Load-bearing reserves of existing bridges

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Summary

In the same way as the Technical Bending Theory (TB) for linear-elastic materials, the Extended Technical Bending Theory (ETB) makes it possible to calculate the state of strain of reinforced and pre-stressed concrete cross-sections stressed by any combination of the stress resultants \( M_x, M_z, T, V_z, V_y \), and \( N \). When using the ETB, no additional models are required to determine the shear-bearing capacity of these cross-sections. Furthermore, the theory describes the serviceability limit state (SLS) as well as the ultimate limit state (ULS).

In the first part of the paper, the differences between the ETB and the classical calculations based on the TB and empirical truss models will be explained. Additionally, comparisons of test results and calculations with the ETB will point out convincing correspondence in SLS as well as in ULS.

In some specific cases, the more realistic results obtained using the ETB identify additional load bearing reserves in existing bridges. Because of this, the ETB sometimes makes it possible to verify the structural integrity of existing bridges even when the classical dimensioning concept has failed. As the maintenance and repair of bridges becomes more and more important, the ETB can help to keep bridges in service. In the second part of the paper, some current examples for the application of the ETB will be presented. The theory has made it possible to save several bridges without the need for expensive repair and alteration work.

Keywords: extended technical bending theory; cross-section dimensioning; shear-bearing capacity; load-bearing reserves; maintenance of bridges; concrete bridges.

1. Introduction

For the calculation of concrete bridges according to [1], the linear elastic analysis of the stress resultants is decoupled from the subsequent verifications of the cross-sections in ULS and SLS. A plastic or non-linear analysis is not yet allowed in bridge design. The advantage of this decoupling is that it makes it possible to superpose the stress resultants of the numerous load cases.

For verifications of the cross-sections, there are normally two main combinations of stress resultants which have to be analysed: bending moments and/or normal force \((M+N)\) and shear forces and/or torsional moment \((V+T)\). This differentiation is shown in Fig. 1. The reasons for these separate verifications are the non-linear material behaviour of reinforced concrete and the restricted capabilities of the theories on which these verifications are based. For linear elastic materials, the assumption of a plane state of the longitudinal strains (Bernoulli Hypothesis, TB) delivers the distribution of the longitudinal stresses inside the cross-section. By using equilibrium conditions, it is then possible to determine the corresponding distribution of the shear stresses. Because the principle of superposition is allowed for linear elastic materials, it is possible to subsequently determine the resulting state of stress and state of strain. But for non-linear materials like reinforced concrete, superposition of these separate results is not allowed. This is why the TB can only deliver the corresponding state of stress and state of strain for the combination \(M+N\).
Several different models have been developed to describe the structural behaviour and the shear capacity of reinforced and pre-stressed concrete beams loaded by shear forces. The truss model with crack friction is state-of-the-art for the current German standards. It is based on a model developed by Reineck [2]. While the TB fulfils the equilibrium criteria as well as the compatibility criteria for the combination M+N even for non-linear materials, the truss model only fulfils the equilibrium criteria for the combination V+T. The compatibility criteria should be fulfilled in the ULS by empirical equations for the inclination $\theta$ of the compression struts. For this reason, truss models can only deliver the approximate stresses and strains in SLS.

Because a superposition of the separate results for M+T and V+T is not permissible, it is not possible to determine the correct state of strain in this way. It is only possible to consider the approximate interaction of the different stress resultants in ULS. This is why the inner lever arm $z$ resulting from the TB is used for the truss model. Also, the inclination $\theta$ of the compression struts depends on the longitudinal stresses in the centre line of the cross-section. The shift rule resulting from the truss model must also be taken into consideration when determining the required longitudinal reinforcement. At the moment there are no such interaction rules for the calculation in SLS.

2. The Extended Technical Bending Theory

A solution for this problem is the Extended Technical Bending Theory, first derived by Hartung [3] and later enhanced by him for the engineering consultancy Krebs und Kiefer [4, 5]. This theory makes it possible to calculate the state of strain of thick-walled cross-sections stressed by any combination of the six stress resultants, even for non-linear materials like reinforced concrete. Because the ETB fulfils the equilibrium criteria as well as the compatibility criteria, it describes both limit states, SLS and ULS. This enables to base all verifications for reinforced and pre-stressed concrete cross-sections on a single theory. For the verifications in SLS, the stress limits defined in [1] must be checked and for those in ULS, strain limits based on [1] must be defined.

To integrate the shear forces and the torsional moment into the strain calculation, it is necessary to consider the transverse strain $\varepsilon_t$ and the shearing strain $\gamma$. This integration means that other models for the calculation of the shear-bearing capacity are no longer required. For a combination of bending moments and/or normal force, the ETB delivers exactly the same solution as the TB, showing that the TB is a special case of the ETB.
Preconditions for applying the TB are a plane state of strain, a perfect bond between steel and concrete and the preservation of the cross-section shape. The first precondition must be changed when considering the shearing deformations. Instead of a plane state of strain, a constant curvature of neighbouring cross-sections is assumed. This means that no additional longitudinal stresses caused by constrictions of this curvature are considered. Variations of the curvature resulting from uniform loads are disregarded. Basically, like the TB, the ETB is not valid in disturbed parts of beams as found near supports and concentrated loads.

Due to the complexity and the non-linear correlations, the state of strain can only be determined by iteration. For the iterative calculation, the cross-section is divided into single panels and panel elements. The equations which describe the behaviour of these panel elements and the entire cross-section are sorted into different iteration levels which are embedded in each other. To obtain a solution for these very complex correlations, it is assumed that the principal stresses and the principal strains are coaxial. For this reason, according to the calculation, cracks only appear perpendicular to the principal tension stresses. The cracks open only in a direction perpendicular to the crack plane and no crack displacement parallel to the crack plane occurs, so that the ETB is based on a rotating crack model. For this reason, only cross-sections with shear reinforcement can be calculated in cases of acting shear forces. The calculation is also based on smeared cracks.

As an example, Fig. 2 shows some of the resulting strain and stress distributions for a T-beam cross-section stressed by a bending moment $M_y$, a prestressing force $P_0$ and a shear force $V_z$. The ETB delivers the strains orientated at the coordinates $x$ and $z$ ($\varepsilon_x$, $\varepsilon_z$ and $\gamma$), the principal strains and stresses ($\varepsilon_1$ and $\varepsilon_2$, $\sigma_1$ and $\sigma_2$) with their orientation ($\varphi$ and $\theta$) and the reinforcement stresses ($\sigma_{sl}$ and $\sigma_{sw}$). Additionally the curvature $u$ can be displayed.

![Fig. 2 Strains and stresses of a T-beam cross-section stressed by $M_y$, $P_0$ and $V_z$.](image)

There are two different kinds of longitudinal strains $\varepsilon_l$ displayed in Fig. 2: the result of the TB (only $M_y$ and $P_0$) and the result of the ETB ($M_y$, $P_0$ and $V_z$). A comparison of these strains shows an additional bar extension $\Delta \varepsilon_l$ resulting from the shear force. It causes an earlier transition into a cracked state and a premature yielding of the longitudinal reinforcement. To counter the premature yielding, the design codes specify an additional tensile force $\Delta F_{td}$ that calls for additional longitudinal reinforcement in ULS ("shift rule"). But this additional bar extension in SLS also has effects that should be considered, for example for the verification of tendon couplings. There, the earlier transition into a cracked state causes larger stress amplitudes at a lower load level. This is especially important for the fatigue verification of the couplings.
3. Verification of the ETB

To verify the ETB and the necessary simplifying assumptions, several documented tests were recalculated by Krebs und Kiefer using the ETB. The comparison of the failure loads and the failure reasons demonstrates convincing conformance between test results and calculations. For example, in [6], a comparison of the failure loads of seven different tests with the results of the ETB shows at most relative differences of 10.6%. The corresponding relative differences between tests and calculations according to the current German standard [7] are documented in [8]. They amount to at most 36.8%. Some recalculations done at the Kaiserslautern University of Technology show even greater unutilised load-bearing reserves. For example, in the test TA4 documented in [9]: the test failure load was 468 kN; the failure load according to [7] is documented in [8] as 274 kN (41.5% difference) but the failure load according to the ETB is 443 kN (5.3% difference).

Furthermore, at the Kaiserslautern University of Technology, two large-scale tests were performed to compare the stresses in the reinforcement over the whole load range. The results were published in [5]. The comparison also shows very good conformance. The photograph on the left of Fig. 3 shows the controlling shear crack at the state of failure of an investigated reinforced concrete beam. On the right, a comparison between the calculated and measured strains in the controlling stirrup is shown.

As shown by these comparisons, the ETB can describe the structural behaviour more realistically than the combination of TB and truss model. There is very good conformance in ULS as well as in SLS, and the failure loads according to the ETB are often higher than the failure loads according to [1] or [7]. These additional load-bearing capacities arise from the stiffness-oriented distribution of the internal stresses and the resulting possibility of stress redistribution inside the cross-section. This is why the theory can be very useful in cases where load-bearing structures cannot be verified by traditional methods. In some of these cases, the ETB makes it possible to verify the structural integrity.

4. Applications of the ETB

The ETB allows realistic calculations to a degree of precision that has never been achieved until now. The theory makes it possible to mobilize load-bearing reserves that cannot be recognized using traditional calculation methods.

Where are the areas of application of the ETB? Apart from the fact that all new bridges can be designed more cost-effectively using the ETB, the main area of application of the ETB is in the assessment of existing structures.
The theory can be applied in the following areas:
- assignment of existing structures to bridge categories,
- calculation of the residual load-bearing capacity in cases of on-site concreting errors or in special site-conditions,
- avoidance of reinforcement measures when these would be required according to traditional recalculation methods.

The two examples below show that the application of the ETB can lead to considerable advantages.

4.1 Example 1: DB AG railway crossing bridge near Stelle (Lower Saxony)

This bridge was built in 1976 and consists of 12 single-span pre-stressed concrete superstructures as single-rib T-beams with individual span widths varying from 16 m to 24 m. The transverse beams at the ends are also pre-stressed and transfer the loads from the superstructures to reinforced concrete columns with shallow foundations. Some of the columns are single columns located in the middle of the transverse beam. However, in some cases, two columns transfer the loads in one supporting axis, so that in these areas, the transverse end beam functions as a one-field pre-stressed beam with a span of up to 12 m (Fig. 4). There is no visible damage to the superstructures and transverse end beams.

Fig. 4 Railway crossing bridge Stelle.

According to the DB AG guideline 805 [10], assessments of the structural safety of existing railway bridges are carried out at different levels; as the level number increases, so does the effort involved and the required precision of the assessment methods.

The following assessment levels have been defined:

Level 1 Estimates of the structural safety,
Level 2 Approximate calculation of structural safety,
Level 3 Exact calculation of structural safety,
Level 4 Measurement-supported calculation of structural safety.

The load coefficient \( \beta_{UIC} \) represents the ratio of the difference between the permissible stresses and the stresses resulting from the dead weight, and the stresses resulting from the live loads. A structure is considered to be structurally safe under full UIC traffic loads if the coefficients are greater than 1.0.

According to assessment level 2, “Approximate calculation of structural safety”, the values of the load coefficient \( \beta_{UIC} \) calculated on the basis of the existing structural design were too low, especially for the transverse end beams described above (shown in Fig. 4).

In the present case and according to assessment level 2, \( \beta_{UIC} \) values of 0.55 and 0.57 were calculated in these areas [11]. DB AG then decided to carry out an assessment according to assessment level 3 “Exact calculation of structural safety” as specified in [12] using a structural model with maximum accuracy. Using traditional methods, \( \beta_{UIC} \) values of 0.72 and 0.82 were calculated. The critical aspect was once again the shear strain in the transverse end beams. Using traditional methods, it was therefore impossible to prove sufficient structural safety even in level 3.
To obtain a realistic picture of the load-bearing and deformation behaviour in addition to the exact structural model in level 3, the ETB was used for subsequent calculations, by agreement with DB AG. Using the ETB and thus taking the existing longitudinal, transverse and shear reinforcements into consideration during the strain calculations, additional load-bearing reserves could be activated which had not previously been utilized.

Using a load multiplication factor for the UIC traffic load, the stress resultants at the relevant points were determined in several steps. For stress resultants calculated in this way, the calculation program supplies the overall state of strain in the cross-sections at the relevant points. If the principal strains are smaller than 5 ‰ under tension and/or larger than 2 ‰ under compression, structural safety is considered to be proven.

The load coefficients calculated using the ETB reflect the actual stresses and strains within a building component more precisely than the load coefficients calculated for shearing stress using the framework analogy. The comparable load coefficients according to level 3 using the ETB result in values of $\beta_{UIC} = 1.2$ and 1.5. The safety of the structure is therefore proven. The ETB calculations were verified and confirmed by the checking engineer, Dr. Mündecke by means of an independent derivation of the theory and independent comparison calculations [13].

### 4.2 Example 2: Two-span road bridge near Viernheim (Hesse)

In the case of a two-span road bridge near Viernheim (shown in Fig. 5), a decision was to be made on whether the bridge should be repaired or replaced by a new bridge. The data on the existing bridge was as follows:

- **Year of construction:** 1967
- **Continuous beam:** 2 spans
- **Span widths:** 18.05 m and 23.55 m
- **One superstructure in each direction**
- **Two-rib slab-and-beam**
- **Superstructure width:** 10.75 m and 10.00 m
- **Bridge category:** 60.

One special characteristic of this bridge was that there were only very small amounts of shear reinforcement (2 x Ø 12 – 25 per rib with 9.0 cm² per meter). The reason for this is that according to DIN 4227, which was in force when the bridge was built, shear reinforcements were not required where certain levels of main tensile stresses were maintained.

Recalculations according to the DIN technical report 102 [1] showed that shear forces could only be partly absorbed by the existing reinforcements. Fig. 6 shows the shear force that can be absorbed by the existing stirrups according to [1]. The diagram clearly shows that sufficient safety reserves cannot be proven in accordance with [1], neither for bridge category 30 nor for load model 1.

When the same bridge was recalculated using the ETB, the limiting stress resultants were calculated at the point where the permissible strain was reached. The following permissible strains were applied:
Concrete: longitudinal compressive strains: \( \varepsilon_c > -3.5 \, \% \) or \(-2.0 \, \% \)

Longitudinal reinforcement: tensile strains: \( \varepsilon_{sl} < 25 \, \% \)

Shear reinforcement: tensile strains: \( \varepsilon_{sw} < 2.1 \, \% \)

The results of the ETB recalculation were as follows:

- For bridge category 30/30, the safety reserves required by [1] could be proven in all areas.
- For load model 1, the safety reserves required by [1] cannot be achieved, even using the ETB. The minimum safety reserve calculated using the ETB is 1.08, which is smaller than the value required in [1].

To summarize, considerable load-bearing reserves could be mobilized using the ETB, but these were not sufficient to meet the required safety levels. The decision on whether to repair the bridge or to demolish and replace it was influenced not only by considerations of structural safety but also by financial considerations and by the overall state of preservation of the bridge. In the end, the road construction authorities decided to demolish the bridge and replace it.

5. Conclusion

The two examples presented here – and many others – demonstrate that the ETB is particularly suitable for calculations involving existing reinforced concrete structures and pre-stressed concrete structures with varying cross-sections. Using the ETB, load-bearing reserves can be utilized which could not be identified using traditional methods.
The *ETB* is suitable in principle for all cases in which there is no apparent damage to the structure. It is a useful addition to the results of the calculation methods used up to now. The *ETB* can be used to advantage for recalculating bridges made of reinforced concrete and pre-stressed concrete in cases where the bridges are in good condition and demolition is to be avoided if possible.

Another area of application is in recalculating structures that have been damaged (e.g. through vehicle impact or as a result of concreting errors during construction, as documented in [5]) and under special construction conditions.

As the preservation of existing structures becomes increasingly important, the *ETB* represents an important tool for realistic assessments of the load-bearing capacity of such structures. It can be crucial to a cost-effective decision on the further use of the structure.

6. References


