Train-track interactions over vulnerable railway turnout systems exposed to flooding conditions

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Train-track Interactions over vulnerable Railway Turnout Systems exposed to Flooding Conditions

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Abstract

The definition of twist is used in modern railways to determine the warpage of a particular track plane to identify track quality. In some cases, twist is intentionally introduced on tracks to facilitate motion in curves. Nevertheless, twists values above certain thresholds, twist faults, are a direct risk to safety and a potential cause for derailments. Twist faults are commonly observed in ballasted tracks, which consists of crushed rock particles and have low endurance to resist against dynamic track forces. In general, the deterioration in ballast structure has slow progress. However, in reality, there are some catalysts such as extreme events that can speed up the deterioration of the ballast bed. Extreme events have rare occurrences but a high potential to damage structures and environment in a short duration. Even though the adjective 'rare' is still used to define extreme events, a consensus among the environmental scientists on the increased frequency of extreme events could be found in the literature. In this study, the impacts of flooding, one of the most common extreme events, on the dynamic behavior of a turnout structure is investigated in terms of dynamic twist. The reason to select a turnout as a basis for the simulation is the asymmetrical structure of turnout that is expected to amplify twist values. A 3-dimensional finite element method (FEM) model was developed and many hypothetical scenarios ranging from various materials to vehicle speeds were tested in FEM environment. It should be emphasized that the developed model is the modified version of a previously validated model and therefore, validation of the model is done by a comparison with the parent model. The results of the simulations, first time, show that the performance of 'fiber-reinforced foamed urethane' (FFU) bearers is relatively poor in comparison to concrete bearers in terms of twist values. Results also demonstrate that partially damaged structures in the case of flooding is the most critical situation. Regarding the limitations in FEM modelling, it is recommended to halt any railway operations and avoid the approach of 'reach the station first' in emergency cases.

Keywords: railway; switches; crossings; turnouts; flood; extreme weather; train-track interaction; vulnerability.

1. Introduction

Many debates on whether climate change is a global conspiracy or a scientific fact are ongoing in the public domains [1, 2]. It would not be surprising to see the public unconvinced that climate change will be a concrete reality to deal with in the near future. Admittedly, concerning the fact that as most decision-makers are strongly reliant on the general public opinion [3], the actions to mitigate the impacts of climate change are not expected to be taken in the foreseeable future. Hence, the global eco system would possibly continue to deteriorate, leading to severe and frequent weather-oriented natural disasters [4].

According to an earlier report published by UN [5], floods have the largest share in the frequency of global extreme events among the others and it is the second effective one in terms of economic impact costing 656 billion dollars in only one decade. Actually, the economic impact would have been higher if the areas susceptible to flooding were more economically developed. In the report, it was also estimated that 2 billion people were affected and over 100 thousand people lost their life due to flooding. Those numbers clearly emphasize the significance of precautions taken against flooding risks and remind the fundamental principle 'Safety First'. In this regard, all kinds of efforts are welcomed by any stakeholders, particularly, railway infrastructure managers.

Any major disruption in railway service always draw public attention and increase pressure on political and administrative authorities. In such cases, thousands of stranded people in stations, interruptions in trades, financial burdens on infrastructure managers, the loss of reputations are the only few of the consequences. Hence, railway systems are one of the key infrastructures that should be protected against any major disruption such as flooding [6]. Floods could originate from several events (i.e. heavy rainfall, rise of sea levels, rapid snow/ice melt, etc.) and damage several structures (buildings, bridges, tracks, overhead lines) of a railway system, causing each for a consequent assessment, maintenance and repair process.

A considerable number of studies have been dedicated to investigating the impact of extreme event on railway tracks [4, 7-19]. In most cases, the focus is made on those studies to estimate the likelihood of occurrence; to mitigate the risks of such extreme events, to design an optimum maintenance schedule or response plan and so on. In other words, most of those studies were conducted from the risk assessor's point of view. Nevertheless, it is a necessity to understand the limits of physical behaviors of the infrastructure in case of any extreme event. In that manner, a proposed method addressing the issues of flooding being destructive on railway tracks is to produce a replica of the flooding scenario in a controllable test environment to analyze the outcomes of such phenomenon. But obviously, it is not possible to estimate the exact magnitude and form of extreme event owing to its unpredictable complex behavior and limitations in its simulative environments. However, it is feasible to follow an experimental or numerical approach that enables to understand the physical outcomes/measures of the events to minimize the potential risks and hazards. For instance, structural damages on a ballasted railway track caused by flooding could be simulated by means of numerical methods, including but not limited to: finite element methods, discrete element methods and smoothed particles hydrodynamics.

The commonly known damages in railway tracks associated with flooding are mostly related to embankment failures (i.e. washed away ballast, landslides, degradation in soil properties). In practice, infrastructure managers take broad spectra of measures including: flood response plans, prediction models, flexible maintenance plans, deploying trouble-shooting teams, establishing warning system and so on. More importantly, infrastructure managers can also halt any rail operation to avoid further catastrophic failures (i.e. derailments) after any sign of threat. Nonetheless, a damage could be inevitable in some cases as many accident reports could be found in the related literature [6, 20]. Hence, it is pivotal to examine and discuss the effects of embankment damages with respect to track-train interactions. For this purpose, “the maintenance limit approach” could be the suitable one to be used. In maintenance manuals, certain threshold values for track parameters (i.e. alignment, cross-level, gauge, longitudinal level) are defined to indicate the track quality and also to specify the maintenance intervals. Among those parameters, cross-level is relatively more meaningful to be used for the flood conditions as other parameters may already be remaining within the limits for small scale damages, owing to sufficient track resilience.

A twist defect is defined as the intolerable variation in the cross-level values at a certain interval, where the cross-level is the height difference between right and left rails. According to the reports, 10% percentage of reported derailment in the UK is linked to twist defects [21]. Each infrastructure manager categorizes track measurements into several risk levels that settle the maintenance intervals. Measurements could be either dynamic, as obtained by track measurement cars, or static, as measured by trolleys or hand gauges. In general, infrastructure managers use both measurement methods and interestingly, it seems that they use the same thresholds for both methods [22, 23].

In this study, a specific component of a railway track, a turnout is investigated which inherently tends to be twisted under vehicle loading due to its asymmetric topology. Turnouts are deployed at junction points of tracks enabling to divert railway traffic. Their critical role entails frequent and high-quality maintenance to ensure a high level of safety standards, which is costly for Infrastructure Managers. Numerous number of studies have been conducted to gain new insights into understanding the dynamic behavior of railway turnouts [24-28]. However, the specific outcomes of those studies are often relying on the ideal environments (dry and mid-temperature) and do not coincide with the extreme weathers, in this case, flooding in particular. Therefore, this paper aims to examine the dynamic behavior of a turnout subjected to flooding conditions by a comprehensive 3D beam oriented FEM model in terms of dynamic twist.

2. Methodology

The methodology of the current study is illustrated in Figure 1 and explained in the following sections.

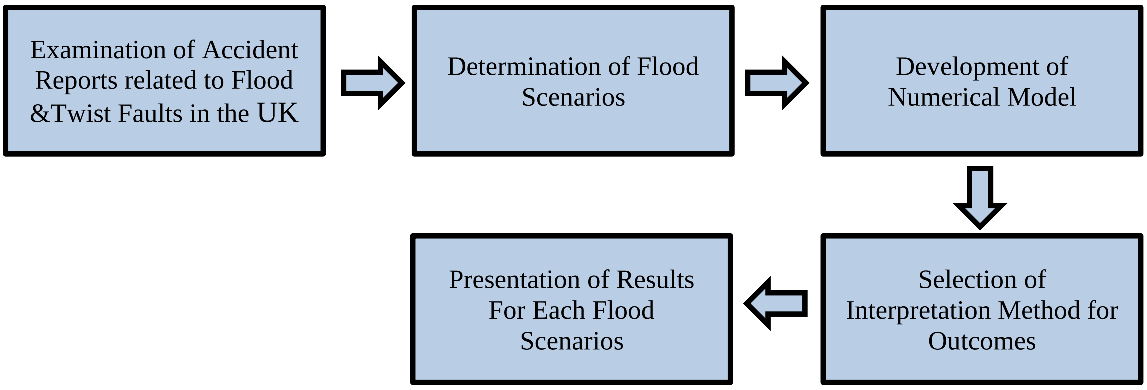

Figure 1. A block diagram of the methodology followed in the current study

2.1 Flood Scenarios

A previously validated FEM model [25] is modified to mimic flood scenarios. The model is illustrated in Figure 2 descriptively. Beams' elemental properties have been updated to simulate complex switch and crossing sections. Three flood scenarios, similar to the ones observed in accident reports [21], are tested for two bearer/sleeper types (i.e. concrete and composite bears/sleepers). In this study, the focus is placed on the severe ballast washaway or when the track support is lost fully or partially. It should be note that concrete bearers/sleepers are the most common technology and composite bearers is particularly promising technology which is expected to replace its counterparts, owing to its advantages such as long lifecycles, vibration properties and so on. In the first scenario, the structural integrity of track is protected but the track itself is exposed to different levels of surface water. The water level is normalized with respect to ballast top surface. The 0% water level indicates the dry condition whereas 100% means the top surface of the ballast structure is just beneath the surface of water. In the second scenario, the flood damages ballast bed severely and the track is unsupported at the certain locations of switch and crossing sections. The length of the damaged section is assumed to be three, five and ten bearer length. In the final scenario, flood has dragged some of the ballast particles and therefore track structure is supported partially. Multiple partial support conditions are tested, and then the worst case is selected. Later, parametric simulations for a turnout with the worst support condition are made to show the effect of partial support.

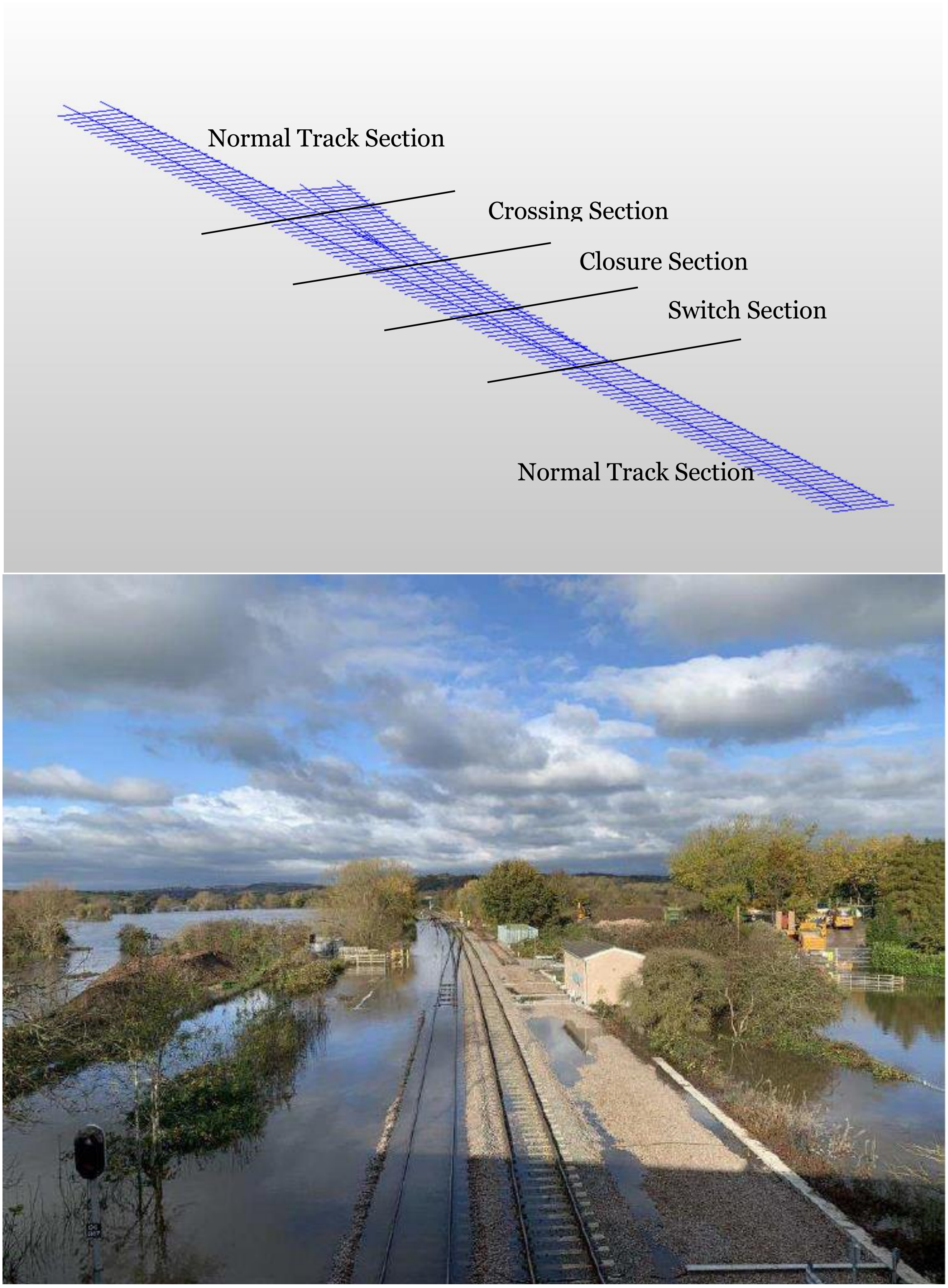

Figure 2. A descriptive view of the numerical model (top). A flooded turnout (bottom - the picture courtesy of East Midlands Railways).

2.2 Finite Element Model

Prior to introducing the FEM model, it is worth mentioning the modifications first. Two minor and two major modifications have been applied to the parent model. The first modification covers the alteration of the material properties of ballast bed which has several material properties with respect to many flooding conditions as reported in [18]. The second modification is related to the aforementioned flood scenarios, where the ballast support is removed completely or partially to simulate the damage caused by flooding. The third but the important modification is the replacement of the spring model. In the parent model, the model was established on elastic foundation theory, which involves elastic springs. The application of elastic springs yields accurate results when the focus is made on unidirectional displacements of tracks. In most cases, this approach coincides with the practice. Meanwhile, in practice a common approach is to consider vertical displacements of rails in the direction of the gravity vector. In other words, the displacements in opposite direction such as rail lifts are neglected. Nevertheless, here the basis parameter is the cross-level value which is defined as the height difference between two rails. Hence, the replacement of the elastic spring model is imperative to consider the rail displacement in both directions. In this regard, an inelastic spring model is used, applying force only in one direction but not to restrict the movement in the opposite direction. It is noteworthy that elastic spring models apply forces in two directions and underestimate the displacement in the opposite directions. The fourth modification is the extension of normal track sections in order to provide valid boundaries for the turnout with inelastic springs. The total length of the extension is 45 m, which is found to be valid for wheel-rail impact [29] . Finally, the application of new spring model and the extension of the turnout implies a separate validation process for the model, whereas it is assumed that the minor changes protect the validity of the model itself. Within this work, the selected validation method refers to a comparison with the validated parent model. The outcomes of two models are presented in terms of contact forces in Figure 3. As also seen in the figure, the replacement of the spring model causes no meaningful deviation from the validated parent model.

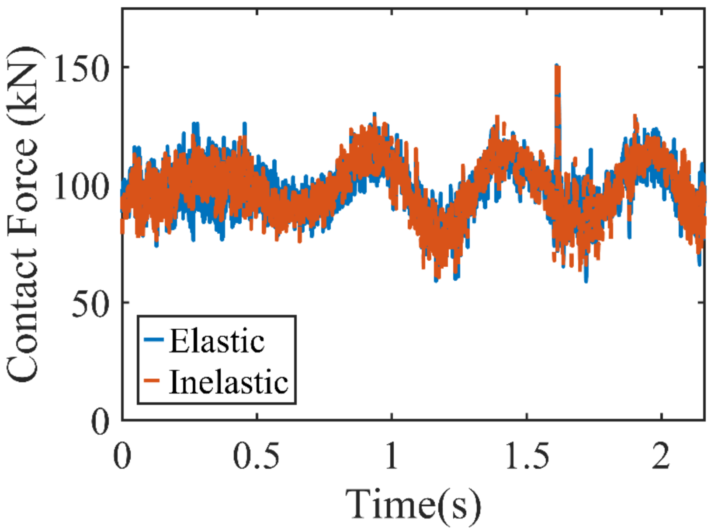

Figure 3. A comparison of contact forces for elastic and inelastic material models

The properties of the model are as follows. A simplified model of 83 m track is built, including a standard turnout with 1:9 crossing angle. A simplification is a necessity to achieve optimum simulation times for such a large structure. In this regard, the model utilizes so-called a beam-grillage method, in which 3D models consist of beams, springs and dashboards. The rails and bearers are represented by equivalent beam structures and the rest by spring-dashboard couples. Rails in both diverging and through routes are modelled, excluding the check rails due to boundary conditions. The total of 77 bearers with a length ranging from 2.4 m to 4.7 m and 48 sleepers with a length of 2.6 m are considered with an average of spacing of 0.71 m. The fastening system is defined by only elastic properties of the rubber rail pad, neglecting minimal elasticity provided by other components as well as the damping properties of the rail pad which have a negligible contribution into total track damping in comparison to ballast bed. Ballast bed and substructure is composed of inelastic springs due to the aforementioned reason. Each bearer is supported by a varying number of spring elements related to their lengths. The properties of inelastic springs are given in Table 1 along with other parameters inherited from the parent model. It should be emphasized that ballast properties in flood conditions are collected from a previous study [18], which is a rare study considering the ballast properties under flooding conditions.

Table 1. Material Properties

Element	Properties	Value
Rail	ρ^1 E^2 PR^3	7800 210 0.3
Bearer (Concrete, FFU)	ρ^1 E^2 PR^3	2500, 740 38, 8.1 0.2, 0.25
Rail Pad	k^4	1300
Ballast* (0%, 29%, 57%, 100%, 114%)	k^4 c^5	14.6, 13.7, 13.4, 11.3 ,6.5 1.16, 1.77, 1.82, 2.37, 3.3
Primary Suspension	k^4 c^5	1.15 2.5

[1] Density (kg/m3). [2] Modulus of elasticity (GPa). [3] Poisson Ratio. [4] Stiffness (MN/m). [5] Damping coefficient (kNs/m).
* Values at different water levels

The boundary conditions to solve differential equations are adjusted as follows. The ground beneath substructure and ballast bed is assumed to be rigid and therefore, the inelastic springs are fixed at one end. The beam elements are allowed to move in their vertical plane and rotate in their pitch axis. In the case of loss of ballast contact, the spring elements are removed from the model to represent the damage. The boundary conditions for the vehicle allow directional and rotational movements, excluding lateral displacement and rotation around the vertical axis.

The vehicle used in the simulation has one bogie and two wheelsets. The geometry of car body and secondary suspensions are neglected as they have limited contribution to the dynamic forces. The weight of the car body is also added to the vehicle model and the mass per wheel is 10 tons.

The vehicle is assumed to be in motion on the through route of the turnout in facing direction. Four vehicle speeds (i.e. 25, 50, 75 and 225 kph) are applied in the simulation. It is noteworthy that no infrastructure manager would operate at 225 kph under flooding condition. The aim of a simulation with 225 kph is to test the effect of high speeds since the other values are expected to present a quasi-static behavior. Later, 225 kph will be referred to 'test speed' whereas the rest as 'operational speeds'. Lastly, the interface between wheel and rail is associated with Hertzian Contact Theory, where the contact forces are the product of contact stiffness and penetration.

The mathematical solver setting is set to the explicit time integration, a well-known method to solve differential equations in the time domain, offering a stability in high-frequency responses and large displacements. However, the time constant in the explicit method is determined by the length of the minimum element. Hence, the mesh size in the model is adjusted to obtain highly accurate results in a feasible simulation time. The other settings (i.e. output settings, termination time and so on) is either parameterized or used with recommended settings. Prior to the analysis, preloads are applied by a method called 'Dynamic Relaxation' to reach an equilibrium in terms of avoiding absurd vibrations as a result of gravitational forces. Apart from that, a profile of track surface irregularities is introduced into the model to mimic the real field conditions. The process of extracting the profile is detailed in [25]. As aforementioned, the model is validated by comparison with the validated parent model. The reason of that is the shortage of sufficient field data for extreme events within the literature.

2.3 Interpretation of Outputs

A commercial program, LS-Dyna is used to solve that specific problem mentioned above. Despite its powerful solver, the program has no strong interface to visualize the results. Thus, the outputs from the simulation are transferred to another commercial program, MATLAB for the interpretation. The code is written to scan, categorize all the nodal information and then calculate the twist values.

As previously explained in the introduction, the twist is the difference between cross-level at certain intervals. It is calculated by the formula in Eq. 1, in metric units, or Eq. 2, in terms of gradient. Following the calculations, the measurements are compared to the thresholds specified by the experience of infrastructure managers. In this study, the guidance on twist faults [22] is followed here. In this guidance, the basis length of interval is 3 meters (distance between point A and point B) and the same thresholds are implemented for both static and dynamic measurements. Here, the dynamic twist concept is adopted. One should bear in mind that in practice, static measurements are conducted by a cross-level gauge at every 2 measurement points at the interval. Whereas, dynamic measurements are obtained by the interpretation of the data (i.e. position information, speed and so on) at a resolution length (i.e. 0.5 m). To further explain the nuance, the railway signaling terms could be referred. Basically, if the static measurement is a fix blocked system, then dynamic measurements are a moving block system. Finally yet importantly, the thresholds and their interpretation for twist faults are given in Table 2.

$$\text{Twist} = \text{Crosslevel at point B} - \text{Crosslevel at point A} \tag{1}$$

$$\text{Twist} = \frac{\text{Crosslevel at point B} - \text{Crosslevel at point A}}{\text{Distance between point A and point B}} \tag{2}$$

Table 2. Thresholds for twist defects [22]

Gradient	Difference (mm)	Action
>=1 in 600	<=5	No action
1 in 508	6	Monitor and Repair
1 in 218	14	Monitor and Repair
1 in 200	15	Repair in 10 days
1 in 127	24	Repair in 10 days
1 in 122	25	Repair in 36 hours
1 in 92	33	Repair in 36 hours
1 in 90	34	Close the line
<=1 in 71	>=43	Close the line

3. Results

In this section, the outcomes of the simulations representing various flooding scenarios are presented for concrete and FFU bearers. As aforementioned, concrete bearers are the most common bearer technology and well-known for their durability in adverse environments. On the other hand, FFU is a promising bearer technology which might replace concrete bearers [27]. In other words, the results are shown for current and potential bearer technologies.

3.1. Comparison of Elastic and Inelastic Models in terms of Twist Behavior

The parent model has adopted the elastic beam foundation theory with elastic springs, which has been confirmed as sufficiently valid by many past and present studies but is believed to be underestimating the displacements against gravity vector. To show the difference between elastic and inelastic beam models, a comparison at 75 kph of vehicle speed is presented in Figure 4. The results in Figure 4a belong to fully supported turnout structures with both material models. As can be seen from the figure, they yield similar results. But as differently, the elastic model has minor and hardly perceptible fluctuations at crossing section (sleeper 50-65), having the maximum discrepancy of 0.2 mm. As no any disagreement observed in Figure 4a, it can be concluded that applications of both elastic and inelastic springs could be acceptable in fully supported structures.

Large deviations in twist values by using elastic and inelastic springs could be observed in Figure 4b and 4c, where a ballast section of 10 bearer length is completely (Figure 4b) or partially (Figure 4c) failed at crossing section. It is noteworthy that the selection of the case is based on the fact that it is likely to have higher twist values at the selected position where the impact forces occur. Furthermore, the figure of the supported case (Figure 4a) shows any signs to select a case which considers failures at the switch section. As illustrated in both figures, the magnitudes of discrepancies are 1.7 mm and 3.2 mm for fully and partially damaged sections, respectively. Associated with the reference the guidance, the magnitudes of the deviations are prominent and could change the category of the potential failure. Hence, it is recommended to apply inelastic spring models to similar simulations to avoid any underestimation at the outcomes.

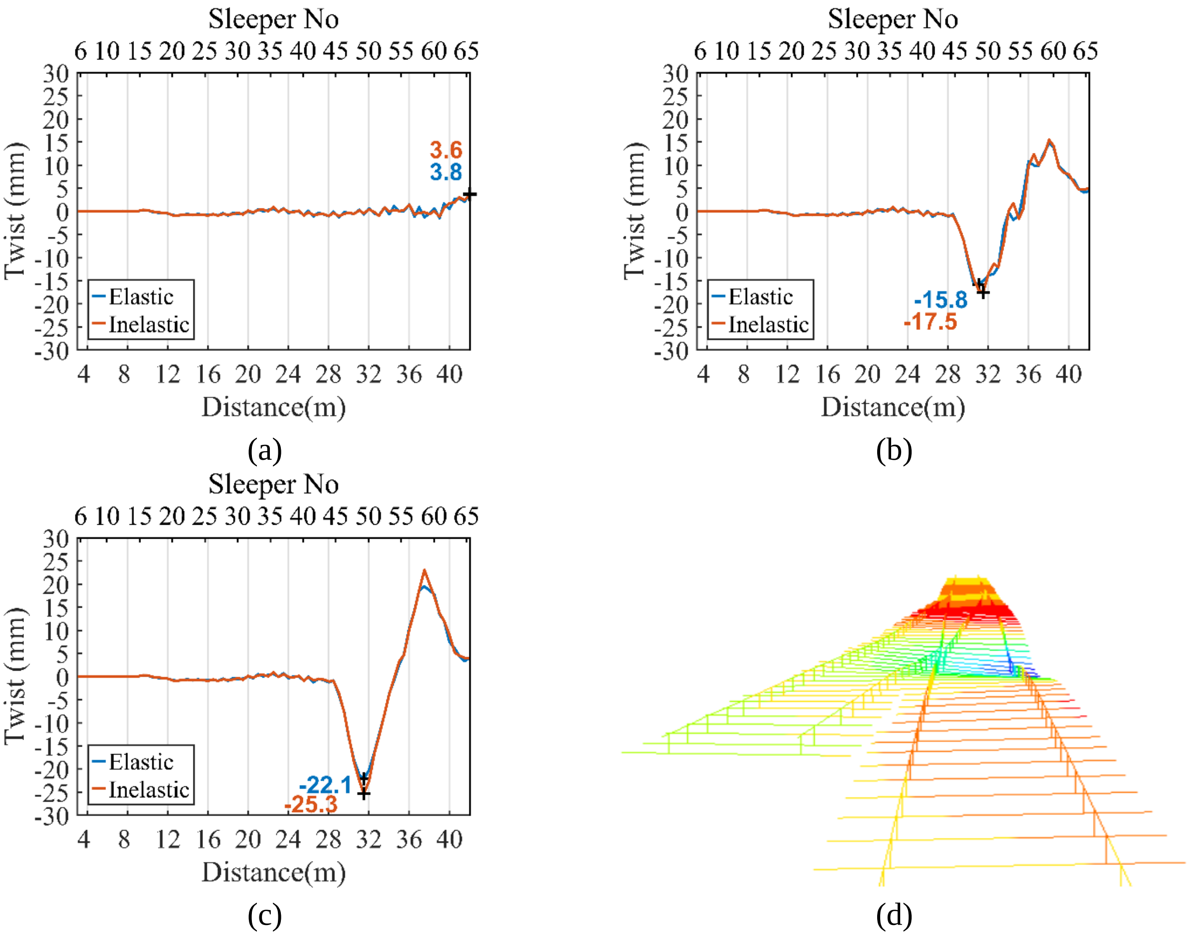

Figure 4. *Comparison between elastic and inelastic models (a) no ballast damage (b) complete ballast damage (c) partial ballast damage and (d) a view from simulation environment showing vertical displacements for partial damage scenario*

3.2. Relation between surface water level and twist values

It was reported in [18] that the surface water level could change the ballast properties and influence the dynamic behavior of the vehicle and track structure. Considering the asymmetrical structure of turnouts, the effects of various water levels could play a critical role and therefore, it is tested in the simulation for a well-supported turnout.

3.2.1. For a Turnout with Concrete Bearers

The outcomes of the simulations show that the variations in ballast properties are less likely to cause substantial twist values on well-supported turnout structures with concrete bearers regardless of the vehicle speed as presented in Figure 5. Despite oscillations caused by high test speed in Figure 5d, there was still no reliable link between the surface water level and twist values.

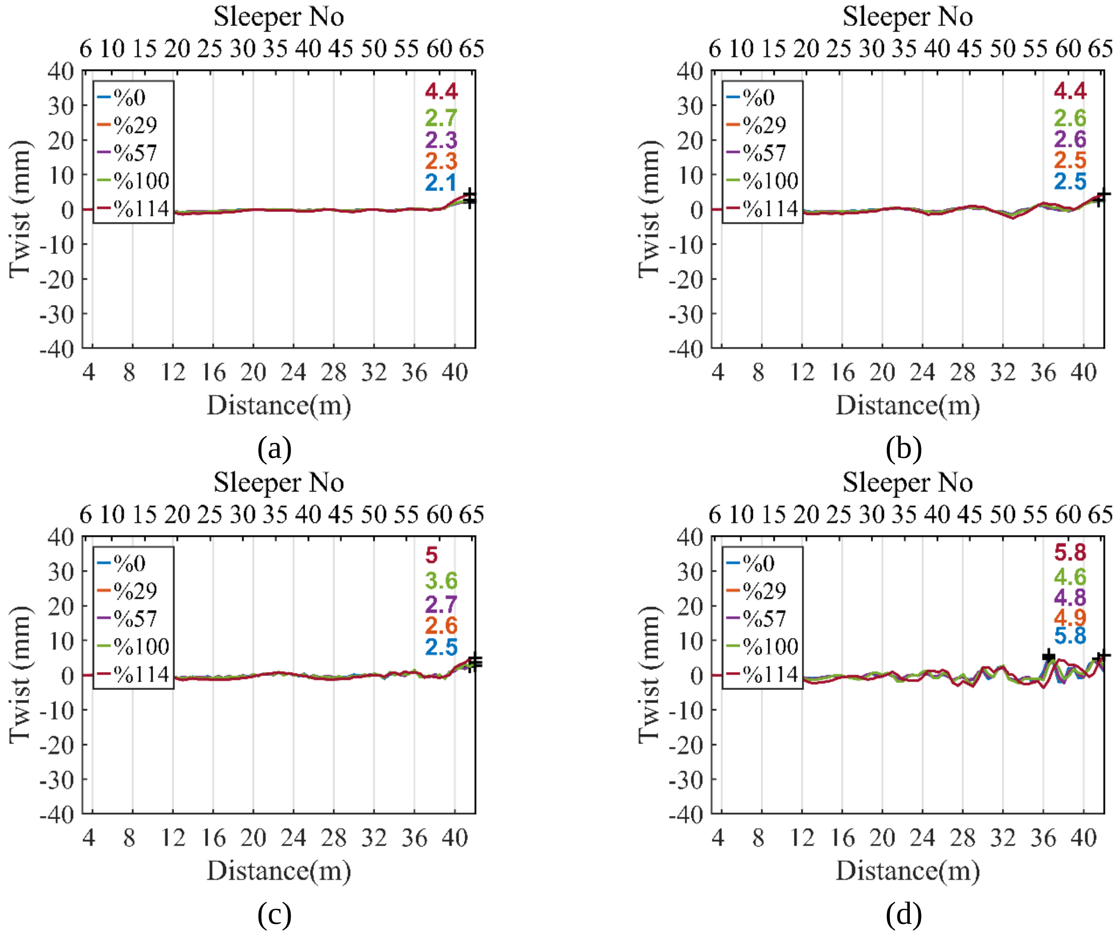

Figure 5. *The relation between surface water levels and twist value for a turnout with concrete bearers (a) at 25 kph (b) at 50 kph (c) at 75 kph and (d) at 225 kph*

3.2.2. For a Turnout with FFU Bearers

To address the question of whether the response of a well-supported turnout system with FFU bearers differs from the one with concrete bearers, further simulations are conducted with FFU bearers and presented in Figure 6. As illustrated in the figures, no evidence was found for remarkable differences in the comparison of two models since the turnout structure with FFU bearers have the twist values in the range of normal operation limits, close to the design limit of zero twists. The impact of vehicle speed might be perceptible in Figure 6d as small changes in twist values, similar to the case with concrete bearers. In contrast to the previous simulation, the magnitude of twist values are relatively smaller for the turnout structure with FFU bearers. With respect to the outcomes of both simulations, it can be concluded that variable surface water level could pose a limited threat unless the ballast bed loses its structural integrity.

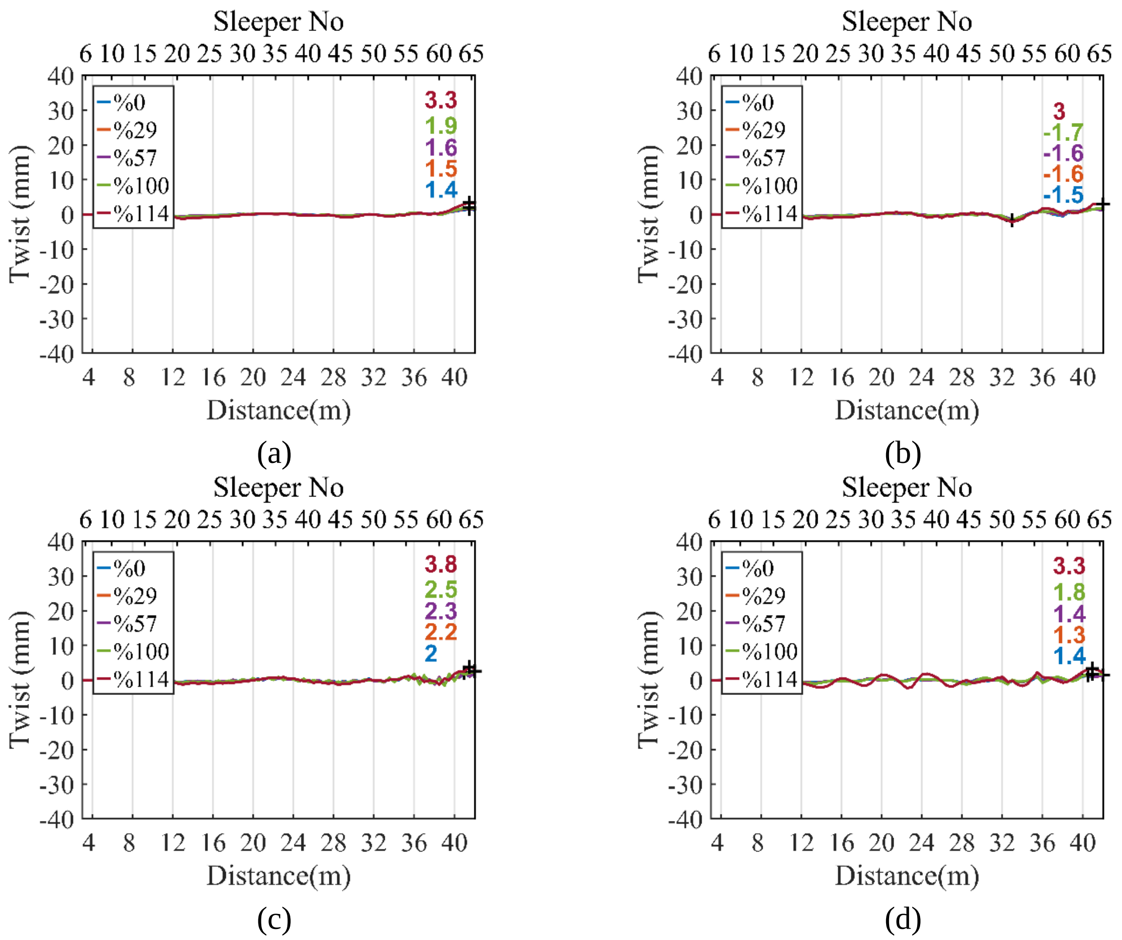

Figure 6. *The relation between surface water levels and twist value for a turnout with FFU bearers (a) at 25 kph (b) at 50 kph (c) at 75 kph and (d) at 225 kph*

3.3. Completely damaged ballast structure

Based on the observations from [21], it could be said that one of the expected damage type after flooding is the destruction of ballast bed, completely. Therefore, scenarios for that possibility is tested in the simulation and results are presented in this section.

3.3.1. At the Switch Section For a Turnout with Concrete Bearers

The relation between twist values and hypothetical cases of turnouts with concrete bearers which have no ballast support at the switch section is lower than expected, as depicted in Figure 7. Based on the guidance, it can be stated that the twist values here appear to be in the range of normal operation limits, including test speed 225 kph. Moreover, no correlation is observed between the magnitude of vehicle speed and twist value. Nevertheless, it is perceptible from the figure that the higher vehicle speed and lack of ballast support result in instability of the turnout-vehicle system, in relation to the dynamic behavior of the vehicle. In other words, the volatility in figures is striking and the vehicle is possibly bouncing on one rail and then the other rail, consecutively. This behavior is dominant when the length of the unsupported section is equal to ten bearers. As can be seen from the figure, there is an interdependence between the length of the unsupported section and twist value. The longer the unsupported section is, the higher the twist value would be.

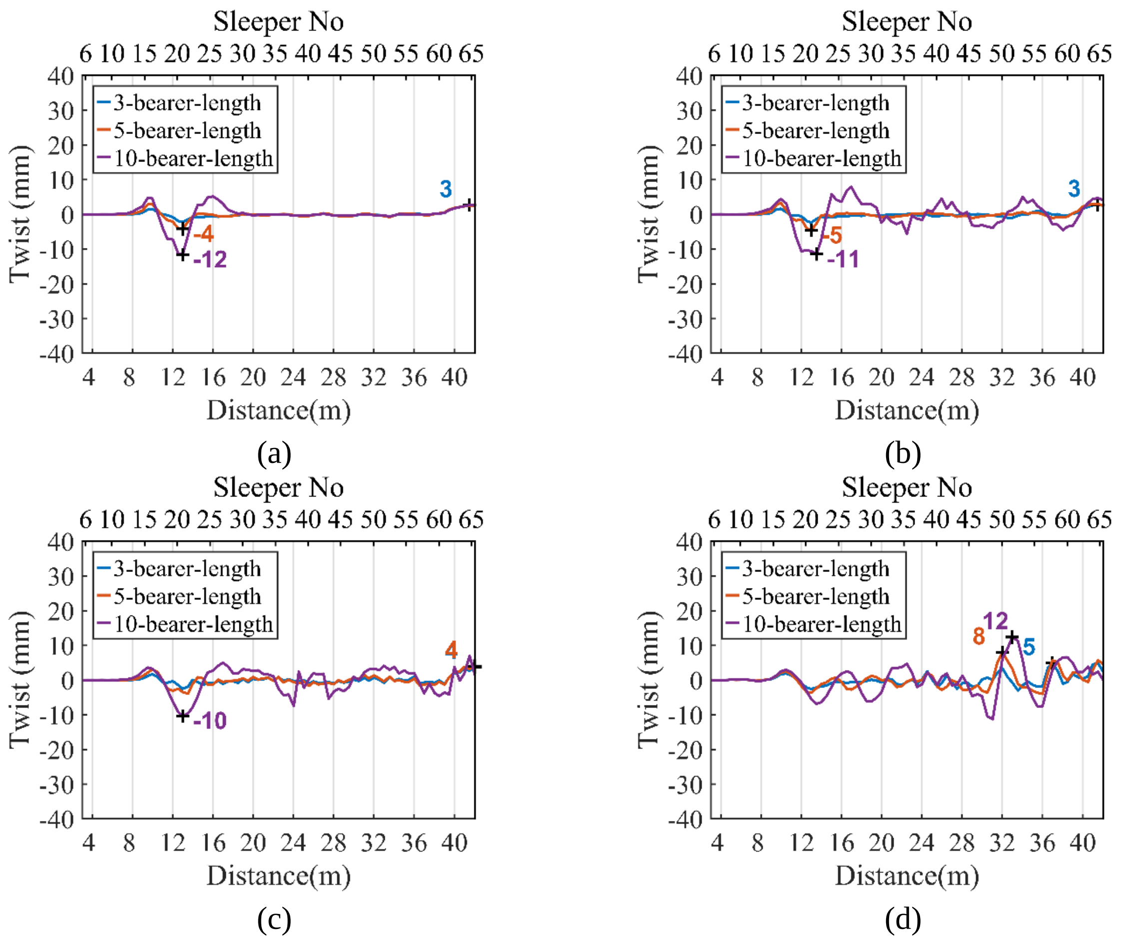

Figure 7. *Twist values for a turnout with concrete bearers when the ballast structure is completely damaged at switch section (a) at 25 kph (b) at 50 kph (c) at 75 kph and (d) at 225 kph*

3.3.2. At the Crossing Section For a Turnout with Concrete Bearers

In case of the fully unsupported bearers at the crossing section, the response of the turnout system to dynamic loadings become more distinctive (Figure 8). The magnitudes of twist could reach 19 mm at operational speeds and 33 mm at test speed. With reference to guidance, 19 mm twist fault requires corrective action in 10 days and 33 mm twist fault must be removed within 36 hours. A substantial difference between operational speeds and test speed indicates a correlation between vehicle speed and degree of twist fault when the ballast support is missing at crossing section. Interestingly, such correlation was missing in the previous section which has no ballast support at switch section. A closer investigation on the simulation model clarifies the reason as the impact forces introduce a significant rolling motion for the vehicle, influencing load distributions on rails. Furthermore, the structure of the turnout at that point have a more stiff structure at center rather than the edges. What would be similar is that, the length of the unsupported section has a strong influence on the twist value as longer sections produce higher twists. The guidance offers that the cases of 5 and 10 bearer length at operational speed need maintenance within 10 days whereas the case of 3 bearer length has no time limit for maintenance but a remark stating ‘monitor and repair’. At the test speed of 225 kph, maintenance must be scheduled within 10 days and 36 hours for both 5 and 10 bearer lengths, respectively. It is noteworthy that the dynamic behavior of the vehicle after passing the turnout is ignored due to the settings to decrease the simulation time since the focus is made on the turnout structure rather than vehicle behavior. However, vehicle instability as observed in the previous section is expected to continue here. Consequently, it seems that an operation in the cases of 5 and 10 bearers are highly risky and therefore it is recommended to halt any operations immediately in such a cases.

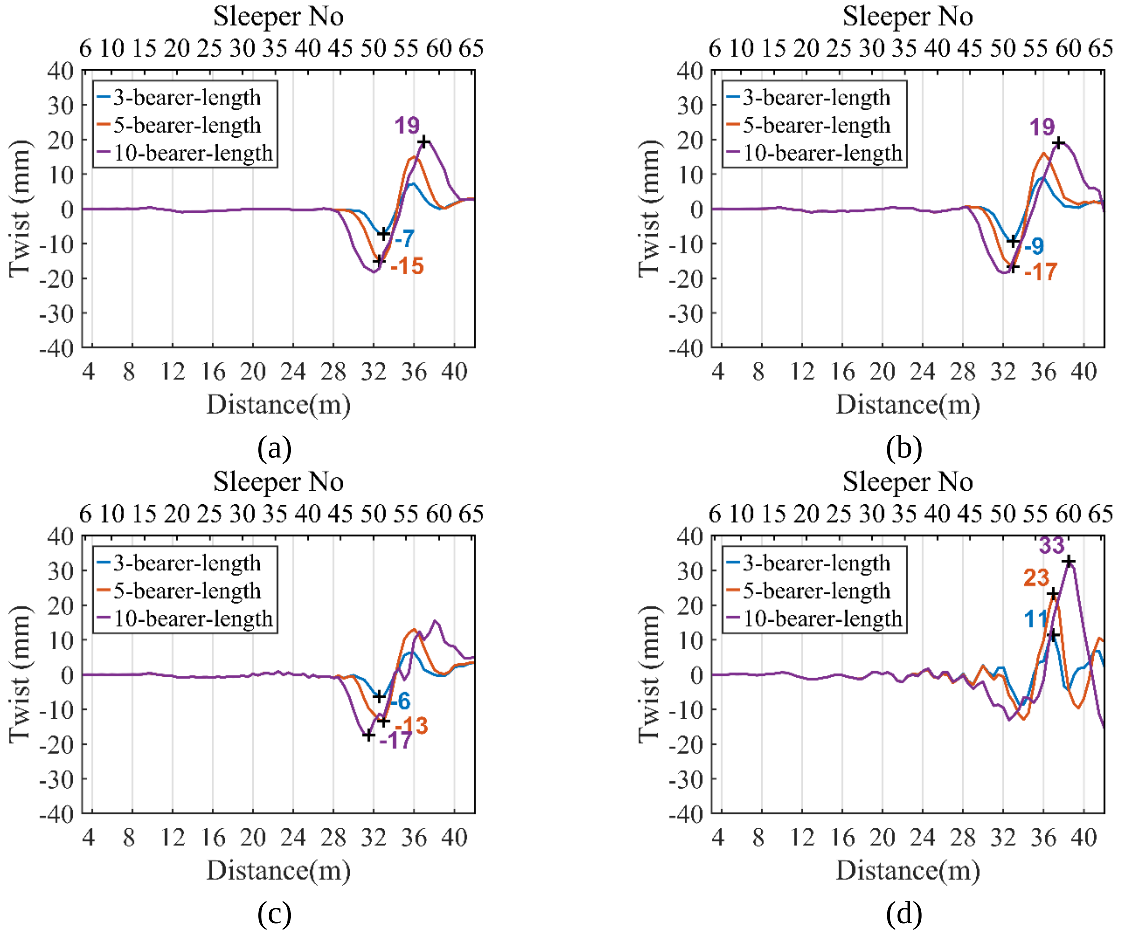

Figure 8. *Twist values for a turnout with concrete bearers when the ballast structure is completely damaged at crossing section (a) at 25 kph (b) at 50 kph (c) at 75 kph and (d) at 225 kph*

3.3.3. At the Switch Section For a Turnout with FFU Bearers

In comparison to simulations with concrete bearers, no clear differences were observed when the simulations were conducted for a turnout with FFU bearers that has no ballast support at switch section. The results are presented in Figure 9. The magnitudes of twist values are in a similar range in all cases. Likewise, the vehicle speed has no recognizable impact on twist values and vehicle instability seems to be effective and decisive. Consequently, there appears to be not advantageous in terms of using FFU or concrete bearers at switch section when the structural integrity of the ballast structure is completely damaged.

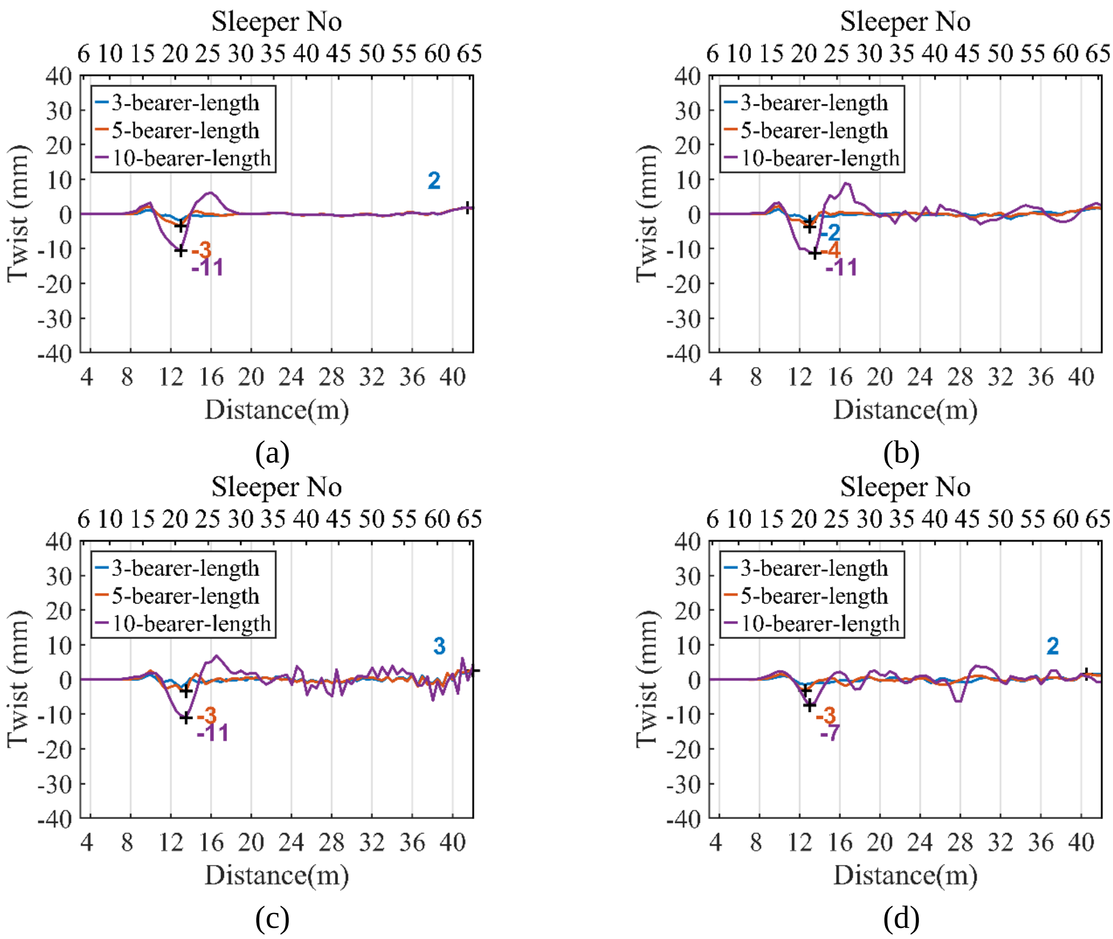

Figure 9. *Twist values for a turnout with FFU bearers when the ballast structure is completely damaged at switch section (a) at 25 kph (b) at 50 kph (c) at 75 kph and (d) at 225 kph*

3.3.4. At the Crossing Section For a Turnout with FFU Bearers

When a turnout structure with FFU bearers has a completely damaged ballast structure or fully unsupported bearers, the magnitudes of twist values are in similar but can reach up to 23 mm at operational speeds requiring maintenance within 10 days (Figure 10). Moreover, there are clear discrepancies in twist values with respect to the number of unsupported bearers. Here the longer unsupported section is, the more severe is the twist. Furthermore, twist values considerably increase at the test speed, indicating the interconnection between vehicle speeds and twist values, and have the highest peak in the case of 10 unsupported bearers. Strangely, the difference between peaks of the 5 and 10 unsupported bearers is smaller than the its similar case of a turnout with concrete bearers. A closer look on the simulation environment shows that vehicle suffers from severe rolling motions and therefore, the reason why two cases produce such close results is likely related to the vehicle behavior which results in deviations over the load distributions during the simulation. Lastly, the results in the figures present evidence for the relatively poor performance of FFU bearers at crossing section when a turnout has no ballast support at that section.

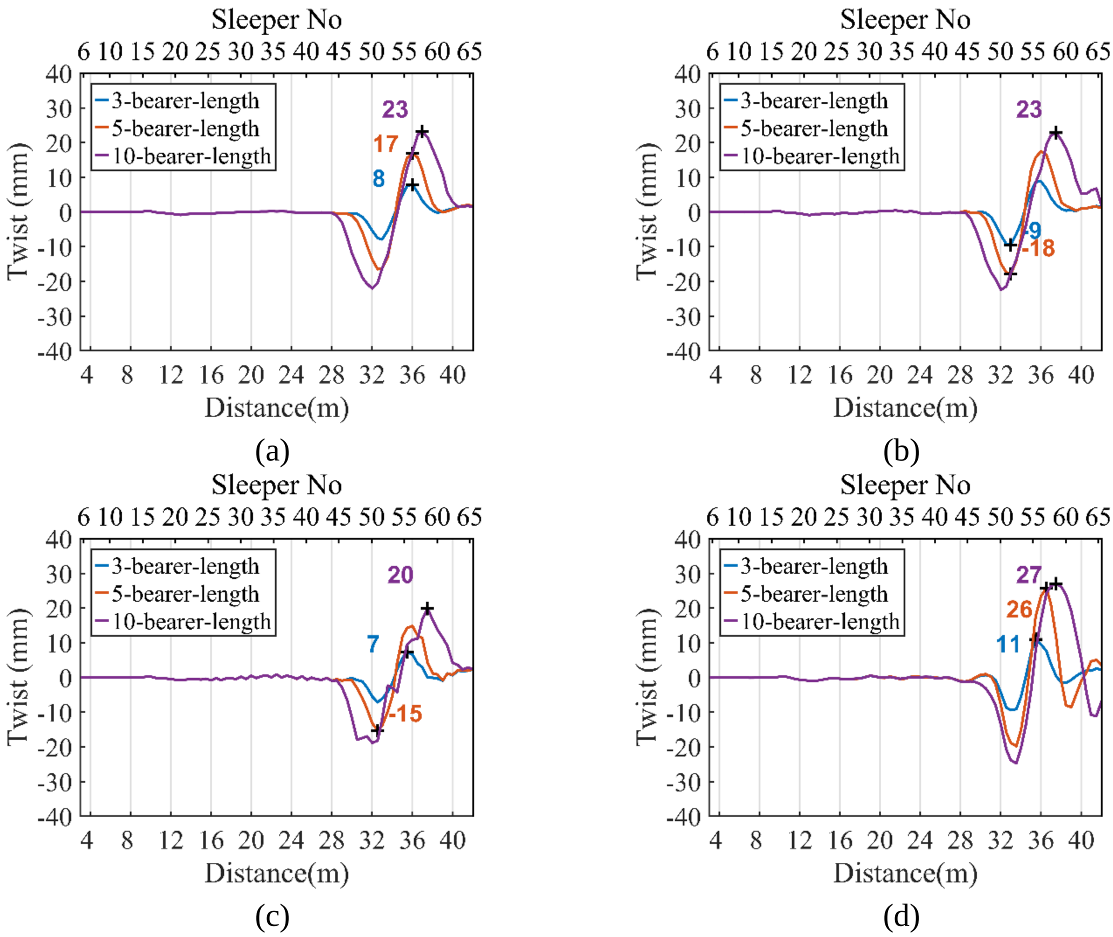

Figure 10. *Twist values for a turnout with FFU bearers when the ballast structure is completely damaged at crossing section (a) at 25 kph (b) at 50 kph (c) at 75 kph and (d) at 225 kph*

3.4. Worst Case Testing for Partially Damaged Ballast Structure

Several studies have reported that the response of a track or a turnout with partially supported sleepers and bearers rely on the length of the support that is available per bearer [30-33]. Those studies describe the challenge as either static or eigenvalue problem, posing constraints in terms of reflecting dynamic effects. Furthermore, many studies within the literature assume that the worst case happens when the sleepers/bearers are fully unsupported [34-37]. However, the twist thresholds and measurements in practice indicate that the magnitudes seem to be relatively higher than the magnitudes that are calculated for fully unsupported cases in the previous section. This outcome indicates that track structure suffer from a partial ballast support more, which amplifies twist fault. Hence, it is crucial to include and examine the effect of partial support, which is at high possibility during a flood, within this study. For that purpose, the first step is defined as determining the critical support length to run parametric simulations. Otherwise, the number of simulations covering all possible cases will be incredibly high and infeasible. Hence, a basis is selected among the aforementioned cases and the critical length is determined by testing that case with another parametric study associated with available support lengths per bearer. The selected case has the vehicle speed of 75 kph and the length of 10 bearers. The results are presented in terms of support loss, where 0% means the turnout structure is supported and 100% means completely failed ballast bed.

The results of the parametric study illustrated in Figure 11 demonstrate that the completely damaged ballast structure (100%) produces more twist than the cases of well-supported turnout structure and 25% support loss; on the contrary, it causes less twist in comparison to the cases of 50%, 75% and 90% support loss. It seems that 25% loss of the ballast support is relatively sufficient to support both rails and requires no action to repair according to the manual. On the other hand, the case of fully unsupported bearers is close to the maintenance limit that compels repairment within 10 days. It should be noted that fully unsupported case produces large variations in twist values along turnout as presented in figures, particularly in the case of unsupported bearers in switch section. This behavior is explained previously as an indication of vehicle instability, which would be investigated thoroughly but neglected in this study due to its specific coverage of scope. In other words, the completely damaged ballast structure could result in poor performance in terms of other parameters (i.e. derailment coefficient, etc.), in comparison to partially supported cases. Hence, it is imperative to remind the scope of the study where the performance of the system was assessed by twist values. According to Figure 11, the cases of 50%, 75% and 90% support loss are worse than fully unsupported bearers and within the threshold of the repair within 36 hours. Furthermore, 75% support loss appears to be the worst-case in both switch and crossing sections. Consequently, the assessment of partially supported bearer is grounded on the case of 75% support loss as a worst-case scenario.

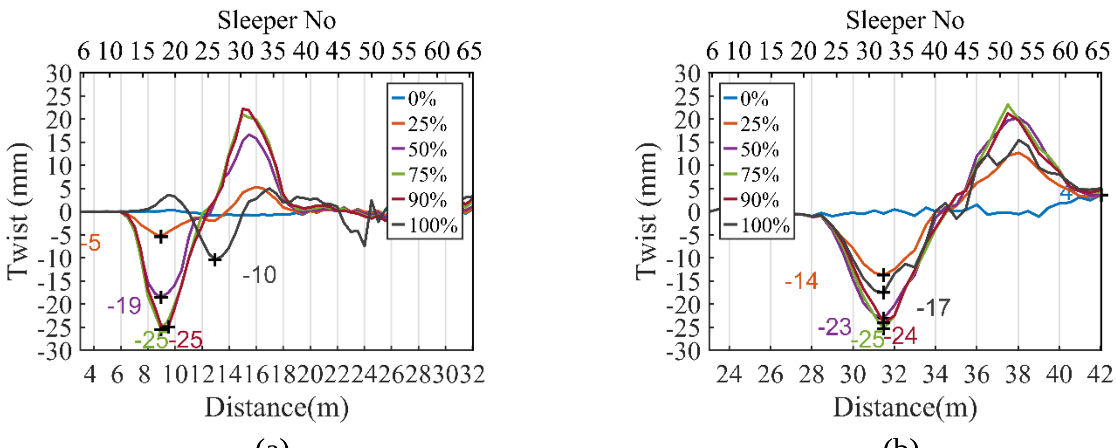

Figure 11. *Results of the parametric study on available support length for a turnout with concrete bearers at 75 kph (a) 10-bearer-length ballast structure damaged at switch section (b) 10-bearer-length ballast structure damaged at crossing section*

3.5. Partial Damaged Ballast Structure

With reference to the previous section, partially supported turnout structures seem to be more critical in terms of twist fault. The worst case scenario is when the ballast normalized support loss is 0.75. Therefore, worst case scenarios are tested in the simulation environment and the result for those cases are presented in this section.

3.5.1. At the Switch Section For a Turnout with Concrete Bearers

The outcomes of the simulation for a turnout with partially supported structure at switch section are presented in Figure 12. As illustrated in the figures, variations in the length of partially damaged section produce large discrepancies in the magnitude of twist value. The magnitudes are around 7, 16 and 25 mm for operational speeds; 11, 24 and 30 mm at test speed. As previously explained, the 225 kph is to test the effect of vehicle speed on twist values figuratively. It is obvious that higher speed significantly contributes to twist value at switch section in the case of partial support. Furthermore, the fluctuations in twist values exhibit strong indications of instabilities in the vehicle-track system. With reference to the guidance, the cases of three partially supported bearers at operational speeds are within the acceptable limits on the contrary to the case of five and ten partially supported bearers, which should be corrected within 10 days and 36 hours, respectively. Apart from those, the results show that the three partially supported bearer case at test speed is permissible; however, the five partially supported bearer case requires maintenance within 10 working days and ten partially supported bear case has to be repaired within 36 hours.

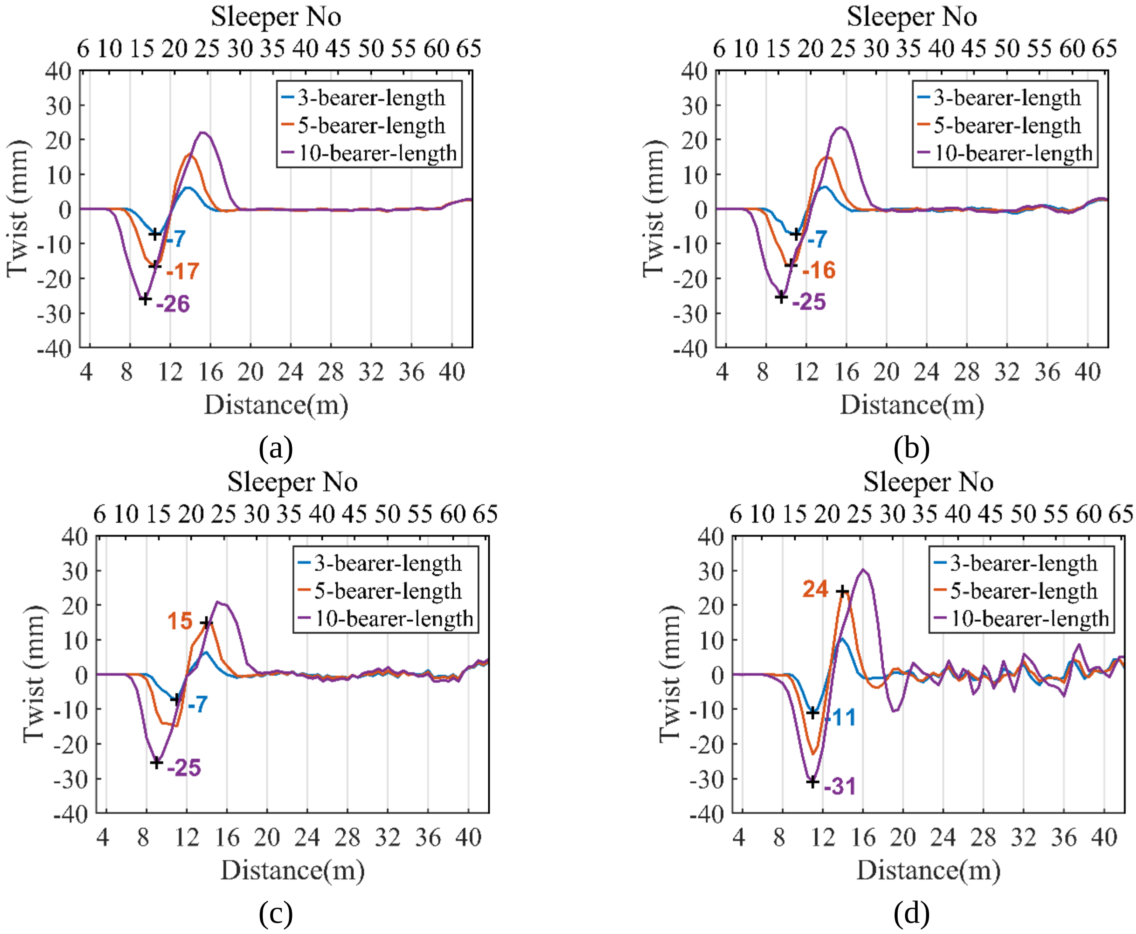

Figure 12. *Twist values for a turnout with concrete bearers when the ballast structure is partially damaged at switch section (a) at 25 kph (b) at 50 kph (c) at 75 kph and (d) at 225 kph*

3.5.2. At the Crossing Section For a Turnout with Concrete Bearers

When partial ballast support is available at crossing section for a turnout with concrete bearers, twist behaviour, as illustrated in Figure 13, seems to have similar characteristics with the corresponding cases in the previous section. The magnitudes of twists are close to each other in all scenarios at operational speeds. The only case confirming different outcomes is observed at 225 kph, which also presents the contribution of the vehicle speed into the magnitude of twist. The vehicle speed amplifies the results up to 6 mm, depending on the number of unsupported bearers.

Furthermore, it produces oscillations in twists values before the crossing nose, which is recognizable in the figures. Considering the thresholds in the guidance, all cases with three partial bearers are below the maintenance thresholds. The cases with five partially supported bearers at 25, 50 and 75 kph are within the limit of 10 working days. The rest of the cases are subjected to the classification that necessitates the repair in 36 hours.

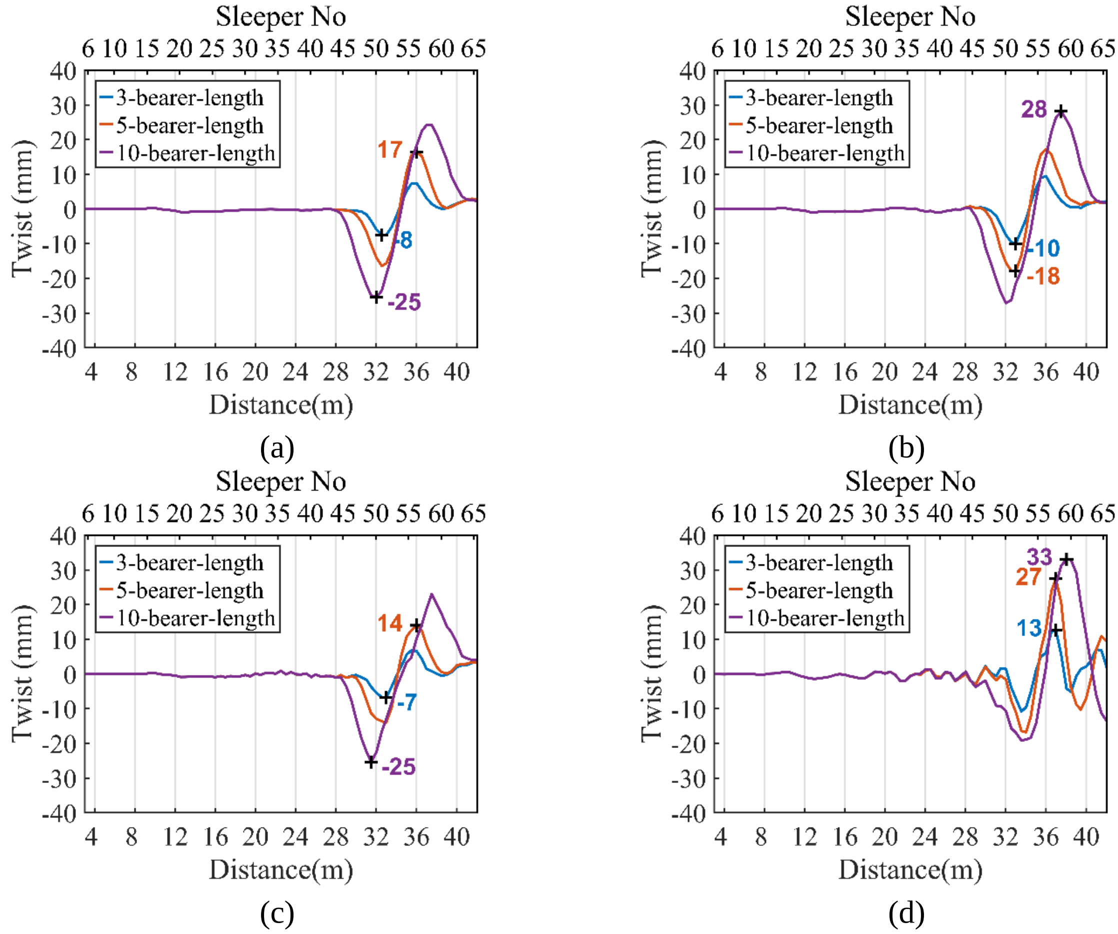

Figure 13. *Twist values for a turnout with concrete bearers when the ballast structure is partially damaged at crossing section (a) at 25 kph (b) at 50 kph (c) at 75 kph and (d) at 225 kph*

3.5.3. At the Switch Section For a Turnout with FFU Bearers

FFU bearers show poor performance in terms of dynamic twists in comparison to concrete bearers in a similar case. Figure 14 presents the performance of a turnout having partially supported FFU bearers at switch section. In the worst case, the dynamic twist could reach 38 mm which requires an immediate response to close the line and stop the traffic. Aside from that, similar trends observed in concrete bearers are also valid for FFU bearers. The length of the partially supported section and the vehicle speed have recognizable impacts on twist values. Moreover, the effect of vehicle speed is not clear at lower speeds, except for negligible up and downs in twist values. The only positive outcome of FFU bearers in this scenario would be that FFU seems to provide more stability, considering the less instability at 225 kph after passing the partially supported section in comparison to the similar case for concrete bearers.

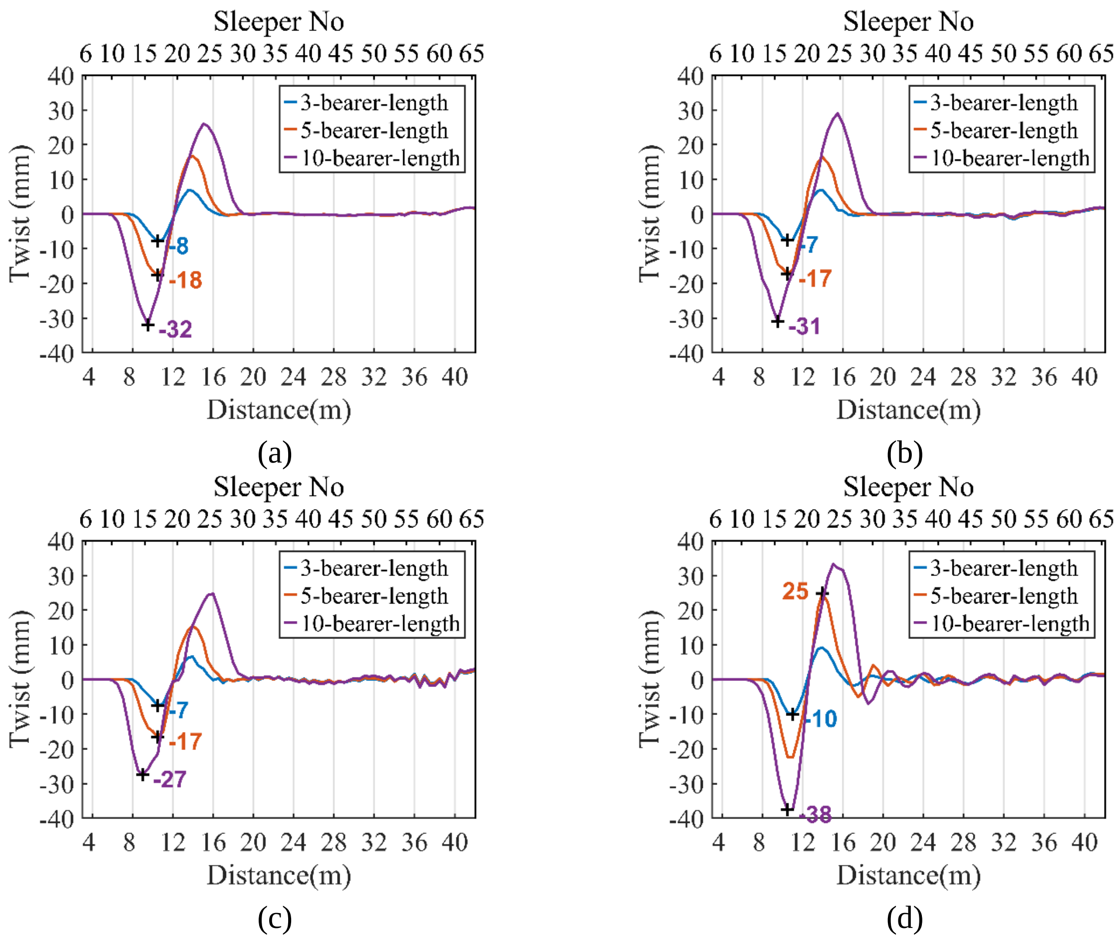

Figure 14. *Twist values for a turnout with FFU bearers when the ballast structure at switch section is partially damaged (a) at 25 kph (b) at 50 kph (c) at 75 kph and (d) at 225 kph*

3.5.4. At the Crossing Section For a Turnout with FFU Bearers

Dynamic twist values in the case of partial support for FFU bearers at the crossing section are illustrated in Figure 15. It seems that the performance of FFU bearers at crossing section surpasses the performance at switch section since the magnitudes of twists are lower at crossing section. Interestingly, the case with ten-bearer-length partial support at 225 kph produces less twist value than the expected. The twist values for that case are frankly close to the case with five-bearer-length partial support. This is believed that it has been resulted from the vehicle instability once again, affecting load distributions between rails.

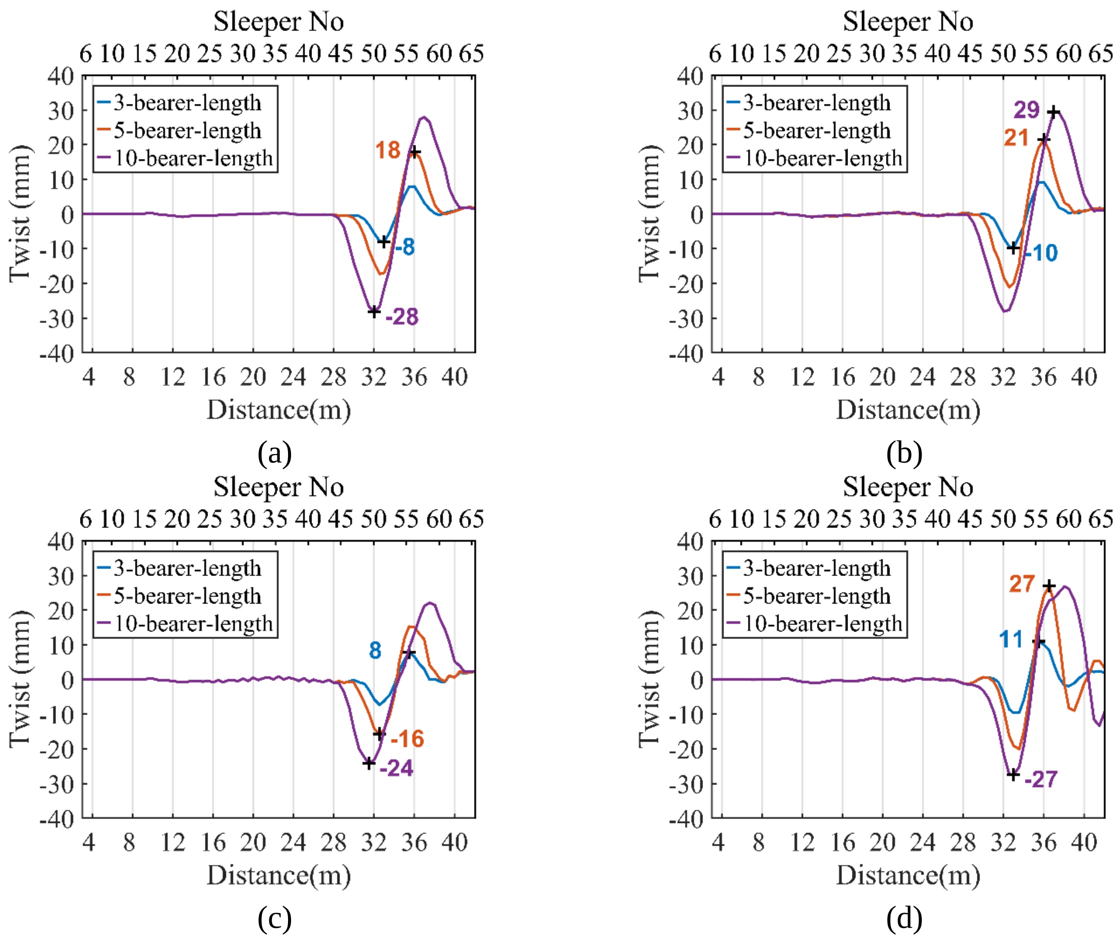

Figure 15. *Twist values for a turnout with FFU bearers when the ballast structure at crossing section is partially damaged (a) at 25 kph (b) at 50 kph (c) at 75 kph and (d) at 225 kph*

4.Discussions

In the last 15 years, 132 derailment reports have been published in the UK by Railway Accident Investigation Branch [21]. 10% of those reports addresses the cause of the accidents as 'twist defects'. Furthermore, surprisingly, 80% of those accident reports of twist defects is directly related to the turnouts. Those numbers show the impact of twist defects despite the standards in effect [38, 39, 40], which ensure certain capabilities for vehicle-track couple such that the vehicle should be able to run safely on a twisted track within the acceptable level according to maintenance manuals. Aside from twist faults, 5% of the accident reports is linked to damages caused by flood, half of which is embankment failure, including derailment due to excessive twist. Consequently, this study is crucial as it highlights the risks of running a vehicle on a twisted track under flood conditions.

The study recommends that employing inelastic spring models as track support is more appropriate for the simulations of turnouts since they include the displacements against the gravity vector and excludes unrealistic tensile forces produced by elastic springs.

The stages of derailment due to twist defect involve the dynamic behavior of not only the track but also the vehicle. A twist defect disturbs the load distribution of vehicle and cause lateral forces due to cross-level difference. Accordingly, the derailment criteria (Y/Q) exceeds the critical threshold and wheel flanges climb the rail, ending up with a derailment. As the study neglects the dynamic behavior of the vehicle, including constraints on the lateral direction, the study itself has limitations to comment on the risk of derailment. However, it is capable of showing the potential risk from the maintenance point. In other words, Table-3 only summarizes the capacity of turnout system under flooding condition associated with the risk level indicated in the reference guidance.

In the current study, comparing several surface flood levels showed that the turnout structure exhibits low-risk behavior in terms dynamic twist as far as the structural integrity of the track is protected. . However, it is noteworthy that the validity of the results is strongly related to the validity of the study that was used to obtain material properties. In reality, determining the ballast and subgrade material properties under flooding is a challenging task since flooding could alter the properties of those in magnitudes at several locations. Nevertheless, if it is assumed that the properties homogenously change along the turnout in case of flooding, then the results for surface water levels could be accepted as valid ones and the operation might be conducted on a fully supported turnout structure, particularly in inevitable situations.

What is surprising outcome of the study is that there is a weak correlation found between the vehicle speed and dynamic twist. A more detailed investigation shows that the vehicle stability become problematic at higher speed; load distributions are uneven and unique in each case and therefore, twist values exhibit unexpected patterns in some cases. Furthermore, track surface irregularities introduce a nonlinear track response. To show the correlation between vehicle speed and dynamic twist, further simulations could be conducted for many more speed values, nevertheless, it might be infeasible requiring a huge number of simulations.

The correlation between the length of damaged section and the system response is also confirmed in this study. The longer the damaged section is, the stronger is the system response (higher dynamic twists). It was found that the damaged ballast section with 3-bear-length in all cases induce twists below maintenance limits due to sufficient track stiffness whereas other cases pose threats to the operation. Further investigations in the field could be conducted to confirm this outcome that might be useful while arranging maintenance schedule. Likewise, field tests could be conducted to investigate on another outcome of this study, which shows that the worst case in terms of dynamic twist is when the available ballast support is 25%. Here, it is noteworthy that the damage propagation is assumed from one side. In other words, 25% percent ballast support is available at bearer edges not at the center of the bearers.

Finally yet importantly, the performance of FFU bearers is relatively lower than the concrete bearers in terms of twist defects in most cases. Indeed, it might be the first time as is exhibited that FFU could show poor dynamic performance under partially damaged ballast structure, particularly in the event of flooding. Thus, it is recommended to conduct field experiments whether the partial support conditions for FFU bearers entitle higher risks or not when compared to concrete bearers. It should also be emphasized that FFU bearers produced lower twist values when the structure is well supported. Hence, FFU bearers could be effective where the risk of flood or structural damage is low. Further field investigations are also recommended for this outcome.

Table 3. Summary of maximum dynamic twists in mm, presented in color background related to risk profiles in the reference guidance (green: no action, blue: monitor, yellow: repair in 10 days, orange: repair in 36 hours, red: close the line)

Flood Scenarios	Cases		Concrete Speed (kph) 25	50	75	225	FFU Speed (kph) 25	50	75	225
Well-Supported	Normalized Water levels	0%	2.1	2.5	2.5	5.8	1.4	1.5	2	1.4
		29%	2.3	2.5	2.6	4.9	1.5	1.6	2.2	1.3
		57%	2.3	2.6	2.7	4.8	1.6	1.6	2.3	1.4
		100%	2.7	2.6	3.6	4.6	1.9	1.7	2.5	1.8
		114%	4.4	4.4	5	5.8	3.3	3	3.8	3.3
Partial Support	Switch	3-bearer-length	7	7	7	11	8	7	7	10
		5-bearer-length	17	16	15	24	18	17	17	25
		10-bearer-length	26	25	25	31	32	31	27	38
	Crossing	3-bearer-length	8	10	7	13	8	10	8	11
		5-bearer-length	17	18	14	27	18	21	16	27
		10-bearer-length	25	28	25	33	28	29	24	27
No Support	Switch	3-bearer-length	3	3	4	5	2	2	3	2
		5-bearer-length	4	5	4	8	3	4	3	3
		10-bearer-length	12	11	10	12	11	11	11	7
	Crossing	3-bearer-length	7	9	6	11	8	9	7	11
		5-bearer-length	15	17	13	23	17	18	15	26
		10-bearer-length	19	19	17	33	23	23	20	27

5.Conclusions

This study highlights the dynamic behavior of a turnout exposed to flooding condition and associates the outcomes with the dynamic twist concept that is measured to evaluate track performance. It also presents a comparison between the applications of concrete and FFU bearers in terms of performance against twist faults. The study involves FEM simulations at two locations, four vehicle speeds and three hypothetical flooding scenarios.

Prior to the main simulations, the discrepancy between elastic and inelastic spring models was presented by comparing three scenarios. It was found that the elastic spring model underestimated the displacements owing to the presence of tensile forces.

Results indicate that the effects of surface water levels have limited impacts on dynamic twist values when the turnout structure is supported. Furthermore, no clear evidence was found for a discrepancy between the uses of concrete and FFU bearers in that case.

The relevance of damaged ballast structure with the dynamic twist value is clearly supported by the outcomes of the simulations. In the case of completely damaged ballast bed or fully unsupported bearers, the dynamic twists values were lower than the expected values associated with the maintenance manuals. Therefore, partial loss of ballast support was also tested in the study. The study confirms the findings within the literature that there is a relation between the length of available support and the system response. It clearly presents the impact of partial support on the dynamic behavior of the turnout system. The results show that 25% of partial support is the worst case in terms of dynamic twist values, which is accepted as a basis for parametric simulations of the partially damaged structure.

Parametric simulations indicate that twist values seem to be independent from vehicle speeds below 75 kph, showing that the risks are same for any operations at such speed levels under flooding. Nevertheless, the vehicle stability seems to be a problem at higher speeds, particularly when longer sections are damaged. The shorter the section is, the lower is the twist and the more stable is the structure. Aside from those outcomes, the study emphasizes the disadvantage of application of FFU bearers in terms of dynamic twist value. In most parametric studies covering the fully or partially damaged ballast structure, it was found that concrete bearers yield lower twist values. Interestingly, parametric studies also reveal that the twist values in the case of partial support are higher at the switch section rather than crossing section, indicating the likelihood of the derailments is higher at switch section.

Even though the strong reliance exists between the validity of the model and the material properties collected from the literature, the study clearly demonstrates the effect of support conditions on dynamic twist values resulted from flood conditions. In other words, the study could also be accepted as a reference work that considers a turnout system laid on a relatively soft support structure. In conclusion, the study provides important findings that could help infrastructure managers while designing and maintaining turnout structures and selecting materials as support elements.

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