Columns Replacement in the Church of Saint Fatima

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ABSTRACT

The town of Beit Sahour is an important biblical site in Palestine. Saint Fatima's Church, built more than one hundred years ago and reconstructed in the year 1949, is owned by the Latin Patriarchate and serves the small catholic community of the town. The church carries the name of Our Lady of Fatima which was the title given to the vision of the Blessed Virgin Mary when she appeared before three shepherd children in the town of Fatima in Portugal in 1917. In the summer of 2008 the parish priest of the Latin Congregation embarked upon a challenging and an aggressive project of changing the pillars within the church in order to widen the viewing angle to the altar area. The following is a detailed account of the design process together with the construction methodology that proved to be feasible in cost and effective in execution time. The sectional area of the new pillar is about a quarter of the original size. The success of the undertaking was measured by the extent of the induced cracks in the vicinity of each column. These cracks were either minimal or invisible. The section of the new pillars was made of composite material; structural steel profiles imbedded in concrete. The intermediate shoring system was an ad hoc steel structure. Replacing the columns was also an opportunity to develop a more contextually consistent and complementary theme within the church.

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INTRODUCTION

Saint Fatima's Church, which belongs to the Beit Sahour Latin community and with a 300-seat capacity, is an older heritage structure which is elegantly designed and very well constructed. It was initially built in 1863 to serve the small catholic community of the town universally acknowledged to be the site of the Shepherds' Field where the angles appeared to herald the birth of Jesus Christ in the neighboring town of Bethlehem. In the year 1949 a newer church was built on top of the older one by the renowned Italian Architect Barluzzi who designed the Church of all Nations at the Gethsemane Garden in Jerusalem. The church main structure is comprised of three vaulted bays identical in shape, except that the middle bay is narrower. Its massive walls are built of native stones. The structure is a typical masonry one with a high ceiling supported on a grid of arches that span in both the short and the long directions. It radiates the loving, exacting architecture of the Holy Land. No apparent mathematical shape could be accurately defined to describe the curvature or the topology of the arches; however, their general shape approximates a pointed arch, which is known for its effective ability to transform vertical loads into lateral reactions causing minimal values for moments within the arch element itself. Within the said arches, elegant vaults span the area to create an architecturally attractive ceiling and produce a charming inner isometric view. Intricate architectural details add to the solemn atmosphere, prerequisite for serious worship vigils.

OBJECTIVE

The original diameter of the columns within Saint Fatima's Church stood at about 60 centimeters; their height was about 275 centimeters; they supported a system of two way arches and vaults which characterized the masonry structure of the church. The exaggerated size of the columns blocked worshippers from viewing the altar area, as illustrated in Photo 1.

Photo 1. General Interior Original View

The columns were made of five round blocks of pink native Palestinian Mizzi Yahoudi lime stone taken from Bethlehem area quarries. The top of the columns had an attractive hand carved crown and the bottom had an effective stone pad. The purpose of the engineering exercise was to replace the bulky masonry stanchions by new ones but of narrower diameter. (Photo 2 is a snap shot for one of the targeted columns). The new stanchions would have a diameter of no more than 30 centimeters. The esthetics of the final delivered product was as important as the safety of the structure proper together with the safety of the church parishioners. The high risk involved in the undertaking had to be very well considered.

Photo 2. The Original Column

ENGINEERING APPROACH

It was observed that the bases as well as the crowns of the church columns were made of quality pink stone with elegant carvings. This implied that the capital stones were either one piece extending from one end to the other or that the crowns and the bases were made up of two adjacent stone pieces. These elements were to remain in place. Each column was comprised of five round stones of 60 centimeter diameter. The architectural normally layout and the vertical section around a typical column give a good estimation of the loads involved. The loads that supported by each column included a dead load, a minimal live load as well as a minimal snow load as the town of Beit Sahour is seldom known for severe snow storms. A typical tributary area and a relevant section from which loads were quantified are shown in Figures 1 and 2. While the masonry column was being replaced an effective temporary shoring system was put in place. The operation involved the transfer of loads at two stages. The new column had a composite section made of structural steel embedded in concrete, shown in

Figure 3. Chiming with steel chims was the suggested method for establishing snug anchor for the newly mounted column. Extra attention to details and comprehensive safety precautions had to be taken; these included, but were not limited to, skilled workers and proper insurance coverage. Furthermore, perhaps more intricate in design and construction was devising a moveable shoring system capable of supporting the church ceiling while the columns were being replaced. This was accomplished by the versatile rigid steel frame shown in Photo 3.

THE MATHEMATICAL MODEL

The Finite Element Program SAP2000[1] was judged suitable for the present analysis undertaking. This was primarily based on the availability of its Section Designer feature. A one dimensional column element was constructed with 2 loading cases and one load combination. The section was entered into the model via the section designer wizard. Since no standard section was readily available to meet the existing conditions. A built up section was the prudent choice. The prototype column was envisaged to be topped by a steel plate and another base plate at the bottom end. Heavy loads and friction on the end plates were expected to render the column as fixed at both ends. The Allowable Stress Design (ASD) method was selected as the course of action in spite of the high dead load to live load ratio. Conservatism was judged necessary in the present undertaking. Therefore, the bottom end was modeled as a pinned end while the upper one was modeled as a roller in the z direction and it was restrained in both the x and the y directions. This is illustrated in Figure 4.

LOAD ESTIMATION

The tributary area on each interior column of the church structure was conservatively estimated at 20 square meters. The depth of the roof ceiling is about 1 meter, in addition to the bulky arches. Therefore the applied service loads were estimated as follows[2]:

- Dead load
  400 KN (ceiling) + 200 KN (arches) = 600 KN
- Live load
  $20m^2$@$1.50 KN/m^2$ = 30 KN
- Snow load

$20m^2@1.50\ KN/m^2 = 30\ KN$

Seismic loads were not considered because the entire structure is not seismically designed. Furthermore, the temporary supporting steel structure was designed to carry the dead load only.

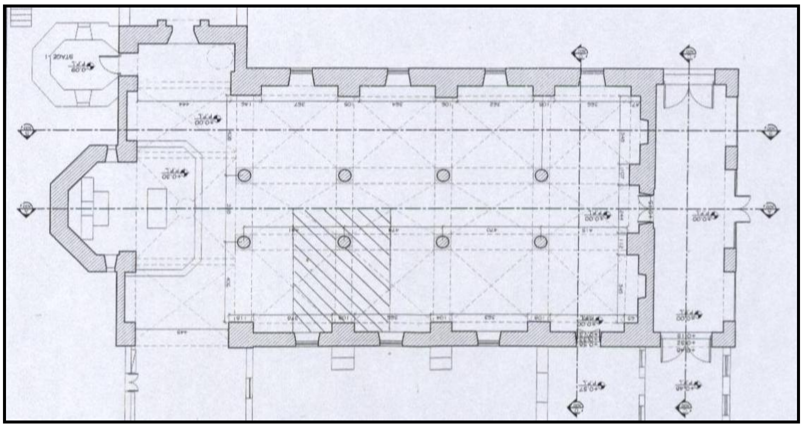

Figure 1. The Floor Plan of the Church

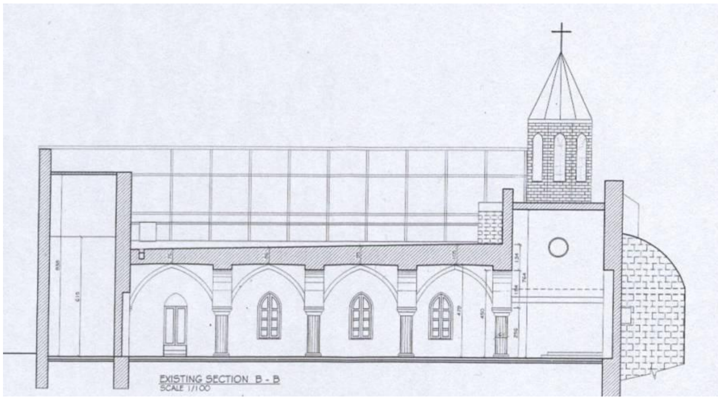

Figure 2. A Longitudinal Section

The column was expected to be subject to vertical axial loadings; however, accidental eccentricity of 7 centimeters due to the possible unsymmetrical loading in either direction was assumed for added safety. This could be perpetual or transient happening during construction. The induced bending moments in the x and the y directions were estimated at 42 KN-m. The section as defined in the computer model is shown in Figure 3.

The proposed section was a composite made of one hollow steel tube of 324 mm diameter and 8 mm wall thickness holding 2 orthogonal UB 254x146 sections with 4 stiffener plates of 100x20 mm welded to all flanges. One UB section had to be cut along the middle for obvious reasons. The rest of the volume was filled with concrete. The column had a round steel base plate of 350 mm diameter and 20 mm thickness. The column was topped by a 30 mm round steel plate. Good quality concrete adds to the durability of the section, provides confinement to the steel elements in addition to the added compressive strength that it provides.

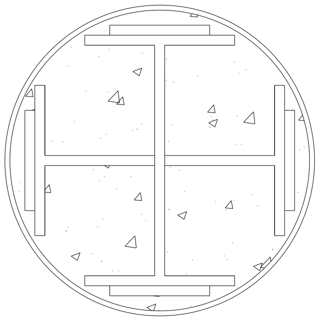

Figure 3. The Section of the New Column

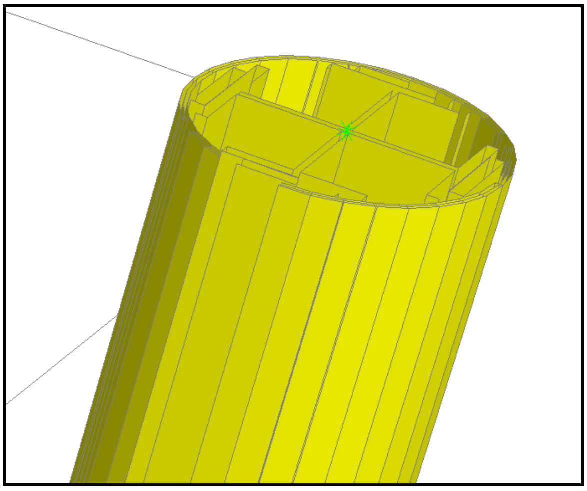

Figure 4. The Mathematical Model

The following are the section properties as computed by SAP 2000; followed by the Euler buckling load which is defined as the maximum axial load that a column can support when it is on the verge of buckling:

Area = 330.2 cm^2	R_{major} = 9.18 cm
I_{major} = 27,835 cm^4	R_{minor} = 9.16 cm
I_{minor} = 27,715 cm^4	E = 200,000 MPa
Z_{major} =7,110 cm^3	F_y = 25 MPa
Z_{minor} = 7,071 cm^3	

Euler Buckling load for a column is given by the following equation

$P_{cr} = \pi^2 E I/(kL)^2$

For k=1, which is the effective length factor for a pin ended column[3]. The column is conservatively assumed having pinned ends thus unable to resist end moments.

$P_{cr} = 3.14^2 \times 200{,}000 \times 27715 \times 10^4/(3000)^2$

$= 60{,}724$ KN

Due to the nearly symmetrical section the buckling load is nearly equal in both the x and the y directions. Euler buckling load is based on the assumption that the column is perfectly straight, both ends are pinned and the load is perfectly axial. This rather large value was indicative of the ultimate compressive strength of the selected element, thus the built-in safety.

THE INTERMEDIATE SHORING SYSTEM

With the new column comprehensively designed and ready for mounting, the column replacement exercise hinged upon the removal of the existing column with absolutely no damage to the main church structure. Any small damage would be irreversible, would pose tremendous hazard and would render the premises unsafe. The load resting on the column had to be effectively shored. Figure 5 and Figure 6 show the steel framed shoring system whose design and analysis were meticulously accomplished using SAP 2000 to guarantee minimal, if any, deflection under dead load action on the directly supporting elements. Moment resisting braced elements and truss elements were specified together with appropriate shear or moment connection patterns using 20 mm A325 bolts[4]. Impact loading under any circumstance was completely ruled out. Furthermore, the shoring structure had to be mobile, light and fully stable.

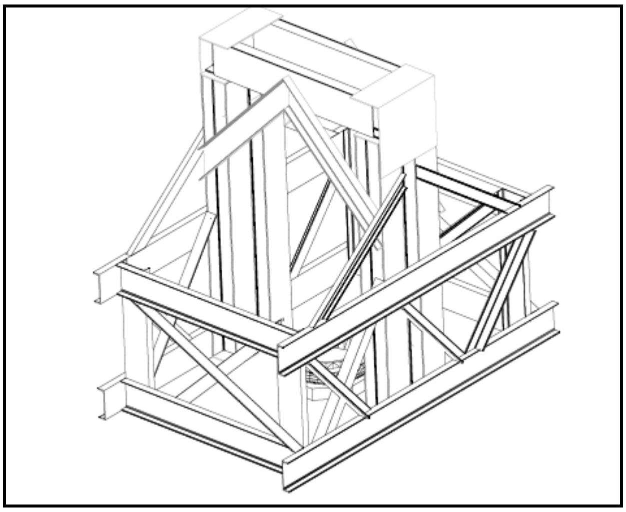

Figure 5. A Perspective View of the Temporary Shoring System

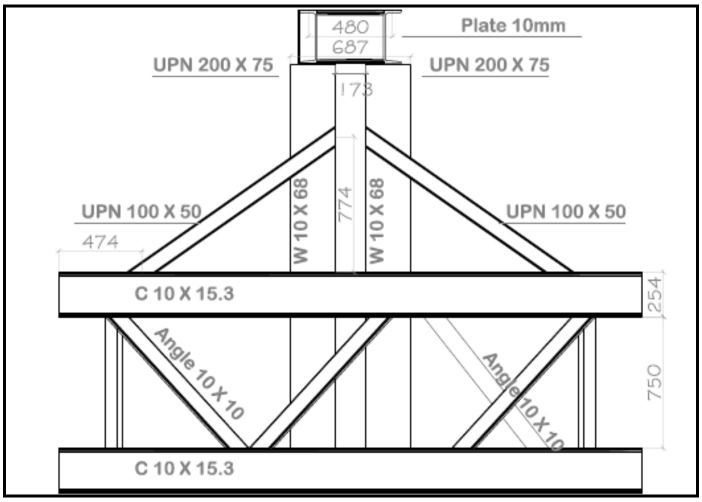

Figure 6. Details of the Shoring System

FIELD EXECUTION PROCEDURE

The column replacement project was quite intricate in nature and held a wide margin of risk for the structure itself and for personnel involved in work execution. It was therefore essential that a detailed procedure be mapped out a priori and precisely followed.

a) The procedure was applied to one column at a time.
b) Each of the new columns weighed about 500 kilograms. Therefore, enough manpower had to be available to handle such a load during the mounting procedure. No load carrying machinery could be summoned.
c) The top round stone of the existing pillar was cut by a Power Saw along two parallel lines and at 35 cm spacing. The part of the stone, which was cut, was discarded while the inner part remained in position.
d) The temporary supporting steel structure was to be effectively and promptly put in place as shown in Photo 3. This is a rigid frame, with braces modeled as truss elements, designed to yield minimal deflection under the applied dead load. Connections made of 20 mm A325 bolts, allowed for system mobility.
e) To overcome the slight variation in column height the shoring system was placed on level struts of lumber that were cautiously adjusted in height at each successive column location.
f) Center lines of the existing columns were accurately marked on the existing stone column capital. Every attempt was followed to locate the geometrical centroid of the section. The centroid of the column had to fall exactly under the centroid of the existing column capital. There had to be absolutely no tolerance in this aspect.
g) The shop-constructed shoring frame was 5 mm too short than the available clearance. The extra narrow space was reserved for tapered steel chims that were custom made and hammered to hold the new column snug in place.
h) The new composite column was shop-constructed 5 mm too short. Chimming allowed it to hold snug in place. It had to be hauled to location prior to the completion of the assembly of the temporary steel shoring structure.
i) Grouting was the last step to be implemented around the base and the crown plates. The temporary structure was removed the day after. This seemingly innocent step proved to be quite intricate and cumbersome as disassembling a loaded frame required that the

whole shoring structure be fitted on a system of bolts, clearly shown in Photo 4, between the floor and the lower beams. Unscrewing the bolts relieved the stress from the loaded structure thus the disassembling effort became a rather simple exercise.

j) Painting and decorating the body of the new stanchion to make it blend in with the rest of the church architecture was the final touch to the general esthetics of the final delivered product. This is shown in Photo 5.

k) The success of the operation could be measured by the extent of crack widths of the gravity arch and the adjacent church walls. Such cracks were either totally absent or minimal. The church went back to serve the community of parishioners, albeit with a better interior design.

Photo 3. The shoring steel frame

Photo 4. The Shoring Steel Frame

Photo 5. The New Interior View

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