



## Continuously welded rails on temporary bridge decks

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### Abstract

The possibility of omitting rail expansion devices from the track configuration, when continuously welded rail is continued over temporary bridge decks, is investigated in detail. More specifically, the related rail track to temporary bridge interaction phenomena are analysed using finite element modelling. A parametric analysis assesses the additional rail stresses due to moving trainloads and temperature variations, based on stipulations provided in the unit identification code 774-3R. In addition, the model is expanded to a more complex structure that is able to simulate the buckling behaviour of the rail track using non-linear methods. It can be concluded that, depending on the magnitude of two main factors, the lateral ballast resistance and the amplitude of the initial track misalignment, a considerable reduction of the track stability might arise.

**Keywords:** railway track design, simulation, railway technology

## 1 Introduction

### 1.1 Continuously welded rails and track-bridge interaction

In recent years, continuously welded rails (CWR) have become an essential part of modern railway track structures due to their better maintainability, safety and riding quality compared to the former fish-plated track [2]. Because of these advantages trains can travel at higher speeds and with less friction. Therefore, this type of track is especially used for high-speed trains. For the general case of a continuously welded rail on an embankment, the displacement of the rails is prevented through the track fastening elements. These fasteners exert a clamping force onto the rail to transfer all vertical, longitudinal and transversal loads to the sleepers. This clamping is realised in such a way that all the

longitudinal movement of the rail is transmitted to the sleepers, since the longitudinal sliding resistance between rail and sleepers is far greater than the longitudinal sliding resistance between sleepers and ballast. If consequently, a thermal or traffic load is applied onto the rail, longitudinal forces will arise in the tracks as a result of the prohibited movement of the rails. Due to these longitudinal forces, continuously welded rail track can typically be split up into three different zones: a central zone, in which no displacements of the rails occur as a result of the ballast/sleeper resistance, and two so-called breather zones at each end of the central zone where displacements augment when approaching the expansion devices resulting in decreasing normal forces. If now continuously welded rails are continued over bridges the track and bridge are interlinked to each other, regardless of whether the track is directly fastened or laid on a ballasted bed [3, 4].

This interlinking affects the behaviour of one component on the other, which can be termed as the interaction between the bridge and the track. Movement of either one of them will result in forces on the other. As a result, interaction between track and bridge is manifested in the following way [1]: forces applied to a CWR track induce additional forces into the track and/or into the bearings supporting the deck and movements of the track and of the deck; any movement of the deck induces a movement of the track and an additional force in the track and, indirectly, in the bridge bearings. In order to prevent the need for rail expansion devices in CWR, zero longitudinal restraint (ZLR) fasteners might be useful [5]. These ZLR fasteners do not prevent any longitudinal displacement of the rails and therefore no (longitudinal) forces will be transferred to the bridge. However, a first disadvantage of this technique is that, if a rail fracture would occur, the gap at fracture could enlarge as a result of the absence of longitudinal anchoring and movement of trains passing over it, resulting eventually in a possible derailment. Also, when train loading is present, the longitudinal restraint will increase, reducing the favourable effect of these fastenings. Despite these disadvantages, this method has already been successfully applied several times [5-7]. Examples can be found at the 'Olifants River Bridge' in South Africa, the high-speed line Brussels-Lille [5], the Mission Valley West light rail extension in San Diego [3], etc.

### 1.2 Continuously welded rails and track-bridge interaction

In order to be able to perform maintenance works on the track bed, (e.g. when constructing a pedestrian tunnel or a sewer passage underneath the train tracks) without having to stop train passages, contractors often use a technique in which they entirely replace the existing track bed by a temporary bridge deck. Additionally, in situations where maintenance works on the track bed cannot be done while assuring stability of the tracks, temporary bridge decks are used as well. While a number of alternative maintenance methods exist internationally [6], the method described here is the standard solution for Belgium and most of the neighbouring countries in

Western Europe. Consequently, these temporary bridges are used and re-used almost daily on the railway network, since they are such an easy stabilisation for most of the regular localised maintenance and small tunnelling works.



Figure 1. Example of a twin girder (left) and a tube girder temporary bridge deck (right)

### 1.3 Continuously welded rails and track-bridge interaction

Three different temporary bridge deck configurations are frequently used in Belgium: a twin girder, a tube girder and an assembled girder configuration. The first two types are applied most frequently [7]. In the twin girder configuration, the temporary bridge deck is composed of two separate decks, each consisting of two metal girders. Depending on the length of the span, different sizes of metal girders can be used. These girders also include additional stiffening plates welded to both flanges. Transversal stiffeners connect both girders, as shown in *Fig. 1a*. These transversal stiffeners carry the bearing plates for the train tracks. The separate bridge decks for each rail are connected to each other using cross girders and bolts. The temporary bridge deck is usually placed on neoprene, or sometimes wooden, bearings, which in turn are situated on a concrete beam. The temporary bridge deck of tube girder configuration, shown in *Fig. 1b*, is composed of one massive tube girder, equipped with multiple bearing plates to which the train tracks can directly be connected. This massive tube girder is stiffened using additional steel plates, welded to the upper flange, and additional transversal stiffening plates. Similar to the twin girder configuration the tube girder temporary bridge deck is also installed on neoprene bearings, which are situated on a temporary concrete beam. In both situations, the ballast layers, as well as foundations, etc. are entirely replaced by the temporary bridge decks.

#### 1.4 Track-bridge interaction

If continuously welded rails (CWR) are continued over bridges, temporary or permanent, the track and the bridge are influencing each other, regardless of whether the track is directly fastened to the bridge or loosely installed on ballast. This mutual influence affects the overall behaviour of both, which is generally defined as bridge-track interaction. Movements of either one of both components will result in additional loads on the other component. More specifically, interaction between track and bridge is manifested in the following way. On the one hand, forces applied to a CWR track induce additional forces into the bridge and/or into the bearings supporting the deck and movements of the track and of the deck. On the other hand, any movement of the deck induces a movement of the track and an additional force in the track and, indirectly, in the bridge bearings.

Research has focussed extensively on the subject of this interaction during the past few years [4]. The ERRI (the European Rail Research Institute) Committee D 213 carried out an extensive study to analyse and assess these interaction forces [8]. The results of these studies were subsequently summarized in the form of a report, which is called UIC code 774-3R [1]. This report contains the loads to be considered, the detailed configuration of test models (geometry, boundary conditions, etc.) and the design requirements needed in order to prevent damage due to track-bridge interaction effects and has been accepted in Europe<sup>6</sup> as the general guideline for all interaction analyses and will therefore be the starting point as well as the calibration method for

all finite element models in this research. The forces causing the interaction between track and bridge are those that cause relative displacements between the tracks and the bridge, such as thermal expansion, braking, accelerations, deck rotation, creep, shrinkage, temperature induced movements, etc. Three different load cases should be considered according to the UIC code 774-3R [1, 5] for both driving directions of the moving train. All other loads will result in negligible differential movements between the tracks and the bridge. These relevant load cases, modelling specifically the track-bridge interaction, are:

- A distributed vertical load of 80 kN/m, situated on the tracks on one of both embankments, left or right, and on the tracks of the entire temporary bridge;
- A distributed brake load of 20 kN/m, working in the longitudinal direction of the tracks on one of both embankment as well as the entire temporary bridge;
- The influence of a temperature variation in the temporary bridge deck, quantified by both a temperature increase and a temperature decrease of 35°C.

Due to these forces, additional rail stresses will arise in the tracks and these additional stresses might, in some cases, become too large. This can result in track buckling when the additional loads result in large compressive stresses or rail breaks when the additional loads result in large tensile stresses. It should therefore be verified whether these additional rail stresses remain within acceptable limits, which are imposed by the UIC code 774-3R [1].

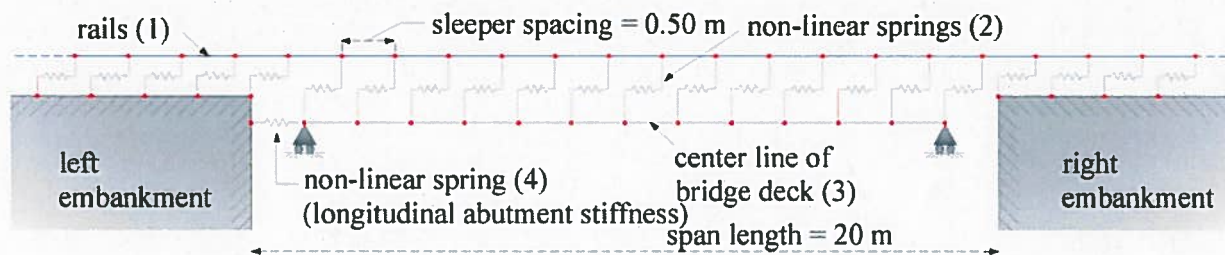


Figure 2. Example of a twin girder temporary bridge deck (Top) Example of a tube girder temporary bridge deck (Bottom)

## 2 Assessment of additional rail stresses in first parametric study on two-dimensional model

### 2.1 General composition

Using the model configuration shown in *Fig. 2* a first parametric analysis is performed in which the influence of multiple parameters of both the temporary bridge deck and its boundary conditions are analysed. Especially their influence on the arising stresses in the rails due to the three different interaction load cases is investigated. At the same time, the calculated rail stresses and relative rail/bridge displacements are compared with the limitations given in the UIC leaflet 774-3R [1]. This verification will be a clear indication that the basic assumptions of the finite element model are valid. Although a rail track consists out of 2 rails connected by sleepers, a one-dimensional element model represents the track. However, this beam model is designed to have identical beam behaviour as two UIC 60 rails, connected by sleepers, by modifying the stiffness parameters. The vertical position of the track, shown in *Fig. 2* as (1), in the finite element model coincides with the upper surface of the temporary bridge deck, which is an allowable simplification [1]. The temporary bridge deck itself is modelled using one-dimensional horizontal, flexible beam elements. These beam elements have the same properties (bending stiffness, material properties, etc.) as the actual temporary bridge elements. These flexible elements are vertically positioned at the centre of gravity of the temporary bridge and are depicted in *Fig. 2* as a thick line (3). A tube girder temporary bridge, such as the one shown in *Fig. 1b*, has a bending stiffness of about  $0,011 \text{ m}^4$  and are positioned at a distance of 237 mm under the rails. Additional vertical rigid beam elements, which have an infinite bending stiffness, connect the horizontal elements of the temporary bridge to the bearings or to the horizontal beam elements of the track. These rigid beam elements, which do not represent any actual part of the geometry, are shown in *Fig. 2* (thin lines) and take care of the vertical load transfer between rail track and temporary bridge. In addition, the horizontal connection between the track elements

and the embankments, which are ballasted, or the temporary bridge deck elements is modelled using bilinear longitudinal spring elements, shown in *Fig. 2* as (4). They are placed at a longitudinal distance of 50 cm between springs. The connection between track and ballast should follow a bilinear behaviour characterized by a threshold displacement  $u_0$  and a maximum longitudinal resistance  $k$ . This resistance depends on whether the track is loaded or unloaded and the corresponding values are 20 kN/m for unloaded track with good maintenance level and 60 kN/m for loaded track with a threshold displacement of 2 mm. If the displacement is higher, the ballast is assumed to have lost cohesion and no resistance is left. In case of a temporary bridge deck, wooden or elastomeric bearings are used. For this finite element model, two types of elastomeric bearings are used. A fixed bearing at the left side of the bridge is modelled by restricting the vertical displacements and by adding a bilinear spring in the longitudinal direction. The stiffness of this spring has been estimated as the stiffness of the compressed elastomeric material between deck and abutment and assuming an abutment with infinite stiffness [9]. In addition, a sliding bearing is installed at the right side of the bridge. This bearing is modelled by again restricting vertical displacement but now allowing horizontal displacement. In these finite element models, possible friction because of the longitudinal deformation of the sliding bearing is not taken into account. It is opted to model the embankments over a length of 100 meters. It is assumed that the influence of the track-bridge interaction on the continuously welded rail will have died out by then, which is also proven by the calculations. The maximum mesh size for all calculations is smaller than 5 cm.

### 2.2 Model properties

*Table 1* shows the model properties and reference values of the parameters investigated in this study. The longitudinal ballast resistance on the embankments is the resistance in the longitudinal direction of the tracks, acting on the sleepers because of the ballast, assuming good maintenance of the ballast [1]. The longitudinal fastener resistance on the temporary bridge deck

resistance represents the longitudinal resistance of the fasteners working on the tracks, assuming a good clamping force is present [1]. During the analysis of this model, each parameter is varied between values that are still acceptable from a practical perspective. Only the track properties remained constant. The properties of the temporary bridge deck used for this study are those corresponding to a typical temporary bridge deck used by the Belgian railways for a span length of 12 metres [1].

Table 1. Properties of the two-dimensional model

Span length	20 m
Longitudinal ballast resistance on embankments	Char. resistance (unloaded): 20 kN/m
	Char. resistance (loaded): 60 kN/m
	Elastic limit: 2 mm
Longitudinal fastener resistance on the temporary bridge deck	Char. resistance (unloaded): 40 kN/m
	Char. resistance (loaded): 60 kN/m
	Elastic limit: 0,5 mm
Longitudinal stiffness of the longitudinally fixed bearing	6000000 kN/m

### 2.3 Validation

Comparing the results from this finite element model with multiple test cases provided by the UIC code 774-3R[1, 5] as shown in Table 2, validated the final assembly of this finite element model. The names of these examples (first column) and the different load types are based on UIC code 774-3R. The load types are, e.g. vertical train load (left and right), braking or accelerating and temperature load of 35°C and were described earlier. This validation uses the properties of an actual permanent bridge deck as input parameters for both the finite element and the UIC calculations. The fact that the finite element model is able to predict the occurring stresses with an average error margin smaller than 10% ensures that the FEM results correspond to realistic interaction behaviour. Afterwards, the validated model was then adjusted and expanded to the specific case of a temporary bridge deck.

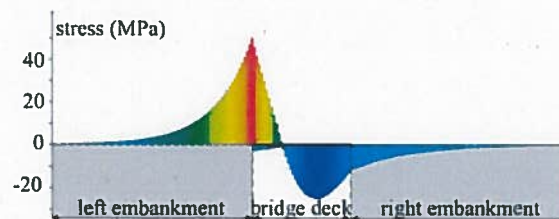


Figure 3. Longitudinal normal stresses in the rails for a train moving from left to right

### 2.4 Results of parametric analysis

Looking at the calculation results for a temporary bridge deck, based on the reference model of Table 1, a large variation of the longitudinal rail stresses can be noticed, when compared with a situation of ballasted track without a temporary bridge [3]. Fig. 3 illustrates the resulting longitudinal normal stresses in the rails due to the rail-bridge interaction when a train crosses the tracks going from left to right. In all figures, trains moving from left to right are characterised as 'direction 1', while 'direction 2' has the trains moving the other way around. The direction is relevant because of the longitudinally fixed bearing being situated at the left abutment: at this location, a high stress peak of about 50 MPa can be found. The stress at the other end of the bridge is lower, since the bearing at this embankment allow for longitudinal movement. The additional stress because of rail-bridge interaction is thus spread out over a certain length.

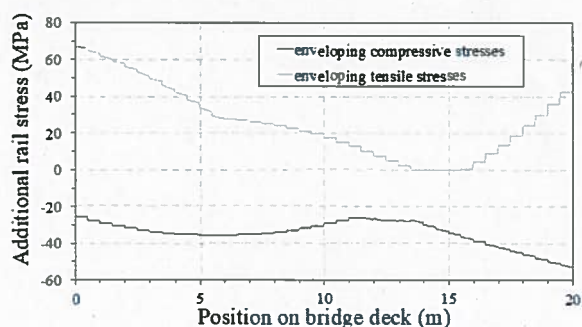


Figure 4. Envelope lines for additional rail stresses for trains moving in direction 1

If these values are compared with the rail stresses without rail-bridge interaction (i.e. ballasted track without temporary bridge deck), envelope lines for the maximal additional compressive and tensile stresses in the rail can be proposed, as

shown in *Fig. 4*. For this load combination, the maximal additional rail stress in tension equals 68 MPa, while the compression value equals 54 MPa. Both values occur at opposite ends of the temporary bridge decks, which seems quite logical. *Fig. 5* shows the main results when several parameters of the initial model are varied individually. The considered variations are: (1)

1. Span length of the temporary bridge, shown in *Fig. 5a*. The variation is kept within a realistic range of 10 to 30 meters. All other stiffness characteristics of the temporary bridge deck are kept constant. While this might seem strange, it is actually quite representative for practical applications. These temporary bridge decks are not specifically constructed for each application, but reused as often as possible. Temporary bridge decks with the cross-section used can in reality be used for span lengths of 10 to about 30 meters;
2. The variation of the bearing stiffness is shown in *Fig. 5b*. The parameter can be varied between a value that is 10 times lower and higher than the standard bearing stiffness for neoprene bearings. While this variation might seem large, temporary bridge decks can sometimes be supported directly on the abutments, on wooden blocks or on actual elastomeric bearings. The standard value corresponds with a rather stiff neoprene bearing;
3. Ballast quality (*Fig. 5c*). Depending on the degree of maintenance and compaction of the ballast, a different longitudinal ballast resistance may occur. Therefore, the influence of the longitudinal resistance of the ballast is investigated. Since no ballast is present on the temporary bridge deck only the longitudinal resistance of the part of the track situated on the embankments is altered. The characteristic longitudinal resistance for loaded tracks thus varies between 50 kN/m for a loaded ballasted track under moderate maintenance, over 60 kN/m for a loaded ballasted track with good maintenance to 70 kN/m for a situation with frozen ballast, representing

excellent maintenance and longitudinal resistance [1]. The resistances for unloaded track are 10 kN/m, 20 kN/m and 30 kN/m respectively [1];

4. The result of a variation of the longitudinal fastener resistance is shown in *Fig. 5d*. The influence of a reduced/enhanced clamping action of the fasteners situated on the temporary bridge deck is investigated here. The values are based the assumptions for moderate, good and excellent clamping force in UIC code 774-3R [1];
5. Finally, the variation of the temporary bridge deck bending stiffness is in *Fig. 5e*.

It becomes quite obvious that the span length and the bending stiffness of the temporary bridges are the most determining parameters when it comes to the resulting additional rail stresses. The other parameters that were examined, bearing stiffness, ballast quality and longitudinal fastener resistance, also have a clear influence on the resulting rail stresses but of a much smaller magnitude. The standard configuration used in this parametric analysis corresponds to the case of a temporary bridge deck with a span length of 20 metres. The bending stiffness of the temporary bridge deck is chosen equal to the stiffness of an actual temporary bridge deck used by the Belgian railways for a span length of 12 metres. The characteristics for the longitudinal track resistance and the loads applied to the model are both based on the values imposed by the UIC code 774-3R [1]. Therefore, if this configuration complies with the limitations given by the UIC code 774-3R [1] (maximum additional compressive rail stress 72 N/mm<sup>2</sup> and maximum additional tensile rail stress is 92 N/mm<sup>2</sup>), one can assume that for this span length and temporary bridge deck configuration it is allowed to use CWR track over the bridge structure without providing an additional expansion device in the tracks. When considering the graphs shown in *Fig. 5* it can be conclude that not only for the standard configuration, but also for all the other test cases examined (except for the one with a span length of 30 metres), the additional rail stresses remain below the limitations imposed by the UIC code 774-3R [1].

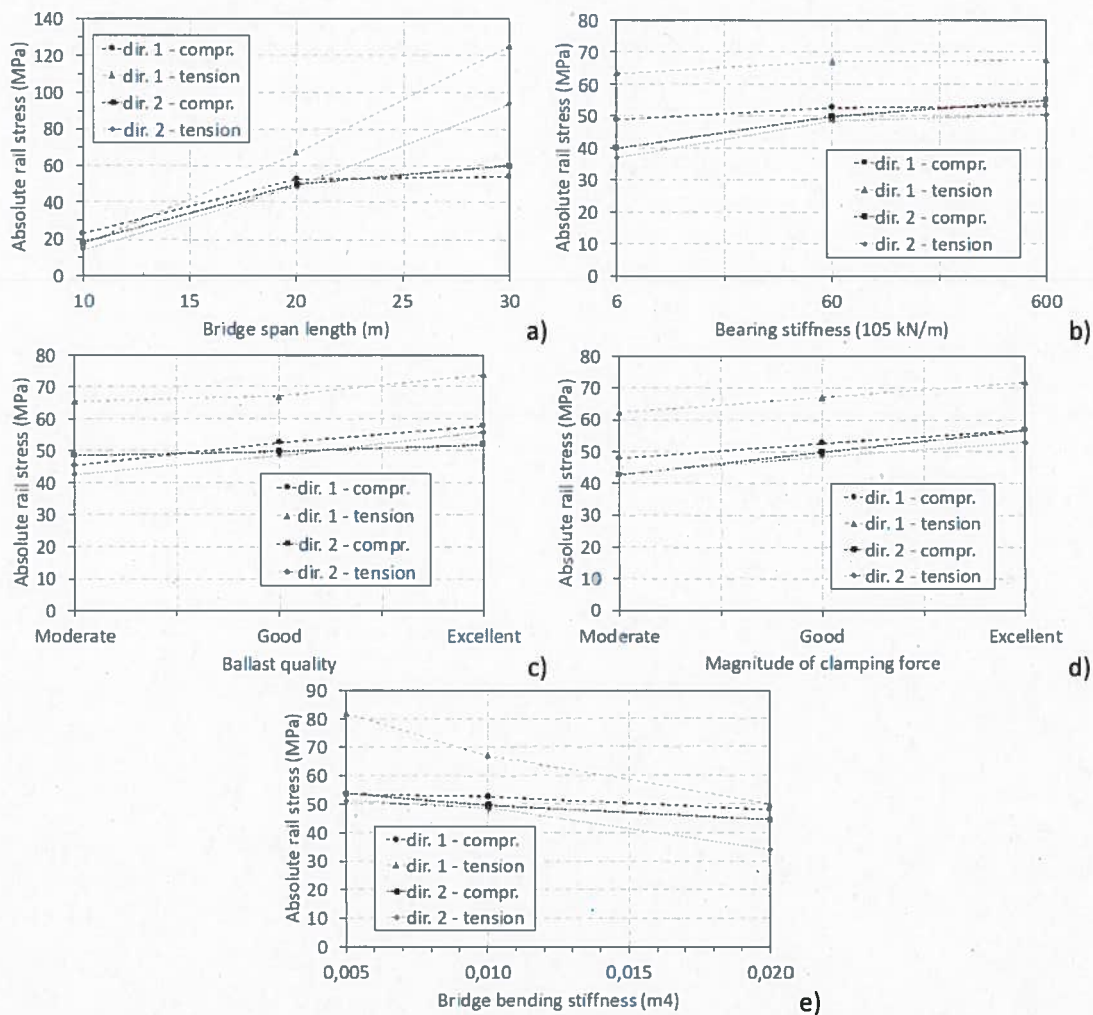


Figure 5. Variation of the additional rail stresses in the two-dimensional model with a) span length, b) the bearing stiffness, c) the ballast quality, d) the clamping force and e) the bending stiffness of the temporary bridge deck

The logical conclusion is that it is allowed to omit the expansion device from the structural configuration of the track for these configurations. Furthermore, it could even be concluded that, if the temporary bridge decks used by the Belgian railways are only used in correspondence with their official span range (which equals 12 m), there is still a large safety margin with respect to the additional rail stresses and displacements.

### 3 Conclusions

Based solely on the two-dimensional model assembled according to the UIC code 774-3R [1] the conclusion would be that it is allowed to continue CWR track over a temporary bridge deck without providing expansion devices in front and after the temporary bridge. However, if one would also consider the results obtained from an analysis in which the track is loaded with temperature loads, then this conclusion would not be so straightforward

Table 2. Comparison of FE results with UIC<sup>1</sup> examples: additional stresses due to several load types

		Load type 1	Load type 2	Load type 3	Load type 4	Sum
E1-3	UIC (MPa)	-126,0	-30,7	-16,4	-17,0	-64,1
	FEM (MPa)	-126,0	-31,5	-19,3	-17,6	-68,4
	Error (%)	0	3	18	4	7
E4-6	UIC (MPa)	-126,0	-30,7	-15,9	-28,2	-78,8
	FEM (MPa)	-126,0	-31,5	-14,5	-25,7	-71,7
	Error (%)	0	3	-9	-9	-4
A1-3	UIC (MPa)	-126,0	-13,2	-12,7	-15,0	-41,0
	FEM (MPa)	-126,0	-12,6	-11,5	-15,4	-38,1
	Error (%)	0	-5	-10	3	-7
A4-6	UIC (MPa)	-126,0	-13,2	-12,7	-24,7	-50,7
	FEM (MPa)	-126,0	-12,6	-13,3	-22,0	-47,9
	Error (%)	0	-5	5	-10	-5

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