SURGERY IN EXISTING STRUCTURES

MAJOR CHANGES IN STRUCTURAL SYSTEMS FOR ADAPTATION OF NEW FUNCTIONS OR REPLACEMENT OF EXTREMELY DETERIORATED MEMBERS

BY REFAAT M. SALLAM

The word "surgery" in the title may represent abnormality to the traditional verbiage used in the repair of structures. This word, however, comes to mind when treating problems such as those mentioned in the title. In medicine, surgery is concerned with treatment of injuries, diseases, and disorders of the body by incision, manipulation, or alteration of organs. Most of these operations are practiced in a similar manner in structural engineering when major changes in structural systems are required. Such resemblance suggests that this kind of project needs special experience; precision like that practiced by surgeons in major operations includes thorough investigation and an understanding of all details of the existing structure (which might need some field and laboratory tests), and awareness of the problems that might be encountered and the approaches to their solutions. These major changes are sometimes incorrectly thought to be costly but they can solve problems, especially concerning the two cases mentioned previously: adaptation of new functions and replacement of extremely deteriorated members, without removing the whole structure or important parts of it.

Main Hall at the beginning of work

CASE 1—ADAPTATION OF NEW FUNCTIONS

THE PROBLEM

This example deals with the removal of two large adjacent columns in a central main hall of an old classical building of historic value. The three-story building, located in the center of the town of Alexandria, Egypt, was constructed in the beginning of the 20th century to serve as a club for the elite. In the last decade of the same century, the Egyptian ministry of culture, who adopted the building, decided to transform it to serve as a cultural center.

The central main hall of the building, which is composed of only one floor with a high-rise roof, is the largest free area in the building. Square in shape with sides of 54.13 ft (16.5 m), it was planned as a theater. The plan and vertical section of the hall is shown in Fig. 1. The high-rise roof is carried by large granite columns and arches spaced at equal distances of 11.4 ft (3.5 m) on the periphery of the hall. The cross section of all granite columns is 2.3 x 2.62 ft (0.7 x 0.8 m), whereas the cross section of the granite arches is 2.3 x 2.3 ft (0.7 x 0.7 m). Figure 2 represents the elevation of the main hall.

The stage of the theater, with a width of about 24.4 ft (8 m), is obstructed by two such columns, which are marked on Fig. 1 and 2. These two columns have to be demolished after being replaced by a new bearing system (the replacement system).

ADOPTED SOLUTION

The replacement system has to satisfy the following conditions:

1. Because this system has to be constructed prior to the removal of the two columns, it has to be aligned so as not to interfere with any of the two columns and the arches they support;
2. The system must be able to carry the loads and forces assigned to it, and transfer them to its new foundation safely and with minimal deformations; and
3. The system has to provide the free space required by the stage according to the architectural drawings. Steel was chosen as the construction

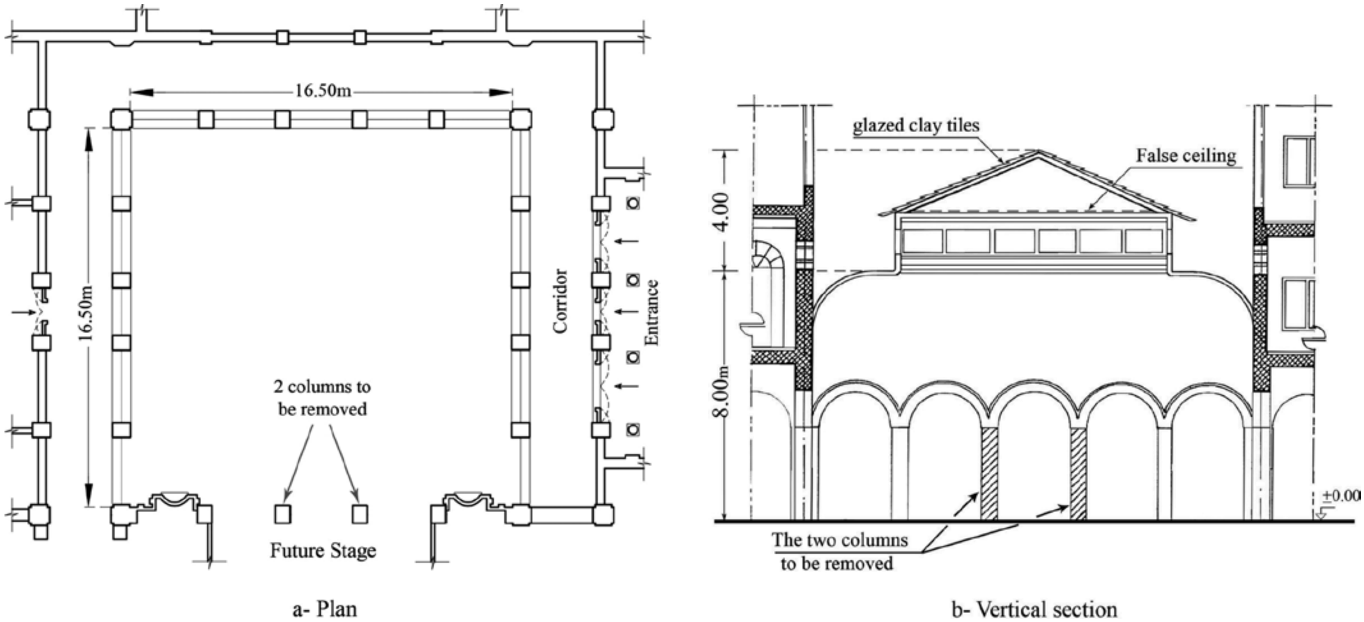


Fig.1: Main hall to be transformed to a theater (Note: 1 m = 3.28 ft)

material for the replacement system because it possesses, in this case, more than an advantage over reinforced concrete (RC).

First, the steel structure can be easily manufactured in a specialized workshop, transferred to the site, and assembled in place. On the contrary, RC meets difficulties in mixing and casting in restricted areas at the center of the building. Second, RC needs a special consideration to the problems caused by both plastic and drying shrinkages when casted in contact with other materials in an existing structure. Third, the time factor is obviously in favor of the steel. Fourth, steel is more cost effective than RC in this case.

In this century-old historic building, the materials and techniques used in the original construction were quite different from those used today. The granite columns carry very stiff granite arches, which in turn carry the thick walls 24 and 16 in. (600 and 400 mm) and the roofs of the building. The roof of the main hall is formed of steel main beams and purlins in perpendicular directions; the infill material is brick work 4.8 in. (120 mm) thick, bound up in between and covered with lime mortar in the lower part of the roof. Glazed clay tiles cover the upper pyramid part of the roof, with a false ceiling on the bottom surface.

Simplified working drawings of the replacement system are shown in Fig. 3 and 4. The replacement system comprises the following components:

1. The first component (Component 1), placed on the bottom surfaces of the three arches connected to the two columns to be removed, are composed of built-up sections of steel arched I-beams cut exactly to the same curvature and width of this surface. Later on, before demolishing the two columns, small accidental gaps between the two surfaces were closed by injection under pressure with cement slurry modified by styrene butadiene latex;
2. The second component (Component 2) are steel I-beams laid under the arched I-beams (Component 1) running perpendicular to their planes and resting on either side on the two main truss frames;
3. The third component (Component 3) of the replacement system is the two main truss frames

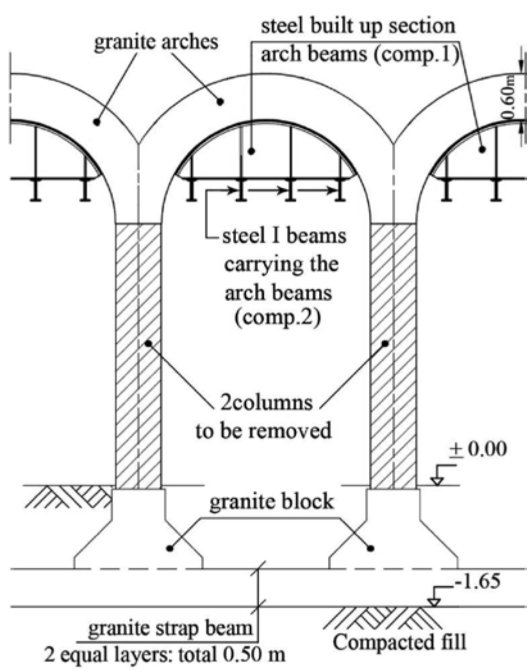


Fig. 2: Part elevation of main hall, also showing Components 1 and 2 of replacement system (Note: 1 m = 3.28 ft)

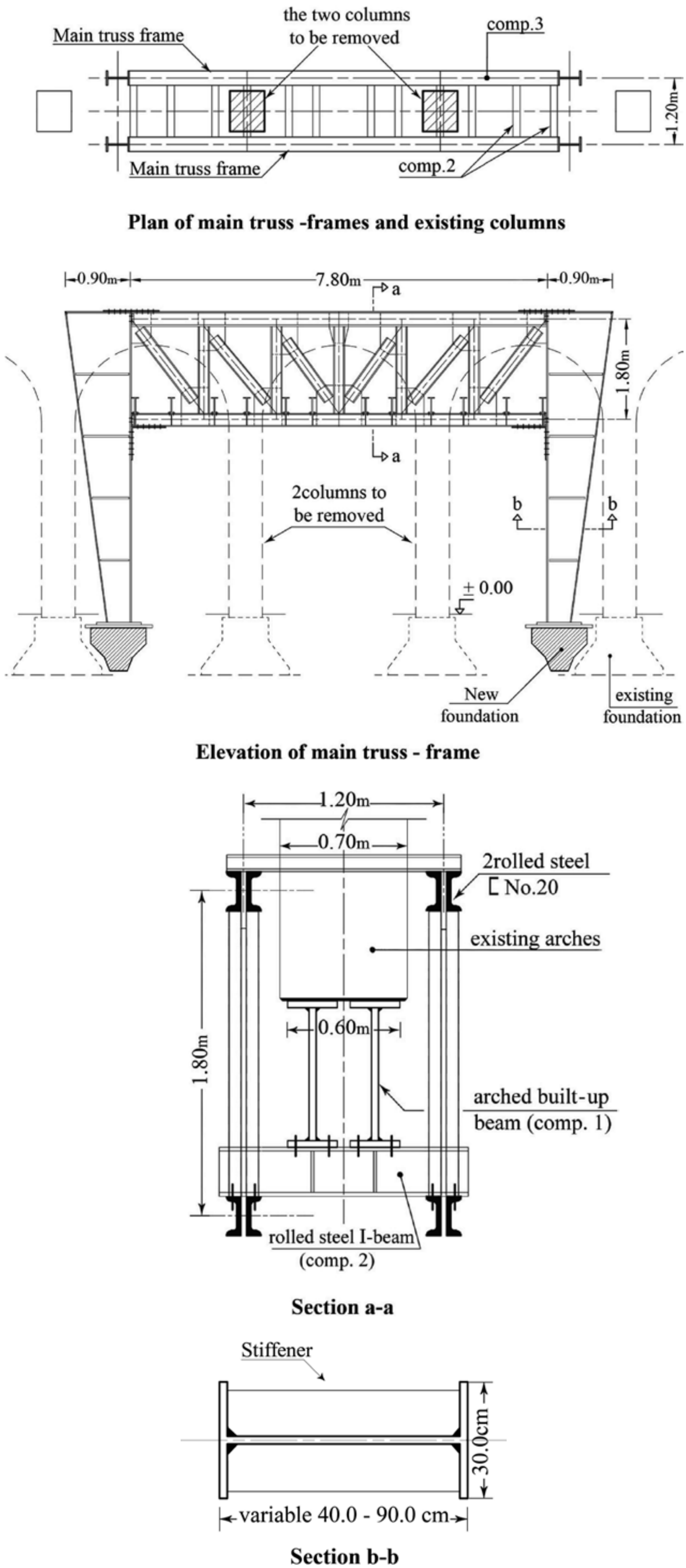


Fig. 3: Main truss frames (Component 3) (Note: 1 cm = 0.40 in.; 1 m = 3.28 ft)

Main truss frame after erection. Granite columns shown before removal, indicated by a dot (●)

laid on either side of the two columns to be removed and parallel to the line joining their axes; and

4. The fourth and last component (Component 4) of the replacement system is the new foundation of the two main truss frames. Soil borings taken at the site showed that the soil profile is composed of two layers. The top layer is a non-uniform earth fill mixed with some sand, broken limestone pieces, and broken earthenware pieces to a depth of about 20 ft (6 m) from the ground level. The second layer is composed of dense sand with agglomerated sand pieces, which continues up to the full depth of the borings—50 ft (15 m) from ground level.

The chosen type of foundation, its design, and the techniques used in the field for execution should all be considered with the safety measurements of the existing foundation of the building. This means that the new foundation should not interfere with the existing one. There should be no added stresses under the existing foundation due to the spreading of stresses from the new foundation in the soil and, finally, the field works of the new foundation should not yield vibrations, which might disturb the existing shallow foundation.

Bored piles of RC manually driven to avoid any disturbance to the existing foundation and resting on the sand layer at a depth of 23 ft (7 m) from the ground surface were chosen for the new foundation. Figure 4 indicates its conformity to the aforementioned safety measurements of the existing foundation.

NOTES ON WORK PROCEDURE

In dealing with problems of this kind, the first step is to pay a visit to the site of the building and its environment. The aim of such a visit is to establish an idea about the status of the building. Signs of structural distress, such as cracks, excessive deformations, foundation settlement, deterioration caused by aging, and environmental effects, should be detected. In the case in hand, it was noticed that mortar for rendering and joining

Load test on single pile. Foundation of existing granite columns also shown

the stone blocks of the bearing walls over the arches, for joining filling bricks in the lower curved part of the hall roof, and for the rendering of the suspended false ceiling of the top pyramid part of the roof were almost all decayed. Because interest in this stage was only in keeping the unity and the strength of different existing elements of the hall in a good state, instructions were given to remove loose mortar from the joints in between stone blocks and bricks to a minimum depth of 1.2 in. (30 mm) and to replace it by rich cement-sand mortar. Decayed rendering will be treated later on when general finishing processes of the building take place after the end of structural works.

Generally, it can be inferred that old structures (of age equal to or more than about 80 years) are massive but not monolithic—that is, the joints between different members are not rigid—and relative motions and rotations can only be resisted by the masses imposed upon these members (refer to Fig. 3 for the existing granite columns and their strap beam foundation of the same material).

The design of the replacement system was carried out keeping eye on the deformations—especially the vertical deflections—which must be kept as small as possible. This is because in the existing structure the forces due to own weight (DL) and live load (LL) are transferred to the columns mainly by arch action (direct compression), not accompanied by significant deflections. The replacement system in turn transfers the loads (DL + LL) to the new foundation by beam action (bending moment + shearing force), accompanied by relatively significant deflections. The dimensions of the main truss frame, which satisfy the requirement of estimated minimal vertical

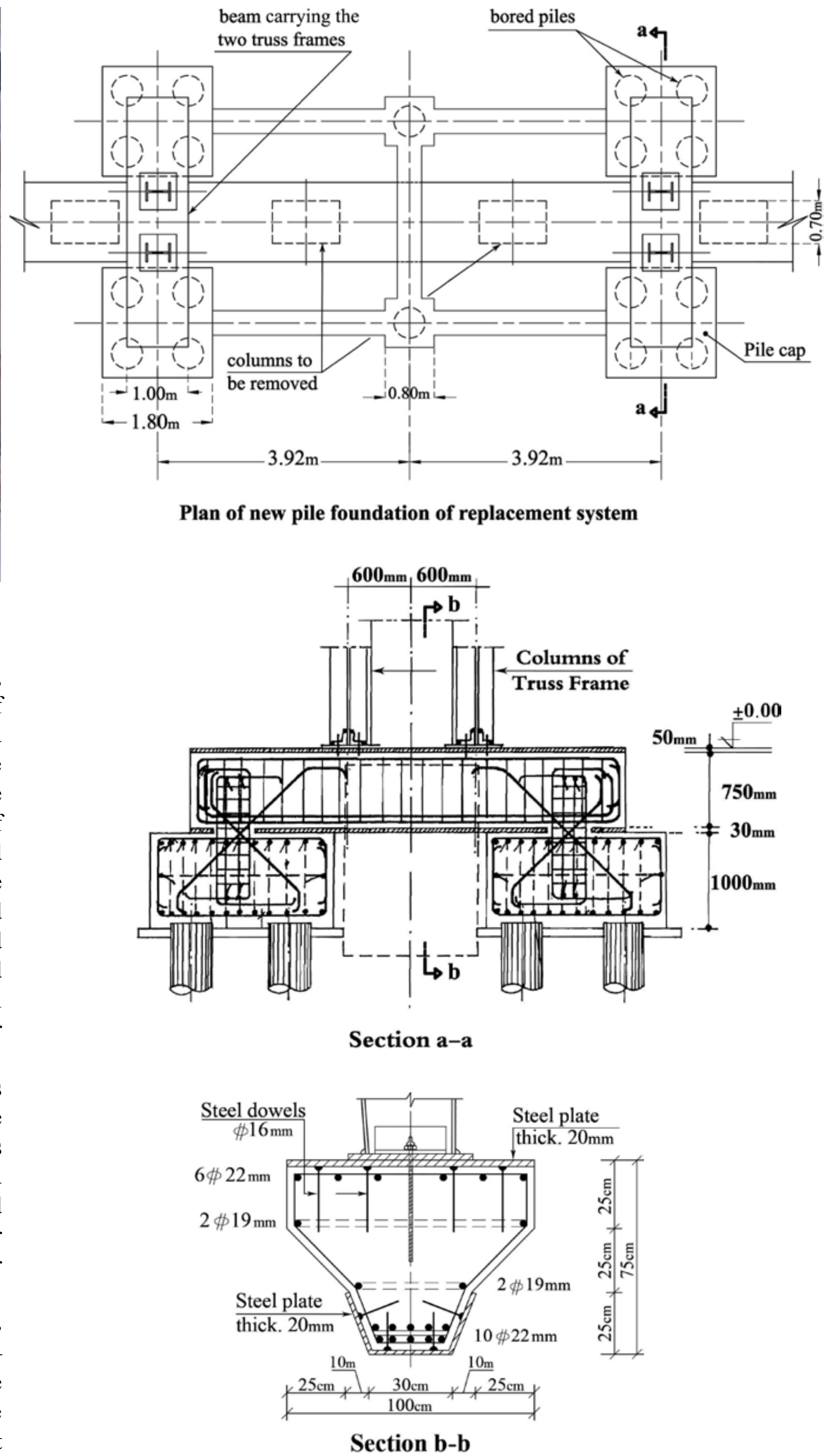
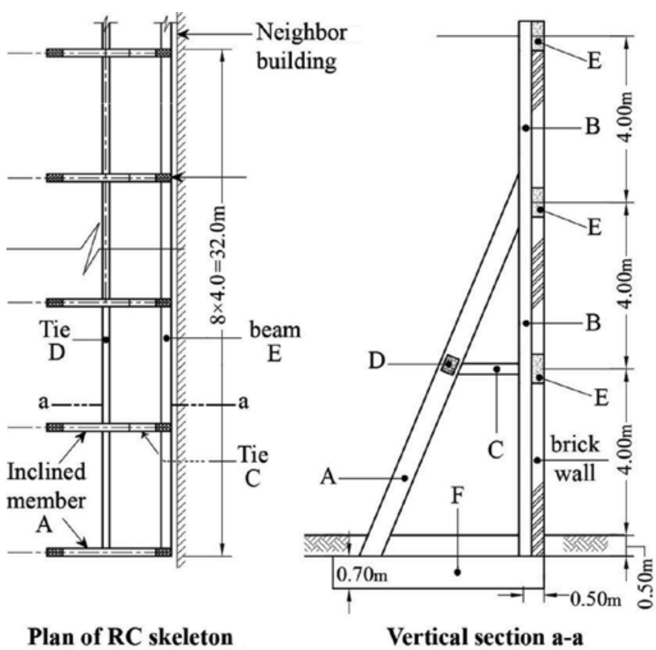


Fig. 4: Pile foundation for new replacement system (Component 4) (Note: 1 mm = 0.04 in.; 1 cm = 0.40 in.; 1 m = 3.28 ft)

deflection, are shown in Fig. 3. Maximum vertical deflections of the middle span sections of the trusses for DL + LL was calculated and found to equal 0.023 in. (0.57 mm). If these truss frames were in new construction, the Egyptian code of practice of steel structures and bridges allows a maximum

Stitch drilling to separate granite columns from the rest of the structure

Truss frames after removing the two granite columns, indicated by a dot (●)



A Inclined member of main frame
B Vertical member of main frame
C Tie member between A and B
D Tie member between members A of frames
F Foundation raft

Fig. 5: Details of existing curtain wall and bearing RC skeleton (Note: 1 m = 3.28 ft)

vertical deflection of the truss to equal 7/8 in. (22 mm). The average maximum deflection of the two trusses measured after the removal of the two columns was 0.03 in. (0.7 mm) only.

REMOVING THE TWO ADJACENT COLUMNS

Before removing the two columns, a thorough visual inspection of all the components of the replacement system was carried out by experienced engineers aided by simple tools. The inspection included examining the quality of the field welds of the steel member's joints.

A similar inspection was carried out on the building's main hall, once again to visualize if there were any changes in its condition due to the running works.

When all checks proved positive, the removal of the two columns was carried out. The removal was simplified because each of the main hall columns was composed of ten blocks—nearly of the same height—fitting exactly to the upper and lower blocks and stuck to them by an adhesive. This meant that what was required was only to separate the columns at their top and bottom horizontal sections from the rest of the structure. This was done using stitch drilling by overlapping bore holes—beginning at the top section—after which the different blocks of a column were manually separated.

In the course of execution of the new pile foundation, a load test on a single pile was performed. The settlement under a load of 1.5 the working load of the pile was measured and found equal to 0.1 of the value allowed by the local code of practice of soil mechanics. This was expected in the course of a design for minimal deformations.

PROJECT SUCCESS

A visit to the cultural center 3 years after completion of the repair and rehabilitation works proved the success of the new adaptation of the main hall as a theater. This was confirmed by some officials of the cultural center.

Recently, another visit to the theater was made by an experienced engineer. The visual inspection proved that the structure is intact and functioning well. This visit was 11 years from the completion of the work.

Recent photo of the stage

CASE 2—REPLACEMENT OF EXTREMELY DETERIORATED MEMBERS

THE PROBLEM

This project dealt with the replacement of some extremely deteriorated members of an RC skeleton formed of main frames, secondary beams, and tie members. The RC skeleton supports a curtain brick wall with large dimensions. Other members of the skeleton were moderately damaged and could be repaired. The main cause of the damage was corrosion of the reinforcement and the associated cracking and spalling of the concrete.

The brick curtain wall was constructed to provide privacy to an old small palace constructed on the shore of the Mediterranean Sea in the city of Alexandria, Egypt. The palace should not be disclosed to a neighboring public building.

The dimensions of the wall are: length (l) = 105.0 ft (32 m), height (h) = 39.4 ft (12 m), and thickness (t) = 0.82 ft (0.25 m). Figure 5 represents a plan and a cross section of the existing curtain wall and the supporting structure. Different members of this structure are defined by letters on the same figure.

THE STATE OF DAMAGE

Visual inspection and a condition survey of the curtain wall and its supporting structure revealed the following information:

1. There is no evidence of the presence of foundation problems, such as uneven settlement of the supporting structure or concrete deterioration of the raft, which is actually protected against the marine environment by being an underground level (Fig. 6).
2. The curtain wall of clay bricks is covered by a cement-sand rendering layer 0.8 in. (20 mm) thick, which suffered from decay and separation in some places, whereas the brick blocks and the mortar adjoining them generally appeared to be in a good state.
3. The RC skeleton: generally, all RC members suffered from reinforcement corrosion but with diverse degrees of deterioration. Also, the concrete cover over the reinforcement was much less for some members than that required by the codes for structures subject to the marine environment. For example, 0.8 in. (20 mm) was measured on most of Members A, C, and D, and about 1.2 in. (30 mm) for Members B and E.

Because taking photographs was prohibited at this site, it was decided to map cracking and spalling of the main members of the structure as close as possible to the in-place condition to illustrate their states of damage (Fig. 6).

The damage of different members can be summarized as follows:

- Member A, the inclined member of the main frames, and the horizontal tie Members C and D, were severely damaged to the extent that they could not be repaired and needed be replaced. Besides nearly continuous spalling and cracking in the longitudinal and perpendicular directions caused by reinforcement corrosion (Fig. 6), the decay of concrete surfaces was caused by severe salt weathering. Horizontal and inclined exposed surfaces were more liable to salt-weathering attack than vertical surfaces.

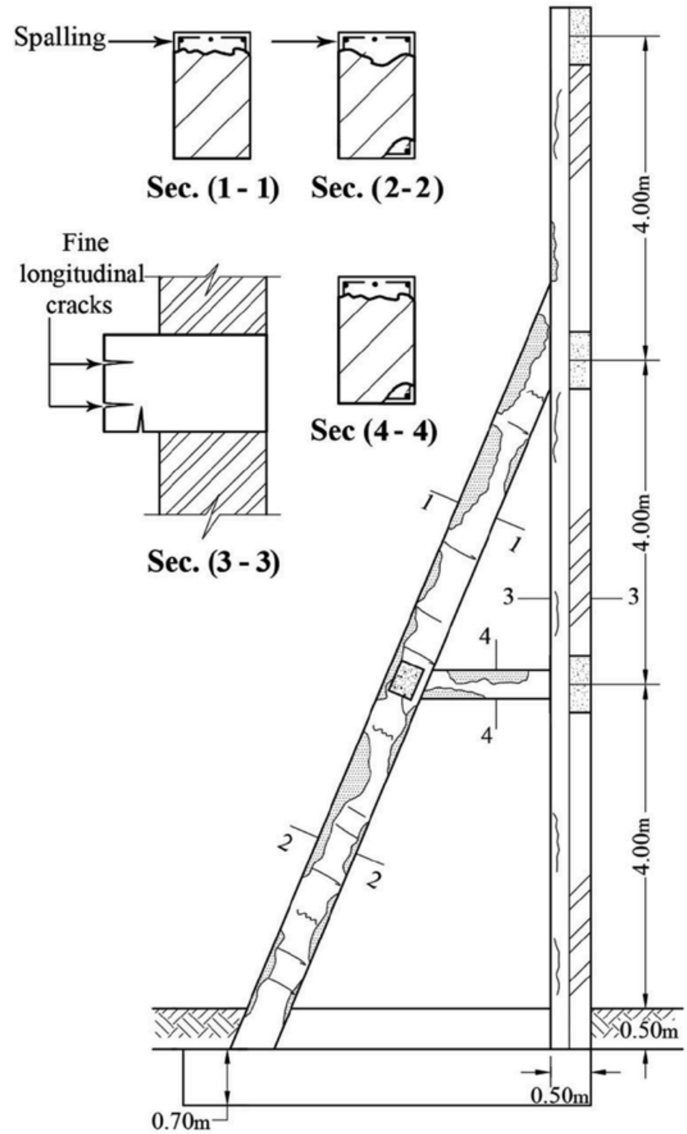


Fig. 6: Cracks and spalling of existing RC members due to corrosion and salt weathering (Note: 1 m = 3.28 ft)

The weather conditions at the site promote attack by salt weathering. When airborne seawater is deposited on the surfaces of concrete due to the temperature decrease at night to the dew point, the next day, when the temperatures rise to higher degrees, it leads to rapid drying of the surface layer of concrete. Thus, pure water evaporates, leaving salt crystals in the pores of this layer. This phenomenon is progressive and leads to the continued growth of salt crystals in the pores, causing disruption of the cement mortar to a considerable depth from the surface and leaving behind loosened coarse aggregate particles.

- For Member B, the vertical members of the main frames, and Member E, the horizontal curtain wall beams, unlike Member A, are mostly protected against the aggressive agents by the curtain wall itself (refer to Fig. 5). The corrosion of reinforcement was manifested as narrow, intermittent, mainly longitudinal cracks on the exposed parts of their surfaces.

It should be emphasized that, according to the state of damage of Members B and E, the decision to keep the whole structure as is—with some repair—and replacement of only Members A, C, and D (Option 1), which leads to considerable economy and meets the demand of the owner, or demolishing the whole structure, including the voluminous curtain wall and reconstructing a new complete one (Option 2). So if all the surfaces of Members B and E are required to be available for repair, because almost all their reinforcement are corroded, then (Option 2) is a must.

To reach a decision about this, the following field and laboratory tests were carried out:

1. The concrete cover over the reinforcement in places of vertical cracks was removed (in six different places). The reinforcing steel bars appeared to be in generally good condition with a slight change in color due to mild corrosion. The reduction in bar diameters was very small and nearly not noticeable by the naked eye.
2. Small parts of the bricks of the curtain wall adjacent to the RC frames were removed (in six different places) to a depth of 6.0 in. (150 mm) for visual inspection. This proved that there were no vertical or horizontal cracks in these areas, and that there was no corrosion in the protected parts of Members B and E. This conclusion was promoted by the fact that there were no deformations or cracks in the wall in the vicinity of Members B and E.
3. Core samples were drilled in places of vertical cracks in the exposed faces of Members B and E and the samples (in six different places) were extracted at depths of 1.2, 2.4, and 4 in. (30, 60, and 100 mm) from the surface. These samples were laboratory tested for chloride ion content. Results of the tests were as follows:

Depth, in. (cm)	Value of chloride ion content per kg/m^3 of concrete	
	Maximum	Minimum
1.2 (3.0)	0.80	0.30
2.4 (6.0)	0.20	0.05
4.0 (10.0)	0.05	0.0

Chloride ions (cl$^-$) intruding into RC from seawater or other sources can cause corrosion to reinforcing steel if oxygen and free moisture are available to allow the required electrochemical reactions to take place. The possibility of corrosion increases with the increase of the concentration of cl$^-$, and when the concentration exceeds a certain limit—known as the threshold level—the possibility of corrosion rises and needs be considered.

For many reasons, the concentration of cl$^-$ in concrete from all possible sources, and also the oxygen and humidity levels, are not uniform. This is why different codes allow different values of the threshold level of cl$^-$. The initiation of corrosion, however, not only depends on the total amount of chloride ions, but also depends on the chloride differential along a steel bar or between adjacent bars. Thus, the results of tests for the cl$^-$ content in concrete need to be considered with sound engineering judgment as being one of the factors that controls the potential corrosion in concrete, on which the repair program is designed.

Many investigators consider that if the chloride content in concrete exceeds 1.5 lb/ft^3 (0.7 kg/m^3), then the amount of cl$^-$ is sufficient to cause corrosion.

From the results shown in the previous table, Members B and E, abutting against the curtain wall, need not be removed and only repaired from corrosion on exposed surfaces. In this case, Option 1 is valid. Member A and its ties, Members C and D, need to be demolished and replaced by new members.

ADOPTED SOLUTION

As a principle, the demolition of extremely deteriorated members should not take place until after the construction of their replacements and after properly connecting them to the existing RC structure to guarantee their monolithic action. For this reason, the replacement members should follow new paths other than those of their prototypes. Figure 7 shows the configuration of both systems.

The repair of mildly corroded members (Members B and E) should precede the construction of the new members (Members A, C, and D) to guarantee proper connections of new to existing repaired members. First, Member E was repaired because the repair of Member B required shoring with steel elements fixed closely on either side under the lower repaired Member E, and rest on the raft foundation.

The repair of Members B and E were carried out in consecutive order; the nine vertical frames in Member B were repaired in three stages and adjacent members were not repaired at the same stage, as shown in Fig. 8. Member B was also repaired in three nearly equal stages along its length, beginning from the foundation level. A similar approach was followed for Member E.

Repair of Mildly Corroded Members

The repair process comprised the following steps:

1. Cutting and removing deteriorated concrete cover over reinforcement and extending the cut to about 0.8 in. (20 mm) beyond the inner faces of the reinforcement. For this step, hand-chipping tools were used because local experience proved that well-trained laborers could perform with suitable speed and guarantee the safety of the remaining concrete;
2. Removing the corrosion on the steel bars by sand blasting;
3. Preparing the remaining concrete surfaces to receive and bond firmly with repair materials. This step included removing the debris of deteriorated concrete; removing loose aggregate and mortar pieces with the help of compressed air and wire brushes; and flushing with potable water, then verifying that the roughness of the concrete surface was suitable for good bond with the repair material. Generally, hand-chipping tools lead to suitable roughness of the surface;
4. Add additional reinforcement if the bar diameters were reduced, by corrosion, to more than 10%; and
5. Before casting the repair concrete, the clean surface of the remaining concrete was covered by an adhesive polymer—epoxy resin—used to provide good bond between the two materials.

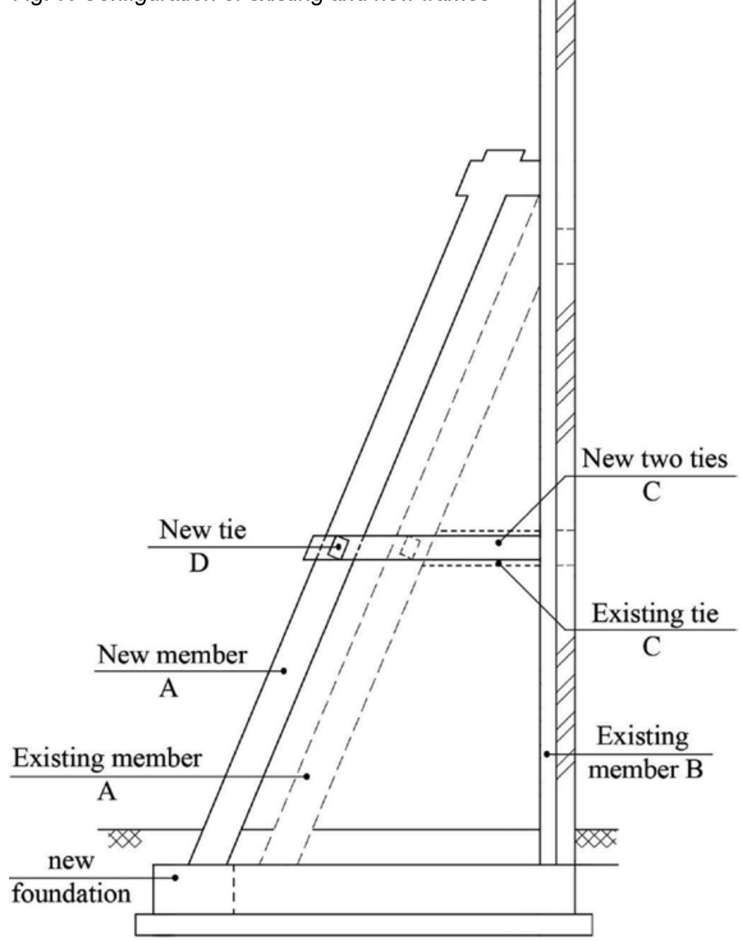
The mixture design for repair needed to satisfy the important condition of low permeability required for structures in a marine environment. Low permeability was achieved mainly by reducing the water-cement ratio (*w*/*c*) to a chosen value of 0.37. To keep a suitable workability for shotcrete application (in the dry mix process), high-range water-reducing admixture, chosen as the most suitable for the repair, was added to the mixture. Aggregate grading followed the recommendation for shotcrete mixtures with a maximum nominal aggregate size of 3/4 in. (19 mm).

Laboratory and field tests yielded the following suitable mixture proportions:

Cement, lb (kg)	772 (350)
Sand, lb (kg)	1290 (585)
Gravel, lb (kg)	2865 (1300)
Water, gal. (L)	34.2 (130)
w/c	0.37
High-range water-reducing admixture, gal. (L)	2.03 (7.5)

Trial batches of shotcrete at the site proved to be of suitable consistency to support itself in the vertical, sloped, and horizontal positions with good compaction without excessive rebound.

Fig. 7: Configuration of existing and new frames



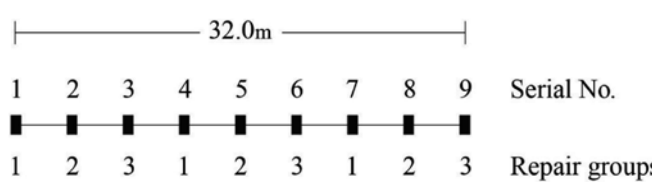


Fig. 8: Sequence of repair of Member B (Note: 1 m = 3.28 ft)

The resistance of the repaired members to future corrosion was also enhanced by good curing of the fresh shotcrete, characterized by large surface area compared to its volume.

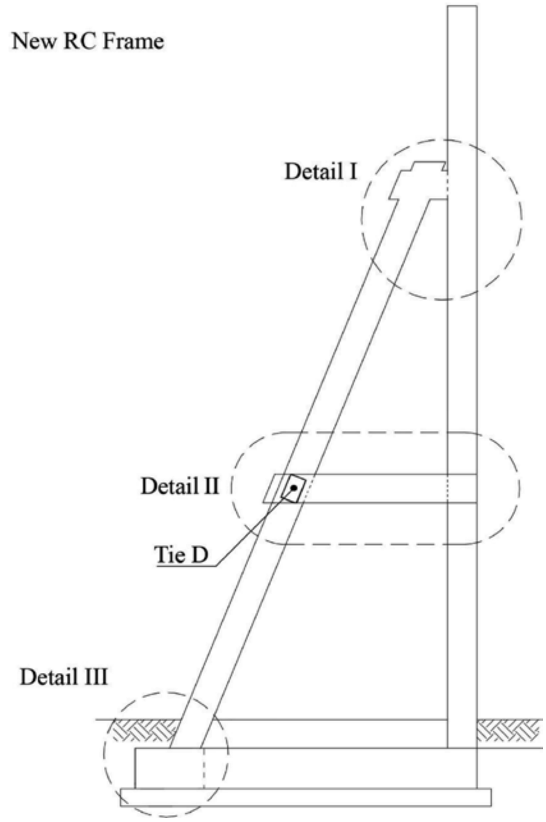
The increase of the concrete cover over reinforcement to conform to the standards for concrete in a marine environment is also of great value in magnifying the resistance to corrosion.

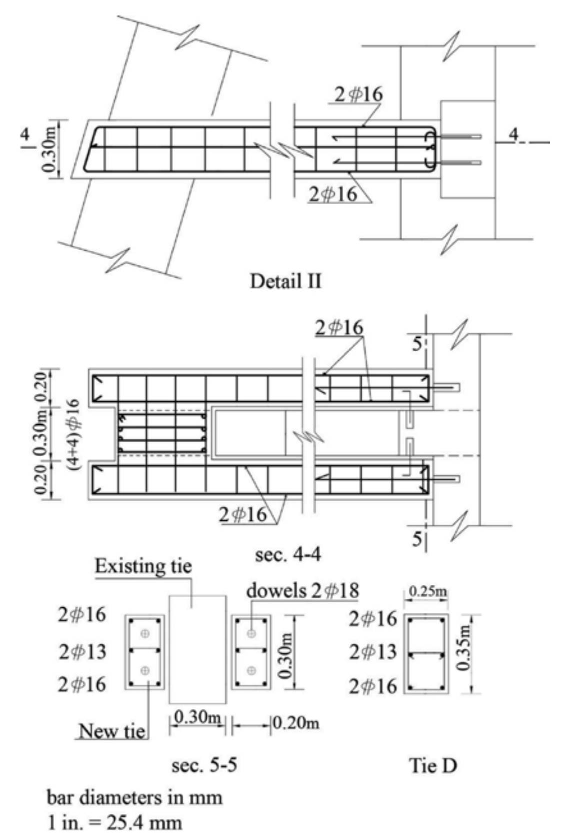
These mixture proportions were used with a slight modification for the concrete of the new members due to increasing the maximum nominal size of the aggregate to 1.2 in. (30 mm).

Replacement of Highly Deteriorated Members

Referring to Fig. 7, the new Member A was laid in the same vertical plane of the existing Members A and B because it has to form a frame action with Member B.

The deteriorated Tie C was replaced by two ties on either side with clear spaces of 3/4 in. (20 mm) and at the same level. The new longitudinal Tie D was placed freely at its proper position.





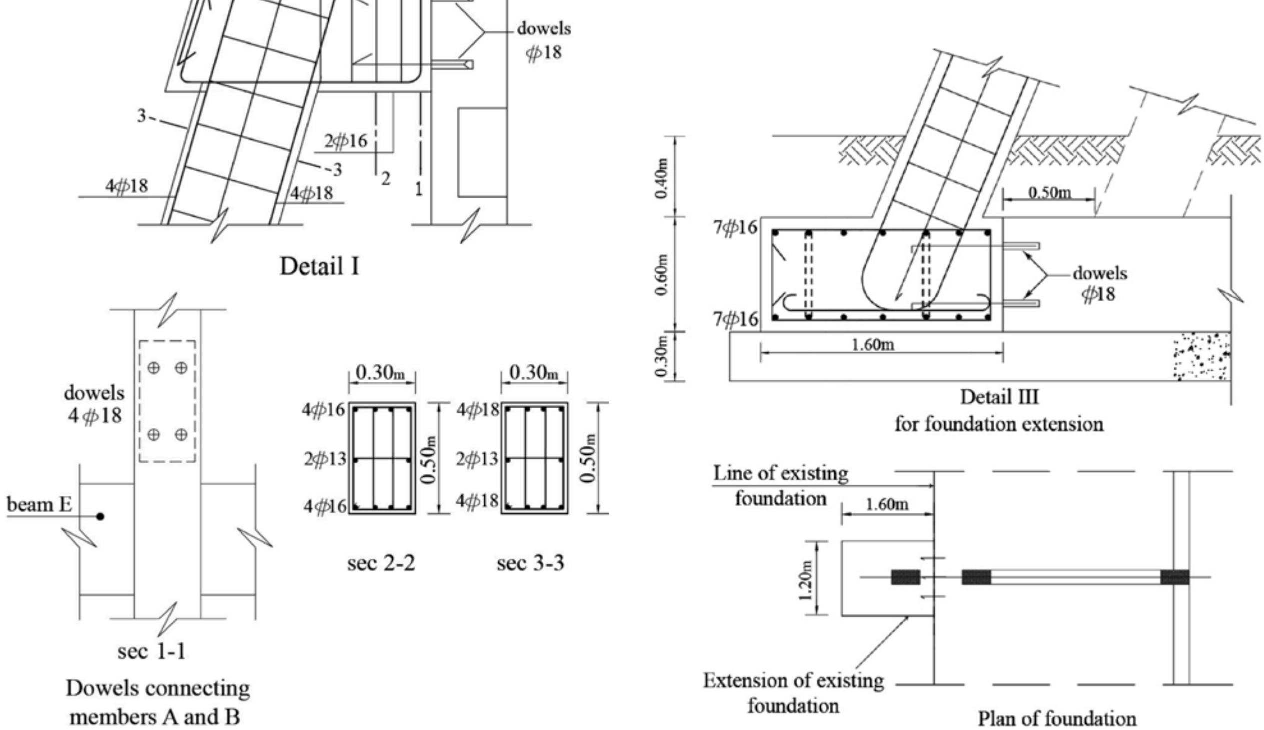


Fig. 9: Details of new Members A, C, and D (Note: 1 m = 3.28 ft)

Structural analysis and design of different sections of the members of the new RC frames were performed under the effect of vertical load (own weight of the whole structure), horizontal wind load, and temperature differential. Details of these new members are shown in Fig. 9, which also includes an extension of the existing foundation to accommodate the new Member A.

The joint between an existing and a new member was first performed by drilling circular holes of diameters of 1 in. (25 mm) in the existing members to a depth of 8 in. (200 mm). A glass capsule of two separate compartments each filled with one of the two components of adhesive epoxy was placed in each hole. A dowel of ribbed high tensile steel rod, 0.72 in. (18 mm) in diameter, was placed concentric with the hole—pushed and rotated to nearly its full depth—so that the capsule was broken and the two components of epoxy were mixed and filled the hole, thus bonding the dowel bar to the existing member.

Three laboratory pullout tests on replicas of the prototype joint were performed; the constant pullout force was three times the maximum joint force in the frame. All the tests were successful because no displacement, either between the rod and the adhesive or between the adhesive and the surrounding concrete, were measured.

The joint was then completed by preparing the surface of existing concrete to bond with a new one. This was achieved by roughening the surface using wire brushes, cleaning the surface by compressed air, and flushing by clean water. Epoxy resin, as solvent-free adhesive, formulated to act properly through a range of atmospheric temperatures of 72°F (40°C) to suit the ambient temperatures at the site, was brush applied to the surface shortly prior to placement of the new concrete.

PROJECT SUCCESS

Fifteen years after the time of restoration of the structure, it is still in good condition. The solution adopted Option 1 (including replacement of highly deteriorated members only and repair of other members) and was cost effective, an important condition to the success of the project. The cost savings was 35%, compared to the estimated cost of Option 2 (including demolishing and reconstructing the whole structure). The project was also time effective, as the time needed to complete the work was 15 weeks, whereas that estimated for Option 2 was not less than 22 weeks, taking into consideration the difficulties in demolishing the curtain wall.

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