

Safety Assessment of Existing Highway Bridges and Viaducts

Johan Maljaars, Dr.ir.; Raphaël Steenbergen, Dr.ir.; Linda Abspoel, Ir., TNO, Delft, The Netherlands and Henk Kolstein, Dr., TU Delft, The Netherlands. Contact: johan.maljaars@tno.nl

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Abstract

The assessment of the structural safety of existing bridges and viaducts becomes increasingly important in many countries owing to an increase in traffic loads. Most existing standards, however, are developed for the design of new structures. For this reason, an assessment method for determining the actual safety level of highway bridges and viaducts has been developed. The method focuses on the determination of the effect of traffic actions and consists of a number of levels. The first level requires the least work from the engineer but is the most conservative. Each of the next levels is less conservative. Some levels require actual measurements at critical parts of the structure. This paper consists of two parts. The first part explains the basis of the safety assessment method and the second part shows the step-by-step application of the method to an existing highway bridge in The Netherlands.

Keywords: traffic loads; existing structure; reliability; residual lifetime; measurements; bridge.

Introduction

Existing infrastructure represents a large economic value. This infrastructure is often subjected to severely increasing traffic loads and this was not anticipated during the design in many cases. In addition, the material used for civil engineering structures may have degenerated over time, for example, due to corrosion and fatigue. Replacement of the structure with a new one is in many cases not preferred because of the costs and traffic hindrance involved. The safety assessment of existing infrastructure is therefore a concern for authorities, engineers and researchers.

The (traffic) loads that should be taken into account in the design of bridges or viaducts are described in standards such as EN 1991-2.¹ These standards usually provide some generally applicable load models that can be used independently of the layout of the bridge and the road. This means that the design value of the traffic load in standards often includes some implicit conservatism when an individual bridge is considered. Besides, the standard accounts for an expected

increase in axle loads and intensity of traffic for the next 50 or 100 years. In the case of assessment of an existing bridge, the required residual lifetime might be reduced as compared to these 50 or 100 years. These and other factors may be used in the assessment of an existing bridge.

This paper describes an assessment method for the structural safety of existing infrastructure. The method is aimed at determining the actual and / or the required safety level of an existing bridge or viaduct in highways by focussing on the accurate determination of the actual traffic load effects for the ultimate limit state (ULS) and the fatigue limit state. The paper shows the application of this method to an existing highway bridge.

Ultimate Limit State Assessment Method for Existing Bridges

The assessment method for checking the ULS consists of a number of levels. In each level the effect of traffic actions and/or the required safety level is determined more accurately. This means, on the one hand, that the amount of work for the engineer increases with each level and, on the other hand, that the conservatism decreases, allowing for an increasingly optimal assessment. At each level, the structural safety can be evaluated. If

sufficient safety cannot be demonstrated, the engineer may carry out the assessment of the next level.

Level 1—Partial Factors and Load Reduction Factors for Existing Infrastructure

The point of departure of level 1 assessment is the traffic load model provided in standards for new structures. This load is multiplied by load reduction factors that represent the difference between new and existing bridges. In addition, partial factors that are based on the required safety level for existing bridges are considered. The assessment is given in detail in the following text.

The required reliability indices for new structures are provided in standards or legislation. EN 1990² recommends a reliability index of $\beta = 4.3$ for ULS of important structures (Consequence Class 3 or CC3). From the economy point of view, a lower safety level may be acceptable for existing structures because increasing the safety levels usually involves relatively more costs for existing structures than for structures that are still in the design phase.³ However, a minimum safety level is required from the point of view of human safety. ISO 2394⁴ provides the maximum allowable yearly probability for loss of human life due to structural failure. This requirement can be translated into a minimum reference period of 15 years for existing structures.⁵

In Ref. [5] reliability indices are provided for existing structures on the basis of the economic and human safety considerations mentioned earlier. Distinction is made between a disapproval level for existing structures—below which the structure is unsafe and should be closed for traffic—and a repair level for existing structures—which is the minimum required safety level in case an existing structure is strengthened or extended. Further, partial factors for weight and traffic loads are derived for these reliability indices. These partial factors are based on full probabilistic calculations, using



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distribution functions for the traffic loads resulting from weigh in motion (WIM) measurements on a representative Dutch highway.⁵ Additional WIM measurements have shown that the distribution functions of the traffic loads on other (Dutch) highways are similar. Therefore, these partial factors are considered representative of highways in The Netherlands and probably also of many other European highways. Both the reliability indices and the partial factors are summarised in *Table 1* for structures in CC3. The partial factors relate to the fundamental combinations of actions 6.10a and 6.10b as mentioned in EN 1990:

$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} \gamma_P P \gamma_{Q,1} \psi_{0,1} Q_{k,1} \quad (6.10a) \quad (1)$$

$$\sum_{j \geq 1} \xi_j \gamma_{G,j} G_{k,j} \gamma_P P \gamma_{Q,1} Q_{k,1} \quad (6.10b) \quad (2)$$

In addition to the reduced partial factors, the engineer may account for two traffic load reduction factors:

- A reduction factor related to the reference period. The probability that a certain load value is exceeded depends on the number of lorries per year and the required lifetime of the bridge. The traffic load in standards such as EN 1991-2¹ is provided for a reference period of $T = 100$ years. For a shorter reference period—such as for an existing bridge—the reference period reduction factor ψ_{rp} on the load in EN 1991-2 can be determined using Eq. (3). The exponent (0,45) in this equation is based on curve fitting of results from WIM measurements in The Netherlands.⁶ The minimum reference period to consider for an existing bridge is $t = 15$ years (*Table 1*). The number of lorries per year n_a can be obtained from measurements or a standardised value can be used. As an example, EN 1991-1-2 provides an indicative number $n_a = 2 \times 10^6$ per year for the

slow lane of roads with two or more lanes per direction and with high flow rates of lorries.

$$\psi_{rp} = \left(\frac{\ln(n_a \times t)}{\ln(n_a \times T)} \right)^{0,45} \quad (3)$$

- The axle loads and intensity of traffic are expected to increase in time. This trend is already taken into account in the traffic load values in standards such as EN 1991-2. Elaboration of extensive WIM measurements in The Netherlands has provided insights into the trend.⁶ The study concludes that the loads provided in EN 1991-2 are expected to represent the traffic situation around 2050 on the basis of trends that are foreseen at this moment. Further, the increase in time of the maximum vehicle weights is larger than the increase in time of the maximum axle weight. Starting from the expected traffic load in 2050, factors have been derived for shorter life spans that take into account the trend in traffic loads. These trend reduction factors, ψ_t , are dependent on the influence length in order to consider the difference in trend between axle load and traffic load. The trend reduction factors ψ_t are graphically shown in *Fig. 1*. Note that the factors are based on WIM measurements in The Netherlands. However, the types of lorries do not vary significantly between countries and the number of lorries on the Dutch highway system is relatively large compared to that in most other European countries. For these reasons, the factors are expected to be in the same order for other European countries.

Level 2 – Current Use of the Structure

In addition to the partial factors and the load reduction factors of the level 1 assessment, the level 2 assessment considers the actual use of the bridge.

The current use of the structure may differ from the use anticipated in the

design. As an example, a bridge may accommodate four lanes while the present layout of the bridge consists of three lanes. In the level 2 assessment, the engineer may assess the structure for current use, considering the “normal” traffic flow. This implies that the most heavily loaded lanes—with lorry traffic—are the slow lanes and that the hard shoulders are considered as the remaining area. However, in case of a calamity it is possible that this layout of lanes is temporarily different. For this reason, an additional assessment is required using the most adverse lane layout, that is, the most heavily loaded lane may be any lane, including the hard shoulder. A reduced reference period may be applied in such a situation. A short reference period results in a low characteristic value. This low characteristic value combined with the standard partial factors may result in failure probabilities that are higher than the maximum allowable probability from the point of view of human safety, according to Ref. [4]. As a consequence, a minimum reference period that is longer than the duration of the calamity itself results.⁵ For this reason, a reference period of 1 month is prescribed for calamity durations of maximum 1 day in the Dutch standard.⁷

It is evident that this procedure reduces the flexibility for future changes to the layout of lanes. Each anticipated change should be preceded by a level 2 assessment for that situation. In addition,

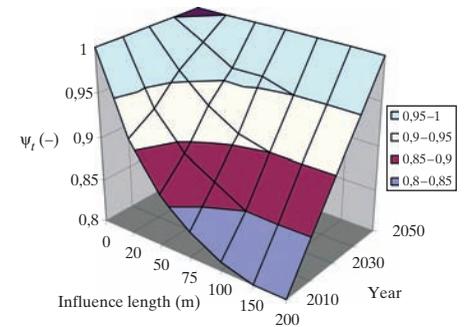


Fig. 1: Graphical presentation of the trend reduction factor ψ_t according to Ref. [12]

| | Reference period (years) | β for indicated reference period* | β per year* | Partial factors | | |
|---------------------------------------|--------------------------|---|-------------------|---------------------------|-------------------------------|--------------------|
| | | | | Weight γ_G (6.10a) | Weight $\xi \gamma_G$ (6.10b) | Traffic γ_Q |
| New structure | 100 | 4,3 | 5,2 | 1,40 | 1,25 | 1,50 |
| Existing structure, repair level | 15 | 3,6–3,8 | 4,3–4,4 | 1,30 | 1,15 | 1,30 |
| Existing structure, disapproval level | 15 | 3,3 | 4,0 | 1,25 | 1,10 | 1,25 |

*The reliability indices are provided for action combinations where wind action is not dominant.

Table 1: Partial factors for traffic bridges in highways, CC3 (Source: Ref. [5])

tion, the level 2 assessment is only feasible if the layout of lanes is generally respected by drivers.

Level 3—Design Stress Based on Measurements

Level 3 assessment requires the execution of measurements. For concrete bridges or viaducts, traffic loads can be measured using WIM systems. For steel bridges, strain gauge measurements on critical details can substitute or complement WIM measurements. Load cells are used to determine reaction forces at supports.

In level 3 assessment, the design value of the traffic load is based on the measurements instead of standards such as EN 1991-2. The measured loads or stresses need to be extrapolated to arrive at the design load level corresponding to the required reliability index, that is, with a sufficiently large return period. Since measured data is unavailable for this large return period, the design load is obtained by extrapolation of measured data. For this purpose, the measured data needs to be fitted to a distribution function. This procedure is similar to the one used to derive the design traffic loads in EN 1991-2,¹ but here the design stress (or design load) is determined for one specific bridge or even one specific location in a bridge. The procedure is elaborated for the case of measured stresses.

- Plot the probability of exceedance of the measured variable stresses as a function of the stress level (dots in Fig. 2).
- Fit the measured variable stresses to a distribution function. The distribution function itself (e.g. Weibull or Gamma function) and its parameters should be selected in such a way that they correspond best with the measured spectra using methods such as the maximum likelihood. In most cases, it is not feasible to obtain a perfect

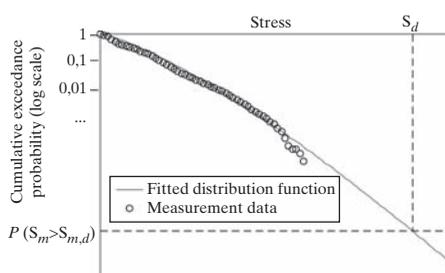


Fig. 2: Schematic presentation of the determination of the design stress based on extrapolated measurements

fit for the entire range of measured stress peaks. Since we are interested in the tail of the function, the engineer may decide to fit the distribution to those measured stress peaks that are larger than a certain truncation limit. The fitted distribution function is presented as a curve in Fig. 2.

- Determine the exceedance probability of the variable stress level corresponding to the required reliability index. The exceedance probability follows from $P(S > S_d) = \Phi(-\alpha_E \beta)$, where α_E is 0,7 in agreement with EN 1990² and ISO 2394⁴ and β is the required reliability index for existing structures according to Table 1. The reference period of the structure and the duration of the measurements need to be accounted for. Suppose that the reference period is t and that x number of stress peaks exceeding the truncation limit have been measured during period i . The exceedance probability of the design stress related to the measurements is equal to

$$P(S_m > S_{m,d}) = \frac{\Phi(-\alpha_E \beta)}{x \times (i/t)} \quad (4)$$

- Determine the design value of the stress due to traffic loads $S_{m,d}$. This is the stress in the best fitting distribution function that corresponds with the exceedance probability calculated earlier (indicated with dashed lines in Fig. 2). Whether the uncertainties in the stress measurement and in the extrapolation need to be accounted for can be a point of debate. In the extrapolation of Fig. 2 we have not considered this uncertainty because the procedure is expected to provide a sufficiently conservative design stress for this case. For example, the fact that the maximum axle weight is subjected to physical limitations is ignored. In some specific cases, it may be required to account for uncertainty in the extrapolation.
- Combine the design value of the variable stress level S_d with the permanent stress and the variable stresses due to wind and temperature actions.

Experience based on WIM measurements have indicated that the variation of traffic load between weeks outside public holidays is insignificant for highways. Consequently, a measurement period of approximately 3 weeks outside public holidays is usually representative for the traffic distribution during a whole year and is thus sufficient for the level 3 assessment.

In case the measurements are used to determine the design loads for longer periods, the resulting design loads could be multiplied by the inverse of the load reduction factors of the level 2 assessment. However, we advise against extrapolation for a long period. Instead, the measurements should be repeated after a certain period of time (e.g. 3 years) because traffic flows may change over time.

The attention of the engineer is required for situations where the measurements may not be representative, causing this level 3 assessment to be invalid. Examples are as follows:

- significant changes to the road system (e.g. opening of a new highway) that may affect the traffic flow on the structure considered;
- large construction works in the vicinity of the bridge that started after the measurement period;
- new legislation on, for example, the maximum axle loads or maximum traffic speed.

Note that the measurements only refer to the current use of the structure with “normal” traffic flows. A level 2 assessment remains necessary for the calamity situation.

Level 4—Full Probabilistic Assessment

A final possibility is to carry out a full probabilistic assessment. In this case, the distribution functions of all relevant load and resistance variables and uncertainties need to be determined. Probabilistic methods such as Monte Carlo or FORM are used to determine the actual failure probability or reliability index. This reliability index can be compared with the required reliability index for existing structures according to Table 1. The most difficult step in this assessment is the selection of distribution functions. In general, distribution functions need to be selected for at least the following variables. Note that additional variables may be relevant depending on the type of failure mechanism.

- loads, such as permanent loads, traffic loads, temperature loads and wind loads;
- material properties, such as the yield strength and ultimate tensile strength (steel) or compressive strength (concrete);
- dimensions and eccentricities, such as the member thickness, initial curvature and out of plumbness;

- uncertainties in the load models and response models.

The distribution function for traffic loads is determined in the level 3 assessment. In some cases, expert opinion is required for estimating the parameters of the distribution functions of several variables. The JCCS Probabilistic Model Code may provide guidance on these parameters for a number of variables.

Fatigue Assessment Model for Existing Bridges

Apart from a check on the static strength, the structure also needs to be checked for fatigue. A state-of-the-art procedure for assessing the remaining fatigue lifetime of existing bridges is provided in Refs [8 and 9]. The input required for the assessment is an accurate fatigue load model (FLM).

Several FLMs exist for structures subjected to traffic loads. For example, FLM 4 in EN 1991-2 consists of a set of standard lorries with prescribed weight, axle distances and wheel contact areas. The percentage of lorries in this model depends on whether the road is mainly used for long or medium distance traffic (Table 2). Fig. 3 provides the corresponding axle types.

A more refined FLM based on WIM measurements on highways in The Netherlands has been developed.¹⁰ The model distinguishes between traffic actions that have occurred before 1990, traffic actions between 1990 and 2010 and traffic actions for the period after 2010 (Table 3). Expectations for trends in future lorry types are considered in the model for the period after 2010. Two remarks apply to this model:

- The model is based on WIM measurements in The Netherlands. However, the types of lorries do not vary significantly between countries. Therefore, the load characteristics are expected to be representative for other European countries as well.

- The model is derived for steel bridges. As a simplification of the load model, heavy vehicles with low percentages of occurrence were neglected. This is justified during fatigue checking of steel structures, but may not be justified in the case of concrete structures. The model should be used with caution in the case of concrete structures. On the other hand, the model gives a more accurate representation of real traffic loads than FLM 4 in EN 1991-2 does even for relatively heavy vehicles, as will be demonstrated in the next section.

Application on a Highway Bridge in The Netherlands

The Galecopperbridge is a steel bridge that functions as the crossing between one of the busiest highways in the Netherlands—E30/A12—and a main canal for vessel transport—the

Amsterdam-Rijnkanaal. Currently, over 2 million lorries pass the bridge per traffic direction per year. Renovation of the bridge is planned in 2012. The assessment procedure described in the previous sections is applied to determine the actual safety level of this existing bridge.

Description of the Bridge Structure

The Galecopperbridge consists of two skewed, cable stayed bridges with a total span of 320 m with two intermediate supports (Fig. 4). Two pylons with two stay cables running over each pylon support six main girders per bridge. The pylons are hinged with the piers in the plane of the stay cables and are clamped out of that plane. The height of the main girders is 3300 mm but the thickness of the web and the bottom flange varies along the span and between the main girders. Cross-beams are positioned perpendicular to the main girders at a centre to centre distance of 3333 mm. An orthotropic deck spans the distance between the cross-beams.

Fig. 5 presents the lane layout for one of the two bridges. All traffic is separated into traffic continuing on the highway E30/A12—directed onto the main lanes—and traffic entering or leaving the highway—directed onto the parallel lanes. A hard shoulder is present next to the main lanes and an acceleration/deceleration lane is present next to the parallel lanes. In 2009, $1,27 \times 10^6$ and $0,69 \times 10^6$ lorries per traffic direction were counted on main lane 1 and parallel lane 1, respectively. The first of the two bridges was opened for traffic in 1971 and the second bridge in 1976. A renovation of the bridge is planned in 2012. Prior to the renovation, the structural safety of the bridge in its current state has been assessed. The description in the following sections provides a part of this assessment, being the determination of the effect of traffic actions in main girders 1 and 3 at midspan.

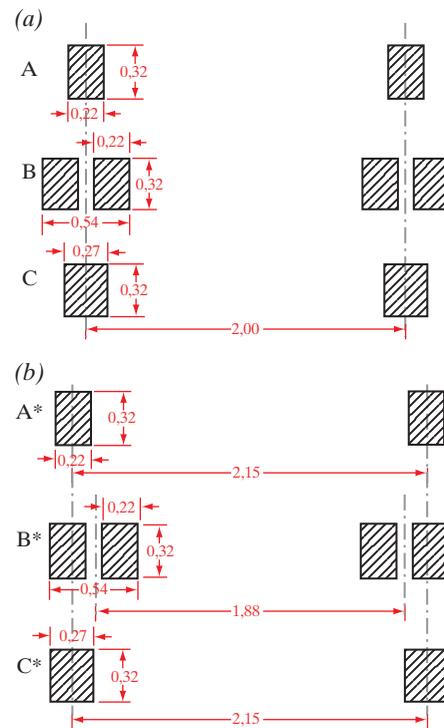


Fig. 3: Axle types in FLMs (Units: m)

| Lorry type no. | No. of axles | Axle spacing (m) | Axle load (kN)* | Axle type (Fig. 3) | Long distance traffic (%)** | Medium distance traffic (%)** |
|----------------|--------------|--------------------|---------------------|--------------------|-----------------------------|-------------------------------|
| 1. | 2 | 4,5 | 70; 130 | A; B | 20 | 30 |
| 2. | 3 | 4,2; 1,3 | 70; 120; 120 | A; C; C | 5 | 5 |
| 3. | 5 | 3,2; 5,2; 1,3; 1,3 | 70; 150; 90; 90; 90 | A; B; C; C; C | 40 | 15 |
| 4. | 4 | 3,4; 6,0; 1,8 | 70; 140; 90; 90 | A; B; C; C | 25 | 10 |
| 5. | 5 | 4,8; 3,6; 4,4; 1,3 | 70; 130; 90; 80; 80 | A; B; C; C; C | 10 | 40 |

*The load model includes dynamic load amplification for pavements of good quality.

**In 10% of the cases a lorry on the slow lane is accompanied by another lorry on the adjacent lane.

Table 2: FLM 4 of EN 1991-2

| Lorry type no. | No. of axles | Axle spacing (m) | Axle load (kN) ¹ | Period < 1990 ^{2,3} | | Period 1990–2010 ^{2,3} | | Period > 2010 ^{2,3} | |
|----------------|--------------|---|---------------------------------------|------------------------------|------------|---------------------------------|------------|--|------------|
| | | | | Axle type (Fig. 3) | Percentage | Axle type (Fig. 3) | Percentage | Axle type (Fig. 3) | Percentage |
| 1. | 2 | 5,2 | 35; 40 | A*; B* | 2,0 | A*; B* | 2,0 | A*; B* | 2,0 |
| 2. | | | 55; 70 | | 1,4 | | 1,4 | | 1,4 |
| 3. | | | 70; 100 | | 0,6 | | 0,6 | | 0,6 |
| 4. | 3 | 3,8; 1,3 | 55; 50; 40 | A*; B*; B* | 4,0 | A*; B*; B* | 4,0 | A*; B*; B* | 3,5 |
| 5. | | | 75; 80; 60 | | 2,8 | | 2,8 | | 2,45 |
| 6. | | | 90; 125; 100 | | 1,2 | | 1,2 | | 1,05 |
| 7. | 4 | 3,8; 6,6; 1,3 | 55; 55; 35; 35 | A*; B*; B*; B* | 11,5 | A*; B*; C*; C* | 11,5 | A*; B*; C*; C* | 8,5 |
| 8. | | | 60; 75; 55; 55 | | 8,05 | | 8,05 | | 5,95 |
| 9. | | | 70; 110; 85; 85 | 3,45 | 3,45 | 2,55 | | | |
| 10. | 5 | 3,8; 5,6; 1,3; 1,3 | 60; 50; 25; 25; 25 | A*; B*; B*; B*; B* | 28,5 | A*; B*; C*; C*; C* | 28,5 | A*; B*; C*; C*; C* | 27,0 |
| 11. | | | 70; 95; 60; 60; 60 | | 19,95 | | 19,95 | | 18,9 |
| 12. | | | 80; 125; 90; 90; 90 | | 8,55 | | 8,55 | | 8,1 |
| 13. | 6 | 2,8; 1,3; 5,6; 1,3; 1,3 | 60; 40; 60; 45; 45; 45 | A*; B*; B*; B*; B*; B* | 4,0 | A*; B*; B*; B*; B*; B* | 4,0 | – | – |
| 14. | | | 70; 60; 90; 80; 80; 80 | | 2,8 | | 2,8 | | – |
| 15. | | | 80; 90; 115; 105; 105; 105 | | 1,2 | | 1,2 | | – |
| 16. | 6 | 4,2; 1,3; 4,2; 3,8; 1,3 | 60; 70; 45; 45; 40; 40 | – | – | – | – | A*; B*; C*; C*; C* | 4,0 |
| 17. | | | 75; 95; 70; 80; 65; 65 | | – | | – | | 2,8 |
| 18. | | | 90; 125; 95; 100; 85; 85 | | – | | – | | 1,2 |
| 19. | 8 | 2,8; 1,3; 5,6; 1,3; 1,3; 4,2; 1,3 | 60; 40; 60; 45; 45; 45; 35; 35 | – | – | – | – | A*; B*; B*; C*; C*; C*; C* | 5,0 |
| 20. | | | 70; 60; 90; 80; 80; 80; 60; 60 | | – | | – | | 3,5 |
| 21. | | | 80; 90; 115; 105; 105; 105; 85; 85 | | – | | – | | 1,5 |

1) The loads need to be multiplied with a dynamic amplification factor of 1,1. Further, the loads need to be multiplied by a trend factor of 1,2 per 100 years in relation to the reference year 2000.

2) In 10% of the cases a lorry on the slow lane is accompanied by another lorry on the adjacent lane.

3) In 10% of the cases a lorry on lane 1 is attended by another lorry on the same lane with a distance in between of 40 m.

Table 3: Refined FLM according to Ref. [10]

Measurement Programme and Results

A measurement programme is carried out. Strain gauges are applied at the bottom flanges of the main girders at midspan. Two types of measurements are carried out:

- Measurements during the crossing of a single, calibrated lorry with a mass of 120 t. These measurements have resulted in influence curves (Fig. 6). The horizontal axis of this figure presents the distance along the bridge and the vertical axis presents the stress in the structure due to the passing of a unity weight of 1 kN. The influence curves of the main girders were approximately equal for passages with a velocity of 20 km/h and 80 km/h (difference approximately 5–10%).
- Measurements during the crossing of actual traffic. The duration of these measurements was 3 weeks during a period without public holidays. Major

calamities have not occurred during the measurement period. The measurements have yielded stress spectra.

Design Value of Stresses Due To Traffic Loads for ULS

The assessment levels 1 to 3 described in Ultimate Limit State Assessment Method for Existing Bridges section are used to determine the design value of the stress due to traffic actions in main girders 1 and 3 at midspan.

Level 1 – Partial Factors and Load Reduction Factors for Existing Infrastructure

EN 1991-2 specifies traffic load model 1 (LM1) for general verification in ULS. The model consists of a combination of a tandem load and a distributed load on every notional lane, as indicated in Table 4. The adjustment factors α_{qi} and α_{Qi} provided are in accordance with the Dutch National Annex for highways of more than two lanes. Dynamic effects are already covered in Table 4.

Numbering of notional lanes and parts to consider as loaded should be chosen in such a way that the effects from the load models are the most adverse.

The bridge accommodates five lanes of 3 m width for the main carriageway and four lanes of 3 m width for the parallel carriageway. The width of the remaining area is 2,1 m. Using the influence curves (Fig. 6) we are able to calculate the stress due to the characteristic value of the traffic action. Subsequently, the stresses are multiplied by the partial factor and the traffic load reduction factors.

- The partial factor for traffic loads, the disapproval level, is $\gamma_Q = 1,25$ for load combination 6.10b (Table 1).
- The minimum reference period of $t = 15$ years for the normal traffic situation is to be considered. For $n_a = 1,5 \times 10^6$ lorries per year on the main lanes this results in a reference period reduction factor equal to $\psi_{rp,15yr} = 0,95$ (Eq. (3)).

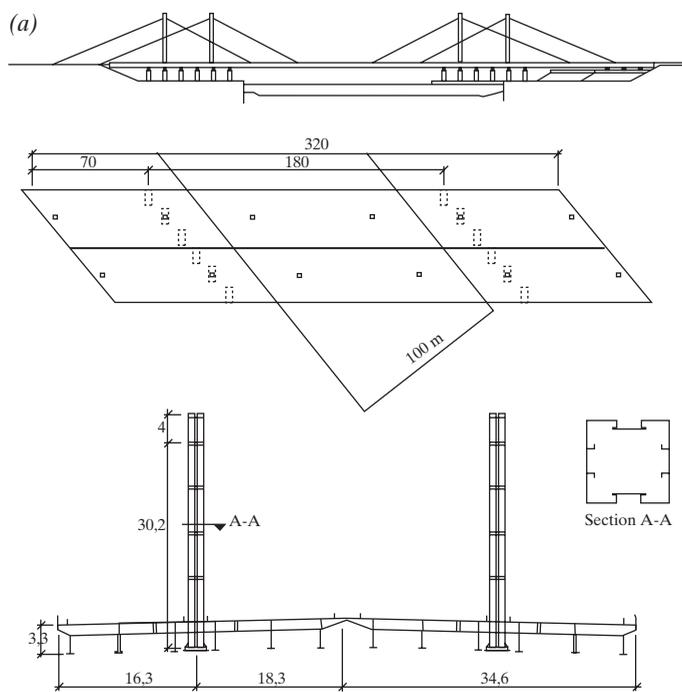


Fig. 4: Side view, plan and cross-section of the two bridges

– The influence lengths are obtained from Fig. 6 and are 175 and 120 m for main girders 1 and 3, respectively. The corresponding trend reduction factors are equal to $\psi_l = 0,81$ for both girders (Fig. 1).

This results in the design stresses due to traffic loads according to the second

rows in Tables 5 and 6 (other rows are introduced later).

Level 2—Current Use of the Structure

In the normal traffic situation main lanes 1, 2 and 3 coincide with notional lanes 1, 2 and 3, respectively. The parallel lanes and the acceleration/deceleration lane

are notional lanes 4, 5 and 6. The residual surface of the carriageways—including the hard shoulder—is the remaining area. Again, the stress due to the characteristic traffic action is determined using the measured influence curves (Fig. 6). The resulting stress is provided in the third rows in Tables 5 and 6.

In the calamity situation the most adverse lane layout is considered, that is, the characteristic value of the stress is equal to the level 1 assessment. The reference period is now reduced to 1 month, resulting in $\psi_{rp,1mo} = 0,81$. The resulting stress is provided in the fourth rows in Tables 5 and 6.

Level 3—Design Stress Based on Measurements

Figure 7 provides the cumulative probability of exceedance of the stress peaks that are resulting from the 3 week measurements with actual traffic. Distribution functions are fitted to the measured data. The truncation limit considered in the fitting is 30 and 40 N/mm² for main girders 1 and 3, respectively. The resulting stress is provided in the fifth rows in Tables 5 and 6.

The level 3 assessment results in considerably lower stresses than the level 2 assessment for the normal traffic situation (third and fifth rows in Tables 5 and 6). This difference can be explained by the following:

1. Analysis of WIM measurements has shown that the actual loads on large span bridges in the Dutch highway system are lower than the load according to EN1991-2 and α -factors according to Table 4.¹¹ Apart from the load reduction factors of the level 2 assessment the extra reduction on the traffic load in EN1991-2 is approximately $\psi_{NL} = 0,80$ for large span bridges in Dutch highways with three or more lanes.
2. The three lanes of the highway split into three main lanes, two parallel lanes and an acceleration/deceleration lane just before the bridge. The

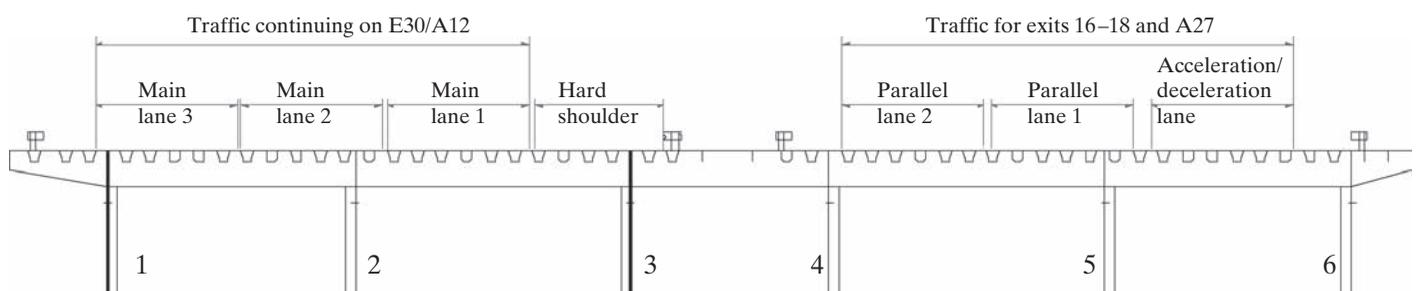


Fig. 5: Current lane layout for one bridge

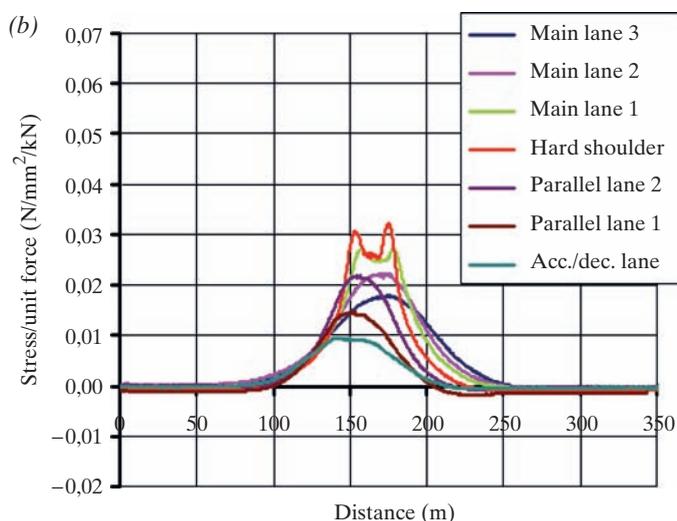
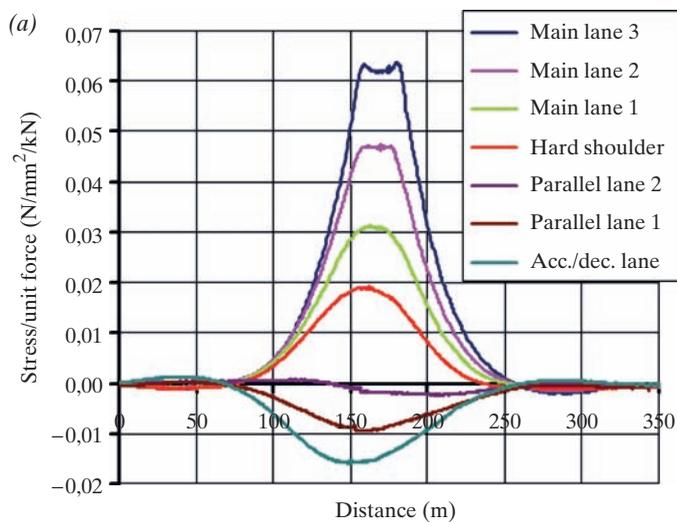


Fig. 6: Influence curves determined with calibrated truck measurements (mass 120 t, velocity 20 km/h)

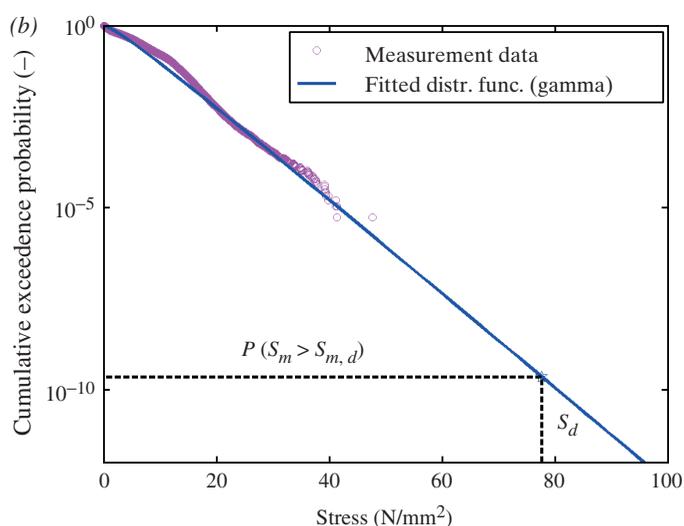
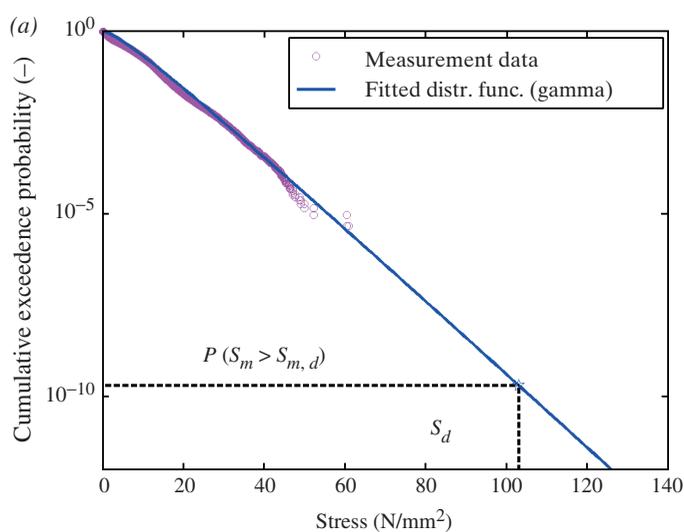


Fig. 7: Cumulative exceedance probability of peak stresses measured under actual traffic load for main girders 1 and 3 at midspan (measurement period 3 weeks)

traffic is expected to divide between parallel and main lanes. This means that the distributed loads per lane i are expected to be lower than $\psi_{rp}\psi_{NL} \times \alpha_{qi}q_i$. The effect is expected

to be significant, but difficult to quantify. A factor χ is introduced to account for the distribution, where the traffic load is assumed to be $\psi_{rp}\psi_{NL} \times \chi\alpha_{qi}q_i$.

| Notional lane no. | Distributed load (for a notional lane width of 3 m) (kN/m ²) | Tandem load (two axes; the specified load is per axle) (kN) |
|-------------------|--|---|
| 1 | $\alpha_{q1} \times q_1 = 1,15 \times 9$ | $\alpha_{Q1} \times Q_1 = 1 \times 300$ |
| 2 | $\alpha_{q2} \times q_2 = 1,4 \times 2,5$ | $\alpha_{Q2} \times Q_2 = 1 \times 200$ |
| 3 | $\alpha_{q3} \times q_3 = 1,4 \times 2,5$ | $\alpha_{Q3} \times Q_3 = 1 \times 100$ |
| Other lanes | $\alpha_{q>3} \times q_{>3} = 1,4 \times 2,5$ | — |
| Remaining area | $\alpha_{qr} \times q_r = 1,0 \times 2,5$ | — |

Table 4: LM1 according to EN 1991-2 and values according to the Dutch National Annex

| Assessment | Traffic situation | Calculation S_d | Value S_d (N/mm ²) |
|------------|-------------------|--|----------------------------------|
| Level 1 | — | $\gamma_Q \psi_{rp,15yr} \psi_t S_{k,a} = 1,25 \times 0,95 \times 0,81 \times 307$ N/mm ² | 295 |
| Level 2 | normal flow | $\gamma_Q \psi_{rp,15yr} \psi_t S_{k,n} = 1,25 \times 0,95 \times 0,81 \times 247$ N/mm ² | 237 |
| | Calamity | $\gamma_Q \psi_{rp,1mo} \psi_t S_{k,a} = 1,25 \times 0,81 \times 0,81 \times 307$ N/mm ² | 252 |
| Level 3 | normal flow | $S_{m,d}$ (i.e. based on measurements) | 103 |
| | calamity 1 | $\gamma_Q \psi_{rp} \psi_{t,1mo} \psi_{NL} \chi S_{k,a} = 1,25 \times 0,81 \times 0,81 \times 0,80 \times 0,67 \times 307$ N/mm ² | 135 |
| | calamity 2 | $\gamma_Q \psi_{rp} \psi_{t,1mo} \psi_{NL} S_{k,a} = 1,25 \times 0,81 \times 0,81 \times 0,80 \times 247$ N/mm ² | 162 |

Table 5: Design stress due to traffic actions S_d in main girder 1 at midspan for the load combination 6.10b

3. The hard shoulder is loaded with q_r (Table 4) in case of design stress assessment with influence curves and EN 1991-2 traffic load. Since there were no major calamities during the measured period, effects of loads on the hard shoulder are not included in the stress assessment based on extrapolated spectra measurements.
4. Only the positive parts of the influence curves are taken into account in the level 2 assessment. It is more realistic to assume that traffic is also present on the other parts of the bridge. In this respect, it is noted

| Assessment | Traffic situation | Calculation S_d | Value S_d (N/mm ²) |
|------------|-------------------|--|----------------------------------|
| Level 1 | — | $\gamma_Q \times \psi_{rp,15yr} \times \psi_t S_{k,a} = 1,25 \times 0,95 \times 0,81 \times 172$ N/mm ² | 165 |
| Level 2 | normal flow | $\gamma_Q \times \psi_{rp,15yr} \times \psi_t S_{k,n} = 1,25 \times 0,95 \times 0,81 \times 169$ N/mm ² | 163 |
| | Calamity | $\gamma_Q \times \psi_{rp,1yr} \times \psi_t S_{k,a} = 1,25 \times 0,81 \times 0,81 \times 172$ N/mm ² | 141 |
| Level 3 | normal flow | $S_{m,d}$ (i.e. based on measurements) | 78 |
| | calamity 1 | $\gamma_Q \times \psi_{rp,1mo} \psi_t \times \psi_{NL} \chi S_{k,a} = 1,25 \times 0,81 \times 0,81 \times 0,80 \times 0,67 \times 172$ N/mm ² | 76 |
| | calamity 2 | $\gamma_Q \times \psi_{rp,1mo} \psi_t \times \psi_{NL} \times S_{k,a} = 1,25 \times 0,81 \times 0,81 \times 0,80 \times 169$ N/mm ² | 111 |

Table 6: Design stress due to traffic actions S_d in main girder 3 at midspan for the load combination 6.10b

that the load model in EN 1991-2 is originally calibrated on bridges with single and double spans and not on more complicated bridge layouts.¹² The influence curves in Fig. 6 show that a small reduction of the stress in main girder 1 is to be expected if the negative parts of the influence curves are also loaded.

5. The load model in EN 1991-2 includes a dynamic amplification factor (DAF) equal to 1,1. The measured DAF for the main girders was equal to 1,05–1,10 for a lorry with mass 120 t and a speed of 80 km/h. Although other lorry masses or speeds may result in a different DAF, the DAF considered in EN 1991-2 is expected to be a reasonable assumption for the bridge.

Aspects 1 and 2 of the list can be quantified by considering modified distributed loads for the normal traffic situation equal to $\psi_{rp} \psi_t \psi_{NL} \times \chi \alpha_{qi} q_i$. Aspect 3 is considered by leaving the hard shoulder unloaded. Aspects 4 and 5 are not considered because their influence is expected to be small. The resulting design stress agrees well with the level 3 assessment for all main girders (difference between 2 and 10%) if distribution factor χ is assumed to be 0,67. Another way to express this is that modified adjustment factors α_i^* are derived for the Galecopperbridge from the measurements. These factors are equal to $\alpha_{q1}^* = \psi_{NL} \chi \alpha_{q1} = 0,62$ for notional lane 1 and $\alpha_{qj}^* = \psi_{NL} \chi \alpha_{qj} = 0,75$ for the other notional lanes $j > 1$. As a comparison, a recent study of traffic loads on existing bridges in Switzerland has resulted in adjustment factors α_{qi} between 0,4 to 0,5.¹³

Two calamity situations have been considered in this level 3 assessment for main girders 1 and 3.

– A calamity on the main carriageway: For this calamity, the most adverse lane combination is considered as well as the load reduction and traffic distribution factors $\psi_{rp} \psi_t \psi_{NL} \chi$. The resulting stress is provided in the sixth rows in Tables 5 and 6.

– A calamity on the parallel carriageway so that all traffic is directed onto the main carriageway: For this calamity the normal traffic situation is considered for the main carriageway, that is, main lanes 1, 2 and 3 coincide with notional lanes 1, 2 and 3. Load reduction factors $\psi_{rp} \psi_t \psi_{NL}$ are considered but the distribution factor χ is not considered since all traffic continues on the three lanes of the main carriageway. The resulting stress is provided in the seventh rows in Tables 5 and 6.

Comparison of Levels 1 to 3

The considerable difference in stresses due to traffic loads between assessment levels 1,2 and 3 (Tables 5 and 6) indicates that there is room for checking or even extending the lifetime of an existing structure by using the proposed assessment procedure.

Comparison of Measured Spectra with the Fatigue Load Models

In total, 33×10^6 and 18×10^6 lorries have crossed the bridge on main lane 1 and parallel lane 1, respectively, during the period from the opening of the bridge until this moment. These numbers are used in combination with the lorry weights and distributions (Tables 2 and 3) to complement the FLM. The fatigue stresses in main girders 1 and 3 are determined using this FLM in combination with the influence curves of Fig. 6. Rainflow counting is used to determine the stress ranges. The resulting stress spectra are provided in Fig. 8. These spectra are derived assuming that 15% of the lorries on the main carriageway are accompanied by a lorry on the carriageway at the same moment in time. Modification of this percentage with a factor 2 does not significantly influence the spectra.

In addition, Fig. 8 gives the measured spectrum with corrected number of cycles representing the period between the opening of the bridge and this moment. The figure indicates that FLM4 in EN 1991-2 gives a non-

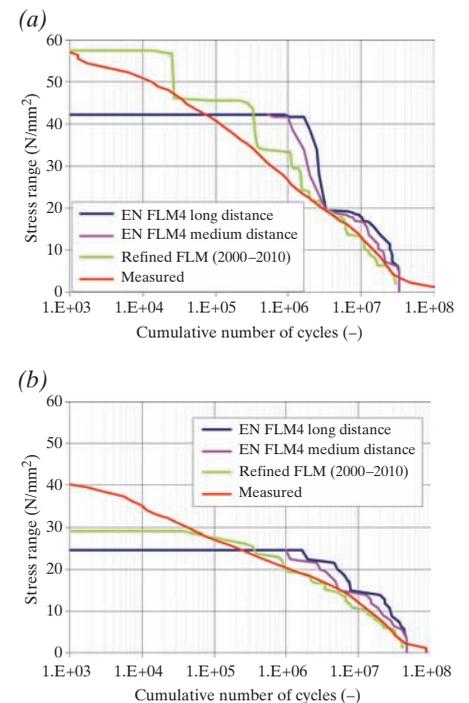


Fig. 8: Stress spectra according to various FLMs and measured spectra, period opening now

conservative spectrum for large stress ranges and a conservative spectrum for intermediate and small stress ranges when compared with the measured spectrum. The refined FLM agrees well with the measured spectra for intermediate and small stress ranges. For high stress ranges the agreement is not so good. However, the corresponding number of cycles is relatively small so that the contribution to the cumulative damage D according to the Palmgren Miner rule is relatively small. Note that the refined FLM is determined from WIM measurements on Dutch highways other than the highway that comprises the Galecopperbridge.

Values of the cumulative fatigue damage D for the FLMs are compared with D for the measured spectrum. The comparison is carried out for various types of steel connections. The ratio $D_{FLM}/D_{measured}$ varies as follows:

- EN 1991-2 FLM4 long distance: $2,6 \leq D_{FLM} / D_{measured} \leq 3,1$;

- EN 1991-2 FLM4 medium distance: $1,7 \leq D_{FLM} / D_{measured} \leq 2,1$;
- refined FLM, period 2000–2010: $0,9 \leq D_{FLM} / D_{measured} \leq 1,3$.

FLM4 in EN 1991-2 is dedicated to the design of new bridges. Considering the distribution of lorry types on the Galecopperbridge at this moment, the conclusion is that FLM4 in EN 1991-2 overestimates the fatigue damage. The refined FLM results in a cumulative fatigue damage D that agrees reasonably well with D for the measured spectrum. This conclusion is valid for steel structures. For concrete structures it may be required to extend the refined fatigue model with a small number of heavy vehicles. The FLM spectrum should then agree better with measured spectra for high stress ranges. Research in this area is under way.

Conclusions

This paper provides an assessment method for existing highway bridges and viaducts, which is based on the determination of the required safety level on the one hand and the effect of traffic loads on the other. The procedure consists of a number of levels where each of the next levels is less conservative than the previous one:

1. Use the required reliability index and reference period for existing structures. The corresponding partial factors, the reference period factor and the trend factor for traffic loads, are provided in this paper.
2. Assess the structure with the current layout of lanes for the normal traffic situation. In addition, consider a calamity situation where the layout of lanes is ignored and the reference period is further reduced. This assessment level is only permissible in case the layout of lanes is generally respected by drivers and in case the layout of lanes will not be changed during the residual lifetime of the bridge.
3. Measure the stress spectrum due to actual traffic at critical locations and derive the design value of the variable stress by extrapolation of the stress spectrum.
4. Carry out a full probabilistic analysis with relevant distribution

functions for the loads, for the structural response and for the model uncertainties.

Acceptance of authorities and bridge managers is required for this procedure.

The application of this procedure to an existing highway bridge in the Netherlands has shown that the effect of actions in a level 3 assessment is considerably lower than that in a level 1 assessment. Hence the lifetime of existing bridges and viaducts can be extended by using the assessment procedure.

FLM 4 according to EN 1991-2 is conservative for small and medium stress ranges and unconservative for large stress ranges when compared to the actual lorry distribution at this moment. As a consequence, the current fatigue damage of steel bridges is overpredicted. A refined FLM is put forward, which gives a reasonable prediction of the actual fatigue spectrum at this moment.

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Nomenclature

Main symbols

| | |
|------------|--|
| G | Permanent action |
| P | Prestressing action |
| Q | Variable action or concentrated load |
| S | Stress level due to traffic actions |
| T | Reference period of 100 years |
| n_a | Number of lorries per year |
| t | Reference period for an existing structure |
| q | distributed load |
| γ | Partial factor |
| ξ | Reduction factor |
| ζ | Factor for a combination value of a variable action |
| ψ_0 | Traffic load reduction factor |
| ψ | FORM sensitivity factor for effects of actions |
| α_E | Load adjustment factor |
| α | Reliability index |
| β | Load distribution factor |
| χ | Cumulative distribution function of the standardised Normal distribution |
| Φ | Normal distribution |

Subscripts

| | |
|--------------|--|
| G, P, Q, q | (refer to main symbols) |
| a | most adverse lane lay-out |
| k | characteristic value |
| d | design value |
| m | measured value |
| n | lane lay-out for 'normal' traffic flow |
| rp | reference period |
| t | trend |
| NL | Netherlands |

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