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The impact of design and executive errors affecting the damage to the floor of the concert hall

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ABSTRACT

Purpose: The aim of the article is to demonstrate the influence of errors and performance errors that affect the damage to the concert hall floor.

Design/methodology/approach: When the project was launched, cracks appeared which made the floor unsuitable for further use. In order to determine the cause of the damage, the team conducted a detailed structural health analysis.

Findings: The documents used for the analysis included the submitted project records, as well as the actual state of the structure.

Research limitations/implications: First, the design team checked the design records for the rooms used for statistical calculations. The arrangement of the floor layers, which was not in accordance with the original design documents, overloaded the load-bearing structure of the Ackerman ceiling.

Practical implications: Engineers identified other flawed design guidelines that added to the damage. As a solution, the client followed one of the solutions proposed by the original renovation team.

Originality/value: Before the commencement of construction works, an analysis of design solutions and executive assumptions is required, as they may affect the previously made elements of the building structure.

Keywords: Historic buildings, Renovations, Construction works

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PROPERTIES

1. Introduction

Scrapes and cracks are a fairly common issue that occurs during renovation or redevelopment of historic building objects. This sort of damage in historic constructions objects usually appears due to errors in establishing properties of built-in building materials [1]. This article sets out a methodology of examining root causes of the risk of construction disaster that developed in the renovated concert hall.

Renovations of historical building objects require a set of design solutions that ensure that repair work carried out bring expected results. The most common projects given to heritage preservationists require securing the original building blocks and the overall look of modernized object [2,3]. Appearance of scuffs and cracks on brick-built elements of load-bearing structures is a common problem. This sort of constructional damage on bearing walls may be caused by building object settling unevenly, local exceeding of permissible strains (e.g. increase in payload) , loss of longitudinal stability and rigidity of walls, also due to thermal strain caused by temperature change. Stone partition walls are quite often removed or new floor layers are added during the execution of construction work. That repeatedly causes significant constructional damage to a supporting structure and in extreme cases, to construction. Building appraisers that compile assessments and evaluations of technical damage to a brick construction in some cases as a solution recommend only surface reinforcement of damaged walls [4]. Performing a significant change in the distribution of loads in the reconstructed buildings of classic construction, without perimeter rings is a complex subject and in certain cases the above solution might insufficient in order to ensure safety of a supporting construction or securer exploitation of a building construction. This study presents a diagnostics and analysis carried out on a construction of a -story ceiling that weighed down got with additional floor layers. The case that is depicted in this paper describes a methodology of evaluating reasons behind the identified faults and presents an effective way of removing the risk of construction disaster. There are different methods that are used to enhance a construction of a building object and it is usually the use of lightweight ultra-durable materials which was captured in many publications [5-7]. These are correct solutions to a problem but in this case it was decided to relieve the building structure relief. Reasons behind it were based on a cost of this sort of solution as well as due to design-reasons. Additional weight of incorrectly made layers floor layers added to the construction could have caused local overstrains of other structural components. It is worth noting that cracked stone floor tiles had to be removed regardless of a taken approach.

2. Account of a state of the concert hall floor

Stone-slabs floor – the last floor layer in the concert hall was replaced as a part of a reconstruction that was done in 2008. As a result of construction work the layout of existing floor layers at the time (according to project documents from 2007 [8]) that consisted of:

- ceiling Ackerman – 27 cm,
- 2 x fibreboard,
- screed – 4 cm,
- woodstave – 2.2 cm,

was swapped with redesigned floor layers with stone-slabs floor/designed floor layers with a stone floor):

- ceiling Ackerman – 27 cm (as above),
- stabilized sand– 7 cm,
- stone-slabs floor – 2 cm.

In order to verify the actual state of the layout of the floor layers engineers took five different outcrops from the top level of the floor cavity blocks (Fig. 1).

Basing on the outcrops it was established that premises of the statistical formulas attached to project documentation of a building redevelopment were incorrect. Thickness of ceiling concrete Ackerman is ~ 3 cm (see Fig. 1c) while in the project estimation [8] and technical expertise preceding the renovation [9] it was assumed to be 9 cm. It significantly increased arm of span bending and thereby overstated lifting capacity of the floor (load-bearing capacity of this floor). Internal forces arm for span bending that was used in project calculations is 23.0 cm and the actual one calculated for the analysed outcrops is – 18.5cm which means reduction of this arm the by over 24%.

Additionally, floor layers done by construction team aren't consistent with project documentation. Project specification [8] calls for 7 cm stabilized sand with a characteristic load 1.33 kN/m^2 when in reality there's an average of 6.5 cm of concrete screed and average of 1.0 cm of glue mass which is the characteristic load 1.74 kN/m^2.

The real intersection of floor layers established on the grounds of local outcrops presents as follows:

- cement and lime plaster – 1.5 cm,
- Ackerman ceiling – 21 cm,
- screed (gentry) – 4 cm
- fibreboards and vapor barrier – 1.0 cm,
- concrete screed – 6.5 cm,
- adhesive mortar – 1.0 cm,
- stone floor – 2.0 cm.

Fig. 1. Examples of outcrops from floor layers: a, b) no 2, c, d) no 4

Fig. 2. View at the construction of Ackerman ceiling from the bottom

Engineers assessed the technical condition of inter-story ceiling during the site inspection, looking at it from the bottom of the construction – Figure 2.

Looking at the bottom of the construction engineers didn't notice any signs of damage on the supporting structure – suspended ceiling and electrical installation are attached to the it. Ferromagnetic device was used to measure the thickness of jacket on reinforcing bars (cover of reinforcing bars) which is 2.0 cm and the diameter of reinforcing bars of ceiling ribs was 10 mm.

Based on the conducted research of floor layer outcrops and visual evaluation of the damaged marble floor surface in the concert hall of 'Dziekanka' building in Warsaw it was ascertained that:

- cracks on the stone-slabs of the floor are consisted with cracks of the layers of adhesive and base surface – execution fault (finishing works),
- layout of the floor layers is inconsistent with project documentation which overloaded the supporting

structure of the Ackerman ceiling, including payload (of the crowd of people) that was taken into account – execution and design faults
- amount and thickness of dilatation is insufficient – execution fault
- dilatations and working breaks of the base surface and inconsistent with floor surface dilatations – execution fault.

3. Root cause analysis of observed irregularities

Analysis of the technical documentation of the renovation it brought to light a series of faults and errors. In his calculations from 1997 a designer estimated load-bearing capacity of the beam of a ribbed floor as M_{Rd} = 2.84 kNm basing on an incorrect premise of computational spread of this bar/joist being set to 3.0 m.

The same author in the project from 2008 significantly alters computational premises without performing any research or outcrops. He assumes static diagram of a three-span continuous beam with a span 3.0 m, calculation loads: weight of layers (after laying stone slabs) 7.20 kN/m^2; utility 3.90 kN/m^2, on the rib 3.44 kN/m. He determines carrying capacity still basing on thickness of the concrete overlay layer (concrete topping) hf = 9.0 cm, top and bottom surface reinforcement of 0.8 cm^2, concrete cover (lagging from below) 4.0cm which results in carrying balk capacity of M_{Rd} = 4.04 kNm. It is important to stress that there was no evidence of top or bottom surface reinforcement found in the outcrops and when demolition work was carried out.

There was an 'old' screed found in the outcrops from under the marble floor Ackerman celling that was estimated to be 6.0 cm thick (combined thickness of concrete overlay and old screed came to 9.0 cm as a result) that was covered with a fibreboard 1.0 cm thick. When comparing it with construction project from 2007 it was stated as two even layers of 2.5 cm. The new floor surface was made of approx. 6.5 cm concrete floor and 2.0 cm slabs of stone on 1.0 cm layer of adhesive. Basing on the floor outcrops engineers estimated the computational value of self-weight load equal to 8.67 kN/m^2 compared to significantly lower (difference of 1.47 kN/m^2) value equal to 7.20 kN/m^2 that was incorrectly estimated by the designer.

The process of reparation work carried on the floor was being monitored to make sure it was executed according to the project recommendations. Assumptions that were made during the expert evaluation of discrepancies of the actual layout of the floor compared to project documentation were confirmed as seen on the Figure 3 demolition area below.

Fig. 3. View on floor layers during scheduled demolition work

It needs to be noted that adding weight of ceiling layers happened as a result of replacement of floor layers that resulted from the solution included in the project from 2007 [8]. Deployment of layers of increased weight significantly exhausted. It is worth noting that the author of expert opinion [9] and the project [9] approved of the replacement of floor layers – entry in the construction log and building contractor executed it differently to the instructions that was given by the project. Execution faults that essentially resulted from inappropriate completion of layers under stone-slabs floor were elaborated in the paper listed below [10]. That paper recommends replacing the damaged stone slabs and reparation of the existing ones as well as additional dilatation.

Engineers performed new calculations for load capacity of the Ackerman ceiling and agreed on geometrical representation of the floor intersection as in Figure 4.

Based on the adopted parameters for strength of concrete and steel materials engineers set effective height of the concrete compression zone:

$$x_{eff} = A_{s1} * f_{yd} / b_d * f_{cd} \tag{1}$$

where:
f_{yd} – design yield strength of steel – 190 MPa,
f_{cd} – concrete compressive strength – 8.0 MPa,
x_{eff} = 0.61 cm.

The height of the compression zone is smaller than the plate thickness, i.e. the cross-section was apparently T-shaped.

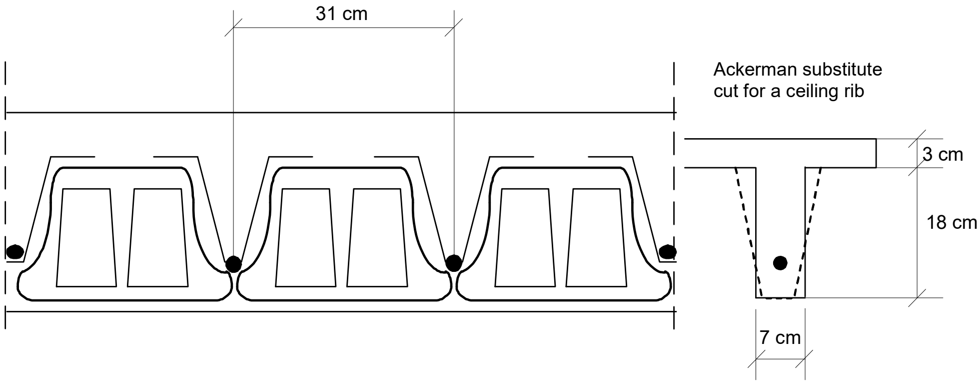

Fig. 4. Adoption of the equivalent intersection of the Ackerman ceiling rib

The design resistance of the cross-section is:

$$S_{cc,eff} = A_{cc,eff} * (h_0 - x_{eff}/2) = 34.47 \text{ cm}^3 \quad (2)$$

where:
$S_{cc,eff}$ – static moment of the effective concrete field of the compression zone,
resulting in: $M_{Rd} = 2.77$ kNm.

Floor own load increased by individual layers was established based on uncovered outcrops. Two parameters were calculated based on the data from the obtained results: characteristic loads of 7.16 kN/m^2 and design loads 8.67 kN/m^2.

Assuming the diagram of a continuous beam, the value of the maximum moment from the calculated self-weight load to the beam width bd = 31 cm (for the extreme span) was determined:

$M_{cw} = 2.42$ kNm,

hence the reserve of load capacity for service loads was: 0.34 kNm per floor rib. The value of the design utility loads was 1.22 kN/m^2 – the design assumed 3.90 kN/m^2.

The next logical step was to check carrying capacity of the floor taking into consideration load assumed for the purpose of the project (it included designed floor layout) [8], meaning computational load of floor layers (in this case floor slabs 2 cm thick) would amount to 6.90 kN/ m^2.

$M_{cw\_proj} = 1.93$ kNm

therefore, reserve of carrying capacity for payloads would be 0.84 kNm on the rib of the ceiling. The maximum value of calculated would amount to 2.99 kN/ m^2 which would be less than value projected in the project documents, any room within a concert hall could be then used as an office space.

The moment of inertia of the cross-section of the Ackerman ceiling rib of the reinforced concrete floor was calculated on the basis of the relation: Further step was to estimate

$$I = \frac{b_d \cdot x_{eff}^{\;3}}{3} + \alpha \cdot A_{s1}\left(h - x_{eff}\right) = 1914 \text{ cm}^4, \quad (3)$$

where:
E_{cm} – Young modulus of concrete,

$$\alpha = \frac{E}{E_{cm}} \quad (4)$$

ceiling deflection (simplified method) was calculated from the relationship:

$$f = \frac{5Ml^2}{48E_{cm}I} = 0.32 \text{ cm}. \quad (5)$$

acceptable value for ceiling deflection:

$f_{dop} = l_0/300 = 300\text{cm}/\ 300 = 1.0$ cm – so the condition of utilisation was fulfilled.

Design engineer changed premises of his calculations again after he received the expert opinion written by the authors of this paper in order to prove that ceiling load-bearing capacity is sufficient. To validate his opinion he uses the wrong thickness of concrete overlay as well as effective spread. Additionally, he put forward in his document different thickness of floor layers in outcrops collected that suggest another weight of construction of the floor.

4. Discussion and proposal of remedial works

Solutions that was presented in the paper [10] consisted of replacing damaged stone labs with the new ones and covering cracks in the bed surface only was incorrect due to the observed overload of the floor – the overload was a result of faulty execution of the renovation work that was non-confirmative in reference to the project recommendations [8] and design errors.

Based on technical state of the floor and statistical data in regards to strength of the material engineers working on the project recommended four different technical solutions for floor layers on Ackerman ceiling, including changing the stone-slabs floor layer.

The first solution describes removing stone-slabs floor layers down to fibreboard and replacing it with a light wooden floor. Removing the top layers would significantly reduce the dead weight of the floor to the calculated value of 5.79 kN/m^2 which would ensure acceptable level of design load (computational loads) for the new wooden flooring and payload at the level of 4.10 kN/m^2.

The second solution would mean removing all the floor layers down to the concrete overlay of Ackerman ceiling. Removal of these layers would reduce their dead weight to the design level of 3.92 kN/m^2. The margin of the allowed design load for the new top layer of the floor and the floor payload would come to 5.97 kN/m^2. Due to using a dry cement mortar (terrace cements and river sands in 1:3 ratio) 4 cm thick and relaying new stone slabs that are 2 cm thick reserve of bearing capacity for design payload would come to 4.39 kN/m^2.

The third solution means taking of top layers of the floor as described in the second solution. That enables making a floor that consists of wooden surface on a double cross joist. In this case after using levelling screed 4 cm thick and layering 22 mm wood staves there would be enough bearing capacity left to estimate payload a the level of 4.88 kN/m^2.

Fourth solution is hypothetical only and was prepared on a specific request of the object's business user. It involves reinforcing the supporting structure Ackerman ceiling in order to shift designed payload – weight of the crowd attending concerts. This reinforcement could be achieved by using steel grillage based on existing construction posts. It could be also accomplished by reinforcing the surface bedrock (ceiling) using state-of-the-art reinforcing materials in a form of carbon ribbon (or glass or aramid fibres). But even if the floor was reinforced it would still require demolition of the existing stone-slabs floor as well as adhesive and concrete floor layers before reconstructing the floor according to the project on the stabilized sand.

All four solutions for the new layout of floor layers on Ackerman ceiling involved replacement of the stone-slabs floor. Engineers recommended the third solution from the list which offered possibly the greatest load relief for the ceiling structure biggest relief of the surface bedrock construction. Based on the control computations and on the grounds of the technical evaluation that was carried out, it was recommended to put the concert hall out of use until the reconstructive work was completed. Adding designed imposed loads (payload) to possible weight of a crowd attending a concert and a dead weight of the existing floor construction could create a risk of construction disaster. Engineers advised to carry a continuous technical supervision while the floor was being renovated. Its results confirmed engineers' conclusions from the expert opinion that was shared when the project team investigated the discrepancies between the actual state of the construction layers and the project documentations and t thickness of the concrete overlay.

5. Conclusions

Carrying renovation or redevelopment of historic building objects that are a part of country's national heritage is remarkably complex. It is so complicated due to legal regulations concerning protection of monuments and a technical issues surrounding interference in civil structure of an object. Special attention needs to be paid to parameters of the building materials, construction characteristics and possible interplay between construction and its environment [3].

The calculations that were carried out to check the statistical and strength attributes (Verification tests, static-strength calculations) of the Ackerman concert hall ceiling and the assessment of the floor structure show that:

- The completion of additional floor layers by the contractor and incorrect definition of concrete overlay thickness in Ackerman ceiling floor done by the designer caused an overload of a load-bearing structure of Ackerman floor – breach of permitted payload foreseen not only for concert halls but also for office accommodations,

- It was advised that the concert hall room was taken out of service in the building until the renovation works of floor layers was completed (or ceiling was strengthened). Construction supervision office was also presented with the discoveries of the report as recommended by the Construction Law [11].

When planning renovation of a historical building it is usually required to take into account an actual technical state of a construction and parameters of pre-existing built-in materials [1,3,4,12]. It is unacceptable to apply solution that alters state of a structure. It is a good practice to select a repair solution to ensure the construction complies with requirements of permissible compressive stress, especially if it would cause tensile or additional shear stress (emergence of tensile or additional shear stresses).

User of the concert hall decided to choose an option that lightens the roof structure by taking down all the floor layers in the mentioned room. Construction works were carried with the use of light manual equipment in order to avoid violating the construction of Ackerman dense-ribbed ceiling Construction works were carried out by the book and ensured safety of building's structural components. Currently the concert hall is used in accordance with its intended purpose.

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