig. 18) and in dynamic finite element analyses by Yang et al. [73].

The significance of a dynamic system response will depend on the accelerations induced, which will increase with increasing frequency of loading (for a given load and system stiffness), reducing system stiffness (for a given load and frequency of loading), and increasing load (for a given frequency of loading and system stiffness) (Eq. (2)).

$$a_{max} = \omega^2 .A = \frac{\omega^2 F}{k} \quad (6)$$

where A is the amplitude, F is the peak cyclic load, ω is the circular frequency of loading (=2π × the actual frequency f) and k is the simple system stiffness defined as the load required to cause a unit displacement. (The dependence of ballast acceleration on the inverse of the stiffness k was recognised by Sato [68], who suggested that the rate of ballast degradation varies in proportion to $1/\sqrt{k}$ to account for higher ballast accelerations at lower stiffness). Applying too high a load or loading at too high a frequency in a laboratory rig or element test will give unrealistically high accelerations and overstate the significance of a dynamic system or material response.

The dependence of the system response on the system stiffness and the frequency of loading is well illustrated by the results of cyclic load triaxial tests on specimens of railway ballast at different cell pressures, reported by Sun et al. [70]. Key data and outcomes are summarised in Table 7, which indicates the peak acceleration a_{max} at each stage of each test according to Eq. (6), the cell pressure and whether the specimen exhibited plastic shakedown to a stable state (no cell shading) or continuing displacements/ratcheting and eventual failure (grey cell shading).

The data in Table 7 show that the acceleration needed to cause ratcheting is generally at least 1 g, and increases with both frequency of loading and with stiffness (confining pressure). In other words, the duration of loading is important, because a short duration acceleration (at a high frequency) will not transfer enough energy to have a significant effect. This is recognised in design codes for structures such as Eurocode EN1990:2002 + A1 (British Standards Institution [11]), which specifies a maximum allowable acceleration for bridges of a_{max} = 0.35 g at frequencies up to 30 Hz.

The influence of rail support system stiffness on accelerations can be demonstrated with reference to the beam on an elastic foundation model for the track. Fig. 19 shows calculated accelerations during passage of a Class 395 Javelin train travelling at a speed of 80 m/s, on (a) well-supported track with a rail support system stiffness k_{system} = 40 MPa, and (b) poorly supported track, with k_{system} = 10 MPa.

Fig. 19 shows that, for normally performing track without hanging sleepers, accelerations do not usually approach g, even if the rail support system stiffness is low (10 MPa). The duration of the acceleration is also of potential significance. The accelerations shown in Fig. 19 were calculated every 1/500 of a second: evaluation every 1/30 of a second would reduce the calculated peak accelerations by a factor of roughly 2.

Table 6
Representative ground stiffnesses, critical and train speeds, and static and dynamic deflections. [1,39,2,15,3,30,4,10] (in review). Calculations carried out by Vangelis Ntotsios using the program MOTIV (Modelling Of Train Induced Vibration), following the approach and using the parameters for ballasted track model Case B1 of Ntotsios et al. [52]: Railpad stiffness 120 MN/m; Railpad damping loss factor 0.15; Ballast mass 1740 kg/m; Ballast stiffness 4640 MN/m^2; Ballast damping loss factor 0.1; Sleeper mass 300 kg; Sleeper spacing 0.6 m; Subgrade Poisson's ratio 0.158.

Representative soil type	Soft/ organic clay/silt	Stiff clay	Dense sand/ gravel	Weak rock (mudstone)
Typical density ρ kg/m^3	1550	2000 [2]	2040	2200 [4]
Shear wave velocity v_S at low confining stress (<c50 kPa), m/s	30 [1]	100 [2]	200 [3]	300 [4]
Rayleigh wave velocity v_R (Eq (5), m/s	27	90	180	270
Train speed $v = v_R$, km/ hour	97	324	648	972
Inferred shear modulus $G = \rho.(V_S)^2$, MPa	1.4	20	82	198
Young's modulus $E = 2G(1 + \nu)$, MPa, for Poisson's ratio $\nu = 0.158$	3.24	46.33	190	459
Static deflection for a 20 tonne (200 kN) axle load, mm	18	2.0	1.0	0.62
Deflection at train speed $v = v_R$ for a 20 tonne axle load, mm	31	4.0	1.4	0.64

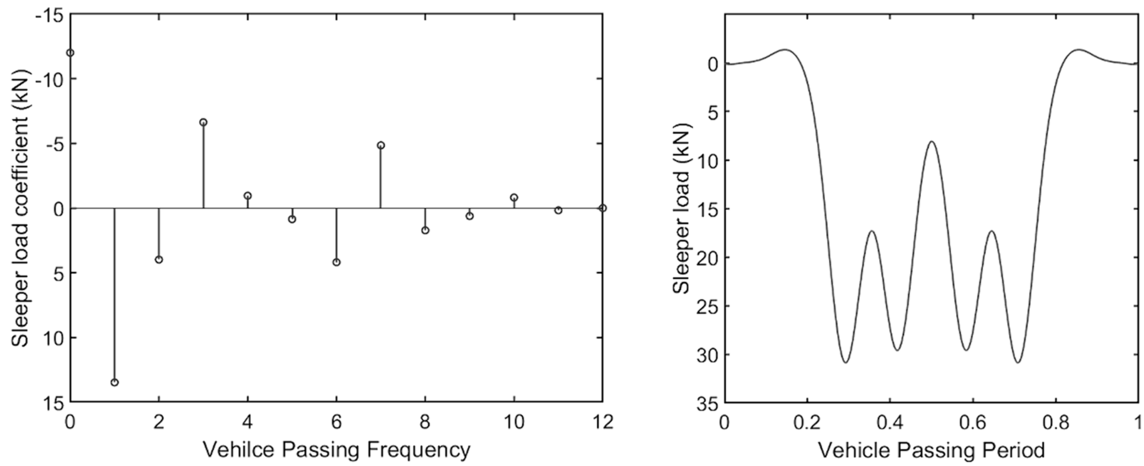


Fig. 17. Loading coefficients (per rail or per sleeper end) for sinusoidal loading at different multiples (harmonics) of the train vehicle passing frequency (a), making up the loading pattern experienced by each sleeper end per vehicle passage (b), for a 65 kN wheel load. Figure: Dr D Milne.

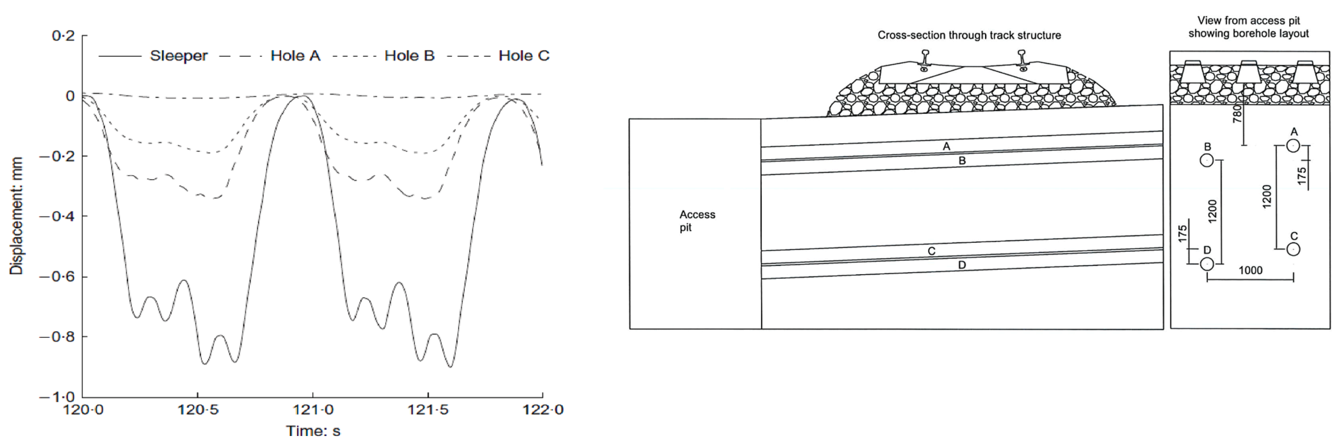


Fig. 18. Attenuation of higher loading frequencies with depth in field measurements [60].

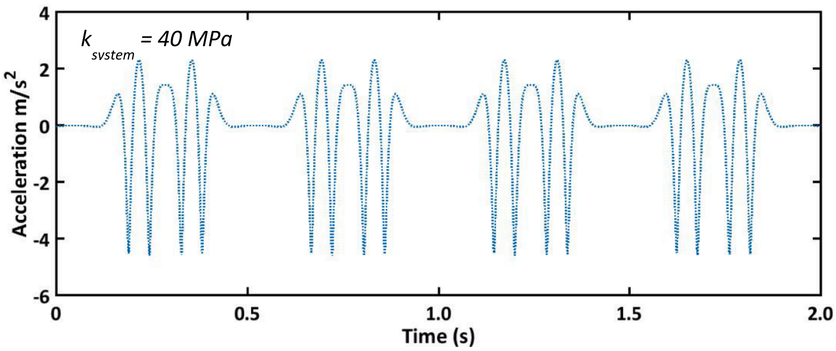
Table 7
Maximum accelerations in triaxial tests on ballast specimens at different loading frequencies and cell pressures. Unshaded cells indicate plastic shakedown to a stable state; grey shaded cells indicate continuing displacements (ratcheting). Data from Sun et al. [70].

	Frequency of loading				
Cell pressure	**6 Hz**	**12 Hz**	**18 Hz**	**24 Hz**	**30 Hz**
10 kPa	0.09g	0.35g	0.90g	3.6g	5.0g
30 kPa	0.12g	0.47g	0.86g	2.3g	5.0g
60 kPa	0.08g	0.40g	0.72g	1.5g	3.6g

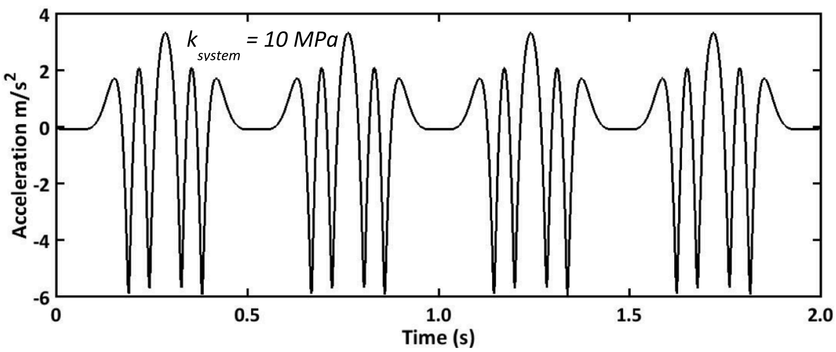
Conclusions

The paper has presented a number of key insights into railway track substructure behaviour that have arisen from research carried out mainly by the rail research group at the University of Southampton over the past decade or so. They relate to the key geotechnical performance indicators for railway track of (i) the rail support system stiffness; (ii) the vertical alignment or level; and (iii) the rate at which the level deteriorates due to plastic settlement. The work summarised has shown that:

1. Variations in track level and hanging sleepers are likely to be much more significant than variations in a continuous rail support system stiffness, in terms of their effect on the wheel-rail contact forces.
2. It is especially important to avoid unsupported (hanging) sleepers. Such localised defects can be assessed, repaired and the effectiveness of the repair confirmed with the aid of targeted monitoring using geophones, accelerometers or direct measurement of deflection through digital video image analysis.
3. For conventional railways, the range of acceptable rail support system stiffness is quite wide (10 to 50 MN/m^2 based on field measurements), as long as it is reasonably uniform.
4. Various interventions can reduce the rate of plastic settlement of ballast under train loading. Some (under sleeper pads, revised ballast grading) have been adopted in countries around the world. Reducing the ballast shoulder slope (or containing the ballast laterally) is perhaps the most effective but least adopted. This may be at least in part a result of limited space alongside the railway.



(a) well-supported track with a rail support system stiffness k_{system} = 40 MPa



(b) poorly supported track, with k_{system} = 10 MPa.

Fig. 19. Accelerations calculated using the beam on an elastic foundation model for a Class 395 Javelin train travelling at a speed of 80 m/s, on (a) normally performing track with a rail support system stiffness k_{system} = 40 MPa, and (b) poorly supported track, with k_{system} = 10 MPa. Figure: Dr L M Le Pen.

5. The potential for critical velocity effects, even on ground that would normally be considered to be stiff, increases with train speed. However, the conventional normalised plot of dynamic displacement divided by static displacement against train speed divided by critical velocity masks the general reduction in displacement with increasing ground stiffness. A dynamic displacement that is three times a static displacement of 15 mm would be a concern, but a dynamic displacement of three times 0.5 mm is not.
6. Train dynamic loading is properly represented by a spectrum of sinusoidal loads at different frequencies that are multiples of the car passing frequency, some of which will be negative (180° out of phase). Train loading cannot be adequately represented by a single sinusoidal load at a unique frequency; and to do so in laboratory tests, especially at what might be interpreted as the highest axle passing frequency, risks overstating the real loading regime.

The paper has also highlighted some current and future research challenges that need urgently to be addressed to ensure that railway systems continue to deliver safe, resilient, affordable, low-carbon transport in a world where our climate and societal expectations are seemingly changing faster than our willingness, ability or capacity to adapt. These are:

1. Preventing plastic settlement of the subgrade at an affordable cost (in terms of both money and carbon).
2. Better models for material dynamic behaviour that can be used to estimate and drive the development of methods to control the accumulation of plastic settlement in subgrade, embankment and ballast materials. These should be rooted in fundamental soil mechanics principles, and validated with reference to high-quality field data.
3. Better models for understanding and quantifying system damping, by the material properties of the track system and the ground and through the attenuation of higher frequency components with depth.

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CRediT authorship contribution statement

William Powrie: Writing – review & editing, Writing – original draft, Visualization, Supervision, Resources, Project administration, Methodology, Funding acquisition, Conceptualization.

Declaration of competing interest

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Data availability

Data will be made available on request.

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