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Experimental Investigation of a Ballastless Asphalt Track Mockup under Vertical Loads

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This study presents experimental results from a laboratory investigation into the mechanical behaviour of a ballastless asphalt track under vertical loads and isothermal conditions. A full-scale test section was constructed inside a steel box, consisting of three wide-base sleepers resting on an asphalt layer that was underlain by an unbound granular layer (UGL) supported on a rubber mat (representing subbase and subgrade). Sensors were installed to measure diverse responses, consisting of vertical stresses at the bottom of the UGL, horizontal strains at the bottom of the asphalt layer, relative vertical displacements between various track components, and vertical surface accelerations. Sleepers were loaded directly on top of the rail pads by using servo-hydraulic actuators. Cyclic loads were applied to investigate the effects of different excitation amplitudes and frequencies. It was found that all measured responses displayed a strong frequency dependence. Vertical stresses below the UGL varied linearly with the load amplitude, while other responses showed a non-linear relationship. Train passages with a maximum speed of 200 km/h and axle loads up to 200 kN were simulated by sequentially loading the three sleepers. From this load type, it was found that ballastless asphalt track exhibited time-dependent behaviour such as delayed recovery of strains in-between axle passes. Furthermore, the majority of the vertical actuator displacement was absorbed by rail pad compression. Lastly, measured stresses and strains were of very low magnitudes, suggesting marginal long term mechanical damage under service loads for such a ballastless asphalt track structure.

Keywords: Asphalt pavement; Ballastless asphalt track; Cyclic loading; Full-scale laboratory testing; Railroad testing; Simulated train passage; Track vibration.

1. Introduction

There are increasing demands for faster and heavier trains, and subsequently, the expansion in railway line capacity because of the rise in passenger and freight railway traffic volume. Traditional ballasted tracks are reaching their performance requirements, owing to ballast breakage and deterioration and, in turn, frequent maintenance activities [1, 2]. Consequently, ballastless tracks have emerged, which are primarily made of Portland cement concrete (i.e., slab tracks). Slab tracks are associated with high initial costs, amplified noise and vibration levels, and difficulties in accommodating maintenance activities even though they provide improved support conditions [1, 3]. Recently, an alternative ballastless track-type based on asphalt concrete has attracted research and industry attention, as it could combine the
advantages of both ballasted and slab tracks [4]. Asphalt is a well-researched material that is traditionally used for construction purposes within the transport infrastructure, but its application in the rail sector has so far been limited.

The current practice of using asphalt in ballasted tracks is primarily as a subballast layer [5-7], while in ballastless tracks, it is used as a support for the concrete slab tracks and for waterproofing purposes [8-12]. Only a few occasions were identified, mainly in the German railway industry, wherein ballastless tracks were built with asphalt as the uppermost support layer (also called asphalt overlayment) [13-15]. Such tracks were mainly constructed in tunnels, with standard highway asphalt and specially designed sleepers, which were wider, heavier, and included a geotextile at the bottom as well as an anchor block.

The mechanical response of ballast and slab tracks has been extensively investigated and researched. Initial models for studying track behaviour under moving loads considered a beam on spring foundation or a halfspace, either elastic or viscoelastic [16-20]. Thereafter, more advanced models were developed for a more precise estimation of the dynamic track response and ground-borne vibrations [21-24]. This advancement was enabled by a better understanding of the track behaviour by conducting small-scale element tests [25-28] and mockup tests [29-37]. Field tests have the advantage of evaluating as-built track conditions, including the response of the subsoil and assessment of ground-borne vibrations and noise [38-43]. Nevertheless, the testing of new materials or design concepts in the live railway network requires extensive certification. In this context, full-scale mockup testing can be a relatively cost-effective way to gain insight into the functionality of the different components under various loading conditions, while in a controlled environment. Besides, the ability to simulate the effects of a train passage in a limited test section [31, 34] has also been achieved within reasonable precision. This has enabled to get an estimate of the pattern and magnitude of the anticipated response in the live network. Finally, such testing could help to advance modelling efforts and increase their reliability.

There is limited scientific literature on the mechanical behaviour of ballastless asphalt tracks. In one of the early studies, a 3D program called KENTRACK [44] was developed by combining the finite element method for track superstructure and multilayered elastic theory for track substructure. The program was used to design the service life of asphalt-based tracks. The model included rails and sleepers (as beam elements), fasteners (as linear springs), and a two-layered track structure consisting of an asphalt layer resting on a subgrade. Design charts were developed using the maximum tensile strains at the bottom of asphalt and the maximum compressive stresses on top of subgrade as criteria to estimate the allowable...
number of load repetitions. The estimations were based on damage laws developed for highway pavements. The study concluded that the required thickness of the overlayment was governed by fatigue cracking, ranging between 0.25 m to 0.40 m.

A full-scale test section of a ballastless asphalt track was built in Korea with specially designed wide sleepers consisting of a geotextile at the underside [4, 45]. Vertical loads were applied to the rails, incrementally to a maximum value of 200 kN, and then unloaded back in stages. Measurements of vertical stresses, horizontal strains, and vertical displacements were done at different locations. The responses showed a non-linear behaviour with respect to the load level. Negligible hysteresis was recorded between the loading-unloading curves in stress measurements, while it was prominent for strains and displacements. Based on this study, a 0.3 m asphalt layer thickness was suggested, complying with the allowable stress requirement (133 kPa) at the top of the base layer.

Subsequently, the long term settlement of this proposed design was investigated [46] under moving wheel loads that were approximated as sinusoidal loads applied to the rails for several million consecutive cycles. The loading frequency was calculated based on the wheel-to-wheel distance and simulated train speed. No clarification was given on whether the load was applied at a fixed point or multiple points along the rails. The relative settlement between the rail and sleeper was reported to be small, indicating an elastic behaviour of the rail pads, while that between the sleeper and the asphalt trackbed was quite large, which was attributed to the compression of the geotextile and the track layers.

In a recent field study [47], a test track was built, and responses were monitored for a year. The study reported that vertical stresses in the base layer and horizontal strains in the asphalt layer were influenced by temperature, showing a significant increase in summer compared to winter. Nevertheless, no reference was made to the loading conditions under which the peak responses were measured. As part of this study, a 3D linear elastic finite element model of the test track was also developed, including all major components and layers. The asphalt layer was additionally modelled as linear viscoelastic. The study did not explain how the input material properties were obtained. A quasi-static analysis was performed with distributed loads acting on the rails, and model responses were compared with field measurements, in which the asphalt strains showed a poor match.

This study derives motivation from the limited research available on ballastless asphalt tracks. The understanding of the mechanical behaviour of such tracks can be strengthened by testing full-scale mockups, and further, developing advanced models by translating the know-how available in the context of the more traditional trackforms. The
The main objective of this study is to experimentally investigate the mechanical behaviour of a ballastless asphalt track under vertical loads. As a starting point, a full-scale mockup is constructed, instrumented, and tested indoors. The focus of this study is on investigating the mechanical track responses without impairing the system integrity. Service behaviour under nominally pristine conditions is targeted, and thus design limit states are not investigated as in existing studies. First described are the construction and instrumentation of the mockup. Presented next are measured results across different loading conditions. Finally, observed behaviours of general interest are highlighted and discussed.

2. Ballastless asphalt track mockup

2.1 Construction

A full-scale mockup of a ballastless asphalt track was constructed having a length of 4.00 m and width of 2.21 m. From bottom-up, the track layers included, (i) a 0.025 m thick polyurethane mat utilized as a substitute for subgrade and the subbase, (ii) a 0.275 m thick UGL, and (iii) a 0.280 m thick asphalt layer. Utilizing a thin mat as a replacement for subbase and subgrade has been done in other related studies, e.g., [36]. The layers were built inside a steel box in an outdoor location. Subsequently, the box was craned inside a test hall and placed on a thick concrete floor. A wet mix of cement mortar was spread on the floor before the box placement to ensure full contact between the bottom of the box and the floor. A schematic view of the test facility is shown in Figure 1, and the different stages involved in the construction of the mockup are shown in Figure 2. The mockup size was limited to the current dimensions to allow for the largest transportable facility that can be built with full-scale construction equipment. The steel boundaries are not expected to influence significantly the properties of the constructed track layers.

The selection of the polyurethane mat was based on guidelines [48] that specified a minimum value of modulus of deformation \( E_{v2} \) of 120 MPa on top of the subbase layer for a newly constructed track. The static elastic modulus of the mat based on the technical specification data provided by the manufacturer was 20 MPa. Light Weight Deflectometer (LWD) tests were performed to characterize the stiffness of the mat inside the box (see Figure 2a). The measured surface deflection and the applied force were used to calculate the dynamic deflection modulus \( E_{vd} \) using static, linear–elastic theory for a homogenous and isotropic half-space. The resulting \( E_{vd} \) values varied between 40 MPa and 50 MPa. The variation obtained is attributed to the boundary conditions of the test, as the thickness of the mat does not comply with the homogeneous half space assumption applied for the estimation of \( E_{vd} \).
The corresponding $E_{v2}$ value was obtained to be approximately 100 MPa based on correlations found in literature between $E_{v1}$ and $E_{v2}$ [49].

The UGL constructed on top of the mat was categorized as Danish type SGII [50] with a maximum aggregate size of 31.5 mm and fine content of 3.7 %. The UGL was compacted in two lifts using multiple passes of a vibratory tandem roller (see Figure 2b). The achieved dry density was estimated based on in-situ measurements at three different locations with a nuclear moisture density gauge as 98.1 % on average, relative to a vibrating table reference compaction [51].

The asphalt concrete layer constructed on top of the UGL (see Figure 2c and 2d) consisted of three identical base course lifts with a total thickness of 0.24 m, and a surface course with a lift thickness of 0.04 m. The base course lifts were coarse graded with a maximum aggregate size of 16 mm (Danish GAB 0 type 16). They included 5.1 % of unmodified binder graded as 70/100. The surface lift was a stone mastic asphalt (Danish SMA 11), characterized by an S shaped aggregate gradation curve with a maximum size of 11 mm. This mix included a styrene based polymer added to 70/100 bitumen (during asphalt production) with binder content of 5.4 %. The void content was estimated for the base course and the surface course as 3.1 % and 2.7 % (average values from 8 samples) respectively, which complies with the EN 16432-2 requirements [48].

Three wide concrete sleepers (type BBS 3 W60 of the GETRAC A3 system) were placed on top of the asphalt layer with a center to center spacing of 0.6 m (see Figure 2e). The sleepers were each 2.40 m long and 0.57 m wide at the base, and with a mass of 540 kg. They had a thin geotextile layer of thickness 7 mm at the underside, over the two ends (each over an approximate area of 1.00 m $\times$ 0.57 m). The sleepers were provided with a built-in Vossloh 300 fastening system. According to the manufacturer’s datasheet, the static stiffness of the rail pads was 23 kN/mm ($\pm$ 10 %).

### 2.2 Loading setup and Instrumentation

Vertical loads were applied to the mockup via three identical 100 kN servo-hydraulic actuators (MTS Model No. 244.22) that were fixed to a rigid reaction frame built over the box (see Figure 2e). The spacing between the actuators corresponded to the sleeper spacing of 0.6 m (see Figure 1a). Small rail segments (type UIC60) of length 0.20 m were mounted onto the sleepers with the built-in fastening system to aid in load distribution. As there was one actuator for loading each sleeper, a spreader beam was installed to distribute the load equally on top of the two rail segments (see Figure 1b and 2e). The contact between the spreader
beam and the rail segment was established through a circular steel disc of diameter 10 mm

 glued at the top of the rail-segment at its centre in order to ensure the best possible alignment

 and symmetry in the loading.

 The sensors employed for monitoring the different mechanical responses in the

 mockup are depicted in Figure 3. These included: earth pressure cells (PCs), asphalt strain

 gauges (ASGs), potentiometers (POTs), linear variable differential transformers (LVDTs),

 and accelerometers (ACCs). Figures 3a and 3b show a schematic layout of the embedded

 sensors at two different elevations inside the box when viewed from the top. Figure 3c shows

 a schematic layout of the surface sensors. These figures also show a Cartesian coordinate

 system placed at the center of the box, which is defined for the location reference of the

 various sensors. The images of the instrumentation as installed in the setup are shown in

 Figures 3d-3g. Full details about the embedded and the surface sensors are listed in Table 1

 and Table 2 respectively. Additional strain gauges (not listed in Table 1) were mounted along

 the sides of the steel box and at the bottom flange of the spreader beams to monitor the

 boundary conditions in the setup. It was assessed through a separate numerical analysis that

 the boundary effects caused by the steel box on the sensor measurements (under a pulse load)

 was less than 10 % at the peak recorded values.

 2.3 Test plan

 In general terms, the mockup was interrogated in two phases, by two types of tests: (i) cyclic

 loads applied only to Sleeper 2, and (ii) simulated moving train loads, which consisted of

 loading all three sleepers sequentially. The cyclic loads were applied only to the central

 sleeper, as in this case, the maximum response magnitudes were expected to occur in the

 sensors directly below it. The influence of the boundary on these sensors was anticipated to be

 minimum amongst all the deployed sensors. The experiments were performed by imposing a

 deformation controlled time history at the respective actuators, and at an average temperature

 of 22 °C. More details on each of the test types are provided hereafter.

 2.3.1 Test Phase 1: Cyclic loads

 In this phase of testing, the displacement of Actuator 2 was controlled to apply sinusoidal

 excitations of different amplitudes and frequencies to the central sleeper for 200 cycles. The

 aim was to establish steady state conditions and analyse the sensor readings for the effects of

 different loading frequencies and amplitudes. The tests carried out during this phase are

 shown in Table 3. The tested load amplitudes ranged between 10 and 30 kN, and three

 different frequencies: 0.1 Hz, 1.0 Hz, and 10.0 Hz were considered.
2.3.2 Test Phase 2: Simulated moving train loads

In this phase of testing, the loading induced by the Danish IC3 train was simulated, with a load arrangement as shown in Figure 4a. This train has a length of 56 m, consisting of three wagons and eight axles. Let \( j \) be an index denoting the axle number, then \( j = 1 \) represents the rear axle while \( j = 8 \) denotes the front axle. Let the individual axle positions relative to the rear axle be denoted as \( \Delta x_j \) (i.e., \( \Delta x_1 = 0 \)) and the respective axle loads be \( 2P_{Z,j} \), wherein \( P_{Z,j} \) denotes the wheel load on one side of a sleeper. The simulation assumes that during the IC3 train passage any random sleeper is symmetrically loaded by a dual-force. Let this vertical load history on top of a random sleeper (on one side, excluding the weight of the rail) be denoted as \( S_{Z}^{\text{train}} \). Using the simplified track model of an infinite beam on Winkler foundation (see Figure 4b), and based on equations previously described \[52\], \( S_{Z}^{\text{train}} \) was calculated as:

\[
S_{Z}^{\text{train}} = \frac{s}{2} \sum_{j=1}^{8} p_{Z,j} e^{\beta_z \left(x + \Delta x_j + V t\right)} \left(\cos(\beta_z \left(x + \Delta x_j + V t\right)) + \sin(\beta_z \left(x + \Delta x_j + V t\right))\right)
\]

wherein \( s \) denotes center-to-center sleeper spacing, \( t \) is simulation time, \( V \) denotes train travel speed, and

\[
\beta_z = \frac{4 k_z}{EI}
\]

in which \( k_z \) is the track modulus, \( EI \) is the bending rigidity of the beam (properties as per rail section UIC 60), and \( x \) is the initial \((t=0)\) distance between the sleeper and the rear axle of the train. Figure 4c shows the calculated load history of a random sleeper (on one side) during the IC3 train passage. Figure 4c also depicts that upward beam bending produces tensile forces on the top of the sleeper when an axle just approaches or leaves the location.

In the mockup, passage of IC3 train was simulated by the principle of sequential loading \[30, 33\]. The rationale behind this method is explained as follows. When the train moves on the rails, the wheel loads are transferred from the rails to the substructure below via the sleepers. Every sleeper along the railway track direction experiences the same load history \((S_{Z}^{\text{train}})\) but with a time delay \((\Delta t)\) depending on the train speed \((V)\) and the spacing between the sleepers \((s)\) where \( \Delta t \) is simply given by \( \Delta t = s/V \). Consequently, the vertical loads induced by a moving train can be approximately reproduced by sequentially loading the individual sleepers with the exact same vertical load history. Figure 4d illustrates the target
actuator load in the mockup set as \( 2S_{train} \) so that load distributed on each side of the sleeper was \( S_{z}^{train} \) (after applying a cut-off to the tensile part of the load history). Tensile loads could not be applied in the set-up as the spreader beams were only resting on the rail segments. The loading scheme was sequential (from Sleeper 1 towards Sleeper 3), with a time delay between Actuator 1 and 2 as \( \Delta t \) and that between Actuator 1 and 3 was \( 2\Delta t \). Based on a separate numerical analysis [53], the value of \( k_{z} \) was adopted to be 42 MPa for the speed and load range simulated in this study. This value is very close to what would be obtained if the track modulus was calculated based on the stiffness of the rail pads. As in test Phase 1, the target loads were achieved by controlling the displacement of the individual actuators. Because the assumptions of the Winkler model do not perfectly comply with the mockup, and also to eliminate the coupling between the three actuators, the displacement functions required some manual fine-tuning in order to reach the target load. Table 4 lists the tests carried out in this experimental phase. As can be seen, the test in the list (No. 10) included a single axle loading history and not a full train. This single axle passage was simply calculated according to the explanation above with \( j=1 \) in Equation 1.

3. Test results and discussions

3.1 Responses for cyclic loading

In this section, the results of the tests carried out in Phase 1 are presented and discussed. The sign convention adopted is as follows: downward load and displacement are considered positive, and compressive stresses and strains are positive.

3.1.1 Influence of load frequency

First addressed are results from tests 1, 2, and 3 (see Table 3) where cyclic loads of amplitude 19 kN (on average) were applied by Actuator 2 at three different frequencies: 0.1 Hz, 1.0 Hz, and 10.0 Hz. The resulting responses are presented in Figure 5 for the last ten cycles, during which nearly steady state conditions were achieved. In all the three tests, the maximum increase of the mean load and the load amplitude for the last 100 cycles was observed to be less than 5%. The individual steady state sensor responses from the three tests were zeroed at their respective minimum values to bring them into a common origin (for ease of graphical comparison).

Figure 5a depicts Actuator 2’s force-displacement curve; there are three plots in this figure, each corresponding to a different frequency – each illustrating a closed hysteresis loop. It should be noted that the actuator’s displacements refer not only to the displacements of the test section, but they also include the deformation of the spreader beam and the loading frame.
However, the latter is purely elastic, which means that the area enclosed within these loops can be considered as a measure of the overall energy dissipated in the track system over a single load-unload cycle. The loop areas calculated for the three frequencies are 3.55 J @ 10.0 Hz, 3.46 J @ 1.0 Hz, and 3.43 J @ 0.1 Hz. The values are nearly similar, indicating that the dissipation energy is weakly linked to the applied frequency. The overall stiffness of the system is complex, with a norm indicated by the inclination of the hysteresis loop. The norm of the complex stiffness is estimated to be: 35.0 kN/mm @ 10.0 Hz, 33.4 kN/mm @ 1.0 Hz, and 32.0 kN/mm @ 0.1 Hz, indicating a minor increase with the applied load frequency.

Figure 5b illustrates the force exerted by Actuator 2 against the relative vertical displacement recorded by POT 40. Nearly identical readings were obtained by POT 39 and POT 41 (and therefore not shown). The potentiometers measure the vertical compression of the rail pads. The three curves in this figure correspond to the three test frequencies. A closed hysteresis loop is formed for a load-unload cycle, which is similar in shape for the different applied frequencies. The area of the loops remains similar for a frequency of 0.1 Hz and 1.0 Hz and decreases at 10.0 Hz. The norm of the complex stiffness of the rail pads is estimated to be: 30.7 kN/mm @ 10.0 Hz, 29.4 kN/mm @ 1.0 Hz, and 28.3 kN/mm @ 0.1 Hz. Thus, the behaviour of the rail pads is observed to be mildly sensitive to the excitation frequency within the considered range.

The force exerted by Actuator 2 against the relative vertical displacements recorded by LVDT 1 and LVDT 2 is shown in Figure 5c. Both LVDTs measured the relative displacement between Sleeper 2 and the asphalt surface, i.e., the combined effect of the geotextile compression and the surface displacement at these two locations. LVDT 1 (as well as LVDT 4) was located at the edge of Sleeper 2, while LVDT 2 (as well as LVDT 3) was located in the middle of Sleeper 2 (see Figure 3). The readings from LVDT 3 and LVDT 4 are not shown as they contain virtually identical information to the readings of LVDT 2 and LVDT 1, respectively. There are six curves in Figure 5c, three of which correspond to the readings of LVDT 1 across the three frequencies, and the other three correspond to LVDT 2. All six curves display a pronounced hysteresis loop for a load-unload cycle. With an increase in the loading frequency from 0.1 Hz to 10.0 Hz, the calculated area of the loops (area bounded by the loading-unloading curves) shows a gradual decrease. Further, the amplitude measured by LVDT 1 and LVDT 2 also shows a decrease by 30% and 47%, respectively, with an increase in the loading frequency. These results depict that the relative vertical displacement between the sleeper and the asphalt surface is very sensitive to the excitation frequency. At a given frequency, the amplitude measured by LVDT 1 is higher than of LVDT 2. The difference
between the displacement amplitudes can be attributed to the difference in surface deflection at these locations, i.e., at the edges and in the middle of Sleeper 2 (neglecting the effect of sleeper bending).

Figure 5d presents the force exerted by Actuator 2 against the vertical stress recorded by pressure cell PC 16. PC 14 gave similar readings, and therefore the measurements are not included in the figure. The three curves in this figure correspond to the three test frequencies. The stress and the load curves also form a closed-loop, the area of which is lower at a higher frequency. Due to its location, PC 16 records the maximum stress amplitudes at the top of the mat (or equivalently at the bottom of the UGL). The measured stress amplitudes show a 38% decrease with an increase in frequency from 0.1 Hz to 10.0 Hz. This behaviour is a manifestation of the frequency-dependent response of the overall track structure.

Figure 5e depicts the force exerted by Actuator 2 against the measurements of all the five pressure cells at a frequency of 1 Hz (i.e., Test 2 in Table 3). A progressive decrease in the stress amplitude can be observed with increasing distance from Sleeper 2 in both X and Y directions. The peak stress amplitudes are registered by PC 14 (7.3 kPa) and PC 16 (7.1 kPa), which are below the loading positions (i.e., below the rail pads). PC 15, which is below the middle of Sleeper 2 (i.e., in the unloaded region), measures about 4.7 kPa. PC 17 and PC 13, which are below the middle of Sleeper 1 and Sleeper 3 respectively, measures about 1.5 kPa. The pressure cells below the loaded areas (i.e., PC 14 and PC 16) show a closed hysteresis loop between the loading and unloading curves, which are similar in shape and area. In contrast, the pressure cells below the unloaded regions (i.e., PC 13, PC 15, and PC 17), do not display any noticeable hysteresis between loading and unloading curves. Further, as expected from symmetry, the curves are very similar for PC 13 and PC 17.

The horizontal strains measured by ASG 6 are shown in Figure 5f. The strain amplitude measured by ASG 6 is the maximum expected strain along the X direction at the bottom of the asphalt layer (amongst the functioning sensors). The sensor ASG 7 records similar measurements as ASG 6 and is not shown in the figure here. The trend of the other strain gauges with changing frequencies could not be reliably estimated from these tests. This was because the recorded strains were quite small and within the precision limits of the sensors. It is observed that the strain amplitudes measured by ASG 6 decrease by about 48%, with an increase in the frequency from 0.1 Hz to 10.0 Hz. This indicates that the horizontal strains generated below the asphalt layer show a strong frequency-dependency. A pronounced hysteresis loop is formed between the loading and unloading curves at frequencies of 1.0 Hz and 10.0 Hz, respectively, while no noticeable hysteresis occurs at a lower frequency of 0.1
Hz. This implies that at the higher frequencies (1 Hz and 10 Hz), a noticeable time delay occurs between the applied loads (at the top of Sleeper 2) and the generated horizontal asphalt strains. Comparatively, at a slower loading rate (0.1 Hz), the time delay between the applied force and the horizontal asphalt strains is much smaller.

3.1.2 Influence of load amplitude

In order to investigate the influence of load amplitude on the track behaviour, cyclic loading was performed at 1 Hz at an approximate mean load level of 31.2 kN, while the load amplitude was varied. The load amplitude was increased by a factor of two between Test 6 and Test 4, and a factor of three between Test 8 and Test 4 (see Table 3). The same procedure was repeated at 10 Hz in between Tests 7 and Test 5, and between Test 9 and Test 5. The corresponding steady state sensor amplitudes were computed from the last ten cycles (average value) and used to analyse the influence of load amplitude. The sensor amplitude ratios computed from Test 6/Test 4 (@ 1 Hz) and Test 7/Test 5 (@ 10 Hz) are displayed graphically in Figure 6a. They are compared to the corresponding load amplitude ratio of approximately 2. The sensor amplitude ratios computed from Test 8/Test 4 (@ 1 Hz) and Test 9/Test 5 (@ 10 Hz) are shown in Figure 6b and compared with the load amplitude ratio of approximately 3.

From Figure 6a, it is observed that the LVDT amplitude ratio is 30 % higher than that of the load amplitude ratio. This indicates that the relative vertical displacement between the asphalt surface and the sleeper varies non-linearly with the load level. The amplitude ratios of the potentiometers and the asphalt strain gauges are only slightly higher (about 10 %), while the pressure cells are almost similar to that of the load. This indicates that the rail pad compression, horizontal strains below the asphalt layers, and the vertical stresses below the granular layer scale almost linearly with the load amplitude. The sensor amplitude ratios are similar at both the frequencies of 1 Hz and 10 Hz. As the load amplitude ratio increases to 3 in Figure 6b, the non-linearity showed by the LVDT readings is pronounced and is higher at a frequency of 1 Hz. The amplitude ratios of the potentiometers and the asphalt strain gauges are slightly higher than before (about 15 %). This indicates minor non-linearity in the rail-pad compression and horizontal asphalt strains. In contrast, the amplitude ratios of the pressure cells are similar to that of the load as before.

3.2 Simulated moving loads

In this section, the results from simulating a moving load are presented. First, the findings from the simulation of a single moving axle load (Table 4, Test 10) are presented and discussed. Thereafter, the outcomes from simulating the Danish IC3 train (Tests 11 to 14 in
Table 4) are discussed. All the responses that are presented graphically in this section are a manifestation of the externally applied load only without the initial conditions.

3.2.1 Response for a single axle passage

The results from the simulation of a single axle load of magnitude 120 kN, moving at a speed of 120 km/h from Sleeper 1 towards Sleeper 3, are shown in Figure 7. The time history of the loads applied by the three actuators is illustrated in Figure 7a along with the theoretical values calculated from the Winkler model using Equation 1 \( S^\text{min}_Z \) shown in dashed lines. The shape of the applied force and the corresponding theoretical target force are similar overall, with some deviations at the beginning of the loading curve and in the unloading region. These differences are because the setup was excited in displacement-control mode. The peak forces applied by the three actuators are 38.0 kN, 38.6 kN, and 38.4 kN. The time delay between the peak force in Actuator 1 and Actuator 2 is measured as 19.2 ms, while that between Actuator 2 and Actuator 3 is 18.5 ms. Comparing these values with the theoretical values (given in Table 4 for Test 10) shows that the simulation of the moving axle load is achieved reasonably well in the mockup. According to the analysis of an equivalent Winkler model, for \( K_z = 42 \) MPa, when a wheel load is located on top of any sleeper, the axle load is mainly distributed between five sleepers, i.e., the central (loaded) sleeper plus two on either side; the relative amounts are (%): 10.3, 24.2, 34.0, 24.2, 10.3. For a case where only three sleepers are present, the force in the loaded sleeper (40.9 kN) and the adjacent ones (29.1 kN) amounts to 82.4 % of the axle load. In Figure 7a, it is seen that corresponding to the peak force of 38.6 kN in Actuator 2, the average forces in Actuators 1 and 3 were 22.5 kN amounting to a maximum of 70.5 % of the axle load.

The time history of the relative vertical displacement recorded by the three potentiometers (POT 39, POT 40, and POT 41) is presented in Figure 7b. The potentiometer readings follow the overall shape of the applied load, increasing gradually with the force level and then unloading to initial conditions without any residual displacement, indicating an elastic behaviour for the rail pads. As expected, the overall behaviour of the three sensors is quite similar, although POT 40 records slightly higher values. The average peak value is 0.626 mm, the corresponding spring stiffness of the rail pads is therefore 30.8 kN/mm, calculated as 0.5×38.6/0.626.

In Figure 7c, the time history of the relative vertical displacements recorded by four LVDTs (LVDT 1 to LVDT 4) are displayed. The LVDT readings follow the shape of the applied load during the loading phase. However, in contrast to the potentiometers, they do not
revert to the initial conditions on unloading, and within the experimental window, their recovery is still ongoing. This reflects the time-dependent nature of the mockup, particularly the geotextile and the asphalt concrete. LVDT 1 and LVDT 4 recorded similar peak values of around 0.195 mm, while LVDT 2 and LVDT 3 recorded similar peak values of 0.130 mm. The variation in the peak values can be attributed to the difference in surface deflection below the edges and around the middle of Sleeper 2 (neglecting the bending of the sleeper). Comparing the LVDT and the POT readings, it can be concluded that most of the vertical displacement in the track (75%) is due to the rail pad compression.

The time histories of the vertical stresses recorded by pressure cells (PC 13 to PC 17) are illustrated in Figure 7d. The five sensors follow the overall shape of the applied load without any noticeable time-dependent behaviour on unloading. The time delay between the peak stresses recorded by the pressure cells indicates the axle movement from Sleeper 1 to Sleeper 3. PC 17, located below Sleeper 1, registers the first stress peak of about 13 kPa. The next peaks co-occur for PC 14 (18.5 kPa), PC 15 (15.6 kPa), and PC 16 (20.3 kPa), which are all located below Sleeper 2. The last stress peak of 9.5 kPa is recorded by PC 13, located below Sleeper 3. As a first-order approximation, the stresses below the sleepers can be assumed to be uniform over the entire loaded area. This amounts to a calculated average value of 33.9 kPa below Sleeper 2 and about 60% of this value below Sleeper 1 and Sleeper 3. At this loading condition, the peak vertical stress recorded below the UGL by PC 16 is approximately 60% of the average stress below Sleeper 2. The peak stresses recorded below Sleeper 1 (PC 17) and Sleeper 3 (PC 13) are lower than those recorded under Sleeper 2 (PC 15) because these two sleepers are at the boundaries, and the load from adjacent sleepers is missing on one side. PC 17 records peak stresses that are 35% higher than that of PC 13. This is not consistent as these sensors are expected to show similar readings because of the symmetry in positioning (Figure 3) and applied loads (Figure 7a). The differences may be attributed to slight variations in the installation of the individual pressure cells as these sensors are very sensitive to it.

Figures 7e and 7f display the time history of the horizontal strains at the bottom of the asphalt layer in the X and Y directions, respectively. The axle movement can also be visualized here from the time delay between the peak readings of the different strain gauges. In Figure 7e, the peak strains occur successively from ASG 11 (below Sleeper 1) to ASG 6, ASG 7 (below Sleeper 2), and ASG 3 (below Sleeper 3). Similarly, in Figure 7f, the peaks occur consecutively from ASG 10 (below Sleeper 1), ASG 9 (below Sleeper 2), and then ASG 2 (below Sleeper 3). The overall behaviour of the strain gauges reflects the shape of the
applied load, with a gradual increase in the strains and then a time-dependent recovery to the initial conditions without any permanent strain accumulation. In some of the strain gauges, the recovery is still ongoing within the test window.

Among the functioning sensors in the X direction, the peak tensile strains are expected under ASG 6 and ASG 7, which record similar values of around 16.7 microstrains. ASG 3 (below Sleeper 3) illustrates a similar peak value, while ASG 11 (below Sleeper 1) records a peak strain that is 50% more (24 microstrains), which is not consistent with the other sensor responses. When the load is applied on Sleeper 1, tensile strains are recorded below it by ASG 11, and compressive strains are measured below Sleeper 3 by ASG 3. The compressive strains measured by ASG 3 reach a peak value of 5 microstrains at approximately 25.1 s, when peak tensile strains of magnitude 24 microstrains occur below Sleeper 1 as measured by ASG 11. Thereafter, as the load gradually moves towards Sleeper 2 and then Sleeper 3, the strains measured by ASG 3 progressively reverse from compression to tension.

The peak tensile strain recorded by ASG 9 (below Sleeper 2) in the Y direction is 18 microstrains. This value is similar to the peak tensile strain along the X direction. The peak strains measured by ASG 2 and ASG 12 are similar to ASG 9, while the peak strains recorded by ASG 10 (below Sleeper 1) are lower by 35%. This is not consistent as these sensors (ASG 2, ASG 10, and ASG 12) are expected to provide similar peak responses considering the nearly symmetric applied loading history (Figure 7a) and their installed positions (Figure 3).

3.2.2 Response for a full train passage at different speeds and axle loads

In this section, selected results from the simulation of Danish IC3 train passage at different speeds and axle loads are presented and discussed. The load and displacement curves of only Actuator 2 from Test 11 to Test 14 (see Table 4) are shown in Figure 8. There are four charts in this figure corresponding to simulations involving two different axle loads: 120 kN and 200 kN, and two train speeds: 120 km/h and 200 km/h. The loading part of the four curves appears as a straight line with an average slope of about 28.3 kN/mm; this slope is similar in all the four cases. The unloading part is curved and forms a hysteresis loop generating similar overall load-displacement curves at the two test speeds. The corresponding traces of all sensor readings in the mockup (except for the accelerometers), were also very similar for the two speeds. These observations indicate that an increase of train speed from 120 km/h to 200 km/h has little effect on the track responses. Therefore, in the following only results from a simulated train passage at a speed of 120 km/h at two different axle loads (120 kN and 200 kN) are presented and addressed (Tests 11 and 12 in Table 4).
Figure 9a shows the force history of Actuator 2 in Test 11 and Test 12. The forces in Actuators 1 and 3 were similar (and therefore not shown), but with a simulated time delay of 18 ms as given in Table 4. Eight distinct peaks can be seen in the Figure, each corresponding to one of the eight IC3 train axles (see Figure 4a). For the axle load of 120 kN, the average value of the peak force applied on top of Sleeper 2 was 38.2 kN, while for the axle load of 200 kN, it was 64.3 kN, i.e., very close to their respective theoretical peak values (see Table 4).

The time history of relative vertical displacement recorded by three potentiometers (POT 39, POT 40, and POT 41) at the two load levels are shown in Figure 9b as an average. The overall shape of the two curves is similar, consisting of eight peaks corresponding to the eight axles and conforming to the shape of the applied force shown in Figure 9a (both during loading and unloading). The average values of the peaks (i.e., rail pad compression) in the two tests are 0.94 mm, and 0.62 mm. From this, the average stiffness of the rail pads is calculated to be 30.5 kN/mm and 34.0 kN/mm respectively. This slight difference in rail pad stiffness indicates the minor dependence on loading rate and amplitude. The average peak value observed at an axle load level of 120 kN is very similar to the case of a single moving axle at the same load level and speed (see Figure 7b).

Figure 9c shows the relative vertical displacement measured by the LVDTs for the two tests. The LVDT readings at similar locations are separately averaged, i.e., LVDT 1 and LVDT 4 are averaged, and LVDT 2 and LVDT 3 are averaged. In each of the four charts, eight distinct peaks corresponding to the train axles are identified. The measurements show minor accumulation with successive axle passes, causing the consecutive peaks to be higher. A time-dependent recovery is observed after the passage of sets of closely spaced axles on the same bogie. Within the test window, the recovery of the LVDT readings is still ongoing. The average peak value noted at an axle load level of 120 kN is again very similar to those in Figure 7c, simulating a single moving axle load.

The vertical stress history of PC 16 and PC 15 (located below Sleeper 2) are shown in Figures 9d and 9e, respectively. The overall behaviour of each chart resembles the applied load signal shown in Figure 9a. For the two load levels, the peak stress (average) below the UGL is measured by PC 16 to be 35 kPa and 21 kPa, respectively. It amounts to approximately 60% of the calculated average stress below Sleeper 2, assuming a uniform stress distribution. The stresses at the same elevation below the unloaded part of Sleeper 2 (i.e., at the middle) are measured by PC 15 to be 25 kPa and 15.3 kPa, which are about 45% of the calculated surface stress.
Figures 9f and 9g show the horizontal strain history measured at the bottom of the asphalt layer by ASG 7 (in the X direction) and ASG 2 (in the Y direction), respectively. ASG 7 is located below Sleeper 2, where less boundary influence is expected on the strain readings in the X direction. Readings of ASG 2 are displayed in Figure 9g instead of ASG 9 (where higher responses and lower boundary influences are anticipated) as the latter showed unreliable measurements in these tests. The choice of ASG 2 was also based on the reliability of measurements established from Test 10 (see Figure 7f).

The peak tensile strains recorded by ASG 7 in the X direction at the two load levels are 25 microstrains and 15 microstrains, respectively. The peak tensile strains measured by ASG 2 in the Y direction are 39 microstrains and 20 microstrains, respectively. ASG 2, which measures horizontal strains in a direction transverse to the moving load, shows accumulation resulting in consecutive strain peaks to be higher (see Figure 9g). In contrast, ASG 7, which measures horizontal strains along the direction of the moving load, shows no noticeable accumulation, and the consecutive strain peaks are of nearly similar magnitude. Another noticeable difference in the readings of the two strain gauges is the faster strain recovery observed along the direction of the moving load (see Figure 9f) compared to the transverse direction (see Figure 9g). Within the experimental window, the strain recovery is ongoing, especially in the readings of ASG 2. The peak strains measured by the two sensors at 120 kN axle load are similar to the single moving axle simulation, as shown in Figures 7e and 7f.

The vertical surface acceleration history, as recorded by the sensor ACC 36, is illustrated in Figure 9h. The passes of the successive train axles can be observed from the recordings. The peak accelerations at the two load levels were measured to be about 0.9 m/s² and 0.7 m/s², respectively.

In order to investigate the effects of speed, some additional test results are analysed in the time and frequency domain. The force time history of Actuator 2 in Tests 11 and 13 (Table 4) is converted to the normalised amplitude spectra by Fourier transform and illustrated in Figures 10a and 10b, respectively. In these two tests, the simulated train speeds are 120 km/h, and 200 km/h and the axle load is 120 kN. The corresponding normalised amplitude spectra of the sleeper force history, as calculated from the Winkler model, are also displayed in Figure 10. This figure demonstrates that the frequency content of the sleeper force history calculated from the Winkler model was adequately reproduced by Actuator 2 in the two tests in the mockup. Similar observations were verified with the forces applied by Actuator 1 and Actuator 3 (therefore not shown).
The normalized amplitude spectrum of actuator forces at an axle load of 200 kN (at both the speeds) was also verified to be similar to the theoretical spectrum. It is noted that the frequencies associated with vehicle dynamics, wheel-rail contact, soil layer are not captured in the Winkler model and consequently not in the mockup as well. In Figure 10a, for the simulated speed \(V\) of 120 km/h, the dominant frequency in the spectrum corresponds to the bogie passage frequency \(f_b\) and its higher-order harmonics, which is calculated as:

\[
f_b = \frac{V}{L_b} = 1.9 \text{ Hz}, \text{ where } L_b \text{ is the distance between bogies} = 17.7 \text{ m (see Figure 4a)}.\]

The spectrum is modulated around the axle passage frequency \(f_a\), which is calculated as:

\[
f_a = \frac{V}{L_a} = 13.3 \text{ Hz}, \text{ where } L_a \text{ = distance between the axles} = 2.5 \text{ m (shown in Figure 4a)}.\]

In Figure 10b, the frequency spectrum at a speed of 200 km/h is presented where the dominant frequency is at \(f_b = 3.1 \text{ Hz}\) and its higher-order harmonics with modulations at \(f_a = 22.2 \text{ Hz}\).

The effect of speed on the track responses can be visualized in Figure 11a that shows the vertical surface accelerations as recorded by sensor ACC 36 (see Figure 3) in Tests 12 and 14 (see Table 4). These tests simulated an axle load of 200 kN at two different speeds of 120 km/h and 200 km/h. Peak accelerations were recorded by this sensor on the track centre line in the mockup, and it increased from 0.9 m/s\(^2\) to 1.8 m/s\(^2\) with an increase in the simulated train speed. The corresponding frequency domain responses are shown in Figure 11b. The spectrum shows the frequency peaks that are already associated with the load signal (see Figure 10a and 10b). In addition, higher frequency content is observed in the range beyond 60 Hz, which could be attributed to the free vibrations of the system.

**4. Summary of findings and conclusions**

This paper focused on the experimental investigation of a ballastless asphalt track under vertical loads and constant temperature conditions. For this purpose, a full-scale mockup of size 4.00 m × 2.21 m was built inside a steel box, consisting of a rubber mat (that simulates subbase and subgrade), UGL, and asphalt concrete layer. Three wide sleepers were placed on top of the asphalt layer, and servo-hydraulic actuators were used to load the individual sleepers directly at the top of the rail pads. A wide array of sensors, both embedded and surface, were installed to measure different mechanical responses in the mockup.

The instrumentation effort has provided some insight regarding the design of such large scale test sections: (i) the chosen pressure cells exhibited reliable behaviour across all tests, though the actual working range was very small (<less than 50 kPa) relative to the operating range (up to 1 MPa) that was so selected in order to withstand the construction
process, (ii) the asphalt strain gauges showed somewhat erratic recordings - coupled with the fact that the measured strains were very small, almost within the precision limits, and (iii) including redundancy in the mockup, of about 50 %, helped to extract reliable measurements especially for the strains.

Testing under cyclic loads of different frequencies has demonstrated a frequency-dependent behaviour of all measured responses, with the load-displacement response of the track developing an increase of the complex stiffness norm at increasing frequencies. The majority of measured track responses displayed a nearly linear behaviour when comparing steady state responses at different load amplitudes. Strong non-linearity was demonstrated only by the relative vertical displacement between the sleeper and the asphalt surface. Testing under simulated moving loads, e.g., axle load passage and train passage, has further confirmed the pronounced time-dependent behaviour of the track responses, such as strain accumulation in-between successive axle passes (e.g., horizontal strains below asphalt layer and relative vertical displacement between sleeper and asphalt surface). Under simulated train loading, the measured substructure responses (stresses and strains) were of very low magnitude; well within the standard requirements for ballastless tracks [48], considering peak vertical stresses and peak horizontal tensile strains at the bottom of the asphalt layer. Peak vertical acceleration measured on the track centre line was shown to increase with the increase of the train speed. Nevertheless, the peak vertical acceleration was less than 2 m/s², which is comparable to reported values in ballasted tracks but at a much larger distance, approximately 3 m away from the track centre line [54].

Based on the findings from the experimental investigations, modelling for ballastless asphalt tracks should include the effects of time dependence and non-linearity in material behaviour. Also, special attention should be drawn to the rail pad since the majority (75 %) of the actuator stroke in the current experimental investigation could be attributed to the rail pad compression. The geotextile compression also noticeably contributed in this connection.

As part of future study, resilient track responses can be investigated in the mockup for other sleeper designs with different fasteners and geotextiles. Besides, the response of such a track to horizontal loads can also be studied by using a modified loading and instrumentation scheme. Nonetheless, the findings of the current tests provide an overall intuitive understanding of the mechanics of ballastless asphalt tracks under anticipated train loads and support the development of reliable models for further analysis and design purposes.

Acknowledgement
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Figure captions

Figure 1: Schematic view of the test facility showing: (a) cross section and (b) longitudinal section (All dimensions are in mm).

Figure 2: Major construction stages of the test section. (a) Placement of the mat, (b) compaction of the unbound granular layer, (c) placement of the asphalt mixture, (d) compaction of the asphalt layer, and (e) constructed test facility.

Figure 3: Schematic layout (top view) showing location of: (a) pressure cells on top of the mat, (b) asphalt strain gauges at the bottom of the asphalt layer, and (c) surface instrumentations. Images of installed instrumentation from the mockup showing: (d) pressure cells (PC), (e) asphalt strain gauges (ASG), (f) potentiometer (POT) (View B), and (g) LVDT and accelerometer (ACC) (View A) (All dimensions are in mm).

Figure 4: Illustration of the methodology for simulating moving train loads in the mockup. (a) Configuration of the simulated train, Danish IC3, (b) model of infinite beam on Winkler foundation, (c) time history of sleeper force (on the top and at one side) during IC3 train passage as calculated from the above model, and (d) sequential loading of the three actuators in the mockup (Dimensions are in m).

Figure 5: Cyclic forces applied through Actuator 2 versus different mockup sensor readings at steady state conditions. (a) Displacement of Actuator 2, (b) POT 40, (c) LVDT 1 and LVDT 2, (d) PC 16, (e) all pressure cells (@1 Hz), and (f) ASG 6 (see Figure 3 and Tables 1 and 2 for sensor positions and other details).

Figure 6: Effect of the load amplitude on the steady state sensor response. The amplitude ratio of the different sensors readings corresponding to: (a) load amplitude ratio of 2 and (b) load amplitude ratio of 3 (see Figure 3 and Tables 1 and 2 for sensor positions and other details).

Figure 7: Time history of sensor responses measured during simulation of a moving axle load \( 2P_{z,j} = 120 \text{ kN} \) and \( V = 120 \text{ km/h} \). Responses are shown for: (a) actuators, (b) potentiometers (c) LVDTs, (d) pressure cells, (e) asphalt strain gauge along X direction and, (f) asphalt strain gauge along Y direction (see Figure 3 and Tables 1 and 2 for sensor positions and other details).

Figure 8: Force displacement curve of Actuator 2 from simulation of train passage at two different axle loads and speeds \( 2P_{z,j} = 120 \text{ kN} \) and 200 kN, \( V = 120 \text{ km/h} \) and 200 km/h.

Figure 9: Time history of sensor responses measured during simulation of IC3 train passage at two different axle load levels (for \( V = 120 \text{ km/h} \) and \( 2P_{z,j} = 120 \text{ kN} \) and 200 kN). Responses are shown for: (a) Actuator 2, (b) average reading of potentiometers, (c) average reading of LVDTs, (d) - (e) pressure cells, (f) asphalt strain gauge along X direction, (g) asphalt strain gauge along Y direction, and (h) accelerometer (see Figure 3 and Tables 1 and 2 for sensor positions and other details).

Figure 10: Normalized spectral magnitude of vertical sleeper force history during simulation of train passage \( 2P_{z,j} = 120 \text{ kN} \) at two different speeds of: (a) \( V = 120 \text{ km/h} \), and (b) \( V = 200 \text{ km/h} \).
Figure 11: Accelerometer responses during simulation of train passage ($2P_{x,j} = 200$ kN) at two different speeds in: (a) time domain and (b) frequency domain (see Figure 3 and Table 2 for sensor position and other details)

List of Figures

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Figure 2: Major construction stages of the test section. (a) Placement of the mat, (b) compaction of the unbound granular layer, (c) placement of the asphalt mixture, (d) compaction of the asphalt layer, and (e) constructed test facility
Figure 3: Schematic layout (top view) showing location of: (a) pressure cells on top of the mat, (b) asphalt strain gauges at the bottom of the asphalt layer, and (c) surface instrumentations. Images of installed instrumentation from the mockup showing: (d) pressure cells (PC), (e) asphalt strain gauges (ASG), (f) potentiometer (POT) (View B), and (g) LVDT and accelerometer (ACC) (View A) (All dimensions are in mm)
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Figure 11: Accelerometer responses measured during simulation of train passage (\(2P_{Z,j} = 200\) kN) at two different speeds in: (a) time domain and (b) frequency domain (see Figure 3 and Table 2 for sensor position and other details).
List of Tables

Table 1

*Embedded sensors installed in the setup*

<table>
<thead>
<tr>
<th>Sensor name</th>
<th>Location [in m]</th>
<th>Elevation</th>
<th>Model No</th>
<th>Measured entity</th>
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<tr>
<td>PC 13, PC 17</td>
<td>X = ±0.60, Y = 0.00</td>
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<td>GEOKON</td>
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*ASG 4, 5, 8 were unresponsive in all the tests.*
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<td>Relative displacement between Sleeper 2 and the asphalt surface (at the sleeper middle).</td>
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<td>Vertical surface accelerations</td>
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*POT 38 was unresponsive in all the tests.*
Table 3

*Steady state cyclic loads applied by Actuator 2*

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<th>Frequency [Hz]</th>
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<td>60.00</td>
<td>31.00</td>
<td>28.45</td>
<td>1.0</td>
</tr>
<tr>
<td>9</td>
<td>2.80</td>
<td>60.00</td>
<td>31.00</td>
<td>28.60</td>
<td>10.0</td>
</tr>
</tbody>
</table>

*Mean load = 0.5 × (Min load + Max load); Load amplitude = 0.5 × (Max load – Min load)*
### Table 4

*Simulated moving loads*

<table>
<thead>
<tr>
<th>Test No</th>
<th>Axle load ( P_{z,j} ) [kN]</th>
<th>Speed ( V ) [km/h]</th>
<th>Load type</th>
<th>Peak value of ( S_{x}^{\text{train}} ) [kN]</th>
<th>Time delay ( \Delta t ) [ms]</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>120</td>
<td>120</td>
<td>Single axle</td>
<td>40.87</td>
<td>18</td>
</tr>
<tr>
<td>11</td>
<td>120</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>200</td>
<td>120</td>
<td>Danish IC3 train</td>
<td>65.5</td>
<td>18</td>
</tr>
<tr>
<td>13</td>
<td>120</td>
<td></td>
<td>Danish IC3 train</td>
<td>39.3</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>200</td>
<td>200</td>
<td></td>
<td></td>
<td>10.8</td>
</tr>
</tbody>
</table>
Figure 4

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Figure 10

(a) Normalized spectral magnitude vs. Frequency (Hz) for Winkler Model and Actuator 2.

(b) Normalized spectral magnitude vs. Frequency (Hz) for Winkler Model and Actuator 2.