# THE INSTALLATION OF A SEISMIC ISOLATION SYSTEM FOR BUILDING RETROFIT


by

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# Abstract

Seismic isolation systems have been extensively studied with regards to their analysis and design, yet in contrast it seems remarkably difficult to obtain any literature describing the construction and physical implementation of these systems. An understanding of construction being so essential to efficient and effective design, it is remarkable that this aspect has been left entirely within the private sector.

This first part, of a two-tier thesis describes construction methodology, specifically as it pertains to the installation of a base isolation system for the purposes of building retrofit. To achieve this, a study of construction practise, as applied in the Western United States, has been detailed along with its express application in a variety of case studies taken both from industry and from a retrofit project, namely *"The Rehabilitation of Mitchell Hall for Seismic Upgrade"*.

This particular case study is detailed on a website, which forms the second part of this thesis, from the investigation through the analysis, design and finally the construction phase of the intervention. This website may be accessed at both infra.mit.edu and moment.mit.edu.

Thesis Supervisor: Jerome J. Connor
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## Acknowledgements

There are several people without whom this thesis, and more so this entire year would not have been possible. I would like to thank each and every one of them for the effort and time they individually put into helping me to further my education.

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In addition, I wish to thank Lisa Grebner for her continued participation in the M.Eng. H.P.S. Programme. She bought a practical and real aspect to the course, that is not achievable in the classroom alone.

I particularly want to thank the entire M.Eng. Class of 2001. The midnight posse deserves a special mention for all those late nights involved in finishing this thesis. Cheers guys! To the M.Eng. Rehabilitation Group, Leonardo, Tai and Noelle, we finished the report in the end.

Finally, the most deserving of thanks are my parents, both for getting me here and through my entire education. None of this would have ever been possible without their support and encouragement.

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# *Chapter 1* – Introduction

## 1.1 Introduction

Seismic isolation is a technology that has only in recent times become accepted as a viable means of earthquake damage mitigation, and even this is only realistically so in Japan and the Western United States. There are many barriers to the implementation of seismic isolation as a retrofit strategy, such as stringent code requirements imposed upon design and the necessary systems testing, as required in the Uniform Building Codes (UBC-97), involved with every project during and after completion. High construction costs are a serious barrier to entry for many clients, and thus the supply has remained small keeping these artificially high implementation and specialised equipment costs. In addition, the more traditional methods of seismic retrofit are hard won over due to the long standing conviction of "the stiffer, the better". The concept of inserting an artificially weak layer in to a structure goes largely against the grain, but resorting to the most one of the most basic equations in dynamics, it is clear that it makes perfect sense.

$$\varpi = \sqrt{\frac{k}{m}} \qquad \ldots(1.1)$$

The term $\varpi$ represents the fundamental circular frequency, k the stiffness and m the mass of the structure. Thus it is evident that as k increases, so too does the natural frequency of vibration, moving it closer toward what is often the high frequency range containing the majority of the energy, in terms of accelerations, of an earthquake.

## 1.2 General Overview

With the spate of earthquakes that have recently struck all around the world, e.g. Taiwan 1999 ($M_w$ 7.6), Seattle 2001 ($M_w$ 6.8), India 2001 ($M_w$ 7.4), El Salvador 2001 ($M_w$ 7.6), Turkey 1999 ($M_w$ 7.4) and Greece 1999 ($M_w$ 5.9), seismic retrofit and rehabilitation are being recognized as not only a valuable tool, but an essential instrument in maintaining today's infrastructure. A number of varying design codes constitute regulations for buildings in seismically sensitive areas, the common factor amongst them being the preservation of life. Few of these address the conservation of a building itself, resulting in design strategies that often give little thought to operability during and after seismic activity (e.g. hospitals, motion sensitive fabrication facilities, essential services such as police, fire, rescue etc.). For instance many reinforced concrete buildings rely on plasticity at the joints as an energy dissipation technique, rendering the structure effectively useless after an event. In today's fiscally oriented and resource driven environment, the loss of a building through lack of foresight, is simply not an acceptable alternative.

## 1.3 Mission Statement

The purpose of this thesis is to document and suggest improvements for current construction practise with regards to the implementation of base isolation as a retrofit strategy. Literature and information on this topic is scarce, and generally resides only within the heads of those practising engineers, actively involved within this specific area of the retrofit construction industry. With this in mind, several have been contacted and interviewed with their input and insights analysed and evaluated as they fit into the development of the construction industry as a high technology field, where constant innovation and value engineering will make isolation available to the widest possible variety of projects, and not simply the elite.

Thus the main goal of this project is to make this information more widely available, thus opening the field and removing the "specialist only" idea from the engineering population, both in design and construction, encouraging more to become involved in promoting isolation as a viable and effective, and most of all cost-efficient option for building retrofit. If the design code requirements can be demystified, a large step will have been taken in this direction.

For the purposes of this thesis it is assumed that the reader is reasonably familiar with the concept of Base Isolation, and it is therefore described only minimally. For further reading on this topic, please see the Reference Section.

## 1.4 Why the High Cost of Implementing Isolation as a Retrofit Strategy?

There are several reasons for the expense involved when using isolation to retrofit a structure. These are perhaps most easily explained using an Industry Analysis as suggested by Porter, and described diagrammatically below in Figure 1.

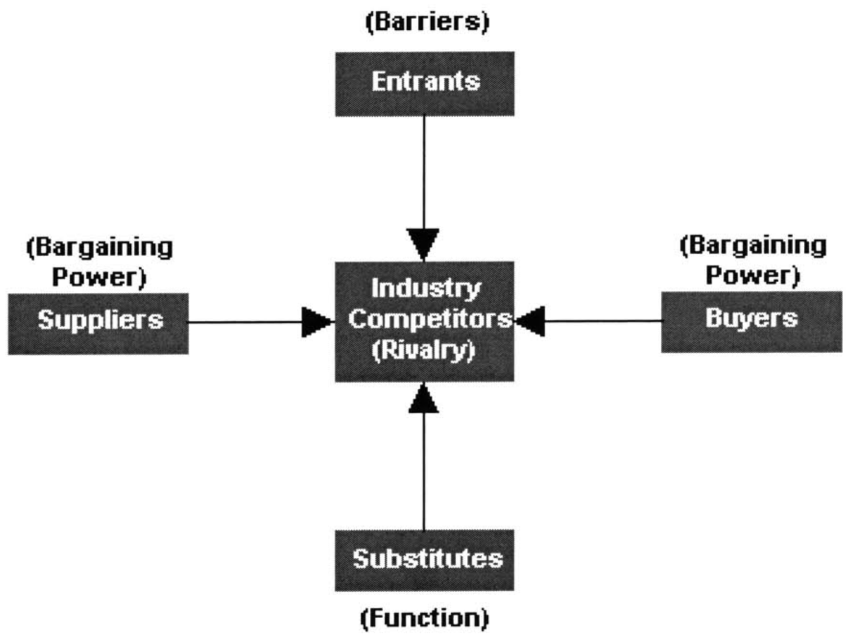



**Figure 1** – Industry Analysis according to Porter.

Porter recognises five forces that affect and drive competitive strategy within an industry, each of which is listed below. The inverse relationship between profit margins and intensity of competition is key to understanding the concept behind these forces: "*as the intensity of competition goes up, margins and returns are driven down.*"

1. The threat of entry by new competitors.
2. The intensity of rivalry among existing competitors.
3. Pressure from substitute products.
4. The bargaining power of buyers.
5. The bargaining power of suppliers.

Unfortunately the market for base isolation is relatively closed, due to both stringent design codes and the high cost of necessary equipment made by sole source manufacturers, thus creating a vicious circle and keeping prices artificially high. Analysing the market as suggested above does not provide a strong outlook for this technology to become more widespread in the near future.

Entrants:

There are few potential opportunities to enter into this small market place, and thus no long queue of competitors waiting to do so. The necessary experience designing and installing these units is possessed by relatively few, allowing current competitors to maintain their high prices. Design codes have, until the advent of the International Building Code 2000 (successor to UBC-97), turned what was once straightforward into a complex and over-conservative set of requirements. For instance, as the basic static formula is intended for use in any direction, and isolators are always isotropic, it seems strange that the dynamic analysis necessitates bilateral analyses.

These factors make it difficult for a firm to establish itself firmly within the market, and until there is more encouragement for them to do so, this factor of the industry analysis remains against cheaper costs of implementation.

Suppliers:

The suppliers in this case are viewed as the manufacturers of isolation units. Various systems are available, such as natural rubber bearings (NRB), lead rubber bearings (LRB), friction sliders and friction pendulum systems (FPS). By far the most popular, in the United States, are LRB's. This is largely because they were one of the first available options that made this concept a reality, since their development in New Zealand in the 1980's.

Their supply in the US is dominated by a single company, who can therefore dictate their price in the relative absence of other competition. Other international competitors exist in Japan and Europe, but neither have the luxury of controlling such a large market. This is a second barrier to increased competition and the resulting reduction in prices within the said market place.

In addition, competition is restricted due to in legislation regarding the use of American made products for publicly funded projects, which comprise the majority of the large retrofit projects on the West Coast to date. Such simple regulations, such as the requirement for any internationally made product to be used on a public project to be shipped on a vessel flying the U.S. flag, make it increasingly difficult for U.S. based firms to compete at an international level, and vice versa, providing a further barrier to entry.

Buyers:

The buyers in this case are the clients who wish to consider seismic isolation as an option for their structure. Isolation has in the US thus far been largely confined to more high profile or historic structures, allowing the construction companies to price high, which in turn allows manufacturers to do the same and lastly prevents more clients or buyers considering isolation as a feasible alternative. This creates a vicious circle, as in keeping the market small a barrier to new entrants is erected. A smaller threat of entry by new competitors allows current industry to continue as usual.

In addition, the large expense forces many clients to look at other mitigation alternatives such as stiffening, damping and any other proven traditional solution. These markets are more than ready to take advantage of this, effectively exploiting the isolation market while expanding their own market share. This is evident with the largest current player in the damping market aggressively pursuing this avenue.

Substitutes:

The above issue lead to the question of what are the cheaper available substitutes? These are solutions such as stiffening and damping as means of seismic hazard mitigation. As many are available, and code requirements are less stringent and construction practise more familiar, it is easier for potential competitors to enter these as opposed to the isolation market. This again leads to reduced rivalry amongst existing competitors.

## 1.5 Thesis Overview

A case study will be referred to throughout this thesis, and used to provide examples of the implementation of many of the techniques described herein. Mitchell Hall is shown below in Figure 2. This was a design project carried out as part of the requirements for completion of coursework. The entire analysis and design have been completed, but construction has not taken place.

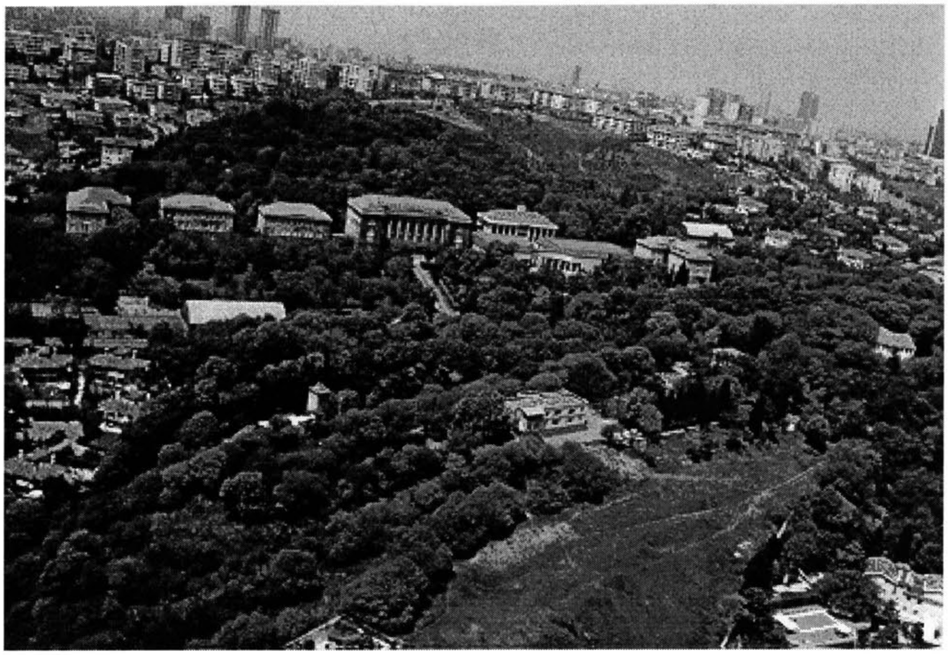


**Figure 2** – Location of Mitchell Hall

**Figure 3** – Elevation of Mitchell Hall.

Mitchell Hall was designed by Shepley Rutan and Coolidge Architects (Boston) and was completed in 1913. The external walls are two feet thick, unreinforced concrete around the entire perimeter. The foundations are simply these walls extended down to bedrock, which lies at approximately 10-ft depth. The internal columns are concrete encased steel (A36) riveted H-sections on concrete pile caps, which again extend to bedrock. Mitchell Hall, a three story building (with a basement), has a 32m by 16m footprint and is 16m in height. For more details about Mitchell Hall, please see the website submitted as part of this thesis.

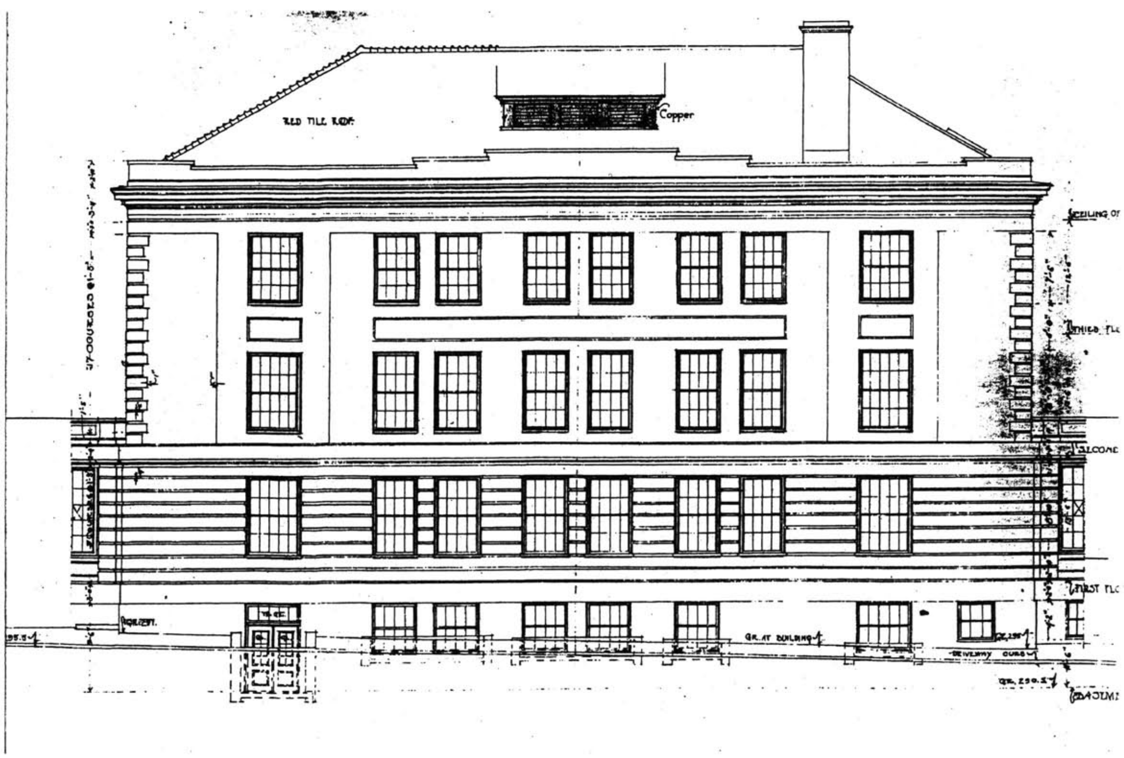


**Figure 4** – Front Elevation of Mitchell Hall.

# *Chapter 2* – Basic Concepts of Isolation

## 2.1 Introduction

The advantages of base isolation have been long recognized within the realms of mechanical engineering, in the use of spring mountings to reduce the transfer of accelerations from vibrating machinery into building floors. In recent times, the benefits of this concept have been realised throughout the civil engineering community as a means of seismic hazard mitigation. Essentially, isolation is implemented to minimise the interaction between two or more objects, that is to say one feels a force or acceleration imparted to the other only minimally. A common example is the shock absorbers on a car. While this is designed to resist impact only, if a car drives over a bumpy road, the acceleration experienced by the wheels is greater than that felt by the car, even though the car may be displaced. Similarly, ground accelerations felt by a building are reduced through the introduction of a soft layer between the ground and superstructure. This directly contradicts what was previously, and still is to a lesser extent, common practice. That is either the stiffening of the superstructure to control imposed motion or the use of dampers to dissipate input energy without damage to the structural system. Figure 5 demonstrates the observed behaviour of fixed-base and isolated buildings respectively.

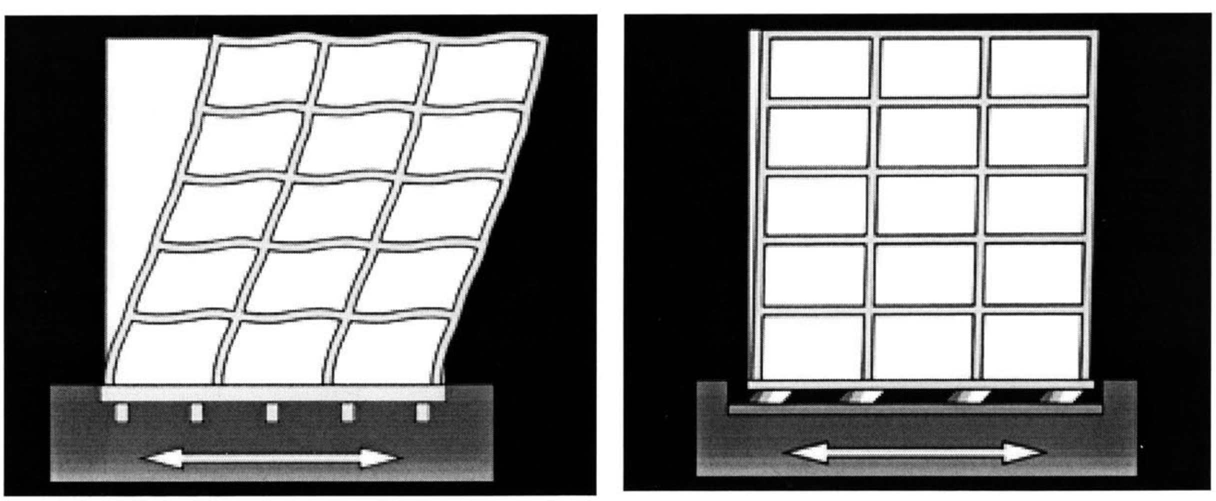


**Figure 5** – Fixed Base vs. Isolated Behaviour

Isolating a building effectively increases the fundamental period of the structure, or reduces the range of input frequencies that can cause resonance thus mitigating the damage potential of an earthquake. Earthquakes in the United States, and particularly along the San Andreas Fault generally have the majority of their energy content at high frequencies corresponding to periods of approximately 0.15 – 0.5 seconds. This phenomenon is shown in the frequency response spectrum below in Figure 6, which is a plot of frequency against its corresponding amplitude for an earthquake recorded near Ismit, August 1999. If the fundamental frequency is for example 10 Hz. (corresponding to T = 0.1s), then all frequencies below this will excite the fundamental mode of vibration. If on the other hand this value can be reduced to 0.4 Hz. (T=2.5s), then effectively the range of frequencies that affect this mode of the structure has been dramatically reduced.

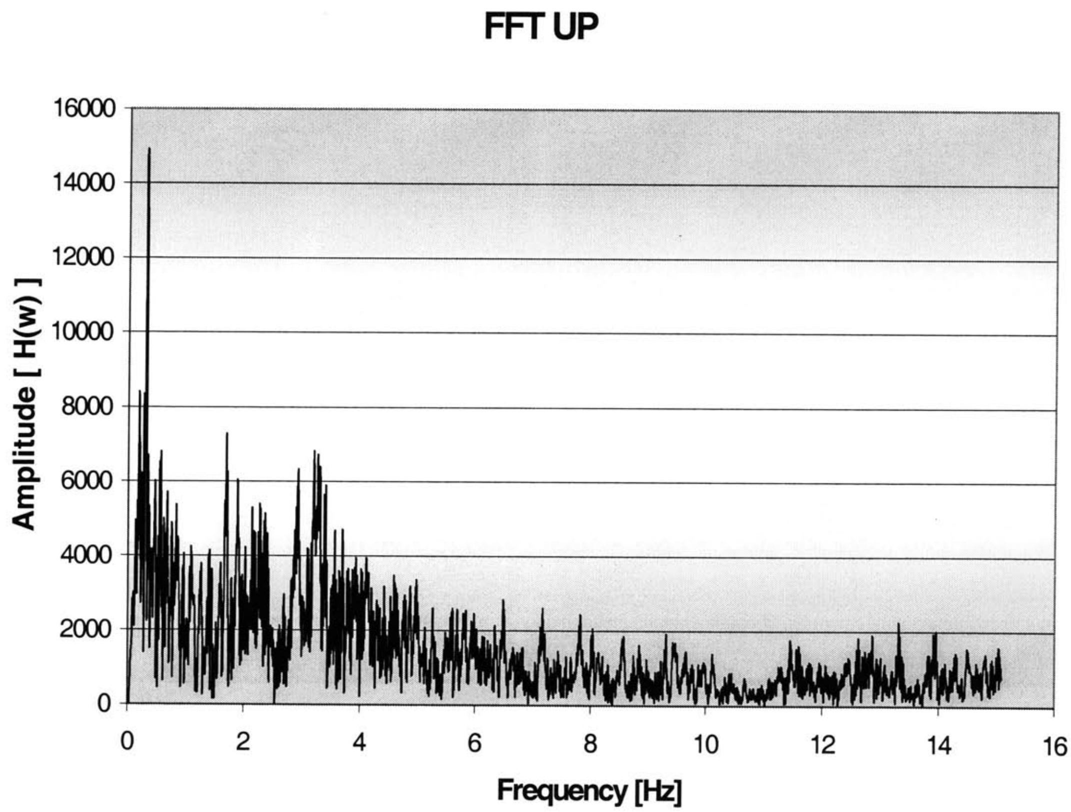



**Figure 6** – Frequency Response (Fourier-Transform) Spectrum

Conversely by stiffening a fixed base structure, the period is reduced and the frequency increased, often moving the building farther inside this damaging range of frequencies. Thus in the theoretical condition of infinite rigidity, there is 100 percent transfer of ground acceleration into the building, or the building moves as if part of the ground with an equal magnitude of acceleration.

# *Chapter 3* - Construction Methodology

## 3.1 Introduction

The construction scheme has been split into several steps, which can help to differentiate between the various milestones and the required labour teams and equipment for any given phase of the retrofit construction sequence. The following six steps have been identified as being relatively important points throughout the process, and will be used as a basis for relating each phase to a certain skill set. The final product should appear as shown in the schematic shown below in Figure 7.

Step 1: Basement Excavation.
Step 2: Moat Excavation.
Step 3: Insertion of Rigid Layers and Jacking.
Step 4: Cutting of External Concrete/Masonry Walls.
Step 5: Placing of Isolators.
Step 6: Separation of building from Base.

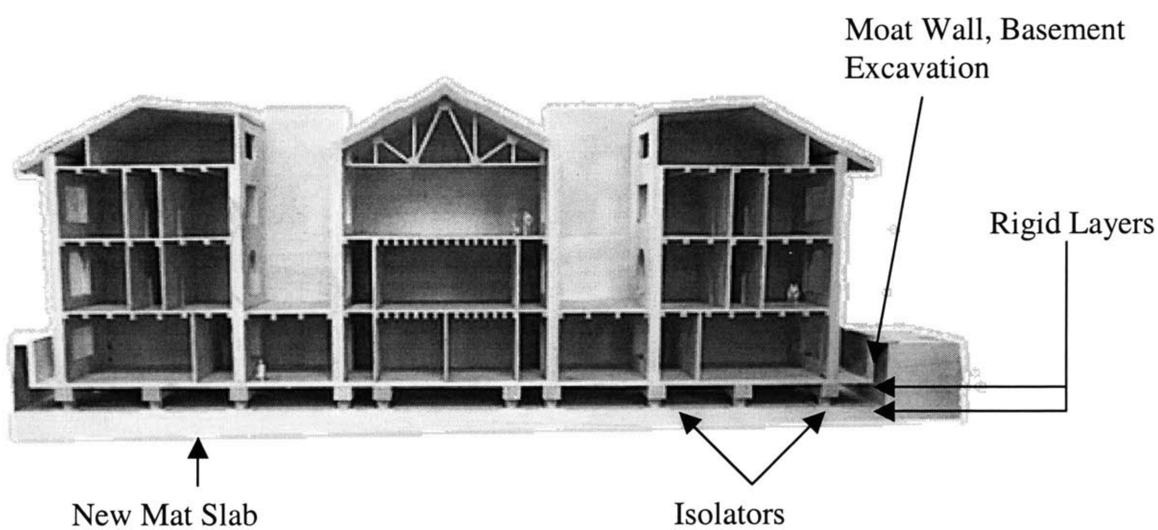



**Figure 7** – Model of an isolated structure

Each of these will be described in turn and related to the case study of Mitchell Hall in terms of how the strategies suggested might actually be implemented.

## 3.2 Step 1: Basement Excavation

Logistics are the most complicated part of excavating a basement in order to gain access to either the foundations of a structure, or remove a sufficient depth of soil to be able to install the isolators themselves. Problems such as site accessibility for excavation and removal equipment, entry into the basement itself and the co-ordination of various teams setting up around the area are probably the most usual kinds of trouble run into during the execution of this stage. Access problems can cause serious time delays to a tight construction schedule if not properly planned for.

The first step is the total demolition of the basement. Total demolition consists of several stages:

- Soft demolition, in which any built in furniture, cabinets and carpets are removed.
- Hazardous material removal and abatement e.g. asbestos.
- Salvage of any historic fabrics or finishes, to be placed in storage.
- Hard demolition in which ceilings, floors, plumbing, electrical and HVAC are stripped out, with the aid of small Bobcats ® if required.
- Removal of Debris from the building.

For large buildings, such as the Los Angeles City Hall, CA, shown below in Figure 8, it is typical to cut a doorway in the exterior wall with a ramp enabling access into the interior of the structure. Large-scale excavators are common, even in the case of historic retrofit, where it is often not financially feasible to use anything else. Occasionally, in smaller buildings, it may not be possible to use this kind of equipment, but rather than dig by hand it is usually viable for a Bobcat © or its equivalent to gain access. For a worst-case scenario, as was encountered in the case study of Mitchell Hall, hand excavation can be effected if easy entrance and exit is provided for.

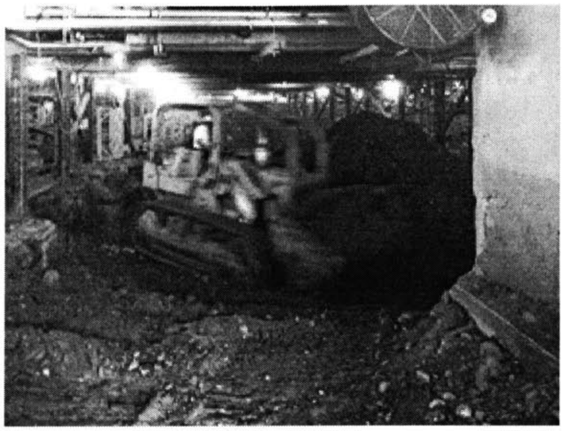


**Figure 8** – Basement Excavation in L.A. City Hall Retrofit

For a building resting upon strip foundations, excavation continues down to until the tops of the footings have been exposed. At the same time the retaining wall, which will form the moat surrounding the building, is poured. This is necessary to prevent an excessive build up of soil pressure external to the walls when much of the internal support has been removed. The moat is dug at a specified distance around the perimeter of the building with a backhoe, and the trench backfilled with bentonite slurry to prevent soil settlements. The retaining wall is then cast in place via one of several methods described below.

A cantilever retaining wall might be used if bedrock is at sufficient depth to allow for sufficient embedment, otherwise a gravity wall or soil tiebacks could be used to provide the required stability under seismic loading. The moat wall must also be of sufficient strength to withstand a minor imposed seismic load through the shoring in case of an event during construction.

The increased column lengths as a result of the basement excavation can be tackled in two ways. Firstly the use of corbels and secondly provision of the rigid layer early in construction. These will be discussed later.

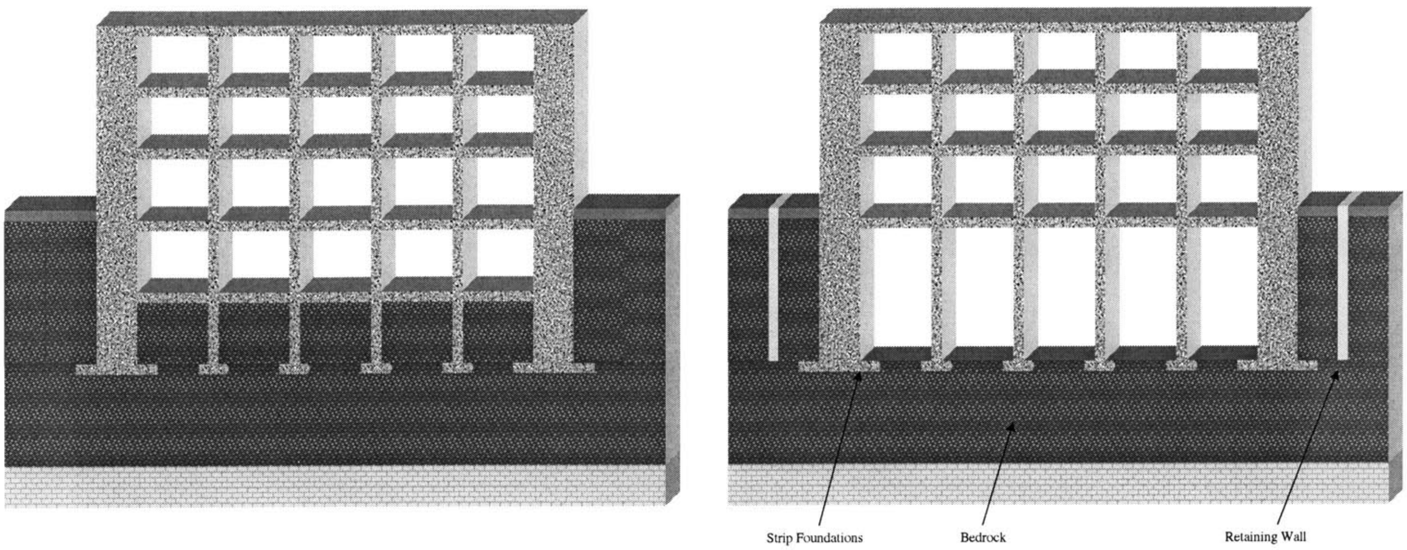



**Figure 9** – Building in its (a) initial state and (b) with basement excavated

For many older buildings, the foundations simply consist of the often-thick external walls simply extending down to bedrock. If this is the case, the basement must be excavated to a sufficient depth to allow the isolators to be installed, a new floor put in place and utilities to be connected. A typical depth to accommodate this scenario might be 6 – 7 ft.

An example of the level to which column footings are typically exposed in the excavation process is shown below in Figure 10.

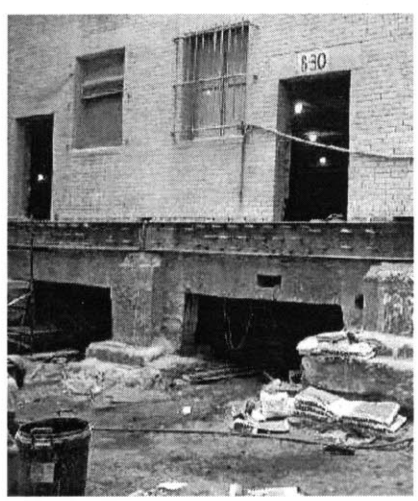
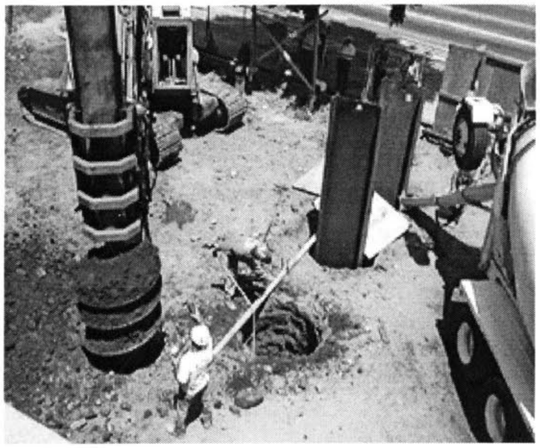


**Figure 10** – Example of exposed footings from L.A. City Hall, CA.

## 3.3 Step 2: Moat Excavation

This phase sees the initial shoring and temporary bracing of the exterior walls against the retaining walls put in place to ensure stability against relative building movements under construction loading. Once the retaining wall is in place, excavation can begin around the perimeter, while at the same time steel bracing is put place to prevent relative movement between any internal and external columns, while the soil around the footing is excavated to facilitate its enlargement. This is often necessary for it to be able to withstand the increased demand of seismic loading. Alternatively, for a smaller structure a mat slab might be poured to supply this resistance in addition to providing the necessary rigid layer below the level of isolation. A mat slab, in the event of an earthquake also has the advantage of having lower surface stress due to the movement of surrounding soil and will as a result settle less in the event of liquefaction. From a construction standpoint it also makes logistics around the interior of the building far easier, but in the case of larger structures is simply not feasible due to the shear volume of concrete necessary. Occasionally, a layer of blinding (lean mix concrete) may be poured along certain areas of a large excavation to facilitate the movement of construction equipment around the site more quickly.

Often, the shoring of the moat begins before any excavation takes place. Again, taking the example of L.A. City Hall, 30 ft deep post-holes were bored, into which steel soldier piles were lowered, before finally being filled with concrete. This sequence is detailed in Figure 11. These were used to support shoring as the excavation took place. This is possible only if the bedrock is at sufficient depth. In the case of Mitchell Hall, bedrock lay at 10 ft depth. A solution similar to that just mentioned was to bore down to rock, and then drill a further 4 ft. Steel I-Sections were grouted in place and these used to support the shoring.



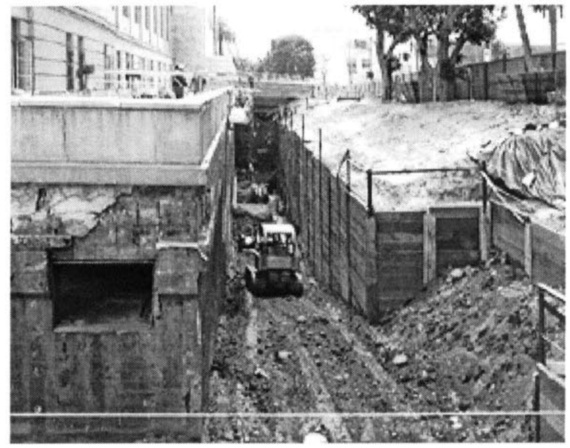


**Figure 11** – Boring of post-hole and resultant shoring as used in L.A. City Hall, CA.

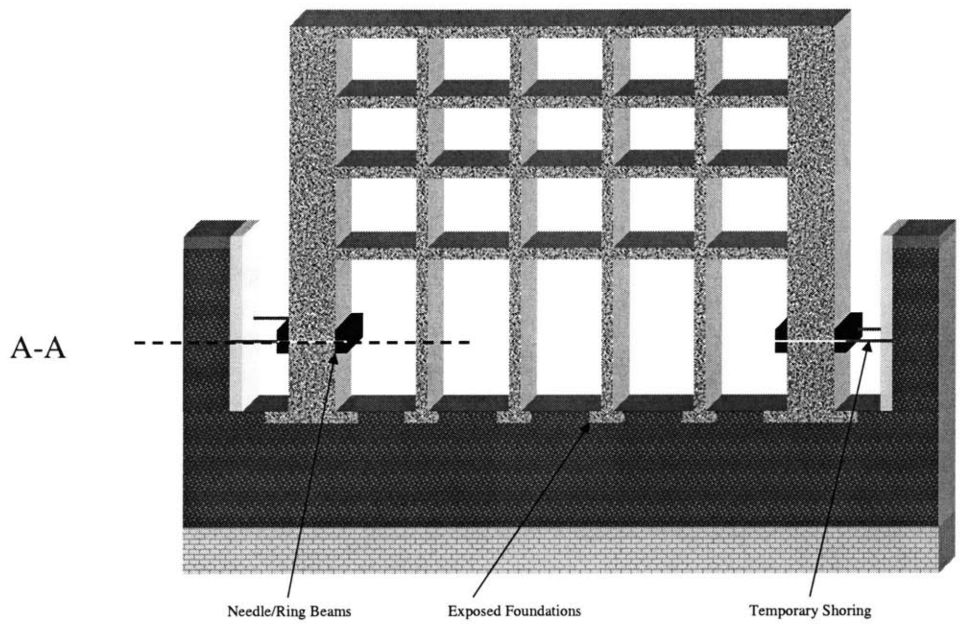



**Figure 12** – Building after moat has been excavated and shoring put in place.

The Ring Beams are in place for several reasons. Primarily they serve to confine the concrete around the area where local stresses will build up due to imposed jacking stresses, and to help maintain building rigidity during construction. They will also help confine the concrete around the area where the connections from the steel isolator mounting plates will be made to the supporting walls. In addition a band will be placed around the base of a concrete column to serve the same purpose (assuming we are not dealing with a concrete encased H-Section). Finally, the ring beams provide for connection detailing, or where the rigid layer above the layer of isolation connects to the exterior walls. At this point it is important to note that in the majority of bas isolation retrofit cases, the building consists of continuous concrete (either reinforced or unreinforced) or masonry walls, while the internal columns may be as described above or traditional reinforced concrete. Typically, the rigidity of the superstructure lens itself well to baseisolation theory of an isolated SDOF structure, but unfortunately, this eas does not carry over into construction.

The connection details of the ring beam are as follows for the case of Mitchell Hall. The exterior wall is core drilled at specified locations, and a steel plate of a particular size is placed either side. A practicality regarding the ring beam is that no two surfaces are perfectly flat. The AISC codes allow for an out of flatness of 1/120 for steel, and for a concrete wall, it is unlikely that it was perfectly formed. Combined these result in a worst-case gap of 1 – 1.5 in. Thus a layer of grout must be poured between the two.

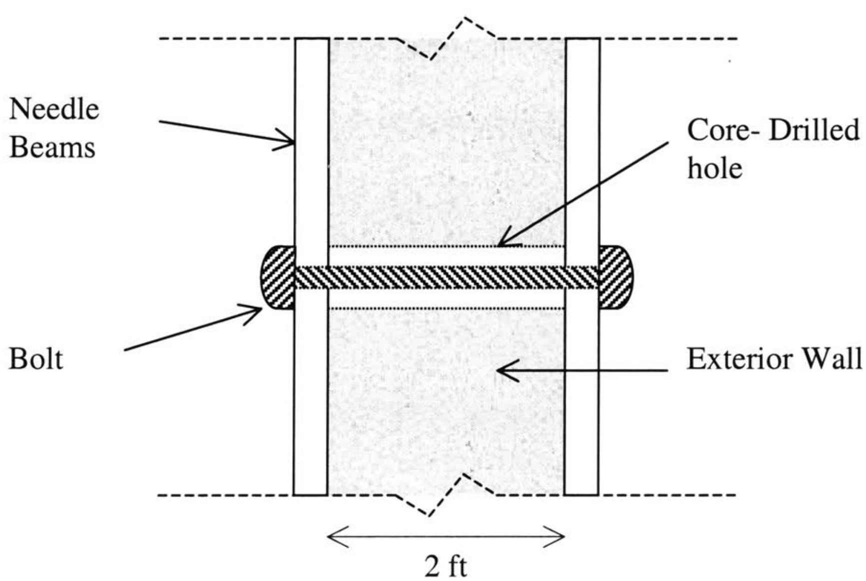
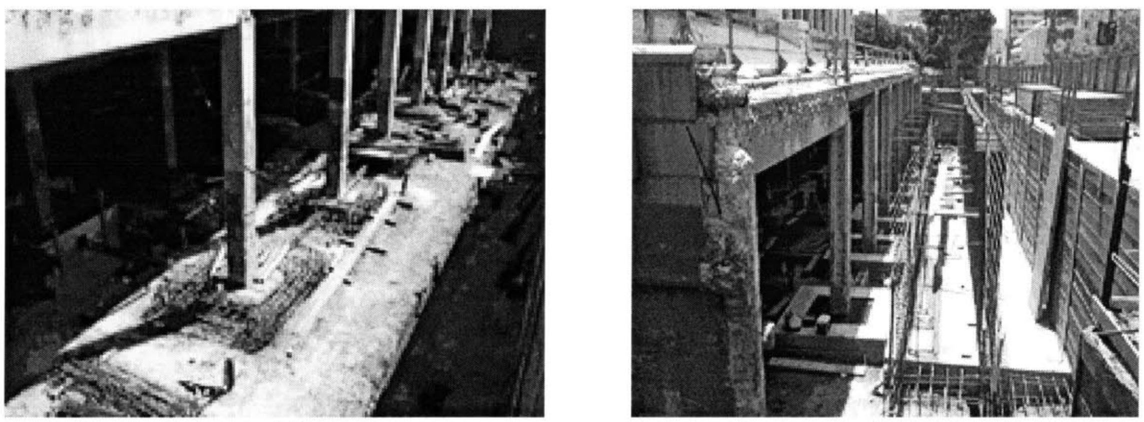



**Figure 13** - Connection detail for ring beam around Mitchell Hall (Section A-A).

The size of the boltholes is dependent upon the bearing pressure they will exert upon the concrete under seismic loading, whereas the size of the bolts will be determined by the necessary tensile capacity (bearing of concrete critical over bearing of bolts).

Thus if necessary, in order to mitigate costs, the bolt holes will be drilled oversized, and the void filled with a high strength grout to distribute the load over a larger area.

The final part of this segment is to drill through the base of the retaining wall and put in place a drainage system around the perimeter of the retaining wall. This will prevent the build up of hydrostatic pressure on the outside of the moat in the event of a high water table or heavy rains, as well as providing drainage for any water that does find its way inside the moat. Once this is complete, the moat slab can be poured, as shown in Figure 14. The vertical rebar in Figure 14 (b) marks where the moat wall will stand.



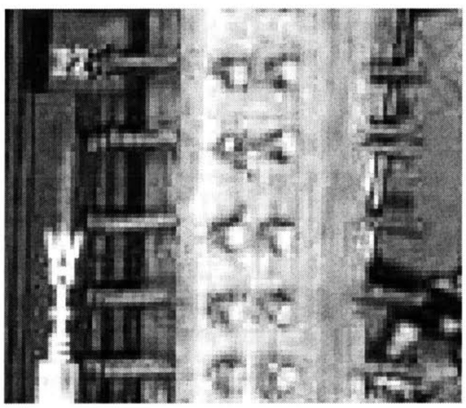
(a) (b)

**Figure 14** – Pouring of Moat Base Slab in L.A. City Hall, CA.

### 3.4 Step 3: Insertion of Rigid Layers and Jacking

Two rigid layers must now be inserted into the building, one above and one below the intended soft isolation layer. There are several options below this level, the two most common being first a mat slab, and second reinforced cast-in-place concrete grade beams spanning the excavation. In addition to providing rigidity and in the common absence of any alternative, these are also required for founding the jacks, that is taking the load to earth, and thus any proposed set-up must take this into consideration.

The problem with either solution is the connection detailing to both the internal columns and the external walls. A possible solution, for the exterior walls, is to core drill around the inner perimeter and grout in steel reinforcing bars, with a specified pull-out resistance. A set length of these could remain exposed, with or without a bob, to provide sufficient anchorage to the slab. Although expensive, this is one of the more effective ways of providing for connection. The internal columns, with regards to steel encased H-Sections, generally have the steel exposed and studs welded in place, around which the concrete is cast in place, to resist induced shear from lateral translation or ground movements.



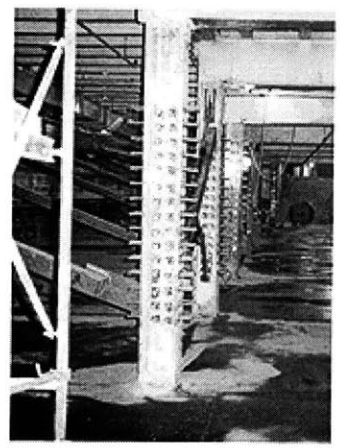


**Figure 15** – Steel Studs Welded onto Columns to improve "Grip".

If a mat slab is chosen, a large mesh of steel will have to be fixed in place, a large volume of concrete vibrated and thus a large amount of labour required. With a mat slab it may also be necessary to drive piles underneath where the mat slab is to be placed, to act as shear keys and prevent relative movement between the slab and the soil in the event of minor liquefaction occurring, as shown in Figure 17. An ideal scenario would be where the soil was of sufficient strength that the expanding of the pile caps and spread footings would be sufficiently rigid. This rarely being the case, grade beams, or formed concrete beams cast in place on grade are an attractive option. These are reinforced dependant upon the base shear imposed upon the structure under the worst load case. This is the approach that was used in the case study, and a typical arrangement might be as shown below in Figure 16. The duplicity facilitates both connection detailing as well as providing a symmetric foundation for jacking.

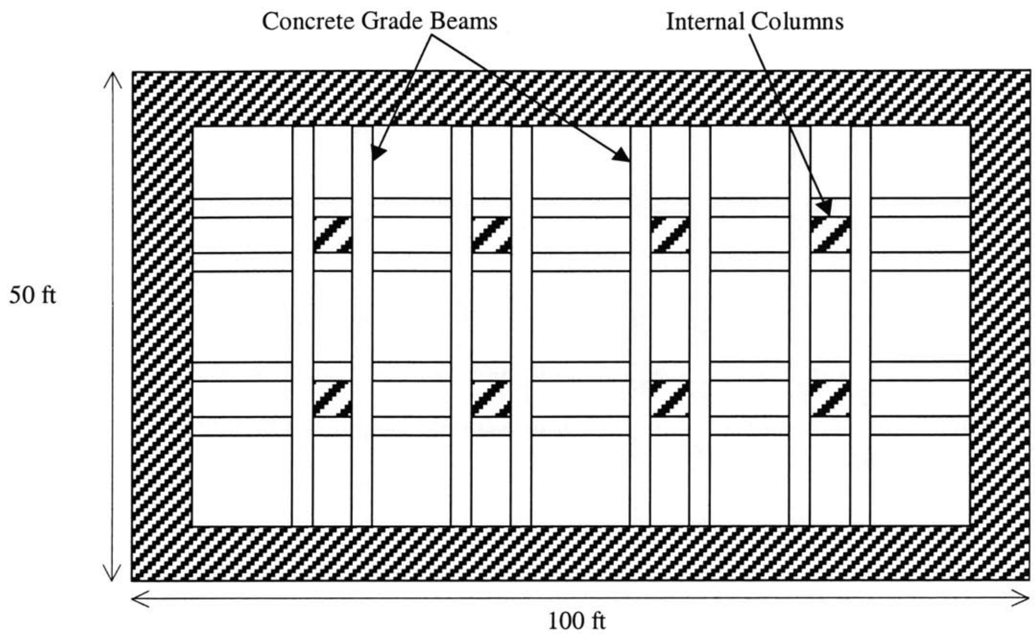



**Figure 16** - Framework for cast in place grade beams for Mitchell Hall

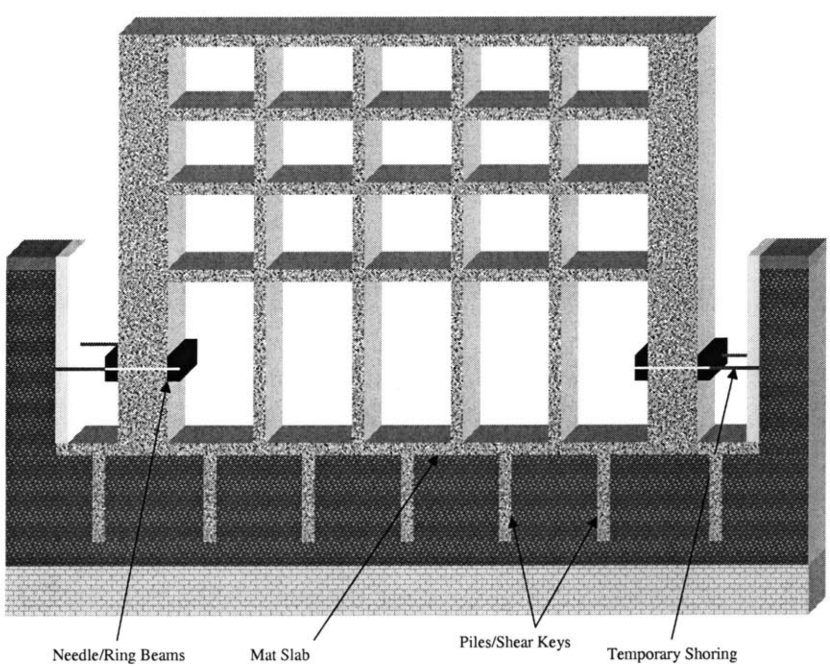



**Figure 17** – Example of a building with a mat slab just poured.

Once the base rigid layer has been placed, a foundation is available to position the slightly heavier upper layer. This layer generally consists of standard steel I-Sections in a grid spanning both lateral directions of the building between the external walls and internal columns. An alternative that is used for large-scale structures is a steel truss fabricated in situ and encased in concrete. Dependant upon seismic demand, and the height vs. the footprint of the building, this section can be in the region of 6 ft by 8 ft, as was the case in the retrofit of San Francisco City Hall. Angles are attached to the top of the isolator mounting plate to facilitate connection. A final method is to simply increase the thickness of the first floor. This layer is shown schematically in Figure 18. This can in turn also be used as the support for the new basement floor once the retrofit project is complete. In the case of the Mitchell Hall case study, the ring beams provide for connection detailing, which for instance might be so simple as a bolted steel angle plates. Connection of these beams to individual columns (either external or internal) can be made by chipping off the surrounding concrete of an encased H-Section and welding or bolting, or by attaching a steel collar to a reinforced concrete column. With reinforced concrete columns in retrofit projects, additional strengthening is often required in order for them to meet the seismic demand placed upon them. This might be achieved through the use of unidirectional composite wraps, shotcrete or any other similar technology.

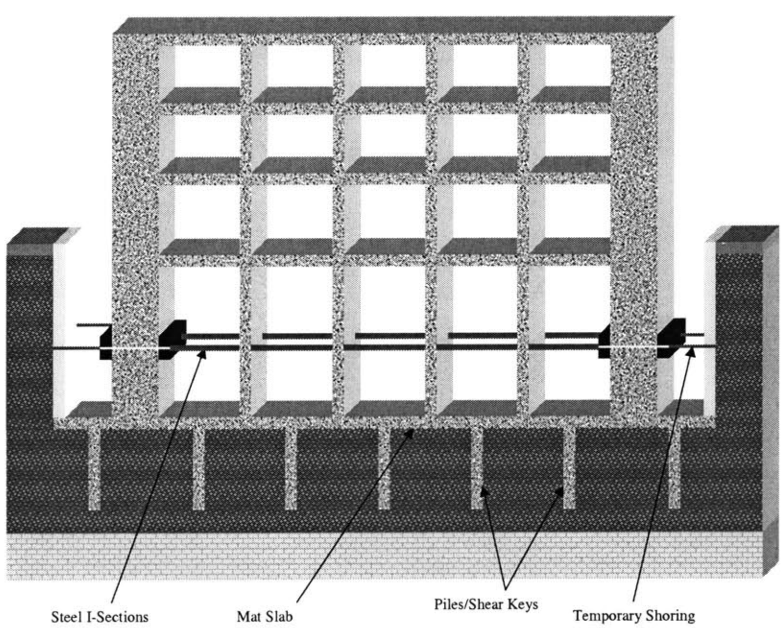



**Figure 18** – Provision of upper rigid layer.

A problem often encountered at this point, is again accessibility to the site. One side of the moat can be expanded, or a bubble cut, beyond what is necessary, and a crane set up to lift and lower equipment and debris over the moat wall. If the building is of sufficient size, an access ramp to a cut out of the wall and supported on shoring may be used to import manageable sections. For a smaller structure, such as Mitchell Hall, these are cut to the same length as a column span (in this case ≅ 6 m).

For a worst-case scenario when installing the upper rigid layer, where access to the building basement is severely restricted, the only option is to resort to the old style methods of building that have been in place since the time of the Egyptians. On occasion where it is not possible to dig out a bubble of sufficient size to get a steel section in place, it is simply set down on the side of the moat wall and tipped over the edge. A small, rounded steel angle is attached to the front, and using block and tackle the section is literally dragged to the desired location using an air-tugger, essentially an air powered winch. Steel rollers might also be used if a solid foundation is available. Once in the desired location, anchor bolts are set in the surrounding walls, and the floor above, or temporary shoring, and the section is chain hoisted into place. This can be to a W36 standard section done with relative ease by an experienced crew. A section can be in the region of 5000 lbs, and taking into consideration the coefficient of friction between steel and sand, this necessitates a horizontal force of about 2500 – 3000 lbs, which is well within the capabilities of these winch systems.

This rigid layer may sometimes also be positioned so as to be in the optimum position with regards to minimising the effective length of a column to be jacked. This reduces the required strengthening in the form of corbels or stiffening plates to facilitate either the friction grip or the welded collar.

As the upper rigid layer is being completed, work can begin on jack placement. Depending upon the size of a project, it is generally required that, in order to maintain stability, no more than six to eight columns be jacked concurrently. Thus the respective number of teams, including concrete, grouting, steel, demolition, jacking and craning, are bought in and work in a specified sequence around the building. Thus as demolition crews finish one segment, the steel team moves in and fixes the collar or friction grip, described below, while the jacking and craning teams prepare for load transfer and unit installation respectively on a prior segment. This process will be covered in more detail in the next section. As one column is completed, the demolition may move to the next, ensuring this condition remains satisfied.

The procedure for placing and loading jacks is as follows. Firstly considering a steel encased H-Section, the concrete team chips off a section of the column exposing the steel. Once exposed lugs are welded in place to form a collar by which beams running North South can be used to transfer the column load through the jacks to earth. For a reinforced concrete column, or an external masonry wall, a friction grip in the form of a steel collar with a thick wood lining tightened around the section and then used to transfer the load to earth. In practice some slippage is noted (often more than 1", as noted in the Martin Luther King Jr. Centre in CA) and this is simply compensated for by restoring the jack pressure.

Generally, for continuous concrete wall jacking, as was the problem presented in the Mitchell Hall case study, the following procedure is used. At the locations where the isolators themselves are to be placed, two to four holes are core drilled along the top of where the mounting plate will rest, shown in Figure 19.

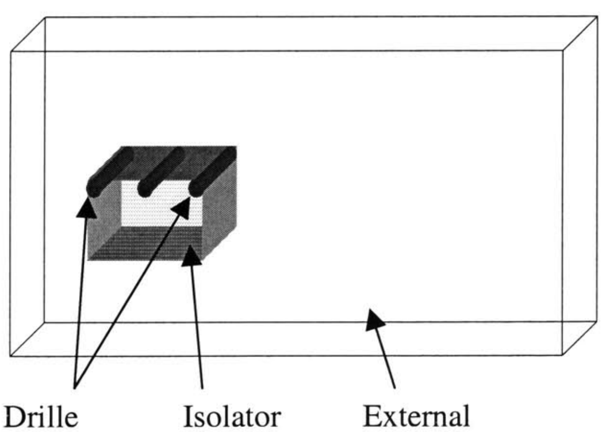



**Figure 19** – Location of Drilled Holes.

Rebars are placed these holes, which are in turn used to support the steel beams that transfer the load through the jacks to earth. Steel I sections span the rebars and rest upon four hydraulic jacks placed at the corners and resting upon either the mat slab or grade beams. This is shown in Figure 20, along with a schematic of what portion of the load is actually taken by the jacks in a continuous wall. Evidently, there will be some spreading of the actual induced stresses, but the figure below is simply intended to describe the rationale for the design pressure to be taken by the jacks.

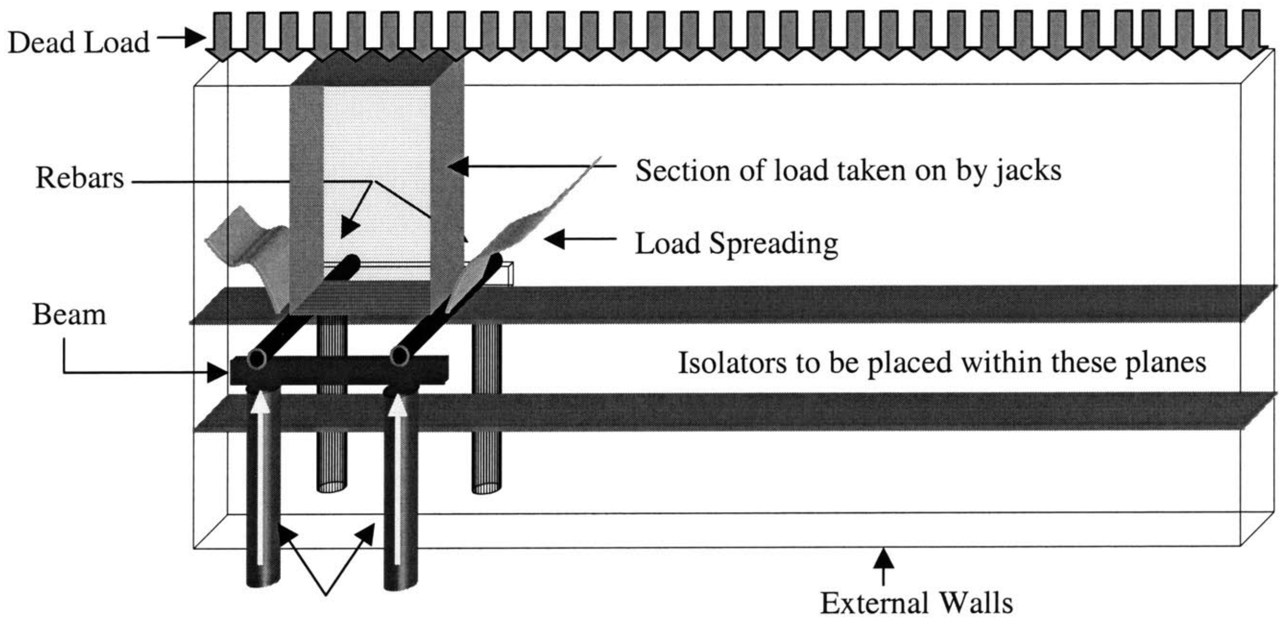



**Figure 20** – Schematic of Hydraulic Jack Placement.

When jacking a column, the load transfer is very straightforward. Hydraulic locking collar jacks are shimmed up until they carry the load predicted by the design engineer to be taken by the column, but restricted to a maximum displacement of about 1/8 in. This is a safeguard against imposing a relative displacement along the floor being jacked and any resultant damage such as cracking. In practise, this limit could realistically be about 1 in. for a 30 ft. span beam (L/360 deflection), as design engineers allow for this deflection of beams under service loading. Soil settlements can be a problem at this stage if not over consolidated, but this effect can be mitigated by constantly monitoring the jack pressure. For large-scale projects, such as the retrofit of San Francisco City Hall, these are often computer controlled and monitored.

Jacking columns subject to excessively large loads can lead to additional problems, such as those encountered in the retrofit of Oakland City Hall, CA, in which some of the central columns were carrying dead loads in excess of 4,000,000 lbs. To lift the entire dead load, it was necessary to attach heavy steel corbels (vertical steel plates) to the bases of the riveted steel columns. A noteworthy problem was encountered with the weldability of the riveted steel sections, forming the column, that is described by William Honeck, Senior Principal and Mason Walters, Senior Associate of Forell/Elsesser Engineers Inc., CA. The original steel fabricator had coated the faying surfaces of the column components with a tar like substance before riveting them together. The tar was heated when the corbel was fillet welded to the section, resulting in what was an unacceptable porosity in the weld. The problem was solved through the introduction of a small stringer pass at the seam and by using a less heat intensive welding process. Such unforeseen problems are frequently met during a retrofit job. The isolation bearing assembly from the Oakland City Hall job is shown in Figure 21.

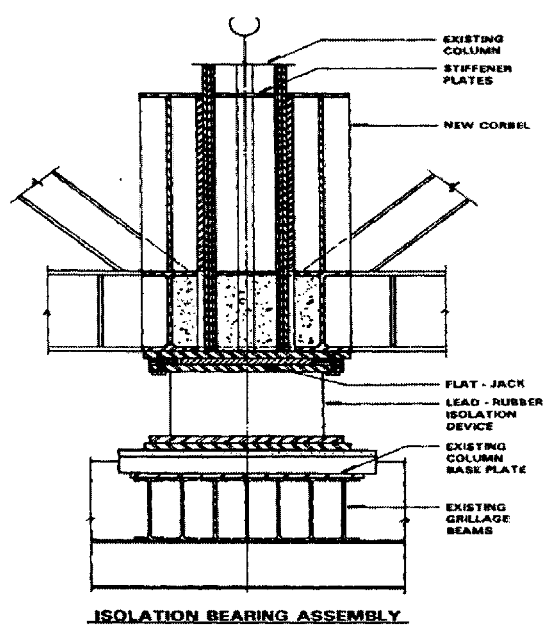



**Figure 21** – Isolation Bearing Assembly from Retrofit of Oakland City Hall

A second problem encountered in this project was a tearing of some of the old column flanges. As this had not quite reached the first row of flange rivets, it could be solved by back gouging and rewelding the affected sections and simultaneously reducing the shrinkage stresses by resequencing the weld placement.

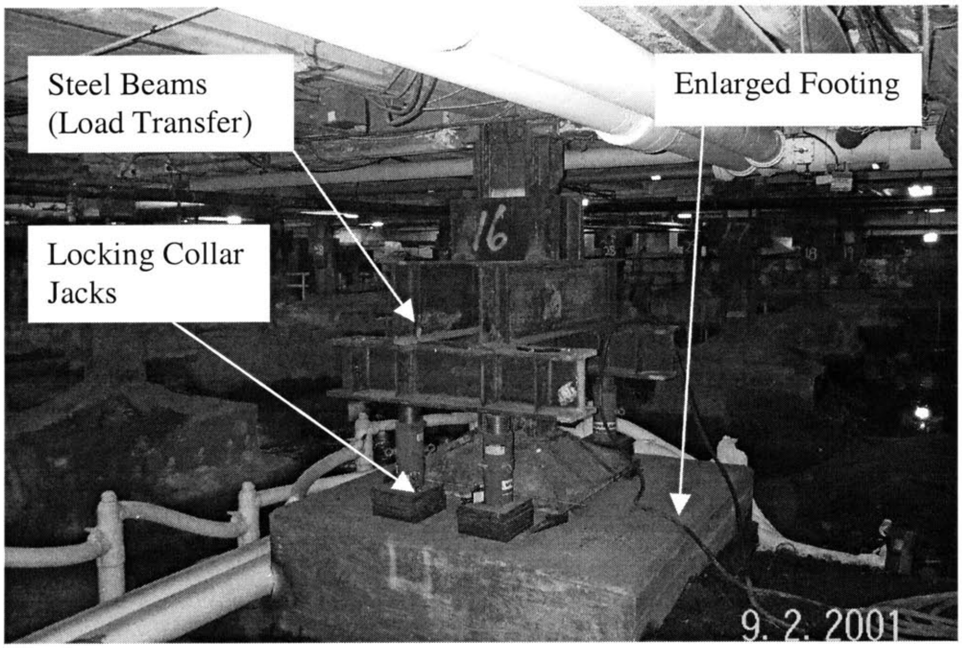



**Figure 22** – Jacking of a column at Russel Wharf by Modern Continental, MA.

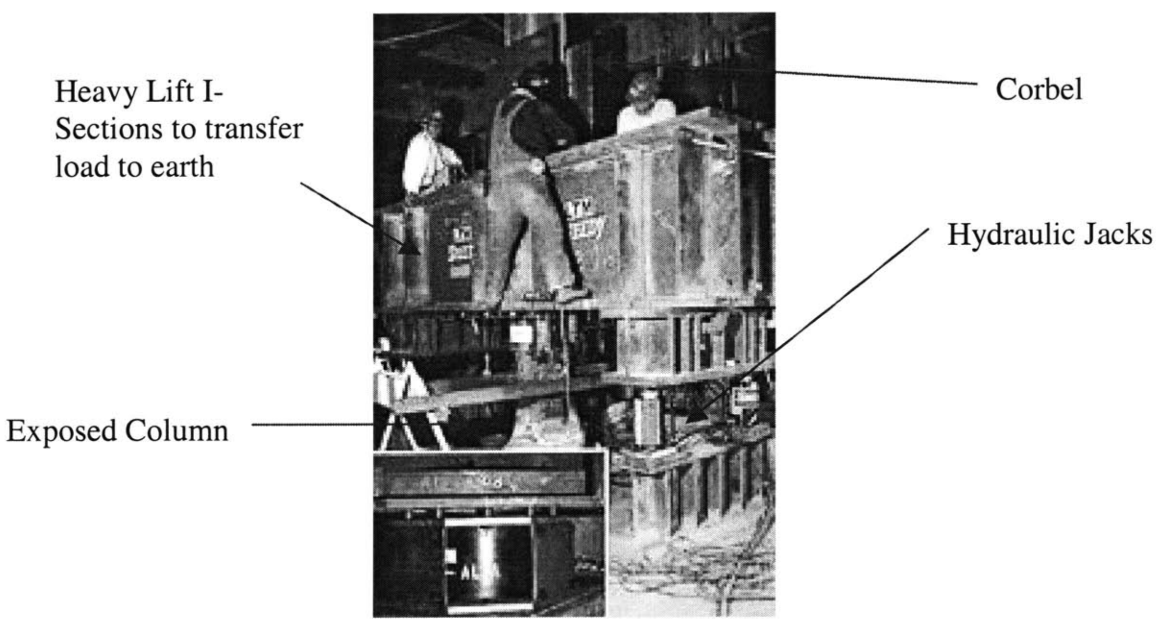



**Figure 23** – Sheedy Company cutting a Column in San Francisco City Hall, CA.

Figure 22 shows the method by which the column load is transferred to ground, before the base is cut out to facilitate the placement of the isolator. Once the jacks have been pressurized to the prescribed load, the collars are locked and the hydraulic pressure released, thus safeguarding against possible leaks and easing the heavy usage over a long period of time. Figure 23 shows a column being cut after jacking is complete.

### 3.5 Step 4: Cutting of External Concrete/Masonry Walls

This step is only applicable to structures with a continuous outer wall around the perimeter, and not one with external columns spanned by some external cladding. The latter kind of structure is facilitates the implementation of seismic isolation far better than the first. Once the jacks have been loaded and locked, the sections of concrete/masonry to be removed for isolator placement may be cut using either a circular, wire or plunge saw. The implement used is largely dependant upon the thickness of the walls.

In the Mitchell Hall case study, the external walls were 2 ft. thick unreinforced concrete, and thus a wire saw was necessary to make the cut (a 60" diameter circular saw would have been required!). The hole has to be of sufficient size to accommodate the isolator, which can be 3 ft. in diameter and maybe 2 ft. tall, as well as room for connection details and manipulation. A typical size, and that used in the Mitchell Hall design project, could be 54" in width and 18 – 22 in. in height.

The next step is to decide upon the sequence in which the elements are to be cut, that is which team is where and when and what the resultant stresses on the building will be as a result. As there are restrictions on how many columns can be cut at one time, it is occasionally useful to split the building into segments. This again relates to the previous section, in that the jack placement will be representative of this sequence.

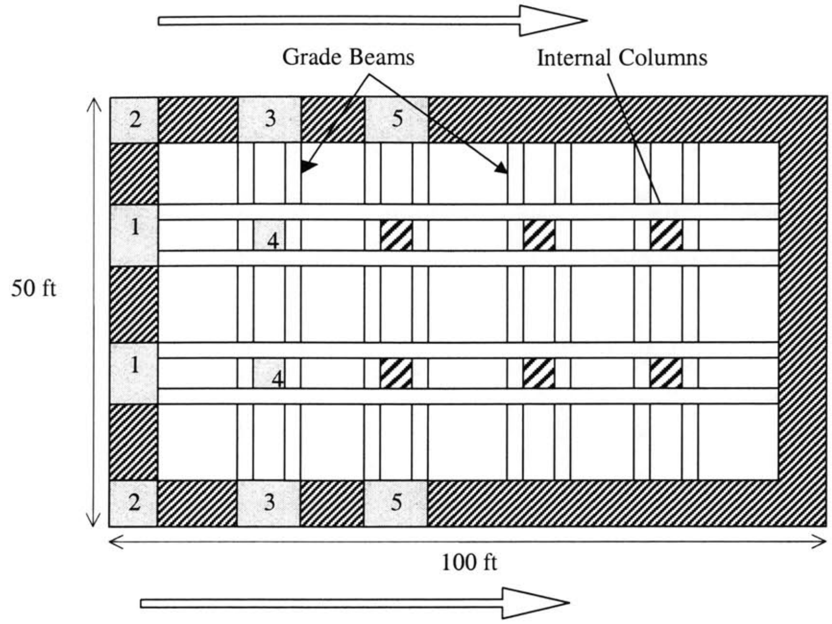



**Figure 24** – Jacking and Demolition Sequence for Mitchell Hall.

The removal of these concrete blocks can be a task in itself, as was the case in the retrofit of the Parliament building in New Zealand, where seven tonne blocks of concrete needed to be removed, as shown in Figure 25. These were removed by drilling holes in both bottom corners and inserting rails attached to a hydraulically raised platform. Once the load is transferred, the blocks are slid onto this platform, towed out to the bubble in the moat where they were removed by crane.

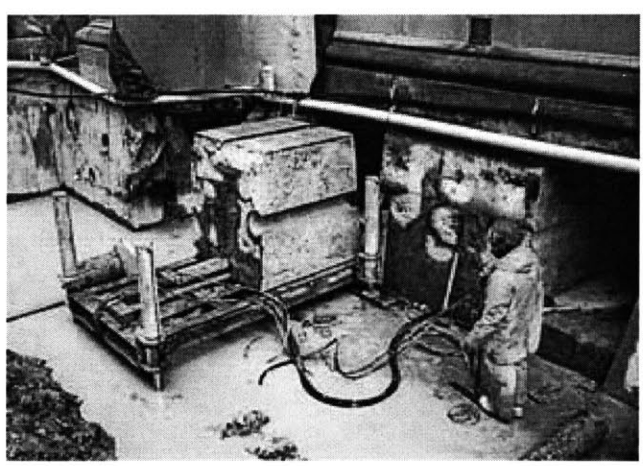
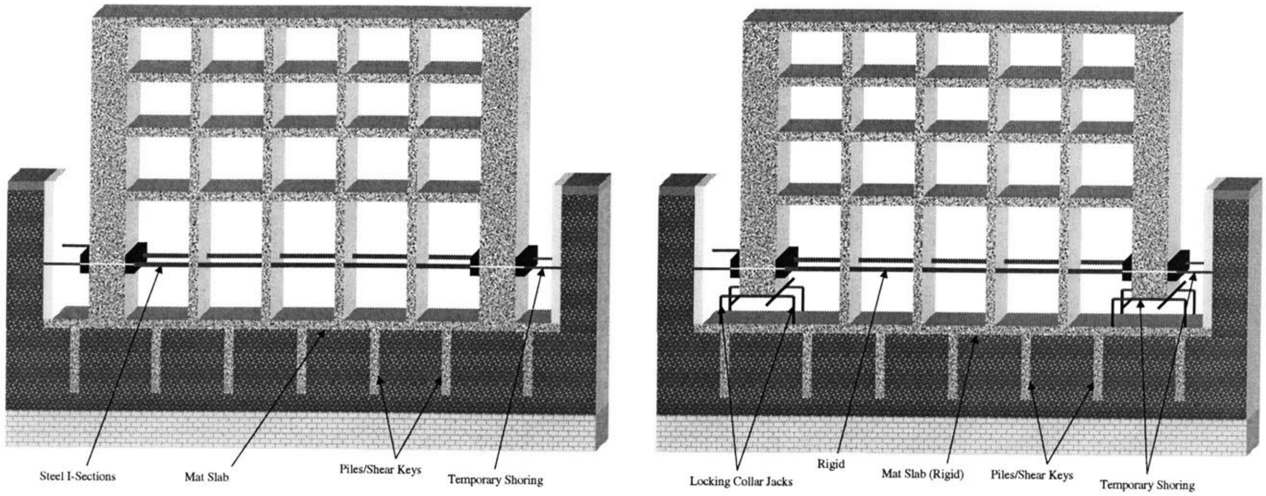


**Figure 25** – Removal of Concrete Cut Out at Parliament Building, New Zealand.




(a) (b)

**Figure 26** – Mitchell Hall upon completion of Phase 4.

Figure 26 (a) is a schematic of how Mitchell Hall, the case study, would appear after the installation of the upper rigid layer comprising of standard rolled W-Sections. As these are reasonably heavy, lifting equipment is necessary to manipulate them into place. The same piece of equipment used to remove the concrete cut outs might be employed to this end (b)

Figure 26 (b) shows Mitchell Hall when the locking collar jacks are in place and the concrete section removed. It is important that the previous step be completed at this point to ensure the building has sufficient stiffness to withstand any soil settlements or other relative movements within the structure without sustaining any kind of serious damage. Practically in this kind of operation, it is impossible for the structure not to be infringed upon in some manner or another. The slots will be cut in the same sequence as jacking takes place as detailed in Figure 24. Again this sequence depends upon the geometry of the building and any restrictions placed on jacking by the design engineer.

## 3.6 Step 5: Placement of Isolators

Once again, the major problem here comes down to the accessibility of the basement to construction equipment. The manipulation and installation of 2.75 metric tonne (6000 lb.) isolators is no easy task in confined conditions. The procedure for large-scale retrofit projects is reasonably straightforward. The isolator is lowered by crane through the bubble in the moat onto the device cart used to remove the concrete cut outs, which is then towed to the desired location and the isolator positioned on the enlarged footing or foundation.

For a smaller structure, such as Mitchell Hall, this difficulty is increased several fold. The following option was selected for use in the Mitchell Hall case study. An entire side of the moat was to be expanded beyond what was actually necessary as dictated by the expected displacement, that is the bubble used in the above case was extended along the entire length of the side with easiest access for construction equipment. A rail system was placed across the secondary span, as shown in Figure 27, which runs underneath the cut out in the external walls and the cut columns. This could be founded on the grade beams or mat slab, whichever is applicable to the respective structure. Thus the proposed demolition sequence needed to accommodate this requirement for a clear run underneath the structure. Thus a crane moves sequentially along the enlarged portion of the perimeter and lowers the unit onto these rails, which is then manipulated using a pulley system across the width of the structure and finally into the desired position.

The time span associated with the installation of each unit can in simple terms be thought of taking a four man team one working two weeks to complete. This includes each team and covers the process in its entirety from initial demolition to the connection details and finally the concrete finishes. The tasks assigned to each team may be summarised as in Table 1. The connection detailing is as specified by the manufacturer and is standard for any given set of units.

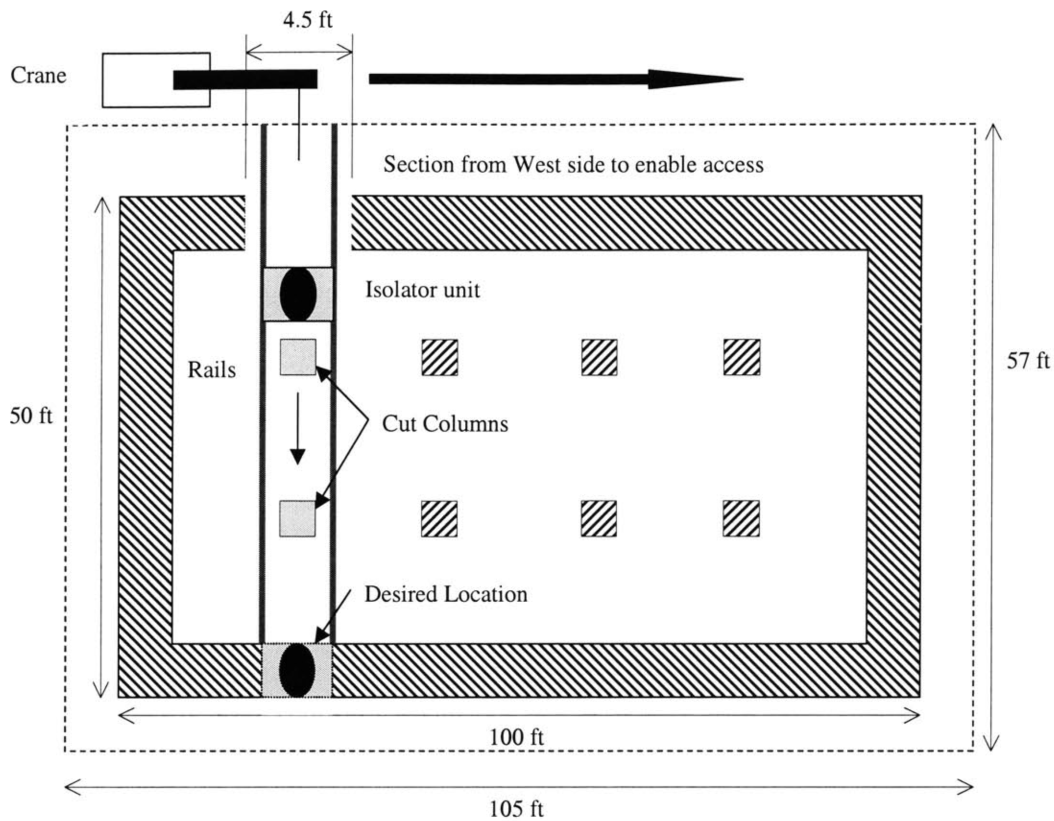



**Figure 27** – Proposed Rail System for Isolator Installation in Mitchell Hall

Generally, however, they are either bolted and grouted into the external walls, as shown in Chapter 5, or to a column with mounting plates as detailed below.

**Figure 28** – Connection detail for an isolated column.

Once the isolators have been positioned, the flat jacks can be inserted. These are used to transfer the load from the locking collar jacks onto the isolators, or effectively back onto the column itself, as well as to take up any initial deflection of the isolators when loaded for the first time.

| Team | Task |
| --- | --- |
| Demolition: | Concrete cutting, and chipping from columns to expose steel (2 days) |
| Steel: | Welding/bolting of collars and corbels to columns /walls to facilitate jacking, and setting up of temporary bracing system (1 – 3 days) |
| Jacking: | Operation of jacks and maintaining correct pressure distribution over building (1/2 day) |
| Craning: | Lower isolators into building and manipulate into place (1 – 3 days) |
| Grouting: | Connect the isolators to the building as specified by manufacturers (1 – 2) |
| Concrete: | Finish on cut outs and columns, ensure good connection and stability against future cracking. (1 – 2 days) |

**Table 1** – Assignment of tasks to installation teams

This deflection is stated by manufacturing companies to usually be in the region of about 1/8 in., however in practice deflections of up to ¼ in. are not uncommon. This is due to the variability associated with the vulcanisation process used to manufacture the units, which is often likened to baking a cake. That is to say, the ingredients are always in the same proportion, but when you bake it in the oven, no two ever come out the same. For column load transfer, this is a relatively simple procedure, however when jacking a continuous wall, particularly in a historic structure where materials such as unreinforced concrete or masonry are common, a variety of complications present themselves.

The locking collar jacks, for the most part, take up only the portion of load over the width of the cut out, with some spreading. As the support structure for the wall is changing from continuous to discrete, that is over several point supports, each isolator will finally be subject to more load than was originally over that portion of wall. Thus when jacked to this pressure to prevent settlement, the wall will be subject to negative bending stresses and potential cracking. Here, the proposed ring beams will go some way to mitigating this effect, however a better solution is the use of oil transfusion flat jacks. These are inflated initially with oil to sufficient pressure to transfer the load from the locking collar jacks to the isolators, and then further pressurised as required when the walls are finally cut.

Typically, a flat jack is shimmed up over about twenty minutes to a pressure of 50 psi, while the pressure in the hydraulic jacks is simultaneously reduced. It is important not to over extend them, in order that the contact area between the steel thrust plate and the jacking surface is not reduced. The operation of these will be further discussed, along with that of other equipment, in the next chapter. Once loaded, the locking collar jacks are left at a pressure of 200 - 300 psi in case there is a leakage problem with the flat jack. This is left for a period of about 24 hours, after which any leak is likely to have occurred. Once construction is completed, the oil is exchanged for epoxy through a transfusion nozzle, effectively making the flat jack a permanent fixture. The epoxy takes between 4 and 6 days to dry. These jacks are operated independently of each other, and monitored to allow for any differential settlements. The building is literally sitting on oil during this phase. For construction purposes, it is preferable to place the flat jacks above the isolator, simply to facilitate easier access and operation, both for the initial inflation and the transfusion periods.

Before the isolators are installed, steel angles should be welded on two opposite sides, both top and bottom of the mounting plates. This is demonstrated in figure. The angles must be sufficiently small to allow for the largest expected displacement under the maximum capable event (MCE), as they become a permanent fixture. These angles are used to attach steel plates to the isolator mounting to provide stiffness and prevent movement during construction.

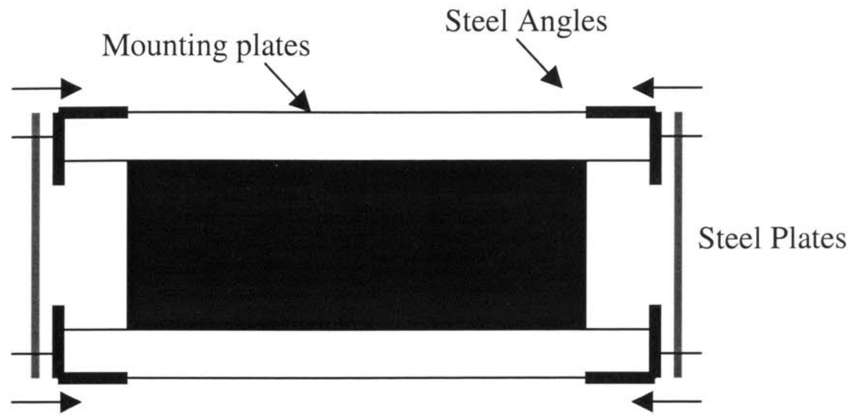



**Figure 29** – Stiffening of isolator during construction


**Figure 30** – Installation by Sheedy Drayage, San Francisco City Hall, CA.

Some examples of isolators being installed, and in operation are included below. Figure 28 shows a cut column on temporary support. It is clear that a rail system, if feasible in terms of building geometry, could be quite effectively implemented in this case.

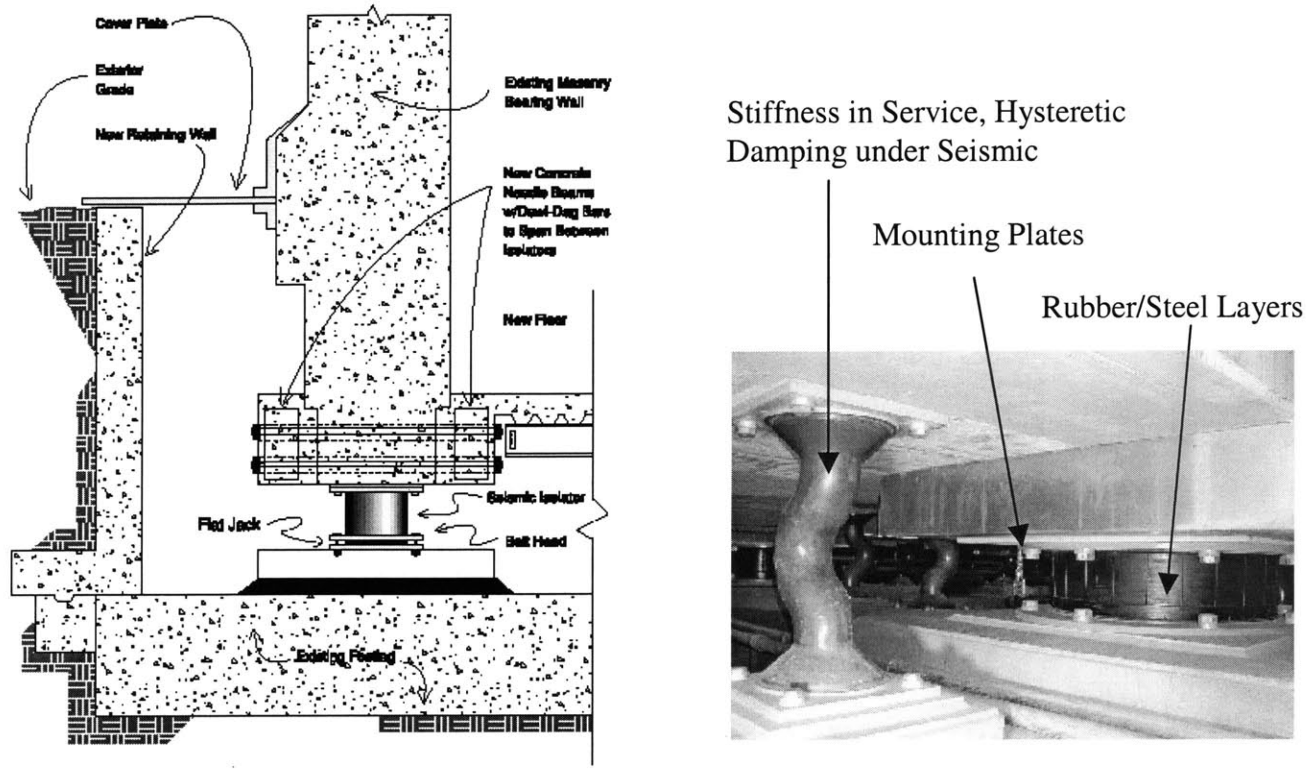



**Figure 31** – Schematic of (DIS INC) and installed isolator.

Once again a worst-case scenario might be dealt with in a similar manner to the installation of the rigid layers. With restricted access the isolators, generally about 6000lbs, are lowered over the side of the moat and placed on a sled. This might be a ¼ in. thick steel plate bent into shape, with steel stiffeners.

## 3.7 Step 6: Separation of Building from Base

Before separation, a new basement floor has to be installed and the utilities, electrical and mechanical need to be reconnected. Flexible connections are required for all of these, and some schematics of these shall be listed in Section 5.1. The basement floor rest upon the framework of steel I-Sections used to provide the rigid layer above the isolation. A hybrid construction is often used, both to prevent compression in the steel when subject to large seismic demand, and also for ease of construction. Corrugated steel sheets are places spanning the secondary spans which are supported on the Steel I-Sections, and concrete pored in-situ. Steel studs are welded in place to provide a full, or the necessary shear connection as determined by the design engineer. An example is shown in Figure 32.

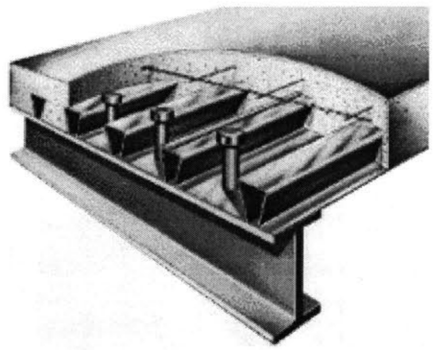


**Figure 32** – Steel/Concrete Composite Floor Slab

The final stage in the construction scheme is the disconnection of the building with its base. The remaining portions of concrete between the isolators are cut away with a wire saw, on a plane with the top and bottom of the isolation units respectively, in order to allow total freedom of movement under seismic loading. Once the cutting is complete, the steel stiffening plates shown in Figure 29 may be removed from the isolators, and the building set free. The tool used to cut the walls is again dependant upon their thickness, and the material. If the concrete is reinforced, this adds to the expense and difficulty of this phase. The wall is cut along two planes, approximately on a line with the top and bottom mounting plates of the isolators. At this stage the isolators are bearing the full load of the wall (column load already fully transferred) and the oil in the flat jacks is sequentially exchanged for epoxy as each segment is removed.

For a thin wall, a circular saw or a plunge saw might be used, but as the thickness increases, a diamond tipped wire track saw is probably necessary. The external shoring can now be removed. While it might seem illogical to remove it after the building is cut, the load placed on the structure during cutting can be substantial, and thus the greatest degree of stiffness attainable is desirable. The steel stiffening plates on the isolators themselves are the last to be removed, finally separating the building from its foundations.

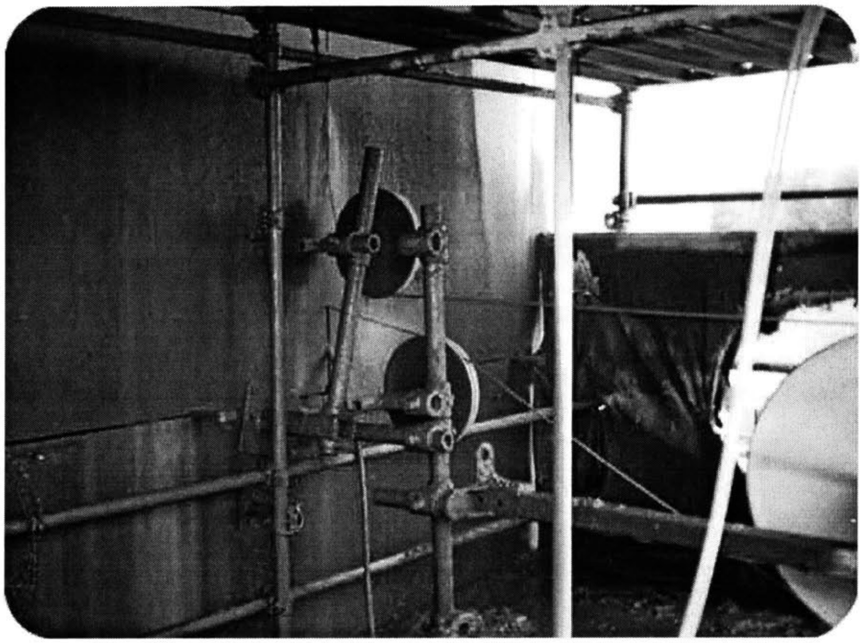


**Figure 33** – Example of a wire saw being used to cut a concrete wall

Often viscous dampers are placed between the footings and the building to provide some energy dissipation under service loading, such as high winds. The amount required is largely dependant upon the height and weight of the structure. An analysis of the structure, using a finite element programme such as SAP 2000 or its equivalent, can quite easily determine what the predicted behaviour of the structure will be under severe wind loading for a given location, and thus the necessity for viscous dampers or energy dissipation capacity in addition to service stiffness. Stiffness can be increased in the service condition through the introduction of low strength steel sections, as shown in Figure 31, which are designed to yield under seismic loading.

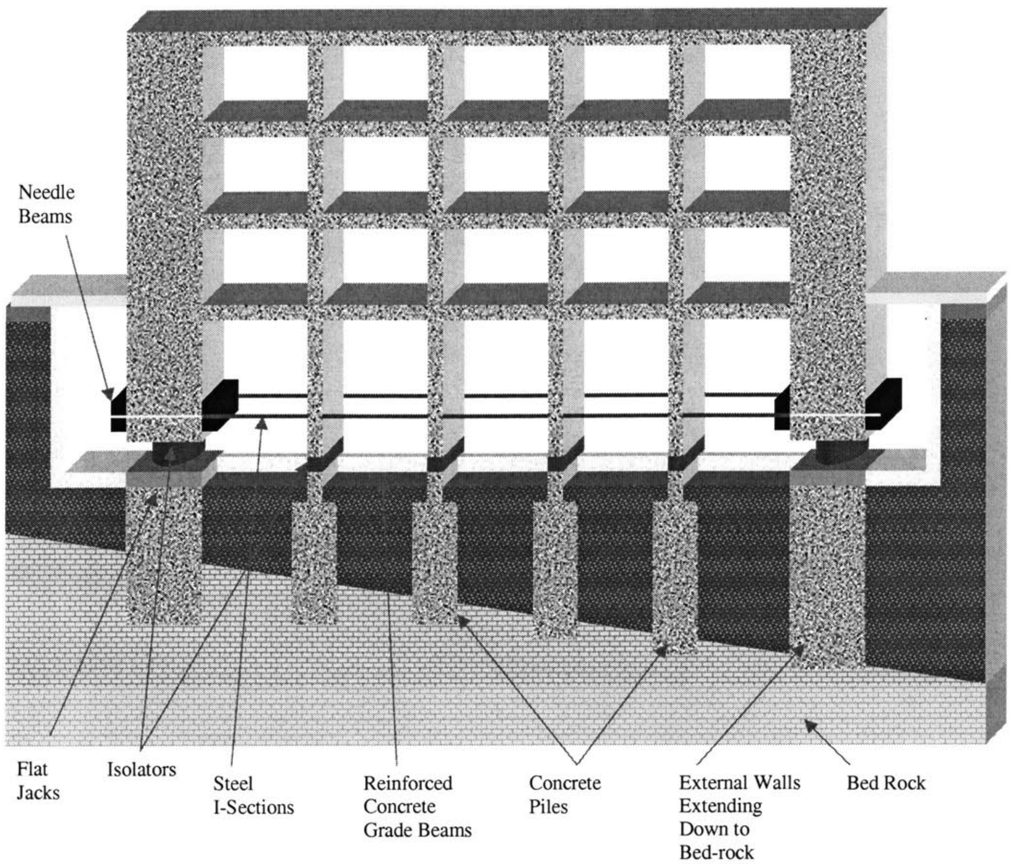



**Figure 34** – Mitchell Hall after completion (excluding placing of basement floor)

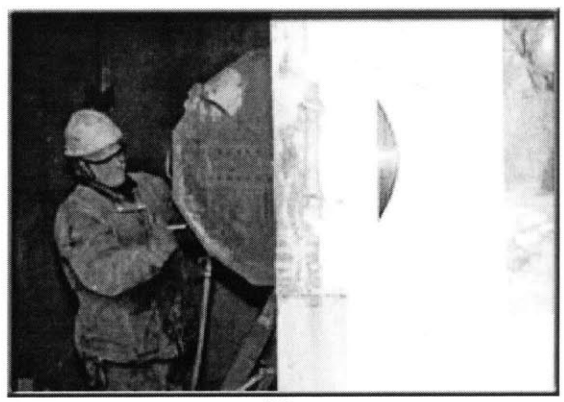
The last aspect of the sequence is debris removal and site cleanup. This stage can be quite costly. Sliding plates must be placed around the entire perimeter of the structure. These are attached by a hinge to the external wall and rest on ground level, with no fixed connection. There is the possibility of these causing injury when moving, however this cost is far outweighed by the overall advantages of the system.

# *Chapter 4* - Construction Equipment

## 4.1 Concrete Cutting Equipment

The circular saw is a fairly standard concrete cutting tool, but only for a certain thickness of wall, as the increase in blade diameter required as thickness increases makes the saw a liability rather than a benefit. For Mitchell Hall, a 60 in. diameter blade would have been required for full penetration, and a this stage it is no longer reasonable to expect to have any degree of accuracy with the cuts. Confined conditions can also restrict the use of this piece of equipment, as shown below in (a).

(a)



(b)

**Figure 35** – Use of a circular saw

Another common method is the plunge saw, which is simply a diamond tipped, hydraulically operated chain saw. Precision cuts are relatively easy to achieve, but again thickness is a restriction.

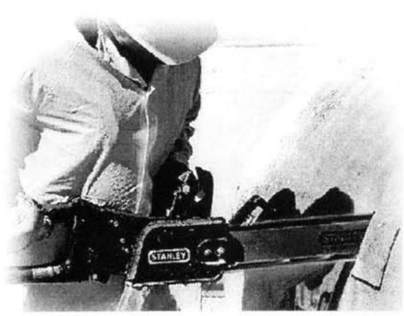



**Figure 36** – Operation of a plunge saw.

For thicker walls it is necessary to use a wire saw. Basically, tracks are set up on both the interior and exterior of the wall, which move in unison cutting along a specified path. As the system is mechanised, precision in not a big problem, however setting the equipment up can be time consuming.

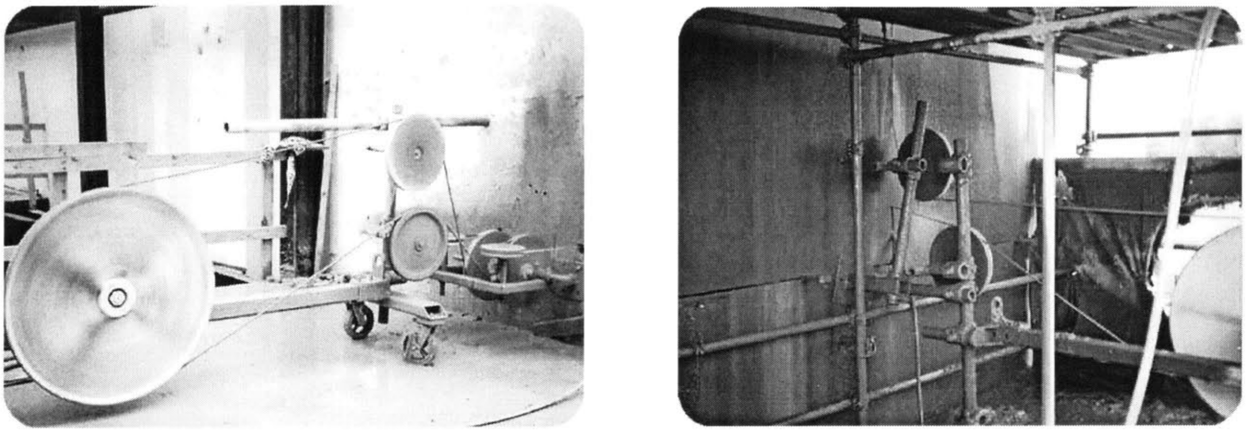


**Figure 37** – Use of a wire saw.

## 4.2 Locking Collar Jacks

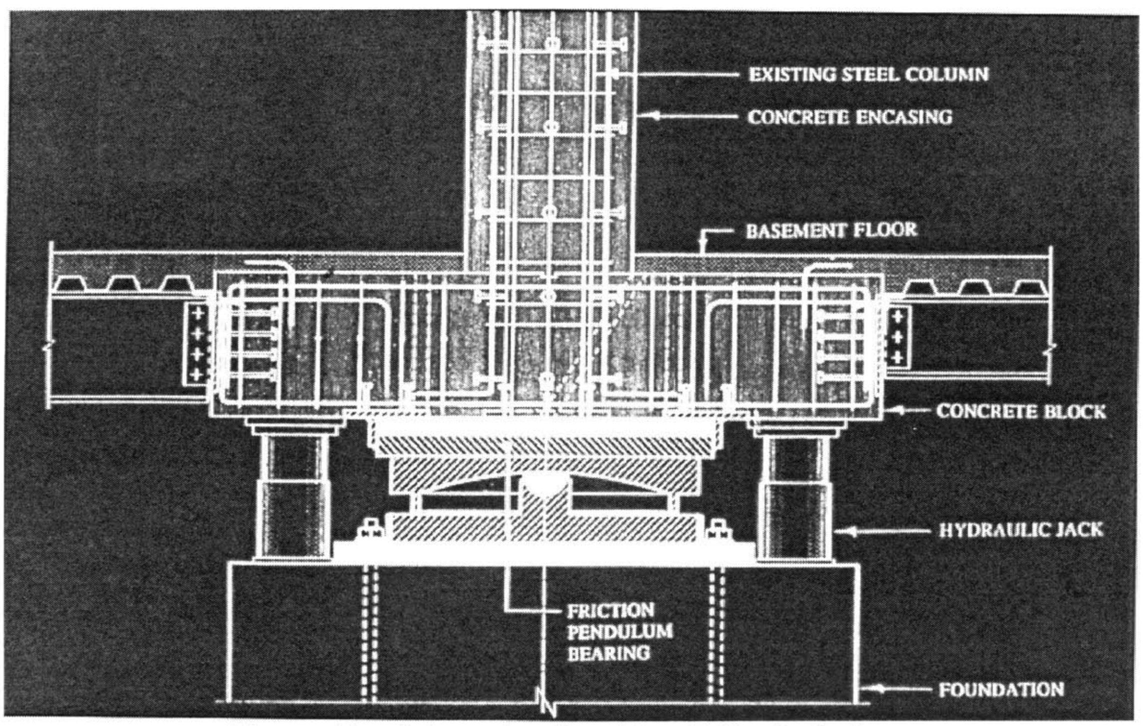



**Figure 38** – Schematic of Locking Collar Jacks in use.

Hydraulic Locking-Collar Jacks are the most frequently used for this kind of project. This is simply a standard hydraulic jack, which once pressurised can have a collar locked and the fluid pressure released. This prevents any instability inherent in resting a building on fluid, as well as mitigating possible damage from potential leaks etc.

The general procedure is to transfer the load from the column to the hydraulic jacks, cut the column, place the isolator and then transfer the load from the jacks to the unit itself, via a flat jack as described in the next section.

## 4.3 Flat Jacks

Flat jacks are the apparatus used to transfer the load from the hydraulic jacks to the isolators, while preventing any movement of the building. They generally consist of a circular mild steel capsule, with a cross section that is initially shaped like a dumb-bell. The inset portion is fitted with ground steel thrust plates, before the jack is positioned for loading. When the jack is loaded, or inflated with liquid grout, the hydrostatic pressure is transmitted by the flat portions of the capsule through the thrust plate, while the toroidal rim deforms allowing the flat plates to move apart. It is important not to over inflate the jacks as the jack will become oval in shape and burst, and the usable bearing surface decrease. The flat jacks are permanent fixtures when filled with epoxy, through the use of a special resin injection cylinder.

These flat jacks are often fitted with a pressure gauge in order that the weight upon it may be measured. This is essential when performing a jacking sequence with multiple co-ordinated units.

More recently units known as Oil Transfusion Flat Jacks are being used in construction projects, for the reasons described earlier. That is to say, the difficulty in jacking a continuous wall, due to the excess force exerted upwards by the fully "loaded" isolator, can be overcome using these parts. Bending stresses that cause crack propagation are no longer induced, as there is a variable uplift pressure.

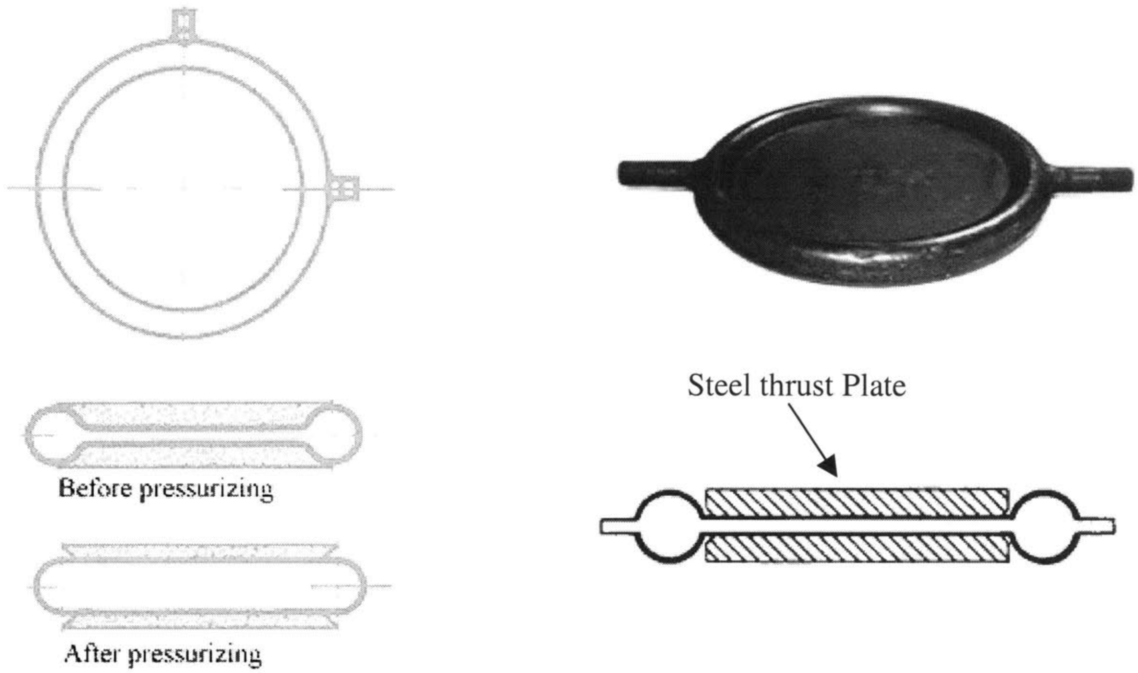



**Figure 39** – Schematic of regular flat jacks.

To mitigate this a second chamber is added to the flat jack that may be filled with oil. The initial load is taken up in the primary chamber with epoxy and subsequent load increases, as the walls are cut, are taken up with hydrostatic pressure induced by pumping oil into the secondary chamber. This may be varied at any appropriate time, and is monitored to allow for differential soil settlements due to the new loading conditions on the foundations. Once the final load has been reached, the oil is exchanged for epoxy through a transfusion nozzle, and the sequence completed. This effectively negates one of the reasons for placing the needle beams, but due to connection difficulties and safety considerations, it was decided to leave them in place.

## 4.4 Isolation Units

A variety of isolators are available, such as Lead Core Rubber Bearings, Friction Pendulum Bearings, Natural Rubber Bearings, Fibre Reinforced Bearings, Natural Rubber Bearings, High Viscous Damping Rubber Bearings and a host of others. By far the most common in the United States and Japan are the Lead Rubber Bearings (LRB). These were originally developed in New Zealand in the 1980's, and were the first apparatus to make the long standing concept of seismic isolation a reality. In their most rudimentary form they consist of alternate layers of steel plates and vulcanised rubber mounted between two ½ in. thick steel plates. The steel bonded to the rubber provides a very large stiffness in the vertical direction, but very low in the horizontal. A schematic is given in Figure 40.

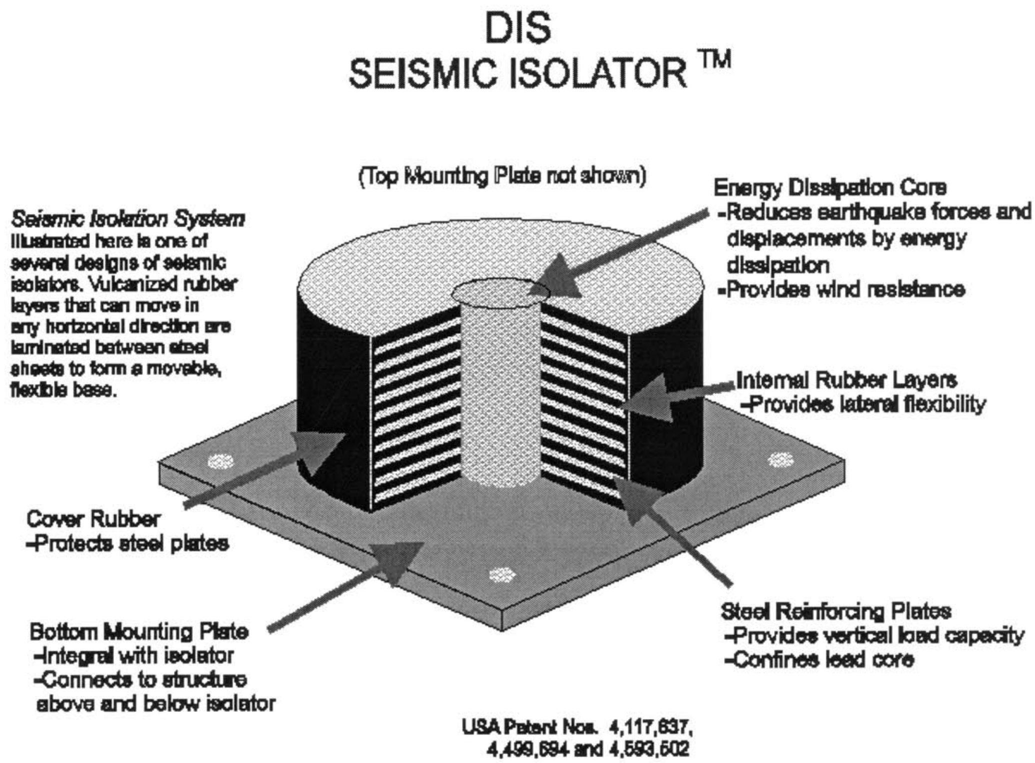



**Figure 40** – LEAD rubber Bearing as manufactured by DIS Inc.

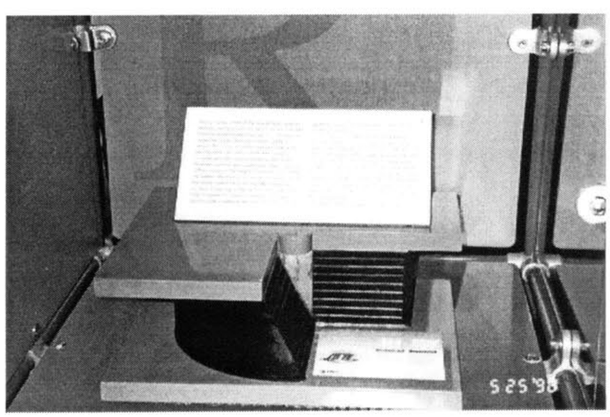


**Figure 41** – Lead Rubber Bearing cut away.

The next most widely used system is a friction pendulum bearing such as that manufactured by Friction Pendulum Systems (FPS). These consist of a Teflon on stainless steel slider, and have the advantage of being substantially smaller in the vertical dimension than LRB's. In addition they provide damping through frictional force. A major disadvantage however is that the damping capacity varies as the system heats up, and is difficult to predict. A big problem with friction damping, in addition to the above, is that there is a sudden change in acceleration at the maximum displacement, as opposed to the gradual change provided by the LRB system (i.e. acceleration is zero at maximum displacement).

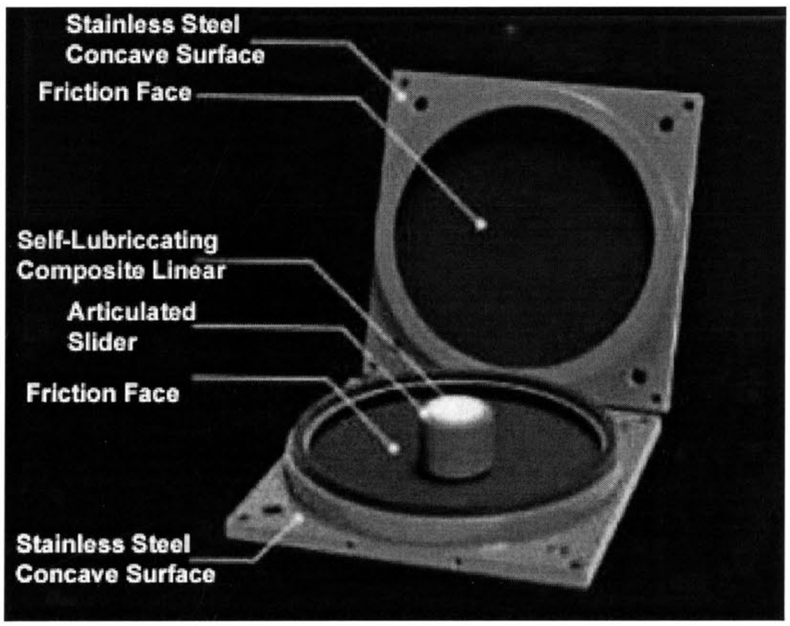



**Figure 42** – Detail of a Friction Pendulum Bearing

When subject to seismic loading, the bearing, or articulated slider, moves along the concave surface inducing small pendulum loads to the supported structure. A dynamic friction force is created, which provides a damping force thus dissipating some of the input energy from the imposed ground motions.

# *Chapter 5* - Unique Problems to Isolation

## 5.1 Flexible Connections

One of the most evident problems when considering base isolation is that, if the building can move relative to the ground, so to must all utilities and services to the building be able to do so. Dynamic Isolation Systems (DIS) Inc. provides some detailing describing methods by which these can be implemented. Some of the more relevant have been included for the sake of completeness.

It is important that every connection have several degrees of freedom, namely in both lateral directions, the vertical direction, and finally rotational freedom in both the horizontal and vertical planes.

All of the schematics following are as detailed by DIS.

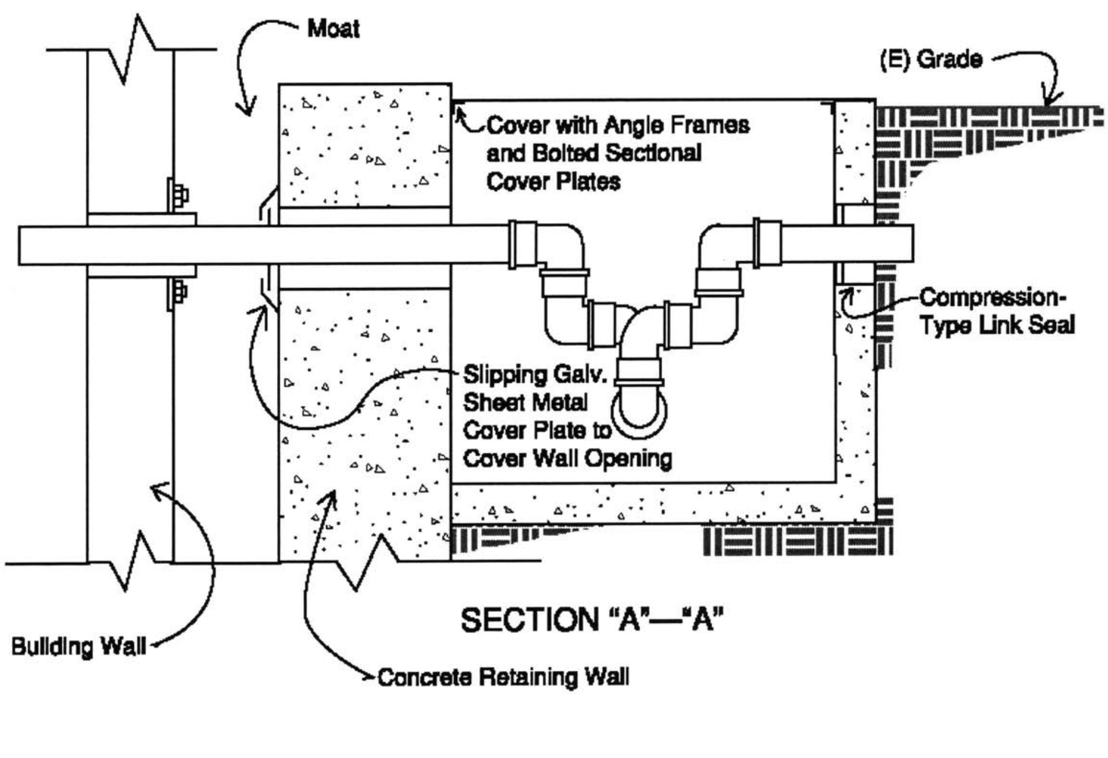



**Figure 43** – Elevation of Sewer Connection

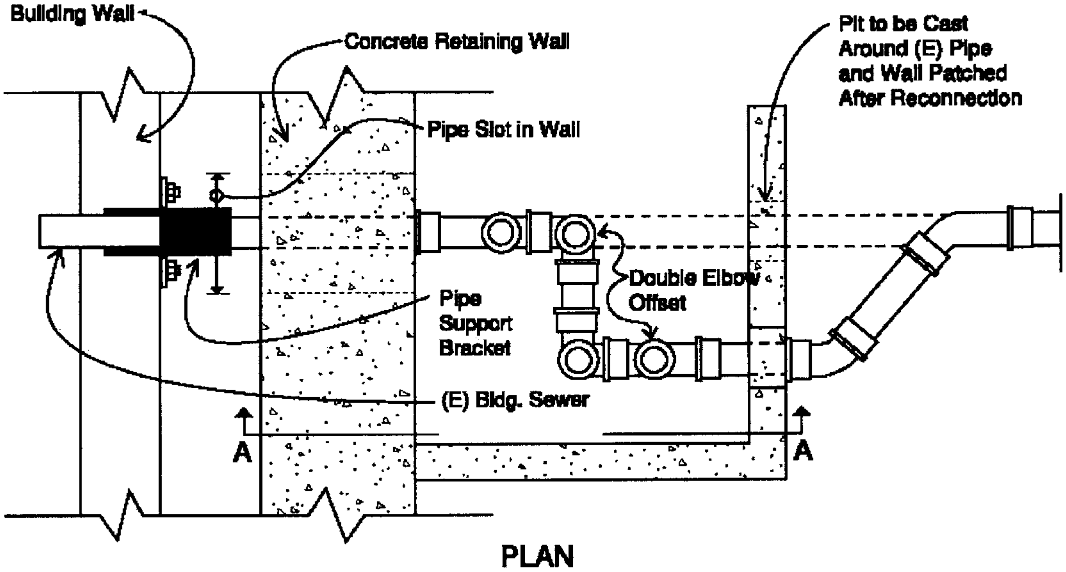



**Figure 44** – Plan view of sewer connection.

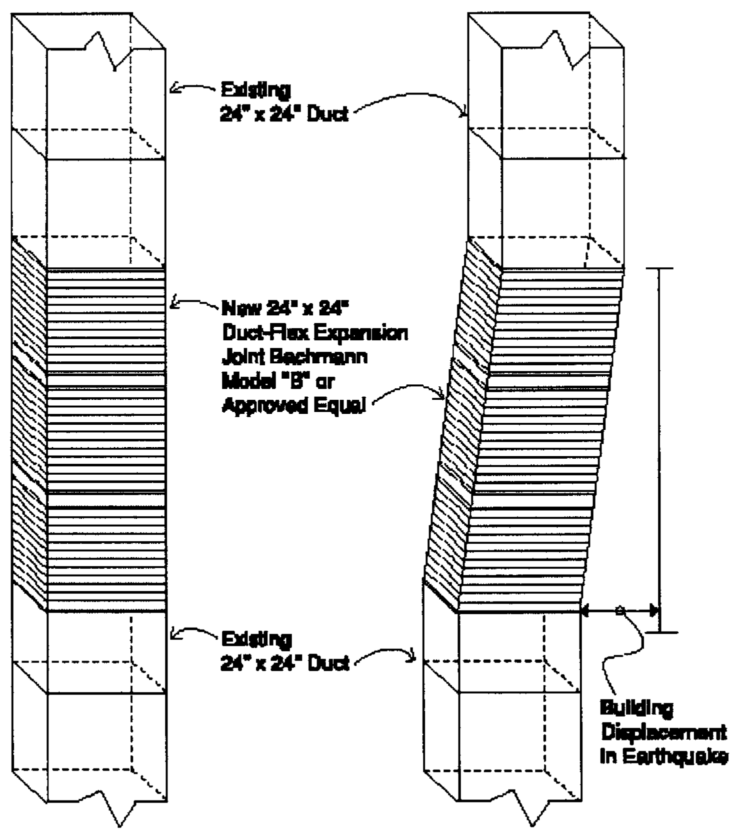



**Figure 45** – Flexible Ducting through Layer of Isolation.

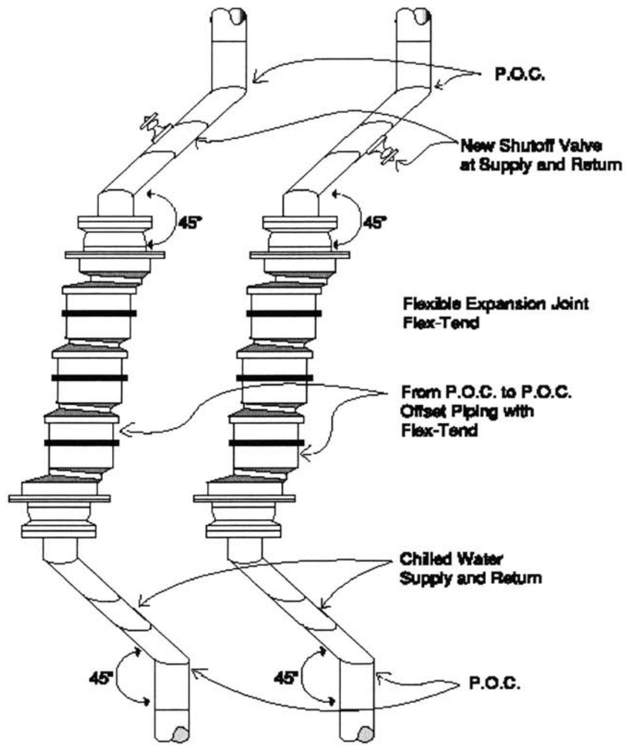



**Figure 46** – Flexible Chilled Water Risers.

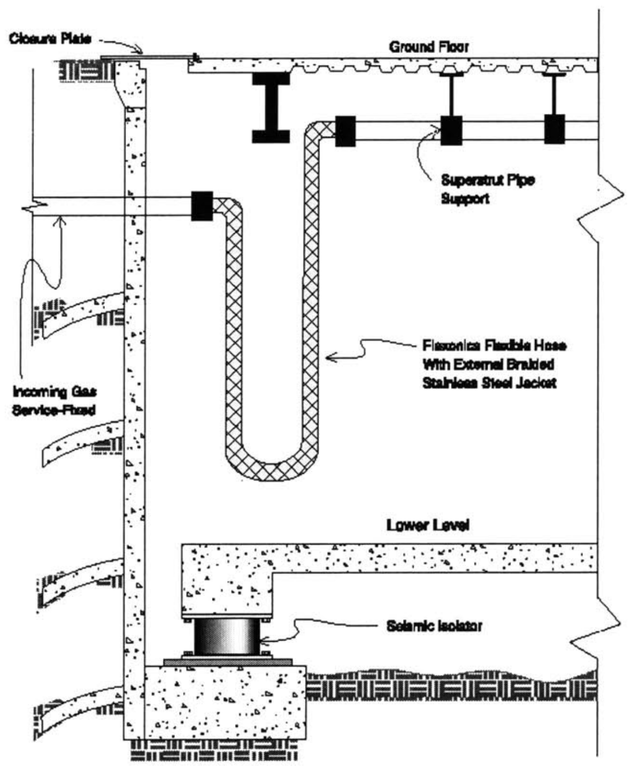



**Figure 47** – Flexible Gas Line at moat Wall

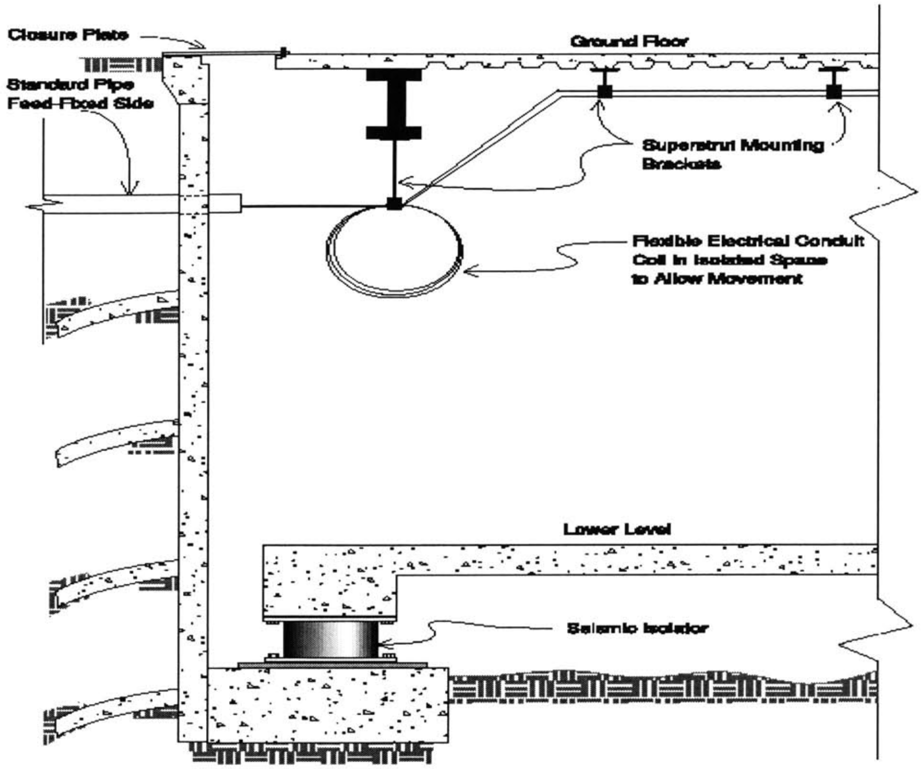



**Figure 48** – Electric Stub at Moat

## 5.2 Connection Details

The connection detailing of the isolator to the structure is generally specified by the manufacturer, who provides both the equipment and instructions on how best to apply their respective technology. However, as each and every construction project is unique unto itself, many situations arise where perhaps more than one isolator is needed per column, or the diameter of the isolator exceeds by half the continuous wall that it is intended to support.

In cases such as these, the specifications can only be used as guidelines. Some of those considered more relevant to retrofit have been included in this section, as well as some interesting instances of connection details from projects around the country.

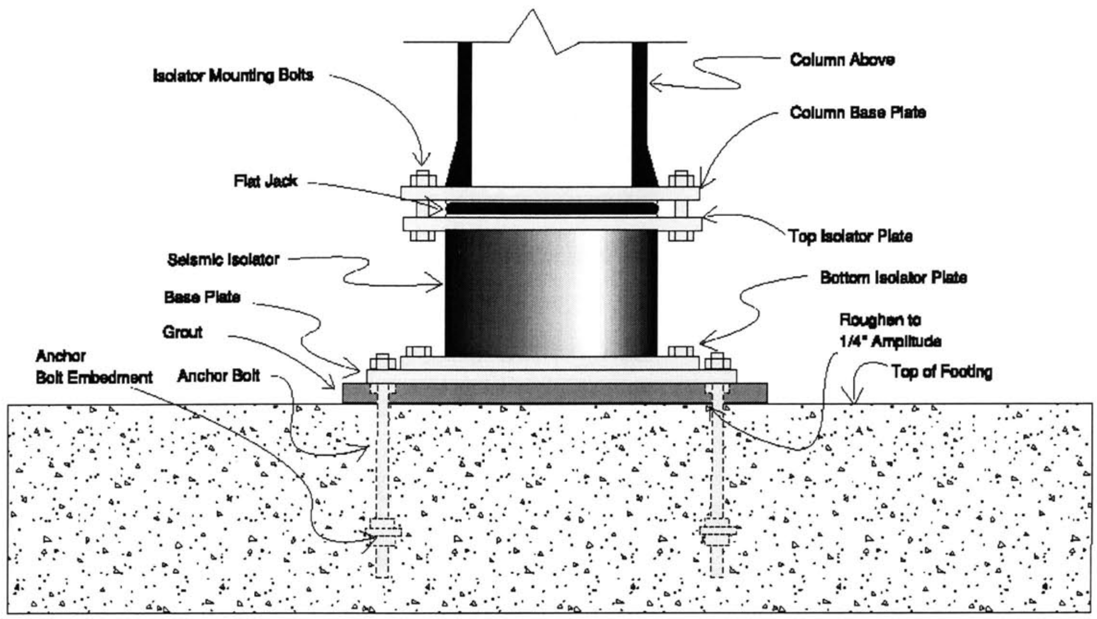



**Figure 49** – Seismic Isolator Assembly for Retrofit Construction (DIS)

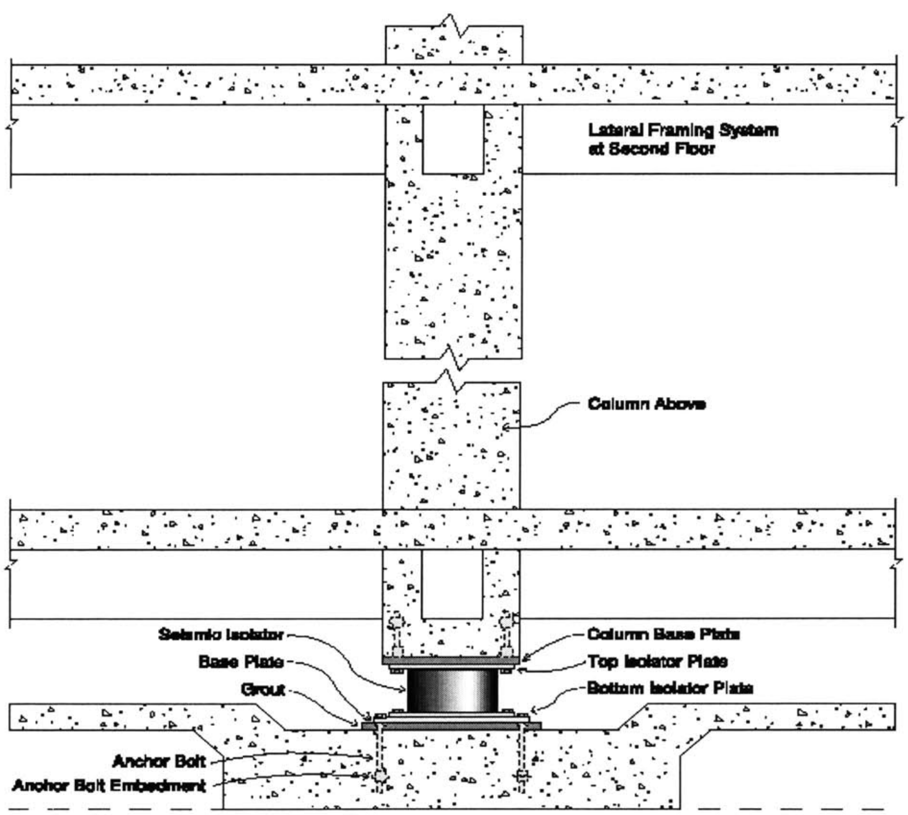



**Figure 50** – Column Connection Details (DIS)

As shown in Figure 51, the connections for a standard isolator are fairly substantial, which can cause occasional problems when dealing with a dilapidated material, such as are commonly found in retrofit projects.

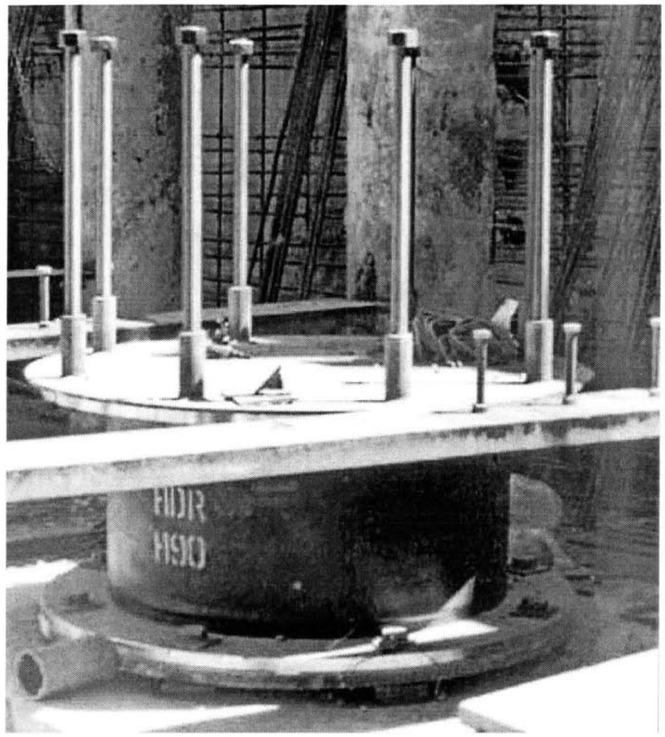



**Figure 51** – Isolator with connection bolts.

## 5.3 Other Practicalities of Base Isolation Construction

The theory of isolation is sound when considering seismic hazard mitigation, but a disadvantage to this solution is that some parts of a structure simply have to remain fixed at the base. For instance, the steps leading to the entrance of a building and the elevator shafts cannot realistically be flexible and at the same time serviceable. In terms of stairs, the solution is easily solved through the use of sliding plates at the entrance to the building, leaving the steps rigid at the foundation. A schematic, again as suggested by DIS Inc., is shown in Figure 52. The details for an elevator are somewhat more complex and shown in Figure 53.

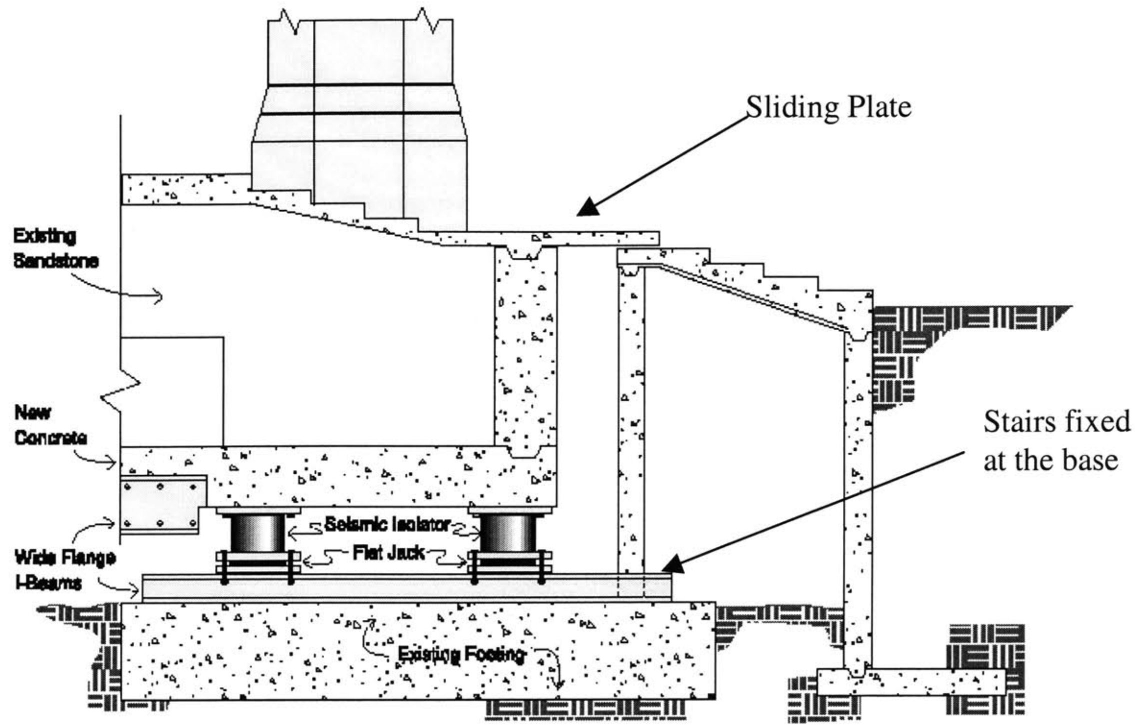



**Figure 52** – Option for Retrofitting Stairs

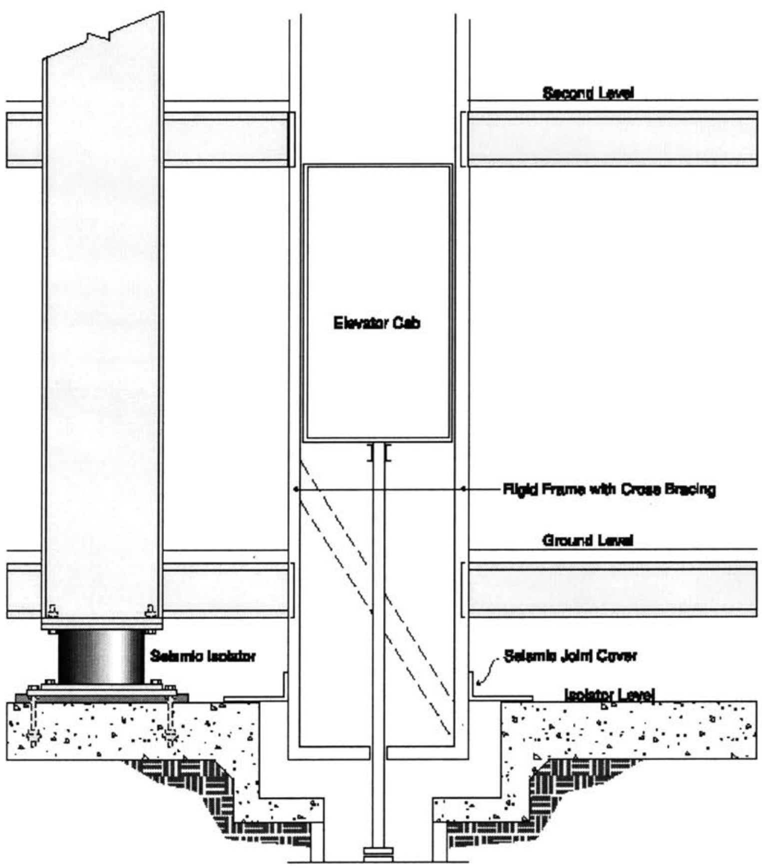



**Figure 53** – Retrofit Strategy for an Elevator Shaft.

# *Chapter 6* - Typical Costs

There are several rules of thumb used by contractors to give rough estimates of jobs to potential clients without going through the rigors of a full cost estimation. This is built up from an experience base from what a similar project cost before. However as isolation is still an emerging field, these prices still vary wildly, and are by no means certain. One would expect and hope that over the next few years this will stabilise. An interesting example cited by Brandon & Johnston Associates Design Engineers, CA, was one of a job they bid for the retrofit of a historic structure. The bids came in over a wide range, so the client disregarded the two highest and two lowest, and in doing so, unknowingly the most experienced competitor who had bid the highest price. The job began, and the cost overruns ran wild, and ended up very close to the highest original bid. This simply demonstrates the uncertainty, not only in a retrofit project, but also the need for experienced people to do the work, that generally comes with an isolation project.

In terms of the isolators themselves (considering rubber bearings), the cost can be approximated from the relationship of US$1 per cubic inch. For instance, a 29.5 in. diameter isolator that is 18 in. high would cost he volume in cubic inches multiplied by US$1, or US$13,000. This is a fairly typical cost for such a unit manufactured by DIS Inc.

In terms of installation, dependant upon the size of the project and the number of isolators to be installed (i.e. reuse of equipment), a single isolator costs between US$15,000 and US$25,000 to install from start to finish. This price includes the demolition, jacking, isolator connection, flat jacking and finishing In the case of Mitchell Hall, this would be around the higher end of the scale. This does not include the cost of the unit itself. This cost is as quoted by Sheedy Drayage Co., CA, in 2001.

Jacking costs usually run about US$5,000 to US$7,000 per column for a standard section, where US$5,000 is cheap and US$6,500 is about standard. This includes the provision of jacks, materials and take down. Complicated jobs, or columns supporting excessively loads can run significantly higher, sometimes in the order of US$70,000.

The main factor in determining the actual cost is: who provides the jacking surface, that is the surface by which the section will be lifted and the base upon which the jack will be founded. With these fully in place, the lower range of US$6,000 is relevant, but this rises significantly, in the order of US$10,000 to US$15,000, if the surfaces not already in place.

Probably the largest single cost associated with a retrofit project is site access and temporary earth support, however these costs are very much situation dependent.

# *Chapter 7* – Conclusion

Base Isolation today is used primarily only in high profile or large-scale public projects. The relative expense of this option as compared with more conventional method dictates that it must be so. Any part of construction, forming such a large part of this expense, that may be reduced can thus significantly impact upon the viability of a project. A single concept lies behind this occurrence, that being the notion of competition. As manufacturers lose part of their market share, the cost of entry decreases and the primary barrier to would be entrants is removed.

It is important for contractors to fully understand the technology involved with base isolation. The many design manuals and codes go a long way toward achieving this, and this thesis is intended to take that process one step further in providing a small resource that was, in the author's research, difficult to obtain in a single document.

The practises discussed herein are not beyond the capabilities of many contractors, and as this technology becomes more widespread and more familiar, it is likely that it will take over as the prevalent seismic hazard mitigation strategy, not only in the United States and Japan, but throughout Europe and Asia as well.

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