



SEISMIC RETROFITTING OF MANI MANDIR COMPLEX AT MORBI, GUJARAT, INDIA

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SUMMARY

The Mani Mandir complex ($100m \times 100m$ in plan) is an important historic monument of the town of Morbi in the western state of Gujarat, which suffered significant damage during the M7.7 Bhuj earthquake of 2001 in India. As part of the earthquake reconstruction program, the Government of Gujarat decided to seismically retrofit this complex. The project was divided into two phases of design and execution; this paper discusses the evaluation and design procedures recommended for execution. A detailed condition survey was carried out and measured drawings were prepared. A comprehensive retrofit program was formulated. Conservation principles, minimum intervention and consonance with the heritage character of the building were important considerations in selecting the retrofit program. The complex was modeled using finite elements and behaviour was studied of the existing structure as well as retrofit structure. The retrofit measures recommended included discriminate use of internal reinforced concrete skin walls, providing a rigid diaphragm behaviour mechanism in existing slabs, introducing stainless steel reinforcement bands in the existing masonry walls, cross-pinning and end-pinning in walls and pillars, and strengthening of arches and elevation features.

INTRODUCTION

Morbi is situated in the western region of Gujarat at a distance of about 125 km from the epicenter of the 2001 Bhuj earthquake. The city is famous for its beautiful architecture, which is a harmonious blend of Gothic, Saracenic, Mughal and Rajput styles. The imposing complex is made up of the Willingdon Secretariat and the Mani Mandir temple (Figure 1a). It is scenically located on the western banks of the

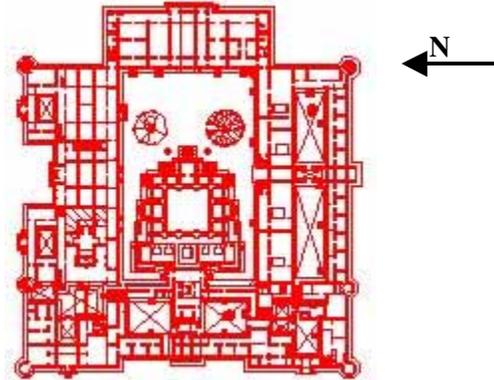
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(a)
Fig. 1: Mani Mandir Complex (A) View from Southwest Corner and (b) Plan of Complex.

Macchu River, just outside the fortified walls of the old Morbi town. The complex was built in the 1930s by the ruler of Morbi. It comprises of a very ornate masonry building built in yellow sandstone in the tradition of the Indo-Saracenic style of architecture. The Secretariat building has a large central courtyard housing the Mani Mandir temple (Figure 1b). The total area of the Willingdon Secretariat is 4900m² on the ground floor and 4150m² on the first floor. There is a part second floor of about 255m². Prior to the earthquake, important government offices occupied a portion of the complex. The extensive damage suffered during the earthquake has left the building unfit for occupation and some parts of the building are in imminent danger of collapse.

Seismic retrofit of historic buildings is not often carried out in India and there are no codes to guide such retrofit. The project brief specified that the buildings be upgraded to meet the current seismic code specifications. These are stringent and punitive for unreinforced masonry structures and the challenge lay in a harmonious integration of seismic retrofit program with the architectural restoration and conservation program (Penelis [1]).

The principles that governed the seismic retrofit program were:

- * Avoid intervention to maximum extent possible.
- * Introduce retrofitting measures in consonance with the heritage character and principles of conservation.
- * New elements must be non-intrusive and compatible with existing materials.
- * New elements must not be a cause of further damage (such as corrosion).
- * Retrofit measures must be easy to implement.

DAMAGE DOCUMENTATION

Original drawings for the building were not available; fresh measured drawings were prepared. Three sets of condition surveys were carried out with the help of these drawings. Firstly, a macro-survey was carried out, which helped in identifying grossly the areas of severe, moderate and minor damage, and also the areas that required emergency interventions. Secondly, a detailed survey was conducted floor-wise, wing-wise to identify the types of damage. Thirdly, a micro-detailed structural survey was carried out room-wise exhaustively documenting all damages including the extent of corrosion, the location of structural members and their sizes, and the length and width of cracks.

Existing Structure

The Willingdon Secretariat was built originally as a two-storey complex with load bearing walls of soft, yellow sandstone above plinth and black basalt stone below plinth. The walls are of 450mm thickness above plinth. The stone is dressed and exposed on the external side and coated with paint or lime wash internally. The masonry is of Ashlar-type. Stone blocks are not physically bonded to each other by a clearly identifiable mortar layer and stay in place almost purely by bearing friction. Some of the stones are locked to each other by means of wooden keys. Such keys are few and far between and were found primarily in the *chhatris* (ornamental canopies) and *shikhars* (decorative towers) above the roof. The stone blocks of pillars are socketted into each other by a small tongue and groove detail. The walls are punctured at numerous places for doors and windows.

The floors are built of stone slabs of about 750mm width and of 200mm thickness. These slabs are wedged between flanges of steel joists (Figure 2a). The joists rest on a stone cornice (made of cornice pieces) running along wall lengths. A floor finish (of about 150mm thickness) was provided over the stone slabs. There are arches along the external façade walls and internally across passages.



Fig. 2: Details of Flooring (a) Stone Slab Wedged between Steel Joists, and (b) Separation of Joist Flanges and Failure of Stone Slab.

Damage Prior of 2001 Earthquake

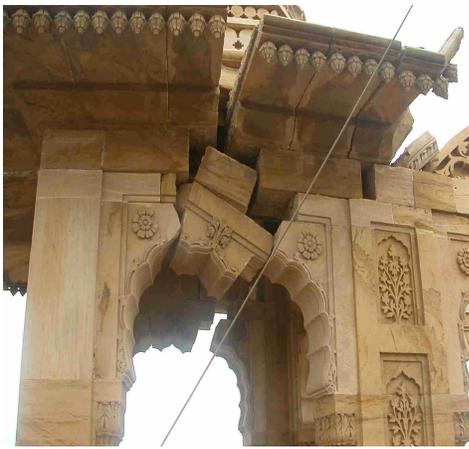
The condition survey showed considerable damage to the buildings even prior to the 2001 earthquake. General disregard and poor maintenance of the monument had caused deterioration in the form of corrosion of steel joists, damage to cornice pieces at the steel joist locations, weathering and flaking of sandstone, roof leakage and peeling of internal paint. The steel joists were corroded in almost all locations, though to a varying extent. At the roof level, the joists were very heavily corroded and showed separation of the flanges from the web (Figure 2b). Roof leakage appeared to have been a recurrent problem. Numerous attempts to remedy this situation were evident; each attempt resulted in the addition of a fresh layer of waterproofing. Three separate layers of waterproofing were observed, one on top of the other, together adding up to more than 300 mm thickness. This terrace waterproofing had cracked heavily. The general damage due to deterioration and weathering was less intensive on the first storey and very limited on the ground storey.

Damage in 1956 Earthquake

The structure appears to have suffered some damage in the 1956 Anjar earthquake (M6.1). The extent of damage during the earthquake is not known but displacement of keystones of portals, arches and movement of stones of walls seems to have occurred. Retrofitting measures were undertaken at that time.

Damage in 2001 Earthquake

The earthquake caused severe damage and collapse of a large number of elements above the roof, extensive damage at the roof level, moderate damage at the first storey and little damage on the ground storey. Staircase cap slabs, parapets, *shikhars*, arches, portals and *chhatris* above the roof were very badly damaged; a large number of them were partially or completely destroyed (Figure 3a). The bastions at the extreme corners of the structure sustained severe damage including partial collapse; the portions standing are precariously balanced and have wide, through-cracks (Figure 3b). Similar observations were noted in the temple where the corners have been severely damaged (Figure 4a). Stone weather-sheds at the roof level were broken and destroyed at some places.



(a)

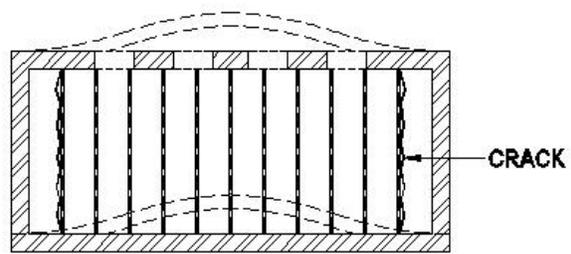


(b)

Fig. 3: Damage above Roof Level: (a) Destruction of Arches in Elevational Elements, And (b) Partial Collapse of Bastions.



(a)

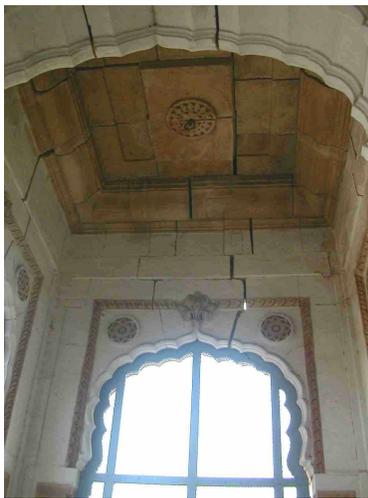


(b)

Fig. 4: Floor Level Damage: (a) Out of Plane Collapse at Corners of Temple Portion, And (b) In - Plane Separation in Slab along End Joist.

Some portions of the roof slab suffered collapse. This was primarily observed in the regions where the joists supporting the slabs rested on capitals of individual stone columns. These joists were significantly corroded. A large number of cornice stones at the junction were cracked, locally crushed or dislocated. A typical crack pattern was observed in the floor slabs: In many bays, there was a crack running parallel to the joists all along the junction of the end joist (Figure 4b). Weather sheds were broken at this level due to the falling debris.

At the first storey, almost all joints in arches and pillars opened out (Figure 5a). A large number of the joints in walls also opened out in a typical diagonal pattern (Figure 5b). Cornices below steel joists were damaged at many locations (Figure 2a). The arches cracked heavily and the keystone were dislodged in most arches. Deformation of the arches was also observed. Many decorative galleries and some walls show a distinct tilt; they have been pushed out and away from the building. Numerous wide cracks were formed in walls. Many walls opened out at corners.



(a)



(b)

Fig.: 5 Damage to Vertical Elements: (a) Openings of Joints in Arch, and (b) Diagonal Cracks in Walls.

In the ground storey, there was no major seismic damage; the overall condition is reasonably good except for minor cracks in walls and a few separation cracks at the crown of the arches. On the south and southwest walls of the temple, heavy weathering was observed, possibly due to the effects of the monsoon.

The foundations were excavated at two locations and inspected. No damage or differential settlement was observed in the structure.

Previous Attempts of Retrofit

As mentioned earlier, there have been some earlier efforts to retrofit the buildings. Keystones of some arches and stone blocks of a few walls and portals were stapled to adjacent stones, possibly as a response to the 1956 Anjar earthquake. The steel stitching being stronger than the parent stone, caused local crushing of the stone even though it prevented complete dislocation of the stone from the adjacent one. (Figure 6a). There were attempts to contain the weathering of the buildings (Figure 6b). This was done by insensitively plastering the surface with cement mortar. At one location, some arch pillars that had weathered badly, had been fully jacketed in concrete.



(a)



(b)

Fig. 6: Earlier Interventions towards Restoration: (a) Fastening of Stones Using Mild Steel Staples, and (b) Repairing of Weathered Stone.

BEHAVIOUR ANALYSIS

Poor Bonding Between Stones

As mentioned earlier, the Secretariat exhibits extensive damage at the roof level, moderate damage at the first storey and little damage on the ground storey. One of the reasons for such behaviour is the poor bonding between the stones. There is almost no mortar in the joints and in the absence of a bonding material, the bond between the stones is derived from bearing friction. At the roof level as there is very little load on the walls, the friction force will not be sufficient to resist the lateral forces.

Lack of Rigid Diaphragm Action of Slab

The structural system of the floor comprises of stone slabs with steel joists bearing on the cornice. The cornice is marginally socketted into the wall. There is thus almost *no diaphragm action* of the floor slab. As a result, lateral inertia force generated due to the floor mass is not transmitted to the walls in proportion to their stiffness. Instead, it gets applied as a horizontal thrust to the masonry walls in their weaker direction. This lateral force is in addition to the lateral force generated due to self-weight of these heavy walls. The combined lateral forces cause significant out of plane deformations and subsequent damage to the walls.

Relative Displacement in Slabs

As noted earlier, cracks parallel to joists are seen at some places on the underside of the slab, along the junction of the joist and stone slab adjacent to wall. These cracks occur due to the relative displacement between the rigid wall (parallel to the joist) which undergoes very little displacement, and the joist which undergoes significant displacement (Figure 4b). There is also maximum damage to the cornice at this location due to local crushing. Shear failure in the form of diagonal cracks (opening out of joints) was observed in many walls. There was no crushing of stone in the walls. The damage is most severe in walls that exhibit variation in the stiffness due to random locations of openings on different floor levels.

Reentrant Corners

The Willingdon secretariat is a building of immense proportions with a large central courtyard and several internal courtyards in each of the wings. There are many reentrant corners thus formed. Expectedly, these corners suffered damage of much higher magnitude than other areas. The highly stiff bastions at the extreme corners of the building attracted very large forces and were severely damaged and collapsed partially.

Arches

The arches in the structure have cracked heavily and the keystone has dislodged due to the induced tension in these arches during reversal of stresses. Dislocation (displacement) of the arch supports have also caused the first hinge to form at the center of the arch in the seismic event.

STRUCTURAL ANALYSIS

Material Tests

Core tests on the stone were carried out to obtain the crushing strength and tensile strength. These were found to be between 20-40 MPa and 1.6-3.9 MPa respectively. Water absorption was found to be in the range of 6-7 % and the density of the stone was 22.5 kN/cu. m. Chemical analysis of the stone was also carried out. The results confirmed that the stone was sandstone with 92% silica and 5.5% alumina content. Soil excavations were conducted to verify the type of foundation soil. The founding depth was approximately 2.6 meters from the ground level and the soil encountered was medium stiff clay.

Modeling and Analysis of the Existing Structure

Design Force Level for the Buildings

There are two seismic codes in India that deal with the design and detailing of masonry structures, IS 1893 and IS 4326. The IS 1893 deals with the earthquake design criteria for all types of structures. The 1984 revision (IS 1893[3]) of this code has separate multiplying performance factors (K) for ductile (K=1.0) and non-ductile (K=1.6) reinforced concrete buildings. There was no separate performance factor for masonry buildings. In the absence of such a factor, K=1.6 was typically used for masonry structures. Subsequent to the 2001 earthquake, a revision of one part of IS 1893 was released (IS 1893[4]). The method of calculating seismic coefficient has been changed. A response reduction factor (R) has been introduced. For unreinforced masonry structures, the response reduction factor is 1.5 while that for ordinary concrete structures is 3. Thus, seismic force levels in masonry structures have doubled as compared to the previous IS 1893: 1984. The response reduction factor for reinforced masonry with seismic features such as lintel and plinth bands and corner reinforcement is 3.

The IS 4326:1993 (IS 4326[5]) is a prescriptive code for seismic detailing. The code provides empirical provisions for seismic strengthening features such as lintel and plinth bands and corner reinforcement regardless of building size. It was realized that such prescriptive rules would not be valid for a historical monument such as the Mani Mandir complex. The project brief thus required the more stringent specifications of IS 1893-2002 to be followed. The applied design seismic loads were based on an importance factor of 1.0 as per the design brief and seismic Zone IV as per the seismic zoning map of India. The elastic modulus for stone masonry was taken as 3500 MPa, in view of the Ashlar type (Brookes [5]) masonry. The damping ratio of 0.05 was used for the analysis. Based on the above parameters, the seismic coefficient worked out to 0.20g. The features above the roof were designed for an increased seismic coefficient equal to five times that of the structure.

Modeling

There is no structural connection between the Willingdon Secretariat and Mani Mandir. Hence, they were modeled separately as three dimensional structures using the analysis and design software program 'ETABS' (Wilson [6]). Shell elements were used to model walls to simulate both membrane and plate bending behaviour. The openings in the walls and the slab openings (for the central courtyard as well the numerous internal courtyards) were modeled as per actual locations. Dynamic analysis was carried out on the model. The model was analysed without a rigid diaphragm to obtain the realistic behaviour of the original structure. A rigid diaphragm was subsequently introduced to study its effects on the structure. Based on the dynamic analysis of the structure, the fundamental period of the existing structure when

modeled without a rigid diaphragm was found to be 0.49 seconds. With the introduction of rigid diaphragm, the structure had a fundamental period of 0.17 seconds. This compared well with the time period of 0.12s calculated as per empirical formula of IS 1893 (Part1):2002 ($T_a=0.09H/\sqrt{d}$, where T_a is approximate fundamental period, H = Height of building in m., d= base dimension of the building at plinth in m.). The fundamental period of the retrofitted structure with reinforced concrete skin walls was 0.13s.

Verification of the Analysis and Structural Adequacy

The results of the computer model showed that the existing structure was adequate in almost all places for gravity loads but was highly inadequate to take the code specified lateral seismic forces. In some areas, the overstress ratio was in the range of 5. The maximum deflection at roof for the original structure without diaphragm was 79 mm, for the original structure with diaphragm was 8.6 mm and maximum deflection for the retrofitted structure at roof was 2.8 mm

Limitations in Analysis

It was recognized that there were approximations in the modeling of the materials and the structure. In reality, the walls were not made of a homogenous material and the effect of the almost non-existence of mortar in walls was not accounted for in the analysis. The modulus of elasticity considered was an approximation based on subject literature and was not established separately for this structure. The modeling of walls as finite elements with shell behaviour was also an approximation of the actual behaviour of these elements.

METHODOLOGIES FOR REPAIR, RESTORATION AND RETROFITTING

Elements to be Added/Enhanced for Improved Seismic Behaviour of Structure

Introducing Rigid Diaphragm Action of Slab

For improved seismic performance, a rigid diaphragm was required to be put in place at both the floor levels. The diaphragm action was proposed to be developed by means of introducing diagonal bracing elements on the underside of the first floor (Figures 7 and 8). Such a diagonal bracing would ensure that there was a well-defined path for transfer of the lateral inertia forces from the slab to the walls in their stiffer direction. The detailing of the diagonal bracing was complicated due to the presence of the cornice element. As a result bracing in the form of a complete truss (in plan) spanning between the cross walls was proposed. The brace elements were connected to the steel joists for lateral support. A different strategy was adopted for the roof slab as explained later.

Enhancing Strength of the Structure

From the failure pattern, it was quite apparent that the stone masonry walls with their poor bonding were unable to provide the adequate lateral strength. It is not possible to improve the existing mortar. Hence an alternative means of improving seismic performance was required. Various options were considered. These were: introducing vertical diagonal bracing, introducing new shear walls, strengthening existing walls (by shotcreting), and providing base isolation.

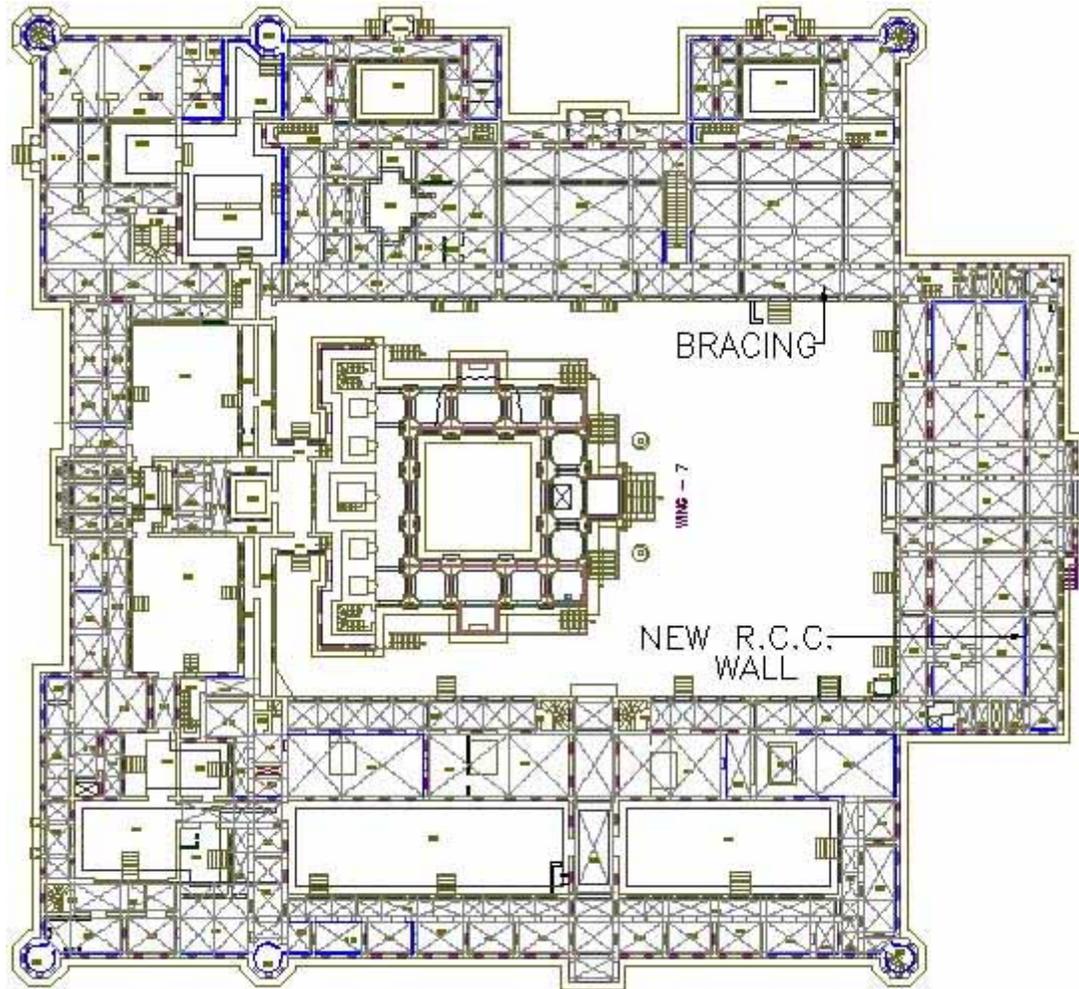


Fig. 7: Location of Proposed Bracings and R.C. Skin Walls at First Floor Level.

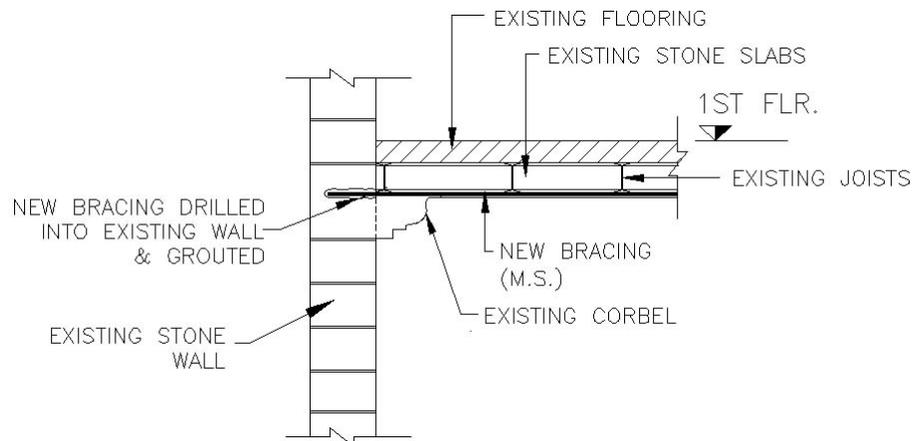


Fig. 8: Sectional Details of Anchoring of Diagonal Floor Bracing in Walls.

The option of diagonal bracing was an attractive one as it meant the least intervention into the existing structure and could be designed as a sleek, honest and aesthetically pleasing retrofit. However due to the existing wall foundations, it was not possible to provide foundations for the large axial tension and compression forces generated at the base of the braces. Hence this option was discarded. Typically,

shotcreting in India is limited to a thickness of 40 mm primarily due to equipment limitations. The option of base isolation was considered to be a very expensive one and is not available indigenously; hence this option was not considered. Thus, the process of elimination led to the choice of additional reinforced concrete skin shear walls. (Figures 7 and 9a).

Based on the results of the analysis and the structural behaviour, reinforced concrete skin walls were introduced at select locations. A computer model was generated considering these new skin walls and ignoring the capacity of the masonry walls. The design force levels were reduced based on the response reduction factor for ductile detailed RC shear walls ($R=5$ for ductile shear walls). Initially a large number of thin walls of 100mm were introduced internally adjoining existing walls. Such an intervention would not be obvious as the internal walls were already painted. In the second iteration it was decided to reduce the total length of the RC skin walls and increase their thickness to 150mm. This model was further refined to arrive at the optimal positions and lengths of new RC skin walls of 150mm thickness. Existing foundations were suitably strengthened (Figure 9b).

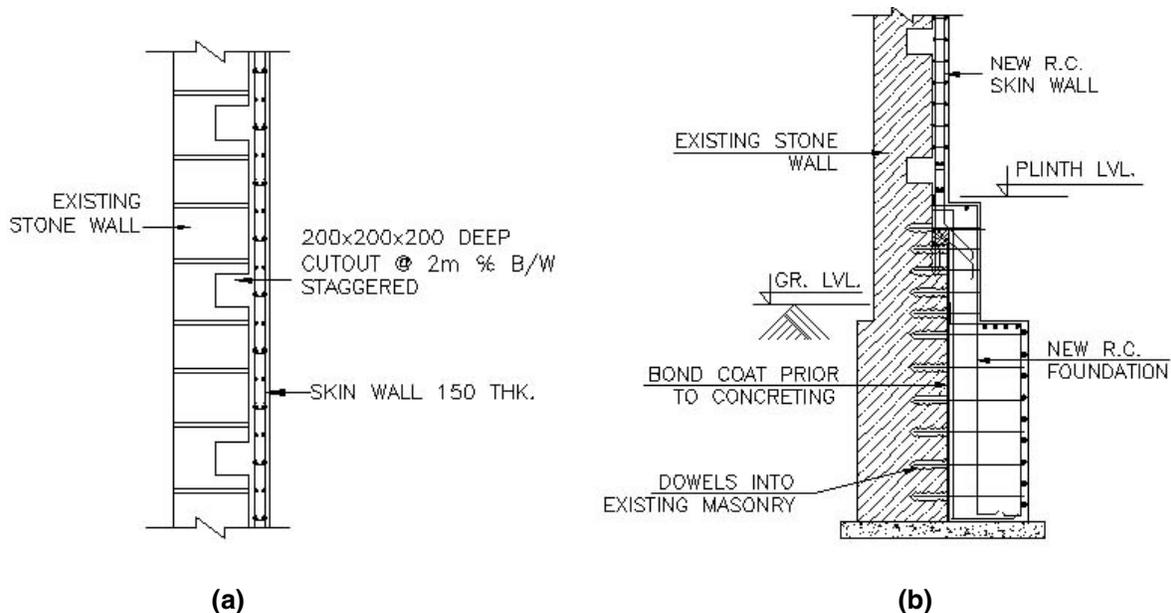


FIG. 9: Details of New R.C. Skin Wall (a) R.C. Skin Wall Nogged in to Masonry Wall, and (b) New R.C. Skin Wall Foundation.

End-pinning of Wall Corners

Many of the external and internal wall corners had opened out during the 2001 Bhuj earthquake, and are liable to do so in a future seismic event due to the poor masonry detail at the corner. Thus, it is proposed to securely connect the perpendicular walls to each other by cross-pinning with stainless steel rods of 8mm and 10mm diameters at every 600 mm along the height (Figure 10a).

Introducing Horizontal Reinforced Bands To Existing Masonry Walls

While the new RC walls have been designed to take the full lateral loads, the original structure (without RC skin walls but with slab diaphragm action) was checked for its capacity to take 25% of the design seismic loads. This was found to be adequate. However, it was necessary to provide seismic features in the form of horizontal stainless steel bands to the existing masonry walls to improve their performance and ductility. Accordingly, 6mm stainless steel reinforcing rods were proposed between the stone courses at a spacing of about 600 mm along the height (Figure 10b).

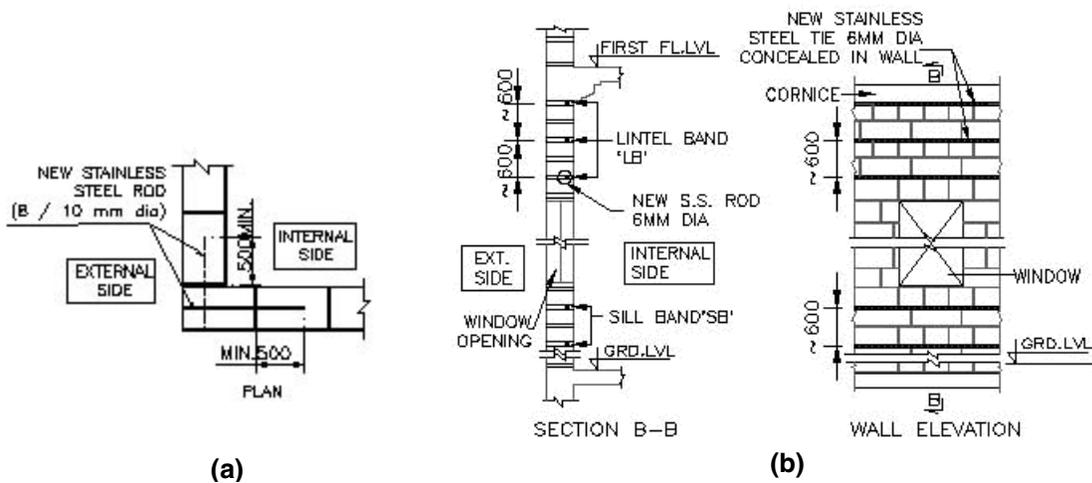


Fig. 10: Intervention in Walls: (a) End Pining at Wall Corners, and (b) Seismic Reinforcement Bands.

Strengthening of Arches

The arches suffered three types of failures: dislocation of keystone, severe cracking of arches and movement of supports. Two retrofit details for arch strengthening have been recommended based on the type of damage. Where there is movement of support and the arches are internal, a stainless steel tie with a turnbuckle is proposed (Figure 11a). In areas where there is no discernible movement of supports, the keystone is tied back into the wall above by means of stainless steel rods (Figure 11b). A lintel is formed by means of providing stainless steel reinforcing bars in the courses between the stones above the arch.

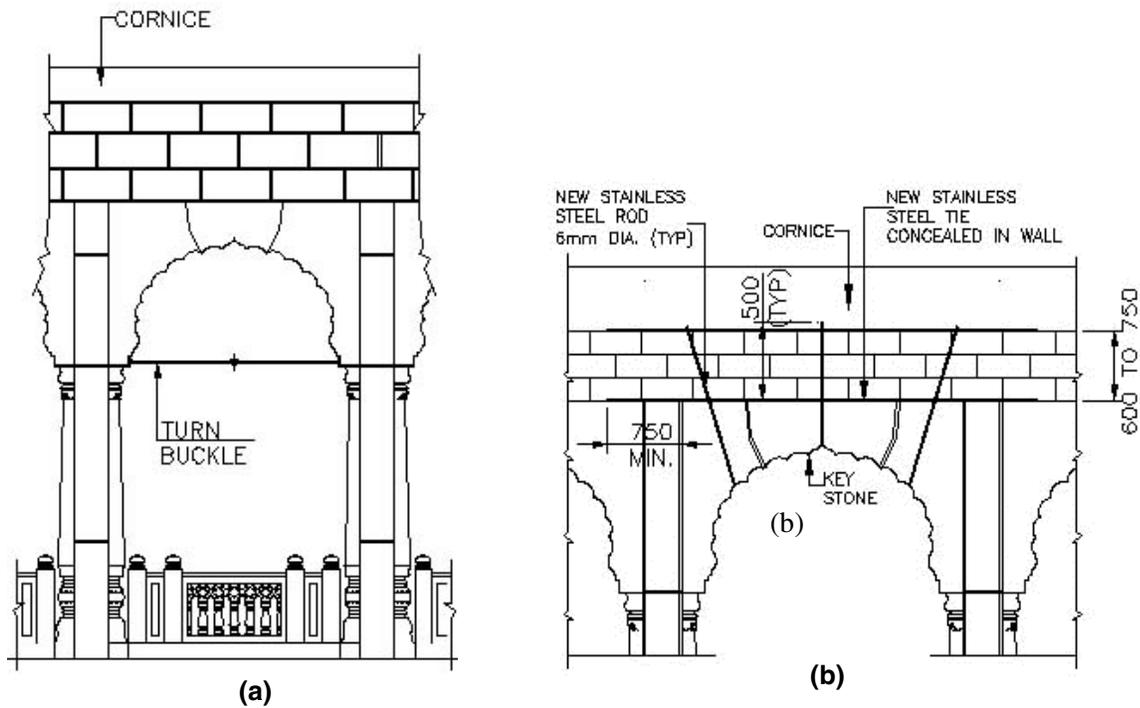


Fig. 11: Detail of Interventions in Arches: (a) Strengthening of Arches Using Ties, and (b) Strengthening Of Arches by Pining And Reinforcement Band.

Cross-Pinning of Corridor Columns

The columns in corridors towards the interior courtyards were significantly damaged during the 2001 Bhuj earthquake. The columns are made of three or more pieces of stone and negligibly socketted into each other. It is proposed to connect these stones to each other by cross-pinning (Figure 12a).

Stitching and Grouting of Cracks in Walls

The cracks in the walls are to be grouted with a low-strength grout compatible with the stone. Walls with diagonal cracks are to be stitched using stainless steel bars before the cracks are grouted (Figure 12b). The grout needs to be tested before use for its compatibility with the stone.

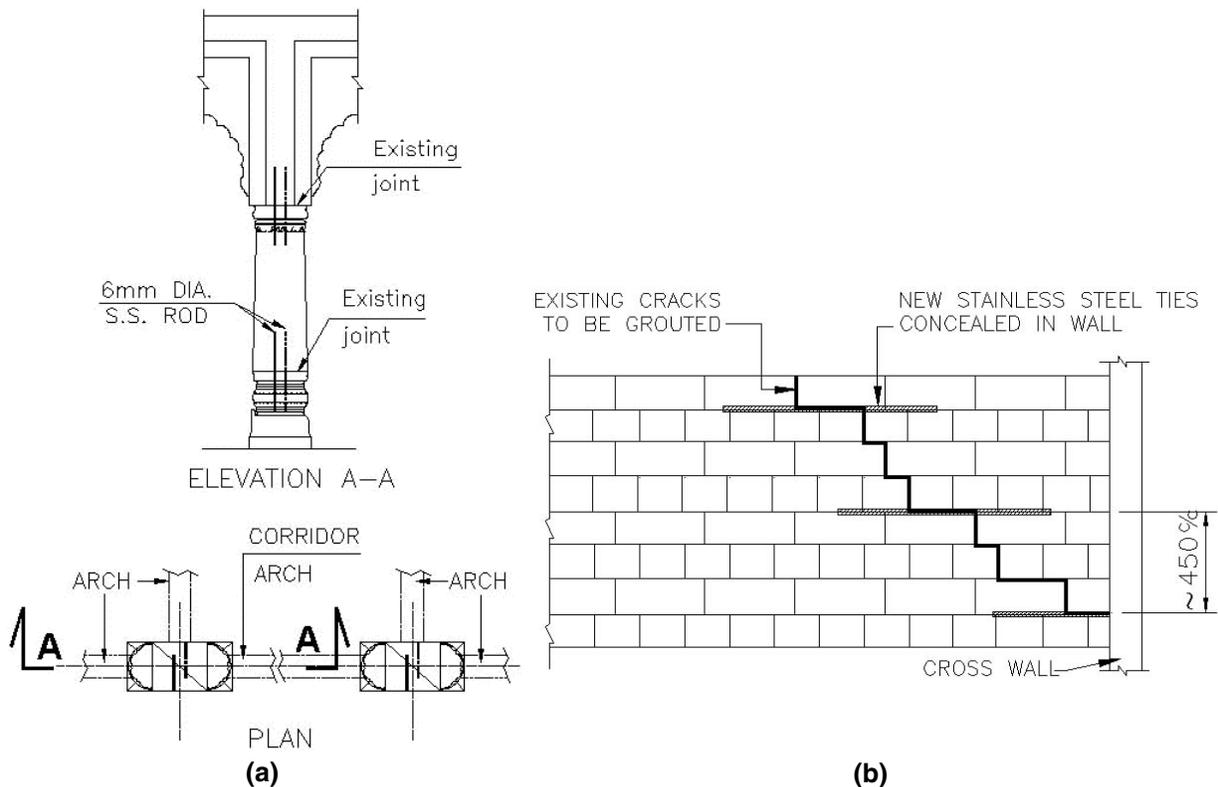


Fig. 12: Interventions in Vertical Elements: (a) Cross Pining of Corridor Columns, And (b) Detail of Stitching Cracks in Walls.

Areas Requiring Demolition and Rebuild:

The roof slab was badly damaged and had partially collapsed; this needed to be replaced. All *chhatris* and most *shikhars* above terrace floor required to be demolished & rebuilt with sufficient seismic detailing. Parapets on terrace floor needed to be removed, marked and re-fixed with an appropriate method or rebuilt where destroyed. The corner bastions and the internal staircases within them had collapsed partially and required complete rebuilding. The following is a description of the details being considered to achieve the above.

Roof Slab

The entire roof slab showed considerable corrosion of steel joists and numerous deep cracks on the roof surface. As mentioned earlier, the waterproofing on roof slab was done in three layers amounting to about total 300mm thick; this additional load had caused sagging in some steel joists and subsequent cracking in the adjacent stone slabs (Figure 2a). Considering that more than 70% of the slab was partially or

completely damaged, it was recommended that the roof slab be completely removed and replaced with a suitable slab system. The replacement of the slab provided two challenges. The proposed new slab should be reminiscent of the original construction methodology as well as provide the necessary diaphragm action. The use of precast slabs in lieu of the stone slabs was considered but rejected as it would not provide the necessary diaphragm action and further costs would be entailed to provide this. Finally reinforced concrete in-situ slab system spanning between composite beams of concrete and epoxy coated steel joists was adopted as a compromise solution which essentially retained the philosophy of the earlier structural system besides appearing similar to the original ceilings.

Bastions

Analysis showed that the corner bastions are very severely stressed; any amount of rebuilding with masonry, even with provision of seismic reinforcement such as horizontal steel bands would be inadequate. A 150 mm thick reinforced concrete (RC) skin wall was introduced internally. Double layer of reinforcement in both directions was provided based on the analysis of the retrofitted model. Cutouts were provided in this wall to match with the bastion windows. The RC wall was bonded to the external masonry bastion walls by noggling the RC wall into the existing masonry wall at intervals (Figures 9 and 13). The staircases in the bastions that had collapsed were to be rebuilt in structural steel and realigned to ensure the presence of a continuous diaphragm to allow for transfer of lateral forces from the floor slab to the RC bastion walls.

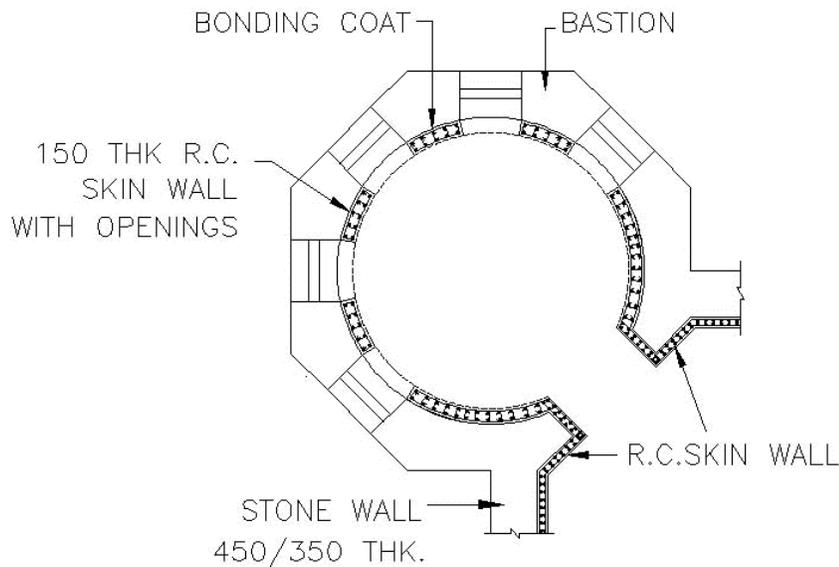


Fig. 13: Plan of Bastion with New R.C. Skin Wall.

Elevation Features above Roof (Chhatris and Shikhars) and Decorative Balconies

The partially collapsed *shikhars*, *chhatris*, decorative balconies and their walls which were pushed out or tilted are in a very precarious state and require emergency measures for their complete dismantling and shoring/strutting. Each stone has to be carefully numbered, removed and stored carefully. Many decorative stone blocks have broken. Identical carvings on similar sandstone are to be carried out by special artisans of the area. As there would be no bonding between the stones pieces, the stones will be connected to each other by means of crosspinning using 8mm stainless steel dowels.

Roof Parapets

The roof (stone) parapets have collapsed and broken partially. The parapets are to be dismantled and numbered. The broken parapets will need to be reconstructed of matching sandstone and carved by local craftsmen. The pilasters of the parapets shall be pinned into the slab by use of epoxy coated rebars and grout.

Weather Sheds

A significant number of stone pieces which constitute weather sheds have broken due to overturning or due to the impact of falling debris. These have to be dismantled and rebuilt similar to the roof parapets. All weather sheds are to be anchored to the supporting corbel and tied back into the supporting wall (Figure 14).

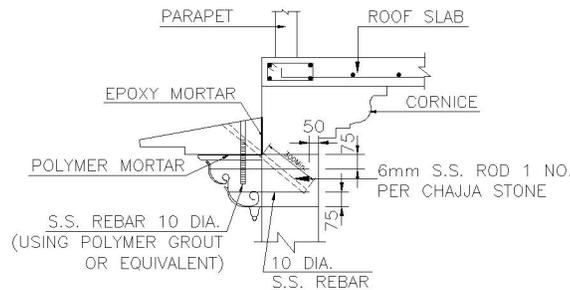


Fig. 14 Pinning Details For Weather Shed.

CONCLUDING REMARKS

The 2001 earthquake exposed the seismic vulnerability of the Willingdon Secretariat and Mani Mandir Complex. This was also established from the dynamic analysis of the structure using finite element modeling. The structure showed several deficiencies such as inadequate lateral strength, lack of a diaphragm action in slab, reentrant corners, poor bonding between stone blocks, vulnerable arches, unreinforced tall elevation features on roof.

A comprehensive retrofit program was formulated to address these deficiencies. Measures such as reinforced concrete skin walls, diagonal bracing on the underside of the floor slabs for diaphragm action, horizontal stainless steel reinforcement bands in existing masonry walls have been proposed to improve lateral strength and behaviour. Measures such as cross-pinning and end pinning have been recommended to improve the seismic behaviour of walls, weather sheds and stone pillars.

The structural retrofit program has tried to limit the intervention to the most minimum. The total area of new reinforced concrete skin walls introduced is less than 10% of the area of the existing masonry walls. The estimated cost of the proposed retrofit worked out to less than Rs. 4300/m² (\$90/m²). This is economical when compared to the structural cost of a standard new reinforced concrete office building (Rs. 3000/m²) or \$63/m²). The cost of constructing a similar ornate masonry building would be many times over; if at all it could be built. The actual execution of the retrofit program will bring out a whole new set of challenges.

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