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Adjustable fastener**

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1 **Corrective Countermeasure for Track Transition Zones in Railways: Adjustable** 2 **Fastener**

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5 **Abstract**

6 Transition zones in railway tracks are the locations with considerable variation in the vertical
7 stiffness of supporting structures. Typically, they are located near engineering structures, such as
8 bridges, culverts, tunnels and level crossings. In such locations, the variation of the vertical stiffness
9 and the differential track settlement result in amplification of the dynamic forces acting on the track.
10 This amplification contributes to the degradation process of ballast and subgrade, ultimately
11 resulting in the increase of maintenance costs.

12 The paper studies a corrective countermeasure that can mitigate the track degradation in
13 transition zones when differential settlement appears. The countermeasure is the adjustable rail
14 fastener and its working principle is to eliminate the gap under hanging sleepers by adjusting the
15 shims (height of the fastener). The adjustable fasteners are first tested on three transition zones,
16 wherein the adjusted heights of fasteners (accumulated voiding) are recorded after the 2-month and
17 5-month operation. The test results show the adjustable fasteners are effective to mitigate the track
18 degradation in the transition zones. The effect of the adjustable fasteners on the dynamic behaviour
19 of transition zones is analysed using the FE method. The results show that the adjustable fasteners
20 are effective to reduce the amplification of wheel forces, achieve a better stress distribution in
21 ballast, and decrease the normal stresses in rails in transition zones. Parametric studies are also
22 performed to study the applicability of the adjustable fasteners.

23 **Keywords:** Adjustable fasteners; Railway track modelling; Transition zone; Corrective
24 countermeasures; Finite element method.

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1. Introduction

Transition zones in railway tracks are locations with considerable variation in the vertical stiffness of supporting structures. Typically, they are located near engineering structures, such as bridges and slab tracks. An example of a typical transition zone is shown in Figure 1.

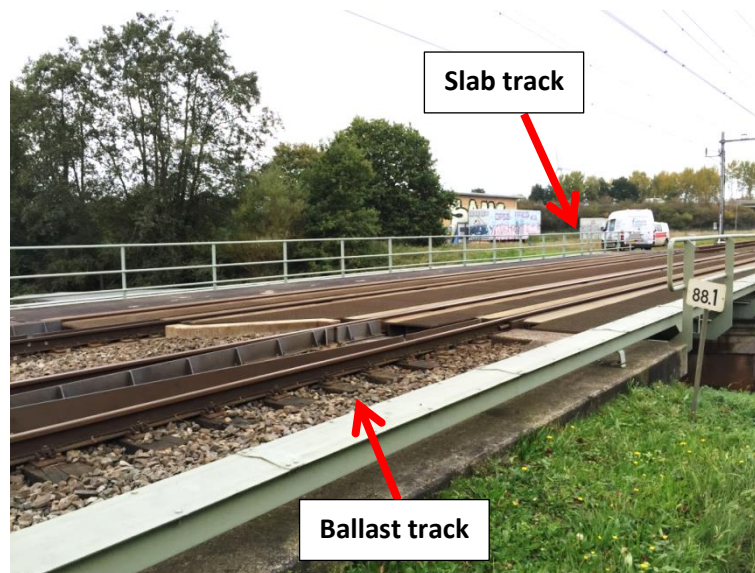


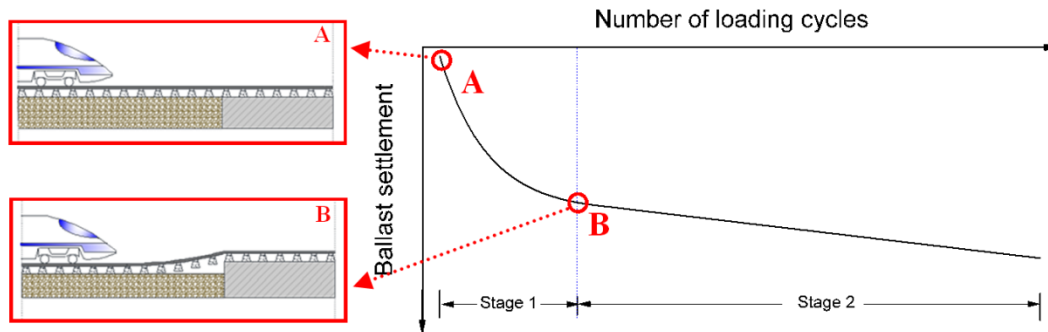
Figure 1. Track transition zones

In those locations, the variation of the vertical stiffness together with the differential settlement of tracks (when the foundation settles unevenly) results in amplification of dynamic forces, which contributes to the degradation of ballast and subgrade, ultimately resulting in deterioration of the vertical track geometry or even the damage of track components. To keep track transition zones in operation, more maintenance is required as compared to free tracks [1, 2]. For instance, in the Netherlands, the maintenance activities on the tracks in transition zones are performed up to 4-8 times more often than that on free tracks [3, 4]. Transition zones in the other countries of Europe and the US also require additional maintenance [5, 6].

Even though many countermeasures have been used in transition zones, severe track deterioration in transition zones is still often observed [7, 8]. In [9], three countermeasures including geocell, cement, and hot mix asphalt were applied on three similar transition zones. Compared to a plain transition zone (no countermeasure is applied) as a reference, it has been found that all countermeasures were not sufficient to reduce the settlement in the transition zones. In [10], the countermeasure, which uses an approaching slab linking the ballast track onto a concrete culvert, 'has exacerbated rather than mitigated the (transition) problem'. These findings are in agreement with [11], where the authors indicated that 'the problem of track degradation associated with stiffness variations is far from being solved'.

According to the settlement behaviour of ballast tracks [12-17], the track settlement process can be divided into two stages (as shown in Figure 2). Stage 1 is the rapid settlement process, caused by the volumetric compaction and abrasion of ballast particles. Stage 2 is the standard settlement process (until the end of the maintenance interval) caused by the frictional sliding of particles. The settlement of ballast, subballast, and subgrade in Stage 1 is: (1) fast, which happens only after few months; (2) large, accounts for approximately 50% of the total settlement in a maintenance interval;

1 (3) somewhat inevitable, which happens even though it was compacted. On the contrary to the large
2 settlement appearing in ballast tracks, the engineering structures barely settle, which creates a
3 considerable geometry irregularity (differential settlement). After the differential settlement appears
4 (corresponding to the beginning point of Stage 2, see Point B in Figure 2), one end of the rails is
5 settled together with the ballast track, while the other end is constrained by the engineering
6 structures, creating gaps under sleepers (also known as hanging sleepers or voiding) on the
7 embankment side.



8

9 Figure 2. Schematic permanent settlement curve of ballast as a function of loading cycles (only for
10 open ballast tracks), from [18].

11 Due to the existence of the gaps, the dynamic responses in transition zones are significantly
12 increased [19, 20]. For instance, a 1mm gap can increase the sleeper-ballast contact force in adjacent
13 locations by 70% [21]; and the 2mm gap can lead to 85% increase of wheel forces [19]. It should be
14 noted that the settlement curve in Figure 2 is only for open ballast tracks. This is because the
15 dynamic responses (e.g. wheel-rail interaction forces and ballast stress) are increased by the
16 differential settlement and alter the settlement curve (mainly in Stage 2). An example of the
17 transition zone with a large differential settlement is shown in Figure 3 [18].



18

19 Figure 3. Transition zone with a large differential settlement, from [18].

20 Therefore, the countermeasures can be categorized according to the settlement behaviour into
21 the preventive countermeasures and corrective countermeasures. The preventive countermeasures
22 are implemented during the construction prior to the track operation, while the corrective ones are
23 used when the track has already settled (the differential settlement is visible). The studies of the
24 transition zones with the perfect geometry, i.e. considering the beginning point of Stage 1 (Point A in
25 Figure 2), are more suitable for the preventive countermeasures, such as the studies in [8, 22-27];
26 while for the analysis of corrective countermeasures, the numerical models have to take the
27 differential settlement into account. The corrective countermeasures that can timely mitigate the

1 transition zone problems caused by differential settlement (at Point B in Figure 2) are required
2 further studies.

3 This study focuses on the corrective countermeasures for transition zones which should meet the
4 following requirements:

- 5 • The corrective operations should be performed in short track-possession windows
6 manually or by using small machines.
- 7 • The corrective countermeasures should be able to mitigate the track degradation in
8 transition zones.

9 The paper presents the experimental and numerical analysis of a corrective countermeasure - the
10 adjustable fastener. The adjustable fastener intends to fill the partial gap between a sleeper and
11 ballast by shims, whereas it prevents a large operation such as tamping. Even though (unloaded)
12 track alignment is not restored, the hanging sleepers in the vicinity of engineering structures are
13 eliminated, which can slow down the track degradation in transition zones.

14 The paper is organised as follows. The preventive and corrective countermeasures for transition
15 zones are reviewed in Section 2, including the introduction of the adjustable fastener. The
16 measurement results of three transition zones with the adjustable fasteners are discussed in Section
17 3. In Section 4, the dynamic behaviour of the transition zone (with differential settlement) with the
18 adjustable fasteners is analysed using FE method. Finally, conclusions are given in Section 5.

19 **2. Countermeasures for transition zones**

20 The countermeasures for transition zones can be divided according to their application period,
21 which is either the design stage (preventive measures) or the operation stage (corrective measures).

22 When designing a transition zone, the primary goal is to construct a zone with smooth changes of
23 the vertical stiffness from the embankment to the engineering structure. A thorough review of the
24 countermeasures for transition zones can be found in [28]. The countermeasures are applied to
25 embankments or/and engineering structure. The intent of the countermeasures on the embankment
26 is to reinforce the ballast track on different levels using various measures such as:

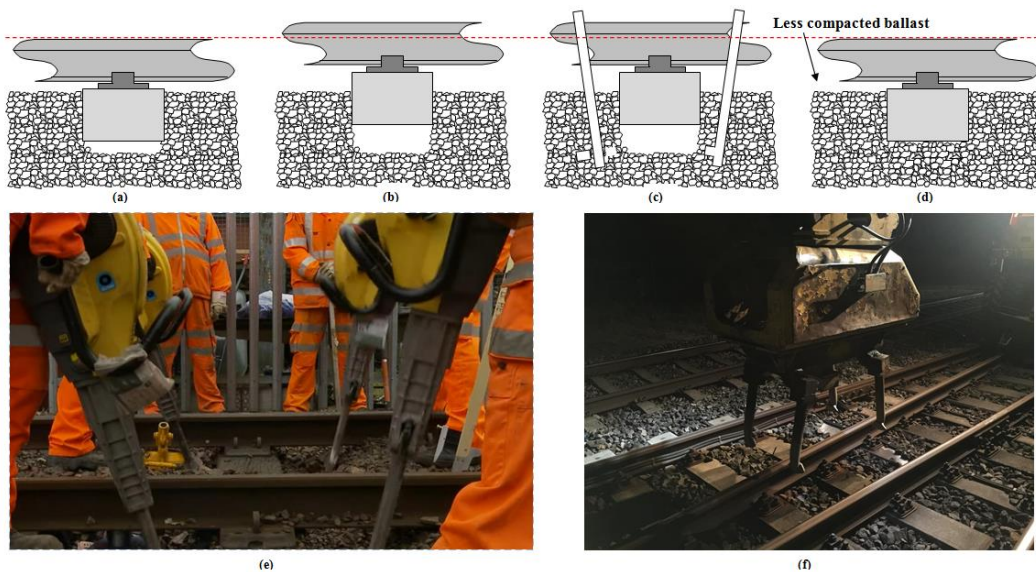
- 27 • Subgrade: the geocell, geotextile, cement, hot mix asphalt [9, 29], and transition wedge
28 (special backfill) [30];
- 29 • Ballast: the ballast glue [31], under ballast mat [32], pile or steel bar underneath the
30 ballast [8, 33], and ballast containment wall [34];
- 31 • Sleepers: sleeper modifications such as increasing its length and reducing the spacing [27,
32 34] and weight [35, 36].

33 The countermeasures on engineering structures are intended to decrease the stiffness of the
34 tracks, for instance using rail pads [37], under slab pads [34] and under sleeper pads [24, 25]. In
35 addition, some countermeasures increase the intergrade stiffness of transition zones such as using
36 auxiliary rails [27, 34, 36] and approaching slabs [38]. In some cases, a combination of several
37 countermeasures is used [28].

1 When the preventive countermeasures do not mitigate the track degradation efficiently, e.g. [9,
2 10], or no countermeasures are used, e.g. [7, 39], critical differential settlement may appear in a
3 maintenance cycle. The differential settlement may result in the damage of track components and
4 deterioration of the passenger's comfort. To mitigate the existing differential settlement, the
5 corrective countermeasures are necessary. Besides, the hanging sleepers also appear in the vicinity
6 of the engineering structure (0.9m [22] or 1.5m [40] from the engineering structure). However, due
7 to the abutment or the transition structure, it is not always possible for tamping machines to
8 perform track maintenance near engineering structures. In these situations, corrective
9 countermeasures that can be performed manually or by small packing machines can be applied. The
10 general principle of the corrective countermeasures is to fill the gaps between the sleepers and
11 ballast to eliminate hanging sleepers.

12 2.1.Existing corrective countermeasures

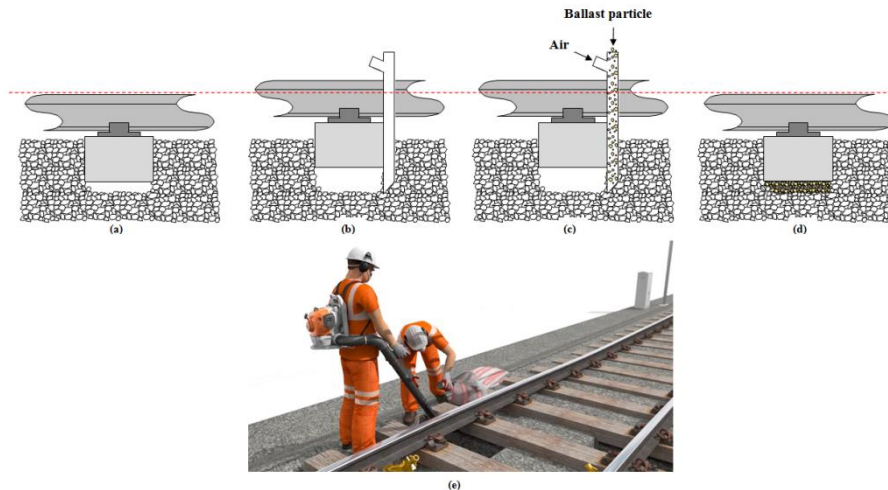
13 The most common corrective countermeasure is tamping. During the process, sleepers are lifted
14 to the required level and the ballast around the sleepers is packed into the void below the sleepers,
15 either manually or by mechanical means (when possible). When the mechanised on-track tamping
16 machines are used, vibrating tamping tines are introduced into the ballast on both sides of the
17 sleeper. The vibration frequency is chosen so as to fluidise the ballast, which then is compacted
18 inwards and upwards towards the bottom of the sleeper [41]. In case of some special structures, the
19 tamping machines are difficult to employ. Tamping using manual or mechanised vibrating hammers
20 is therefore necessary. During the process, sleepers are lifted by hydraulic rail jacks (sometimes
21 sleepers are not lifted) and then four vibrating hammers are used, with consolidating heads opposing
22 each other on both sides of the track. It should be noted that the manual tamping is prohibited in
23 some countries (e.g. the Netherlands) due to health and safety rules. Since tamping does not add
24 additional material to the track and make ballast less compacted, hanging sleepers will re-appear at
25 the relatively short time. The principle of tamping and operations using manual vibrating hammers
26 [42] and mechanised vibrating hammers are shown in Figure 4.



27

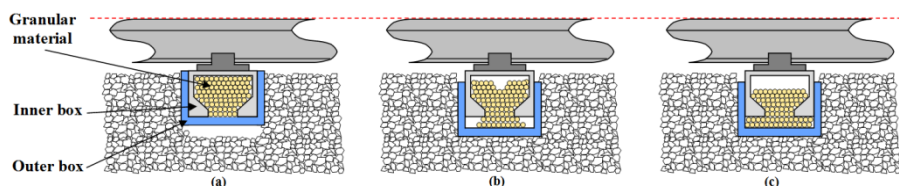
28 Figure 4. Schematic diagram of tamping: (a)-(d) Working principle, (e) Operation using manual
29 vibrating hammers [42], (f) Operation using mechanised vibrating hammers.

1 One way to fill the gap under hanging sleepers is to use hand-held stoneblower [41, 43] or on-
2 track stoneblower (when possible) [7, 44-46]. The principle is first to lift the sleeper to the required
3 level with minimum disturbances; and then to blow a pre-determined quantity of stones into the
4 void under the sleeper using compressed air. The amount of the added stones is determined by the
5 basis of the sleeper displacement before the stone blowing (using the void meter) [43]. The size of
6 the blown stones is 14-20mm, which is smaller than the ballast particle [41]. The principle of ballast
7 blowing and a hand-held stoneblower are shown in Figure 5 [42].



8
9 Figure 5. Schematic diagram of stone blowing: (a)-(d) Working principle, (e) Demonstration from [42].

10 Another corrective countermeasure is to use the Automatic irregularity-correcting sleeper [47].
11 This kind of sleeper consists of a fibre-reinforced polymer sleeper and two automatic subsidence
12 compensating devices under each rail. The automatic subsidence compensating device has two
13 nested boxes (see Figure 6(a)), which allows relative vertical movement. The inner box is connected
14 to the rail through the sleeper, while the outer box is laid on the ballast. The inner box is filled with
15 the (2mm-diameter) granular material. When the differential settlement appears in the track, the
16 outer box sinks together with the ballast. At the moment, the gap between the inner box and the
17 outer box is filled with the granular material as shown in Figure 6(b). Therefore, the differential
18 settlement is compensated by the granular material as shown in Figure 6(c).

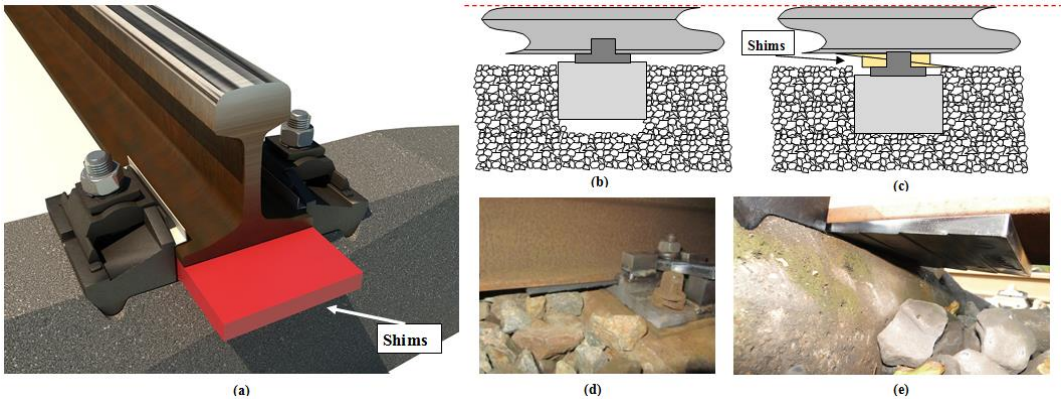


19
20 Figure 6. Working principle of the Automatic irregularity-correcting sleeper.

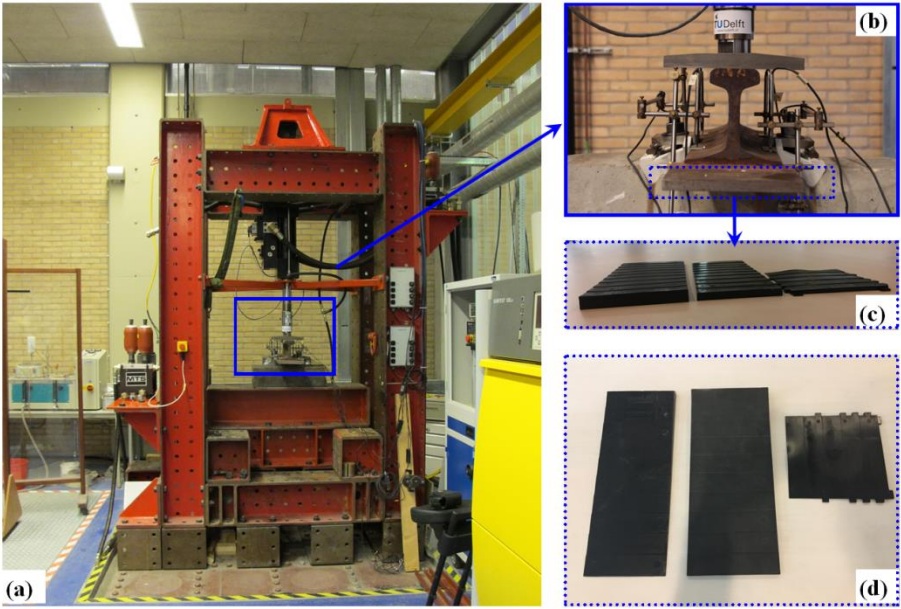
21 **2.2. Proposed corrective countermeasure**

22 An innovative way to remove the gap under the hanging sleepers is by using adjustable fasteners
23 which consist of two plastic wedges (shims) inserted between sleepers and rails (Figure 7) instead of
24 the rail pad. The height of the adjustable fastener can be changed by adjusting the position of the
25 two wedges (shims) to fill the gap, which is 7mm-37mm (i.e. the net adjustable height is 0mm-
26 30mm). When hanging sleepers appear, the height of the shims can be manually adjusted. As a result,

1 the hanging sleepers can be restored to be fully supported by the ballast under. The principle of
2 using adjustable fasteners is shown in Figure 7(b) and (c). An example of the adjustable fastener used
3 in the track is shown in Figure 7(c). The stiffness of the shims (see Figure 8(c)) has been tested in the
4 laboratory of TU Delft [48] prior to the field implementation as shown in Figure 8, which follows the
5 test procedures of rail fasteners stated in [49]. The load was applied by the hydraulic press machine
6 (see Figure 8(a)) and the displacements were measured by LVDTs (Linear Variable Differential
7 Transformer, see Figure 8(b)). During the test, the height of shims (adjusted height of the fasteners)
8 was set to the maximum (37mm including 30mm adjusted height). The shims were installed in the
9 fastener attached to an NS90 concrete sleeper and a section of UIC 54E1 rail. The test results show
10 that the vertical stiffness of the shims is 8.02E8 N/m and no heavy visible damage was found after
11 loading.



12
13 Figure 7. Schematic diagram of Adjustable fastener: (a) Adjustable fastener, (b)-(c) Working principle,
14 (d)-(e) Examples of the adjustable fastener used in the track.

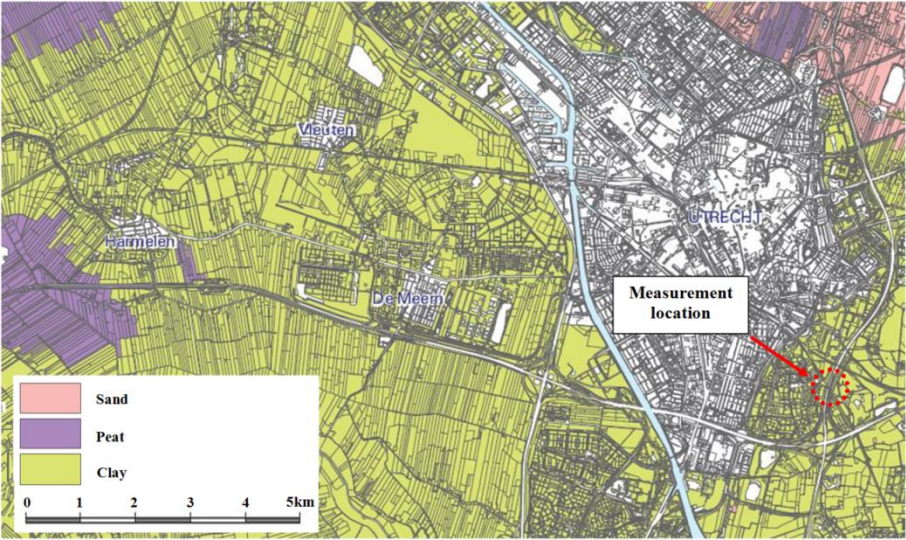


15
16 Figure 8. Laboratory test of the shims: (a) Hydraulic press machine, (b) Zoom-in photo at the actuator
17 with LVDTs, (c) Side view of the shims, (d) Top view of the shims.

18

1 **3. Experimental analysis**

2 To study the effectiveness of the adjustable fasteners, they were tested on three transition zones
3 in the track between Utrecht and Houten, the Netherlands. The natural ground in that region mainly
4 consists of soft soil. According to the ground map of the location measured in 2012 [50], the natural
5 ground of the measured lines is mostly clay, as shown in Figure 9. A detailed field survey including
6 Cone Penetration Tests and Vertical Seismic penetration Tests at a nearby location can be found in [2]
7 and [37]. The tracks were newly constructed and no transition slabs were employed in the transition
8 zones. Heavy traffic was expected on the tracks, including both passenger trains and freight trains.
9 The change of the traffic amount was negligible during the whole measurement period. Since the
10 fasteners are adjusted to fill the gaps under sleepers, the adjusted height equals the accumulated
11 voiding, which can represent the degradation process of tracks at the moment. Therefore, the
12 adjusted heights of the fasteners are recorded to analyse the track degradation.



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Figure 9. Ground map of the measurement location.

15 **3.1.Measurement set-up**

16 The measurements were performed on the transition zones which consist of ballast tracks and
17 slab tracks (the rails are directed fastened on metallic or concrete structures) [51]. The three
18 (Embankment-Slab track-Embankment) transition zones are named Transition Zone A, B, and C, as
19 shown in Figure 10.



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Figure 10. Photographs of the measured transition zones.

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The adjustable fasteners had been installed on ten sleepers on both sides of slab tracks when the tracks were constructed. At that moment, the fasteners were set to their lowest position (i.e. adjusted height is 0mm). The new tracks including the transition zones were levelled and aligned by tamping machines before the tracks went into service, as shown in Figure 11.



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Figure 11. Tamping in a transition zone.

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The fasteners were adjusted after the 2-month operation and after the 5-month operation. During the adjustments, the fasteners were first unfixed (see Figure 12(a)); then the rails were lifted to the required level by standard rail jacks (see Figure 12(b)) or hydraulic rail jacks. After that, the fasteners were adjusted to the desired height by sliding shims. The schematic diagram of using adjustable fasteners in transition zones are explained in Figure 13. Since there was no operation specification for the adjustable fasteners, the adjustment mainly depended on the experience of the working staff. The operation for one side of a slab track (ten sleepers) required approximately fifteen minutes with three or four working staff. The adjusted heights of fasteners after two months and five months were recorded to analyse the development of the settlement in transition zones. It should be noted that due to practical reasons the fasteners on the embankment-slab track side of Transition Zone A were not possible to adjust after five months.



Figure 12. Adjustment of the fasteners.

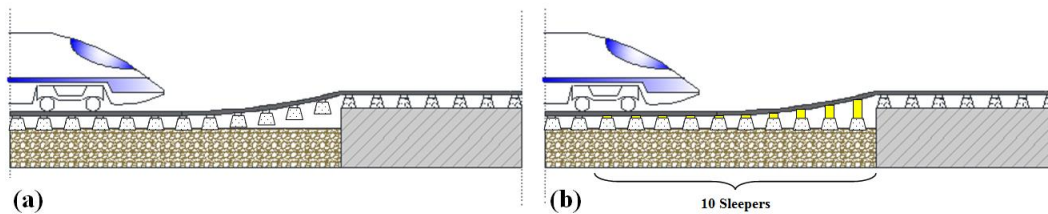


Figure 13. Schematic diagram of using adjustable fasteners in transition zones.

3.2. Measurement results

The adjusted heights of fasteners (accumulated voiding) of Transition Zone A, B, and C are shown in Figure 14, Figure 15, and Figure 16, respectively. It should be noted that since the adjusted height of all fasteners is 0mm at 0 month, it is omitted in the figures. For convenience, the sleepers are numbered starting from the one closest to the slab track. The numbers are negative on the left side of the slab track (the embankment-slab track transition) and positive on the right (the slab track-embankment transition). The moving direction of the trains are indicated in the figures.

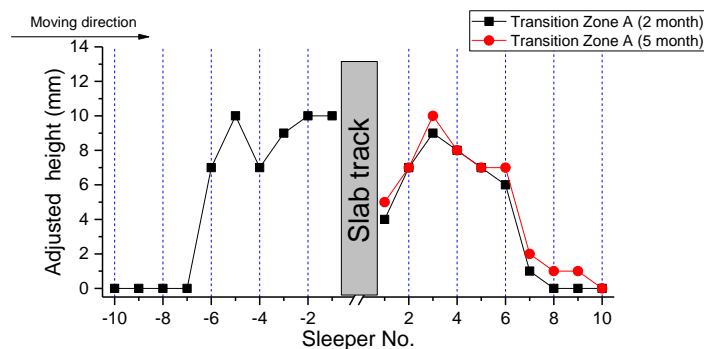
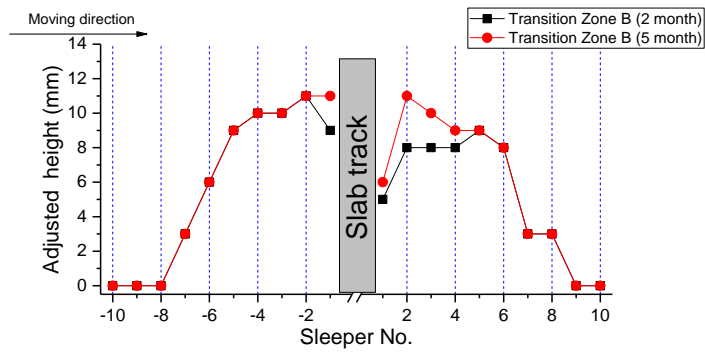


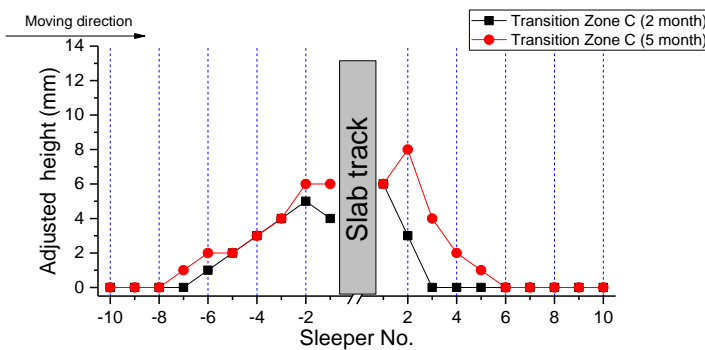
Figure 14. Adjusted heights of the fasteners in Transition Zone A.



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Figure 15. Adjusted heights of the fasteners in Transition Zone B.



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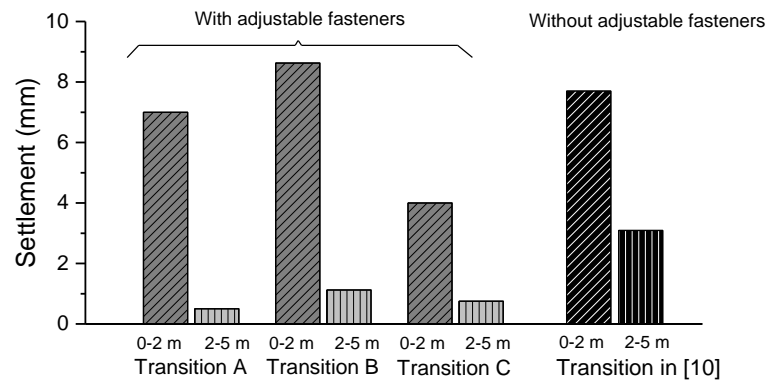
Figure 16. Adjusted heights of the fasteners in Transition Zone C.

5 As it can be seen from Figure 14-Figure 16, the settlement of ballast tracks appears after the 2-
6 month operation where the maximal settlement values within 5mm (in Transition zone C)-11mm (in
7 Transition zone B) can be found. The settlement is comparable with the measurement results in [2,
8 10], where more than 10mm settlement were also found after the 2-month operation. This is
9 reasonable since both locations have the soft subsoil. The track settlement close to the slab track is
10 higher in comparison with that farther from slab tracks. The settlement reaches the largest amount
11 at the 2nd or 3rd sleepers from the slab track (Sleeper±2, Sleeper±3); after that, it gradually reduces to
12 0mm at the 8th or 9th sleepers (Sleeper±8, Sleeper±9). This is mainly because the ballast/subballast
13 layer was not full stabilised before the operation, i.e. the rapid compaction process (Stage 1 in Figure
14 2) happened. This confirms the theory of the appearance of the different settlement in transition
15 zones which is explained in Section 1 that the settlement near engineering structures is much higher
16 than that farther from engineering structures. It is also consistent with the measurement results in
17 [52].

18 It should be noted that the settlement at the 1st sleeper is less than that at the 2nd or 3rd sleepers.
19 It is because the rail is constrained by the slab track. As a result, the 1st sleeper has less movement
20 under train loads and generate smaller impact to the ballast. In addition, the wheel forces are
21 increased when the bogie is half on the slab track and half on the embankment, the distance of
22 which corresponds to the 2nd or 3rd sleepers [18].

23 At the 2-month operation, the ballast tracks were compacted while the slab tracks were almost
24 unchanged, which caused a differential settlement in the transition zones. If no corrective
25 countermeasure was implemented, the tracks in the transition zones would continue degrading due
26 to the amplified dynamic wheel loads and ballast stress [20, 53-57]. However, after filling the voiding

1 using adjustable fasteners at the 2nd month, the growth rate of the settlement in ballast tracks was
2 significantly reduced, which can be seen by comparing the accumulated settlement in the 0-2nd
3 month and that in the 2nd-5th month in Figure 14-Figure 16. Also, the measurement results of
4 Transition Zone A-C (averaged settlement measured at the locations of the 1st to 4th sleepers to the
5 slab track) are compared with the transition zone without adjustable fasteners (the settlement at 4m
6 from a culvert) from [10] in Figure 17. As shown in Figure 17, large settlements are found both in the
7 transition zones with and without adjustable fasteners after the 2-month operation, which is mostly
8 caused by the rapid compaction of ballast/subballast. However, there is also a relatively large
9 settlement generated in the 2nd-5th month in the transition zone without adjustable fasteners, which
10 accounts for 40% of the settlement in the 0-2nd month. On the contrary, the proportion is only within
11 7% (Transition Zone A)-19% (Transition Zone C) in the cases of the using the adjustable fasteners. It
12 indicates that the adjustable fasteners reduce the growth rate of the settlement (i.e. degradation
13 rate) in the transition zones.



14

15 Figure 17. Comparison between the settlement transition zones with and without adjustable
16 fasteners.

17 4. Numerical analysis

18 Since the adjustable fasteners are proved to be effective in the field measurement, its effect on
19 the dynamic behaviour of transition zones is analysed using the FE method.

20 4.1. Introduction of FE model

21 The FE model used in the paper is based on the model proposed in [58] and further developed in
22 [18, 57]. In this study, it is tuned according to the measured transition zones. The model is pre-
23 processed in ANSYS and solved in LS-DYNA. The modelling method and the parameters are briefly
24 described here. The model consists of two ballast tracks and a slab track (in between), as shown in
25 Figure 18. When the railway vehicle moves from one ballast track to the other (from left to right), it is
26 possible to analyse both the embankment-slab track and the slab track-embankment transition with
27 a single calculation. In the model, the 'slab track' is symbolical and not analysed to reduce calculation
28 costs, because the purpose of the paper is to study the ballast track degradation in the transition
29 zone rather than the slab track itself. The effect of the mortar layer [59] and the temperature [60] is
30 not considered.

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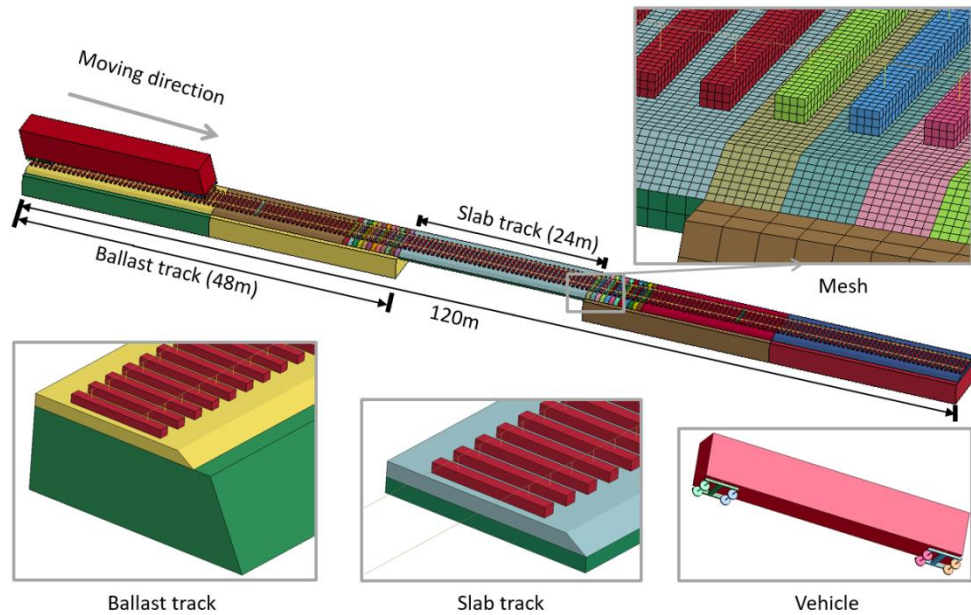


Figure 18. FE model of track transition zones.

As the length of the ballast track is 48m and the slab is 24m, thereby, the total length of the transition zone is 120m. The components of ballast tracks are rails, fasteners, sleepers, ballast, and subgrade. The rails are modelled by the beam elements with the cross-section and mass properties of the UIC54 rails. The adjustable fasteners are modelled by spring-damper elements which have bilinear material property. In compression, the elements can model elastic rail pads; and in tension, the elements can model the clamping effect of fasteners. In this way, hanging sleepers can be attached to rails, leaving gaps underneath.

Ballast, sleepers, and subgrade are modelled using the selective reduced integrated solid elements (element type 2 in LS-DYNA) with elastic material properties. This solid element assumes that the pressure is constant throughout the element to avoid pressure locking during nearly incompressible flow. The depth of ballast and subgrade layer is 0.3m and 2m, respectively.

The railway vehicle is a passenger vehicle, which is idealized as a multibody system consisting of one carbody, two bogies, and four wheelsets. The parameters of the vehicle are based on [61] and adapted to a Dutch passenger train [2, 62]. The velocity of the vehicle is 144km/h (standard operation velocity). The contact between the wheels and rails is modelled using the Hertzian spring [43]. The stiffness is calculated according to Equation 1.

$$k_H = \sqrt[3]{\frac{3E^2Q\sqrt{R_{wheel}R_{railprof}}}{2(1-\nu^2)^2}}, \quad (1)$$

where: E is the modulus of elasticity of the wheel and rail; ν is the Poisson's ratio; Q is the static vertical wheel load; R_{wheel} is the radius of the wheel; $R_{railprof}$ is the radius of the railhead [43, 63].

The connection between sleepers and ballast is modelled by the Penalty-based contact [64] in order to accurately present the spatial movement of sleepers and consequent ballast stresses. The Penalty-based contact employs the penalty method, which places normal interface springs between all penetrating nodes and the contact surface, as shown in **Error! Reference source not found.**

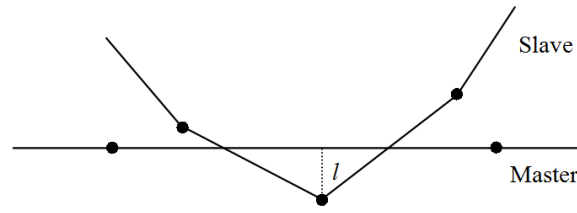


Figure 19. Schematic diagram of the penalty contact method.

According to the method, the search for penetrations between the bottom surface of sleepers and the top surface of ballast is made for every time step during the calculation. In the case that the penetration happens, no force is added. When the penetration between contact surfaces has been found, a force proportional to the penetration depth is applied to resist and ultimately eliminate the penetration [65]. If slave node n_s has penetrated through master segment s_i , the interface force vector f_s can be expressed as:

$$f_s = -lk_i n_i, \quad \text{if } l < 0, \quad (2)$$

where l is the penetration; k_i is the stiffness factor for master segment s_i ; n_i is normal to the master segment at the contact point. The stiffness factor k_i is:

$$k_i = \frac{f_{si} K_i A_i^2}{V_i}, \quad (3)$$

where f_{si} is a scale factor for the interface stiffness; K_i is the bulk modulus; A_i is the face area of the element that contains s_i ; V_i is volume [65].

On the contrary, the Constraint-based contact [64] (tied contact) is used between the ballast and subgrade, and between the slab and the support layer. In the tied contact, the slave nodes are constrained to move with the master surface. The non-reflection boundaries [65] are applied to both ends of the model in order to reduce the wave reflection effect. The translation and rotation freedom of the nodes on the bottom of the subgrade and slab track are fixed. The material properties of the track components and the vehicle used in the model are suggested by [24, 30, 34, 66-69] and tuned according to the field measurements described in [18]. The main parameters used in the model are collected in Table 1.

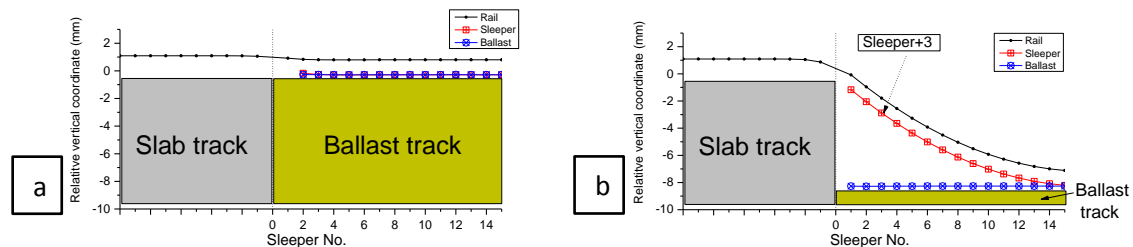
Table 1: Material properties of the track components.

Parameter	Value
Sleeper Elastic Modulus (Pa)	3.65E+10
Sleeper Poisson ratio	0.17
Sleeper Density (kg/m ³)	2.50E+3
Ballast Elastic Modulus (Pa)	1.20E+08
Ballast Poisson ratio	0.25
Ballast Density (kg/m ³)	1.80E+3
Subgrade Elastic Modulus (Pa)	1.80E+08
Subgrade Poisson ratio	0.250
Subgrade Density (kg/m ³)	2.30E+3
Concrete slab Elastic Modulus (Pa)	3.50E+10
Concrete slab Poisson ratio	0.17
Concrete slab Density (kg/m ³)	2.50E+3
Support layer Elastic Modulus (Pa)	3.30E+10
Support layer Poisson ratio	0.17
Support layer Density (kg/m ³)	2.50E+3

Fastener Vertical (compression) Stiffness (N/m)	1.20E+8
Fastener Vertical (compression) Damping (N*s/m)	5.00E+4
Fastener Vertical (tension) Stiffness (N/m)	1.20E+11
Fastener Vertical (tension) Damping (N*s/m)	5.00E+4
Primary suspension stiffness (N/m)	4.25E+5
Primary suspension damping (N*s/m)	1.00E+6
Secondary suspension stiffness (N/m)	4.68E+5
Secondary suspension damping (N*s/m)	6.50E+4
Secondary suspension Bending stiffness (Nm/rad)	1.05E+4
Distance between wheels (m)	2.5
Distance between axles (m)	20.0
Length of train body (m)	23.0
Axle load (t)	21.4
Carbody Mass (t)	64.6
Bogie Mass (t)	2.4

1

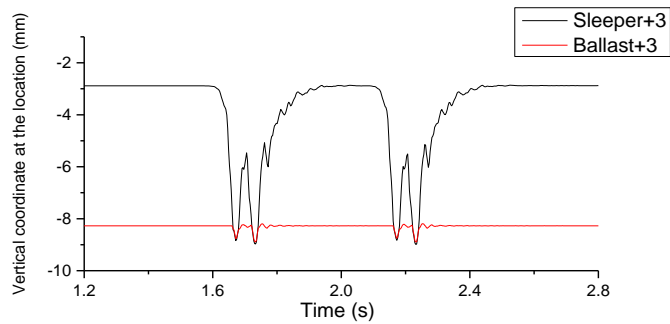
2 The FE model considers the settlement of the ballast track due to the rapid compaction after the
 3 operation (settlement in Stage 1 as illustrated in Figure 2). Since the exact settlement value is difficult
 4 to acquire, it is assumed to be equivalent to the adjusted height of the fasteners after 2 months. The
 5 maximal adjusted height varies from 5mm (in Transition zone C) to 11mm (in Transition zone B). As a
 6 result, the medium number 8mm is adopted for the differential settlement value in the model. After
 7 that, the cases of 4mm and 12mm are also analysed in the parametric study. The rail longitudinal
 8 level, sleeper and ballast coordinates in the transition zone model before and after the
 9 implementation of the differential settlement are illustrated in Figure 20.



10

11 Figure 20. Coordinates of track components before and after the implementation of the differential
 12 settlement: (a) Before, (b) After.

13 At the equilibrium state of the model, due to separable contact between the sleepers and ballast,
 14 as well as the application of the gravity and the resistance of the rail, the gaps between the sleepers
 15 and ballast appear in the vicinity of the slab track, as shown in Figure 20(b). The hanging distance of
 16 Sleeper+1 is the highest and the hanging distance gradually reduces as the distance from the slab
 17 track increases. The hanging values of the sleepers are presented as the differences between the
 18 sleepers and ballast. The same situation is observed on the other side of the slab track. After the
 19 equilibrium state, the vehicle moves over the transition zone. The movement of a hanging sleeper
 20 (Sleeper+3) is shown in Figure 21.



1

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Figure 21: Time history of vertical coordinate due to the passing vehicle at Sleeper+3.

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The gap under the hanging sleeper is eliminated under train loads, where an increase of wheel-rail contact forces can be expected. When the sleepers contact the ballast, the penetrations between the bottom surface of the sleepers and the top surface of the ballast occur. The stress distribution in the ballast can be therefore calculated. Because the voids under sleepers are different depending on the location (see Figure 20(b)), the stresses in the ballast elements under different sleepers also vary according to their locations.

9

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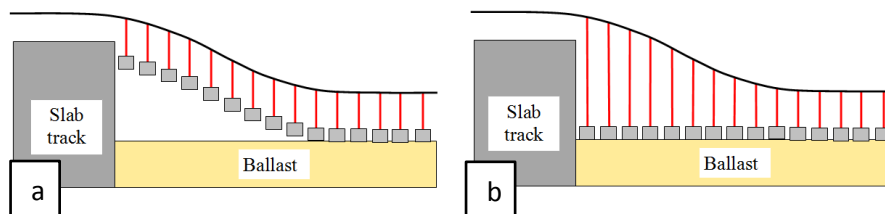
13

14

15

16

In the transition zone using adjustable fasteners, the spring-damper elements are extended to compensate the gaps between the sleepers and ballast, as shown in Figure 22. It should be noted that the stiffness and damping of adjustable fasteners are assumed to be same as normal fasteners (normal rail pads) and constant at various adjusted height. This simplification is made to reduce the computational expenses based on the findings that the effect of the differential settlement on dynamic responses is much larger in comparison with the stiffness variation [20, 54, 55, 57]. The differential settlement is magnified in Figure 22. In the actual case, the differential settlement (8mm) is much smaller compared to the size of a sleeper (240mm in height).



17

18

19

Figure 22. Schematic diagram of the simulation of the countermeasure: (a) Reference, (b) Using adjustable fasteners. Fasteners are indicated by the red lines.

20

21

22

23

The study to find the optimal mesh sizes was performed prior to this study and it is not included due to the limited space. The model contains 675686 nodes, 489,547 solid elements and 3,612 beam elements. The time-step is 1.3E-5s. The calculation takes around 14 hours using a 4-core (I7) workstation with 4 High Parallel Computing (HPC).

24

25

4.2. Dynamic responses of transition zones with the adjustable fasteners

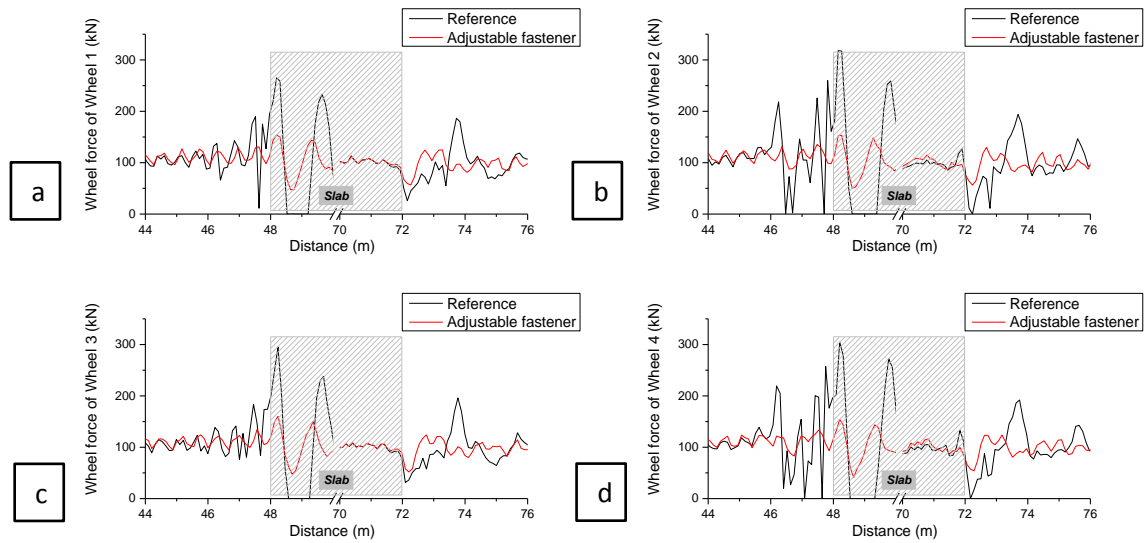
Wheel force

26

27

The calculated wheel forces of four wheelsets in the reference case and the adjustable fastener case are shown in Figure 23. The wheel loads on the slab track are not considered in this study and

1 therefore their responses are covered by the shaded area. The maximal wheel forces acting on
 2 ballast tracks are collected in Table 2.



3
 4
 5 Figure 23. Wheel-rail interaction force of four wheelsets: (a) Wheel 1, (b) Wheel 2, (c) Wheel 3, (d)
 6 Wheel 4. The wheel loads on the slab track are covered by the shaded area.

7 Table 2. Maximal wheel forces in the ballast track in transition zones from Figure 23.

Wheel No.	Embankment-Slab track transition			Slab track-Embankment transition		
	Reference	Adjustable fastener	Reduction	Reference	Adjustable fastener	Reduction
1	190.0	130.8	45%	186.4	124.9	49%
2	259.8	135.4	92%	194.0	129.7	50%
3	183.6	133.9	37%	196.1	124.0	58%
4	257.6	133.3	93%	192.0	123.7	55%

8
 9 Figure 23 shows that the wheel loads are slightly vibrated around 105kN, which is the static wheel
 10 load ($210/2=105\text{kN}$); while they are increased considerably at both the embankment-slab track
 11 transition and the slab track-embankment transition in the reference case. This indicates the
 12 differential settlement lead to the amplification of wheel loads if no adjustable fastener is
 13 implemented, which is consistent with [68]. It can also explain the track degradation in transition
 14 zones often reported in [7, 9].

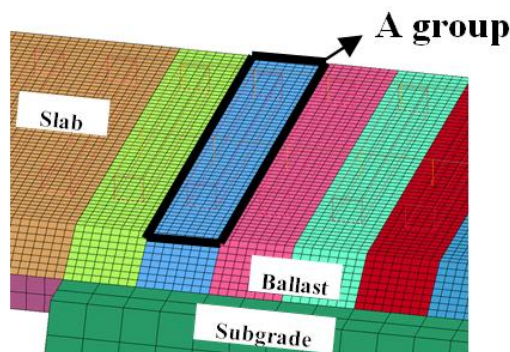
15 In [68], the dynamic responses of a train passing the transition zone (consisting of a ballast track
 16 with backfill and a long bridge) were studied. When the different settlement was 10mm, the wheel
 17 load of Wheel 1 was increased on 50% the embankment and 64% on the engineering structure in
 18 comparison with the static load. In this study, the increases of the wheel load of Wheel 1 are 81%
 19 and 152%, respectively, which is comparable with the study in [68]. The discrepancy is mainly caused
 20 by the difference in contact settings and the constrain condition of the engineering structures (the
 21 slab track in this study is fixed by all the nodes on the bottom layer; while the bridge in [68] is fixed
 22 only at the ends of the bridge deck).

23 Comparing to the reference case, the wheel loads are reduced significantly when adjustable
 24 fasteners are in use (see Figure 23 and Table 2). The reason is that the hanging sleepers in the

1 transition zone are eliminated by the adjustable fasteners. This moment corresponds to the first
 2 adjustment of the fasteners in the measured transition zones after the 2-month operation (Section 3).
 3 The decrease of the amplified wheel loads (in Figure 23 and Table 2) explains the substantial
 4 reduction of the track settlement growth rate in the measurement after using the adjustable
 5 fasteners.

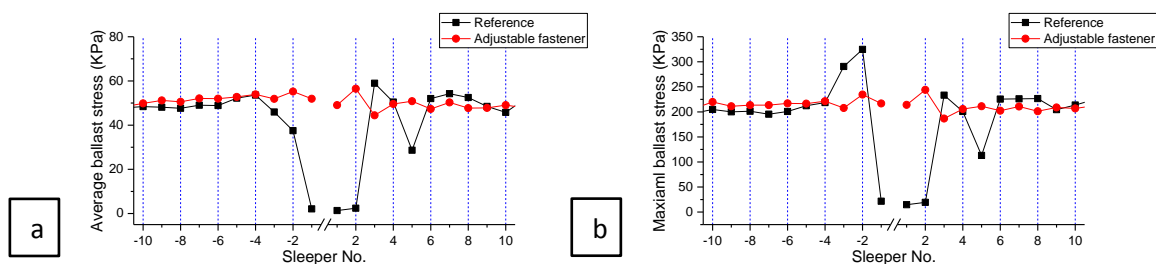
6 Ballast stress

7 Since the ballast settlement depends on the ballast stress, the ballast stress in the transition zone
 8 is studied as well. It should be noted that the response of unbound granular materials like ballast is
 9 non-linear, which affects the calculated distribution of stresses inside the ballast. However, for the
 10 transient behaviour analysis in this study, the linear model is used as a first attempt, which is a
 11 typical approach, e.g. in [20, 69], due to the limitation of the computational power. Moreover, the
 12 ballast bed in this model is assumed to be well compacted (after the rapid compaction). At the
 13 moment the sleeper contacting the top of the ballast, the ballast on the top can be simplified as the
 14 linear material. In order to compare the ballast responses at the different locations, the ballast
 15 elements under one sleeper are considered as a group (unit), which are 416 elements ($52 \times 8 = 416$,
 16 there are 8 elements on the wide side and 52 elements on the long side, see Figure 24).



17
 18 Figure 24. Collecting method for ballast stresses. Red lines are sleeper frames.

19 The vertical stresses in ballast under sleepers are analysed. The average and maximal stresses
 20 calculated for the ballast elements in each group along the track are shown in Figure 25.

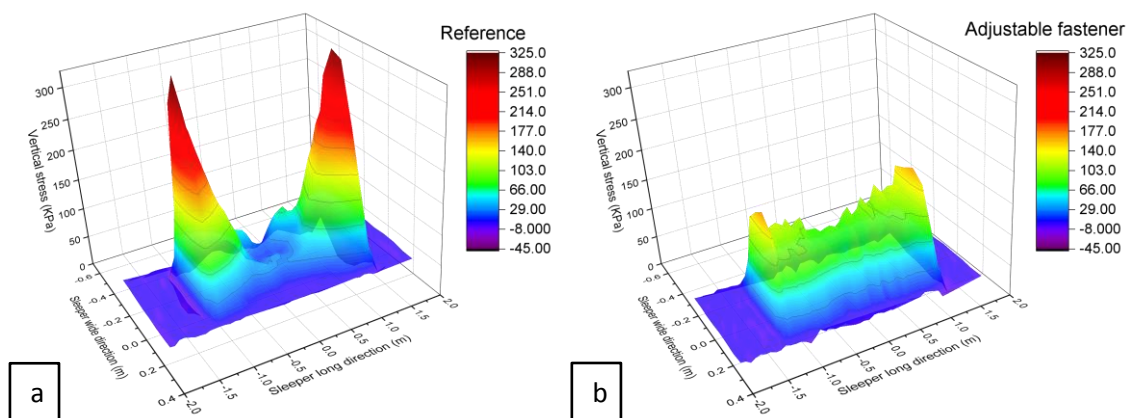


21
 22 Figure 25. Ballast stresses along the track: (a) Average; (b) Maximum.

23 The maximal ballast stress is around 200KPa at the location less affected by the slab track (e.g.
 24 Sleeper ± 10), which is reasonable in comparison with the measurement results in [70]. In [70], the
 25 ballast stress was measured during the passage of a coal train with 25t axle load. It is found that the
 26 maximal ballast stress was in the range between 160-230kpa (with an outlier at 415kPa due to a
 27 wheel-flat).

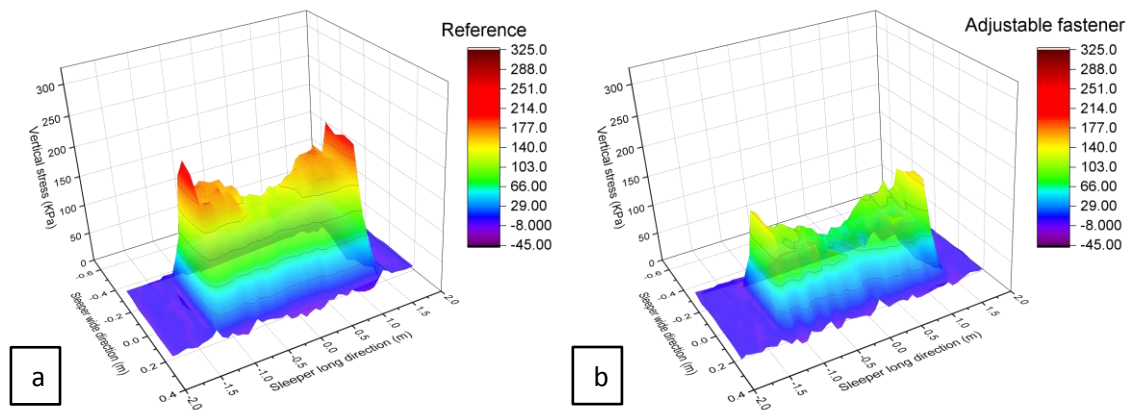
1 It also can be seen in Figure 25(a) that, after using the adjustable fasteners, the average of ballast
2 stresses are more evenly distributed along the track. In the reference case, the ballast stress closer to
3 the slab track is lower than that at a greater distance from the bridge. It is because the hanging
4 values of the sleepers are so large that the bending stiffness of rails resists the sleepers to fully
5 contact the ballast. This confirms the finding from the measurement that the settlement at the 1st
6 sleeper is less than that at the 2nd or 3rd sleepers. The stresses in rails at the moment can be expected
7 high which will be discussed later.

8 Due to the poor support condition, the hanging sleepers can move irregularly, causing the stress
9 concentrations in ballast, which can be proved by Sleeper-2 in Figure 25(b). The maximal ballast
10 stress reaches 325KPa at Sleeper-2, which is approximately 60% higher comparing to the maximal
11 ballast stress in a well-supported location. The stress distributions in ballast under Sleeper-2 in the
12 two cases are shown in Figure 26. The reduction of ballast stresses can be clearly seen from Figure 26,
13 where the maximal stress in ballast is reduced from 325KPa to 235KPa. A similar situation can also be
14 found under Sleeper+3, as shown in Figure 27, where the maximal stress in ballast is reduced from
15 233KPa to 187KPa. Since the ballast stress is proportional to the settlement rate of ballast [71, 72],
16 the amplified stress may lead to the permanent settlement in ballast, which indicates the settlement
17 (track degradation) in the transition zone may increase continuously if the adjustable fasteners are
18 not implemented.



19

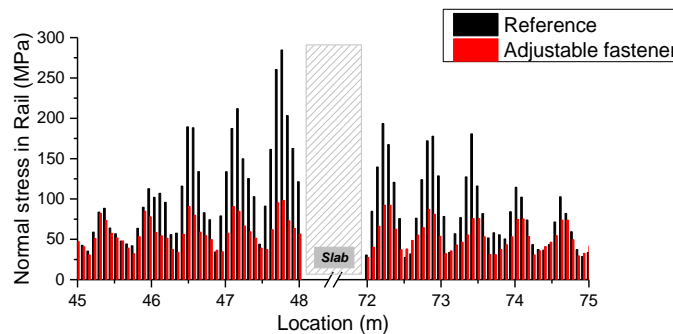
20 Figure 26. Stress distribution in ballast under Sleeper-2: (a) Reference case, (b) Using adjustable
21 fasteners.



1
2 Figure 27. Stress distribution in ballast under Sleeper+3: (a) Reference case, (b) Using adjustable
3 fasteners.

4 **Rail stress**

5 The maximal normal stresses in rails in the two cases are shown in Figure 28. The rails are
6 simulated by the beam elements with the length of 75mm (eight elements in a sleeper space). As
7 expected, the results show that the normal stresses in rails in the reference case are amplified near
8 the slab track. When the adjustable fasteners are in use, the normal stresses are again decreased
9 significantly. It shows the adjustable fasteners are also beneficial to rails.



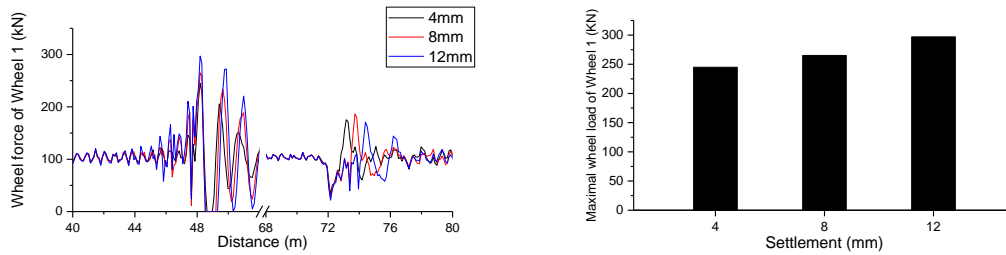
10
11 Figure 28. Normal stresses in rails in the reference case and the adjustable fastener case.

12 To conclude, the adjustable fasteners are effective to reduce the amplification of the wheel forces,
13 to achieve a better ballast stress distribution under the hanging sleepers, and to decrease the normal
14 stresses in rails in transition zones.

15 **4.3. Parametric study**

16 **Differential settlement value**

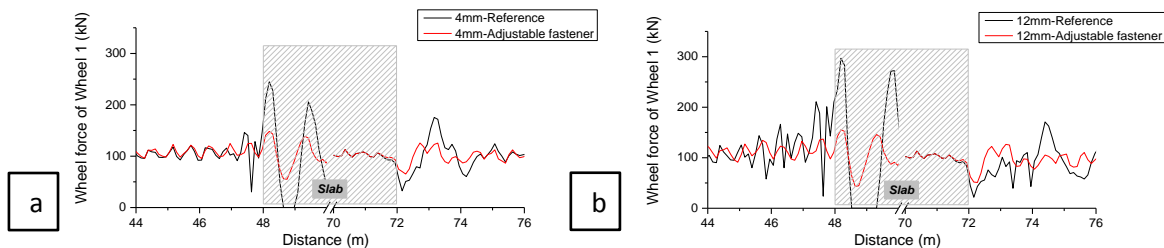
17 Since the differential settlement value strongly depends on the material properties of tracks, the
18 construction and operation condition, the differential settlement in transition zones may vary. To
19 study the applicability of the adjustable fasteners, the transition zones with various differential
20 settlements are modelled. Three values of the differential settlement are calculated, including 4mm,
21 8mm (the reference case in Section 4.2), and 12mm. The wheel forces of Wheel 1 in three cases are
22 compared in Figure 29.



1
2 Figure 29. Wheel force of Wheel 1 in three cases: (a) Time history; (b) Maximum.

3 As it can be seen in Figure 29, the wheel forces are increased as the differential settlement value
4 grows near the slab track. It indicates that the adjustable fasteners should be used in the early stage
5 of the differential settlement initiation. It should be noted that since the rapid compaction stage
6 (Stage 1 of the settlement in Figure 2) is somewhat inevitable, the fasteners should be adjusted as
7 soon as the ballast track is compacted, namely, at the end of Stage 1 or the beginning of Stage 2.
8 Stage 1 of the settlement ends at 0.5MGT according to [16]. Assuming five vehicles in a train, four
9 trains in an operational hour, and fifteen operational hours in a day, it takes approximately twenty
10 days to complete Stage 1 of the settlement.

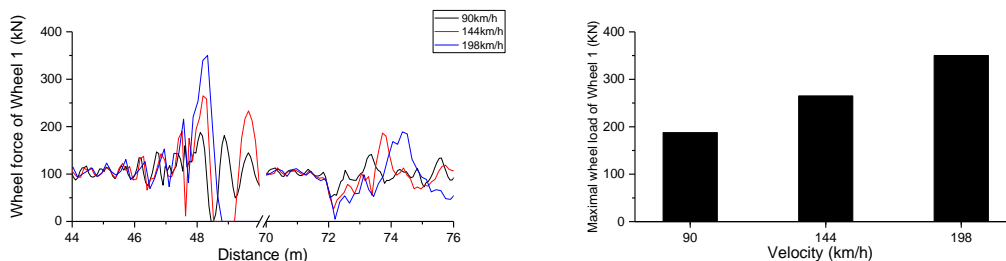
11 The transition zones with adjustable fasteners in the cases of 4mm and 12mm are also studied. The
12 wheel forces of Wheel 1 are shown in Figure 30. It shows that the adjustable fasteners can
13 significantly reduce the amplification of wheel forces in both cases, which indicates the applicability
14 of the adjustable fasteners is relatively large.



15
16 Figure 30. Wheel forces of the transition zones using adjustable fasteners: (a) 4mm differential
17 settlement, (b) 12mm differential settlement.

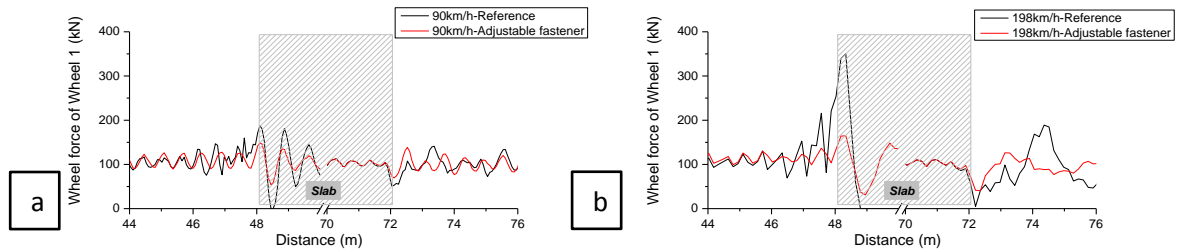
18 **Velocity**

19 The effect of the adjustable fasteners is also studied in the cases of different velocities, including
20 90km/h, 144km/h (the reference case in Section 4.2) and 198km/h. The wheel forces of Wheel 1 in
21 three cases are shown in Figure 31.



22
23 Figure 31. Wheel force of Wheel 1 in three cases: (a) Time history; (b) Maximum.

1 As expected, the wheel forces are amplified significantly near the slab track as the velocity
2 increases, which can be seen in Figure 31. The comparison of the wheel forces in the transition zones
3 with and without adjustable fasteners is shown in Figure 32.



4
5 Figure 32. Wheel forces of the transition zones using adjustable fasteners: (a) 90km/h, (b)
6 198km/h.

7 Figure 32 shows the wheel forces are reduced in both cases. A considerable reduction can be
8 found in the case of 198km/h, which is over 50%, from 350kN to 165kN. It indicates that the
9 adjustable fasteners work in both low-velocity and high-velocity range and the benefit is more
10 significant when the velocity is high.

11 5. Conclusions

12 The paper presents the experimental and numerical analysis of a corrective countermeasure-the
13 adjustable fastener. Its working principle is to eliminate the gap under the hanging sleepers by
14 adjusting the height of the fastener (relative position of two shims). The following conclusions can be
15 drawn.

- 16 (1) Large differential settlements in the three transition zones were found after the 2-month
17 operation, the maximum of which is ranged from 5mm to 11mm. The track settlements near
18 the engineering structures (slab tracks) were much higher in comparison with that farther
19 from the engineering structures.
- 20 (2) The growth rate of the settlement (i.e. degradation rate) of ballast tracks is reduced
21 significantly after the voiding in the track is filled with the adjustable fasteners.
- 22 (3) Numerical analysis shows that the adjustable fasteners are effective to reduce the
23 amplification of wheel forces, achieve a better stress distribution in ballast, and decrease the
24 normal stresses in rails in transition zones.
- 25 (4) The parametric studies show that the applicability of the adjustable fasteners is relatively
26 wide. The adjustable fastener works effectively at both low-velocity and high-velocity and its
27 benefit is more significant when the velocity is high.
- 28 (5) It is recommended that the fasteners should be adjusted once the ballast track is compacted.

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34 authors would like to thank Jan Moraal (from TU Delft) for providing the laboratory test report of the

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