Svensk Djupstabilisering

Swedish Deep Stabilization Research Centre

Report 10

Mitigation of Track and Ground Vibrations by High Speed Trains at Ledsgård, Sweden

Göran Holm
Bo Andréasson
Per-Evert Bengtsson
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Svensk Djupstabilisering

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Swedish Deep Stabilization Research Centre

The Swedish Deep Stabilization Research Centre coordinates research and development activities in deep stabilization of soft soils with lime-cement columns. A joint research programme based on the needs stated by the authorities and the industry is being conducted during the period 1996 – 2004. Members of the Centre include authorities, lime and cement manufactures, contractors, consultants, research institutes and universities.

The work of the Swedish Deep Stabilization Research Centre is financed by its members and by research grants.

The Swedish Deep Stabilization Research Centre is located at the Swedish Geotechnical Institute and has a Steering Committee with representatives choosen from among its members.

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Preface

The Swedish Deep Stabilization Research Centre performs a comprehensive research programme comprising theoretical research at universities and research institutes and also research and development in connection with actual projects, e. g. infrastructure projects.

Excessive vibrations were observed at a section (Ledsgård) of the West Coast Line between Gothenburg and Kungsbacka in Sweden when the traffic with high speed trains started. Banverket (the Swedish National Rail Administration) initiated a research and development project and comprehensive measurements and analyses were performed. The dry deep mixing method was chosen to reduce the vibrations. In this report the design, execution and measurements of vibrations before and after the deep mixing are presented.

The report is financed by Banverket (the Swedish National Rail Administration).

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Linköping August 2002

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1. Introduction

Railways belong to the oldest transportation systems. Construction of many of the existing railway lines started some 150 years ago under different traffic conditions as far as speed, axle loads, traffic intensity and subsoil conditions are concerned. Today, there are urgent demands from the railway organisations and the industry to increase axle loads and train speeds on the existing railway lines both for economic and environmental reasons. During the last ten years, a number of new railway lines have been built and in the years to come extensive infrastructure investments will be made in many countries. These projects comprise a large number of high speed lines. Many of the existing and new railway lines are built on soft soil deposits with low railway structures.

An example of this is the section of the West Coast Line between Göteborg (Gothenburg) and Kungsbacka, see Figure 1. Here, a new track (western track) was laid at the beginning of the 1990's close to the old track (eastern track) built about 100 years ago. Traffic with high speed trains (X2 trains) started in spring 1997 with a speed of 200 km/h. Shortly afterwards, excessive vibrations were observed at the Ledsgård site, located some 25 km south of Gothenburg. These vibrations were in the order of ten times greater than those measured earlier from heavy train traffic in soft soil conditions and had been regarded as "worst case". Train speed was reduced to 160 km/h and later to 130 km/h. In summer the same year, Banverket (the Swedish National Rail Administration) initiated an R&D project to thoroughly investigate the phenomenon. Extensive tests and investigations were thus carried out in autumn 1997. Together with analyses of the phenomenon, these have been reported in Banverket (1998) and Banverket (1999).

In 1999, the consulting company J&W designed countermeasures at the site. These were carried out in summer 2000. To investigate the effect of these countermeasures, additional measurements in high speed traffic were carried out both before they were introduced (May 2000) and afterwards (December 2000). Furthermore, measurements were made shortly after the countermeasures, in

Fig. 1. Location of the Ledsgård site.

regular traffic, to study the long term continuous increase of the soil improvement with time.

This report presents an overview of the problem, the investigations, the analyses, the measurements and the deep mixing method used to reduce the vibrations, together with the results obtained.

The whole countermeasures program was carried out in June and July 2000. Train speed was increased to 160 km/h in August 2000 and will be increased to 180 km/h later this year (2002). Trains were in operation for the old track during the countermeasure period.

2. Vibration problems in connection with high speed trains

2.1 INTRODUCTION

This chapter contains a short survey of different problems connected with track and ground vibrations induced by high speed train traffic.

When describing train-induced track and ground vibrations, the problem is often divided into three parts; source, medium and object. The source consists of train, track, pads, sleepers and the railway structure. The medium is the soil, which is normally layered/inhomogeneous. The objects are buildings, human beings and equipments, but can also be the track, catenaries and other structures belonging to the railway itself. The three parts do not behave independently of each other: there is an interaction between the source, the medium and the objects. The interface relations between railway structure and soil and between soil and buildings play an important part.

2.2 TRACK VIBRATIONS

2.2.1 Train

It is the behaviour of the train with its engine and cars that excites the track system, which in turn excites the soil and the objects. Important parameters determining the properties of the waves emitted from the source are the axle configuration, i.e. their location within the train, their loads and their spring connection to the bogie and the car itself. The velocity and acceleration of the train are also of importance. The acceleration may originate from braking (negative acceleration, retardation), positive acceleration or centrifugal acceleration when the train passes curves. There are observations indicating that braking of a train will produce relatively strong ground shaking.

Most wheels are not completely circular. Under forced braking, the rail will produce flat areas on the wheel rims. When the wheels rotate, non-circularities will produce a periodic force, which will load the rail with the wheel frequency.

At higher train speeds, the interaction of the railway structure with the surrounding soil will produce increased displacements of the railway structure. There will also be a lift, or heave, of the track in front of, between and after the wheels. At a particular train speed, the critical speed, the downward deflection will reach a maximum and so will the lift. If the train speed is increased beyond this speed, the downward displacements and the lift will decrease. This process can be regarded as a form of resonance, where the speed of the train acts as the driving frequency. At higher train speeds, the maximum downward displacement is not located below the wheel but is situated behind the wheel (in the sense of train direction). This means that the wheels are moving uphill creating a horizontal resisting force. Figure 2 shows the downward displacement and the heave measured at different train speeds at the Ledsgård site. The best fit lines for these measurements and the isolated downward dynamic amplification are also shown. From the figure it can be seen that the dynamic amplification is insignificant up to about 140 km/h and that a significant increase occurs at about 180 km/h.

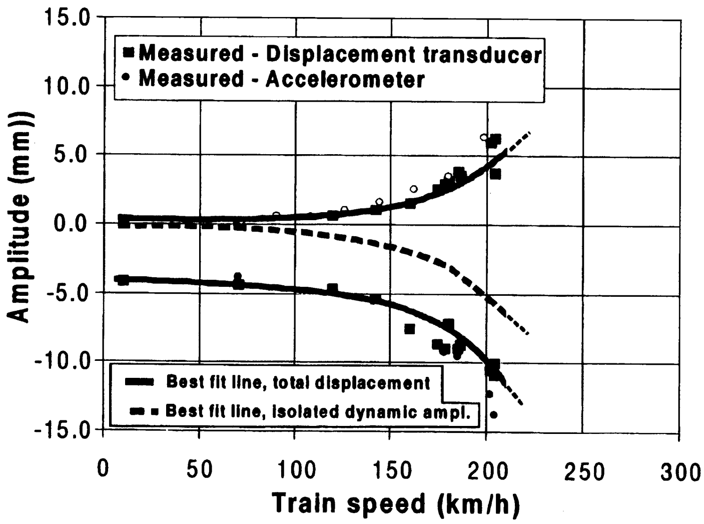

Fig. 2. Downward displacement of the track at different train speeds (lower solid line), lift (upper solid line) and dynamic (vertical) contribution (dashed line) at Ledsgård. Banverket (1999)

2.2.2 Rail and sleeper system

The system of ballasted track is used in Sweden. The track contains inhomogeneities of two different types; inhomogeneities in geometry and inhomogeneities in stiffness. These cause reaction forces in the track and the vehicles. The forces are dispersed to the environment as waves in the ground through the rails-pads-sleepers-railway structure system. The waves are emitted from a stationary point in the ground, i.e. the location of the sleepers. They will therefore be noticed as propagating waves both by an observer on the ground and a passenger on the train.

The geometric inhomogeneities are mainly surface corrugations of the rails. These corrugations have very different scales of length (wavelengths), ranging from a few millimetres to several kilometres. When a train passes the corrugations, the emitted waves will therefore have very different time scales; the frequencies of the waves will cover a large interval. When a train passes a curve, the centrifugal forces will produce horizontal forces on the rail, which in turn will produce horizontal movements of the railway structure and the ground. Switches are another type of inhomogeneity which can produce excessive vibrations in the ground during the passage of a train.

The sleepers mainly cause inhomogeneities of stiffness. Even if the rails were completely smooth, a wheel would produce time-varying forces when moving from the part of the rail which is supported by a sleeper to a part that is not supported. At constant train speed, the forces, both vertical and horizontal, will be periodic with a frequency called the sleeper passage frequency.

The vertical and horizontal forces acting on the rail system will produce reaction forces acting on the bogies and the engine and cars. These will therefore be put in motion, relative to a co-ordinate system moving with the train, and produce additional forces on the rails, and so on. The relative movement of the engine and the cars will thus excite the ground and the environment.

2.2.3 Railway ballasted system

Railway structures normally consist of friction material. The upper part is often a ballast material of crushed rock of 32–64 mm. The properties of such material under dynamic loads are not well known. The shear strains of the material can be expected to be high during the passage of a train, which means that the stress-strain behaviour will be non-linear. The shear modulus will be much lower than at low strain. The internal damping will be high in such a material. This sets additional demands when calculating the response of a train to the railway structure / the response of the railway structure to the passage of a train.

The dimensions of the railway structure are in the order of the lengths of the emitted waves. This may indicate that certain wavelengths (frequencies) are more easily emitted than others. The railway structure can therefore be expected to filter the frequency content of the emitted waves.

A railway structure often changes its dimension along the direction of the track. This may cause particular wave emission phenomena. A train travelling at a speed of 160 km/h covers a distance of 90 meters in two seconds. The subgrade modulus can change rapidly underneath a train and this will produce vibrations in the ground. There are two occasions when this problem arises; for example when a train enters and leaves a piled bridge foundation or a section with soil improvement. The soil is then loaded by a transient force, which will produce vibrations in the soil.

If the material is loaded many times at a fairly high level of strains, the material may degrade, i.e. each load cycle will reduce its moduli to small values. This may cause stability problems in the subsoil and the railway structure. Degradation is often accompanied by pore pressure build-up in the material.

Since trains have dimensions in the order of the emitted waves, there will probably be directional effects in the emission of the waves, i.e. the waves are more strongly emitted in certain directions than in others, both horizontally and vertically down into the soil.

2.2.4 Source-soil interface

The source and the soil around it interact dynamically. If the source is considered as constant, the change in the properties of the soil will alter the character of the emitted waves. The vibrations in the environment will thus also be different. It is therefore important to study this interaction. Soft soil adjacent to the railway structure may also undergo large shear strains. Because of non-linear effects, the shear modulus will decrease, sometimes to a value only one quarter of that at small shear strains. This means that the shear wave velocity in this volume decreases to half its value at small strains.

Another effect of great importance is the pore pressure build-up in the soil beneath the railway structure. This can lead to settlements and may even cause stability problems in some cases. This problem was studied at Ledsgård.

2.3 ENVIRONMENTAL VIBRATIONS

The passage of a train will produce a static elastic settlement of the railway structure and the ground. This settlement, sometimes called a 'trough', will accompany the train during its motion along the rail. If the speed of the train increases, the trough will be enlarged in the direction perpendicular to the track; the geometric damping will be less in that direction. In the direction of the track, the trough will be shorter. The trough will also be deeper under the train. When the speed of the train is equal to the speed of the Rayleigh surface waves, the trough will theoretically be very deep and will change in the direction perpendicular to the track to a lesser degree than it changed at low speeds. In that direction, there is a smaller geometric damping. This is illustrated in Figure 3.

One property which plays an important role for train-induced ground vibrations is the velocity of the different waves, (P-, S- and R-waves) propagating in the soil.. If the speed of the train exceeds any of these three wave velocities, the character of the propagation of the waves is drastically changed. If the train speed is lower than the Rayleigh wave velocity of the ground, the speed is called sub-seismic and the waves will interfere to create the 'trough' that was mentioned above. If the speed is higher than the Rayleigh wave velocity but less than the P-wave velocity of the soil, the speed is called trans-seismic. The Rayleigh waves will then form a shock front similar to an aeroplane when flying faster than the speed of sound. If the speed of the train is higher than the P-wave velocity of the soil, the speed is called super-seismic. Both the Rayleigh wave and the P-wave will produce shock fronts. The shock fronts will give rise to high levels of vibration of the soil. Normally, the waves are attenuated ac-

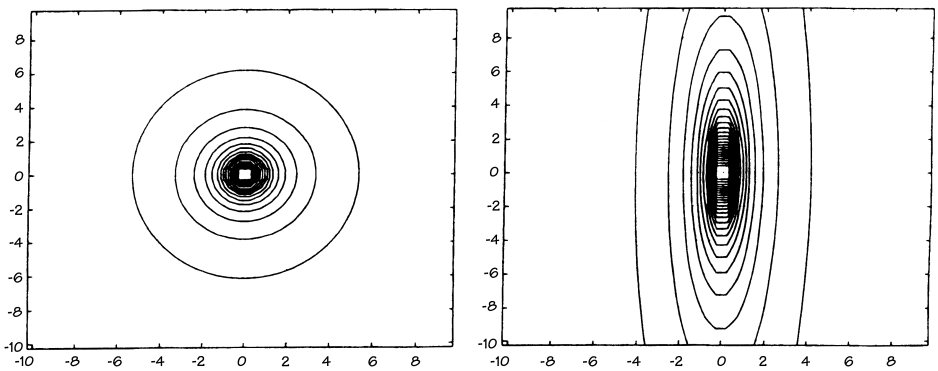

Fig. 3. Iso-displacement lines for the surrounding soil when the force moves from left to right in the figure with 25 % of the Rayleigh wave velocity (left) and 98 % of the Rayleigh wave velocity (right).

cording to the common inverse distance law. However, resonance and focusing effects may produce other attenuation relations. Note that this effect is completely different to that of 'critical velocity', as described in Chapter 2.2.1.

Focusing of wave energy can occur in two different ways. The first is if the layers, particularly the bedrock, are concave and act like a parabolic reflector. The second way is that also horizontal parallel layers may refract the downward rays upwards so that they meet at the surface or close to it in limited areas.

The topography of an area will also affect the propagation of waves. It can amplify as well as attenuate the level of the vibrations. Topographic effects that amplify the level of vibration can be found on hilltops and in the soil at the foot of outcropping rock. Also at the rim of a river valley or gully there may be an increased level of vibrations.

The internal damping (dissipation) of the medium plays a major role in wave propagation. As the strain increases, the internal damping also increases. For the same shear strain, the damping is higher for frictional materials (sand, silt) than for cohesive materials.

2.4 VIBRATIONS IN CATENARIES AND OTHER RAILWAY STRUCTURES

Vibrations and dynamic movements in the track structure may be transmitted to the catenaries and the overhead contact wire. Another dynamic source is the moving train and collector influencing the contact wire. A prerequisite for safe operation of high speed as well as low speed trains is perfect contact between the overhead contact wire and the pantograph collector. Qualitative analyses of the variation in the characteristics of the system with train speed and the main parameters of the wire-pantograph system are needed.

There are several critical speeds that are dependent on the stiffness and support of catenaries, stiffness of the wire, tension force of the wire, properties of the pantograph collector, etc. The dynamic process in the wire and in the contact with the collector is generated by the movement of the pantograph collector itself and by imperfections in the geometry of the wire and the track. These processes are influenced by the parameters of the collector, the wire, etc. An unfavourable combination of these parameters and of driving speed may result in a loss of stability of the motion, whereby various resonance phenomena may occur, which results in excessive wear and danger of failure of the whole wire-collector system and reduced efficiency of the electrical supply system. Fluctuation in the contact force may result in short-term losses of contact between the wire and the collector and, consequently, in arcing and mechanical impacts at contact. This phenomenon must be minimised as far as possible, since it results in high mechanical wear and reduced capacity of transmission routes.

2.5 DESCRIPTION AND EXTENT OF THE VIBRATION PROBLEM

Problems in connection with large track motions imply that the trains cannot run at the designed operating speed, which causes delays and extra costs for the track owner. Severe vibrations also have a detrimental effect on railway equipment and cause greater wear, so that maintenance costs increase further.
The problems of modelling track motions arise from insufficient knowledge of the dynamic material properties in the railway structure, the ballast.
Environmental vibrations disturb people both outdoors and inside buildings. There may also be cosmetic cracks in walls. Sometimes, permanent settlements will take place. Measures to combat these problems often entail extensive engineering work and a financial burden for the track owner.

The problems in modelling the environmental vibrations arise from the material properties of the soil, particularly in the vicinity of the track, where non-linear effects may be obtained. The profile of the material properties may also change along the line of the track.

Severe motions in the catenaries and other railway structures may jeopardise the security and safety of traffic since signal systems, for example, may be affected.

At a number of places on certain railway lines in Sweden extensive vibrations from high speed trains have been observed. The problem has also been noticed in a number of European countries.

3. The Ledsgård Site

3.1 GENERAL

The Ledsgård site is situated north of Kungsbacka about 25 kilometres south of Gothenburg on the West Coast Line in Sweden. The site is near a river, the Kungsbacka ån, in the centre of a large plain. The ground is lowest close to the river and rises towards the north.

At the Ledsgård site there is a very soft organic soil (gyttja) below the dry crust. Below the gyttja there is soft clay. The river is probably the origin of the gyttja layer described below. At some time, the river has probably been re-routed so that today it passes outside the gyttja area where the railway line passes.

At the site there are three tracks (a western track, a middle track, here called the eastern track, and a station track). Figure 3 shows a cross-section of the railway structure. High speed traffic travels along the western and middle/eastern tracks. The western track was built in the early 1990, whereas the eastern track is more than 100 years old. The western track experienced the most severe vibrations. Countermeasures to reduce the vibrations were performed only for the western track.

3.2 GEOTECHNICAL CONDITION

From a geotechnical point of view, the Ledsgård site is rather special, with a pocket of very soft organic soil (gyttja) of maximum 3 m thickness below the dry crust. This gyttja extends approximately 200 m along the track. The layer has its maximum thickness in the measurement section 24+265. The maximum cone liquid limit is 250 % and the maximum organic content 20 %. The gyttja is underlain by soft clay and the depth to bedrock is more than 60 m. Geotechnical investigations have been carried out in phases. Detailed information on the Ledsgård site, including the results of extensive field and laboratory investigations, is presented in Banverket (1998) and Banverket (1999).

In connection with the R&D project of 1997/98, the test section and its immediate surroundings were investigated with penetration tests, sampling, shear wave velocity measurements and laboratory investigations, including dynamic triaxial testing. The results were presented in Banverket (1999). In connection with the design of countermeasures, additional soil investigations were carried out along a 400 m section with a potential need for countermeasures to reduce the vibrations, thus identifying the extent of the gyttja pocket. In the measurement section (24+265), the railway structure is approximately 1.4 m thick and consists of crushed rock (macadam). Figure 4 shows the soil conditions in the test/measurement section.

Special soil investigations were performed. Cross-hole and down-hole measurements were made to determine the shear wave velocity in the different soil layers. The shear wave velocity is in the order of 40 m/s in the gyttja layer. In the underlying clay, the shear wave velocity is approximately 60 m/s, increasing to 90 m/s at 14 m depth. Figure 5 shows the measured shear wave velocity.

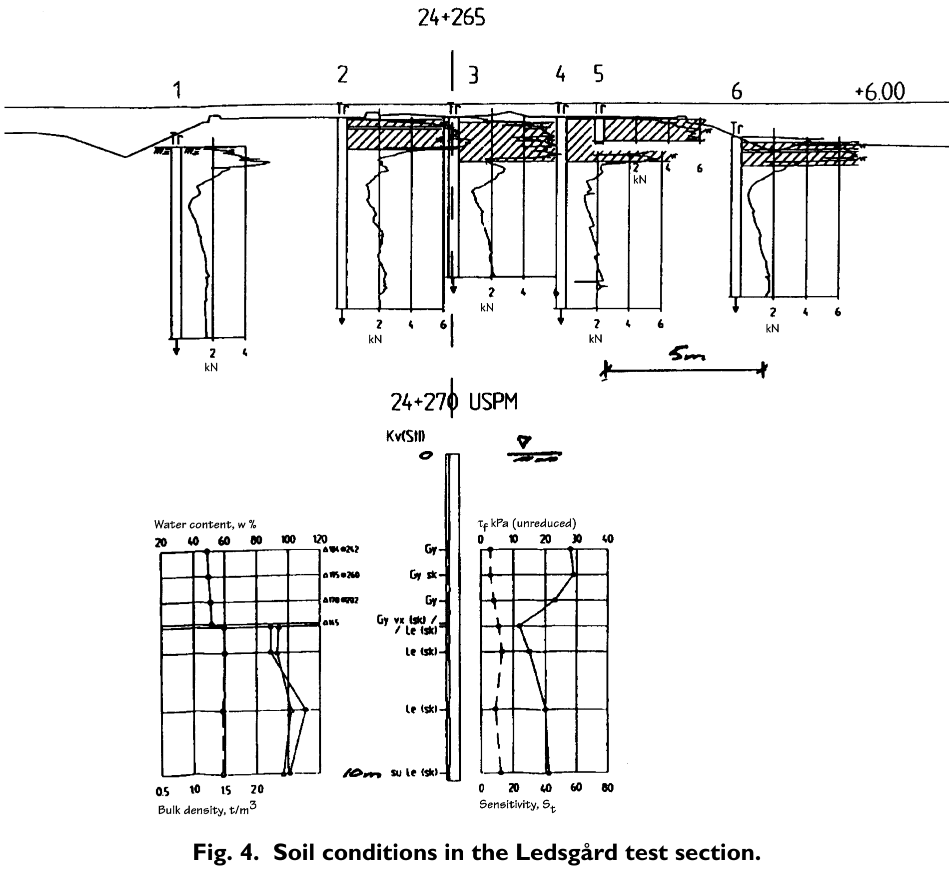

Fig. 4. Soil conditions in the Ledsgård test section.

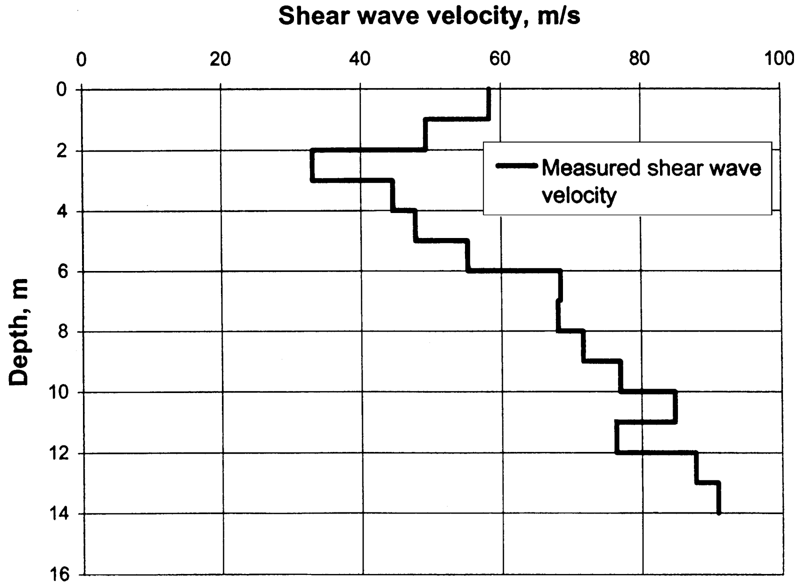

Fig. 5. Shear wave velocity at Ledsgård.

The main result of the additional soil investigation carried out in connection with the design of countermeasures is presented in Figure 6. The figure shows a section along the line with the gyttja pocket sketched in (based on 10 sampling points). The gyttja is subdivided into two groups with the natural water content above and below 150 %. The maximum water content observed is slightly in excess of 200 %. (200 % water content corresponds to approximately 17 volume-% solid material and 83 volume-% water, i.e. a 1:5 ratio of solid material/ water).

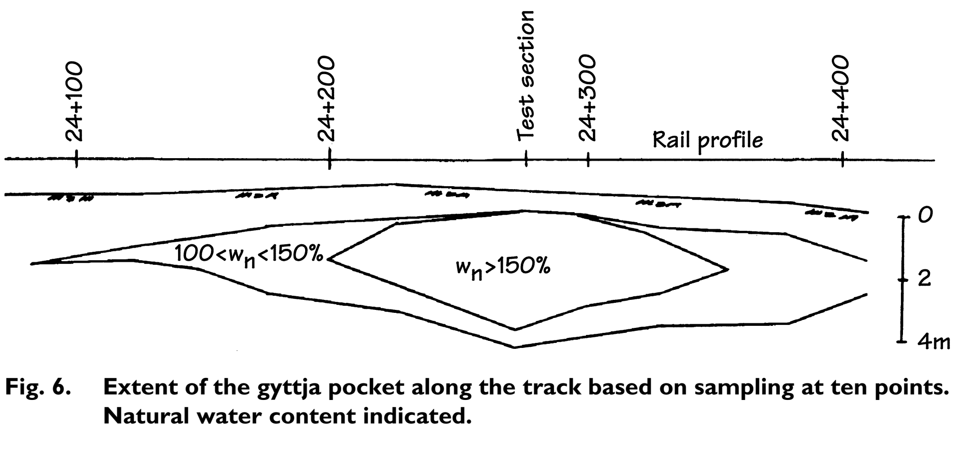

Fig. 6. Extent of the gyttja pocket along the track based on sampling at ten points. Natural water content indicated.

To limit the countermeasures in the northern direction, additional vane tests (in combination with sampling) were carried out in three boreholes (sections between 24+000 and 24+100). All borings yielded shear strengths (corrected with respect to cone liquid limit – 70 to 100 %) of approx. 15 kPa below the dry crust to some 8 m depth. These results, together with additional information, were used in designing/limiting the countermeasures, see Chapter 4.

3.3 SETTLEMENT AND VIBRATION OBSERVATIONS

To follow the behaviour of the railway line at Ledsgård, Banverket carried out settlement measurements on the section by levelling the western track at regular intervals. The result of such measurements for an eight-month period in 1998/99 are shown in Figure 7. The figure clearly shows that the settlements start in Section 24+000. The figure also shows the result, in terms of relative level, of vibration measurements carried out under regular traffic at 160 km/h. The measurements were made at 50 m intervals by accelerometers mounted on 30 cm diameter concrete blocks cast between sleeper ends. These measurements clearly show the extremely soft behaviour in the test section and the local extent of high vibration levels.

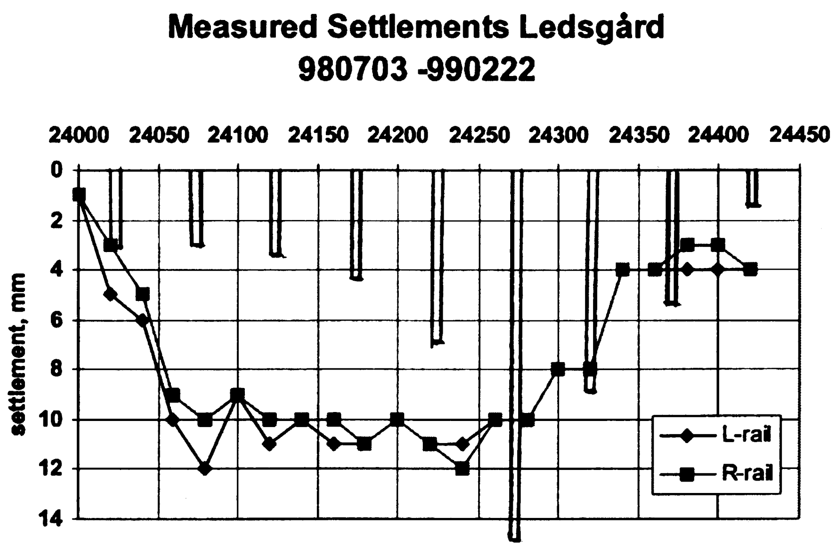

Fig. 7. **Settlement observations of the western track at Ledsgård (both rails) together with relative vibration levels along the studied section.**

3.4 RAILWAY BALLASTED STRUCTURE

The railway structure for the new track was constructed at the beginning of the 1990's without any ground improvement measures in the area with gyttja. The track consists of UIC 60 rail placed on Pandrol rubber pads (10 mm) and concrete sleepers with a spacing of 0.67 m. The total height of the structure was 1.4 m, consisting of ballast (0.5 m) and sub-ballast (0.9 m). The old track was built about 100 years ago, also without any ground improvement. Close to the river, Kungsbackaån, soil improvement by deep mixing and piling was performed for constructing the new bridge.

3.5 TRAINS AND TRAFFIC

The traffic on the western and middle/eastern tracks includes high speed trains, com-muter trains and freight trains. The high speed trains were passenger trains (X 2000) with a maximum speed of 200 km/h.

4. Countermeasures

4.1 REQUIREMENTS AND REGULATIONS

The appearance of the high-speed phenomenon at Ledsgård in spring 1997 was a new occurrence for Banverket. Thus, there were no requirements or regulations dealing with the problem at the time. Based on the discovery and early studies, a Technical Note “Geodynamic Analyses” (TM 97-059 “Geodynamiska analyser”) was published by Banverket in January 1998. Recently, Banverket Regulation BVF 585.13 “Jorddynamiska analyser” (“Soil Dynamic Analyses”) has been accepted.

The Technical Note, which was valid at the time of design of the countermeasures at Ledsgård, addressed the high-speed phenomenon by specifying the minimum stiffness, or shear wave velocity, in the soft soil beneath the railway structure. The note also presented methods for mitigating the effects of the phenomenon, suggesting the dry deep mixing method (normally lime/cement columns), preferably in a wall configuration, as an attractive method. In the new Regulation, the design approach is to increase the critical speed (the train speed producing the maximum deflections in the track) to a certain level above the maximum train speed (normally by a factor of 1.5 – 2). With a detailed analysis of maximum developing deflections, the above factor can be somewhat reduced. Maximum track deflection is 4 mm for new lines.

4.2 COUNTERMEASURE METHODS

Based on the good results of soil improvement with the dry deep mixing method (dry DMM) achieved in the area, also in gyttja type material, and the recommendation in the Technical Note mentioned above, the dry DMM (normally lime/cement columns) was a prime alternative from the beginning of discussions on countermeasures at Ledsgård. However, laboratory investigations had to be carried out to verify the effect of the improvement, especially in the gyttja.

Other methods which can be considered for solving the high-speed phenomenon include a stiff, piled concrete deck and a stiffening beam in the railway structure itself. Certain calculations were carried out for the latter method of a beam structure corresponding in bending stiffness to a plate with a thickness of approximately 0.45 m.

4.3 MATERIAL PROPERTIES

In analysing the high-speed phenomenon and designing suitable countermeasures, the most important parameter is the stiffness of the various components of the railway structure and the ground. The stiffness, together with the geometry and density, governs the wave velocity, which in turn governs deflections and the critical speed of the whole system. The dynamic properties of the soil material at Ledsgård have been thoroughly investigated in the Banverket studies, (Banverket, 1998) and (Banverket, 1999). In regard to the material in the railway structure, the stiffness properties are deduced with empirical methods presented in the literature. The stiffness of the structural components such as the rail and concrete (beam solution) is quite well known.

However, of prime importance in the design of the dry DMM improvement is the stiffness of the improved soil. To estimate this property, laboratory tests on samples of improved soil were carried out. The stiffness, together with the undrained shear (compressive) strength, was determined through a series of unconfined compression tests. The test results in a stress/strain curve from which the undrained shear strength and the E-modulus can be deduced. The tests were carried out at SGI and an example of the results is presented in Figure 8.

Important topics in designing effective soil improvement with the dry DMM are to determine the amount and proportion of binder components as well as the required mixing energy. An initial recommendation, which proved so successful that it was kept throughout the project, was to use 150 kg binder per treated m^3 gyttja with the unslaked lime/cement in a proportion of 25/75. Outside the gyttja pocket, the amount was reduced to 120 kg/m^3 in a proportion of 50/50.

The unconfined compression tests showed a very good effect of the treatment. Figure 9 shows the results in terms of undrained shear strength and E-modulus versus time for three prepared batches (two of gyttja and one of clay). Corresponding tests on untreated samples resulted in an undrained shear strength of 20 kPa and an E-modulus of 1.1 MPa. (The E-modulus is deduced from a

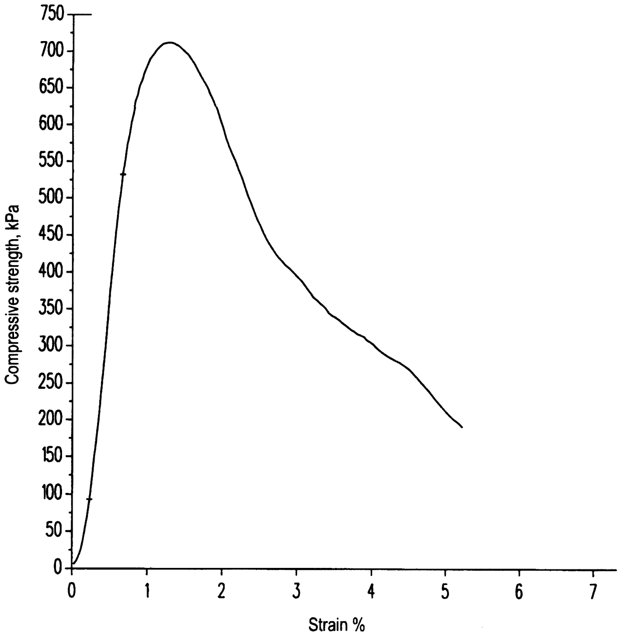

Fig. 8.
Unconfined compression test on laboratory mixed sample.

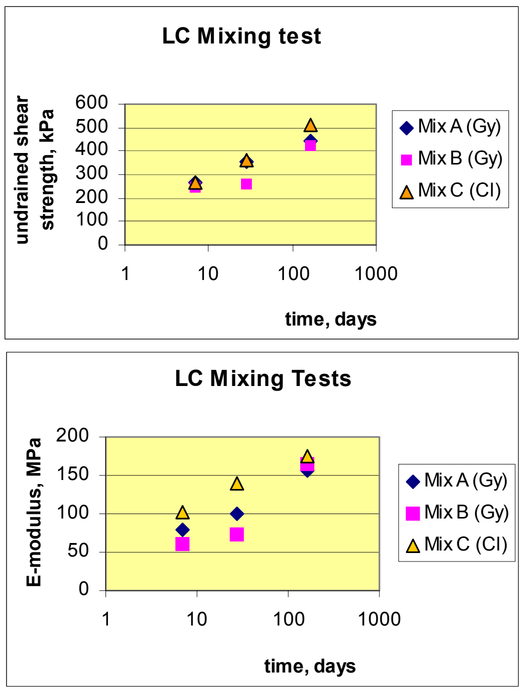

Fig. 9. **Results of unconfined compression tests on lime/cement laboratory mixed samples of gyttja (Gy) and clay (Cl) (150 kg/m^3 lime/cement in a proportion of 25/75).**

straight portion of the compression test curve corresponding normally to a compression in the order of 0.5 %). The compression at failure is approximately 1 % with a brittle failure mode.

Figure 8 clearly shows the very good effect of the lime/cement stabilisation. The compression tests were carried out 7, 28 and 162 days after mixing and the increase with time of both strength and stiffness is very clear. (The samples were stored in room temperature for the first 7 days, then in a climate room at 7 °C).

The value of stiffness of the treated soil to use in the dynamic design of the railway structure/foundation is a complex and important question. In situ, the stabilised soil is confined and the stress/strain levels are very low compared to the laboratory tests. This indicates higher values than in the laboratory. On the other hand, the mixing process is very different in the field compared to laboratory mixing.

4.4 CALCULATION METHODS AND RESULTS

4.4.1 Initial estimates

Different approaches/methods have been used in designing the countermeasures. Initially, based on the recommendations in the "regulations" at the time, the approach was to increase the stiffness, or rather the strength, of the soil to a certain specified extent. The increase in stiffness was assumed to be proportional to the increase in strength, which is usually estimated to be in the order of ten times the strength of the unstabilised soil. With this approach, the required stabilisation ratio (percentage of soil being stabilised) was approximately 40 %. Should the stabilisation effect be only five-fold, a complete (100 %) stabilisation would be required.

4.4.2 FLAC analyses

A key question when studying the high-speed phenomenon is the critical speed or the wave propagation velocity in the railway structure/ground system. One way of calculating the wave velocity of the coupled system of the stiff railway structure and a sub-soil of variable stiffness is to use the finite difference program FLAC (2D). This program can also study the effect of countermeasures, in terms of the wave velocity.

The result of the FLAC calculations, assuming a conservative ten-fold increase in stiffness and a 40 % stabilisation ratio, was that the wave velocity (corresponding to the critical speed) increased by approximately 25 and 50 % with columns in single pattern and in walls along the track, respectively. With a more optimistic/realistic approach, corresponding to the actual measured behaviour of the track after reinforcement, the critical speed increases from an initial 60 – 65 m/s to approximately 150 m/s (with columns in walls). However, the FLAC analysis does not answer the question of developing deflections; neither does it fully represent the three-dimensional system of the railway structure and its surroundings. Therefore, the FLAC3D program was used as final design tool.

4.4.3 FLAC3D analyses

The FLAC3D finite difference program has been extensively used in the studies of the Ledsgård high-speed phenomenon within the Banverket R&D projects, see Banverket (1999). The correlation between calculated and measured behaviour was found to be quite good.

FLAC3D is a versatile program by which the railway structure, the subsoil and the surroundings can be effectively modelled. Different material models can be introduced. However, in these studies only elastic soil properties have been utilised. Material damping can also be introduced (of limited effect on the resulting behaviour since the radiation damping dominates).

In short, the method of analysis is to apply a certain moving load on top of the model representing the railway structure on the ground. Depending on the level of refinement, the load represents one bogie, two bogies, two times two bogies or the full trainload. The moving load is applied with specified speeds and the developing deflections along the track are studied. At low speed, the deflections are "static", i.e. they directly represent the applied load. At increasing speed, the deflections increase and a dynamic part is added. At the critical speed, the deflections reach a maximum. With a further increase in speed the deflections decrease again. The development in dynamic behaviour is coupled to a wave field in the surroundings. The finite difference model used for the railway structure improved with the dry DMM is shown in Figure 10. The size of the model is 15 x 50 x 16 m^3 (length/width/depth) with 23,000 elements. Two element sizes were used, 0.4 x 0.4 x 0.5 m^3 and 1 x 1 x 1 m^3.

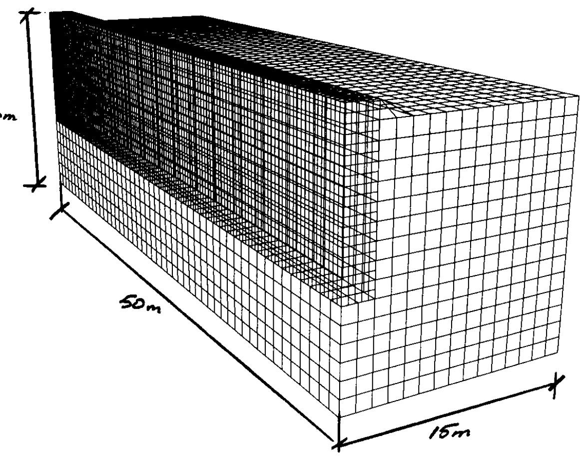

Fig. 10. FLAC3D model used in the analyses of dry DMM improvement.

4.4.4 FLAC3D results

The results of the FLAC3D analyses can be presented in many ways. Figure 10 shows the contour of vertical displacements on the surface of the model.. As shown in Figure 9, the model is symmetrical about the track centre. Figure 11 thus shows the vertical deflections on the ground surface on one side of the symmetry line/plane after a certain travelled path, here 35 m. The vertical displacement history at one location along the path (point 4) is shown in the figure. Analyses of the unreinforced track and the track improved with dry DMM (with the layout shown in Figure 13) at 55 m/s (200 km/h) train speed are shown.

The figures clearly show the difference in behaviour of the track. Unreinforced, the deflections in the track are maximum 7 mm downwards and 3 mm upwards. Accounting for multiple loads (successive double bogies in the full train) the deflections are estimated to increase to approximately 15 mm. With reinforcement, the deflections decrease to 3 and < 0.5 mm downwards and upwards respectively, and the total deflection of the full train is estimated to be approximately 4 mm. Bearing in mind that the calculations were carried out with very conservative values of stiffness of the improved soil, the calculated deflections were considered acceptable.

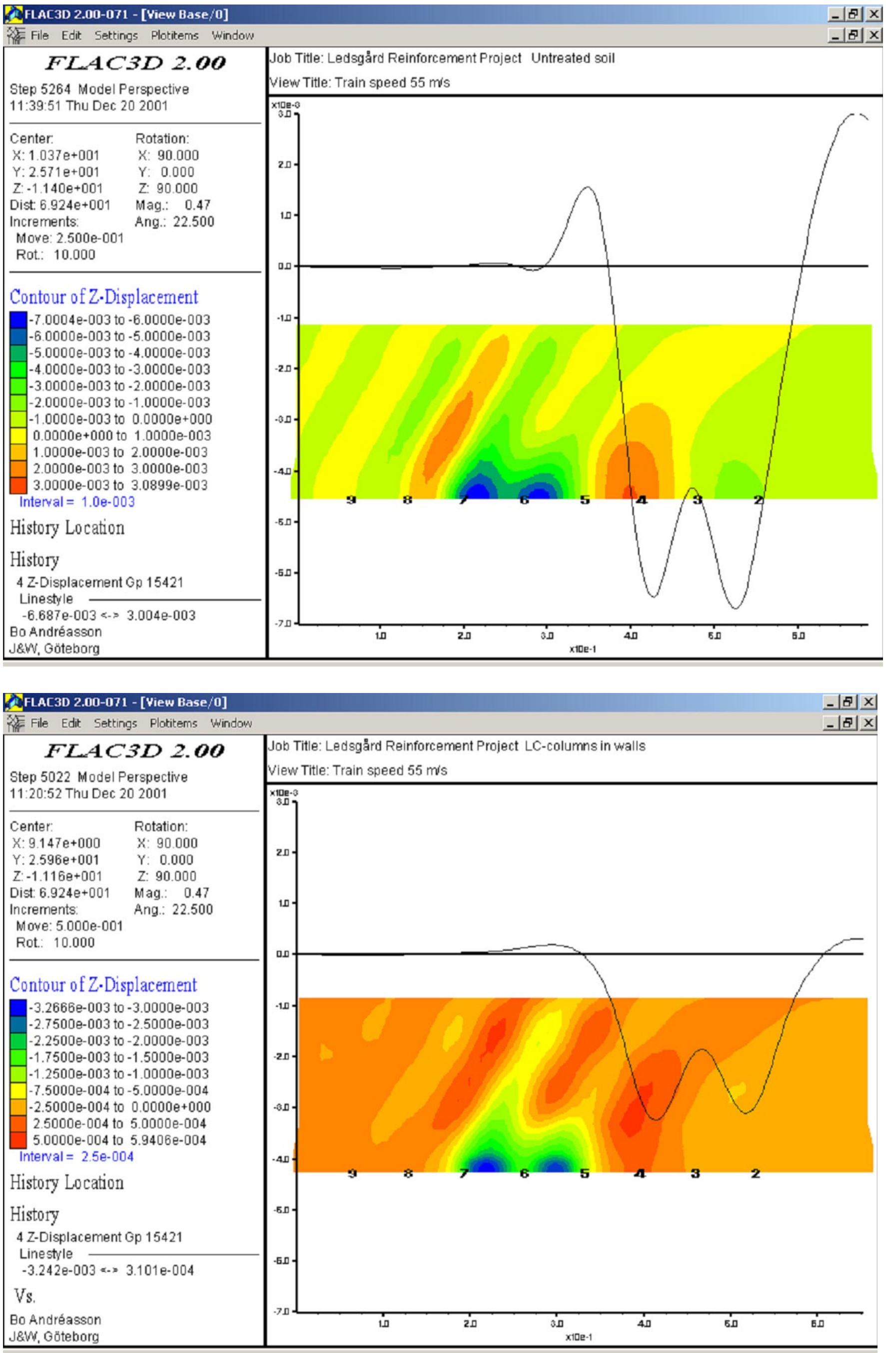

Fig. 11. Results of FLAC3D analyses of a two-bogie load travelling along the symmetry line at 55 m/s - unreinforced track above and dry DMM improved below. Deflection pattern on the surface shown together with deflection history at registration point 4.

From Figure 11, it can be seen that the vibrations at a distance from the track still show the same “high-speed”, plough-shaped pattern as before reinforcement. However, the amplitudes are considerably reduced. This is explained by the fact that, due to the limited surface wave velocity in the soil profile, the deflections produced by the track system cannot be transferred to the surroundings in pace with the train. This effect develops even for a stiff track structure, although the amplitude will be reduced.

A summary of similar calculations with various train speeds for the untreated and the reinforced track is presented in Figure 12. The figure clearly shows that for the untreated track the deflections start to increase when speed exceeds 40 m/s. This is the “high-speed” phenomenon and is in good correspondence with actual measurements at the site. With dry DMM columns in walls, the deflections are considerably reduced, in particular the heave. However, a high-speed phenomenon is still observed.

As mentioned above, the stiffness of the dry DMM columns was assessed in a very conservative way assuming only a tenfold increase in shear modulus relative to the stiffness in the untreated material. The actual measured deflections in the track after reinforcement, as described in Chapter 6, was only approximately 0.8 mm, with no visible increase with train speed. Updated FLAC3D analyses have produced good correspondence with the measured behaviour if the dry

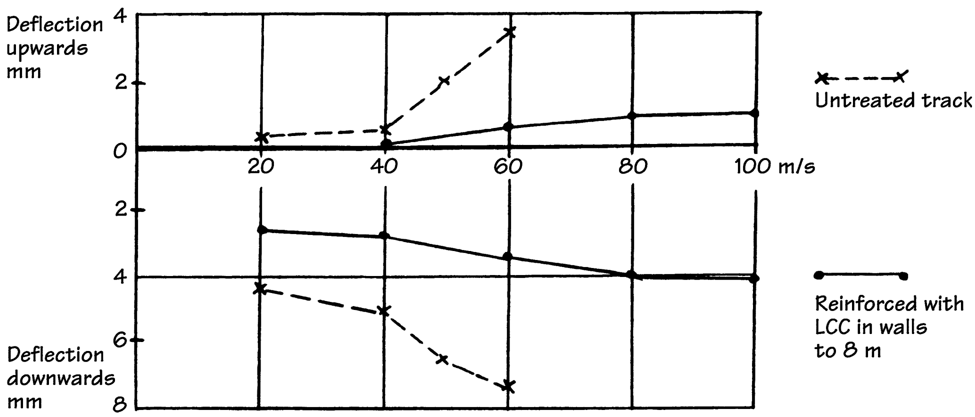

Fig. 12. Summary of FLAC3D analyses at Ledsgård. Vertical displacement, upwards and downwards, under the moving load of two bogies. Untreated track and track stabilised by columns in walls (LCC = Lime Cement Columns).

DMM column stiffness is increased ten times more i.e. to be in the order of 100 times the untreated stiffness values. Thus, a shear modulus of 150 MPa for the stabilised gyttja and 200 MPa for the stabilised clay, corresponding to a Young's modulus of 300 and 500 MPa, respectively, produce the best fit to the measured data.

4.5 LAYOUT OF IMPROVEMENT WITH DRY DM

The high-speed phenomenon is primarily caused by the soft gyttja pocket. As described in Chapter 3, the gyttja pocket starts in approximately Section 24+150 and extends down to the river, see Figure 5. Due to observed settlements all the way back to Section 24+000, improvement was carried out from Section 24+000 to 24+372, where the existing dry DMM improvement commences.

The first 150 m, from Section 24+000, is stabilised by dry DMM in a single pattern, see Figure 12, which shows the column layout in the area around Section 24+150. From Section 24+150 down to the existing lime/cement columns close to the bridge, the columns are installed in a ladder-type configuration with two longitudinal walls extending (in principle) to 7 m and transverse walls extending to 6 m depth below the rail. To accommodate the existing reinforcement at the bridge and to take settlements into account every second column in the longitudinal walls extends to 13 m depth, as indicated by the figures within the "column" circles in Figure 13.

The demand for proper transmission to the stabilised soil required a good connection between the columns and the railway structure. This was achieved by installing the columns from a "working platform" to be removed after completed installation to expose the stiff column heads.

As mentioned above, the amount of binder was totally 150 kg/m^3 with the components unslaked lime and cement in a ratio of 25/75 in the "ladder" part from 24+150 on and 120 kg/m^3 with a binder ratio of 50/50 in the single column part.

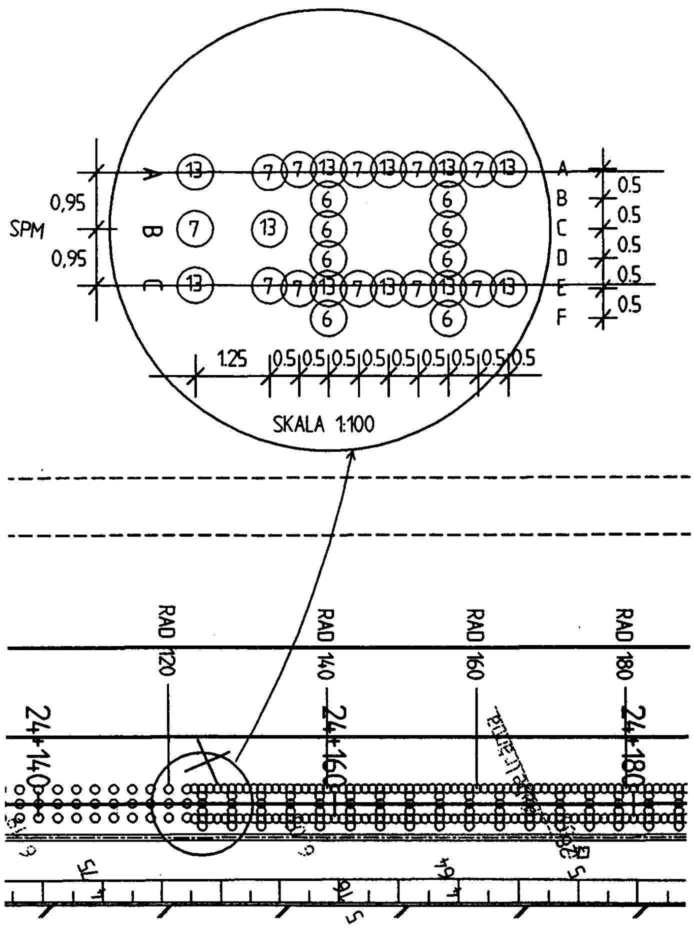

Fig. 13. Layout of soil improvement with the dry DMM columns (lime/cement columns). Also showing the transition at 24+150 from "settlement" stabilisation (from 24+100 to 24+150) and "high-speed" stabilisation. Column length below rail level given by figures within circles.

5. Execution

5.1 INTRODUCTION

In view of the widespread usage of dry deep mixing for soil improvement (DMM = Deep Mixing Method) and the reduction of permanent deformations in soft soils in Sweden, it is a natural development to use the technology in vibration reduction purposes where possible.

Dry deep mixing is often used to improve the soil beneath railway structures and thereby increase the stiffness of the soil-railway structure system. The principle is shown in Figure 14. Another important effect is that the improved soil increases the critical speed for the system so that extreme vibration levels due to train passages at this speed will be avoided. The dry deep mixing method (DMM) has been described with respect to e.g. applications and execution in Bredenberg, Holm, Broms (1999).

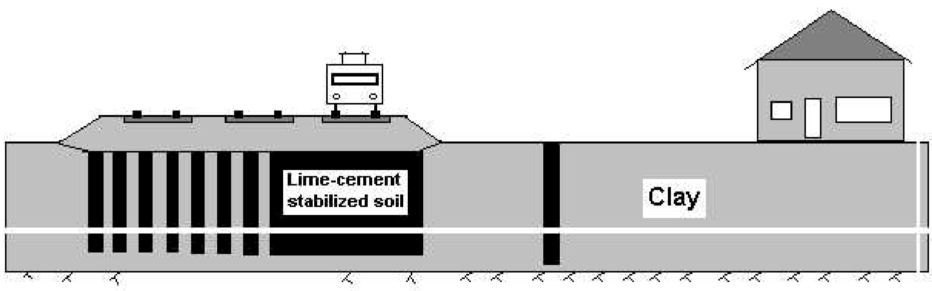

Fig. 14. Principle of improvement of the soil beneath the track with the dry deep mixing method.

5.2 THE DRY DEEP MIXING METHOD

The stabilisation of soft clay, organic soil, loose sand and silt by the addition of dry binders is a well-known technique in the Nordic countries. The dry deep mixing method has been used frequently during the last three decades for soil improvement of road and railway structures. It is also used as a foundation for buildings such as shopping malls, oil tanks and industrial buildings. The volume of soil stabilised in Sweden and Finland during recent years has been approximately 600,000 m^3/year, corresponding to approximately 2 million metres of columns.

Mixing dry binder with soft soil results in an increase in strength and stiffness. These improvements are immediate but are also followed by a long-term increase.

A mixing tool, mounted on a leader, is rotated down into the ground to the required depth. During the slow withdrawal of the mixing tool, a dry mixture of binders is injected into the soil, thus creating an improved soil column. The columns interact with the non-stabilised surrounding soil, resulting in an improved composite soil volume.

5.3 METHOD DESCRIPTION

5.3.1 Purpose of soil improvement

At the Ledsgård site, deep mixing was performed to increase the stiffness of the soft soil layers. By installing the columns in an appropriate configuration and assuring a good connection between the columns and the railway structure, an effective soil improvement system was to be achieved.

5.3.2 Plant and equipment

The equipment used for installation consisted of an installer, a carrier and a materials supply station. The installer was a crawler-based rig, with a rotary engine mounted on a leader. A purpose-designed mixing tool was fitted to the leader and the binder was distributed into the ground by compressed air. The machine generates a very low ground pressure and the working range is up to 7 meters. The machine can reach ± 3 m in height. The total weight of the installer was 48 tons, generating a ground pressure of approximately 35 kPa. The Limix rig for dry deep mixing soil improvement is shown in Figure 15.

Fig. 15. Limix rig for soil improvement with the dry deep mix method.

The binder is fed from the carrier to the installation machine. The carrier consists of two silos, one with cement and one with lime. A load cell system is placed under each silo and supplies the on-board computer with information on the amount of material fed each second through the system from each silo. The two silos have a maximum capacity of 12 tons each. When the silos are empty, the carrier is refilled at the materials supply station. The total weight of the carrier is 30 tons (54 tons when filled with binder, generating a ground pressure of 55 kPa).

The supply station consists of two bulk tankers, each with a capacity of about 40 m^3. The Hercules Limix system is based on a completely computer controlled installation process. Initially, site-specific installation parameters, such as rotational speed, withdrawal speed and binder amount, were loaded into the onboard PC. The operator enters the column number, adjusts the mixing tool into position and drills down to the required depth. From there onwards, the computer controls the installation process. The rotation speed and the withdrawal speed of the mixing tool are governed by the rate at which the binder leaves the silos. Due to the frequent variations in distributed binder, the withdrawal speed is constantly re-calculated and varies throughout the column. This ensures that the binder is correctly distributed at each level in the soil. The operator overviews the process on his screen and can abort it if necessary. All relevant data for each column are stored for quality control.

One advantage of a computer controlled system is that each column is manufactured in the same way, regardless of the operator's skill or attentiveness.

5.3.3 Procedure

The soil improvement work using dry deep mixing at Ledsgård consisted of upgrading 370 m of the western track.

The following principal procedure was followed for the Ledsgård site:

1. Disassembly of tracks and removal of cable groover
2. Excavation of top ballast and disposal to depot
3. Excavation of sub-ballast to 800 mm below base of rail and disposal to depot
4. Setting out of dry DMM columns
5. Installation of dry DMM columns
6. Removal of non-mixed soil above columns and exposure of the top of the columns
7. Back filling and compaction of ballast

The time schedule for the total project was approximately three months (starting in May 2000), where the deep mixing occupied two weeks (excluding trial columns).

Since the binders used to form the columns are soil-dependent, each individual site has to be evaluated separately. The first step consisted of mixing trials in the laboratory on soil samples from the site, see Chapter 4. These trials provide the initial design parameters. Trial columns were then installed on site in order to:

- Verify the design parameters
- Optimise the installation parameters
- Detect possible layers/areas with poor stabilisation effect.

The trial columns were installed within the construction area under conditions similar to those during construction. A total of 12 trial columns were installed with a length of approximately 8 metres.

The final binder recipe was established. Soil improvement was executed with two different patterns, grid pattern and single columns according to Chapter 4. The following summarises a number of parameters:

- Column diameter 600 mm
- Column length 6, 7 or 13 m (average length 7.6 m)
- Total length 13,000 m
- Binder quantity 120 and 150 kg/m^3
- Binder type: unslaked lime and standard Portland cement with 50/50 and 25/75 blend
- Area coverage: about 54 % in the grid part and about 21 % in the single column part
- Column overlap for wall pattern: 0.1 m
- Installation capacity per 16 hours: 1,000 m/rig
- Selling price: 23 •/m^3 of improved column, excluding tests

5.3.4 Control of column quality

Field tests were performed on six of the trial columns in order to verify their homogeneity and quality. The tests were carried out with the Standard Column Penetration Test (SCPT) and Reversed Column Penetration Test (RCPT), see Figures 16 and 17.

In the SCPT, a steel probe fitted with vanes is pressed down into the trial column. The vanes span the main part of the column diameter, thus giving a good estimate of the column strength. A pre-drilled hole is made in the centre of the column before the SCPT. The measured tip force (in kN) is plotted against depth (m). The homogeneity and shear strength of the column are then evaluated.

An alternative when installing long columns is to use the reversed column penetration test (RCPT). A preinstalled steel probe fitted with vanes is pulled through the finished column, from the bottom of the column to the surface. The measured force (in kN) is plotted against depth (m). The homogeneity and shear strength of the column can then be evaluated.

Three of the twelve trial columns were provided with a reversed sounding probe for later pull tests. Six of the columns were tested – the three with pre-

Fig. 16.
SCPT for quality control of dry Deep Mix Method columns.

Fig. 17.
RCPT for quality control of dry Deep Mix Method columns.

installed probes (RCPT) and three by pre-drilled standard column penetration test (SCPT). Only the lat-ter three tests were considered reliable.

The tests were carried out 5 and 14 days after installation. The penetration resistance of the 20 x 400 mm^2 penetration wing (measured at the point) in one test (SCPT) carried out after 14 days is shown in Figure 18. The trial columns were installed to 8 m depth, which was verified by the tests, as can be clearly seen in the figure. The gyttja layer extends to approximately 4 m depth.

Because of the probe geometry (20 x 400 mm^2 and pre-boring of a 55 mm diameter hole) the penetration resistance from Figure 18 can be converted into an undrained shear strength. The evaluated undrained shear strength from the three reliable SCPT tests and the two RCPT tests is presented in Table 1.

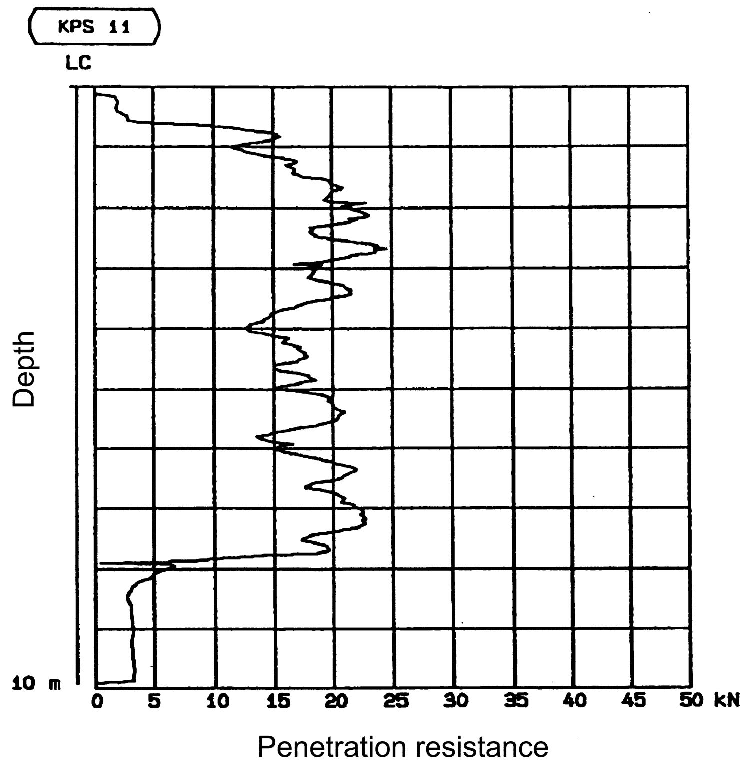

Fig. 18. Result of field test on a dry DMM column. Penetration resistance of prebored standard column penetration test (SCPT).

Table 1. Results of field tests on dry DMM columns.

Column No.	Method	Time [days]	Shear strength [kPa]
2	RCPT	5	(110 to 460)
3	SCPT	5	120 to 180
4	RCPT	5	(110 to 460)
5	SCPT	14	180 to 260
11	SCPT	14	150 to 200

As judged from Figure 18 and Table 1, the effect of the improvement is quite good. The undrained shear strength is in the order of 150 - 250 kPa two weeks after installation. Figure 17 also shows that the effect is as good in the gyttja as in the underlying clay.

A comparison of the shear strengths evaluated from the field tests, in Table 1, with the tests of the laboratory mixed samples in Figure 9 shows that the laboratory values are somewhat higher. Using "average" values, the field test shear strengths are in the order of two-thirds of the corresponding laboratory tests. In the design of countermeasures, the stiffness of the material is of prime interest and such data cannot be deduced from the field column penetration tests.

5.4 PRACTICAL ASPECTS

The dry deep mixing was carried out over a period of two weeks in summer 2000 by the Swedish contractor Hercules Grundläggning. Two photos from the works are shown in Figure 19 and Figure 20. The works were carried out according to plans. Execution of the soil improvement was a routine type of job. The crucial part lies in the dynamic design of the railway structure as presented in Chapter 4. However, it is important to achieve an accurate overlap of the columns as well as relatively homogenous columns versus depth so that the requirements on static and dynamic stiffness are fulfilled. A relatively stiff drilling rod, an accurate inclinometer on the leader and precise positioning of the mixing tool make it easier to obtain column overlap.

Driving rain and gusts during installation of the trial columns reduced neither the installation capacity nor the quality of the columns. However, it is important to have efficient equipment for dehumidification of the compressed air so that the binder remains dry until reacting with the water in the soil.

Fig. 19.
The dry DMM column installation works with a commuter train passing on the neighbouring track.

Fig. 20.
Limix rig for dry DMM soil improvement.

Substantial heave was observed during column installation, which lifted the neighbouring track to some extent and necessitated its re-levelling. The columns were installed from a working bed (sub-ballast material), which was later removed to expose the tops of the columns. Had the stiff, treated soil not been reached with this procedure, the plan was to dig a trench to ensure good contact with the treated soil.

5.5 TIME SCHEDULE AND COST

The time schedule for the total works was approximately three months starting in May 2000. The deep mixing with lime-cement columns took two weeks (excluding test columns). Traffic was interrupted between July 9 and July 31, 2000 on the one track, where the track was removed and subsoil improvement performed.

The total cost was 5.1 million SEK (about 0.56 million •). Figure 21 shows the cost distribution, which indicates that the cost for the dry DMM soil improvement itself (lime/cement columns) is only a minor part of the total cost.

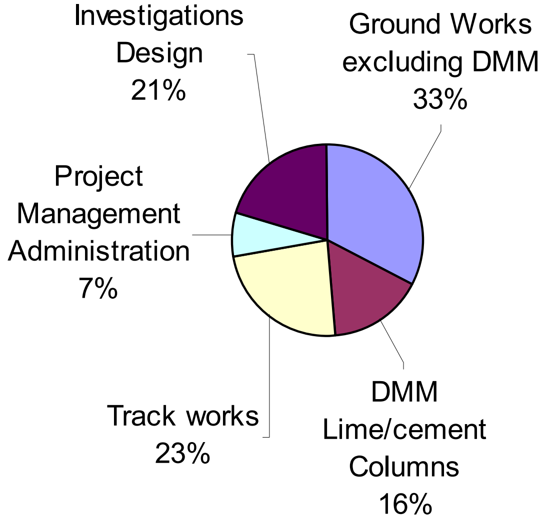

Fig. 21. Distribution of cost of countermeasure.

6. Measurement of vibrations

6.1 GENERAL

Extensive measurements have been performed since the excessive vibration levels were observed in spring 1997. Comprehensive tests and investigations were thus carried out in 1997 and 1998.

To investigate the effect of the countermeasures with dry DMM improvement (lime-cement columns) additional measurements during the passage of high speed traffic were carried out before and after the measures. Furthermore, measurements were carried out in September and October 2000 and a number of measurements were performed in March 2001 to study the increase in the effect of the soil improvement with time.

The prime aspects of the measurements described in this chapter are track/trackbed vibrations and vibrations in the surroundings before and after the countermeasures. Some information on the influence of the countermeasures on the western track on the behaviour of the eastern track under high speed traffic is also presented

6.2 MEASUREMENTS PERFORMED

As mentioned above, extensive measurements have been carried out at the Ledsgård site since the high-speed phenomenon (rapidly increasing vibration levels in the track with increasing train speed) was first observed in spring 1997. To verify the results of the countermeasures carried out in July 2000, vibrations in the track and surroundings have been measured before the measures (May 2000) and after the measures (December 2000).

Four parties have been involved in the measurements:

– Banverket (Swedish National Rail Administration), Borlänge
– Chalmers University of Technology, (CTH), Gothenburg
– Jacobsson & Widmark AB, (J&W), Gothenburg
– Royal Institute of Technology, (KTH), Stockholm

Thorough presentations of the measurements of the respective party are presented in separate reports, Banverket (1998, 1999, 2001), CTH (2001), J&W (2001), KTH (2001) and a general report J&W (2002). The detailed results are presented in these reports and are summarised here.

The excitation sources employed have been:

- X2 trains travelling at specified speeds
- regular trains, including, X2, commuter and freight trains
  – the Banverket Track Loading Vehicle (TLV) imposing different types of load on the track/rails.

Measurements in October 1997

In October 1997, measurements were performed to investigate the principles behind the high-speed phenomenon. A chartered X2 train passed over the test section at different speeds of up to 200 km/h and in different directions (southbound and northbound). Since one important aspect was to determine whether there were any evidence of soil degradation, measurements of displacements (vibrations), pore pressures were also made. Figure 22 shows the measured vertical displacement of the track for an X2 train passing at different speeds. From the figure, it can be seen that at low speeds, far below the critical speed, the displacement pattern is static. At high speeds the pattern of displacement is changed. At high speeds, substantial heave is observed, in addition to attenuating performance when the last axle has passed. The total displacement is approximately tripled at high speed compared to low speed.

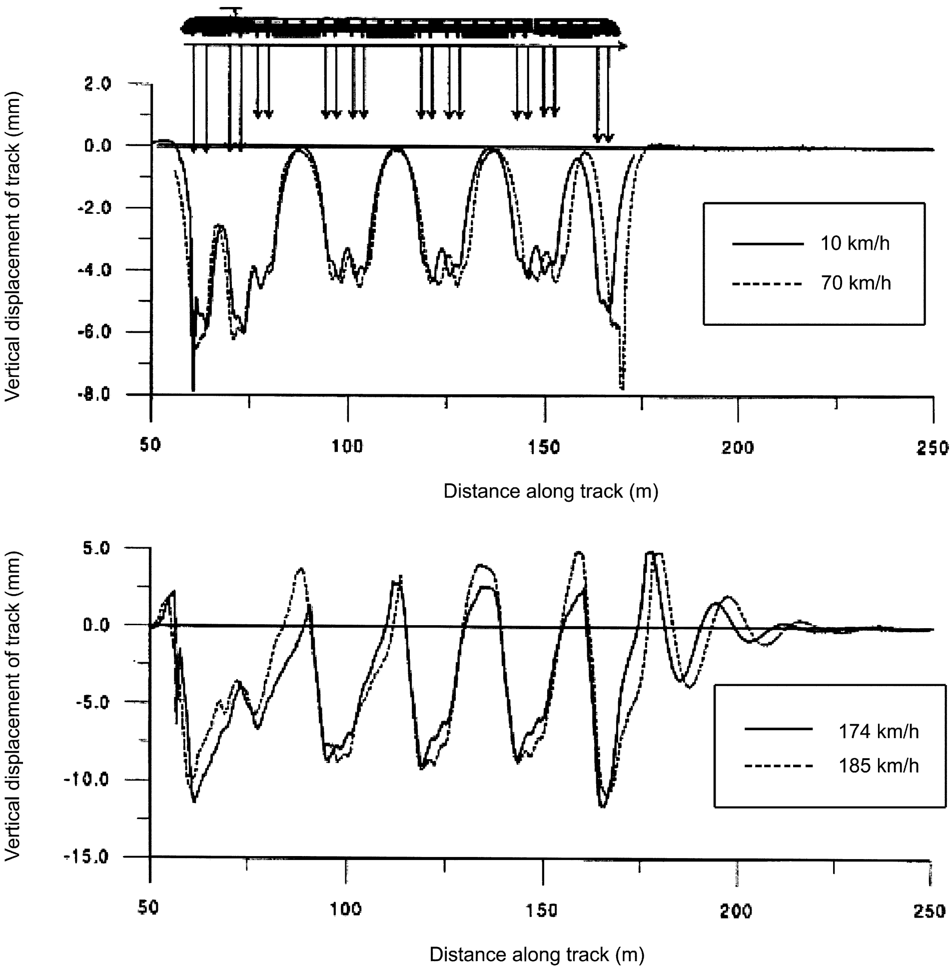

Fig. 22. **Measured vertical displacement at different speeds at Ledsgård.**

Measurements in May 2000

In May 2000, all the above parties were involved in measuring the behaviour of the untreated tracks and their surroundings before the countermeasures. The vibration source was primarily a chartered X2 train passing at specified speeds of up to 190 km/h, as well as regular trains. The Banverket Track Loading Vehicle (TLV) was used to apply specific cyclic/dynamic loads to the track.

Movements in the track and trackbed were measured, Banverket (2001), KTH (2001) and J&W (2001). Measurements were performed on both tracks - primarily of vertical displacements but also of lateral displacements along the track.

Vibrations in the surroundings were also measured, CTH (2001) and KTH (2001). The sensor arrays were relatively extensive. CTH used 42 accelerometers in two separate paths, one perpendicular and the other parallel to the track, at the ground surface and at some points at depths of 2 m and 6 m. Maximum distance from the track was 34 m. KTH used accelerometers as well as geophones in three lines perpendicular to the track at 7.5 m intervals and with a maximum distance of 30 m from the track. KTH geophones and accelerometers were also installed in the track.

Measurements in September 2000 and October 2000

In September 2000, i.e. five weeks after completed dry DMM improvement, a limited series of measurements was performed on the vertical displacements in the trackbed shortly after increasing the train speed to 160 km/h, J&W (2001). In October 2000, measurements of vibrations in the surroundings under regular train traffic were carried out, KTH (2001). One line of geophones/accelerometers extending 30 m from the track was employed.

Measurements in December 2000.

In December 2000, an extensive measurement program was again carried out to finally study the effect of the countermeasures. The vibration source was primarily a chartered X2 train travelling at a specified speed. The measurements were performed on both tracks and regular trains were also measured.

Movements in the track and trackbed, on both tracks, were measured by Banverket (2001) and J&W (2001, whereas vibrations in the surroundings were measured by KTH (2001) using a single line geophone/accelerometer array extending 30 m from the tracks.

6.3 DEFLECTIONS IN THE TRACKS

Vibration measurements in the tracks were carried out on both tracks before and at different times after the countermeasures. Measurements were performed on rails and sleepers as well as on deflections in the trackbed at sleeper ends. Vertical deflections were the prime object. The excitation source was primarily X2 trains and the Track Loading Vehicle (TLV).

A summary of measured deflections in the trackbed on both tracks before and after the countermeasures is presented in Figure 23 (traffic on the respective/ measured track). The figure shows peak-to-peak deflections in the middle part of the X2 train travelling at various speeds from May (before) and December (after) measurements. "West" indicates the track where dry DMM improvement (lime/cement columns) were performed, whereas the "East" track is not reinforced.

The results of the countermeasures are very clear in regard to the dry DMM improved track. The vertical deflections at low speed have been reduced from 4 mm to approximately 0.8 mm (practically constant deflections with speed after countermeasures), i.e. a reduction by a factor of 5. At higher train speeds, the reduction is even more pronounced, since the dynamic amplification of the

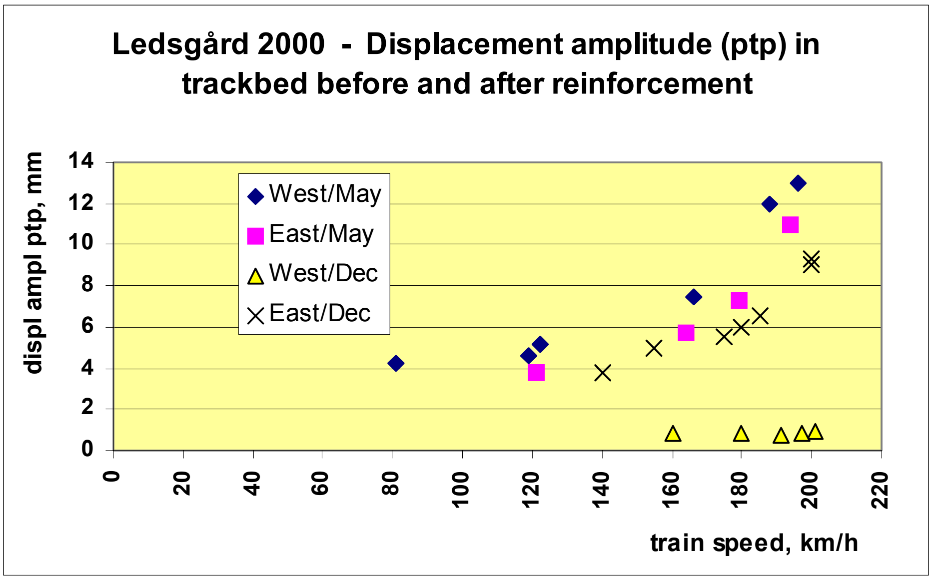

Fig. 23. Summary of measured displacements in the trackbed before (May) and after (December) reinforcement of the western track. Central part of X2 trains, J&W (2001).

trackbed movements has disappeared, resulting in a 15 to 20-fold reduction in vertical deflection amplitude. The vertical deflections in the eastern track were initially somewhat lower (about 15 %) than in the western track and have decreased further (15 – 20 %) as a consequence of the countermeasures in the western track. Thus, the vertical deflections in the untreated track after countermeasures in the western track are in the order of two-thirds of those of the untreated western track before the countermeasures.

A comparison of early measurements of the reinforced track six weeks (September 2000) after dry DMM column installation with the December 2000 measurements (4.5 months after installation of columns) indicates that the vertical deflections decrease with time. The measured vertical deflection amplitude (peak-to-peak in the middle part of the train) was 1.0 mm in September 2000 and decreased to 0.8 mm in December 2000.

The ratio between the lateral displacement amplitude along the track and the corresponding vertical deflections was essentially constant with train speed, 0.2 – 0.3, J&W (2001). The measurements were carried out on both tracks only before the countermeasures.

Measurements with the Track Loading Vehicle, Banverket (2001), showed that the track receptance at 1 Hz was reduced by a factor of 3.4 and 1.2 for the western and the eastern track, respectively, due to the countermeasures in the western track. Figure 24 shows the track receptances of the western track before and after soil improvement. The excitation and response were measured on the rail head mainly by means of accelerometers. The receptance was extremely high before soil improvement. There was a resonance of 2 – 4 Hz. The track secant stiffness showed similar results with an increase corresponding to a factor of 3.3 for the western track and a factor of 1.1 for the eastern track.

The displacement of one sleeper end during train passages, before DMM improvement, is shown in Figure 25. The passage of each wheel is indicated. At the low speed train passage (80 km/h), a normal, although high, displacement pattern arose. Maximum displacement occurred when a wheel passed the sleeper. For the high-speed train passage (192 km/h), extremely high displacement occurred with a different pattern. The maximum displacement occurred after the bogie passages. After the improvement with dry DMM, the displacement pattern was normal for all velocities tested (max 200 km/h).

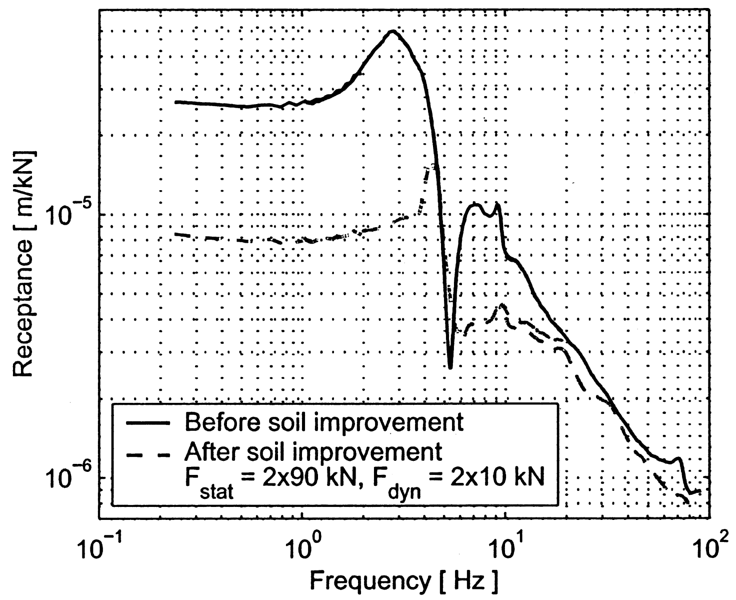

Fig. 24. Track receptance of the western track before and after dry DMM improvement.

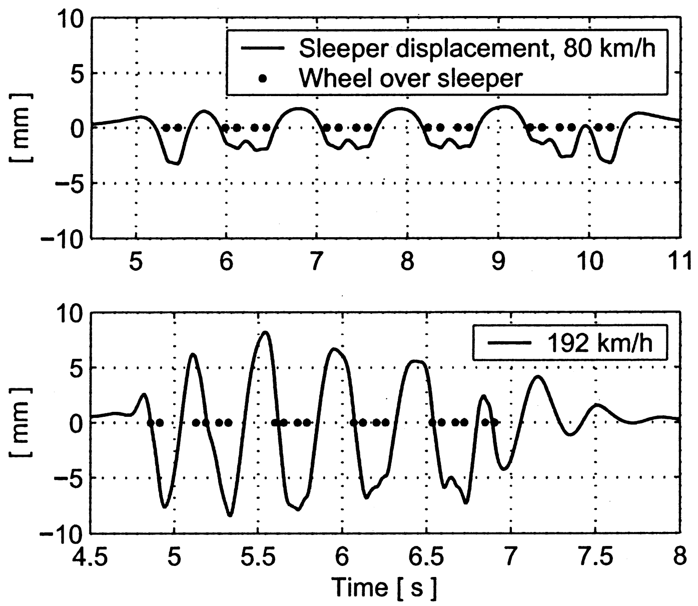

Fig. 25. Displacement of one sleeper end during train passage at speeds of 80 and 192 km/h.

6.4 VIBRATIONS ALONG TRACK SECTION

When designing the countermeasures the extent of the gyttja pocket was an important boundary for the different layout of the dry DMM soil improvement. To study the varying vibration conditions along the reinforced track section, vibration measurements were carried out in three sections along the track using geophones placed 3 m from the western track, J&W (2001). Measurements were performed in Section 24+265 (the test section where essentially all measurements were carried out), section 24+175 (also with dry DMM wall stabilisation) and section 24+075 (with dry DMM columns in single pattern). Figure 26 shows the measured vertical particle velocity amplitude in the middle of the X2 train versus train speed before and after countermeasures.

From Figure 26, it is clear that the high-speed phenomenon exists in the two southern sections and is shown by a sharp increase in vibration level with increasing speed. In the northernmost section, however, there is no trace of such behaviour. This was expected, since the soft gyttja pocket does not extend to this section.

After the dry DMM soil improvement, the vibration amplitude decreases to approximately the same low level in all three sections. When comparing the results shown in Figure 22 with the vertical displacements amplitudes in Figure 23, it should be borne in mind that for constant displacement amplitude with train speed (static type deflection) the particle velocity will increase linearly with the train speed. A deviation from this trend is thus an indication of a developing high-speed phenomenon.

In spring 2001, Banverket carried out "Rolling measurements" with the TLV, exposing the track to a cyclic load of 1.4 and 6 Hz while passing the area at 5 and 20 km/h, respectively. This was a test run of this novel procedure, but showed very promising results. For further information, see Banverket (2001).

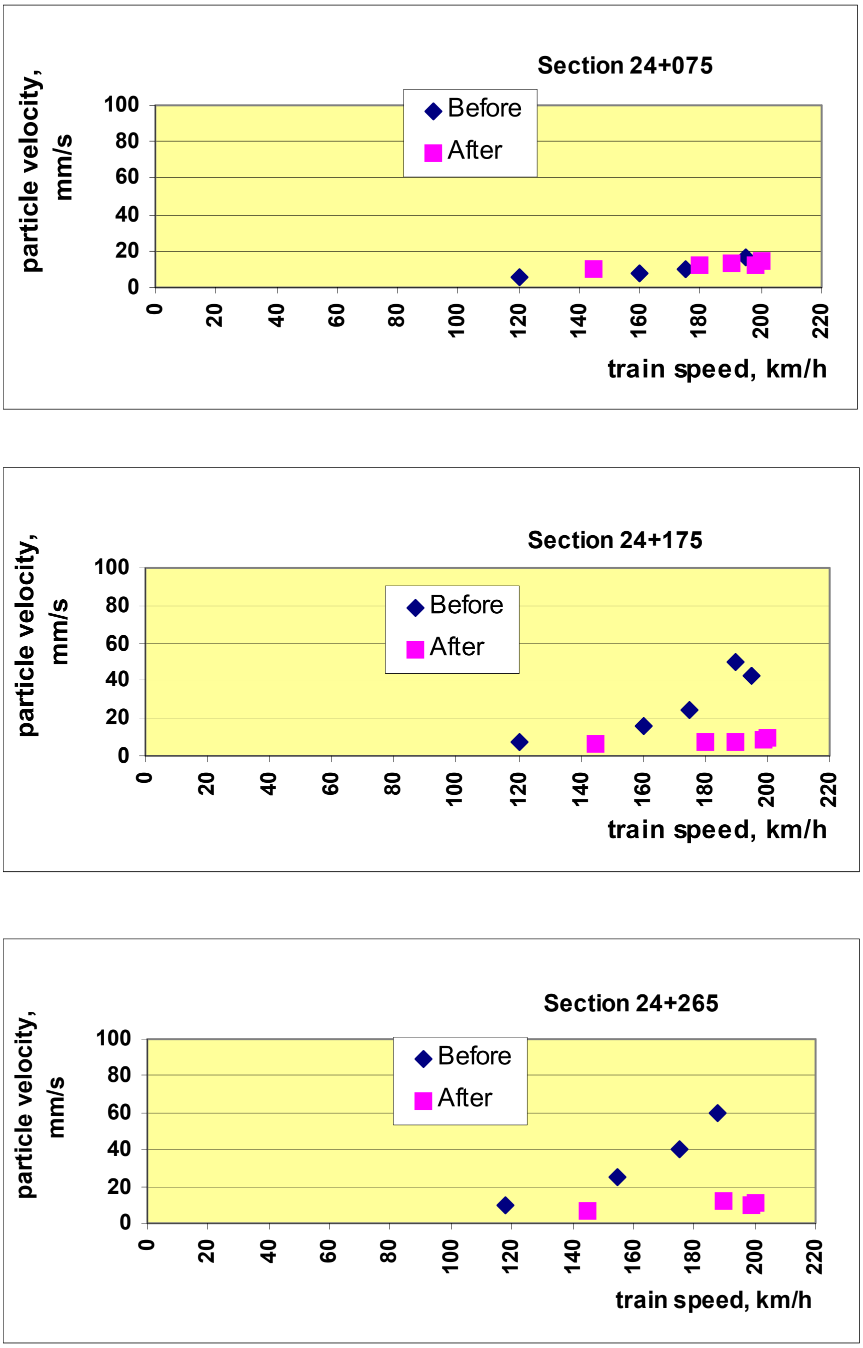

Fig. 26. Particle velocity vs train speed in Sections 24+075, 24+175 and 24+265 before (May 2000) and after (December 2000) dry DMM improvement of the railway structure, J&W (2001).

6.5 VIBRATIONS IN THE SURROUNDINGS

Vibrations in the surroundings were measured, CTH (2001) and KTH (2001). The measurements are included in extensive research projects at the respective institution and together with the results they will contribute to future doctoral theses.

The key results of the measurements are reproduced in Figure 27. The figure shows calculated vertical particle displacement amplitudes (double integration of acceleration records) at different distances from the western track versus train speed before and after the countermeasures.

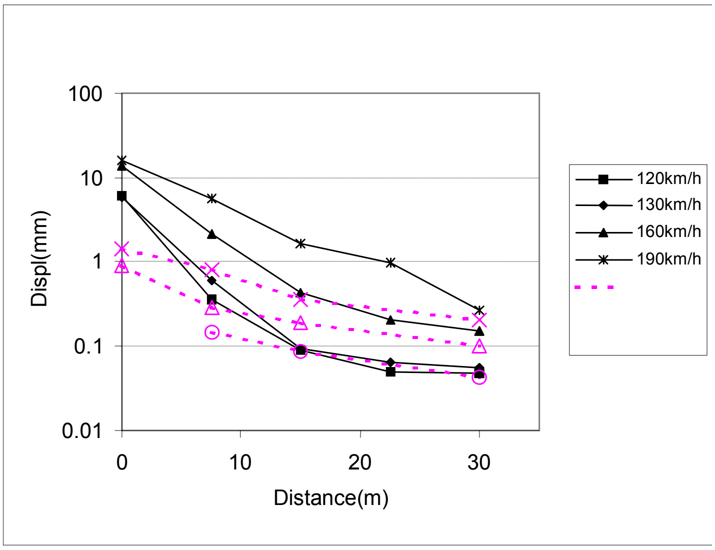

Fig. 27. Attenuation of particle displacement (peak-to-peak) with distance from the mid-point of the western track at different train speeds before (solid lines) and after (dashed lines) dry DMM soil improvement, KTH (2001).

From Figure 27, a number of observations can be made:

- There is a large increase in vertical displacement amplitude with increasing speed for an untreated track. The increase between train speeds 130 and 190 km/h is in the order of 10 – 20.

- At large distances from the track, 22.5 – 30.0 m, a remarkable (unexpected) reduction in amplitude is observed at maximum speed before reinforcement.

- At high speeds (190 km/h), the reduction of displacement amplitude level due to the countermeasure is very clear (a factor of approximately 5), with the exception of the 30 m recording.

The extensive measurements carried out by CTH before the countermeasures showed similar results, with a remarkable reduction in displacement amplitude with larger distance at maximum speed (190 km/h), see Figure 28.

The sudden drop in vibration magnitude between 25 m and 35 m distance at 190 km/h train speed, observed only before the countermeasures, is mysterious and should be given further attention. One explanation is a possible variation in the ground in terms of the gyttja pocket. The soil conditions in the studied section may not be homogeneous over the entire measured distance.

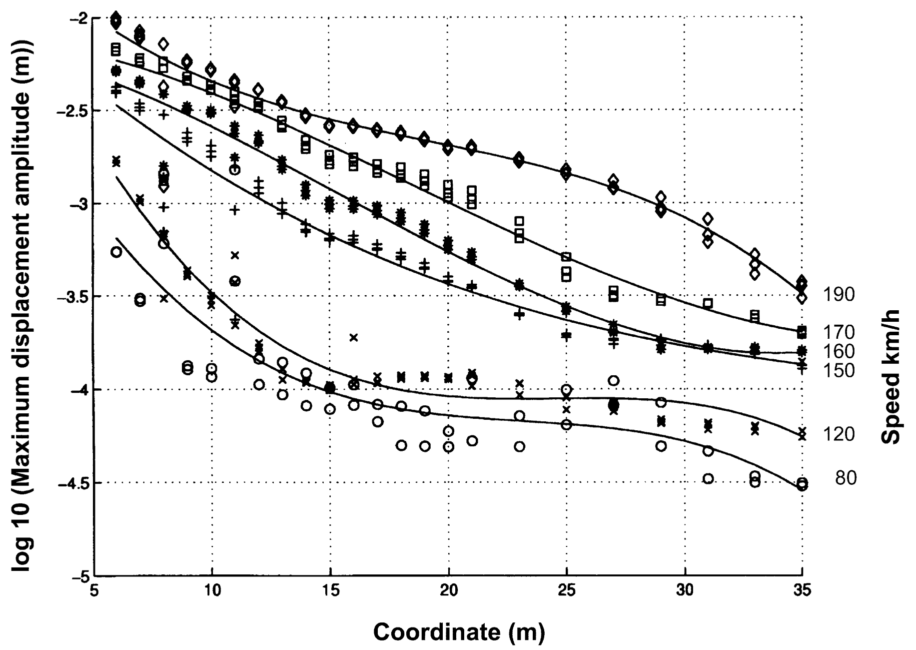

Fig. 28. Maximum displacement amplitudes with distance from track and train speed before countermeasures, CTH (2001).

6.6 BARRIER EFFECT

The barrier effect of the installed dry DMM columns was studied, KTH (2001). The columns/walls are considered to act as a barrier reducing the vibrations from traffic on the unreinforced track behind the barrier such that the vibration environment is improved. Figure 29 shows this beneficial effect clearly. The large vertical displacement amplitudes recorded before dry DMM column installation are almost eliminated.

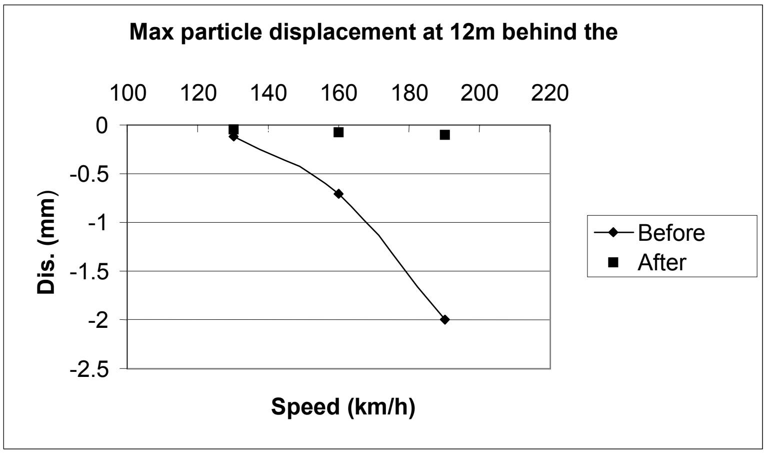

Fig. 29. Comparison of ground vibrations before and after installation of the dry DMM barrier with train speed, KTH (2001). Train traffic on the unreinforced western track.

6.7 CATENARY POLE DISPLACEMENTS

An important consequence of the excessive vibration levels is the behaviour of the catenary poles. Measurements of the displacements of the catenary poles have been performed before and after the countermeasures.

Measurements of acceleration in three directions were performed at the catenary poles close to the test section, Banverket (2001). The displacements before the countermeasures were clearly dependent on train speed, with a maximum peak-to-peak vertical displacement of approximately 10 mm for a train with a speed of 200 km/h on the nearby western track. The corresponding values in the horizontal direction were 4 and 2 mm in the perpendicular direction and along the track, respectively. After dry DMM soil improvement of the track, the corresponding displacements were approximately 1 mm vertically and 0.5 mm laterally.

Traffic on the east (middle) track before the countermeasures also caused considerable displacements in the nearby catenary poles. These displacements seem also to have been effectively reduced (limited data available).

7. Conclusions

The extensive measurements at Ledsgård carried out before and after the countermeasures against the high-speed phenomenon have shown the good effect in reducing the vibrations and have provided good insight into the effect of the actual measures. The results can be summarised as follows:

- The dry DMM soil improvement has eliminated the vibration problem in the track/trackbed by creating a stiff ground below the ballasted track structure.
- The efficiency effectiveness of this countermeasure is very good, with a reduction of the displacements in the track/trackbed by a factor of approximately 5 at low speed and 15 at high speed (200 km/h)
- The trains were able to operate on the adjacent track while the dry DMM soil improvement program was executed.
- The level of vibrations in the vicinity of the track has also been reduced, especially in the immediate proximity. The efficiency Effectiveness in reducing the vibrations at greater distance requires further studies.
- The walls of dry DMM columns are effective as a barrier reducing the vibrations behind the wall. The effect decreases with distance from the barrier.
- The high speed phenomenon at the Ledsgård site is geometrically confined to a limited length of track due to the local extent of the soft gyttja pocket, which is the cause of the problem.
- A dry binder consisting of unslaked lime and cement has proved to be effective in the very soft organic soil (gyttja).
- Dry DMM column installation over the improved length (150 m) was performed within two weeks.

- From these measurements, it is not possible to state whether the configuration used is optimal. However, the extra cost for a certain over-design is relatively limited in the present case. The cost of the DMM works constituted only a minor part of the total costs.

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