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Session C3

General Repair of Buildings
Aspects généraux de la réparation des bâtiments
Allgemeine Fragen der Instandsetzung im Hochbau

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Extending the Lifespan of the Architectural Heritage

Prolongement de la durée de vie de l'héritage architectural

Verlängerung der Lebensdauer des architektonischen Erbes

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Giorgio Croci, born in 1936, has carried out research, studies and projects for the strengthening and restoration of historical buildings, e.g. the Coliseum and the Senatorio Palace in Rome, the Ducal Palaces in Modena and Genoa, the Castle of Spoleto, the Basilica of St. Ignatio de Loyola in Spain, the minarets and mosques of Samarkand and Cairo, Chephren's Pyramid, Egypt, and temples of Angkor.

SUMMARY

The paper describes some of the possibilities for strengthening and restoring historic buildings. Regardless of the cause of damage, the strategies for intervention are greatly facilitated when the actions are stabilised, rather than due to sudden or evolutionary phenomena. In this latter case it may be expedient to intervene on a step-by-step basis and to use a monitoring system to ensure that the desired results are attained. The potential offered by new techniques and modern technology, such as synthetic fibres, ropes and stainless cables and the possibility to regulate induced forces and deformations artificially mean that structural engineering can make a substantial contribution to extending the lifespan of historic buildings and monuments.

RÉSUMÉ

L'auteur décrit les possibilités de renforcement et réhabilitation des ouvrages historiques. Indépendamment des causes de dégradation, l'intervention est d'autant plus facile que la stabilisation des effets dégradants a pu se faire au préalable, afin de ne pas avoir à faire face à une reprise ou à une progression subite du phénomène destructeur. Dans ce cas, il peut être plus avantageux d'intervenir au coup par coup et, à l'aide d'un système de surveillance, d'observer un arrêt éventuel des dommages. Les ingénieurs civils disposent actuellement de techniques innovatrices - comme la régulation artificielle des forces et des déformations induites - et de nouveaux matériaux - comme les fibres synthétiques, les fils et les câbles en acier inoxydable - qui permettent d'intervenir de manière significative pour prolonger la longévité de monuments et constructions historiques.

ZUSAMMENFASSUNG

Der Beitrag beschreibt Möglichkeiten zur Verstärkung und Restauration historischer Bauten. Unabhängig von der Schadensursache wird der Eingriff sehr erleichtert, wenn die Einwirkungen stabil sind, anstatt dass sie plötzlich oder fortschreitend auftreten. Im letzteren Fall kann es vorteilhaft sein, Schritt für Schritt einzugreifen und dabei zu beobachten, ob sich die erhofften Ergebnisse einstellen. Dank neuer Materialien wie synthetischen Fasern, Seilen und rostfreien Kabeln und Techniken, wie der künstlichen Regulierung der erzeugten Kräfte und Deformationen, können Ingenieure einen bedeutsamen Beitrag zur Verlängerung der Lebensdauer historischer Bauten leisten.

1. INTRODUCTION

Ancient monuments are delicate structures that bear witness to our culture through the centuries, and even millennia, despite the fact that their original designs not always gave attention to the aspects of durability and safety.
However to extend the lifespan of a monument is not always necessary to carry out substantial works; it is often possible, after a thorough inspection, to intervene with a few carefully chosen measures that respect the monument's historical value.
On 17 November 1994, a scientific mission made an ascent of Chephren's Pyramid (Figure 1) to evaluate the role of temperature and wind in the decay of the stone and to assess the stability of some blocks that have been shifted by earthquakes in the past (Figure 2). On the one hand the survey contributed new and important elements for deeper understanding of the construction of the pyramid and, on the other, showed that, apart from the problem of weathering, no radical structural interventions were necessary, it being sufficient to add some supports in the same limestone to ensure the stability of blocks at risk.
A thorough survey of any monument is only the first step in studies that often include in-depth investigations and, in the case of evolutionary phenomena, careful monitoring of data. Three classes of actions may affect the stability of a building, either independently or in combination: static actions (forces, loads, ...), indirect actions, such as imposed deformations (soil settlement, temperature variations, ...) and dynamic actions (mostly in connection with earthquakes).
From the strategic point of view it is useful to consider the relationship between the actions involved and time; in other words, whatever the cause of the damage, different strategies should be followed according to whether the actions are currently stabilised, in evolution or of the type that may appear suddenly.

Fig. 1: The ascent of Chephren's Pyramid

Fig. 2: Shifting of a block on the top of Chrephren's Pyramid

2. STABILISED ACTIONS

2.1 The origins of damage

Some supplementary forces or stresses usually ramain even after the phenomena that have caused damage are over. Sometime one of the main causes may be the original design, since in the past the form and dimensions of buildings were not always decided on the basis of analyses and calculations: rather, past experience and the observation of instances of collapse and damage were the only guides available to the architects.
Safety levels may therefore be very poor, especially when well-tried designs were abandoned for experimentl ones. Gothic cathedrals are an outstanding example of such structures. The radical nature of their design not only contains intrinsic weaknesses, but also renders them sensitive to even low-level actions such as minor soil settlements, low-intensity earthquakes or wind pressure.

In the cathedral of Vitoria in Spain the main columns have been affected by significant levels of curvature and deformations: at mid-column level, these measure about 30 cm (Figure 3) and the deformations have detached some of the voults (Figure 4).

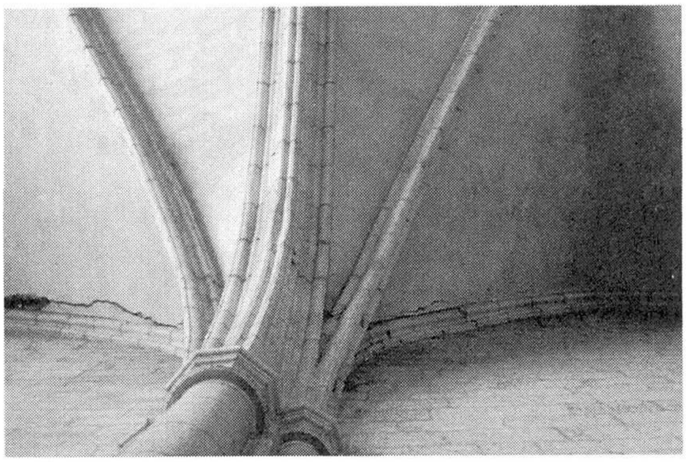

Fig. 4: Disconnections between the vaults and walls

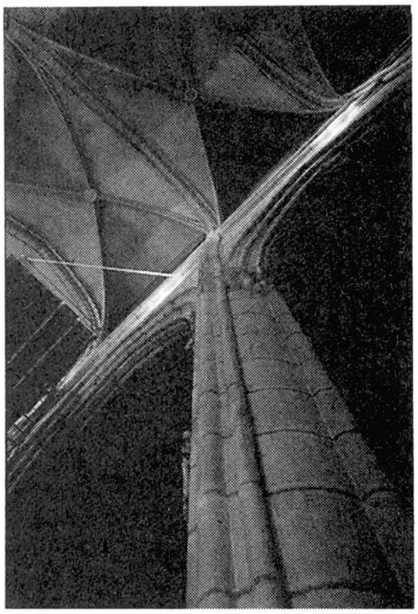

Fig. 3: Deformations on the columns of Vitoria Cathedral

A monitoring system recorded the evolution of temperatures, deformations and out-of-plumb, for one year, showing that the phenomena are now substantially stabilised.
A general elastic finite element model was used to clarify first the scarce spatial cooperation in this kind of structures and, subsequently, the influence of the sequence of construction on the resulting stresses and deformations (Figures 5 and 6): the results obtained were found to correspond well with the actual state of the building (Figure 3). However, it was also necessary to carry out a non-linear analysis, taking into account the influence of the weakest zones, such as the triforium, and reductions in the resistant sections of the columns in order to define present safety levels in terms of ductility. The diagram in Figure 7 shows the equilibrium conditions in the middle section of the column, where the line "r" represents the resistant moment-curvature relationship and the line "a" is the external moment, taking account of the second order effects; the point "E" represents the equilibrium situation and the related curvature X is very close to the actual curvature measured. AMμ indicates the moment increment necessary to reach collapse, thus giving a very low real safety margin, especially taking into account possible errors in the evaluation of the limit deformations.

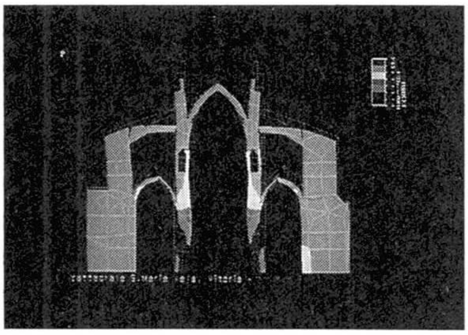

Fig. 5: Stresses and deformations where the buttresses were built before the vault

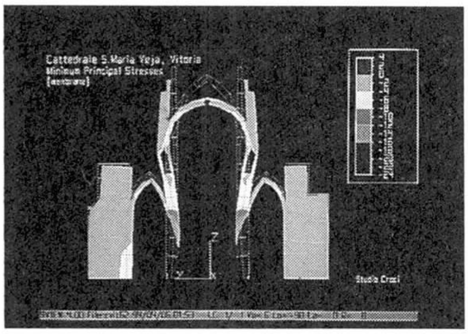

Fig. 6: Stresses and deformations where the buttresses were built after the central vault

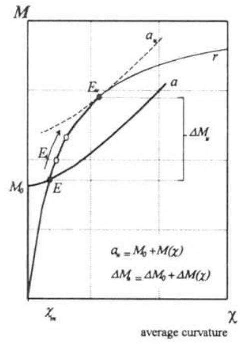

Fig. 7: Equilibrium between external and resistant moments

2.2 The strategy for intervention

Historic buildings in masonry often suffer from cracks and excessive deformations, of which the principal causes are usually the limited tensile strength and unbalanced thrusts.
The interventions can therefore conveniently follow these two main strategies: basis for the insertion of tie rods or cables and the introduction of artificial forces through the use of jacks or by prestressing.
The reinforcement measures carried out in the Cathedral of Vitoria are centred mainly in three areas (Figure 8):
- the tops of the pier extensions (a) will be longitudinally and transversely connected by trusses inserted in the garret to improve collaboration between the two sides of the nave and between the bays;
- the thrust assured by the flying buttresses in zone (b) may have been reduced by visco-elastic deformations: this will first be verified and if it is necessary to adjust the thrust by the use of flat jacks, a bidirectional counteraction will be assured by post-tensioned bars;
- finally, to stiffen the areas of the triforium and above the aisle vault springing (c), and make them more shear resistant, a truss will be inserted in the aisle garrets, which will be fixed to the buttresses and to the pier extensions.

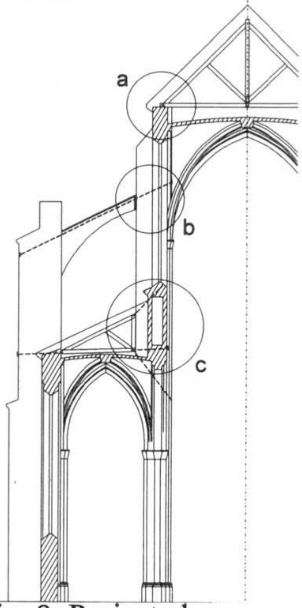

Fig. 8: Projected reinforcement measures

3. EVOLUTIONARY ACTIONS

3.1 The origins of damage

Soil settlement is the most common cause of evolutionary phenomena, often causing damage to monuments built generally on superficial foundations.
Although the soil itself may have thickened over the centuries and the original settlements are therefore extinguished, variations in the soil conditions and, particularly, in the hydraulic conditions (alterations to the natural drainage, raising or lowering of the water table, leakage from sewage systems, etc.) may well generate new settlements that are not easily stopped, since they often involve large areas, and whose further developments cannot easily be predicted.
Many of the monuments of Samarkand (Uzbekistan) are affected in this way and both minarets and disconnected walls have dangerous cracks or lean heavily. In the Tilla Kari Mosque on Rejistan Square (Figures 9 and 10) is an example in which the outward top displacements amount to tens of centimetres, creating unstable situations for both the walls and the connected vaults.

Figg. 9, 10: Leaning and deformations in minarets and mosques, Samarkand

3.2 The strategy for intervention

In consideration of the subsidence that involves much of the area of Samarkand and the difficulty of establishing a priori the entity of the measures necessary to stabilise the situation, it was proposed a step-by-step solution, using a minitoring system to check the improvements made during and after the completion of each phase and, finally, the stabilisation of the settlements.

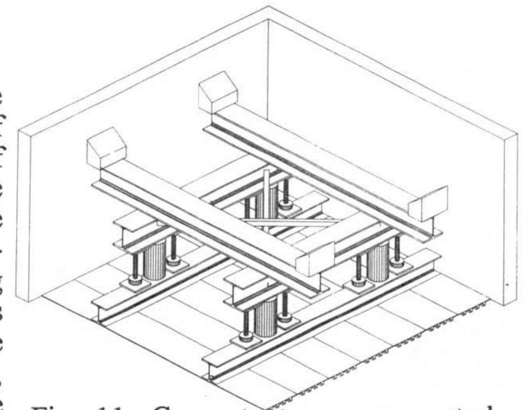

Fig. 11: Concrete boxes connected to the walls to reduce the pressure on the soil

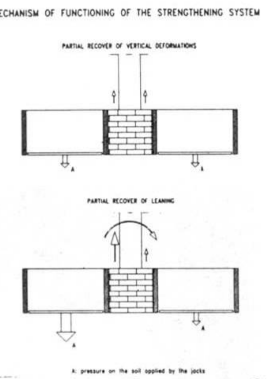

Fig. 12: System of Jacks to regulate pressure on the foundations

The solution consists in removing a certain amount of soil inside and outside the walls of the mosque and filling the spaces with empty reinforced concrete boxes, which are connected to the masonry foundations, so as to create a continuous rigid raft.
This has the dual advantage of reducing the weight and enlarging the foundations. A further significant advantage is obtained by placing a movable foundation slab on the bottom of the concrete boxes, loaded using provisional jacks (Figure 11 and 12) in order to induce and regulate the pressure necessary to correct the leaning on the basis of the data recorded.
If the settlement does not appear to stabilise, the movable and removable bottom slabs make it possible to dig more deeply in to the ground in different zones of the box girder and further reduce the loads, possibly reaching firmer strata.
This "observational criterion", based on a programme of step-by-step measures, by regulating the load and pressure on the soil at each stage, can eliminate the uncertainties inherent in predicting the possible evolution of soil settlements, making it more cost-effective.

4. SUDDEN OR UNEXPECTED ACTIONS

4.1 The origins of damage

These types of action include unexpected events such as bombing or landslides, or others that may be relatively probable, such as earthquakes or hurricanes.
General measures to mitigate the effects of disasters can prevent much of the loss involving ancient monuments. Earthquakes are the most prevalent phenomena for which preventive measures can substantially reduce damage and avoid collapses. The damage to monuments in Cairo caused by the earthquake of 12 October '92 offers an interesting example. Many minarets collapsed not only because the masonry was of poor quality but also on account of the intrinsic weakness of the structural behaviour, especially in the upper parts where slender columns cannot withstand both flexural and twisting actions (Figures 13 and 14). The connection between the minaret and the main part of the mosque is another weak zone, as is shown both by mathematical

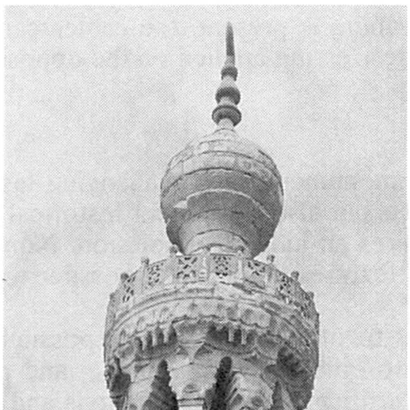

Fig. 13: Slender columns in some minarets

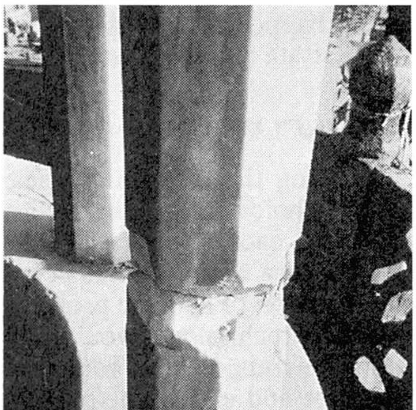

Fig. 14: Important damages in the upper part of a minaret

models (Figure 15) and direct observations, although the spontaneous cracks (Figure 16) automatically mitigate the seismic effects.

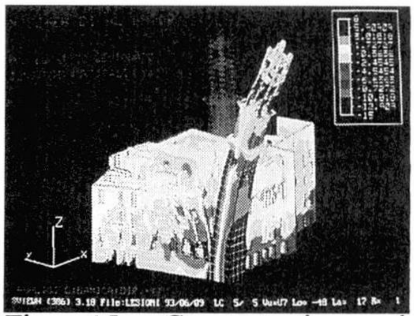

Fig. 15: Concentration of stresses and deformations revealed in a mathematic model

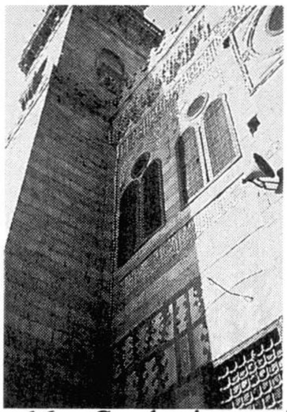

Fig. 16: Crack in a joint between the minaret and the mosque

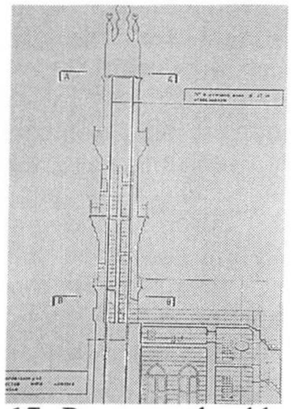

Fig. 17: Prestressed cables for the reinforcement of the minaret

4.2 The strategy for intervention

It is well known that seismic behaviour can be improved not only by strengthening measures, but also by reducing natural frequencies, improving the capacity to dissipate energy and reducing volumetric asymmetries etc.. In reality, however, it is not easy to reduce the natural frequencies (mainly stiffness) in historic buildings without penalising their strength; nor can the dissipation of energy usually be improved without introducing significant structural alterations such as creating joints between the structure and the foundation and replacing rigid connections with special dissipating devices.
The most suitable approach remains therefore strengthening, without renouncing anyway, when it is possible, to render different volumetric parts of the building independent of one another, such as by disjunction between the minaret and the mosque.
Besides offering an indispensable improvement in the masonry in specific areas where it has deteriorated, the reinforcement can usually be accomplished by tie bars, prestressed cables, etc. placed so as to confer appropriate tensile resistance. Figure 17 shows the strengthening of a minaret using prestressed vertical cables inserted inside the wall and requiring only small holes on the perimeter of the winding staircase. If the bearing capacity of the foundations is insufficient and if, as often happens, some degree of leaning is present, the cables can be anchored in deeper and more solid strata and a higher level of prestressing applied on the opposite side to the slant.

5. CONCLUSIONS

Extending the lifespan of ancient monuments is a challenging task that must take into account not only technical and economic aspects but also respect for historical values and the original concepts of building, each of which thus requires an individual solution. Nonetheless, each of the categories of phenomena examined has peculiar features that make it possible to define some general guidelines for the solution of specific problems.
Modern technology would appear to offer two principal possibilities: the use of steel or synthetic fibre tie rods, cables, ropes, etc. to give tensile resistance; and prestressing, springs, iacks, etc. to regulate and ensure the means of artificially inducing actions and, if need be, recovering part of the permanent deformations.

Stone Restoration at the Houses of Parliament in London

Rénovation de la maçonnerie en pierre du Parlement de Londres

Steinrestaurierung am House of Parliament in London

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SUMMARY

This paper describes the deterioration which occurred in the masonry of the Houses of Parliament in London as a result of air pollution, and the restoration program which has been carried out over twelve years. The policy governing the conservation work is set out and details are given of stone cleaning and repair techniques. The temporary works adopted for the restoration of the Victoria Tower are described, with an outline of the cast iron roof repairs and regilding. Dynamic methods are described for testing the structural integrity of slender pinnacles.

RÉSUMÉ

Cette communication traite de la dégradation qu'a subi la maçonnerie du Parlement à Londres par suite des effets de la pollution de l'air. Elle présente le programme de rénovation de ces douze dernières années. Elle montre la stratégie adoptée pour les travaux de protection ainsi que divers détails sur les techniques de nettoyage et de réparation de la maçonnerie en pierre. Elle décrit également les mesures de construction auxiliaires prévues pour la Tour Victoria, en fournissant des informations succintes sur les travaux de réparation et d'enjolivure de la toiture. Enfin, elle fournit des indications sur les méthodes d'essais dynamiques utilisées pour évaluer l'intégrité des flèches de tour très élancées.

ZUSAMMENFASSUNG

Der Beitrag beschreibt den Verfall des Mauerwerks am Londoner House of Parliament infolge Luftverschmutzung und das Restaurierungsprogramm der zurückliegenden zwölf Jahre. Es werden die Strategie der Konservierungsarbeiten und Details der Steinreinigung und -reparatur vorgestellt. Die Baumassnahmen für die Reparatur des Victoria Tower werden beschrieben, mit kurzen Ausführungen zur Reparatur des gusseisernen Daches und der Verschönerungsarbeiten. Ferner werden dynamische Testmethoden zur Beurteilung der Integrität der schlanken Turmspitzen beschrieben.

1. INTRODUCTION

1.1 The Houses of Parliament, properly known as the Palace of Westminster, stand on the historic site of the medieval royal palace where Parliament originated. The previous palace was almost completely destroyed by fire in 1834 except for Westminster Hall with its magnificent hammer beam roof dating from 1394. The present Houses of Parliament were built between 1840 and 1865 by the architect Charles Barry, assisted by the gothic designer Augustus Welby Pugin.

1.2 The building is founded in the Thames gravels on a mass concrete raft up to 3m thick. The structure is of loadbearing masonry supplemented by cast iron beams and columns, with brick jack arches to support internal floors. The roofs are trusses of cast and wrought iron covered by 1000 x 750mm cast iron tiles. The area of the site is about 30,000m^2 and there are 13 internal courtyards. The clock tower, Big Ben, is 93m high and the 119m Victoria Tower was the tallest building in the world when it was completed in 1860.

2. CONSERVATION, RESTORATION AND REPAIR

2.1 Stone Deterioration

Although Charles Barry was aware that the polluted air from coal burning in Victorian London had a severe effect on stone buildings, the stone he chose was a magnesian limestone from Anston in Yorkshire. It carved well and could be supplied in blocks up to 1300mm thick, but stone decay was apparent even before the building was completed. The surface of the stone was eroded over the decades and by 1926 some 200T of loose fragments had been handpicked, denuding much of the rich decorative carving. Because the stone was not marked at the quarry so that it could be laid in its natural bedding plane exfoliation occurred where stones were laid with the bed parallel to the surface of the masonry. A programme to restore some of the most noticeable defects was begun in the 1930s but it was interrupted by the second world war. Clean air legislation of the 1950s eliminated the smoke, smogs and airborne acidity caused by coal burning, so the rate of decay reduced and buildings which had been cleaned then remained clean.

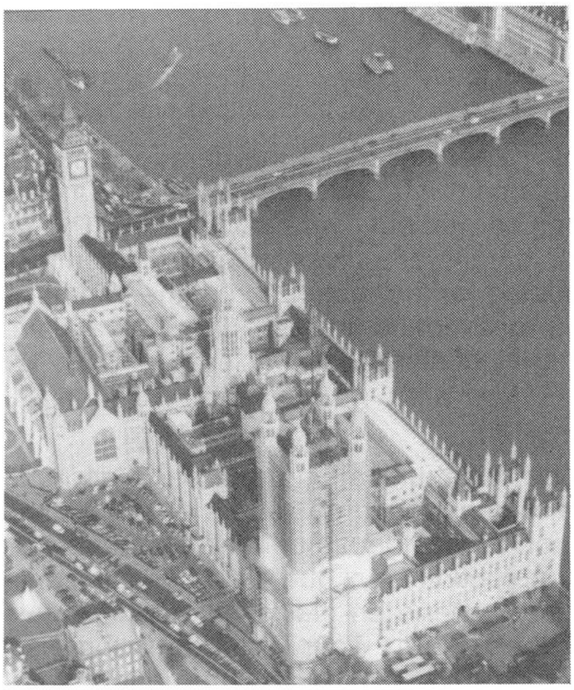

Fig. 1 The Houses of Parliament

2.2 The Repair Programme

2.2.1 The work required was in three categories: conservation, restoration and repair, and a programme covering all the external facades has been carried out as follows:

Phase 1	New Palace Yard	1981 - 1982
Phase 2	West Front	1982 - 1984
Phase 3	North Elevation	1982 - 1984
Phase 4	The Clock Tower - Big Ben	1982 - 1984
Phase 5	River Front	1985 - 1986
Phase 6	The Central Tower	1986 - 1989

Phase 7	South Elevation	1990 - 1991
Phase 8	The Victoria Tower	1990 - 1994

2.2.2 The original Anston quarry was worked out many years ago and the repairs were made using an oolitic limestone from Clipsham in the county of Rutland which is a good colour match and somewhat harder than Anston. It was carefully marked at the quarry to ensure correct bedding in the building.

3. CONSERVATION POLICY

3.1 From the outset a conservation policy was established with the following principles:

- Retain and conserve as much of the original fabric and decorative detail as possible.
- Research the history of the original work before specifying any repairs.
- Make careful records before, during and after repairs and prepare an archive document detailing all restoration.
- Where possible, use the traditional techniques of the original builders.
- Where possible, use non-destructive testing to avoid loss of original fabric.
- Avoid irreversible processes.
- Aim for a 40 year maintenance cycle.

3.2 It was decided to allow weathered stone to remain provided that it was sound with no water traps and when cleaning stone, to accept a "grubby clean" sufficient for repairs to be carried out, rather than risk unnecessary damage to the surface in trying to achieve a pristine finish. Although stone cleaning has aesthetic benefits, it can cause further damage and its main purpose in conservation terms is to reveal the full extent of the repairs which are needed. When limestone fresh from the quarry is first laid, moisture dries out and carries salts to the surface of the masonry forming a protective crust. If cleaning removes this crust the durability of the stone is reduced.

4. STONE CLEANING TECHNIQUES

4.1 Water Washing

4.4.1 Trials conducted with the assistance of the Building Research Establishment concluded firmly in favour of water washing. However, it is always important to avoid saturating the stone and to avoid water penetrating into the building. Just sufficient water was applied by very fine nebulae sprays to soften the dirt so that it could be gently and carefully brushed off. Close supervision of the contractor was invariably necessary to avoid excessive watering. To prevent water penetration through mortar joints repointing was necessary before water washing and all mortar joints were cleaned with a pencil blast gun before repointing. Despite trying many methods, no entirely satisfactory way was found to prevent water getting in at windows so someone was always present on the scaffolding whenever water was being sprayed and a second person with a radio was stationed inside to report any ingress of water.

4.1.2 Attempts to measure dampness at various depths in the stone did not yield reliable results, but it was clear that the masonry becomes saturated very quickly indeed and is extremely slow to dry out. To prevent water running down the face of the building and saturating lower parts, plywood slurry gutters were erected with pipes taking the water to settling tanks.

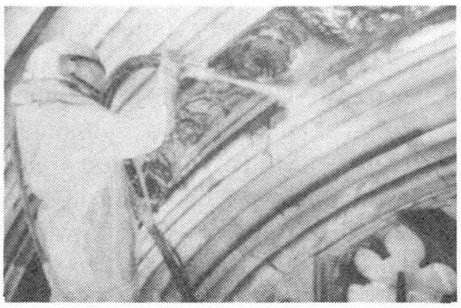

Fig.2 Blast Cleaning

4.1.3 In some situations where stone was sound and smooth, relatively light soiling was removed simply by wet sponging.

4.2 Blast Cleaning

4.2.1 For some parts of the building such as the libraries, oak panelled fine rooms and air conditioned archives of irreplaceable historic documents, the risk of water penetration could not be accepted and semi-dry blast cleaning was adopted using silica grit graded from 0.5mm down. The grit was selected after trials of various gradings and of other abrasives such as aluminium oxide and crushed nut shells.

4.2.2 A disadvantage of blast cleaning is that the natural crust on the stone is damaged. Other disadvantages are the noise and dust, and a mottled shading which is sometimes caused by the way the gun nozzle is manipulated or by markings beneath the original surface of the stone. Advantages are that work can continue through the winter and the rate of progress is more predictable than for water washing.

4.2.2 The technique entails a variety of nozzle sizes and careful control of the pressure and water content depending on the stone characteristics and whether the surface is ashlar or fine carving. The pressures used were of the order of 1.5 bar with water at the rate of 1 litre/minute. Staff responsible for specifying and supervising the work were given practical experience off-site before the contract began so that they could appreciate the difficulties of working in heavy protective clothing with air fed helmets in a confined space, dust and noise. This experience led to a decision to use pressure gauges on hypodermic needles to insert into the air lines for independent pressure checks.

4.3 Chemical Poultices

Alkaline clay poultices were used to a limited extent, particularly to remove the staining from anti-pigeon gel and for the very delicate medieval carvings on Westminster Hall.

4.4 Other Stone Cleaning Methods

4.4.1 Acid cleaning methods were not appropriate to this carbonaceous stone, although on granite and brick buildings 5% hydrofluoric acid solution has been used with success. Very high pressure water jets were also inappropriate with such a relatively soft stone.

4.4.2 Small scale laser cleaning trials were conducted and showed promise. The laser readily removes sooty deposits and sulphate encrustations leaving the surface of the stone undisturbed. Lasers present their own safety hazards and considerable development work will be needed to scale up the system for areas of ashlar, but the method has proved very satisfactory for cleaning fine carvings and was used to remove Victorian limewash from medieval limestone statues in Westminster Hall.

5. STONE REPAIR AND REPLACEMENT

5.1 After cleaning, each area was reinspected and a detailed schedule of repairs was prepared. Defects included sulphate attack, stone spalling. clay beds and faults within the stone, missing pieces and water traps. The aim was to retain as much of the original stonework as possible, accepting weathered profiles. Repairs were undertaken where failure was likely during the 40 year maintenance cycle, to prevent water traps, to reduce the rate of decay and to reinstate important architectural details.

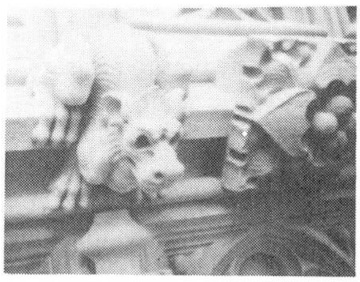

Fig.3 New Carvings

5.2 The stone repairs ranged from repointing, to dressing out water traps, repairing with stone dust mortar, piecing in small areas, the renewal of ashlar blocks and the replacement of elaborate carvings. For the Victoria Tower, which was by far the largest phase and included particularly rich ornamentation, a stone carving workshop was established on site staffed by masons and stone carvers who were able to recreate accurately the decayed decorative carvings, heraldic devices, statues and building details which it was necessary to cut out and replace.

5.3 Individual replacement ashlar blocks were fixed with four stainless steel dowels: one in each face. While the stones were being placed the vertical dowels were held up with thin blades and then dropped into place under gravity. The horizontal dowels were pulled into place by a technique known as "mousing". This entails overdrilling the hole and inserting the dowel full depth with a thin string wound spirally around it. When the stone is in place the string is pulled, drawing the dowel into its working position. When some original stones were cut out it could be seen that the Victorian masons had used exactly the same method.

5.4 Pinnacle Stability Testing

The inadequate structural stability of several very slender pinnacles was a matter of concern. Load tests and dynamic resonance tests were carried out from which it was possible to identify failed bedding joints and pinnacles which should be rebuilt. With careful dismantling the majority of original stones were able to be reused, but with stainless steel dowel bars inserted. Having established the value of measuring the natural vibration frequency of the stone pinnacles to indicate failed bed joints and pinnacles which might eventually topple in extreme winds, a sophisticated laser monitoring technique was developed. Vortex shedding from even a light wind vibrates a pinnacle sufficiently for the natural frequency to be measured with a laser mounted on a remote rooftop. By traversing the spot of laser light down the pinnacle from top to base, fault planes are indicated by discontinuities in the vibration frequency of the pinnacle. In this was all the pinnacles on the Palace can be checked in two days and the technique is now used for planned maintenance inspection at 4 year intervals.

6. THE VICTORIA TOWER

6.1 Temporary Works

6.1.1 The access scaffolding for the 119m high Victoria Tower, which weighed 1000T and used 110km of scaffold tube, was founded on bored piles and supported over the adjacent roofs of the Palace on Bailey bridge towers and trusses. In view of the prominence and importance of the site, it was designed to the wind loading and other standards of a permanent structure. At upper levels the scaffold was tied back into the masonry using stainless steel drill anchors which were later made good with stone plugs and the positions were recorded for future use.

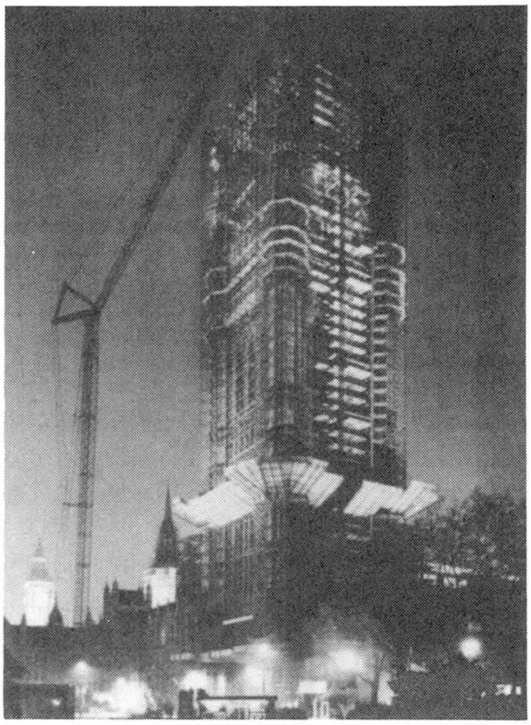

Fig.4 Victoria Tower Scaffold

6.1.2 A safety fan projected from the scaffold just above Palace roof level. Slurry gutters were installed below alternative working platforms, with gullies and downpipes leading to settling tanks at ground level. The working areas of the scaffold were totally enclosed with fire resistant translucent sheeting and the leaded windows were protected with plywood cut to the gothic profiles. To ensure that stable environmental conditions were maintained for the archive rooms within the tower, a mechanical ventilation and filtration system was provided to feed air through the working area to the existing air conditioning plants.

6.1.3 The tower floodlighting remained in operation during the works and there was favourable press comment on the scaffold as a work of art in itself.

6.2 Victoria Tower Roof

6.2.1 The Victoria Tower is roofed with large cast iron tiles on a trussed framework of cast and wrought iron. This is surmounted by a 28m high flag mast supported by ornate ironwork. Originally the tiles had a zinc anti-corrosion treatment which had failed very quickly in the acid Victorian city air and had been overcoated with a patent treatment. Research showed that many layers of lead based paint had also been applied over the years.

6.2.2 The aim was to put the roof in repair and provide a protective treatment which would last some 40 years. The accumulated layers of dirt and paint were removed back to bare metal by air abrasive blasting. Lead precautions included a full working enclosure, protective suits with air supply, air sampling outside the enclosure, disposal of the spent grit and dirt under controlled conditions and a decontamination unit for workmen. Broken cast iron tiles and jointing rolls were repaired by stitching, cold cast welding and off-site welding in an oven with controlled heating and cooling. Some new iron tiles, rolls and missing decorative features were cast, and after reassembly the roof surface was shot blasted and hot zinc sprayed. Pitting in the metal surface was stopped with an epoxy filler, followed by a sealing coat and finally a colour finish of vinyl co-polymer paint.

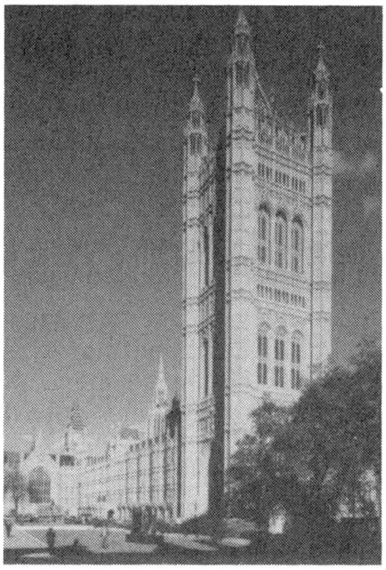

Fig.5 Restoration Complete

6.3 Gilding

From analysis of paint scrapings, research in the Public Record Office, old photographs and contemporary descriptions it became clear that the gilding had originally been much more extensive and it was decided to restore the original decorative scheme. The gilding design was faithfully restored using 4,000 sheets of gold leaf and the completed scheme achieves a balance with the restored clock tower at the opposite end of the building, as Barry and Pugin originally intended.

7. BIRD PROTECTION

Birds, particularly pigeons, roosting on buildings are a problem throughout London. Stone coloured nets and sprung stainless steel wires were therefore fixed over balconies, around statues and across other perching areas. These measures are not generally visible from ground level.

Repair Methods for Deteriorated Sandstone Facades

Procédés de réparation de façades en grès endommagées

Reparaturtechniken für beschädigte Sandsteinfassaden

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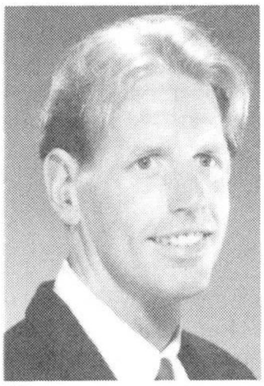

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SUMMARY

A local sandstone was used to clad many San Francisco buildings constructed around the turn of the century. Today, many of these facades are deteriorating, and the stone is cracking, spalling and weathering away, creating a hazard for those walking below. This paper describes the field and laboratory investigations and the facade repair at the St. Francis Hotel. The sandstone cladding had deteriorated due to the effects of water on the stone's mineral constituents. The lifespan of the facade was extended by repairing damaged areas, applying a surface treatment to the stone and by reducing the water infiltration into the stone at horizontal ledges.

RÉSUMÉ

Un grès dur local fut utilisé vers la fin du siècle dernier pour le revêtement d'innombrables immeubles de la ville de San Francisco. De nombreuses façades ont été endommagées par les intempéries, faisant apparaître des fissures et des éclatements dont les projections sont dangereuses pour les piétons. Cet article présente l'examen effectué pour l'hôtel St. Francis, sur le site et en laboratoire; le grès de ce bâtiment s'est décomposé en partie, par l'effet de l'eau. Il a été prévu de réparer les parties de la façade endommagée par un traitement superficiel de la pierre et une étanchéité pour réduire les infiltrations d'eau de pluie dans les moulures.

ZUSAMMENFASSUNG

Für die Verkleidung vieler Gebäude wurde im San Francisco der Jahrhundertwende ein örtlich gewonnener Sandstein verwendet. Heute sind viele dieser Fassaden verwittert, Risse und Abplatzungen treten auf, die die Fussgänger auf der Strasse gefährden. Der Beitrag beschreibt Feld- und Laboruntersuchungen am St.Francis Hotel, wo der Sandstein sich infolge wasserempfindlicher Mineralbestandteile zersetzte. Die Lebensdauer der Fassade wurde durch Reparatur der geschädigten Flächen, durch eine Oberflächenbehandlung des Steins und durch Abdichtung der horizontalen Leisten verlängert.

1. INTRODUCTION

Sandstone was a popular cladding material for many buildings constructed in the United States around the turn of the century. The stone was readily available and easily carved into ornamental shapes. A great belt of "Colusa Sandstone" is located just north of San Francisco, and many prominent San Francisco buildings, including the St. Francis Hotel, were constructed of this local stone.

Today, approximately ninety years later, the sandstone cladding on these buildings is experiencing severe deterioration. The deterioration represents a potential safety hazard for those walking below these buildings, and once the deterioration process has started, the damage progresses at an accelerated rate. This paper summarizes the field investigation, material analysis, design of repairs, and the ongoing restoration of the sandstone facade of the St. Francis Hotel.

1.1 Building Description

The St. Francis Hotel, shown in Figure 1, is a thirteen-story steel framed structure with brick infill and sandstone cladding on the street facades, and brick walls on the west facade. The typical sandstone walls have 30 cm courses with alternate courses projected out 2.5 cm (projecting course). The building was constructed in three stages: 1904, 1907, and 1913. The original drawings of the building were not available.

2. INVESTIGATION

The investigation consisted of a stone-by-stone survey of the entire building and a laboratory analysis of sandstone samples. First, the material analysis is presented then the field observations are related to the mechanisms causing the deterioration.

2.1 Sandstone Material Analysis

2.1.1 Composition of Colusa Sandstone

Petrographic analysis of the stone found it to be a graywacke, or low grade sandstone, with a siliceous-argillaceous matrix. The matrix contains the clays kaolinite, illite and montmorillinite and small amounts of carbonate materials. The stone has fine grains of quartz and feldspars. It is moderately soft, quite porous and readily absorbs water.

Figure 1 - St. Francis Hotel

2.1.2 Deterioration Mechanism

The deterioration mechanism, shown in Figure 2, begins when water enters the stone. The water dissolves elements in the binding matrix while acidic pollutants in the water alter the calcite portion of the binder into gypsum. As the water evaporates during repeated wet-dry cycling, the gypsum is deposited at the surface of the stone and forms a surface crust approximately 2 mm thick. The process of forming the surface crust is known as "surface induration." Behind the crust, a layer of disaggregated stone is formed as the calcite binder is lost. Once formed, the crust will expand in all directions due to two mechanisms: -1 the gypsum expands as it incorporates water into its crystal structure, and -2 the montmorillinite clay expands in the presence of water. The crust is not as hard as the original stone, but it is much harder than the layer of disaggregated stone. Expansion of the crust cannot be restrained by the stone and eventually the crust exfoliates, and falls away from the body of the stone.

2.2 Field Investigation

In 1987, a condition survey of the entire sandstone facade was made from swing stages, and the condition of over 30,000 stones was recorded. The condition of each stone was recorded on scaled building elevations prepared using rectified photography. During the survey, loose pieces of stone were removed and stainless steel anchors were installed to alleviate immediate hazards. Four years later, selected areas were resurveyed to evaluate the progress of the sandstone deterioration and to set priorities for the repairs.

The most severe deterioration was found in areas experiencing frequent wet-dry cycles. The resurvey revealed that once a stone forms surface cracks, more water can penetrate behind the surface and the deterioration spreads rapidly into the stone. The most common forms of deterioration include:

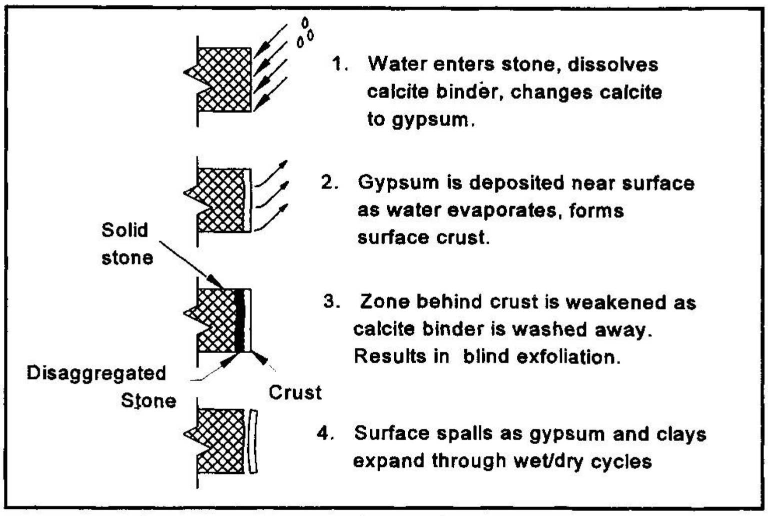

Figure 2: Deterioration Mechanism

2.2.1 Surface cracks, blind exfoliation and shallow spalling.

The most widespread type of deterioration results from the surface induration process. The deterioration first appears as fine, shallow cracks near the perimeter of the blocks. Tapping on the stone near the crack often reveals a hollow sounding area, indicating the presence of a disaggregated zone and a "blind exfoliation." The blind exfoliation increases in size until the crust falls away, creating a shallow spall or exfoliation.

Stones that are particularly vulnerable to this type of deterioration are the quoins at the corners of the building, due to their many exposed edges, and the parapet, due to the harsh exposure. Roughly 60% of all stones had formed surface cracks and roughly 45% of all stones required repair due to the surface cracking. The number and length of cracks, as well as the amount of blind exfoliation increased from 1987 to 1991.

2.2.2 Weathering

Weathering appeared as a disintegration of the stone surface. It occured most often at the bottom edge of stones that are set above the small ledge created by a projecting course below. The ledge collects water which then wicks up into the stone above. The weathering leads to loss of surface detail such as carvings and sharp corners.

2.2.3 Deep spalls

Deep spalls were found on stones with multiple surfaces exposed to the weather. The intricate shapes of these ornamental stones create areas where dirt and moisture can collect. Cracks originating from several exposed surfaces joined to spall off projecting corners. This spalling especially occured on watertables, where deteriorated mortar joints exposed multiple stone surfaces. Deep spalls were typically limited to projecting ornamental stonework, but they are one of the more hazardous types of deterioration.

Deep spalls can also be caused by expansion of corroding embedded steel anchors. These spalls were found on the balcony railing supports and on the sandstone brackets under the balconies.

3. REPAIR DESIGN

The Colusa sandstone is inherently flawed in that it deteriorates in the presence of water. The objective of the repairs was to reduce potential hazards and reduce the rate of deterioration. The life span of the facade will be extended by: -1 repairing already damaged areas, -2 reducing the amount of moisture entering the stone, -3 improving details to reduce water seepage into the stone, and -4 implementing a periodic inspection and maintenance program. Throughout the design process the following issues had to be considered for each repair: -1 the historic nature of the building; -2 material compatibility and -3 constructability and cost.

Seismic considerations for the building's structural system had been previously addressed by structural engineers Chin & Hensolt. Lateral forces were considered in the design of anchors for existing and replacement stones.

3.1 Historic Nature of the Building

The St. Francis Hotel is a designated Historic Landmark, and the impact of all repairs on the historic fabric and character of the building had to be considered. The most important issue was the visual impact of the repairs. Nearly every square meter of the facade needed repair, so it was important that the repairs minimize any changes to the appearance of the facade.

Some deterioration was due to poor original detailing. The restoration is intended to correct the detailing while altering the appearance as little as possible. Since most of the original ornamentation was severely deteriorated, it was difficult to find stones to replicate. Bracing of stone elements had to be accomplished without exposing the brace and the introduction of new elements such as flashings had to be made without changing watertable profiles.

3.2 Material Compatibility

The facade was constructed at the turn of the century using traditional materials. Repairs using modern materials had to be carefully considered. Where new materials were used, the initial and weathered appearance of the new materials had to match the existing stone's color and texture as closely as possible. Mechanical properties such as thermal and moisture expansion and compressive strength had to be compatible with the original stone to avoid the introduction of new stresses. In general, the intent was to avoid introducing new materials when the original stone could be economically repaired. However, certain elements were so severely deteriorated that complete replacement of the unit was required. When large areas of new materials were introduced, the new elements were isolated from the adjacent stone to accomodate thermal or moisture movements.

Finally, the properties of the existing materials were required for the design of new attachments. For example, anchors for some replacement pieces were welded to the existing structural steel. Coupons of the steel were tested for chemistry to determine the welding procedures.

3.3 Constructability and Cost

The issue of constructability was especially important since this is an older building and no drawings were available. The general relationship between the stone, brick backup and steel frame was known from previous work, but the information was not specific enough for detailing purposes. In addition, many subtle, but important differences in the configuration existed between the three buildings. Issues such as the location of the steel frame, whether the stone blocks were supported by the steel frame or were self supporting, the location of internal bracing of ornamentation, the keying of the stone with the brick backup, and the construction of the cornice had to be determined by inspection openings before repairs could be detailed.

The repair details needed to be flexible to account for tolerances in the original construction. Design of replacement blocks considered such issues as hoisting and scaffold load capacities, scaffold clearances, and the ability of workers to lift and handle materials. Identification of deteriorated stone, color matching, lead time to revise details, and adequate cure times for cementitious materials were critical to the repair sequence.

The work on the building was divided into phases for cost monitoring purposes. The cost of each type of repair was monitored in the first phase, then the costs were projected for the entire building. Subsequently, adjustments were made to the repair procedure, the amount of stone repaired, or the budget to bring the project in at budget.

4. REPAIR SCHEMES AND IMPLEMENTATION

The restoration work was a monumental undertaking for the Hotel, felt by all staff members ranging from the general manager to the service personnel. Close coordination between the contractors, design professionals and hotel staff was required to maintain the quality of service provided by the Hotel.

Most of the repairs fall within three broad categories: -1 repair of deep spalls -2 repair of surface cracking, blind exfoliation and shallow spalls caused by surface induration, and -3 preventive repairs intended to delay and reduce future deterioration.

4.1 Deep Spalls

Several different repairs were used for deep spalls, depending upon the type of stone. Deep spalls on ashlar stones were typically repaired by use of mortar patches, sandstone dutchman patches were used at highly visible stones near street level, and parging was used at window sills. Mortar patching with a color matched cementitious mortar was used for most deep spalls.

The Colusa sandstone is notorious for its tendency to reject mortar patches. A good bond between the stone and patch is essential to achieve a reasonable service life. Thin, feather edge patches are the most prone to failure. A minimum patch thickness of 2.5 cm was established to insure that all deteriorated stone was removed, and good keying between the patch and the stone was provided. The compressive strength of the patch material was also checked to verify that it was not stronger than the stone.

Parge coatings are a specific type of mortar patch that cover the entire top surface of a window sill. The entire top surface is covered to avoid exposed joints between the sandstone and mortar that will eventually open due to mortar shrinkage and allow water infiltration below the parging.

Dutchman patches were much more costly and were used only at highly visible stones. The dutchman patches were made by cutting out the deteriorated area of the stone and inserting a new piece of stone into the void. Joints around the inserted dutchman were as thin as possible. The surface of the dutchman was then tooled to match the tooling pattern on the adjacent stone.

Large areas of projecting ornamentation such as quoins and watertables had extremely deep spalls that could not be effectively or economically patched. These areas were repaired by removing the deteriorated stone (sometimes the entire unit) and installing glass fiber reinforced concrete (GFRC) cover panels. GFRC was chosen since it provides the best color and texture match and is most easily adapted to the varying support conditions. Since such large areas of the facade received GFRC panels, considerable effort was spent developing the best color and texture match. Accelerated weathering tests were done to expose the potential of severe degradation in the color or texture of the panels due to long term weathering. Care was taken to isolate the GFRC from the adjacent materials to eliminate the buildup of internal stresses caused by temperature and moisture expansion and contraction. The GFRC panel connections were made either with anchors set in the backup masonry walls or with anchors welded to the existing steel frame. Special inspection was performed for all welding.

4.2 Surface Cracking, Blind Exfoliation, Surface Spalling.

Where deterioration had not progressed deeply into the stone, the stone was repaired by retooling the surface of the stone. This repair method is superior to mortar patching since it retains the original material, corrects detailing problems, provides an ideal surface to receive preventive surface treatments and has a low visual impact. The insitu retooling of stone, particularly on this scale, had never been attempted in San Francisco; therefore considerable effort was spent by the contractor in developing tools and procedures to make the repair cost effective. The initial retooling cost was quite high since the commonly available tools were not adapted to nor sturdy enough for the production work needed to achieve a reasonable cost. In addition, highly paid and skilled stone sculptors were needed to perform the work. During the initial phase of the project the sculptors developed tools and procedures that significantly increased the production rate and could be learned by typical masons on the job. Eventually, the unit cost for retooling became less expensive than patching.

Retooling by its nature changes the stone profile, only certain stones can be retooled without disturbing the overall appearance. Retooling was used at specific stones as follows: For surface cracks at the top edges of projecting ashlar blocks, the top edge of the stone was cut off with a beveled edge to remove the loose piece and provide a slope away from the building. Where the disaggregated zone extended down the face of the stone, the surface of the stone was planned back roughly 1.25 cm until sound stone was reached. Then, horizontal tooling marks were applied to match adjacent stones. Ornate stones carved with floral patterns were retooled by hand using low impact dallet chisels.

4.3 Preventive Repairs

Several repairs were done to delay future deterioration. The underlying causes of deterioration cannot be completely eliminated but the service life of the stone can be increased by reducing the amount of water and pollutants that enter the stone. To reduce the pollutants on the surface of the stone, the entire facade was cleaned. Care was taken to choose a product and procedure that would not harm the stone. A mild alkaline cleaner was used and care was taken to monitor the pH level of the stone so that no chemical residue was left on the stone. The use of a mild cleaner reduced the amount of protection required by the contractor, and the cleaner used was mild enough that it could be disposed of directly through the city sewer system.

To reduce the amount of water infiltration into the stone, all mortar joints were repointed and a penetrating liquid water repellant was applied to all stones. Test sample panels were installed in 1988 to test the effectiveness of various treatments. The panels were reviewed two years later and cores were analyzed to determine if the treatments were effective. The analysis revealed that the water repellant cannot penetrate deeply into the surface crust of the original stone surface but it is still effective in preventing water infiltration directly through the face of the stone. The shallow penetration will result in a relatively short service life for the water repellant, but the material and installation costs are low so it can be reapplied as needed. The newly retooled stones will benefit more from the repellant because of better penetration of the repellent on these stones.

Horizontal surfaces are especially vulnerable to deterioration due to increased exposure to water. To protect these areas, sheet metal flashings were added to the top surface of all watertables and a polyurethene coating was installed on the top surface of the balconies. The flashings and coatings provide a durable, low maintenance repair with minimal visual impact. In some areas, decorative moldings were added to the flashings to simulate the intricate carvings on the original stone.

5. PERIODIC MAINTENANCE

The forces that caused the initial deterioration of the stone will continue to act on the building and continue to cause deterioration. A periodic maintenance program is necessary to keep the preventive repairs intact and to address the ongoing deterioration. The maintenance program will incorporate the repairs used for the restoration, but the volume of maintenance work will be significantly less than was required for the restoration. The maintenance work can be performed from hanging scaffolds that are much less costly and less intrusive than the fixed scaffolds used for restoration.

At each maintenance interval, the facade will be cleaned and the water repellent reapplied. The integrity of the pointing, joint sealants, flashings, and coatings can be evaluated and repaired as required to keep water out of the stone. In addition, any new deterioration found will be repaired.

Repair and Renovation of the Reading Terminal in Philadelphia

Réparation et consolidation de la Grande Gare de Philadelphie

Reparatur und Erneuerung des Reading Terminal in Philadelphia

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SUMMARY

The Reading Terminal was constructed with wrought iron and hot rivets in 1893. This renovation dealt with the repair of deteriorated elements and the addition of framing for a ballroom and meeting rooms. The roof arch trusses were preserved, as the three-hinged structure was one of the world's largest when originally constructed. Evaluation of existing conditions and development of repair techniques are discussed.

RÉSUMÉ

La Grande Gare de Philadelphie, communément appelée "Reading Terminal Station", a été construite en 1893, d'acier forgé et de rivetage. La réparation et la consolidation de la structure existante a permis la création d'un nouvel ensemble multifonctionnel et de plusieurs salles de conférences. Les arcs en treillis à trois articulations, dont la portée était l'une plus grandes à l'époque, ont été préservés. L'évaluation des conditions existantes et les techniques de réparation utilisées sont présentées.

ZUSAMMENFASSUNG

Der Zugschuppen des Reading- Hauptbahnhofs in Philadelphia wurde 1893 aus Schmiedeeisen und Heissen gebaut. Bei dieser Renovierung handelte es sich um die Reparatur der verfallenen Bauteile sowie die Hinzusetzung des Balkenwerks für einen neuen Ballsaal und verschiedene neue Versammlungszimmer. Die ursprünglich gebauten gebogenen Dachträger sind als wesentlicher Bestandteil eines der grössten dreischarnierigen Bauwerke der Welt erhalten geblieben. Die Entwicklung der Reparaturtechnik sowie die Auswertung der vorhandenen Zustände sollen hier diskutiert werden.

1. DESCRIPTION OF PROJECT

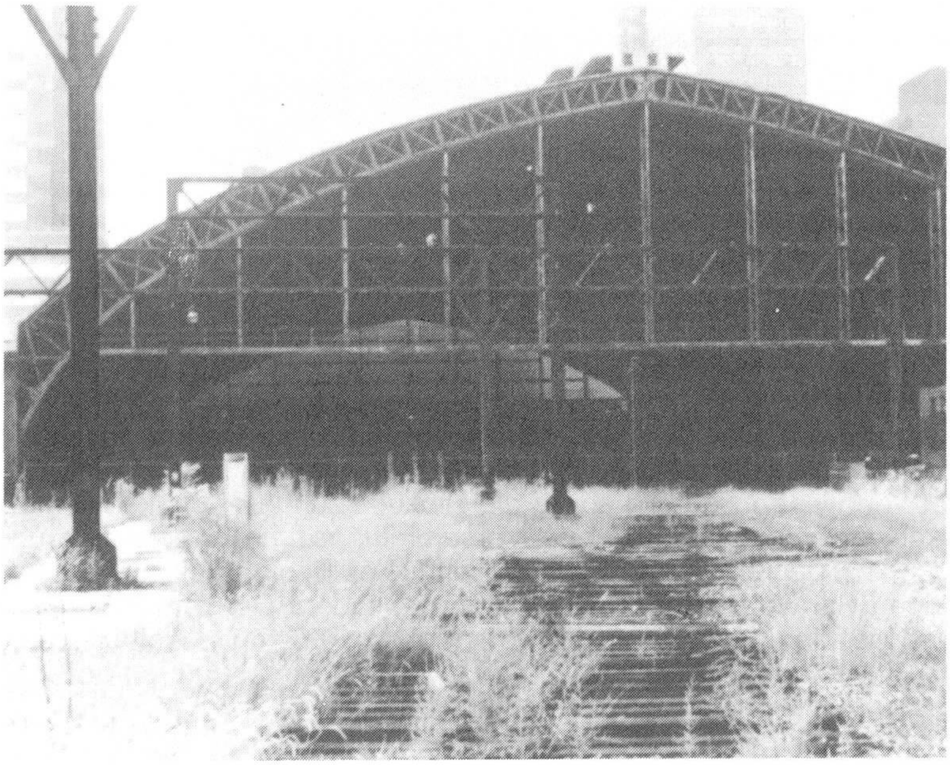
In 1983, the City of Philadelphia and the State of Pennsylvania held a competition to select a site for a convention center in Philadelphia. The site they chose was the Reading Railroad Terminal and four downtown city blocks. This combination repre-sented a site with historical significance that was also choice business district property.

Originally, the Reading Terminal project was to be financed by capital from the Reading Company, the real estate company that remained after the railroad went bankrupt. As the project changed from a private development, the Reading Terminal had to be purchased from the Reading Company. The initial purchase consisted of the upper portions of the structure, leaving the first floor market and structure in place. Some renovation of columns was required and necessitated work in the Market area.

The Terminal renovation would prove to be challenging for several reasons, not the least of which was contamination with PCBs, which delayed the existing conditions investigation several years. The historical significance of the Terminal was basic to its inclusion in the project. To provide some background for the project, that issue is examined thoroughly below.

2. READING TERMINAL HISTORY

In the 1890s, the Philadelphia and Reading Railroad decided to locate a passenger terminal in downtown Philadelphia. The commuter and passenger traffic into Philadelphia necessitated such a development. A site was selected at the northeast corner of Market and 12th streets. The site had a frontage on the south of Market Street and extended north past Filbert Street to Arch Street.



Rail lines in the city were elevated on rubble-filled viaducts with cut stone gravity retaining walls. Arches of steel rail and brick infill crossed streets so that neither rail nor street traffic would impede each other. For that reason, the tracks were required to enter the terminal on the second level which was 7.6 meters above street grade. Placing the tracks this way enabled the first floor to be available for other uses. As it happened, the site contained two important market houses between Filbert and Market streets which were to be demolished and relocated by the terminal construction. The markets were offered space on the first floor. The terminal building was to begin north of the Head House on Market Street. The construction of the Terminal building before the Head House allowed the markets to operate continuously, which set a precedent for the latest renovation when the markets were also kept open.

By the 1980s, the basement was no longer in use. Wastewater was discharged directly through the first floor to the basement, which had 0.6 meters of standing water. The foundations for the terminal were 3 meters below the basement floor. The riveted wrought iron columns sat on a base plate which transferred the load to a cut granite criss crossed stack of lengthening stones that rested on a cast concrete base footing. The presence of water over the years had not caused any appreciable settlement. The bearing level remained intact and capable of 65,000 kilograms per square meter bearing capacity.

The Terminal was constructed of hot riveted wrought iron plates and angles. Tension elements in the trusses were to be double rolled from the muck bar, and no scrap was allowed to be added in the rolling. The wrought iron for tension elements was to be capable of sustaining an ultimate stress of 358.5 MPa on a full section of test piece with an elastic limit of 179 MPa. Wrought iron for other elements was to have an ultimate stress of 331 MPa and an elastic limit of 179 MPa except for plates of over 61 cm. width for which the ultimate was 317 MPa.

The Terminal was designed by Wilson Brothers & Company and reported to the ASCE annual convention of 1895 by Joseph M. Wilson. The Terminal roof is the most significant part of the building. The track area covered was 79.25 meters x 168 meters. Pairs of arch trusses were placed at 15.3 meters apart. These elements formed a three hinged arch, which at the time of its construction was the longest such span in the world. Compound curves were used to form the shape of the arch trusses which sprang up 26.8 meters from their base pins to the center pin of the arch.

3. EXISTING CONDITIONS STUDY

Determining the base line for the beginning of restoration is a critical step in any restoration project. Access to the terminal was limited. Due to its contamination with PCBs, a release was signed each time access was granted to the space. During this phase, access could not be obtained to place high reach equipment in the terminal for examination of the truss arches and the wooden roof framing. Visual observation of the roof was performed from the track floor. The roofing membrane had failed in previous years, which allowed water to flow through the roof. Sheathing and purlins were stained from years of water flow. Our estimate was that complete sheathing replacement would be required, as would 60% of the 10 x 15 cm wood purlins.

Access to the Terminal became more constricted because the transfer of ownership was bogged down. Reading had not reached an agreement with the Convention Authority for sale of the Terminal. The delay ran on for several years with no access to the Terminal for evaluation. Fortunately, a historical repository of documents, the Athenaeum, was available where the original ink on linen drawings were stored. The documents proved to be very accurate. The truss arches were designed by a graphical solution which was contained in the original documents. The graphical solutions for the load cases were very helpful in fine tuning our computer analysis of the arch trusses. Six radii were used to develop the compound curve of the arch. As the computer model was constructed graphically, the original loading cases were applied and the analysis values for member forces were checked against the graphical solution.

During this phase of the project, a survey of the basement and Market floor was performed to determine the condition of the structure and confirm the dimensions on the design drawings of the Athenaeum. Any renovation must begin with a survey to confirm layout and accuracy of construction. The original drawings depicted a building which was slightly out of square. The sheet layout around the property was off 90° square by 19 minutes. This allowed the Terminal to completely fill the site with a slight variance side by side and end to end. The field survey determined that the building was built in accordance with the plans. This left two options as far as any added framing. One option would have been simply to frame new columns on top of old ones and accept the 19 minute variation. The other option was to square up the new design and superimpose it on the existing framing. Unfortunately, the second option was chosen by the Architects. This required the use of control lines to lay out the newly added structure. The contractor never caught on to this approach and coordination problems plagued fitup during construction.

A condition of the sale of the Reading Terminal was that all contaminants be removed before the property was turned over. This required the removal of all roadbed top surfaces and all ties and other associated material and left only the structural framing to move onto for examination of arch trusses. Wooden pads of 30.5 x 30.5 cm bolted together were purchased to provide a work platform for the high reach equipment. We could not cover the areas completely where we wanted the high reach, so we used a hydraulic crane to leap frog the platforms down the tracks. This allowed us to lift our equipment into the Terminal and move it down track lines so each truss could be examined.

In the past, truss arches had been repaired by welding by the Reading Railroad maintenance crews. Repairs were functioning well, but we still needed some test weld specimens to confirm design strengths that could be used. Coupons would be taken from beams and/or girders to provide for such testing. This was done during the demolition phase using beams that were being removed from the track floor. Weldability was confirmed, although a strength loss of 30% was experienced when welds ran across the grain. Consequently, weld values were lowered for such a case.

The track floor had large areas which were in reasonable condition. Deterioration was focused at certain points such as the following:

- Side stringers of passenger platforms;
- North end framing where water was blown in by storms; and
- Side framing, particularly side wall trusses, where water was not discharged by the drainage pipes.

The 2 x 14 cm pine sheathing began to break down over the years as the roofing failed and water gained access to the sheathing. The pieces of sheathing flowed down to the roof drains and gradually filled them. Because these pipes were filled with toothpick-like fragments, the drainage system because useless. The rainwater directed to the downspouts would spill over onto the track floor and run down onto the Market ceiling. The Market ceiling had a waterproof coating at one time, but this too had failed, allowing water to run through the ceiling into the Market.

4. DESIGN OF TERMINAL RENOVATION

The architectural plan for the terminal involved adding two levels of new framing at the north end. These new levels would frame meeting rooms with movable partitions on the track floor and a ballroom with a movable wall on the upper level. The track floor required renovation of deteriorated beams and the placement of a new concrete slab. Once the new slab was in place, it would serve as a work platform for new framing and renovation of the terminal roof above.

Renovation of track floor beams or girders was done by replacement or welding of auxiliary plates to an existing section after it was cleaned. Since lead-based paint was used, the beams could not be sandblasted to clean them. A chemical compound called Stripeze was used to dissolve and remove paint and rust as a semi-liquid.

Two components of the Terminal roof structure had to be addressed in the track level renovation. Eye bars provided the tension tie at the base of the three hinged arch trusses. It was obvious that some deterioration of the eye bars had occurred by oxidation. To evaluate the condition of the eye bars would mean placing a redundant system to take the tension forces and then removing and examining the eye bars. The design decision was to place a redundant system of rods anchored at each end to the arch bases. Anchor plates were attached to the back of the truss arch base and the rods passed through these plates into a nut on the back side. These 57 mm diameter A36 rods were tightened slightly so that any tension which developed at the arch bases would flow into the rods.

The truss arches were fixed at the east side but sat on rollers to form an expansion bearing at the west. These roller bars were 5 cm diameter with a center groove. The base shoe of the arch had a corresponding plate tab which fit in the groove. The six roller bars would then roll east or west as the structure expanded and contracted. Heavy oxidation was present on the rollers and shoe. A replacement system was needed to recreate the expansion bearing. Teflon™ bonded to stainless steel plates was chosen to form the joint. Teflon on Teflon would provide the expansion capability.

To place the Teflon bearing assembly would require lifting the expansion end of the truss arch. Dead weight of the existing arch was 55 kips which was a very reasonable lift. The structural drawings were prepared showing a lifting frame with hydraulic jacks to raise the truss. During the renovation, the lifts proceeded quickly. A truss arch base was raised by the hydraulic jacks on the lifting frame which allowed removal of rollers, cleaning of the bearing, and placement of the Teflon assembly. As an added precaution, these lifts were made after the tension rods were in place.

4.1 Column Design Review: Renovation or Upgrade

In our existing conditions study, we found a number of columns with reduced section at the Market floor due to oxidation. The loss was enough that capacity of some columns had to be limited to dead weight only above the Market. Plating was to be used at the Market floor to reestablish the column cross section. This work had to be phased with Market tenants and had to proceed the new construction within the Track floor above. Phasing of tenants was done by quadrants to allow a segment of columns to be repaired.

Exposed wrought iron columns in the Market required a rational fire protection system. A flooding technique with four sprinkler heads around each column both above and below the Market ceiling was proposed and accepted by codes. The columns below the Market did not have to be exposed so a concrete encasement was chosen. This concrete encasement allowed a composite column for added capacity and effectively fireproofed the wrought iron. To renovate each column, the Market floor was cut away to allow plating. The column encasement was cast and the Market floor was recast with concrete after the plating was completed.

4.2 Roof Systems

The original arch truss roof system was designed for the following conditions:

First: Snow (0.6 kPa) on one side + dead load
Second: Snow (0.6 kPa) on both sides + dead load
Third: Wind on one side (1.7 kPa vertical surface) + dead load
Fourth: Snow and wind (1.7 kPa vertical surface) one on one side; snow only on the other side + dead load

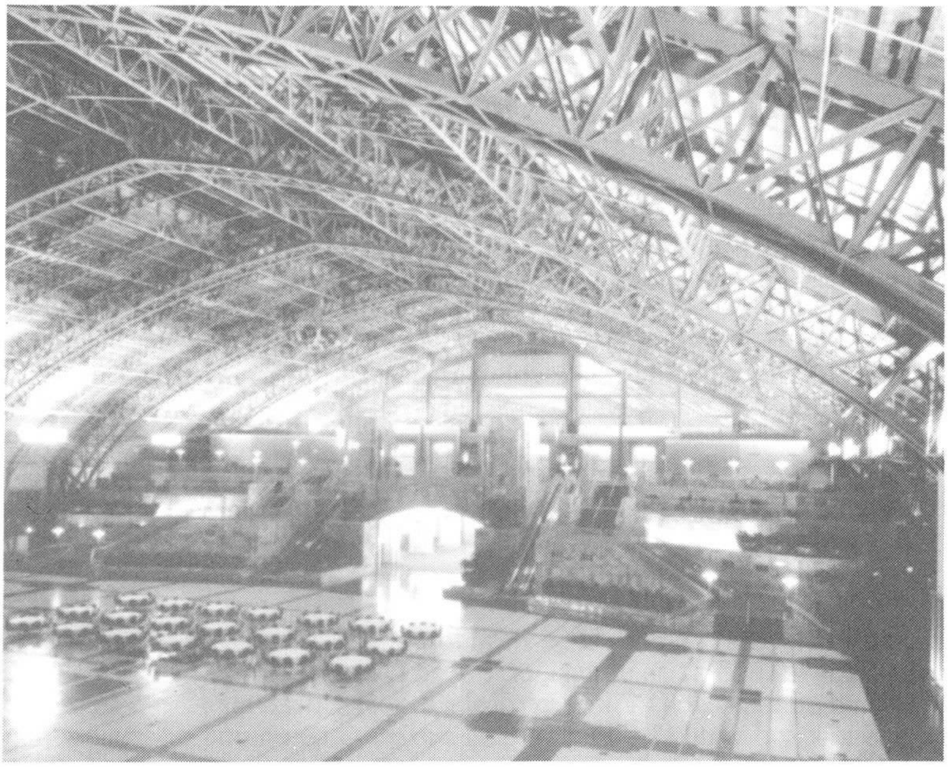
Current code requirements show ground snow load at 1 kPa, but considering exposure and roof slope, the minimum snow load is 0.6 kPa. The significant change is design for snow drifting at changes in roof elevation. Vertical faces at skylights result in drift loads. The roof eaves along the east and west sides have the roof drains in crickets. The parapet along the eave also creates drifting which calls for higher snow loads along the edges. Snow guards were placed on the roof to hold the snow in place and allow melting at the roof surface. The snow guards help prevent ice dams in the drainage crickets as long as they hold the snow. Wind loading is much different now than it was in 1893. Primary roof loading is negative in wind conditions. This was beneficial in the current analysis for load combinations.

Joseph Wilson set allowable tension for the wrought iron at 96.5 MPa and compression at 83 MPa. The wrought iron test specimens revealed a material reasonably consistent with ultimate strengths of 248 MPa (+) and yields of 228 MPa (+). Computer analysis indicated only a few members with stresses above the original 96.5/83 MPa allowable. A modest increase of allowable to 110/96.5 MPa provided a capacity envelope.

5. CONSTRUCTION ISSUES

During the existing conditions phase, the wood purlins, though stained, appeared sound from the bottom. We reduced our original estimate of 60% replacement based on this data. Unfortunately, the purlins had rotted from the top from the water migrating along the sheathing. As each three boards of sheathing stopped over a purlin, the water collected and promoted decay. Ultimately, just over 60% of the purlins were replaced based on rotted cores. Obviously, first thoughts were more accurate in this instance.

Other segments of the renovation and construction proceeded on course. Opening was in the spring of 1994 after 17 snow storms in the northeast during the winter of 1993. Snow loading was frequent and drifting along the eaves was present. The roof performed well and is ready for the next 100 years.



Riveted Steel Repair Methods at a Historic Train Station

Méthodes de réparation d'une structure en acier riveté dans une gare

Methoden zur Reparatur einer vernieteten Konstruktion an einem Bahnhof

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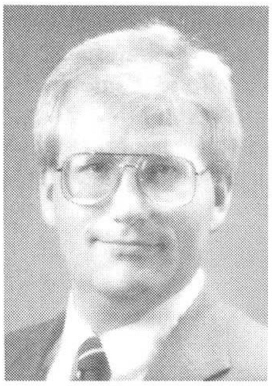

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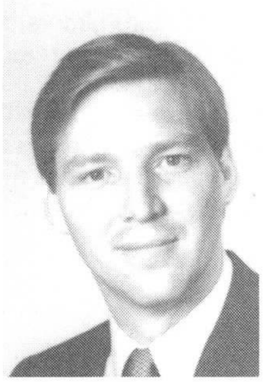

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SUMMARY

This paper describes the variety of methods used to repair the 85-year-old riveted steel in a historic train station in downtown Chicago. Exposure to salt-laden moisture took a severe toll on the steel members. Corrosion damage was extensive; complete flange angle deterioration and large web rust holes were a common occurrence. The steel rehabilitation approach on this project is addressed. Factors influencing repair need and guidelines for repair selection are provided. Examples of repair design and details are described, including a discussion of welding criteria, replacing rivets with bolts, composite construction and supplemental members.

RÉSUMÉ

L'article présente différentes méthodes utilisées pour la réparation d'une structure en acier riveté, construite il y a 85 ans dans la gare ferroviaire du centre de Chicago. L'exposition à une atmosphère humide saline entraîna la corrosion de nombreux éléments en acier. Les auteurs traitent de la réhabilitation de la partie métallique de l'ouvrage; ils indiquent les facteurs qui influent sur la réparation et sur les directives du choix de la réparation; ils décrivent diverses études de remise en état, traitent des critères de soudure, des possibilités de remplacer les rivets par des boulons, une construction composite et des éléments complémentaires.

ZUSAMMENFASSUNG

Dieses Referat beschreibt die verschiedenen Methoden, die zur Reparatur für den 85-jährigen vernieteten Stahl im Zugbahnhof der Innenstadt Chicago benutzt wurden. Die Aussetzung an der salzhaltigen Atmosphäre hat schweren Schaden an vielen Teilen angerichtet. Der Ansatz zur Stahlreparatur wird gezeigt. Die Faktoren, die die Reparatur und die Richtlinie für Reparaturenauswahl beeinflussen, sind erwähnt. Beispiele für Reparaturentwurf und Einzelheiten sind beschrieben, eingeschlossen ist eine Besprechung über Schweiss-Kriterien, das Ersetzen von Nieten mit Bolzen, gegliederten Aufbau, und zusätzlichen Teilen.

1. DESCRIPTION OF STRUCTURE

Built in 1910, the Chicago and North Western Trainshed covers a 2½ square city block area in downtown Chicago. The downtown terminus of the C&NW/METRA commuter rail lines is a two-story structure serving 200 trains per day, which enter one story above street level on the first supported level. To accommodate 45,000 daily commuters, the historic trainshed building is being rebuilt in nine construction phases under a four-year, $72 million (US) renovation project. Construction phasing is commencing in longitudinal slices so only 3 of 16 tracks are out-of-service at any one time. Each construction phase includes canopy replacement and rehabilitation at track and street levels.

The trainshed structure is constructed of built-up, riveted steel members. Transverse bent lines, typically spaced on 7.8 m (25 ft-6 in.) centers, consist of double web, floor beams spanning between columns. These double channel columns below each track are founded on "wedding cake" pedestals supported by timber piles. Track stringers, curb girders, and platform stringers forming a built-up, I-section frame into the floor beams (Figure 1). In most areas, a track stringer is located below each rail and a 33 cm (13 in.) concrete slab forms the track deck. Canopy columns, located at the platform centerline, fit between floor beam webs at the bent lines. The train boarding areas are covered with a 2.4 hectare (6 acre) canopy structure containing smoke slots for engine exhaust. Ballasted track supported by a corrugated steel and concrete deck is used at the structure's north end where the individual tracks merge. The structure is enclosed with granite and brick masonry walls.

2. NATURE OF DETERIORATION

With limited maintenance, 85 years of exposure to moisture and salts took a severe toll on the steel members. Deterioration of the trackbed and platform concrete led to moisture leakage through the track level. The built-up, riveted steel members provided several horizontal surfaces, crevices, pockets, and gaps for salt-laden moisture collection, which led to extensive corrosion damage.

Corrosion damage from the top-side moisture migration was often severe but highly localized. Common corrosion damage included complete flange angle deterioration, rust holes or extensive section loss on floor beam webs, and section loss at critical connections. Overall section loss on some members caused enough loss in member properties that buckling, or severe deformations and distortions resulted.

Street level corrosion damage was also a problem. In each street viaduct, salt water splashing on the exposed steel columns gradually reduced their cross-sections at the ground line. Moreover, water flowing through track level breaches oftentimes collected at interior column bases, sometimes causing rust holes.

To address the steel damage from corrosion, an extensive steel repair program was planned and later refined as the project progressed. As discussed in the next section, several factors guided the decision on where to call for repairs.

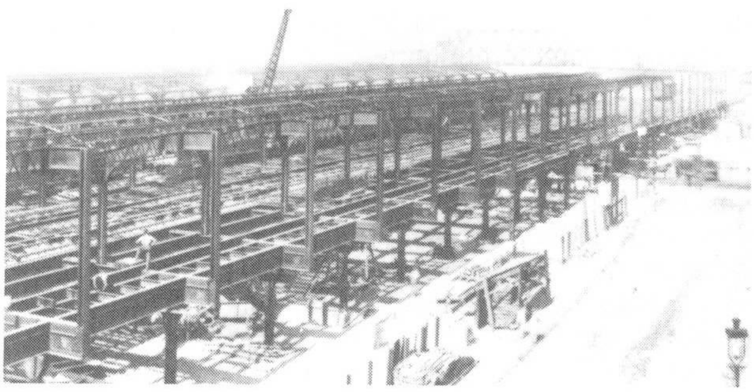

Fig. 1 C&NW Trainshed looking along the east side (circa 1910)

3. DECIDING WHERE REPAIRS WERE NEEDED

3.1 Structural Review

It was very important to have a sense of the reserve strength at typical locations when deciding on repair necessity. For example, we found that the original design was generally conservative, and therefore some strength loss could be accepted without repair. On the other hand, some new canopy column loads were greater than the original. In these locations, significant strength loss could not be tolerated.

3.2 Effect of Corrosion on Strength

The effect of section loss on strength of riveted steel is not always obvious. At a typical floor beam, a 75 percent horizontal leg thickness reduction in the top flange angles reduces the section modulus (S_x) by only 21 percent with no effect on shear strength. However, this section loss could generally not be tolerated because the flanges would be prone to compression buckling and out-of-plane bending from top flange loads.

It was also difficult to evaluate the effect of corrosion of rivet heads, which sometimes occurs before appreciable section loss of the adjacent steel. Testing by the authors in conjunction with a prior investigation[1] showed corrosion of rivet heads had very little effect on shear strength provided the rivet shanks were still intact.

3.3 Aesthetic Considerations

From a structural standpoint, web rust holes in low shear areas and flange deterioration near a simple support can be tolerated. However, the appearance and public perception of rust holes in a recently rehabilitated facility was not acceptable. Also, there is a possibility that future inspectors may not reach the same conclusion as to the structural necessity for repair. For these reasons, rust holes were almost always repaired where the member would remain exposed to view.

3.4 Inspection

A reasonably thorough inspection of the trainshed was made before repair work began. However, most members were at least partially concealed, and where exposed, thick layers of rust byproducts generally concealed the deterioration. It was therefore anticipated that final determination of repair type and location would be made after the steel members were exposed *and* sandblasted to remove rust by-products. In some locations, the contractor prime painted immediately after sandblasting to avoid re-sandblasting flash rust. The prime-painted surfaces were easiest to inspect, although a sandblasted surface could be inspected with close access and good lighting. The degree of corrosion damage could then be accurately assessed visually, sometimes with the aid of simple tools such as a straight edge and calipers.

4. REPAIR GUIDELINES

A repair decision usually comes down to finding the most economical repair that provides an acceptable level of strength, durability, and appearance. If the repair work effects the project schedule, fabrication and installation time becomes an extremely important factor. There were four basic repair approaches that recurred on this project: *1) fix the damage, 2) make the member composite, 3) replace the member,* and *4) find another load path.* To decide among these alternatives, the following guidelines held true for the trainshed repairs:

- *If the damaged area was accessible, it could be fixed* - Provided there was access, the ironworkers skillfully installed intricate welded and bolted repairs.
- *Where possible, make the member composite* - Shear studs are inexpensive compared to structural steel repairs. Of course, enough steel must remain for stud welding.
- *If a badly damaged member comes out easily, replace it* - Provided the member could be easily removed, it proved less expensive to replace an entire member than make extensive repairs, due to high labor costs involved in the repairs. On the contrary, it was all but impossible to replace a majority of members that were partially encased by the track deck or framed into several other members.

- *If there is no other choice, examine alternate load paths* - Much of the corrosion damage at the trainshed occurred in inaccessible regions of members that could not be easily replaced. In these instances, the most viable solution was to bypass the damage by providing a load path to supplemental members.

The following sections describe some of the difficulties with the steel repairs and their solutions.

5. REPAIR DETAILS

5.1 Welding to 85-Year-Old Corroded Steel

Provided the live-load stress range was low, welded doubler plates and stiffeners worked well to repair corrosion-damaged areas where the deteriorated steel had few interferences from rivets, connections, or other framing members. Metal thickness beyond the deterioration had to be sufficient for welding. If corrosion pitting was minimal, minor grinding was used to remove paint and surface rust. Heavier pitting on the existing steel required more aggressive surface preparation for welding. Heavy grinding with disc grinders was employed to even out surface amplitudes and building-up weld metal was allowed to fill gaps. Testing revealed the 1910 vintage trainshed steel had a chemistry closely matching ASTM A7[2] steel. A higher-than-usual phosphorous content required an adjustment in welding procedures.

5.2 Field Bolting

Areas with rivet or connection interferences lent themselves to combination welded and bolted plates, or completely bolted repair plates. Heavily pitted or moderately distorted steel usually precluded welded plate use. In the latter case, bolted repairs were not only used to strengthen an area, but also to draw the steel plys together, thus reducing distortion.

Reusing existing rivet holes for bolting provided an efficient repair means to address difficult repair situations containing many interferences, as shown in Figure 2. The rivet pattern was field measured before fabricating repair plates or flange angles. Additional bolts and interior stitch fasteners used on these repairs were field drilled. On bolted repairs without existing rivet holes, new holes were field drilled in the existing steel using the new repair plate with holes as a template.

The shear strength of high-strength bolts is greater than the existing rivets. Bearing connections using high-strength bolts, with and without shear plane thread exclusion, were often specified to increase

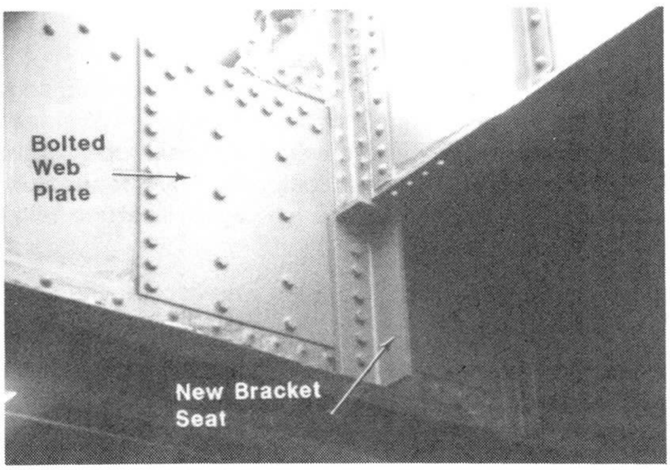

Fig. 2 *Bolted web and bracket seat repairs using existing and new holes*

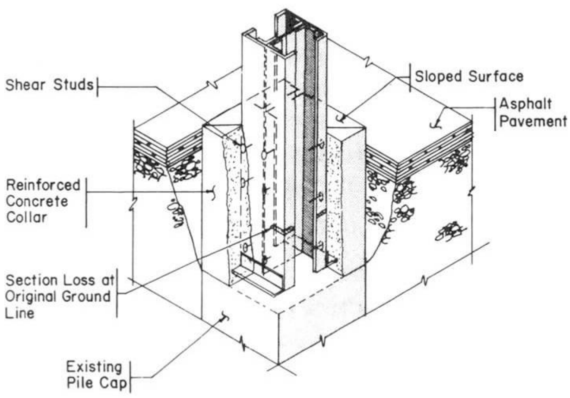

Fig. 3 *Composite concrete column encasement*

connection capacities at several repair locations. Bolted connections were also used on doubler plates and flange angle replacements.

5.3 Fatigue Considerations

On train-carrying members, where train live load was the predominant stress, the use of welded repairs was avoided. Even though live load stresses were below accepted allowable levels for fatigue, there was concern about residual stresses in the welds and their influence on the existing steel. Bolted repairs were the predominant repair type on these members.

Welded repairs were used throughout the trainshed for the past 30 years, some in tension zones where the live load stress range was relatively high. Although the fatigue resistance was dubious, these repairs performed well where the weld quality was good. The only significant crack found at the trainshed occurred at a poor quality butt weld between two existing patch plates. The butt weld cracked and drove a crack through a *compression* flange to which the plates were welded.

5.4 Composite Construction

Making a steel member composite with concrete is an effective and inexpensive means of repair because welded studs are inexpensive. Two repair types used this principal quite extensively.

To address column base corrosion, the column bases were excavated to their foundation and steel studs were attached. Concrete column encasements were then cast to the final grade elevation, while founded on the footing. The composite column encasement provided a supplemental load path for any minor section loss and gave the deteriorated column steel additional protection (Figure 3). At locations of severe section loss, welded and bolted repair plates were installed prior to stud installation.

The other composite repair was located on curb girders. The top half of these members were designed to be encased in concrete curbs that would later support one precast platform edge. Demolition around the curb girders revealed heavy top flange section loss on several members. To increase the member capacity, additional studs were installed to make the curb girders fully composite with the curb.

5.5 Member Replacement

At the trainshed, steel members that were candidates for replacement were platform stringers and floor beams in the platform area. During in-phase demolition, the existing platform was demolished, thus exposing these members. Replacement members were easily installed by lowering them into position. Little or no shoring was required.

5.6 Supplementing Members

Supplementing members to strengthen deteriorated connection regions or provide alternate load paths was a repair method used on numerous floor beams at the trainshed, as an economical alternative to member replacement. In its most basic form, this repair consisted of adding bearing brackets or stiffeners below deteriorated framed beam connections (Figure 2).

Figure 4 illustrates an example of two innovative means used to supplement the column to floor beam connection region by providing an alternate load path. At many locations, this connection experienced various degrees of deterioration, and using standard repairs was very impractical. Each trainshed column contained a knee brace plate directly below the floor beam connection. Instead of using a bolted web plate in this region with extensive shoring, the knee brace plate was stiffened with angles to give an alternate load path. Besides providing lateral resistance, the knee brace plates were activated to carry a portion of the gravity load.

Another repair shown in Figure 4 consisted of placing a channel against the deteriorated floor beam web. This repair supplemented the existing connection region by providing an alternate live load path that bypassed the original connection; additional live load was transferred to the stiffened knee brace below.

New sister girders (Figure 5) were used where web deterioration on floor beams was quite extensive, and transverse framing members and the concrete trackbed precluded using bolted web plates. The sister beams were installed from below when the track above was closed.

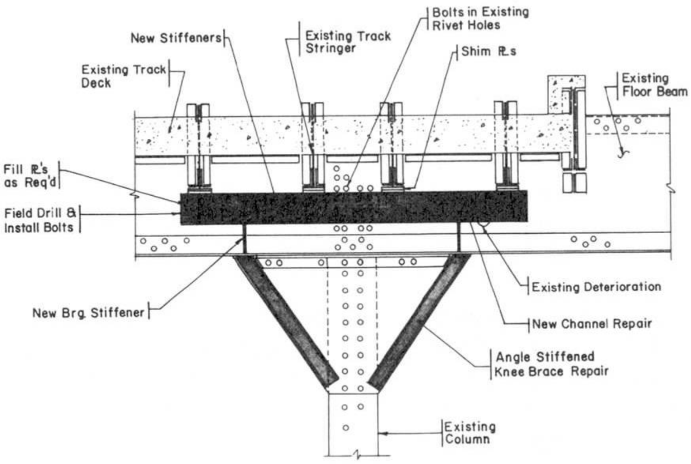

Fig. 4 *Alternate load path repair utilizing a channel and stiffened knee brace plate*

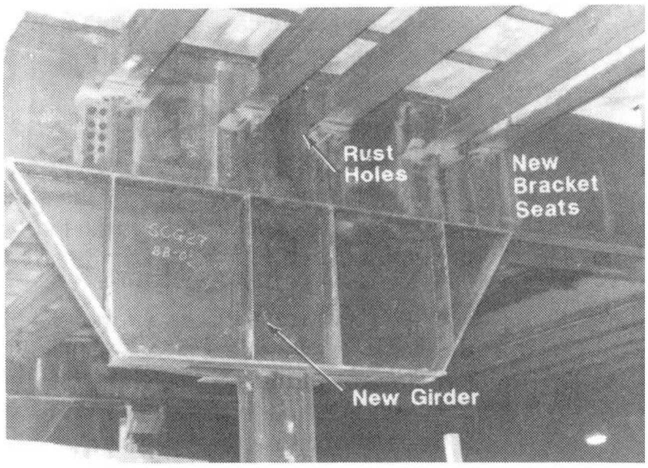

Fig. 5 *Supplemental girder below the existing floor beam*

6. FUTURE PROTECTION

To insure the repairs will last, several protection means were employed:

- Cleaning the entire track deck framing steel to grade and repainting with a two-component epoxy coating. Viaduct areas are receiving a polyurethane top coat for added protection.
- Replacing the entire cast-in-place concrete passenger platforms with new precast platform panels with a "belt and suspenders" joint seal system.
- Casting a new, high performance concrete overlay on the trackbed. A heavy-duty, urethane waterproofing system is applied atop of the overlay where the trackbed is exposed. In the ballasted area, the track deck and overlay are protected with a fully adhered, butyl-rubber membrane below the ballast.

7. ACKNOWLEDGEMENT

The authors wish to acknowledge the support of METRA (Metropolitan Rail), the owner of the historic trainshed. Their initiative in rehabilitating this structure to modern standards while preserving the past is commendable. Also, the authors wish to thank the project architect, Harry Weese Associates; the Construction Manager, CRSS Constructors; and the General Contractor, Mellon-Stuart MKK Joint Venture.

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Strengthening and Repair of a Large Industrial Building

Renforcement et réparation d'un large bâtiment industriel

Verstärkung und Reparatur einer grossen Industriehalle

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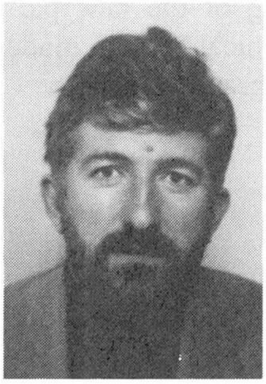

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SUMMARY

The strengthening and repair of a large industrial building for cold mill plant (over 100'000 m2), built in structural steelwork, is presented. The main columns and crane girders were strengthened because of the increased loading due to the change of plant technology. The structural components of the industrial building were repaired because of damages due to a fire. The applied strengthening and repair interventions are described.

RÉSUMÉ

Le renforcement et la réparation d'un large bâtiment industriel pour une usine de laminage à froid (surface de plus de 100'000 m2), en charpente métallique, sont présentés. Les colonnes principales et les poutres de roulement sont renforcées pour supporter les augmentations de charges, dues aux changements de la technologie. Les éléments structuraux du bâtiment sont réparés pour pallier les détériorations causées par un grand incendie. Les interventions de renforcement et de réparation, appliquées à la charpente métallique du bâtiment, sont décrites brièvement.

ZUSAMMENFASSUNG

Es werden Verstärkungen und Reparaturen einer grossen Industriehalle dargestellt. Die aktuelle Grundrissfläche der Stahlkonstruktion beträgt über 100'000 m2. Zuerst wurden die Hauptstützen und der Krahnbahnträger wegen der Lasterhöhung durch die Produktionsänderung, verstärkt. Dann wurden auch verschiedene Elemente der Tragwerke repariert, diesmal wegen grossen Brandbeschädigungen. Die verwendeten Verstärkungen und Reparaturen werden dargestellt.

1. STRENGTHENING INTERVENTIONS

1.1 Introduction

The multi-bay building for cold rolling mill plant "MKS" in Smederevo (Yugoslavia) was built in steelwork. This industrial building, having original area 60747 sq.m, was constructed twenty years ago according to the applied technological process of that time. Because of the introduction of the new technology, the industrial building was strengthened, reconstructed and enlarged by new useful area of 43.594 sq.m. The new technological process requested the significant increasing of total number of cranes (from 18 to 37) with greater (40-110%) bearing capacities and more intensified working regimes; that resulted in the strengthening of the main columns and crane girders of the original building structure. The strengthening and reconstruction of the original industrial building as well as the building of the new part (second stage) was designed by "Projmetal Belgrade", consulting and technical control made by Institute for materials and structures of Civil Engineering Faculty of Belgrade University.

1.2 Structural Steelwork of the Industrial Building

The plan view and the sections of the industrial building (first stage is shaded part of plan drawing), having unite total useful area of 104.241 sq.m and structural steelwork of total weight 17.000 t, is given in Fig. 1.

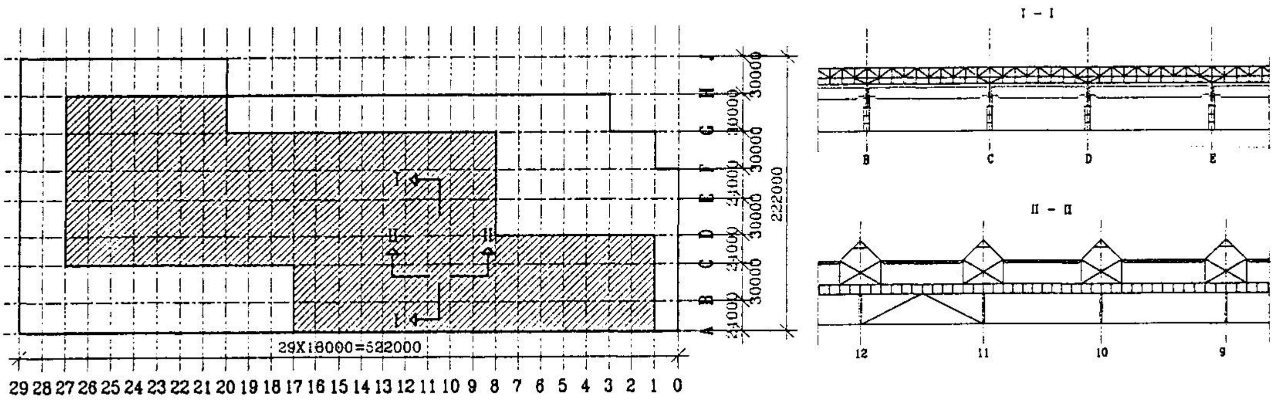

Fig. 1 Plan view and sections of the industrial building

The main columns of the industrial building are spaced on 18,0 x 30,0 m and 18,0 x 24,0 m. Two roof trusses (of span 30,0 m or 24,0 m each), inclined at the angle of 45 °, are coupled forming the transverse lanterns (of width 6,0 m and height 3,0 m) spaced on 18,0 m. The roof trusses are carried by the braced supporting structures, located on the gantry girders; consequently they are not supported directly by the main columns. The purlins, spanning 12,0 m, spacing on 3,0 m, are supported by the roof trusses. The crane girders are constructed as plate girders (depth 2000 m), having static system as continuous beam on n-supports; the distance between the expansion joints defining its length. The entire structure of the industrial building is separated in three parts by the expansion joints, at the column lines 8 and 17.

The roof cladding consisted of profiled sheets, made from aluminum (first stage) and from steel (second stage) combined with the polyurethane insulated panels. The slope of the roof surface (except the pitch-parts with the lanterns) amounted 1,7 % in the drain direction, and it was formed by the longitudinal inclinations of purlins. The wall cladding (second stage) consisted of the "Luxalon" panels, made as aluminum sandwich with foamed polyurethane core as thermal insulator.

1.3 Strengthening of Main Columns

The partial strengthening of main columns F8-F27 as well as the strengthening of column bases (transformation into box section) of main columns D9-D17, E8-E17 were carried out (Fig. 2). The additional longitudinal web stiffeners were welded in all columns. The amount of the additional steel material for the main columns is 44,4 t (i.e. 4,1% more steel in columns).

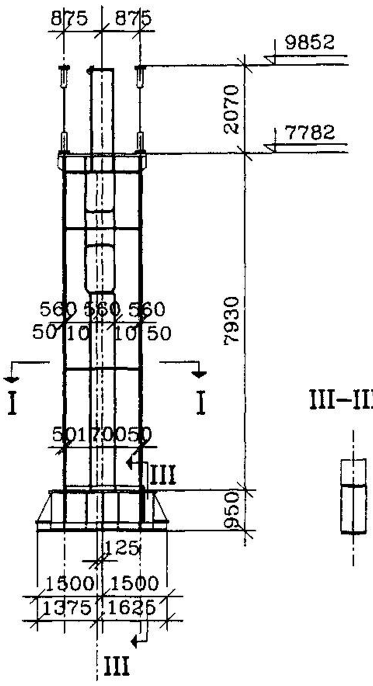

Fig. 2 Strengthened main column

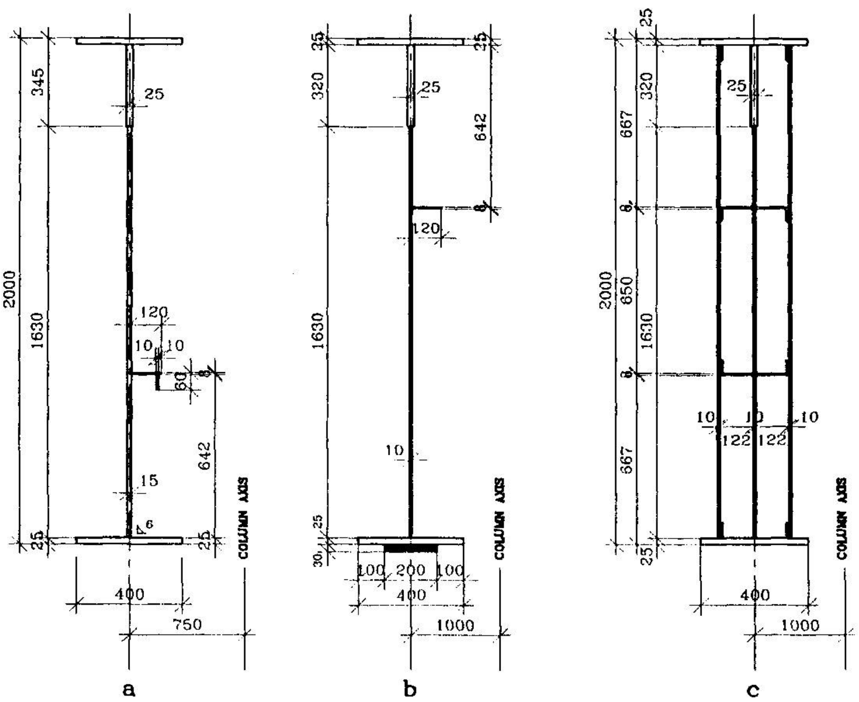

Fig. 3 Strengthened crane girders

1.4 Strengthening of Crane Girders

The numerous different types (31) of the crane girders (total number 502), were applied in the original building. Thus the crane girders were classified in three types, concerning the strengthening works. The type "a" represented the crane girders (411 pieces) where the additional elements were put in for the longitudinal web stiffeners; the crane girders of type "b" required the additional plate to be welded on the bottom flange; the transformation of plate girder into box girder was applied to the crane girders (81 pieces) of type "c" (Fig. 3). The expansion joints of crane girders were strengthened as well. The additional steel material for the strengthening of crane girders amounts 256,1 t (i.e. 10,0% more steel in crane girders).

1.5 Conclusion Remarks

The all strengthening and reconstruction works were carried out practically without any interruption of the current plant production. Having applied the adequate methods of design and optimal constructing procedures, the effective solution in the functional, technical and cost benefit aspect was attained.

2. REPAIR INTERVENTIONS

2.1 Damages of the Building Caused by Fire

The second building stage, was about to finish when the fire accident occurred and caused a lot of damages of structural steelwork and equipment as well. Next day after the fire accident the special technical commission for building rehabilitation was formed by team of experts (including the co-authors of the paper). The detailed geodesic measurements were carried out in order to determine exactly the actual state of the structural steelwork. The special photogrametric method from air (by plane) was used as well.

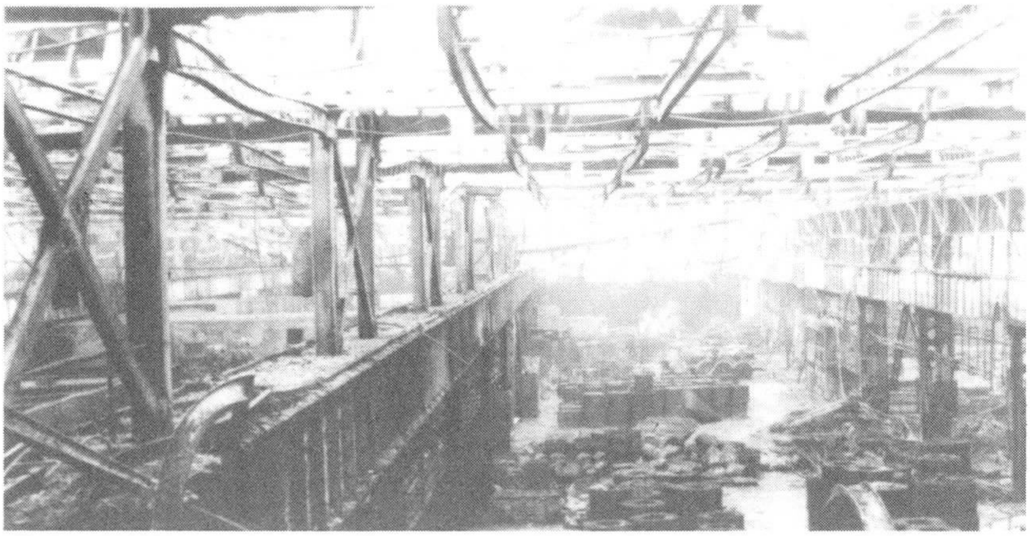

Fig. 4 Building structure damaged by fire

The following damages were noted:
- Roof cladding is either completely destroyed (aluminum profiled sheets), or partly damaged remaining on the roof (steel profiled sheets).
- More than 90% of purlins are largely deformed and they should be changed.
- About 50% of roof trusses and supporting roof structures were significantly deformed (Fig 4.). The roof trusses in the zone of the greatest damages (bays: D-E, E-F and F-G between the columns 20 - 28) fell down.
- The global and local deformations of the main columns were noticed, especially large deformations were present in the column row F, where the crane girders were located on different levels, with the presence of the column twisting consequently.
- The crane girders underwent the significant global and local deformations. The expansion joints were open excessively, some of them even more than 500 mm.

- The brake bracing structures, especially in the field 21-22, got excessive deformations, that somewhere resulted in the member ruptures.
- The wall cladding was damaged in the large area of external walls.

2.2 Repair of Roof Structure

Based on the analysis of test results concerning fire resistance, spread of flame and propagation of harmful gases, the new roof cladding (sandwich made from profiled steel sheets and "vunisol JM" insulated panels), having less self weight is applied. The retained old purlins were repaired and reconstructed (Fig. 5a). The destroyed purlins were replaced by the new type of purlins (Fig. 5b). These new types of purlins enabled the transformation into pitch-roof surfaces (of slope 10%) between the lanterns (having other pitch-roof transparent surfaces of slope 45 °), that improved the drainage from the roof.

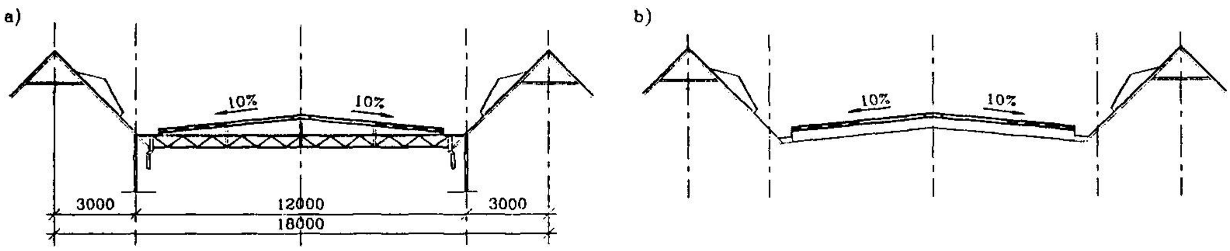

Fig. 5 Rehabilitation of purlins

The increased tolerances concerning the out-of-straightness were specified for the members of roof trusses. The additional flexural moments (with respect to weak bending axis) were introduced for stress control, because of the specified tolerances. All compression members that could not sustain the specified limit eccentricity (e<L/500) were strengthened by welding the additional plates. Where the greater eccentricities (e>L/500) were registrated, the member were straightened or replaced. The limit eccentricity of L/100 was specified for all tension members for stress verification, and the members with greater values of eccentricities were either straightened or strengthened. The repaired roof structures were tested.

2.3 Repair of Main Columns

The repair treatments were defined after the detailed inspection and geodesic measurements. The main columns were classified into two categories:
a) Main columns where the damaged crane girders are disassembled - " free columns ",
b) Main columns carrying the crane girders -"linked columns".
The geometry of the "free columns" was adjusted up to the increased tolerances (1.5 times grater than the allowable values), concerning the deviation of column axis from the vertical. The "linked columns" were submitted to straightening up to the specified tolerances, or if it is not possible the columns were strengthened. The correction of column geometry was done by heat treatment only. The strengthening concept comprised the transformation of the column from I - section to box section (Fig. 6).

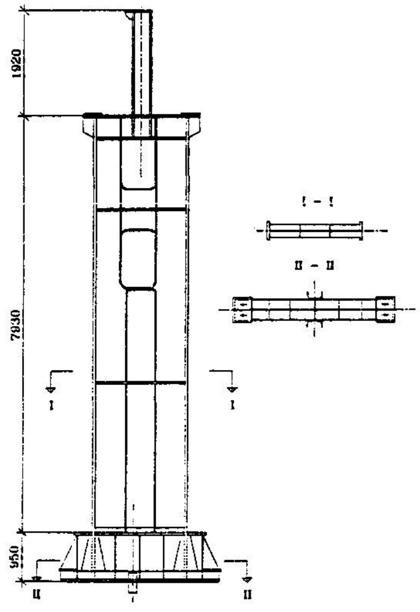

Fig. 6 Repaired main column

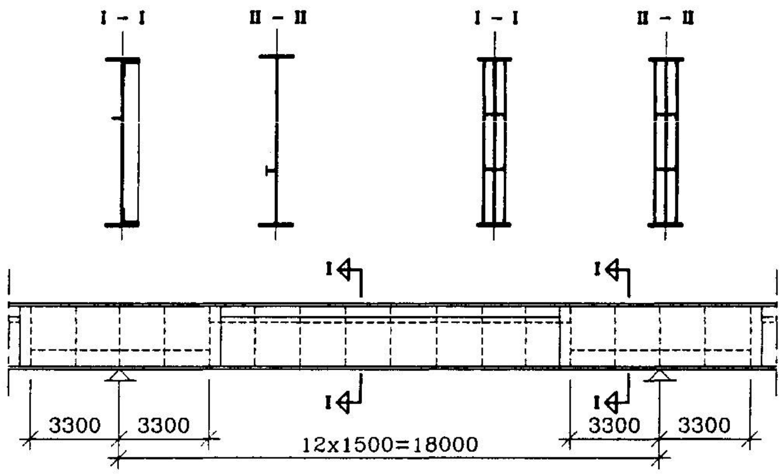

Fig. 7 Repaired crane girder

2.4 Repair of Crane Girders

The tolerances for crane girders were specified in the following increased amounts: - L/50, i.e. 36 mm, concerning the out-of-straigthness in horizontal or vertical direction, - b/200, i.e. 7 mm, concerning the web buckling. The other tolerances, including the deviations of distances and levels between the rails, were taken according to the code. The detailed geodesic measurements were done (measurement spacing in L/4), concerning the following registrations: the axis distance between the rails, the out-of straightness of upper flange in horizontal plane, the deviation of crane girder in vertical sense, the local deformation of the web (out-of-plane displacements) and the relative deviation of upper flange with respect to lower flange. Based on the geodesic measurements, it was tried to adjust the crane girders (to the position that fulfill the prescribed tolerances) by the heat treatment and by the transversal movements at the supports. If it was not possible, and where the excessive measured eccentricities required the strengthening of section due to stress or stability reasons, the I-section was transformed into box section by adding the vertical plates (Fig. 7). The unrepairibly damaged crane girders (42%) were replaced. The expansion joints, open after fire, were repaired. The repaired crane girders were tested by static and dynamic loads.

2.5 Conclusion Remarks

The rates of the rebuilt, repaired and retained components of main structural steelwork are:
- purlins (94% rebuilt, 6% retained), roof trusses (25% rebuilt, 42% repaired, 33% retained)
- main columns (28% repaired, 72% retained),
- crane girders (42% rebuilt, 25% repaired, 33% retained).

The overall rehabilitation works can be illustrated by the following data:
- 5.000 t of ex-steel elements disassembled and the same quantity of new steel structural elements rebuilt and reassembled, 3.000 t of steel structural elements repaired and reerected,
- 70.000 sq.m of new roof, i.e. 25.000 sq.m of new wall steel sandwich panels built and erected.

The all repair and rehabilitation works were carried out simultaneously in due time.

Repair of an Existing Prestressed Concrete Shell of a Mill in Egypt

Réparation d' un voile en béton précontraint en Egypte

Reparatur eines vorgespannten Schalentragwerks in Ägypten

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Magdi R.A. Youssef, born 1941, received his civil engineering degree at Ain Shams University in Egypt in 1964. He is now the head of the technical design office of a leading construction company in Egypt. He is a committee member for the Egyptian Code for Reinforced Concrete.

SUMMARY

The repair and strengthening of an existing prestressed concrete building has been performed by careful inspection of each element of the building, structural design and the choice of the best repair method. Use of external prestressed tendons and flat jacks has been mostly preferred.

RÉSUMÉ

La réparation et le renforcement d'une construction existante en béton précontraint a été réalisée sur la base d'une inspection attentive de chaque élément de la construction, du calcul statique et de la meilleure méthode de réparation. L'utilisation de câbles de précontrainte extérieure et de vérins plats a été choisie.

ZUSAMMENFASSUNG

Die Reparatur und die Verstärkung eines bestehenden vorgespannten Betonbauwerks wurde auf Grund einer genauen Untersuchung jedes Bauwerkselementes, der konstruktiven Bemessung und schliesslich der Auswahl der besten Reparaturmethode vollzogen. Es wurde vor allem Vorspannung und Spannpressen verwendet.

1-INTRODUCTION

Buildings constructed in EGYPT in the forties and fifties, specially heavy industrial factories & bridges have been exposed to environmental changes & different weather conditions along with changes of the levels & chemical properties of underground water . Such factors together with lack of regular maintenance, improper rainwater drainage systems & inadequate isolation of roofs have affected the buildings. Since then, regular examination of important establishments has proved to be of great importance to detect damage before it is too late. New methods of repair & and new materials were introduced to make buildings more durable. This paper deals with the repair done to a spinning & weaving factory constructed in 1956 in Egypt and has been affected by the aforementioned factors. In 1990 it was necessary to perform the repair work without suspending work and production in the factory.

2-DESCRIPTION OF THE PROJECT

A prestressed concrete shell 164.9 x 173.9 m. for a spinning mill that was constructed and later repaired by "The Misr Concrete Development Co.". The roof consisted of concrete shell 7 to 25 cm. thickness supported on truss diaphragms with post tensioned tie beams in the longitudinal direction and on valley beams in the short direction. There was also a concrete flat plate slab hanging from the shell roof by high tensile steel bars. The shell was supported on 64 columns hinged up and down as link members and 16 fixed columns on axes 7 and 8.The mill foundations were supported on piles. Fig.(1).

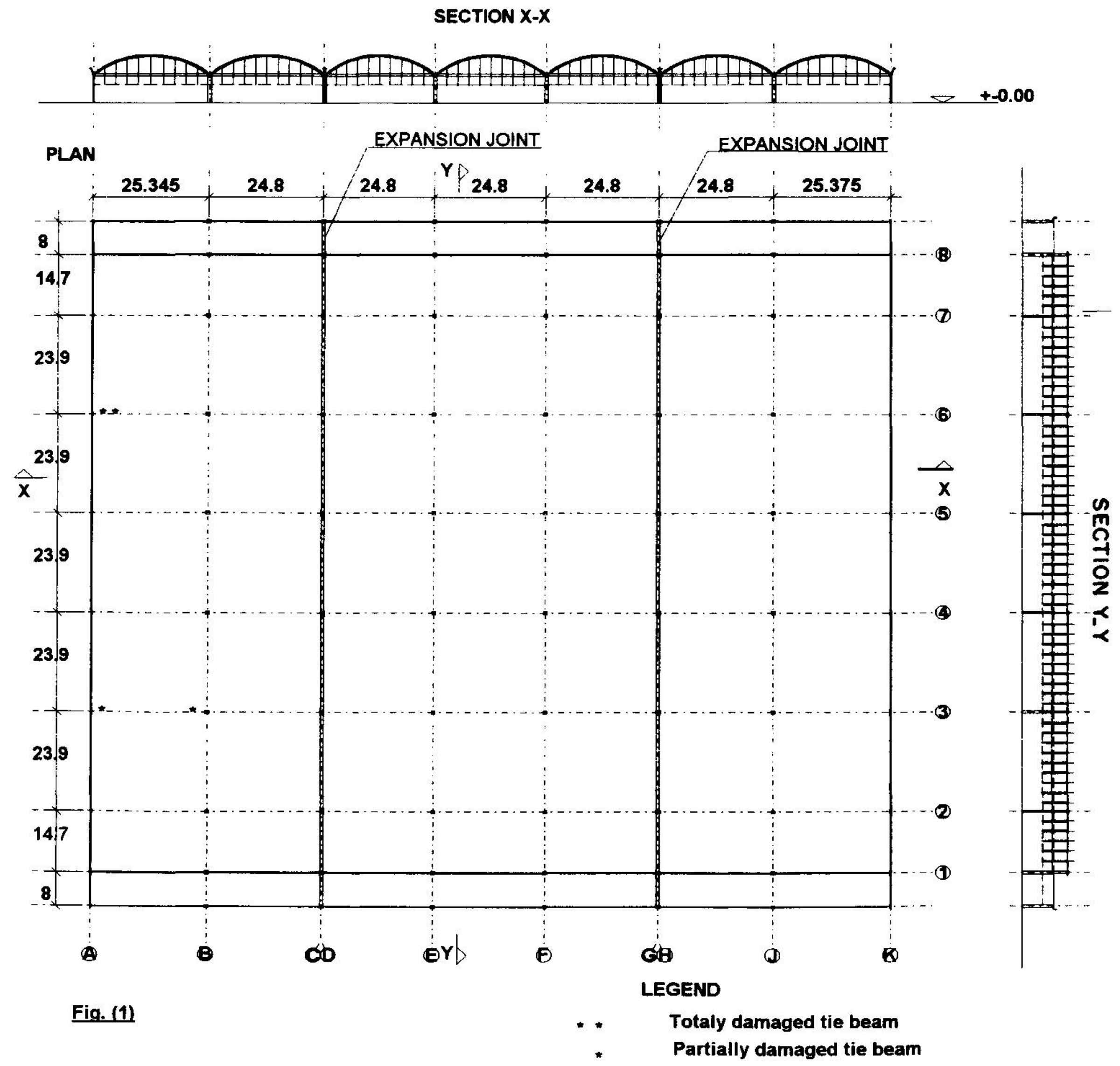


Fig. (1)

3-INSPECTION OF THE STRUCTURE

3-1 Inspection procedure followed:

3-1-1 All the cracks or fractures, damaged concrete, broken or rusty rebars and prestressing tendons were indicated on specific drawings for each element.

3-1-2 Site survey for the columns concerning the relative displacement between the lower and the upper edges as well as survey for the tie beams and the valley beams.

3-1-3 Scratching at random spots, (10x10x3 cm.) to show the condition of the prestressing tendons. Spots with undamaged tendons were directly covered by epoxy mortar.

3-1-4 Checking of two pile caps to investigate the condition of the foundations.

3-1-5 Defining the strength of the existing concrete for all elements by non destructive tests.

3-1-6 Checking the roof isolation and the expansion joints.

3-2 Inspection results

The isolated layers and the expansion joints filler were damaged causing leakage of rain water. Inspection of the roof showed presence of cracks, damage in concrete parts of the shell and corrosion of reinforcing bars and prestressing cables. Two tie beams were seriously damaged.

4-DESIGN

A finite element model for the whole structure (shell and columns) has been carried out by "Europe Etudes Gecti" of France [1] to check the stresses and displacement of the actual structure under permanent, superimposed loads and the effect of additional prestressing on shell element. The checking of the permanent and the superimposed loads on the existing structure showed that there were residual tension stresses (about 1MPa) at the tie beams and valley beams that might have been one of the causes of the cracks. Several repair methods and systems have been thoroughly studied and considered. Using external prestressing with flat jacks proved to be the most suitable system in this case. To minimize the displacement and the stresses at the ends of the columns, especially at the fixed columns during prestressing operation, two steps were followed using flat jacks inserted in the expansion joints. Fig.(4).

4-1 Percentage of the force on the tie beam due to an applied prestressed force

The finite element models show that 85% of the total applied prestressed forces are carried by the tie beams.

4-2 Strain value and stresses on columns due to displacement

step no.	distance from center line of building (m.)	87.5	62.5	37.5	12.5
	Column dimension (cm.)	40x75	75x75	40x75	75x75
1	Flat jack displacement (m.) = 0.005	0.005	0.005	0.000	0.000
2	Prestressed force (KN) = 1300	-0.00694	-0.00353	-0.00512	-0.00171
3	Flat jack displacement (m.) = 0.009	0.00206	0.00547	-0.00512	-0.00171
4	Prestressed force (KN) = 1300	-0.00957	-0.00305	-0.01023	-0.00341
	Displacement without flat jack prestressed force (KN) = 2600	-0.02387	-0.01705	-0.01023	-0.00341

Table (1) Columns displacement in meters.

Strain = stress / E = 0.85 P / A E
P = prestressed force
A = tie area = 0.75 x 0.30 = 0.225 m^2
E = Young's modulus (taking into consideration the age and strength of the concrete)
Strain / KN = 0.85 / 0.225 x 36 x 10^6 = 1.049 x 10^{-7}

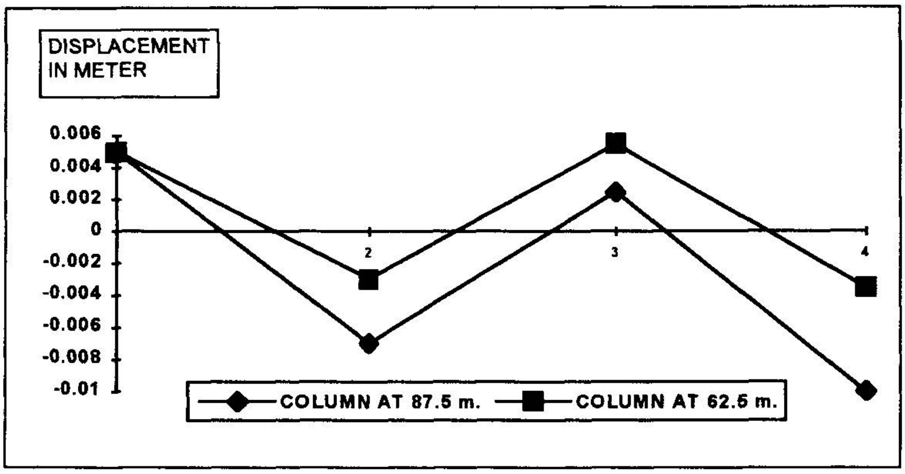


Fig.(2) : Displacement of the columns

From the final calculation of the displacement the stresses on the columns were calculated

5- REPAIR AND STRENGTHENING

5-1 Shoring

A suitable system of shoring was erected under all the damaged parts for the safety of the building and to allow the production to continue in the mill.

5-2 Repair of cracks

Epoxy resin with two components without solvent was injected for re-establishment of the structure's monolithism, for protection of the reinforcement bars and prestressed tendons against corrosion and for water proofing of the structure.

5-3 Repair of damaged concrete

Mortar with epoxy resin as a base was applied for the roof. Expansion joints were filled around the flat jacks with pure cement mortar with admixture to minimize shrinkage.

5-4 Protection of tie beams at the damaged parts

Before strengthening, repair of the damaged concrete was done for the tie beams where the concrete was damaged over a part of the total length. This repair consisted of constructing a concrete jacket that was post tensioned using mono-strand anchorage of compensated type. This jacket overlapped an undamaged part of the tie beam. Fig.(3)

5-5 Installation of the flat jack

The dished surfaces of the flat jacks were filled and leveled with a hard setting epoxy resin mortar. The copper piping, the valves and the manometer were connected to the flat jack. The flat jacks were then installed inside the expansion joints in the structure at the intersection of axes C,D and G,H with the axes from 1 to 8. The flat jacks were connected to one pump. Fig.(4) Detail (3)

5-6 Prestressing cables

Tie beams and valley beams were strengthened by external prestressing cables, consisting of unbonded plastic sheathed strands. All strands of one cable (4K15) were threaded in one steel tube of 76 mm. external diameter, and 2mm. thick. The cables were hung by temporary steel supports on the tie beams or the valley beams and were ended at concrete anchorage blocks on the outer grid lines. (For tie beams at grid lines A, K and grid lines from 1 to 8,for valley beams at grid lines 1,8 and grid lines from A to K). Fig (4).

PLAN FOR JACKET

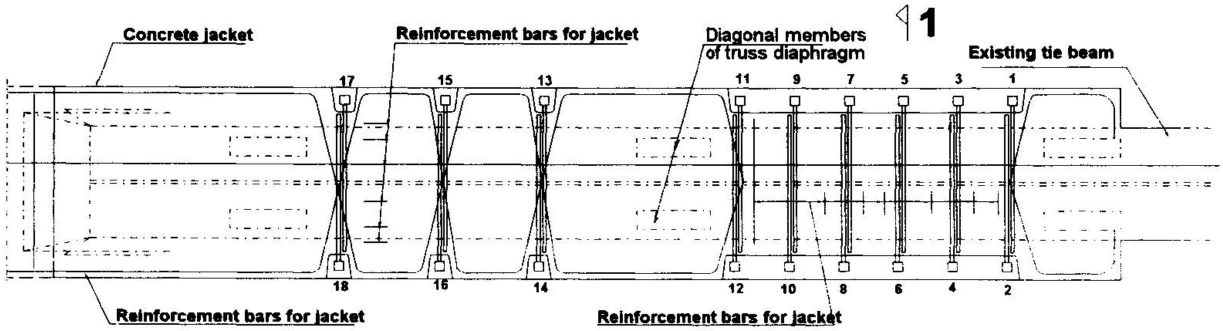
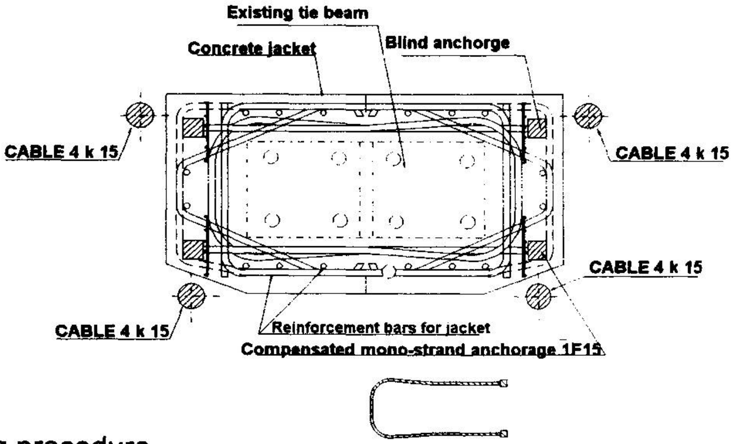


Fig.(3)

SECTION 1-1



5-7 Strengthening procedure

The following steps were followed

5-7-1 Tensioning of the strands of each tendon up to jacking pressure 50 bars.

5-7-2 Injecting the prestressing cables with cement grout.(the prestressing strands were individually greased and sheathed)

5-7-3 The first stage of flat jacks loading : each flat jack was inflated by 5 mm. they were then shimmed by wedges to maintain the inflation.

5-7-4 The first stage of tensioning of prestressed tendons by half the prestressing force up to 1300 KN. At this stage structural deflections were measured to check the assumed value for Young's modulus of elasticity for the concrete.

5-7-5 The second stage of flat jacks loading : each flat jack was inflated by 9 mm. then they were shimmed by wedges.

5-7-6 The second stage of tensioning of tendons by full prestressing force up to 2600 KN.

5-7-7 Injecting the flat jacks by epoxy resin.

5-7-8 Tensioning of valley beams tendons.

5-8 Isolated layers

All the isolated layers over the valley beams were renewed to protect the shell roof .

SCHEMATIC DETAILS ARRANGEMENTS

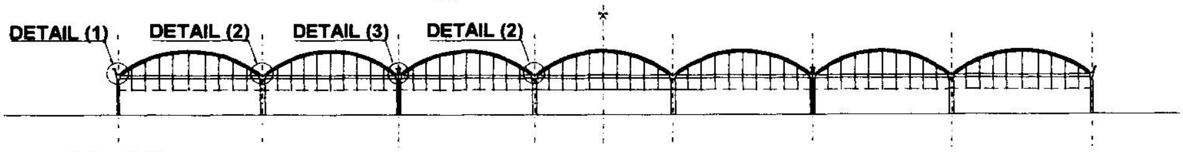
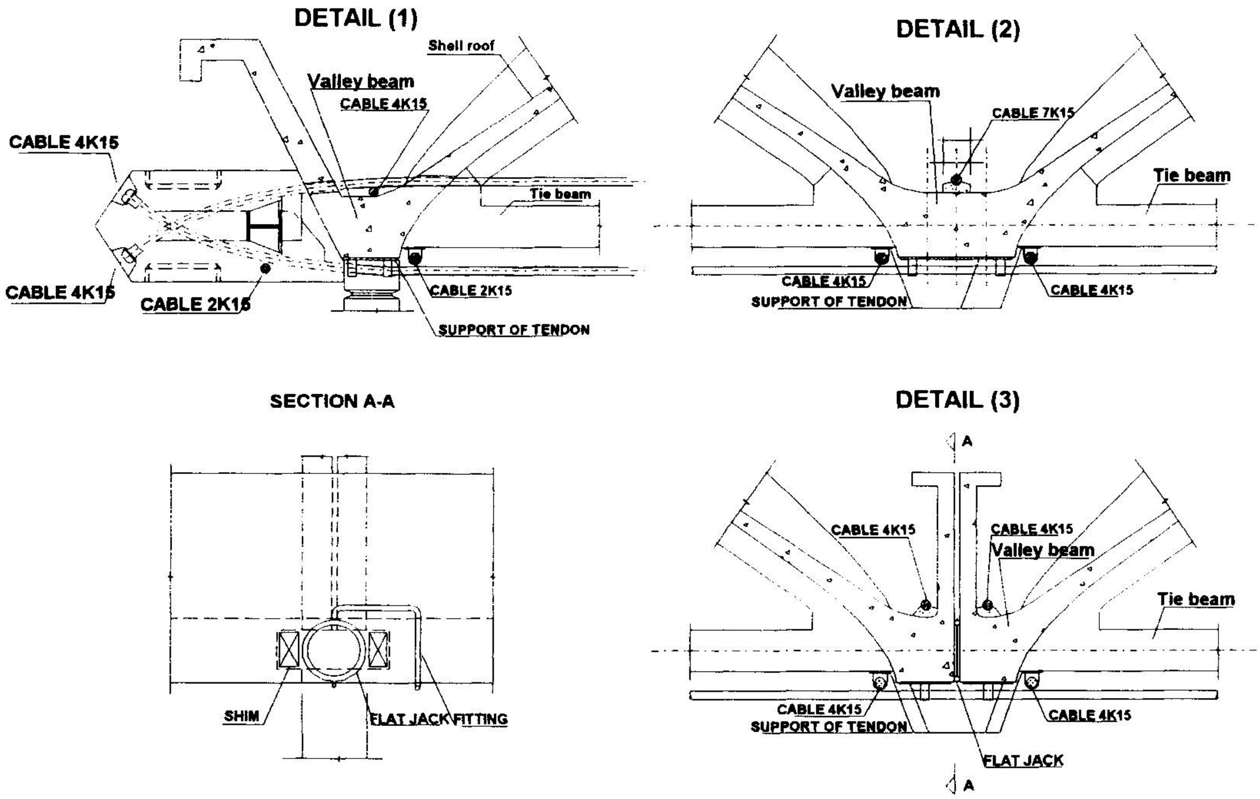


Fig.(4)



6-CONCLUSION

Repair of existing concrete structures using external prestressing forces is considered a procedure that needs a high degree of accuracy .Measurement of deflections and displacement during all stages should be under strict control. If the calculated displacements are different from the measured ones, the tensioning forces for the prestressing of tendons and the inflating pressures for the flat jacks must be modified accordingly

REFERENCES

[1] A.G. LABIB and M.H.C. ADIB ,finite element model for the shell roof of Mill in Misr Mehalla Co. Europe Etudes Gecti of France 1990

Repair of a Building Damaged by Foundation Settlement

Réhabilitation d'un bâtiment endommagé par des tassements différentiels

Sanierung von Gebäudeschäden infolge Setzungen

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Jaime Gusmao, born in 1932, received his civil engineering degree at UFPE, Brazil, and his graduate degree from the University of Illinois. For many years, Jaime Gusmao has been responsible for the design of foundations and research in soil-structure interaction.

SUMMARY

The paper describes a simple and practical solution used to compensate for settlement caused by different foundations at the same building. Fix supports were replaced by moveable ties on sand jacks that descended periodically in accordance with the settlement measured between the blocks The whole device, the basic control elements, the operative sequence for load transfer, the problems raised and the solutions given are presented in this paper.

RÉSUMÉ

L'auteur décrit une solution simple et pratique pour compenser les tassements différentiels se produisant dans un immeuble comportant divers types de fondations. Pour ce faire, des supports fixes ont été remplacés par des tirants mobiles posés sur des presses à sable, qu'il faut descendre périodiquement en fonction des tassements mesurés entre les différents blocs. L'article fournit de multiples informations sur l'ensemble du dispositif, les éléments fondamentaux de commande, l'ordre opérationnel de transfert des charges, les difficultés rencontrées et les solutions correspondantes.

ZUSAMMENFASSUNG

Der Beitrag beschreibt eine einfache und praktische Lösung, um Setzungsdifferenzen infolge unterschiedlicher Fundationsarten innerhalb desselben Gebäudes auszugleichen. Feste Auflager wurden dabei durch bewegliche Zugbänder auf Sandpressen ersetzt, die periodisch entsprechend den zwischen den Blöcken gemessenen Setzungen nachgelassen wurden. Die gesamte Vorrichtung, die grundlegenden Steuereinrichtungen, die operative Reihenfolge bei der Lastumlagerung, die auftretenden Problemen und ihre Lösung werden behandelt.

1. INTRODUCTION

One building that is a medical center in Recife, Brazil, started having cracks due to foundation settlement soon after the end of its construction. It was a reinforced concrete structure with two types of foundation. The central block is an eight floors tower having deep foundation of steel piles near 40 m long. The outter block has three floors in spread foundation at the depth of 0.85 m with columns spaced 6m per 5m and takes 90 % of the ground area. The blocks are separated by a joint but the beams of the lower block were set on consols at the columns of the tower. Figure 1 shows the general lay-out.

Damage was concentrated at the spans next the tower where the differential settlement was larger.The rest of the building had a satisfactory behavior. Cracks started at the mansonry walls and soon reached the structural beams showing the need of repair to overcome the design error.

Soil tests were made to predict the settlements that were found to be not less than 150 mm. The excessive cost of underpinning of the spread foundation by the use of deep piles postponed the execution of this solution. Then it was necessary to extend the lifespan of the structure until the definite repair was made. It was decided to relieve the structure using a settlement-control device by moveable ties hung from the tower. The ties allow to compensate the settlement already occurred as well as the settlements that may occur later on. The paper outlines some features of the solution.

2. BASIC CONCEPTION OF THE SOLUTION

The basic conception of the temporary solution is to compensate the settlement due to consolidation of the 18 m thick layer of soft organic clay. The solution implies on some procedures as to predict and measure settlement regularly, to loose the outter block from the tower where it is supported replacing the original fix supports of the beams by moveable ones and to descend the lower structure from time to time, according to the settlement occurred in the meantime. Figure 2 shows the succeeding phases of construction that was followed. Phase 1 is the initial situation. Phase 2 is the construction of consols in turn of the tower's columns at the 4th floor, above the roof level of the lower block. Phase 3 is the placement of moveable ties hung from these tower's consols, resting over sand jacks and supporting the three level's beams of the outter block. After the load transfer, the primitive consols are cut off giving way to the descent of the beams.

However the success of the solution depends on having control over the value of some data, as load and settlement, and the performance of some elements as the structure and the conceived devices.

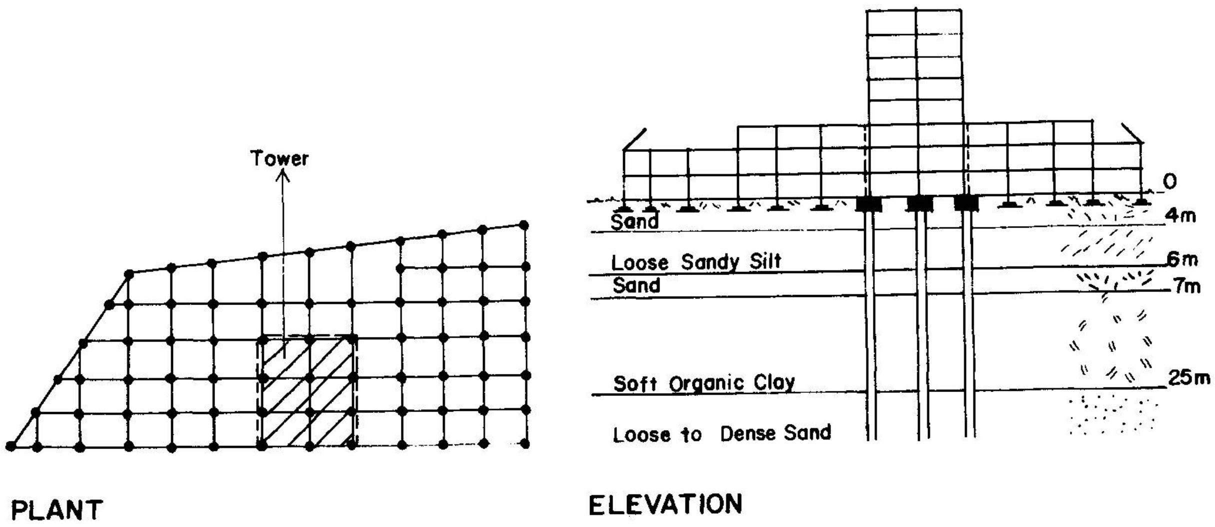

Fig. 1 Lay-out of the structure and foundation conditions

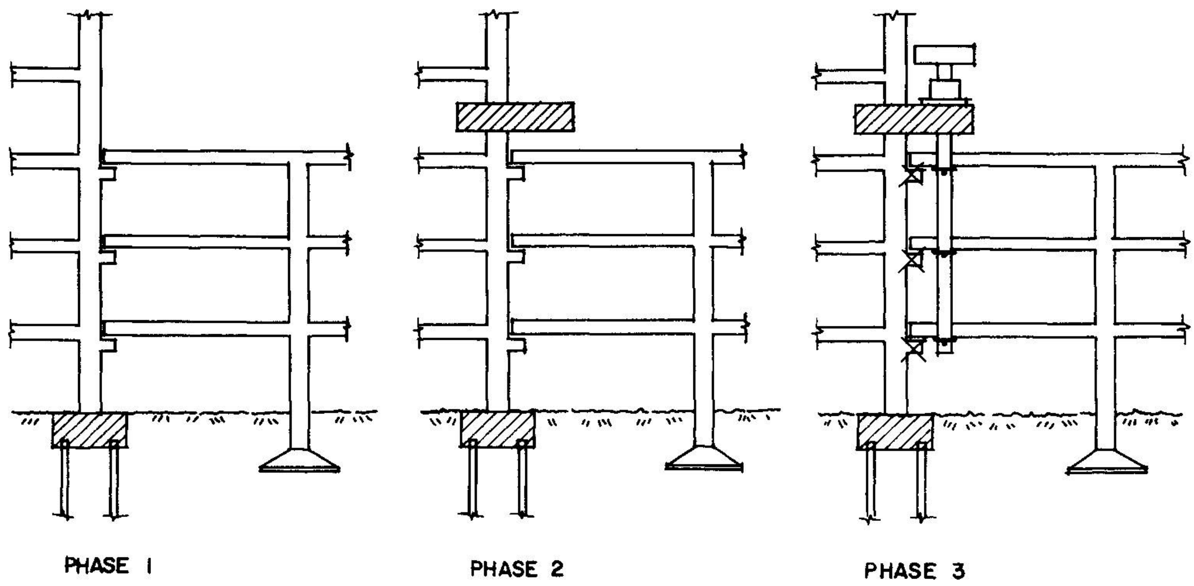

Fig 2 Construction sequence of the solution

3. ANALYSIS OF THE SOLUTION

3.1 The tower structure

The principal effect of the solution is on the reinforced concrete frame of the tower. The structure system should be able,to resist to the new load distribution with safety. The maximum tie load is 266 kN. Loads are applied symmetrically to the frame at the 4th floor, when originally they are distributed along the lower floors. The structure was calculated to this new stage of loading and found adequate ro support it. The new consols in turn of the tower's columns were designed using the provisions in the Reinforced Concrete Brazilian Code (1) to fck = 15 MPa. Epoxy was used to bond the external encasement and the surrounding concrete. A summary of the detail is given in Figure 3.

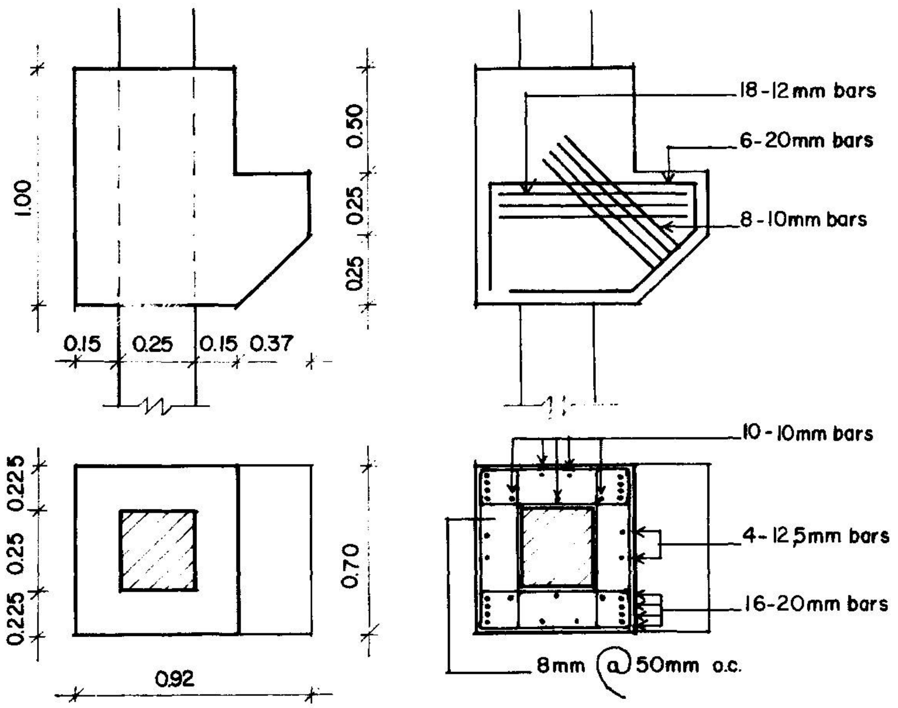

Fig. 3 Dimension and reinforcement of the consol

3.2 Settlements

Loads of columns in front of the tower varied between 500 to 700 kN using spread footings to transmit the pressure of 100 kPa to the soil at 0.85 depth. Since the first notice of damage, pins were fixed at these columns and the settlements started to be measured. At this time, the settlement already occurred and causing mansonry and structural damage was evaliated larger than 20 and 40 mm respectively, by the distortion criterium of Skempton and Mac Donald (2).Once taken the decision of compensating settlement, the solution relyied as much on the measured settlement as on the predicted ones. The total settlement was predicted to be higher than 150 mm by laboratory tests on soil samples. It was also used the early part of the time-settlements curve as the only datum for its extension, and the rest of it as a check of the prediction to the remaining settlement (3). This method was applied to the lectures of settlement made at different times. The results were dispersed, but the prediction was mantained.

A total of 28 settlement lectures were made along 9 years at interval of 3 to 6 months. The annual average settlement was 15 mm. Settlement was compensated when it reached a maximum of 20 mm or less to release the structure.

3.3 Sand Jacks

Sand Jack is much used to descend strut of reinforced concrete form. Should this jack be used at the tie device, it had to work lowering the beams under control in order to compensate the settlement measured.

Figure 4 gives detail of the sand jack Its 160 mm diameter piston should get out an extension of 150 mm which is the settlement predicted to be compensated along time. The cilinder below has an internal diameter of 180 mm. It is filled in by fine sand of quartz with uniform grain-size and pre-loaded to 350 kN. This load is higher than the maximum one to be transmitted by the tie system. The device has two little holes at the bottom. The sand takes out smoothly using a thin steel pointer through them. Two lateral defletometers measured the descent up to the value of the settlement to be compensated. Laboratory tests permitted to define the procedures to assure an even lowering of the piston. The field operations proved to be the sand jack a practical, simple and safe device to lower the conceived system.

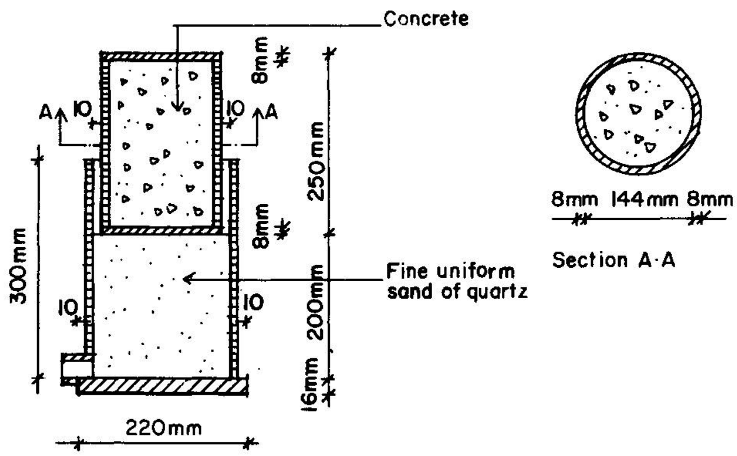

Fig. 4 Detail of the sand jack

3.4 Moveable Ties

Rigid ties were used instead of cables having intermediate supports to receive the beams. The transfer of the beam's load from the fix consols to the tie should be made by parts, starting from the top beam to the one down, with the use of inflate jacks put at the contact between the beam and its support in the tie. Epoxic resin was used as fluid. In each level, the load transfer was made symmetrically in order to assure the load equilibrium at the central tower.

The system is shown in Figure 5. All parts of it as splice, hook, bolt, welding, connection, etc. were tested to failure. The system's details should facilitate the antecipated construction sequence, including provisions for its erection and descent, installation through openings made at the existing structure, change of the sand jacks, space to operate them and the placement of inflate jacks to support the beams. Finally, load test made at each installed tie system determined the load transfered by each beam using strain gages fixed on the ties.

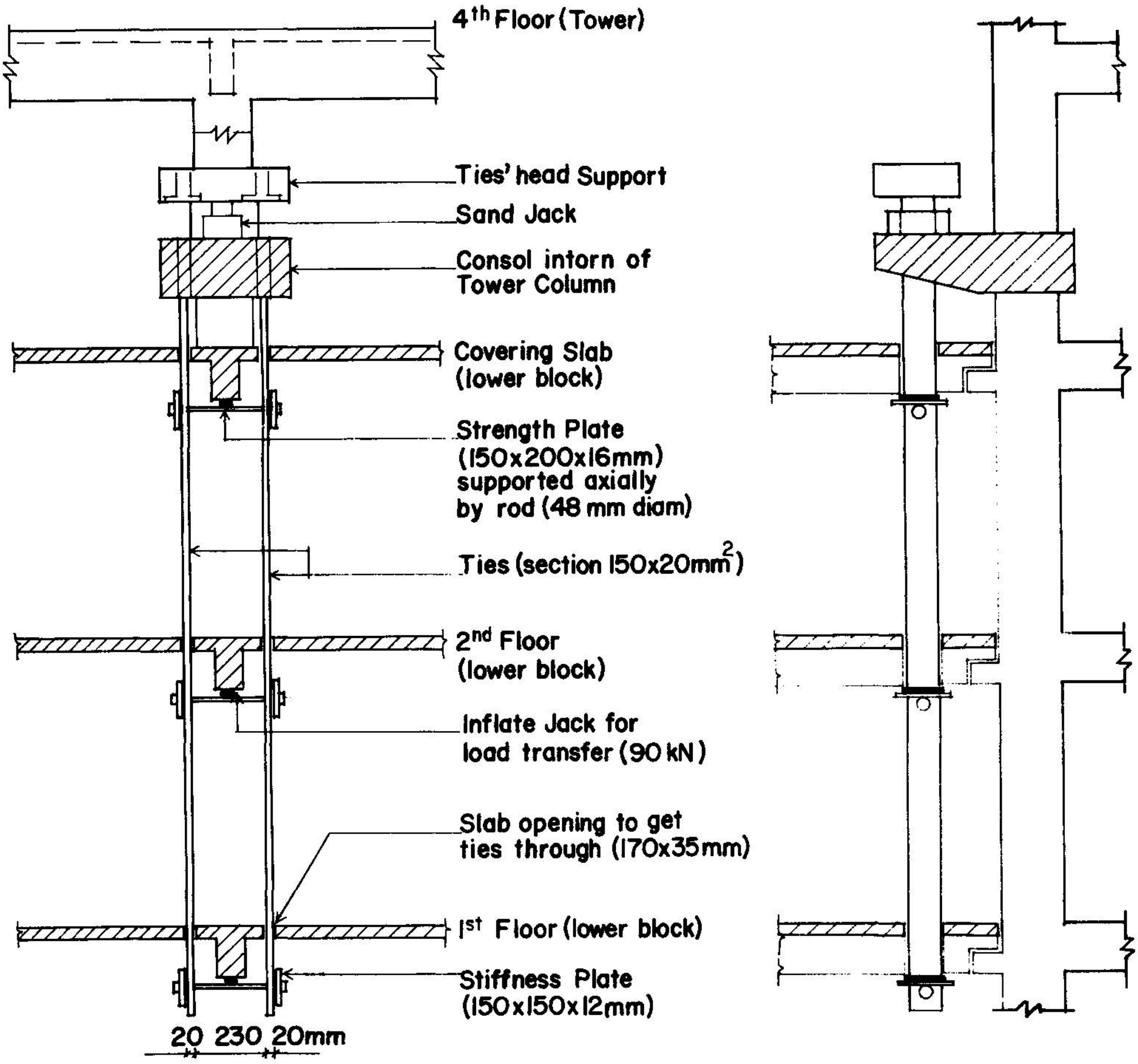

Fig. 5 Tie device to compensate settlement

4. RESULTS AND DISCUSSION

The following sequence was set up to the construction: 1. consols in turn of the tower; 2. openings at the slab to descend the tie system; 3. moveable tie system laid down over the sand jack; 4. placement of the inflate jacks below the beams; 5. instrumentation; 6. load transfer; 7. break of the original consols; 8. structure relaxation by the sand jack lowering the whole device as much as the measured settlement to be compensated.

The load transfer is a delicate operation guided by instrumentation. The process starts with the toric jack applying 90 % of the dead load at the support. In the case of adhesion between beam and its fix support, the jack needs to apply an effort higher than the beam reaction. When the defletometer indicates movement, one stops to pressure the jack, the load is recorded and the register is closed. Then the fix support is broken and the beam is free to descend. Operation is repeated to the beam at the level below.

The criterion adopted for structure relaxation was to proceed a maximum descent of 5 mm per tie in each round with a timelag of 24 hours between the descents. The first relaxation compensated 60 mm of settlement. Sometime on, when the settlement measurement neared 20 mm, it was compensated to release the structure. The total settlement was more than 100 mm, in average, along seven years of work on the direction of the Author.

In the meantime, the sand jacks were removed once. In some jacks, their pistons had got the final course. In others, the strong confinement of the sand avoided its remotion during relaxation. The jacks were repaired and turned back to the tie system.

The difference of level between the blocks was overcome using flexible by-pass for pipes and cables as well as moveable ramps at the doors. Ties were encased in each floor, together with the corresponding tower's column, having enough space to move inside without beeing seen.

When the settlement reached 200 mm it was decided to drive piles at all columns and the tie device was disactivated. It had extented the lifespan of the structure for more than 10 years.

5. CONCLUSIONS

A simple and practical device was used to compensate settlements of one building having two different types of foundations. Moveable supports have the advantage to compensate settlements and to release the structure rapidly. The cost is low and construction procedures are quite simple. Otherwise, blocks remain with different level and periodical correction is needed, causing some disturbance to the users of the building.

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Strengthening of Existing Piers and Jetties

Renforcement des structures portuaires existantes

Verstärkung bestehender Anlegekonstruktionen

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SUMMARY

The paper presents the ways of adapting and strengthening of existing harbour structures to fulfil conditions imposed by increases in the size and draught of modern ships. Three ways of adapting are analysed, namely, through recalculation, through adequate deepening of the harbour bottom and through rehabilitation and strengthening of existing structures.

RÉSUMÉ

L'article présente les moyens de transformation et de renforcement des structures portuaires existantes, afin de les adapter à des conditions imposées par l'augmentation des dimensions et du tirant d'eau des navires modernes. Trois moyens d'adaptation à de nouvelles exigences sont présentés, soit un nouveau calcul adéquat, un approfondissement du fond marin avoisinant, et le renforcement et la reconstruction de structures existantes.

ZUSAMMENFASSUNG

Vorgestellt werden in diesem Vortrag die Methoden der Anpassung und Verstärkung der bestehenden Anlegekonstruktionen in den Häfen zur Erfüllung der neuen Bedingungen, die durch die Vergrößerung der Abmessungen und des Tiefganges der modernen Schiffe verursacht werden. Drei Methoden dieser Anpassungen und Verstärkungen wurden charakterisiert, nämlich die Durchrechnung der bestehenden Konstruktion für neue Bedingungen, die entsprechende Vertiefung der Hafensohle in der Nähe der Konstruktion und der Umbau der Konstruktion, verbunden mit entsprechender Verstärkung und Anpassung.

1. INTRODUCTION

The development of marine transport and the construction of ships in recent years indicate that ships are highly specialized and equipped with handling gear which causes high vertical loading. They have special shapes with a large longitudinal surface area, leading to high loadings on mooring devices as the result of large wind forces. The fleet of ships now in existence and being developed, particularly these vessels built to meet the requirements of certain seeways (e.g.Panamax) impose changed demands on harbour facilities, particularly on different types of berthing structures. It requires readjustment of harbour facilities to assure adequate depths, handling and mooring facilities, as well as adequate strength and stability for the significantly increased loadings.

In order to avoid any confusion caused by different terminologies, the terms wharves, quays and piers are introduced for marine structures which are used for the mooring or tying of vessels while they are loading or discharging cargo and/ or passengers. Wharves and quays are backed by warehouse areas, marshalling and storing areas, industry areas, roads, rails, etc.-areas often created by extensive fill operations. A pier is usually a rectangular wharf structure which projects out into the water while jetty is synonymous with wharf and pier [1].

In the paper the readjustment of harbour facilities, particularly piers and jetties is analysed. This analysis deals with two main factors, namely, the increase of bearing capacity of the overall structure and the increase of the depth at the berth.

When analyzing the aforementioned problem the following three methods of adapting berthing structures to the new requirements may be considered, namely:

i) The application of modern calculation methods, which take into consideration the development of theory in the field, as well as the latest results of experimental on-site and model studies.

ii) Deepening of the harbour bottom at such a distance from the berth that the stability factor of the structure and that of the underwater slope will not be changed.

iii) The reconstruction of the existing structure with the use of that structure as a structural element of a new deeper pier or jetty.

The application of the above three ways of adaptation of the existing berthing structures to the new requirements can take place only in that case in which the influence of steel corrosion and concrete deterioration is negligible.

Generally it is observed that corrosion of steel structures (e.g. steel piles, steel sheet walls, etc.) reaches the highest degree at the mean low water level while mechanical wear caused by waves, currents, ice and suspended particles of sand can cause significant reduction of the strength of the structures or stuctural elements into consideration. This means that strengthening of existing piers and jetties can be considered in such cases only if a strengthening of existing structures took place. In almost all cases, when the corrosion is observed, the existing corroded structural elements are replaced or the whole structure, having a lower strength, is introduced as a part of the new structure.

The deterioration of reinforced concrete berthing structures is caused by impact, overloading, structural movement, abrasion, lack of durability of the concrete and reinforcement, frost, chemical and electro-chemical attack. All of these causes have long-term consequential results, in that hostile environment it is allowed to penetrate the damaged structure through cracks and physical damage.

Damage of berthing structures from the marine enviroment can not in many cases be entirely avoided. It is therefore necessary, particularly in the case when the loading acting on the structure will be increased, to select design solutions so that the structure function will be fulfilled throughout its entended new lifetime despite the effects of the marine enviroment. Such results can be more easily achieved if the design of the strengthened structure is adapted to the enviromental conditions prevailing in the area, and that such solutions will be introduced which can be implemented with a high standard of engineering. In addition the design should also consider, apart from the technical strength, the durability of the structure.

It has to be metioned here that the successful design and construction of harbour structures is highly dependent on a complete knowledge of the wave "field", much more complete than the wave height and the associated period. Some of the necessary additional parameters from designer's point of view are: wave evolution over the arbitrary bottom profile, amplitude of wave oscillation at the structure wall, influence of possible erosion hole on the wave field in the vicinity of the structure and others.

2. CALCULATION METHODS

Pier and jetty strength and stability conditions can be divided into the estimation of loads, the dimensioning of the structural elements e.g. piling, coping beams, slabs, walls, etc., and the checking of the overall stability for the most unfavourable conditions of loading.

Loads on a pier or jetty originate mainly from moored ships, cargo handling equipment, transportation means, as well as merchandize stored on the structure. These loads may be analyzed with a view to their possible decrease in order to allow the structure to be used for larger vessels, which may, however, exert loads of the same magnitude, as those formerly exerted by smaller vessels.

Loads due to moored ships are mainly estimated as a function of the ship's displacement. Several national norms exist which estimate the mooring force on a bollard, or the tension and pressure on a certain length of the berth (say 1.0 m) as functions of the water depth at the structure under consideration. Both ways, however, give the values of the force or load on a bollard for a very wide range of displacements, a lower value may be 100,000 t while the upper value may be 200,000 t. This means that between the two extreme values considerable reserves may exist. Thus, it is generally recommended not to use the so-called table values, but for each particular case to calculate real force acting on a bollard as a function of the area of the ship's side, the maximum wind velocity acting perpendicular to the ship's mooring line, screening of buildings, hills, etc. of the structure under consideration and the parameters of current and waves in the harbour area. The above calculations made for a ship with a displacement of 100,000 t show that for wind speed of 20 m/s the maximum force on a single bollard may be about 650 kN in the direction perpendicular to the mooring line. This would mean that when using the Recommendations of the Committee for Waterfront Structures EAU 1990, which gives a force on a single bollard of 1000 kN, there may be a reserve of 350 kN on a single bollard with certaing arrangements of the mooring devices (perpendicular mooring ropes).

Considerable reserves also exist in crane loads, mainly because the cranes are either totally withdrawn from use (e.g. Ro-Ro ships with their own reloading ramps) or the cranes are replaced with modern ones which generally have much smaller loadings per running metre of the berthing structure. Thus if the overall stability of the structure is sufficient, the change from old cranes to modern ones may often bring a reserve in vertical loads, and consequently horizontal loads as well. In addition it must be pointed out that with a change from general purpose berth to a specialized berth, e.g. coal discharging, the number of cranes and other means of loading and transport are not only reduced but can be specified. This means that the conversion of a pier to specialized use may also allow considerable reductions in load. However, independent on the above reductions, a possible increase of the fundamental vertical bearing capacity of a berth may be considered, namely, in function of loads hitherto applied. It may be argued here that if accurately known vertical loads have been acting several years on the structure, and no significant deformation has taken place, an increase of about 30% of the permissible vertical load with respect to the maximum loads hitherto applied may be allowed.

As for the dimensioning of berthing structure elements, two main areas of interest may be touched upon, namely, the calculation of the earth pressure, if acting on the structure elements, and the bearing capacity of anchoring systems used. In the first area very simple earth pressure calculation methods have been used when designing the old structures. In the modern solutions the flexibility of the bearing elements can be taken into consideration giving considerable increases of allowed loadings. With respect to the anchoring system the possibility of reducing the anchor force applied, or increasing the allowable anchor force may be analyzed here. The first is of course connected with the reduction of all horizontal forces mentioned above together with the loads due to earth pressure if occuring. Here, only one course of action would be recommended, namely, checking the shape of the fill behind the wall. It is namely often found in berthing structures, that after many years of service, the fill has settled and has thus not only decreased in height but that also the values of the main soil parameters have increased. As for the increase of permissible anchor forces, the recalculation taking into consideration the increase of bearing capacity of compression and tension piles of a pile anchoring system may be suggested here.

The next area of berthing structure checking, when an increase in load or depth is foreseen, concerns the stability recalculation. Here it can be recommended that checks are made not of the stability of the plane running section of the structure, but of a section of the length equal to the distance between expansion joints. This allows to consider a sum of all the loads acting on the berthing structure section, which will also mean that when, for instance, possible crane loads as well as ship's mooring loads are taken into consideration, the total section of the berthing structure may absorb much larger loads then indicated by using stability checking for one running meter.

Taking into consideration all the above statements, it is possible to assume that an increase of minimum 10% of the bearing capacity of the existing berthing structure can be in any case introduced. This also means that an adequate increase of water depth is possible. Generally, if the strengthening of an existing berthing structure has to be made to increase its permissible loadings to about 15%, the recalculation of the strength, taking into consideration the above mentioned parameters, can allow to meet such requirements.

3. HARBOUR BOTTOM DEEPENING

The simplest way to reach the required depth at the pier or jetty is of course to deepen the harbour bottom either at the berthing structure or at a certain distance from it. However, regardless of the method employed it is first of all necessary to check carefully the real depth at the structure. It has namely often been pointed out that the actual depth at the berthing structure is much greater than was required even during the preliminary dredging work. This is caused mainly by natural scour in harbours and currents arising from ship's propellers. Once the actual depth is known, a full set of berthing structure recalculations must be carried out, which take the projected final loads into account. If the calculations of the existing depth allow it to be compared with the required depth, proper protection for the harbour bottom is necessary in order to ensure that this depth is maintained. This protection may take either the form of crushed stone layer, or, when it is provided at berthing structures where considerable currents are present, of concrete slabs placed on a layer of coarse sand or gravel. Using these methods a deepening of about 1.0 m with respect to the design water depth at the existing structure is possible. Of course in this case no corrections for dredging works are possible. In Fig.1 [2] an example of a scour-protection using geotextiles is given. It consists of woven geotextile (1) covered by non-woven needle punched geomembrane (2) treated mainly as a sedimentation layer (filled later by sand grains - increase of stability of the scour - protection layer), reinforcing woven geotextile (3) and balasting layer consisting of colloidal mortair units (4) (0.5 m diameter and 0.10 m height) giving an average weight of the geomembrane of about 100 kg/m^2.

The deepening of the harbour bottom at a certain distance fom the existing berthing structure should be carried out only if it can be assumed that the bearing capacity and the stability of the structure will be or is sufficiently high to take the loads projected in the new programme decided on for the structure in question. The distance depends on the berthing structure, the size of the ship, the shape of the hull, and on the mooring and fendering devices. The easiest way is to install a row of flexible dolphins at such a distance from the berth that the slope of the harbour bottom reaches the desired depth. The dolphins can be equipped with fenders of considerable diameter or thickness. This reduces the distance between the dolphin and the pier or jetty which in turn determines the length of transport bridges and other structures. The existing or required outreach of cranes is a prime consideration here. Whatever the case may be, the proposed method is recommended for berthing structures intended for the transshipment of oil, corn, and similar products, as these only require loading arms supported on special platform which may be erected between the mooring dolphins and the existing berthing structure.

An additional way of deepening of existing structures is to strengthen the bottom part of the structure allowing on necessary dredging. In Fig.2 [5] an example is given of a jetty strengthened through building of a submerged concrete retaining wall along and in between the front row piles of the jetty. After strengthening the berth pocket in front of the jetty was deepened without additional risk for the jetty foundation. It has to be metioned that for the strengthening of the bottom part of a berthing structure the high pressure jet grouting can be used.

In recent years through the introduction of bow thrusters, the harbour structures are attacked by jet velocities, causing intensive bottom scouring. This requires not only additional protection systems, e.g. using flexible revetments, but also special shape of the berthing structure together with the adjacent bottom. The main task is, however, to maintain the necessary depth without danger of bottom scouring.

4. USE OF THE EXISTING BERTHING STRUCTURE AS A STRUCTURAL ELEMENT OF A NEW DEEPER PIER OF JETTY

The deepening and strengthening of an existing berthing structure can be fairly easily carried out by replacing the old berthing structure with a new one fulfilling the new requirements concerning depth and strength. The other way is to leave the existing structure as it is and construct a new independent berth at a certain distance from the old one. The first solution is very expensive because it requires the demolition of the old structure and the construction of an entirely new one.

The prime requirement for the second solution means an adequate space in front of the existing berthing structure. This is usually coupled with a reduction in the width of the harbour basin, navigation channel, etc., which may cause considerable and unacceptable problems.

As a result of the above considerations the use of the existing pier or jetty as a structural element of a new, deeper berthing structure which will be deeper and will meet all the requirements of the authorities apper to be the best approach. Existing berthing structures may be used in various ways and for different purposes which can be listed as follows:

i) As support of a part of the new berthing structure or as a support for the main structural element of the new structure.

ii) As a support for the new berthing structure and for the reduction of the earth pressure on it.

iii) As an anchoring element for the new berthing structure.

iv) As an anchoring and earth pressure reduction structure for the new berthing structure.

The first solution (Fig.3) [3] was adopted for the reconstruction of the Ore Terminal in the Harbour of Gdańsk. In the first phase the existing structure was equipped with a new crane connected with the construction of a new crane truck; in the second phase the construction of a new coping slab supported by the old structure and new prefabricated reinforced concrete sheet wall was made. The reconstruction allowed to increase the depth at the structure from 8.0 to 11.5 m and to load it by two crane tracks, one railway track and surcharge of 30 kN/m^2.

In the second case, concerning the berthing structure of the Vistula Terminal in Gdańsk (Fig.4) [3], the existing structure consisting of anchored timber sheet wall was used to decrease the influence of earth pressure and to support partially the new coping slab. An increase of the depth of 4.0 m was reached while the loadings from cranes through constructed crane tracks were allowed.

The third solution (Fig.5) [3] was introduced for the strengthening of the Swedish Quay in the Harbour of Gdynia. The reconstruction (increase of depth, new installations, new cranes) was made by driving in the front at a distance of 1.5 m a steel sheet wall of Larssen V type, strengthened by box piles LVP. The space between the existing and new sheet pile wall was filled by compacted sand while the new upper structure was used to connect and anchor the new structure with the old one.

The last case (Fig.6) [4] concerns a structure consisting of a new inclined sheet piling wall, driven in front of the existing structure and attached to the remainder of the old structure by prestressed anchors. The bottom at the previous seabed level is secured by an underwater concrete plug. The strengthening of the existing berthing structure allows on the increase of the depth from -7.0 m to -12.5 m. The front wall is a combined HZ-steel wall in which the Z-sheets are driven from the level -3.5 m to -14.5 m.

The presented cases clearly indicate the possibilities of strengthening of the existing berthing structures to meet the requirements arising from the changes occuring in recent years in the marine transport.

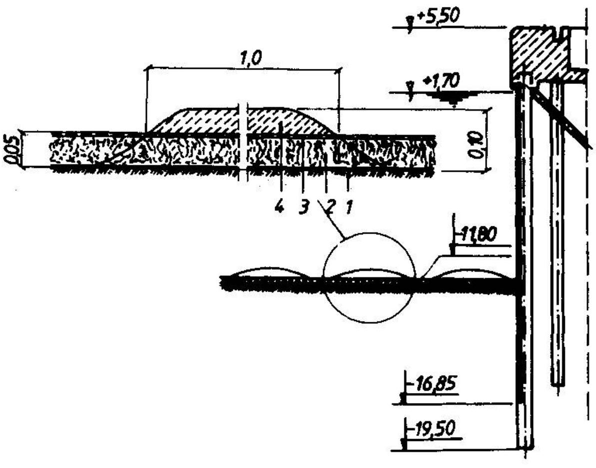

Fig.1 Bottom protection using geotextiles [2]: 1-woven textile, 2-geomembrane with sand, 3-reinforcing geotextile, 4-mortair

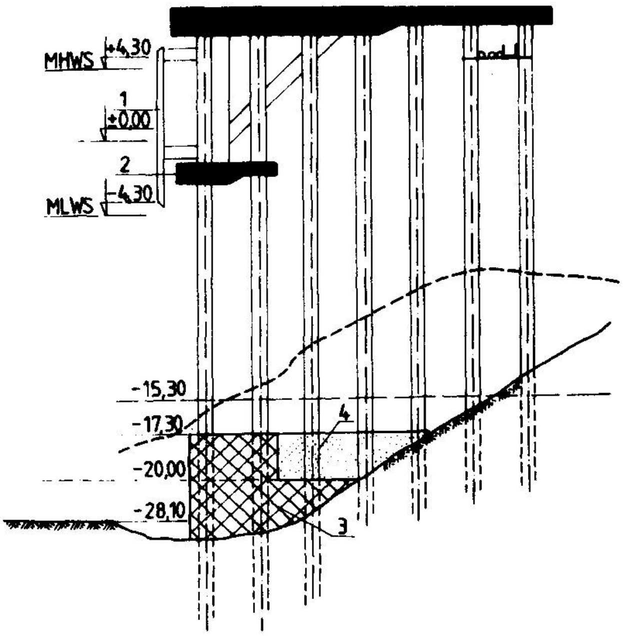

Fig.2 Strengtheing of an existing jetty through a submerged retaining wall [5] 1-main fender, 2-concrete fender tray, 3-concrete retaining wall, 4-soil refill

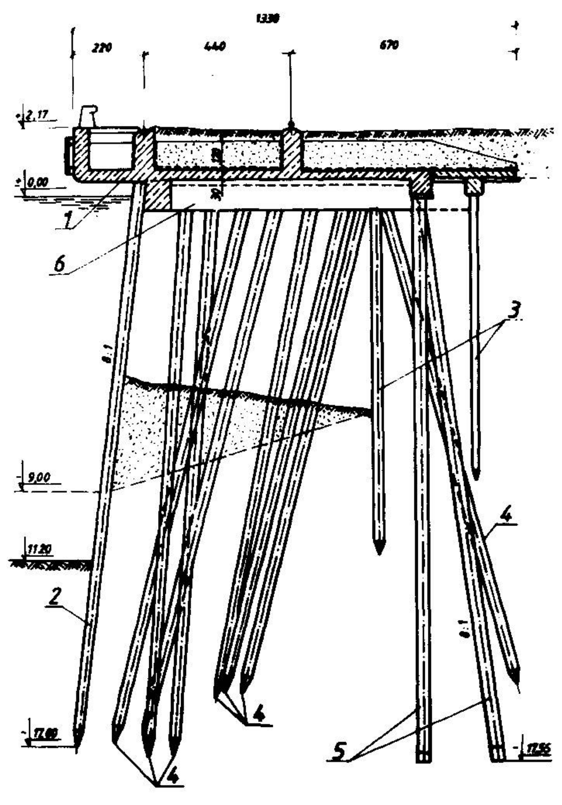

Fig.3 Reconstruction of the Ore Terminal berthing structure in Gdańsk [3]: 1-new reinforced concrete slab, 2-concrete sheet wall, 3-existing timber sheet wall, 4-existing timber piles, 5-concrete piles, 6-existing concrete slab with reduced thickness

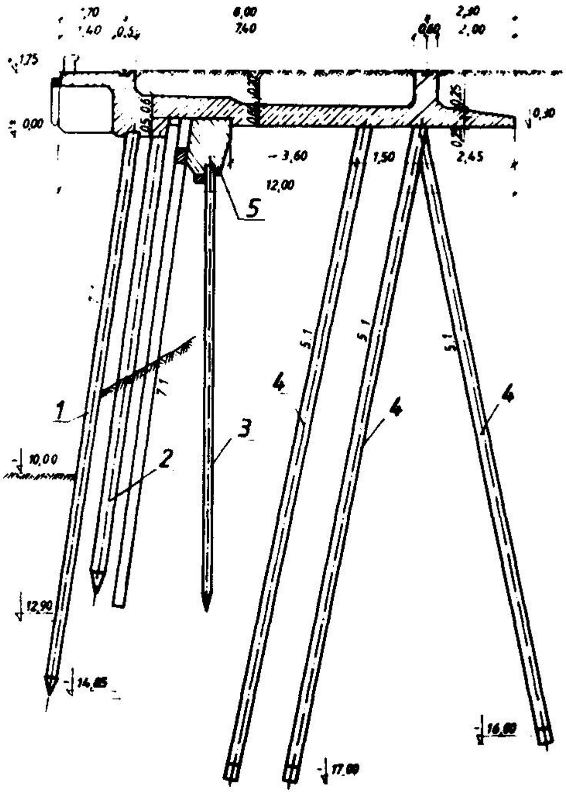

Fig.4 Strengthening of the berthing structure of the Vistula Terminal in Gdańsk [3]: 1-concrete sheet wall, 2-existing timber piles, 3-existing timber sheet wall, 4-concrete piles, 5-existing concrete coping beam

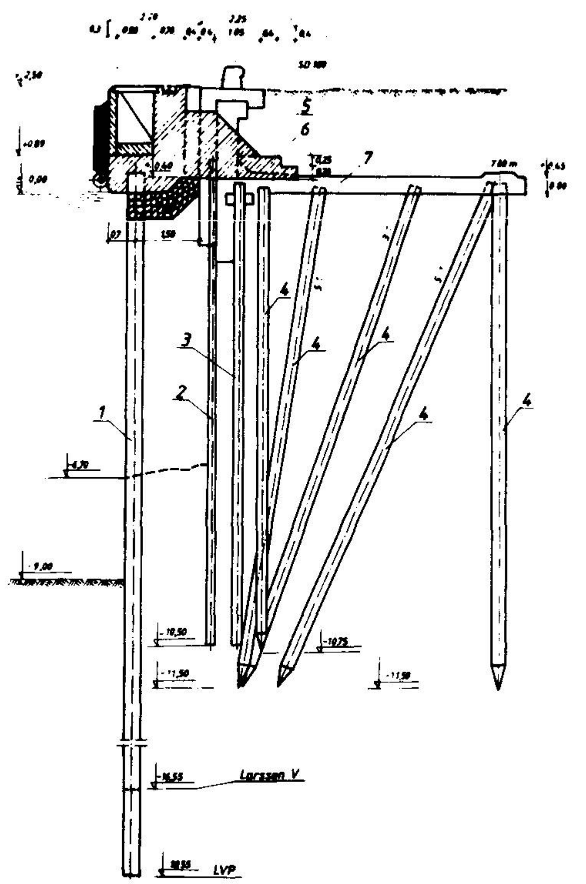

Fig.5 Strengthening of the Swedish Quay in Gdynia [3]: 1-steel sheet wall Larssen V strengthende with box type piles LVP, 2-existing timber sheet wall, 3-steel sheet wall, 4-timber piles, 5-longitudinal anchoring beam, 6-transverse anchoring beams, 7-existing cover plate

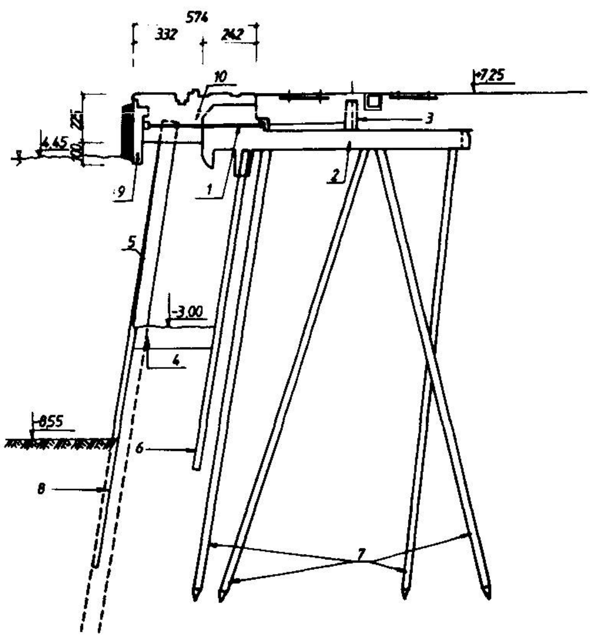

Fig.6 Cross-section of the strengthened berthing structure [4]: 1-prestressed anchor, 2-existing cover plate, 3-new crane-rail beam, 4-concrete, 5-H-piles, 6-existing BZ-sheet piling, 7-existing concrete piles, 8-Z-sheets, 9-prefabricated element, 10-new head beam.

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