FORENSIC ENGINEERING: NEED FOR A NEW PROFESSIONAL PROFILE

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Abstract: Since decades structural engineers have been educated to design new buildings. This is certainly a good start of a career as a structural engineer since it is an appealing perspective to use creativity and to support society with structures that are safe and serviceable during a long service life. However, structures are often not built as they should be, which may result in malfunctioning, insufficient structural safety or even collapse. In such a case the cause of the problem has to be determined a quickly as possible, in order to eliminate the risk or inconvenience for the users. To this aim experienced engineers are required, who are able to come to a quick assessment based on reliable judgement. In the past, problems with structures were mostly incidental, but nowadays new tendencies are observed, like aging of structures and their consequences, the use of software without understanding its background, and changes of the function of a structure during service life. That means that there is a growing need for another profile in structural engineering: the forensic engineer. The need for, and the requirements to such a new professional profile are treated in this paper

Keywords: forensic engineering, damage, collapse, assessment, structural failure, diagnosis, concrete structures

1. Introduction

An important task of the structural engineer nowadays is not only to design new structures, but as well to find solutions for existing structures. In a number of cases there is a need for immediate intervention, such as in the case of the damage or even collapse of a structure. Then it is not only important to eliminate the reason of the structural unsafety as quickly as possible, but as well to indicate which measures are necessary to restore the structure to such a level, that further use is possible, respecting the required structural safety level. An important task for the expert involved is to give an answer on the question who is guilty, because for restoring a structure a financial budget is required. This is not an easy task, since the problem mostly involves more aspects than just a violation of the building code. That means that the forensic experts involved should have considerable experience in structural engineering. Moreover they should not only have knowledge on the governing building codes, but as well on their background. In general they should able to well understand the behaviour of a structure under increasing load and/or deformation, and be aware of the fact that not only the material properties of a structure change in time, but often also the magnitude and configuration of the load and the function of the structure.

2. Why structures fail?

Analyses focusing on the reasons of structural failure have been carried out already for many decades. In a very interesting analysis of 250 structural failures and their causes, Blévot in 1974 concluded that the reasons for structural failure in concrete structures can be subdivided as follows:

– Errors in the structural concept: 3,5%,
– Errors in calculation hypotheses: 8,5%,
– Errors in reinforcement design or placement 2,5%,
– Ignorance of the effect of deformations 20,0%,
– Ignorance of time dependent effects 44,0%,
– Construction errors 15,5%,
– Chemical reactions or frost 4,0%,
– Miscellaneous 2,0%.

From this classification it is obvious that underestimating and/or not understanding the effect of deformations, in combination with time dependent effects, represent the utmost part of the damage (together 64%). A substantial part of this damage is due to not understanding the effect of imposed deformations: deformations by creep, shrinkage and temperature often cannot freely occur, because the structural element, as a part of the structure, is connected to other parts of the structure.

Fig. 1 shows the floor of an underground parking garage made in reinforced concrete. The floor has been designed for the load of the cars and on the upward pressure of the soil water, which can increase in winter. The floor was cast in summer; the mean temperature in winter is lower and moreover the floor is subject to shrinkage. Free shortening, however, is not possible because of the heavy columns, which are connected to foundation blocks in the sand below the floor. Because the design did not take account of shortening of the floor in its plane and its restraint, large cracks through the structure occurred through which the soil water could easily propagate from below.

Fig. 1. Leaking floor of a parking house due to ignoring the effect of imposed deformation

Another investigation, carried out in Switzerland, by Matousek and Schneider in 1978 [2], after investigating 400 cases, gave the following classification for the reasons of failure:
– Errors in design, calculation and planning 37%,
– Errors in construction 35%,
– Errors in design and construction 18%,
– Damage during service 5%,
– Miscellaneous 5%.

Even much more important than this classification was their remarkable conclusion that „about 1/3 of all damage cases and about ½ of all cases with human injuries could have been avoided, without any additional control, by normal attention, and adequate reaction, of the person who is next in the chain architect, engineer, contractor and executor. Moreover they stated that „by a well-organized control system more than 75% of all errors with material consequences and 90% with human consequences would have been detected in due time”.

That this important conclusion is still actual, turned out in a study focusing on structural safety by CUR (2005). Fig 3 shows the chain of participants in the process from initiative to the final structure. Many parties are involved: architect, design office, consultants, contractor and subcontractors. The information has to be transmitted from partner to partner. If, say, there are 12 activities and 21 communications, and all of them have a reliability of 99%, the probability of a good result is only 110-12-21 = 67%.

Fig. 2 shows the collapse of a part of a shopping centre under construction (Rotterdam, 2010). When casting the third floor, the scaffolding system failed. It turned out that this was due to the absence of a significant part of the bracings in this system, that should ensure stability. According to Terwel (2014) an advisor of the supplier noticed this and warned, but no amendments were introduced. The main contractor asked for a final check, but this was not carried out. According to Terwel „The underlying factors of this case are a lack of technical competencies of the assembly team, unclear responsibilities regarding structures, insufficient risk management for the temporary structure and inadequate checking. No party felt responsible for the quality of the supporting structures"

Fig. 2. Collapse of a shopping center under construction due to an unstable scaffolding system

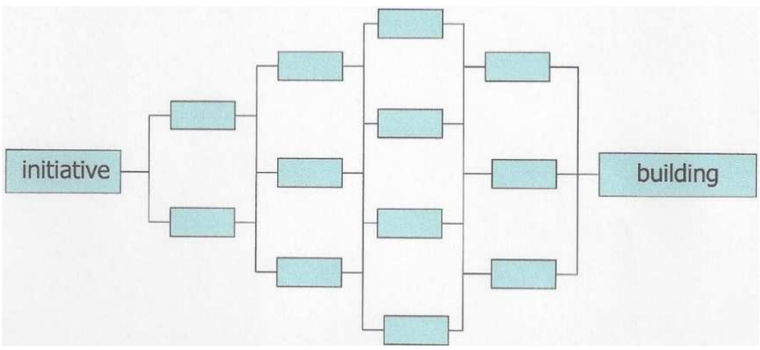

Fig. 3. Chain in design and construction (CUR 2005)

The following new classification of causes for structural failure was presented by CUR (2005). Three main levels are distinguished, all including a number of causes for errors:

– Micro level:
  - Errors by insufficient skill

– Meso-level:
  • Errors caused by inadequate organization, like:
    – Unclear commitment of tasks and responsibilities,
    – Insufficient interface management,
    – Inappropriate communication,
    – Inadequate quality of the control procedures,
– Macro-level:
  • Increasing specialization,
  • Pressure to work faster and cheaper,
  • Increasing fragmentation by increasing delegation of tasks,
  • Deficiency or reduction of general knowledge,
  • Complexity of building codes,
  • Lowest bid system (focus on cost instead of quality).

Fig. 4 shows the example of a series of balconies which failed by progressive collapse (Maastricht 2003). The balconies were supported only at one side by a steel column (Fig. 4 top right). The forces were all transmitted to the lower column at the 1th floor, which was supported by a corbel with inappropriate reinforcement detailing. This corbel failed in shear, resulting in the progressive collapse of the whole series of balconies. The first obvious cause of failure was the inappropriate detailing of the corbel and the risky support of balconies on a single corbel. Those can be regarded as errors on a micro-level, but a further consideration reveals that also on the meso- and macro levels errors have been made. The full survey is:

– Micro level:
  • Errors in detailing of the corbel,
  • Insufficient robustness of the structure,
– Meso level:
  • Insufficient communication between designer, construction team and the producers of the precast balcony slabs,
  • Insufficient steering of the process (too many independent partners),
  • Inadequate reaction on warning signs of the structure during construction,
  • Responsibilities not clearly defined,
– Macro-level:
  • Too small capacity of control authorities,
  • Insufficient skill,
  • Too many subcontractors.

Fig. 4. Progressive collapse of a series of balconies in a high rise building, Rotterdam 2003

3. Important capabilities of forensic engineers

3.1. Understanding the behavior of structures under loads and imposed deformations

Structures are in general designed on the basis of codes of practice. Very often codes are not fully transparent since they are a product of a compromise at the table of a code committee. Especially when compromises have to be made between different countries, like in case of the Eurocodes, the resulting rules do not fully reflect a rational model of the physical behavior. Furthermore the rules are simplified, so that the designer only has to make a number of checks. An example is the design in shear, where shear forces are represented by shear stresses in cross-sections, which hide the real behavior. So, the designer knows the rules but not the truth behind them. An illustrative example is given in Fig. 5, showing the collapse of a roof of a school under construction. Fig. 5a shows a part of the building under construction. Fig. 5b shows the collapse of the auditorium roof during roof during construction. Fig. 5c shows a cross section of the prestressed beam that should carry the auditorium roof and Fig. 5d shows a side view of the same prestressed beam, with large web openings. In the construction stage the beams are placed in parallel, so that the lower flanges form a closed plane, on which in-situ concrete is cast. Through the web openings transverse reinforcement connects the beams. After hardening of the in-situ concrete a composite roof is obtained, with a large bearing capacity. The problem however, is the situation just after casting the in-situ concrete. In this situation the beams have to carry the fresh concrete, which only contributes to the load and not yet to the strength. In this situation the shear capacity of the beam is decisive. However, the large web openings prohibit the formation of truss action, because the inclined compressive struts cannot develop.

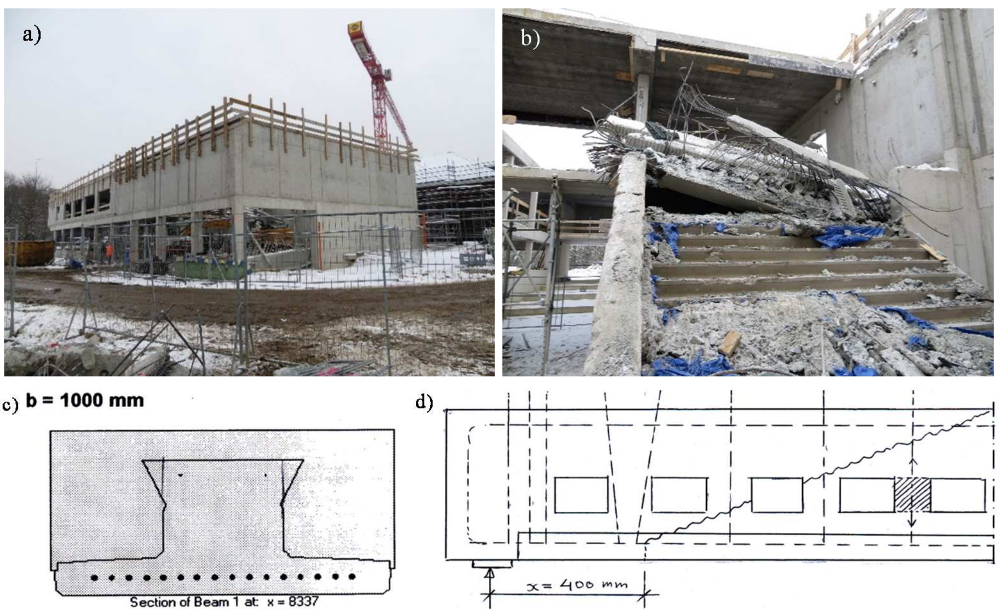

Fig. 5. a) School building under construction, b) Collapse of roof for auditorium, c) Cross section of prestressed beam, d) Side view on prestressed beam with web openings

3.2 Being able to determine the actual condition of a building by adequate measuring the material properties

When the structural reliability of a building is under discussion, it is of importance to be aware of the most advanced measuring techniques, in order to be able to judge upon the bearing resistance of the structure. A special case is shown in Fig. 6. The picture shows a new building on a Caribbian island just before finalization. However, the developer went bankrupt, and the building became the property of the bank. When the bank investigated the state of the building it turned out that the builders had decided during construction to build one story more than initially planned. Moreover it turned out that there were no drawings of the cross-sections of the columns, so that the reinforcement in the columns was unknown. Finally there were no data on the strength of the concrete.

In the first stage of the investigation concrete cores were drilled from a number of columns and beams. Those cores were used to determine the strength, but also to calibrate a rebound hammer, so that this device could be used to collect more information elsewhere in the building. The position and the diameter of the reinforcing bars in the columns were measured by a so-called Ferroscan (Fig. 7). The results could verified using pictures made during construction, showing the reinforcement of some columns. Finally it turned out that the structural safety was sufficient. A favourable condition was that the building had been designed for a seismic load assuming a too high seismic class (the design calculations had been made in another country where the local seismic design requirements were more severe).

Fig. 6. Building to be investigated

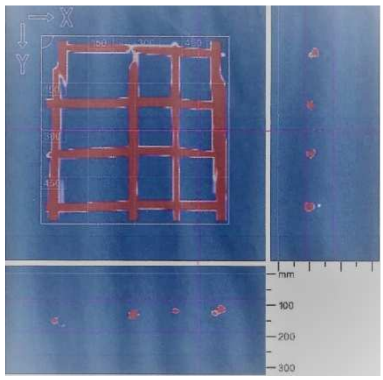

Fig. 7. Reinforcement in column determined by Ferroscan

3.3 Being able to make quick and reliable decisions in cases of potential danger

In situations in which suspect details or damage in a structure are detected, the consequences should be investigated as soon as possible and decisions have to be made with regard to further use of the structure and even evacuation. An example of such a situation was the la Concorde Overpass in Laval Canada in 2006. On a Saturday afternoon it was reported to the authorities that pieces of concrete fell down from the viaduct on the road below. The authorities decided to send inspectors on Monday morning, but this was too late. The bridge collapsed in the weekend.

A situation in which a quick decision had to be taken as well was the discovery that the reinforcement in the concrete bottom of the Maas Tunnel in Rotterdam was heavily corroded. In some places the reinforcement cross section was reduced to about 50%.

The tunnel, which was at the moment of the discovery of the corrosion about 75 years old has an important role in the city traffic system. It connects the northern part of the city of Rotterdam with the southern part and immediate closure would lead to an enormous traffic collapse. A quick evaluation of the bearing capacity is shown in Fig. 8b. It shows the representation the bearing system by an arch model carrying the upward load at the bottom of the slab due to water pressure to the walls. This quick check showed that, even when all reinforcement in the top of the slab would have been disappeared by corrosion, the bearing mechanism still has considerable residual bearing capacity. A favorable circumstance is that when the water level of the river Maas increases, not only the upward load at the bottom increases, but as well the lateral support of the arch, by the increased lateral water pressure at the walls of the tunnel (force N_W in Fig. 8b). Therefore it was decided that there was sufficient residual bearing capacity to drop the option of immediate closure and take some more time to investigate the structural safety more in detail. Afterwards two independent nonlinear finite element analyses confirmed that this decision was right.

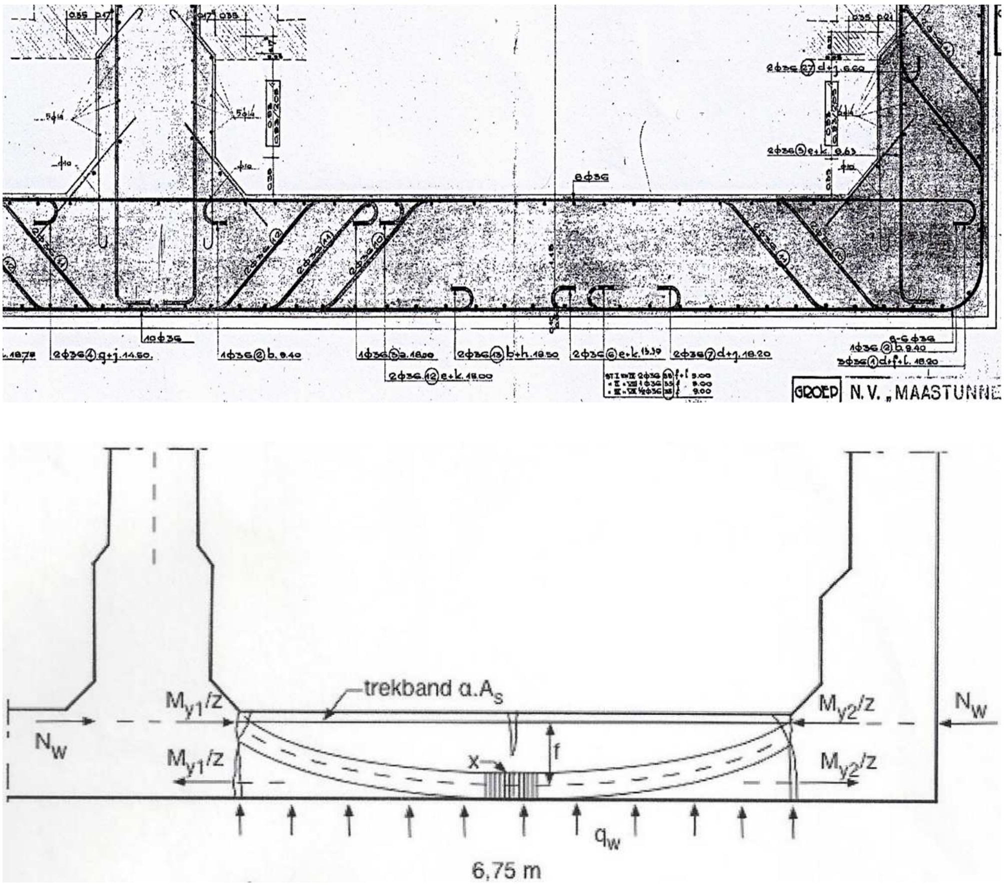

Fig. 8 a) Original design drawing of bottom slab of Maas Tunnel Rotterdam, b) Simplified representation of bearing mode based on arch-action

3.4. Being able to deal with the effect of aging of structures on structural safety

For many decades concrete structures have been designed for predominantly two criteria: structural safety and serviceability. There were no considerations about the service life of structures. The observation of substantial damage to concrete structures developing in time due to deterioration processes became however gradually a point of large concern. In the new *fib*-Model Code for Concrete Structures 2010 design for service life is a central issue. According to this principle structures are designed to satisfy the demands for safety and serviceability for a specified period of time. Structures serving the infrastructure of a country, like tunnels and bridges, are mostly designed for a low maintenance service life of 100 years. In structures like buildings and offices the functional service life is often the governing criterion, so that a physical service life of 50 years is most often required. Nowadays we dispose of considerable knowledge with regard to deterioration processes, and we are able to design our new structures for service life by making sure that we use the right materials, that the concrete cover is sufficient and that the cracks are well controlled. We dispose of adequate quality control procedures for the material concrete and its constituents to avoid unforeseen problems like e.g. alkali aggregate reaction.

A problem which is left, however, is that most of the existing structures have not been designed based on service life considerations. The actual structural safety of those structures is a point of concern. If the actual safety turns out to be sufficient, the question is for how long this is guaranteed. That this uncertainty is justified is often illustrated by unexpected and unwanted surprises, like the collapse of gallery slabs in an apartment building in Leeuwarden, the Netherlands, 2012, Fig. 9.

Fig. 9. Unannounced collapse of gallery plates in an apartment building in Leeuwarden, The Netherlands, 2013

The collapse turned out to be caused by corroded reinforcement. A remarkable observation was that the structure fulfilled the code criteria of today. There were no cracks and no signs of upcoming problems due to for instance spalling of the concrete cover. That illustrates that obviously our knowledge about deterioration processes is not yet complete.

Thousands of concrete structures, especially belonging to the infrastructure, are subject of deterioration due to a variety of detrimental effects. One of this is chloride attack. After considerable research, there seems to be consensus about the mechanism of chloride intrusion, and the criteria that should be met in order not to lose structural safety due to reinforcement corrosion. Reinforcement corrosion is expected to occur if the chloride concentration at the reinforcement exceeds a certain value. This can be calculated with the equation is the aging factor giving the decrease over time of the apparent diffusion coefficient. Depending on the type of binder and the micro environmental conditions the ageing factor is likely to be between 0,2 and 0,8.

$$C(x,t) = C_S - (C_S - C_i) \cdot [erf \frac{x}{2 \cdot \sqrt{D_{app}(t) \cdot t}}] \tag{1}$$

where:

$C(x,t)$ – is the content of chlorides in the concrete at a depth x (structure surface: $x = 0$ m) and time t [wt.-%/binder content],

C_s – is the chloride content at the concrete surface [wt.-%/binder content],

C_i – is the initial chloride content of the concrete [wt.-% binder content],

x – is the depth with a corresponding content of chlorides C(x,t) [mm],

D_{app} – is the apparent coefficient of chloride diffusion through concrete [m^2/sec.] at time t,

t – is the time (years) of exposure,

erf – error function,

$$D_{app}(t) = D_{app}(t_0)(\frac{t_0}{t})^{\alpha}$$

where:

$D_{app}(t_0)$ – is the apparent diffusion coefficient measured at a reference time t_0.

It is a remarkable observation that the crack width obviously does not play a role in this respect. On the other hand design codes give limits to the maximum crack width not to be exceeded. This looks controversial. In order to get a better understanding of the role of crack width for the service life of reinforced concrete structures, at TU Delft a series of tests was carried out, concentrating on the role of crack width on the development of corrosion. Altogether 32 concrete beams (1500×100×150 mm) were tested, see Fig. 10.

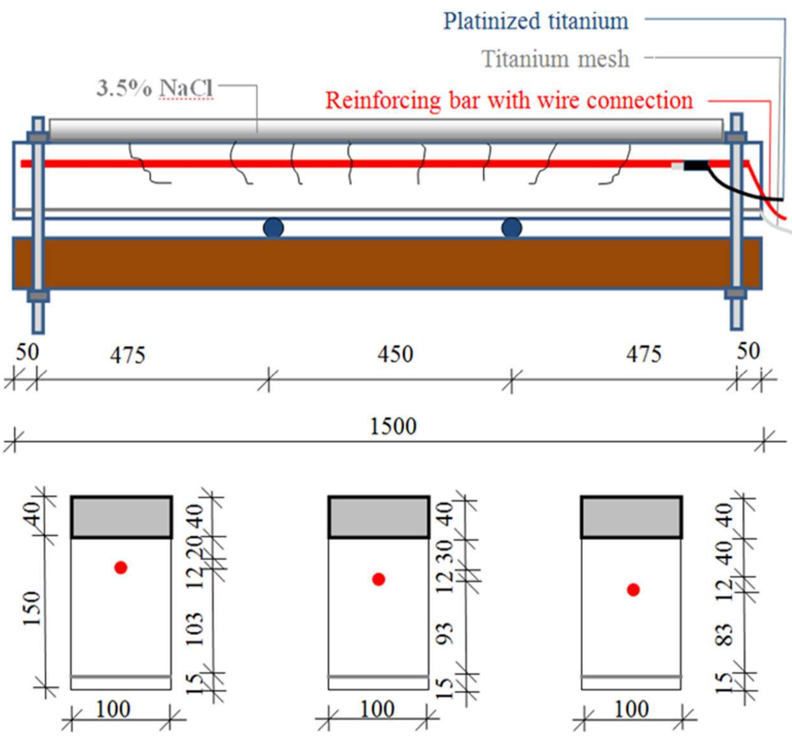

Fig. 10. Test specimens for the determination of the role of crack width on durability (Blagojevic, 2016)

Parameters were the main crack width, the number of cracks, the concrete cover and the type of loading (static versus variable). The cracked concrete beams were exposed to alternately wetting and drying cycles once a week for 2 days ponding using a 3,6 % NaCl solution and a 5 days drying phase to simulate an aggressive environment. After 2 years the beams were sawn open and the areas into which chlorides have protruded were highlighted. Fig. 11. shows that the chloride intrusion is substantially influenced by the presence of cracks. The bending cracks facilitate the ingress of chlorides over a large area, because the crack faces function as areas in direct contact with the environment. The areas where corrosion was observed are related to the crack pattern. In a number of cases the corrosion occurred at the reinforcing bar at the location where the bar intersects the crack, but in other cases at some distance from the crack, showing that chlorides, oxygen and water can infiltrate the concrete to a more remote location from the crack along the bar across the area adjacent to the crack, damaged by micro- and mesocracking. These observations show that theoretical models, used for the probabilistic determination of service life, based on the assumption that serious damage only occurs after the end of the initiation time, and that cracks do not play a role, should be reconsidered.

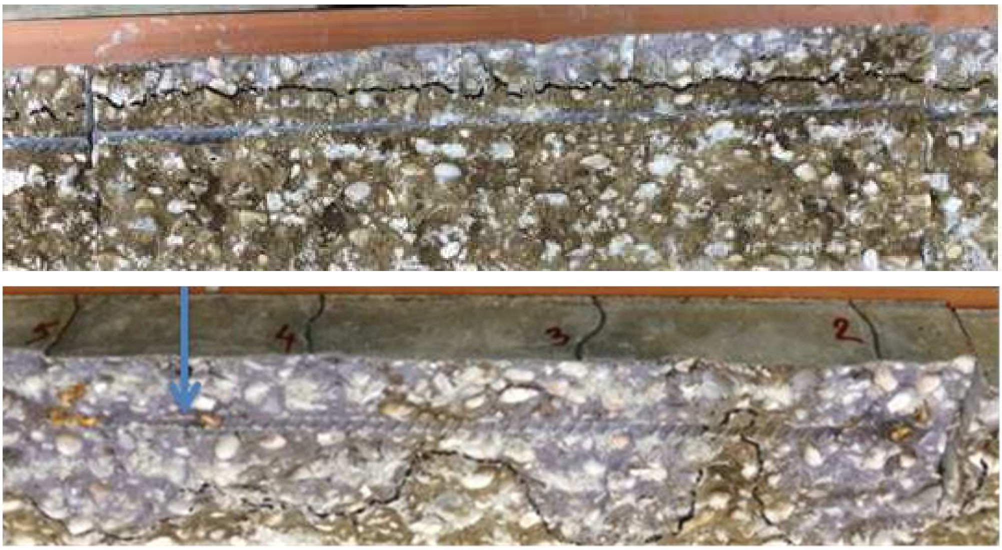

Fig. 11. a) Chloride intruded area in a beam without bending cracks (light grey area),
b) Chloride intruded area in a beam with bending cracks (light grey area) (Blagojevic, 2016)

The research showed that the maximum stress in the reinforcing steel, occurring in the service life, is a better indicator for the start of reinforcement corrosion than the maximum crack width. Nevertheless in an existing structure without analytical data, the maximum crack width is certainly an indication for the location, where the reinforcement should be investigated for corrosion.

3.5 Determining the residual bearing capacity of structures

Due to the increase of the traffic load on many infrastructural systems, like bridges, an accurate determination of the bearing capacity of those structures is necessary, in order to avoid unnecessary investments in strengthening. It should be noted that design equations found in codes are not the best option to decide whether a structure is strong enough or not. Design equations are developed for the realization of new structures. If simplification of rules leads to conservatism in the design of new structures this is not a disadvantage. The additional safety margin obtained might proof to be very useful in future, when some residual capacity may pay off. However, in order to be able to determine the residual capacity and use it to demonstrate that a structures has sufficient bearing capacity for loads larger than adopted in the original design, it is necessary to dispose of more accurate means in order to reliably determine the real bearing capacity. The *fib* Model Code 2010 introduced the concept of various „Levels of Approximation". The lowest level can be used when high accuracy is not required, such as in a preliminary design. The highest Level of Approximation should be used if decisions have to be taken about investing money in upgrading or strengthening of a structure, with considerable financial consequences.

A good example of this is the punching shear resistance of thin bridge decks, supported by prestressed beams, Fig. 12.

The deck is provided with transversal beams, by virtue of which the thin upper slab, subjected to a high wheel load, takes profit of compressive membrane (confining) action. This action substantially adds to the punching shear capacity of the deck. However, the code rules for the punching shear capacity have been derived on the basis of tests (circular or rectangular slabs with free edges) which did not allow compressive membrane action to develop. Therefore they underestimate the real bearing capacity which in reality is significantly larger than the design capacity. This demonstrates that new code rules to be developed should move away from the empirical approach that has been used often up to now, and be developed as much as possible on the real physical behavior. In the *fib* Model Code 2010 this was a leading principle. An example is the calculation model for the punching capacity. From the research program quoted before (Amir, 2014) it turned out that the fib approach can be extended, introducing the effect of compressive membrane action. By using this modified model it could be demonstrated that the bearing capacity of 70 large span bridges in The Netherlands was enough, contrary to earlier expectations. This demonstrates that forensic engineers should base their expertise on a solid understanding of the real physical behavior of concrete structures, and not rely on superseded code rules.

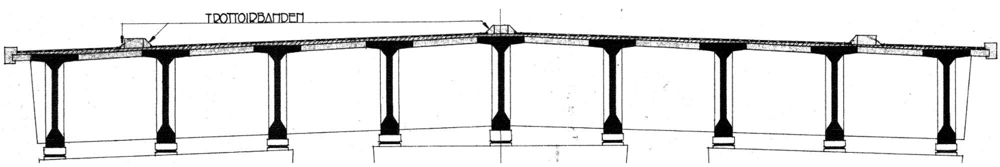

Fig. 12. Slender bridge deck with unexpected residual capacity against punching shear (Amir 2014)

If the determination of the real bearing resistance is getting high priority, it is logic that numerical FE-simulations will be applied more frequently. It should be realized, however, that the reliability of FEM calculations depends on the choices made when developing the FEM schedule, the appropriateness of the constitutive relations and the reliability of the input values. An interesting example is shown in Fig. 13.

The figure shows a cross-section over the Maas Tunnel, already mentioned before. Fig. 13 shows the results of the additional analysis carried out with the program ATENA. The picture shows that during the increase of the water load the tunnel cross-sections starts to act like a set of circular pipes, after crack formation and local crushing of the concrete. It turned out that even in the hypothetical case that all reinforcement would have become inactive due to corrosion, the ultimate water load on the tunnel at failure would be equal to 4,3 times the maximum service water load on the structure. The program ATENA gave realistic results because of the advanced formulation of the behavior of the concrete under multiaxial loading: a 3D combined plastic fracturing model was used according to Cervenka (2008). The analysis confirmed moreover that the initial „quick-check" on the basis of a concrete arch model was as well a reasonable approach. With this model a ratio 3,2 between ultimate load and maximum service load was calculated. This shows that the forensic engineer should not only be able to simplify the behaviour to such a level that a first provisional decision can be taken (closing the tunnel for traffic or not), but should additionally be able to use the tool of NLFEM analysis in an optimum way, in order to determine the reliability as accurately as possible (and to decide on upgrading or not) and to verify the results of the „quick check".

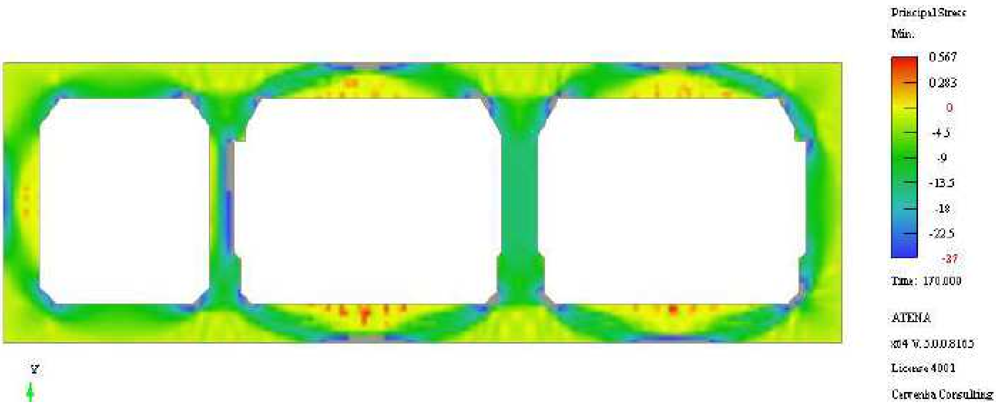

Fig. 13. Numerical FE analysis of a submerged tunnel with corroded reinforcement in the bottom slab

An important task to increase the reliability of NLFE analysis is to reduce the model- and user factor. A reduction of the model factor is achievable by developing „tailor-made" NLFE Models, which are especially suitable for certain well-defined applications, see e.g. (Belletti 2013). In The Netherlands NLFEM programs, with preselected element type- and size, constitutive equations, crack models and multi-axial stress states have been developed, which were calibrated against tests results focusing on the same aspect of the analysis to be carried out. Within this scope e.g. guidelines were produced for the determination of the shear capacity of large prestressed T- and I-beams. It was demonstrated that such an analysis, as a level IV approach, gives more accurate results than the best analytical models.

A next challenge is the determination of the bearing capacity of structures against seismic loads with non-linear finite element time history analysis. Calibration of programs to certain types of houses is costly, since it requires shaking table tests on a sufficient number of prototypes, Fig. 14.

In northern part of the Netherlands the buildings have to be verified for seismic resistance, since gas extraction from deeper layers in the soil have turned out to result in seismic agitation, which is governing for the structural safety. At this moment about 180.000 buildings are concerned. Some of them can be classified as prototypes with similar properties, but they are not identical. Analyzing any house with a nonlinear time history analysis would not be a viable option. Therefore expert systems are under development in order to supply a „simple approach" based on the insight gained from testing, inspection and nonlinear time history analyses of representative buildings. If the structural reliability is insufficient upgrading is required. An appropriate choice has to be made between options like base isolation, window frames, façade replacement etc. This project asks for a solid theoretical base, experience and creativity of the engineers in charge.

Fig. 14. Shaking table test on a typical Dutch house in Pavia, Italy, 2015

4. Conclusions

1). Forensic engineers able to judge upon deficiencies in structures, their cause and the treatment for upgrading are more and more needed.

2). In order to carry out their task of finding the cause of deficiencies, damage or even partial or full collapse, the forensic engineer should dispose of profound knowledge about the real behavior of structures.

3). It is important to realize, that the cause of failures should not only be explained from a technical point of view, but can also be found in bad communication and lack of quality control

4). The aging of concrete structures asks for a lot of additional expertise in order to be able to assess the structural reliability, at the moment of investigation and during the remaining service life.

5). Large scale evaluations of structural safety are necessary due to changes in the magnitude of loads, like related to increased vehicle loads, or unforeseen influences like seismic loads.

6). Building codes should offer different levels of approximation to satisfy not only the need to design new structures but as well to assess existing structures.

7). Education of students should be more directed to understanding structural behavior, stimulating the ability for adequate assessment of buildings.

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