Damage investigation of thin flexible pavements to Longer Heavier Vehicle loading through instrumented road sections and numerical calculations

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ABSTRACT

Longer Heavier Vehicles provide an improvement in energy efficiency and environmental performance compared to traditional Heavy-Duty Vehicles. In Sweden, the maximum permissible vehicle gross weight has been increased from ~64 to ~74 tonnes without increasing the axle load limits. The consequence of this is investigated in this study. Response from two instrumented thin flexible pavements subjected to loading from three types of heavy vehicles (~64, ~68 and ~74 tonnes) has been measured and the recordings were compared with numerical calculations based on 2D multilayer elastic calculations. Pavement damage contribution by the three vehicles was thereafter investigated. As long as the number of axles is increased to compensate for the increased vehicle loading and dual wheels are used, ~74 tonnes vehicle are not more aggressive to the two thin pavement structures compared to the lighter vehicles with fewer axles but higher average axle loads and tyre pressure.

ARTICLE HISTORY

Received 9 July 2020
Accepted 2 March 2021

KEYWORDS

Pavement response; instrumented road section; falling weight deflectometer; multilayer linear-elastic theory; heavy vehicle

Introduction

The popularity of Longer Heavier Vehicles (LHVs) has increased worldwide in recent years. LHVs offer more efficient and environmentally friendly transportation of goods compared to conventional trucks (Christidis & Leduc, 2009; de Saxe et al., 2019). Most analyses on LHVs focus on the economic aspects of the transportation of goods and vehicle-related costs without considering the effects on the pavement structures (Christidis & Leduc, 2009; Ortega et al., 2014; Pålsson et al., 2017). A quantitative assessment of any pavement damage related to the increase in gross vehicle weight (GVW) magnitude could assist in predicting more accurately the pavement lifetime and the pavement-related economic costs. Road structures are usually designed for the heavy traffic that is expected in the forthcoming 20 years. Introducing new truck configurations that were not considered in the design procedure of the road structure requires an evaluation of the impact of the new load combination for the preservation of the road assets.

Empirical methods based on the long-term experience have been traditionally used for the design of flexible pavements. Newer, mechanistic-empirical design methods capable of predicting the behaviour of pavements more accurately are being developed and implemented worldwide (ARA Inc., 2004). Their development requires accurate initial assumptions, reliable data collection, method validation and calibration. The development of new pavement design methods is typically supported...
by full-scale road tests and deflection measurements (Pereira & Pais, 2017). Full-scale field testing by instrumented road sections has been proven to be a reliable pavement structural performance data collection method (Blanc, Hornych, et al., 2019; Chadbourn et al., 1997; Gusfeldt & Dempwolff, 1967; Hicks & Finn, 1970; Swett et al., 2008; Terrell & Krukar, 1970; Timm, 2009; Ullidtz, 2002). Instrumentation typically consists of sensors measuring mechanical stresses and strains and sensors measuring the impact of environmental variables, mainly temperature and moisture. The collected data can be used to validate various numerical models (Al-Qadi et al., 2004; Li et al., 2016), to evaluate the performance of the structure in-situ (Hugo & Epps-Martin, 2004; Solanki et al., 2009), to monitor the long-term behaviour of pavement structures (Blanc, Hornych, et al., 2019), to evaluate the performance of new materials (Blanc, Chailleux, et al., 2019), and to facilitate maintenance operations (Gaborit et al., 2014).

Multilayer elastic theory (MLET) based calculations are integrated into several mechanistic-empirical design guidelines. MLET assumes that the layers are isotropic and homogenous, circular loading is applied, and the material extends infinitely in the horizontal direction (Erlingsson & Ahmed, 2013; Huang, 2004). Other approaches are available such as the finite element (FE) method. However, a much larger computational time is required which may not be practical for pavement design practice (Loulizi et al., 2006), and the differences in the resulting response values obtained by both methods are negligible (Saevarsdottir & Erlingsson, 2015).

Historically, LHVs have been permitted to operate in Sweden (Erlingsson et al., 2018). Since 1996 and up to the beginning of 2015, the maximum GVW of 60 tonnes has been permitted. In 2015, the GVW was increased to 64 tonnes. In 2018, The Swedish Transport Agency decided to further increase the maximum GVW to 74 tonnes on part of the road network, planning to allow it on the entire road network if studies and experience indicate that the impact of the LHVs on traffic safety, economy and the infrastructure augur well. As a part of testing, two thin in-service pavement structures were instrumented in order to get a direct measurement of the impact of LHV on the road structures (Erlingsson & Carlsson, 2018). The structures were subjected to falling weight deflectometer (FWD) loading and three different LHVs (with GVW of approximately 64, 68 and 74 tonnes). In this study, the FWD and LHV mechanical response have been evaluated. The objectives were to quantify the effect of the new 74 t LHV in comparison with the previously allowed 64 tonne LHV. Furthermore, an intermediate LHV of 68 tonnes with a high number of single tyre axles was investigated for comparison.

**The test road site**

Two in-service thin flexible pavement structures located in the Piteå municipality in the northern part of Sweden along the roads Lv373 and Lv515 were instrumented in the autumn of 2017. Both road structures were old and have had some maintenance history. The site on Lv373 is located east of the Långträsk village at N65.33741, E20.4675, while the site on Lv515 is located south of the intersection between Lv373 and Lv515 at N65.31813, E20.29687. The test sites are located 10 km away from each other. Their geographical location is shown in Figure 1. The filled square shows the location of the BWIM station and the filled triangles show the locations of the two weather stations. A bridge weigh-in-motion (BWIM) station is located on the Lv373 in the vicinity of the test sites across Lake Storlångträsk. A meteorological station is located approximately 15 km north of the test sites and the Swedish transport administration operates a weather station at 8 km distance west from the Lv373 site. Both sites are located in a relatively flat marshland type terrain. The test site at Lv373 is 6.5 m wide and has an annual average daily traffic (AADT) of about 630 vehicles, while the test site at Lv515 is 6 m wide and has an AADT of around 100 vehicles.

The installation procedure in the existing roads was identical at both sites (Erlingsson & Carlsson, 2018). The asphalt concrete was removed along a 20 m long stretch along the lane. Thereafter two excavations 1.2 m wide were made at 4 m distance from each other down to 1.4 m depth. The excavated material was stored layer by layer. All existing layer thicknesses were determined during the excavation process. The sensor installation was performed from bottom to top, while the removed material was placed subsequently in their original position and compacted with the unbound base
course. Finally, a new asphalt concrete (AC) layer was repaved. The reason for two trenches was to get two set-ups of the instrumentation for both sites.

The instrumentation for the mechanical response measurements included asphalt strain gauges (ASG) to obtain the transversal and longitudinal strain at the bottom of the asphalt layer, strain measuring unit coils (ε-MU) to measure the vertical strains in the structure, and soil pressure cells (SPC) to measure the vertical stresses in the granular layers. The ε-MU coils were installed in one side of the vertical walls of the excavation trenches along a vertical line in the outer wheel path. The SPCs were installed on the other side of the vertical excavation walls, also in the outer wheel path. The ASGs were finally placed on top of the base course material and the asphalt layer was placed in position. The ε-MU coils and the SPCs were therefore installed inside the excavation trench while the ASGs were installed on top of the unbound base course outside the excavated part. A hand vibrator compaction equipment was used in 10 cm thick layers when refilling the trenches. A normal road construction equipment was used to place the AC layer. The installation procedure of the instrumentation for the cross-section of road Lv515 is illustrated below in Figure 2. The same installation procedure was implemented in the case of road Lv373.

After the installation of the pavement mechanical sensors, the installation of the instrumentation for the monitoring of the climatic variables was carried out. It consisted of three temperature sensors installed in the asphalt layer, a frost rod (VTI Tjäl 2004) made of 41 temperature sensors placed on a 200 cm log rod with 5 cm interval spacing to measure the temperature distribution in a vertical cross-section, and five time-domain reflectometer (TDR) sensors installed on a 200 long cm PVC rod to measure the moisture content in the granular layers and the subgrade. Figure 3 shows the layer thicknesses and material types at both sites with a schematic view of the placement of the mechanical instrumentation.

As seen in Figure 3 the test site on road Lv515 is a very thin and simple structure with only a 4 cm AC layer on top of a 15 cm granular base resting on a sandy subgrade. The site on road Lv373 is in an old part of the road that has been overlaid two times. Originally this was a cut-back asphalt concrete road that has since been rebuilt by placing a surface treatment layer and unbound base course on top of it. Later a new 47 cm thick pavement structure was added, consisting of two AC layers and an unbound base course and subbase layer.

Falling weight deflectometer (FWD) backcalculations

Falling weight deflectometer (FWD) measurements were performed for both pavement structures to obtain an estimation of the mechanical properties of the layers by backcalculating the stiffness.
The FWD measurements were performed at three different load levels, 30, 50, and 65kN. The values obtained from the measurements at 50 kN loading were averaged and compared to deflection bowls obtained from MLET backcalculations. An iterative error minimisation algorithm was implemented for
the backcalculation procedure. Upper and lower boundaries for the iteration were set for the stiffness values of each layer based on catalogue values with a tolerance of ±15%. The root mean square error (RMSE) was calculated comparing the measurements against the calculated deflection values for the assumed stiffness values at each step of the iteration. Thereafter a new set of layer stiffness parameters within the boundaries was tested. The parameters giving the minimum RMSE were then reported and thereafter used for comparison of the calculated and observed deflection bowls from the 30 and 65 kN load levels to ensure a good fit for a broad range of load levels.

As the FWD measurements were made during two consecutive dates, slightly different temperatures of the AC layers were observed. The stiffness of the AC layer was therefore iterated in the ±15% range to the value obtained by the following formula

$$E_T = E_{T, \text{ref}} \cdot e^{-b(T-T_{\text{ref}})}$$

where $E_T$ is the calculated stiffness of the AC at the target temperature $T$, $T_{\text{ref}}$ is the reference temperature (10°C), $E_{T, \text{ref}}$ is the asphalt concrete stiffness 6500 MPa at 10°C, and $b$ is a regression constant of 0.065 (Erlingsson, 2012). The resulting stiffness values obtained from the backcalculation procedure are shown in Table 1 for the cross-section at road Lv515 and in Table 2 for the cross-section at road Lv373.

At the test site Lv373 the temperature during the FWD measurements varied between 13°C and 16°C during the testing. At Lv515 the temperature varied between 15°C and 18°C, therefore a slightly lower stiffness modulus was backcalculated for the AC layer at Lv515. The recorded volumetric moisture content values at the time of measurement for Lv515 were 9.12% at 36 cm depth, 7.59% at 76 cm depth, 9.86% at 116 cm depth, 11.94% at 156 cm depth, and 5.40% at 196 cm depth. For Lv313, the recorded volumetric moisture content was 10.93% at 36 cm, 13.71% at 76 cm depth, 18.31% at 116 cm depth, 47.64% at 156 cm depth, and 37.69% at 196 cm depth. No precipitation occurred during the measurement campaign.

The FWD measurements revealed that the observed deflection bowls were larger within the trenches than outside, results attributed to the differences in compaction. Thus in all analyses here, stiffness values within the trenches were used for the modelling of the eMU coils and SPCs, whereas the values outside the trenches were used when the ASGs are modelled.

| Table 1. Backcalculated layer stiffness for the test site at road Lv515. |
| --- |
| Layer | Thickness [cm] | Outside the trench | Inside the trench |
| Asphalt Concrete | 4 | 5166 | 5166 |
| Base Course | 15 | 230 | 150 |
| Replaced Subgrade | 131 | 105 | 80 |
| Subgrade | ∞ | 105 | 105 |

| Table 2. Backcalculated layer stiffness’s for the test site at road Lv373. |
| --- |
| Layer | Thickness [cm] | Outside the trench | Inside the trench |
| Asphalt Concrete | 10 | 5883 | 5883 |
| Base Course | 12 | 450 | 280 |
| Subbase | 26 | 320 | 150 |
| Base Course | 12 | 180 | 140 |
| Tarmac | 13 | 120 | 120 |
| Subbase | 30 | 160 | 90 |
| Natural Gravel | 15 | 150 | 80 |
| Subgrade | ∞ | 73 | 73 |
The observed deflections and the calculated deflection bowls using the ERAPave software (Erlingsson & Ahmed, 2013) and the stiffness values from Tables 1 and 2 are shown in Figure 4 for all three load levels (30, 50 and 65 kN).

The stiffnesses were validated by comparing the observed registrations of the induced vertical strain of the $\epsilon$MU coils from the 50 kN FWD measurements with the MLET calculations. The results are shown in Figure 5. Multiple FWD measurements were carried out for all load levels at both locations on top of each sensor group. The figure shows the average value of the 50 kN measurement over the depth intervals and the error bars show one standard deviation.

As shown in Figure 5, the values of vertical strain measured by the $\epsilon$MU coils were relatively similar to the MLET-based calculated values except lower registrations were observed in the subbase layer in the Lv373 test site over the depth range 25–39 cm. The reason for this is probably a malfunction of the sensor. A similar accuracy was obtained for the 30 and 65 kN loading. Since the degree of the fit was considered acceptable, a linear elastic material model was used for all layers of both structures in the response calculations for the heavy vehicle loadings.

The heavy vehicle testing

The responses of three different LHV with a gross weight of approximately 74 tonnes (LHV1), 64 tonnes (LHV2), and 68 tonnes (LHV3) were measured to assess the effect the vehicles imposed on the two structures as they drove over the test sections with a speed of 60 km/h. The vehicle drivers were instructed to drive in such a manner that the outer wheel would follow the wheel path. It is therefore assumed in the calculations that the vehicles passed directly over the top of the sensors which

![Figure 4. Deflection bowls at (a) 30 kN, (b) 50 kN and (c) 65 kN for the test site at road Lv515 and (d) 30 kN, (e) 50 kN and (f) 65 kN for the test site at road Lv373.](image-url)
were marked on the surface, although minor deviations in position due to lateral wander might have occurred.

Images of the vehicles directly before entering the test site Lv515 are shown in Figure 6. The yellow marks on the road surface in front of the vehicles were used to mark the position of the instrumentation in the structure which is required for accurate positioning of the loading placement. The schematics of the half-axial configurations of the three LHVs are shown in Figure 7 where distances between axles are given in centimetres. The axles of the vehicles have been divided into four groups and a number has been assigned within the groups, as a naming convention. The axle loading and tyre pressure are further given in Tables 3 and 4. The tyre pressure was measured by a pressure meter, and axle loads were obtained by static weighing of each axle of the vehicles. The pavement response measurements at the two sites were performed on two consecutive dates. The same vehicles were used both days but they were slightly differently loaded each day, therefore there is a difference in axle loads but not in tyre pressures.

LHV1 had 9 axles, total weight approximately 74 tonnes and a c/c distance between the centre of the first and last tyre was 20.50 m. LHV2 had 7 axles with a total weight of approximately 64 tonnes and a length of 19.74 m. Finally, LHV3 had 8 axles, total weight approximately 68 tonnes and a 20.80 m distance between the centre of the first and last tyre.

There were noticeable differences in tyre pressures, axial load magnitudes and configuration between the three vehicles. Overall, LHV1 had the lowest variation in axle loads and the lowest average axle load. It had further generally the lowest tyre pressures. LHV2 had the lowest number of axles and the highest average axle load but high tyre pressures in the trailer tyres. LHV3 had the largest single axle load but the average axle load was lower than LHV2. Furthermore, LHV3 had single tyre configurations on all multiple axles except for the front-rear driving axle of the tractor where dual tyre configuration was used.
Table 3. Tyre pressures $p$ and axle load levels $F$ for the heavy vehicles during the measurements at the test site on road Lv515.

| Axle ID | Type | $p$ [kPa] | $F$ [kN] | Type | $p$ [kPa] | $F$ [kN] | Type | $p$ [kPa] | $F$ [kN] |
|---------|------|-----------|----------|------|-----------|----------|------|-----------|----------|
| A1-1    | S    | 840       | 83.36    | S    | 850       | 74.53    | S    | 890       | 93.16    |
| A2-1    | D    | 750       | 83.85    | D    | 700       | 93.65    | D    | 700       | 114.25   |
| A2-2    | D    | 750       | 82.38    | D    | 700       | 91.20    | D    | 700       | 114.25   |
| A2-3    | S    | 840       | 65.70    | –     | –         | –        | –    | –         | –        |
| A3-1    | D    | 850       | 87.77    | D    | 900       | 91.20    | S    | 870       | 94.14    |
| A3-2    | D    | 850       | 90.22    | D    | 900       | 89.73    | S    | 860       | 95.61    |
| A4-1    | D    | 850       | 78.45    | D    | 900       | 101.50   | S    | 870       | 69.14    |
| A4-2    | D    | 850       | 79.43    | D    | 900       | 100.03   | S    | 890       | 72.08    |
| A4-3    | D    | 850       | 80.41    | –     | –         | –        | S    | 890       | 74.53    |
| $\Sigma F$ |       | 731.58   | 641.85   |       |           |          |       | 680.58    |          |

Note: Type S and Type D stand for single and dual tyres, respectively.

Table 4. Tyre pressures $p$ and axial load levels $F$ for the heavy vehicles during the measurements on road Lv373.

| Axle ID | Type | $p$ [kPa] | $F$ [kN] | Type | $p$ [kPa] | $F$ [kN] | Type | $p$ [kPa] | $F$ [kN] |
|---------|------|-----------|----------|------|-----------|----------|------|-----------|----------|
| A1-1    | S    | 840       | 79.43    | S    | 850       | 75.02    | S    | 890       | 93.16    |
| A2-1    | D    | 750       | 84.83    | S    | 850       | 96.60    | D    | 700       | 112.78   |
| A2-2    | D    | 750       | 82.38    | D    | 700       | 88.75    | D    | 700       | 112.78   |
| A2-3    | S    | 840       | 65.70    | –     | –         | –        | –    | –         | –        |
| A3-1    | D    | 850       | 83.85    | D    | 900       | 95.61    | S    | 870       | 91.20    |
| A3-2    | D    | 850       | 86.30    | D    | 900       | 94.14    | S    | 860       | 97.58    |
| A4-1    | D    | 850       | 79.43    | D    | 900       | 96.11    | S    | 870       | 68.16    |
| A4-2    | D    | 850       | 76.00    | D    | 900       | 94.14    | S    | 890       | 69.14    |
| A4-3    | D    | 850       | 80.90    | –     | –         | –        | S    | 890       | 68.16    |
| $\Sigma F$ |       | 718.83   | 640.37   |       |           |          |       | 666.85    |          |

Note: Type S and Type D stand for single and dual tyres, respectively.

**Response modelling**

Multilayer elastic theory-based software ERAPave (Erlingsson & Ahmed, 2013) was used for the response modelling. The stiffness values obtained from the FWD backcalculation process were used as inputs in the modelling process. Linear elasticity was assumed for all the layers on both road cross-sections. All axle contact areas were assumed to be circular and the tyre pressure was assumed...
to be constant at 850kPa, which is close to the average tyre pressure of all tyres. Typical comparisons
between the measured signals and the MLET simulated pavement responses generated by ERAPave
are shown in Figures 8–10 for LHV1, 2 and 3, respectively.

The best fit between the measured and the calculated values was observed for the longitudinal
ASG, and the SPC sensors, (a), (b) and (e), (f) in Figures 8–10. The largest deviations between the
sensor observations and the calculations were associated with the transversal strain gauges (ASG) at
the bottom of the AC layer. The transversal tensile strain measurements were highly sensitive to the
positioning of the vehicles and slight lateral deviations as the tyres passed over the sensors led to
inaccurately recorded values, typically providing lower registrations than the expected (calculated)
values.

Figure 8. Sensor response registrations (black line) compared to MLET calculations (red dotted line) for (a) and (b) ASG longitudinal,
(c) and (d) ASG transversal, (e) and (f) SPC, and (g) and (h) MU coils. Sensor registrations from road LvS15 are shown on (a), (c), (e)
and (g) and from road Lv373 on (b), (d), (f) and (h). All registrations are from LHV1.
Figure 9. Sensor response registrations (black line) compared to MLET calculations (red dotted line) for (a) and (b) ASG longitudinal, (c) and (d) ASG transversal, (e) and (f) SPC, and (g) and (h) $\epsilon$ MU coils. Sensor registrations from road Lv515 are shown on (a), (c), (e) and (g) and from road Lv373 on (b), (d), (f) and (h). All registrations are from LHV2.

The recordings from the $\epsilon$ MU sensors were noisy and mainly provided the peak values after a noise filtering process; hence only measured peaks are shown along with the calculated curve for the vehicle response. It should be noted that recordings from two deepest $\epsilon$ MU sensors in the Lv373 structure were omitted as they gave unrealistic values.

Figure 11 shows all calculated peak values against all measured peak values for both structures and all four sensor types. The total number of observed data points is given in the figure as well as the coefficient of determination $R^2$ which provides the degree of accuracy of the modelling approach.

In the case of longitudinal tensile strain the agreement between the calculated strain and the ASG recordings is quite acceptable with $R^2 = 0.950$. The points for the two test sites are clustered together since recordings were made by sensors located at the same depth for each structure. The strain values
Figure 10. Sensor response registrations (black line) compared to MLET calculations (red dotted line) for (a) and (b) ASG longitudinal, (c) and (d) ASG transversal, (e) and (f) SPC, and (g) and (h) e-MU coils. Sensor registrations from road Lv515 are shown on (a), (c), (e) and (g) and from road Lv373 on (b), (d), (f) and (h). All registrations are from LHV3.

were higher at Lv515 as the AC layer is thinner (4 and 11 cm depth, respectively). The variations in the registered sensor values are due to different tyre pressures (700–920 kPa) and axle configurations (single vs. dual tyres) and to some extent differences in axle load level.

The degree of accuracy between recordings obtained from transversal ASG sensors and calculations was lower due to the difficulties in maintaining the correct transversal position of the vehicle. The measured strains were generally lower than the calculated strains, as shown in Figure 11(b). For the calculated value it is always assumed that the sensor is under the middle of the tyre for single tyre axles and between the two tyres for dual configurations and therefore providing the peak values. However, due to inaccurate positioning as the vehicles pass over the sensors, the measurements g
Figure 11. Validation of calculations against measurements for (a) ASG longitudinal tensile strain, (b) ASG transversal tensile strain, (c) vertical stress from SPCs, and (d) vertical strain from $\epsilon$ MU coils. Green dots are from the Lv515 structure and yellow dots from the Lv373. Data from all axles and sub-axles from all the three LHV are included.

frequently gave lower values. Calculations show that only a few centimetres of lateral movement had a great impact on the tensile strain observations and it is difficult even for experienced drivers to keep that accuracy at a speed of 60 km/h on a narrow road. The degree of prediction accuracy for the SPC sensors was relatively high ($R^2 = 0.917$), however, and the points were dispersed equally around the equality line on the chart. The SPC sensors were located at three depths providing stress levels over the entire range up to 120 kPa. The same was observed in the case of $\epsilon$ MU coils, though the accuracy here was slightly lower ($R^2 = 0.882$). The graph shows registrations from eight and seven sensors from structures Lv515 and Lv373, respectively.

**Damaging effect of different vehicles**

Two failure criteria were used in order to compare the damaging effect of the three different LHV’s to the pavement structures. They correspond to the allowable number of load repetitions to prevent
bottom-up fatigue cracking of the AC layer $N_f$ and limiting the accumulated permanent deformation in the subgrade $N_d$, respectively expressed as (Chatti et al., 2009; Hajek & Agarwal, 1990)

$$N_f = f_1 \varepsilon_t^{-n_1}$$

$$N_d = f_2 \varepsilon_v^{-n_2}$$

where $\varepsilon_t$ and $\varepsilon_v$ are the tensile strain at the bottom of the AC layer and the compressive strain on top of the subgrade respectively and $f_1, f_2$ and $n_1, n_2$ are material parameters.

By using Miner’s rule a damage factor $D_f$ with respect to bottom-up fatigue life for each axle group for the three LHV’s can be defined through the summation over all sub-axes in the axle group as

$$D_{f\text{LHV}kl} = \sum_{j=1}^{j_{\text{end}}} \frac{1}{f_1 \varepsilon_t^{-n_1}}$$

where $j$ stands for the number of sub-axes in the axle group and $j_{\text{end}}$ is the total number of sub-axes in the axle group. Further is $l = 1, 2, 3, 4$ the axle group number and $k = 1, 2, 3$ stands for the three LHV under consideration. A similar expression for a damage factor $D_d$ can be received for the subgrade deformation criteria by replacing the tensile strain $\varepsilon_t$ with the vertical compression strain $\varepsilon_v$ and $f_1$ and $n_1$ with $f_2$ and $n_2$, respectively. Now we can define in a similar manner a relative damage factor $D_{rf}$ for the axle group with respect to fatigue cracking using the four axle groups of LHV2 (∼64 t vehicle) as normalisation as

$$D_{rf\text{LHV}kl} = \sum_{j=1}^{j_{\text{end}}} \frac{\varepsilon_t^n}{f_1 \varepsilon_t^{-n_1}}$$

Again, a similar expression $D_{rd}$ can be received for the subgrade deformation criteria.

Calculated critical strain values using the values from Tables 3 and 4 for the axle loads and configuration along with the tyre pressures and distances between axles from Figure 7 are given in Figure 12. These values were further used to estimate the relative damage factor of the different axle groups of the three vehicles as shown in Tables 5 and 6. Here the exponent $n = n_1 = n_2 = 4$ was used for both criteria. The depth for the subgrade criteria was assumed as 118 cm for the Lv373 structure but a 60 cm depth was assumed for the Lv515 structure as the structure is unusually thin and a 19 cm depth was not considered representative for the Swedish conditions for a low volume road in a cold region (see Figure 3).

| Axle group | $D_{rf-ag}$ | $D_{rd-ag}$ | $D_{rf-ag}$ | $D_{rd-ag}$ | $D_{rf-ag}$ | $D_{rd-ag}$ |
|------------|-------------|-------------|-------------|-------------|-------------|-------------|
| 1          | 0.970       | 1.529       | 1.000       | 1.000       | 1.000       | 1.000       |
| 2          | 3.047       | 0.999       | 1.000       | 1.000       | 1.000       | 1.000       |
| 3          | 0.784       | 0.931       | 1.000       | 1.000       | 1.000       | 1.000       |
| 4          | 1.093       | 0.604       | 1.000       | 1.000       | 2.674       | 1.091       |

Obviously there is a large variation in the damaging effect of the different axle groups. It is further difficult to compare the different axle groups together as the combined effect of tyre pressures, sub-axle loads and configurations have an impact on the distress contribution and therefore the damaging effect. By looking at the difference in the damaging effect of LHV1 compared to LHV2 there are two things that stand out. The effect on the steering axle of the LHV1 on the subgrade criteria is about 50% higher than for the LHV2. This is attributed to the higher axle loading of that axle. As axle group 2 of LHV1 is a triple axle, of which the last sub-axle has a single tyre configuration, it had a significant impact
Figure 12. Calculated peak strains at (a) and (c) the bottom of the asphalt layer and (b) and (d) the top of subgrade for the different axles of three LHV's. Values from road Lv515 are shown on (a) and (b) and from road Lv373 on (c) and (d).

Table 6. Relative damage factors $D_{rf}$ and $D_{rd}$ for each axial group for Lv373.

| Axle group | $D_{rf-ag}$ | $D_{rd-ag}$ | $D_{rf-ag}$ | $D_{rd-ag}$ | $D_{rf-ag}$ | $D_{rd-ag}$ |
|------------|-------------|-------------|-------------|-------------|-------------|-------------|
| 1          | 1.181       | 1.541       | 1.000       | 1.000       | 1.636       | 2.456       |
| 2          | 4.221       | 0.997       | 1.000       | 1.000       | 4.937       | 1.136       |
| 3          | 0.828       | 0.952       | 1.000       | 1.000       | 8.534       | 0.931       |
| 4          | 0.856       | 1.135       | 1.000       | 1.000       | 6.669       | 0.933       |

on the LHV’s 1 2nd axle group on the fatigue cracking life of the structure compared to the comparable axle group of LHV 2. The main contributor here was the third sub-axle of LHV1 as well as the lower tyre pressure of LHV 2. This effect could be mitigated by replacing the single tyre of LHV1 with dual tyres and distributing the loading evenly over all sub-axles. Finally, the LHV3 had a much larger damaging impact on the two pavement structures than both LHV1 and LHV2 except for axle groups 3 and 4 for the subgrade criteria of Lv373. The main reason was that single tyre configuration was mainly used on that vehicle with relatively high tyre pressure.

In a similar manner, a damage factor with respect to bottom-up fatigue life for each vehicle can be defined through the summation over all axle groups for the respective vehicle. In addition, normalizing as before with the LHV 2 then revealed a relative damage factor for the entire vehicles as

$$D_{H_{LHVk}} = \frac{\sum_{j=1}^{\text{end}} \sum_{j=1}^{\text{end}} \varepsilon_{nLHVx_{ij}} \varepsilon_{T_{LHV2i}j}^{n}}{\sum_{j=1}^{\text{end}} \sum_{j=1}^{\text{end}} \varepsilon_{nLHVx_{ij}}^{n}}$$

Again, a similar expression can be used for the subgrade criteria. Results where values from Tables 5 and 6 have been used are given in Table 7. The exponent $n = n_1 = n_2 = 4$ was used as before.

As shown in Table 7, the damaging effect of LHV1 was about 22% and 39% higher compared to the LHV2 with respect to bottom-up fatigue cracking for the two structures. The subgrade criteria were highly dependent of the depth where it was applied. At Lv515 the LHV1 courses caused about 8% less damage than the LHV2, whereas at Lv373 the LHV2 courses about 6% more damage. This is attributed
Table 7. Vehicle relative damage factor $D_{rf}$ and $D_{rd}$ at both test sites.

| Vehicle | Road Lv515 | | Road Lv373 | |
|---------|------------|---|------------|---|
|         | $D_{rf}$  | $D_{rd}$ | $D_{rf}$  | $D_{rd}$ |
| LHV1 (∼74 t) | 1.216 | 0.921 | 1.390 | 1.055 |
| LHV2 (∼64 t) | 1.000 | 1.000 | 1.000 | 1.000 |
| LHV3 (∼68 t) | 2.109 | 1.520 | 4.283 | 1.072 |

to the fact that axle group 4 of LHV1 is a tridem axle group and heavier that the corresponding tandem axle group of LHV2. Further, the subgrade was reached at considerable depth (118 cm) at Lv373 but the effect of superposition of loading from closely spaced axles increased with depth. Therefore this is seen in structure Lv373 but not in Lv515 where the subgrade criteria were applied at a 60 cm depth.

The LHV3 in all cases had the highest damage effect and for the bottom-up fatigue cracking the effect was quite detrimental. The main reason for this was that all sub-axles except one had a single tyre configuration with relatively high tyre pressure.

Conclusions

GVW up to 64 tonnes have been allowed on the road network in Sweden since 2015. In 2018 the allowed load limits were increased to 74 tonnes on part of the road network. In this paper, the response of two in-service instrumented thin flexible pavement structures subjected to loading by FWD and three types of LHV (∼74, ∼64 and ∼68 tonnes, respectively) were analysed in order to investigate the effect of LHV with different axle configurations, axle loads, and tyre pressures have on the damage accumulation. FWD measurements and 2D axisymmetric MLET-based backcalculation software were performed to obtain the stiffness of the pavement layers. The obtained stiffness values were used in the response modelling of the LHV loading. By comparison between the measured and the calculated response values, the modelling strategy was proven to be accurate. The damaging effect of the different vehicles was compared, first by axle groups and thereafter for the three vehicles. Two criteria were examined: the bottom-up criteria of the AC layer and the permanent deformation criteria of the subgrade.

The LHV1 (∼74 tonnes vehicle) resulted in being more aggressive to the two pavements than the previously allowed 64 tonne vehicles (LHV2). It was mainly the axle group 2 that caused a high contribution to the fatigue life reduction of the two structures. This could be mitigated by replacing the last sub-axle single tyre of LHV1 with dual tyres and distributing the loading evenly on all sub-axles in the axle group.

There was also a 50% higher impact of the steering axle of the LHV1 on the subgrade criteria compared to LHV2 due to the higher axle loading. The fourth axle group of LHV1 was a triple axle and also had a great impact on the subgrade criteria in the deeper layers as seen by their effect superimposed with depth at the test site on structure Lv373. This was seen as a 13.5% higher damage effect on the subgrade for LV373 than at LHV1. By applying the criteria at even greater depth this damaging difference between the two vehicles would have increased even further (not shown here).

The highest damaging factor values were calculated for LHV3 (68 tonne vehicle) due to the high number of axles with single tyres and relatively high tyre pressure, and hence the loading applied over a smaller contact area resulting in high impact on the pavement structures.

This was a limited study including only two pavement structures and three LHV. Other combinations of axle load configurations, load distribution and tyre pressures might reveal different results. Furthermore, only two criteria are looked at, fatigue cracking and rutting accumulation in the subgrade. Other distress mechanisms such as accumulation of plastic deformation in the AC layer or the granular materials were not part of this study.
Acknowledgements

The authors would like to thank Trafikverket (Swedish Transport Administration) for their financial support as well as Håkan Carlsson and Mikael Bergqvist for carrying out the field testing.

Disclosure statement

No potential conflict of interest was reported by the author(s).

Funding

This work was supported by Trafikverket.

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