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Protection Buildings for Ruins and Monuments

Bâtiments de protection pour des ruines et monuments

Schutzbauten für Ruinen und Monumente

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INTRODUCTION

The degradation of our cultural heritage has been increasing rapidly during the last decades. Examples from Italy clearly demonstrate the increased rate of degradation.

There are a number of cases that warrant the concern shown in recent years, and if urgent measures are not taken, historical buildings of great significance will inevitably be lost.

The obvious conclusion is that the increasing air pollution must be the cause of this disturbing development. Therefore, the problem of preservation has met with new challenges.

During the last decade a few proposals for protection buildings for cultural objects have been presented, e.g. a shed roof over Parthenon on Acropolis, Athens, and a protective shell over the Column of Marcus Aurelius in Rome, (Museum, Quart. rev., Unesco, I53, 1987).

In Norway, the ruins of the ancient cathedral at Hamar have been degrading since 1567, when the church roof burned down. During the last decade the Norwegian Central Office of Historical Monuments and Sites decided to build a protection building over the ruin, and an architectural competition was held in 1987.

In 1990 a research project, Eurocare Carebuild, was started, having the objective of developing a technology package which may serve custodians having objects that need protection.

THE PROTECTION BUILDING AT HAMAR

The protection building at Hamar, designed by the architects Lund & Slaatto is an aluminium/glass building having warped, skew glass walls, see Fig. 1.
For the project a new aluminium space deck system for triangular glass panels has been developed. The system can adjust itself to form a warped surface, see Fig. 2.

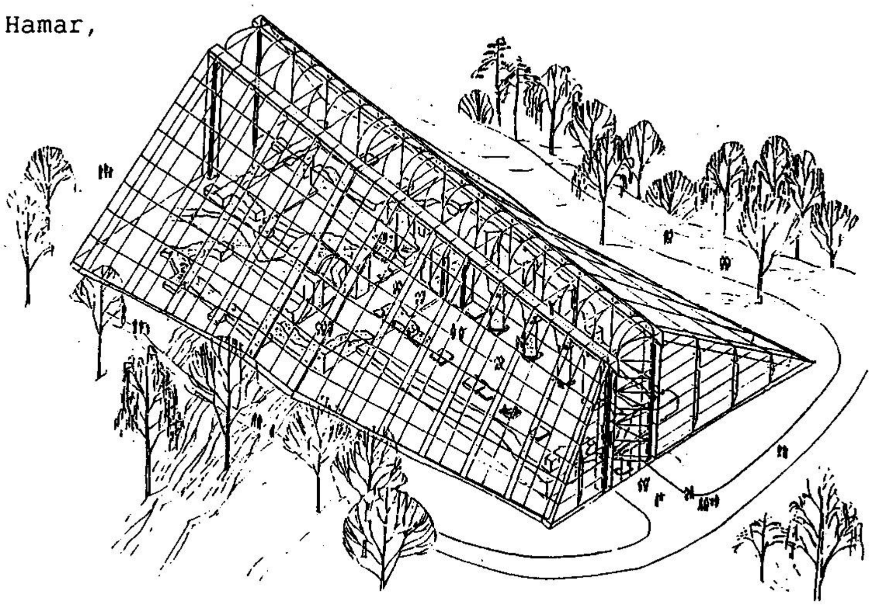

Fig. 1 Drawing of the protection building

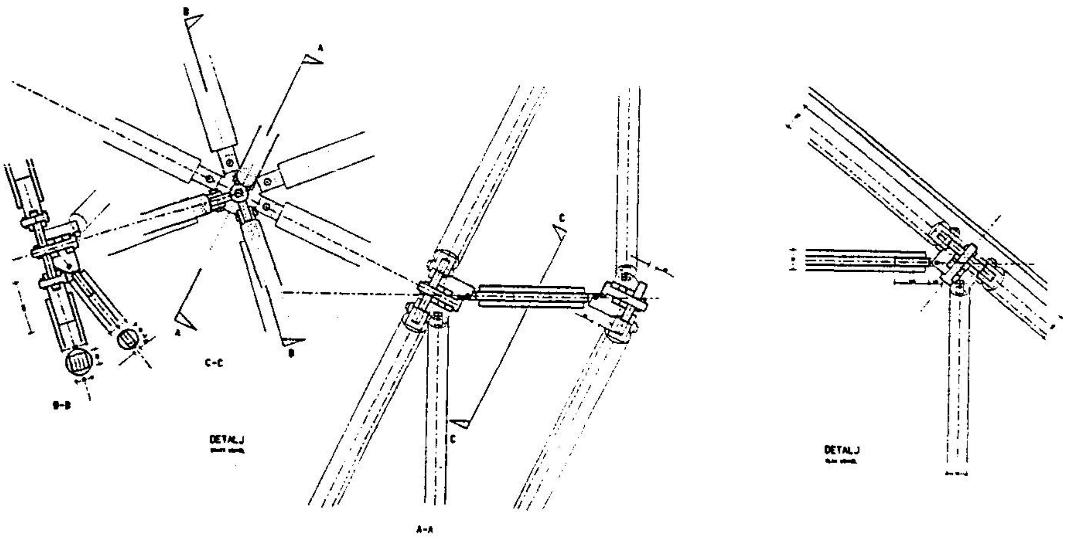

Fig. 2 Joints of the designed protection building at Hamar

EU 446 EUROCARE CAREBUILD

The research project has the following project profile, see Fig. 3, and is planned to run till 1995.

Title	: EUROCARE CAREBUILD Envelope buildings for historic buildings, monuments, stone ruins, etc.
Supported by	: Norwegian Council for Scientific and Industrial Research
Participants	: Norwegian Central Office of Historical Monuments and Sites NILU (Norwegian Institute of Air Research) Lund & Slaatto Arkitekter A/S Erichsen & Horgen A/S, HVAC-consultants Dr.techn. Kristoffer Apeland A/S, structural consultants University of Lund, Sweden (Professor Bo Adamson)
Project leader	: Professor Kristoffer Apeland, Oslo School of Architecture

Fig. 3 EU 446 Eurocare Carebuild Project Profile

So far, interesting results have come up in connection with:

- Consequences for archeological layers when a protective shell is built over the layers (published in Norwegian).
- Special energy use and conservation aspects for protective shells over stone ruins and buildings (to be published).
- Application of RILEM/CIB method for prediction of service life.
- New design of space decks in aluminium for warped surfaces.

The technology package will be further developed during the project.

Two Aspects of Structural Reinforcement of Historic Monuments

Deux aspects du renforcement structural des monuments historiques

Zwei Aspekte struktureller Verstärkung historischer Bauwerke

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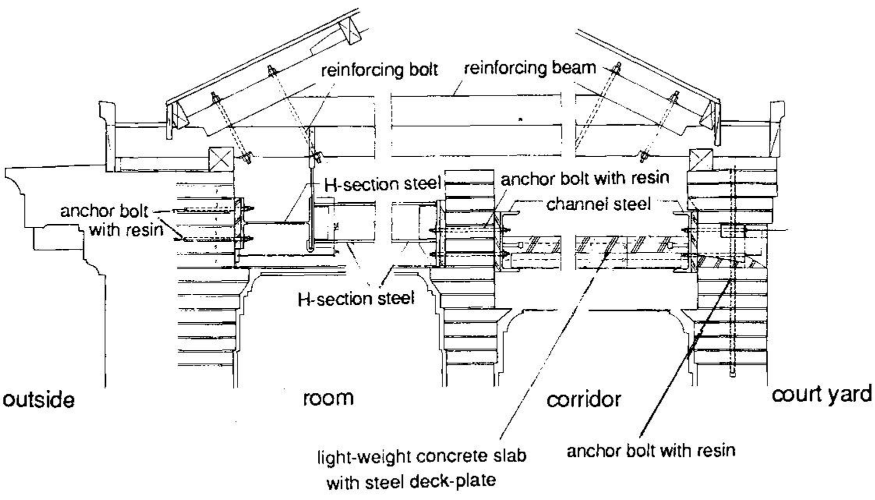
Structural reinforcement of historic monuments can be inducted to preserve buildings either in their authentic condition or only in their outer appearance. Among recent restoration-conservation in Japan, remarkable examples are introduced in the following:

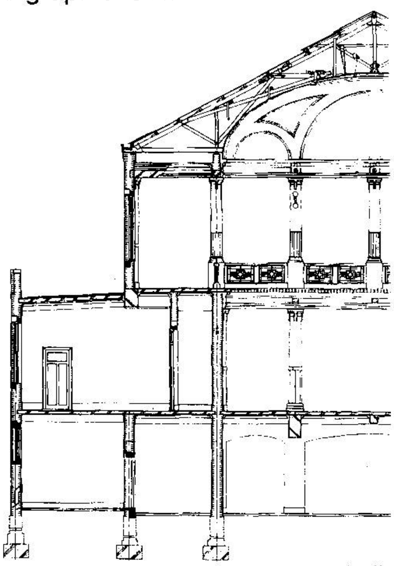
Example 1. Nagoya High Court and District Court Building (Important Cultural Property) Completed in 1922, Baroque Revival Style, building area: 2,241.8m^2, 3 stories, base: brick & concrete, wall: brick, slab: reinforced concrete, beam: reinforced concrete, arch: brick, roof truss: timber, roof: copper, restoration: commenced in 1984 and completed in 1989,

Reference: Japanese Association for Conservation of Architectural Monuments, "Restoration of Nagoya High Court and District Court Building 1984-1989", Nov. 1989, Nagoya City.

On the poster, the whole structural reinforcement will be shown in detailed graphic form.



reinforcement on the top of longitudinal walls



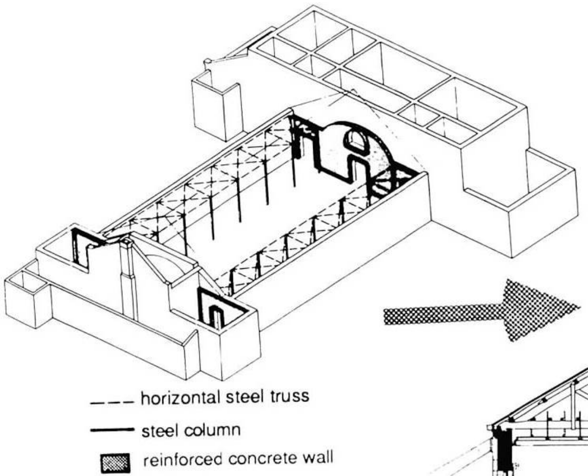
section of hall

The highlight of this building is the central hall, a square of approximately 17x11 meters, which has an open space, 10.9x8.2 meters, from the ground floor to the ceiling of the 2nd. floor covered with a big top light. This space supported by brick columns and walls bound together with reinforced concrete beams and slabs was insufficient concerning resistance against earthquake. In order to prevent at the top of brick wall from bending, it was necessary to achieve a monolithic horizontal reinforcing structure. As important as the hall's spatial qualities as a cultural property are, as weak they are seen from the structural point. After thoroughly examination and discussion, they innovated careful and proper ways that the original appearance of hall was maintained without reminding any trace of reinforcement. To achieve a monolithic structure against the earthquake, the top of hall and roof trusses were reinforced by using buttresses, reinforcing bar, anchor bolt with resin and steel. The buttresses along the 2nd floor's wall face to the court yard can be seen only from the 2nd floor's veranda behid the hall. To connect walls/columns to beams/slabs, reinforcing bars and anchor bolts with resin were inserted in the brick columns, walls and beams, channel steel beams were used along the top of the brick walls and H-section or channel steel beams were used along the top of the façade and back walls. These reinforcements were put inside the sturctural frame and roof trusses.

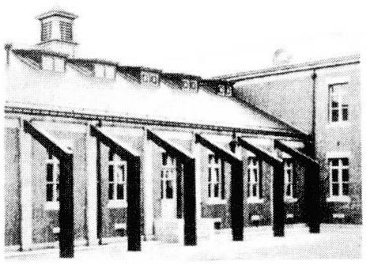
Example 2. Yamagata Prefectural Government Assembly Hall (Important Cultural Property)
Completed in 1916, Gothic Revival Style, building area: 866.4m^2, 1 story/partially 2 stories,
base: brick, wall: brick, floor framing: timber, roof truss: timber, roof: slate,
restoration: commenced in 1986 and completed in 1990,

Reference: Japanese Association for Conservation of Architectural Monuments, "Restoration of Yamagata Prefectural Government Building and Assembly Hall Vol.1 / The Assembly Hall Report on the work 1986-1990", Mar. 1991, Yamagata Pref.

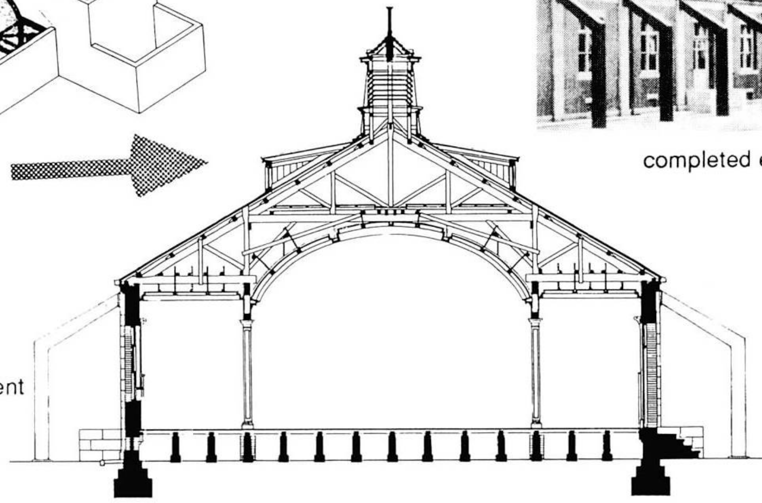
On the poster, the whole structural reinforcement will be shown in detailed graphic form.



horizontal steel trusses reinforcement not realized



completed exterior



buttresses reinforcemment realized

The design of this building, which has a Basilica style vaulted ceiling with top lights, is unique for governmental assembly halls in Japan. Although the interior had been changed, the close investigation during dismantling made it possible to reconstruct the original condition with original materials. Reinforcing bars and structural steel were needed to strengthen the structure against earthquakes. For there was not adequate space to put horizontal reinforcing trusses along the top of the brick walls, the common reinforcing way like Ex.1. could not be applied. It would have been inevitable to change the original interior condition with additional new walls to support the trusses and they would have overlaid important vestiges on the original wall. After many examinations and discussions, it was decided that the walls of the hall were to be reinforced by using exposed structural steel such as flying buttresses on the exterior. Although buttresses are very striking and disturbing on the exterior, it was judged that the authenticity of the interior with its historical materials is more valuable and has to be given preference against an outer appearance with no reinforcing additions. This was the first time of this kind of restoration of a designated cultural property. In almost all previous restoration cases, preserving the historical outer appearance as a landmark in the townscape and preserving the historical materials was considered as equally important. The way of reinforcement in this case has given the chance to reconsider when restoring and preserving historical architecture.

Recently, conservation of Western style architecture built of brick during the Meiji and Taisho periods (1868 ~1912/1912~ 1926) is increasing. Brick structures which are relatively new to Japanese architectural tradition were not as highly developed as wooden structures. As was the case with wooden buildings in Japan, brick structures have also been refined in a principle that conceals the reinforcement in the interior and façade appearance, using reinforcing bars, braces and trusses in the roof or floor framing. But reinforcement sometimes deprives the architecture of some authenticity, in which the historical meaning which shows the particularity of that period is inherent. Reinforcement and authenticity are often contradictionary.

When we decide what and how we must conserve, it is inevitable to define in detail what is authentic in the architectural construction, i.e., authentic style and authentic design. Yamagata's example, where the reinforcing structural steel is exposed, is in this sense remarkable. Planning the conservation and active reuse of historic monuments, we must primarily consider their "authenticity". It is very important to identify the historical value and meaning from various points of view. Suitable reinforcement should be decided according to these considerations.

Seismic Evaluation of a Brick Masonry Building of 1895

Comportement aux séismes d'un bâtiment en maçonnerie de 1895

Erbebenverhalten eines Backsteingebäudes von 1895

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1. Introduction

This papar describes the seismic appraisal of exisiting masonry building and the measures needed to ensure the structural meets modern Tokyo seismic requirements. Fig. 1 shows the first plan.

2. Response Analysis of the building

As the structural characteristic in the plan, X and Y directions are different, separate models were created for each direction (see Fig. 2). Each floor was assumed to consists of 5 lumped masses, connected by assumed stiffness for floor slab derived from the test-recorded stiffness value for the wall (see Part 1) . Thus vertically, the masses are connected by the brick wall stiffness value based on the shear modulus, and horizontally the masses connected by the floor slab stiffness having both shear and axial components.
The calculation models are shown Fig. 3.

By Comparing the buildings dynamic characteristics, the input seismic waves adopted for analysis were EL CENTRO (1940 NS), HACHINOHE (1968 NS), TAFT (1952 EW) and TOKYO (1956 NS).

The fundamental natural frequency of the structure was calculated as 5 Hz (approx.) and the peak value of input acceleration normalized to 200 cm/s^2. The base of the structure's foundation was assumed as fixed against rotation in consideration of the restraint provided by the soil and the soil's damping factor ratio assumed as 7%.

From the analysis, the maximum response anaylsis in the X direction was 561 cm/s^2 (TAFT), representing an amplification factor of 2.81, and in the Y direction was 610 cm/s^2 (HACHINOHE), an amplification of 3.05. (Table 1)

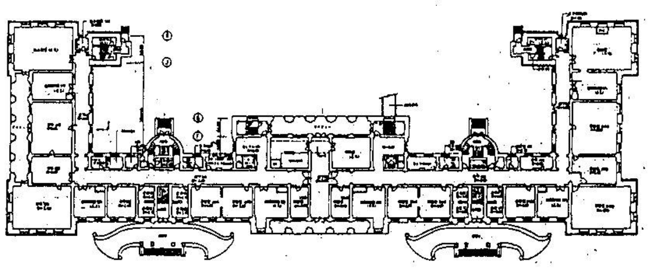

Fig 1 Building Plan (1st Floor)

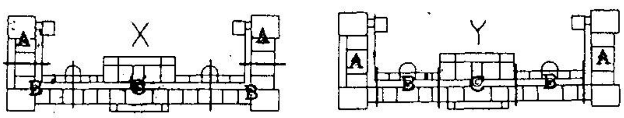

Fig 2 Building Sub-division for Modeling

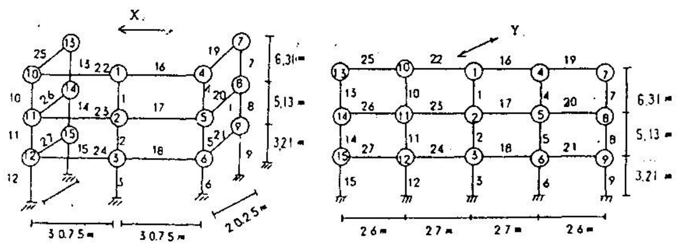

Fig 3 Building Model for Calculation

Table 1 Maximum Response of Mass Points

Max. Response		X- direction TAFT 1952 EW 200cm/s²						Y- direction HACHINOHE 1968 NS 200cm/s²					
Block	No.	DISP.	(TIME)	VEL.	(TIME)	ACC.	(TIME)	DISP.	(TIME)	VEL.	(TIME)	ACC.	(TIME)
C	1	0.33	(4.69)	11.1	(4.64)	406.	(4.60)	0.64	(4.16)	14.2	(4.10)	580.	(4.15)
	2	0.20	(4.69)	6.6	(4.64)	272.	(4.69)	0.46	(4.16)	10.3	(4.10)	462.	(4.16)
	3	0.08	(4.69)	2.5	(4.64)	202.	(3.71)	0.15	(4.16)	3.1	(4.10)	278.	(4.16)
B	4	0.37	(4.69)	12.4	(4.65)	451.	(4.69)	0.67	(4.16)	14.9	(4.10)	610.	(4.15)
	5	0.25	(4.69)	8.0	(4.65)	319.	(4.69)	0.43	(4.16)	9.6	(4.10)	438.	(4.15)
	6	0.09	(4.69)	2.9	(4.65)	205.	(6.55)	0.18	(4.16)	4.0	(4.10)	295.	(4.16)
A	7	0.47	(4.70)	15.7	(4.65)	561.	(4.69)	0.50	(4.16)	11.1	(4.10)	490.	(4.15)
	8	0.31	(4.70)	10.1	(4.65)	386.	(4.69)	0.34	(4.16)	7.5	(4.10)	389.	(4.15)
	9	0.10	(4.70)	3.3	(4.65)	214.	(6.56)	0.13	(4.16)	2.7	(4.10)	265.	(4.15)
B'	10	0.40	(4.69)	13.4	(4.65)	484.	(4.69)	0.63	(4.16)	14.1	(4.10)	587.	(4.15)
	11	0.26	(4.69)	8.6	(4.65)	339.	(4.69)	0.40	(4.16)	8.9	(4.10)	417.	(4.15)
	12	0.10	(4.69)	3.1	(4.65)	210.	(6.55)	0.17	(4.16)	3.7	(4.10)	286.	(4.16)
A'	13	0.47	(4.70)	15.7	(4.65)	558.	(4.69)	0.47	(4.16)	10.4	(4.10)	472.	(4.15)
	14	0.31	(4.70)	10.1	(4.65)	387.	(4.69)	0.29	(4.16)	6.4	(4.10)	355.	(4.15)
	15	0.11	(4.69)	3.3	(4.65)	216.	(6.55)	0.11	(4.16)	2.3	(4.09)	253.	(4.15)

3. Structual Assessmemt from Results of Response Analysis

Masonry allowable stresses are obtained directly from testing and divided by a safety factor of 1.5 for short term (seismic) conditions. (Table 2)

Maximum responses shear forces and average shear stresses, based on the 200cm/s² input acceleration, are shown in Table 3.

Areas exceeding the allowable stress are also indicated (mark *).

The stresses from the maximum response forces in the slab are in all cases less than allowable stresses.

From the results discussed, it was desided to reinforce those walls which were shown to be over stressed, by constructing reinforced concrete strengthning walls · connected by shear stud bolts to the existing walls. The maximum shear stress in the upgraded wall,which in all cases are less than the allowable stresses.

Regarding out-of-plane direction (perpendicular to masonry walls),shear forces based on the maximum response acceleration of inplane direction are adopted as the external forces to check the wall bending bearing capacity (Fig.4). By means of this calculation, at thin walls such as 380mm THK.,510mm THK. steel plates (3.2mm THK.) are installed at both sides of the wall surface to strenghthen flexural capacity.

Table 2 Allowable Stress (MPa)

		Testing Value	Short Term
Compression		6.0	4.0
Bending		0.15	0.10
Tension		0.15	0.10
Shear	3rd fl.	0.30	0.20
	2nd fl.	0.35	0.23
	1st fl.	0.40	0.27

Table 3 Maximum Shear Stresses in Wall

X-Dir	FL	Mem. No.	Weight (KN)	Shear Area (m²)	Sh. Force (KN)	Shear Stress (MPa)	
A	3	7	8480	20.7	4150	0.20	
	2	8	9650	24.1	7520	0.31	*
	1	9	8750	35.4	9500	0.27	
A'	3	13	8780	22.5	4390	0.20	
	2	14	10570	26.0	8040	0.31	*
	1	15	9720	37.6	10300	0.27	*
B	3	4	11360	34.6	5660	0.16	
	2	5	13820	41.3	10010	0.24	*
	1	6	11830	51.9	12990	0.25	
B'	3	10	11430	31.9	5610	0.18	
	2	11	14010	39.7	10040	0.25	*
	1	12	13060	49.7	13290	0.27	
C	3	1	26410	75.8	13240	0.17	
	2	2	33150	115.9	23730	0.20	
	1	3	29230	140.1	31330	0.22	

Y-Dir	FL	Mem. No.	Weight (KN)	Shear Area (m²)	Sh. Force (KN)	Shear Stress (MPa)	
A	3	7	14660	49.6	8700	0.18	
	2	8	16750	54.1	16070	0.30	*
	1	9	14990	73.3	20630	0.28	*
A'	3	13	15020	43.2	8510	0.20	
	2	14	18490	64.5	16180	0.25	*
	1	15	17200	87.9	21180	0.24	
B	3	4	10350	17.6	4740	0.27	*
	2	5	13440	29.7	10190	0.34	*
	1	6	11180	31.6	12790	0.40	*
B'	3	10	10370	18.2	4790	0.26	*
	2	11	12180	29.7	9370	0.32	*
	1	12	11160	31.6	11890	0.38	*
C	3	1	16050	49.3	9700	0.20	
	2	2	20340	43.0	18680	0.43	*
	1	3	18060	74.6	24220	0.32	*

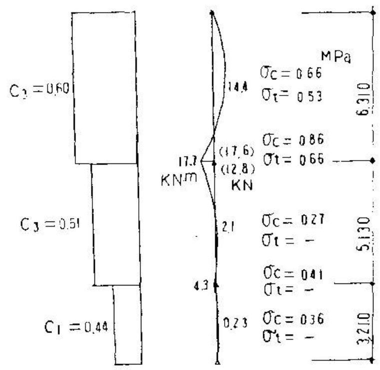

Fig 4 Bending Diagram Perpendicular to Wall

4. Conclusion

From the response analysis, it was shown the the natural period of the structure is 0.2 seconds as compared to 0.33 seconds for the surrounding soil. This large difference would appear to partly explain why the building didn't suffer any severe damage when struck by the Kanto earthquake.

Thus, structural stability is maintained for an input level up to 200 cm/s² at the ground surface. Further, if the ultimate strength is assumed to be equivalent to the material strength obtained from testing and some of the walls are upgraded as described above, the structure should withstand ground surface accelerations up to 300-400cm/s².

Despite the building's 100 years of age,it can be seen that this famous old building can remain in their masonry building for many years to come. This study also illustrates how masonry (or indeed other materials) can be engineered to create seicmic resistant structures.

Restoration of an Ancient Masonry Building after the 1990 Earthquake in Macedonia

Restauration d'un bâtiment ancien en maçonnerie après le séisme de 1990 en Macédonie

Instandstellung eines antiken Mauerwerksgebäudes nach dem Erdbeben von 1990 in Mazedonien

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1. STRUCTURAL SYSTEM AND STATE OF DAMAGE OF THE BUILDING

The analysed two storey residential building, dating from the beginning of the 20^{th} century, was one of the buildings to suffer most from the earthquake in Gevgelija in December, 1990, that struck the frontier regions between Macedonia and Greece (intensity VIII - IX).
The building is quite impressive itself, measuring 100 meters in length and 11.8 meters in width, with a decorated entry on the south side and a well preserved elegant roof wooden structure.
Its structural system is quite complex: at the ground level a cast in situ concrete slab connects the massive brick facade walls with a row of steel profiled columns at midspan; at the first level a wooden floor strucure is supported by the walls and the columns.
The partition walls are of different dating and origin: the recently introduced ones are made of concrete, while the original ones consist of wooden frames filled with compacted earth.
The structure survived the earthquake but was considerably damaged. The damage was concentrated at the first level and at the roof, the concrete slab at the ground level preserving the lower part of the structure from any considerable damage. In the upper part the partition walls have been seriously damaged, some have collapsed. The facade brick walls, although quite massive, have cracked, especially at the roof level. The wooden floor structure has been seriously damaged, and in places has collapsed. The gable walls have been dislocated at the corners and have cracked, especially at the roof level. This changes were detrimental to the integrity of the structure as a whole.

2. PROPOSALS FOR REPAIR AND STRENGTHENING OF THE STRUCTURE

The project for repair consists of partial grouting at the places of damage on the facade walls, respecting at the same time the original facade and wall finishes. At the corners where important cracking and dislocation have taken place, vertical cast in situ elements have been embedded. All the original partition walls have to be replaced by new modern walls, made of brick. The old wooden floor structure has to be entirely replaced by a new one.

As for the strentghening of the structure genuiting of the key perpendicular walls with a 5 cm layer of concrete has been proposed. Thus, a sufficient capacity of stength and deformability in the perpendicular direction of the structure can be achieved, the capacity of the structure in its original state due to the limited length of the walls (11.8 m.) being very low.The stiffening of the structure at the level of the first floor is to be provided with genuiting of all the facade walls, from the inner side between the celling and the lentels, a vertical distance of 70 cm. In such a way the wooden floor structure is going to be braced by a concrete belt all around the perimeter of the building. Futhermore, a thin concrete slab is to be embedded at the top of the structure, connecting the individual wall elements and thus contributing to the overall stiffness of the structure. With these measures a synchronized behaviour of the individual wall elements will be obtained, thus enlarging the posteleastic capacity of the structure and preventing its collapse.

3. STABILITY VERIFICATION OF THE RETROFITTED STRUCTURE TO A MAXIMUM EXPECTED SEISMICITY LEVEL

The structure in its original and repaired state has been modeled using a simple cantilever system fixed at the base, with masses concentrated at the two floor levels. With this mathematical model a dynamic time history analysis with different earthquake records has been performed . The level of the maximum expected earthquake action, as well as the different types of seismic records to be applied, have been determined with a special seismological study. In the analysis, certain ductility capacity has been allowed to the masonry walls (1.8 for the genuited perpendicular walls and 1.5 for the longitudinal facade walls).

Even though the maximum expected earthquake level is very high (42 % of "g"), a satisfactory response for the retrofitted structure has been obtained. The required ductility by the earthquake does not exceed the ductility capacity of the walls, in longitudinal and perpendicular directions.

The analysis shows that the retrofitted structure posseses sufficient capacity of strength and deformability - ductility to survive strong ground motions without considerable damage.

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Repair and Strengthening of a Medieval Brickwork Bell Tower

Réparation et renforcement d'un clocher médiéval en maçonnerie

Reparatur und Bewehrung eines mittelalterlichen Glockenturmes aus Mauerwerk

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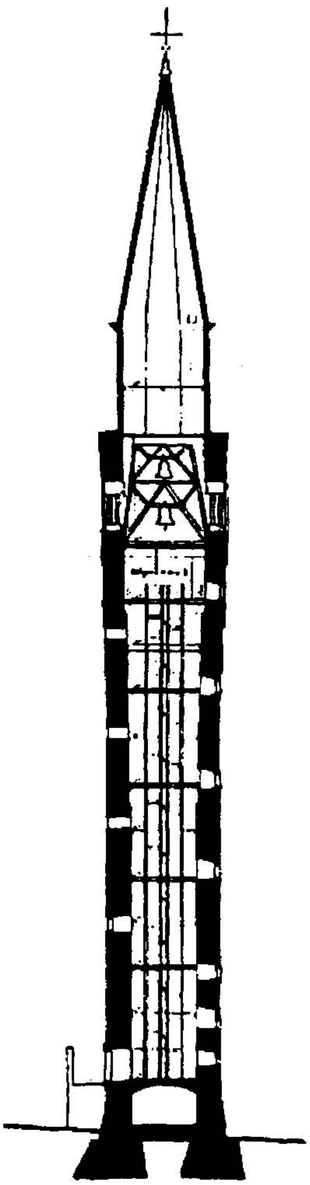


Figure 1

The described case study is the evaluation, repair and strengthening of the bell tower of the Cathedral of Pordenone, in Friuli, the Italy's region stricken by a strong earthquake in 1976. The structure, built during the last years of the 13th century, is 72 m high (Figure 1) and dominates with its imposing mass the historical center of the town.

The masonry walls are sufficiently strong and generally well constructed and preserved, and demonstrated to be tough enough to survive the earthquake.

Some major concerns about its actual safety level were however justified by the not negligible inclination (about 1%) and by evident traces of past heavy damages, often caused by lightnings, which were repaired between the end of the last century and the beginning of the present one.

The tower history and the repairs performed during the centuries have been found to be well documented.

In particular, pictures exist of the damages at the northern corner of the tower, which was near to collapse at the beginning of this century, and the inner part of such corner still presented damages and cracks in 1990.

The previous interventions appeared to be in general very efficient. Such are in particular the strong iron ties applied at five different levels and the reconstruction of the masonry in the outer part of the northern corner.

The preliminary investigations made in order to decide if new strengthening interventions were necessary were first of all based on the study of the existing documentation and on the accurate survey for detecting all the damages and cracks of the masonry walls.

Soil characteristics were investigated by spt tests and water table was monitored with piezometers.

The mechanical characteristics of the existing materials, masonry and iron ties, were then assessed by means of adequate in situ tests (flat-jacks for masonry).

Measures were also made of the dynamic response properties of the overall structure. Based on the obtained data FEM numerical models were constructed and used for analysing the structural behaviour of the whole structure and the relevant local states of stress in the masonry walls: in static conditions, during the bells motions and during the earthquake. The results showed the need of a strength improvement. Interventions were finally decided and executed aimed to improve the actual conditions of the structure without however substantial changes.

This was obtained first by reconstructing damaged masonry portions using traditional materials and techniques. In particular hand made solid clay units and mortars and injections made with hydrated lime mixed with "cocciopesto", which gives hydraulic properties to the admixtures, have been used.

Then, the inactive ties have been substituted and some new ties have been added. Finally, the confinement of the inner parts of the masonry walls have been substantially improved where an external confinement is already provided by the existing iron ties by means of five steel diaphragms in the positions indicated in Figure1. They are included in the system of the internal stairs as shown in Figure 2 a. Their principal characteristics are shown in Figure 2 b.

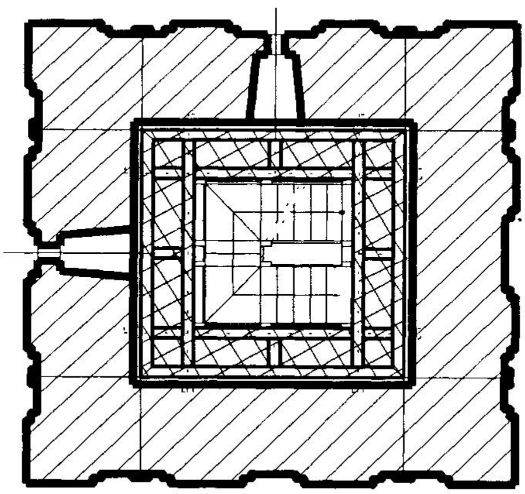


Figure 2 a: section of the tower at the level of one of the diaphragms.

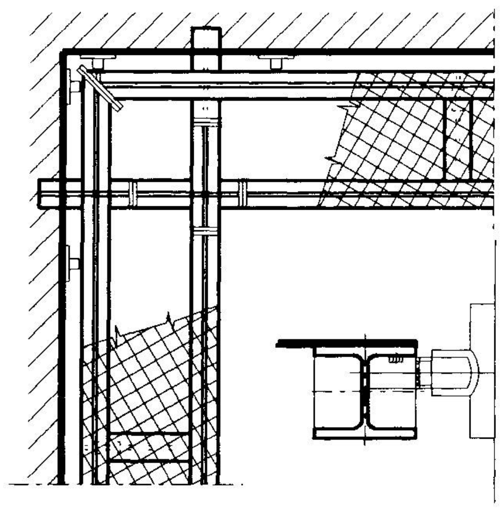
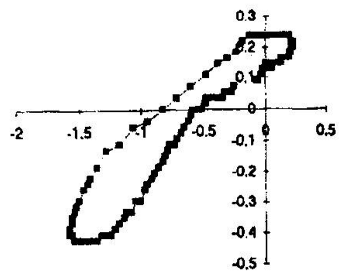


Figure 2 b: details of the steel diaphragms and of the devices used to put them in contact with the masonry walls.

The installation of the diaphragms inside the tower has been made possible by the use of several bolted joints. This construction technique allows the maintenance and, in case, the removal or substitution to be easily executed.



A monitoring system was finally installed to realise an automatic control of: tower inclination, cracks movements, temperature in the walls thickness, water table level and dynamic excitations.

In Figure 3 the daily variation of the tower inclination is shown, which is typically measured through the displacements of the pendulum, in sunny days.

Strengthening of Pisa Tower by External Post-Tensioning

Renforcement de la Tour de Pise par précontrainte extérieure

Verstärkung des Turmes zu Pisa mittels externer Vorspannung

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In 1991, VSL International Ltd. was entrusted with a study of the temporary circumferential prestressing of the Pisa Tower.

The job was to present a solution with temporary hoop prestressing tendons at the first level "Loggia" of the Tower. Those tendons were intended to prevent buckling of the masonry on the South Side of the Tower.

The idea was to wrap a number of prestressing strands around the circumference of the tower in the area of the first loggia as shown in Fig. 1. It was decided to place monostrands distributed over the height of the loggia wall. In addition, 8 monostrands were placed above the arches of the lowest level columns. The tendons were initially stressed to 50 % of their guaranteed ultimate strength. Very strict requirements with regard to the visual and functional effect on the tower were set:

- temporary tendons
- small visual impact on the tower
- no detrimental effects on the marble surface
- long term corrosion protection
- no grease in the monostrand
- strand overlength nicely hidden
- resistance against microorganisms and similar
- resistance against UV-radiation
- proven technology must be used
- stressing system insensitive to human error

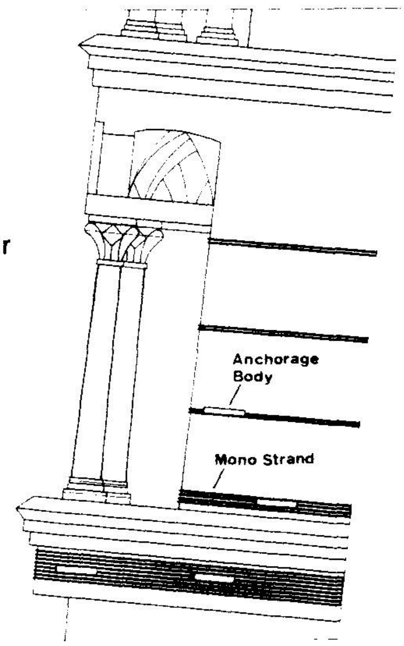

Fig. 1: Tendon Lay-Out at First Loggia

These requirements called for a design which allows the tendons to be force-monitored, retensioned, detensioned and removed. The chosen prestressing system does not interfere with existing materials. The tendons are finally held in place by their prestressing force.

The anchorage bodies were machined out of full material St 52-3 and then hot dip galvanized with a zinc thickness of min 80 μm.

The 7-wire prestressing strand according to Euronorm 138-79 is hot dip galvanized after drawing of wire with a zinc coating of min. 50 μm.
The strand is sheathed by a sleeve made of PVDF (Polivinylidenflouride), diameter 20/16.2 mm. This material is highly resistant against chemicals and weathering, and allows low-friction sliding of the strand in the sleeve inspite of the absence of any grease. The wedges were treated with a new corrosion protection method called "Dacromet 320", similar to hot-dip galvanizing but 2.5 times more effective for the same coating thickness. Because of the requirement that there must not be any interference with existing materials the anchorage had to be a "flying" anchorage, i.e. it slides with respect to the marble surface while stressing. A special PVDF pad was therefore provided between the anchorage body and the structure. This pad allowed low friction sliding, at the same time protecting the marble.

The anchorage system has minimized outside dimensions, thus causing hardly any visual impact on the structure. After stressing, the strand overlength is hidden in the anchorage body and covered by closing the lid as shown in Fig. 2.

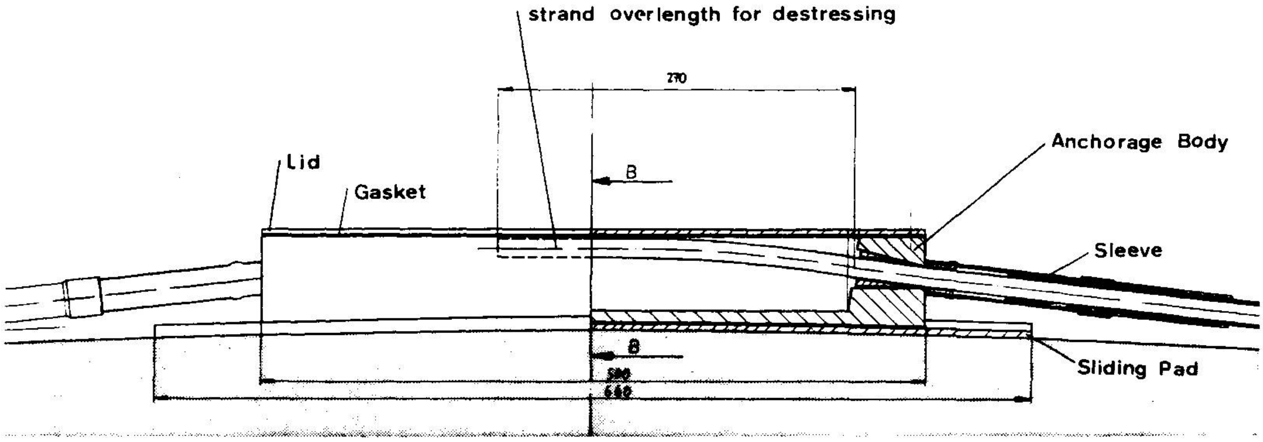

Fig. 2: Specially Designed Monostrand Anchorage Body on Sliding Pad

Although "proven technology" was used, the importance of the structure justified the execution of extensive testing. A full-scale stressing test was carried out on a circular silo structure with a radius similar to that of the Pisa Tower. The aim of the test was to demonstrate the adequacy of the entire system, in particular the anchorage body and the PVDF sliding pad. The experience gained during the installation and stressing confirmed that the special anchorage body, and the stressing procedures work as intended. The friction coefficient of the assembly of non greased strands was approximately $\mu = 0.11$.

The installation of the hoop tendons was executed in June 1992 by the VSL licensee PRECO.

Structural Reinforcement of the Cathedral in Cava dei Tirreni

Renforcement structural de la cathédrale de Cava dei Tirreni

Statische Festigung der Kathedrale von Cava dei Tirreni

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1. SUBJECT OF THE INTERVENTION.

The main building of St. Adiutore's Cathedral in Cava dei Tirreni is composed of a large broad nave, two side aisles, a transept and an apseidal area. The construction of the Church, lasted about 55 years, begun in the second decade of the XVI century. However, many subsequent interventions caused during the time a remarkable "static confusion" of the resisting elements which, added to the original high seismic vulnerability of the structure, led to the complete closing of the monument after the Irpinia earthquake of November 23, 1980.

2. DIAGNOSIS OF THE DAMAGES.

The main diseases were: deep fractures in the arches and in the vaults of both the side aisles, compression failure of some masonry pillars and a substantial foundation settlements in a wide zone between the principal facade and the right side aisle.

The masonry buttresses, built to contrast the seismic lateral loads, fulfilled their function during last earthquake event, but disjoined from the bearing walls and plasticized in the cross sections of lowest strength. The principal facade presented dangerous slipping surfaces and disjunction from the aisles walls, and the outbuildings were diffusely damaged. The wooden coverings were in a severely degraded state.

3. SOLUTION ADOPTED.

The proposed intervention therapy, aimed at the reduction of the building seismic vulnerability through retrofitting and strengthening the existing structures.

First of all, with reference to the central nave and the lateral aisles, it was decided to prop the fractured vaults and arches, to demolish the heavy masonry buttresses, and to disassemble the wooden covering.

Retrofitting of the existing masonry structures has been performed by:

- confinement of the damaged masonry pillars, using closed steel plates as stirrups and an external cover of spritz-beton;
- sewing of the arches fractures through insertion of radial steel bars and injection of grouting mortar;
- strengthening of the vaults through superposition of a reinforced concrete slab, connected to the vaults by means of glued steel nails.

The task of contrasting the seismic lateral loads is now performed by reinforced concrete frames effectively shaped, which are much lighter than the demolished tuff buttresses and have a much higher strenght per unit volume. These elements bear the new r.c. pitch roof of the lateral aisle, and allow a complete connection between the pillars-arches complex and the masonry walls of the central nave: r.c. and masonry are linked together by means of steel nails.

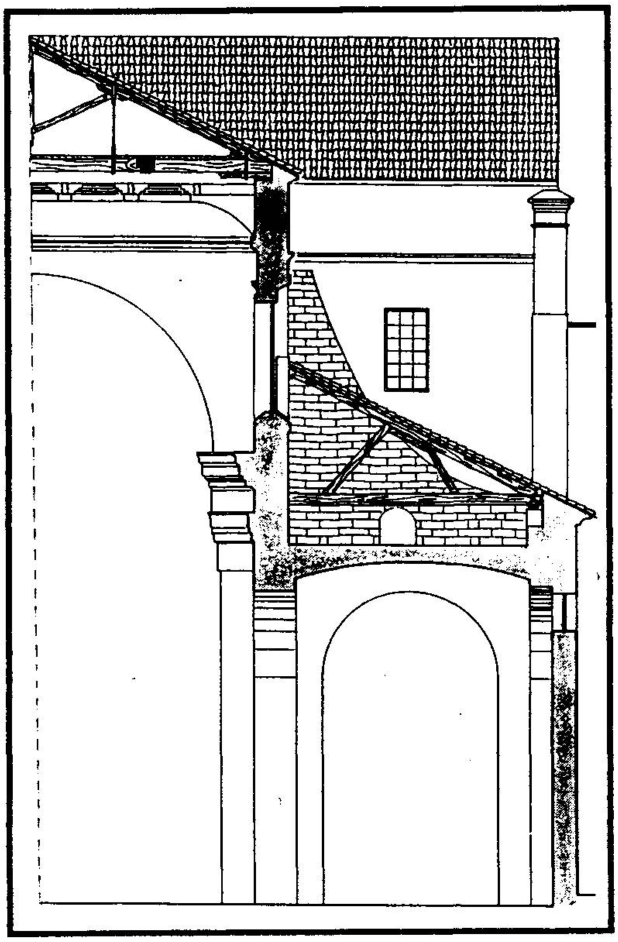

Fig. 1 Cross-section of the right side aisle before structural rehabilitation

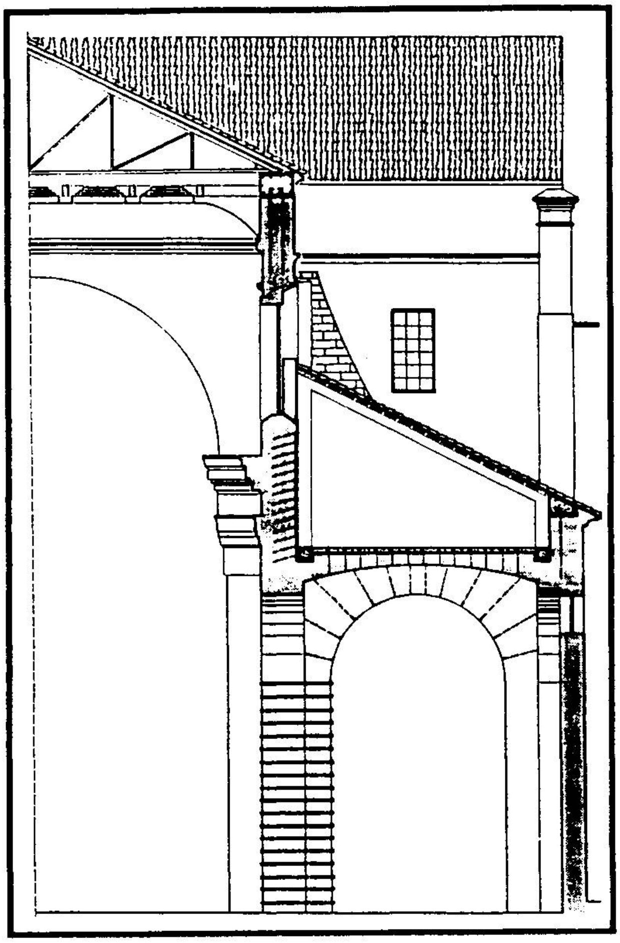

Fig. 2 Cross-section of the right side aisle after structural rehabilitation

A finite element model of the entire system has been developed, assuming that masonry elements are unable to resist tractions. An equivalent linear dynamic analysis has been performed, selecting the design earthquake time histories among the accelerograms recorded in the nearby stations during the Nov. 23, 1980 event. Results show a satisfactory reduction of the seismic vulnerability.

The new roof structures of the central nave consist of steel trusses, which were built assembling with bolts at the Cathedral site shop-welded elements, and then hoisting them onto the r.c. curb built on top of the existing walls.

The static recovery of the monument ended with the strengthening of the principal facade, performed through insertion of vertical and horizontal steel bars.

Strengthening of the Minutolo Chapel in Naples Cathedral

Renforcement de la chapelle Minutolo de la cathédrale de Naples

Verstärkung der Minutolo-Kappelle in der Kathedrale Neapels

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Historical notes

The Minutolo Chapel was probably built during the same period as the opposite Illustrissimi Chapel. Before the construction of the Cathedral was finished, it was decided to increase its size and importance by widening the transept and adding the two chapels to either side of the apsis.
The date of the frescos in the Minutolo Chapel, almost wedged in between the already existing structure of the right apsis, leads us to believe that it was therefore built after the Cathedral rostrum, though not after the last 10 years in the 13th century. Inside the chapel the underlying hypogeum is even clearer, the external buttresses and the lower base inclined towards the upper cornice of the siding and the square tower of the transept are evident.These walls, in squa re tufa blocks,show signs of a lenghty period exposed to atmospheric conditions. This leads us to believe that the Minutolo Chapel, already in use as originally designed on the aisle, was brought about after the construction of the apsis.
The inside of the chapel, built with a rectangular design with an ogivally crossed vault, ends with a polygon apsis added in the second phase of construction, which took place in the beginning of the 15th century. Even the small vestry is an addition, exploiting an area between the Cathedral apsis and that of the chapel. Nearly all of the pavement is decorated with precious marquetry with policrome marble inlays of the Cosmos. Throughout the various centuries the internal walls have been completely covered with frescos.

State of conservation and erosion.

As can be gathered from the history of the chapels' construction, the walls in the supporting structures were built in various periods and with various characteristics. The wall on the left and that of the entrance, is the transept pillar in the shape of a polygon, also in tufa blocks. The nave is roofed by two bay-vaults with crossed ribbing, while the apsis is formed by a large gothic style arch and a semivault with ogival ribbing.
On the left wall, the irregular external apsis buttress is hidden by part of a wall situated between the two pillars and the cornice of a depressed arch. A small chapel has been built under this arch.
The above floor is vaulted in tufa stones with a strongly depressed curve that is thin and flat at the extrados, almost like a "plate". Considering the use of

small stones of an excellent quality, there are signs of considerable erosion. The technical solution used in the past was that of a supporting element in relation to the crown by a large pillar in stone masonry.
Two large brick arches were then added, converging towards the centre of the pillar, creating a larger support area in the vault. About 30 years ago, in the hope to block the increasing fissures, it was considered necessary to increase the width of the pillar (by at least 4 square meters), without considering the need for a suitable foundation for this heavy structure.
In this period, structural work was also carried out on the previous pillar that had obviously pulled away from the vault intrados trhough sinking. The foundations for the converging walls in the south corner were also realized.

Intervention for consolidation.

The problem concerning the static restoration of the main structure of the Calpestian vault in the chapel, was formed with the birth of the actual vault due to its dimensions and form. The bulky supporting walls, built in the past to the hy pogeum quota, have continually proved to be not only insufficient, but also damaging by creating ad excessive weight to the supporting ground, thus causing sinking.
The object of this restoration, carried out by the Superintendant for the Architectural and Environmental Treasures of Naples, and Suburban Areas, is to eliminate the enormous pillars and to use a crypt method that will allow the chapel to be opened to the public and, with time to ensure the definitive strenghtening of the vault. Apart from being a piece of particular architectural work with precious mosaics of the Cosmos, it is without a doubt a significant structural element in the original building.
The technical solution used was that planned with the advise of Eng. Bruno Pandolfi and Paolo Falasca. This method created the supporting structure (for both above and below ground) by a frame of metal beams, positioned on the shortest side of the chapel, and interconnected at the right-angles by another series of steel sections placed according to where they are required. These steel sections were welded to the inferior sides of the beams and supported the vault by stays bolted above and a distribution plate at the intrados. The use of this metallic frame, limited in width and disposition, has given both support to the fissured vault and to an overload equivalent to that determined by overcrowding, (a rare occurance but not to be excluded). This solution has been positively valued, because it does not detract from the architectural image and has provided a quick and easy execution. The last phase was to progressively demolish the preceding reinforcement structures in the crypt. This operation was carried out by evaluating and controlling, with the use of a computerized monitoring system, the eventual static alterations in the vault structure.

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Restoration of the Altemps Palace in Rome

Restauration du Palais Altemps à Rome

Restaurierung des Altemps-Palastes in Rom

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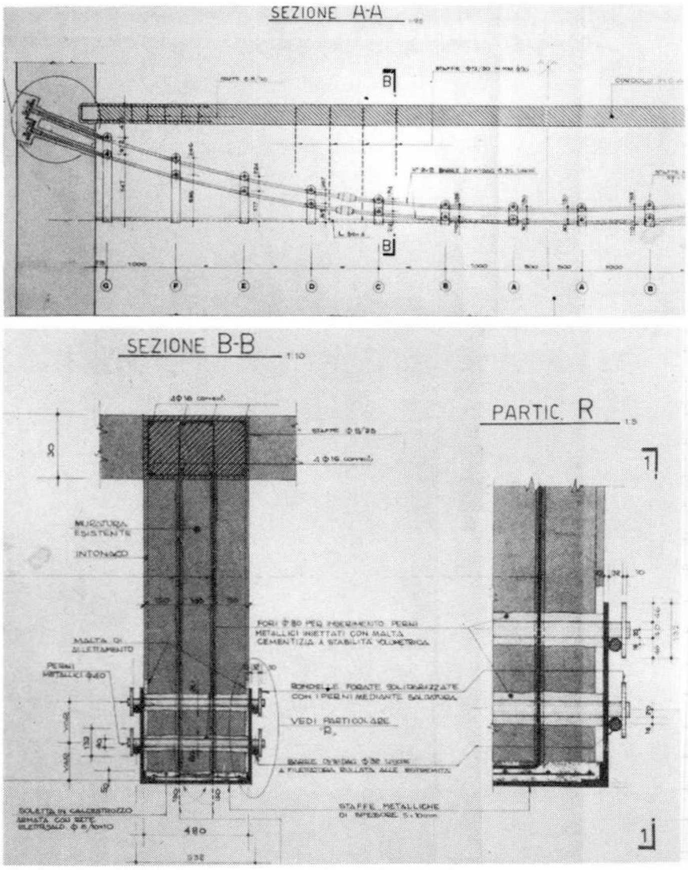
1. INTRODUCTION

The restoration of this historic, medieval-renaissance, block in centre of Rome involved the work of different disciplines: from Engineering to Architecture, from the History of Art to Archaeology. The project of structural consolidation was carried out by Prof. G. Croci, of the University of Rome with the collaboration of Engineer M. Biritognolo.
The problems of restoration and consolidation were tackled with an inter-disciplinary approach.
In particular, the structural intervention concerning the general restoration of the building, was preceded by an analysis using a mathematical model. Besides this there were a series of local analyses and interventions carried out on single structural elements.
It is important to specify the artisan nature of the restoration works, which is present in the individual details as well as in the global approach.
Some local structural interventions rise to a particular importance from a structural point of view. These interventions are related with the strengthening of structural elements (to assure the bearing capacity requested for the future function of the Palace) whilst preserving of the original structural and architectural typologies.
A monitoring system to watch the behaviour of a strengthened masonry element was also installed.

2. LOCAL INTERVENTIONS

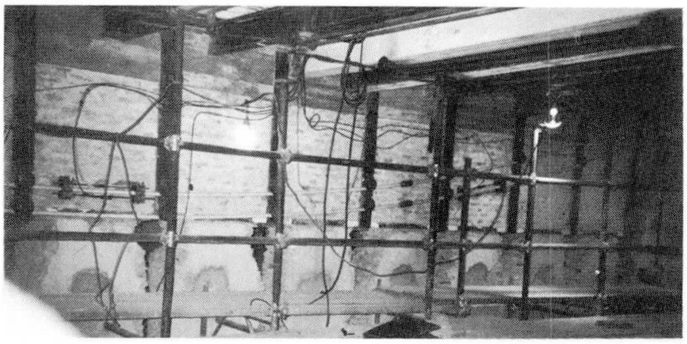
2.1 Strengthening of the masonry walls and of the floor structure in the hall

Three adjacent rooms separated by two internal walls, bearing the upper floor structures, had to be enclosed to obtain a hall of great dimension, originally named "Sala delle feste".
The intervention carried out in each one of the two walls was the demolition of the lower portion and the creation of a masonry beam supporting the upper floor.
This beam is composed by a wall panel having height of about 2,00 meters and span length of about 13.50 meters; over this beam, at the level of the upper floor, a concrete beam was cast (pic. 1, 2 and 3).
Four parabola shaped diwidag bars, (two at each surface of the wall), were placed as reinforcement to ensure the required bearing capacity.
A similar intervention was carried out on the principal structure of the upper floor of the hall. This structure is composed of wooden or steel beams. (see pic. 7)

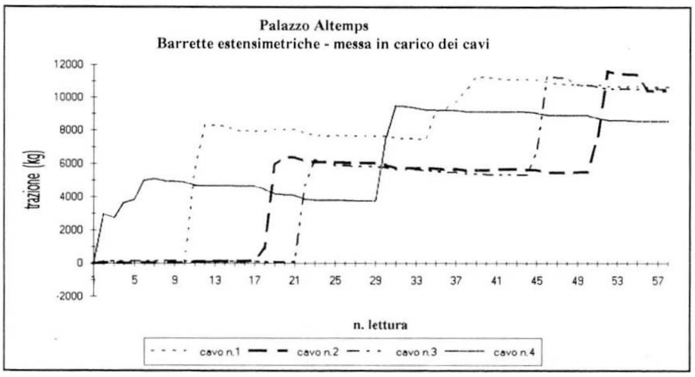


Pic. 1 and 2

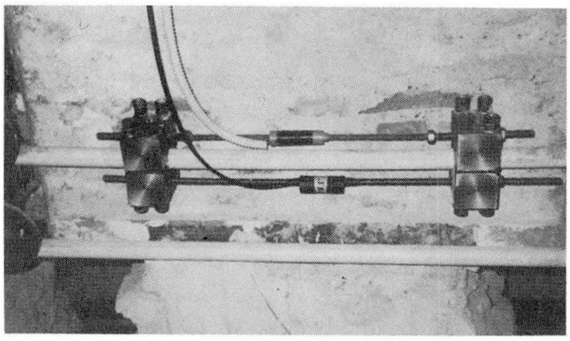
A monitoring system was installed to observe the values of the tensile forces in the dywidag bars in the principal masonry beams during and after the works.
Four pairs of strain guages (pic. 4) were attached to the bars, as well as a thermal sensor to measure the temperature variations, which permitted the control of the structural behaviour, comparing it with the theorical analysis (pic. 5).



Pic. 3



Pic. 5

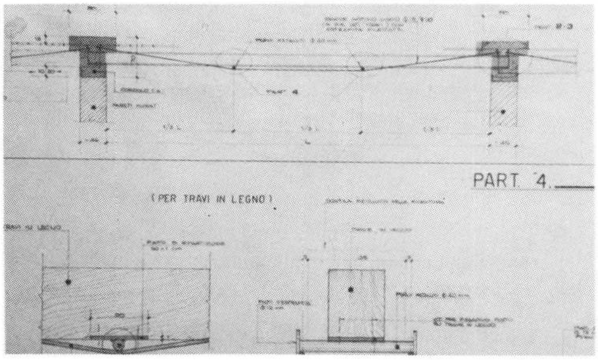


Pic. 4

2.2 Strengthening of the timber beams supporting the floor of the "Perspective Hall"

The floor of the "perspective hall" is supported by two timber beams, over these there is a secondary wooden structure bearing floorboards and an upper paving.
The bearing capacity of timber beams had to increased according to the foreseen utilization of the hall.
In order to maintain the original timber elements, a load text and an extensive examination of their state of conservation were performed.
The increase in bearing capacity was obtained by a steel section placed over the principal beam within the height of, and connected to, the secondary structure.

Pic. 6



Pic. 7

2.3 Reinforcement of some masonry panels with external steel bars

These interventions shown in the photo 6, allow the avoidance of any interference and damage the reinforced structure.

Strengthening and Control of the Dome of Vicoforte Sanctuary

Renforcement et contrôle de la coupole du sanctuaire Vicoforte

Verstärkung und Kontrolle der Kuppel des Vicoforte-Heiligtums

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The elliptical masonry dome of the Sanctuary of Vicoforte near Mondovì, Italy, built in 1731 is the largest of its kind (major axis "37,15 m, minor axis 24,80 m, maximum height of the monument 84 m) and is in absolute the fith largest dome in the world. The original project of the monument is due to Ascanio Vitozzi (1539-1615) who was responsible of the early part of the construction.
The unfortunate selection of the site from a geothechnical view point is responsible of the structural damages that the monument suffered throughout his life. Only the north-east section is founded in fact on sufficiently consistent marls, whereas the remaining parts of the monument, and in particular the south-western sections, rest on compressible clay-silt layers of variable thickness (up to 3÷3,5 m). The monument was therefore exposed to the effects of large differential setlements during the various phases of its construction and life. Construction itself - started by Vitozzi in 1596 - after continuous compensation of initial settlements during the construction process and the establishment of a drainage system of clay layers, was practically abandoned at elevation 11,10 m in 1600, with a slow prosecution untill el. 19 m during the whole XVIIth century.
Architect Francesco Gallo (1672-1750), after new levelling of structures to compensate further settlements due to progressive consolidation of clay layers, as well as to unsufficient maintenance of the drainage system, started again construction works in 1701 and completed in 1731 the daring large elliptical dome, inspite of a negative opinion expressed by Filippo Juvarra asked for consultancy. New settlements due to immediate and delayed effects of large added loads, magnified in time by recurrent lack of maintenance of the drainage system, were responsible in the following centuries of statical disorders with the appearence of large cracks in the dome and in the lower parts of the monument. Maximum differential settlements of the west-side foundations with respect to the north-east side, developed during the whole history of the monument, were estimated in 1962 to be the order of 55 cm. Maximum amplitude of cracks measured at the base of the dome was 82 mm with a total amplitude of 413 mm on the dome perimeter. Total increment of major west side cracks (extending from el. 14 m to the top of the dome) was of 14 mm in the period 1935-60.
A monitoring, rehabilitation and structural strengthening program was started in 1976 with the following objects:
- consolidation and stabilization of foundations,
- structural strengthening of the dome through the formation of a post-tensio-

ning ring at the base of the drum,

- monitoring of the principal parameters characterizing the structural disorder and of the response of the monument after strengthening (with particular regard to the time-dependent stress response of the post-tensioning ring).

The post-tensioning ring is formed by 14 interconnected tangential tie-rods, each one consisting of 4 prestressing bars hidden in the drum masonry tensioned to a convenient limited fraction of the calculated circumferential stresses in the dome.

Diagrams of the variation in time of the stresses in the bars over 5 years (as obtained by the monitoring system) have been analysed and a separation between the relaxation of the stresses in the bars (due to plastic flow of the masonry) and parasite stress effects due to the differential thermal dilatation of masonry and steel bars has been tried. Maximum value of mean relaxation of stresses in the 56 tensioning bars has reached the order of 20% of initial values (stress decrease from 50 to 40 KN per bar) in the four years following initial tensioning. (fig.1)

Correlation of these data with monitored movements of the main cracks of the dome leads to the conclusion of the need for a recalibration of the stress levels in the post-tensioning system to be performed in 1993. The need of more reliable data on the internal temperatures in the dome masonry has led to a program for measuring and monitoring these parameters to be operative also in 1993.

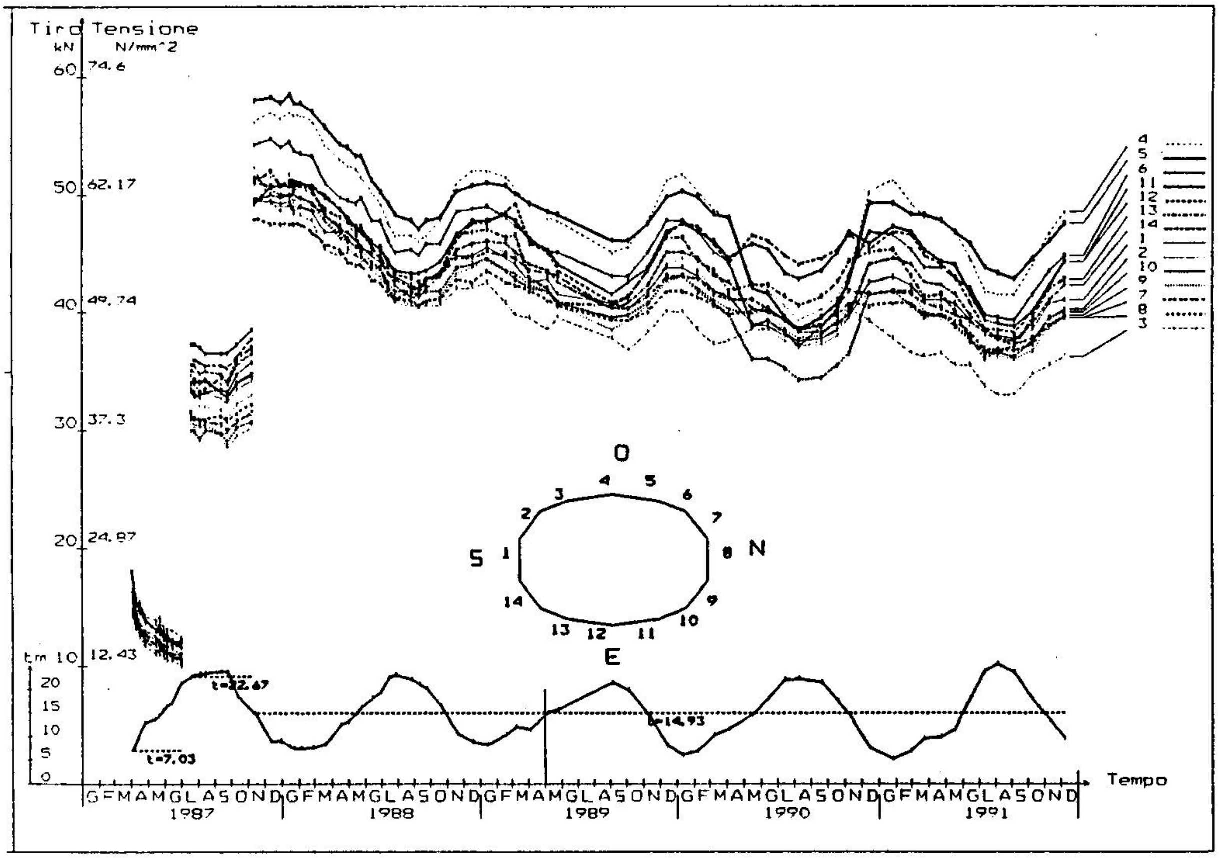

Fig.1 Variation in time of forces and stresses in the post-tensioning ring (mean values of 4 bars for each of the 14 sections of the ring) and variation of inside temperatures.

Conservation of the Lions Court at the Alhambra of Granada

Conservation de la Cour des Lions à l'Alhambra de Grenade

Erhaltung des Löwenhofs in der Alhambra von Granada

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1. INTRODUCTION

Decay studies on marble artifacts, relative to different architectonic structures of the Arab Palaces in Alhambra, show that the severe deterioration processes have been primed by static displacements. It is evident in the Court of the Lions [1], in which the environmental thermohygrometric influences have an important, [2] and [3], but secondary part on the decay evolution. Weathering processes weaken, above all, the marble artifacts already affected by micro-cracking conditions [1].

2. MARBLE STRUCTURES

2.1 Material

The Macael marble, widely used in the monumental complex of the Alhambra, is an anisotropic material both in texture and physical-mechanical properties, as the relative petrofabric analyses and investigations of the directionality of certain physical and mechanical parameters (Vp, E, σc and σt) have verified.
This anisotropic behaviour and its relative magnitude have been preliminarly determined in laboratory on correlated quarry and monument marble specimens (from working scraps of the "fuente de los Leones" unexposed copy, and from damaged original artifacts, removed during ancient restorations).

2.2 Colonnades in "Patio de los Leones"

On-site valuations of the physical-mechanical behaviour and decay evolution on marble artifacts (under external influences and loading stresses) have been based on the following non-destructive investigations: i) Textural orientations of the single structural elements, related with their specific geometrical conformation, working and laying. Generally the foliation plane, mechanical weakness plane, lies horizontally for base, capital and architrave elements, while it is in vertical placement for the shafts. ii) Ultrasonic pulse velocity measurements, to value the relative decrements due to incipient or moderate and severe decay.
The whole of the Macael marble artifacts, in different structures of the Arab Palaces, show a satisfactory conservation stage, with some exceptions: e.g., colonnades in "Patio de los Leones"(figure 1).
Selective decay evolution, relatively to columnar structures in the Court of the Lions has been verified

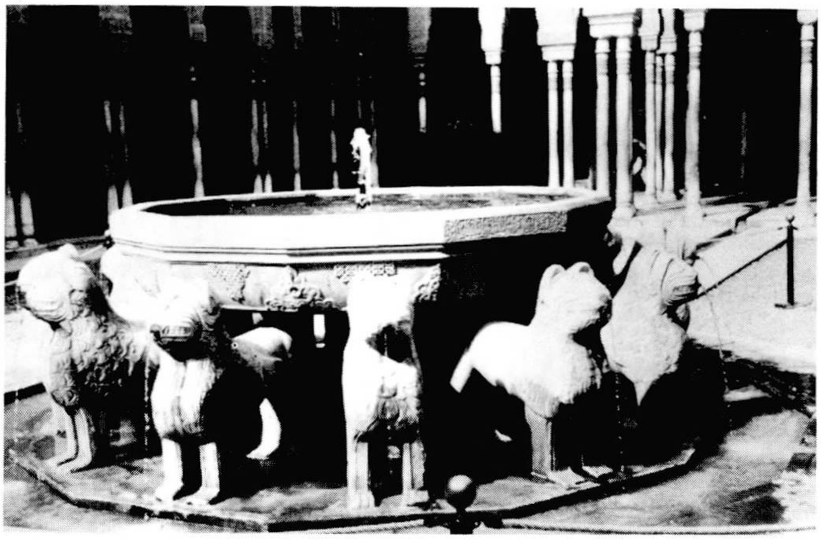

Fig. 1 "Patio de los Leones", Alhambra.

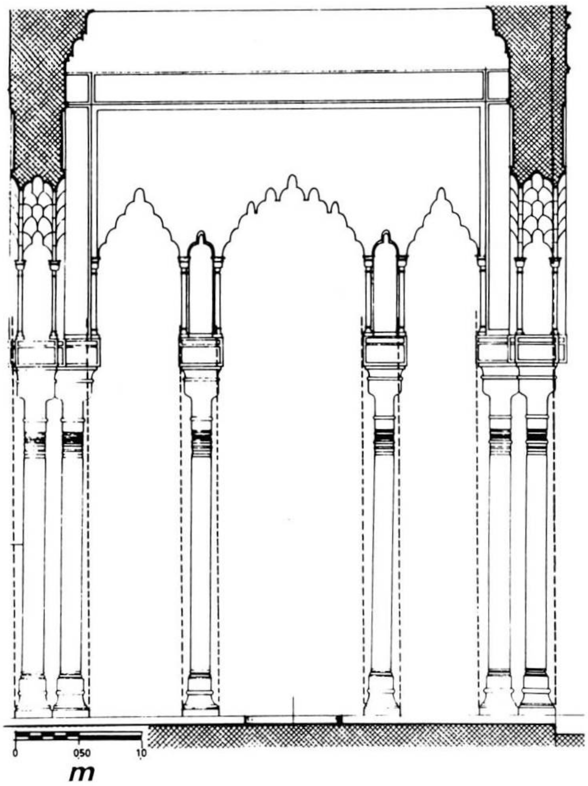

Fig. 2 Static situation, drawn in 1976, for "Templete de los Mocarabes", Court of the Lions.

Fig. 3 Progressive reduction in height of bases, and damaged shafts according to mechanical weakness plane.

by direct correlations between the increases of the apparent decay degree and total anisotropy index, measured in marble artifacts. Moreover, this evolution has been related to: a) rotation of the foliation plane, consistent with the shaft placement, influencing the durability of the artifacts under the same exposure conditions; and b) evident displacements of the original static conditions (figure 2), and new signs of differential settlement and plumb-line diversifications in severe damaged shafts, recognized particularly in the "Galeria de Dos Hermanas" colonnade (figure 3).

3. CONCLUSIVE REMARKS

A convenient control on stability conditions of the colonnades appears useful to found the adequate maintenance interventions, before carrying out replacements of damaged marble artifacts and/or protective treatments.
(This research was supported by contract EV4V-0108-I from the Commission of European Communities, and Research Group N° 4065 of the "Junta de Andalucia" Government).

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Structural Preservation of St. Paul's Facade at Macau

Conservation structurale de la façade de St. Paul à Macao

Strukturerhaltung der St-Pauls-Fassade in Macao

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1 - INTRODUCTION

The St. Paul's Facade at Macau is part of a church whose construction was concluded in 1644. Its architect was the italian jesuit Carlo Spinola who leaded a team of christian japanese workers.

The facade built with pink granitic stones is very imposing due not only to its dimensions, but also to the fact of being marvellously sculptured with decorations both in western and eastern style.

The church, whose interior (columns and ceiling) was made of wood, was destroyed by a fire in 1835 and only the stone facade remained. The fire deteriorated some stones namely those placed around the window frames of the facade. So, afterwards, these frames were strengthened with brick arches placed in their interior.

In 1935 a general restoration campaign was carried out and most of the joints between stone blocks were refilled with a strong cement mortar that covered part of the block edges contiguous to the joints.

By 1990 the facade had the following pathology:
- displacement of some stones from their original positions
- infestation by plants rooted in the joints between stone blocks
- bad drainage conditions
- dirtiness due to traffic pollution, rain water and "graffitti".

Thus the Macau Municipality and the Macau Cultural Institute formed a joint-venture in order to promote the following actions for the facade rehabilitation:
- stability studies
- preservation works

In order to valorize the monument and to establish a future museum on site an archaeological search was carried out in order to find the remains of the interior foundations of the original church.

2 - STABILITY STUDIES

Studies were made in order to evaluate the facade stability conditions.

First a radar search was undertaken in order to evaluate the soundness of the facade stone blocks and to verify if there were voids between them.

Then a survey to the foundations bottom was made by excavating inspection pits.

Finally, penetrometer tests were executed in order to quantify the soil capacity bellow the seating level of the foundations.

With all the collected data a stability analysis was made mainly for the severe local wind conditions (typhoons). The strongest wind ever recorded in Macau was in September 1964 (typhoon Ruby) with peak velocities of 211 km/h. The "Code of Practice on Wind Effects" of Hong Kong, edited in 1983, prescribes wind preassures that correspond to peak velocities of 214 km/h.

The conclusions obtained from the referred studies were:
- a strengthening of the foundations should be executed since only one half of its width was in contact with the granitic bed-rock. A solution with micropiles was proposed
- the resistance of the superstructure is sufficient provided that actions to correct the detected pathology are undertaken.

3 - PRESERVATION WORKS

The preservation works carried out were mainly:
- replacement of displaced stones using hydraulic jacks. After reaching the convenient position stones were kept by using stone wedges thus allowing the removal of the jacks before the refilling of joints with mortar
- removing of the existing vegetation by using adequated chemical produtcts (herbicides)
- repair of damaged joints between stone blocks using lime mortar. Particular care was taken in order to place the mortar withdrawn from the vertical plan of the facade
- improvement of the drainage conditions by correction of existing mortar slopes and execution of new ones
- hand cleaning of the facade mainly using low pressure water and plastic or nylon brushes. In the case of greasy stains detergents were also employed. For the removal of the "graffitti" some organic solvents (chloride of methylene, for instance) were required
- removal of the exceeding part of the cement mortar placed in the joints during the 1935 restoration campaign. This was made by cleaning the joints to a depth larger than their width with a minimum of 25 mm. Metallic hand tools, as scratchers, hammers and chisels or wire brushes, were employed carefully in order to avoid damages on the granitic blocks. After the excessive cement mortar was removed the joints were just cleaned with compressed air.

Preserving Frescos while Substituting the Wooden-Beam Floor

Conservation des fresques lors du remplacement de planchers en bois

Erhaltung der Fresken während dem Ersetzen des Holzbodens

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In many old buildings of the "Ancient Centre" in Naples, the false ceilings carried out to mask the intrados of the wooden floors, were usually vault shaped and painted in fresco, in order to give greater solemnity to entertainment rooms and greater prestige to the owuer. The usual shape was a "gaveta" vault that gives larger plane surface. The false ceiling, originally, was made of reeds ("canne") with a wooden trellis, from which derive the Italian term "incannucciata", then, in the course of time, the reeds were replaced with canvas. The wooden trellis was fixed to the wooden beams of the floor through little wooden bars, which was a remaining of the wooden trellis working. The trellis was usually made of chestnut.the section of the wooden elements, which constitued the trellis, was from about cm 3 x 3 to about cm 7 x 7, even if the section usually employed was about cm 3 x 5. The reeds or the canvas were coated with gesso and painted in fresco. The interstice resulted from wooden floor and the "incannucciata", to present that the latter went rotten, was ventilated through air intakes, called "ventarole", that shut by wire gauzes, connected the several rooms among them and outside.

Nowadays the engineers often are asked for giving their opinion on the stability of these old wooden floors. The problems of diagnosis of impairment due to the impossibility of access to the floor soffit because of the "incannucciata" are omitted in this paper in order to speak about the possible intervention to make on the floors once they are considered unsafe. To make this procedure clear, the traditional thecnics of construction of the wooden floors in Naples is described briefly. The floors were made of raw beams, usually of chestnut, leaned on longitudinal walls for a lenght not less than 1/20 of the span. The distance of the floors from one axis to another was 80 - 90 cm. On the beams, in the zone included between two adjacent beams, called "valera", some halfround pieces, usually made of chestnut, called "panconcelli" or most generally "chiancarelle" were put orthogonally and in contact. On these "chiancarelle", a layer of scraps or masonry debris ("sfabbricina") mixed with scanty mortar was laid. This layer is called "riccio". A layer of good mortar or a light mix of lapillus, which formed a sort of cover, called "masso" was spread on the "riccio". On this "masso" the proper flooring is placed. Once the floor must be replaced, the choice inevitably fall on the steel beam floor. It is made of steel beams, hollow flat tiles and a light concrete filling casting. The beams are connected together with a net, o 8 with a mesh of cm 20 x 20, soldered to the beams.Everething will be completed by a cm 4 concrete slab casting. The choise is inevitable for a different reasons. First of all, the lightness of this kind of floor, next its pecularity of having, like the wooden floor, points of suppot, and finally, as we can see latery, the possibility of assembling it into parts connecting it to the "incannucciata".

After making the dimensional choices and reducing the members with the well known methods of the Science and Technics of costruction, we examine the operative phases of sustitution. The first phase consists in removing the flooring, next the "masso" and finally the "ricco" in order to lay the "chiancarelle" bare. This is the most dilcate phase, because, generally the "chiancarelle" are very damaged, so any sharp movement or the use of wrong tools, can give rise to the fall of "sfabbricina" on the "incannucciata" with the its resulting damage. Thus the operation must be performed with great care using only bush-hammer and trowel and not other tools, such as shovels or picks. To preserve the "incannucciata" the floor should not be uncovered as a whole, but only a "valera" at a time, while at the beginning, two at a time in order to have two spans opened. After this, the "chiancarelle" should be removed. The "incannucciata" could be very damaged, in this case a skilled restorer should be called. Nevertheless if the trellis and the canvas, or reeds, are in good conditions, the only advisable operation is to stiffen the connections between the reeds or canvas and the wood. At the vault extrados, the connection of the reeds to the trellis takes place sticking strips of glass fibre cloth, firstly impregnated with bicomponent exposy resin, or more economically, using strips of cloth sticked with no watery glue. Before removing the beams it is necessary to create temporany supports for the beam and to protect the canvas from possible small masonry debris and above all from the water casting. The temporany supports for the canvas could be made with wires fixed on bars orthogonally placed to the floor frame. The simpler and cheaper protection of the canvas is to put first a plywood sheet on the trells, second cardboards and papers and finally a sawdust layer. To remove the beams it is necessary to widen the hole of housing to sling the beam and to saw in two parts. Now the wooden beams is replaced with the steel beam. The profile used is generally a hot rolled steel Fe 360 section IPE or NP. The profile must not be leaned directly against the tuff masonry, because, obviously, the pressure concentration could break the support stone. That's why a concrete or solid bricks bearing is created to support the beams. The support of the beams must be about 1/20 of the span and never less than cm 15. After placing the beams, the trellis, which support the "incannucciata", is connected to them. The connection, in the case of particulary deformed "incannucciata", is made with steel wires and thread-tensioner which will be used to settle the vault again. But if the vault is in good condition it is sufficient to bring back the connection into use through zinc plated steel wires tied to the trellis and the beam. Then the hollow flat tiles are placed and the holes of housing are sealed with a sand and cement mixture. When the concrete is set, the completing concrete is cast. After the setting of the last one, the plywood steels are removed, the cardboards and the sawdnust are sustituted and the operation for the next "valera" is repeted. At the last bay the cardboards and the sawdnust are not employed, except the playwood sheets which will not be got back after casting. After completing the last span, the electro-saldered net is placed and the slab of cm 4 is cast, taking care of not mixing the concrete with the surplus water compared to the stoicheiometrical necessary water in order to avoid damp stains on the "incannucciata".

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Controlling Methods Applied to a Roman Dome to be Restored

Méthodes de contrôle pour une coupole romaine en cours de réparation

Kontrollmethoden für eine römische Domkuppel

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1. BACKGROUND

The Temple of Romulus, named in honor of Romulus, the son of the Emperor Maxentius, was built in 311 AD on the site of the Temple of the Penates, which had been torn down to make way for the emperor's great basilica on the Via Sacra in the Roman Forum [1]. Much of the original building is extant, including a cylindrical entrance hall, approximately 15 meters in diameter, topped by a semicircular dome, the main hall, and two smaller halls, which, however, are in ruins. In the 6th century, the temple was transformed into the atrium of the church of Sts. Cosmas and Damian. In 1631, a Baroque vaulted ceiling was added, which is sustained by the outer drum walls and inner pillars at the church floor level. During this campaign, the temple was seriously damaged when a communicating door was opened to connect the rear of the church to the temple dome area.

2. PRESENT STATE OF THE DOME

The static condition of the dome is poor. This is the result of several factors: the collapse of the smaller halls, which laterally sustain the pressure of the dome; deterioration wrought by time; the 19th century excavations in the Forum; and the various modifications made to the building.

An examination of the cracking patterns has revealed the static and mechanical changes undergone by the dome over the centuries. The building contains serious lesions, which start at the top of the dome and continue downwards, almost vertically, to the ground. Moreover, the drum walls are rotated outward, visibly diverging from vertical. A major lesion, which from the arch keystone propagates throughout the dome up to the lantern, is the result of the construction of the communicating door. Also, the vaulted ceiling may also have contributed to the poor condition of the whole.

3. RESTORATION AND STRUCTURAL REPAIR OF THE BUILDING

Restoration of the Temple of Romulus is being undertaken by the Soprintendenza Archeologica in Rome, in conjunction with the Soprintendenza ai Beni Ambientali e

Architettonici. The project will entail restoration of the domed hall and demolition of the Baroque vaulting. The church communicating door will be left. An automatic monitoring system will be installed. In view of the precarious state of the whole, special care will be taken in the removal of the vaulting. To limit the effects of the modification, a temporary external encircling of the dome will be added at drum level. The encircling could be made permanent, should the results of the automatic monitoring warrant.

4. NUMERICAL FINITE ELEMENT METHOD USED IN RESTORATION

The mechanical history of the building will be determined as part of the forthcoming restoration project. A finite element model has been developed to accurately duplicate the geometry of the structure. Using a numerical analysis carried out on the virtual model of the integral structure, the static state of the building in its original geometry has been investigated. This has made it possible to determine the mechanical consequences of the evolutions from modifications (the church communicating door, the vaulted ceiling), structural cracking, and the collapse of various elements. Hence, using structural identification methods, we have been able to arrive at an exhaustive representation of the monument's present cracking state. This has entailed:

1. Carrying out tests for determining the stress state using flat jacks and assessing the deformability and resistance of the walls

2. Developing a permanent monitoring system to check the progress of crack propagation and structural movements.

The finite element numerical model together with mechanical measurements has enabled diagnosing the building's physical condition. The numerical model has given indications regarding the effects of removing the vaulted ceiling and the encircling system on the damaged structure. In simulating the some encircling system, beneficial effects have also emerged with regard to the existing state of cracking.

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Reconstruction of Bridges in the Historical Centre of St. Petersburg

Reconstruction de ponts dans le centre historique de Saint Petersbourg

Wiederaufbau von Brücken im historischen Zentrum von Sankt Petersburg

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1.INTRODUCTION

St-Petersburgh, founded in 1703, occupied the territory in the mouth of the Neva river, which included about 40 islands, separated by rivers, channels and canals with their total number now reaching 86. Due to such situation, the builders had to erect bridges for connection of different parts of the new city from the very beginning. First of them were wooden-made and don't exist. First stone bridges appeared in 1760ies and some of them are still working. Later on first russian cast-iron, steel and concrete bridges were built in St-Petersburgh, forming distinctive outlook of the city's centre, contributing much to it's graceful silhouette.
For the last period of time city authorities have faced the necessity of reconstruction of the bridges, built between 1760 and 1916, because they were no longer capable to withstand the needs of transport from the points of their width, reliability and shipping conditions underneath. Serious deformations and damages made further service of some bridges dangerous. During the restoration work there appeared several methods of approach, depending upon architectural and transport requirements and conditions of existing structures.

2.PRESERVATION OF EXISTING STRUCTURES

This method was applied, when it was possible to use old structures after their strengthening. Examples:
- the Laundry bridge across the Fontanka river, built in 1769 as 3-span stone arch, faced with granite. It's piers' foundations were strengthened after the appearance of serious deformations and cracks of stone superstructure;
- the Upper Swan bridge across the Swan canal, built in 1768 as one-span stone arch. All it's constructions were remained and only new water-protection and asphalt were laid;
- the Kazan bridge across the Catherine canal, built in 1805 as one-span brick-work arch.After the guniting of the arch it's still working at the main city's thoroughfare-the Nevsky avenue.

3.RESTORATION OF OLD CONSTRUCTION IN NEW MATERIAL

This method was applied in the cases, when serious damages made further usage of old material impossible, but the old bridge had architectural value. Examples:
- the Hermitage bridge across the Winter canal, built in 1766 as one-span stone arch. It's stone arch was changed for concrete one, faced with granite, of the precisely same dimensions, after the appearance of serious destructions of old superstructure and piers' foundations;

- the General Post Office pedestrian bridge across the Moika river, built in 1824 with one-span suspension superstructure. After serious damages of the chains and pylons two extra piers were built, completely spoiling it's appearance. Recently it received original outlook after capital reconstruction.

4.ERECTION OF A COMPLETELY NEW BRIDGE

This method was used, when existing bridge hadn't got any architectural value, but a new bridge had to provide harmonious unity with surrounding ensemble. Examples:

- the Italian pedestrian bridge across the Catherine canal. It's new one-span steel girder, decorated in the classical traditions of 19 century, changed wooden construction and became integral part of the heart of St-Petersburgh;
- the Second Winter bridge across the Winter canal. It's concrete arch, faced with granite, replaced old wooden superstructure, repeating the outline of the First Winter and the Hermitage bridges and giving an excellent finishing touch to the ensemble of the Winter canal.

5.CHANGING OF ELEMENTS OF BRIDGE'S CONSTRUCTION

This method was used, when a part of a bridge had to be replaced due to certain circumstances, with a new contruction becoming an integral part of the whole bridge. Examples:

- the Trinity bridge across the Neva, built in 1903. It's swing span failed to withstand the requirements of shipping, being only 22,8m wide. It was replaced by the bascule span, providing 43m clearance for ships, going by the Volga-Baltic water way. The outline of the new span produced the impression of continuation of old constant superstructure. Newly-built concrete arch, connecting the pier of the bascule span with the left bank of the Neva, made the whole construction completed, coinciding with the outline of the existing arches near the right bank;
- the Old Kalinkin bridge across the Fontanka river, built in 1780ies. It's removable central span, once used for shipping of sailing vessels, was changed by the stone arch, similar to the adjacent;
- the Big Okhta bridge across the Neva, built in 1911. During forthcoming reconstruction only the bascule span and the deck will be changed and it's 136m arch trusses will be preserved.

6.CHANGING OF THE WHOLE OLD SUPERSTRUCTURE

This method was applied during the reconstruction of the Liteiny bridge across the Neva, built in 1879. It's old piers were widened, using the starlings, which made possible to install new continuous steel girder instead of former iron arches, having the carriageway's width increased from 18 to 28m. The swing span with the width 19,8m was replaced by the bascule one, providing 50m clearance. The new superstructure had got the original length of the spans and curvilinear outline, resembling old arches. The new bascule span with the outline, similar to the constant one, provided the completed and continuous silhouette to the whole construction.

7.CONCLUSION

The choice of the method of reconstruction was made in each case after thorough inspection of the bridge's condition, archive studies, consultations with architects and art critics. This provided the opportunity to preserve unique architectural ensembles, strict and graceful view of St-Petersburgh.