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# SEISMIC SAFETY IN ANCIENT STONE DOMES REINFORCED BY AN ORIGINAL METHOD

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## SUMMARY

The present article discusses an original structure for strengthening ancient stone-built domes, whose seismic resistance is to be enhanced. It consists of a thin reinforced concrete reinforced concrete shell, provided with a support ring and placed above the dome. Connecting members project downwards from the shell and are introduced in the midst of the dome's stone. Resulting reinforcement is achieved by creation of an interconnected stone reinforced concrete structure. Stress concentrations in the connection areas are a specific problem of the interconnected structures. While said structures are subject to static and thermal loads and seismically originated forces, problems relating to the stress - strain state of the stone dome and its dynamic characteristics are considered. The issue of stress concentration in surrounding connecting-member locations is discussed. The problem under study is considered through examples of real conservation cases: the ancient stone dome in the town of Ahaltsihe, in the seismic zone of Georgia; and a dome of the Hagia Sophia structure. The stress-strain states are analyzed using the finite-element method.

## INTRODUCTION

The ancient stone domes are usually architectural works of art. They are a part of buildings, which are usually architectural and sometimes historical monuments. These buildings may have utilitarian value as well. At present, there is a great number of stone domes in the Mediterranean countries, in the Caucasus and in the East (India, Iran, etc.) (DOMES [1], Heyman [2], Krautheimer [3], Kuznetzov [4], etc). In the recorded past, these territories have come under the influence of severe and destructive Earthquakes, accompanied by significant destruction and human victims. Keeping of architectural-historical monuments such as ancient stone domes in their proper state is a duty of civilized society. Today there are certain samples of research, conservation and restoration of stone domes in the seismic regions of the world (DOMES [1], Krautheimer [3], Kirikov [5], Lefandeux [6], Aoki [7], Emerson [8], Gabrilovich [9], HAGIA-SOPHIA [10], Karaesmen [11]), and in particular in Georgia (Danieli (Danielashvili) [12], Danielashvili [13], Engineering [14], etc). Their study and analysis are important.

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## PROPOSED METHOD OF STRENGTHENING OF STONE DOMES

### Construction of strengthening structure

Taking into account the historical and architectural value of the stone domes, there is a need for conservation in all cases where there is the presence of cracks or any other damages in them, as well as a need for absorption of seismic load in the possible event of severe earthquakes without significant damage. An original strengthening construction is proposed (Fig. 1) for such conservation of the stone domes. In order to preserve the lower surface of the domes in their present ancient and authentic interior appearance–for example, the stone dome of Ahaltsihe (Fig. 2), with its stone masonry containing cracks in the lower surface (Fig. 3); and the Hagia Sophia Dome, with its rich, colorfully ribbed decoration in the lower surface–as well as the significant advantages in carrying out the construction work itself, it is proposed to carry out the strengthening of the dome from the lower surface. Advantages of such a method, besides the above factors, are as follows: the buried character of the strengthening construction under roofing cover, and use for the work of the strengthened stone constructions. The discussed construction of dome strengthening consists of a new thin-walled reinforced concrete shell over the existing stone dome, with supporting ring (Fig. 1) size and reinforcement to the pattern is given with reference to the dome in Ahaltsihe). The supporting ring is placed in the thickness of the stone dome. The necessary link for joint work of the stone dome with the reinforced concrete shell is done by means of reinforced concrete connection elements. As pins they stand out from the reinforced concrete shell, introduced into the stone dome, have a form of a truncated pyramid or cone (with the enlarged part in the stone dome) and are distributed through the entire dome surface. The connection elements may be made from various kinds of reinforced concrete. An example of analogue strengthening of very lightly sloping reinforced concrete shells, with a floor diameter of 10.0m is given (Danielashvili [13]). A characteristic feature of the deflected mode in such a structure is concentration of stress at the placement points of connection elements. The following should be taken into account: non-uniform modularity of materials in the connected constructions, and significant differences in their strengthening index for pressure stretching. As a result of strengthening by the proposed method, stress in the stone dome may be significantly decreased, thus significantly raising its earthquake resistance; the thrust is absorbed by the reinforced

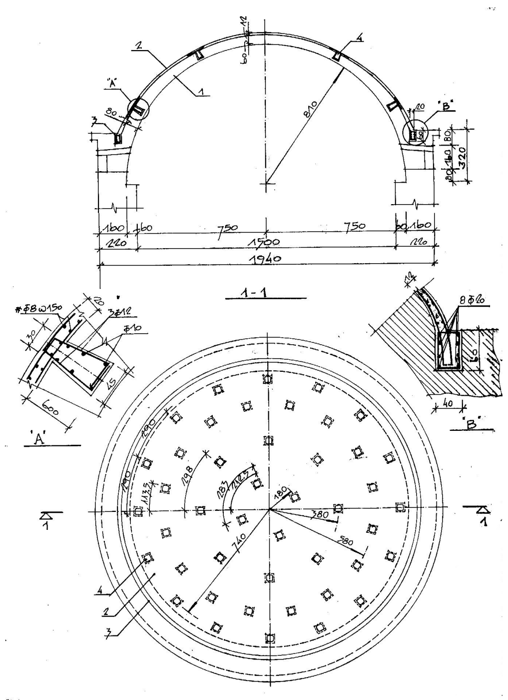


Fig. 1. Dome strengthening construction. Vertical meridional section on 1-1. Plan. 1 - stone dome; 2 – reinforced concrete shell; 3 – supporting ring; 4 – connection elements.

Fig. 2. View of the dome (former mosque) in Ahaltzikhe without roof.

concrete ring and the supporting structures are unloaded from the effect of horizontal forces. Execution method of strengthening is described in (Danieli [12]).

a b

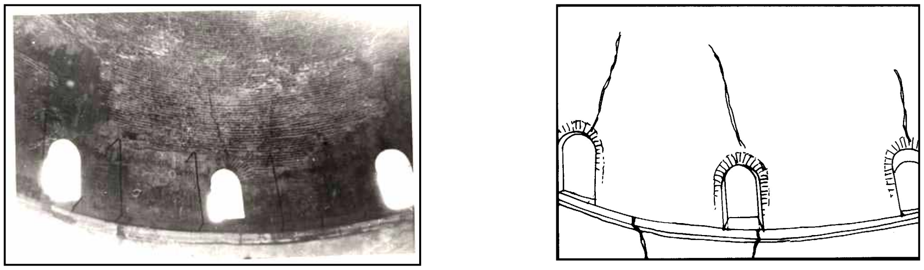

Fig. 3. Cracks at the inner surface of the dome in Ahaltsihe: a) picture; b) scheme

## Description of analytical model

An extensive structural analysis was done, to specify characteristics of the deflected mode of the stone dome's strengthened construction under dead (self-weight) and seismic load, , including a series of static and dynamic (seismic) analyses. The complete series included: analysis of the stone dome (only) for action of vertical (self-weight), thermal and seismic loads due to the stone dome's mass; analysis of the reinforced concrete shell – construction for strengthening of vertical (self-weight) loads of the reinforced concrete shell and joined stone dome, and seismic loads due the total mass of the stone dome and reinforced concrete shell; analysis of the strengthened, interconnected structure – stone dome and reinforced concrete shell in the action of vertical load (self-weights of the stone dome and reinforced concrete shell) and seismic load due to the general mass of the stone dome and reinforced concrete shell, joined to the interconnected structure. The series of structural analyses are performed by the finite-element method (FEM) using the software "MAG-STR" (NIIASS, Kiev, Republic of Ukraine, International Certificate of Inspection – IMS CONSELL, June, 1992). The following spatial model is employed for analysis of the connected construction (Fig. 4);3-dimensional (3-D) structural models that consist of the following groups of linear finite elements: a) triangle plane-shell finite elements to represent surfaces of dome and shell; b) frame-type finite elements to represent supporting ring beams; c) spring-type finite elements to represent influences of surrounding structures (walls) in vertical and horizontal directions; d) spring-type finite elements to represent connections between stone dome and concrete shell. The proper elements are brought to zero, for analysis of the single construction (of the stone dome itself, and the reinforced concrete shell itself) in the consideration of a model of analysis It should be noted that thickness of the reinforced concrete shell, according to analysis, is estimated not by the demands of buckling and safety, but mostly by possible redistribution of efforts between the stone dome and reinforced concrete shell. It depend on the necessary degree of unload for the stone dome (absorption of efforts from inside the stone dome). In the widespread cases of spans and thickness of stone domes, it is recommended to accept the reinforced concrete shell thickness as 8-16 cm.

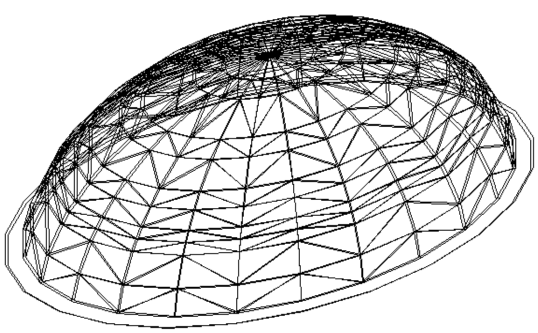

Fig. 4. Model of connection joints construction

## DOMES

### Stone dome in Ahaltsihe

**Seismic characteristics of the building site** - The building of the former mosque in the town of Ahaltsihe was constructed in 1758 (Fig. 2). At present it is an architectural-historical monument of Georgia and is a part of the architectural-historical complex of Ahaltsihe. The complex is located on the height of the rock base, in an active seismic zone – the Dzhavakhetskoye Plateau in South Georgia. In this region, the maximum possible earthquake according to the Richter scale is estimated at a magnitude $M_{max}$ =7.0 (ENGINEERING [14]). According to research (CONSTRUCTION [15]), the estimated seismicity in Ahaltsihe was $7_1$. After the Spitak Earthquake (Armenia, December 7, 1988, Richter scale magnitude Ms = 6.9; distance from Spitak to Ahaltsihe about 125 km), with its tragic results, and the Rachinsky Earthquake (Georgia, April 29, 1991, Richter scale magnitude Ms = 6.9; distance from epicenter to Ahaltsihe about 100 km), the estimated intensity under the temporary scheme of seismic zoning of the Georgia territory is $9_2$ (ENGINEERING [13]). It should be noted that during the period of its existence, Ahaltsihe suffered from several significant earthquakes, including the Akhalkalaki Earthquake in 1899 (distance between the towns of Ahaltsihe and Akhalkalaki is about 45 km), whose intensity according to modern estimation may be considered as 8(ENGINEER. [13]). with a Richter scale magnitude Ms = 5.4.

**Strengthening** - The stone dome in Ahaltsihe (Fig. 2) is one of the most significant stone domes of its size among those existing today in Georgia. The inner diameter of the supporting contour of the dome is about 16.0 m, inner height (rise) about 8.0m, thickness of the dome's wall is 0.6-0.8m. The dome was constructed from the thin clay brick with dimensions of 24x24x4 cm, on a lime-clay solution. On the upper surface of the dome, the brick edges stand out horizontally in steps (Fig. 2). The dome has typical cracks distributed from the supporting zone, with meridional direction and opening crack width on the lower surface of 1.5-2.0cm (Fig. 3). The cracks are caused by the existence of stretching ring tensions (about 4 $t/m^2$) as a result of the domes own weight and a weakening of the dome's masonry by window openings in this region (Fig. 3). Taking into account the historical and architectural value of the building, the existence of developed cracks, and the need for absorption of seismic load during possible severe Earthquakes without significant damages, there is a clear need for dome conservation. To preserve the inner ancient look of the dome in its present real state of brick masonry with cracks, it is proposed to carry out strengthening of the dome according to the proposed method – with the arrangement of a new reinforced concrete dome on its upper surface, connected with the existing stone dome.

**A structural analysis** - To develop the project of conservation (strengthening) and to specify the deflected-mode characteristics of the stone dome construction, reinforced concrete shell and strengthened (connected) construction under the influence of dead (self-weight) and seismic load, as well as to estimate the efficiency of the accepted strengthening method, a series of analyses is done by FEM (see above for details) (Danieli [16]). In addition, the following analyses are performed: immediate influence on stone shell of the load from self- weight, according to the approximate-shell theory of Geckeler I.W. (Timoshenko [17]) (preliminary control analysis); design of reinforced concrete shell for influence of load from the combined self-weight of the strengthened (connected) construction (intensity of combined estimated load at the top of the dome is 2.0 $t/m^2$), according to the theories of limited equilibrium (Akhvlediani [18]) and reinforced concrete.

Input data for analysis was drawn from results of inspection and measures, according to design proposals: For stone dome: density 1.8 t/m3; module of deformations $1.5 x 10^4$ $kg/cm^2$; span of a dome – D = 15.8 m; height (at apex) f = 4.9 m (above the level of the top of light openings); thickness of stone masonry from 0.8 m (at bottom zones) to 0.7 m (at top zone, apex). For concrete shell and concrete connection elements: density 2.4 $t/m^3$; compressive strength for concrete 300 $kg/cm^2$; module of deformations $3.00 x 10^5$ $kg/cm^2$; span of a shell –D = 16.7 m; height (at apex) f = 5.26 m; thickness of shell 0.12 m (in the supporting ring 0.20 m); cross-section of ring beam b x h = 0.40 x

0.60m; for concrete connection elements (joints): cross-section from 0.30 x 0.30m at surface of reinforcement shell, up to 0.45 x 0.45 m at surface of a stone dome; length 0.60 m. Total number of connection elements is 44. Joints are distributed throughout the dome surface by 4 circular lines: 16 (for 1st ring, near the edge beam), 16, 8, 4 (for last ring - near the apex of a dome). Seismic intensity is 9 (acceleration – 0.40 g) (CONSTRUCTION [15]).

### Hagia Sophia dome

**Preliminary notes** - The Hagia Sophia as a whole, and the dome construction in particular, have been considered great structures of the civilized world for almost 15 centuries. Thus, many publications are devoted to their study and research, for example, (Aoki [7], DOMES [1], Emerson [8], HAGIA SOPHIA [10], Krautheimer [3], Kuznetzov [4], etc.). The study and analysis showed that finding a correct solution for the problems of estimating deflected-mode characteristics involves significant difficulties. The main reasons for the difficulties are: construction of the dome contains quite significant initial asymmetric deformations; some authors (HAGIA SOPHIA [10]) consider (and we concur) that the dome's construction from its very beginning was actually divided into some strips in its lower zone by meridional cracks, and it was reasonable to take this into account in modeling the dome's construction; during its existence, the construction was exposed to numerous seismic influences, including at least six destructive events (years 553-557-558; 869, 989, 1344, 1766, 1894), two of which (558 and 1344) led to partial destruction of the dome; during 15 centuries of its maintenance, the structure was completed, restored and changed repeatedly and asymmetrically, using materials of different characteristics in each case, etc.

**Description of employed models** - Three types of finite-element models were used for the series of structural analyses (static and dynamic analysis) (Danieli [16]):

1. Preliminary Model "l" of main structural elements for the entire building.
2. Detailed model of a stone dome only.
3. Model of stone dome connected with concrete shell by connection joints (like a model described above, concerning the strengthening of a dome, Fig. 4).

General characteristics and content of structural analysis for Model "3" were the same as described above for the Ahaltsihe dome (Fig. 2). Input data for the model (geometry, characteristics of materials and so on) were prepared using information from (DOMES [1], HAGIA SOPHIA [10], Kuznetzov [4]) and Models "2" and "3". For stone dome: density is 1.8 t/m$^3$; module of deformations 1.50x10$^4$ kg/cm$^2$; span of a dome –D = 32.0 m; height to apex f ≈14.8 m; thickness of stone masonry from 0.8m (at bottom zones) to 0.7 m (at top zone, apex). For concrete shell and connection elements: density is 2.4 t/m$^2$; concrete compressive strength 300 kg/cm$^2$; module of deformations 3.00x10$^5$ kg/cm$^2$; span of a shell –D =32.90 m; height to apex f ≈15.25 m; thickness 0.16 m (support ring beam 0.20 m); cross section of ring beam b x h = 0.40 x 0.60 m. For concrete connection elements (joints): cross-section is from 0.50 x 0.50 m at surface of reinforcement shell, to 0.60 x 0.60 m at surface of a stone dome; length 0.60m.Total number of connection elements (joints) is 94. Joints are distributed throughout the dome surface by 6 circular lines: 24 (for 1st ring, near the edge beam), 24, 16, 12, 12, 6 (for last ring - near the apex of a dome). Seismic intensity - 9 (acceleration – 0.40 g) (CONSTRUCTION [17]).

## SOME RESULTS OF STRUCTURAL ANALYSIS AND DISCUSSION

### Preliminary remarks

A series of structural analyses were performed to study the main characteristics of behavior in stone and concrete shells and connection elements. These series included: analysis of the original stone dome and the dome connected with a concrete shell, for vertical (self-weight), seismic (Danieli [16]) and thermal loads, and for various numbers of connecting elements (Bloch [19]), as well as analysis of a stone dome fragment to study problems of stress concentration around the connecting elements.

## Vertical and seismic loads

**Stone domes at Akhaltzikhe** - The main results of structural analysis for stone dome in Akhaltzikhe are presented in Table 1. It was found that for the stone dome connected with the reinforced concrete shell, in comparison with a single stone dome: a) For vertical loads, membrane compression stresses are reduced by 25-27%; membrane tension stresses are reduced by 3.5 times. b) For seismic loads: maximum membrane stresses (+/-) are reduced by 3.1-4.1 times; stresses due to bending moments are reduced about 2.0 times. Maximum vertical-end horizontal displacements (+/-) are reduced about 3.0 times. Maximum internal forces in connection joints were: for vertical load 1.72 t-shear, 12.9 t-axial tension; for seismic action 3.75 t–shear, 0.285 t–axial.

**Stone domes at Hagia Sophia** - The main results of structural analysis for stone dome at Hagia Sophia are presented in Table 2. It was found that: a) For vertical loads: 1) maximum horizontal displacements of a ring beam are reduced 2.1-2.4 times - meaning that horizontal reactions that are transferred from dome to columns and surrounding structures also decrease significantly; 2) membrane compression stresses are reduced by 15-25%; 3) membrane tension stresses are reduced by 2.8 times. b) For seismic loads: 1) maximum horizontal displacements of a dome are reduced 4.2 times, and vertical displacements about 5.0 times; 2) membrane stresses (+/-) are reduced by 1.5-2.2 times; 3) stresses due to bending moments are reduced 1.2-1.6 times. Maximum internal forces in connection joints were: for vertical load 65.9 t-shear, 24.7 t-axial; for seismic action 21.6 t–shear, 21.8 t–axial.

The obtained results show the efficiency of the proposed strengthening method. A detailed description of the use of the structural models and the main results of these analyses were presented (Danieli [16], Bloch [19]).

## Thermal problems

A special series of static finite-element analyses was performed to estimate the level of deformations and stresses in stone and concrete domes and their connecting elements for thermal loads. The structural model of domes (Fig.4) consisted of triangle plane-type finite elements (FE type # 41) to simulate shell surfaces, space frame-type bar elements (FF type # 5) to simulate edge beams and spring-type finite elements (FE type # 51) for simulation of connecting elements between stone and concrete domes. Geometry and rigidities of stone domes were taken according to 2.1.3 and 2.2.2: modulus of elasticity $1x10^5$ kg/cm$^2$; factor of Poison 1/6 (0.17); factor of thermal expansion $0.5x10^{-5}$. Cross sections of the reinforced concrete dome and connecting elements were presented in (Danieli [16], Bloch [19]): modulus of elasticity $3x10^5$ kg/cm$^2$; factor of Poison 1/5 (0.20); factor of thermal expansion $1x10^{-5}$. The performed series of structural finite-element analyses for thermal problems included the solution of two main problems:

1. Structural analysis for a gradient of overall temperatures (overall temperatures were taken equal to cross sections) loaded by warm (or cold) season.
2. Structural analysis loaded by a gradient of temperatures of shell cross sections at warm (or cold) season.

Corresponding regulations and methods were used for estimating the main parameters of thermal loads: values of the overall thermal gradient (problem # 1) and the thermal gradient by shell cross section (problem # 2). Design values of the main parameters were taken as following:

- overall thermal gradient ( problem # 1 ): $20^0$ degrees;
- thermal gradient by shell cross- section (problem # 2): + $10^0$ /-$10^0$ degrees.

The series of finite-element structural analyses of stone domes only were performed additionally for the same values of thermal gradient. It allowed us to estimate the influence of the reinforced concrete shell on results (deformations, stresses in stone dome and connecting elements). The main results of the performed analysis are presented in Table 1 and Table 2 and Fig.5, 6, 7. Corresponding values of deformations and stresses due to vertical and seismic loads are included in Tables 1 and 2 also, for the sake of comparison. As could be demonstrated, the values of deformations and stresses due to thermal loads are on the same order as the corresponding values due to vertical and seismic

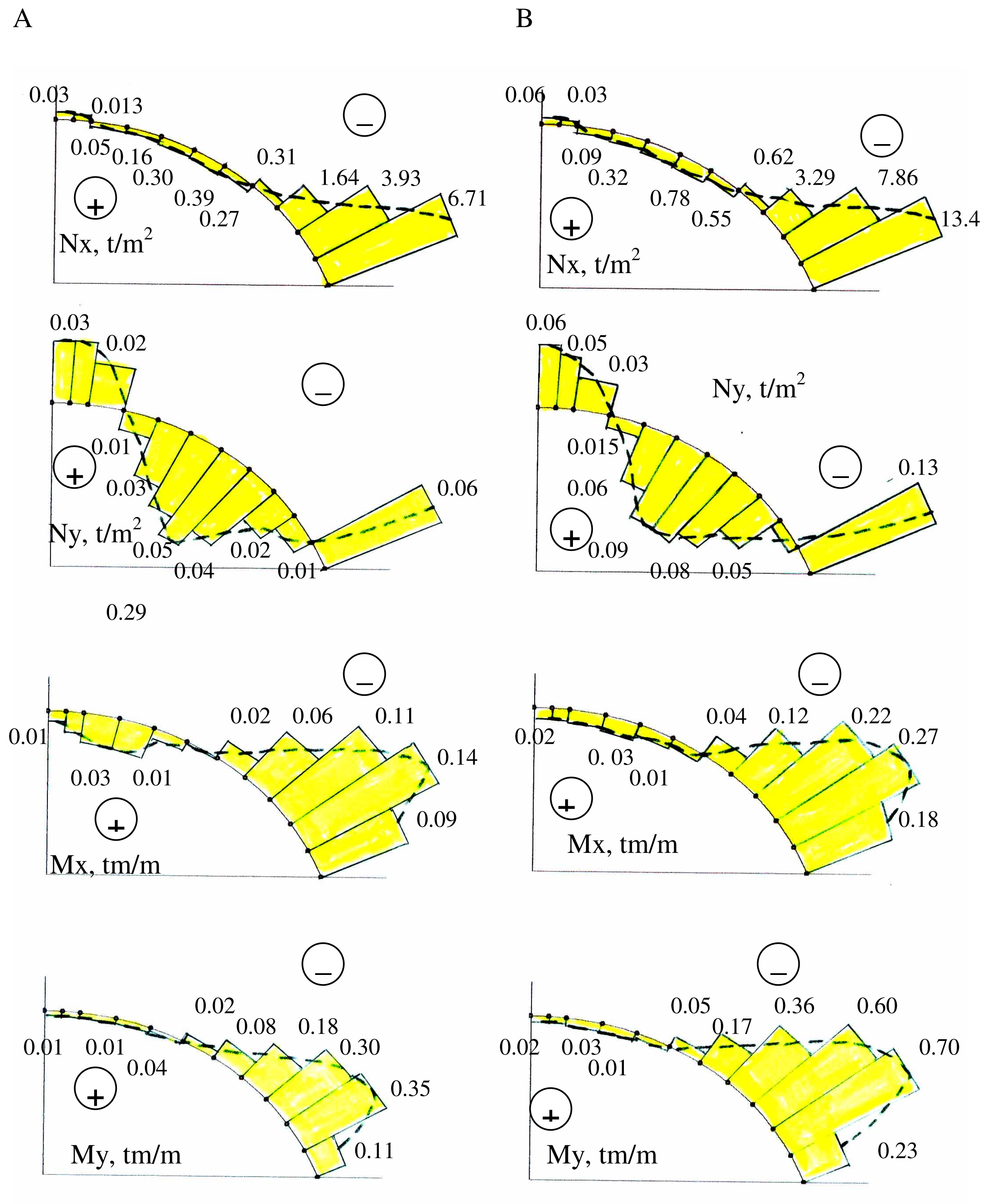


Fig. 5. Dome in Ahaltsihe

Thermal loads. Stone dome only. Internal stresses and moments in a stone dome.
A – overall (uniform) thermal gradient 20 degrees
B – non-uniform thermal gradient +10 degrees/-10 degrees

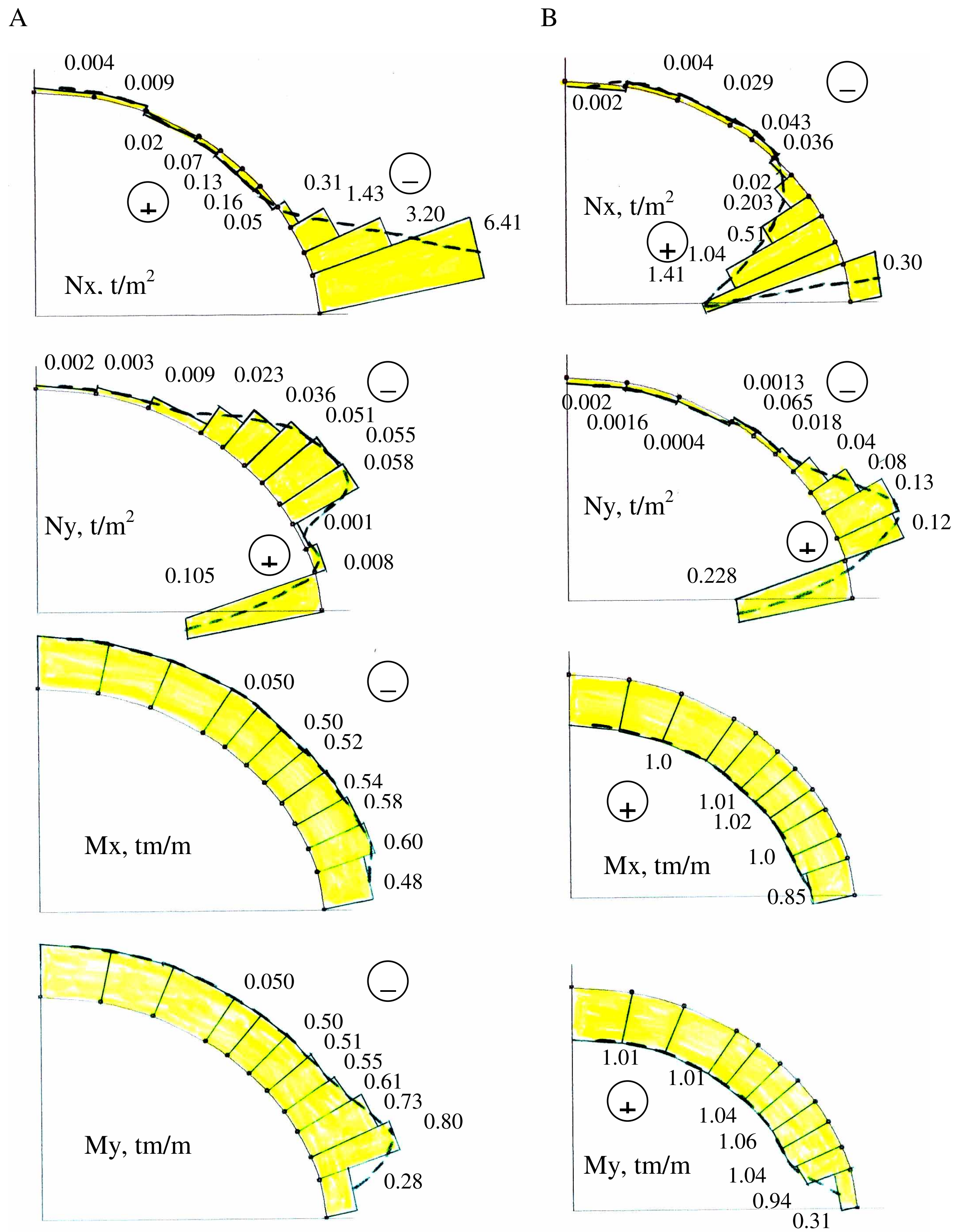


Fig. 6. Dome in Ahaltsihe

Thermal loads. Interconnected stone and concrete domes. Internal stresses and moments in a stone dome.

A – overall (uniform) thermal gradient 20 degrees

B – non-uniform thermal gradient +10 degrees/-10 degrees

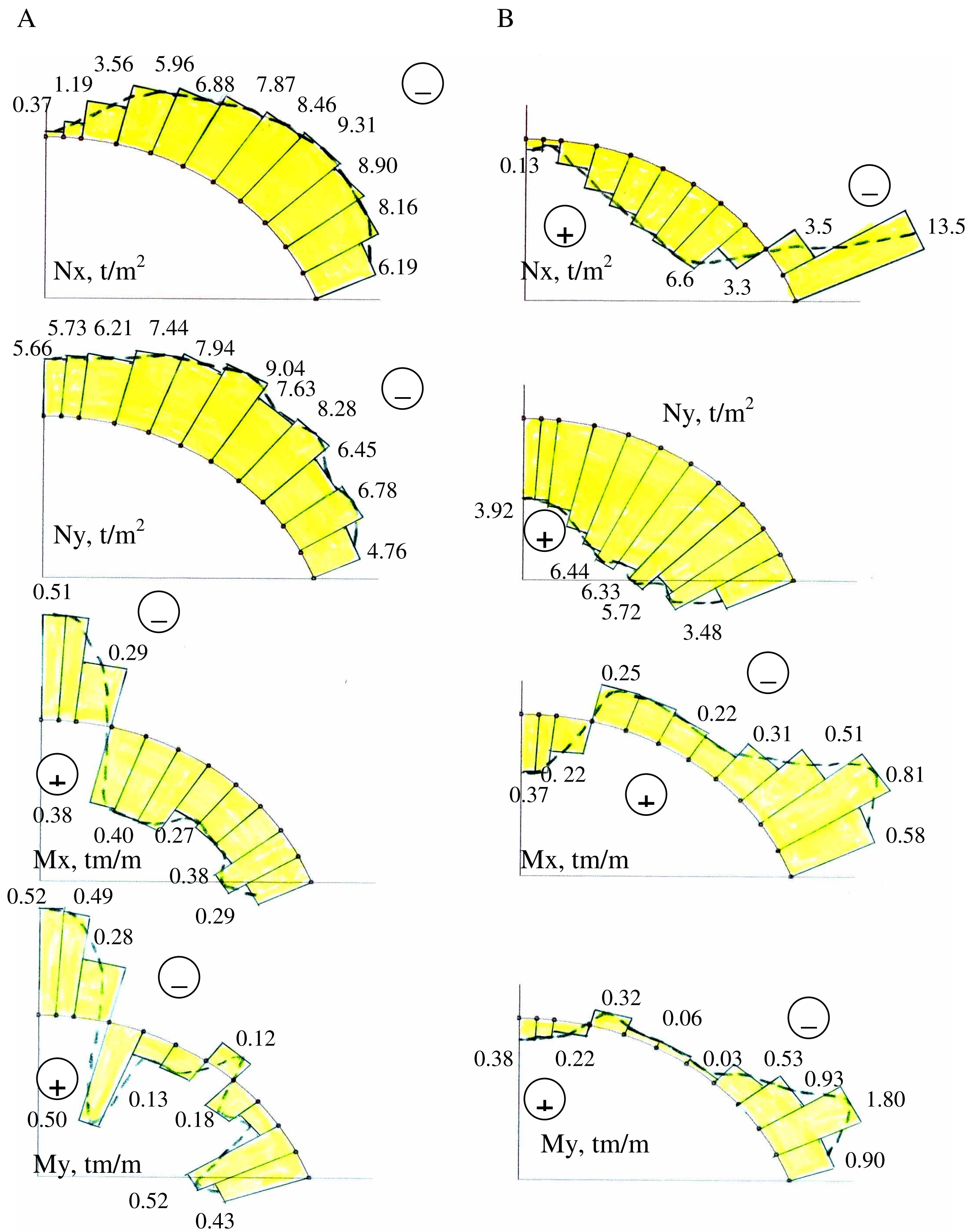


Fig. 7. Hagia Sophia dome

Thermal loads. Interconnected stone and concrete domes. Internal stresses and moments in a stone dome.

A – overall (uniform) thermal gradient 20 degrees

B – non-uniform thermal gradient +10 degrees/-10 degrees

loads. Therefore, for such problems it is important to take into consideration the thermal loads as independent load cases. Values of additional forces due to thermal loads for some zones of shells and connecting elements exceed the corresponding values for seismic load. In these cases, thermal loads are to be included into load combinations.

## Stress concentration problem: Physical non-linear problems

It is known that high levels of stress and a complex character of stress distribution are typical for connection zones. The same is confirmed by results of linear analysis (Timoshenko[20]). On the other hand, fracture often did not occur practically at connection zones for these levels of stresses. This could be explained by non-linear behavior of material and redistribution of stresses. Hence, it was decided to perform a series of non-linear structural analyses for connection zones. Proposed engineering solutions for strengthening of a stone dome foresee the use of protruding reinforced concrete connecting elements; these connecting elements are joined to the existing stone dome (to be strengthened) and the newly cast reinforced concrete shell (the strengthening structure. These protruding connecting elements are of prismatic shape and are executed in cavities, which are specially drilled into a stone dome. The performed series of structural analyses for connected stone and reinforced concrete domes allowed us to find the main parameters of deformed and stressed states for domes and connecting elements. On the other hand, the shape of the protruding connecting elements leads to stress concentration in stone dome zones adjacent to these connecting elements. Evaluation of the stress concentration problem was the aim of the performed series of non-linear finite-element structural analyses. Plane strain-stress state non-linear problems were considered in order to simplify the structural models being used.

The structural models were extracts of a structure, which consisted of a reinforced concrete connecting element and surrounding zones of the stone dome. Two main schemes of load were examined for a connecting element: 1-axial load, 2-pure shear (Fig.8). The software "MAG-STR" was used for analysis, as a physical non-linear element library; and step-by-step loading procedures are available. Two linear test problems were solved as a preliminary to define more precisely the parameters of a finite-element mesh. The scheme for the first test problem corresponded to a famous problem (Timoshenko [20]): a mesh of 35 x 26 elements was used here. For the second test problem, the scheme was the same as that used for non-linear analysis. The influence of mesh density was tested here: meshes of 64 x 42 elements and 34 x 22 elements were used. Finally, a mesh of 34 x 22 elements was chosen for non-linear structural analysis, according to the results of these preliminary test problem solutions. The non-linear structural model of a fragment consisted of non-linear plane triangular and rectangular finite elements; total number of nodes is 960, total number of finite elements 1001. "Deformation-stress" streak lines with a number of steps (18) were used for the description of non-linear properties of materials (stone, concrete).Following characteristics of materials were used for analysis: Stone: compression –breaking stress 30 $kg/cm^2$; limit of deformations 0.0030; tension-breaking stress 3 $kg/cm^2$; limit of deformations 0.0020; Concrete: compression –breaking stress 200 $kg/cm^2$; limit of deformations 0.0036; tension-breaking stress 20 $kg/cm^2$; limit of deformations 0.00036. The procedure for obtaining a non-linear solution is to check the values of general deformation for every plane finite element at every step of load. These values of general deformation are compared with the limit values of deformations defined by the "deformation-stress" streak line. In some cases, values of generalized deformations exceed the limit values at a current load step, and corresponding finite elements are switched off. Switching off is simulated here by zero values for the corresponding properties of materials for these finite elements. The step-by-step load procedure included 22 steps of load, with correction of results at every step. The state of the material was checked at every load step during load increases. Output information about the state of the material, values of deformations and stresses is available; it allowed us to follow the progress of damage accumulation and the effects of stress redistribution. The main results of the series of non-linear FEM analyses are presented in Tables 3 and Fig.9 and.10. In reviewing the results of the non-linear analysis, it was found that stresses in a stone dome could reach high values for zones near concrete connection elements. Therefore, it could be recommended to provide special preliminary preparation of a stone surface for the zones of connection elements. For example, concrete spraying of high-strength material (compression-breaking stress 200kg/cm$^2$ and more) with a thickness of 2-3 cm could be used.

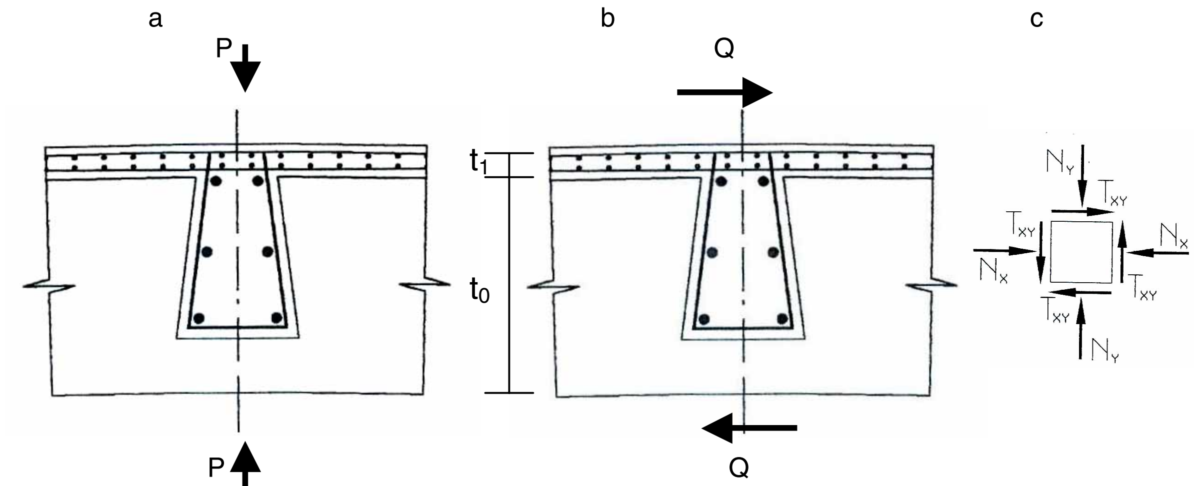


Fig. 8. Load schemes for non-linear finite element analysis.
a) axial load (P = 50 t); b) shear load (Q = 50 t); c) stressed state components

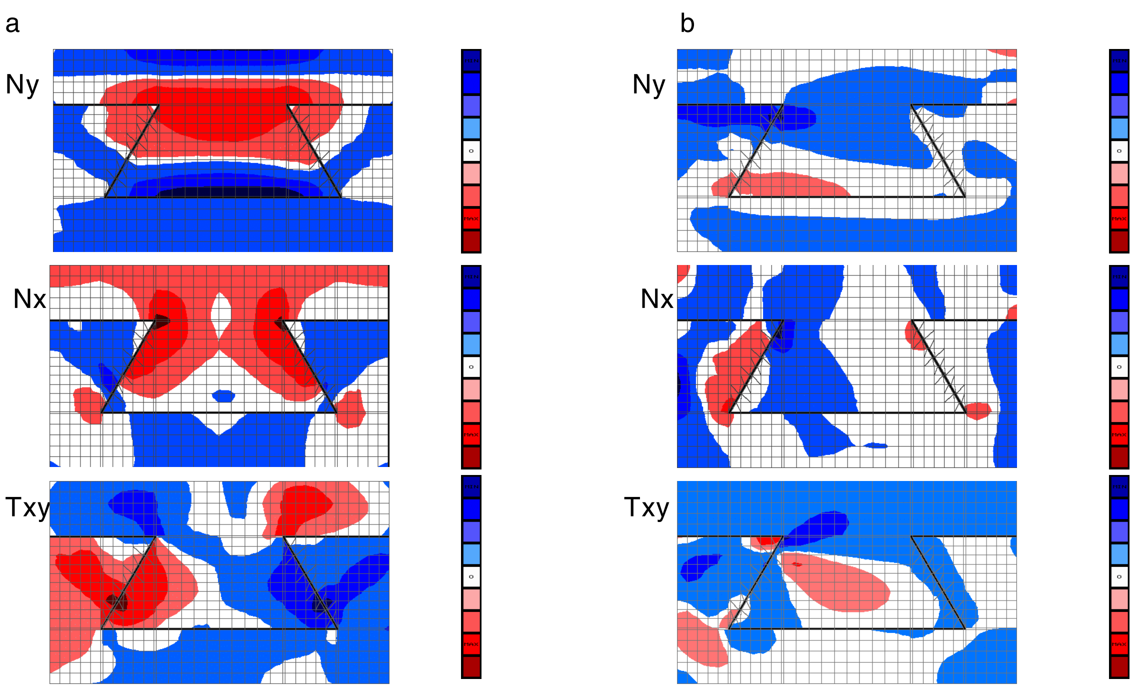


Fig. 9. Stress concentration problem. Connection zone fragment. Non-linear analysis.
Distribution of normal (Nx, Ny) and shear (Txy) stresses.
a) axial load (P = 50 t); b) shear load (Q = 50 t)

Table 1. Dome in Ahaltsihe. Main results of analysis*

| No | Parameters | Thermal load uniform $\Delta t=+20^o$ | Thermal load non-uniform $\Delta t_1=+10^o$ $\Delta t_2=-10^o$ | Vertical loads | Seismic loads(Ground acceleration – 0.4 g ) |
|---|---|---|---|---|---|
| 1 | Horiz.displ, mm, U, V | | | ±0.21/±0.16 | ±0.37/±0.11 |
| 2 | Vertic.displac.mm, W | | | –1.36/–1.08 | 0.16/±0.05 |
| 3 | Period of free vibr.,s | | | | |
| | $T_1$ | | | | 0.092/0.044 |
| | $T_2$ | | | | 0.068/0.003 |
| 4 | Inter.membran.str, t/m$^2$ | | | | |
| | + Nx | +0.78/+6.90 | +0.39/–0.37 | +3.8/+0.9 | |
| | – Nx | –1.34/–13.5 | –6.71/–9.31 | –10.9/–8.0 | ±2.8/±0.9 |
| | + Ny | –0.13/+6.40 | –0.06/–0.04 | – | |
| | – Ny | +0.09/+3.48 | +0.05/–4.76 | –10.5/–8.2 | ±1.3/±0.3 |
| 5 | Bend. moment, tm/m | | | | |
| | + Mx | +0.03/+0.28 | +0.03/+0.24 | | |
| | – Mx | –0.27/–0.50 | –0.14/–0.08 | –0.176/0.376 | ±0.163/±0.08 |
| | + My | +0.03/+1.35 | +0.01/+0.77 | 0.188/0.368 | |
| | – My | –0.70/–1.22 | –0.35/–0.56 | | ±0.27/±0.131 |
| 6 | Int.forc.in st ring beam | | | | |
| | Axial force,N,t | -/7.01 | /+5.87 | +26.4/+18.7 | |
| | Bend.moments,t.m | | | | |
| | My(vert.) | /0.28 | /0.07 | | |
| | Mx(hor.iz.) | /1.37 | /0.89 | | |

Remark: *Maximum values (stone dome only/stone dome in interconnected structure)

Table 2. Hagia Sophia Dome. Main results of analysis*

| No | Parameters | Thermal load uniform $\Delta t=+20^o$ | Thermal load non-uniform $\Delta t_1=+10^o$ $\Delta t_2=-10^o$ | Vertical loads | Seismic loads(Ground acceleration – 0.4 g ) |
|---|---|---|---|---|---|
| 1 | Horiz.displ, mm, U, V | ±1.47/±1.20 | ±0.15/±0.55 | ±1.28/±0.31 | 20.6/±0.65 |
| 2 | Vertic.displac.mm, W | 1.86/1.86 | 0.043/0.53 | –5.39/–1.19 | 4.4/±0.07 |
| 3 | Period of free vibr.,s | | | | |
| | $T_1$ | | | | 0.321/0.183 |
| | $T_2$ | | | | 0.216/0.111 |
| 4 | Inter.memb.str, t/m$^2$ | | | | |
| | + Nx | +1.41/+15.5 | +0.16/(–6.5) | +7.5/+2.0 | |
| | – Nx | –0.30/–18.0 | –6.41/–17.2 | –13.4/–5.9 | ±6.4/±5.8 |
| | + Ny | +0.23/+14.8 | +0.11/(–5.8) | –/+6.7 | |
| | – Ny | –0.13/–19.6 | –0.06/(17.9) | –24.0/–15.7 | ±28.0/±21.8 |
| 5 | Bend. mom, t.m/m | | | | |
| | + Mx | +1.02/+0.28 | (–.48)/+0.24 | –0.45/–0.23 | ±1.56/±0.97 |
| | – Mx | (+0.85)/–0.5 | –0.60/–0.08 | | |
| | + My | +1.06/+1.35 | (–.80)/+0.77 | –0.80/–0.90 | ±2.48/±2.14 |
| | – My | (+0.31)/–1.2 | –0.28/–0.56 | | |
| 6 | Int.forc.in st ring bea | | | | |
| | . Axial force,N ,t | -0.60/+7.85 | +1.84/+3.68 | 14.0/10.8 | ±2.3/±2.6 |
| | Bend. Moments.,t.m | | | | |
| | My(vert.) | 0.17/+0.015 | +/-0.48/-0.01 | 1.92/1.9 | 0.06/±0.3 |
| | Mx(hor.iz.) | 0.002/+0.043 | 0.001/-0.043 | 1.7/2.6 | 0.23 /±2.4 |

Remark: *Maximum values (stone dome only/stone dome in interconnected structure)

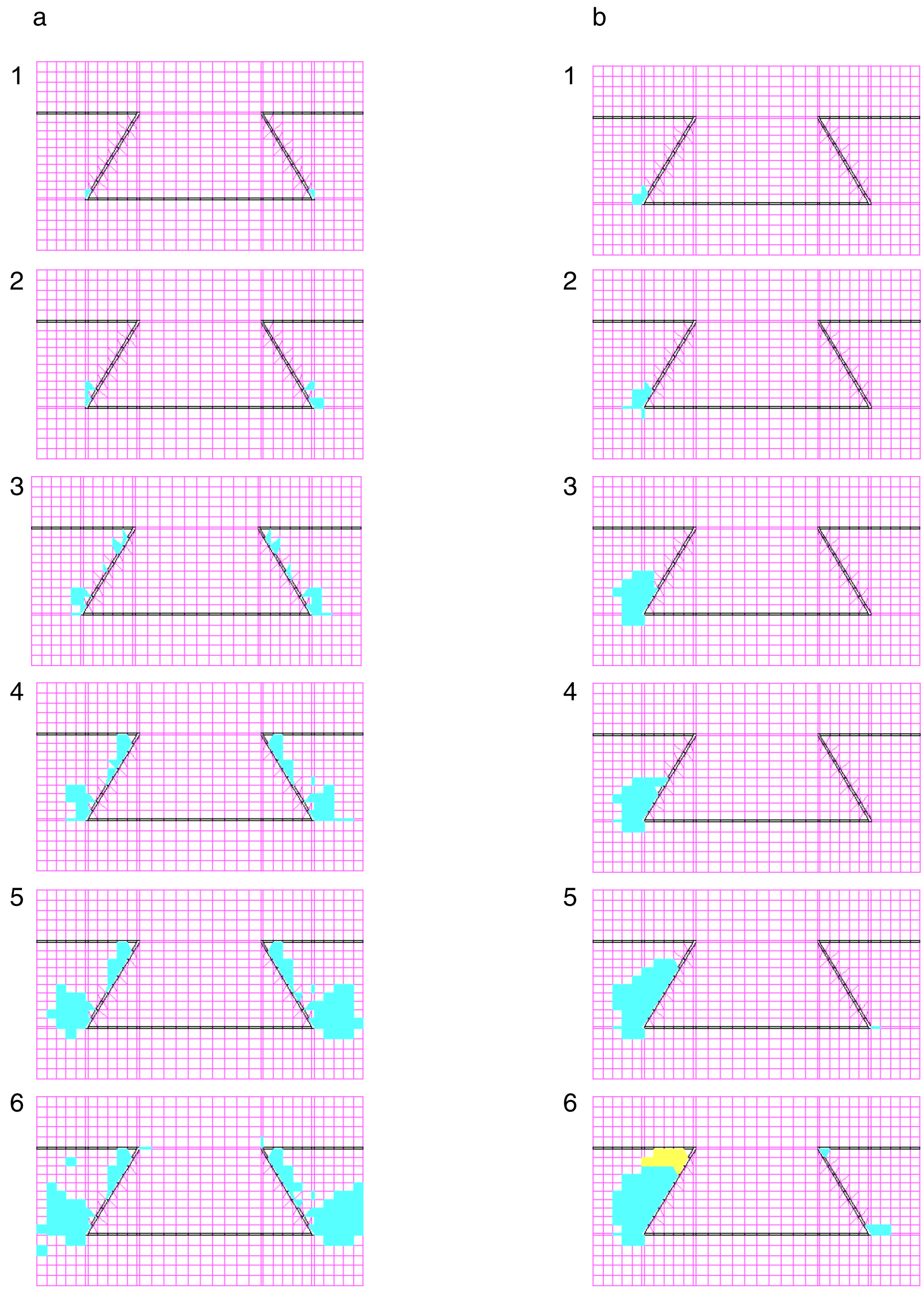


Fig.10 Stress concentration problem. Connection zone fragment. Non-linear analysis. .
Development of failure stress regions versus applied load level.

a – axial loading: P=50 t. 1 – step 3, $P_3$=0.2P; 2 - step 7, $P_7$=0.4P; 3 – step 11, $P_{11}$=0.6P; 4 – step 14, $P_{14}$=0.75P; 5 – step 18, $P_{18}$=0.89P; 6 – step 26, $P_{26}$=P.

b – pure shear loading: Q=50 t. 1 – step 2, $Q_2$=0.4Q; 2 - step 7, $Q_7$=0.7Q; 3 – step 12, $Q_{12}$=0.8Q; 4 – step 16, $Q_{16}$=0.84Q; 5 – step 20, $Q_{20}$=0.88Q; 6 – step 22, $Q_{22}$=0.9Q.

Table 3. Main results of analysis: maximum values of internal forces in connections. Interconnected structure*

| No | Ring No | Number of connections of ring | Forces, t (axial R**/shear S) | | | |
|---|---|---|---|---|---|---|
| | | | Thermal load uniform Δt=+20o | Thermal load non-uniform Δt1=+10o Δt2=–10o | Vertical loads*** | Seismic loads (Ground acceleration – 0.4g ) |
| Dome in Ahaltsihe (total number of connections 44) | | | | | | |
| 1 | 1 | 16 | +3.5/±13.5 | -3.23/±12.0 | +1.8/±1.72 | ±0.285/±3.75 |
| 2 | 2 | 16 | +4.22/±6.90 | -5.99/±10.35 | +3.5/.±0.25 | ±0.255/±2.92 |
| 3 | 3 | 8 | +5.12/±4.41 | -7.55/±7.34 | +6.8/±0.44 | ±0.20-/±1.60 |
| 4 | 4 | 4 | +6.04/±2.65 | -8.42/±4.01 | +12.9/±0.78 | ±0.075/±2.34. |
| Hagia Sophia dome (analogue) (total number of connections 94) | | | | | | |
| 5 | 1 | 24 | +34.1/+22.1 | +12.9/–6.0 | +24.7/±65.9 | ±1.4/±13.8 |
| 6 | 2 | 24 | –24.6/–34.7 | +18.4/+6.3 | +13.9/±24.6 | +1.1/±7.1 |
| 7 | 3 | 16 | +41.5/–41.2 | –23.2/+22.7 | +15.5/±22.0 | ±21.8/±21.6 |
| 8 | 4 | 12 | +12.8/+17.5 | –24.5/–21.3 | +17.1/±17.7 | ±6.3/±6.6 |
| 9 | 5 | 12 | +14.3/+22.4 | –19.3/–12.2 | +11.2/±14.2 | ±2.4/±3.0 |
| 10 | 6 | 6 | +19.1/+29.6 | –21.9/–8.4 | +11.1/±13.0 | ±0.5/±2.2 |

Remarks:
* stone dome only/stone dome in interconnected structure
** Resulting, in direction perpendicular to connection axis
***Load from weight reinforced concrete shell + weight stone dome

## CONCLUSION

For perception of seismic load during possible severe earthquakes without significant damages, and preservation of existing ancient appearances of the lower surface of the dome in its present state of antiquity, this original dome conservation (strengthening) structure is proposed. It consists of a new thin-walled reinforced concrete shell with a supporting ring, arranged above the existing stone dome. The necessary connection for joint work, as an interconnected structure between the stone dome and reinforced concrete shell, is done by means of reinforced concrete connections. They protrude as pins from the reinforced concrete shell, have a form of a truncated pyramid with broad contact in the stone dome, and are distributed throughout the entire surface of the strengthened dome. Thus, an interconnected stone-reinforced concrete structure is created. The efficiency of the proposed method is shown on the grounds of numerical analysis of vertical, thermal and seismic load with the use of FEM, on examples of Ahaltsihe and Hagia Sophia (analogue) stone domes. Its essence lies in: the potential regulation of the stress-strain state of the stone dome by means of changing the quantity and location of connecting members; stress strains in the stone dome are significantly decreased; supporting constructions are significantly unloaded due to the influence of horizontal forces, as the thrust is wholly absorbed by the reinforced concrete supporting ring. As a result, the earthquake resistance of the stone domes increases significantly.

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## REFERENCES


1. Domes from Antiquity to the Present. Proc. of the IASS-MSU Int. Symposium. Istanbul, Turkey, May 30-June 3, 1988. Publisher: Mimar Sinan Universitesi, Istanbul, 1988.
2. Heyman J. The Stone Skeleton – Structured Engineering of Masonry Architecture, Cambridge University Press, 1995.
3. Krautheimer R. and Ćurčić S. Early Christian and Byzantine Architecture. Penguin Books Ltd., 1986.
4. Kuznetzov A.B. Tectonics and Structures of Symmetrical Buildings (in Russian). Moscow, 1951.
5. Kirikov B. Earthquake Resistance of Structures: from Antiquity to Our Times. Mir Publishers, Moscow, 1992.
6. Lefandeux G. Les cupoles en maconnerill. Genie civil, 1967, 7, 144 (in French).
7. Aoki T., Kato S., Ishikawa K., Hidaka K., Yorulmaz M. and Gili F. Principle of Structural Restoration for Hagia Sophia Dome. Proc. of STREMAH Int. Symposium, San Sebastian, 1997, 6, pp. 467-476.
8. Emerson W. and Van Nice, R. L. Hagia Sophia, Istanbul: Preliminary Report of a Recent Examination of the Structure, Supplement to American Journal of Archaeology, 1943, Volume XLVII.403-416.
9. Gabrilovich P. and Richard P. Methodology for Strengthening and Repair of Earthquake Damaged Monuments in Pagan – Burma. Proc. of Eight World Conference on Earthquake Engineering, San – Francisco, U. S. A., Prentice Hall Inc., vol. 1, 1984, 609-616.
10. Hagia Sophia from the Age of Justian to the Present, Edited by Mark R. and A. Ş. Calmak, Cambridge University Press.1992.
11. Karaesmen E., Unay A.I., Erkay C. and Boyaci N. Earthquake Engineering. Proc.of Tenth World Earthquake Engineering Conference. Balkema, Rotterdam, 1992.
12. Danieli (Danielashvili) M., Gabrichidze G., Goldman A. and Sulaberidse O. Experience in Restoration and Strengthening of Stone Made Ancient Domes in Seismic Regions. Proc. of 7th US National Conference on Earthquake Engineering, Urban Earthquake Risk. July 21-25, 2002, Boston, Massachusetts, USA. Vol. II, 2002,. 1167-1175.
13. Danielashvili M., Gabrichidze G., Melashvili Y. and Sulaberidze O. Study of Some Reinforced and Metal Spatial Structures, in Seismic Regions of Georgia, Proc ICSS-98, Spatial Structures in New and Renovation Project of Buildings and Construction. Vol. I, Moscow, 1998, 396-403.
14. Engineering Analysis of the Racha Earthquake Consequences in Georgia, 1991. Editor Gabrichidze G., Metsniereba, 1996, Tbilisi, Georgia.
15. Construction Codes and Regulations, SNIP II.7.81, Part II, Chapter 7 Construction in Seismic Regions (in Russian), Stroyizdat, Moscow, 1982.
16. Danieli (Danielashvili) M., Goldman A., Aronchik A., Bloch J., Method of Increasing Seismic Resistance in Ancient Stone and Brick Domes. Proc. of Fifth National Conference on Earthquake Engineering, 26-30 May 2003, Istanbul, Turkey, 2003, pp. 1-10, Paper No: AE - 008 (SD-R).
17. Timoshenko S.P. and Woinovsky-Kriger S. Theory of Plates and Shells. 2nd edition, McGraw-Hill, New York, Toronto, London, 1959.
18. Akhvlediani N.A. and Danielashvili M.A. Limit Analysis of Reinforced Concrete Shells, Arkhivum Inzynezii Ladowey, Tom XXXVI, Z 3, Warshava, Poland, 1990, 187-205.
19. Bloch J., Aronchik A., Goldman A., Danieli (Danielashvili) M. A Method of Strengthening Ancient Domes and Vaults, and Problems of their Stress – Strain States in Seismic Regions. Wessex Institute of Technology, UK. ERES IV Earthquake Resistant Engineering Structures, 22-24 September 2003, Ancona, Italy. Publl., In .HPSM 2004 Second International Conference on High Performance Structures and Materials, 31 May -2 June 2004, Ancona, Italy WIT Press Southampton, Boston, 2004.
20. Timoshenko S. Strength of Materials. Part II. 3rd Edition. Toronto, N.Y., London, 1956.