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ARTICLE

Approach to assessment of existing structures in the *fib* Model Code 2020

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Abstract

Existing structures often no longer meet the demands formulated in contemporary design codes with respect to structural safety and serviceability. This occurs, for instance, if the loads on existing structures, like traffic loads on bridges, are larger than assumed in the original design. A second potential reason is that structures are subject to deterioration, like alkali–silica reaction within the concrete or corrosion of the reinforcement due to chloride attack or carbonation. A third possible reason is that in recent codes, additional criteria have been introduced based on new theories and/or negative experiences with older structures. The new *fib* Model Code for Concrete Structures 2020 will be valid both for the design of new structures and the assessment of existing structures. This paper shows how the design and assessment of concrete structures are integrated into this new code concept.

KEYWORDS

deterioration, existing concrete structures, MC 2022, structural safety

1 | INTRODUCTION

In the past concrete structures have predominantly been designed for structural safety. Meanwhile, it has turned out that the magnitude of the loads on structures often has been underestimated due to unforeseen developments. This applies, for instance, to traffic loads on bridges: in traffic there is a trend that transport of goods is carried out in less but larger vehicles, which has had a significant influence on the axel loads. Simultaneously the bearing capacity of structures has been influenced by the effect of aging, resulting in a reduction of the bearing capacity and as such a reduction of structural safety. It is clear that these opposing trends may lead to future problems with respect to structural safety.

This raises the interest in another aspect of the actual structural behavior of an element, being the potential availability of residual capacity. For the design of new structures, models have been developed based on experiments carried out in university laboratories. Models have been developed with the aim to describe the structural behavior as accurately as possible, supported by the test results. However, those theories are often too complex for design purposes and have therefore been simplified to generate more user-friendly rules and principles to be implemented in codes. A consideration was here that simplified and transparent design rules are appropriate for design and in general lead to less misinterpretations and design errors than more complex models. Those simplified design rules are, admittedly, more conservative than their more advanced alternatives, but the resulting additional costs and required materials involved in design and construction are rather limited. A very important spin-off of this development is that there may be "hidden" residual capacity not taken into account during design, but which might be proven in an assessment. By virtue of this "hidden" residual capacity, the required structural safety can still be proven, in spite of the increase of the traffic loads in time. Figure 1 shows this development. In the figure, the curve $S(t)$ shows the increase of the load in time, whereas the descending curves $R(t)$ represent the decrease of the (design) resistance due to aging effects. The upper descending curve $R(t)$ in the figure includes the contribution of "hidden" residual capacity. The possible availability of residual capacity has been the main reason to offer different levels of approximation (LoAs) in the *fib* Model Code 2010. The lower LoAs are suitable for basic design purposes, whereas the highest LoAs are more appropriate for assessment of existing structures, based on which important decisions about the eventual strengthening of the structure should be taken. The more refined intermediate-level LoAs can be used, for instance, for a more accurate design or for a first estimation of the residual bearing capacity with limited effort required.

To understand and quantify the decrease of bearing capacity in time, adequate knowledge of the possible deterioration processes is required. With this knowledge not only the present safety level can be better determined, but also its future development. This may also give indications for inspection (type and interval), monitoring, and eventual strengthening measures.

Another aspect, relevant for the position of the curves $R(t)$ is that the original design may have been based on an outdated design code. This might imply that the structure contains non-compliant details, which may have a negative effect on the bearing capacity. All those aspects are regarded in the draft of the *fib* Model Code 2020. Later in this paper, they will be addressed in more detail.

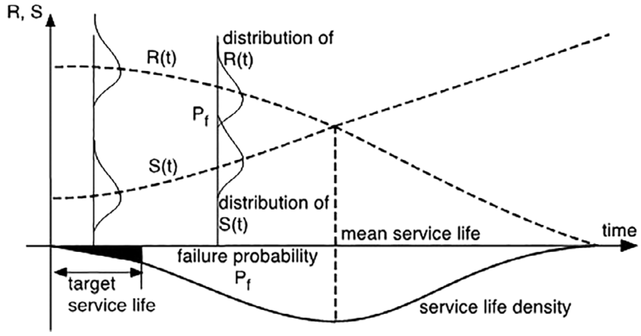

FIGURE 1 Increase of the load on a structure in time $S(t)$ in comparison with two curves showing the reduction of structural resistance due to aging $R(t)$, where the upper $R(t)$ curve includes the contribution of residual capacity.

2 | GENERAL APPROACH TO ASSESSMENT

General considerations on carrying out an assessment are given in chapter 28 of *fib* MC 2020. Various levels are considered.

- Level A0: Informal qualitative assessment

A preliminary assessment, essentially based on visual inspection and engineering experience, allows making a preliminary evaluation of the condition of a structure. In this way, it is possible to see the possible effects of deterioration, such as corrosion of reinforcing steel, as soon as there are visible indications of damage (e.g., cracks and/or spalling).

- Level A1: Measurement-based determination of load effects

This enables the assessment of serviceability by measuring performance values and their comparison with threshold values. No structural analysis is carried out. The threshold values can be those given in codes or can be individually specified. The assessment can be made under working loads or a defined proof load.

- Level A2: Partial factor method, based on document review and visual inspection

Assessment of load-carrying capacity and serviceability is undertaken using information from codes, design, and visual inspection documents. Structural analysis is generally carried out using simple (design) methods, but more refined analysis may be used. Safety and serviceability verifications are based on partial factors methods.

- Level A3: Partial factor method, based on supplementary investigation

Assessment of load-carrying capacity and serviceability is undertaken using additional information gained from site-specific detailed surveys, investigations, non-destructive testing procedures, and (potentially) load models, if appropriate. Structural analysis is carried out using refined methods and detailed models. Safety and serviceability verifications are based on partial factors.

- Level A4: Modified target reliability—Modification of partial factors

Verification of the load-carrying capacity is undertaken using site-specific modified partial safety factors. Structural properties, as well as external circumstances, can influence the safety evaluation. Practically, modifications of the partial factors would be carried out for groups of structures with similar structural behavior of load influences, where such a portfolio of structures exists (e.g., bridges owned by a particular authority).

- Level A5: Full probabilistic assessment

The assessment is made taking into account the statistical properties of all the basic variables. Structural reliability analysis is used directly, instead of partial factors. Uncertainties are modeled probabilistically.

3 | EVALUATION OF STRUCTURAL SAFETY OF DETERIORATED STRUCTURES

3.1 | General

Deterioration of concrete structures can occur due to various reasons: the properties of the concrete may suffer from internal effects, like alkali–silica reaction (ASR) and internal sulfate attack. Moreover, external influences can play a role: the alkalinity of the concrete around the bars, which fulfills a protecting function for the reinforcement, can be lost due to carbonation of the concrete or due to chloride ingress to the reinforcement exceeding a critical level. Other external processes that may play a role are frost-thaw cycles, external sulfate attack, or leaching and acid attack.

For the evaluation of the structural safety of existing concrete structures and their expected development in time, as much as possible the same basic models are used for the design of new structures. If necessary, those models will be extended to take into account the effect of material degradation.

3.2 | Deterioration by ASR

ASR is a chemical degradation mechanism that impacts existing concrete structures because it causes the expansion of concrete. ASR-affected structures commonly display map cracking. The cracking pattern observed by the structure is influenced by loading (stress) effects and restraint conditions. In this context, restraint is provided by the reinforcement in the concrete structure (i.e., internal restraint) and boundary conditions of the structural component under investigation (e.g., abutments). The directions in which there is minimal or no restraint are the directions where ASR-affected expansions reach their maxima. So, the properties of the concrete itself are influenced by crack mapping. This effect strongly depends on the composition of the concrete and the expansion of this as shown in Figure. 2. In general, the modulus of elasticity and the tensile strength are influenced to a greater extent than the compressive strength (Esposito[1]).

A second effect occurs in structures due to imposed deformations, resulting from different reinforcement directions and concentrations of reinforcement or imposed deformations on a structural level.

Strategies on how to deal with the effect of ASR in an assessment are given in section 30.1.11.2 of *fib* MC 2020. Charts are given for lower bound relations for the concrete compressive strength, the tensile strength, and the E-modulus as a function of the free expansion of ASR-affected plain concrete. However, a warning should be given that using those values in structural analysis without modeling the beneficial effects of confinement, will result in gross underestimation of structural resistance.

Short general descriptions are given with regard to the expected impact of ASR on various aspects of structural behavior:

a. Impact of ASR on axial capacity:
   The impact of ASR on axial compressive capacity shall be considered to be small. The adverse influence of chemical prestress to structural capacity in axial compression shall be accounted for in structural analysis;
b. Impact of ASR on flexural capacity:
   The influence shall be considered to be negligibly small. Chemical prestress does not influence the flexural capacity of a reinforced concrete section. Moreover, it shall be regarded in structural analysis;
c. Impact of ASR on shear strength:
   The impact of ASR on the shear capacity of members with a sufficient quantity of shear reinforcement shall be considered to be negligibly small;
d. Impact of ASR on punching shear strength:
   The impact of ASR on the punching shear capacity of members without shear reinforcement shall be considered to be negligibly small;
e. Impact of ASR on anchorage of deformed reinforcing bars:
   The impact of ASR on anchorage of deformed reinforcing bars in plain concrete shall be considered to be about 40%. This reduction in capacity is associated with the ASR-induced volumetric expansion limit of 1.5%. Linear interpolation between 40% reduction and no reduction for 0% volumetric expansion shall be permitted;

As mentioned already, the role of confinement is essential. Miyagawa[2] formulated this in the following way: "As long as the reinforcement is not fractured due

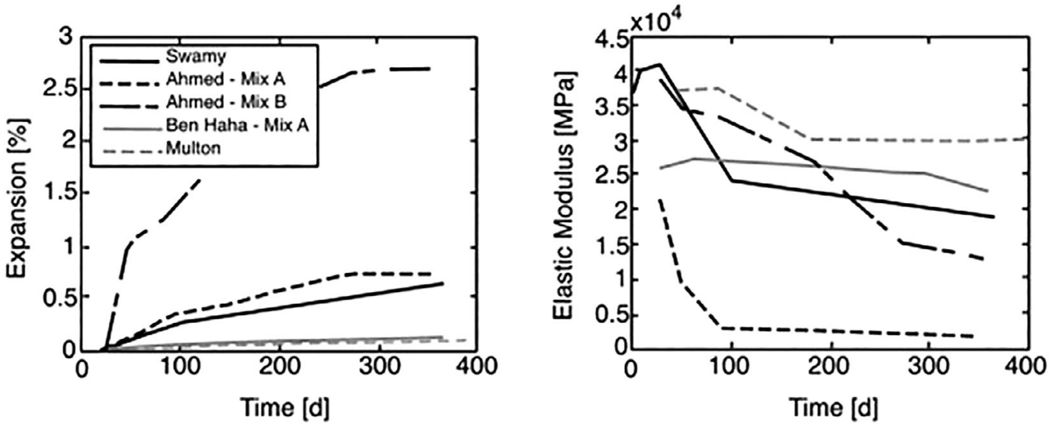

FIGURE 2 Expansion due to ASR and E_c-modulus as a function time, from different sources. ASR, alkali–silica reaction.

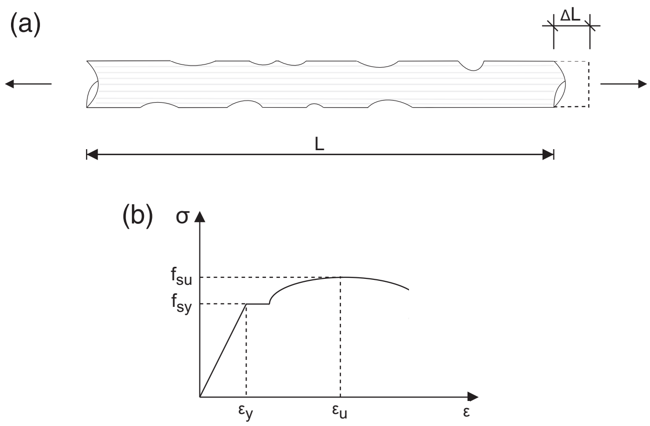

FIGURE 3 (a) Reinforcing bar suffering pitting corrosion. (b) σ–ε relation for undamaged steel bar.

to ASR-related expansion, the safety of a structure is not considered to be seriously compromised. However, the safety of a structure becomes questionable when confinement of the concrete degrades due to fracture of reinforcement."

The ASR part of the MC2020 was contributed by O. Bayrak.

3.3 | Structures with corroded reinforcement: Change of properties of reinforcing steel due to corrosion

To be able to assess the influence of the effect of corrosion of reinforcing steel on the resistance of a concrete structure, it should be verified whether the corrosion is uniformly distributed along the reinforcing bar(s), which may occur in the case of carbonation, or is localized, which may happen in case of pitting corrosion due to chloride penetration. In the first case, the effect of corrosion results predominantly in a reduction of the cross-sectional area of the steel, whereas the ultimate strain will not be significantly influenced. In case of pitting corrosion, the geometry of the pits and their distribution along the reinforcing and prestressing steel plays a role, since here not only the reduction of the tensile capacity of the reinforcing steel is relevant but also the reduction of the ultimate strain.

Figure 3 shows a schematic representation of a reinforcing bar, suffering pitting corrosion (Zeng et al.[3]) in combination with the stress–strain relation of an undamaged bar (Figure 3b). For determining the effect of corrosion on the strength characteristics and ductility of the reinforcing steel, the loss of mass of a steel bar due to corrosion can be used. The mass loss ratio M_{corr} is defined as:

$$M_{corr} = (m_0 - m_1)/m_0 \tag{1}$$

where m_0 is the (estimated) weight of the reinforcing bar before corrosion and m_1 is the weight of a bar with the same diameter and length after corrosion.

The loss of strength is determined assuming that the bar diameter remains the same and that the yield stress, the tensile strength, and the ultimate strain are reduced. For the description of the loss of yield stress and tensile strength, linear descending relations have been derived. For uniform corrosion, the following decay relations can be used for determining the reduction of yield stress and ultimate steel strain at the maximum tensile force (Imperatore et al.[4]):

$$\frac{f_{sy,corr}}{f_{sy}} = 1 - 1.43M_{corr} \tag{2a}$$

$$\frac{\varepsilon_{su,corr}}{\varepsilon_{su}} = e^{-2.0M_{corr}} \tag{2b}$$

For pitting corrosion, the following linear reduction expressions can be used:

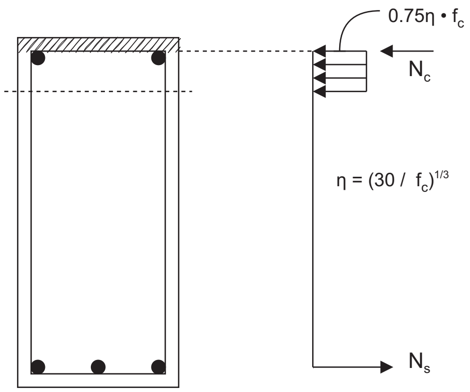

FIGURE 4 Assessment of bending capacity of a cross-section with corroded reinforcement.

$$\frac{f_{sy,corr}}{f_{sy}} = 1 - 2.0M_{corr} \tag{3a}$$

$$\frac{\varepsilon_{su,corr}}{\varepsilon_{su}} = e^{-5.5M_{corr}} \tag{3b}$$

3.4 | Structures with corroded reinforcement: Determination of remaining resistance

For the determination of the resistance of structural concrete members with corroded reinforcement, as much as possible use is made of the (design) models given for structures with undamaged reinforcing steel. In the following, some examples are given.

3.4.1 | Members subjected to bending

When the bending resistance of a structure with corroded reinforcement is determined, the following aspects should be considered (Figure 4)

- The effect of reduction of concrete strength in the compression zone due to the splitting effect of the compression reinforcement caused by the volume increase due to corrosion product;
- The reduction of the cross-sectional area of the reinforcing steel and/or its ductility;
- The reduction of bond strength due to corrosion

For low to moderate levels of corrosion, the effect of the expansion of the reinforcement in the compression zone can be taken into account by reducing the concrete compressive strength with a factor $k_c = 0.75{\cdot}\eta_{fc}$ where $\eta_{fc} = (30/f_{ck})^{1/3}$ is a factor taking into account the more brittle behavior of such a concrete for $f_{ck} > 30$ MPa. As a conservative approach, it can be assumed that in the compressive zone, the concrete cover should be ignored in the calculation of the bending resistance.

For the tensile resistance of the reinforcement, the residual cross-sectional area should be determined using the minimum bar cross-sections measured along the whole length of the tensile chord. In case of broken stirrups, the compressive reinforcement may not be fully effective and should not be considered in the calculations. For the determination of the yielding force in the reinforcement in the tensile zone, only the reduction of the cross-sectional area of the bars has to be taken into account.

3.4.2 | Shear capacity of slabs without shear reinforcement with corroded longitudinal reinforcement

In this case, the basic expression for the shear resistance of similar members with non-corroded steel can be used. In section 30.1.3.2 of the draft of *fib* MC 2020, the design shear resistance of the web of a beam, or a slab without shear reinforcement is given as:

$$V_{Rd,c} = k_v \frac{\sqrt{f_{ck}}}{\gamma_c} b_w z_v \tag{4}$$

where $\sqrt{f_{ck}} \leq 8$ MPa and $z_v \leq 800$ mm, were in general $z_v = 0.9d$ may be assumed. There are two LoAs. According to the second level, the parameter k_v follows from:

$$k_v = \frac{0.4}{1 + 1500\varepsilon_x} \times \frac{1300}{1000 + k_{dg} z_v} \; (z \,\text{in}\, \text{mm}) \tag{5}$$

In this equation, ε_x is the longitudinal strain in the structural member calculated at mid-height of the effective shear depth according to eq. (30.1.10) of Model-Code 2020:

$$\varepsilon_x = \frac{1}{2E_s A_{s,det}} \left\{ \frac{M_{Ed}}{z} + V_{Ed} + N_{Ed}\left(\frac{1}{2} \pm \frac{\Delta e}{z_v}\right) \right\} \geq 0 \tag{6}$$

For corroded longitudinal reinforcement, the expression is extended with a bond reduction factor k_{bond}, according to:

$$\varepsilon_{x,det} = \frac{1}{2E_s A_{s,det}} \cdot \frac{1}{k_{bond}} \cdot \left\{ \frac{M_{Ed}}{z} + V_{Ed} + N_{Ed}\left(\frac{1}{2} \pm \frac{\Delta e}{z_v}\right) \right\} \tag{7}$$

The bond factor k_{bond} accounts for the effect of reduced bond due to corrosion. Where $k_{bond} = 1.0$ for

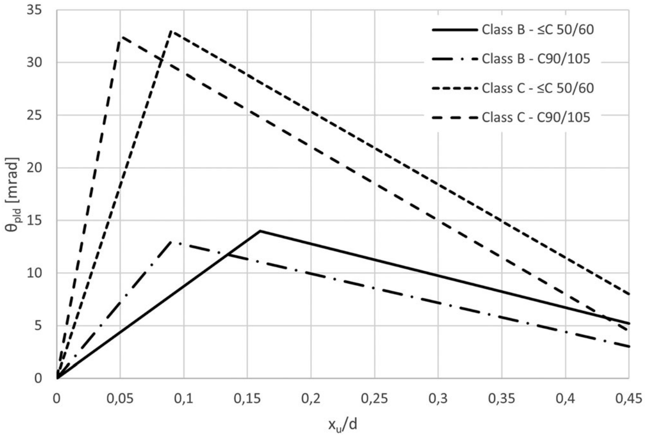

FIGURE 5 Allowable rotation θ_{pld} for reinforced concrete beams and slabs with uncorroded reinforcing steel bars class B and class C.

non-corroded bars, for bars with a moderate level of corrosion less then 5% weight loss kbond = 0.75 may be assumed.

Where $A_{\mathrm{s,det}}$ is the cross-sectional area of the bar reduced by deterioration (in case of uniform corrosion). In case of pitting corrosion, $A_{\mathrm{s,det}}$ is the minimum cross-sectional area of the tensile reinforcement in the area where flexural shear cracks occur.

In case of severe corrosion, causing crack formation along the longitudinal bars, strongly reduced values of k_{bond} apply. Accurate values cannot be given here. If the tensile tie consists of more than one layer of rebars, different values of k_{bond} per layer may apply, which should be combined into one value for the whole tensile tie.

3.4.3 | Rotation capacity

In the design of statically indeterminate beams and slabs often use is made of the possibility of redistribution of bending moments. To this aim, the rotation capacity of the plastic hinges should be checked. The allowable rotation is calculated as the product $1.2h{\cdot}\theta_{\mathrm{f}}$ where h is the effective depth of the section and θ_{pld} is the allowable rotation, which can be read, for reinforced beams and slabs, from the diagram shown in Figure 5. However, for the steel classes B and C, the following limit values apply:

Class B: $(f_t/f_y)_k \geq 1.08$ and $\varepsilon_{uk} \geq 5‰$;

Class C: $(f_t/f_y)_k \geq 1.15$ and ≤ 1.35 and $\varepsilon_{uk} \geq 7.5‰$;

It should be verified whether the corroded steel in the plastic hinge region (with a defined effective length of $1.2h$) satisfies those demands, taking into account the shape of the pits and the hardening capacity of the reinforcing steel used. Here, Equations (2a), (2b), (3a), and (3b) can be used.

3.4.4 | Models to determine the resistance related to other failure modes

Models to determine the resistance related to other failure modes are given as well, like shear capacity of members with shear reinforcement, punching shear capacity, and bearing capacity under combined bending and normal forces. In all cases, the behavioral models for undamaged structures serve as a starting point.

4 | STRUCTURES WITH NON-COMPLIANT DETAILS

4.1 | General considerations

If the contemporary code for structural design is used for the assessment of an existing structure, it may turn out that the existing structure satisfies the detailing demands of the original (old) code but not those of the current code. The detailing of such structures is then defined as "non-compliant."

Where non-compliant detailing is identified, the behavior of the respective region should be investigated, including its resistance and its ductility. If the detailing of the structure is locally non-compliant, its effect on the overall structural behavior should also be investigated. This should result in a decision as to whether the structural safety requirements are met or not.

Various possibilities can occur:

a. Structures designed using former codes may have a satisfactory residual resistance, even in spite of the effect of non-compliant detailing. So it might be that the structure still satisfies the demands expressed in the current governing design recommendation.
b. The structure does not meet the requirements of the current code governing design, but the difference is limited and the structure has demonstrated satisfactory performance over many years and it functions appropriately. In such a case, a slight decrease of the structural safety level for an existing structure can be accepted, whereas the prospective satisfactory performance over the remaining service life is a point of further consideration.
c. The (estimated) structural safety is below an acceptable level. In this case, the structure should be strengthened to meet the requirements for future use

FIGURE 6 Plain steel applied in many older structures in case of ribbed steel as used nowadays.

under the governing loading conditions or closed for further use.

4.2 | Examples of non-compliant details

Non-compliant details to be considered include:

- Smooth or weakly ribbed reinforcement (Figure 6) associated with lower bond strength which is inferior to indented (ribbed) steel with regard to crack width control, while leading to wider cracks; lower bond strength can give rise to increased ductility of the structure. Furthermore, smooth and weakly ribbed reinforcement can result in larger deflections of structures than would be the case with ribbed reinforcement. The lower bond can lead as well to a reduction of the flexural capacity. However, the lack of bond of the longitudinal steel does not lead to a decrease of the shear—or punching shear—capacity of members without shear reinforcement. The reason is that if the steel is smooth, no inclined shear cracks can develop. At the building site, however, the steel is generally not perfectly smooth, partially due to slight rusting, which increases the bond strength. The difference in shear capacity between members reinforced with "smooth" or ribbed bars, is therefore generally small (Yang et al.[5]);
- Reinforcing bars with low ductility properties, which can lead to reduced ductility of the structure;
- A small concrete cover, which may result in reduced bond strength in the longitudinal bars, but also to an increased ductility at the structural level;
- Small bar spacings or a high number of bar layers, which can result in reduced bond strength and a higher risk of concrete cover spalling over large surface areas;
- Excessively small bending radius of reinforcing bars in the anchorage region, which may result in reduced fatigue strength and increased transverse tensile forces;
- An excessively pronounced staggering of longitudinal reinforcement, which can lead to loss of bending resistance of structural members;
- Insufficient anchorage of transverse reinforcement in flexural tensile zones, which can lead to loss of anchorage capacity;
- Shear reinforcement not enclosing the longitudinal reinforcement reducing the shear capacity;
- Shear reinforcement following the shape of the outer surface of structural members with variable width;
- Indirect supports with no, or insufficient, suspension reinforcement;
- Dapped-end beams (half-joints) with insufficient anchorage of reinforcement and/or insufficient suspension reinforcement;
- Supports with insufficient sliding capacity due to unsuitable design;
- Insufficient reinforcement, which does not accord with minimum reinforcement requirements in bending, shear, and punching.

Nowadays, structural details are often designed according to the principle of strut and tie models. That implies distinguishing D-regions and B-regions, where in the D-regions, the struts are oriented into the direction of the compressive stress trajectories in the uncracked stage, and the tensile ties should be as short as possible. However, the recommendations for strut and tie modeling have only been part of code recommendations in the last decades but not before. Figure 7 shows, as an example, three design options (a–c) where Figure 7a leads to an underestimation of the force in the tensile tie *H* and as such to a too low cross-sectional area of the corresponding tensile tie. Figure 7b shows the correct model, whereas Figure 7c gives an alternative for a good solution using the available horizontal stirrups over the height of the corbel.

Existing structures that are assessed often show sub-optimum reinforcement in the D-regions. In such a case, the strut and tie method can be used as well, but now looking for allowed alternative ways for the transmission of forces which have to be tailored to the given reinforcement configuration.

For complicated cases, an NLFEM analysis could be considered. This method is especially efficient in combination with experiments. For the application of NLFEM analysis, guidance is given in the draft of the *fib* Model Code 2020 as well.

Finally, proof loading a structure with non-compliant details is an option: it can be considered in the case that available behavioral models, in combination with the relevant material properties, may provide a reliable estimation of the structural resistance.

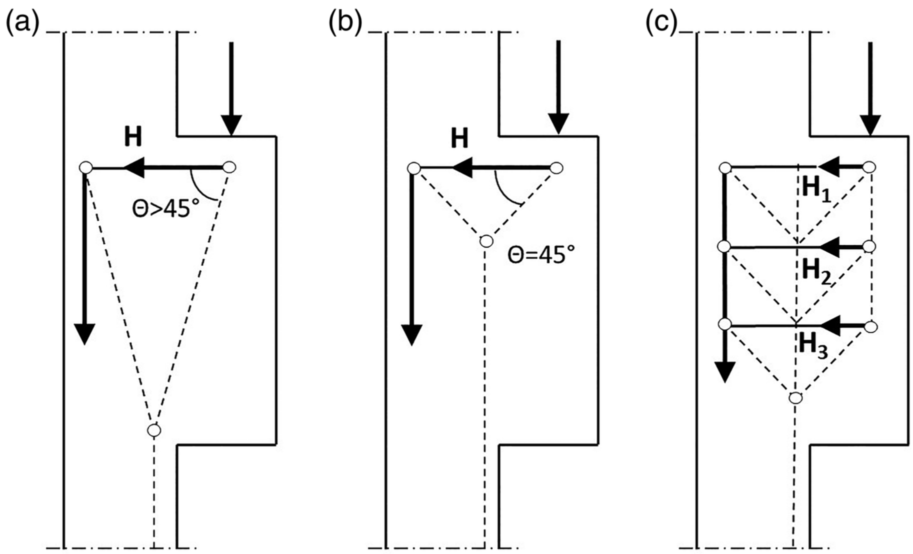

FIGURE 7 (a) Wrong assumption of strut inclination leading to underestimation of H; (b) correct assumption of strut inclination; and (c) alternative representation by model, taking into account additional stirrups over the height of the corbel.

5 | PREDICTING THE REMAINING SERVICE LIFE OF EXISTING STRUCTURES

Until now, this paper focused predominantly on determining the actual condition of existing structures, including their structural safety. The results are used to make important short-term decisions, related to safety for the users, eventually allowing an increase of the loads, or using the structure for another purpose, etc. However, an important aspect is as well the development of the condition of the structure in time. How long will the structure still satisfy the demands of structural safety? How should inspection and monitoring be arranged, and what should be measured? Should the structure be strengthened and how should this be done?

To be able to answer such questions adequately, in chapters 14 and 30.6 of the draft version of the Model Code 2020, descriptions are given of different types of deterioration processes. This may help to anticipate on the future development of the condition of structures and choose an optimum repair and strengthening strategy.

An example is the prediction of the service life of structures subject to chloride attack. In section 30.6.3.2 of *fib* MC 2020, the following expression is given for chloride penetration:

$$C(x,t) = C_0 - (C_{s,\Delta x} - C_0) \cdot \left[erfc \frac{x - \Delta x}{2 \cdot \sqrt{D_{app}(t) \cdot t}} \right] \quad (8)$$

where C(x,t) = content of chlorides at depth x and time t, $C_{s,\Delta x}$ = chloride concentration at depth Δx (equivalent surface chloride concentration), C_0 = initial chloride content of the concrete, and D_{app}(t) = apparent chloride diffusion coefficient, which can be calculated by means of an

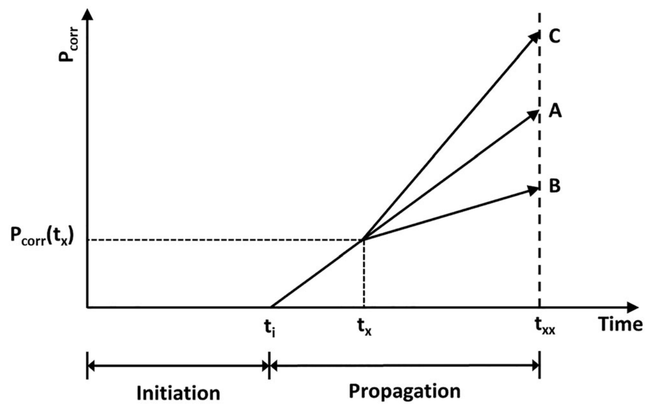

FIGURE 8 Development of depth of corrosion in reinforcing steel as a function of time (courtesy C. Andrade).

inverse analysis from a measured chloride profile at the structure. By measuring D_{app} at various places and measuring as well the values of the concrete cover, the mean value and the variability of those parameters can be determined. For a critical chloride content (mostly assumed to be 0.6% of the cement weight), a probabilistic analysis can be made, showing which part of the reinforcing steel is in the corroding stage, with which probability, and how this can be expected to further develop in time.

A more simple analysis can be made if at an existing structure, the depth of corrosion P_{corr} in the reinforcing steel and its progress in time is measured (Tuutti[6]). The bilinear relation, according to Tuutti, is shown in Figure 8 as line A, in combination with the horizontal part 0-t_i (where t_i is the initiation time of steel corrosion). The inclined part of the relation (line t_i – A) can be derived from the measurements taken at the structure.

However, it should be realized that the assumption of a linear increase of the corrosion penetration in time is a simplification. During corrosion, the volume of the rebar increases since the volume of rust is larger than the original steel volume. This leads to radial crack formation around the bar and finally loss of the concrete cover, which may result in higher rate of corrosion. However, this depends on whether the subsequent time of wetting of the reinforcement was increased or reduced by the formation of the crack. A longer time of wetting would usually be expected to increase the corrosion rate. This is represented by line C. However, it can also happen that the circumstances develop in a more favorable way, as a result of an intervention. The link to the mechanical properties of the steel bars can be made using the mass-loss ratio η according to Equation (1). The reduction of the tensile strength of the bar and the expected reduction of the ultimate strain can then be determined by Equations (2a), (2b), (3a), and (3b) depending on the type of corrosion. So, in this way, the loss of strength of the bars and thus the structural resistance in time, and its further development, can be calculated.

6 | CONCLUSIONS

1. The assessment of existing structures should aim at the determination of the structural safety and its further development in time (remaining service life) and the determination of required/possible interventions to the structure or parts thereof to extend the useful service life.
2. For the determination of the structural safety and its development in time account should be taken of a possible increase of the loads on the structure in time and of a reduction of the bearing capacity due to different types of deterioration.
3. Structures built in the past may have been designed according to rules which are meanwhile out of date. The possible effect of non-compliant details on the bearing capacity should therefore get due attention.
4. If design recommendations for new structures, as found in building codes, are used for the assessment of existing structures, conservative results may be obtained. For assessment therefore more advanced rules may therefore be more appropriate. For this reason, in MC 2020, methods of analysis with different levels of accuracy are offered. According to this "Level of Approximation" approach, for any case considered, the most appropriate method can be selected. In general, the higher LoAs require more effort but may lead to more accurate and economic solutions.

DATA AVAILABILITY STATEMENT

Data sharing not applicable to this article as no data sets were generated or analysed during the current study.

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