Structural safety: lessons from the partial collapse of a multi-story car park

JOOST WALRAVEN
Delft University of Technology, Delft, The Netherlands

Abstract: At 27th of May 2017 a large part of a new multistory car park at the Airport of Eindhoven in The Netherlands collapsed. This collapse occurred a few weeks before the official opening. It turned out that the floors of the building were made according to the so-called Bubble Deck system. This consists of precast concrete planks on top of which reinforcement is provided, in combination with polypropylene spheres. This floor system leads to a light and nevertheless stiff floor. In the past the slabs were design to predominantly carry the loads in one direction. In the last decade the system was changed in such a way that it could carry the load in two principal directions. However, the joints were detailed incorrectly, which resulted in the collapse. It was remarkable that in the 15 years that this detail was used, nobody was aware of the risk of its application. That implies that many other existing buildings might be unsafe as well. In this paper the consequences and solutions are treated.

Key words: Car park floor collapse

1. Introduction

Although more and more advanced calculation methods have been developed, structural safety remains a point of serious concern. Even nowadays failures occur in the construction stage, but also deficiencies show up after many years of obviously satisfactory use. It is not only important why failures occur, but most of all how they can be detected in an early stage and be avoided in future.

A recent development is that structures are designed for a defined service life. In order to guarantee that the intended service life is really reached without significant maintenance regular inspection and monitoring is required, the results of which are recorded. This requires as well sufficient knowledge of the “as-built” situation, since initial deficiencies may develop to substantial risks in future.

Another point of concern is still the large number of partners in the design and construction process. Fig. 1 shows a 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 an undisputable result is only 110 - 12 - 21 = 67%.

Furthermore the way in which the whole building process is organized differs from case to case. The relation between initiator, architect, designer, constructor and supplier of materials shows large variability. Moreover often specialists in different areas are added, like installation advisors, fire experts and financial advisors. Engineers may be involved as designers, coordinators or are charged with detailed calculations. Those tasks can be carried out for different parties. The designing engineer will often work for the initiator of the project, but sometimes also for the contractor. The coordinating engineer mostly works for the contractor, but sometimes as well for the initiator. The engineers who concentrate on detailed calculations may work for the suppliers of precast elements. In that case the supplier delivers a complete product on the basis of performance requirements by the contractor. But the designing engineer can also work for the main contractor. In that case the supplier delivers the precast product according to the specifications of the contractor. An independent evaluating expert can be hired by the original initiator, the actual owner, the builder or the supplier. So, engineers are involved in different situations in the design and construction process. An obvious question is therefore: "Who is responsible for what?"

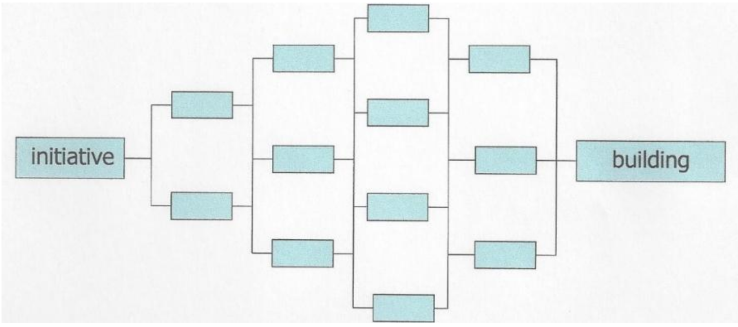

Fig. 1. Chain of participants in design and construction.

2. Failure of a part of a new car park for the Airport Eindhoven in the Netherlands

2.1 Description of the structure and the collapse

At the end of May 2016 a start was made with the construction of the so-called Multi Purpose Building (MBP) of Eindhoven Airport, in which the parking garage P1 is an important component. The MPB has four floors, a width of about 120 m and a depth of about 65 m. The floors were designed with the so-called Bubble Deck system. These are floor elements with internal plastic spheres allowing a significant reduction of the weight of the floors. It was planned to open the building for the public in June 2017. However, the collapse of a part of the building occurred just before that date (fig. 2). When the collapse occurred (in the evening) there were no construction activities and no people were at the site, so there were no victims, in spite of the large scale of the collapse.

Fig. 2. Collapse of a part of the car park at Eindhoven Airport just before completion.

The floors were built as a so-called Bubble Deck floors. Such floors consists of a precast lower plank, on top of which spheres (“bubbles”) are placed made of high density polypropylene. Those spheres are kept in position by reinforcing steel. At the site a concrete top layer is cast so that a floor is obtained with the spheres inside. Fig. 3a shows an element spanning in one direction, with the bubbles, just before casting the in situ concrete. Fig. 3b shows a combination of the use of deck elements spanning in one and in two directions.

Fig. 3a Bubble deck element spanning in one direction.

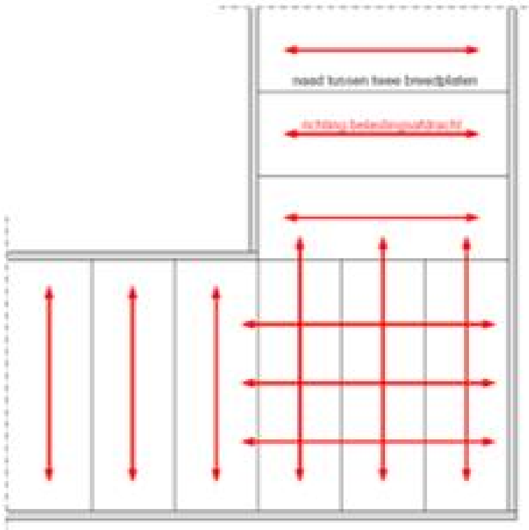

Fig. 3b. Bubble deck elements spanning in one or two directions (corner area).

The floor in the parking garage was designed in such a way that most elements were spanning in the direction perpendicular to the longitudinal joints. This required both bending moments and shear forces to be transmitted across the longitudinal joints. Fig. 4 shows details of the reinforcement in the joints carrying bending moments in perpendicular direction. In this case coupling bars had been placed in the in-situ concrete perpendicular to the bottom joint of the composite precast planks. However, in the case that the joint is cracked (vertical dotted line A in fig. 4), the tensile forces in the coupling reinforcement, due to the bending moments to be transmitted across the joint, have to be introduced into the reinforcement in the precast plank. Those forces have to be transmitted across the interface between the precast and in-situ concrete. So, an important requirement is that the interface between precast and in-situ concrete is able to transmit those tensile forces by in-plane shear. So, the bond properties of the interface between the precast and the in-situ concrete are an essential element in the connection. Indeed insufficient interface-shear capacity turned out to be the major cause of the collapse. This indeed occurred in the floors of the parking garage: the insufficient bond of the interface became governing and failure patterns developed according to the dotted blue line C (fig. 4).

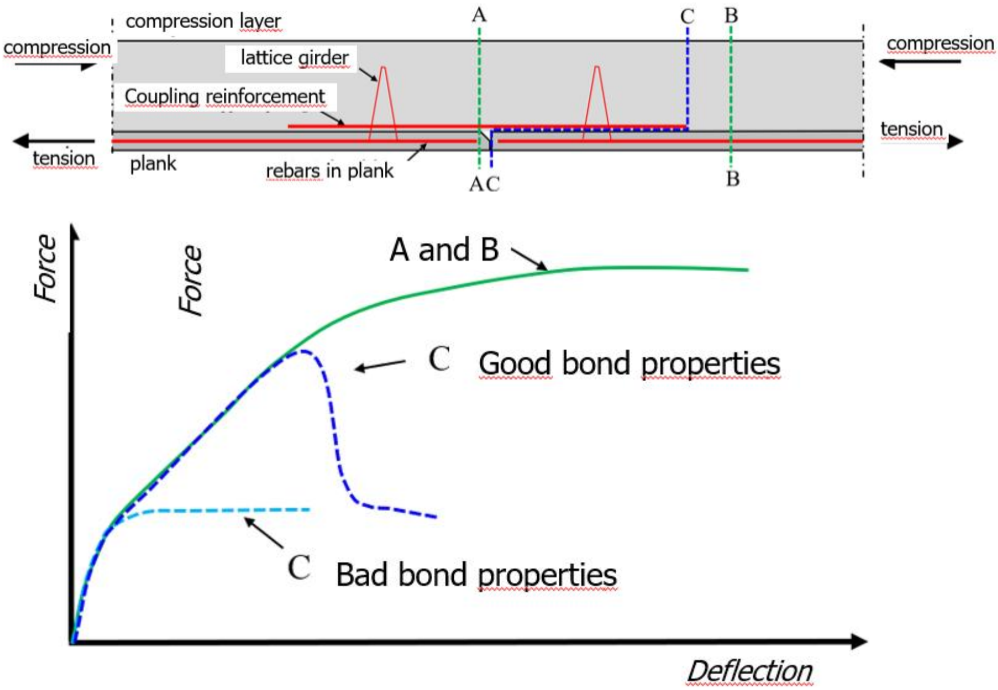

Fig. 4. Load-deflection relations (schematic) for the critical detail in case of failure by yielding of the coupling reinforcement (A), the reinforcement in the precast composited plank (B), and failure along the interface between precast and in-situ concrete with good or bad bond properties (dark blue and light blue dotted relations).

The failure mechanism on a structural level is shown in fig. 5. The first floor which failed was the one on level 4, which led to progressive collapse of the floors on the levels 3 and below.

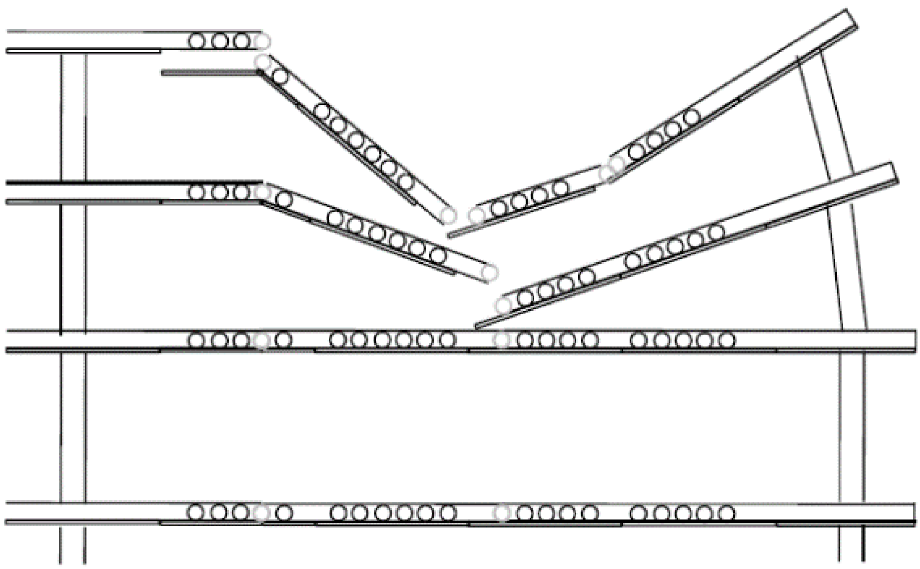

Fig. 5. Collapse mechanism due to progressive collapse of the floors, starting at level 4.

2.2. Detailing of critical joint in comparison with governing code provisions (EN 1992-1-1)

Considering the critical joint as a lapped splice, in section 8.7.2 (2) of EN 1992-1-1 it is recommended that laps between bars should normally be staggered and especially not concentrated in areas with large bending moments (fig. 6). In the case considered the critical joint, leading to collapse, was located near to the maximum bending moment at mid-span, whereas the coupling bars were not staggered. This is already a situation which can be described as “risky”.

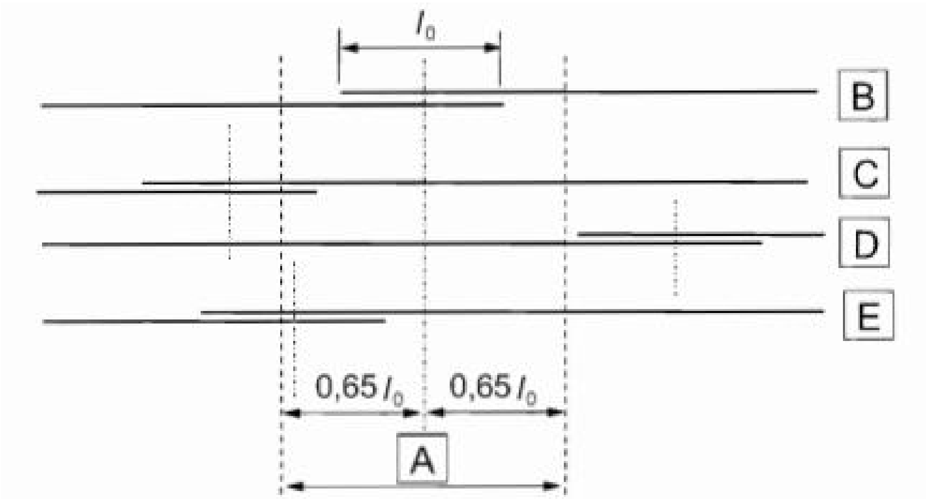

Fig. 6. Avoiding too many lapped slices in one cross-section (according to EN 1992-1-1, Section 8.7.3).

Moreover in section 8.7.2 (1)P it is required that the detailing of laps between bars shall be such that “the transmission of forces from one bar to the next is assured”. In the case considered the lapped bars are positioned in concrete’s cast at different times. The lower bars of any lap are located in the precast plank produced in the factory whereas the upper bars are located in the concrete cast in situ on the top of the plank. So in-between the bars an interface is found, which should be able to transmit the maximum force in any bar from the precast to the cast in-situ concrete (fig. 7).

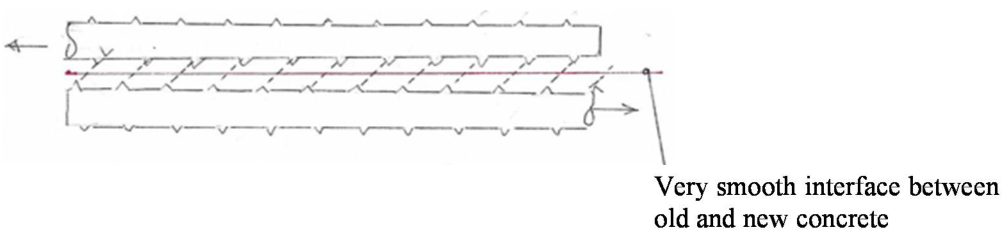

Fig. 7. Lapped splice with low capacity due to very smooh interface between old and new concrete.

The interface shear capacity of a plane between old and new concrete should be checked with section 6.2.5 “Shear resistance in the interface of concretes cast at different ages”. This check was, however, not carried out. If this verification would have been carried out, the interface should have been qualified as “smooth”, because the plank surface was not roughened after casting in the factory. Tests at TU Eindhoven showed however that in reality the interface should be better described as “very smooth”, because of the combination of an unroughened surface of the precast concrete and the use of self-compacting concrete for the in-situ layer. The low tensile capacity of the interface was confirmed by tensile tests on vertically drilled cores.

Another aspect of the “lapped splice” which should have been considered is that the coupling bars (fig. 4) have been placed directly on top of the hardened plank concrete and as such are not homogeneously surrounded by in situ concrete. Also this leads to a reduction of the bond capacity of the lapped splices which was not regarded (comparable to bond reduction in “bundled bars”).

2.3. Further reflection on the reasons of collapse

As suggested before, the shear strength of the interface between precast and cast-in-situ concrete appeared to be the “weakest link” in the bearing system. In order to verify this hypothesis tests have been carried out at the laboratory of TU Eindhoven. This showed that indeed in all specimens failure occurred by the formation of a crack in the interface between the precast plank and the cast-in situ concrete (according to crack type C (light blue dotted line) in fig. 4.

Another observation was that the triangulated lattice girders, which act as couplers between the precast and cast in situ concrete (also visible in fig. 4) had an insufficient clamping effect: they were weakly anchored and were positioned too far from the joint. Therefore cracks starting in the joint could progress and open easily without being counteracted by the triangular lattice girders.

A remarkable aspect was further that the collapse of the floor occurred four months after it had been cast. At the time of collapse there was hardly any variable load, because the parking garage was not yet open to cars. Most probably the high temperature at the day of collapse (one of the hottest days of the year with T_{max} = 33°C) has played a role. This might have caused a temperature gradient over the height of the slab, which has led to upward bending which was partially restrained by the columns. This effect was, however, considered as the potential “trigger” initiating the failure process but not as a major reason for the collapse.

3. Structural safety of existing buildings with similar types of floors

After analysing the collapse of the parking house at the Eindhoven Airport a logic question was whether more buildings have been built with similar types of floors, with bending moments acting on the longitudinal joints with the same type of detailing. Soon it turned out that already since 15 years buildings have been constructed with similar types of joint details. This was never recognized as unsuitable, although those floors did not meet the official rules for detailing lapped splices across the joints, as argued before. Floors designed in this way were simply seen as a product of an evolution of the original structural bubble deck system as applied in the past, where the slabs were designed for carrying the loads predominantly in one principal direction, with limited redistribution between the slabs in transverse direction. Designing bubble deck slabs for carrying loads in two perpendicular directions, with a substantial transmission of bending moments and shear forces in the direction perpendicular to the longitudinal joints, was meanwhile not considered as an exception but rather as an accepted way of designing. As an estimation about 10.000 actual buildings contain bubble-deck floors designed in the original way, and about 100–500 have been designed for transmitting substantial bending moments across the longitudinal joints.

On the other hand, obviously a large number of buildings, containing the wrong detail, have served for many years without problems. It can therefore be concluded that the probability of failure for a floor of this type is obviously smaller than would follow from a direct comparison with the official design rules. An aspect to be considered here is that design rules in codes are mostly simplifications of more advanced calculation models. By using those simplified rules the probability of design errors by designing engineers is reduced. An additional advantage is that some residual structural capacity is obtained for a relatively low cost, which could appear to be very valuable in certain situations where additional capacity is welcome. In the *fib* Model Code 2010 the design and assessment rules have been subdivided into various Levels of Approximation. The lower levels of approximation are suitable for design, whereas the higher levels of approximation are especially useful for the assessment of existing structures, where the accuracy of the determination of the bearing capacity is very important because of the high costs involved in strengthening or even demolishment. Principally this also applies to structures with inadequate detailing.

In order to cope with the unwanted situation of a substantial number of buildings containing the inadequate joint detail a document was written entitled "Investigation of structural safety of composite plank floors delivered after 1999 [2, 3]". This document was developed under the auspices of the Dutch Ministry of Internal Affairs, and was developed in cooperation with a large group of experts. The document describes a stepwise procedure to evaluate the structural safety of this type of floors. The procedure, described in this document, was as follows:

Step 1: collect drawings from the archives of the owner of the building, commissioner, designer, floor supplier and/or responsible building authority;

Step 2: Check whether there are joints subjected to positive bending;

Step 3: Search for additional information, regarding the producer of the plank floors, design drawings and calculations and information on eventual deficiencies;

Step 4: Check whether the composite plank floors at the site have been produced with self-compacting concrete without roughening the upper face of the concrete plank;

Step 5: For the floors with non-roughened plank surfaces and self-compacting concrete cast at the site the shear and normal stresses in the joints should be determined: for this action spreadsheets have been developed. Here the actual load on the floor should be checked (in some offices corridors and room systems had been replaced by open working areas ("office gardens": this could allow a reduction of the variable load);

Step 6: A risk analysis should be carried out: this should imply the probability of the occurrence of a critical load combination (including the ratio between variable and permanent load), the load history (which maximum loads have been carried already before?), concrete mixture and roughness, the design of the floor (do simplifications used imply unused redundancy?). Furthermore the floor should be inspected with regardto delamination between precast and in-situ concrete in the joint area and its eventual influence of the residual capacity. Finally the behaviour of the structure should be simulated for failure of the critical joint: is there any second bearing mode available?

On the basis of this analysis further recommendations should be formulated to allow next steps. This could e.g. be a reduction of the load by an adapted function of the floor, strengthening the floor or applying proof loading to get more information.

4. Proofloading

According to EN 1990 it is allowed to demonstrate the structural safety of a building whether by calculation or by tests, or a combination of both. Proof loading can be considered when there is a reasonable chance, demonstrated by calculation, that the structure satisfies the demands of structural safety. In this calculation also the particular weaknesses of the structure should be accounted for. Moreover it should be assured that the consequences of an eventual collapse of the area subjected to proof loading will be limited, e.g by the provision of temporary supports below the areas subjected to proof loading, which leave sufficient space for the expected deflection of the floor. The loads applied on the supporting structure in case of an eventual failure should be able to be carried by the remaining structure. With regard to the type of loading careful considerations are required. A question is for instance whether the result obtained by proof loading of only one part of the floor between column axes is valid as well to the neighbouring floor areas, since the boundary conditions may be different. Furthermore the probability of the occurrence of weaknesses is larger in case of a larger floor area than in a small area.

Fig. 8a. Proof loading of a floor of an Leidschendam, Netherlands.

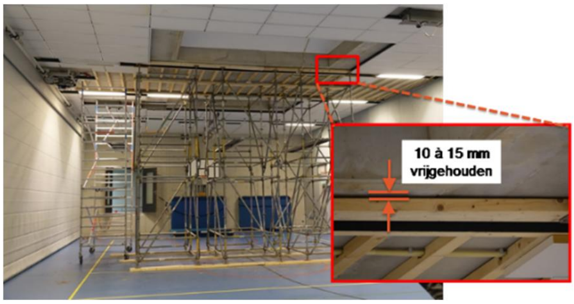

Fig. 8b. Safety frame below floor with 15 mm free deflection.

Fig. 8a shows proof loading of an office in Leidschendam, the Netherlands, where a floor with the critical detail was investigated. All adjacent floors were loaded simultaneously. The proof loading was carried out with water-containers. In order to make sure that failure of the floor would not damage the remainder of the structure a safety frame was built with 15mm free space to allow free deflection of the floor during proof loading (fig. 8b). The floor was loaded in steps. The following measurements were done:
– Floor deflection,
– Joint opening,
– Debonding between plank and in-situ concrete,
– visual inspection where cracks $w > 0.5$mm where registered and monitored.

An important question is as well to which loading level the floor may be loaded in order to demonstrate sufficient structural safety for the case considered. In The Netherlands this can be determined with the national code NEN 8700, which is derived from EN 1990 with its national annex. NEN 8700 offers the possibility to derive the maximum load on the characteristic value corresponding with the actual use.

In [3,5] an example was given for a practical case:
– Consequence class 2,
– Reference period 15 years (NEN 8700, Annex B.3.2),
– Required reliability $b = 2,5$ (NEN 8700, Annex B.3.2),
– Permanent load 8.5 kN/m^2,
– Variable load 2.0 kN/m^2,

The partial factors for the permanent load and for the strength are determined with

$$g = 1 + abV(1+1/n)^{0.5} \quad (1)$$

Where α is a sensitivity factor which may be assumed to be:

α = 0.3 for the permanent load,

α = 0.8 for strength,

β = the reliability index which can be assumed to be 2.5 for the case considered (level of disapproval for consequence class CC2),

V = the coefficient of variation which may be assumed to be:

V = 0.05 for permanent load ,

V = 0,15 for the strength ,

n = the number of floor areas tested,

Substitution of those values in Eq. (1) gives:

– Partial factor for the permanent load: γ_G = 1.04,

– Partial factor for strength γ_M = 1.30.

For a variable load of 2.0 kN/m^2 a load factor γ_Q = 1.15 applies according to NEN 8700.

With those partial factors the following design value of the load is obtained:

$$r_{sd} = \gamma_G G + \gamma_Q Q = 1.04 \times 8.5 + 1.15 \times 2.0 = 11.2 \text{ kN/m}^2$$

For the effect of sustained loading according to NEN 8700 a factor 1.05 should be used. In combination with the uncertainty in strength (γ_M = 1.30), and subtracting the already acting permanent load of 8.5 kN/m^2 the maximum proof load becomes:

$$Q = 1.05 \times 11.2 - 8.5 = 6.8 \text{ kN/m}^2$$

By the proof-loading up to this level it was shown that the office floors where stronger than derived on the basis of the governing code rules, so no strengthening measures were necessary.

5. Strengthening options

For floors in existing buildings where it could not convincingly be demonstrated that the structural safety was sufficient, strengthening options were developed. A first option which was considered was the application of externally bonded FRP reinforcement. The external FRP reinforcement should then take over a part of the tensile force which is now fully allocated to the coupling reinforcement above the plank. A disadvantage of this solution is that the FRP strips have a relatively low axial stiffness, so that they are only activated after the shear capacity of the interface between the two concretes has been exceeded. Furthermore the fire safety of the solution with externally glued FRP strips remains a point of copncern. Three other alternatives have been developed which are described in [1].

5.1 Application of a steel strip as external reinforcement

A steel strip of 10$\times$ 100 mm^2 with a length of 2900 mm is glued against the concrete, after roughening the bottom face of the plank concrete. The strip is further connected by bolts, fig. 9a, b.

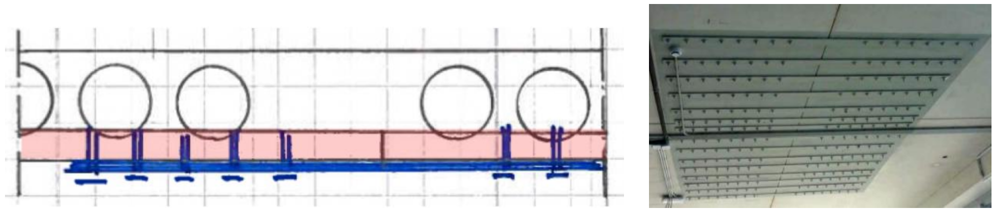

Fig. 9a, b. Strengthening solution by externally glued strips (Font Freide [1]).

Those bolts have a penetration depth of 70 mm, so they cannot damage the coupling bars just above the interface between precast and in-situ concrete. The main function of those bolts is to guarantee the bearing capacity of the floor during a fire (the glue is most probably not fire resistant). The strip itself and the bolt heads are not fire resistant; therefore they should be protected by a fire resistant coating. Principally the steel strip can replace the coupling reinforcement, so that even after delamination of the interface the safety of the floor is guaranteed. When the load on the floor is increased, the tensile resisting force across the joint at the bottom of the cross section is distributed over the coupling bars in the in-situ concrete and the steel strip attached to the plank at the bottom of the cross-section. The smaller the force introduced into the coupling bars, the smaller is the shear stress at the interface. An advantage of this solution is that it is not necessary to place temporary supports below the floor. The space between the bore hole between strips and anchors should be filled with mortar to make sure that all anchors are loaded simultaneously.

5.2 Reinforcing steel bars in sawn grooves

For this option sawn grooves are provided in the bottom face of the slab. The grooves have lengths of 1.6 m and distances of 175 mm, fig. 10).

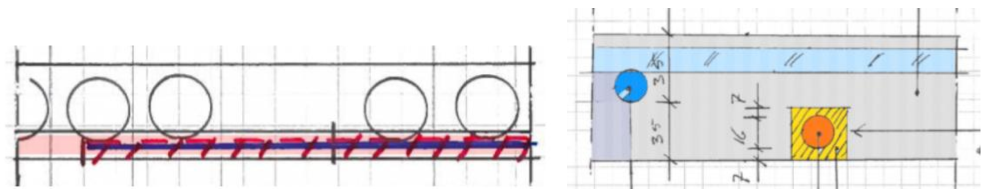

Fig. 10. Reinforcing bars ins sawn grooves (Font Freide [1])

In the grooves reinforcing bars f 16 mm are placed and fixed with a plastic glue. An advantage of this solution is that the position of this additional reinforcement is very near to the coupling bars beyond it in the in-situ concrete. Moreover it is a well-known technique, the fire safety is guaranteed and the lower surface of the repaired slab is flat. A disadvantage is the necessity of the sawing operation, which produces dust and noise nuisance. Furthermore the vibrations in the floor could lead to delamination of the interface between precast and in-situ concrete, which might increase the risk of a collapse during carrying out the strengthening process. Therefore during strengthening supports under the floor have to be applied.

5.3 Anchors in the spheres

For this solution anchors are provided, after the row of spheres in the anchorage zone has been injected for about 50% with mortar. To allow the placement of the anchors and the injection under the spheres an opening with a diameter of about 70 mm is drilled. The anchor is then placed in position and temporarily fixed allowing injection of the sphere. After hardening of the mortar the anchor is prestressed, which increases the shear capacity of the interface between precast and in-situ concrete, fig. 11.

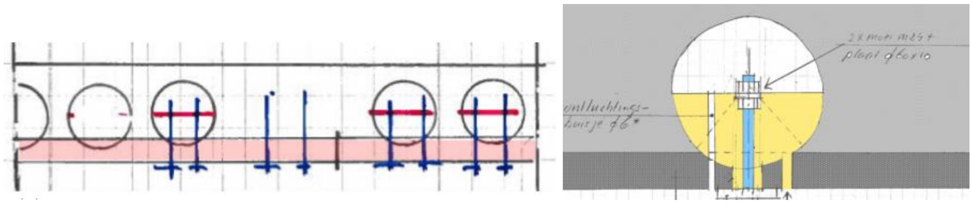

Fig. 11. Solution with anchors in spheres injected with grout (Font Freide (1)].

In areas where no spheres are available the anchors can be provided as lime anchors. Also this option has the advantage that drilling creates nuisance but also implies the risk that the weak interface between precast and in situ concrete is delaminated. Therefore during the strengthening activities the floor should temporarily be supported.

For the strengthening of the Polak-Building, a Faculty Building at the Erasmus University of Rotterdam, the option described under 5.1 was chosen. The most important arguments for this choice were the relatively low costs involved with this experienced technique, which can quickly be executed without the need for supports. Good experience was obtained with this strengthening method.

6. Lessons learned

It occurred that an existing load-bearing system, which has never given any trouble, was further developed to a system with intrinsically wrong details, not meeting the applicable code rules, without that this was noticed by participants in the design and construction process. This resulted in hundreds of buildings with potentially insufficient structural safety.

In most countries it is allowed to deviate from the governing code rules if this has been convincingly demonstrated by tests. This is for instance described in EN 1990, Chapter 5 “Structural analysis and design assisted by testing”. However, no tests had been carried out to verify the behaviour of the modified detail considered. If a design engineer is confronted with a number of realized applications, shown as an example, he/she might assume that this detail has already been substantially verified by testing, or/and has already been approved by various building authorities at different places.

Building processes today are quite complex and the responsibility for mistakes is often unclear. Engineers can have the function of “designer” for the commissioner, taking initiative for the building to be realized, but may also work as a coordinator for the contractor, or for the producer of the precast concrete elements. With so many participants in the design and construction process there is a lack of central supervision, overviewing and controlling the whole process. In the current situation questions and doubts about solutions have not been discussed, because the responsibility is assumed to be somewhere else. Therefore in new building projects there should be one party which has full supervision and takes responsibility for the end result.

The building authorities, which have to approve the designs of new buildings, are overloaded and their capacity is too small. A problem for them as well is that the documents, delivered to them for verification and approval have not a clear structure. They are often incomplete and lack sufficient background information and explanation.

Calculations are nowadays very often carried out with calculation programs. However, those programs assume correct behaviour of areas in the structure which are not regarded in detail. The role of correct detailing is generally underestimated. “The devil is in the detail” applies more than ever to structural safety of buildings.

The assessment of existing buildings is a task with growing importance. Therefore experiences and potential solutions should be shared between the experts involved and made accessible to other interested engineers.

References

1. Font Freide, J., Brouns, F., Vos-Pols, M., “Improvement structural safety of bubble deck floors by externally applied steel strips”, Cement, Nr 1, 2019, pp. 38–47 (in Dutch).
2. Hageman, J.G. Consulting Engineers, “Information document structural safety of composite plank floors in existing buildings delivered after 1999”, Rijswijk, 5 Oct. 2017 (in Dutch).
3. Hageman, J.G., Consulting Engineers, “Explanations of information document structural safety of composite plan floors”, Rijswijk, 21 December 2017 (in Dutch).
4. Vamberski, J., Walraven, J.C., Salet, T., Vrijling, H., Hordijk, D., Vrouwenvelder, T., “Bubble deck floor problems spotlighted”, Cement 4/2020, pp. 42–51.
5. Vossen, S. van der, “Proof loading of bubble deck floors”, Cement 1/2019, pp. 70–75.

Bezpieczeństwo konstrukcji: wnioski z częściowego zawalenia się parkingu wielopoziomowego

Słowa kluczowe: zawalenie się garażu