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INVESTIGATION OF LATERAL TRACK PANEL STIFFNESS THROUGH FOUR-POINT BENDING TESTS

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ABSTRACT
Lateral track stability in ballasted tracks is mainly given through lateral ballast resistance and track panel stiffness. Nowadays this becomes even more important due to widely spread application of continuously welded rails. The lateral ballast resistance is hardly to control, changes over time and is influenced by many environmental factors. On the other hand, a high lateral track panel stiffness was found to be beneficial for curve breathing and buckling. Despite this importance, the empirical knowledge is poor and a general overview is missing in literature. To overcome this issue, four-point bending tests were performed on all the rail tie combinations currently in use in Switzerland. Four-point bending tests were chosen due to their ability to estimate bending and shear deformation separately. The stiffness was calculated from the measured force and displacement data based on the Timoshenko beam theory. The differences in stiffness are discussed based on the mechanical behavior of the track panel. Based on the results high and low stiffness superstructure types can be distinguished. It was found that the lateral track stiffness in case of coupled concrete (ZSX) ties can be increased up to a bending stiffness of $84600 \text{ kNm}^2$ and a shear stiffness of $10435 \text{ kN}$ compared to a bending stiffness of $2440 \text{ kNm}^2$ and a shear stiffness of $3773 \text{ kN}$ as present for conventional concrete ties. The limitations in terms of stiffness of every superstructure type as well as possible improvements are given.

Keywords: Lateral track panel stiffness, Lateral resistance, Railway engineering, Track mechanics, Curve breathing, track stability, track buckling
INTRODUCTION

With the introduction of continuously welded railway tracks (CWR) around 1950 lateral track stiffness became more important to prevent curve breathing or buckling. The lateral ballast resistance and the track panel stiffness mainly influence the lateral track stability. While the former can hardly be controlled and influenced by many environmental factors and changes over the whole lifecycle of a track, the latter was found to have a beneficial effect on track breathing and buckling. Although it is easy to control, the empirical knowledge is poor.

The beneficial effect of lateral track resistance to avoid track buckling was firstly concluded by Meier (1). Many publications, for example Kish and Samavedam (2) and more recently Choi and Na (3) confirmed his findings. Furthermore, it was concluded theoretically by Zimmermann and Braess (4) that a high track panel stiffness has a positive effect on track breathing which was also confirmed experimentally by Braess et al. (5). Despite the importance of accurate estimation of the lateral track panel stiffness, the empirical knowledge is low.

Previous research was exclusively based on three point bending tests and mainly consisted of cross comparison of two different superstructure types (1, 6–10). Direct cross-comparison between different studies found in literature is not possible because they differ significantly in experimental setup and testing methods. Another drawback is that lateral track stiffness was commonly estimated based on three point bending tests, where shear deformation cannot be determined separately. Neglecting shear deformation as it holds for slender beams according to Bernoulli beam theory does not hold for frame-like structures such as track panels or shear buildings as discussed in more detail by for example Bouma (11). As shown in the present work, this becomes even more important for high stiffness types of superstructures such as ZSX ties, where shear deformation accounts for around 50% of total deformation.

This work aims to give a complete overview of the lateral track stiffness of all types of superstructures, which are currently in use in Switzerland. For that, four-point bending tests on 12 m long track panels are performed and the measured force displacement data is evaluated using Timoshenko beam theory to account for bending and shear deformations separately. The stiffness of every type of superstructure is given by means of shear and bending stiffness. Based on the results different types of superstructure are cross-compared to each other and clustered into high and low stiffness superstructure types.

EXPERIMENTAL INVESTIGATION AND METHODOLOGY

Experimental Setup

For the investigation, four-point bending tests were performed on a simply supported track panel with a total length of 12 m as shown in Figure 1. The track panel was lying on the ground for construction and testing. To avoid friction between the ground and the track panel and to enable free longitudinal and lateral motion every tie was vertically supported by two air cushions of type 6LTM-200-1 (DELU, Germany) with a load carrying capacity of 3750 kg each. The horizontal support was achieved by a rigid steel structure, fixed to the ground. The support and the load where applied directly to the ties, in order to ensure evenly distributed forces in both rails. This was done with a steel clamp connected to the ties through friction. The clamp could be adapted in width and height to account the different tie types and was designed for a maximum load of 20 kN.
FIGURE 1 Four-point bending test experimental setup.

Loading was applied through two hydraulic cylinders fixed to the same rigid structure and to the clamp on the tie respectively. The cylinders were powered by a manual hydraulic pump and the load was measured online with load cells directly connected to the cylinder. A linear variable differential transformer (LVDT) in the middle of the span and at the load transmission points measured the displacement continuously. An additional second discrete displacement measurement at each load level was done with the tachymeter based optical system QDaedalus as described in the following section. Therefore, 14 optical targets were deployed at the following locations:

- middle span,
- load transmission points,
- in the middle between the load transition points and the support and
- at the support.

For redundancy, this points were measured on both rails. The targets consist of concentric circles with a maximum outer diameter of approximately 40 mm. The tests were displacement-controlled based on the online LVDT measurements at the middle span. The load levels where chosen at a displacement of 0 mm, 5 mm, 10 mm and then every 10 mm up to either a maximal displacement of 100 mm or a maximal force of 20 kN. The track was unloaded after reaching a displacement of 5 mm and 30 mm, to see the plastic deformation. An exemplary loading curve is shown in Figure 2. At each displacement controlled load stage discrete QDaedalus measurements took place.

The superstructures were built accordingly to Swiss standard. The fasteners where tightened with the electric torque wrench Lösomat (Geodore, Germany) which allows to precisely define and monitor the torque applied to every screw. Tie spacing and their horizontal alignment were measured by hand with a maximum spacing tolerance of 5 mm to each side and an alignment tolerance of 2 mm laterally. For every superstructure type, the ties were first placed
on the ground with the right spacing and aligned properly. The rails were set in place and the fasteners tightened. The clamps for the support and for the load transmission were then mounted to the ties. After completing the construction, the track panel was lifted by air cushions and then the targets for the QDaedalus measurements as well as the LVDT’s were set in place. Data was recorded by means of continuous displacement and force from the LVDT and load cells respectively and 3D coordinates at each load stage from the QDaedalus measurement system.

QDaedalus
The system QDaedalus is developed at the Institute of Geodesy and Photogrammetry of ETH Zurich. It consists of hardware and software components and was firstly deployed for industrial metrology by Guillaume et al (12). The hardware part is composed by a commercial motorized Leica total station (for this project TS60 series) on which, a modified industrial CCD camera is plugged in place of the eyepiece, in a non-destructive way. A single QDaedalus system is capable of measuring automatically very precise horizontal directions and zenith angles (~0.5 mgon or 0.08 mm/10m) of simple targets. For this project, the targets consisted in circles printed on self-adhesive paper which could be detected and measured by a sub-pixel ellipse matching algorithm introduced by Guillaume et al. (13).

In order to be able to determine the absolute 3D position of the targets mounted on the experiment by triangulation, 2 QDaedalus systems (ST1, ST2) have been mounted on the end of the experiments as shown in Figure 2. Furthermore, 6 reference points have been fixed on rigid structures in order to ensure the scale and the absolute referencing of the coordinates system throughout the duration of the experiments.

A single epoch of measurements consisted in the acquisition of angular measurements, from ST1 and ST2 to all targets and all reference points. The duration of the measurements of a single station on 19 targets was approximately 1 minute. The 3D coordinates of all variable points were obtained using the 3D geodetic software TRINET+, Guillaume et al (14), by a 3D least-squares adjustment. The positions of both QDaedalus stations were assumed to be fix only during a single epoch. This generates that the coordinates of ST1 and ST2 are assumed to be variable for each epoch.

The empirical precision (1σ) obtained after adjustment are, for ST1 and ST2, of approximately 0.05 mm and 0.02 mm, for the horizontal and the vertical components, respectively. For the targets on the experiment, the precision (1σ) varies between 0.05-0.12 mm and 0.03-0.04 mm for the horizontal and the vertical components, respectively. In Figure 2, the empirical precision in the horizontal is represented by the 95% confidence ellipses (2.54σ) and shows that the position of ST1 and ST2 have been chosen to promote the precision of the targets in the y-coordinate direction.
Evaluation Methodology
The experiments were carried out as simply supported four-point bending tests (Figure 3). Displacement was measured continuously (LVDT) and at every discrete load stage (QDaedalus). The corresponding peak load to every load stage was extracted manually from the load cell data (Figure 2). The stiffness was calculated based on the QDaedalus displacement data and the according peak load for each load stage.

Possible support deformations were taken into account as rigid body motion of the track panel and subtracted from the measured displacement data. The values of both the inner and the outer rail were cross compared to each other and to the LVDT measurements to achieve redundancy in the measurements. If no reason for rejection was found, the mean of the displacement values at the inner and outer rail where taken. At the load transition points, the mean values of the force and the displacement were used for further calculation.
The bending stiffness $EI$ was calculated according to Timoshenko beam theory as:

$$EI = \frac{ab^2}{8} \cdot \frac{F}{w_c - w_F}.$$  \hspace{1cm} (1)

Where $a$ describes the distance between the support and the load transmission point, $b$ the distance between the load transmission points, $F$ the force, $w_c$ the displacement in the middle of the track and $w_F$ the displacement at the load transmission points (Figure 3). The according shear stiffness $GA_v$ can be described as:

$$GA_v = \frac{Fa}{w_c - \frac{Fa}{EI} \left( \frac{a^2}{3} + \frac{ab}{2} + \frac{b^2}{8} \right)}.$$  \hspace{1cm} (2)

The strongly nonlinear load displacement behavior of the track panel disables direct cross-comparison of the results. To overcome this issue, a characteristic stiffness value was defined at the turning point of the load displacement curve (Figure 2). This denotes the minimal stiffness value and is independent of the geometric parameters, span and maximal deflection. It is also not directly influenced by the fastening system which can lead to very high initial stiffness or high stiffness when the rail contacts the ribbons of the base plate. This value was further found to be representative due to dynamic loading and vibration acting on the superstructure during operation which could lead to a relaxation of the clamp force in the fastening system. Therefore, using the initial slope of the curve would overestimate the system stiffness. Taking into account the increasing slope after the turning point would likewise lead to an overrated stiffness, since this extent of deformations would widely exceed the serviceability of a real track.

**Investigated Superstructure Types**

All the superstructure types currently in use in Switzerland were tested. To get a better insight into the mechanical behavior of the track panels, parametric studies with tie distance and fastening system torque where performed. The following tie types were tested:

- Concrete ties B91 (Standard gauge)
- Concrete ties VöV-M2 (Meter gauge)
- Concrete turnout ties with ribbed base plates (CTS, also for guardrails, Standard and meter gauge)
- Heavy-Duty Concrete Ties (HDS, Standard gauge)
Steel ties with ribbed base plates, also for guardrails (Standard and meter gauge)
Steel Double ties (SDS, Meter Gauge)
Wood ties with ribbed base plates (Standard and meter gauge)
Y-shaped steel ties (Standard and meter gauge)
ZSX-Ties (Concrete, Standard gauge)

Additionally, the following configurations have been modified:
Rail profile (46E1 and 54E2)
Fastening type (with wood ties: Skl 12 and Kpo 3)
Torque moment of fastening
Number of fastening (with HD-ties: 2, 3 or 4 fastenings)
Tie distance (55, 60 or 65 cm)
Guard rails (with Concrete and Steel ties)

RESULTS
The first part of this section gives an overview of the characteristic bending and shear stiffness of the tested superstructure types. In the second part the load bearing behavior for wood, concrete and coupled concrete (ZSX) ties is shown in more detail. The results can be subdivided into high and low stiffness superstructure types (Figure 4). Most of the conventional rod tie tracks such as wood, steel and concrete with a bending stiffness below $30'000\, kN\cdot m^2$ can be assigned to low stiffness superstructure types. Some of the investigated superstructure types were not taken into account for better readability of Figure 4. These were HDS ties with only two fastening systems per side, which behave like conventional concrete ties and concrete turnout ties without guard rails, which are not relevant for practical applications and behave similar to wooden ties. The ratio between bending and shear stiffness is small for low system types of superstructure and the displacement is clearly dominated by bending deformation. Higher stiffness superstructures can be achieved by using ZSX, coupled steel and Y-Shaped ties which allow a much better coupling of the rails. Also the introduction of guard rails (GR) has a beneficial effect on the total stiffness of the superstructure. The ratio between bending and shear stiffness is usually higher for the high stiffness superstructure types, which leads to higher shear deformations compared to the total ones. The superstructure stiffness is mainly influenced by the fastening system, and therefore the coupling of both rails, while the rail profile has only small influence.
The tested wood ties had a characteristic bending stiffness of about $6133 \text{ kNm}^2$ and a shear stiffness of $1935 \text{ kN}$ (Figure 5) at a deformation of about $40 \text{ mm}$ (green). The details of the test are given in the header of the figure in the following form: Date, tie type, gauge, rail profile, fastening system, torque, tie spacing and geometry of experimental setup. The initial
slope of the curve (yellow) takes into account the much higher initial stiffness (Figure 5, left).

Rotation in the fastening system then decreases the stiffness until there is contact between the rail and the iron ribbon at a deflection of about 50 mm and the stiffness increases again. The bending and shear displacement are given in light and dark grey respectively.

For concrete ties our tests yield a characteristic bending stiffness of 2440 kNm² and a shear stiffness of 3773 kN, which are lower in comparison to the wooden ties (Figure 6). After an initial resistance, the stiffness drops and the track panel behaves almost linear elastically. Contact between the plastic guide plate and the rail base are less important as it can be seen at a displacement of about 80 mm.
Coupled concrete ties (ZSX) show a much higher stiffness where shear deformation accounts up for about 50 % of the total deformation (Figure 7). The test had to be terminated at a displacement of 15 mm since the testing apparatus did not allow us to further increase the force. The bending stiffness $EI = 84010 \, kNm^2$ and shear stiffness $GA_y = 10523 \, kN$ are much higher than the ones measured for conventional rod tie tracks. The behavior is almost linear elastic and the tangent stiffness differs only slightly from the initial one.
The tests showed that the load-displacement behavior of the track panel is not linear elastic. Different types of superstructure show plastic deformations accounting for up to 50% of the total deformation. This is mainly due to the joints between the rails and the ties, which are based on friction. Half of the deformation remains after unloading. In order to bring the track panel back into its initial non-deformed state, the screws of the fasteners have to be loosened. No plastic deformations occur in the individual components (rail, fastening system, tie).

**FIGURE 8 ZSX tie normal gauge.**

The experiments showed that the types of superstructure can be clustered into low and high stiffness types. The overall behavior is mainly influenced by fastening system performance. The shear deformation plays an important role, especially when it comes to high stiffness superstructure types. Conventional rod tie track performance is mainly influenced by the torsional stiffness of the fastening system as it was shown by applying different torques. The tie spacing and their material have only a little influence. This is also the case for the rail profile, where two competitive trends take place, one affected by the inertial properties of the rail itself (mainly for stiff track types), and the other affected by the shape of the rail base. For practical applications, the use of a small rail profile might be beneficial, especially because of the smaller resulting axial forces. Within the superstructure types of high panel stiffness one can further distinguish between frame ties like ZSX and coupled steel ties, where the overall performance is again given by the torsional stiffness of connected ties (lateral friction of fastening systems) and the Y-shaped steel ties where the longitudinal slipping resistance is the limiting factor. It is hard
to compare these stiffness values with the ones found in literature since information about characteristical values they used is missing. For a conventional concrete tie superstructure as shown in Figure 6, the bending stiffness was found to be $2440 \text{ kNm}^2$, which agrees with the values proposed by Stieber (12) were the bending stiffness for concrete ties is 2.1 times the stiffness of the single rails which yields an overall stiffness of $2630 \text{ kNm}^2$.

CONCLUSION

The superstructure types were clustered in high and low stiffness track panels. High stiffness means a bending stiffness of above $30000 \text{ kNm}^2$ as for Y-shaped steel ties. Based on the results stiff superstructure types are recommended within particular areas where buckling or breathing are likely to occur. Further optimization could be done by increasing longitudinal slipping resistance for Y-shaped ties and by increasing the fastening system stiffness for ZSX ties.

REFERENCES