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Solutions and examples

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ARTICLE

Assessment of existing structures in *fib* modelcode 2020: Solutions and examples

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Abstract
For the assessment of concrete structures in the new *fib* Model Code 2020 (*fib* MC 2020), three categories are distinguished: (1) the residual capacity of existing structures without damage, (2) the residual capacity of structures suffering deterioration, and (3) the residual capacity of structures with noncompliant details. In the accompanying paper by Walraven and Dieteren in this volume, the backgrounds for this subdivision have been explained, and indications for the assessment of existing concrete structures in those categories have been given. In the actual paper, examples are given how to perform an assessment of concrete structures for the categories mentioned above. These examples have contributed to the formulations of the recommendations proposed for *fib* MC 2020.

KEYWORDS
deterioration, existing concrete structures, *fib* MC 2020, structural safety

1 | GENERAL

The new *fib* Model Code 2020 (*fib* MC 2020) gives methods not only for the design of new concrete structures but also for the assessment of existing structures. As much as possible, it tried to use the same basic models for design and assessment.[1] It should, however, be emphasized that the behavioral models derived from tests in laboratories have been carried out on specimens made of new concrete, whereas structures to be assessed consist of concrete "with a history." For a new standard concrete, the relations between compressive strength, tensile strength, E-modulus, and other properties are reasonably unique. Therefore, in databases, often only the compressive strength is mentioned. In the design rules, this coherency between mechanical properties is "built-in." However, in the existing structures, those "unique relations" are not always realistic/met. In the first place, the concrete properties may have been influenced during casting, vibration, and by the quality of curing. Moreover, the concrete can be damaged by imposed deformation on a large scale (structural level) or on a small scale (swelling of rebars due to corrosion, cracking, or spalling of the concrete cover). When using material strength values in assessment models, special care is required. Understanding the background of behavioral models used for assessment is, therefore, an important element of a good analysis.

In this paper, a number of cases are presented, which show that "second thoughts" are an indispensable part of a reliable assessment.

2 | CONSIDERATIONS ABOUT MATERIAL INPUT VALUES

2.1 | Concrete strength values derived from drilled cores

A usual method to get relevant information for an existing structure is to drill cores from the structure at various

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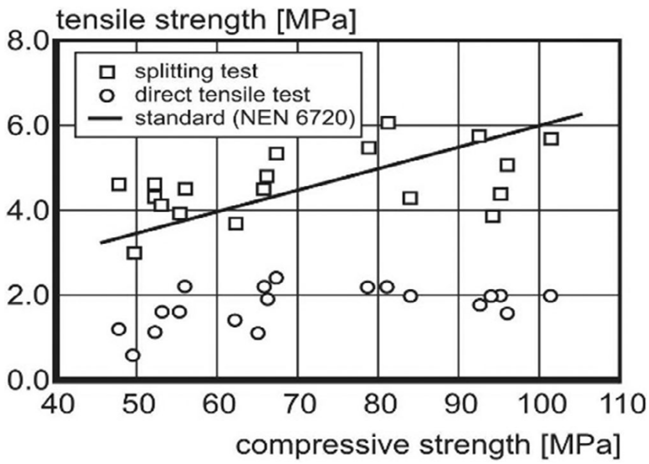

FIGURE 1 Splitting tensile strength and axial tensile strength determined on cylinders drilled from the bottom of an existing slab viaduct.

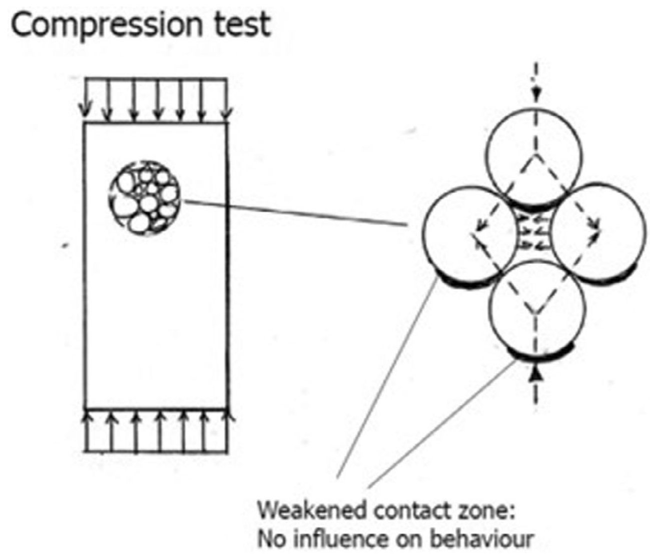

FIGURE 2 Internal bearing mechanism of concrete in compression, where the weakened zones below the aggregate particles do not play a role and as such do not reduce the compressive strength.

locations. Instructions are available about the selection of the core locations, the number and dimension of the cores, the method of drilling, storing, the preparation of the tests, and the way of carrying out compressive and splitting tensile strength tests. In codes, the compressive strength and the splitting tensile strength are given. It is assumed that the uniaxial tensile strength and the splitting tensile strength of concrete are the same. This was confirmed by tests by Maláric s and Müller.[2] However, in existing structures, the effect of anisotropy can play a role, resulting in values that substantially deviate from this concurrence. Figure 1 shows an example of such unexpected results. From a number of drilled cores, taken from the bottom of a solid slab viaduct in The Netherlands, the concrete compressive strength, the splitting tensile strength, and the axial tensile strength were measured. The concrete compressive strength varied between 50 and 100 MPa. This was already a surprise, because the concrete strength class used for casting the viaduct about 60 years ago was only C25. This was explained by the supposition that the concrete delivered at the site had a higher strength than required, but as well because the cement particles in those days were rather coarse, so that hydration continued for a long period after casting. The splitting tensile strength varied from about 3.5 MPa, corresponding to a concrete compressive strength of 50 MPa, to about 6.0 MPa for a corresponding concrete compressive strength of 100 MPa. However, axial tension tests showed values that were only about one-third of the spitting strength values. This gave rise to substantial discussion, realizing that the shear capacity is strongly related to the "tensile strength" of the concrete (although in *fib* MC 2020 the shear equation contains the term ${f_{ck}}^{1/2}$, this is regarded as a direct link to the concrete tensile strength). The explanation was found in the effect of bleeding below the coarse aggregate particles, which obviously had occurred during compaction of the concrete, about 60 years ago, before the introduction of superplasticizers. Figure 2 shows a compression test on a concrete cylinder, with weakened zones below the aggregate particles due to bleeding. The figure shows how the forces are transmitted from particle to particle, kept together by the horizontal tensile stress in the matrix between the aggregate particles.

It can be seen that the weak zones below the particles do not play a role in this equilibrium system. However, they have a considerable influence on the axial tensile strength. In a splitting tensile tests, the weak areas have no influence on the splitting tensile strength. This explains the results shown in Figure 1. Also, in a beam subjected to shear, the weak bleeding areas below the particles will not have a considerable influence on the shear strength, because they are not influencing the formation of the flexure-shear cracks. This was confirmed by comparing the results of shear tests on old concrete beams (including the bleeding effect) and newly cast concrete beams, without bleeding.

2.2 | The effect of sustained loading on concrete strength: Compression, tension, and also shear

In the Netherlands, hundreds of solid slab bridges were expected not to meet the demands of structural safety. Fortunately, tests on drilled cores showed a compressive strength that was substantially higher than expected on the basis of the design strength 60 years ago.

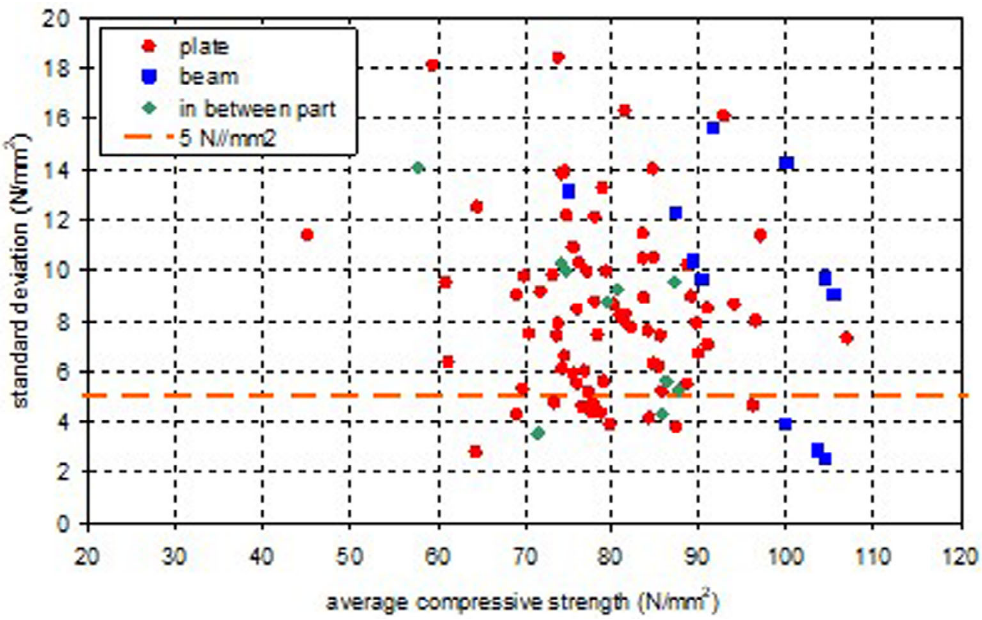

FIGURE 3 Compressive strength as found be testing drilled cores.

This strength was C15–C25, whereas the measured strength values were much higher, as shown in Figure 3. For a ratio $f_{ck,60years}/f_{ck,28days} = 5{:}2$, the design shear capacity according to *fib* MC 2020 would be a factor 1.58 higher than expected on the basis of the original design shear strength. That would mean that the many bridges considered would not need strengthening. However, a basic question remains: does the reducing effect of sustained loading on the compressive and tensile strength of the concrete apply as well to the shear resistance of concrete members without shear reinforcement, which is known as a function of the concrete strength? In the actual version for *fib* MC 2020, in the chapter on concrete properties, the expressions $f_{cd} = \alpha_{cc} \cdot \eta_{fc} \cdot f_{ck}/\gamma_C$ and $f_{ctd} = \alpha_{ct} \cdot f_{ctk}/\gamma_c$ are given. Factor α_{cc} is expressed as a combination of two functions, one of which is a reduction factor for sustained loading, and the other is taking account of increase in the concrete strength after 28 days. So, the effect of sustained loading is (at least partially) compensated by the strength gain after 28 days. For tension, the values $\alpha_{ct} = 0.6$ for normal strength concrete and $\alpha_{ct} = 0.75$ for high strength concrete are defined. That would imply that the effect of concrete strength increase, as measured in the tests on drilled cores, would be nullified by the sustained loading effect. This would have a large impact, since, than the utmost part of the viaducts considered would be unsafe.

It was, therefore, decided to carry out shear tests under short-term loading and long-term sustained loading[3]; see Figure 4. In the short-term tests, shear failure was achieved in 3–5 min. The results of those tests were used as reference values for the sustained loading tests. In those tests, the sustained loading levels were between 0.88 and 0.98 of the short-term shear strength. Those values were gradually increased during the sustained loading period, in order to take account of the estimated increase in the shear resistance in time because of the increase in the concrete strength. The results showed, surprisingly, that there is no sustained loading effect in shear. Recently, the last tests, with sustained load levels of 0.92 and 0.93, respectively, were stopped after 9 years without shear failure.

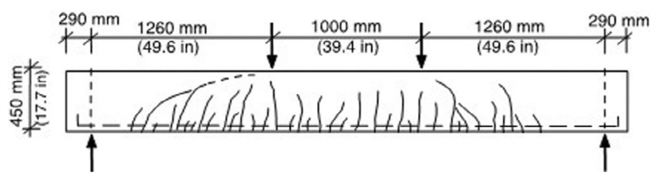

FIGURE 4 Sustained loading tests on beams without shear reinforcement.[3]

The explanation of this conclusion that there is no sustained loading effect in shear was found in the dominating contribution of aggregate interlock in the flexural shear cracks to the shear capacity. This effect is not sensitive to sustained loading: it shows even a hardening effect because of an increase in the concrete contact areas in time between the aggregate and the matrix in the cracks.

3 | ASSESSMENT ASSISTED BY TESTING

3.1 | General considerations

“Design by testing” is a verification method that is already offered in actual codes to demonstrate the applicability of

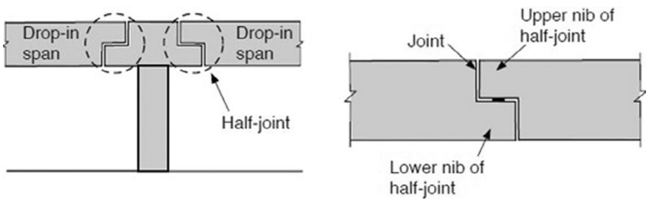

FIGURE 5 Half-joint connection between prestressed beams.[4]

special solutions. This is, for instance, the case if a new innovative type of detailing is proposed that is not yet covered by code provisions. It can also be used to demonstrate the suitability of new high performance materials.

"Assessment by testing" is not yet described in codes, but it can be at least as valuable as "design by testing." This is especially suitable if a large number of typical concrete structures contain one or more non-compliant details, and the question is what magnitude of load those structures still can carry. An example is a large testing program carried out in the UK, investigating the bearing capacity of dapped-end prestressed precast beams ("half-joints") (Figure 5), which has led to a considerable number of problems in a large number of precast bridges.[4] The advantage of this, up to recently, rather popular connection was its suitability for precast construction and reduced cross-sectional depth of the corresponding beams. The disadvantage of this connection, however, was its vulnerability to deterioration at the nib due to seepage of chloride-rich water through the expansion joints. Through the re-entrant crack, the chloride has direct access to the suspension reinforcement behind the nib. Another important disadvantage is the difficult accessibility for inspection or repair, so that deterioration processes are not recognized at an early stage. The investigation described in Ref. [4] showed furthermore that there was a large collection of different reinforcing details, which did not always satisfy the requirements of strut and tie connections. Moreover, there were also construction errors further compounding the reduction in structural reliability arising from the problems with the "as-designed"/"as provided" detailing. So, in this case, there was a combination of noncompliant details with the additional effect of corrosion. For the verification of structural safety of members with those details, special recommendations were developed. This illustrates that "Assessment by testing" is especially useful if the results of the tests have an impact on the judgment of a large group of similar structures.

3.2 | Punching shear resistance of prestressed concrete bridge deck with very thin decks

In the Netherlands, about 70 large bridges were built with prestressed T-beams, and the flanges of which were connected to thin cast in situ decks; see Figure 6a. Because of the gradually increasing traffic loads, a verification was carried out into the structural safety. This verification, for which the actual Eurocode design rules (EN1992-1-1) were used, led to the conclusion that none of the bridges fulfilled the demands of structural safety. The governing failure mode was the punching shear capacity of the deck under the high wheel loads. It was realized, however, that there could be a residual capacity because the effect of compressive membrane actions was not regarded in the design codes. The governing design rules for punching were all based on test results obtained from punching shear tests on specimens without deformation restraint effect at their edges. In actual structures, the area where large punching shear loads are introduced cannot freely deform. So, in reality in-plane and bending resistance by the surrounding slab area will occur simultaneously and expectedly increase the punching shear resistance. In order to investigate this, at TU Delft tests have been carried out at 1:2 scale models; see Figure 6b. The results have shown that by the effect of compressive membrane action, the real punching shear capacity was considerably higher than predicted by applying the original design equation without compressive membrane action. A substantial residual punching shear capacity was demonstrated so that large scale strengthening, the need of which was feared initially, was not necessary.

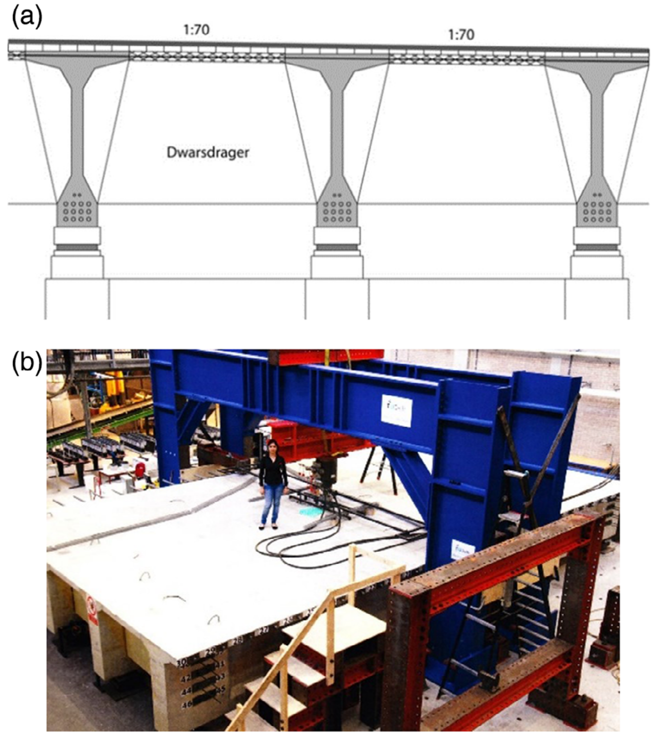

FIGURE 6 (a) Cross section of the bridge deck. (b) Punching shear test on the bridge deck (scale 1:2) (Amir et al.[5–7]).

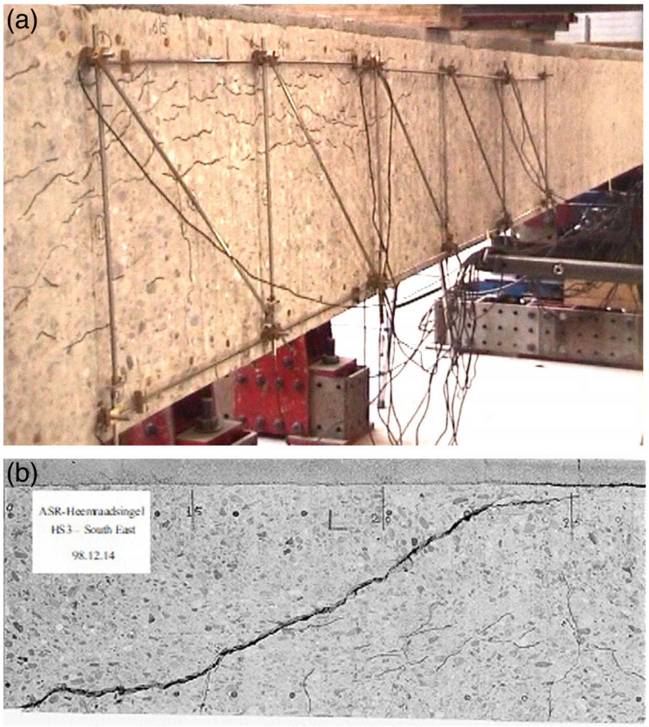

FIGURE 7 (a) Horizontal cracking in a beam sawn-out from a solid slab viaduct.[8] (b) Shear tension failure as observed in laboratory shear test.[8]

Moreover, it was shown that with a suitable NLFEM program, it is quite well possible to calculate the punching shear resistance including the effect of compressive membrane action After calibration of the program to the test results, it turned out that the program was able to reproduce the 14 test results with different loading geometries with a mean value of $P_{\text{exp}}/P_{\text{calc}} = 1.02$ and a coefficient of variation of 0.11.[7] After this calibration, the NLFEM program could be considered as "tailored" to bridges of this type. The program was used for the assessment of existing bridges in the Netherlands. It was possible to demonstrate that the family of 70 large bridges of this type had sufficient structural safety, by virtue of compressive membrane action, which was doubted before on the basis of less advanced calculations. In the new draft of *fib* MC2020, the effect of compressive membrane action has been introduced as the Level II approximation in chapter 30.1.5.4.2.

3.3 | Shear resistance of solid slab viaducts suffering ASR deterioration

In the Netherlands, a number of slab viaducts were attacked by the Alkali Silica Reaction. Those structures showed a characteristic cracking pattern consisting of a large number of short horizontal cracks; see Figure 7a.[8] This crack formation could be explained by the restraining effect of the longitudinal reinforcement at the top and the bottom of the slab: the volume increase in the concrete suffering ASR is counteracted by the reinforcement in axial direction, so that in this direction, the concrete is to some extent prestressed. In the other (vertical) direction, no volume restraining effect was active, so that in the horizontal direction, the cracks could freely form. Drilled cores from the bottom showed a strongly reduced tensile strength in the vertical direction. The concrete compressive strength was on average 45 MPa. For this compressive strength, a concrete axial tensile strength of 3.5 MPa would be expected. However, in the vertical direction, a mean tensile strength of only 1.1 MPa was found with a COV of 42%. This was caused by the horizontal cracks. It was, therefore, expected that those cracks could have a reducing effect on the shear capacity. Therefore, 0.6 m wide beams were sawn from the slabs and transported to the laboratory to be subjected to a concentrated load, near to the supports, leading to shear failure. Figure 7a shows the side face of such a beam, equipped with LVTD's where the horizontal cracks are visible.

The shear tests showed that failure did not occur due to shear flexure failure, as could be expected for undamaged slabs without shear reinforcement but due to shear tension failure; see Figure 7b.[8] The shear tension failure occurred at a load that was about 25% lower than the expected shear capacity for flexural shear failure of undamaged concrete. This shows that cracks due to ASR can reduce the shear bearing capacity in such a way that another failure mechanism becomes governing. But, it also showed that the reduction was less than expected based on the measured tensile strength.

3.4 | Assessment with NLFEM analysis supported by test results

A NLFEM calculation is a tool to assess structural safety using realistic descriptions of the material behavior with actual material properties. By using this method in an optimum way, it is possible to get a realistic assessment of the residual bearing capacity of a structure. However, when preparing a NLFEM analysis, many choices have to be made that may have a considerable influence on the result of the analysis. This has been illustrated by several "round robin blind prediction contests" where the competitors had to predict the bearing capacity of structural members that still have to be tested. For instance, if they use programs with smeared crack models, they have to make a choice between rotating crack models, fixed crack models, partly fixed crack models, micro-plane crack models, and isotropic damage models. Moreover, choices have to be made regarding the constitutive relations for the material properties, like concrete in compression, concrete in tension, bond, tension stiffening, and shear friction in cracks, etc. Guidelines for the use of NLFEM models have been proposed by *fib* already in 2008.[9] In the *fib* MC 2010 and in the new *fib* MC 2020, basic guidelines are given.

An indispensable option to improve the accuracy, and as such the reliability, of a NLFEM analysis is calibration against representative test results. In the scope of a large bridge assessment program in The Netherlands, NLFEM programs were calibrated to specific types of structural members. For instance, many bridge decks consist of prestressed I or T-beams with stirrups. By collecting test results on large prestressed beams from the literature and calibrate the NLFEM program in such a way that for such type of beams the best results are obtained, the model uncertainty of the chosen NLFEM for such types of structural elements can be substantially reduced. In *fib* MC 2020, the safety formats to be applied in NLFEM analysis are further developed and clear guidance is given.

4 | ASSESSMENT ASSISTED BY PROOF LOADING

Proof loading can be defined as "Assessment of a structure at a given limit state by applying an equivalent load on to the structure." If the structure is able to carry the action, and at the same time, none of a set of predefined stop criteria is violated, and the structure fulfills the acceptance criteria related to structural reliability in the predefined reference period, then the structure may be assumed to be safe under that limit state.

When developed and executed correctly, proof loading reduces the model uncertainty of the resistance of the structure. Proof loading can be considered as an option for the case when available research/theoretical methods, combined with the relevant material properties in the structure, cannot provide a reliable estimation of the safety of a structure. Aspects that may be considered as suffering from a lack of relevant information may include details concerning:

- The analytical system of load transmission through the structure;
- The cooperation of structural elements or parts of the structure;
- The influence of damage on structural safety in the actual situation;
- The critical properties of the structure including the dimensions and material properties, etc.;
- The effectiveness of previous interventions (repair measures, etc.).

Figure 8 gives a schematic representation of the strategy to be used during proof loading.[10] At the start of the test, the permanent loads are active with their real magnitude. During the test, an additional external load P_{target} is applied on the structure, to verify whether the design load can be carried. The sum of the load contributions G_1 (permanent load present during the test), G_{dj} (additional permanent load taking into account remaining [reduced] uncertainty during the life of the structure) and Q_{dj} (design value of variable loads), which present the total load P_{target} may not exceed the limit load P_{lim}, for which the criteria are a function of the effect of the load P. The limit load P_{lim} is the highest load for which during proof loading no damage occurs, which might negatively influence structural safety and serviceability in the period after proof loading. Such damage could be categorized into two types, namely,

- Sudden and uncontrollable failure modes such as shear failure, or crushing of concrete during the test, should be avoided: such damage is defined by "stop criteria," which have to be defined before the execution of proof loading.
- Damage that may affect the future use of the structure, such as large crack widths, reduction in structural stiffness, large deflection and local yielding of reinforcement. Such damage is related to the expected utilization period of the structure after proof loading.

Figure 9 shows the proof loading of a bridge in the Netherlands.[11]

To optimize the procedures used in proof load tests, it can be interesting to combine field testing and finite element modeling. Furthermore, acoustic emission measurements can give information on the development of damage occurring in the structures. Further background information and recommendations about proof loading can be found in a *fib* Bulletin produced by Task Group 3.2.[10]

5 | ESTIMATING THE RESIDUAL SERVICE LIFE BY INSPECTIONS AT REGULAR INTERVALS

The assessment of existing concrete structures has most often been carried out to judge the actual structural safety of the structure considered. This allows for taking immediate decisions with regard to the actual use, like "the structure has still sufficient safety," "the structure may only be used up to a certain reduced load level," and "the structure should be strengthened."

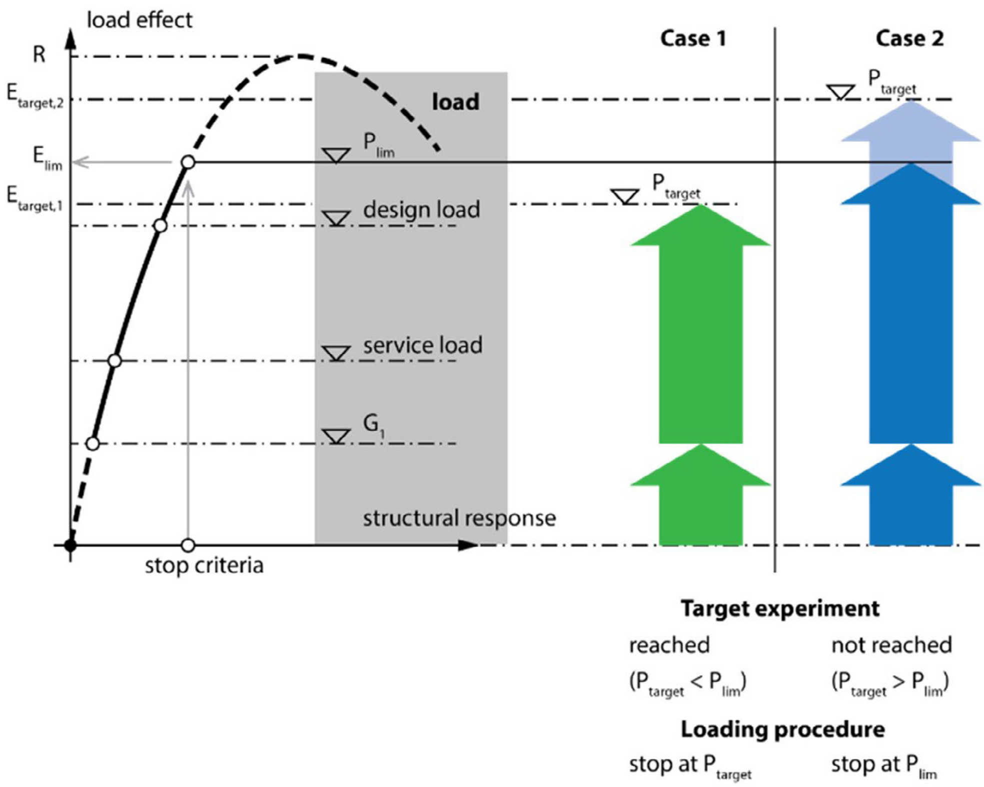

FIGURE 8 Scheme of a proof load test.[10]

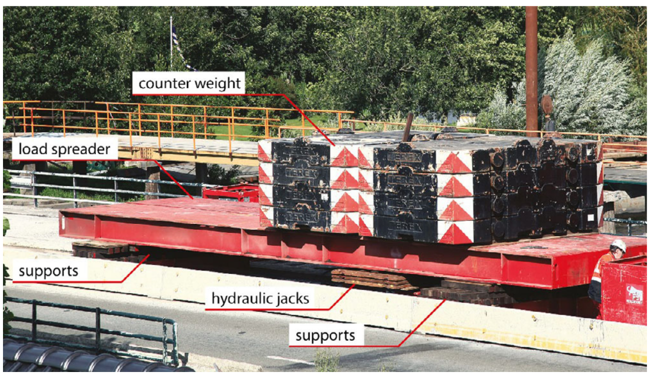

FIGURE 9 Proof loading of the Ruytenschildt Bridge in The Netherlands.[11]

However, another very important aspect is the expected development of deterioration of the structure in future and, in combination with that, the gradual decrease in the structural safety until the end of service life. For structures exposed to chlorides, this means that data should be available about:

- the thickness of the concrete cover;
- chloride content;
- the diffusion coefficient (mean values and standard deviation);
- cracking of the concrete;
- existence of higher corrosion / deterioration risk locations (i.e., vulnerable locations);
- other hazards that might affect concrete deterioration/rebar corrosion.

An example of a first trial to predict the progress of deterioration is related to the bridge Wilpsedijk in The Netherlands, built in 1970.[12] The one span bridge consists of precast concrete T-beams. During an inspection in 2002 rust stains, cracking and spalling of concrete were found at the end of the beams. On the basis of the inspection data, a prognosis was made of the development of further deterioration. The diffusion coefficient (mean value and standard deviation), used to predict the end of the initiation period, was based on diffusion tests in a laboratory on a representative type of mixture. For the propagation period, other parameters were used, like the corrosion rate, based on the recommendations of the Duracrete project.[13] For the propagation state, five classes of damage were defined, based on the state of cracking. In this way, a prediction was made of the probability that parts of the structure would be subject of corrosion and to what extent. Figure 10 shows the overview of the upper and lower bound damage development as calculated.[12] The lower and upper bounds are presented in terms of 20 and 80% fractiles, respectively. This analysis was made in 2002, so that it enabled both "looking back" and "looking forward." In general, it was concluded that the mean prediction was reasonably good but the classification in damage classes in the propagation stage was

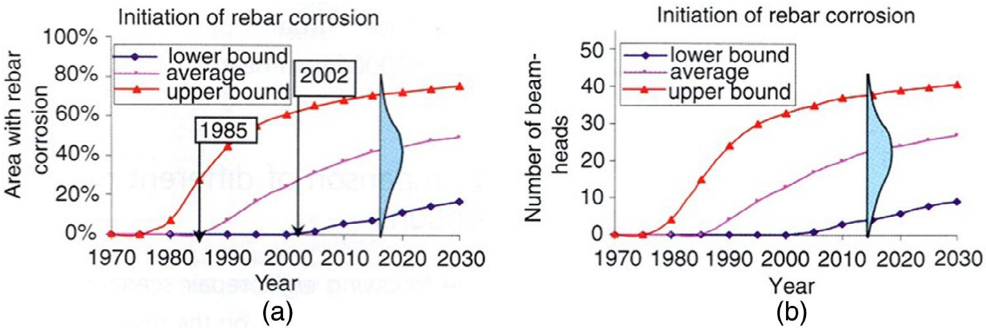

FIGURE 10 Prediction of initiation of rebar corrosion by area (a) and by number of beam heads (b) during 60 years.[12]

less accurate. It should be noted that this prediction was a first step to get experience with a probabilistic type of prediction of deterioration, and enabling an appropriate plan for inspection, maintenance, and eventual strengthening. In the *fib* MC 2010, relations were given for time dependency of the chloride diffusion coefficient, which is in particular essential if pozzolans and slag are used. An important further recommendation in *fib* MC 2010 was the introduction of the "apparent" chloride coefficient D_{app}, which is the diffusion coefficient as determined from measurements at the structure itself. The apparent coefficient of diffusion is determined by an inverse analysis from measured chloride profiles. In this way, many influences are implicitly regarded, like the influence of construction quality, but also effects like the influence of capillary suction and degree of saturation of the concrete under the local conditions. Moreover the effect of "spatial variability" can be considered. Spatial variability includes *systematic* spatial variation (mean value and standard deviation) and *random* spatial variation. The effect of systematic variation can be regarded in a practical way by distinguishing areas with an increased risk of corrosion, where orientation to solar radiation, exposure to strong wind, and temperature effects play a role. The effect of random spatial variation is more complex, but its effects can be regarded by the determination of D_{app} by taking a sufficient number of drilled cores from various locations to determine the chloride profiles. In this way, better results can be expected now, especially with regard to the reduction in the variability of the results (the 20% and 80% values).

The condition of new concrete structures should preferably be monitored and inspected over the life of the structure, with this ideally being instigated during the construction stage. By monitoring and regular testing, the evolving need for maintenance can be followed and the condition of the structure can be known at any time. This action also supports the possibility of adapting the structure to carry other loads or changing its function.

6 | USE OF HIGH LEVELS OF APPROXIMATION FOR THE ASSESSEMENT OF EXISTING STRUCTURES WITH DETERIORATION

The level of approximation approach was introduced in the *fib* MC 2010. It allows for the use of more advanced and complex models for the determination of the behavior of concrete structures and especially their structural safety. But, if structures suffer substantial deterioration, the concrete properties and the constitutive relations, at least for a part, are subject of speculation. For the assessment of existing structures, with deterioration, the question is, therefore, whether the highest levels of approximation are still suitable to generate results with the highest reliability.

As mentioned before, for existing structures assumed relations between concrete compressive strength and other parameters may no longer be valid. As a consequence, this might also influence the validity of the behavioral models, derived from laboratory tests on specimens with new, undamaged concrete. Moreover, contrary to laboratory tests, for existing structures with effects of deterioration, it is more difficult to describe the material properties, since they depend on the state of damage, which cannot always be well described. The state of damage may be clear at the surface of the structure, but the condition of the inner part of the structure is more difficult to describe and evaluate accurately. Furthermore, many measuring systems have insufficient capacity to supply accurate data. As an alternative the assessment of an existing structure with deterioration can be conducted assuming plausible worst case scenarios.

In Ref. [9], an investigation by M. Prieto is reported, aiming at predicting the shear capacity of a series of beams with corroded reinforcement, using the shear equations given in *fib* MC 2010, levels II and III (chapter 7.3.3.3 in Ref. [9]). The results are remarkable: the "less accurate" level II method gave significantly better predictions than the level III method, which is assumed to be more accurate. At the first sight, this shows that it may be advisable not to use the method with the highest accuracy in design, anticipating on "hidden effects" of deterioration or missing parameters in both level II and III methods for structures with deterioration. Further research in this respect is planned.

7 | CONCLUSIONS

1. In the new *fib* MC 2020, assessment of existing structures is a subject of major attention. Although meanwhile substantial experience has been gained in assessing various types of existing structures, it is still an area that needs further development.
2. Using design codes for the assessment of existing structures may lead to considerable conservatism and unnecessary investments in repair and strengthening.
3. The use of higher levels of approximation can be advantageous for the assessment of structures without deterioration, but prudence is needed if those structures are affected by deterioration.
4. For the assessment of existing structures, preferably models should be used that reflect the behavior of structures as realistically as possible. Empirical equations should be handled with care.
5. NLFEM models may be very useful for the assessment of existing structures (including deteriorated structures and structures with noncompliant details) if used in combination with suitable verification tests and appropriate calibration procedures.
6. The technology of proof loading of existing structures should be supported by appropriate recommendations, especially if deterioration effects are involved. Acoustic emission is a subject of further development.

DATA AVAILABILITY STATEMENT

Data sharing is not applicable to this article as no new data were created or analyzed in this study.

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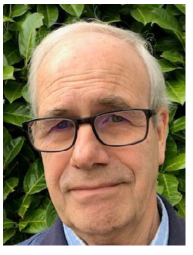

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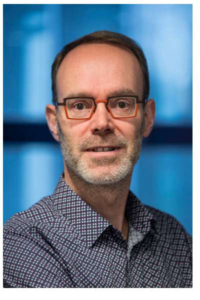

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