# SEISMIC ISOLATION RETROFITTING OF THE SALT LAKE CITY AND COUNTY BUILDING

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*The City and County Building, a massive unreinforced masonry structure completed in 1894, has been seismically retrofitted using base isolation. The isolation system consists of 443 lead-rubber isolators installed underneath the building on top of existing spread footings. The building is isolated from the surrounding ground by a perimeter moat wall, permitting lateral movement to take place during an earthquake.*

*It is believed that this is the first historic structure in the world to be retrofitted against possible seismic damage using base isolation.*

## Introduction

Construction on the Salt Lake City and County Building began in 1890, and the building was dedicated and occupied in 1894. The building is a five-story unreinforced-masonry and stone-bearing wall structure with a 76-m-high central clocktower. Plan dimensions are 40 x 80 m. The style of architecture is Richardsonian/Romanesque Revival.

For many years the building was Utah's tallest. Even today, because of its ornamental architectural style and prominent location in the center of a city park, the building is a highly visible landmark (Fig. 1).

## Structural History

At the time the building was designed and built, earthquake engineering was a nonexistent science. The possible susceptibility of the structure to earthquake damage was likely not even considered.

Occupants became aware of the problem in later years when occasional minor earthquakes would produce cracks in walls and over doorways, loosen stonework, produce visible swaying in the clocktower structure, and set decorative statues askew.

The largest earthquake experienced by the building occurred in 1934, which prompted the removal of the four statues over the main building entrances and the statue atop the clocktower. The March 12, 1934, Hansel Valley earthquake had an approximate Richter magnitude of 6.1. Hansel Valley is about 80 miles from Salt Lake. Accelerations felt by the building at its base were likely not greater than 0.05 g. Although not reported in the local paper, folklore claims that the clockworks were shaken loose and crashed down through the tower floors and fifth-level skylight, finally coming to rest on the fourth floor. Miscellaneous wall cracks have continued to appear through the years, with some attributed to settlement, some to earthquakes, but with no documentation or records kept.

Eventually the condition of the building became the subject of official concern, and in the early 1970's, a modest study of the building was funded by the City and County. The structural portion of the study concluded that the building was, for the most part, structurally sound, and would likely meet all requirements of the 1940 Uniform Building Code without modification. However, the building was woefully inadequate seismically, and would require extensive modification to be brought up to code.

The clocktower, because of its height, weight, and configuration, was the most obvious seismic hazard. A study of the tower concluded that a steel space truss within the tower was the best solution, primarily because it would not add significant weight to a building which had already shown some signs of settlement distress.

During the following decade a certain amount of construction work was undertaken as funds were budgeted by the City and County. Small construction packages were bid that allowed the restoration of most of the exterior of the East Entrance, and implementation of the first sections of the planned stabilization of the clocktower. But the continued deterioration of the building and the inescapable effects of inflation eventually forced a reconsideration of this approach to the building's restoration, and the City decided to have a comprehensive study made on the structure to determine whether the building should, or could, be restored and seismically retrofitted, how the space could best be utilized, and the costs involved with the different approaches.

This study strongly recommended that the building be saved, primarily due to its historical significance and unique architecture. However, costs involved to seismicly retrofit the building, using a conventional Uniform Building Code (UBC) approach, appeared to be prohibitively high. To bring the building up to Zone 3 standards, it would be necessary to totally gut and replace the interior structure, and construct a new building within the original exterior ornamental facade.

## Testing and Determination of As-Built Conditions

Field tests of existing materials were conducted to determine the reliable degree of strength of the as-built structure. The tests included in-place shear, compression prism, core compression, stone compression, and footing-concrete compression tests. Significant findings were that the sand-lime mortar used in the masonry, and the footing concrete, were of generally poor quality, stone used for foundations was of a very high quality, and the laying of brick between wythes through the various wall thicknesses was generally quite good.

Original architectural plans were available for preliminary studies and the development of working drawings. Although inaccurate in many respects, these drawings provided a good starting point in determining many of the as-built conditions.

## Structural Analysis and Design

The first challenge in this preliminary phase was to refine the structural study and explore ways of reducing the anticipated cost of seismic retrofit.

Preliminary analyses were done to arrive at seismic retrofit schemes using the UBC, Base Isolation (BI), and the ABK Method, a relatively new methodology being tried in California for seismic hazard mitigation of existing unreinforced masonry buildings. Common to all three schemes were the following:

1. A steel space truss within the clocktower to stabilize it and transfer seismic forces down into the main building. Conservatively, it was decided to use results of the conventional, nonisolated analysis for sizing of tower steel members and tower wall anchorage for all three solutions because of the significance of the tower as a seismic hazard.
2. A structural plywood diaphragm in the fifth-floor attic spaces to stabilize the top of exterior masonry walls.
3. Lightweight reinforced-concrete topping over portions of existing floor diaphragms to increase their stiffness and strength.
4. Anchorage of exterior masonry walls to floor and attic diaphragms.
5. Plywood shear walls within attic spaces to laterally stabilize the main building roof structure.
6. Anchorage of seismic hazards, such as chimneys, statues, gables, dormers, gargoyles, and balustrades, around the exterior of the building.

7. Diaphragm anchorage to interior masonry walls, for shear transfer and to provide a tension tie through the walls.

The original base isolation scheme consisted of ~500 isolators, to be installed on new footings a couple of meters below the bottom of existing footings. Directly above the isolators, and below the existing footings, would be a series of reinforced concrete beams to carry the weight of the structure between isolators. These beams would all be rigidly interconnected by struts to tie the system together.

Preliminary calculations indicated a dramatic reduction in force levels. Shotcreting of existing walls would not be necessary, and existing floor diaphragms would require only minimal strengthening around their perimeters. It would be necessary, however, to entirely remove the first floor to provide working space for the foundation work.

In essence, the BI solution shifted the focus of the structural work from the shear walls above to the foundation, reducing much of the seismic retrofit work to a massive underpinning project.

Preliminary cost estimates indicated the ABK Method as the most economical solution, although it was questioned whether the method could be extrapolated to a building of this monumental size. In terms of damage control and disruption in the historic fabric of the building, the ABK solution was felt to be inferior to the BI solution. Although the BI solution exceeded the ABK solution in cost by over $1 million, based on rough schematic estimates, it was still considerably less expensive and architecturally disruptive than the UBC solution, which would require total demolition and reconstruction of the interior of the building. Calculations also indicated that the base-isolated building stayed well within the elastic range (0.08 g, vs 0.55 g for a nonisolated building), so that predicted damage would be minimized for the design earthquake.

These findings were presented to the Mayor and City Council, with a recommendation to proceed with base isolation. It was decided to conduct a detailed analytical study, which would include dynamic time-history analyses on a detailed building computer model.

Base Isolation Concept

Although no seismograph records for major earthquakes in Salt Lake exist, earthquake records in other areas, with geology similar to Salt Lake, indicated that the City and County Building might be subjected to amplified force levels as high as 0.55 g. Resisting these forces using such approaches as the UBC and ABK methodologies would require significant construction throughout the building. This work would have great impact on existing architectural finishes, such as moldings, wainscots, door frames, and floor tile. This conventional type of seismic retrofit work would primarily serve to prevent building collapse, but noncatastrophic damage would still be extensive due to seismic energy being absorbed through inelastic deformation of building components.

Base isolators are very stiff in the vertical direction to transfer gravity loads, but are flexible in the horizontal direction, thus isolating the building from the horizontal components of seismic forces. Base isolation would shift the fundamental period of vibration of the structure to a range outside the predominant energy content of the design earthquake. In simple terms, the structure would tend to vibrate at a different frequency than the ground below it, thus avoiding resonance and significantly reducing the level of force experienced by the building. (The concept works in much the same way as shock absorbers do in an automobile, isolating the car and its occupants from road vibrations.)

A typical bearing used on this project is ~43 cm square by 38 cm tall (Fig. 2). It consists of a sandwich of alternating steel and rubber layers with a lead core. The purpose of the lead core is twofold: it acts as a fuse, preventing building motion due to wind loads, but yielding and providing inelastic viscous damping to absorb energy during a seismic event. This damping also helps to control horizontal deflections. The steel plates, which are bonded to the rubber, prevent the rubber from spreading outward under vertical loading, thus making the bearings very stiff in the vertical direction.

For the bearings to work properly, it is necessary to isolate the perimeter of the building from the ground horizontally so that the building could translate relative to the ground during an earthquake. To do this a retaining wall would need to be constructed around the building's exterior with a 30-cm seismic gap. Computer runs indicated that the maximum deflection the building would experience relative to the ground during the design earthquake was about 12 cm. The additional clearance was provided as a factor of safety. A bumper restraint system was also installed in the moat to act as a backup safety device.

## Base Isolation Design

Shortly after approval of the base-isolation schematics, exploratory trenches dug around the foundation revealed significant variance between the original drawings and the as-built conditions. It was discovered that footings beneath the building were roughly 50% wider and thicker than shown on the original plans, with a massive concrete mat, 23 m square x 1.4 m thick, underneath the four main tower piers. The original base-isolation scheme of placing isolators below existing foundations had to be abandoned.

The structural design team revised the scheme so isolators would be installed on top of the existing footings. However, because of isolator installation clearances, it became necessary to raise the new first floor 36 cm.

Although a significant amount of tunneling and undermining work was eliminated by the decision to install the isolators above the existing footings, it was now necessary to cut hundreds of slots through existing walls above the footings to install the isolators. Also, a considerable amount of notching into existing walls for side beams was necessary. Tests had indicated that the stonework at the base of existing walls was extremely hard (1500 kg/cm$^2$ compressive strength), and it would be difficult to cut into it or through it. Also of concern were the types of tools that would be permitted for foundation work. The sand-lime mortar appeared to be of extremely poor quality at many locations, and it was felt that an impact hammer of any significant size would vibrate much of the mortar loose. Because of this concern, it was specified in the contract documents that only nonimpact methods of stone removal would be permitted.

The final design, on which construction documents were based and construction has proceeded, consisted of 443 bearings (Fig. 3) placed on top of the original spread footings, with a new concrete structural system built above the bearings to distribute loads to the isolators. This new structure contains the following elements:

1. New concrete side-beams are poured on each side of all masonry walls. The walls are notched in 10 cm on each side to receive these beams. Post-tensioning rods are drilled through the walls and tightened to clinch the masonry material between the new beams (Fig. 4).
2. At isolator locations, all wall material is removed to accommodate the bearings themselves and a new concrete cross beam is poured over the top of the isolator and connected to the two side beams. The cross beam acts as a double cantilever in transferring the wall load from the side beams onto the isolator.
3. Below the isolators, several small steel beams are welded together to form a grillage to distribute vertical loads from the isolator onto the existing footing (Fig. 2).
4. A new concrete first-floor diaphragm is built, linking all of the isolators so they will act together as a system.
5. After the four steps above are completed, the mortar joints directly below the new side beams are removed, thus transferring the building weight onto the isolators and completing the isolation process.

All isolators were made the same size to cut down on fabrication costs and simplify installation details. The different types of isolators were reduced to two, those with lead plugs and those without. At one time, all isolators had lead plugs, but computer analyses had indicated unacceptably high tower shear for certain earthquake records. The isolators with lead plugs, approximately half of the total, were located around the perimeter of the building to cut down on torsional response.

## Construction

Prior to bidding the final contract, several different methods were tried by prospective contractors in an effort to determine the most efficient means of stone removal using nonimpact tools. Two methods thought to be promising were line drilling, that is drilling a series of holes close enough to each other to form a single cut, and high-pressure water jet cutting. Both methods worked, but neither proved to be cost effective.

As this job was truly unique, with no real precedent, it was difficult to obtain a reliable cost estimate. Experts in stone and concrete coring and cutting were consulted and shown the schemes to get a general idea of the cost, as well as the stone removal methods likely to be employed. Fortunately, as things turned out, the local bidding climate was very favorable and this portion of the work was well within the budget.

The method chosen by the low bidder, and used for a large portion of the stone cutting, was a wire saw. The saw, manufactured in Spain, consists of a 1.27-cm diam diamond-embedded steel wire, guided by a series of pulleys that can be adapted to each cutting situation. A control panel and device mounted on a track maintains tension on the wire. The wire is kept cool with a steady stream of water. Large diamond blade rotary saws, up to 1.22-m diam, were also used to make the vertical cuts. Overcutting at corners could not be permitted, so this method left a considerable amount of partially cut stone within the holes that had to be broken out later. Both line drilling and wire-saw cutting were used to make the horizontal cuts across the top of the slots.

Every effort was made, in preparing the bid documents, to have the contractor account for minor variations in existing footing elevations in his bid. A difference of ±7.62 cm from the average elevation shown on the drawings was to be accounted for by adjusting grout thickness, cross-beam depth, etc. Since the floor was not to be removed prior to bidding, the average footing elevation was determined by studying the existing drawings, and checking actual footing elevations where possible in a very limited number of locations. Preliminary data seemed to indicate that the 7.62-cm tolerance would be adequate. It has been recommended that, perhaps, the first floor should be removed as a separate contract so that existing footing and plinth conditions could be verified, and information thus obtained could be incorporated in the bid documents. This suggestion was denied as it would delay the overall project completion date.

Upon removal of the first floor, however, it became apparent that almost the entire south half of the building foundation was up to 15.24 cm higher than shown on the bid documents. The north half, where footing-elevation measurements had been taken previously, was almost entirely at the assumed elevation. Another problem not anticipated was that the stone used for plinths varied greatly in thickness, both along their length, and from side to side. Uniform stone thicknesses and widths had been shown on the original drawings, and verified in only a few accessible isolated locations prior to bidding and subsequent floor removal.

As luck would have it, the contractor began work on the south end of the foundation, where almost all of the problems were encountered. To stay ahead of the contractor, and answer the myriad of questions generated by varying foundation conditions, it became necessary for the writer to stay full time at the job site for about six weeks. An additional structural engineer was assigned to the project for four months to help solve the steady stream of problems created by unforeseen existing conditions.

Also at the site were two engineers employed by the City and assigned to the project full time. This greatly eased the burden on the project engineer, as these people were on site to spot potential structural problems as they turned up, and to alert the project engineer. A full-time field engineer is a must on any job of this size and complexity.

Another major concern was the possibility of building settlement and potential cracking at time of mortar joint removal. The isolators were preloaded to take up any slack in the isolator assemblage before cutting the mortar joints between isolators. The isolator vertical stiffness is considerable (535,000 kg/cm). It was decided to preload to 2/3 of the estimated dead load, rather than the entire dead load; this would lower the risk of cracking the walls above, which might occur if too much preload were applied.

Preloading was accomplished using Freyssinet flat jacks in a shim space provided for this purpose below each isolator. The particular flat jack used is 41-cm diam x 3.18 cm thick, with a rated load of 172,000 kg. Two workmen could easily preload three isolators in an hour. The procedure used is as follows:

1. Place flat jack with circular shims in space between isolator and spreader beam. The shims are fabricated to fit in the two dished areas located in the center of each side of the flat jack.
2. The flat jack comes with two valve stems. Epoxy is pumped into one of the stems until all air is bled off through the second stem, at which time it is locked off. As pressure increases, the central dished out area of the flat jack expands as it fills with epoxy and presses against the two nested shims.
3. The hydraulic jack for pumping the epoxy is a double-acting type, where the hydraulic fluid, gauge, and assorted hoses are kept separate from the epoxy, thus allowing the hydraulic jack to be reused. A solvent is used to

clean the fittings and the top cylinder which comes in contact with the epoxy. When the gauge reading indicates the desired preload, the flat-jack valve stem is locked off.

4. The remaining space between the two 61-cm square plates not occupied by the flat jack is shimmed tight with steel shim plates, and the four corner bolts are tightened down.
5. Once the epoxy has cured, temporary safety screw jacks are removed and used elsewhere on the job.

Installing individual isolators under the eight cast-iron columns posed special problems. A special column clamping device was designed and fabricated to grip the column in friction as well as bolt double-shear. The clamping device, in turn, bolted to the needle beams fabricated from channel sections which were used to jack against in order to pick up the load. An additional collar was installed below the clamping collar to support the new first-floor beams. These beams were installed first to provide lateral bracing for the column during the underpinning procedure. Once it was verified by surveying instruments that the load was picked up, the bottom of the column and cast-iron baseplate were cut off, removed, and replaced with a base isolator. After isolator preloading, the needle beams, jacks, and clamping device could be removed and reused on the next column. This method worked well for all but one column, which cracked and had to be repaired.

## Construction Staging

Of critical importance was the staging of mortar joint removal once all isolators were in place and preloaded. It would be unwise to allow the contractor to proceed at will with mortar joint removal, since parts of the building might be cut loose while other parts are still rigidly attached to the ground, making the building susceptible to even small tremors. The contractor had to meet the following requirements:

1. The new first-floor diaphragm had to be substantially in place before any mortar joints were cut.
2. All mortar joints must be removed in as short a time as possible.
3. A specific joint cutting sequence must be followed so that one part of the building does not become isolated for an extended period of time while another part is still tied down.

It must be realized that the building is in greater seismic danger during the whole isolator installation process, as significant portions of wall must be removed to install the isolators. An earthquake of any significant size during isolator installation could be catastrophic to the building.

In regions seismically more active than Utah, the added degree of risk during the installation process would need to be studied carefully, as the potential for a significant earthquake during the isolator installation time window would be much greater. Perhaps isolator locking mechanisms could be employed during isolator installation in areas of high seismicity where the degree of risk during construction is judged to be unacceptably high.

**Conclusions**

The experience of designing and constructing a seismic isolation system for the Salt Lake City and County Building has indicated the feasibility of this method for retrofitting existing buildings. It is an approach that offers the possibility of preserving as much of the original architectural fabric as possible, and, at the same time, providing a greater degree of protection from nonstructural damage than conventional strengthening.

But not all existing buildings are equal candidates for base isolation. To be economically retrofitted with such a system, a building must substantially meet the following criteria:

1. The building's shape must be suitable. The plans and elevations must be reasonably regular. The structure's height should be less than its width so uplift is not a major problem. Short, heavy buildings are more suitable than high-rises, as their nonisolated period is in the range most likely to benefit from base isolation.
2. The site must allow the building to move relative to the ground without interference from adjacent structures.
3. The building should have interiors worth preserving. Alternative strengthening techniques require demolition of many interior surfaces and alteration of spaces.
4. The anticipated difficulty or expense of repairing nonstructural damage in an unisolated condition should be substantial, to justify the additional cost of an isolated structure.

Finally, the difficulty of documentation and construction, and the complexity of engineering an isolated structure, make base isolation a technically challenging solution that should not be undertaken casually. For the foreseeable future it will remain an experimental technique that will require resources beyond those typically found on small restoration projects. But, with time and experience, base isolation is likely to find its place as an accepted procedure that can be considered on its own merits and used where appropriate.

Fig. 1. City and County Building, Salt Lake City, Utah, USA

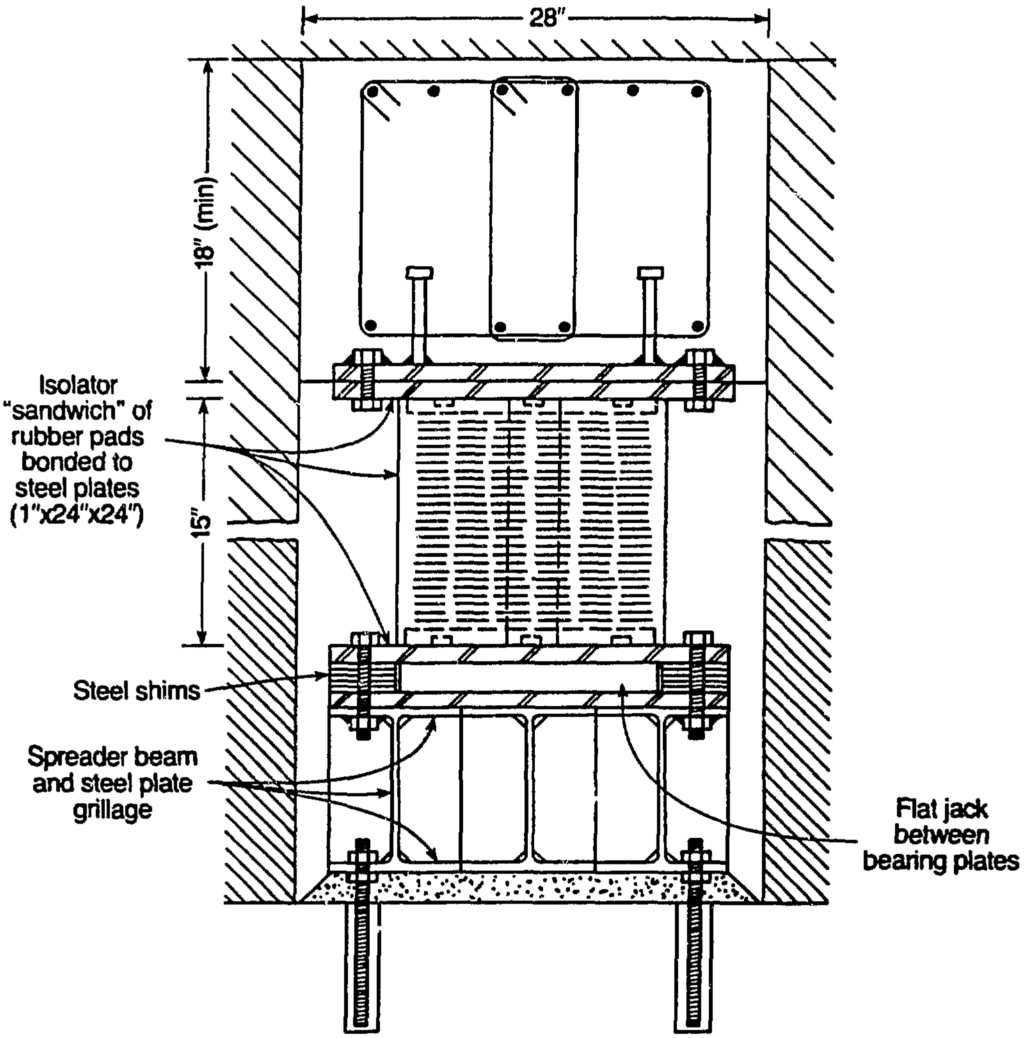


Fig. 2. Typical Installed Base Isolator

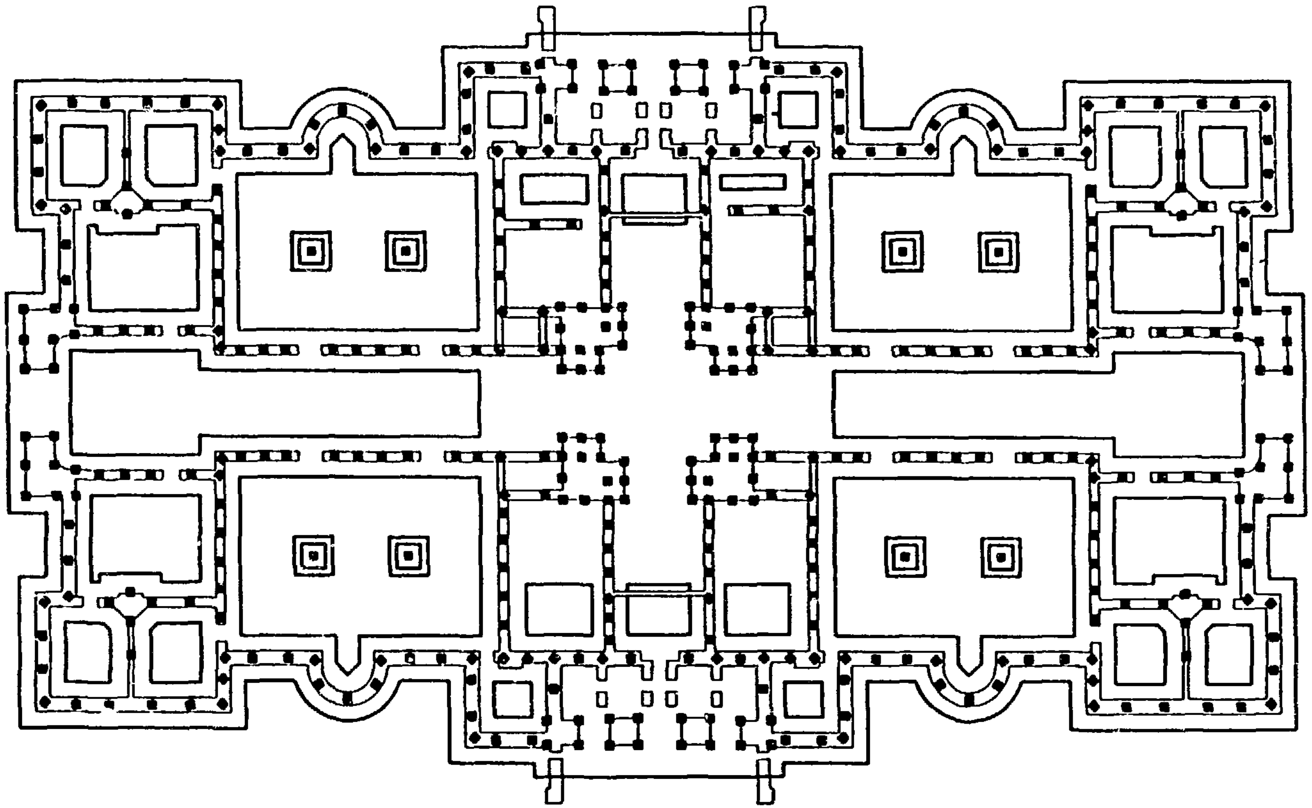

Fig. 3. Plan View Showing Isolator Layout

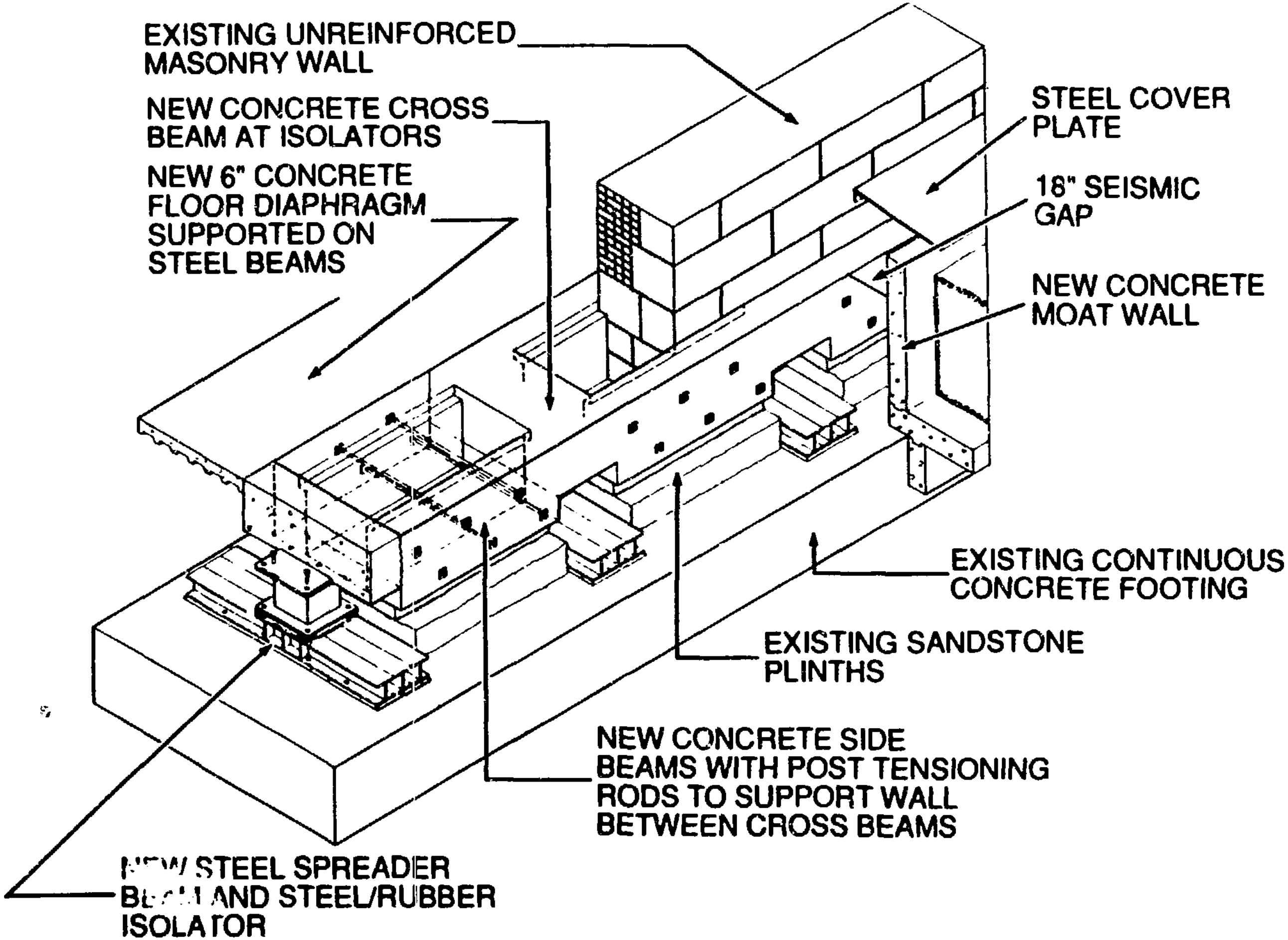


Fig. 4. Isometric Showing how Isolators are Installed at Exterior Walls