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Innovative Method for Repairing Masonry Buildings

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Summary

The paper presents an approach of testing several models of reinforced masonry with polymer grids. Three techniques of reinforcing were used: by insertion, coating and confining. The results of testing programs show an essential improvement of the behavior under static and dynamic loads of the reinforced masonry. Structural performances also increase more than the cost of reinforcing. The method applies to repair the buildings of brick and stone masonry damaged by long service and/or severe earthquakes. It allows replacing R/C or steel structural members with reinforced masonry ones in achieving more homogeneous and long lasting buildings. Nondestructive tests, performed with mobile acoustic equipment, allow assessing the quality of repairing work.

Keywords: coating, confining, insertion, rehabilitation, reinforcing, remodeling, repairing, restoring, retrofitting, strengthening.

The art of construction the masonry buildings is known on Romanian territories since the Roman Empire. In spite of natural disasters many vestiges of the former ancient settlements like Apulum, Callatis, Tomis and Ulpia Traiana are still preserving. There are also proofs that during centuries some of monumental masonry buildings and churches have been anti-seismically shaped. Although brittle masonry remains a preferred construction material. However, this artificial stone should be often repaired and strengthened. During the last decades the behavior of masonry buildings under different loads was intensively searched. A European Masonry Data Bank was also created. Advanced technologies are used for retrofitting masonry works. Romania, as a Balkan country where frequent earthquakes occur, is highly interested in the field. A method of reinforcing the masonry works with polymer grids was recently proposed. There are three ways of using the synthetic reinforcement: by inserting it on horizontal layers, by coating vertical surfaces and by confining structural members or building bodies. The paper presents the results of three testing programs showing the advantages and also the limits of this simple method, so easy to be applied.

A series of six identical short columns with the dimensions 375x375x874 have been made by using solid clay bricks 240x120x60 with typical strength 7.5 MPa and standard mortar with the cement-lime-sand ratio 1:1:12. Three columns by plain masonry remained as reference, while the other three were reinforced with polymer grids *Tensar SS40*. The reinforcement was inserted in the mortar between bricks on three horizontal layers, namely in the joint no. 2, 6 and 10, upwardly. No outer plastering or coating was provided. The columns have been tested to axial compression up to their ultimate limit state. Between the extreme cases, by reinforcing with only three grids weighting in total 166 grams, the compressive strength of one typical short column increased with 24.32%, while its ductility increased with 30.90%.

Another series of twelve identical wall panels, with the dimensions 875x240x874, have been two by two let plain as reference, plastered and reinforced by insertion and coating. Six wall panels were tested to axial compression and the other six were tested to diagonal tension, both up to their ultimate limit state. The results are comparatively shown in drawings and tables. Since through insertion, coating or insertion and coating the masonry with polymer grids the structural performances typically increase with 10 to 30%, while the cost rises only with 0.6 to12%, the method of reinforcing proves to be advantageous. In order to put in value the own strength of bricks the masonry should be confined. That means to close the masonry structural members with synthetic reinforcements in all directions. Confined in this way all vertical and lateral loads induce in them three-axial compression, and their bearing capacity increases several times.

Two testing programs were carried out on confined masonry. First consists in a 3D model of a two-story masonry building reduced to a scale of 1:2. The model was submitted to seismic actions on the shaking table of ISMES in Bergamo. After reaching the limit state of cracking the model was repaired by confining with two reinforced belts. Tested again, to the same dynamic actions, the retrofitted model showed a higher strength. Beside the usefulness of the repairing method it was shown that masonry buildings could be homogenized. Indeed, it allows replacing R/C or steel structural members with reinforced masonry ones improving in this way the behavior of masonry buildings to lateral actions. The second program was an attempt to compare the behavior of plain and reinforced infills with typical dimensions of 2100x1625x150. The infill reinforced by confining with polymer grids *Tensar SS30* was tested together with similar plain infills on the shaking table of LNEC in Lisbon. Due to the weak strength of horizontally hollowed bricks and the poor connection between grids and masonry the test results appeared as irrelevant. However, it was shown that the provision 2.5.6 of Eurocode 8 regarding *Damage Limitation of Infills* could be solved by confinement. This is why the testing program will be resumed through INCO Copernicus project IC15-CT97-0203.

The test results presented above show an essential improvement of the behavior under static and dynamic loads of the masonry reinforced with polymer grids. The method is worth to be used for reconstruction or renewal of any part of a damaged or deteriorated masonry building to provide the same level of strength and/or ductility, which the building had prior to the damage. Nondestructive inspection, performed with mobile Impact-Echo equipment , allows assessing the quality of repairing work.

Investigation and Repair of Old Culturally Valuable Concrete Structures

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Summary

The preliminary investigation and the repair of culturally valuable concrete structures is determined by the demand for preserving the original architectural appearance and surface texture of the altered concrete structure as well as for meeting the technical requirements for a durable structure. The characteristics of such an investigation and corresponding repair strategies are described and illustrated by two examples, the music center "Liederhalle Stuttgart" and the concrete dam "Schluchseesperre" in the Black Forest.

Keywords: investigation methods, concrete surfaces, architectural aspects, preserving the surface texture, repair strategies

1. Introduction

In almost all conventional measures of protection and repair of concrete structures where the surfaces are damaged due to various deterioration mechanisms the repair consists of applying surface coatings or overlays made of polymer or mineral materials to the entire surface of the structure. Sometimes, this is indeed the only method to preserve the load carrying capacity and the functioning of the structure. But in many cases this is not necessary at all, and the only purpose of such a measure is to give the building an immaculate appearance.

In the full paper it will be shown, that, on the basis of accurate and detailled investigations of the structure and its materials, it may be possible to repair old concrete buildings much more carefully. In this context we limit ourselves to the repair of concrete and reinforced concrete structures which have gained historical importance due to their architectural appearance or due to special technical qualities.

2. Investigations

The objective of such investigations is to create a solid basis for the prediction of the progress of the deterioration and corrosion of the old structure so that a restoration concept can be developed which preserves the original concrete surface whenever possible. Furthermore, such investigations are necessary to derive measures for a suitable restoration of the deteriorated parts of the concrete surface as well as to develop and specify repair mortars or concretes which are compatible with the old concrete surface in technological as well as in architectural respects. Finally on the basis of the investigations strategies for the maintenance and the inspection of a particular member or building have to be derived.

3. Repair work

The method of repair of the structure depends on the results of the comprehensive investigation and may differ from conventional methods. In some cases, repair mortars or concretes have to be used which are not commercially available, and which have to be optimized in its particular technological and visual properties. In the full paper the investigation and the repair of two culturally valuable buildings under the aspects described above are illustrated.

4. Conclusions

The preservation and repair of monuments and historical buildings made of plain or reinforced concrete should be designed such that the original appearance and the original surface texture is preserved as far as possible. To accomplish this, preliminary investigations and strategies are required which differ from those usually applied in the conventional repair of concrete surfaces. Hence, special methods and planning tools have to be developed and used to take into account particular architectural and technological aspects. The detailed scientific based investigations create the basis to minimize the extent to which the original surfaces had to be altered in order to preserve the overall impression of the structure. Further damage, which also has to be expected in conventional repair work, should occur only to a very limited extent and may be repaired within the regular maintenance work.

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Evaluation and Preservation of Historic Buildings in Osijek

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Summary:

Tvrda is an old part of town Osijek, built at the beginning of the XVIII century on the ruins of the old fortress from the XIII and an oriental Turkish center from the XVII century. It was a center of civil and military rule, religious and cultural midpoint and an important craft and trade center.

Fig. 1 Tvrda in the XVIII century

The building started immediately after the liberation from the Turkish rule in the year 1689 and was mostly finished by the 1722 year. The outer walls were from that time continuously strengthened and the inner ward was continuously remodeled. At the 20^{th} years of this century most of the outer fortifications were destroyed.

Tvrda presents a specific complex of the baroque architecture and is one of the most important Croatian cultural monuments. Therefore a broad study has been undertaken in order to properly estimate its value as cultural and civilization phenomena and to deepen our understanding from their stylistic and artistic values. Studies, analysis and systematization of the old structures remaining in Tvrda, according to their usage, structural type and materials used, has enabled the recognition of their true historic value, contributed in their preservation and choice of an adequate restoration.

Keywords: Osijek, castle, building types, systematization, restoration, preservation

Tvrda's building types

There are more than a hundred different buildings which are a living witness of the old structural types, crafts and materials.
The biggest buildings in Tvrda, according to their size, number and area which they cover are military fortifications, namely: eight bastions, seven towers, inner and outer wards, keeps, gate houses, buttresses, battlement parapets, palisades, defense dikes and trenches, approach roads, ditches and moats.

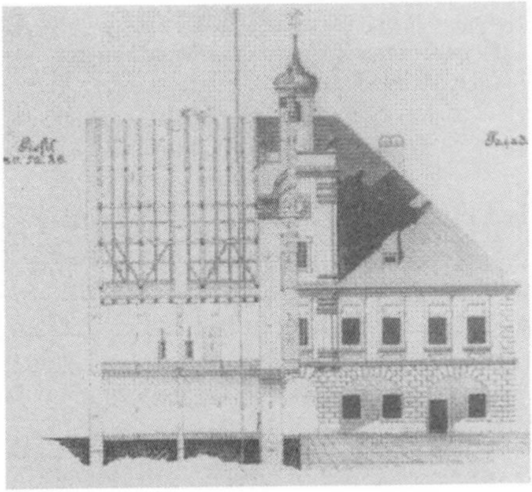

Fig. 2 Tvrda's main keep building

Second group consists of other army structures: keeps, barracks, depots, powder-houses, hospitals and so on. They were mostly large elongated buildings built directly by the fortification line.

The third group are sacral buildings: churches and monasteries. They typically have broad naves with high walls and big ceiling spans.

The most numerous were administrative and civilian buildings in the Tvrda's inner ward. They differed in their use, size and erection time.
Thanks to the military fortification which has caused Tvrda's isolation and solitude from the other town parts, the historic treasure, their typology, architecture, ambient and structural characteristic is preserved. In order to preserve their authenticity and historic value, it is necessary to evaluate, systematize and validate the buildings in their whole and particularity (materials used, construction technology, structural type).

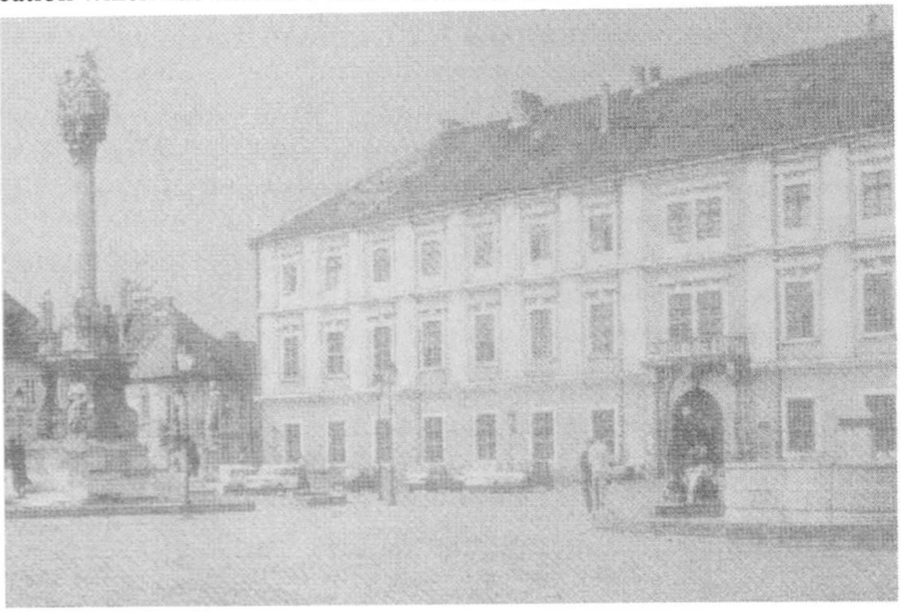

Fig 3. Main Tvrda's square

By knowledge of all of these data it is possible to chose the correct structural strengthening method and use of the latest technologies and materials so not the diminish the buildings' historic value.

Confining of Masonry Walls with Steel Elements

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1. Introduction

The transverse confinement of masonry is generally obtained by means of steel elements, namely tie-bars or tie-beams, conveniently fastened to the masonry with steel end-plates [1]. In this way a quite effective structural system is obtained, in which the existing materials are stressed in the most rational way. In addition, the intervention can be arranged in such a way to be easily controlled or removed, if necessary. This paper is focused upon the definition of a theoretical procedure for the prediction of the effect of confinement. The inelastic behaviour of both masonry and steel is accounted for. The method is applied in order to reproduce the results of a F.E.M. numerical simulation carried out with the non linear code ABAQUS, whose reliability has been checked in turn by means of a direct comparison with some existing experimental data [2]. The case under consideration is that of masonry walls subjected to compressive load and confined by tied steel plates. In spite of some simplifications introduced into the analytical developments, the method proposed can be considered as a first attempt to the direct evaluation of the load bearing capacity of confined masonry.

2. The theoretical model for confined masonry

The model presented hereafter concerns the case of a masonry wall uniformly confined along its transverse direction by means of tied end-plates (fig.1). It is assumed that:

1) the behaviour of masonry is assumed to be isotropic;
2) the steel confining plate is fully rigid;
3) the behaviour of masonry in compression is represented by means of a suitable non-linear σ–ε law, whose parameters are fitted on the basis of experimental evidence;
4) the mutual relationship between the applied stress and the confining stresses and is of pseudo-elastic type, i.e. *Navier*-like equations hold in both elastic and post-elastic range.

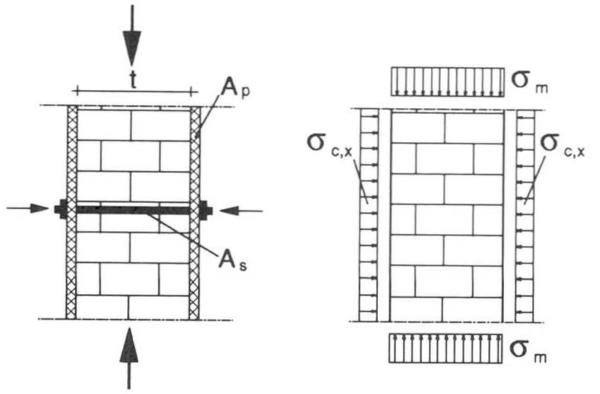

Fig. 1 Idealisation of confined masonry

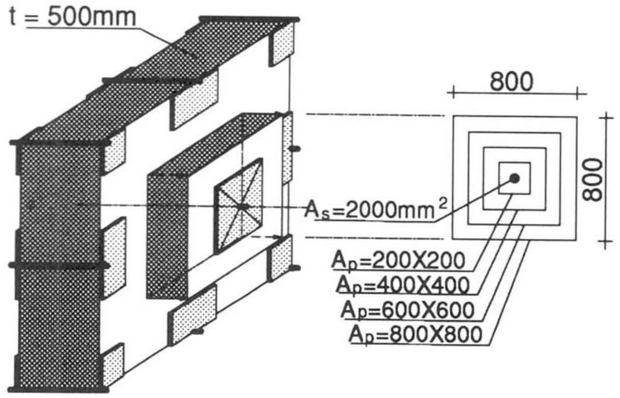

Fig.2 The masonry panel considered with F.E.M. simulation

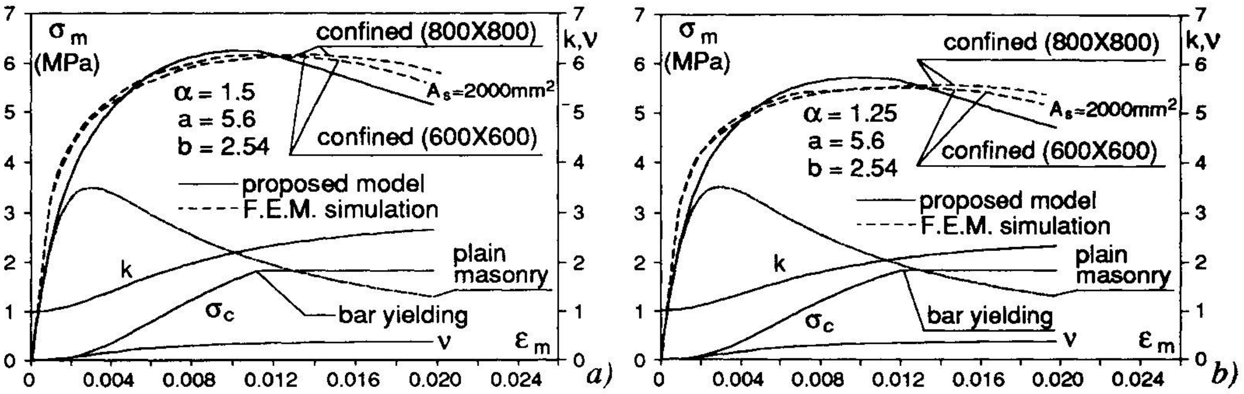

Fig.3 Fitting of the proposed theoretical model for t=500mm (a) and t=300mm (b)

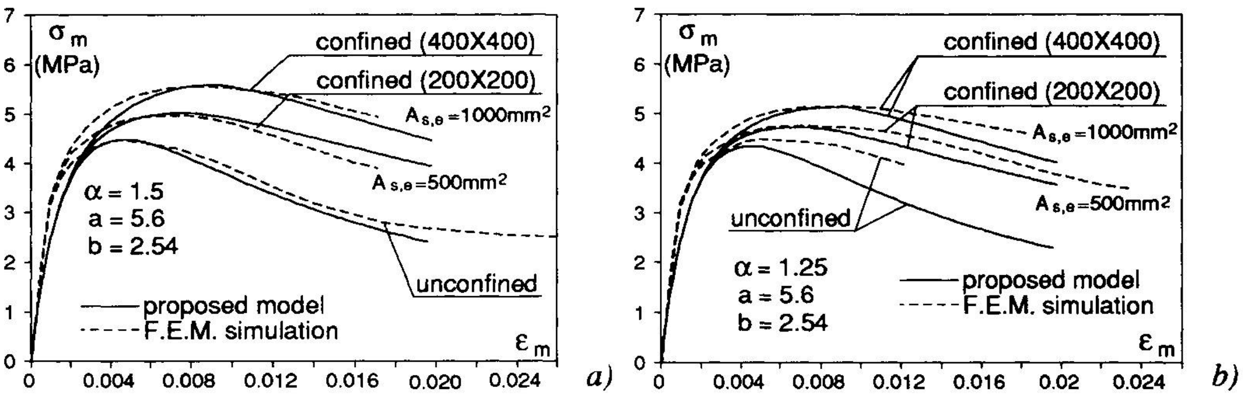

Fig.4 Values of $A_{s,e}$ for t=500mm (a) and 300mm (b) and corresponding behavioural curves

Under these hypotheses the expressions for the state of stress in the confined wall are found. They are able to take into account the yielding of steel bars. The *Saenz*' s law for concrete is considered for the representation of the behaviour of plain masonry. A suitable law is assumed for the *Poisson*' s modulus ν [3] and for the masonry strength enhancement factor k accounting for the combined state of stress. With respect to the existing models, mostly concerned with concrete, in this procedure the number of parameters to be fitted empirically is drastically reduced to k and ν ,only. With a suitable choice of these factors, the proposed model can interpret experimental or numerical results with a satisfying degree of accuracy (fig 3). When a partial confinement with smaller plates is considered, the concept of equivalent steel area $A_{s,e}$ is introduced (fig. 4). This is defined as the cross section area of the steel bar which, in case of global confinement, would have produced the same load bearing capacity for the wall. Nevertheless, as far as this concept is concerned, the general validity of the method has been not completely assessed. When, in fact, partial confining with smaller plates is considered, the failure mechanism of the wall could be different, with a possible collapse due to punching or shear-tension in the masonry. For this reason, an *ad-hoc* experimental program is presently being planned, in order to provide new elements for the set-up of the proposed model.

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Repairs of Vertically Deflected Buildings

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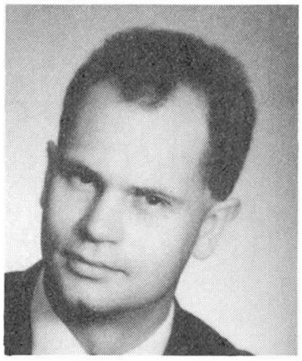

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Summary

The paper presents methods of rectifying vertically deflected buildings and particularly rectifications with the use of hydraulic jacks. The paper contains the characteristic of the method, its comparison with other methods used so far as well as depicts the scope of works necessary to perform rectification. Both the cost-benefit study and the social aspect speak for rectifying buildings by means of the method presented in the paper.

Keywords: deflected buildings, deflections eliminating, surface deformation

1. Methods of eliminating deflections

Mining exploitation results in surface deformation - deformation of a continuous character. The deformations are represented by a number of parameters. The most essential of them influencing the mining area buildings are: strain, mining subsidence, slope, curvature.
Due to its special character , deflection has always been a problem difficult to solve.
Methods of eliminating deflections can be divided into two groups. The first one consists in ground removal from under the part of the building which is placed higher (Fig. 1a), the other one - in lifting with the help of servo-motors (Fig. 1b).
Within the second method of rectifying deflected buildings two ways were practically used.
The first one is rectification with the help of individually operated hydraulic servo-motors. In that method the servo-motors are centrally operated from one oil pump and the steering takes place through force extortion in each jack individually. Precision is crucial in that process, since the success of the operation depends on the experience and the intuition of the person operating the jack. Up to 1997, 20 family houses had been rectified in the Rybnik Mining Company area by using that method.
The other way in the group of methods being discussed is the usage of computer operated hydraulic jacks to rectify buildings. By means of apparatus specially designed for that purpose, proper relocation is forced in each of the jacks. The last four years saw 80 buildings rectified that way in the Rybnik Mining Company area. Two of the above described methods of building rectification are being used side by side, therefore they will be described in detail.

2. Building rectification through lifting

Each object meant for rectification requires a number of preparatory works. The pre-rectification works last about a week and comprise hewing out the servo-motor recesses, cutting off the central heating, gas and water-sewage systems, propping the window- and door- heads and building in the jacks in the cellar storey.
For the time of the rectification the building is specially protected. The most common protection

is two channel bars running on both sides of the torn walls. Fig.2 shows built in jacks in cellar storey.

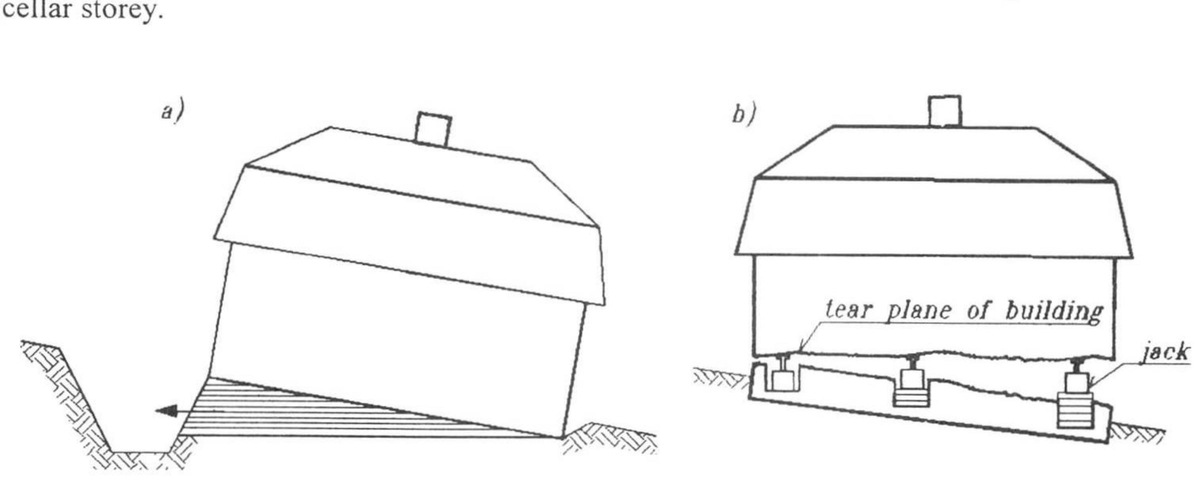

Fig.1 Methods of buildings rectifications

Fig.2 Built in jacks in cellar storey

The process of rectification of the building comprises three stages. During stage one the building is being torn. Stage two is parallel lifting. Levelling stage is the essential rectification stage and it resolves itself into non- uniform lift of the building.
Rectification of buildings is a very interesting issue from the technical point of view. One should keep in mind, however, that it is mainly a social problem. So far, buildings deflected by more than 7% had been dislodged and demolished. Nowadays, thanks to the rectification prospect, they are not demolished but their utility value is fully restored. Thus, there is no need to rehouse people. Inhabitants of Silesia can keep living in their households which , very often, they had built by themselves.
Presented method of rectifying buildings through lifting allows to quickly and faultlessly eliminate mining damage effects such as building deflection, thus preventing them from being demolished or collapsing.
The rectification process is cost-effective both from the economical and the social point of view.

Damages and Rehabilitation of the Orthodox Cathedral in Belgrade

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Summary

The Belgrade Orthodox Cathedral, made of brick in lime mortar 150 years ago has the ceiling structure of its central part supported by longitudinal walls composed of lateral brick arches with steel ties at the level of the vertex and of shallow vaults between them. Loosening of steel ties through loosening of their anchorage in longitudinal walls, caused structural damages of arches, especially of their upper chords together with large deflections. The rehabilitation included the production of additional prestressed ties whereby the disturbed system of arches with ties has been reestablished. At the same time, inconvenient stresses in the arches have been corrected to a certain degree. The strengthening of the sections of the arches has been made by additional recycled masonry concrete elements compatible with the materials of the arches.

The Belgrade Orthodox Cathedral is of a rectangular shape in its base with a semi-circular altar. It is 18.8m wide and 43.8 m long, built of brick in lime mortar with a wooden roof. The ceiling of the central part is composed of four lateral ellipse-shaped arches made of brick with the steel ties.

After detailed inspection, the conclusion has been made that the basic cause of structural damages is the loosening of steel ties of the lateral brick arches in the ceiling, because they have been partially pulled out due to loosening of their anchorages in longitudinal walls.

The solution for rehabilitation included the following: (a) building in a pair of prestressed steel ties ϕ 36mm on every arch whereby the forces of approximately 200 kN would be inserted into the structure; (b) production of concrete anchor blocks for the new ties; (c) making the strengthening on extradoses in the arch vertexes zones in the shape of "П" elements bonded with the existing arches.

The Figure 1 shows the vertical section of an arch whereat the height position of additional ties, as well as several specific details.

It has been decided to make "П" - shaped strengthenings of recycled brick concrete which will be so modelled as to achieve the composite with the properties which will, to a suitable extent, be compatible with the properties of the materials within the existing arches.

In order to obtain the material to meet the mecesaru conditions, four mixtures have been treated the compositions of which are presented in the next table.

Composite	Quantities of the material (kg/m3)				Bulk density (kg/m3)	
	Cement	Hydrated lime	Aggregate	Water	Design	Actual
1	200	200	1300	350	2050	1960
2	300	100	1250	400	2050	2120
3	400	0	1200	450	2050	2030
4	450	0	1150	450	2050	2090

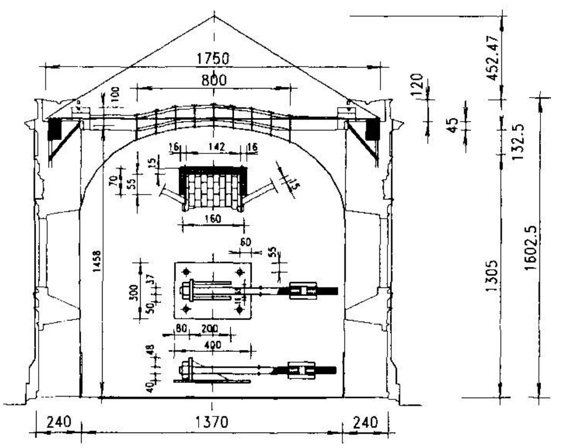

Fig. 1 Additional ties in arches including specific details

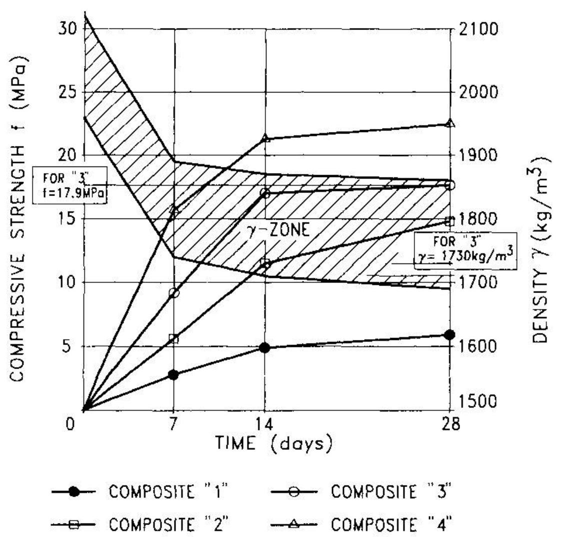

Fig. 2 Bulk densities and compressive strengths of all composites

The presentation of bulk densities and of compressive strengths of all investigated composites - concretes - at the ages of 7, 14 and 28 days is given in Figure 2. As it is obvious, the optimum meeting of the set conditions is achieved by the mixture - composite marked by number 3.

Being made of recycled masonry concrete, the additional "П" elements are compatible with the basic material of the arches whereby a structural system has been obtained containing no outstanding material discontinuities and non-homogeneities that could cause inconvenient states of stresses within the structures which, in some localities have already reached the ultimate limit state.

Reconstruction of Wooden Floors in 19th Century Industrial Buildings

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Summary

The strengthening of wooden floors in the industrial building built towards the close of 19th century is presented in the paper. The strengthening was combined with modernization and adaptation of the building for public purposes. According to this method, the floors in a few buildings, formerly being factories, in Lodz have been strengthened.

1. Introduction

An intensive development of the textile industry in Lodz at the turn of 19th century was accompanied by erecting objects, characteristic for that industry, built from bricks and wood which were available in the region. They were one storey saw-tooth halls or several storeys buildings made of bricks, with wooden floors. These buildings were used in accordance with their assignment during tens of years. Only in the recent period (1980 years), ownership transformations caused the necessity of modernization and adaptation of these buildings for another purposes. The strengthening technique described in the paper concerns wooden floors in several-storeys buildings.

2. Typical building description

The considered buildings are usually of rectangular plan. They have three or four storeys, mostly without cellars. A plan of one of renovated buildings, situated in the downtown of Lodz, is presented in fig. 1.

Floors of these buildings are supported on external brick walls and on two or three rows of internal cast iron columns. Wooden deals of dimensions ca 0.25×0.35m, situated crosswise in spacing of ca $4.5\div5.5$m. Ribs made of wooden deals of the cross section ca 0.10×0.20m are situated longwise in spacing of ca $0.5\div0.7$m. Wooden boards of thickness of 45mm were applied as the floor covering.

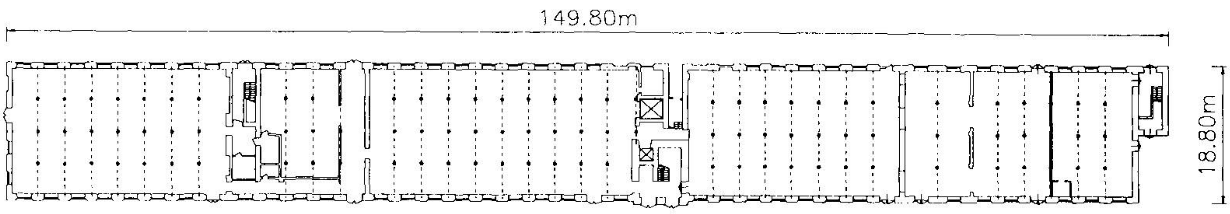

Fig. 1 Ground plan of the renovated building

3. Conception and realization of the floor modernization

As the strengthening and the stiffening of the floors, RC beams were designed. They clamped the wooden main beams and were connected with the RC slab cast on the boards of existing floor. The newly designed continuous four span RC beams of ca 0.50m depth, of 2×0.10m web width and of 0.5m upper part width, are supported on cast iron columns (through the existing wooden main beam and the cast iron head) and in external wall pockets which have been hewn beside and above existing main beams (see fig. 2). Ends of the wooden ribs have been cast in the RC beams.

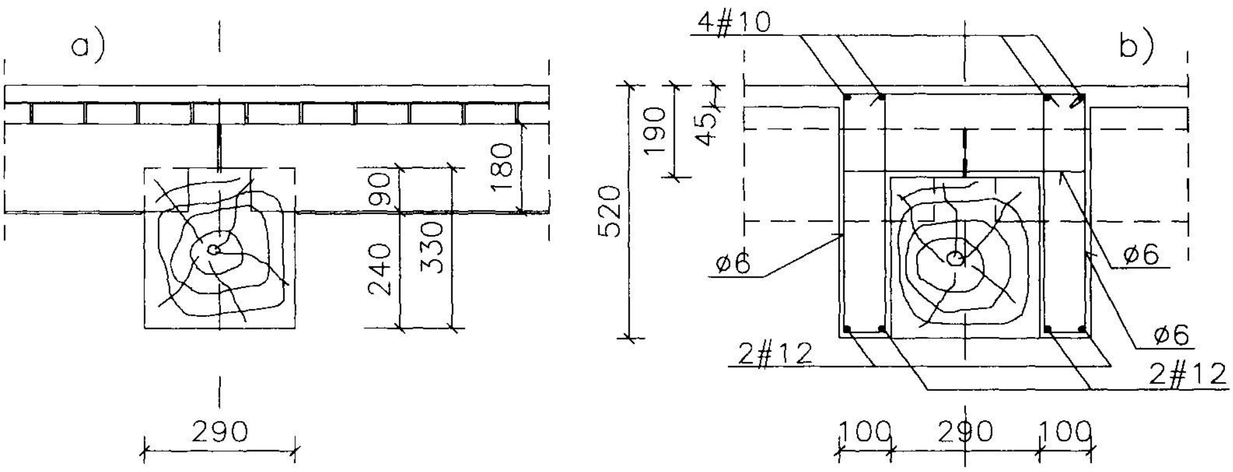

Fig. 2. Strengthening construction

Conclusions

The applied method enables to adapt many old factory buildings for new purposes, considerably saving costs of materials and labour. The constructional result is the building with the floors performing all contemporary requirements relating to load capacity, stiffeness and sufficient fire resistance. The applied method minimizes demolition works and is environment-friendly because it does need the utilization of removed structure elements.

Timber Floors Strengthened with Concrete

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Summary

Timber-concrete composite (tcc) beams may be used for the renovation of old timber floors. Although these systems are not new (Poštulka, 1997) and form a simple and practical solution, they are not widely adopted. One of the reasons for this is the lack of uniform design rules. In a research programme shear tests on four different fastener types were performed as well as bending tests on tcc beams, manufactured with these fasteners. A non-linear simulation model was built that is able to perform a Monte Carlo simulation on single tcc beams and tcc floor systems. The model was successfully verified with the bending tests before other simulations were performed. Several geometries were simulated resulting in a statistical distribution of the load-carrying capacity of each geometry. This model now allows for the calculation of the characteristic system strength and stiffness values.

1. Introduction

Timber-concrete composite (tcc) floors may be regarded as an alternative renovation method of timber floors in which the timber floor is integrated and thus still functions. Metal fasteners are drilled into the top of the existing timber beams before the concrete is poured upon the planks of the timber floor. After hardening of the concrete a timber-concrete composite (tcc) beam has been realised.

In order to set up design rules for these structures and to obtain the strength and stiffness of some fastener types, a joint research programme was started in 1992. Shear tests on these fastener types and bending tests on tcc beams, in which these connectors were utilised, have been carried out at the University of Karlsruhe in Germany. A simulation model, partly based on the finite element method DIANA, has been developed at TNO and Delft University of Technology that was then used to analyse the beam tests. In this way it was possible to test the validity of the simulation model, to predict the behaviour of other tcc geometries and to obtain the statistical distribution of the short term load-carrying capacities.

2. Shear Tests

The load-displacement curves for four different connector types were determined. These connector types are respectively screws, nailplates and two kinds of dowels. Two of these joint types, screws and one kind of dowel as represented in figure 1, will be discussed here since the other connector types are not suitable for renovation purposes.

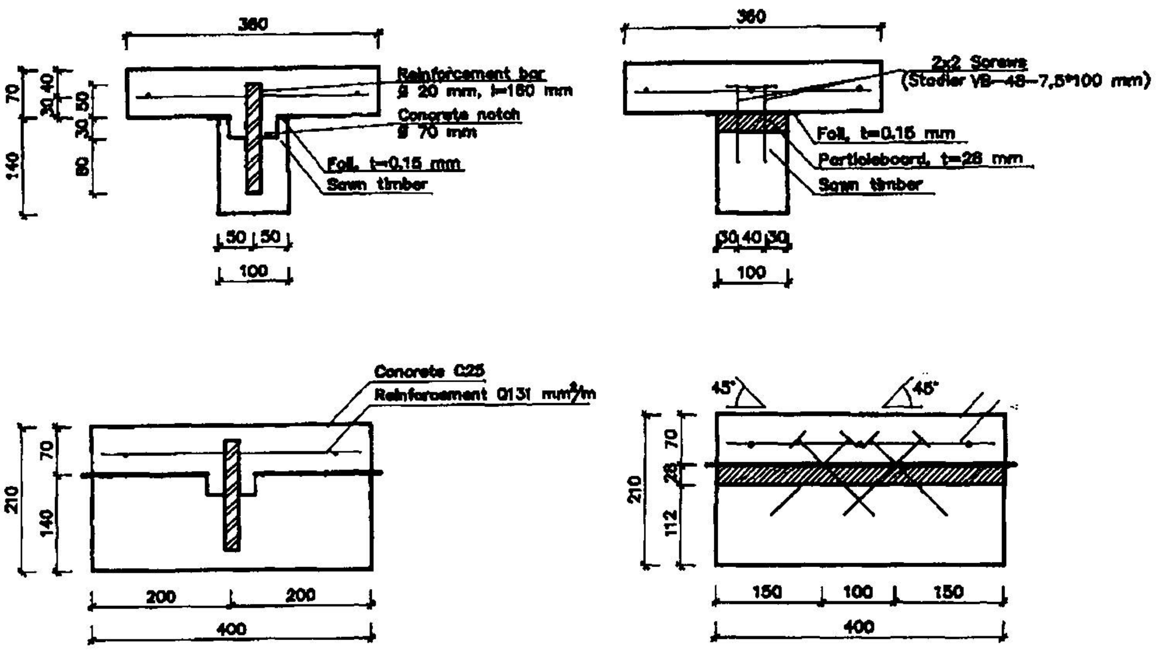

Fig. 1 Two fastener types: reinforcement bar with concrete dowel and screws at 45^0.

The screws are arranged in such a way that half of them is placed at an angle of 45^0 and the other half at an angle of -45^0 with the timber beam axis. The screws are 150 mm long and are driven into the timber for about 100 mm, the remaining part of the screw forms the connection with the concrete. The dowels consist of a reinforcement bar with a diameter of 20 mm and a length of 160 mm, which is driven into the timber for 110 mm. An extra hole with a diameter of 70 mm and a depth of 30 mm surrounds the reinforcement bar and is filled with concrete during moulding. This concrete dowel decreases the stresses in the timber caused by the reinforcement bar.

3. Bending Tests

The bending tests were performed on beams with a span of 5.4 m. These beams were loaded in four-point bending and the slip of several connectors as well as the vertical displacement at midspan was measured. For some connector types the vertical displacement between the timber and the concrete was measured as well midspan and/or near the supports. Due to the type of connector a horizontal gap could occur between the timber and the concrete. Figure 2 shows the test-set-up for the bending tests.

Construction and Design of Composite Concrete - Timber Floors

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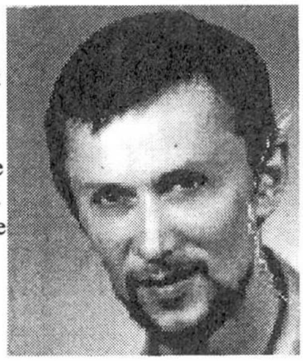

Summary

The concrete-timber floor consists of timber joists connected with concrete slab by means of mechanical connectors. The concrete-timber technic is specially predestinated for rehabilitation of timber floors in old buildings. In comparision to timber floor double load bearing capacity and triple stiffness may be expected on an average. An overall stiffness of the building significantly increases, concrete slab makes the floor more resistant to fire and acoustic is improved.

1. An experiance from applications of concrete-timber technic to rehabilitation of old timber floors in Poland

In Poland concrete-timber floors were first applied in 1980 for rehabilitation of timber floors in state nursery school. Since then several other apartment and public buildings were rehabilitated. The most important element for succesfull rehabilitation of timber floor with concre-timber technic is precise assessment of timber joists for biological corrosion and material properties of timber. The best way to carry out such assessment was by total removing all floor boards and clay pugging until all floor joists were visible. The joists with damaged ends at the supports had to be reconstructed by joists nailed to the beam from both sides. The demaged ends were cut off or precisely cleaned from biological corrosion. The timber joists may also be reconstructed by channel iron steel or by so called „concrete shoe".
When the floor joists were excessively deflected, the depth of the slab over the joists were increased and additional reinforcement (loop shaped) given.
In order to decrease the biological corrosion all wooden elements were impregnated by fungicide. It was recommended to impregnate the joist ends by pouring the agent into preliminary drilled skew holes at boths ends of each joist.
Becouse the floor joists and other wooden elements are covered by concrete slab, closed space is formed and has to be ventilated. In order to make the movement of air in that closed space possible it was necessary to leave openings in concrete slab adjacent to walls at both ends in all interbeam spaces.

2. Recommendations for design

It is recommended to assume slab depth from 6 to 8 cm and the grade of concrete minimum B20. Reinforcement of the slab made of round oridinary steel diameter 6 to 8 mm should be placed in the middle of the slab depth. Main bars should be laid perpendiculary to the timber joists at spacing less than 12 cm, distributing bars at spacing less than 33 cm.

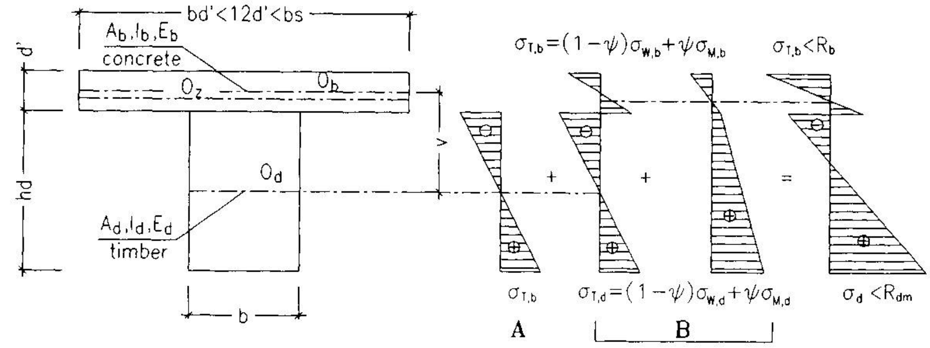

Fig. 1 The distribution of stresses in concrete slab and timber joist:
A - erection phase, B - service phase

The concrete-timber sections must be designed on the assiumption the joint at adjacent sides of joist and concrete is flexible, Fig. 1. The stress in timber joist section is calculated from equation:

$$\sigma_d = \sigma_m + \sigma_{T,d} = \sigma_m + [(1-\psi)\cdot\sigma_{W,d} + \psi\cdot\sigma_{M,d}] \qquad (1)$$

where:

σ_m - the stress in the timber joist before connection with concrete slab caused by erection load (moment M_m), when the floor is supported σ_m is relatively small,

$\sigma_{T,d}$ - the stress in the timber joist after connection caused by moment $M=M_o-M_m$ calculated from the live load and part of the dead load (load of the floor layers),

$\sigma_{W,d}$ - stress in the timber joist caused by moment M in the multilayer section ($C = 0$),

$\sigma_{M,d}$ - stress in the timber joist caused by moment M in combined section (rigid joint, $C = \infty$),

ψ - coefficient taking into account the influence of joint flexibility on distribution of stresses in the section.

For the notations on Fig. 1 the coefficient ψ is calculated as follows:

$$\psi = 1 / (1 + \pi^2\cdot\Sigma EJ\cdot\beta / (C\cdot l^2 v^2 (1+\beta)) \qquad (2)$$

$$\beta = (E_oJ_o / \Sigma EJ - 1) \qquad (3)$$

where:

E_oJ_o - flexural rigidity of the combined section, [kNm^2], $\Sigma EJ = E_bJ_b + E_dJ_d$ - the sum of concrete and joist section flexural rigidities, l - calculated span, [m], C - joint flexibility modulus, [kN/m^2].

The stresses in the extreme fiber of the timber joist and concrete slab must not exceed the permissible value.

The increment of concrete-timber floor deflection over the preliminary timber floor deflection may be calculated from equation:

$$f = (1 + \varphi)\cdot f_M \qquad (4)$$

where:

f_M - deflection calculated as for combined section (rigid joint, $C = \infty$),

$\varphi = (1-\psi)\cdot\beta$ - the coefficient taking into account the influence of flexibility on deflection.

In-plane Stiffness of the Prussic Caps

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Summary:

Prussian caps (flat masonry arches) were used very often as a masonry slab structure in living houses on the verge of the century. While transfer of the vertical loads is clear, for the in-plane acting horizontal loads (wind, earthquake, etc.) remain several doubts to be cleared. Floor slab is commonly assumed as an infinitely stiff membrane which performs force transfer and distribution to the vertical bracing elements. In this work, we have tried to analyze: a.)stiffness of the Prussian caps and their ability to withstand the in-plane loading; b.) meaning of the strengthening with an additional thin concrete slab; c.) stiffness and load distribution among the two systems.

Keywords: masonry, Prussian caps, strengthening, stiffness distribution, justification

Analyzed models

Analyzed were slab structures made as Prussian caps (flat masonry arches) with a slab span of 6m and width of 4m. They consist of 4 flat masonry arch fields supported by steel I beams which span over the shorter span. Each span is 1,50m long and its height at its middle is 0,15m (f/l=1/10). Walls were modeled as having a symmetric stiffness distribution (Model S) and non-symmetric stiffness distribution (walls with the openings-MODEL N).

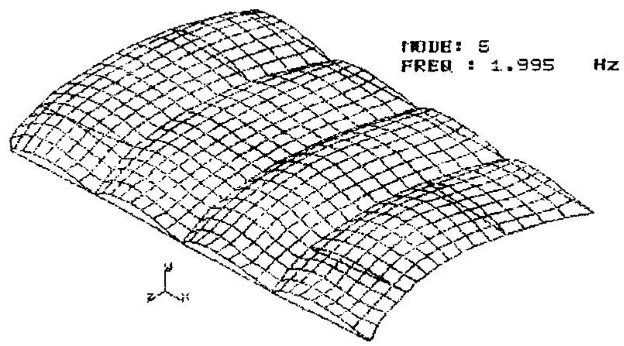


Fig. 1 Modal form of PKS

For both models, several sub-models were defined: (1) MODEL PK where the Prussian caps are the only horizontal diaphragm; (2) MODEL PK-PL are the Prussian caps with the added dead weight load due to the reinforced/concrete slab. Stiffness was not changed; (3)MODEL PL where 6cm reinforced/concrete slab was the only stiff diaphragm and the loading included total load. All models were exposed to the horizontal design earthquake response acceleration in both directions (longitudinal -MODEL-X and transverse MODEL-Z). Contribution of the single modes to overall structural response is estimated with SRSS mode combination method.

Results of analysis

Calculated were natural frequencies, forms and response spectra analysis has been performed for all analyzed models. Mode contribution was analyzed with SRSS combination method.

Fig. 2 SRSS displacements in X direction

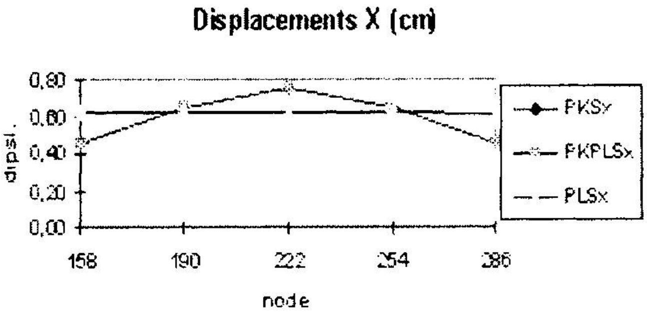
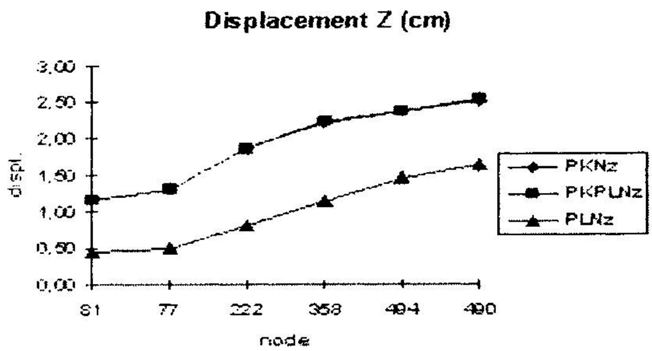


FIG. 3 SRSS displacements in Z direction



The basic slab system, consisting of the flat masonry arches and steel beams (Prussian caps) is not infinitely stiff in comparison with the walls. The buildings made with that system should be, for horizontal loading, analyzed as buildings with a flexible diaphragm.

Additional thin reinforced/concrete slab has contributed to the overall stiffness, while at the same time, its additional dead load did not change basic characteristics of the main system. Average displacements were the same for all models, but their distribution along the span was different (Fig. 2 and 3). Reinforced/concrete slab contributes to the uniform displacement distribution along the span and it has activated the complete bracing system in resisting horizontal loads.

The reinforced/concrete slab, although relatively thin, makes a significant and important contribution to the in-plane stiffness of the basic Prussian cap structure. That seems to be reasonable and justified solution for increase of the in-plane structural stiffness. In that case reinforced concrete slab has to be designed to take over and transfer the complete horizontal loading. Due respect must be paid to the joining details among the new slab, Prussian cap and the walls.

This study is limited and its value is only qualitative. For a quantitative value it should be expanded on the systems having different geometry, thickness, stiffness and mass distribution and excitation models.

Consolidation Works for a Historic Building

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Summary

The paper presents consolidation works used for the historic monument. The consolidation of the foundation ground was carried out by injections of cement suspension. The consolidation of the deteriorated masonry was also injected in order to make it regain its monolith character.

Keywords: Consolidation, injections, ground, electrodraining, cement paste, building, sodium silicate, respiration, rehabilitation, foundation.

Consolidation Work

Among the buildings existing in the area of the town of Cluj Napoca, historic patrimony buildings are the ones with special problems. Some of these buildings are situated in the historical centre of the town. Such is the building erected in 1789 – 1810, on the remains of an older building, recorded in 1607 as " the Redoubt" and used as an inn, later on as barracks and more recently as the premises for some institutions and art centre.

The historical monuments witnessed important events such as the works of the Transylvanian Diet (1849 - 1865) the Trial of The Memorandum Writers (1894), concerts held by Franz Liszt (1846, 1847), J. Brahms (1879) and George Enescu. The front was built in 1789, then it was modified, completed and finally renovated in 1959. At present, the building shelters the Ethnographical Museum of Transylvania, a prestigious cultural institution in Romania.
The building had to be consolidated due to numerous structural damages. The works carried out intended to regain the performance required under normal operation and the provide the stiffness of the building as a whole and its components.
The building is made up of a basement, ground floor, upstairs and penthouse. Its structure consists of brick walls. The foundations and basement walls are from 80 –

100 cm thick stone masonry with lime mortar. The foundation is at 4.10 – 5.60 m and the basement level at 3.40 – 5.10 m compared to the ground.

The foundation ground is made up of a earth landfill layer, 3.00 m thick, then a layer of fine, loose, grey – yellowish clay sand, 3.40 – 3.80 m thick, deposited above a grey sand gravel layer.

The underground water is at 3.5 – 6.0 m depth and its fluctuations are up to 2.0 m in intervals full of rains.

The foundations rests on the fine, loose clay sand. The basement and ground floors are made from brick cylindrical vaults. The ground-floor walls contain both brick and stone masonry with lime mortar. The upstairs floor has wooden beams of 20 x 40 cm; the framework is wooden, while the roof is of tiles.

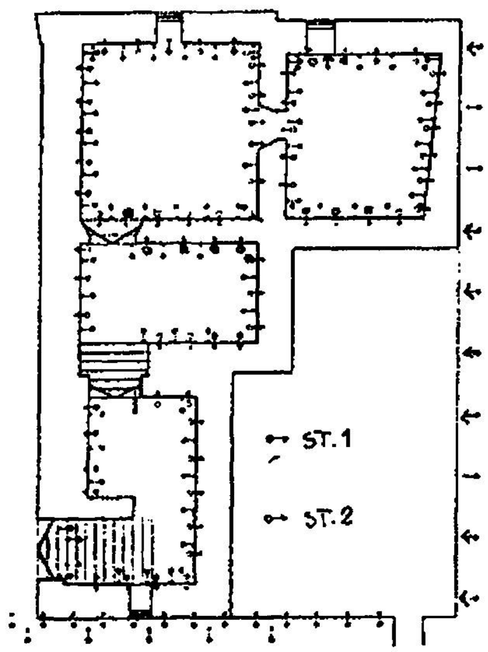

Fig.1 The consolidation of the foundation in two stages

As time went on, the building underwent structural defects whose repair was much delayed. Fissures and cracks appeared in the basement, ground and upper floor walls, in the cylindrical vault of the floor over the ground-floor and the entrance vault.

The size and intensity of these deteriorations required consolidation measures for the foundation ground and structural members.

The consolidation of the foundation ground was carried out by injections of cement suspension, made in tow stages. (Fig. 1)

In the first stage – the precints were injected to insulated the area and to prevent potential leakage leading to fracture pressure in the second stage.

Roof Structure Renewal of the St. Peter and Paul Church in Osijek

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Summary:

Described is the timber roof structure of the St. Peter and Paul parish church in Osijek as well as the damage this building suffered from missile attacks during war operations in 1991/92. The consequences of rain penetration into vital parts of the wooden roof structure are presented. It is emphasized that such penetration and leakage was caused to the impossibilities of proper maintenance of the church in the war period and after the damage done. Results of the technical evaluation of the damage to the main load bearing structural element, as well most significant repair solutions are presented.

Keywords: timber roof structure, church, Osijek, reconstruction

1. INTRODUCTION

The parish church of Peter and Paul was built in 1898. German constructor Franz Langenberg conceived it as a three-nave basilica with a transept of a ground-plan formed like a Latin cross in neogotic style. Its dominated accent is the 90 m high tower dominate the Osijeks panoramic view. The church is a registered monument of culture, situated in the urban center of Osijek, which is entirely also registered as historical monument too. During the war in 1991/92 the church was severely damaged in missile attacks on Osijek. The parish church was hit by more then hundred mortar and rocket shells. Damages are scattered all over the building: from stained-glass windows (vitrages), stone figures and reliefs, to the roofing tiles and the roof timber structure. Estimated total damage to the object, caused by direct and indirect hits, amounts to about 4,500,000.00 DEM. To protect the object from atmospheric agents as well as to prevent further damages of the interior the roof structure has to be repaired. Therefore a through repair and restoration was planned. Professional expertise has been done to get acquired with the real conditions

of the object. On the bases of professional expertise blueprints and design of the repairs are planed and successfully performed.
Beside the roof cover tiles damaged by the shells explosions the damage was done to the wooden structure. The deformations of the vital structural elements have been observed too. Some of the damages are caused by deterioration of wood, and long lasting loads.

2. The main nave, small towers and the north and south transepts

The bearing roof structure is a timber frame made of the first class fir. The static conception is based on a set of compound three level main triangular attic-formed timber frames with steel tie rods above the first level of tie beams. The distances of the main structural frames are unusually spacious and different from field to field: the largest about 10.43 m on the touching point of the nave and the transept. Long lasting dead load has caused excessive deformation of the not adequately designed structural elements.
The principal structure is in the statically sense a compound manifold triangular frame - truss of very clear outline. The bond beams of the roof frames (trusses) are bent and settled on the brick arches of the main nave therefore causing additional loading of the brick arches. The most damaged are the bond beams of the frame - trusses alongside gable walls. The bearings of the structure show a previous intervention done with cleats which have been slacken owing to vibration caused by shell explosions.
The wooden roof rafters are continued girders across three spans accomplished from one piece of timber. They are connected at the ridge by bolts. Some of the rafter are destroyed by the bombings, and most of rafters in the vicinity of gable walls are destroyed by rot.
The purlines are accomplished as Gerber beams across few fields, with spans ranging from 7.10 to 10.43 meters. In the vertical direction they are supported by hands that considerably lessen the static spans. The vertical deflections are in the range of 3.5 cm to 4.3 cm. The horizontal deflections developed due to insufficient stiffness of the purlines and due to the effect of the long lasting loads are in the range of 3.0 to 5.5 cm. The biggest total deflection measured is 6.98 cm which is L/111 considerably larger than permissible. Due to the value of the total deflection several supporting hands slipped out from joints. Owing to the eccentric connection of the rafters to purlines rotation of sections and longitudinal cracks occurred due to torsion.

2. Repair work done

All of the repairs were done successfully without any accidents, according to detail design and all of the damaged parts were replaced by parts of original dimensions.
The sanation of the roof structure was coordinated with the exchange of roof slates slabs. In this way the repairing of the timber structures and elements could be accomplished with the gross reduction of the load for more than 45%, which contributes, among other things, to the security in the phase of repair work execution. To protect the church's interior from atmospheric agents during the works it was necessary to erect a movable lightweight prefab protective cover construction.
The repair work of the bearing structure assembly commenced with the sanation of the main structure and continued with the exchange of bearing plates, rafters and other secondary elements.

The Dynamic Buffer Zone System

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Summary

The main intent of preserving the building envelope of vintage buildings is not to alter the environmental conditions previously experienced by the wall assembly. The technology known as the Dynamic Buffer Zone (DBZ) introduces a layer of warm, dry, pressurized air into the existing wall. The DBZ provides a high degree of containment by controlling the air pressure differentials that cause the moisture flow. The dry, pressurized air also offers opportunities for energy management when it is extended to or through the glazing, and/or combined with the building envelope ventilation air. The introduction of the DBZ between the environmental separator and the interior finish is potentially a reliable way to compensate for minor defects in the building envelope during and after construction.

Introduction

The renovation of buildings with traditional load-bearing masonry walls usually involves window and mechanical upgrades intended to pressurize and humidify the interior space.
This strategy inevitably generates environmental loads not experienced by the original enclosure. There is growing evidence to suggest that sustained air pressurization is damaging to vintage building envelopes even when mechanical pressurization is low. The risks of poor performance and damage to the building envelope are high if air leakage and vapour diffusion are not adequately controlled. One proven restoration approach introduces a layer of dry pressurized air into the existing wall assembly. This technology provides a high level of containment by controlling the air pressure differentials that cause moisture flows. The dry pressurized air layer, known as the Dynamic Buffer zone, also offers energy management opportunities when it is extended to the glazing and/or combined with the building ventilation air.

Dynamic Buffer Zone System

YBSS is uniquely experienced in the development, testing and implementation of the Dynamic Buffer Zone (DBZ) wall system in several historical buildings in Canada. These buildings include the Canada Life Building and Rogers Cantell Office Campus in Toronto, and the East Memorial Building, a major historical government edifice in Ottawa. The challenge presented

by these buildings was to develop and design a system that would protect the aged, but preserved, external cladding and at the same time allow for high humidity levels within the interior spaces. The initial proposal of the Architect and Mechanical Engineer was to insulate the inside face of the external walls. However, YBSS determined that although this proposal increased the interior building temperature it decreased the outside wall temperature, thus causing interstitial condensation within the wall as a result of moisture movements. The freezing of the condensate would inevitable result in cracking, spalling and its eventual destruction of the historic wall cladding . Any solution to this problem had to maintain the external appearance of the building.
YBSS proposed the Dynamic Buffer Zone solution. Extensive research and testing led to solution that utilized the "Dynamic Buffer Zone" (DBZ) principle. The DBZ is essentially a controlled environment around the perimeter that prevents the moisture, heat and exterior pollutants from migrating through the wall.

CONCLUSION

The main purpose of the Dynamic Buffer Zone is to ensure comfortable interior environmental conditions in a building that is undergoing a major interior retrofit in order to extend its service life. The DBZ approach was initially reserved for moisture sensitive buildings, however, there are many other benefits that can be derived from this technology. For example, the control of air movement within the interior enclosure cavities can recuperate energy losses from the building, or dissipate solar gain before it becomes a problem that the mechanical system must address. The conditioning of the wall or window cavities would reduce the need for processing the large volumes of air that are required to generate comfort. The system also ensures an effective wall renovation without modifying the architecture of the original building facade. This air management system has benefits that go beyond operating cost. A technology that controls all aspects of energy and moisture transfer within the building envelope will ultimately produce capital cost savings and an occupant comfort level that is otherwise unachievable through the conventional upgrades. This simple, yet durable, building envelope presents a fundamental component of an exceptional building design.

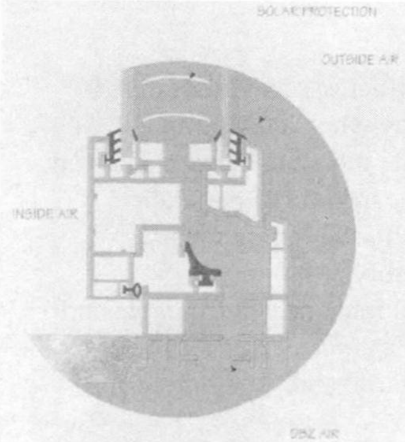

Fig. 2: Sill detail of proposed dynamic window East Memorial Building

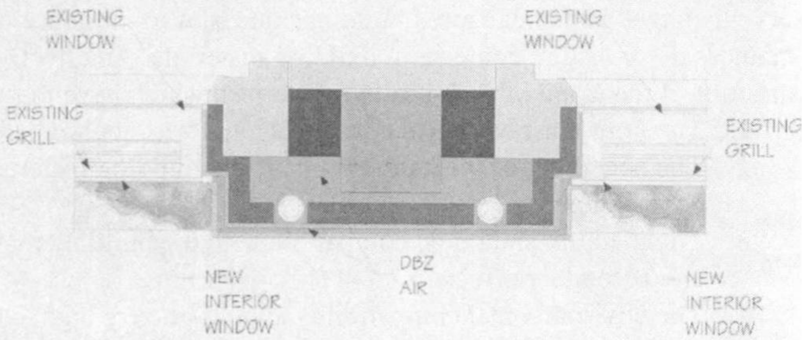

Fig. 1: Plan detail East Memorial Building

Injection Methods for Retrofitting of Moisture Damaged Constructions

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Summary

For the reconstruction of buildings damaged through rising moisture exist a great variety of methods. These methods based on mechanical, electrical, physical an chemical effects. Whether these effects generate a sufficient drying effect depends on certain factors, like moisture content, conditions of evaporation, zeta-potential and so on. Before using these methods it is necessary to determine by diagnostic analysis which method can be apply. Another possibility to increase the safety of success of reconstruction is the improvement of the effectiveness of the methods. By this way injection methods can be used for building materials with high moisture content, while a warming up - and drying process forces the penetration of the injection material in the construction.

Keywords :reconstruction, retrofitting, damage due to humidity, injection, warming up process, paraffin, wax

1. Introduction

Moisture is the main reason for deterioration of stones, bricks, constructions and buildings. It penetrates into the pores and causes by several processes. That is why when attempting to preserve damaged and endangered buildings one tries to influence the moisture balance in a way that reduces the moisture contents in building materials. An effective method to maintain construction is the using of pore-sealing materials, which have to fulfil the requirements:

1. The pore must be sealed completely and with high reliability.
2. The spreading of the medium inside the building material must be determinable and controllable.
3. The medium should be compatible with the building material and should not cause any secondary effects.
4. The environmental compatibility should be very high.

Paraffin is a medium fulfilling these demands to a high degree. It is compatible with the building materials and the environment. After a warming up process it can penetrate deeply in the construction and fill the pores of building materials completely.

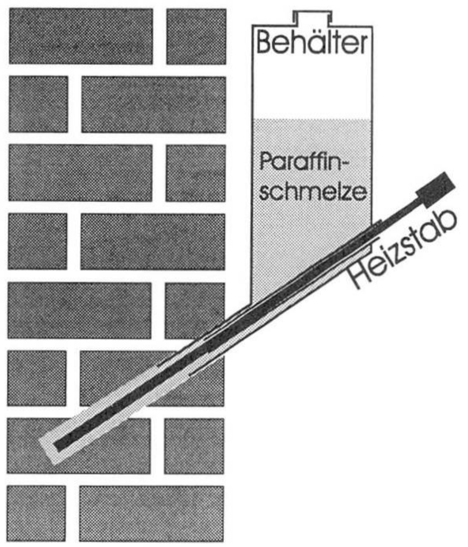

Fig. 1: Device for paraffin injection

2. Thermally Stimulated Injection of Paraffin to Build up Subsequent Moisture Barriers inside the Wall

Liquid paraffin is able to penetrate by means of capillary forces or pressure support. Therefore the treated wall has to be warmed up to a temperature above the melting-point of paraffin before, or while, the treatment takes place. Through this heating is process the moisture vaporises and the moisture-damaged masonry becomes dry. Paraffin injection is practised in the following way: heating sticks are introduced into the bore holes and after a sufficient drying and warming the paraffin filled in. Fig. 1 represents a injection methods using a heating stick inside the bore hole, which one is continuously surrounded by liquid paraffin.

3. Increasing the Durability of Porous Stones by Paraffin Impragnation

A large number of damaging processes are produced by water entering in the pores and causing several damaging reactions. By a Paraffin treatment the constructions are drying and a moisture barrier is produced. Fig. 2 shows a result of a paraffin injection in a moisture damaged wall.

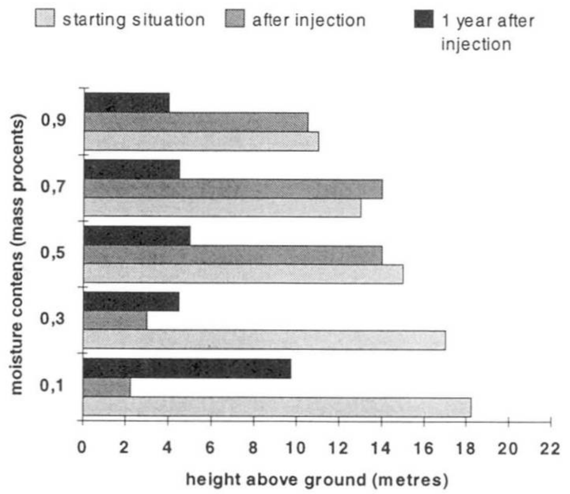

Fig.2: Moisture distribution inside a wall before and after a paraffin injection

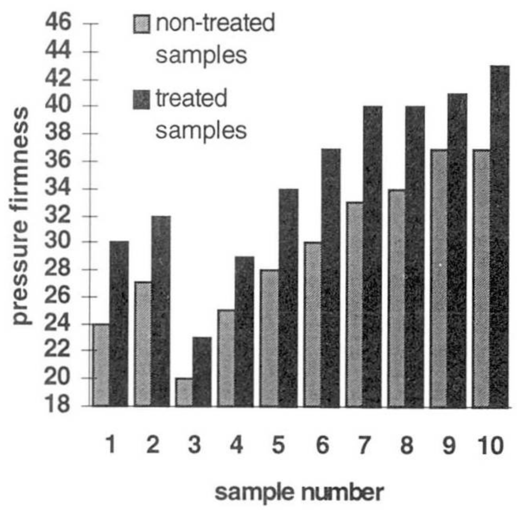

Fig. 3: Increase of compression strength

Filling the pores with paraffin also changes the mechanical properties of the treated building material. Fig. 3 demonstrates the increasing of pressure firmness of bricks by paraffin impregnation. The paraffin's ability to penetrate depends only on the temperature, therefore long treatment times are technically possible and in principle uncomplicated. Through this it is possible to determine the spreading zones of paraffin penetration and adapt them to the requirements of moisture- and stone protection.

Consolidation of the Church of St. Mihajlo near Ston

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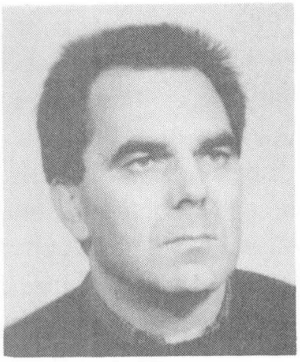

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Summary

Since the structural system is a constitutive part of a piece of architecture because it materializes the architectural idea, the significance of its form for the transmission of all foreseeable loads makes the bearing system a factor in the design of architecture, which requires multiple criteria for the consolidation and the solution requires both rational and emotional factors. The consolidation of the pre-Romanesque church of St. Mihajlo near Ston is an example of a "soft" strengthening of a small, but culturally important building. The church is one of the finest buildings of the specific "Southern Dalmatian type" of the Croatian pre-Romanesque churches: small-scale, aisleless longitudinal building, barrel-vaulted, originally with a miniature dome above the central of its three bays. Therefore, we decided to strengthen it with a minimal possible intervention, in order to prevent further deterioration caused by the seismic and atmospheric action.

Keywords: architectural heritage, structural system, "soft" strengthening, tension ring, minimal possible intervention, St. Mihajlo, Ston, Croatian pre-Romanesque

1. Basic Principles of Consolidation

The basic principle of consolidation can be defined as the need to re-establish the consolidated bearing system, making it resistant to all foreseeable events. By materializing the architectural idea, the bearing system enables its existence in the space and time. The meeting point of the two aspects of architecture, emotional and rational, is the form, because this is how the architecture is realized, both conceptually and really. Due to the spatial compatibility of the form as a determinant of architectural specificities, and of the form as a decisive principle of the structural system's behaviour in the rational domain of architecture, its consolidation becomes respectable not only as a technical problem.

Therefore, consolidation must in no way disrupt the specificities of the structural system as a whole, i.e. in order to preserve it in its fundamental principles, possible interventions must have both material and formative features of the existing bearing system. The consolidation solution should therefore give the structural system the possibility of utilizing the bearing potentials from its spatial configuration, and thus provide sufficient resistance to all relevant influences.

2. Consolidation of the Church of St. Mihajlo

The church of St. Mihajlo near Ston, very modest in dimensions (vault span about 1.7 m), is one of the finest churches of the specific "Southern Dalmatian type" of the Croatian pre-Romanesque architecture: small-scale, aisleless longitudinal building, barrel-vaulted, originally with a miniature dome encased within a rectangular turret above the central of its three bays. [1]

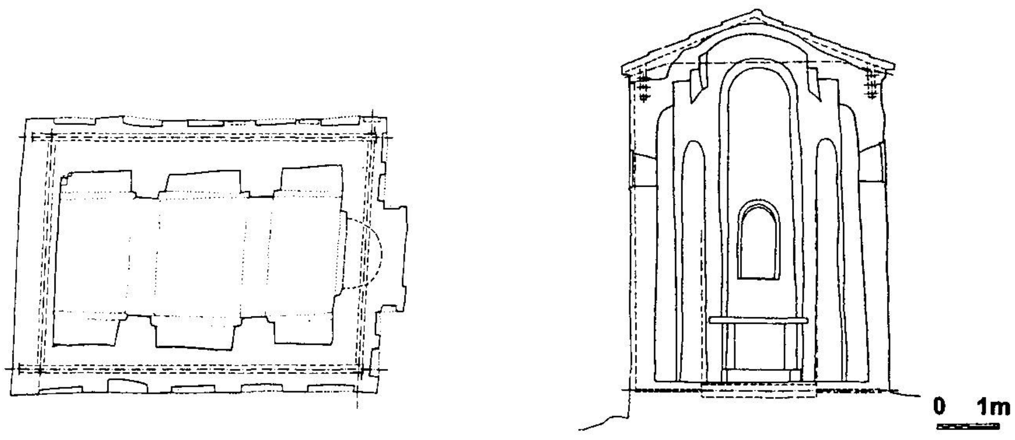

Fig. 1 Strengthening of the church of St. Mihajlo: plan (left) and section (right)

The small church, built of rough stone in lime mortar, using primitive building methods in the 10th century, proved its structural strength by resisting weathering, remodelling and strong earthquakes in the region for a millennium. Even a series of earthquakes in 1996, which caused serious damage to the ancient town of Ston (a small town north to Dubrovnik), provoked but a scant damage to the church of St. Mihajlo, situated on the top of a nearby hill - in spite of its poor condition even before the earthquakes. Therefore, we decided to strengthen it with a minimal possible intervention, in order to prevent further deterioration.

Due to their shape, the vault and the dome create important horizontal thrust on the top of the walls, unusually tall in relation to the church width. The bearing potential of the walls is used by the church builder to the full, nearly to the endurance limit. Since the horizontal thrust of the vault and the dome on a deteriorated wall seem to make a jeopardy to its stability, a horizontal tension ring consisting of iron bars placed in lime mortar, is inserted at appropriate height, as a possible and suitable strengthening of the existing system. The same approach is applied in strengthening the church foundations by adding two transversal connections consisting of iron ties, protected by concrete coating, between the north and the south walls, enhancing their mutual action, the more so since the stone base on which the church lies is gently inclined to the south. Because of very valuable wall paintings the wall consolidation will be done by careful grouting of lime mix under low pressure as the basic ingredient in order to comply with the original binder.

The purpose of all the interventions is to provide an integral bearing system for the church, because this is a crucial factor in transferring all relevant loads.

3. References

[1] Goss, V. P., *Pre-Romanesque Architecture in Croatia*, A. Mutnjaković, Zagreb, 1996

Preservation of the State Oswiecim-Brzezinka Museum

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Summary

There are various buildings and engineering structures on the site of the former Nazi biggest concentration camp Oswiecim-Brzezinka (Auschwitz-Birkenau). These structures gradually undergo deterioration and as far as technology is concerned they are being worn out as a result of use, exploitation, natural environment influence and physic - chemical processes which take place inside materials. The article presents actions which are being conducted in order to maintain the museum structures in possibly good technical condition.

1. Characteristics of structures, their technical condition, causes of deterioration and damage

Brzezinka - on 175 ha of Brzezinka Museum 500 structures are situated. All structures were built during the war and had temporary character. Predominant part of all the structures is wooden and brick. Fig.1 shows cross section of a habitable building made by timber and brick.

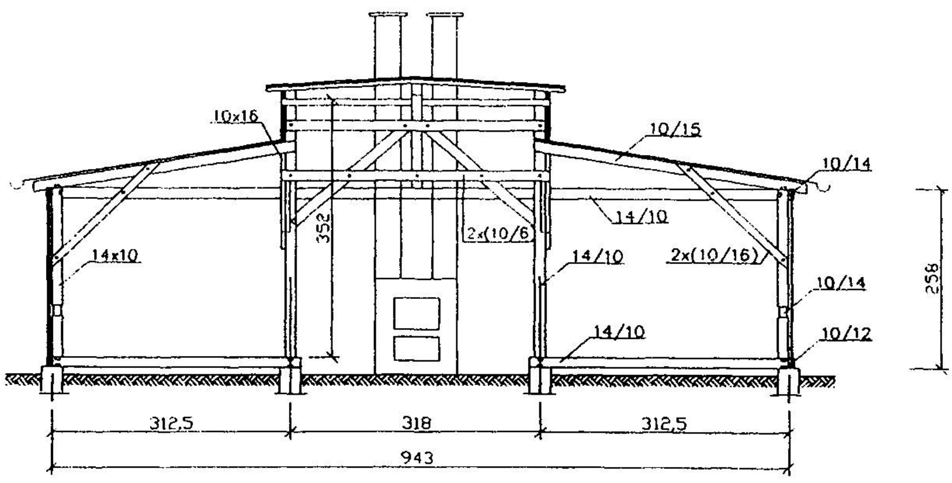

Fig.1 The cross section of a habitable timber building

The technical condition of the Museum buildings and structures in Brzezinka is differentiated. Most of them are in bad shape. Technically safe and in satisfactory condition are only a few of them, well maintained, repaired and renovated in recent years. They are: the Death Gate, the watchtowers, some wooden barracks. Parts of structures like chimneys, remnants of barracks, pieces of fences, engineering structures are in disastrous condition and irreversible destruction.

The basic cause of most observed damages and deterioration of structures situated in the museum area at Brzezinka is intense environment influence, i.e. precipitation, temperature and humidity changes, frost, insolation, and wind which cause erosion of materials and structures of which they are made as well as biological erosion of wooden elements, walls, mortars and concrete.

Oswiecim - The Museum in Oswiecim covers the area of about 15 ha where over 60 structures are situated. They are mainly two storey buildings without cellars which are made of brick with reinforced concrete floors and also timber or concrete loft floors. Buildings were built in 1918 and they were initially designed to serve as caserns. Some of them were built and rebuilt (the first floor was added) during the second world war.

The buildings in Oswiecim Museum are in far better technical condition than those in Brzezinka. Most of them undergo current repairs, are modernised and are relatively well maintained. However, in recent years also in these buildings one can notice growing damages, flabs and construction defects, particularly in less durable elements like roof coverings, wooden constructions, plasters, floors, joinery, etc.

2. Repair and modernisation works

In order to maintain the museum structure, repairs and modernisation, conservatory and protective works are being conducted. In recent years the following works have been accomplished on the site of Oswiecim Museum:

- Capital repairs of most roofs.
- A program of heating the museum consisting in making a heat distribution networks both inner and outer. At the same time the loft ceiling insulation was carried out, mainly with the use of mineral wool which was placed among wooden ceiling beams or in case of ceramic floors it was placed under wooden loft floors.
- Providing some of the buildings with air conditioning system. Air conditioning was introduced in some museum blocks which are to house archives and remembrances collections.
- Conservation of face brick façades by means of cleaning, filling, jointing and covering with transparent silicon hydrophobic preparations.

Part of the structures is being reproduced and reconstructed on the basis of archival documents or listing of similar ones still existing - it concerns for example wooden towers and guard platforms, some of wooden barracks and others.

It is separate issue to maintain the structures situated at Brzezinka on the area of 175 ha. Preservation of all of the structures there was not possible in the past and it will probably be not possible in the future. It should be decided which of them are to be maintained and which should be catalogued and liquidated.

Repairs of Coal Processing Buildings under Continuing Production

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Summary

In the paper the technology of repair of corrosion damaged, reinforced concrete floor of level +6.00 m in coal processing building. Repair of the floor was carried out under continuing production; no breaks in production occurred. The range and degree of corrosion damage, the methods of reinforcement of load-bearing structure and sealing of tile floor were described.

1. Introduction

Production floors in coal processing building are endangered to intensive mechanical influence (static and dynamic ones), chemical and physical ones, related to compound dry and wet coal processing enrichment. This causes that floor undergoes damage in shorter time reducing safety condition of construction. Then, the unavoidable repair, for the sake of large areas of floor, numerous mechanical devices located on it (screens, crushers, conveyers, channels and troughs) as well as continuous affect of brine make very difficult technical problem. In the paper an example solution of such problem is presented for highly damaged reinforced concrete floor of + 6.00 m level taking into account additional requirements of the user:

- devices mounted on the floor must be kept in operation all the time and longer breaks in operation are not allowed,
- one may assume weekly stoppage of these devices from Saturday up to Monday, that provides 48 hours breaks,
- of 4 railway loading flights situated over the floor - only one could be stopped - the extreme one,
- it shall be possible to carry out the repair work beyond the reach of devices within the whole week, however assuming that the greater part of the floor is continuously flooded with brine - only disassembly work could be accepted.

2. Description of floor construction and its damage

Intensely operated floor for 30 years has monolithic, reinforced concrete, slab-rib structure supported with skeleton studs of coal processing building structure. Length of the floor is 9 x 6.0 = 54 m and width is 4 x 6.0 = 24 m. Due to difficult operation conditions and first of all continuous influence of aggressive washing water on concrete and steel, the floor has various degree of corrosion damage on most part of area. The most severe damage occur where brine leaks through the floor slab. Aggressive water flowing down on beam surface make sulfate and magnesium corrosion as well as pitting of reinforced steel. It had been observed that some parts of beams have pit damage of concrete structure in area of leakage, sometime in form of through openings.. On upper part of the floor the numerous defects of concrete floor were found.

3. The repair technology

Authors suggested to unload the floor with stay ropes and horizontal steel beams. Fig.1 presents the idea of unloading. The feature of this solution was that instead of main beams the secondary beams or floor slab were underslung to traverse beam. This enabled the possibility of free access to parts being repaired. After unloading, the corroded weak concrete was removed without removing reinforcement steel and surface was cleaned using wet sand blasting.

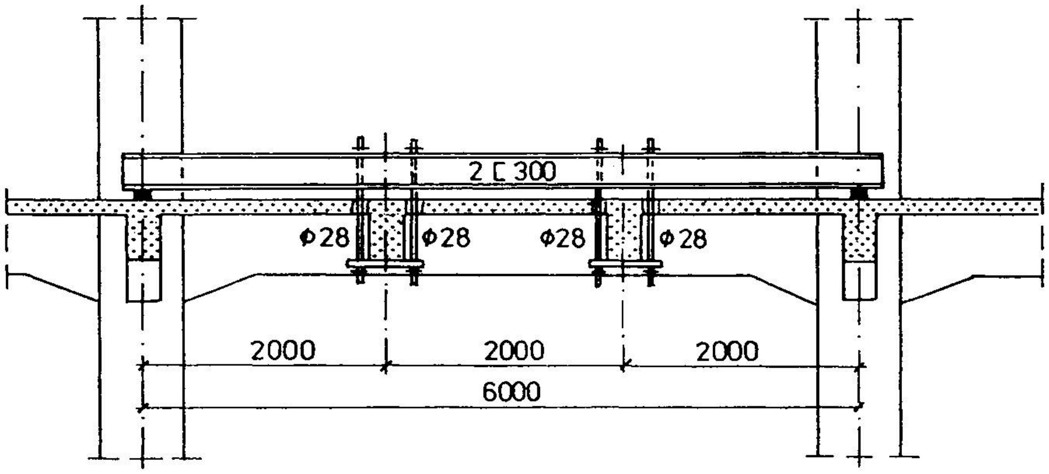

Fig.1 The method of suspending of secondary beams and unloading the stay ropes

After boarding was made, the removed part of concrete was filled with fast-setting cement. When concrete achieved required strength the boarding and unloading steel construction were dismantled and openings made for strings of suspensions were sealed. The next stage included repairing of damaged floor and sealing of construction to eliminate the leakage. Due to necessity to continue the production and short break of wet processes (48 hours in week) application of standard solutions of watertight insulation could not be taken into account.

4. Conclusion

The example presented in the paper proves that having modern insulation and renovation materials available , it is possible to repair the reinforced concrete industrial structures, strongly damaged with corrosion, under continuing production.

Stress Analysis in Masonry Arch Structures

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Summary:

Masonry arch structures are often encountered in old historical buildings as carrying elements for the vertical loading. Damage to these structures occurs due to the abutment settlement and their load carrying ability is very difficult to define in that stage. An acceptable computational model has to include cracking of the masonry units in direct tension, splitting of the units due to mortar dilatancy at high values of normal stress and sliding in contacts at low values of normal stress between masonry units. In this work we have tried to analyze range of the applicability of common numerical methods for analysis of these complex structures. The observed parameters were: model definition, mechanical constants of constitutive materials, changes in loading and changes in boundary conditions. Conclusions about various analysis methods, required assumptions, modifications, model sensitivity and suggestions for further study are outlined.

Keywords: masonry arch, analytical models, linear and nonlinear, load, displacements, stresses.

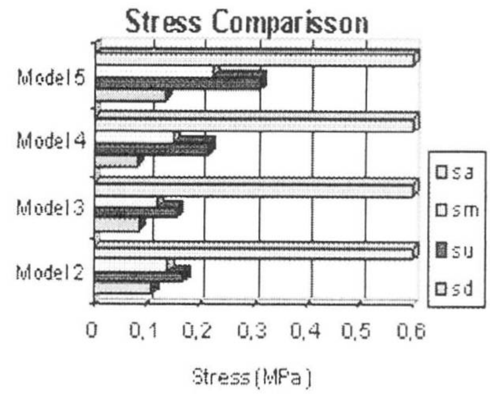
Analyzed models

There are several structural shapes common in the masonry arch systems: half circle, elliptical, oval and segment arch. The oval arch was selected for the studies presented in this work on the ground of the preliminary analysis and its frequent use.

Fig.1 Calculated stress distribution

In the work we have tried to check various analysis methods and their applicability for everyday use.
When the loading conditions are normal, such as increase in live load and limited vertical settlements, generally available mathematical and elastic numerical models give results of the same value. The results were not so sensitive to variations in the input parameters within one model.
Among the elastic FEM models the ones with plane shell FEM (shell elements) simulated the stress change within the cross section and were therefore closer to the real stress distribution.

Fig. 2 Comparison of calculated stresses at arch crown



The calculated stresses for Model 1 to 4 were of the same order and occurred at the same section. Between linear elastic and nonlinear models stresses occurred at the same place but their intensity varied for about 30%. Observed general trend was that as the model became more refined, stresses were growing bigger for the same loading. Sensitivity of the response results, within one model type,due to changes in input data in the range of customary values (number and size of FEM, variations in material characteristics) can be neglected.

Nonlinear model has shown far greater sensitivity. The model's sensitivity was in direct correlation with its complexity (which can partly explain the increase in the stresses in the same section and for the same loading for Model 5).

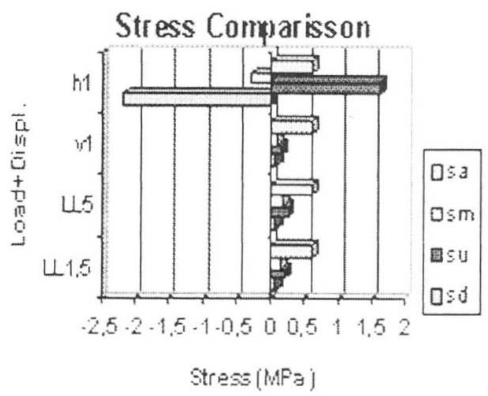


Fig 3. Comparison of the stress values caused by changes in live load (1,5 and 5 kN/m2) and abutment displacements (1cm)

Changes in live load do not endanger the system stability and present at the maximum 20% of the total load. Inherent carrying reserves are significant and for most live loads that occur in the buildings total stresses remain well under the allowable. The increase in live load for 300% has caused the increase in total stress for about 20%. They remained well under the allowed values. Massive arch structure is much more sensitive to the changes of boundary conditions.

Vertical abutment settlement of 1/300 can be still considered as tolerable and inside the elastic range, while the stresses did not change their sign and were under the allowed values. Stresses caused by horizontal displacement of 1cm are 26 times greater than the stresses caused by vertical settlement of the same abutment and for the same amount. By comparing the horizontal and vertical abutment settlements, horizontal settlements are very undesirable and should be prevented.

Nonlinear analysis which can include cracking of the masonry units in direct tension, splitting of the units due to mortar dilatancy at high values of normal stress and sliding in contacts at low values of normal stress between masonry units (such as Model 5) must be used. Reasonable insight in the true structural behavior by changes in the boundary conditions can be presented only through the nonlinear analysis. Linear analysis can give us only a hint of the structural behavior at that stage.

While the linear elastic models are relatively easy to use and give reliable results for standard material values, the present stage of the nonlinear modeling techniques requires a broad inside view in the material data, numerical methods and are not applicable for everyday use.