YU ISSN 0543-0798 UDK: 06.055.2:62-03+620.1+624.001.5(497.1)=861

2013.
GODINA
LVI

GRAĐEVINSKI MATERIJALI I KONSTRUKCIJE

2

BUILDING MATERIALS AND STRUCTURES

ČASOPIS ZA ISTRAŽIVANJA U OBLASTI MATERIJALA I KONSTRUKCIJA
JOURNAL FOR RESEARCH OF MATERIALS AND STRUCTURES

DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA SRBIJE
SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA

Odlukom Skupštine ***Društva za ispitivanje materijala i konstrukcija***, održane 19. aprila 2011. godine u Beogradu, promenjeno je ime časopisa **Materijali i konstrukcije** i od sada će se časopis publikovati pod imenom **Građevinski materijali i konstrukcije**.

According to the decision of the Assembly of the ***Society for Testing Materials and Structures***, at the meeting held on 19 April 2011 in Belgrade the name of the Journal Materijali i konstrukcije (Materials and Structures) is changed into **Building Materials and Structures**.

Professor Radomir Folic
Editor-in-Chief

DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA SRBIJE
SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA

GRAĐEVINSKI MATERIJALI I KONSTRUKCIJE

BUILDING MATERIALS AND STRUCTURES

ČASOPIS ZA ISTRAŽIVANJA U OBLASTI MATERIJALA I KONSTRUKCIJA
JOURNAL FOR RESEARCH IN THE FIELD OF MATERIALS AND STRUCTURES

INTERNATIONAL EDITORIAL BOARD

PUBLISHER

Society for Materials and Structures Testing of Serbia, 11000 Belgrade, Kneza Milosa 9
Telephone: 381 11/3242-589; e-mail:dimk@ptt.rs, veb sajt: www.dimk.rs

REVIEWERS: All papers were reviewed
COVER: Crveni magacin u Pančevu - spomenik kulture značajne arhitektonske i istorijske vrednosti
Red storage in Pancevo - Cultural monument of significant architectural and historical value

Financial supports: Ministry of Scientific and Technological Development of the Republic of Serbia

YU ISSN 0543-0798 GODINA LVI - 2013.

DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA SRBIJE
SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA

GRAĐEVINSKI MATERIJALI I KONSTRUKCIJE

BUILDING MATERIALS AND STRUCTURES

ČASOPIS ZA ISTRAŽIVANJA U OBLASTI MATERIJALA I KONSTRUKCIJA
JOURNAL FOR RESEARCH IN THE FIELD OF MATERIALS AND STRUCTURES

SADRŽAJ

CONTENTS

CIP - Каталогизација у публикацији
Народна библиотека Србије, Београд

620.1

GRAĐEVINSKI materijali i konstrukcije : časopis za istraživanja u oblasti materijala i konstrukcija = Building Materials and Structures : journal for research of materials and structures / editor-in-chief Radomir Folić. - God. 54, br. 1 (2011)- . - Beograd (Kneza Miloša 9) : Društvo za ispitivanje i istraživanje materijala i konstrukcija Srbije, 2011- (Novi Beograd : Hektor print). - 30 cm

Tromesečno. - Je nastavak: Materijali i konstrukcije = ISSN 0543-0798
ISSN 2217-8139 = Građevinski materijali i konstrukcije
COBISS.SR-ID 188695820

SEIZMIČKO POJAČAVANJE ISTORIJSKIH ZIDANIH ZIDOVA KOMPOZITIMA – EKSPERIMENTALNA ISTRAŽIVANJA

SEISMIC STRENGTHENING OF HISTORIC MASONRY WALLS WITH COMPOSITES: AN EXPERIMENTAL STUDY

Miha TOMAŽEVIČ
Matija GAMS
Thierry BERSET

ORIGINALNI NAUČNI RAD
ORIGINAL SCIENTIFIC PAPER
UDK: = 861

1 INTRODUCTION

Although masonry, being the main building material for centuries, has been replaced in great part by concrete and steel, masonry buildings still represent the largest part of the building stock in most European countries. Depending on local resources and tradition of construction, a great variety of old masonry types exist, worldwide, the basic building materials being local stone and clay brick laid in lime mortar.

Stone masonry walls in Slovenia are typically made of rubble or river-bed stone, lime-stone or slate, built in two outer layers of irregularly sized bigger stones, with an inner infill of smaller pieces of stone, in poor mud mortar with a little lime. In the city centres and towns, the walls are made of relatively compact mix of stone, brick and mortar, with no distinct separation between the individual layers of the walls. Regularly cut, or partly cut stone is rarely used. Connecting stones are also rare.

Typically, stone-masonry houses are 3−4 stories high in the cities and towns, whereas their height is limited to 2 stories in rural areas. Structural layout is usually adequate. The distribution of walls is uniform in both orthogonal directions, and thus, due to the thickness of load bearing and cross-walls and relatively small rooms, the wall/floor area ratio is very large, in many cases exceeding 10 %. Floor structures and lintels are traditionally wooden, without any wall-ties provided to connect the walls. Wooden floors are sometimes replaced by brick vaults above cellars, staircases and corridors. Roof structures are wooden and covered with ceramic tiles, sometimes laid in mortar. As a rule, the buildings are built without any foundation, whereas foundation walls are of poorer quality than the walls of the structure above the ground level.

Traditionally, brick has been used in the north-eastern part of the country, which belongs to the non-seismic Pannonic plane. It was not but before the mid-19th century that clay brick replaced stone in the cities, where residential and public brick masonry buildings are generally higher than stone-masonry buildings in historic city-centres. Depending on the period of construction, the height is limited to 4−5 stories before the First World War, to 6−7 stories between both world wars, and attains even 12 stories in the fifties of the 20th century. All buildings are built in plain masonry structural systems. Floor structures vary from wooden or a combination of timber and steel beams, to monolithic reinforced concrete slabs or prefabricated clay or concrete beam systems. As regards structural layout, residential buildings are better than public, where mixed systems can be found with r.c. columns not able to withstand horizontal loads replacing the walls.

Brick masonry buildings were initially built without any seismic provisions, but after the earthquake in 1895, which struck Ljubljana, the country's capital which at that time belonged to Austro-Hungarian empire, requirements to improve the seismic resistance of buildings have been introduced into the building code, such as requirements regarding the quality of masonry, structural configuration of buildings and the tying of the walls (Vidrih 1995). The first contemporary seismic code in Slovenia (at that time Yugoslavia) was introduced in 1963.

Miha Tomaževič, Professor
Slovenian National Building and Civil Engineering Institute, Dimičeva 12, 1000 Ljubljana, Slovenia
miha.tomazevic@zag.si
Matija Gams, Researcher
Slovenian National Building and Civil Engineering Institute, Dimičeva 12, 1000 Ljubljana, Slovenia
matija.gams@zag.si
Thierry Berset, Sika Services AG, Switzerland

As the earthquakes prove, masonry buildings belonging to historical architectural cultural heritage are vulnerable to earthquakes. The majority of damage and buildings' collapse which occurred during seismic events in the last several decades was the consequence of inadequate seismic resistance of old masonry buildings. Seismic vulnerability studies indicate that, in general, not only the structural system and configuration, but also the actual resistance of more recent existing masonry buildings fails to meet the demand of today's codes (Lutman 2010). Consequently, seismic upgrading of such buildings is needed should the buildings remain in use.

Among different methods of strengthening of stone masonry walls, injecting the walls with cementitious grouts proved to be most efficient. Masonry friendly grouts have been developed where cement is replaced by lime or inert materials to be suitable for strengthening the wall of historic importance. The coating of stone masonry walls with reinforced cement coating has been also used. Although sometimes used, repointing in the case of the stone masonry did not prove to be as successful. In the case of brick masonry, various methods of strengthening, based on the use of traditional materials, such as cement and reinforcing steel (reinforced cement/concrete/shotcrete coating, repointing, injecting) have been also developed. The efficiency of the proposed methods for strengthening both, stone and brick masonry walls has been already experimentally verified. These methods have been also successfully applied to buildings damaged after the earthquakes.

However, in the last couple of decades, synthetic materials, such as carbon (CFRP) or glass fibre reinforced polymers (GFRP) are replacing the traditional reinforcing materials. Various techniques of strengthening the masonry walls with polymers have been developed and their efficiency tested in the laboratories (e.g. Schwegler 1994, Triantafillou 1997, ElGawady 2006, Konthesingha 2010, Tomaževič 2011). As a consequence of great differences in mechanical properties of fibre reinforced polymers and brick masonry (compressive and tensile strength, elastic and shear modulus), the efficiency of application of various methods on the resistance and displacement capacity of masonry walls is sometimes dubious.

Much interest exists for using composite materials for strengthening both types of masonry, because the application is time-effective and relatively clean, and the costs of polymers, especially GFRP, is dropping. In order to investigate the efficiency of some recently developedmethods and prepare recommendations for their practical use, a large experimental campaign has been launched also at Slovenian National Building and Civil Engineering Institute in Ljubljana, Slovenia. Test results will be presented and discussed in this paper.

2 PROGRAM OF TESTING, TYPES OF STRENGTHENING AND MATERIALS USED

To investigate the efficiency of different strengthening types, 12 stone masonry walls with dimensions 1500/1000/500 mm (height/length/thickness), and 28 brick walls with dimensions 1500/1000/250 mm (height/length/thickness) have been built in the laboratory. The walls have been built on reinforced concrete foundation blocks, and had reinforced concrete bond-beams on the top. The dimensions of the walls are shown in Figure 1.

Stone masonry walls were built as typical Slovenian historic rural three-wythe stone masonry. Coarse lime stone (compressive strength about 220 MPa), delivered from a demolished stone-masonry house in the region of Posočje has been used for the construction of 10 wall specimens. The stones, up to 30 cm in size, have been laid in lime mortar with small amount of cement added to accelerate hardening. The compressive strength of mortar, consisting of river bed sand (maximum aggregate size 4 mm), hydrated lime and cement in volumetric proportion 8:1:0.5, was 3.3 MPa (c.o.v. = 0.35).

To simulate historic brick masonry, normal format (250/125/60 mm) bricks, available on the market, with nominal compressive strength 20 MPa (actual 29 MPa) and cement-lime-sand mortar, with a small quantity of cement added to accelerate hardening (volumetric proportion 0.25:1:8, compressive strength 1.14 MPa) have been used.

To determine the compressive strength and modulus of elasticity of masonry, two walls of each type have been tested by compression in accordance with European standard EN 1052-1. The compressive strength,, (f_c) of stone and brick masonry was 1.26 MPa and 4.1 MPa, respectively. The elastic modulus (E) of stone and brick masonry was 470 MPa and 1094 MPa, respectively.

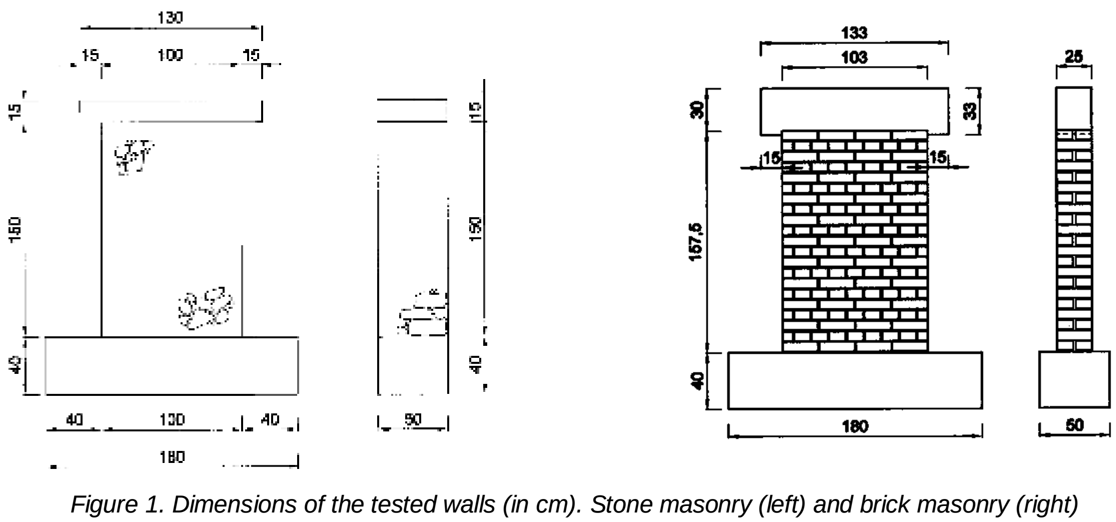


Figure 1. Dimensions of the tested walls (in cm). Stone masonry (left) and brick masonry (right)

The remaining walls were tested as vertical cantilevers, subjected to constant pre-loading, simulating the gravity loads which induced compressive stresses (σ) in the horizontal section of the walls, and programmed cyclic displacements, simulating the horizontal seismic loading, acting at the level of the bond-beams in the plane of the walls. The level of compressive stress (preloading ratio σ/f_c) of stone and brick masonry walls was 0.26, and 0.3, respectively.

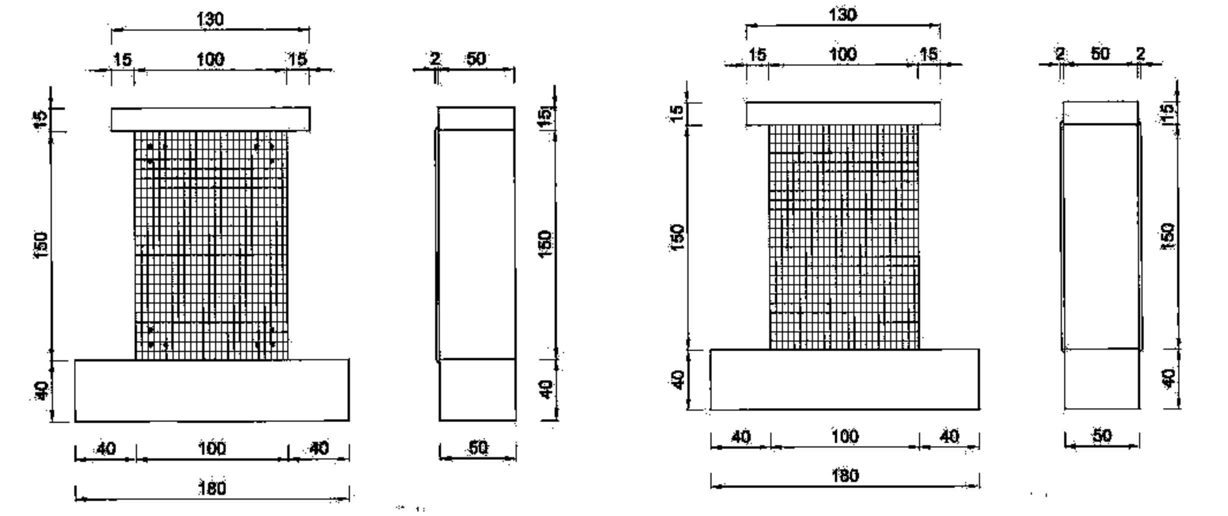
For each type of masonry, two walls were tested for reference (i.e. without any strengthening), and the rest were strengthened by different types of strengthening solutions. Before applying the strengthening solution, some walls have been tested up to the maximum resistance and then repaired, strengthened and retested to simulate the application of strengthening techniques to previously damaged walls. However, the effect of some strengthening solutions was investigated without previously damaging the walls.

Four different types of strengthening were used in the case of stone masonry walls. In the case of strengthening type 1, the coating consisted of vertical GFRP grid as reinforcement and 15−20 mm thick fibre reinforced cementitious mortar as a matrix. The coating, anchored to the wall in the corners, was placed on one side of the wall only. In the case of strengthening type 2, the same coating was placed on both sides, however without being anchored to the wall. In the case of strengthening type 3, the grid was placed diagonally on both sides and the coating was anchored to the wall in the corners. In the case of strengthening type 4, 30 cm wide GFRP fabric strips have been used as reinforcement, laid in epoxy resin matrix. They were placed vertically and diagonally on both sides of the wall and anchored to the wall in the corners. Before coating, the surface of the walls has been levelled with fibre reinforced cementitious mortar. The coating has not been anchored into the r.c. foundation blocks or r.c. bond beams on the top of the walls. Strengthening types are presented in Table 1 and schematically shown in Figures 2 and 3.

Table 1: Types of strengthening

*Masonry**	*Strengthening type*	*Sides*	*Anchors*	*Reinforcement*	*Matrix*	*Orientation of reinforcement*	*No. of walls tested*	*Previously damaged*	*Undamaged*
S	*1*	*1*	*4x3 at corners*	*GFRP grid*	*Mortar*	*Vertical: over entire surface*	*2*	*2*	*0*
S	*2*	*2*	*-*	*GFRP grid*	*Mortar*	*Vertical: over entire surface*	*2*	*2*	*0*
S	*3*	*2*	*4x3 at corners*	*GFRP grid*	*Mortar*	*Diagonal: over entire surface*	*2*	*0*	*2*
S	*4*	*2*	*4x4 at corners*	*GFRP fabric*	*Epoxy*	*Diagonal and vertical 30 cm strips*	*2*	*0*	*2*
B	*5*	*1*	*-*	*GFRP grid*	*Mortar*	*Vertical: over entire surface*	*2*	*2*	*0*
B	*6a*	*2*	*-*	*GFRP grid*	*Mortar*	*Vertical: over entire surface*	*1*	*0*	*1*
B	*6b*	*2*	*5 anchors*	*GFRP grid*	*Mortar*	*Vertical: over entire surface*	*2*	*1*	*1*
B	*6c*	*2*	*8 anchors*	*GFRP grid*	*Mortar*	*Vertical: over entire surface*	*2*	*1*	*1*
B	*6d*	*2*	*13 anchors*	*GFRP grid*	*Mortar*	*Vertical: over entire surface*	*1*	*0*	*1*
B	*7*	*2*	*4x4 at corners*	*GFRP grid*	*Mortar*	*Diagonal: over entire surface with 25 cm vertical strips*	*2*	*2*	*0*
B	*8*	*1*	*4x4 at corners*	*GFRP fabric*	*Epoxy*	*Diagonal and vertical 30 cm strips*	*2*	*0*	*2*
B	*9*	*2*	*4x4 at corners*	*GFRP fabric*	*Epoxy*	*Diagonal and vertical 30 cm strips*	*2*	*0*	*2*
B	*10*	*2*	*4x4 at corners*	*GFRP fabric*	*Epoxy*	*Diagonal and vertical 30 cm strips*	*2*	*0*	*2*
B	*11*	*2*	*-*	*CFRP plates*	*Epoxy*	*Diagonal and vertical plates*	*2*	*0*	*2*
B	*12*	*2*	*8 anchors*	*GFRP fabric*	*Epoxy*	*Horizontal: over entire surface*	*2*	*2*	*0*
B	*13*	*2*	*-*	*GFRP grid*	*Mortar*	*Diagonal: over entire surface*	*2*	*2*	*0*
B	*14*	*2*	*8 anchors*	*GFRP grid*	*Mortar (thin)*	*Vertical: over entire surface*	*2*	*2*	*0*

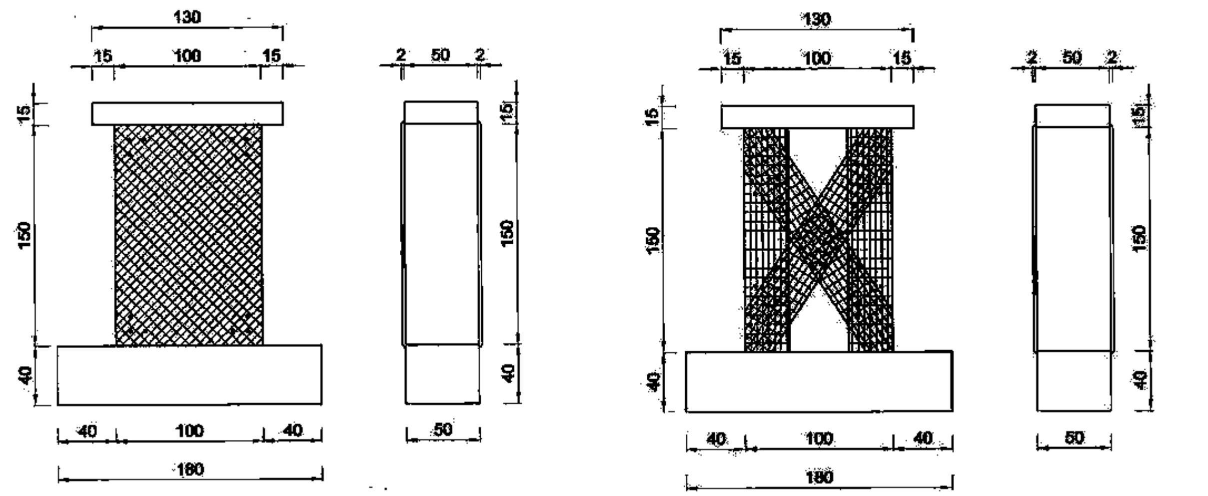
** S – stone masonry, B – brick masonry*



Strengthening type 1: vertical GFRP grid placed on one side of the wall, anchored in corners

Strengthening type 2: vertical GFRP grid placed on both sides of the wall, no anchors

Figure 2. Schematic presentation of stone masonry strengthening types 1 and 2 (dimensions in cm)

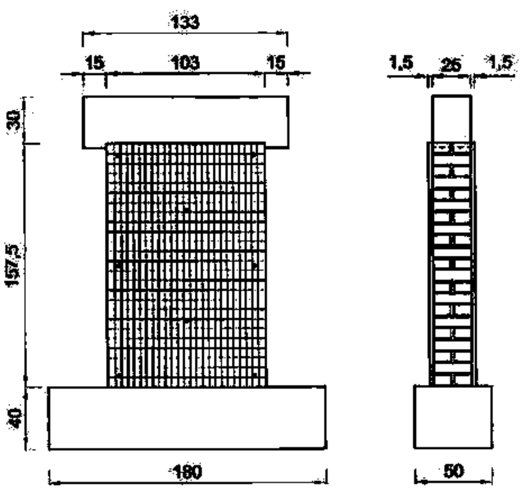


Strengthening type 3: diagonal GFRP grid placed on both sides of the wall, anchored in corners

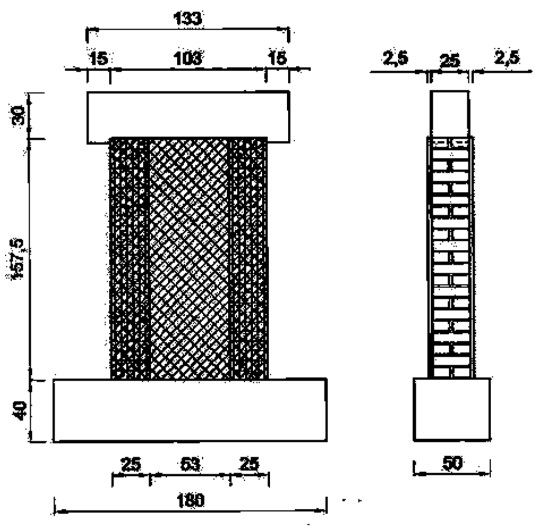
Strengthening type 4: diagonal and vertical GFRP fabric strips on both sides of the wall, anchored

Figure 3. Schematic presentation of stone masonry strengthening types 3 and 4 (dimensions in cm)

The brick walls have been also strengthened by different types of coating, namely GFRP grid laid in cementitious, 15 and/or 25 mm thick mortar (strengthening types 5–7, 13, and 14), GFRP (types 8 and 12) or CFRP uni-directional fabrics laid in 2 mm thick epoxy matrix (types 10 and 11), or by CFRP strips (plates), glued to the masonry and anchored into the foundation blocks and bond-beams with epoxy resin (strengthening type 11). The number of anchors which connected the coating to the masonry also varied. Typical characteristics of the tested strengthening types are given in Table 1, whereas the layouts of representative strengthening types are schematically presented in Figures 4 and 5.

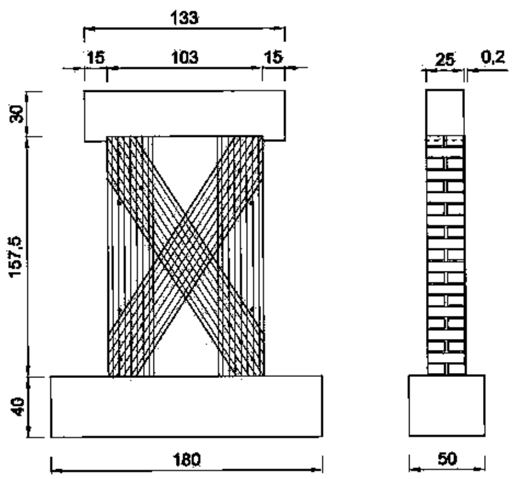


Strengthening type 6c: vertically placed GFRP grid, 8 anchors

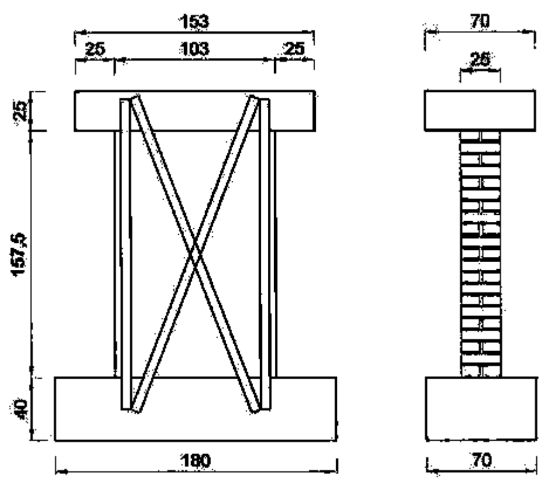


Strengthening type 7: diagonally placed GFRP grid reinforced with vertical strips

Figure 4. Schematic presentation of brick masonry strengthening types 6c and 7 (dimensions in cm)



Strengthening type 8–10: diagonal and vertical GFRP and CFRP fabric strips



Strengthening type 11: diagonal and vertical CFRP plates/strips

Figure 5. Schematic presentation of brick masonry strengthening types 8-11(dimensions in cm)

Commercially available materials have been used to strengthen the walls. Bi-directional glass fibre grid, SikaWrap®-350G, is used in combination with Sika® MonoTop®-722 Mur cementitious mortar. SikaWrap®-350G grid with approximately 17/15 mm windows (nominal 15.7/10.1 mm) is a glass fibre grid with an alkali resistant coating. The tensile strength, measured on virgin filament, is 3.4 GPa. The ultimate load in longitudinal direction is 77 kN/m; in transverse direction, 76 kN/m tensile stiffness expressed as the load at 1 % elongation are 20 kN/m and 25 kN/m in the longitudinal and transverse direction, respectively. Elongation at rupture is 3 %.

Sika® MonoTop®-722 Mur is a fibre reinforced mortar with reactive pozzolanic components, selected aggregates and special additives. The compressive strength at 28 days, tested in accordance with EN 196-1, is 22 MPa. Flexural strength is 7 MPa, and modulus of elasticity, tested in accordance with EN 13412, is 8 GPa.

Glass fibre fabric, SikaWrap®-430G, is uni-directional woven glass fibre fabric, which is applied to the walls using SikaDur-330 epoxy resin. The fabric is 0.17 mm thick and has a tensile modulus of elasticity 76 GPa with 2.8 % strain at rupture. The tensile strength of fibres is 2.3 GPa. Carbon fibre fabric, SikaWrap®-230C, is uni-directional woven carbon fibre fabric, which is also applied to the walls using SikaDur-330 epoxy resin matrix. The fabric is 0.131 mm thick and has tensile modulus of elasticity 238 GPa with 1.8% elongation at rupture. The tensile strength of fibres is 4.3 GPa.

Sikadur®-330 is a 2-component epoxy impregnation resin used for application of glass or carbon fabric reinforcement to the masonry. Its properties measured according to DIN 53455 at 7 days are: tensile strength 30 MPa, flexural elastic modulus 3.8 GPa, tensile elastic modulus 4.5 GPa and elongation at rupture is 0.9 %.

CFRP plates (strips), Carbodur® S 512, are 50/1.2 mm (width/thickness) strips, applied to the walls with a 2-component epoxy adhesive, SikaDur®-30. The modulus of elasticity of CFRP strips is 165 GPa, the tensile strength is 2.8 GPa, and elongation at rupture 1.7 %.

The anchors, SikaWrap® Anchor C, are carbon fibre strings, contained in elastic gauze wrapping, 10 mm in diameter. Before being placed into the holes drilled through the wall, the strings are cut into the pieces, about 200 mm longer than the thickness of the wall, the wrapping is removed and the fibres are soaked in epoxy resin. The anchor is inserted into the hole, filled with epoxy resin, by means of a simple tool. The fibres are spread in the form of a circular fan and glued on the prepared surface of the wall with epoxy resin at the protruding ends on each surface of the wall.

3 TESTING PROCEDURE AND TEST RESULTS

3.1 Testing procedure and instrumentation

To study the efficiency of the proposed strengthening solutions, the walls have been tested as vertical cantilevers, subjected to constant vertical and cyclic in-plane lateral loading, induced by hydraulic actuators acting on the bond beam on the top of the walls. Reinforced concrete foundation blocks on which the walls were constructed, were fixed to the strong floor by means of bolts. Test set-up consisted of a steel testing frame and hydraulic actuators, fixed to the frame in order to simulate constant gravity loads and cyclic lateral in-plane seismic loads. Compressive stresses in the walls' horizontal section, equal to 26 % or 30 % of the compressive strength of masonry for stone and brick masonry, respectively, were kept constant during the tests. In-plane lateral loads were simulated in the form of cyclic horizontal displacements, imposed by means of programmable hydraulic actuator acting at the mid-height level of the bond-beam. The displacement amplitudes were step-wise increased up until the collapse of the walls. The loading was repeated three times to study the resistance and stiffness degradation at each displacement amplitude. All walls were instrumented with load cells and displacement transducers (LVDT-s) to measure relevant forces and displacements. Testing arrangement and instrumentation can be seen in Figure 4.

In the case of brick masonry, the rocking of the unreinforced masonry walls was prevented with a system of prestressed vertical steel ties placed at the ends of walls. 10 % of the total vertical force, acting on the wall, was induced in the ties on each side of the specimen, manually adjusted after each cycle of loading when necessary. Test set-up consisted of a steel testing frame and hydraulic actuators, fixed to the frame in order to simulate constant gravity loads and cyclic lateral in-plane seismic loads.

3.2 Behaviour and failure mechanisms

Stone masonry

Shear governed the behaviour of all, control and strengthened stone masonry walls. In the case of control walls, diagonally oriented cracks occurred in mortar joints in the central part of the walls. By increasing the amplitudes of imposed lateral displacements, the width of the cracks increased as they propagated over the entire surface of the walls (Figure 5). Ultimately, separation of the walls' wythes took place and individual stones started falling out.

Generally speaking, the failure mechanism of the strengthened stone masonry walls was similar in all cases. The mechanism was characterized by diagonally oriented and uniformly distributed cracks in the coating as well as separation of individual wythes of stone masonry at ultimate state. However, some differences, typical for each particular strengthening type, have been observed.

a.) b.) c.)

Figure 6. Testing arrangement and instrumentation (a.), stone masonry control wall (b.), brick masonry control wall (c.)

In the case of the single-side coated walls (strengthening type 1 – Figure 7), similar distribution of cracks as in the case of control walls has been observed on the uncoated side of the wall. Cracks which developed in the coating, anchored in the corners, followed more or less the same pattern. Ultimately, the unstrengthened wythe separated and partly collapsed, whereas the coated wythe remained monolithic.

Although final separation of coated stone-masonry wythes was the reason of collapse in all cases of specimens where coating has been applied on both sides (Figures 8b, 9b, and 10b), the crack pattern depended on the type of strengthening. In the case of strengthening type 2 (vertical grid without anchors), diagonally oriented cracks developed, distributed over the entire surface of the wall (Figure 7a), whereas in the case of strengthening type 3 (diagonal grid, anchored in the corners), the area with diagonally oriented cracks was concentrated in the central part of the wall (Figure 9a). In the case of diagonally and vertically placed fabric strips, anchored in the corners (strengthening type 4), visible cracks occurred only on the part of masonry, not covered by coating (Figure 10a).

a.) b.) c.)

Figure 7. Stone masonry, strengthening type 1: a.) damage on the uncoated side of the wall at ultimate state; b.) cracks in the coating; c.) separation of the wall's wythes

a.) b.)

Figure 8. Stone masonry, strengthening type 2: a.) cracks in the coating at ultimate state; b.) separation of the wall's wythes

a.) b.)

Figure 9. Stone masonry, strengthening type 3: a.) cracks in the coating at ultimate state; b.) buckling of coating and separation of the wall's wythes

a.) b.)

Figure 10. Stone masonry, strengthening type 4: a.) cracks in the uncoated part masonry at ultimate state; b.) separation of the wall's wythes

Brick masonry

Similarly as in the case of stone masonry walls, shear behaviour was dominant in case of all, control and strengthened walls. In the case of the control walls, diagonally oriented cracks occurred in the central part of the walls. By increasing the amplitudes of imposed lateral displacements, new cracks developed, passing through mortar joints and bricks and propagating over the entire surface of the walls. Ultimately, lateral parts of the walls, separated by diagonal cracks, started falling out (Figure 5 b).

In the case of the strengthened walls, two phases can be distinguished in the behaviour mechanism: the behaviour before and after the delamination of the coating. Before the delamination of coating, the strengthened wall behaved as a monolithic, though composite, structural element. Once the delamination occurred and the coating partly or completely delaminated, buckled, and/or peeled off, instantaneous strength and stiffness degradation took place. Ultimately, the remaining resistance capacity was due to the original masonry wall.

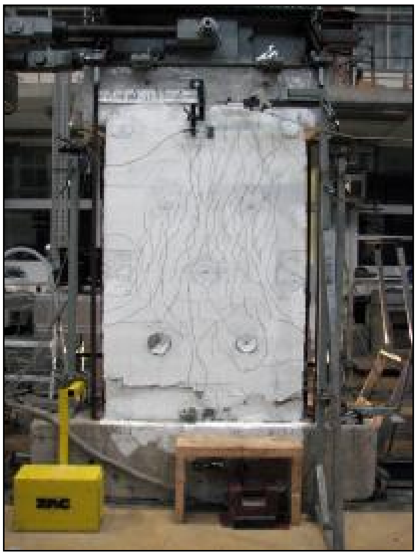
In the case of the walls strengthened by coating made of cementitious mortar, reinforced with vertically placed GFRP grid (strengthening types 5, 6, and 14), the coating delaminated depending on the number and position of anchors. In the case of the walls, where the coating has been applied on the walls without anchors (strengthening type 6a), cracks in the coating, predominantly oriented in the diagonal direction, have been first observed. When the imposed displacements attained the values of ultimate displacements of control walls, the coating without any anchors completely delaminated (Figure 11).

a.) b.)

Figure 11. Brick masonry: GFRP grid in cementitious mortar, not anchored. Internal side of delaminated coating removed from the wall (a.) and damage to the wall, visible when the coating was removed after the test (b.)

The anchors (strengthening types 6b, 6c, and 6d) prevented complete delamination of coating. In such a case, horizontal crack developed just above the bottom line of anchors. Although diagonally oriented cracks developed over the entire surface of coating, the coating in the bottom part of the wall delaminated between the anchors. At ultimate state, part of the coating just above the horizontal crack, where after several additional cycles of loading the masonry crushed, buckled and ruptured around the anchors in a circular shape (Figure 12).

In the case where the mortar was thin (strengthening type 14), relatively large, diagonally oriented crack developed at ultimate state. Filaments of the glass fibre grid ruptured along the crack. As a consequence, practically no improvement in lateral resistance and displacement capacity with regard to the control wall was observed (Figure 13). In the case of the walls, strengthened with coating made of cementitious mortar reinforced with diagonally placed GFRP grid and additional vertical strips of the same grid at the borders, anchored into the walls with carbon fibre string anchors (strengthening type 7), the mechanism was different. Uniformly distributed, diagonally oriented shear cracks developed in the central part of the walls between the vertical strips. When the imposed displacement amplitudes exceeded the ultimate displacements of the



a.) b.)

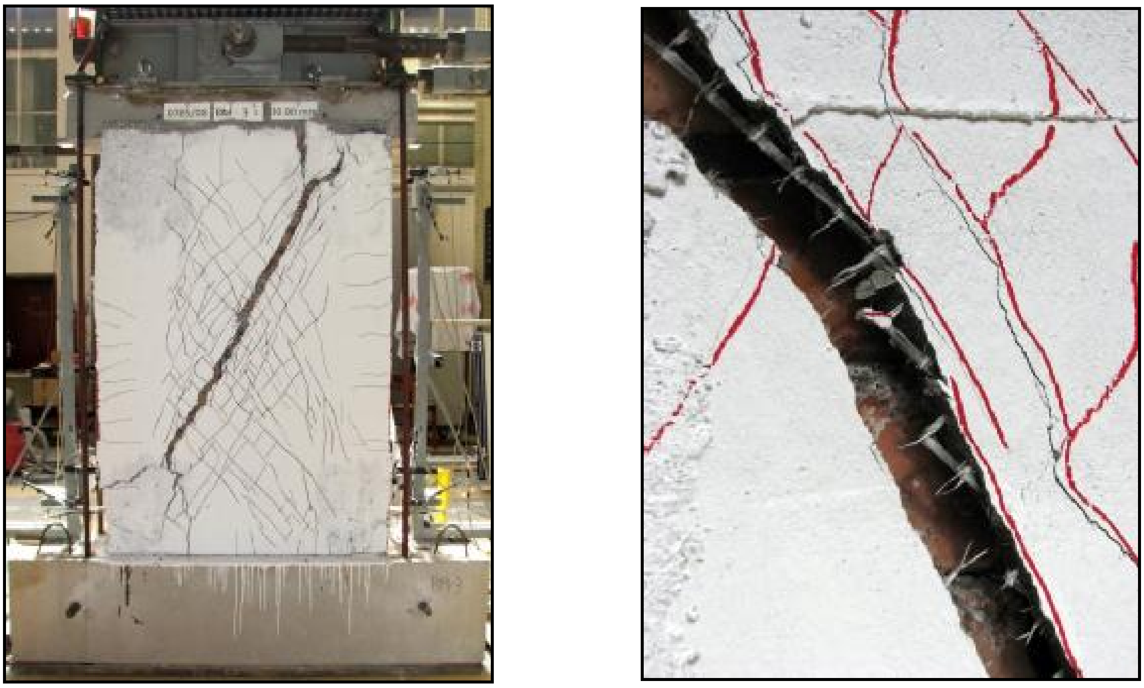
Figure 12. Brick masonry: GFRP grid in cementitious mortar, anchored. Damage at ultimate state: strengthening type 6b (5 anchors, left) and 6d (13 anchors, right)

control walls, horizontal tension cracks developed in vertical strips at both ends of the walls. Ultimately, the walls failed in shear due to rupture of diagonally placed glass fibre grid along the shear crack developed in the central area of the wall (Figure 14). In this case, too, the delamination of the coating took place in some parts of the walls' surface. However, both, lateral resistance and displacement capacity of the wall were significantly improved.

Considering the failure mechanism of walls, strengthened by coating consisting of GFRP or CFRP fabric in epoxy resin matrix (strengthening types 8–10 and 12), it can be seen that the mechanism depended on the distribution pattern and number of anchors. In the case where the anchors were concentrated in the corners of the walls, the coating either stripped off (Figure 15, a) or buckled over the entire surface of the wall (Figure 15, b), pulling off a relatively thick layer of masonry. In the case where the anchors were distributed over the entire surface of the wall (strengthening type 12), local buckling of coating took place. Once the delamination of the coating occurred over a large enough area, the effect of strengthening was lost.

In the case where the walls were strengthened by CFRP strips/plates (strengthening type 11), the strips started to delaminate and buckle already at relatively small imposed displacement amplitudes (Figure 16). Although the anchoring of CFRP strips into the concrete of foundation blocks and bond beams did not fail, the delaminated plates had no effect on the lateral resistance and displacement capacity of the tested walls.

Figure 13. Brick masonry: strengthening type 14. Rupture of GFRP grid at ultimate state



a.) *b.)*

Figure 14. Brick masonry: strengthening type 7. Shear cracks in the central part and tension cracks in vertical strips, anchored to the wall (left) and rupture of filaments of GFRP grid along the crack at ultimate state (right)

a.)

b.)

Figure 15. Brick masonry: delamination of coating with GFRP fabric in epoxy resin matrix. Stripping off (left) and buckling over the entire surface (right)

Obviously, the observed delamination and buckling phenomena are the result of great differences in strength and deformability properties of masonry and coating materials. The phenomena are emphasized in the case of strengthening the walls with CFRP strips/plates, directly glued to the masonry. In addition to that, masonry exhibited substantial deformations in vertical direction when subjected to in-plane cyclic lateral load reversals in a damaged state. The measurements indicated that at preloading ratio σ/f_c = 0.3, the walls compressed by 20 mm at cycling from the beginning of test to ultimate state. The contact stresses at such strain (about 1 %) exceeded the pull-off strength of masonry. As a result, the synthetic coating materials delaminate by pulling off weak masonry material and buckle.

Figure 16. Delamination of CFRP strips at ultimate state

3.3 Lateral resistance and displacement capacity

To assess the efficiency of strengthening, the resistance and displacement capacity of the walls at three limit states have been compared, namely crack (damage) limit state, where the first cracks occur in the walls, causing evident changes in stiffness of the wall, maximum resistance, and ultimate limit state of collapse, defined by severe degradation of resistance at repeated lateral load reversals or collapse of the wall.

The test results are summarized in Table 2, where the values of lateral load (*H*) displacement (*d*), and rotation angle, $\Phi = d/h$ (in % of *h*; *h* = height of the wall), at characteristic limit states are given as the average values, measured at the first amplitude peak in positive and negative direction of loading. Values, measured on strengthened walls, are compared with respective values, obtained by testing the control walls. The results are analyzed in Table 3, where the resistance and displacement capacity of the strengthened walls are compared with the average values, obtained by testing the unstrengthened, control walls. In addition, the values of effective stiffnesses, defined as the ratio between the lateral load and displacement at the damage limit state, $K_e = H_{cr}/d_{cr}$, are also compared.

Lateral load - displacement hysteretic relationships, measured during the testing of *stone masonry* walls, are shown in Figures 17 and 18. For comparison, lateral load - displacement relationships, obtained during the testing of control walls, are also plotted in the figures (green line).

Table 2. Test results: lateral resistance, H, and displacement, d, and rotation, Φ ,at characteristic limit states

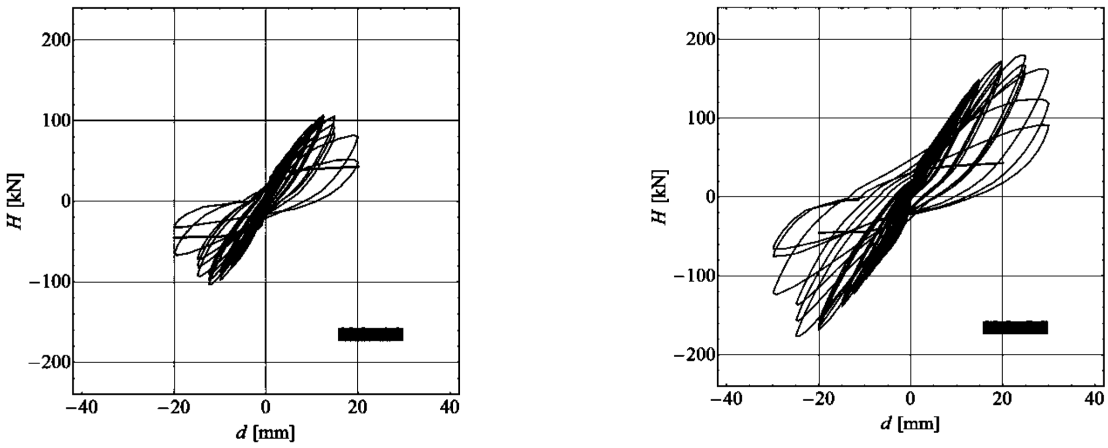
Masonry*	Strength. type	Damage limit			Maximum resistance			Ultimate state		
		H_{cr} [kN]	d_{cr} [mm]	$Φ_{cr}$	H_{max} [kN]	d_{max} [mm]	$Φ_{max}$	H_u [kN]	d_u [mm]	$Φ_u$
S	Reference	27.5	1.5	0.05	45.2	11.13	0.75	26.6	20.0	1.35
S	1	32.3	1.5	0.10	123.6	14.8	1.00	45.6	22.5	1.52
S	2	35.1	1.3	0.08	169.8	22.2	1.50	64.7	27.5	1.86
S	3	39.0	1.1	0.08	191.5	23.5	1.60	79.9	30.3	2.06
S	4	40.3	1.0	0.07	188.6	24.3	1.84	93.2	32.5	2.20
B	Reference	54.74	3.00	0.19	89.54	9.01	0.58	34.33	20.00	1.28
B	5	44.41	1.75	0.11	107.79	11.02	0.71	39.89	20.00	1.30
B	6a	56.52	1.50	0.10	114.34	6.68	0.43	36.51	14.99	0.96
B	6b	59.75	1.50	0.10	132.93	9.15	0.59	35.62	22.50	1.46
B	6c	57.87	1.75	0.11	143.09	12.35	0.80	54.72	22.49	1.46
B	6d	63.97	2.00	0.13	125.10	13.45	0.86	18.60	24.97	1.60
B	7	94.70	3.50	0.23	204.18	21.69	1.40	17.47	30.05	1.93
B	8	44.44	1.50	0.10	104.39	9.82	0.63	40.63	17.51	1.12
B	9	39.65	1.50	0.10	108.82	11.71	0.75	31.80	19.98	1.28
B	10	44.86	1.50	0.10	115.77	9.89	0.64	21.54	19.99	1.29
B	11	42.67	1.50	0.09	76.12	11.03	0.69	33.77	17.49	1.10
B	12	63.18	2.75	0.17	126.58	13.68	0.85	29.34	19.99	1.25
B	13	81.38	3.00	0.19	153.27	10.80	0.70	42.67	17.48	1.13
B	14	53.95	2.00	0.13	125.41	15.97	1.00	34.10	27.48	1.72

* S – stone masonry, B – brick masonry

Table 3. Effect of strengthening methods

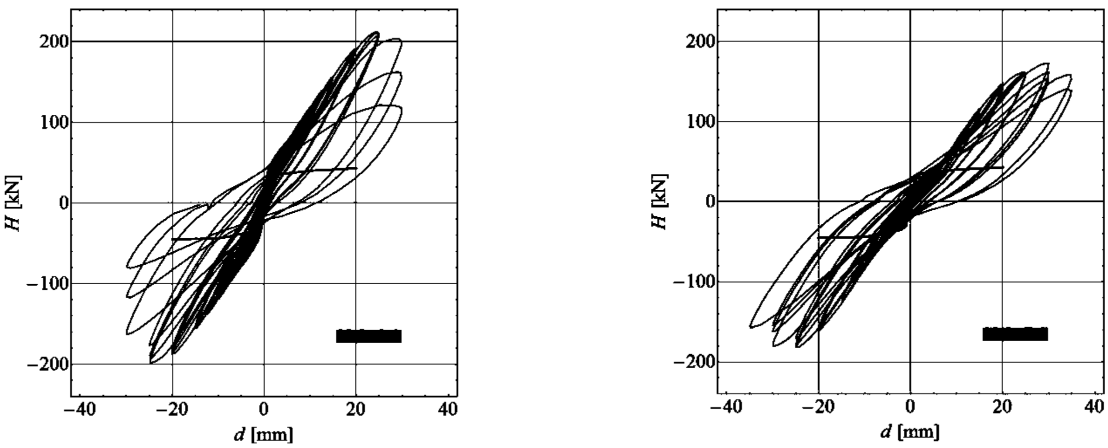
Masonry*	Strengthening type	Stiffness		Resistance capacity		Displacement capacity	
		K_e [kN/mm]	Strengthened/ control	H_{max} [kN]	Strengthened/ control	d_u [mm]	Strengthened/ control
S	Reference	18.33	-	45.2	-	20.0	-
S	1	21.53	1.17	123.6	2.73	22.5	1.13
S	2	27.00	1.47	169.8	3.76	27.5	1.38
S	3	35.45	1.93	191.5	4.24	30.3	1.51
S	4	40.30	2.20	188.6	4.17	32.5	1.63
B	Reference	18.20		89.6		20.00	
B	5	25.23	1.38	107.79	1.20	20.00	1.00
B	6a	37.68	2.06	114.34	1.28	14.99	0.75
B	6b	39.83	2.18	132.93	1.48	22.50	1.13
B	6c	33.29	1.82	143.09	1.60	22.49	1.12
B	6d	31.98	1.75	125.10	1.40	24.97	1.25
B	7	27.47	1.51	204.18	2.28	30.05	1.50
B	8	29.62	1.62	104.39	1.17	17.51	0.88
B	9	26.43	1.45	108.82	1.22	19.98	1.00
B	10	29.91	1.64	115.77	1.29	19.99	1.00
B	11	28.45	1.56	76.12	0.85	17.49	0.87
B	12	25.82	1.42	126.58	1.41	19.99	1.00
B	13	28.57	1.57	153.27	1.71	17.48	0.87
B	14	27.94	1.53	125.41	1.40	27.48	1.37

* S – stone masonry, B – brick masonry



a.) b.)

Figure 17. Stone masonry. Lateral load−displacement hysteresis loops, obtained by testing the strengthened walls. a.) Strengthening type 1, b.) Strengthening type 2



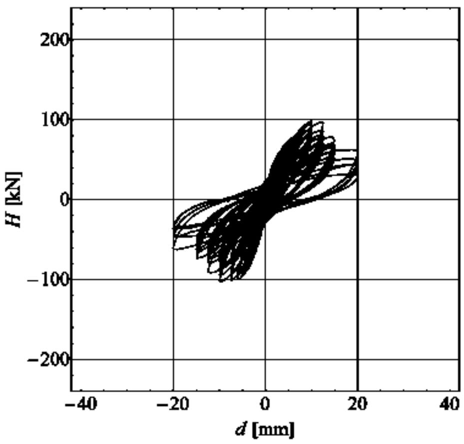
a.) b.)

Figure 18. Stone masonry. Lateral load−displacement hysteresis loops, obtained by testing the strengthened walls. a.) Strengthening type 3, b.) Strengthening type 4

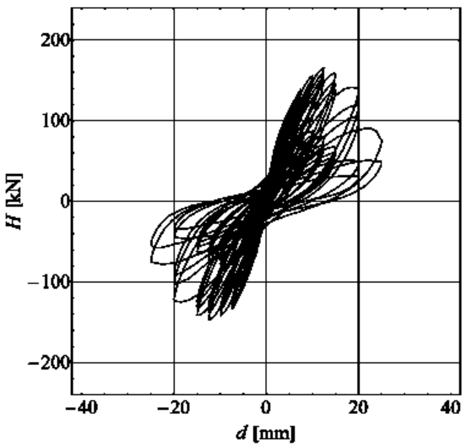
As shown in Table 3, the strengthening of traditional three-leaf stone masonry by application of polymer coating significantly improved lateral resistance and displacement capacity of the tested walls. The efficiency did not depend much on the type of coating (vertically or diagonally placed polymer grid in fibre reinforced mortar; polymer fabric in epoxy resin matrix), but depended mainly on the method of application. Analyzing the test results, no indication can be obtained regarding the influence of damage state of the wall at the time of application of coating (previously damaged, undamaged) on lateral resistance and displacement capacity. On the basis of the observed mechanism, the difference in resistance of walls where strengthening types 1 and 2 have been applied cannot be attributed to the previous damage, but to the method of application.

The application of coating on only one side of the wall, although anchored in the corners, improved the resistance to a lesser degree than the application of coating on both sides of the wall. Moreover, the application of coating on only one side did not improve the displacement capacity. The analysis of test results has shown the importance of anchoring the coating at least in the corners of walls. As can be seen, the improvement in both, resistance and displacement capacity was greater in the case of strengthening types 3 and 4 where the coating was anchored in the corners than in the case of strengthening type 2 without any anchors. The difference in resistance and displacement capacity between the strengthening types 3 and 4 (grid versus fabric) cannot be considered significant.

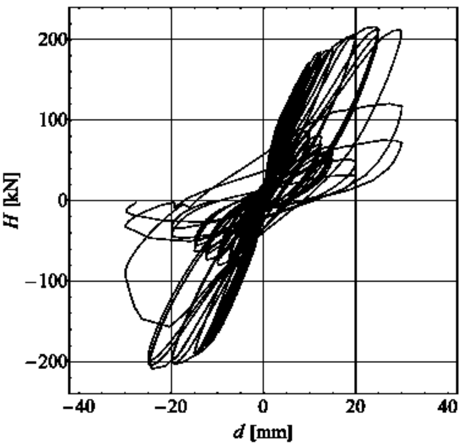
The types and direction of placing of the coating (grid−fabric; vertical−diagonal) influence the position and distribution of cracks. The coating, however, did not prevent the delamination of the wall wythes at ultimate state. Because of delamination, falling out of stones and compression of masonry which resulted in consequent sudden buckling of coating, severe resistance degradation takes place during the ultimate phase of behavior (Figures 17 and 18).



a.)



b.)



c.)

Figure 19. Brick masonry. Typical hysteretic lateral load-displacement relationships, obtained by testing the walls strengthened by strengthening types 8 (a), 6c (b), and 7 (c). Red: control wall

Lateral load - displacement hysteretic relationships, measured during the testing of *brick masonry* walls, strengthened by typical strengthening method, are shown in Figure 19. For comparison, lateral load - displacement relationships, obtained during the testing of control walls, are also plotted in the figures.

In the case of the walls, strengthened with vertically and diagonally placed CFRP strips (plates), glued on the masonry and anchored at the bottom and top into reinforced concrete foundation block and bond-beam with epoxy resin (strengthening type 11), only an increase in lateral in-plane stiffness, but no improvement in lateral resistance and ductility capacity has been observed. In all other cases of application of composite coatings, however, besides an increase in stiffness, resistance capacity of the walls has been improved to a greater or lesser degree (20–130 %). Because of prevailing delamination of the coating, however, little improvement in displacement capacity has been obtained (10–50 %) and the failure mechanism was brittle in all cases.

Analyzing the results given in Table 3, the effect of strengthening the walls with either thicker (cementitious mortar, reinforced with GFRP grid) or thinner coating (GFRP or CFRP fabric in epoxy resin matrix) was similar. In the first case, the resistance was improved by 20–70 % of the original, whereas in the other by 17–42 %. As expected, however, the thickness of coating influenced the lateral stiffness of the walls: the walls strengthened with cementitious mortar, reinforced with GFRP grid were more rigid than the walls strengthened with GFRP or CFRP fabric in epoxy resin matrix. Against expectations, the number and distribution of anchors did not significantly influence the improvement in lateral resistance.

No difference in the behaviour between the previously damaged and undamaged walls, strengthened by the same strengthening type, has been observed. Surprisingly, the difference in the behaviour of walls, strengthened by applying the same type of coating on either only one or both sides of the wall, was also not significant (compare strengthening types 5 and 6a, as well as 8 and 9).

Among all tested strengthening types, the walls, strengthened with coating made of cementitious mortar reinforced with diagonally placed GFRP grid and additional vertical strips of the same grid at the borders, anchored into the walls with carbon fibre string anchors (strengthening type 7), exhibited best behaviour as regards both, the lateral resistance and displacement capacity.

4 CONCLUSIONS

A series of three-leaf stone and brick masonry walls, constructed in the laboratory in the traditional way, and strengthened by application of different types of polymer coating, have been tested by subjecting them to constant vertical load and cyclic shear, simulating seismic loads. The coating consisted of different composite reinforcement and either cementitious mortar or epoxy resin as a matrix. The methods of application of coating also varied (one side, both sides, anchored, not anchored).

In the case of stone masonry the tests have confirmed the efficiency of composite coatings. The in-plane lateral resistance has been improved by more than four times and the displacement capacity by up to 50 %, depending on the type of strengthening. No significant difference in the efficiency of various types of coating has been observed (application on one or both sides of the wall, anchoring or not in the corners). The application of coating increased also the rigidity of the walls.

The efficiency of strengthening types in the case of the brick masonry varied. Depending on the strengthening type, the in-plane lateral resistance of brick masonry walls was improved to a greater or lesser degree (20–130%). However, because of the failure mechanism, which was due to the delamination and buckling of coating, the improvement in displacement capacity was not significant, and the failure mechanism was brittle, with large resistance and stiffness degradation at ultimate state. The application of coating increased the rigidity of the walls.

In all cases, the delamination and buckling of coatings were critical for the behaviour of the walls when subjected to in-plane lateral load reversals. Obviously, these phenomena are the result of great differences in strength and deformability properties of masonry and coating materials. The phenomena are emphasized in the case of strengthening the walls with CFRP strips/ plates, directly glued to the masonry. As the measurements during the cyclic lateral resistance tests indicated, in addition to lateral deformations, masonry exhibited substantial deformations also in vertical direction. At preloading ratio σ/f_c =0.3, at which the walls have tested, the walls compressed by up to 20 mm at cycling from the beginning of test to ultimate state. The contact stresses at such strain (about 1 %) exceeded the pull-off strength of masonry. As a result, the synthetic coating materials delaminate by pulling off weak masonry material and buckle.

Although the composite-based coatings proved to be efficient as regards the resistance of traditional three-leaf stone masonry walls, further efforts should be made to develop techniques which will prevent the separation of stone masonry wythes and prevent large resistance and stiffness degradation at ultimate state. The efficiency of the composite-based coatings significantly depended on the type of application in the case of the brick masonry walls. It is believed that by developing more flexible coatings and means to prevent delamination and buckling, the observed large strength degradation and brittle failure mechanisms can be prevented. Consequently, not only the lateral in-plane resistance, but also the displacement and energy dissipation capacity of the strengthened brick masonry walls can be significantly improved.

ACKNOWLEDGEMENTS

The research, presented in this contribution, has been part of the applied research project L2-0578, financed by the Ministry of Higher Education, Science and Technology of the Republic of Slovenia, and co-financed by the Sika d.o.o., Prevale, Slovenia, branch company of Sika AG, Switzerland.

5 REFERENCES

[1] ElGawady, M., P. Lestuzzi, M. Badoux (2006). Shear Strength of URM Walls Retrofitted Using FRP. *Engineering Structures* **28:12**, 1658−1670.

[2] Konthesingha, C., M. Masia, R. Petersen, N. Mojsilović, G. Simundic, A. Page (2010). Cyclic In-plane Shear Behaviour of Unreinforced Masonry Panels Retrofitted with Fibre Reinforced Polymer Strips. *8th International Masonry Conference*.

[3] Lutman, M, Seismic Resistance Assessment of Heritage Masonry Buildings in Ljubljana, *International Journal of Architectural Heritage*, **4:3**, 2010, pp. 198-221.

[4] Schwegler, G. (1994). Masonry Construction Strengthened with Fiber Composites in Seismically Endangered Zones. *10th European Conference on Earthquake Engineering*.

[5] Tomaževič, M., Gams M., Berset, T. (2011). Strengthening of historic brick masonry walls with GFRP coating. *11th North American Masonry Conference*.

[6] Triantafillou, T.C., M.N. Fardis (1997). Strengthening of Historic Masonry Structures With Composite Materials. *Materials and Structures* **30:2**, 486−496.

[7] Vidrih, R., M. Godec, The 1895 Ljubljana Earthquake and its Influence on the Development of Technical-construction Regulations, *Ujma*, **9**, 1995, pp. 231-237 (in Slovene).

REZIME

SEIZMIČKO POJAČAVANJE ISTORIJSKIH ZIDANIH ZIDOVA KOMPOZITIMA – EKSPERIMENTALNA ISTRAŽIVANJA

Miha TOMAŽEVIČ
Matija GAMS
Thierry BERSET

U radu je istraživana efikasnost pojačavanja kamenih i zidova od opeke, na seizmička dejstva, primenom različitih tipova kompozita, kao što su vlaknima ojačani polimeri (FRP). Obuhvaćeni su elementi sa staklenim (GFRP) i karbonskim (CFRP) vlaknima. Zidovi su ispitivani kao vertikalne konzole izložene prethodno konstantnom i ponovljenom cikličnom bočnom opterećenju u ravni. Pojačani troslojni kameni zidovi srušili su se usled odvajanja pojedinih slojeva, dok je za mehanizam zida od opeke karakteristično odvajanje obloge ili traka usled istiskivanja ili izvijanja i pod pritiskom kada su zidovi bili izloženi bočnom ponovljenom opterećenju. Značajno poboljšanje bočne nosivosti i kapaciteta deformisanja je zabeleženo kod kamenih zidova. Međutim, kod zidova od opeke nagla degradacija krutosti i otpornosti usled odvajanja obloge dovela je do rušenja zidova. Pošto je povećanje bočne nosivosti zabeleženo u većini slučajeva, naglašena je potreba adekvatnog sidrenja obloge radi povećanja kapaciteta deformisanja.

Ključne reči: kameni zidovi, zidovi od opeke, pojačavanje, obloga, mreža od staklenih vlakana, stakleno/ugljenično-karbonski prefabrikati, seizmičko ponašanje

SUMMARY

SEISMIC STRENGTHENING OF HISTORIC MASONRY WALLS WITH COMPOSITES: AN EXPERIMENTAL STUDY

Miha TOMAŽEVIČ
Matija GAMS
Thierry BERSET

The efficiency of strengthening of stone and brick masonry walls for seismic loads by application of different types of composite-reinforced coating, such as GFRP grid/fabric, CFRP fabric in either cementitious mortar of epoxy resin matrix, and CFRP strips, has been investigated. The walls have been tested as vertical cantilevers by subjecting them to constant pre-loading and cyclic in-plane lateral load reversals. The strengthened three-leaf stone masonry walls failed due to the separation of individual masonry wythes, whereas the failure mechanism of brick masonry was characterized by delamination of coating or strips, which pulled off the masonry and buckled as soon as compression of the damaged masonry took place at repeated lateral load reversals. Significant improvement in lateral resistance and displacement capacity was observed in the case of the stone masonry walls. In the case of the brick masonry walls, however, sudden resistance and stiffness degradation took place as a result of delamination of coating, leading to collapse of the walls. Whereas improved lateral resistance has been observed in most cases, adequate anchoring of coating was needed to improve the displacement capacity.

Key words: stone masonry, brick masonry, strengthening, coating, glass fibre grid, glass/carbon fibre fabric, seismic behaviour, testing

KOMPARATIVNA ANALIZA METODA ZA PROCENU POMERANJA FLEKSIBILNIH SIDRENIH BETONSKIH DIJAFRAGMI

COMPARATIVE ANALYSIS OF EVALUATION METHODS OF THE DISLOCATION OF FLEXIBLE ANCHORED CONCRETE DIAPHRAGM WALLS

Radomir FOLIĆ
Mirza MEMIĆ
Adnan IBRAHIMOVIĆ

ORIGINALNI NAUČNI RAD
ORIGINAL SCIENTIFIC PAPER
UDK: = 861

1 UVODNE NAPOMENE

U savremenoj građevinsko-geotehničkoj praksi, pri izgradnji konstrukcija za zaštitu iskopa, sanaciju klizišta, izgradnju podzemnih etaža objekata u urbanim područjima i slično, sve češće se koriste armiranobetonske (AB) dijafragme [8] i [9]. One su, uglavnom, stalne potporne konstrukcije, a često i sastavni deo noseće konstrukcije budućeg objekta. Moraju se projektovati tako da obezbede stabilnost pod dejstvom sila koje nastaju nakon iskopa tla, od podzemnih voda, opterećenja od okolnih objekata i dr. Shodno tome, nameće se potreba za formulisanjem adekvatnog proračunskog modela za proračun tih građevina, da bi bile sigurne u svim fazama građenja i eksploatacije [5].

U sadašnjim uslovima, koristi se nekoliko različitih metoda geomehaničkih analiza i postupaka za izračunavanje bočnih savijanja fleksibilnih potpornih konstrukcija, zavisno od toga da li je reč o projektovanju i/ili istraživanju. Nijedna od tih metoda nije opšteprihvaćena, bilo da su u pitanju linearne ili nelinearne analize [1]. Savremene metode proračuna i softverski paketi zahtevaju takve podatke o tlu, kakve je teško obezbediti zbog praktičnih ili pak ekonomskih razloga [7]. Zato se često javljaju znatne razlike između izračunatih i prognoziranih vrednosti u odnosu na stvarne, dobijene merenjem već izvedenih objekata [13].

Radomir Folić, Univerzitet u Novom Sadu
Fakultet tehničkih nauka, Srbija
Mirza Memić, Direkcija cesta Tuzla, BiH
Adnan Ibrahimović, Univerzitet u Tuzli, RGGF, BiH

1 INTRODUCTION

In modern construction and geotechnical practice, when building structures for protecting trenches, rehabilitating landslides, constructing underground floors in urban areas etc. reinforced concrete (RC) diaphragm walls [8], [9] are being increasingly used. They are mostly permanent retaining structures, often an integral part of future facility's retaining structure. They need to be designed so as to ensure stability under the influence of forces that occur after the excavation of soil, as a result of ground water, loads from surrounding structures/objects, etc. Consequently, there is a need for formulating an appropriate calculation model that enables the safety of these structures in all phases of their construction and service/operation [5].

Under current conditions, there are several different methods of geotechnical analysis and procedures for calculating lateral bending of flexible retaining structures, depending on the purpose, i.e. design or research. None of these methods is universally accepted, whether it is a linear or non-linear analysis [1]. Data on soil required by modern calculation methods and software packages are difficult to obtain either for practical or economic reasons [7]. This often leads to significant differences between the calculated and predicted values on one hand and the actual values, obtained by measurements performed on existing buildings, on the other [13].

Radomir Folic, University of Novi Sad
Faculty of Technical Sciences, Serbia
Mirza Memic, Road Institution of Bosnia and Hezegovina
Adnan Ibrahimovic, University of Tuzla
Faculty of Mining, Geology and Civil Engineering, Tuzla, Bosnia and Hezegovina

U ovom radu komparativno su analizirani rezultati dobijeni primenom nekih karakterističnih metoda za iznalaženja horizontalnih pomeranja fleksibilnih potpornih konstrukcija, kao i pritisaka tla koji deluju na nju. Uz teorijske osnove, formiran je proračunski model, a zatim su upoređeni rezultati proračuna klasičnim metodama (analitički izrazi za analizu) s rezultatima dobijenim metodom konačnih elemenata (MKE). Dijagrami pomeranja dijafragme, dobijeni primenom pomenutih metoda, upoređeni su i sa izmerenim vrednostima horizontalnih pomeranja na izgrađenim objektima. Proračunski model, geometrijski, geomehanički i svi drugi ulazni podaci – parametri za analizu, preuzeti su iz dokumentacije korišćene pri ispitivanju jedne lamele, označene sa 18D [6], koja je element potporne konstrukcije vertikalnog zaseka za plato nove železničke stanice u Beogradu. Ti podaci su plod istraživanja u okviru doktorske disertacije Čedomira Vujičića, kasnije profesora Građevinskog fakulteta u Beogradu.

2 EMPIRIJSKI POSTUPCI PRORAČUNA FLEKSIBILNIH ZIDOVA

Praktični proračun zadovoljavajuće tačnosti fleksibilnih zidova, bilo da se oni koriste kao stalne ili kao privremene građevine, još uvek ne postoji. Rezultati modelskog proračuna i terenskog merenja su pokazali da su deformacije takvih potpornih zidova takve da se iza njih javlja nehidrostatički oblik dijagrama pritiska tla. Oni se tako projektuju da zadovolje računske napone u materijalima od kojih su sagrađeni i imaju deformacije reda veličine koje utiču na raspored pritisaka tla iza njih [2]. Analiza opterećenja na potpornu konstrukciju, naročito tla i objekata u njenoj blizini, detaljno je opisana, uglavnom prema DIN-u, u radu [3]. Dimenzije i karakteristike preseka, odnosno krutost savitljivog zida, deformacije oslonaca – razupirača, odnosno istezanja ili skraćivanje ankera (sidara), promene karakteristika slojevitog tla iza fleksibilnog zida, oscilacije nivoa podzemne vode, opterećenja stalnih ili pokretnih objekata uz iskop i slično, izazivaju preraspodelu pritisaka tla na kontaktu fleksibilnog zida i tla [9].

Naučno istraživanje u ovom području sporo napreduje zbog izuzetne komplikovanosti prirode činilaca koji utiču na proračun fleksibilnih zidova. Sve više se razjašnjavaju svojstva pojedinih faktora, koja su presudna u rešavanju problema. Uprošćavanjem nekih činilaca i prilagođavanjem modelima tla i na osnovu njih – proračunskim metodama za definisanje naponsko-deformacionih stanja u tlu i njegovom interakcijom s drugim objektima, danas imamo dovoljan broj teorija i postupaka za svakodnevnu upotrebu. Prve teorije i postupci proračuna fleksibilnih zidova nisu vodili računa o veličini deformacija zida i detaljnijim podacima o geotehničkim karakteristikama tla. Najviše su korišćene, a i danas se koriste, teorije koje polaze od hidrostatičkog oblika raspodele horizontalnih pritisaka tla na fleksibilne zidove, s tim što se koriguju izračunate statičke veličine, na osnovu čisto empirijskog poznavanja fenomena pritiska tla. Usvaja se to da su zadovoljeni potrebni uslovi pomeranja, i da se u području posmatranog zaštitnog zida javljaju pritisci tla prema Rankinovoj teoriji.

This paper is a comparative analysis of results obtained on some typical methods for indentifying the horizontal displacements of flexible retaining structures, as well as the soil pressures acting upon them. In addition to the theoretical basis, a calculation model was established, and simulation results obtained using conventional methods (analytical expressions for analysis) were compared with the results obtained using the finite element method (FEM). Diaphragm displacement diagrams obtained using the above methods were compared also with horizontal displacements of the already constructed facilities obtained by measurements. The calculation model, as well as the geometrical, geotechnical and all other input-parameters for the analysis were taken from the documentation used in the examination of strip 18D [6], which is an element of the retaining structure of a vertical cut for the plateau of the new railway station in Belgrade. These data consist of research result obtained by Čedomir Vujicic, Professor at the Faculty of Civil Engineering in Belgrade, for his doctoral dissertation.

2 EMPIRICAL PROCEDURES FOR CALCULATING FLEXIBLE WALLS

We still lack a sufficiently accurate practical procedure for calculating flexible walls, regardless of whether they are used as permanent or temporary structures. Results of model-based calculations and field measurements have shown that deformations in these retaining walls lead to the occurrence of non-hydrostatic soil pressure diagram behind them. They are designed to meet the calculated stresses in materials they are constructed, and the order of magnitude of their deformation affects the distribution of soil pressures behind them [2]. The analysis of loads acting upon the retaining structure, especially that of the soil and buildings in its vicinity, is described in details, mainly according to the DIN standards, in [3]. Dimensions and properties of the cross-section, i.e. stiffness of the flexible wall, deformations of supports - shores, i.e. stretching or shortening prestressed anchors, changes in properties of the soil layers behind flexible wall, fluctuations in groundwater level, loads from the fixed or mobile facilities near the trench, etc. all of them cause soil pressure redistribution at the flexible wall-soil interface [9].

Given the highly complex nature of factors influencing the calculation of flexible walls, scientific research in this area advances slowly. The properties of crucial problem-solving factors are increasingly clarified. By simplifying some of the factors and adjusting the feasible methods to the soil, today we have a sufficient number of theories and procedures for a daily use. The early theories and procedures of flexible wall calculation lack to take into account the magnitude of wall deformation and the more detailed data on the geotechnical characteristics of soil. They mostly relied (and still do) on theories that are based on the hydrostatic form of distribution of horizontal soil pressures on flexible walls, while correcting the calculated static dimensions on the basis of the purely empirical knowledge of the soil pressure phenomenon. It is assumed that the required conditions of displacement

Pretpostavlja se da dalja pomeranja zida ne utiču bitno na promenu ovih pritisaka tla. Takođe, ne vodi se računa o savitljivosti zida i o elastičnim svojstvima tla.

Danas raspolažemo s dovoljno podataka o uzajamnom uticaju tla i potporne konstrukcije, kako bismo mogli da prognoziramo veličine i raspored pomeranja i pritisaka tla, odnosno da bismo mogli izračunati potrebne statičke uticaje, bez obzira na to što su mnoga pitanja ostala nerazjašnjena. S ciljem proširivanja saznanja i građenja ekonomičnijih fleksibilnih potpornih konstrukcija, a pritom još uvek sigurnog građenja, preporučuje se – kad god je to moguće – obavljanje merenja na njima.

3 POSTUPCI PRORAČUNA ZASNOVANI NA TEORIJI ELASTIČNOSTI

Horizontalna pomeranja fleksibilnih armiranobetonskih dijafragmi često su takvog reda veličina da nisu u stanju da u tlu mobilišu plastičnu ravnotežu s jedne i druge strane zida. Zbog toga se u takvim slučajevima za proračun dijafragme koriste „elastične” metode proračuna. Ovakve metode omogućuju izračunavanje pritisaka tla na zid dijafragme, i za slučajeve kada je otpornost na smicanje tla u mogućim kliznim površima samo delimično mobilisana.

Za proračun po elastičnim metodama najčešće se koriste:

– metoda koeficijenta k_h reakcije tla (Vinklerov postupak);

– metoda konačnih elemenata (MKE) [3] i [13].

Metoda koeficijenta k_h reakcije tla zasniva se na pretpostavci da je opterećenje od tla iza dijafragme poznato. Usvaja se da je to aktivni pritisak tla, ili neka vrednost između aktivnog pritiska i pritiska tla u miru (slika 3). Prema ovoj metodi proračuna, u svakoj tački konstrukcije koja je u kontaktu s tlom, reakcija tla je srazmerna deformaciji u posmatranoj tački. Takođe, promena pritiska zida na tlo, počevši od pritiska tla u stanju mirovanja, proporcionalna je pomeranju zida. Koeficijent proporcionalnosti (srazmernosti) između pritiska i pomeranja dijafragme predstavlja koeficijent reakcije tla k_h. Ova proporcionalnost važi do pasivnog stanja u tlu, koje je određeno karakteristikama tla (γ, φ *i* c). Ovakvim zakonima obuhvata se samo tlo u neposrednom kontaktu sa zidom. Pomeranja i naponi u tlu i u masivu oko zida ovim postupkom proračuna ne mogu se obuhvatiti.

Koeficijent k_h ne može se direktno izmeriti, niti mora da ima istu vrednost u horizontalnom i u vertikalnom pravcu. On se može po dubini menjati i u sloju tla konstantnih karakteristika. Ovaj koeficijent je funkcija veličine dodirne površine objekat–tlo. Netačna procena vrednosti koeficijenta kh manje utiče na računske veličine napona, a više na računske veličine pomeranja dijafragme. Vinklerov koeficijent kh danas se određuje izrazima koje je predložilo više autora [5].

are met, and that pressures in the area of the observed protecting wall occur according to Rankine theory. Further wall displacements are assumed as not affecting significantly the change in these soil pressures. Also, wall flexibility and elastic soil properties are not accounted for.

Nowadays, we have sufficient data on the mutual influence of soil and the structure, enabling us to forecast the magnitude and displacement the soil pressure, i.e. to calculate the required static influences, despite the fact that many questions still remain unanswered. In order to enhance knowledge and build more cost-effective (while still safe) retaining structures, it is recommended to perform measurements on them, whenever possible.

3 CALCULATION PROCEDURES BASED ON THE THEORY OF ELASTICITY

Horizontal displacements of flexible reinforced concrete (RC) diaphragm walls are often such magnitudes that they fail to mobilize the plastic balance in soil on both sides of the wall. Therefore, in such cases, the diaphragm wall is calculated on "elastic" methods of calculation. These methods enable the calculation of soil pressures acting upon the diaphragm wall also in cases when the shear strength of the soil in the potential sliding surface is only partially mobilized.

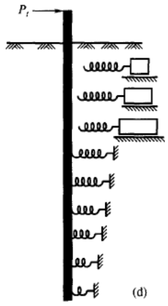
The commonly used elastic methods are the following:

– Method of the subgrade (soil) reaction coefficient k_h, (Winkler's procedure), and

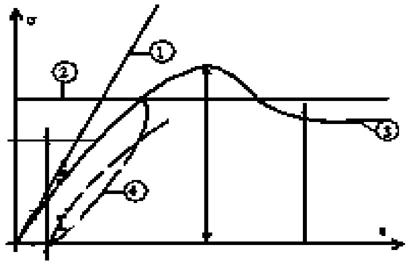
– Finite element method (FEM) [3] and [13].

The method of soil reaction coefficient k_h is based on the assumption that the loading resulting from the soil behind the diaphragm wall, is known. It is assumed to be active soil pressure, or some value between active pressure and soil pressure at rest (Figure 3). According to this calculation method, in each structure-soil contact point, the soil reaction is proportional to deformation of the observed point. Also, changes in the pressure exerted by the wall to the soil, starting from the pressure of soil in rest, is proportional to the displacement of the wall. The coefficient of proportionality between pressure and the displacement of the diaphragm wall is the soil reaction coefficient k_h. This proportionality applies to the passive state of soil, which is determined by its properties (γ, φ and c). These laws apply only for cases when the soil is in direct contact with the wall. Displacements and stresses in the soil and the mass around the wall cannot be identified using this calculation procedure.

The coefficient k_h cannot be measured directly, and its value is not necessarily the same in both horizontal and vertical direction; it can change with depth also in a soil layer of constant properties. This coefficient is a function of the size of the facility-soil contact surface. If the value of the coefficient k_h is assessed incorrectly, this rather affects the calculated magnitude of displacement of the diaphragm wall, than the calculated stress magnitudes. Today, the Winkler's coefficient k_h is determined using expressions proposed by several authors [5].



Slika 1. Vinklerov model
Figure 1. Winkler`s model



1 – idealno elastično/*ideal elastic*
2 – idealno plastično/*ideal plastic*
3 – realno tlo/*real soil*
4 – linija rasterećenja/*unloading line*
5 – relativna deformacija/ *relatively deformations*

Slika 2. Krive odnosa naprezanja i deformacija
Figure 2. Stress-strain relationship

$$p_{hi} = k_{hi} \cdot y_i \tag{1}$$

gde je:

p_{hi} – horizontalni pritisak na savitljivu dijafragmu u posmatranoj tački *i*;

y_i – horizontalno elastično pomeranja zida, odnosno tla u posmatranoj tački *i*;

k_{hi} – koeficijent reakcije tla u tački *i*, u horizontalnom pravcu, pri zbijanju tla.

Metode proračuna sa Vinklerovim modelom (slika 1.) ograničene su na relativno manje deformacije na dnu građevinske jame. Pri većim pomeranjima, neophodno je uvesti nelinearne veze između opterećenja i pomeranja. Da bi se prikazalo stvarno ponašanje ugrađenog dela dijafragme i interakcija konstrukcija–tlo, neophodno je obuhvatiti plastične deformacije u tlu, odnosno tretirati tlo kao nelinearnu deformabilnu sredinu [1]. Otpor tla opisan je koeficijentom reakcije tla, koji se menja s dubinom [2].

Prema predloženom postupku Čedomira Vujičića, proračun horizontalnih pomeranja dijafragme ne zahteva pretpostavke o obliku i veličini deformacije dijafragme, odnosno postupak proračuna ne uslovljava da se pretpostavi opterećenje zida od dejstva tla iza zida. Proračun dijafragme ovim postupkom zahteva jedino definisanje elastoplastičnih karakteristika tla na pritisak, kao i na zatezanje, duž kontakta dijafragma–tlo. Stanje napona u tlu određeno je teorijom granične ravnoteže na bazi Kulonovog zakona. Ako se masa tla ne kreće, kažemo da u njoj deluje horizontalni pritisak tla u stanju mirovanja. Tačan odnos između vertikalnog napona u tlu i ovog horizontalnog pritiska tla u stanju mirovanja danas se može odrediti dovoljno precizno – eksperimentalnim merenjima i računskim postupcima. Ako se zid odvaja od tla, omogućuje se tlu iza zida da se širi, a bočni pritisak tla postepeno opada. Posle nekog konačnog intervala pomeranja zida, u tlu se javlja minimalna vrednost horizontalnog pritiska tla. Ukoliko se zid „ugiba” u tlo, ono biva pritisnuto i zbija se, otpor tla raste i posle nekog intervala pomeranja zida, u tlu se javlja maksimalna vrednost horizontalnog pritiska tla.

where:

p_{hi} – horizontal pressure acting upon the flexible diaphragm wall in the observed point *i*;

y_i – horizontal elastic displacement of the wall or the ground observed point *i*;

k_{hi} – soil reaction coefficient in *i* in the horizontal direction during the compaction of soil.

Calculation methods based on Winkler's model (Figure 1) are confined to relatively small deformations at the bottom of the construction pit. Larger displacement requires non-linear load-displacement relationship to be introduced. To indicate the actual behaviour of the embedded part of the diaphragm wall, as well as the structure-soil interaction, it is necessary to include plastic soil-deformations, i.e. soil needs to be treated as a nonlinear deformable medium [1]. The soil resistance is described by the soil reaction coefficient which changes with depth [2].

According to the proposed procedure C. Vujicic, calculating the horizontal displacement of the diaphragm wall does not require assumptions about the shape and size of the diaphragm wall deformation, that is, the calculation procedure does not require to assume that the wall is loaded as a result of the action of the soil behind the wall. Calculation of the diaphragm wall based on this procedure requires only defining the elastoplastic properties of soil under pressure and tension along the diaphragm-soil contact surface. The stress state in soil is determined by the limit equilibrium theory based on Coulomb's theory. If the mass of the soil is stable, we say that it is dominated by horizontal soil pressure at rest. Today, it is possible to identify the exact relationship between the vertical stress in soil and this horizontal soil pressure at rest, with sufficient accuracy using experimental measurements and calculation procedures. If the wall begins to detach from the soil, the soil behind the wall is allowed to expand, and the lateral soil pressure gradually decreases. After some finite wall displacement intervals, a minimum value of horizontal pressure occurs in the soil. If the wall is "deflected" into soil, it is pressed and compacted, its resistance grows and some interval after the displacement of wall the maximum value of the horizontal soil pressure occurs in the soil.

$$EI \cdot \frac{d^4 y}{dt^4} = q(t)\,; \qquad q(t) = p_d - p_l\,, \tag{2}$$

gde je:

y - nepoznata funkcija kojom se opisuje elastična linija posmatrane savitljive dijafragme;

$q(t)$ - funkcija kojom je data promena ukupnog poprečnog opterećenja posmatrane fleksibilne dijafragme;

p_d i p_l - horizontalni pritisak tla s leve, odnosno desne strane zida, koji se sastoji od stalnog i promenljivog člana, jeste:

where:

y – unknown function describing the elastic line of the observed flexible diaphragm wall;

$q (t)$ – function which describes the changes in overall lateral loading of the observed flexible diaphragm wall;

p_d and p_l – horizontal soil pressure on the left and right side of the wall consisting of a constant and a variable member:

$$p_i = p_{HO} + p_{HK}\,; \tag{3}$$

gde je:

p_i - horizontalni pritisak tla s leve, odnosno desne strane zida;

p_{HO} - poznat zakon opterećenja tla na posmatranoj dubini, horizontalni pritisak tla u stanju mirovanja;

p_{HK} - promena horizontalnog pritiska tla u zavisnosti od bočnog pomeranja zida.

Za rešenje ovog problema najpogodnija je primena numeričkog postupka. Diferencijalna jednačina rešava se metodom konačnih razlika, tj. diferencnom metodom i dobija oblik:

where:

p_i – horizontal soil pressure on the left or right side of the wall;

p_{HO} – the known law of soil load on the observed depth, horizontal soil pressure at rest;

p_{HK} – change in the horizontal soil pressure, depending on the lateral displacement of the wall.

Numerical procedure is the best way of resolving this problem. The differential equation is to be resolved using the finite difference method (i.e. the differential method); it has the following form:

$$EI \cdot \frac{d^4 y}{dt^4} - k_h \cdot y = p_{HO}(t) \tag{4}$$

U ovoj jednačini nepoznato je:

– jednačina elastične linije $y(t)$, i
– zakon promene reakcije tla k_h.

Pretpostavljeno je da deformacije zida prate deformacije tla. Elastična linija zida poklapa se sa horizontalnim pomeranjima tla. Prema tome, izjednačavanjem tih vrednosti, određuje se njihov uzajamni uticaj. Od uzajamnog uticaja, deformacije zida i deformacije tla, zavisi i raspored pritisaka tla na kontaktu zida i tla i obratno – tla i zida [6].

Predloženi postupak proračuna Čedomira Vujičića [13] zahteva samo usvajanje koeficijenta k_h reakcije tla, koji se može odrediti sa zadovoljavajućom tačnošću, a kojim se – u ovom slučaju – idealizuju elastična svojstva tla u području od aktivnog pritiska tla do pasivnog pritiska (otpora) tla, tj. ceo interval elastičnih napona u tlu (slika 3a.).

Zakon promene horizontalne reakcije tla usvojen je kako pri zbijanju tla, tako i pri njegovom širenju, po Vinklerovoj hipotezi, pa je na taj način uspostavljena veza između pritisaka tla i horizontalnih pomeranja zida (slika 3b.).

Postupak se sastoji u tome da se problem, definisan diferencijalnom jednačinom, svede na sistem algebarskih jednačina. Za linearnu diferencijalnu jednačinu četvrtog reda, diferencijalni količnici do četvrtog reda, izraženi preko odnosa konačnih veličina, imaju sledeće oblike (slika 4.):

In this equation the following is unknown:

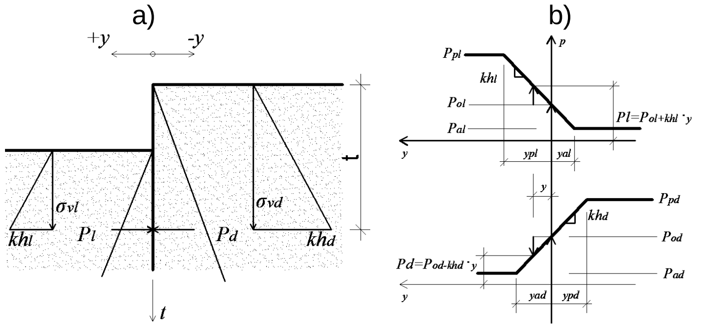
– The equation for the elastic line $y(t)$, and
– The law of change in soil reaction k_h.

Wall deformations are assumed to follow soil deformations. The elastic line of the wall is in the agreement with the horizontal soil displacements. Thus, by equalizing these values, their mutual influence can be determined. This mutual influence, i.e. wall deformation and soil deformation, defines the distribution of soil pressure on the wall-soil and soil-wall interface [6].

The calculation method proposed by Čedomir Vujicic [13] requires only the soil reaction coefficient k_h to be adopted, which can be measured with sufficient accuracy, and by which, in this case, the elastic properties of soil are idealized in the area between the active and passive soil pressure (resistance), i.e. the whole interval of elastic stresses in soil (Figure 3a).

The law of changes in the horizontal soil reaction is adopted both for the case of compaction and for the case of spread of the soil, according to Winkler's hypothesis, establishing thereby a relation between soil pressure and the horizontal displacement of the wall (Figure 3b).

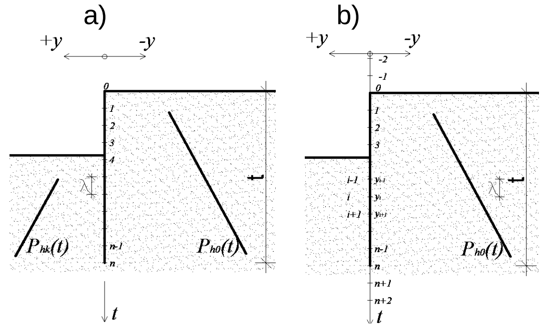
The differential equation is solved by applying the method of finite differences, i.e. the differential method. The procedure consists of reducing the problem defined by the differential equation to a system of algebraic equations. For a fourth order linear differential equation, the differential quotients up to the fourth order, as expressed through the ratio of finite dimensions, are the following (Figure 4):



a) Promena opterećenja
Change of loading

b) Zakon promene pritiska tla
Law of soil pressure change

Slika 3. Promena ukupnog opterećenja savitljivog zida
Figure 3. Changes in the total load of the flexible wall



a) Podeone tačke
Parting points

b) Indeks funkcije
Function index

Slika 4. Podeoni elementi–diferencna metoda
Figure 4. Parting elements-differential method

$$\frac{dy}{dt} = -\frac{y_{i-1} - y_{i+1}}{2 \cdot l}; \tag{5}$$

$$\frac{d^2y}{dt^2} = \frac{y_{i-1} - 2y_i + y_{i+1}}{l^2}; \tag{6}$$

$$\frac{d^3y}{dt^3} = -\frac{y_{i-2} - 2y_{i-1} + 2y_{i+1} + y_{i+2}}{2l^3}; \tag{7}$$

$$\frac{d^4y}{dt^4} = \frac{y_{i-2} - 4y_{i-1} + 6y_i - 4y_{i+1} + y_{i+2}}{l^4}; \tag{8}$$

Odakle sledi:

Hence, it follows that:

$$EI\,\frac{y_{i-2} - 4y_{i-1} + 6y_i - 4y_{i+1} + y_{i+2}}{l^4} + k_{hi} \cdot y_i = p_{HOi}(t) \tag{9}$$

Ili

or:

$$y_{i-2} - 4y_{i-1} + 6y_i - 4y_{i+1} + y_{i+2} + \frac{l^4}{EI} \cdot k_{hi} \cdot y_i = \frac{l^4}{EI} \cdot p_{HOi} \tag{10}$$

Prema tome, sistem linearnih jednačina, iz kojih se dobijaju nepoznata pomeranja y_i, odnosno nepoznate veličine i raspored horizontalnih pritisaka tla na dijafragmu, ima sledeći oblik:

Therefore, the system of linear equations, from which the unknown displacements y_i, i.e. the unknown dimensions and distribution of horizontal soil pressures acting upon the diaphragm wall are obtained, is as follows:

$$
\begin{aligned}
&2y_0 - 4y_1 + 2y_2 && + k_{h0} \cdot y_0 \cdot \frac{1}{EI} = \frac{1}{EI} p_{HO0} \\
&-2y_0 + 5y_1 - 4y_2 + y_3 && + k_{h1} \cdot y_1 \cdot \frac{1}{EI} = \frac{1}{EI} p_{HO1} \\
&y_0 - 4y_1 + 6y_2 - 4y_3 + y_4 && + k_{h2} \cdot y_2 \cdot \frac{1}{EI} = \frac{1}{EI} p_{HO2} \\
&y_1 - 4y_2 + 6y_3 - 4y_4 + y_5 && + k_{h3} \cdot y_3 \cdot \frac{1}{EI} = \frac{1}{EI} p_{HO3} \\
&y_2 - 4y_3 + 6y_4 - 4y_5 + y_6 && + k_{h4} \cdot y_4 \cdot \frac{1}{EI} = \frac{1}{EI} p_{HO4} \\
&y_3 - 4y_4 + 6y_5 - 4y_6 + y_7 && + k_{h5} \cdot y_5 \cdot \frac{1}{EI} = \frac{1}{EI} p_{HO5} \\
&y_4 - 4y_5 + 6y_6 - 4y_7 + y_8 && + k_{h6} \cdot y_6 \cdot \frac{1}{EI} = \frac{1}{EI} p_{HO6} \\
&y_5 - 4y_6 + 6y_7 - 4y_8 + y_9 && + k_{h7} \cdot y_7 \cdot \frac{1}{EI} = \frac{1}{EI} p_{HO7} \\
&y_6 - 4y_7 + 6y_8 - 4y_9 + y_{10} && + k_{h8} \cdot y_8 \cdot \frac{1}{EI} = \frac{1}{EI} p_{HO8} \\
&y_7 - 4y_8 + 5y_9 - 2y_{10} && + k_{h9} \cdot y_9 \cdot \frac{1}{EI} = \frac{1}{EI} p_{HO9} \\
&2y_8 - 4y_9 + 2y_{10} && + k_{h10} \cdot y_{10} \cdot \frac{1}{EI} = \frac{1}{EI} p_{HO10}
\end{aligned}
\tag{11}
$$

Pri ispisivanju jednačina za tačke 0, 1, n-1 i n, pojavljuju se i ordinate elastične linije u tačkama koje se nalaze izvan dijafragme, na odstojanju l i $2 \cdot l$ od krajeva zida. Jednačine za ove tačke glase:

When writing the equations for points 0, 1, *n*-1 and *n*, ordinates of the elastic line also occur in points outside the diaphragm at the distances λ and 2λ from the wall ends. The equations for these points are as follows:

za tačku 0:

for the point 0:

$$y_{-2} - 4y_{-1} + 6y_0 - 4y_1 + y_2 + \frac{l^4}{EI} \cdot k_{h0} \cdot y_0 = \frac{l^4}{EI} \cdot p_{HO0}, \tag{12}$$

za tačku 1:

for the point 1:

$$y_{-1} - 4y_0 + 6y_1 - 4y_2 + y_3 + \frac{l^4}{EI} \cdot k_{h1} \cdot y_1 = \frac{l^4}{EI} \cdot p_{HO1}, \tag{13}$$

za tačku n-1:

for the point n-1:

$$y_{n-3} - 4y_{n-2} + 6y_{n-1} - 4y_n + y_{n+1} + \frac{l^4}{EI} \cdot k_{hn-1} \cdot y_{n-1} = \frac{l^4}{EI} \cdot p_{HOn-1}, \tag{14}$$

za tačku n:

for the point n:

$$y_{n-2} - 4y_{n-1} + 6y_n - 4y_{n+1} + y_{n+2} + \frac{l^4}{EI} \cdot k_{hn} \cdot y_n = \frac{l^4}{EI} \cdot p_{HOn}. \tag{15}$$

Kada su se u nekim tačkama kontakta tla i dijafragme jave naponi zatezanja, proračun se ponavlja, usvajajući da je u tim tačkama pritisak tla na dijafragmu jednak nuli, tj. zona zatezanje se isključuje iz proračuna

When tensile stresses occur in some points of soil-diaphragm wall interface, the calculation should be repeated, assuming the soil pressure acting upon the diaphragm wall in these points is zero, i.e. the tension

[3]. Ukoliko izračunati pritisci tla dostignu granične napone i prelaze u nelinearno područje, proračun se ponavlja, usvajajući u tim tačkama vrednosti graničnih pritisaka tla na dijafragmu [13].

4 NUMERIČKI PRISTUP I METODA KONAČNIH ELEMENATA

Pri proračunu armiranobetonskih fleksibilnih dijafragmi metodom konačnih elemenata, potrebno je definisati zakon ponašanja, napon–deformacija, tla (slika 2.). Određivanje ove veze podrazumeva poznavanje modula elastičnosti tla, i zakona njegove promene sve do sloma tla. Autori su se problematikom numeričkog modeliranja i naprednim modelima tla detaljnije bavili u ranije objavljenim radovima [1], [6-9]. U ovom radu dat je kraći osvrt na osnovne podele i svojstva modela tla, dostupnih u programu „Plaxis V8". Ovaj program razvijen je i namenjen isključivo za proračune podzemnih građevina (tunela, potpornih zidova, sidrenih konstrukcija i slično).

Klasične metode proračuna zasnivaju se na velikom broju pretpostavki na osnovu kojih se definiše raspodela aktivnog i pasivnog pritiska tla na zid, te način određivanja sila u sidrima. One proizlaze iz velikog uticaja interakcije tla i konstrukcije, izrazito nelinearnog ponašanja tla, značajne promene stanja napona u tlu usled iskopa, uticaja redosleda ugradnje sidara (ankera) i iskopa građevinske jame. Ugradnja sidara predstavlja poseban inženjerski izazov, koji je opterećen kako tehničkim zahtevima tako i legilslativom. U tehničkom pogledu posebnu pažnju treba posvetiti zoni sidrenja, koja preuzima opterećenja i prenosi ih u okolnu stensku masu, izazivajući u njoj sekundarna naponska stanja sa neposrednim uticajem na razvoj postojećih i formiranje novih pukotinskih sistema. Poseban problem u takvim slučajevima je postojanje podzemne vode, koja u novonastalim uslovima formira novu hidrološku mrežu (novonastale pukotine postaju novi vodeni putevi), stvarajući uslove za hemijsku dekompoziciju i fizičku dezintegraciju stenske mase, istovremeno narušavajući uspostavljenu hidrogeološku sliku i ravnotežu u masivu. U konačnici se u funkciji od vremena može očekivati narušavanje stabilnosti padine duboko u njenom zaleđu. U pogledu legislativnih ograničenja, ugradnja sidara zahteva saglasnost vlasnika susednih objekata, koji direktno mogu da trpe posledice ugradnje ovih sidara. Stoga, potrebno je sagledati i dati procene uticaja sidra na susedne objekte u toku njihove instalacije i eksploatacije.

Savremeni pristup numeričkom modeliranju usko je povezan s razvojem istraživanja na području:

– naprednih konstitutivnih modela tla;
– savremene teorije mehanike tla;
– savremenih metoda ispitivanja tla (laboratorijska + *in situ*);
– praćenja i proučavanja izvedenih zaštitnih konstrukcija.

Uska povezanost navedenih područja glavni je preduslov za izvođenje:

– povratnih analiza;
– parametarskih analiza;
– uporednih analiza.

zone is excluded from the calculation [3]. If the calculated soil pressures reach the limit stresses and move to the nonlinear zone, the calculation should be repeated, adopting in these points the values of limit soil pressures acting upon the diaphragm wall [13].

4 NUMERICAL APPROACH AND THE FINITE ELEMENT METHOD

Using the finite element method when calculating RC flexible diaphragm walls requires defining the law of stress-strain behaviour of the soil (Figure 2). Determination of this relationship implies knowing the elasticity modulus of soil and the laws of its changes until the soil failure. The authors have pursued the issue of numerical modelling and advanced soil models in more details in their previously published papers [1], [6-9]. This paper provides a brief overview of the basic division and properties of soil models available in the "Plaxis V8" software package. This program has been developed and designed exclusively for calculating underground structures (tunnels, retaining walls, anchored structures, etc.).

Conventional methods of calculation are based on a large number of assumptions based on which the distribution of active and passive soil pressure on the wall are defined, as well as the way of determining forces in the anchors. These arise from the huge impact of the soil-structure interaction, highly non-linear soil behaviour, and significant changes in the stress state in soil due to the excavation, the impact of the order in which anchors are installed and the excavation of the construction pit. Installing the anchors represents a specific engineering challenge, which is a result of both technical and legislation requirements. Technically, special attention should be focused on the anchoring zone, which takes up the load and transfers it to the surrounding rock mass causing secondary stress state, stimulating thereby the development of existing and creation of new crack systems. A particular problem in these cases is the presence of groundwater, which forms a new hydrological network (the emerging cracks become new waterways), facilitating the chemical decomposition and physical disintegration of rock masses and disturbing the established hydro-geological image and balance of the massif. Ultimately, as a function of time, slope instability can be expected to occur in the backing. In terms of legislative restrictions, installing the anchors requires the consent of the owners of neighbouring buildings, which can directly suffer from these anchors. Therefore, during the installation of anchors and their operation it is necessary to consider and assess their impact on neighbouring buildings.

The modern approach to numerical modelling is closely related to the development of research in the following fields:

– advanced constitutive soil models,
– modern theories of soil mechanics,
– modern methods of soil testing (in laboratory and *in situ*)
– monitoring and studying the constructed protective structures.

The close relation between the above areas is a major pre-requisite for performing:

Zbog izrazito nelinearnog i plastičnog ponašanja tla do danas još uvek nije razvijen konstitutivni model koji bi opisivao sve faze–elemente realnog ponašanja tla. Pojedini savremeni modeli tla mogu se, međutim, vrlo pouzdano koristiti za opisivanje pojedinih vrsta tla [5]. Numeričko geotehničko modeliranje i analize inženjerskih problema, u Plaxis-u, obuhvataju konstitutivne modele, od najosnovnijih do najnaprednijih, kojima se simulira linearno ili nelinearno, vremenski zavisno i anizotropno ponašanja tla ili stene. Konstitutivni modeli tla se dele na:

a) Elastične modele: 1) Linearno-elastični model (LE), (slika 5a); 2) Duncan-Chang model (nelinearni hiperbolični elastični modeli); 3) Anizotropno elastični model (model ispucale stene).

b) Plastične modele:1) Mohr-Coulomb-ov model (slika 5b); 2) Drucker Prager-ov model; 3) Von Mises-ov model; 4) Tresca model, od kojih je najjednostavniji i s najvećom primenom u geotehnici Mohr–Coulomb-ov model.

c) Elasto-plastične modele: 1) Idealno elastični – idealno plastični model; 2) Cam Clay i modificirani Cam Clay model; 3) Hardening soil model tla HS (slika 5c); [11].

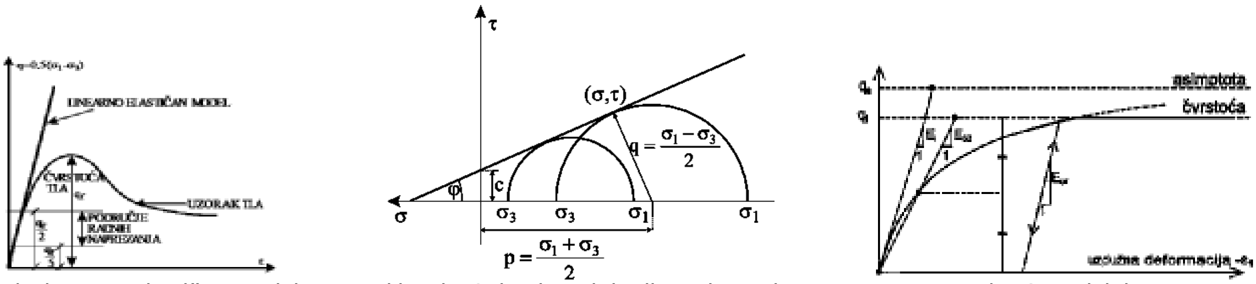
– feedback analyses,
– parametric analyses and
– comparative analyses.

Thus, due to the highly nonlinear and plastic behaviour of the soil, a constitutive model that describes all the phases-elements of the actual soil behaviour has not yet been developed. However, some contemporary soil models may reliably be used to describe certain types of soils [5]. Numerical-geotechnical modelling and analysis of engineering problems in Plaxis includes both the most basic and advanced constitutive models that simulate linear or non-linear, time dependent and anisotropic soil and rock behaviour. Constitutive soil models are grouped as follows:

a) Elastic models: 1) linear-elastic model (LE) (Fig. 5a); 2) the Duncan-Chang model (nonlinear hyperbolic elastic model); 3) Anisotropic elastic model (cracked rock model).

b) Plastic models: 1) the Mohr-Coulomb model (Figure 5b); 2) the Drucker Prager model; 3) the Von Mises model; and 4) the Tresca model, of which the Mohr-Coulomb model is the simplest and have widespread application in geotechnical engineering.

c) Elasto-plastic models: 1) The ideally elastic-ideally plastic model; 2) the Cam Clay and the modified Cam Clay model; 3) Hardening soil model HS (Figure 5c) [11].



a) Linearno-elastičan model
Linear-elastic model

b)Mohr Colomb-ov kriterijum sloma tla
The Mohr Colombov soil failure criterion

c) HS model tla
HS soil model

Slika 5. Modeli tla
Figure 5. Soil models

Modeliranje u Plaxis-u sastoji se od dva osnovna koraka:

1. Određivanje početnog stanja naprezanja u steni/tlu, na osnovu laboratorijskih ispitivanja uzoraka i inženjersko-geoloških podataka.

2. Simulacija iskopa građevinske jame ili neke druge geotehničke građevine, proračun novonastalog stanja napona i deformacija, [6] i [10].

Osnovne faze rada u numeričkom modeliranju:

1. Analiza problema (gustoća mreže, tipovi elemenata);
2. Izbor odgovarajućeg konstitutivnog modela;
3. Određivanje geomehaničkih karakteristika za odabrani konstitutivni model;
4. Određivanje graničnih uslova i opterećenja;
5. Analiza rezultata, [6].

Modelling in the Plaxis software consists of two main steps:

1. Determining the initial stress state in the rock/soil based on laboratory tests and engineering and geological data.

2. Simulation of excavation and excavation pit or some other geotechnical structure, calculating the newly emerging stress-strain state, [6] and [10].

Major phases of numerical modelling:

1. Analyzing the problem (network density, types of elements);
2. Selecting an appropriate constitutive model;
3. Identifying the geotechnical properties of the selected constitutive model;
4. Indentifying boundary conditions and loads;
5. Analyzing the results [6].

5 ULAZNI PARAMETRI I PRORAČUNSKI MODEL

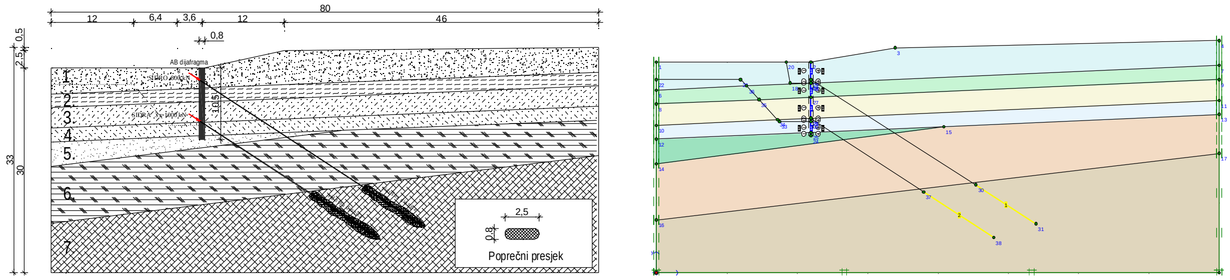
Pri izgradnji staničnog platoa železničke stanice „Beograd”, u Beogradu, projektovano je zasecanje padine u neposrednoj blizini već izgrađenih objekata. Visine zaseka su i preko deset metara. Ovakvi zaseci uslovljavaju obimne geotehničke mere obezbeđenja padine. S obzirom na uslove na terenu, dinamiku građenja potporne konstrukcije i omogućavanje sigurnijih uslova za registraciju rezultata merenja, a imajući u vidu finansijska ograničenja, za ispitivanje je odabrana karakteristična lamela 18D. Proračunski i numerički model ispitivane lamele 18D, geometrijski elementi potporne konstrukcije vertikalnog zaseka za plato nove železničke stanice u Beogradu i profil tla, dati su na slici 6. Aktivna (sadejstvujuća) širina jedne lamele je 4.0 m. Prostor između dve susedne lamele podgrađen je prefabrikovanim elementima.

Tlo koje se zaseca sastoji se, generalno rečeno, od gornjih lesoidnih slojeva, ispod kojih su laporovite sredine, koje leže na proslojcima glinovitih laporovitih krečnjačkih naslaga, a ispod njih je krečnjak „pužarac”. Nagibi slojeva prate nagib padine (nepovoljna orijentacija). Geotehničkim terenskim istražnim radovima nije utvrđeno postojanje podzemnih voda u padini, što bi komplikovalo radove pri izgradnji dijafragmi na ovom lokalitetu.

5 INPUT PARAMETERS AND THE CALCULATION MODEL

During the construction of the station plateau of the Belgrade railway station, it was anticipated to cut up the slope in the vicinity of already existing buildings. At some points, the cuts were more than 10 meters high. Such cuts require extensive geotechnical measures due to secure the slope. Given the conditions on the field, the pace of constructing the retaining structure and thus providing safer conditions for the interpretation of measurement results while bearing in mind the financial constraints, the characteristic strip 18D was selected for testing purposes. The calculation and numerical model of the tested strip 18D, the geometric elements of retaining structure of the vertical cut for the plateau of the new railway station in Belgrade and the soil profile are shown in Figure 6. The active (effective) width of the strip is 4.0 m. The space between two adjacent bays was shored by precast elements.

The soil generally consisted of upper loess layers, followed by marly environments, sitting on interbeds consisting of clay, marl and limestone; below these is limestone with gastropod dominance. The gradient of layers follows the gradient of the slope (unfavourable orientation). Geotechnical in situ investigations did not reveal the presence of groundwater in the slope that would contribute to the complexity of construction works on this site.



Slika 6. Proračunski i numerički model ispitivane lamele 18D
Figure 6. Calculation and numerical model of the tested strip 18D

Prethodnim istražnim radovima, na padini su registrovani tragovi umirenih klizišta. Vodeći računa o važnosti objekta za koji se plato gradi, kao i o objektima na padini iznad zaseka (niz novoizgrađenih stambenih objekata, plitko fundiranih), za obezbeđenje padine usvojene su armirane betonske dijafragme sa zategama. Baze dijafragmi ugrađuju se u laporovito-krečnjački kompleks, ili u krečnjak, a zatege dijafragmi sidre se u krečnjak „pužarac”. Prethodni geotehnički ogledi u laboratoriji, kao i rekognosciranje profila tla u toku iskopa, omogućili su da se detaljno prouče mehaničke i fizičke karakteristike tla, te da se usvoje vrednosti parametara tla za proračun [13]. Geomehaničke karakteristike tla za proračun i modeliranje date su u Tabeli 1.

Neophodno je napomenuti da su za potrebe numeričke analize, usled nedostatka potpunijih geomehaničkih karakteristika tla, usvojeni neophodni parametri tla na osnovu iskustva i podataka iz dostupne literature, a u celosti su dati u tabelama: 2, 3, 4 i 5.

During the preliminary investigations, traces of pacified landslides were found on the slope. Taking into account the importance of the facility to which the plateau belongs, as well as the buildings on the slope above the cut (a row of newly constructed residential buildings with shallow foundations), it was adopted to secure the slope by using a reinforced concrete diaphragm wall with tie rods. Bases of diaphragm walls are implanted into the marly-limestone complex, or limestone, while tie rods of the diaphragm wall are anchored in the limestone with gastropod dominance. Preliminary geotechnical experiments in laboratory, as well as the reconnaissance of the soil profile during excavation made it possible to study in details the mechanical and physical properties of the soil, and adopt the values of soil calculation parameters [13]. Geotechnical properties of soil for the calculation and modelling are given in Table 1.

It should be noted that for the purposes of numerical analysis, and due to the lack of more complete geotechnical properties of soil, the necessary soil parameters were adopted based on experience and data from the available literature; these are presented in full in Tables 2, 3, 4 and 5.

Tabela 1. Geomehničke karakteristike tla – preuzeto iz [13]
Table 1. Geotechnical properties of soil, after [13]

R. br.	Sloj - *Layer*	g_{unstat}	E_{ref}	ν	c_{ref}	φ	*kh*
		[kN/m³]	*[kN/m²]*	-	*[kN/m²]*	°	*[kN/m³]*
1	Les/*Loess*	18,5	13000	0,35	20	20	15000
2	Lesoidni sedimenti/*Loess sediments*	18,5	34000	0,35	20	16	25000
3	Glinoviti sedimenti/*Clay sediments*	19	34000	0,35	30	16	30000
4	Gline i glinoviti laporci/*Clays and marly clays*	20	40000	0,30	30	15	50000
5	Krečnjak sa proslojcima lapora/*Limestone with interbeds of marl*	20	100000	0,30	200	25	150000
6	Laporac sa proslojcima glinovitog lapora/*Limestone with in. of marly clay*	19	120000	0,25	150	20	-
7	Krečnjak „pužarac"/*Limestone with gastropod dominance*						

Tabela 2. Model tla sa ojačanjem (HS)
Table 2. Hardening soil model (HS)

Sloj	*Type*	g_{unstat}	g_{sat}	k_x	k_y	E_{50}^{ref}	E_{oed}^{ref}	E_{ur}^{ref}	c_{ref}	φ	ν_{ur}	K_0^{nc}
		[kN/m³]	*[kN/m³]*	*[m/day]*	*[m/day]*	*[kN/m²]*	*[kN/m²]*	*[kN/m²]*	*[kN/m²]*	°	-	-
1	Drained	18,5	18,5	0.001	0.001	13000	13000	39000	20	20	0,35	0,6
2	Drained	18,5	18,5	0.001	0.001	34000	34000	102000	20	16	0,35	0,59
3	Drained	19	19	$1,010^{-6}$	$1,010^{-6}$	34000	34000	102000	30	16	0,35	0,6
4	Drained	20	20	$1,010^{-6}$	$1,010^{-6}$	40000	40000	120000	30	15	0,30	0,61
5	Drained	20	20	$1,010^{-6}$	$1,010^{-6}$	100000	100000	300000	200	25	0,30	0,47
6	Drained	19	19	$1,010^{-6}$	$1,010^{-6}$	120000	120000	360000	150	20	0,25	0,53
7	Drained	20	20	0	0	250000	250000	750000	250	35	0,20	0,35

Tabela 3. Parametri AB dijafragme
Table 3. Parameters of RC diaphragms wall

ID	Naziv *Name*	Tip Type	EA (kN/m)	EI (kNm²/m)	w (kNm²/m)	v (-)	Mp (kNm/m)	Np (kN/m)
1	Dijafragma/diaphragm2,5x0,8m	Elastic	3.47E+07	1,852E6	4.0	0.15	1.00E+15	1.00E+15

Tabela 4. Parametri ankera
Table 4. Anchor parameters

ID	Naziv *Name*	EA (kN/m)	L (m)	Mp (kNm/m)	Np (kN/m)
1	Sidro/*Anchor*	6.000E+04	1.0	1.00E+15	1.00E+15

Tabela 5. Parametri sidrišne zone
Table 5. Anchorage zone parameters

ID	Naziv *Name*	EA (kN/m)	Np (kN/m)
1	Sidrišna (*Anchrage*) zona	2,88E+06	1.00E+10

Na dužini zaseka padine se pre njenog zasecanja ugrađuju nezavisne armirane betonske lamele dijafragme. Osovinski razmak ovakvih nezavisnih lamela iznosi od 6 m do 8 m; one se na gornjem kraju prihvataju zategama i međusobno su vezane gredom (poklapačom), u visini njihovih glava (slika 7.). Prazan prostor, između nezavisnih lamela, zatvara se montažnim AB elementima. Na taj način, povećana je krutost potporne građevine, a postignuta je i veća lokalna sigurnost zaseka [13].

Radovi ugradnje elemenata zaštitne konstrukcije i iskopa izvođeni su po fazama (slika 8.):

Faza 0. Ugradnja dijafragme dimenzija 80 x 250 cm do dubine 10,5 m;

Faza 1. Iskop do kote - 2.5 m;

Faza 2. Ugradnja i prednaprezanje prvog reda sidara/ankera na koti -2.0 m; F=800 kN/m', u nagibu 1:1.5;

Faza 3. Iskop do kote -8.5 m;

Faza 4. Ugradnja i prednaprezanje drugog reda sidara na koti -8.0 m;F=3x1000 kN/m', u nagibu 1:1.5.

Before cutting up, independent reinforced concrete diaphragm strips were inserted along the length of the slope. The axis spacing these independent strips are the order of magnitude of 6-8 m; at their upper ends, tie rods are fixed and mutually connected by a beam (yoke), at the height of their heads (Figure 7). The empty space between the independent strips is closed with prefabricated RC elements. In this way, the stiffness of retaining structure is increased, and a higher local security of cuts is achieved [13].

Works on the installation of protective elements and excavation were carried out in phases as follows (Fig. 8):

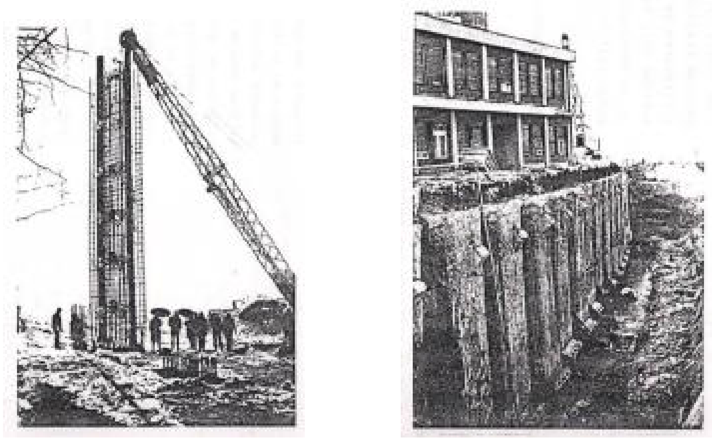
Phase 0: Installing the diaphragm wall dimensions 80×250 cm

Phase 1: Excavating to the level of - 2.5 m,

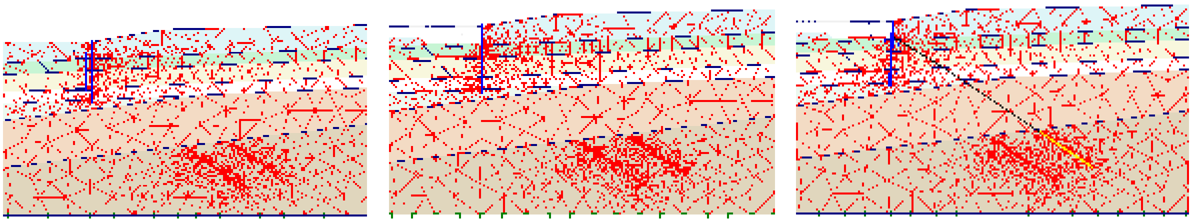
Phase 2: Installing and prestressing the first row of anchors at a level of -2.0 m; F = 800 kN/m², with a gradient of 1:1.5,

Phase 3: Excavating to the level of - 8.5 m,

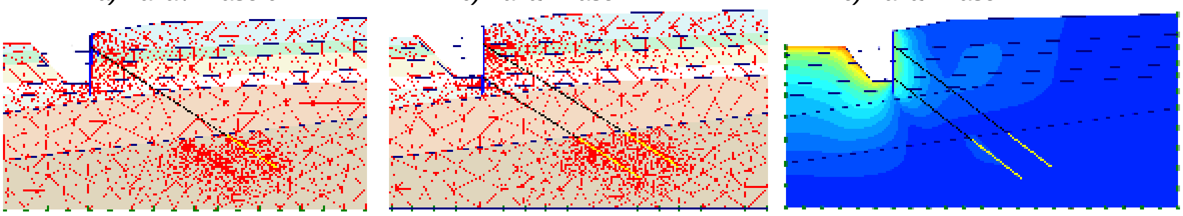
Phase 4: Installing and prestressing the second row of anchors at a level of -8.0 m; F = 3x1000 kN/m², with a gradient of 1:1.5.



Slika 7. Izgradnja staničnog platoa železničke stanice „Beograd”, u Beogradu
Figure 7. Construction of plateau of the "Belgrade" railway station in Belgrade



a) Faza /Phase 0 *b) Faza/Phase I* *c) Faza/Phase II*



d) Faza/Phase III *e) Faza/Phase IV* *f) Ukupna pomeranja/Overall displacements*

Slika 8. Faze izgradnje
Figure 8: Construction phases

6 METODOLOGIJA MERENJA I MERNA OPREMA

Dosadašnja i savremena saznanja o interakciji potporne konstrukcije i tla još uvek uslovljavaju nova ispitivanja i istraživanja, kako u domenu proračunskih modela, tako i u povratnoj analizi podataka dobijenih merenjima na već izgrađenim objektima ili onim u izgradnji. Jasno je da su veoma značajna merenja odnosno ispitivanja na terenu, koja uključuju realne uslove i njihove uticaje uz prirodne dimenzije ispitivanog objekta, a na osnovu kojih se ocenjuje tačnost pouzdanosti rezultata numeričkih modela. Međutim, merenja na gradilištu su uvek skuplja od merenja u laboratoriji, jer zahtevaju veće učešće radne snage, više opreme i materijala, pa se ređe realizuju.

6 MEASUREMENT METHODOLOGY AND EQUIPMENT

Former and current insights into the retaining structure-soil interaction still require new studies and research both in the domain of calculation models and in feedback analysis of measurement data on already constructed buildings or facilities under construction. It is clear that field measurements and tests (including the actual conditions and their impact) using the actual dimensions of the tested object are of great importance, making the basis on which the accuracy and reliability of results obtained from the numerical models. However, on-site measurements are always more expensive than laboratory measurements because they require higher participation of labour, equipment and materials; thus, they are seldom implemented.

Merenja na terenu zahtevaju izbor oblasti u kojoj se meri, određivanje podataka o tlu (parametara tla), obradu dijafragme po fazama građenja, načinu prihvatanja dijafragme, vremenu građenja pojedinih fazaiskopa dijafragme kao i izbor odgovarajućih mernih uređaja. Merenjem treba obuhvatiti sva dejstva/opterećenja od tla i sve deformacije i napone u zidovima potporne konstrukcije. Ona se ostvaruju ili merenjem promene dužina ivičnih vlakana, ili merenjem linija ugiba, odnosno merenjem pomeranja pojedinih tačaka konstrukcije. Podatke o merenju treba dopunjavati podacima u pogledu vremena, postupka građenja, vrste tla, podzemne vode u iskopu i slično. Radi dobijanja što vernijih podataka o tlu, u području ispitivanog elementa AB dijafragme, osim prethodno uzetih uzoraka tla, potrebno je uzimati uzorke tla i za vreme iskopa.

Na proučavanoj lameli instalirana je sledeća oprema:

– za merenje promene pritisaka tla na prednjoj i zadnjoj strani lamele korišćene su merne ćelije;

– za merenje horizontalnog i vertikalnog pomeranja glave lamele, korišćeni su reperi i standardne metode geodetskih merenja;

– za praćenje savijanja lamele projektovana je odgovarajuća oprema;

– za rotaciju glave ispitivane lamele primenjen je precizni inklinometar;

– merenja elastičnih izduženja obavljena su mernim trakama za istezanja;

– za merenje vertikalnog pomeranja tla po slojevima iza ispitivane lamele, projektovani su odgovarajući reperi;

– za praćenje promene napona u tlu oko ispitivane lamele korišćeni su merači pritisaka tla.

U okviru terenskog ispitivanja lamele, merena su i vertikalna pomeranja slojeva tla iza ispitivane lamele. Praćenje ovih pomeranja potporne konstrukcije naročito je važno zbog objekata uz iskop, pošto i oni mogu biti zahvaćeni tim pomeranjima.

Field measurements require selecting the measurement areas, determining soil data (soil parameters), processing the diaphragm wall by phases of construction, the way of fixing the diaphragm wall, the time of construction of individual trenches, as well as selecting the appropriate measuring devices. Measurements shall cover all impacts/loads from the soil and all deformations and stresses in walls of the retaining structure. These are achieved either by measuring the change in the length of edge fibres or measuring the deflection lines, i.e. the displacement of individual points of the structure. Measurement data should be supplemented by data dependent on time, construction procedure, soil type, groundwater in excavation, and the like. Obtaining soil data as valid as possible requires taking soil data also during the excavation, in addition to previously taken soil samples.

The following equipment was installed on the studied strip:

– changes in soil pressures on the front and back of the strip were measured using measuring cells,

– the horizontal and vertical displacement of the strip head were measured using benchmarks and standard methods of geodetic measurements,

– an appropriate equipment was devised for monitoring the strip deflection,

– the head of the test strip was rotated using accurate inclinometer,

– elastic elongations were measured using strain gauges for measuring elongation,

– appropriate benchmarks were designed for measuring vertical displacements of soil layers behind the test strip,

– soil pressure gauges were used for monitoring changes of strain in the soil around the test strip

When testing the strip in the field, vertical displacements of soil layers behind the test strip were also measured. Monitoring these displacements of retaining structures is particularly important because of buildings near to the excavation, as they also can be caught by these displacements.

7 REZULTATI TERENSKIH MERENJA I PRORAČUNA

Programom ispitivanja lamele 18D, u toku građenja po fazama, i u eksploataciji potporne konstrukcije, predviđeno je praćenje promene:

– naponskog stanja u tlu oko lamele;

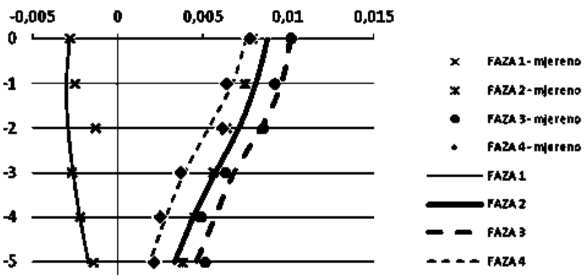
Tabela 6. Rezultati merenja horizontalnih pomeranja dijafragme na ispitivanoj lameli 18D
Table 6: Horizontal displacements obtained by measurements of the diaphragm on the tested strip 18D

	FAZA I	FAZA II	FAZA III	FAZA IV
0	-0,0028	0,0088	0,0101	0,0077
-1	-0,003	0,0081	0,0096	0,0064
-2	-0,0029	0,0071	0,0084	0,0062
-3	-0,0026	0,0057	0,0068	0,0037
-4	-0,0022	0,0044	0,0057	0,0025
-5	-0,0017	0,0033	0,0046	0,0021

7 RESULTS OF FIELD MEASUREMENTS AND CALCULATIONS

By testing the strip 18D during the phases of construction of the retaining structure and its operation, the following was monitored:

– changes of stress conditions in the soil around the strip,



Slika 9. Dijagrami horizontalnih pomeranja
Figure 9. Diagrams of horizontal displacements

– prostornog pomeranja lamele;

– vertikalnog pomeranja tla po slojevima, iza lamele [13];

Rezultati merenja horizontalnih pomeranja preuzeti su iz izvora [6], dati su u Tabeli 1, a deformacione linije dijafragme, sa usvojenim osrednjenim vrednostima, grafički su prikazane na slici 9. Iz ovih priloga se vidi da su horizontalna pomeranja lamele praćena do polovine dubine lamele (gornjih pet metara dužine lamele). Treba naglasiti da mnogo faktora utiče na rezultate merenja. U ovom slučaju isključeni su faktori, kao što su kvalitet građenja, pouzdanost opreme, sigurnost repera, uticaj vremena i atmosferilija i slično.

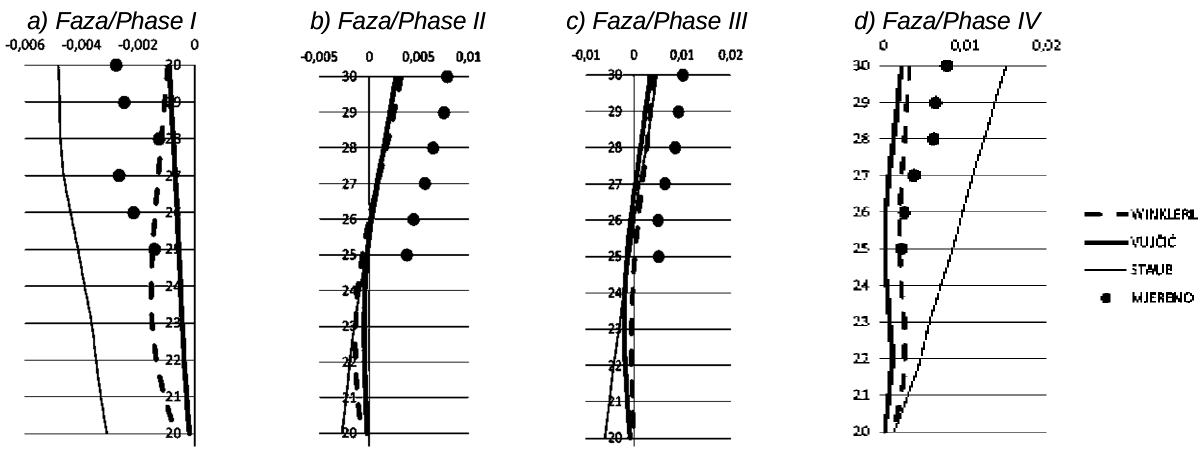
Pri projektovanju zaštitne potporne konstrukcije, primenjeno je nekoliko metoda proračuna. Korištene su metode i softverski paketi koji su bili poznati i dostupni u tom vremenu (1978). Različitim postupcima proračunate su deformacije lamele po fazama njenoga građenja. Za upoređivanje rezultata odabrani su Vinklerov postupak i proračun fleksibilnih konstrukcija prema predlogu Čedomira Vujčića u programu STRESS. Za numeričku simulaciju interakcije konstrukcije i tla, korišćen je savremeni konstitutivni model ojačanja tla (Hardening soil – HS), tj. model tla koji je vrlo često u upotrebi, a dostupan je u primeni Plaxis-a, trenutno jednog od najpouzdanijih programskih paketa koji tretira ovu oblast. Rezultati proračuna prikazani su na slici 9.

– spatial displacement of the strip, and

– vertical displacement of the soil layers behind the strip [13];

Results of measurements of horizontal displacements taken from the source [6] are presented in Table 1, while deformation lines of the diaphragm along with the adopted averaged values are presented in Figure 9. As indicated by these supplements, horizontal displacements of the strip were monitored to its half depth (the upper 5 meters of the strip). It should be noted that measurement results are affected by several factors. In this case, factors such as construction quality, equipment reliability, benchmark reliability, weather and precipitation impact, etc. were excluded from the discussion.

When designing the protective support structure several calculation methods were applied. Methods and software packages familiar and available at the time (1978) to the authors were applied. Strip deformations by phases of its construction were calculated using various methods. Winkler's procedure was adopted for comparing the results, while the calculation of flexible structures was based on a procedure proposed by C. Vujčić using the STRESS software. The structure-soil interaction was simulated numerically using the modern constitutive hardening soil model (HS), i.e. the soil model that is often used and is available when applying the Plaxis, which is now one of the most reliable software packages treating this subject. The calculation results are shown in Figure 9

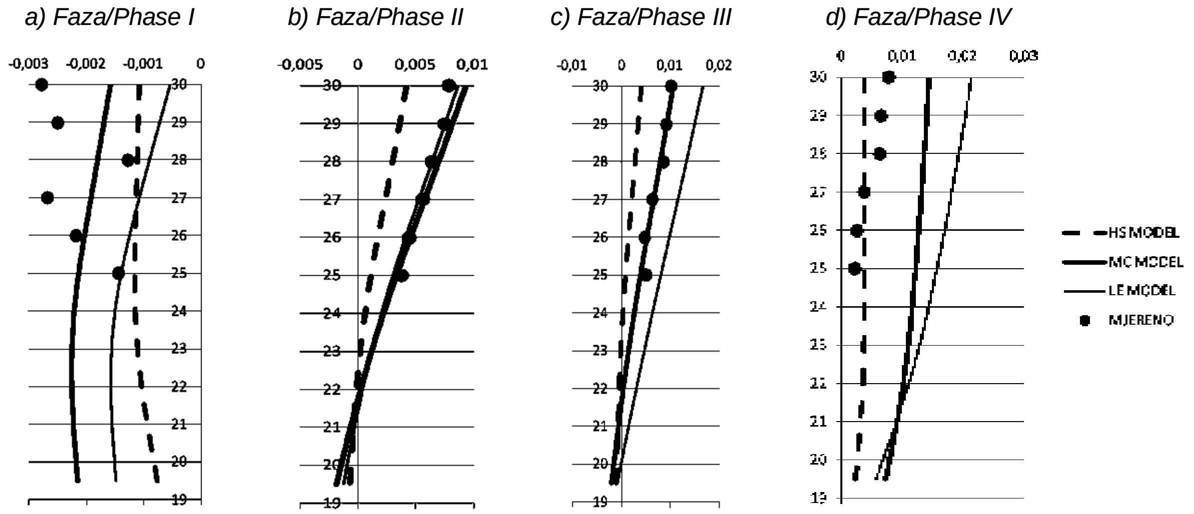


Slika 10. Rezultati metoda proračuna horizontalnih pomeranja dijafragme na ispitivanoj lameli 18D po fazama izgradnje prema izvornoj dokumentaciji [13]

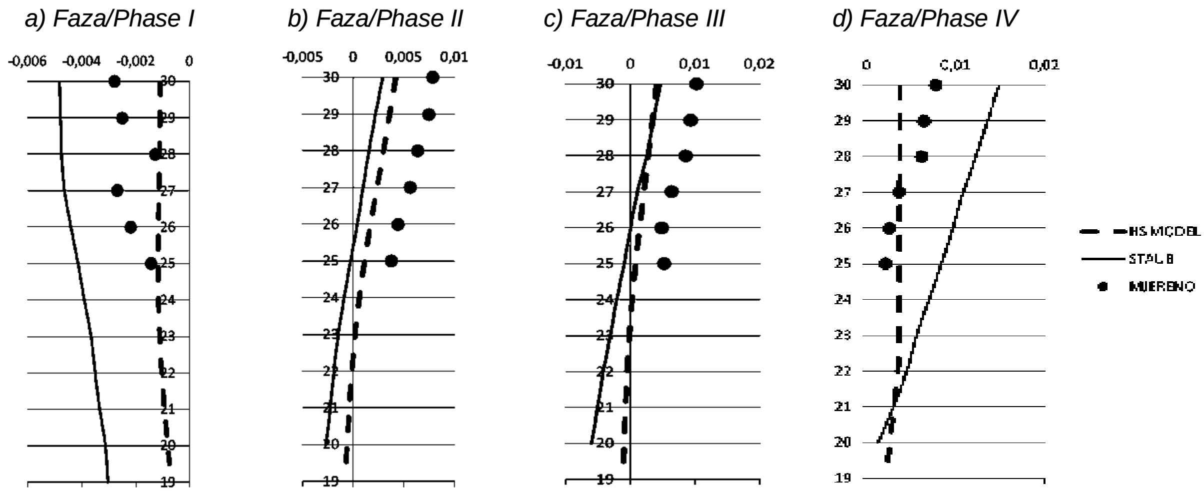
Figure 10. Results obtained by calculation procedures regarding the horizontal displacement of the diaphragm on the tested strip 18D by construction phases according the original documentation [13]

Za numeričku simulaciju interakcije konstrukcije i tla, korišćeni su savremeni konstitutivni modeli tla od kojih su danas najčešće u upotrebi: 1) linearno-elastični model; 2) Mohr–Colomb model; 3) HS model tla, sa očvršćavanjem tla (Hardening soil), slika 11.

Numerical simulation of the structure-soil interaction was based on modern constitutive soil models, of which the following are the most frequently used: 1) linear-elastic model, 2) Mohr Coulomb model, 3) hardening soil model (Figure 11).



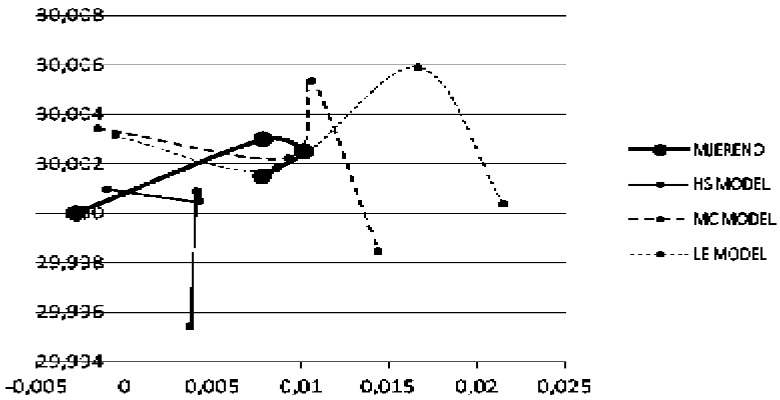
Slika 11. Poređenje rezultata proračuna horizontalnih pomeranja dijafragme po fazama izgradnje, sa izmerenim vrednostima
Figure 11. Comparing results obtained by calculations for horizontal displacement of the diaphragm by construction phases with values obtained by measurements



Slika 12. Poređenje rezultata proračuna horizontalnih pomeranja dijafragme po fazama izgradnje, po različitim metodama: MKE Staub i Plaxis (HS model), sa izmerenim vrednostima
Figure 12. Comparing results obtained by calculations for horizontal displacement of the diaphragm by construction phases, using different methods: FEM Staub and Plaxis (HS) model, values obtained by measurements

Vertikalna pomeranja glave ispitivane lamele 18D praćena su po fazama građenja potporne konstrukcije, preciznim nivelmanskim instrumentom.

Vertical displacements of the 18D test strip head were monitored by phases of construction of the retaining structure using an accurate precise levelling instrument.



Slika 13. Vertikalna pomeranja glave ispitivane lamele po fazama iskopa
Figure 13: Vertical displacements of the strip head by phases of excavation

8 KOMPARATIVNA ANALIZA REZULTATA

Horizontalna pomeranja dijafragme i pritisci tla za ispitivanu lamelu, proračunati su klasičnim i savremenim metodama proračuna. Klasični postupak proračuna napona tla svodi se na izračunavanje aktivnih i pasivnih pritisaka tla. Za izračunavanje ovih pritisaka tla, Rankinovim računskim postupkom, potrebno je poznavati zapreminske težine, parametre smicanja, ugao unutrašnjeg trenja i koheziju pojedinih slojeva tla. Ove parametre sadrže standardni geomehančki elaborati, a neki ulazni parametri se mogu odrediti prema knjizi E. Nonvajlera – *Mehanika tla i temeljenje*. Klasični postupak proračuna pomeranja i pritisaka tla na lamelu služi za dobijanje graničnih vrednosti pritisaka tla, po fazama građenja ispitivane lamele.

Metodom koeficijenta k_h – reakcije tla, tlo se zamenjuje nezavisnim oprugama koje su međusobno vezane samo dijafragmom. Ta pretpostavka je gruba aproksimacija tla. Glavni problem u primeni ove metode proračuna jeste potreba procene odnosno izbor opterećenja od tla iza dijafragme. Drugi nedostatak ovog postupka proračuna jeste nepreciznost određivanja koeficijenta k_h reakcije tla. Najčešće, za opterećenje tla na zaleđe dijafragme, usvaja se dijagram aktivnog pritiska tla. Izbor koeficijenta k_h reakcije tla i njegova promena ne utiču bitno na promene veličina naprezanja tla, ali znatno utiču na računske vrednosti horizontalnih pomeranja. Prema [11], promena koeficijenta k_h za 100%, ima za posledicu promenu momenta savijanja dijafragme za svega oko 10%.

Klasične metode proračuna upotrebljivosti konstrukcije opterećene su pretpostavkama o krutosti tla (Vinklerov koeficijent reakcije tla) i krutosti sidara, koji nisu konstantni za posmatrani materijal nego su rezultat interakcije tla i konstrukcije. Uprkos mnogim pretpostavkama i ograničenjima, klasične metode proračuna svakodnevno se primenjuju u geotehničkom projektovanju zbog brzine i jednostavnosti postupka proračuna. Rezultat proračuna primenom klasičnih metoda na strani su sigurnosti u odnosu na rezultate merenja, a pomeranja zaštitnih konstrukcija su redovno precenjena [12].

U metodi konačnih elemenata posmatra se ispitivani elemenat zajedno sa masivom tla u koji je on ugrađen. Ova metoda bi trebalo da dâ najvernije podatke o

8 COMPARATIVE ANALYSIS OF RESULTS

Horizontal displacements of the diaphragm and soil pressures for the test strip were calculated using conventional and modern methods. The conventional procedure for calculating the soil stress consists of calculating the active and passive soil pressures. When calculating these soil pressures using the Rankine calculation procedure, it is necessary to know the bulk densities, shear strength parameters, the internal friction angle and the cohesion of individual soil layers. These parameters are included by standard geotechnical studies, whereas some input parameters can be determined based on the book of E. Nonvaler (*Soil mechanics and foundations*). The classic procedure of calculating the soil displacement and pressure on the strip is used to obtain limiting values for soil pressures by phases of construction of the test strip.

In the method of soil reaction coefficient k_h, the soil is replaced with independent springs, which are mutually connected only by the diaphragm. This assumption is a rough approximation of the soil. The main problem with the application of this calculation method is the need for assessment and selection of the soil load behind the diaphragm. Another disadvantage of this calculation procedure is the inaccuracy of determining soil reaction coefficient k_h. For soil loading on the diaphragm background, the active soil pressure diagram is the most commonly adopted diagram. The choice of the soil reaction coefficient k_h and its changes does not affect significantly the changes in the soil stress magnitude but significantly affects the calculated values of horizontal displacements. According to [11], a 100% change of the coefficient k_h results in only about 10% change of the diaphragm bending moment.

Conventional calculation methods of determining structure serviceability/usability are burdened with assumptions about soil stiffness (Winkler's soil reaction coefficient) and anchor stiffness that are variable for the given material but resulting from the soil-structure interaction. Despite the many assumptions and limitations, conventional calculation methods are applied on a daily basis in geotechnical design due to the quickness and simplicity of the calculation procedure. Calculation results obtained by conventional methods are in favour of safety as opposed to the measurement deformacijama ispitivane lamele, i naponima u tlu oko lamele, pod uslovom da se pravilno ocene parametri tla. Metoda zahteva uvođenje elastičnih karakteristika pojedinih slojeva tla, u zavisnosti od promena napona i deformacija tla. Proračun ispitivane lamele MKE daje globalnu sliku pomeranja zida i tla, kao i raspored pritisaka tla u posmatranim profilima.

Primenjeni modeli tla imaju svoje prednosti, ali i nedostatke, koji bitno ograničavaju njihovu upotrebljivost za numeričku simulaciju interakcije građevine i tla. Linearno-elastični model daje nerealne deformacije potporne konstrukcije i okolnog tla, što se ispoljava u prevelikom izdizanju tla u području iskopa jame i izrazitoj rotaciji donjeg dela potporne konstrukcije prema iskopu. Mohr-Coulomb model je primenjiv pri početnoj analizi i proračunu fleksibilnih potpornih konstrukcija, jer nisu potrebni opsežni radovi kako bi se obezbedili neophodni geomehanički parametri $(\gamma, \varphi, E, \nu, c, \psi)$. Nedostatak ovog modela jeste u tome što nedovoljno dobro definiše ponašanja tla pri rasterećenju. Model tla sa ojačanjem (HS) omogućuje analizu, uz upotrebu efektivnih parametara tla, te uzima u obzir zavisnost krutosti tla od stanja naprezanja, veliku krutost pri malim deformacijama, kinematičko i izotropno očvršćivanje i sl. Osnovni parametri za HS model tla jesu: $\gamma, \varphi, E, \nu, c, \psi$. Deformabilnost tla detaljnije je definisana s tri različita ulazna parametra krutosti: modul elastičnosti iz triaksijalnog testa, E_{50}, modul elastičnosti iz triaksijalnog testa pri rasterećenju, E_{ur} i modul elastičnosti iz edometarskog testa E_{oed}, što daje realne deformacije tla u modelu u odnosu na one stvarne.

U I fazi izgradnje podgrade zaseka, posle iskopa do dubine -2,5 m za ugradnju gornjeg sidra, izmereno je povijanje ispitivane lamele ka iskopu. Kriva savijanja je nagnuta ka iskopu, ali sa konkavnom krivinom ka zaleđu. Mereno pomeranje glave lamele iznosi oko 3 mm. Pomeranje glave lamele, određeno elastoplastičnim postupcima proračuna (Vinklerov postupak i postupak proračuna Čedomira Vujčića), iznosilo je oko l mm. Izračunato pomeranje glave lamele metodom konačnih elemenata Staub jeste 6mm, Plaxis (HS model) = 1,1 mm. Idući naniže, merena pomeranja opadaju, i prema pravcu izmerene krivine savijanja ona teži ka bazi lamele, koja se može usvojiti kao nepomerljiva. Računska pomeranja postupaka proračuna, posle relativno konstantnih vrednosti pomeranja lamele, duž raspona lamele, praktično su nula u području baze lamele.

Može se zaključiti da su u I fazi građenja računske deformacije ispitivane lamele, izračunate metodom konačnih elemenata (Staub), veće od izmerenih i teško je očekivati da se mogu ostvariti u takvom obliku, u ovoj fazi građenja. U I fazi (slika 12.), iako HS model pokazuje bliskost u dve tačke u odnosu na merene rezultate, rezultati određeni po Staub postupku u ovom pogledu na strani su sigurnosti i pokazuju veću sličnost karaktera promene deformacija po dubini od onih koje su izmerene. Deformacije ispitivane lamele, izračunate u Plaxis-u i elastoplastičnim postupcima proračuna, iako su manje od izmerenih, po svom obliku više odgovaraju obliku izmerenim vrednostima. Pomeranja izmerena u I fazi imaju promenljiv karakter, s neočekivanim skokom (što je najverovatnije posledica odstupanja pri merenju) i sa dobijenim numeričkim rezultatima bitno se razlikuju po karakteru. Međusobna sličnost postoji kod numeričkih results, while displacements of protective structures are regularly overestimated [12].

In the finite element method, the tested element is considered along with the soil mass in which it is embedded. This method should provide the most realistic information on deformations of the test strip and the soil stresses around the strip, provided that soil parameters are properly assessed. This method requires the introduction of elastic properties of the individual soil layers, depending on the changes in stress and soil deformation. The calculation of the test strip using the FEM method provides a global picture regarding the wall and soil displacements, as well as regarding the soil pressure distribution in profiles under consideration.

The applied soil models have their advantages and disadvantages, which significantly limit their usability for numerical simulation of the structure-soils interaction. The linear elastic model gives unrealistic deformations of retaining structures and surrounding soil, which is reflected in the excessive elevation of soil in the pit excavation area and the extreme rotation of the lower part of the retaining structure towards the excavation. The Mohr-Coulomb model is applicable in the initial analysis and calculation of flexible retaining structures because there is no need for extensive works to provide the necessary geotechnical parameters (g, j, E, n, c, y). The disadvantage of this model is that it fails to accurately define the behaviour of soil when unloaded. The hardening soil model (HS) allows analysis performed using effective soil parameters and takes into account the dependence of soil stiffness of the stress state, high stiffness at low deformations, kinematic and isotropic hardening etc. The basic parameters of the HS soil model are g, j, E, n, c, y . Soil deformability is further defined by three different input parameters of stiffness: elastic modulus from the triaxial test, E_{50}, elastic modulus from the triaxial test at unloading, E_{ur}, and the elastic modulus from the edometric test, E_{oed}, providing the model with realistic soil deformations, as compared to actual deformations.

In construction phase 1 of shoring the cut, after excavation to a depth of -2.5m for installing the upper anchor, as indicated by measurements, the strip was deflecting towards the excavation. The deflection curve is inclined toward the excavation, but with the concave curve towards the backing. According to the measurements, the strip head was displaced for about 3 mm, while according to elastoplastic calculation procedures, the Winkler's procedure and the procedure proposed by C. Vujčić, the strip head displacement was about l mm. As determined using the finite element method (Staub), the strip head displacement was 6 mm, while using the Plaxis (HS model), this displacement was 1.1 mm. In the lower sections, displacements obtained by measurements decreased, while the deflection curve tends towards the strip base, which can be adopted as immobile. Differences between calculation procedures, after relatively constant strip displacement values along the strip span, are practically zero in the strip base area.

It can be concluded that in construction phase I, calculation-based deformations of the strip, as calculated by the finite elements method (Staub), were higher than the values obtained by measurements and they are unlikely to occur in such form at this phase of proračuna u karakteru promene deformacija po dubini, ali posebno treba naglasiti da ta sličnost postoji između MC i LE modela, s tim što je MC model rigorozniji (na strani sigurnosti, jer daje veća pomeranja – slika 11a).

U II fazi građenja, posle instalacije gornjeg reda sidara, došlo je do tečenja opterećenog tla iza zida, i konstatovano je deformisanje–ugibanje ispitivane lamele ka njenom zaleđu. Iznad zatege, na delu prepusta lamele, konkavna strana savijanja lamele okrenuta je ka iskopu, a ispod zatege konveksna strana linije savijanja okrenuta je ka iskopu, donji kraj deformacione linije dijafragme teži ka njenoj bazi. Izmereno pomeranje glave lamele ka zaleđu iznosi nešto ispod 9 mm, međutim, elastoplastični postupci proračuna, kao i Staub postupak, dali su da je ovo pomeranje oko 3 mm, dok u Plaxis-u ono iznosi 4 mm. Idući naniže, izmereno pomeranje lamele opada, i teži ka bazi lamele. Svaki računski postupak proračuna daje, u polovini visine lamele, pomeranje lamele *nula*, kao i na donjem delu lamele, krive pomeranja ka iskopu, kojima su najveće ordinate oko l mm, a idući ka bazi lamele teže nuli. Zabeleženo je veoma dobro poklapanje u vrednostima i u karakteru promena deformacija, sa izraženom tačnošću LE modela u pogledu rezultata merenja (slika 11b). Uporedni dijagram pomeranja duž cele lamele za MKE ima sličan oblik, ali su rezultati u Plaxis-u približniji izmerenim vrednostima. Na osnovu navedenog, uočljivo je da su u II fazi građenja računske vrednosti „savijanja" ispitivane lamele, u njenoj gornjoj polovini, istih vrednosti u svim postupcima proračuna, i upola su manje od izmerenih vrednosti. I kod ove faze građenja, u donjem delu lamele, deformacije izračunate metodom konačnih elemenata u Plaxis-u više odgovaraju onima koje bi se mogle očekivati od merenja, nego one koje su izračunate u Stabu. U II fazi postignut je skoro isti karakter pomeranja, ali se mora naglasiti da su obe metode u odnosu na merene rezultate s manjim vrednostima, što nije na strani sigurnosti, posebno u pogledu Staub metoda (slika 12b).

U III fazi građenja, posle iskopa do predviđene dubine građevinske jame, pojavljuje se novi oblik savijanja ispitivane lamele. Linija savijanja dijafragme podseća na onu iz II faze građenja, s tim što je došlo do rotacije dijafragme oko gornje zatege, pa je gornji deo lamele još više krenuo ka zaleđu. Izmereno pomeranje glave lamele, ka njenom zaleđu, iznosi 10 mm. Elastoplastičnim postupcima proračuna ovo pomeranje je izračunato u granicama između 3 mm i 4 mm, metodom konačnih elemenata Staub – 6 mm, a korišćenjem Plaxis-a dobijena je nešto manja vrednost – 4 mm, ali oblik savijanja dijafragme u celosti je približniji onom koji je određen na osnovu izmerenih vrednosti. Idući naniže, merene vrednosti „ugiba" lamele opadaju, i teže ka bazi lamele. Sračunate linije „ugiba", po svim postupcima proračuna, prolaze kroz nulu u srednjoj trećini lamele. Linije „ugiba", određene elastoplastičnim postupcima proračuna i korišćenjem Plaxis-a posle prelaska na stranu iskopa, vraćaju se u dno baze lamele. Međutim, prilikom postupka proračuna metodom konačnih elemenata, Staub, „ugib" ide ka iskopu i u dnu lamele iznosi oko 8 mm. Iz ovoga pregleda se vidi da se računske vrednosti horizontalnih deformacija lamele međusobno dobro slažu do gornje polovine lamele. Na ovom delu lamele računske deformacije manje su od izmerenih. U ovoj fazi (slika 12c), imamo poklapanje obe

construction, i.e. phase I (Figure 12), although the HS model shows similarities in two points as compared to the measured results. Results obtained using the Staub procedure in this respect are in favour of safety, showing higher similarity in the nature of deformation changes along the depth compared the results obtained by measurements. The shape of deformations of the tested strip, as calculated using the Plaxis software and the elastoplastic calculation procedures, although lower than those obtained by measurements, corresponds more to the shape of values obtained by measurements. Displacements measured in phase I are of changing nature with an unexpected jump (which is probably caused by deviations in the measurement) and their numerical results are quite different in nature. There are similarities between the numerical calculation regarding the nature of deformation changes along the depth, but it should be emphasized that this similarity exists particularly between the MC and LE models, noting that the MC model is more rigorous (in favour of safety, given that it yields with higher displacements) (Figure 11a).

In construction phase II, after the installation of the upper row of anchors, a flow of the loaded soil behind the wall occurred, and the tested strip was deformed (deflected) towards the backing. Above the tie, on the part of the strip overhang, the concave side of the deflected strip is turned towards the excavation, whereas beneath the tie, the convex side of the deflection line is turned towards the excavation, and the bottom end of the diaphragm deformation line tends towards its base. The strip head displacement towards the backing, as obtained by measurements, is slightly below 9 mm; however, according to elastoplastic calculation procedures and the Staub procedure, this displacement was about 3 mm, while using the Plaxis software, this displacement was 4 mm. In the lower sections, the measured strip displacement decreases, and tends toward the strip base. According to each of the calculation procedures, displacements at the half-height of the strip, as well as in its base, are equal to zero, displacement curves are turned towards the excavation, their highest ordinates are about l mm, and moving towards the strip base they tend to zero. There is a very good agreement both regarding the values and the nature of deformation changes, with the pronounced accuracy of the LE model as compared to results obtained by measurements (Figure 11b). The comparative displacement diagram along the entire strip obtained using the FEM method is similar in shape, but results obtained using the Plaxis software are more close to values obtained by measurements. As indicated by the above, in construction phase II, the calculated values of "deflection" in the upper half of the tested strip, are the same for all calculation procedures, while being 50% lower than the measured values. At this construction phase, deformations at the lower part of the strip, as calculated using the FEM method in Plaxis also correspond more to those that might be expected from measurements than those calculated by the Staub procedure. In phase II almost the same nature of displacement has been achieved, but it must be emphasized that both methods yielded with lower values compared to results obtained by measurements, which is not in favour of safety (in particular the Staub method) (Figure 12b).

metode, ali i dalje beležimo odstupanje od izmerenih vrednosti, i to na nepovoljan način. Rezultati nešto bliskiji onim izmerenim, javljaju se u donjem delu dijafragme. I po obliku odnosno karakteru promene pomeranja po dubini, oni nisu zadovoljavajući.

Na donjoj polovini dijafragme, računske deformacije, dobijene elastoplastičnim postupcima proračuna i korišćenjem Plaxis-a, bliže su onima koje bi se dobile merenjem, od onih koje su izračunate metodom konačnih elemenata u Staub postupku. U ovoj fazi je zadovoljavajuća podudarnost u pogledu karaktera promena deformacija sa određenim odstupanjima u vrednostima za HS i LE model, dok je podudarnost sa izmerenim podacima i u kvantitativnom i u kvalitativnom smislu izražena kod MC modela (slika 11b).

U IV fazi građenja, posle instalacije donjeg reda sidara, izmereno je pomeranje glave lamele ka njenom zaleđu, između 7 mm i 8 mm. Linija savijanja dijafragme podseća na onu iz III faze građenja. Elastoplastičnim postupcima proračuna, ovo pomeranje iznosilo je 2-3 mm, metodom konačnih elemenata, Staub, nešto ispod 20 mm, a u Plaxis-u – 3,8 mm. Idući naniže, izmereno pomeranje lamele ka njenom zaleđu opada, a deformaciona linija ima tendenciju kretanja ka bazi lamele. Računske deformacije lamele, izračunate elastoplastičnim postupcima proračuna, i korišćenjem Plaxis-a, idući od glave lamele pa naniže, skoro su konstantnih vrednosti. Nešto su veće u području sidara, a manje u polju između sidara, a na dnu lamele teže nuli i relativno su bliske izmerenim. Iz pregleda IV faze građenja vidi se da su merene vrednosti deformacija lamele najpribližnije rezultatima dobijenim u Plaxis-u. Kada su u pitanju numerički proračuni, najbolje poklapanje sa izmerenim vrednostima pokazuje HS model u donjoj zoni visine dijafragme,dok su odstupanja MC i LE modela značajnija i na strani sigurnosti (slika 11b). U ovoj fazi, u odnosu na izmerene rezultate, imamo potpuno odstupanje i po karakteru i po vrednostima za svaku metodu. Nešto povoljniji rezultati su kod HS modela u donjem delu dubine dijafragme (slika 12d).

U I fazi građenja, posle iskopa za gornju zategu, izmerene vrednosti vertikalnog pomeranja glave lamele 18D (slika 13), pokazuju izvesno kolebanje naviše, odnosno naniže, i usvojeno je da u ovoj fazi građenja nema vertikalnih pomeranja glave lamele. U II fazi građenja, posle zatezanja gornje zatege, dolazi do odizanja lamele naviše za oko 3 mm, a u III fazi, posle iskopa za donju zategu, uočljiva je tendencija spuštanja lamele naniže. U IV fazi, posle utezanja donje zatege, ispitivana lamela nastavlja spuštanje naniže, s tim što je i u IV fazi građenja glava lamele viša od početnog, nultog stanja, za nešto preko 1,0 mm.

U poređenju s izmerenim rezultatima, imamo odstupanja po vrednosti pomeranja, ali i u karakteru tih pomeranja. I pored toga, za različite proračunske modele tla, odnosno metode proračuna, vrednosti pomeranja su u uskim granicama vrednosti, tako da i ova raznolikost u karakteru pomeranja nema većeg značaja. To nam, ipak, dozvoljava da se oslonimo na dobijene rezultate proračuna i da na osnovu njih procenimo pomeranja dijafragmi u toku pojedinih faza građenja.

Naučno istraživanje u ovom području sporo napreduje zbog izuzetne komplikovanosti prirode

In construction phase III, after reaching the designed depth of the construction pit, the deflection of the tested strip takes a new shape. The deflection line of the diaphragm resembles that of the construction phase II, with the difference being in that a rotation of the diaphragm occurred around the upper tie, so that the upper strip part is displaced even more towards the backing. The strip head displacement towards its backing, as obtained by measurements, is 10 mm. According to the elastoplastic calculation procedures, this displacement ranges between 3 and 4 mm; the finite element method (Staub) yielded with 6 mm, while the value obtained by the Plaxis software is somewhat lower – 4 mm, but the overall shape of deflection is closer to the shape obtained based on the measured values. In lower parts of the strip, the values of "deflection" obtained by measurements drop and tend towards its base. According to all the calculation procedures, the "deflection" lines pass through zero in the middle third of the strip. According to elastoplastic calculation procedures and the Plaxis software, "deflection" lines, after moving to the excavation, turn back to the strip base. However, according to the finite element method (Staub) of calculation, the "deflection" heads towards the excavation and in the strip base its value is about 8 mm. As indicated by this overview, the calculated values of horizontal strip deformations are in a rather good mutual agreement up to the upper half of the strip. In this part of the strip, the calculation based deformations are lower than those obtained by measurements. At this phase (Figure 12c), the two methods overlap, but still deviate from values obtained by measurements carried out on an unfavourable way. Results are somewhat closer to the measured results at the lower part of the diaphragm. Regarding their shape and character, displacement changes along the depth are also not satisfactory.

On the lower half of the diaphragm, the calculated deformations, obtained based on elastoplastic calculation procedures and the Plaxis software, are closer to those obtained by measurements than those calculated by the finite element method (Staub). At this phase, the agreement regarding the nature of deformation change is satisfactory however there are certain deviations in the values obtained for the HS and LE models, while the agreement with the measured data both in quantitative and qualitative terms is present in the MC model (Figure 11b).

In construction stage IV, after the installation of the lower row of anchors, as indicated by measurements, the strip head is displaced towards its backing for 7-8 mm. The diaphragm deflection line resembles that from the construction phase III. According to elastoplastic calculation procedures, this displacement was 2-3 mm; according to the finite element method (Staub), it was somewhat lower than 20 mm, while the Plaxis software yielded with 3.8 mm. In lower parts of the strip, its displacement towards its backing, as obtained by measurements, drops, while the deformation line moves towards the strip base. Strip deformations are nearly of constant values as calculated by elastoplastic calculation procedures and based on the Plaxis software, moving from the strip head towards the bottom. They are somewhat higher in the anchor zone, while being lower in the zone between the anchors; at the strip bottom they tend to zero and are relatively close to values obtained činilaca koji utiču na proračun fleksibilnih zidova. Sve više se razjašnjavaju svojstva pojedinih faktora, koja su presudna u rešavanju problema. Uprošćavanjem nekih činilaca i prilagođavanjem modelima tla i na osnovu njih, proračunskim metodama za definisanje naponsko-deformacionih stanja u tlu i njegovom interakcijom s drugim objektima, danas imamo dovoljan broj teorija i postupaka za svakodnevnu upotrebu.

Kao što je ranije naglašeno, problem proračuna armiranobetonskih dijafragmi, još uvek je otvoren, pa je neophodno dalje proučavanje ponašanja tih konstrukcija, kako teoretski, tako i na modelima malih razmera, a i merenjima „in situ". Istraživanja u oblasti teorijske mehanike tla, ispitivanja tla, opažanja izvedenih fleksibilnih konstrukcija, kao i razvoj naprednih konstitutivnih modela tla, uz povratne parametarske i uporedne analize, predstavlja osnovu za razvoj modeliranja geotehničkih konstrukcija, ali i razumevanja realnog ponašanja tla.

I u razmatranom primeru, imajući u vidu dobijene rezultate, potrebno je izvršiti dodatne analize i varijacije i ulaznih parametara tla *(c,E, φ)* i parametara koji definišu karakteristike dijafragme, sidara, zone sidrenja i drugih elemenata,a koji su sastavni deo ovakvih konstrukcija ili imaju uticaj na njih. U daljem razmatranju, analiziran je uticaj promene kohezije na horizontalna pomeranja dijafragme (slika 14.).

Odstupanja izračunatih pomeranja u odnosu na merenja, dovodi se u vezu s promenama kohezije tla u toku pojedinih faza instalacije dijafragme. Naravno, to nije jedini uticaj, ali se može smatrati jednim od ključnih. Potvrda za to može se dobiti variranjem početne, reperne, kohezije tla u rasponu od -50% do + 50%, što je pretpostavljena promena u ovim fazama. Variranje je izvršeno u prva četiri sloja proračunskog modela, zato što se pretpostavlja da u slojevima ispod nema značajnih uticaja koji bi prouzrokovali promenu kohezije. Osrednjene vrednosti izmerenih pomeranja, koje se imaju u vidu:

by measurements. As indicated by the overview of construction phase IV, the measured values of strip deformation are the closest to results obtained using the Plaxis software. As for the numerical calculations, the best agreement with the measured values is demonstrated by the HS model in the lower diaphragm zone, while deviations of the MC and LE models are more significant, being in favour of safety (Figure 11b). At this phase, compared to results obtained by measurements, there is a total deviation both by character and values for each method. Somewhat better results were obtained by the HS model in the lower parts of the diaphragm (Figure 12d).

In construction phase I, after the excavation of the upper tie, the measured values of vertical strip head displacement (Figure 13) indicate certain fluctuation upwards and downwards, and it was adopted that no vertical strip head displacement occurs at this stage of construction. In construction phase II, after tightening the upper tie, strip displacement occurs in the upward direction of about 3 mm, while in phase III, after the excavation for the lower tie, the strip has a descending tendency. In phase IV, after tightening the lower tie, the strip continues to descend, but in construction stage IV, the strip head is higher than in the initial (zero) state for somewhat over 1.0 mm.

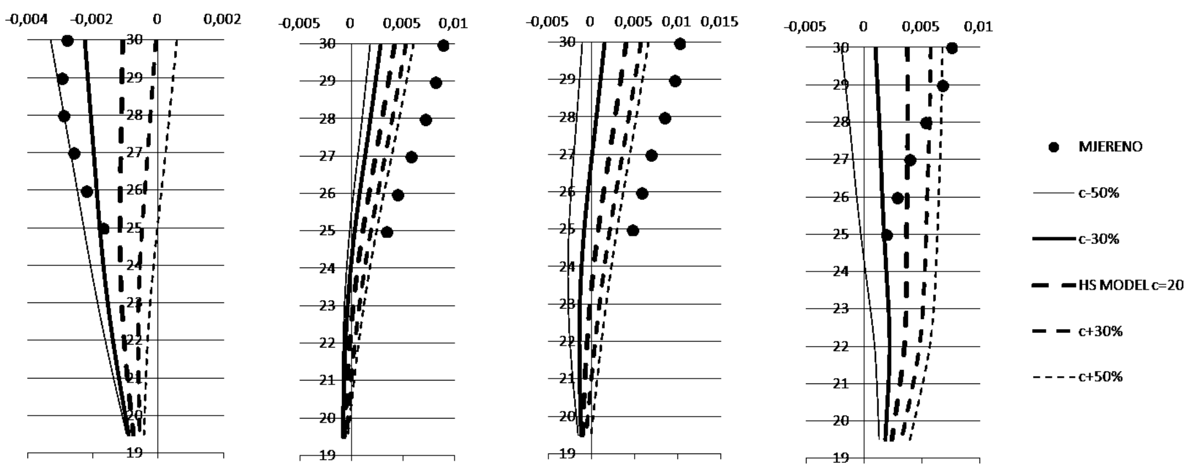
Compared to results obtained by measurements, there are differences regarding the value, but also the nature of displacement. Nevertheless, displacement values obtained using various soil calculation models, i.e. calculation procedures, are in a narrow value-range, so that this diversity in the nature of displacement is unimportant. This allows us to still rely on results obtained by calculation and based on them to estimate the displacement of the diaphragm during the individual phases of construction.

Given the highly complex nature of factors influencing the calculation of flexible walls, scientific research in this area advances slowly. The properties of crucial problem-solving factors are increasingly clarified. Nowadays, there are a sufficient number of theories and procedures for daily practice by simplifying some of the factors and adjusting them to soil models based on calculation methods for defining the stress-strain state of the soil and its interaction with other objects.

As previously noted, the problem of calculation of reinforced concrete diaphragms is still not resolved and further studies are needed regarding the behaviour of these structures, both in theory and small-scale models, as well as using *in situ* measurements. Research in the field of theoretical soil mechanics, soil testing, observations regarding the behaviour of already constructed flexible structures and developing advanced constitutive soil models, along with feedback, parametric and comparative analyses, make the basis for advancing the area of modelling geotechnical structures, but also for understanding the actual soil behaviour.

Taking into account the results obtained by the authors, the example under consideration also requires additional analyses, varying both the input parameters of soil (*c, E, φ*) and the parameters that define the properties of the diaphragm, anchors, anchorage zones and other elements that are parts of this structure or have any impact on it. Furthermore, the influence of changes in cohesion upon horizontal displacements of the diaphragm has been investigated (Figure 14) in this paper.

Differences between the calculated and measured values of displacement are related to changes in soil cohesion during the individual diaphragm-installation phases. Therefore, this can be considered as one of the key influences, but not the only. It can be confirmed by varying the initial (benchmark) soil cohesion in the range of -50% to +50%, which is the assumed change in these stages. Variations were introduced into the first four layers of the calculation model because further layers were assumed without any significant impact that would cause a change in cohesion. The average values of measured displacements were taken into account.



Slika 14. Varijacija kohezije po fazama iskopa
Figure 14: Varying cohesion by excavation phases

Faza 0. Ugradnja dijafragme dimenzija 80 x 250 cm do dubine 10,5m.

U I fazi, za promenu kohezije za -50% značajno su se približile, skoro poklopile merene i izračunate vrednosti. Objašnjenje za to jeste činjenica da se pri iskopu dolazi do otvaranja „slobodne površine", relaksacije tla, što prouzrokuje narušavanje dotadašnje veze između čestica, tj. kohezije. Poklapanje je naročito izraženo u dubljim slojevima koji trpe najviše promena pri pojavi sekundarnih naponskih stanja[1]. Primetna je sličnost promene kohezije i horizontalnih pomeranja, u funkciji od dubine, odnosno javlja se očekivani tok u skladu s promenama u tlu pri iskopu. Time se omogućuje prognoza procene promene kohezije u slojevima ispod iskopa. U tim slučajevima, kohezija se ne može uzeti kao konstantna veličina u toku pomeranja, te se javlja razlika izmerenih i izračunatih vrednosti pomeranja, što treba imati u vidu tokom proračuna.

[1] Zona nedovoljno daleko od površine da bude izolovana od uticaja vremenskih i inženjersko-geoloških promena izazvanih tim pojavama, a ni dovoljno blizu, poput površinskog sloja, koji se najviše menja usled toga. U tim površinskim slojevima nema drastičnih promena sekundarnog naponskog stanja, jer su procesi promena i uspostavljanja sekundarnog stanja u njima permanentna dinamička vremenska pojava.

Phase 0: Installation of an 80 × 250 cm diaphragm to depth of 10.5 m,

In phase I, when cohesion was changed by -50%, a significant convergence between the calculated and measured values occurred, as they almost overlap. This can be explained by the fact that the excavation leads to the disclosure of "free surface", or soil relaxation, causing in turn disturbance in the interconnection of particles, i.e. cohesion. This overlapping is specifically pronounced in deeper layers that suffer the majority of changes during the occurrence of secondary stress states[1]. There is a notable similarity between the changes in cohesion and horizontal displacements as a function of depth, i.e. the expected course takes places in accordance with changes in the soil during excavation. This allows changes in cohesion of layers beneath the excavation to be predicted and evaluated. In these cases, cohesion cannot be seen as a constant

[1] This zone is not far enough from the surface to be isolated from the influence of changes induced by weather and engineering-geological measures, and still not close enough as the surface layer, being the most changed as a result of these factors. In these surface layers, only small changes in the secondary stress state occur as the processes of changes and the establishment of secondary states is ongoing dynamic phenomena.

U slučaju povećanja kohezije, npr. primenom neke od metoda poboljšanja nosivosti tla ili izazivanjem veštačke konsolidacije pre iskopa, evidentna su poboljšanja u smislu horizontalnih pomeranja. Zbog primarnog stanja napona tih slojeva, najveća poboljšanja ostvarena su u površinskim slojevima. Poređenje dijagrama u vezi s povećanjem kohezije tla ukazuje na činjenicu da tehnologiju i parametre ojačanja treba prilagoditi dubini slojeva i da parametri tretmana tla nisu konstantni po dubini. Zbog toga se HS model može smatrati dovoljno pouzdanim za prognozne ocene promena kohezije, odnosno pomeranja u funkciji od te promene, za date uslove tla i dijafragme. Slične pojave, u skladu s realno očekivanim fenomenima tla, uočavaju se i u ostalim fazama izvođenja dijafragme.

U II fazi vidljiva su odstupanja vrednosti dobijenih na proračunskom modelu od izmerenih vrednosti horizontalnih pomeranja, posebno u površinskom slojevima. To se objašnjava nemogućnošću da se proračunskim modelom simuliraju realna pomeranja koja u fazi potiskivanja tla u zaleđe, usled dejstva zatege, imaju i vertikalnu komponentu, što omogućuje veće horizontalno pomeranje tla. S povećanjem dubine, uočava se smanjenje tih razlika, što je i očekivano imajući u vidu manje efekte vertikalnih pomeranja. Stoga, horizontalno pomeranje nije apsolutni i jedini rezultat sabijanja tla iza dijafragme, već su i uslovi koji daju te mogućnosti. Prema tome, iako bismo mogli na osnovu pomeranja da zaključimo da su postignute promene vrednosti kohezije u ovoj fazi veće, vrednosti koje su dobijene proračunskim modelom mogu se smatrati realnim. Karakter dijagrama horizontalnih pomeranja, dobijen proračunskim modelom, ima „mirne" i međusobno „semiparalelne" tokove, saglasne toku tačaka dobijenih merenjem, pa je moguće uspostaviti funkcionalnu zavisnost između proračunatih i merenih vrednosti odnosno doći do koeficijenta popravke, kako bi se moglo pouzdano prognozirati pomeranje u II fazi. U ovoj fazi prisutna je ujednačenija promena kohezije, njena vrednost može se smatrati i konstantnom na celoj dubini.

U III fazi, postignuta kohezija u prethodnoj fazi nije drastično oslabila i može se smatrati da najviše što se može smanjiti jeste vrednost referentne, reperne kohezije, mada se može smatrati da je to kohezija postignuta u prethodnoj fazi. Efekat izdizanja tla, postignut u II fazi, u ovoj i narednoj fazi utiče na razliku merenih i proračunatih pomeranja. Karakterna sličnost merenih i izračunatih pomeranja posebno je izražena u području novog iskopa, pa je moguće postaviti korelacioni odnos između ove dve grupe rezultata, u smislu mogućnosti što realnije prognoze pomeranja.

Nakon postavljanja druge zatege, faza IV, promena kohezije je uspostavljena prema očekivanim vrednostima i ima linearni opadajući karakter, s postignutim vrednostima kohezije u gornjim delovima iskopa, zadržanim vrednostima na nivou reperne vrednosti u srednjem nivou dijafragme i nešto manjim vrednostima od reperne u samom dnu dijafragme, ali ne većim od -30%. Sve je to u potpunoj saglasnosti s horizontalnim izmerenim pomeranjima. Srednja merna tačka (srednji sloj) predstavlja polaznu tačku za utvrđivanje odnosa promene kohezije odnosno pomeranja u slojevima iznad i ispod te tačke. Uzimajući te činjenice u obzir, HS model, uz modifikaciju kohezije

dimension during the process of displacement, and difference occurs between the measured and calculated values of displacement, which should be taken into account in calculations.

With the increase of cohesion, for example using some of the methods for improving the bearing capacity of soil or causing artificial consolidation prior to excavation, there are obvious improvements in terms of horizontal displacements. Due to the primary stress-state of these layers, the highest improvements were achieved in surface layers. As found based on comparing charts of the increasing soil cohesion, the technology and parameters of strengthening should be adjusted to the depth of layers, and soil treatment parameters are not constant along the depth. Therefore, the HS model should be considered sufficiently reliable for predicting the assessment of cohesion changes and displacements as the function of these changes for the given soil and diaphragm conditions. Similar phenomena, in accordance with the reasonably predictable soil phenomena, are observed also in other construction phases of the diaphragm.

In phase II, there are visible differences between the values of horizontal displacements obtained based on the calculation model and those obtained by measurements, especially in surface layers. These differences can be explained by the inability of the calculation model to simulate the actual displacement of soil. When making soil compact towards the backing, due to the action of the tie, these displacements also have a vertical component which allows soil to be displaced horizontally to higher degree. With the depth of soil, these differences are reduced, which is expected given the weaker effects of vertical displacements. Thus, horizontal displacement is not the absolute and sole result of making soil compact behind the diaphragm, but also the conditions that provide these opportunities. Consequently, despite the possible conclusion that could be drawn on the basis of displacements that the obtained changes in values of cohesion in this phase are higher, values obtained using the calculation model are convincing. The nature of the horizontal displacement diagram obtained by the calculation model has its "steady" and mutually "semi-parallel" courses, which are in agreement with the courses of points obtained by measurements; thus, it is possible to establish a functional relation between the calculated and measured values, i.e. to find a repair coefficient that allows predicting the phase II displacement in a reliable manner. At this phase, changes in cohesion occur in a more uniform manner, thus, their value can be considered constant in the target depth.

In phase III, the cohesion achieved in the previous phase is not weakened dramatically and the reference (benchmark) value of cohesion may be considered the sole thing that could be reduced, although it can also be considered that this cohesion is achieved in the previous phase. The effect of soil heave, achieved in phase II, in this and the next phase affects the difference between the measured and calculated displacements. The nature of similarity between displacements obtained by measurements and those obtained by calculations is particularly pronounced in zone of the new excavation, so it is possible to establish a correlation between the two sets of results in terms of ability to predict the tla iza dijafragme, može da bude pouzdan alat za procenu očekivanih horizontalnih pomeranja.

Urađena je parametarska analiza jednog od relevantnih faktora koji utiče na proračun pomeranja tla odnosno dijafragme, a takva se analiza i takvo utvrđivanje uticaja na rezultate proračuna u poređenju s merenim, mogu uraditi i za ostale relevantne faktore. To su parametri: prirodne sredine, dijafragme, potpore (ankera) kao i parametri tehnologije izvođenja dijafragme. Na osnovu tih parametarskih analiza, treba izvršiti procenu i rejting relevantnosti svakog od njih za aplikativnost u inženjerskoj praksi (koliko koji od parametara utiče na proračun i opravdanost njegovog izostavljanja ili neophodnost uzimanja u obzir njegovog uticaja).

9 ZAKLJUČCI I PREPORUKE

Veoma složen problem u geotehničkoj praksi predstavlja određivanje interakcije potporne konstrukcije i tla. U savremenoj praksi, primetna je potreba za pronalaženjem proračunskih modela koji će realno opisati ponašanje tla, pri promeni stanja napona, nastalih pod uticajem različitog opterećenja, jer sigurnost građevine zavisi od deformacija koje se javljaju tokom njene izgradnje i eksploatacije.

Teoretski radovi iz ovog područja mogu biti potvrđeni upoređivanjem s ponašanjem stvarnih potpornih konstrukcija u realnom tlu. Samo na osnovu teorijsko-numeričkih analiza teško je odgovoriti na postavljena pitanja u vezi sa interakcijom konstrukcija–tlo [9], pa je eksperiment veoma značajan.

Pokazano je da se klasičnim metodama proračuna dobija dobar uvid u tok proračuna i praćenje rezultata. One su prihvatljive prilikom početnih analiza i shvatanja problema, iako su opterećene brojnim nedostacima i pretpostavkama.

Metodom konačnih elemenata, uz korišćenje računara, taj problem je umnogome prevaziđen. Numeričko modeliranje pruža široke mogućnosti pri analizi potpornih konstrukcija, te se realnije mogu proceniti raspodela naprezanja, deformacija i pomeranja, i odrediti zone lokalnog sloma tla tokom korišćenja građevine. Trenutno, još uvek nismo u stanju da parametre tla obuhvatimo tako da metoda konačnih elemenata može opisati stvarno stanje napona i deformacija tla neposredno pored i oko dijafragme. Treba napomenuti da dobijanje pouzdanih ulaznih geomehaničkih karakteristika tla nije jednostavno. Razvoj novih modela tla zahteva primenu savremenih metoda ispitivanja mehaničkih i fizičkih karakteristika tla. Kvalitetna analiza glavni je pokretač razvoja savremenog pristupa modeliranja geotehničkih konstrukcija, ali – uopšteno – i razumevanja realnog ponašanja tla.

Razvoj programskih paketa i modela tla doprinosi realnijem određivanju međusobnog delovanja konstrukcije i tla. Napredak u razvoju jasno se vidi pri poređenju rezultata proračuna metodom konačnih elemenata realizovanih programom Staub i Plaxis. Iz uporednih rezultata deformacionih linija dijafragme, može se zaključiti da u svakoj fazi izgradnje napredni HS model tla u Plaxis-u daje realnije rezultate od programa

displacements as accurately as possible.

After installing the second tie in phase IV, the change in cohesion is established according to the expected values and it has a linearly decreasing nature, with the achieved values of cohesion in upper parts of the excavation; values obtained in the middle level of diaphragm remained at the level of benchmark values; while at the bottom of the diaphragm slightly lower values were obtained than the benchmark values, but not lower than -30%. All this is in a complete agreement with the measured horizontal displacements. The mid measurement point (middle layer) is the starting point for determining the ratio of cohesion change, i.e. displacement in layers above and below that point. Taking these facts into account, the HS model can be a reliable tool for assessing the expected horizontal displacements, provided that the cohesion of soil behind the diaphragm is modified.

9 CONCLUSIONS AND RECOMMENDATIONS

Determining the retaining structure-soil interaction is a highly complex problem of geotechnical practice. In contemporary practice, there is a need for finding a calculation model that describes the actual behaviour of soil with the changing stress states resulting from the influence of various loads, given that the structure's safety depends on deformations that occur during its construction and operation.

Theoretical works in this area can be confirmed by comparing them with the behaviour of actual retaining structures in the real soil. Solely on the basis of theoretical and numerical analyses, it is difficult to answer the questions regarding the structure-soil interaction [9]; thus, conducting experiments has particular importance.

Conventional calculation methods are proven good in providing an insight into the course of calculation and monitoring the results. Though burdened with numerous shortcomings and assumptions, they are acceptable for initial analysis and problems comprehension.

Based on the finite element method and using computers, this problem is largely resolved. Numerical modelling provides great opportunities in the analysis of retaining structures, enabling the distribution of stresses, strains and displacements to be assessed more realistically, while zones of local soil failure during the operation of the structure can be established more accurately. Today, we are still not able to cover soil parameters in a way to enable the finite element method to describe the stress-strain state of the soil next to and around the diaphragm in a realistic manner. It should be noted that obtaining reliable geotechnical input properties is a difficult task. Developing new soil models requires the use of modern methods of testing the mechanical and physical properties of soil. A high quality analysis is the main driver of developing a modern approach to modelling geotechnical structures, but also understanding the actual soil behaviour in general.

The development of software packages and soil models contributes to more realistic determination of the soil-structure interaction. Progress in development is evident when comparing the results of calculations obtained using the finite element method based on the Staub and Plaxis programs. Based on the comparative

Staub koji je korišćen u vreme izgradnje dijafragme.

Razvoj novih metoda proračuna i konstitutivnih modela tla zasniva se na sprovođenju povratnih i parametarskih analiza dobijenih merenjem na zaštitnim konstrukcijama. Važno je naglasiti da predložene korelacije treba prihvatiti samo kao grube smernice, a proračunske parametre odrediti primenom savremenih geotehničkih ispitivanja tla, te ih kontrolisati merenjima konstrukcija tokom izgradnje i eksploatacije.

Naposletku, nakon upoređivanja rezultata terenskog merenja i postupaka proračuna, može se zaključiti da se – u slučaju ove ispitivane lamele – izmerene deformacije po fazama građenja najrealnije mogu proceniti proračunom metodom konačnih elemenata sa unapređenim HS modelom tla. Izmerena pomeranja su nešto veća od računskih, ali su ona istog oblika "savijanja", ali translatorno pomerena za nekoliko milimetara, što je dobra aproksimacija za praktične svrhe.

results of deformation lines of the diaphragm, it can be concluded that in every construction phase, the advanced HS soil model in the Plaxis software provides more realistic results than the Staub program that was used in time when the diaphragm was constructed.

The development of new calculation methods and constitutive soil model is based on the implementation of feedback and parametric analyses obtained by conducting measurements on protective structures. It is important to emphasize that the proposed correlation should be taken only as rough guidelines, whereas calculated parameters need to be determined using modern geotechnical soil testing methods and verifying them by measuring the structures during their construction and operation.

Finally, after comparing the results obtained by field measurements and calculation procedures, it can be concluded that deformations of the given strip 18D, as measured by construction phases, can be the most realistically estimated using the finite element method with the improved HS soil model. Displacements obtained by measurements are slightly higher than those obtained by calculations, but they have the same "deflection" shape linearly displaced for a few millimetres, which is a good approximation for practical purposes.

ZAHVALNOST

Ovaj rad je deo istraživanja u okviru projekta TR 36017, koji podržava Ministarstvo za obrazovanje nauku. Autor, Radomir Folić, zahvaljuje na tome.

ACKNOWLEDGEMENTS

The work reported in this paper is a part of the investigation within the research project TR 36017 supported by the Ministry for Science and Education, Republic of Serbia. This support is gratefully acknowledged (R. Folić).

10 LITERATURA

[1] Folić, R., Pavlović, P. (1986): *Numerical Analysis of Anchored Reinforced Concrete Diaphragm Walls*. 2nd International Symposium on Numerical Models in Geomechanics, Gent, M. Jeckson & Son P. Redruth, Cornwall, England, pp. 423–431.

[2] Folić, R., Popović,B., Pavlović, P.: *Prilog proučavanju ponašanja armiranobetonskih ankerovanih dijafragmi*, Drugo savetovanje Društva za mehaniku ta i fundiranje, D. Milanovac, 1984.

[3] Folić, R., Pavlović, P. (1979): *Problematika projektovanja i građenja podzemne etaže novosadske banke*. Savetovanje Specijalna poglavlja iz mehanike tla i fundiranja, SGITS, Cavtat, Zbornik, str. 443–466.

[4] Maquet,M.: *Le quai en paroi mouice de la Darse de l'ocean au port du Havre*. VII International Congress-FIP, London , 1978, p. 134.

[5] Memić, M., Folić, R., Ibrahimović,A.: *Neke metode procjene pomeranjafleksibilnih sidrenih potpornih konstrukcija*, U Zborniku radova (in Proceedings of Intern. Conference, „Assessment, maitenance and rehabiltation of structures and settlements"), Ed. R. Folić, Borsko jezero, 13–16 maja, 2013, pp. 293–302

[6] Memić, M., Folić, R.: *Numeričko modeliranje i analiza sidrenih armiranobetonskih dijafragmi* GEO-EXPO 2012, jun 2012. godine, str. 317–328.

[7] Memić, M., Folić, R., Ibrahimović, A.: *Metode numeričkog modeliranja i sanacije nestabilne padine u izmenjenoj serpentinskoj stenskoj masi* (Numerical modelling and slope reparation methods in an altered and unstable serpentine rock mass), Građevinski materijali i konstrukcije, Beograd, Vol. 50, Br. 4, 2012, str. 23–45.

[8] Pavlović, P., Folić, R.: *Interakcija konstrukcija–tlo kod konzolnih armiranobetonskih dijafragmi*, SGITJ, Građevinski kalendar '03, Beograd, str. 9–102.

[9] Pavlović, P., Folić, R.: *Interakcija konstrukcija–tlo kod ankerovanih armiranobetonskih dijafragmi*, SGITJ, Građevinski kalendar '04, Beograd, str. 256–281.

[10] PLAXIS version 8: Reference manual, Material models manual

[11] Roje-Bonacci, T.: *Modeli tla*, IGH d.d.Zagreb, Split 2003.

[12] Sokolić, I., Szavits-Nossan,A.: Suvremeni pristup numeričkom modeliranju sidrenih zaštitnih konstrukcija, GRAĐEVINAR 63 (2011) 9/10, 847–85.

[13] Vujičić. Č.: *Doprinos optimalizaciji proračuna armiranih betonskih zidova direktno građenih u tlu*, Doktorska disertacija, Građevinski fakultet Univerziteta u Beogradu, 1980. g.

REZIME

KOMPARATIVNA ANALIZA METODA ZA PROCENU POMERANJA FLEKSIBILNIH SIDRENIH BETONSKIH DIJAFRAGMI

Radomir FOLIĆ
Mirza MEMIĆ
Adnan IBRAHIMOVIĆ

Problem proračuna betonskih dijafragmi još uvek je otvoren, pa je neophodno dalje proučavanje ponašanja tih konstrukcija – kako teoretski, tako i na modelima malih razmera, a i merenjima „in situ”. U ovom radu prikazane su i upoređene klasične i savremene metode za procenu ankerovanih (sidrenih) potpornih konstrukcija. Predstavljeni su metod reakcije tla k_h (Vinklerov postupak – *Winkler`s model*) i metod konačnih elemenata (MKE). Pritom, pretpostavljeno je da se horizontalne deformacije tla i potporne konstrukcije poklapaju (da su saglasne). Izjednačavanjem tih vrednosti, određuje se njihov uzajamni uticaj. Od uzajamnog uticaja, deformacije zida i deformacije tla, zavisi i raspored pritisaka tla na kontaktima zida i tla i obratno – tla i zida. Proračun dijafragme zahteva definisanje elastoplastičnih karakteristika tla na pritisak i na zatezanje, duž kontakta dijafragma-tlo. Stanje napona u tlu određeno je teorijom granične ravnoteže na bazi Kulonovog zakona (*Coulomb' s law*).

Komentarisane su osnovne karakteristike ponašanja tla u uslovima koji se javljaju pri iskopu građevinske jame. Prikazan je empirijski i numerički pristup modeliranju i proračunu sidrenih zaštitnih konstrukcija, ilustrovan na primeru simulacije sidrene potporne konstrukcije primenom programa Plaxis V8. Pri tome je korišćen napredni konstitutivni model tla, uz poređenje s realnim rezultatima terenskih merenja izgrađenog objekta [6]. Nadalje, razmatran je i analiziran uticaj promene kohezije na horizontalna pomeranja dijafragme (parametarske analize). Odstupanja izračunatih pomeranja u odnosu na merena, dovode se u vezu s promenama kohezije tla u toku pojedinih faza izvođenja dijafragme. Izmerena pomeranja su nešto veća od računskih, ali su ona istog oblika „savijanja”, a translatorno pomerena za nekoliko milimetara, što je dobra aproksimacija za praktične svrhe.

Ključne reči: Betonske dijafragme, fleksibilne potporne konstrukcije, sidra, teorija elastičnosti, modeli tla, pomeranja, MKE, "Plaxis V8"

SUMMARY

COMPARATIVE ANALYSIS OF EVALUATION METHODS OF THE DISLOCATION OF FLEXIBLE ANCHORED CONCRETE DIAPHRAGM WALLS

Radomir FOLIC
Mirza MEMIC
Adnan IBRAHIMOVIC

The problem of calculation of reinforced concrete diaphragms is still not resolved and further studies are needed regarding the behaviour of these structures, both in theory and small-scale models, as well as using *in situ* measurements. The paper gives an overview and comparison of classical and modern methods of assessment of anchorage flexible retaining structures. Method of the soil reaction coefficient k_h (Winkler's procedure), and Finite element method (FEM) are discussed. Wall deformations are assumed to follow soil deformation. This means that the elastic line of the wall corresponds to the horizontal soil displacements. Thus, by equalizing these values, their mutual influence can be determined, i.e. wall deformation and soil deformation, defines the distribution of soil pressure on the wall-soil and soil-wall interface. Calculation of the diaphragm wall based on this procedure requires defining the elastoplastic properties of soil under pressure and tension along the diaphragm-soil contact surface. The stress state in soil is determined by the limit equilibrium theory based on Coulomb's law.

The basic characteristics of the soil in conditions during construction excavation pit are discussed. The paper presents the empirical and numerical approach to modelling and calculation of anchorage protective structures which is illustrated on the example of simulation of anchorage protective structures using the program Plaxis V8. Thus, an advanced model of constitutional ground is used and compared with the actual results of field measurements of the constructed structure/facility [6]. Furthermore, in parametric study, the influence of changes in cohesion upon horizontal displacements of the diaphragm has been investigated in this paper. Differences between the calculated and measured values of displacement are related to changes in soil cohesion during the individual diaphragm-installation phases. Displacements obtained by measurements are slightly higher than those obtained by calculations, but they have the same "deflection" shape linearly displaced for a few millimetres, which is a good approximation for practical purposes.

Key words: Concrete diaphragms, flexible retaining structures, anchorage, theory of elasticity, soil models, displacements, FEM, Plaxis V8

PROCJENA I REHABILITACIJA DRVENIH KONSTRUKCIJA U SLOVENIJI

ASSESSMENT AND REHABILITATION OF TIMBER STRUCTURES IN SLOVENIA

Tomaž PAZLAR

STRUČNI RAD
PROFESSIONAL PAPER
UDK = 861

1 INTRODUCTION

Timber can be in general considered as one of the oldest construction materials. Several properly designed, constructed and maintained timber structures well demonstrate its durability. However timber is bio-degradable material and its load bearing capacity can be decreased due to visible deterioration, damaging events or change of use. Therefore the timber structures should be – in certain period of time a subject of a thorough inspection which should be an important aspect in a course of action among several alternatives in the building lifecycle. This information is essential for a reliable structural safety analysis which gives guidance to the conservation, replacement or strengthening work necessary to ensure an adequate safety level.

The need for the assessment of an existing structure can originate from various situations [1]:

- Periodical check,
- The expiration of the remaining lifetime, defined in previous assessment,
- If errors in planning or construction process become known,
- If change of use of the building is foreseen,
- In case of doubts of the structural safety, caused by visible damage,
- Due to inadequate serviceability and usability,
- Exceptional incidents or accidental loads which might have damaged the structure,
- In the case of arising suspicion due to material, construction or system inherent impairment of the structural safety,
- If simple, initially unfounded suspicion has to be eliminated.

Tomaž Pazlar, Research Assistant
Slovenian National Civil and Engineering Institute, Dimičeva 12, 1000 Ljubljana, Slovenia

2 ASSESSMENT PROCEDURE AND METHODS

The main objective of the assessment is to identify the actual degree of damage, the reasons for damage and determine the means to rehabilitate the structure. The assessment is constituted from the following sequences [1]: Registration of condition, detection and documentation of damage, determination of causes for damage, evaluation of effects and consequences of damage and specification of necessary actions and rehabilitation measures.

There are a multitude of approaches to classify the assessments, but in general the literature [2] identifies it as a three step approach.

Within the first phase, **preliminary evaluation,** the structure should be checked visually in order to identify the damages and risks and to make a good foundation for the second and third phases. The available documents should also be checked and if necessary an approximate static analysis should be made.

The purpose of the second phase, **general investigation**, is a detailed investigation of structural members with focus on a possible change of action, static system, dimensions, mechanical properties (deterioration) and calculation methods. The result of general investigation can be limitation of use, rehabilitation, reinforcement, demolition or recommendation on more detailed investigation.

The third phase, **detailed investigation**, is based on the same principles as the second phase, but it is far more detailed and may additionally include the laboratory tests of samples taken from the structure. A team of experts might be called upon and additionally more sophisticated methods of investigation should be carried out.

The level of investigation and furthermore the selection of inspection methods should be based on the perimeter of assessment with the economical consideration and from the structural point of view on the remaining level of doubt.

There are different classifications of the assessment methods. Relating to the three assessment phases the following methods are most commonly applied in each phase:

- Preliminary inspection: visual inspection, geometry measurements (thickness gauge, measuring tape), tapping, electrical resistance moisture measuring.
- General investigation: endoscopy, tachometry, complete mapping of cracks, core drilling, shear tests on core samples, drill resistance, penetration resistance, pull out resistance, stress wave and other methods.
- Detailed investigation: photogrammetry, 3D laser scan, load test, macroscopic testing of specimens, microscopic and chemical analysis, dynamic response, strain measurement, X ray, stress wave and other methods.

However the listed methods in practice are more often identified as non-destructive testing methods (NDT) or as destructive testing methods (DT). The gap between NDT and DT is in most cases successfully overcome with the semi-destructive testing methods (SDT) where extraction of small specimens helps in more exact detection of deterioration level.

3 STANDARDS/GUIDELINES FOR THE ASSESSMENT AND ASSESSMENT APPROACH

In order to unify the assessment procedure standards and/or guidelines are welcome to overcome semantic differences in assessing timber structures. Standards which are the point of interest of this paper can be considered as general one. There are some timber specific national standards and assessment guidelines available in USA, Canada, Netherlands, Switzerland (SIA 269/5), and Italy (UNI 11119:2004). There is no specific standard for the assessment of timber structures on international level; some experts recommend the use of general assessment standard ISO 13822:2001 *Bases for design of structures - Assessment of existing structures* [3].

In general the assessment standards should define the following issues: the area of application, general principles of assessment, methods of updating, methods and format for verification, risk acceptance criteria and guidelines for decisions and intervention planning [4]. ISO 13822 is structured according to these principles which reflect in its chapters: General framework of assessment, Data for the assessment, Structural analysis, Verification, Assessment based on satisfactory past performance, Interventions, Report and Judgement and decision. Similarly, the Swiss standard SIA 269/5 includes chapters on General assessment (inspection, monitoring, maintenance, etc), Requirements (use, structural safety, etc), Updating, Structural analysis, Examination, Maintenance interventions, Construction documents (service criteria agreement, service instructions, hazard events, etc) [1]. The Italian standard UNI 11119:2004 *Cultural Heritage - Wooden artefacts - Load bearing structures - Onsite inspections for the diagnosis of timber members* is more practice specific and lays down specific objectives, procedures and criteria to be followed when inspecting timber structures. Additional it has to be taken into consideration that this standard is more related to cultural heritage than having as first goal the protection and conservation of the artefacts. Standard specifies that at least the following information should be gathered in proper inspection environment (accessibility, lightning, cleaning) [5]:

- Wood species,
- Moisture content and gradient,
- Class of biological risk of the wooden member, according to EN 335-1 and EN 335-2,
- Geometry and morphology of the timber members indicating position and extension of main defects, decay and possible mechanical damage,
- Position, shape and dimensions of the critical areas and critical sections,
- Strength grading of critical areas and/or the timber member as whole.

Beside the general procedure more specific objectives are also defined such as determination of geometry and characteristics of timber elements (defects, growth irregularities, position of pith, knots, etc). Standard furthermore defines the grading rules for three structural grades of timber. No specific rules are defined how to detect decay, only the use of NDT methods is recommended. The content of the inspection report is also specified.

Beside listed standards there are also many relevant cultural heritage protection documents (ICOMOS chapters for timber structures for example), but this documents are in general more protection oriented and therefore commonly referenced by the experts of Institute of Protection of Cultural Heritage of Slovenia (IPCHS) when expressing their point of view about the assessment.

In general the adaptability of listed standards in the practice oriented assessments of timber structures is only partial. Since there is no European standard (EN) and no Slovenian standard (SIST) in the specific area of timber structures, the assessments performed at Slovenian National Building and Civil Engineering Institute are based mainly on experiences and internal assessment guidelines taken into consideration the relevant parts of cited standards. Different assessment methods were implemented: non-destructive testing (visual inspection, tapping, electrical resistance moisture content measurements, ultrasound, etc) whereas in some cases damaged elements from the structures were also tested in the laboratory to the failure. The compression and the bending test were two most commonly performed tests in terms of destructive testing. Core drilling and resistance drilling were the most commonly implemented semi-destructive testing methods.

Although the assessments are this paper main topic the general rehabilitation measures are also discussed. To obtain the building permit the rehabilitation measures have to be additionally evaluated by licensed structural engineer. Therefore the common practice as presented with the use cases is only the general overview of possible rehabilitation measures. The choice and all required calculations according to Eurocode standards are further more the task of structural engineer. The investor additionally has to consult about the rehabilitation measures with the Institute for the Protection of Cultural Heritage of Slovenia.

The principles and implementation of most commonly used methods are further on introduced on four typical examples of the assessment: i.) Roof and floor structures of Minorite monastery in Maribor, ii.) Massive timber floor in mansion "Naskov dvorec" in Maribor, iii.) Timber frame structure of secondary school in Kočevje [6] and iv.) Timber bridge over the river Krka in Loke. All presented use-cases can be ranged as general investigations.

4 CASE STUDIES

4.1 Minorite monastery in Maribor

4.1.1 Description of the building

The origins of Minorite monastery in Maribor date in 13^{th} century. The majority of monastery was renovated in the baroque style in 18^{th} century. A three storey building with the shape of a horseshoe has floor area of 700 m^2 and consists of ground floor, two storeys and a high attic which has not been in use. The length of the north and south wing is round 28 m with the widths 6.7 m and 9.4 m accordingly. The eastern wing consists of longitudinally shaped part with dimensions approximately 18 x 11 m and squared shaped SE part, called "tower" with dimensions 15 x 13 m [7].

It is presumed that the spruce/fir timber roof structures originate in the 19^{th} century. The roof structures in all wings are symmetric and non-symmetric trapezoid suspended frames lying on the outer brick walls. The main structural frames on the distances 4 - 5 m carry ridge poles, rafters, and purloins. Red clay roof tiles are used. There are two types of floor structures: above the first floor mainly massive spruce/fir timber floors are installed and under the attic some spruce/fir floors are massive and some are hollow.

The building was firstly used as a monastery, then as barracks (1784 - 1927) and at the end as an apartment building. Although several "ad hoc" renovations were performed, building was visually in a very bad condition, mainly due to poor maintenance, especially when the building was used as barracks and for housing (Figure 1).

The municipal administration was keen on demolishing the monastery and building the residential units. After very long period of discussions it was decided that the building as a historic monument has to be preserved and will be - after renovation - used as premises for the puppet theatre. Although the building was in a bad shape (one of the outside load bearing walls already collapsed (Figure 2)), the architects whish was to preserve the original structures as much as possible and incorporate them into a contemporary structure.

All load bearing structures were assessed before redesigning of the building. The first inspection of roof structures, performed by a consultant company, demanded complete decommissioning of the roof. However when the renovation started, the architect requested the second assessment, described here, which concluded that a great deal of timber structures were sound enough to be rehabilitate.

4.1.2 Assessment of the roof structure

The main attention is on the points where decay is commonly expected, i.e. supports, roof–valley elements, gutters, etc when checking the roof structure. Special concern was also put on signs of biological attack (presence of mould or wooden dust which indicates an attack of wood fungi and insects).

Due to the high level of deterioration practically all elements of roof structure were checked in details. The most problematic parts of the structure were found on the spots where long term wetting was present. Wetting was caused primarily by leaking at damaged elements at the gutters (Figures 3 and 4) and at bad connections of roof planes. Moisture content was measured where necessary. In these parts combined attack of fungi and wood insects caused substantial decay of wood, sometimes even the total loss of cross section and/or strength.

One of the most problematic elements was the main girder with 12 m span which was completely destroyed due to the attack of wood destroying fungi (Figure 5). The temporary structure was not rigid enough which together with the rearrangement of load to other roof members caused substantial deformation. Complete reconstruction of SE part of the building was proposed due to heavy damage on the roof (Figure 6) and floor elements as well as their difficult replacement.

Figures 1 and 2. Minorite monastery before renovation [8]


Figures 3 and 4. Timber deterioration in the gutter valley


Figures 5 and 6. The lower chord of the main girder was completely deteriorated (left). Several ad-hoc renovations were made (right)


Figures 7 and 8. Damage due to insect attack (left). Clear identification of elements that should be replaced (right)

Eastern wing was presumably constructed later and most likely with the less quality wood. Heavy damage due to insect attack was spotted (Figure 7). Not only elements at roof edges and gutter valleys, but also elements in the dry surroundings were damaged by the “house capricorn” (Hylotrupes bajulus). However it was estimated that the roof structure in this part of the monastery was worthwhile to rehabilitate.

Structural elements which could be rehabilitated and elements that should be replaced were clearly marked onsite (Figure 8) and also in the assessment report. The rehabilitation measures in presented use case do not change the structural system – only the decayed members are replaced with the same wood species, length and cross section elements.

4.1.3 Assessment of the floor structures

Above the first floor mainly the massive timber joists were installed adjacent one to another. Under the attics some floors were massive and some were hollow (distances between joists were ca. 0.8 - 1.0 m), all closed at bottom and top with planks). The hollow floors were completely deteriorated and already temporary supported.

The stages of decay of massive timber elements in the second floor differed: in some areas only the surface was affected whereas in some corners practically the whole effective height of joists was affected (Figure 9 and 10). The level of decay was evaluated with inspection with chisel and hammer and on some spots with core drilling.

Figures 9 and 10. Complete deterioration of hollow timber floor (left). Massive timber floor – damage caused by brown rot (right)

4.1.4 Assessment conclusions

The general assessment conclusion was that due to the high level of decay not all timber structures could be preserved. Roof structure and timber floors in SE part of the building were deteriorated in such extend that their preservation was not reasonable. The roof structures in other parts of building could in general be preserved, but all clearly marked elements (double red line) should be replaced. The hollow floors should also be completely replaced – as options the same structural system and timber based composite deck were proposed. The most damaged members of massive timber floors should also be replaced. All other elements should be treated with special restoration resins. Strengthening of timber floor structures should be performed before the timber based composite decking is installed. The detailed description of findings, drawings with elaborated suggestions for replacement/strengthening of particular elements and comprehensive photo-documentation were gathered in the final assessment report [9].

Renovation of Minorite monastery was officially finished in 2009. Today, the building is used as premises for the puppet theatre (Figure 11). High attic with visible rehabilitated timber structural elements of trapezoid suspended frames supposed to be used for schooling puppet performers, for workshops and also for dressing stage scenery, but unfortunately this part of the building was not completed (Figure 12).

4.2 Massive timber floor in mansion Naskov dvorec in Maribor

4.2.1 Description of the building

The mansion “Naskov dvorec” is one of the most important bourgeois houses in Maribor, which when it changed its owner also changed its name (Breunerjev dvorec, Naskova hiša, Berdajsova hiša). Its core dates from 14th century. Most parts of the building were constructed in 17th century and then renovated in a late Baroque style. In 18th and 19th century several renovations took place which gave the building today’s dimensions. The mansion consists of three one storey wings: west (street) wing, north wing and east wing (Figure 13).

Walls and arch floors in some parts of the building are made of brick, whereas the most of floors are made of spruce/fir massive timber joists. Most of floor structures in the north wing, which were subject of inspection, are made of massive timber elements. Hollow timber floors were installed only in some parts of the building. Our estimation is that the roof structure and massive timber floors date in the end of 19th century.

The investor, the municipality Maribor, was committed to preserve as much structure as possible, mainly because of the frescos which were discovered under the limed ceiling (Figure 14). Mansion is like the Minorite monastery considered as urban cultural heritage.

Figures 11 and 12. Monastery after renovation hosts the puppet theatre (left). Render of attic - not completed due to the lack of funds (right)

Figures 13 and 14. Mansion during the renovation (left). Frescos under the limed ceiling (right)

After the renovation the attic will be in use. The structural engineer designed new self supporting concrete slab which will be installed right above the joists. Timber and concrete structures will be separated; the massive timber beams will lose the imposed load bearing purpose. Therefore the evaluation criteria were not as rigid as they would be in case of preserving their origins function or if they would be incorporated in composite structure [10].

4.2.2 Assessment of the floor structures

The assessment was performed during the undergoing renovation. Joists were visible because top planks were already removed (Figure 15). Where partition walls were removed the joists were also visible from underside. Orientation of joists depends on geometry of massive brick wall substructure.

Visual overview of floor joists combined with core drilling revealed typical traces of wood fungi and insects attack (Figure 16). Most of the decay was caused by the insects (Figure 17), mainly the old-house borer (Hylotrupes bajulus L.) whereas the decay caused by fungi was local, mainly on supports and near outside walls. Due to the long term wetting some joists were decayed in such extend that they had to be additionally temporary supported. Brown rot occurs locally, as consequence of long time wetting which can also be identified by spots on limed ceiling in rooms bellow the attic (Figure 18). On all other location the measured moisture content were in the normal range.

4.2.3 Assessment conclusions

The assessment proved that the majority of joists can be rehabilitated - only locally some joists have to be replaced with impregnated new elements. The most damaged top layer (2 - 3 cm) of all massive timber elements should be removed, and the surface should be impregnated with insecticide. After impregnation a layer of PE foil should be installed to prevent wetting of timber. Joists will be used as formwork and therefore temporary supports are recommended (subject of structural calculation by a licensed structural engineer). An additional separation layer should also be installed (expanded polystyrene for example) in order to try to prevent the direct load transfer from concrete slab to joists. This solution will also enable preservation of valuable old frescoes.

It has also been recommended that concrete with low water cement ratio should be used. In order to assure durability of timber elements, porous materials should be used in renovation of ceilings [11].

4.3 Timber floors in town hall in Kočevje

4.3.1 General description of the building

Town hall in Kočevje was built between 1889 and 1899. Its original purpose was a private girl's school, led by the nuns. Building was used as a hospital in the World War I. Today it is a town hall hosting the offices of municipality, public administration and the country court (Figure 19).

Figures 15 and 16. Massive timber floor (left). Core drilling (right)

Figures 17 and 18. Damage caused by house capricion (left). Damage primarily caused by fungi - brown rot (right)

Figures 19 and 20. Town hall in Kočevje (left). Opening spot (right)

The building with floor area of 940 m^2 has a ground level, two storeys and high attics which are also in use. All walls are masonry. The floor structures above the ground level are mason arches and above the upper storeys hollow spruce/fir timber structures. Except general floor plans and cross sections any other documented construction details were unavailable. The subject of inspection was the whole building (including foundation, masonry walls and arches).

4.3.2 Assessment of the floor structures

The building was in use when the inspection was performed. Therefore the time and the points of inspection had to be agreed with the users of the building. This inspection was different than the others presented in this paper. Inspection was performed at reference points and findings obtained were used for the general assessment report.

Two opening spots were chosen - one on the first floor and the second one on the second floor. Both opening spots where located where joists are supported by the masonry walls. This is the location where it is reasonable to expect the highest level of decay because of leaking or because of improper construction – in front of the forehead of beams there should be an air pocket or preferably opening in the façade. Openings with approximate dimensions 1.5 x 1.0 m were made in order to determine relevant dimensions of floor structure (Figure 20). The following layers were identified: parquet, top decking, gravel/sand layer, panelling, joists, bottom decking, reed and plaster. The only difference between the 1st and the 2nd level is in the height of sand/gravel layer: 12 cm in the 1st and 16 cm in the second floor. The structure in the first floor was in very good shape: The gravel/sand layer was completely dry. All visible joists were not deteriorated, only minor side cracks were identified (Figure 21). Gravel/sand layer on the second floor was also dry, but consequences of (long term) wetting were visible: On the bottom side of top planks fungi were present and joists near support were affected by the brown rot (approximate depth 2 cm) (Figure 22) [12].

4.3.3 Ultimate limit state and serviceability limit state

The load bearing capacity and deflections were checked according to EN 1995-1-1:2005 using proposed values from EN 1991-1-1:2004. On the basis of measured floor elements dimensions it has been assessed that the permanent load of 2.1 kN/m' (2.7 kN/m' in second floor) has to be taken into account. The effective beam cross section of 20 x 24 cm and mechanical characteristics for timber strength class C24 were used in calculation. A simple supported beam with 6 m span was used as structural model.

As building is used for offices, the imposed load for Category B (characteristic value of uniformly distributed load 3.0 kN/m^2) has to be taken into account.

Presented ratios indicate that in both floors design actions exceed design resistances. The results do not include masonry partition walls which are sometimes installed above the joists and sometimes above top planks. If partition walls are taken into account, the ratio design action/design resistance increases to 3.7 (or 3.4 in the second floor).

The serviceability limit state proves to be more problematic in both floors. When the regular load is considered, obtained ratios for instant load and final stage exceed allowable values (see Table 2). When the

Figures 21 and 22. Hollow timber floor joist (left). Deterioration of joists (right)

Table 1. Ultimate limit state

		(Bending)			(Shear)		
		Design Action [kNm]	Design Resistance [kNm]	Ratio	Design action [kN]	Design resistance [kN]	Ratio
1st floor	Self weight	13.3	21.3	0.6	8.7	59.1	0.2
	Self weight + Imposed l.	30.8	28.4	1.1	20.1	78.8	0.3
2nd floor	Self weight	16.8	21.3	0.8	11.0	59.1	0.2
	Self weight + Imposed l.	33.7	28.4	1.2	22.0	78.8	0.3

Table 2. Serviceably limit state

<table>
<tr><td></td><td></td><td colspan="3">(Instant)</td><td colspan="3">(Finite)</td></tr>
<tr><td></td><td></td><td>Displacement [mm]</td><td>Allowed disp. [mm]</td><td>Ratio</td><td>Displacement [mm]</td><td>Allowed disp. [mm]</td><td>Ratio</td></tr>
<tr><td rowspan="2">1st floor</td><td>Distributed l.</td><td>33</td><td rowspan="4">20</td><td>1.6</td><td>53</td><td rowspan="4">41</td><td>1.3</td></tr>
<tr><td>Point load</td><td>24</td><td>1.2</td><td>38</td><td>0.9</td></tr>
<tr><td rowspan="2">2nd floor</td><td>Distributed l.</td><td>37</td><td>18</td><td>58</td><td>1.4</td></tr>
<tr><td>Point load</td><td>28</td><td>1.4</td><td>44</td><td>1.1</td></tr>
</table>

masonry partition walls are taken into account, the ratios increase up to 5.3 or 4.2 accordingly. Since the calculation procedure is the same only the ratios are given.

4.3.4 Assessment conclusions

Although any significant deteriorated spots were not identified, the assessed hollow timber floors do not meet current standard requirements. However we estimated that timber joists can be preserved if self weight of floor structure is decreased – the depth of sand/gravel layer should be reduced. More probable solution is the complete removal of sand/gravel layer and installation of new lighter sound isolation material. Structural calculation and detailed plan for renovation is of course required. Masonry partition walls cannot be preserved. They should be replaced with lighter structure (e.g. from gypsum plaster). The re-design and renovation of presented structure has been after the assessment due the lack of funds postponed.

4.4 Timber frame structure of secondary school in Kočevje

4.4.1 General description of the building

The secondary school in Kočevje was built in the mid eighties (Figure 23). The building consisted of two parts: the single storey part was constructed as classic timber frame structure while the two story part was built as concrete wall structure in ground floor and as classic timber frame structure in the second floor. Prefabricated structural members assembled with punched metal plate fasteners presented the building roof structure. The second floor timber frame structure and roof structure were damaged in the fire in December 2012. The single storey part with approximate dimension 18 x 50 m was not directly damage. This part was damaged indirectly due to the fire fighting (Figure 24).

Figures 23 and 24. Secondary school in Kočevje (left). Subject of inspection - red, damaged in fire - blue (right)

ZAG was asked to perform the assessment of the preserved structure. The subject of inspections was concrete structure (not included in this paper), timber roof trusses and timber wall and floor members. The main assessment question was refereeing to rehabilitation of timber based structures. The investor wanted to know if the structure can be preserved and if the load bearing characteristics of structure fulfil the requests defined in the Eurocode standards.

4.4.2 Assessment of roof structure

There are three types of roof structure: pent (Figure 25), gable (Figure 26) and flat roof (Figure 27). With all types the same structural system is used: spruce/fir timber roof trusses jointed with nailing plates. The roof is covered with corrugated sheets "Valovitka". Not all timber member dimensions were compliant with the project documentation - some verticals and diagonal members had smaller cross section than defined in the project documentation. The timber was in perfect condition, no decay could be identified, not even on the most problematic locations (valleys for example). According to the size of knots and other defects the timber could be sorted in strength class C24. The nailing plates were not corroded. Regardless to the fire fighting there were no signs of increased moisture content.

Some installation errors were identified: Batten nails are not always installed in the roof trusses (Figure 28) and the cross section of trusses is locally reduced (Figure 29). The attic is in certain locations used as a warehouse (Figure 30) which was not taken into consideration in the building design. Not all electric installations are set in the installation ducts. Trusses are fixed to the timber frame walls (Figure 31) and concrete walls (Figure 32) using steel brackets. Their load bearing characteristics are not known. More problematic seems to be the fixation of brackets on walls/trusses since only one screw/anchor with relatively small dimension (6 or 8 mm) is used (Figure 31).

Figures 25 and 26. Pent roof (left). Gable roof (right).

Figures 27 and 28. Flat roof (left). Batten nails not fixed in roof trusses (right)


Figures 29 and 30. Reduced cross section of trusses (left). Attic used as a warehouse (right)


Figures 31 and 32. Fixation of trusses to frame walls (left). Fixation of trusses to concrete walls (right)

4.4.3 Assessment of floor and structure

The load bearing floor structure are spruce/fir timber frames, nailed in to the lower chord of timber roof trusses. Thermal insulation is from the top side covered with partly damaged bituminous sheets. Thermal insulation was on some locations still wet due to the fire fighting. More problematic are the bottom particle boards panels which are permanent deformed due to the wetting (Figure 33).

Three different locations were selected to check the dimensions and level of decay and defects of timber frame walls. One external wall, one internal load bearing wall and one internal partition wall were checked. According to the project documentation the dimensions of internal and external timber frame elements and consequently walls should differ. However the inspection proved the opposite. Additionally all particle boards and gypsum boards had the same thickness (Figure 34). Consequently the structure as whole consists of more load bearing elements than taken into consideration in the original calculation.

4.4.4 Assessment conclusions

Presented inspection was together with the project documentation used as the input for new structural calculation according to the Eurocode standards.

The re-calculation of roof trusses proved that due to the buckling some elements are overloaded. However this could be easily solved with reducing the buckling length and roof structures can be rehabilitated.

Since most of the moisture in thermal insulation has dried out there is no need to replace the stone wool. However the particle boards and consequently the whole


Figures 33 and 34. Floor elements (left). Wall elements (right) prefabricated floor structure members will have to be replaced due to their permanent deformation. Approximately 40 % of all floor elements have to be replaced.

Although using the conservative calculation assumptions and Eurocode defined loads (snow, wind and earthquake loads) the structure has sufficient load bearing capacity. This statement is valid only if the load bearing members are properly jointed. Therefore it was recommended that more detailed calculation of joints should be performed by a licensed structural engineer.

Due to the functional requirements the architect eventually decided to completely replace the timber frame structures.

4.5 Timber bridge over the river Krka in Loke

4.5.1 General description of the bridge

Although the massive most commonly oak timber bridges were quite common in the past, the use of timber for bridge construction in Slovenia is drastically reduced. Poor details and improper maintenance are two main reasons why the timber today is not appreciated in the bridge construction. The majority of preserved traditional timber bridges which are today protected as urban cultural heritage are located in the south of Slovenia in the river Krka basin.

For many years Slovenian National Building and Civil Engineering Institute performed periodical bridge inspections for Slovenian Roads Agency (DRSC) [13].

However ZAG Ljubljana is occasionally still asked to perform the assessments of timber bridges, most often when the renovation is planned. This was also the case with presented example. The municipality Straža planned to renovate the bridge (Figures 35 and 36) in 2011 due to the relatively poor condition of the bridge as it was evaluated by the company that maintains the municipality roads.

4.5.2 Assessment of the bridge in Loke

The bridge assessments are performed as visual inspection combined with a simple acoustic emission and supported with the SDT methods [13]. The presented inspection was performed using the boat. Foundations were due to the low water level also inspected visually. If necessary, inspection by diving should be preformed.

Inspection of bridges usually begins with the examination of approaches to structure (Figure 37). With the timber bridges the dilatation profile commonly installed at both ends needs to be retightened periodically. However with this bridge the profiles were not even installed. Asphalt on driveways was crushed in some areas buckled, potholes were present. The drainage on both driveways was not well constructed and not well maintained. River banks in the area of stone pillars were not well maintained, some spots of erosion were present (improper draining from the bridge).

Oak piles were relatively in good shape - regardless to the river debris found at time of inspection (Figure 38). The cross section of piles at the water level was reduced only for couple of centimetres (Figure 40). The piles in the dry riverbed (Figure 39) seem to be more problematic.

Figures 35 and 36. Timber bridge over the river Krka in Loke

Figures 37 and 38. Dilatation profile not installed (left). River debris (right)

Figures 39 and 40. Reduced cross section of pillars

Some oak bracing elements were decayed in such extent that their replacement was proposed (Figure 41). The same conclusion was also proposed for some pile caps – they were mostly damaged at their ends. This might (also) originate from the bad connection detail of bridge railing support (Figure 43). Due to the fungi attack a couple of longitudinal chords were identified as too decayed to be rehabilitated (Figure 42).

Steel fasteners (bolts and clamps) were corroded (Figure 41), but the cross section was not significantly reduced. Only their retightening was recommended.

Oak cross ties connecting the longitudinal chords (lower deck) were the most difficult elements to assess (Figure 45). They were accessible only from the pillars (bottom). From the top side they were covered with upperdecking. Oak lower deck was identified as partly decayed. Improper fasteners (smooth nails) were used for fixation of top deck members (Figure 46). Urgent measurements were proposed regarding to railing – some elements were already missing and some parts of railing completely lost their load bearing capacity (Figures 43 and 44).

Figures 41 and 42. Deteriorated pile bracing (left). Deterioration of longitudinal chords (right)

Figures 43 and 44. Deterioration pile caps (left). Deteriorated railing (right)

Figures 45 and 46. Lower deck (left). Top decking (right)

4.5.3 Assessment conclusions

Regardless to the stated decay in defects which mainly originate in poor detailing and maintenance the load bearing members were in relatively good shape. The following conclusions were gathered in the final report:

- The piles cross section was not significantly reduced and with the exception of pile presented in Figure 39 there is no need to replace or re-strengthen the members.
- Approximately 50% of bracing elements have to be replaced; all bolts have to be retightened.
- The deteriorated part of pile caps have to be at their ends replaced, the detail of railing fixation has to be improved.
- At least six longitudinal girders have to be replaced.
- 60 % (or more) lower decking members have to be replaced.
- Top decking has to be completely replaced, the use of screws or non - smooth nails is strongly recommended.
- Railing should be completely replaced, fixation details should be improved.
- All timber elements should be cleaned at renovation using high pressure water cleaner.
- Driveways should also be renovated, taking special care about the drainage and dilatation profiles.
- River debris should be regularly removed.

With presented rehabilitation the decayed elements are replaced with the same wood species, length and cross section elements. The investor also provided the feedback after the rehabilitation was completed. All our estimations proved to be correct, the investor additionally decided to completely replace the lower decking (Figures 47 and 48).

Figures 47 and 48. Rehabilitated bridge (in 2013)

6 REFERENCES

[1] Dietsch, Philipp, Köhler, Jochan (ed.) 2010. *Assessment of timber structures.* Munchen: Cost Action E55 "Modelling of the Performance of Timber Structures".

[2] Dietsch, Philipp, Kreuzinger, Heinrich 2011. *Guideline on the assessment of timber structures: Summary.* Engineering Structures, Volume 33, Issue 11, November 2011, Pages 2983–2986.

[3] ISO 13822:2001 *Bases for design of structures - Assessment of existing structures.*

[4] Diamantidis, Dimitris 2001. *Probabilistic Assessment of Exisiting Structures* - A publication of The Joint Committee on Structural Safety (JCSS), RILEM Publications S.A.R.L. The publishing Company of RILEM, 2001

[5] Mannucci, Massimo 2011. *The Italian standard UNI 11119:2004 for the in-situ diagnosis of timber structures: pros and cons after 5 years of practical application and proposal for emendations.* SHATIS'11 International Conference on Structural Health Assessment of Timber Structures, Lisbon, Portugal.

[6] Pazlar, Tomaž 2012. *Assessment and rehabilitation of timber structures in Slovenian cultural heritage structures.* iNDiS 2012 - Planiranje, projektovanje, građenje i obnova graditeljstva. Novi Sad, Serbia.

[7] Srpčič, Jelena, Pazlar, Tomaž 2011. *Some examples of the assessment of Slovenian historical timber structures.* SHATIS'11 International Conference on Structural Health Assessment of Timber Structures, Lisbon, Portugal.

[8] Sapač, Igor 2012. *Maribor Puppet Theatre.* The European Theatre Database (http://www.theatre-architecture.eu).

[9] Srpčič, Jelena 2008. Poročilo o pregledu in oceni stanja lesenega ostrešja in lesenih stropnih konstrukcij na objektu Minoritski samostan v Mariboru. Report P105/08-640-1. Slovenian National Building and Civil Engineering Institute. (In Slovenian language.)

[10] Srpčič, Jelena, Pazlar, Tomaž 2012. *Assessment and rehabilitation of Slovenian cultural heritage – case studies.* World Conference on Timber Engineering, Auckland, New Zealand.

[11] Srpčič, Jelena 2009. *Poročilo o pregledu, oceni stanja in predlogu sanacijskih ukrepov za masivne lesene stropne konstrukcije v objektu Naskov dvorec v Mariboru. Report P294/09-640-1.* Slovenian National Building and Civil Engineering Institute. (In Slovenian language.)

[12] Pazlar, Tomaž, Srpčič, Jelena 2010. *Poročilo o pregledu, oceni stanja in ugotavljanju nosilnosti stropne konstrukcije v objektu UE Kočevje. Report P258/10-640-1.* Slovenian National Building and Civil Engineering Institute. (In Slovenian language.)

[13] Srpčič, Jelena, Jarec, Bojan, Bosiljkov, Vlatko 1994 - 2002. *Pregled objekta NM 0127 in NM 128.* Records of period bridge inspections, Slovenian National Building and Civil Engineering Institute.

[14] Pazlar, Tomaž, Srpčič, Jelena 2010. *Poročilo o pregledu in oceni stanja lesenega mostu čez reko Krko v Lokah. Report P258/10-640-1* Slovenian National Building and Civil Engineering Institute. (In Slovenian language.)

[15] Pazlar, Tomaž, Kramar, Miha 2012. Poročilo o o pregledu, oceni stanja in ugotavljanju nosilnosti lesenih strešnih nosilcev in pritličnih montažnih lesenih okvirnih sten objekta Srednje šole Kočevje. Report P075/13-630-1. Slovenian National Building and Civil Engineering Institute. (In Slovenian language.)

REZIME

PROCJENA I REHABILITACIJA DRVENIH KONSTRUKCIJA U SLOVENIJI

Tomaz Pazlar

Potreba za procenom stanja drvenih konstrukcija, zahvaljujući prirodnoj biodegradabilnosti, je još evidentnija i važnija nego kod drugih konstrukcionih materijala. Tokom proteklih 30 godina, Sekcija za drvene konstrukcije Slovenačkog nacionalnog instituta za građevinarstvo (ZAG Ljubljana) sprovodila je procene stanja više drvenih konstrukcija. Ovaj rad prikazuje ciljeve sprovedenih procena i upotrebljene tehnike pregleda na slučajevima drvenih krovova, drvenih okvira zidova, međuspratnih konstrukcija i drvenih mostova. Uz svaku procenu stanja i oštećenja, navedene su i predložene mere rehabilitacije konstrukcije. U radu su diskutovani i aspekti sistemskog pristupa oceni stanja konstrukcija, kao i odgovarajuća važeća građevinska regulativa.

Ključne reči: procena, mere rehabilitacije, noseće drvene konstrukcije, Slovenija

SUMMARY

ASSESSMENT AND REHABILITATION OF TIMBER STRUCTURES IN SLOVENIA

Tomaz Pazlar

The need to assess timber structures is due to its biodegradability even more evident and important as with other construction materials. In the past 30 years Section for Timber Structures at Slovenian National Building and Civil Engineering Institute (ZAG Ljubljana) assessed several timber structures. This paper presents the objectives of assessments and techniques used on case studies of timber roofs / floors, frame walls and timber bridges, together with proposed rehabilitation measures. Aspects of systematic assessment approach and aspects of current building regulative are also discussed.

Key words: assessment, rehabilitation measures, load bearing timber structures, Slovenia

REŠENJA PREKIDA KARAKTERISTIČNIH TERMIČKIH MOSTOVA KOD OBJEKATA VISOKOGRADNJE

RESOLVING THE ISSUE OF DISRUPTING CHARACTERISTIC THERMAL BRIDGES IN BUILDING STRUCTURES

Željko JAKŠIĆ
Norbert HARMATI

STRUČNI RAD
PROFESSIONAL PAPER
UDK: = 861

1 UVOD

Savremene zgrade se izvode sa visokim nivoom izolacije, višestruko zastakljenim prozorima i vrlo često efikasnim sistemom za grejanje. Međutim ovi objekti se ne testiraju tako često na to da li su jednako energetski efikasni kao što su projektovani. Neka zvanična međunarodna istraživanja su pokazala da određeni broj novih zgrada ne ispunjavaju uvek potrebne standarde. Da bi se prevazišle razlike između projektovanog i stvarnog, neophodno je dobro isprojektovati detalje (često se u graditeljstvu sa razlogom kaže da je istina u detalju), obezbediti dobar kvalitet izrade i ugraditi kvalitetne materijale. Ključna pitanja koji su sastavni deo kvalitetnog izvođenja su: termičke performanse, termički mostovi, termička premošćavanja, zaptivenost od produvavanja, proizvoljni raspored slojeva u građevinskom elementu [1].

Toplotne performance. Svaki element omotača ili pregrade zgrade: zid, krov, krovna tavanica, prozor ili vrata, ima određenu ulogu u smanjenju toplotnih gubitaka. Izolacione karakteristike svakog od ovih elemenata su određene na osnovu proračunate vrednosti koefivijenta prolaza toplote (U-vrednosti), pri čemu niža U vrednost pretstavlja bolje toplotne performanse. Pri tome se uzimaju u obzir debljina i izolacione osobine svakog sloja materijala koji čine građevinski element,

Dr Željko Jakšić
Univerzitet u Novom Sadu, Fakultet tehničkih nauka,
Departman za građevinarstvo i geodeziju,
Trg D.Obradovića 6, 21000 Novi Sad.
Norbert Harmati
Univerzitet u Novom Sadu, Fakultet tehničkih nauka,
Departman za građevinarstvo i geodeziju,
Trg D.Obradovića 6, 21000 Novi Sad

1 INTRODUCTION

Houses built today have high levels of insulation, double glazed windows and often highly efficient heating systems. However, new houses are not often tested to see whether they are as energy efficient as predicted. Some of the official research has indicated that some new homes do not always meet the required standard in practice. In order to overcome differences among designed and real partly in better detailing (it is often to say the truth is in a detail), good quality workmanship and good material embedded. The crucial issues related to high quality construction are: Thermal Performance, Thermal Bridging, Thermal Bypass, Airtightness, Sequencing . [1].

Thermal Performance. Each element of the building envelope – a wall, a roof, a floor, a window or a door – has a role to play in minimising heat loss. The insulating effect of each of these elements is measured by its U-value; the lower the U-value, the better its thermal performance. When U-values are calculated, the thicknesses and insulating properties for each of the different layers of material that make up the building element are taken into account, including fixings such as wall ties. The phrase 'thermal mass' has become increasingly common in the context of best practice construction. It refers to the ability of materials to absorb

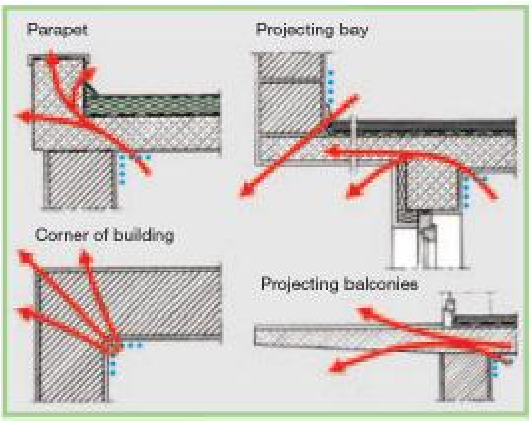
Zeljko Jaksic, PhD
University of Novi Sad, Faculty of Technical Sciences,
Department of Civil Engineering and Geodesy,
Trg D.Obradovića 6, 21000 Novi Sad.
Norbert Harmati
University of Novi Sad, Faculty of Technical Sciences,
Department of Civil Engineering and Geodesy,
Trg D.Obradovića 6, 21000 Novi Sad.

uključujući i spojna sredstva, kao što su zidne veze. Izraz ˝toplotna masa˝ je postao uobičajeni izraz za dobru izgradnju u praksi i odnosi se na sposobnost materijala da apsorbuje i zadržava toplotu. Materijali velike gustine poput betona, opeke i kamena imaju veliku termičku masu, što je od pomoći pri stabilizaciji temperature u prostoriji, otpuštajući lagano akumuliranu toplotu u uslovima niže temperature.

Termički mostovi. Termički ili hladni mostovi su slaba tačka omotača na zgradi, gde su gubici toplote veći nego kroz glavne građevinske elemente. Postoje dva osnovna tipa:

– Toplotni mostovi koji uključuju prozorske štokove u delu prozorske klupice, nadvoja i doprozornika, koji obično predstavljaju vezu unurašnjeg i spoljašnjeg dela fasadnog zida.

– Geometrijski (linijski) toplotni mostovi koji se javljaju na spoju građevinskih elemenata (sl. 1.), kao što je veza zida i krova ili kod promene geometrije kao što je ugao kod zida ili u uglu kose tavanice i zida kod krova.



Slika 1. Hladni mostovi određeni geometrijom [2]
Figure 1. Geometrically determined cold bridges [2]

Hladni mostovi moraju biti svedeni na minimalnu meru kako bi se sačuvala energija i predupredila hladna mesta gde se stvara kondenzacija i plesan. Teško je izbeći sve hladne mostove, ali se njihov efekat može umanjiti pažljivom izradom detalja.

Termički bajpas. Termički bajpas je kretanje negrejanog vazduha unutar šupljine (međuprostora) formirane pregradnim zidom ili kroz prostore kao što su šupljine u međuspratnim tavanicama i potkrovlju. slika 2. prikazuje efekat toplotnog bajpasa u nivou strehe gde postoji mogućnost da hladan vazduh prodre kroz izolaciju i odnese toplotu iz prostora sa zarobljenim vazduhom. Do nedavno termički bajpas nije bio opšte poznat fenomen tako da trenutno ne postoji na raspolaganju nijedan metod kojim bi se procenio njegov efekat. Međutim, postoje jednostavne projektantske mere koje će pomoći da se ograniče ili čak eliminišu toplotni gubici termičkog bajpasa (na primer, zatvaranje zidne šupljine odozgo i odozdo, obezbeđenje da zaptivna barijera vazdušnog prostora prati liniju izolacije kako bi se izbeglo stvaranje negrejanog prostora u krovu).

and store heat. Heavy materials such as concrete, brick and stone have high thermal mass, which can help to stabilise internal room temperatures by absorbing excess heat from the air and releasing it slowly when conditions are cooler.

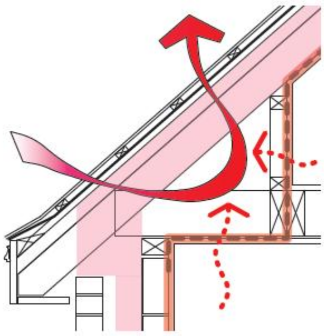
Thermal Bridging. Thermal bridges or cold bridges are weak points in the building envelope where heat loss is worse than through the main building elements. In a well insulated building thermal bridges can account for up to 50% of all heat loss. There are two main types:

– Non-repeating thermal bridges include items such as cills, lintels and jambs, which typically span between the inner and outer skins of a wall.

– Geometrical thermal bridges occur at junctions between building elements, such as between the walls and roof, and at changes of geometry, for example a corner in a wall or a hip in a roof.

To conserve energy and to prevent cold spots where condensation and mould can form, thermal bridges need to be minimised. It is not possible to avoid all thermal bridging, but the effect can be minimised with careful detailing. It is more difficult to avoid thermal bridging caused by poor workmanship, for example mortar snots in the cavity or missing insulation. Even if thermal imaging cameras are used to detect the problem, it will often be too late to avoid expensive remedial works.

Thermal Bypass. Thermal bypass is the movement of unheated air within cavity party walls or through spaces such as under-floor voids and lofts, resulting in heat loss. The Figure 2 shows a thermal bypass effect at eaves level where cold air is able to permeate the insulation and carry away heat from the void between the insulation and the airtightness barrier. Until recently thermal bypass was not widely recognised and so there is currently no method available to estimate its effect. However, there are straightforward design measures that will help to limit or even eliminate thermal bypass heat loss. Ensuring that cavity walls are sealed top and bottom is essential. In other situations such as roofs it is particularly important to ensure that any airtightness barrier follows the line of the insulation to avoid creating unheated spaces between the two.

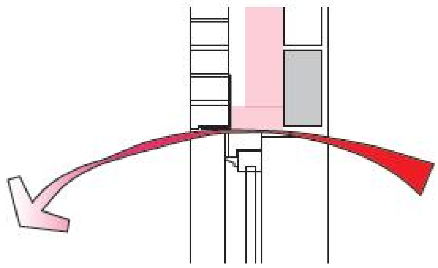


Slika 2. Termički bajpas nastaje kada vazdušno zaptivanje ne prati pravac izolacije [1]
Figure .2 The thermal bypass occurs when the air tightness barrier does not follow the insulation [1]

Zaptivenost. Zaptivenost podrazumeva sprečavanje protoka vazduha kroz praznine i procepe u spoljnom omotaču zgrade i definisan je stepenom pri kojem vazduh izlazi u situaciji kada je zgrada ispitivana na pritisak.

Uobičajena strategija za poboljšanje zaptivenosti od protoka vazduha je da se jasno utvrde barijere za zaptivanje, kao i koja je komponenta, u okviru svakog dela omotača zgrade kojom se obezbeđuje potpuna zaptivenost. Posebnu pažnju je potrebno obratiti na spojevima zaptivnih materijala (sl. 3.). Pri tome je neophodno je obezbediti zdrave životne uslove korisnika kroz odgovarajuću ventilaciju.

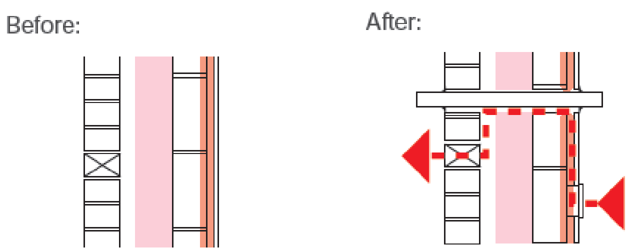
Airtightness. Airtightness means preventing air leaking through gaps and cracks in the external envelope and is defined by the rate at which air escapes when the building is pressure tested. A common strategy to improve airtightness is to clearly identify the airtightness barrier, which is the component within each part of the building envelope that provides an airtight seal. Particular care is needed where one part of the airtightness barrier meets another (Fig. 3.). It is essential to ensure healthy living conditions for the occupants through adequate ventilation.



Slika 3. Vazdušni gubici se mogu javiti oko loše uklopljenih komponenti kakav je doprozornik [1]
Figure 3. Air leakage can occur around badly fitting components such as window frames [1]

Redosled (raspored slojeva). Nepoštovanje redosleda slojeva tokom izvođenja zida može imati ozbiljne posledice po zaptivenost ili termičke performanse omotača zgrade (sl. 4.).

Out-of-sequence. Out-of-sequence work can prevent other work stages from being completed properly or damage work that has already been done, with serious consequences for the airtightness and thermal performance of the building envelope (Fig. 4).



Slika 4. Postavljanje instalacija kroz omotač dovodi do problema u zaptivanju puta protoka vazduha [1]
Figure 4. A services installation through a wall resulting in a difficult to seal air leakage path [1]

Nasumično postavljanje instalacija je osnovni uzrok protoka vazduha iz zgrade. Rupe i tiplovi za razne potrebe mogu probiti zaptivnu prepreku. Pravilno isprojektovana rešenja uključuju ispitivanje i pregled u važnim fazama tako da se bilo koja korektivna mera može sprovesti blagovremeno i na ekonomičan način. Saradnja projektanata i izvođačkog tima u ranim fazama projekta može pomoći u usaglašavanju oko odgovarajućeg pozicioniranja instalacija i razrade planova za efikasnu izradu svih neophodnih otvora.

Cilj ovog rada je da se za sve delove zgrade, karakteristične po tome što se preko njih gubi značajna količina toplotne energije i time predstavljaju problem za postizanje celovite termičke zaštite, obrati pažnja i predlože rešenja koja, do sada, nisu bila dovoljno kvalitetna, ni kao projektantska ni izvođačka.

2 SINDROM BOLESNE ZGRADE

Termin sindrom bolesne zgrade - SBS (sick building syndrome)[3] koristi se za opis situacije u kojoj stanovnici i korisnici zgrade osećaju nedostatke udobnosti ali i akutne zdravstvene probleme koji su direktno povezani sa vremenom provedenim u objektu. Ovo se može odnositi na određenu prostoriju, zonu, ili na celu zgradu. U slučaju kada su identifikovani simptomi određene bolesti i mogu biti povezani direktno sa zagađivačima vazduha tada se koristi izraz BRI (building related illness - bolest prouzrokovana zgradom). Ponekada, problemi su rezultat upravljanja i održavanja zgrade na način koji nije predviđen procedurama za tu vrstu projekta. Nekada problemi vezani za kvalitet unutrašnjeg vazduha mogu biti povezani sa lošim projektnim rešenjem, lošom izradom, primenjenim materijalima ili aktivnostima stanara ili korisnika prostora. Osnovni uzroci simptoma bolesne zgrade su: neodgovarajuća ventilacija, hemijski zagađivači u enterijeru, hemijski zagađivači iz spoljašnjih izvora i biološki zagađivači.

2.1 Biološki zagađivači

Biološki zagađivači su bakterije, buđ, polen, ali i virusi. Ovi zagađivači se javljaju uglavnom tamo gde je povišen nivo vode i vodene pare: plafoni, konzole, kanali za klimatizaciju, spojnice itd. Najčešće pojave intezivnog vlaženja građevinskih elemenata su na mestima hladnih mostova, gde posledice mogu biti širokih razmera (sl. 5). Na formiranje buđi i kondenzacije značajno utiče kvalitet unutrašnjeg vazduha. Bez pravog ventilacionog sistema, koji obnavlja vazduh, nivo vlage će biti veći i mogućnost stvaranja buđi i kondenzacije je još veća.

Buđ formirana u predelu toplotnog mosta, oslobađa spore u prostoriju. Ove spore mogu prouzrokovati različite efekte na zdravlje; od manjih alergijskih reakcija, kao što su iritiranje očiju, nosa, i grla, pa do uvećanih astmatičnih simptoma. . U slučaju dužeg boravka u prostoriji , postoji rizik da ove alergijske reakcije prerastu u hronična obolenja. . Osim na ljudsko zdravlje, formiranje buđi i kondenzacija dovode i do različitih oštećenja zgrade, na primer, može uticati na unutrašnju obradu zida uzrokujući postepeno raspadanje strukturalnih elemenata.

Out-of-sequence services installation is a major cause of air leakage from buildings. Holes and fixings for services can puncture airtightness barriers and can often be too awkwardly located to seal up afterwards. Properly planned projects include testing and inspections at key stages so that any remedial measures can be carried out in a timely and cost-effective manner. Early coordination between the designers and the construction team can help by agreeing suitable locations for services and developing strategies for effective making-good of any necessary openings.

The aim of this work is to find more detailed and better solutions for structural elements characterized by the loss of significant amount of heat. These elements are problematic for achieving a comprehensive thermal protection, focusing more attention and suggesting solutions that previously have not been sufficiently good regarding both their design and construction.

2 SICK BUILDING SYNDROME

The term sick building syndrome (SBS) [3] is used to describe situations in which residents and users of the building experience discomfort and acute health problems that are directly related to the time spent in the building. This may refer to a particular room, a zone, or the entire building. In case when the identified symptoms of a specific disease can be related directly to air pollutants, then the term building related illness (BRI – a disease caused by the building) is being used. Problems are sometimes the result of operating and maintaining the building in a manner not foreseen by procedures for the given type of project. Sometimes, problems related to indoor air quality may be associated with poor project design, poor workmanship, applied materials or activities of occupants or users of the space. The main causes of sick building symptoms are the following: inadequate ventilation, the presence of chemical pollutants in the interior, penetration of chemical pollutants from external sources and the presence of biological pollutants.

2.1 Biological pollutants

Biological pollutants include bacteria, moulds, pollen, as well as viruses. These pollutants occur primarily in places with high level of water and steam: ceilings, cantilevers, air conditioning ducts, fittings, etc. The most common locations on building elements where intensive moisture occurs are cold bridges, where the consequences can be widespread (Figure 5). The formation of mould and condensation is strongly influenced by indoor air quality. Without proper air-restoring ventilation system, humidity level will be increased resulting in turn with high possibility of mould and condensation to occur.

Once the mould is formed in the zone of thermal bridge, it releases its spores into the room. These spores can cause a variety of health effects, from minor allergic reactions as irritated eyes, nose, and throat to increased asthma symptoms. This is because spores are allergens that cause sinusitis, rhinitis and asthma. Given the usually prolonged exposure of people in the room, there is a risk of allergic reactions being developed into chronic diseases. The insufficient level of restoration of the indoor air will lead to the formation of mould and enhance its adverse effects on human health.

a) Razmnožavanje plesni u velikoj razmeri po plafonu na mestu spoja balkona sa tavanicom [4]
a) A large-scale reproduction of mould on the ceiling at the location where the balcony meets the ceiling [4]

b) Kondenzacija se javlja na mestima bez izolacije i vremenom, bez dodatne ventilisanosti, pojavljuje se plesan [5]
b) Condensation occurs in locations with no insulation; over time, without additional ventilation, mould appears in these locations [5]

c) Vlaga uzrokovana termičkim mostovima je dobra podloga za nastanak plesni [22]
c) Moisture induced by thermal bridges is a good substrate for the development of mould [22]

Slika 5. Posledice postojanja termičkih mostova
Figure 5. Consequences of the thermal bridge

3 DIFUZIJA VODENE PARE

Prema definiciji [5] difuzija vodene pare je fizička pojava čija se veličina izračunava za spoljne građevinske konstrukcije i konstrukcije koje se graniče sa negrejanim prostorijama, osim za konstrukcije koje se neposredno graniče sa terenom (pod na tlu, ukopani zidovi, ukopane tavanice). Sve građevinske konstrukcije zgrade moraju biti projektovane i izgrađene na način da se vodena para u projektnim uslovima na njihovim površinama ne kondenzuje.

Zgrada mora biti projektovana i izvedena na način da se kod namenskog korišćenja vodena para, koja zbog difuzije prodire u građevinsku konstrukciju, ne kondenzuje. U slučaju da dođe do kondenzacije vodene pare u konstrukciji, ona se posle računskog perioda isušivanja mora sasvim osloboditi iz građevinske konstrukcije. Kondenzovanje vodene pare je, upravo, jedan od pojavnih oblika vlage u građevinskoj konstrukciji, koja vremenom može dovesti do oštećenja građevinskog materijala (npr. korozija, pojava buđi).

Dozvoljena temperatura unutrašnje površine spoljne građevinske konstrukcije na bilo kom mestu (i na mestima toplotnih mostova), mora biti veća od temperature tačke rose, za date projektne uslove (temperatura i relativna vlažnost vazduha u prostoriji). Minimalna toplotna otpornost za sprečavanje orošavanja unutrašnjih površina građevinske konstrukcije izvan zone toplotnog mosta izračunava se za uslove perioda grejanja (zimski period). Na mestima toplotnih mostova za ocenu opasnosti od orošavanja merodavna je temperatura tačke rose, koja je data tabelarno i u zavisnosti je od relativne vlažnosti vazduha i temperature vazduha. Uvidom u tabelu uočava se da što je niža temperatura vazduha i niža relativna vlažnost vazduha to je niža i temperatura tačke rose. Nakon toga se nameće opšti zaključak da je za istu relativnu vlažnost vazduha neophodno značajno podići temperaturu vazduha oko toplotnih mostova kako ne bi došlo do orošavanja. Zbog toga je potrebno tražiti optimalna rešenja kako bi se neutralisala sva potencijalna pojavna mesta toplotnih mostova.

Na slici 6. se može videti da je, na 20°C u prostoriji, relativna vlažnost vazduha (koja je najviša prihvatljiva relativna vlažnost vazduha kako bi se sprečila pojava kondenza na površini konstrukcije) u funkciji temperature na površini zida (ili temperatura tačke rošenja)

Poor construction quality will lead to the accumulation of inadequacies in the form of many thermal bridges and the lack of proper ventilation. The formation of mould and condensation will cause various damages to the building. This can affect the internal wall surface, leading to the gradual decay of structural elements.

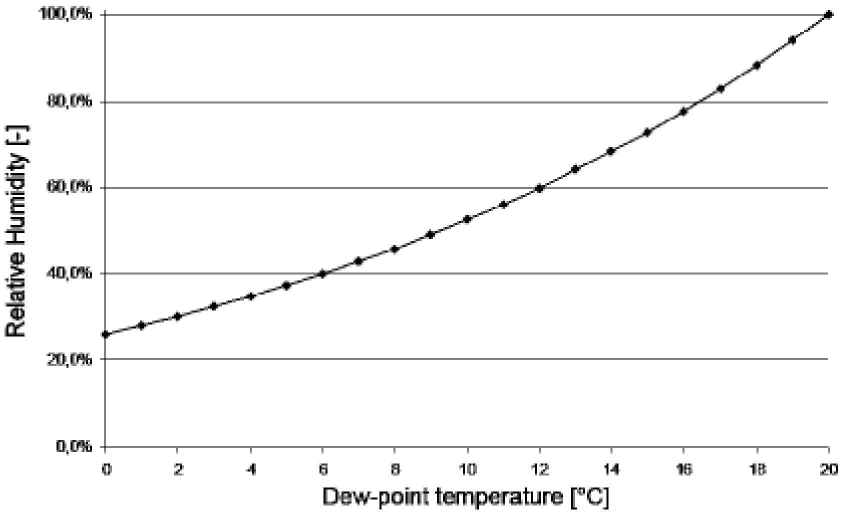
3 WATER VAPOUR DIFFUSION

According to [5], water vapour diffusion is a physical phenomenon the magnitude of which can be calculated both for external building structures and engineering structures in the neighbourhood of heated rooms, except for structures that are directly adjacent to the ground (flooring on the ground, buried walls, dug ceilings). Engineering structures need to be designed and constructed in a way to prevent water vapour from being condensed on their surfaces.

Buildings need to be designed and constructed in a way to prevent water vapour that penetrates the building structure as a result of diffusion, from being condensed when the building is used in a proper manner. In case when water vapour is condensed in the structure, by the end of the calculated period of drying, it has to be completely removed from the building structure. Condensation of water vapour is indeed one of the manifestations of moisture in building structures, which can cause damage to building materials over time (e.g., corrosion, mould growth – Figure 5).

The permissible temperature of the inward surface of the external building structure at any point (and locations of thermal bridges) has to be higher than the condensation point for the given design conditions (temperature and relative air-humidity in the room). The minimum thermal resistance for preventing the interior surfaces of the building structure outside the zone of thermal bridge from dew is calculated for the conditions during heating periods (winter). The risk of condensation on locations of thermal bridges is assessed based on the dew point temperature, which is given in tabular form and depending on the relative air humidity and temperature. Based on the table it can be seen that the lower the temperature and relative humidity, the lower is the dew point temperature. The general conclusion is that the same relative air humidity requires the air temperature around the thermal bridges to be increased to prevent condensation. Therefore, it is necessary to find optimal solutions in order to eliminate any potential location where thermal bridges can possibly occur.

In the figure 6 at 20°C ambient temperature the Relative Humidity (which us the acceptable highest Relative humidity to avoid surface condensation) can be seen as a function of surface the temperature (or dew-point temperature):



Slika 6. Relativna vlažnost vazduha kao funkcija temperature tačke rošenja na 20°C temperature prostorije [17]
Figure 6. Relative Humidity as a function of dew-point temperature at 20°C ambient temperature [17]

Ako je poznata kritična temperatura površine zida (što je najniža temperatura površine zida u prostoriji) maksimalno prihvatljiva relativna vlažnost vazduha može se proceniti na slici 6. To je nivo vlage u prostoriji kada se ne javlja kondenzacija.

4 TERMIČKI MOSTOVI

Jedan od ključnih aspekata izgradnje energetski efikasne zgrade jeste njena toplotna izolovanost. Pod tim se podrazumeva korišćenje adekvatnih materijala, odgovarajuće debljine kako bi se obezbedile zadovoljavajuće toplotne karakteristike. Međutim, to nije dovoljna garancija kvalitetne termičke zaštite objekta, jer je preduslov dobre termičke izolovanosti dobro postavljena toplotna izolacija, u kontinuitetu, po obodu objekta bez prekida. Na taj način se ne umanjuje efekat zaštite objekta, već se na tim delovima izrazitije ispoljava efekat termičke neizolovanosti, sa izraženim posledicama. Takvi prekidi se nazivaju ˝hladni mostovi˝, odnosno termički mostovi. Ovo je jedan od aspekata kome se još uvek u Srbiji ne poklanja naročita pažnja.

4.1 Delovi zgrade karakteristični po termičkim mostovima

Termički mostovi su lokalizovani delovi zgrade koji prikazuju uvećane toplotne gubitke. Mogu biti uzrokovani konstruktivnim rešenjem (npr. ispadi ploče, terase) ili korišćenjem materijala sa većom toplotnom provodljivošću (npr. konstrukcija prozora od aluminijuma bez termičkih prekida). Često su rezultat same strukture zgrade, pojavljuju se na spoju zidova i podova (sl. 5, c-2), između zidova i krova (sl. 5, c-1), u uglovima ili oko prozora ukoliko nisu pravilno postavljeni (sl. 5, c-3).

Praksa je da se za potrebe analize stanja konstrukcije ili preventivnog delovanja u obzir uzimaju nadzemni delovi objekta, dok su podzemni delovi (suteren i temelji) zgrade često zanemaruju (sl. 7).

If the critical surface temperature is known (which is the lowest surface temperature of the environment), the maximum acceptable Relative Humidity can be estimated from Figure 6. This is the moisture level in the internal environment when there is no surface condensation occurs.

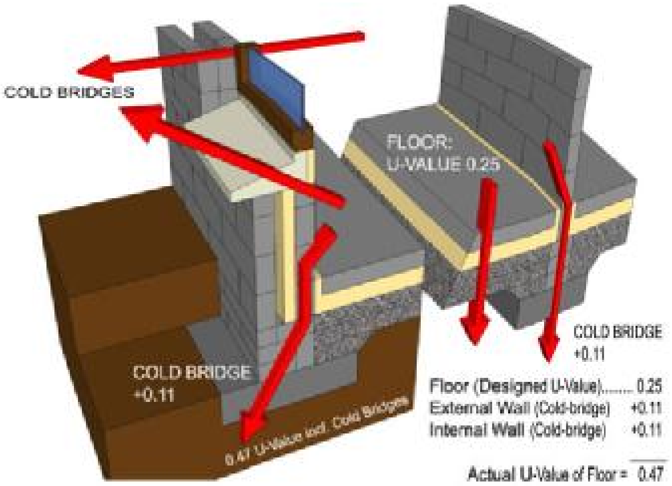
4 THERMAL BRIDGING

One of the key aspects of constructing an energy-efficient building is its thermal insulation. This implies using appropriate materials of adequate thickness in order to ensure proper thermal performances. However, this is not sufficient to ensure a high quality thermal insulation, since good insulation requires thermal insulation to be positioned continuously around the perimeter of the building without any interruption. In this way, the protecting effects are not diminished; instead, in these parts of the building, the lack of insulation is more pronounced, with significant consequences. Such breaks are called "cold bridges" or thermal bridges. This is one of the aspects which still lack proper attention in Serbia.

4.1 Parts of the building characterized by thermal bridges

Thermal bridges are local parts of the building experiencing high levels of heat loss. They can result from design solutions (e.g. missing panels, presence of terraces) or using materials of higher thermal conductivity (e.g. aluminium windows without thermal breaks). They are often the result of the building structure itself, occurring at the junction of walls and floors (Figure 5, c-2), walls and the roof (Figure 5, c-1), in corners or around windows if they are not properly installed (Figure 5, c-3).

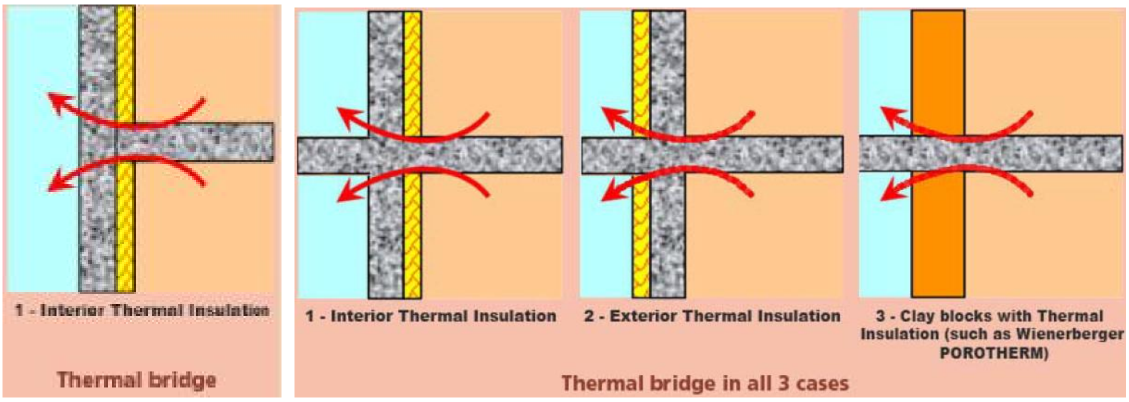
In practice, when analyzing the condition of the structure or taking preventive measures, only the above-ground parts of the building are taken into account, while its underground parts (foundations and basement) are often neglected (Figure 7).



Slika 7. Često zanemarivani termički mostovi u podzemnim delovima zgrade [23]
Figure 7. The often neglected thermal bridges in the underground parts of buildings [23]

Termički mostovi vrlo često ne mogu biti u potpunosti izbegnuti, tako da se intervencija svodi na umanjivanje efekta hladnih mostova na tolerantan nivo. Međutim, postoje i takvi termički mostovi koji se ne mogu tolerisati te se moraju odgovarajuće tretirati. Postavljanjem toplotne izolacije sa unutrašnje strane fasadnog zida ˝otvaraju˝ se mnogi termički mostovi (sl. 8.), koji mogu biti izbegnuti promenom pristupa projektovanja rešenja termoizolacije.

In a majority of cases, thermal bridges cannot be completely avoided, so that interventions are focused on minimizing the effects of cold bridging to a tolerable level, while not producing damage to the building. However, there are thermal bridges that cannot be tolerated and they need to be treated adequately. By installing thermal insulation on the inward looking face of the facade wall many thermal bridges are created (Figure 8), which could be avoided by changing the approach to the design of thermal insulation.



Slika 8. Termički most na spoju između tavanice i u zoni prepusta tavanice u spoljašnji prostor [22]
Figure 8. Thermal bridge at the junction between the ceiling and its overhang area to the outside [22]

5 PREDLOG REŠENJA PREKIDA TERMIČKIH MOSTOVA

5.1 Postizanje toplotnog kontinuiteta i zaptivenost spojeva - opšti principi

Neophodno je obratiti pažnju tokom postavljanja izolacije na kvalitetno izvođenje i obezbeđenje zaptivenosti spojeva koliko god je to moguće. Ovaj pristup se naziva ˝ kontuitet toplotne izolacije˝ i ˝vazdušna zaptivenost zgrada˝. [8]

Da bi dobra termička izolacija bila delotvorna mora biti bez prekida. To znači da nema prekida između izola-

5 A POSSIBLE SOLUTION FOR BREAKING THE THERMAL BRIDGES

5.1 Achieving thermal continuity and air tightness– Principles

In designing and building for low heat loss, both good insulation and control of air infiltration are needed. Good attention to detailing is necessary during installation for insulation to work effectively and to ensure unwanted air infiltration is eliminated as far as practicable. This translates into "thermal continuity of the insulation" and "air tightness of the building". [8]

cionih ploča ili traka, što se postiže na jedan od sledećih načina:

– Izolacione ploča sa zubom (˝falcom˝) a ne ravnim stranama bočnih spojeva daju bolju pokrivenost spoja.

– Izolaciju seći tako da odgovara međuprostoru u konstrukciji. Priljubiti trake izolacije jednu uz drugu i zategnuti ih kako ne bi bile labavo postavljane i uz zatvarače (poklopce) međuprostora konstrukcije.

– Postaviti krovnu izolaciju preko gornjeg niza blokova kod strehe, pre oblaganja krova, nakon što je izolacija postavljena sve do vrha zida, a zatim i na sam vrh zida.

5.2 Zgrada bez termičkih mostova

Brojna istraživanja iz oblasti termičke zaštite pokazuju da se na različitim delovima zgrade pojavljuju gubici neujednačenog intenziteta (sl. 9.). Upravo iz tog razloga, na globalnom nivou u nacionalnim pravilnicima o energetskoj efikasnosti, su definisani maksimalni dozvoljeni gubici karakterističnih delove zgrade. Optimalno rešenje termičke izolovanosti zgrade, u savremenim uslovima, praktično ne znači samo da svaki deo zgrade bude izolovan, već da su ta rešenja tehnički i ekonomski racionalna. To se naročito odnosi na termičke mostove.

For thermal insulation to be effective, it needs to be continuous. Tis will be achieved in one of the next way:

– Insulation boards with stepped rather than flat butt joints give better continuity.

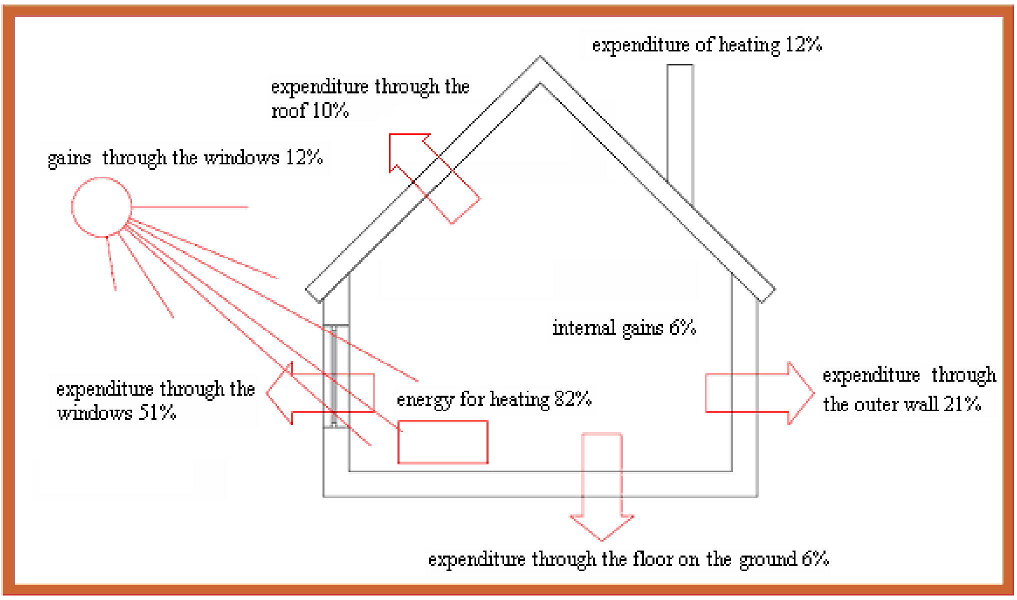
– Cut cavity insulation to suit. Butt the sheets tightly to each other, as well as tight up against cavity closers and loose fill insulation.

– Install roof insulation over the top course of blocks at the eaves, prior to felting at the roof having brought the wall insulation up to the top of the wall and bring the wall insulation right up to the top.

5.2 A building without thermal bridges

As shown by numerous studies from the field of thermal protection, losses appear on different parts of the building with uneven intensity (Figure 9).

For this reason, the maximum allowed losses in characteristic parts of the building are defined at global level in national regulations on energy efficiency. Nowadays, the optimum solution for insulating the building does not mean that its every part is insulated, but that these solutions are both technically and economically rational. This is especially true of thermal bridges.



Slika 9. Delovi zgrade uključeni u bilanse toplotnih gubitaka zgrade [10]
Figure 9. Parts of the building included in its heat loss balance [10]

Pojava termičkih mostova je moguća na svim delovima zgrade gde ne postoji kontunitet konstrukcije i pregrada (obloga).

Kao posebno izraženi problem, termički mostovi se javljaju na sledećim delovima zgrade:

• Krovne terase za nazitkom, bez obzira na njegovu visinu (prohodne, neprohodne, ˝zelene˝[17]).

• Ispusti konstrukcije van fasade, kao otvoreni, negrejan prostor (nastrešica, balkon, lođa isl).

• Temelji, prema spoljašnjim temeljnim zidovima i u zoni mržnjenja, kod objekata sa ili bez podzemne etaže.

Thermal bridges may occur in any part of the building where a discontinuity exists between the structure and the partitions (claddings).

As a distinct problem, thermal bridges occur in the following parts of the building:

• A rooftop terraces with a jamb-wall, regardless of its height (passable, impassable, "green").

• Overhangs on the construction over the facade, as an open, unheated space (eaves, balcony, loggia, etc.).

• foundations, towards the exterior foundation walls and in the freezing zone in buildings with or without underground floors.

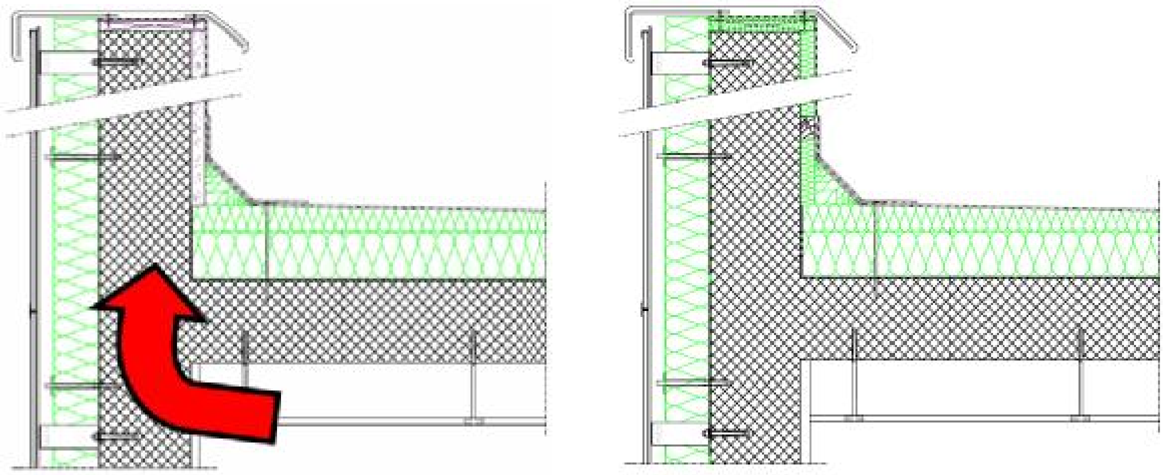
Krovne terase sa nazitkom

Nazidak na krovnim terasama („ravan krovovi“) predstavlja kontinuitet fasadnog elementa i najčešće je bez rešenja za sprečavanje pojave hladnih mostova na spoju sa krovnom tavanicom i njenom termičkom izolacijom (slika 10a). Pored procurivanja, ovo je jedan od razloga zašto se krovne terasa zakrovljuju. Međutim, intervencija je jednostavna i ne preterano tehnički i ekonomsko zahtevna (slika 10b). Ono o čemu treba voditi računa je proračunski definisanje potrebne debljine sloja termičke zaštite, jer se često u praksi proizvoljno određuje debljina ovog sloja. To je potrebno što se, osim sprečavanja stvaranja hladnih mostova, konstrukcija atike štiti od stvaranja dodatnih napona usled temperaturnih razlika, čime se sprečava pojava pukotina na spoju sa završnom, krovnom tavanicom [13].

Roof terraces with a jamb wall

A jamb wall on roof terraces ("flat roof") makes the continuity of the facade element and usually lacks a solution against the appearance of cold bridging at the junction of the roof ceiling and its thermal insulation (Figure 10a).

In addition to leakage, this is one of the reasons for roofing the roof terraces. However, the intervention is simple and technically and financially not demanding (Figure 12b). What we should take into account is to find a calculation-based definition of the required thickness of the thermal protection layer, because in practice, the thickness of this layer is often determined arbitrarily. In addition to protecting against cold bridges, this also protects the attic structure against additional stresses due to temperature differences, thus preventing also the occurrence of cracks at the junction with the final, roof ceiling [13].



Slika 10. Krovna terasa: a) sa termičkim mostom; b) bez termičkog mosta [9]
Figure 10. Roof terrace: a) with thermal bridge, b) without thermal bridge [9]

Ispusti konstrukcije van fasade

Konstruktivni ispusti, u smislu rešenja termičkih mostova, predstavljaju najosetljivije mesto na zgradi. S jedne strane, zbog položaja u odnosu na unutrašnji prostor i uticaja na korisnike tog prostora (zdravlje korisnika, estetika - slika 5,c-4), a s druge strane rešenja su kompleksna zbog konstrukcije ovog dela zgrade.

Brojna su varijantna rešenja. Elementi zgrade van fasade u konstruktivnom smislu predstavljaju konzole, ispuštene delove ploče ili grede (na koje se oslanja ploča). Preko njih se direktno prenosi toplota u spoljašnji prostor. S obzirom da konzolni element mora biti vezan za osnovnu konstrukciju, prvobitna rešenja su usvajana sa termičkom oblogom po obodu ispuštenog elementa. Međutim, vremenom su se pokazali brojni nedostaci:

– U slučaju velike zastupljenosti ispusta na objektu predstavlja značajnu investiciju.

– Dodavanjem termičkog sloja sa gazišne strane elementa povećava se ukupna visina preseka ispuštenog dela u odnosu na ukupnu visinu osnovne konstrukcije u zatvorenom prostoru, čime se formira stepenik, te remeti kontinuitet kretanja iz jednog prostora u drugi.

Structural overhangs above the facade

Structural overhangs as the means of resolving the issue of thermal bridges are the most sensitive location on the building because of their position in relation to the interior space and the impact on their users (health effects, aesthetics – Figure 5,c-4), and because the solutions are complex due to the structure of this part of the building.

There are numerous alternative solutions. Structurally, the elements that fall outside the building facade are cantilevers, overhung parts of the slab or beams (which support the slab). The heat is directly transferred to the outside. Since the only structurally correct solution for the cantilever element is to be attached to the basic structure, the initial solutions are adopted with thermal coating along the edge of the overhung element. However, numerous shortcomings occur over time:

– In the event of major presence of overhangs on the building, they require significant investments.

– Providing the tread side of the element with a thermal layer increases the overall height of the overhang

– S obzirom na izloženost uticajima visokih/niskih temperatura, atmosferilijama, zaštitni sloj termoizolacije je podložan oštećenju, što za posledicu ima provlaživanje termoizolacije i gubljenje njenih svojstava u značajnom procentu.

Sledeći korak u iznalaženju optimalnih rešenja kretao se u pravcu neutralisanja termičkih mostova fizičkim odvajanjem ispuštenih delova konstrukcije od osnovne konstrukcije. Ovaim putem se obezbeđivao visok stepen prekida hladnih mostova, a jedino osetljivo mesto mogla su biti mesta veze za fasadnu konstrukciju matičnog objekta. Međutim i u tom slučaju moguće je iznaći valjana rešenja. Jedno od rešenja je ugrađivanje podloške od estrudiranog stirena ispod nožice veznog elementa. Drugo rešenje je potpuno odvajanje eksterne konstrukcije, čime ona postaje fizički potpuno nezavisna, čime je omogućen visok stepen fleksibilnosti zgrada u promišljanju sadržaja proširenog stanovanja (dodavanje/oduzimanje balkona/lođa, dodavanje kvadrata stambenog prostora kod objekata gde je planiran rezervni prostor za proširenje), slika 11.b. U oba slučaja spoljašnju konstrukciju je potrebno proračunati i dimenzionisati kao samonoseću.

part in relation to the overall height of the basic indoors structure, forming thereby a step that disrupts the continuity of movement between rooms.

– Given the exposure to the effects of high/low temperatures and weather, the protective thermal insulation layer is susceptible to failure, which leads to the occurrence of moisture in thermal insulation and significant drop of performances.

The next step in finding optimal solutions has been directed towards neutralizing thermal bridges by separating the overhang structural parts from the basic structure. This was aimed at securing a high level interruption of cold bridges, while the only sensitive locations could be the connecting points to the facade structure of the parent object. However, in this case it is also possible to find proper solutions. One of them is installing washers made of extruded styrene below the toe of the binding element. Another solution is to completely separate the external structure, so that it becomes physically completely independent thus providing the building with a high degree of flexibility in terms of expanded housing (adding/removing a balconies/loggia, adding square meters of residential space in buildings where a reserve area is planned for extending the space), Figure 11.b. In both cases, it is necessary to calculate and dimension the external structure as self-supporting

a)

b)

Slika 11. Varijante nošenja balkona: a) Konstrukcija balkona nošena osnovnom konstrukcijom zgrade; b) nezavisno nošena konstrukcija balkona [12]

Figure 11. Balcony support options: a) structure suppoted balconies, b) independently supported balconies [12]

Međutim, u savremenoj praksi su se pojavila i smelija rešenja prekida termičkog mosta. Upravo na način za koji se ranije smatrao neprimerenim. Povezivanje elementa ispusta sa osnovnom konstrukcijom preko veznog elementa, na mestu kontaktnih ravni, koji predstavlja kompaktnu strukturu sastavljenu od dve metalne pločice između kojih je umetnut estrudirani polistiren (debljina sloja izolacije u skladu sa proračunatom dimenzijom) ili materijal sa slabo

However, in contemporary practice some bolder solutions have also emerged for interrupting the thermal bridge, which have previously been considered appropriate: connecting the overhang element with the basic structure through a binding element on the location of contact planes, which yields with a compact structure consisting of two metal plates with the extruded polystyrene (the thickness of insulation is in accordance with the calculated dimension) or a material of low heat izraženim svojstvom prenosa toplote. Trenutno postoji mali broj odgovarajućih rešenja termičkih prekida primenljivih za konstruktivne elemente. Međutim, dostupni su proizvodi za termičke prekide kod oslanjanja balkona, nastrešica, spoljašnjih stepenica i veza konstrukcije koji prolaze kroz izolovanu fasadnu oblogu zgrade. Ova rešenja su projektovana tako da imaju minimalnu količinu armature u kontinuitetu sa jedne na drugu stranu, obloženu izolacionim materijalom.

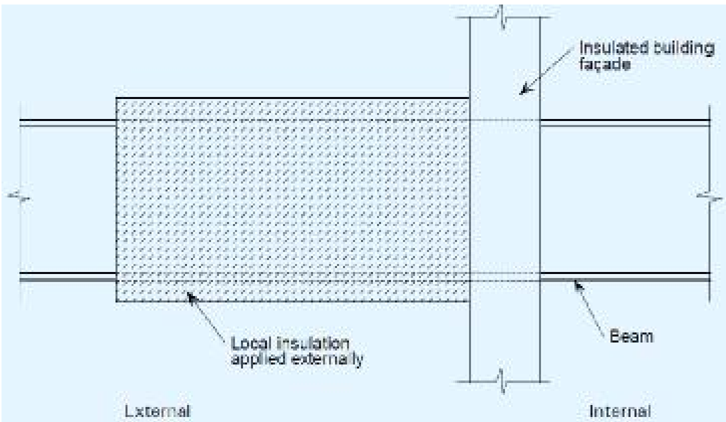
a) Konstruktivni element koji prolazi kroz omotač zgrade bez prekida

Detaljem na slici 12. prikazan je čelični nosač koji probija izolovani spoljni omotač zgrade i lokalno je izolovan spolja do fasadne obloge kako bi podigao površinsku temperaturu dela nosača u prostoriji.

conductivity between them. Curently, there are relatively few proprietary thermal break solutions available for structural elements. However, thermal break products are available to support balconies, canopies, external staircases and structural members that penetrate the insulated building envelope. These proprietary solutions are designed to have the minimum amount of continuos metal from one side to the other and metal components are encased in an insulating material.

a) Structural element penetrating building envelope

At the detail in figure 12 is illustrated a beam that penetrates the insulated envelope of a building and is locally insulated externally to the building envelope to raise the internal surface temperature of the member.



Slika 12. Lokalno spolja izolovana greda [12]
Figure 12. Locally externally insulated beam [12]

Isti nosač bez izolacije i sa dve varijante izolacije je modelovan u materijalu [12]. Rezultati modelovanja su pokazani na Tabeli 1.

The same beam without insulation and the beam with two insulation options were modelled in the paper [12]. The results of the modeling are shovn Table 1.

Tabela 1. Rezultati termičkog modelovanja za nosač sa lokalnom izolacijom [12]
Table 1. Thermal modelling results for a beam with local insulation

Opis modela Description of model	Minimalna temperatura površine nosača u prostoriji Minimun internal surface temperature	Temperaturni faktor Temperature factor f_{Rsi}
Nosač bez izolacije Beam without insulation	8.90 ^{0}C	0.51
Nosač spolja izolovan za 360mm Beam insulated externally for 360mm	9.48 ^{0}C	0.54
Nosač spolja izolovan za 1000mm Beam insulated externally for 1000mm	9.66 ^{0}C	0.55

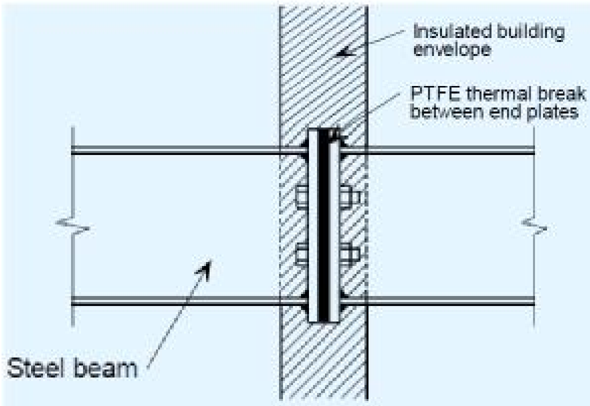
It can be seen from the results that insulating the beam to a lenght of 1m on the external side of the wall only has a marginal effect on the minimum internal surface temperature. The temperature factor for the beam without any insulation is acceptable for storage, office and retail buildings. For other application, the beam should idally be boxed in and insulated along its entire exposed length.

b) Konstruktivni element koji prolazi kroz omotač zgrade sa termoprekidom

Sledeći primer prikazuje model veze sa termoprekidom (sl.13). Modelovane su četiri varijacije termoprekida, sa 5 i 10mm sloja ekstrudirane izolacije sa zavrtnjevima od čelika ili nerđajućeg čelika. Rezultati termičkog modelovanja su prikazani u Tabeli 2.

b) Structural element penetrating building envelope with a thermal break

Next example shows the model of the thermal break connection (Fig 13). Four variations of the thermal break were modelled; 5 i 10mm layers of insulation with either steel or stainless steel bolts. The results of thermal modelling are shown in Table 2.



Slika 13. Nosač povezan završnim pločicama sa izolovanim termičkim prekidom [12]
Figure 13. Beam joined by end plates with insulated thermal break [12]

Tabela 2. Rezultati termičkog modelovanja za nosače sa izolovanim termoprekidima [12]
Table 2. Thermal modelling results for a beam with insulated thermal break

Opis modela Description of model	Ekvivalentna termička provodljivost* Equivalent thermal conductivity* λ_{eq} W/mK	Minimalna temperatura površine nosača u prostoriji Minimun internal surface temperature	Temperaturni faktor Temperature factor, f_{Rsi}
Kontinualni nosač bez termičkog prekida Continous steel beam with no thermal break	3,48	7,5 ^{0}C	0,50
Prekid od 5mm izolacije sa čeličnim zavrtnjevima Break by 5mm insul. with steel bolts	7,60	5,8^{0}C	0,43
Prekid od 5mm izolacije sa čeličnim nerđajućim zavrtnjevima Break by 5mm insul. with stainless steel bolts	5,80	6,8^{0}C	0,47
Prekid od 10mm izolacije sa čeličnim zavrtnjevima Break by 10mm insul. with steel bolts	5,70	6,9^{0}C	0,48
Prekid od 10mm izolacije sa čeličnim nerđajućim zavrtnjevima Break by 10mm insul. with stainless steel bolts	3,90	8,6^{0}C	0,55

Napomena:*Odgovarajući termički konduktivitet preko omotača zgrade. Ovo modelovanje je izvršeno sa unutašnjom temperaturom od 20^{0}C i spoljnom temperaturom od 5^{0}C.
Note:*Equivalent thermal conductivity over the thickness of the building envelope. This modeling was carried out with an internal air temperature of 20^{0}C and an external temperature of -5^{0}C.

Iz rezultata se može videti da korišćenje završnih pločica može eventualno stvarati termičke mostove lošije nego kod nosača bez prekida. To je zbog toga što dodatni prostor za kontakte kreiran od završnih pločica

It can be seen from the results that the use of end plates can potentially make the thermal bridge worse than the continuous beam. This is because the extra area of contact created by the end plates counteracts poništava povećani toplotni otpor izolacionog sloja. Drugi dodatni faktor je da u slučaju nosača bez prekida, izolacija omotača zgrade se formira oko čitavog nosača dok je kod nosača sa spojem sa završnim pločicama nešto od te izolacije uklonjeno i zamenjeno završnim čeličnim pločicama. Jedino model sa slojem izolacije od 10mm i zavrtnjevima od nerđajućeg čelika pokazuje poboljšanje termičkih performansi u odnosu na nosač koji bez prekida prodire oblogu zgrade. Termičke performanse detalja termičkog prekida ovakvog tipa su osetljive na geometriju veze (npr. veličinu grede, debljinu pločice i prečnik zavrtnjeva) i na debljinu sloja materijala za termički prekid.

c) Konstruktivni element koji prolazi kroz omotač zgrade sa ugrađenom jedinicom za termoprekidom

U sledećem primeru je korišćena jedinstvena jedinica za termičke prekide, takođe u slučaju kada nosač probija izolacionu oblogu zgrade. (sl. 14)

the effect of the increased thermal resistance of the insulating layer. Another contributing factor is that for the continous beam case, the insulation of the building envewlope is formed all round the beam whereas with and plates some of this insulation is removed and replaced by the steel end plates. Only the model with a 10mm of insulating layer and stainless steel bolts shows an improvement in the thermal performance over that of a continuos beam penetrating the bullding envelope. The thermal performance of this type of thermal break detail is sensitive to connection geometry (e.g. beam size, and plate thicknes end bolt diameter) and the thickness of the thermal break material.

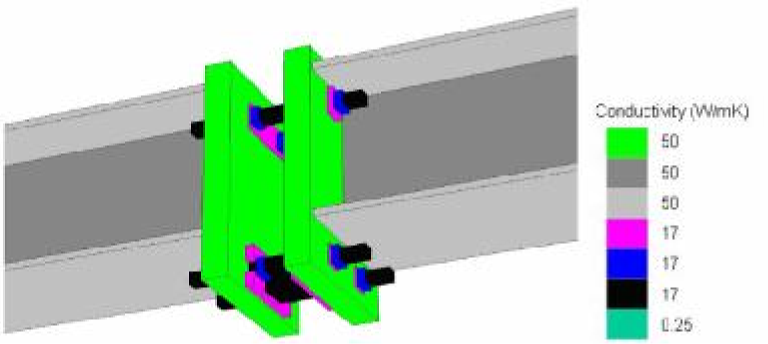
c) Structural element penetrating building envelope with built-in thermal break unit

In the next example is used an unique thermal break unit in a beam that penetrates the insulated envelope of a building (Fig. 14).

Slika 14. "Isokorbova" jedinica za termičke prekide [12]
Figure 14. "Isokorb" thermal break unit [12]

Slika 14 prikazuje "Isokorbovu" jedinicu sa M22 zavrtnjima koji je formiran korišćenjem programa za analizu trodimenzionalnog uobičajenog stanja termičke kondukcije. U modelu je izolaciona obloga bila debljine 80mm, "Isokorbova" jedinica 80mm debljine a nosači su bili pričvršćeni na "Isokorbovu" jedinicu korišćenjem završne pločice debljine 40mm. Termički model je pokazan na slici 15.

Figure 14. shows an "Isokorb" unit with M22 bolts which was modelled using a three-dimensional steady state thermal conduction analysis program. In the model the building envelope insulation was 80mm thick, the "Isokorb" unit was 80mm thick and the beams were bolted to the "Isokorb" unit using 40mm thick end plates. The thermal model is shown in Figure 15.



Napomena: Izostavljena izolacija
Note: Insulation ommited clarity

Slika 15. Termički model za "Isokorbovu˝ jedinicu [12]
Figure 15. Thermal model for an "Isokorb" unit [12]

Rezultati termičkog modelovanja, sa ili bez "Isokorbove" jedinice, predstavljen je u Tabeli 3.[12]

The results of the thermal modelling, with amd without "Isokorb" unit, are presented in Table 3.

Tabela 3. Rezultati termičkog modelovanja za nosače sa "Isokorbom"
Table 3. Thermal modelling results for a beam with "Isokorb"

Opis modela Description of model	Toplotni gubici na termičkom mostu Thermal bridge heat loss	Minimalna temperatura fasadne obloge u prostoriji Minimun internal surface temperature	Temperaturni faktor Temperature factor, f_{Rsi}
Nosač sa "Isokorb" KST 22 Beam with "Isokorb" KST 22	0,43	15,2 ^{0}C	0,81
Nosač bez termičkog prekida Beam without thermal break	1,0	7,5 ^{0}C	0,50

Napomena: Ovo modelovanje je izvedeno sa unutašnjom temperaturom od 20^{0}C i spoljnom temperaturom od 5^{0}C.
Note: This modeling was carried out with an internal air temperature of 200C and an external temperature of -50C.

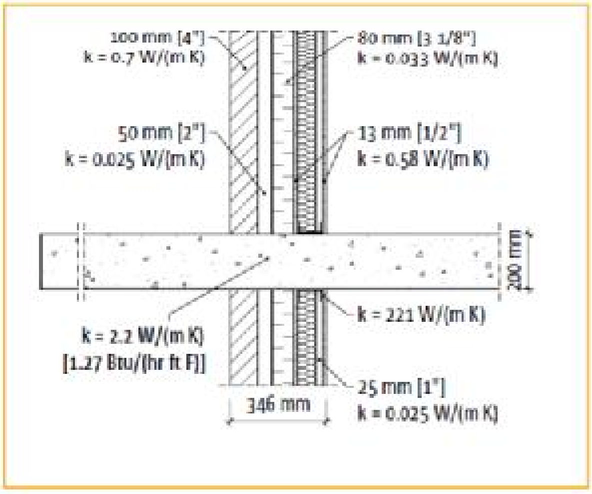
Iz rezultata se može videti da upotreba "Isokorb"-ove jedinice značajno poboljšava termičke performanse nosača koji prodire kroz fasadnu oblogu zgrade. Toplotni gubici su smanjeni gotovo 60% a temperaturni faktor je poboljšan preko 60%. Bez termičkih prekida temperaturni faktor je prihvatljiv za nestambene zgrade (kao što su skladišta, kancelarije i maloprodajni objekti), ali sa "Isokorb"- ovom jedinicom temperaturni faktor postaje više nego dovoljan, na primer, za škole, stambene zgrade i sportske hale (slika 16)

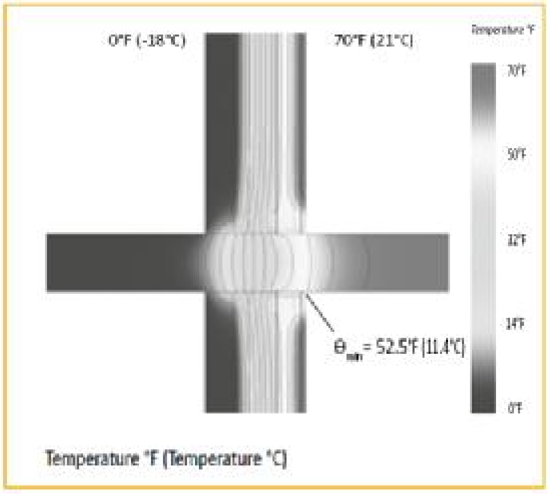
It can be seen from the results that using the "Isokorb" unit significantly improves the thermal performance of the beam penetrating the building envelope. The heat loss is reduced by almost 60% and the temperature factor is improved by over 60%. Without a thermal break the temperature factor is acceptable for non-residental buildings (such as storage, office and retail buildings) but with "Isokorb" unit the termperature factor becomes more than sufficient for schools, residental buildings, sports halls (Fig 16).

Slika 16. "Isokorb˝ova jedinica vezana za čelični ram [12]
Figure 16. "Isokorb" units attached to steel frame [12]

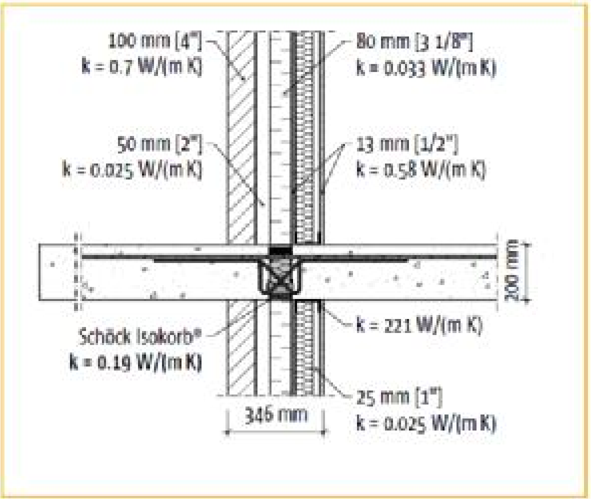
Slična konstrukcijska rešenja prekida termičkih mostova postoje i za AB konstrukcije – grede i ploče (sl. 17, 18, 19).

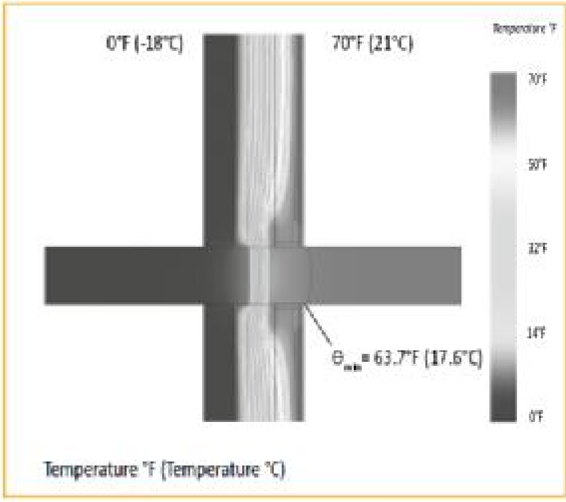
There are the similar solutions in design of thermal bridges interruption for RC structures – between beams and plates. (Sl. 17, 18, 19).



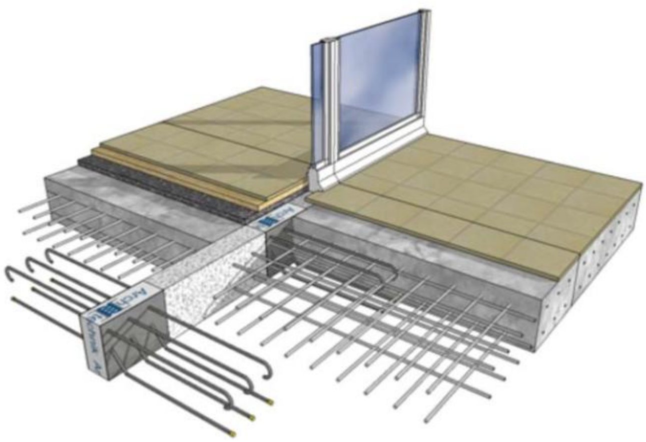


Slika 17. Uobičajena konstrukcija zgrade bez termoizolacije balkonske konstrukcije [14]
Figure 17. Conventional building structure without thermal insulation of the balcony structure [14]





Slika 18. Konstrukcija balkona "Isokorb CM" termičkom izolacijom [14]
Figure 18. Balcony structure with thermal insulation provided by "Isokorb CM" [14]



Slika 19. "Isokorb CM" termičkom izolacijom, kod većine primena potreban je jedan modul termičkog prekida po dužnom metru, što instalaciju čini vrlo isplativom [4]
Figure 19. Most applications would only require one Thermal break module per linier meter, making the installation very cost effective [4]

Čelik za armiranje (λ=50W/(mK)) je zamenjen nerđajućim čelikom (λ=15W/(mK)), a beton (λ=1,6W/(mK)) je zamenjem izolacionim materijalom (λ=0,031W/(mK)). Na ovaj način se redukuje toplotna kondukcija u području veze za oko 90% i smanjuje se gubitak toplote preko balkona oko 80%. Rizik od nastanka buđi je takođe smanjen, a izolovana fasadna obloga zgrade je efikasno izvedena bez prekida.

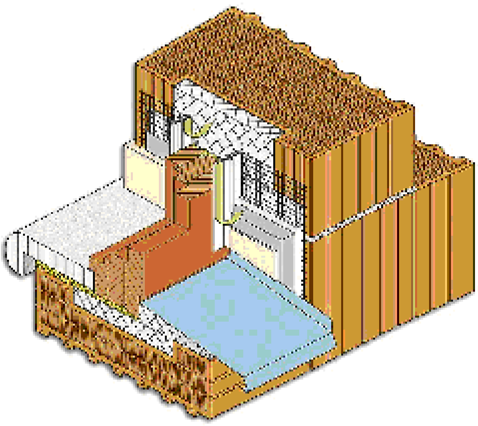
Prozori/Vrata

Elementi prozora predstavljaju specifičan činilac u definisanju mera zaštite objekata od toplotnih gubitaka (ili dobitaka leti). Slaba mesta su spoj elementa sa konstrukcijom zida (zaptivenost spoja i linijski gubici - sl. 20a), struktura konstrukcije i materijal od koga je izveden prozorski element (sl. 5,c-3) i staklena površina, kao najosetljivije i najizraženije područje toplotnih gubitaka (sl. 20b) [15]. Ovo isto važi i za vrata na fasadi koja su jednako koncipirana kao prozor.

Reinforcing steel (λ=50W/(mK)) is replaced with stainless steel (λ=15W/(mK)), and the concrete (λ=1,6W/(mK)) is replaced with insulating material (λ=0,031W/(mK)). This reduces heat conductivity in the connection area by approximately 90%, and minimizes the heat loss via the balcony about 80%. The risk of mold formation is also minimized, and the insulated building envelope is effectively made continuous.

Windows/Doors

Window elements are specific factors in defining measures for protecting buildings against thermal losses (or gains in summer). The weak points here are connections of the elements with the wall structure (joint sealing and line losses – Figure 20a), the structure and the material of the window element (Figure 5, c-3) and the glass surface as the most sensitive and the most prominent area of thermal losses (Figure 20b) [15]. The same applies for the doors on the facade given that they are designed the same as the windows.



a)

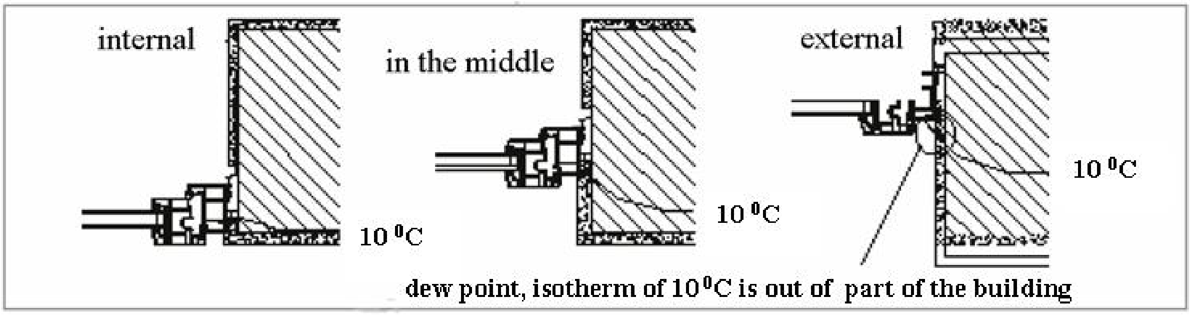
b)

Slika 20. Prozori: a) Fabrički rešen prekid termičkog mosta na spoju prozora i zida, b) Trostruko zastakljen, izolovani okviri, zaptiveni sa zidom [16]
Figure 20. Windows: a) Factory resolved thermal break at the junction the windows and the wall, b) Triple Glazed, insulated frames, sealed to walls [16]

Uticaj toplotnih mostova na tok temperature na spoljnim slojevima povećava rizik pojave rose/ kondenza na unutrašnjim površinama objekta. Očekivana temperatura površina i gubitak toplote zbog prenosa, mogu se dobiti uz pomoć jednačine za provođenje toplote i metode konačnih elemenata u jednom složenom računskom postupku. Ako toplotni faktor tRsi padne na vrednost ispod 0,7 i temperatura unutrašnjih površina na vrednost manju od 12,6°C, potrebno je sprovesti građevinske mere za neutralizaciju toplotnih mostova (sl. 21). Danas se uz pomoć odgovarajućih računskih programa mogu izračunati izoterme za različite građevinske situacije.

The influence of thermal bridges on temperature streams in the outer layers increases the risk of occurrence of dew/condensation on the building's interior surfaces. The expected surface temperature and heat loss due to transmission can be obtained based on a complex computational process using the equation for thermal conductivity and the finite element method.

If the heat factor tRsi falls below the value of 0.7 and the temperature of the interior surfaces below the value of 12.6 °C, it is necessary to carry out engineering measures for eliminating the thermal bridges (Figure 21). Today, aided by computer programs, it is possible to calculate the isotherms for a range of engineering situations.



Slika 21. Uticaj položaja ugradnje na tok izoterme od 10 °C [20]
Figure 21. The influence of position of the installation on the course of a 10 °C isotherm [20]

Poboljšanja su moguća kako na ugrađenim elementima, tako i na samom staklu (sl. 20.). Elementi doprozornika i krila kod drvenih konstrukcija se ne izvode iz jednog komada drveta, već se spajaju iz segmenata koji između sebe imaju termičke prekide, čime se postiže višestruki efekat˝:

– Drvo u jednom komadu provodi značajno više toplote, što omogućava pojavu orošavanja sa unutrašnje strane čime se direktno narušava trajnost elementa

– Povećana nosivost u slučaju većeg broja slojeva stakla.

– Jednostavnije servisiranje oštećenih delova elementa (zamena).

– Povećavanjem slojeva stakla omogućava se formiranje dodatnih komora čime se poboljšavaju izolacione karakteristike sklopa. Ovo pre ako se u stvorene komore upumpa inertni gas (ksenon ili argon).

Temelji, temeljni zidovi i konstrukcije poda na nasipu

Konstrukcije poda na tlu (nasipu) razlikuju se od podnih konstrukcija prema negrejanom prostoru po nosećoj betonskoj podlozi i hidroizolaciji. Toplotni gubici prema terenu iznose do 10% ukupnih toplotnih gubitaka. Kao i kod međuspratne konstrukcije prema negrejanom prostoru (npr. tavan), podnu konstrukciju prema negrejanom podrumu treba odgovarajuće termički izolovati. Kod konstrukcija u zemlji (temelji/temeljni zidovi, podrumski zidovi) primarna mesta gubitka energije je prostor prema spoljašnjem prostoru, a u terenu u predelu zone mržnjenja zemljišta (sl. 22). Kod konstrukcija u terenu se u datim situacijama može iskoristiti pozitivno termičko svojstvo terena koje podrazumeva sniženje razlike u temperaturama sa grejanim prostorom porastom njegove dubine (sl. 23). Međutim i pored toga rešenja se moraju pažljivo projektovati (sl. 24).

Improvements are possible both on the built-in elements and on the glass itself (Figure 23). In wooden structures, the elements of window frames and sashes are not derived from a single piece of wood; instead, they consist of segments with thermal breaks between them, achieving thereby several effects:

– In one piece, wood transfers significantly more heat, which allows the occurrence of condensation on the inward face, which directly undermines the durability of elements.

– Exposure of elements made of one piece to extreme temperatures (e.g. sun in summer) induces stresses which can cause defects in the element. The most common defect is a wing deformation which prevents a high quality sealing between the wing and the window frame.

– Increased bearing capacity in the event of multiple layers of glass.

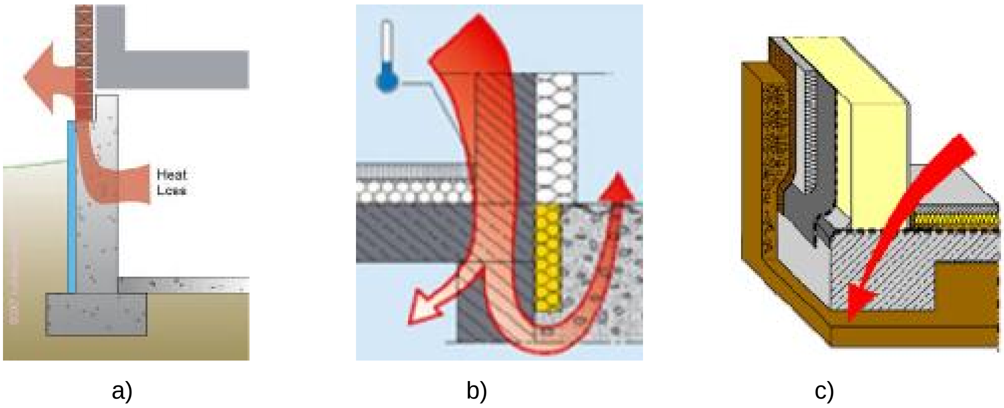
– Easier reparation of damaged parts of the element (replacement).

– Increasing the layers of glass allows the formation of additional chambers, improving thereby the insulation properties of the assembly. This is enhanced by pumping some inert gas (argon or xenon) into the chambers.

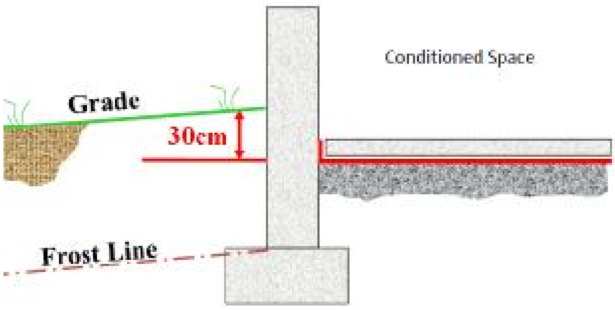
Foundations, foundation walls and floor structures on the fill

The unheated space and waterproofing of floor structures laid on the ground (fill) are different than those in structures based on concrete foundation. Heat losses towards the ground are accounting for as much as 10 percent of the overall thermal losses. Similar to the insulation of the ceiling joist towards the unheated attic, the floor structure also needs to be insulated towards the unheated basement using adequate thermal insulation. Floor structures above the open passages should also be thermally protected but this will be not discussed in this paper.

In underground structures (foundations/foundation walls, basement walls) the main zone of energy loss is the room next to the outer space, and towards the soil-freezing zone (Figure 22). When designing underground structures, the specific situation may require the use of the positive thermal properties of the ground, which implies the reduction of temperature differences with the heated space by increasing the depth of the underground structure (Figure 23). Despite this solution, however, they must be carefully designed (Figure 24).



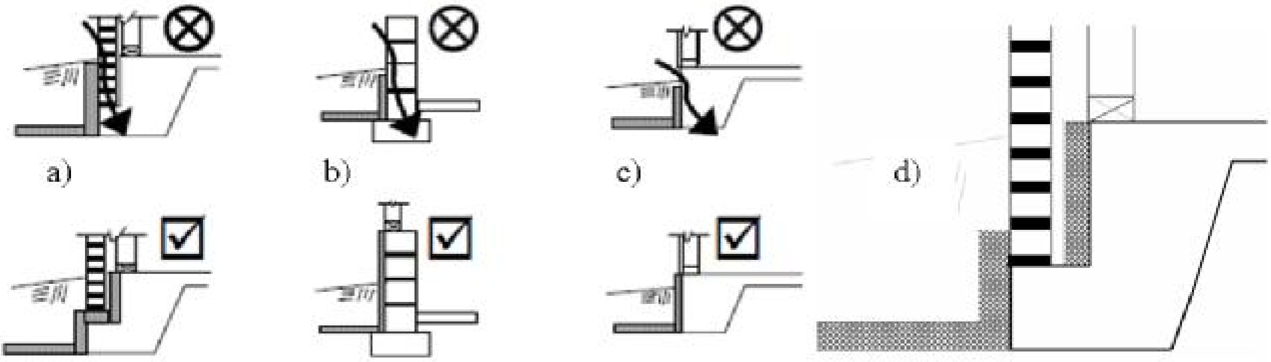
Slika 22. Najčešće pojave termičkih mostova kod izolovanih podzemnih delova zgrade - a) Termička izolacija podzemnih delova zgrade bez kontinuiteta sa nadzemnom građevinskim elementom; b) Gubitak toplote kroz spoljni zid kod tavanice iznad podruma; c) Ploča i zid izolovani, temeljna konstrukcija bez izolacije
Figure 22. The most common occurrence of thermal bridges in isolated underground parts of the building: a) thermal insulation of underground parts of the building without continuity with the above ground structural element, b) heat loss through the exterior wall in the ceiling above the basement, c) a slab and wall insulated from the foundation structure without insulation



Slika 23. Ploče na dubini manjoj od 30cm ispod terena u nagibu treba izolovati [19]
Figure 23. Floor surfaces less than 30cm below grade shall be insulated [19]

Potrebno je utvrditi određena pravila i usvojiti preporuke kada je neophodno izolovati ploče i zidove u tlu. Te preporuke bi trebalo usvojiti kao pravilo građenja za potrebe energetske efikasnosti zgrada.

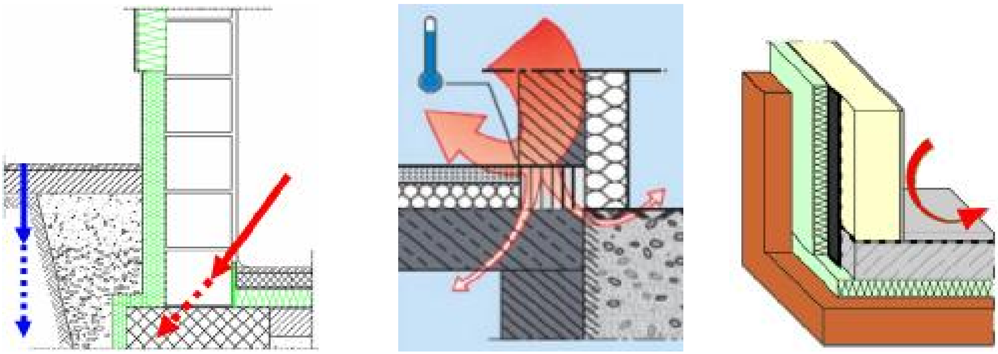
It is necessary to establish specific rules and adopt recommendations regarding the necessity of insulating underground slabs and walls. These recommendations should be adopted as construction rules for achieving energy-efficient buildings.



Slika 24. Izolovanje spoljnih temelja - a) Termički most kroz oblogu od opeke i popravka; b) Termički most kroz podrumski zid i popravka; c) Termički most kroz izloženi temeljni zid i popravka; d) Izolacija nije potrebna na horizontalnom delu temelja koji nosi oblogu od opeke [19]
Figure 24. Exterior foundation insulation - a) Cold-bridge through brick veneer and correction; b) Cold-bridge through basement wall and correction; c) Cold-bridge through exposed foundation wall and correction; d) Insulation not required on the horizontal portion of the foundation that supports a masonry veneer [19]

Najčešći i preporučeni oblici rešenja termičke izolacije podzemnih delova zgrade prikzani su na slici 25.

The most common and recommended solutions for the thermal insulation of underground parts of the building are presented in Fig. 25.



a) b) c)

Slika 25. Najčešći i preporučeni oblici rešenja termičke izolacije podzemnih delova zgrade - a) Izolovanje temelja i podrumskog zida spolja zahteva kontinuitet sa izolacijom zidova viših etaža - opadanjem intenziteta mržnjenja samoniklog tla opadaju termički gubici kroz neizolovanu konstrukciju; b) Smanjenje gubitka toplote alternativnim građevinskim detaljem - termo stopalo; c)Najbolji koncept za smanjivanje toplotnog mosta je potpuna termoizolacija ispod temeljne ploče

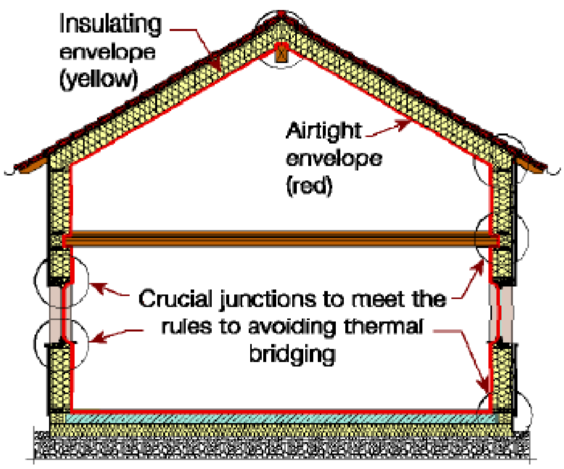
Figure 25. The most common and recommended solutions for the thermal insulation of underground parts of the building: a) The outward insulation of the foundation and the basement wall requires preserving the continuity with the insulation of higher floors – by the decline of freezing intensity of the autochthonous soil, the thermal losses through the non-insulated structure will also decrease; b) Reduces heat loss with an alternative construction detail; c) The best concept for minimizing the thermal bridge is a full heat insulation under floor slab

6 ZAKLJUČAK

Kada se govori o poboljšanju toplotnih izolacionih karakteristika zgrade i mogućnosti smanjenja ukupnih gubitaka toplote zgrade za prosečno za 50-80% , tada se pažnja često fokusira na oblaganje svih površina objekta prema spoljašnjoj sredini, uključujući i pokretne pregrade, elemente prozora i vrata. Oni ˝zatureni˝, neizolovani delovi građevinskih elemenata se zanemaruju, čak u nekim situacijama i namerno, smatrajući da to nema većeg značaja za ukupan energetski bilans zgrade (sl. 26) Ovakav previd se ispoljava sa izraženim posledicama koje su višestruke.

6 CONCLUSION

When it comes to improving the thermal insulation properties of the building and possibility of reducing its overall heat loss for approximately 50-80 percent, then the attention is often focused on cladding the surfaces of the object towards the external environment, including movable partition walls, as well as window and door elements. The "out-of-sight", non-insulated parts of structural elements are neglected, in some cases even deliberately, believing that they have nothing to do with the overall energy balance of the building (Fig.26). This omission results in multitude of severe consequences.



Slika 26. Pravila koja pomažu u prevazilaženju termičkih mostova [9]
Figure 26. Rules to assist in the avoidance of thermal bridging [9]

Sa jedne strane, kod dobro izolovanih objekata pojava neizolovanih delova generiše gubitke kroz njih mnogo više nego da objekat nije izolovan uopšte. U dobro izolovanim zgradama termički mostovi mogu činiti i do 50% ukupnih toplotnih gubitaka. To je iz razloga što se fluid, težeći da pređe sa višeg nivoa temperature na niži, kreće linijom manjeg otpora prolaza toplote te se u znatno većoj količini koncentriše oko hladnijih delova građevinskog elementa. Tako intenzivna koncentracija fluida pojačava i prateće pojave, koje za posledicu imaju oštećenja površinskog sloja građevinskog elementa ili čak čitave strukture elementa u njegovoj ukupnoj debljini, od unutrašnjeg prostora ka spolja.

Sa druge strane, povećana koncentracija vazduha dovodi do pojave ili čak do pojačanog orošavanja sa unutrašnje strane građevinskog elementa, nastalog kondenzovanjem vodene pare kojom je vazduh zasićen u određenom procentu u zavisnosti od namene prostorije (soba, kuhinja, praonica i sl.) Vlažni delovi su dobra podloga za nastajanje i razmnožavanje mikroorganizama, koji utiču na kvalitet mikroklimata i vazduha prostorije. Takvo stanje se odražava na zdravlje korisnika te prostorije.

Zadatak projektanta je da se konstantno edukuje o novim saznanjima vezanim za fiziku zgrade i da je upoznat sa savremenim dostignućima kojima se prevazilaze potencijalna loša mesta na objektu. Savremene tendencije su da se objekat termički izoluje bez prekida (temeljna ploča), čak i na osetljivim delovima gde se ranije, iz razloga sigurnosti konstrukcije, morao vršiti prekid termoizolacionog sloja (konektori koji spajaju izbačeni deo konstrukcije u spoljašnji prostor sa osnovnom konstrukcijom). Tamo gde to nije zaista moguće, uz poznavanje fizičkih zakonitosti, efekti gubitaka toplotne energije se mogu umanjiti u visokom procentu (i do 85%, npr. u slučaju trakastih temelja).

On one hand, in well-insulated buildings the appearance of non-insulated parts induces much higher losses than if the building is not isolated at all. This is because the fluid, striving to shift from a higher to a lower temperature, is moving along the line of lower resistance of heat transfer, being much more concentrated around the colder parts of the building element. Thus, given the intense concentration of the fluid, its side effects are also increased, resulting in damage to the surface layer of structural elements or even the entire depth of the structural element, moving from the inside towards the outside.

On the other hand, the increased concentration of air leads to the appearance of dew on the inward face of building elements, induced by the condensation of water vapour by which the air is saturated. The level of this saturation depends on the purpose of the room (bedroom, kitchen, laundry, etc.). The humid areas are good foundations for the emergence and growth of microorganisms that affect the microclimate and air quality of the room. This situation affects the health of users of the room.

The task of the designer is to be constantly educated regarding the new knowledge about the engineering physics and the modern solutions aimed at overcoming the potentially poor locations on buildings. The newest tendencies are to thermally insulate the building without any disruptions (foundation slab), even in sensitive areas where the insulating layer (connectors connecting the extended structural elements with the main structure) previously had to be disrupted because of structural safety. Where it is impossible with the knowledge of physical laws, the effects of energy losses can be reduced by high percentage (up to 85%, for example, in the case of strip foundations).

ZAHVALNOST

Prikazani rad je deo istraživanja u okviru istraživačkog projekta br. III42012: ˝Poboljšanje energetske efikasnosti zgrada u Srbiji i unapređenje nacionalnih regulativnih kapaciteta za njihovu sertifikaciju˝, koje je pomoglo Ministarstvo za nauku i tehnološki razvoj Republike Srbije, Program integrisanih i interdisciplinarnih istraživanja. Zahvaljujemo se na ovoj podršci.

ACKNOWLEDGEMENTS

The work reported in this paper is a part of the investigation within the research project III42012 "Improving energy efficiency in buildings in Serbia and the improvement of national regulatory capacity for their certification", supported by the Ministry of Education and Science of the Republic of Serbia, Program of integrated and interdisciplinary research. This support is gratefully acknowledged.

7 LITERATURA
REFERENCES

[1] Guide 3: Insulating concrete formwoek (ICF), Edition Building Low Carbon Homes, Richards Partington Architects, *www.rparchitects.co.uk*.

[2] Wall Insulation, StirodurC, BASF SE, Germany.

[3] How to deal with sick building syndrome (SBS), Guidance for employers, building owners and building managers, HSE Books, PO Box 1999, Sudbury, Suffolk, UK. ISBN 978 0 7176 08614

[4] Thermal Break Connections - Puts the 'Break' in Thermal Break, Insula, Hathern Industrial Estate, Rempstone Road, Normanton on Soar, Leicestershire, UK.

[5] Little, J., Arregi, B.: Thermal bridging: Understanding its critical role in energy efficiency, Construct Ireland, issue 6, vol. 5, April/May 2011.

[6] Pravilnik o energetskoj efikasnosti zgrada, Sl glasnik RS, br.6/2011.

[7] Patankar, S., V.; Numerical Heat Transfer and Fluid Flow; Hemisphere Publishing Corporation.

[8] Limiting Thermal Bridging and Air Infiltration, Acceptable Construction Details, Government of Ireland 2008.

[9] Bell, M: Urban Buzz Project—Developing Low Carbon Housing: Design collection: Thermal bridging, Leeds Metropolitan University and University College London.

[10] Vodič kroz energetski efikasnu gradnju, Ministarsatvo zaštite okoliša, prostornog uređenja i graditeljstva, januar 2005., Zagreb.
[11] Heiduk, E.: 04.02_PH Summer School, Thermal bridges, International passive house summer school for students.
[12] Way, A.G.J., Kendrick, C.: Avoidance of Thermal Bridging in Steel Construction, SCI Publication P380, The Steel Construction Institute, Ascot, Berkshire SL% 7QN, 2008.
[13] Moric, K.: Pravilno i pogrešno - kod toplotne zaštite, zaštite od vlage, zaštite građevinskih objekata, Bauverlag GMBH Wiesbaden, Berlin, Neimar, Beograd, 1975.
[14] Schock Isokorb, Technical manual, U.S. Edition, December, 2012.
[15] Todorović, M.: Racionalizacija potrošnje energije primenom mera za poboljšanje energetske efikasnosti na primeru predškolske ustanove, kgh 2, 2012, BIBLID 0350–1426 (206) 41:2 p. 35–70.
[16] Policy and Practice –Setting the Standard, Kirsty Maquire Architect, www.kirstymaguire.com.
[17] Todorović, M., Ećim, O., Martinović, I.: *Izbor prilaza unapređenju energetske efikasnosti i održivosti zidanih zgrada*, Građevinski materijali i konstrukcije, vol. 53, br. 4, str. 5-27, 2010.

7.1 Literatura sa internet pretraživača

[18] Csaba Szikra, Balconies and Thermal Bridges, *http://www.egt.bme.hu/szikra/w_munkatarsak/szikra/szikra_files/B12.pdf*
[19] How to avoid thermal bridges, *http://www.sec.bg/userfiles/file/Guide%20book%20EN/Guide%20book%20EN%206.pdf*
[20] Priručnik za postavljanje PVC prozora i vrata, *http://www.dualstolarija.com/sert/Veka%20Priru%C4%8Dnik%20za%20postavljanje%20prozora.pdf*
[21] How to Properly Insulate a Slab, *http://www.engr.psu.edu/phrc/training/Webinar_Slab%20Insulation.pdf*
[22] Izbegavanje toplotnih mostova ili kako napraviti bolje zgrade, *http://beodom.com/sr/education/entries/fighting-thermal-bridges-or-how-to-make-better-buildings*
[23] Cold Bridge - Thermal Bridge, *http://www.viking-house.us/cold-bridge.html*

REZIME

REŠENJA PREKIDA KARAKTERISTIČNIH TERMIČKIH MOSTOVA KOD OBJEKATA VISOKOGRADNJE

Željko JAKŠIĆ
Norbert HARMATI

Zadovoljavanje zahteva građevinske fizike predstavlja jedan od primarnih zadataka u obezbeđenju zdravog i bezbednog korišćenja prostora u kojem borave ljudi. Fizika zgrade sadrži više činilaca, a difuzija vodene pare je vrlo značajna iz više razloga. Jedan od njih je postojanje hladnih mostova koji u sprezi sa difuzijom vodene pare proizvode oštećenja na zgradi, različitog obima. Najčešći razlozi su nepridavanje dovoljno važnosti fenomenu difuzije i uzrocima pojave vlaženja građevinske konstrukcije, ali i detalji konstrukcije koji su često kompleksni za izvođenje. Pored slabljenja termičkih karakteristika izolacionog materijala i ukupnih termičkih karakteristika fasadnog elementa, pojava vlage sa unutrašnje strane elementa često dovodi do pojave bioloških zagađivača (buđ). Cilj rada je da ponudi rešenja, koja su često i konstruktivno zahtevna, za sprečavanje pojave vlaženja elemenata građevinske konstrukcije karakterističnih kod postojanju hladnih mostova. U radu su korišćena i rešenja koja su rezultat istraživanja drugih autora, koja su primeri unapređenog građenja primenjenog u praksi.

Ključne reči: Građevinska fizika, hladni mostovi, difuzija vodene pare, pojava kondenza, bolesne zgrade.

SUMMARY

RESOLVING THE ISSUE OF DISRUPTING CHARACTERISTIC THERMAL BRIDGES IN BUILDING STRUCTURES

Zeljko JAKSIC
Norbert HARMATI

Meeting the requirements of engineering physics is one of the primary tasks in ensuring a healthy and safe use of space in which people reside. The physics of a building consists of many factors, and the diffusion of water vapour is very important for several reasons. One of them is the existence of cold bridges. The interaction of cold bridges and the diffusion of water vapour lead to defects of various scales in the building. The most common reasons for this is neg*lectin*g the phenomenon of moisture diffusion and its causes throughout the building structure, and the existence of structural details that are often complex to be constructed. In addition to weakening the thermal properties of the insulation material and the overall thermal properties of the facade elements, the appearance of moisture on the inward face of the element leads to the appearance of biological pollutants (mould). The aim of this paper is to offer a comprehensive solution (which is often also structurally demanding) against the occurrence of moisture in structural elements in which cold bridges commonly exist. The paper is based on research resultsof othre autors, as well as on the examples of improved construction practice.

Key words: engineering physics, cold bridges, the water vapour diffusion, presence of condensation, sick building.

UPUTSTVO AUTORIMA*

Prihvatanje radova i vrste priloga

U časopisu Materijli i konstrukcije štampaće se neobjavljeni radovi ili članci i konferencijska saopštenja sa određenim dopunama ili bez dopuna. prema odluci Redakcionog odbora. a samo izuzetno uz dozvolu prethodnog izdavača prihvatiće se i objavljeni rad. Vrste priloga autora i saradnika koji će se štampati su: originalni naučni radovi. prethodna saopštenja. pregledni radovi. stručni radovi. konferencijska saopštenja (radovi sa naučno-stručnih skupova). kao i ostali prilozi kao što su: prikazi objekata i iskustava - primeri. diskusije povodom objavljenih radova i pisma uredništvu. prikazi knjiga i zbornika radova. kao i obaveštenja o naučno-stručnim skupovima.

Originalni naučni rad je primarni izvor naučnih informacija i novih ideja i saznanja kao rezultat izvornih istraživanja uz primenu adekvatnih naučnih metoda. Dobijeni rezultati se izlažu kratko. jasno i objektivno. ali tako da poznavalac problema može proceniti rezultate eksperimentalnih ili teorijsko numeričkih analiza i tok razmišljanja. tako da se istraživanje može ponoviti i pri tome dobiti iste ili rezultate u okvirima dopuštenih odstupanja. kako se to u radu navodi.

Prethodno saopštenje sadrži prva kratka obaveštenja o rezultatima istraživanja ali bez podrobnih objašnjenja. tj. kraće je od originalnog naučnog rada. U ovu kategoriju spadaju i diskusije o objavljenim radovima ako one sadrže naučne doprinose.

Pregledni rad je naučni rad koji prikazuje stanje nauke u određenoj oblasti kao plod analize. kritike i komentara i zaključaka publikovanih radova o kojima se daju svi neophodni podaci pregledno i kritički uključujući i sopstvene radove. Navode se sve bibliografske jedinice korišćene u obradi tematike. kao i radovi koji mogu doprineti rezultatima daljih istraživanja. Ukoliko su bibliografski podaci metodski sistematizovani. ali ne i analizirani i raspravljeni. takvi pregledni radovi se klasifikuju kao stručni pregledni radovi.

Stručni rad predstavlja koristan prilog u kome se iznose poznate spoznaje koje doprinose širenju znanja i prilagođavanja rezultata izvornih istraživanja potrebama teorije i prakse. On sadrži i rezultate razvojnih istraživanja.

Konferencijsko saopštenje ili rad sopšten na naučnostručnom skupu koji mogu biti objavljeni u izvornom obliku ili ih autor. u dogovoru sa redakcijom. bitno preradi i proširi. To mogu biti naučni radovi. naročito ako su sopštenja po pozivu Organizatora skupa ili sadrže originalne rezultate prvi put objavljene. pa ih je korisno uz određene dopune učiniti dostupnim široj stručnoj javnosti. Štampaće se i stručni radovi za koje Redakcioni odbor oceni da su od šireg interesa.

Ostali prilozi su prikazi objekata. tj. njihove konstrukcije i iskustava-primeri u građenju i primeni različitih materijala. diskusije povodom objavljenih radova i pisma uredništvu. prikazi knjiga i zbornika radova. kao i obaveštenja o naučno-stručnim skupovima.

Autori uz rukopis predlažu kategorizaciju članka. Svi radovi pre objavljivanja se recenziraju. a o prihvatanju za publikovanje o njihovoj kategoriji konačnu odluku donosi Redakcioni odbor.

Da bi se ubrzao postupak prihvatanja radova za publikovanje. potrebno je da autori uvažavaju Uputstva za pripremu radova koja su navedena u daljem tekstu.

Uputstva za pripremu rukopisa

Rukopis otkucati jednostrano na listovima A-4 sa marginama od 31 mm (gore i dole) a 20 mm (levo i desno). u Wordu fontom Arial sa 12 pt. Potrebno je uz jednu kopiju svih delova rada i priloga. dostaviti i elektronsku verziju na navedene E-mail adrese. ili na CD-u. Autor je obavezan da čuva jednu kopiju rukopisa kod sebe zbog eventualnog oštećenja ili gubitka rukopisa.

Od broja 1/2010. prema odluci Upravnog odbora Društva i Redakcionog odbora. radovi sa pozitivnim recenzijama i prihvaćeni za štampu. publikovaće se na srpskom i engleskom jeziku.

* Uputstvo autorima je modifikovano i treba ga. u pripremi radova. slediti.

Svaka stranica treba da bude numerisana. a optimalni obim članka na jednom jeziku. je oko 16 stranica (30000 slovnih mesta) uključujući slike. fotografije. tabele i popis literature. Za radove većeg obima potrebna je saglasnost Redakcionog odbora.

Naslov rada treba sa što manje reči (poželjno osam. a najviše do jedanaeset) da opiše sadržaj članka. U naslovu ne koristiti skraćenice ni formule. U radu se iza naslova daju ime i prezime autora. a titule i zvanja. kao iime institucije u podnožnoj napomeni. Autor za kontakt daje telefone. faks i adresu elektronske pošte. a za ostale autore poštansku adresu.

Uz sažetak (rezime) od oko 150 do 200 reči. na srpskom i engleskom jeziku daju se ključne reči (do deset). To je jezgrovit prikaz celog članka i čitaocima omogućuje uvid u njegove bitne elemente.

Rukopis se deli na poglavlja i potpoglovlja uz numeraciju. po hijerarhiji. arapskim brojevima. Svaki rad ima uvod. sadržinu rada sa rezultatima. analizom i zaključcima. Na kraju rada se daje popis literature.

Kod svih dimenzionalnih veličina obavezna je primena međunarodnih SI mernih jedinica.

Formule i jednačine treba pisati pažljivo vodeći računa o indeksima i eksponentima. Autori uz izraze u tekstu definšu simbole redom kako se pojavljuju. ali se može dati i posebna lista simbola u prilogu.

Prilozi (tabele. grafikoni. sheme i fotografije) rade se u crno-beloj tehnici. u formatu koji obezbeđuje da pri smanjenju na razmere za štampu. po širini jedan do dva stupca (8cm ili 16.5cm). a po visini najviše 24.5cm. ostanu jasni i čitljivi. tj. da veličine slova i brojeva budu najmanje 1.5mm. Originalni crteži treba da budu kvalitetni i u potpunosti pripremljeni za presnimavanje. Mogu biti i dobre. oštre i kontrastne fotokopije. Koristiti fotogrfije. u crno-beloj tehnici. na kvalitetnoj hartiji sa oštrim konturama. koje omogućuju jasnu reprodukciju. Skraćenice u prilozima koristiti samo izuzetno uz obaveznu legendu. Prilozi se posebno označavaju arapskim brojevima. prema redosledu navođenja u tekstu. Objašnjenje tabela daje se u tekstu.

Potrebno je dati spisak svih skraćenica korišćenih u tekstu.

U popisu literature na kraju rada daju se samo oni radovi koji se pominju u tekstu. Citirane radove treba prikazati po azbučnom redu prezimena prvog autora. Literaturu u tekstu označiti arapskim brojevima u uglastim zagradama. kako se navodi i u Popisu citirane literature. napr [1]. Svaki citat u tekstu mora se naći u Popisu citirane literature i obrnuto svaki podatak iz Popisa se mora navesti u tekstu.

U Popisu literature se navode prezime i inicijali imena autora. zatim potpuni naslov citiranog članka. iza toga sledi ime časopisa. godina izdavanja i početna i završna stranica (od - do). Za knjige iza naslova upisuje se ime urednika (ako ih ima). broj izdanja. prva i poslednja stranicapoglavlja ili dela knjige. ime izdavača i mesto objavljivanja. ako je navedeno više gradova navodi se samo prvi po redu. Kada autor citirane podatke ne uzima iz izvornog rada. već ih je pronašao u drugom delu. uz citat se dodaje «citirano prema...». Neobjavljeni članci mogu se pominjati u tekstu kao «usmeno saopštenje»

Autori su odgovorni za izneseni sadržaj i moraju sami obezbediti eventualno potrebne saglasnosti za objavljivanje nekih podataka i priloga koji se koriste u radu.

Ukoliko rad bude prihvaćen za štampu. autori su dužni da. po uputstvu Redakcije. unesu sve ispravke i dopune u tekstu i prilozima.

Za detaljnija tehnička uputstva za pripremu rukopisa autori se mogu obratiti Redakcionom odboru časopisa.

Rukopisi i prilozi objavljenih radova se ne vraćaju. Sva eventualna objašnjenja i uputstva mogu se dobiti od Redakcionog odbora.

Radovi se mogu slati i na e-mail: folic@uns.ac.rs ili miram@uns.ac.rs i dimk@ptt.rs

Veb sajt Društva i časopisa: www.dimk.rs

Izdavanje časopisa "Građevinski materijali i konstrukcije" finansijski su pomogli:

INŽENJERSKA KOMORA SRBIJE

MINISTARSTVO ZA NAUKU I TEHNOLOŠKI RAZVOJ SRBIJE

UNIVERZITET U BEOGRADU GRAĐEVINSKI FAKULTET

DEPARTMAN ZA GRAĐEVINARSTVO FAKULTET TENHIČKIH NAUKA NOVI SAD

INSTITUT IMS AD, BEOGRAD

UNIVERZITET CRNE GORE GRAĐEVINSKI FAKULTET - PODGORICA