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Structural Engineering for Northern Pakistan: Indigenous Architecture and Earthquake Resistance

by Jimmy Chi-yi Su
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Abstract

In the Fall of 1993, a joint project began between architectural designers and engineers, for the design of houses in Karimabad, located in the Northern Areas of Pakistan. This thesis records the author's involvement as structural engineer, and how quantitative engineering work was shaped by qualitative considerations of architectural design issues, and the unique culture of Karimabad.

The broad range of structural design challenges in the area was assessed, and included earthquake loading, cultural precedents for building types and spatial use (i.e. live loads on the roofs), material availability, and appropriate technologies. Seismic loads were the main structural concern, and the behavior of single story, masonry buildings under seismic loads was investigated. Houses constructed from reinforced stone masonry with a timber roofing system were selected as the most appropriate technology for the region. Processes to quantify the necessary wall reinforcement were found and developed, and one architectural design was engineered to illustrate these processes, and to gain a general idea of how much steel would be needed in one house. Construction guidelines for all parts of a house, including roofs and foundations, were also researched and developed.

The teamwork between architects and engineers throughout this project was evaluated and discussed. Principles of effective interaction that were learned, and the exchanges that occurred between the architects and engineers in the course of this project, are presented. The purpose of this record is to help future projects, between architects and engineers, to achieve a working relationship which effectively synthesizes the two professions, and produces better designs.

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by
Jimmy Su

Submitted to the Department of Architecture on May 12, 1995 in Partial Fulfillment of the Requirements for the Degree of Master of Science in Building Technology

Thesis Supervisor: Leonard Morse-Fortier

Title: Assistant Professor of Building Technology

My sincere thanks to...

Last may not be least, but here - first is definitely foremost. My thanks to God, the Father/Lord Jesus Christ/Holy Spirit. In my life, and the lives in my family, You have taken care of the most fundamental human need, and through my relationship with You, I've been given so much more than I need. You are constantly teaching me to live for what is most important, and to be what is truly good. I'll do my best to live for You with increasing dedication and fullness, all the way to the finish.

Mom and Dad, thanks for patiently guiding this child into a relatively mature individual (no small feat, considering the child). You are two of the greatest blessings in my life!

Vivian, you've been a consistent friend and more over these many years, and are truly a part of my growth as a person. Thanks for your love which encompasses so much.

To Prof. Len: Thanks for your encouragement, honesty, and teaching - of engineering and more. May your students continue to benefit from your teaching, and your children grow under your fatherhood.

To Professor Jack Myer, and Ken Kruckemeyer: My appreciation and respect to you, for your willingness to work together with engineers to synthesize better designs.

Final thanks to the many brothers, sisters, and friends who have shared themselves with me, and allowed me to give in return!

This work is dedicated to the people of Karimabad: May your growth and prosperity be more than just material.

Unless the Lord builds the house,
it's builders labor in vain.
Unless the Lord watches over the city,
the watchmen stand guard in vain.

- Psalm 127: 1

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Preface

The cover picture shows a Gasirkum roof system, and was taken during the August 1993 team trip to Karimabad. This roof system is based on a traditional skylight opening commonly found over the central living area (*Ha*) of homes in Karimabad. The particular roof shown embodies a lot of what this project was all about. In these times of fast-paced cultural change in Karimabad, due to increased accessibility and modernization, there is a tension between implementation of the new, and preservation of tradition. Through our project, we wanted to help the people of Karimabad, as they faced numerous options and decisions, particularly in the construction of new homes. The Gasirkum roof system is built with local materials, and technology that is understood by the people in Karimabad. It's also a relatively stiff roof diaphragm, and performs well under seismic loads. Seismic stability and structural efficiency can be further improved through engineering analysis and design. In this project, we looked for these potential combinations of functional traditional technologies with new ideas (and materials) to produce designs that would be structurally safe, energy efficient, and compatible with the existing built environment.

Using familiar elements and technologies, combined with the judicious use of new ideas and materials, seems to be the best direction for progress in Karimabad. It is my hope that the information and work done in this thesis, and the project as a whole, will enable the people of Karimabad to retain the best of their traditions, and wisely incorporate the new technologies they can now access.

SECTION 1

INTRODUCTION

"Hope in the future is rooted in the memory of the past, for without memory there is no history and no knowledge. No projection of the future can be formed without reference to the past. Past, present and future, memory and prophecy are woven together into one continuous whole." -Buckminster Fuller

Location and Environment 1.1

Karimabad is a small town of approximately 12,000 people, located in the Hunza Valley region of northern Pakistan. The town sits at a high altitude of 2500m above sea level and the nearby mountains rise over 6700m, forming the bowl shaped valley siting Karimabad. (See Map 1 of Appendix A for an enlarged map of the surrounding region, and Map 2 for a general layout of the town.) Winter temperatures can get as cold as -5°C with little snow fall, but swing around in the summers to reach over 30°C. The local precipitation is only about 15 cm per year, and canals were built to carry glacial run off downward to provide the town's precious water supply. The people of Karimabad have also terraced the valley slopes to provide flat areas for living and planting.

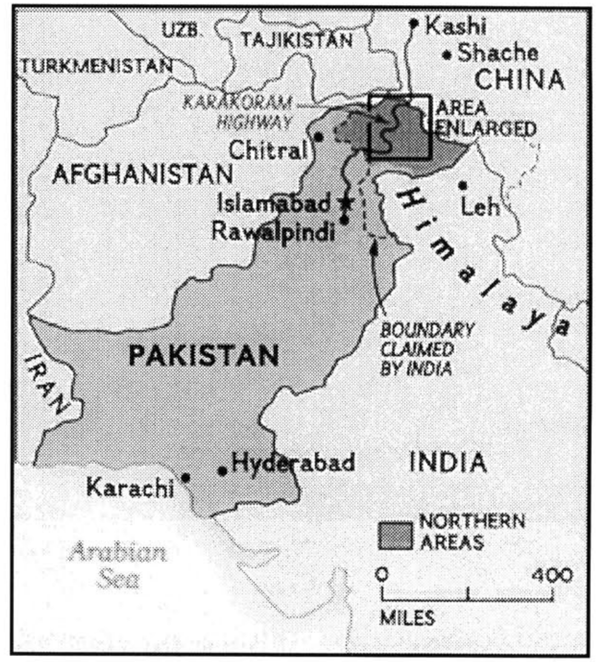

Figure 1.a Location of Pakistan and Northern Areas (McCarry, 1994)

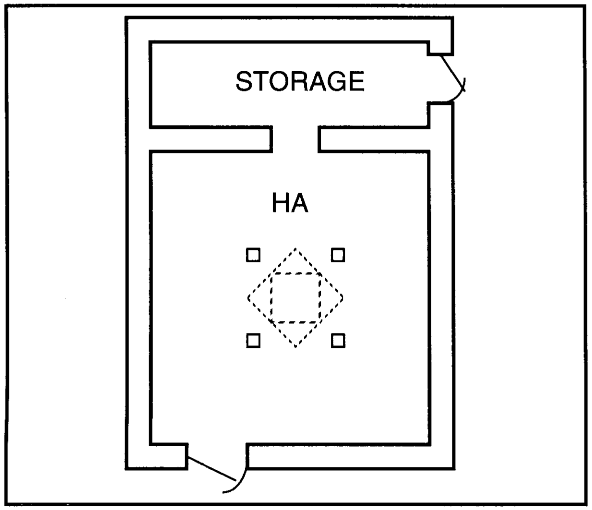

Figure 1.b Simple Layout with Ha (columns, roof opening shown) and Storage

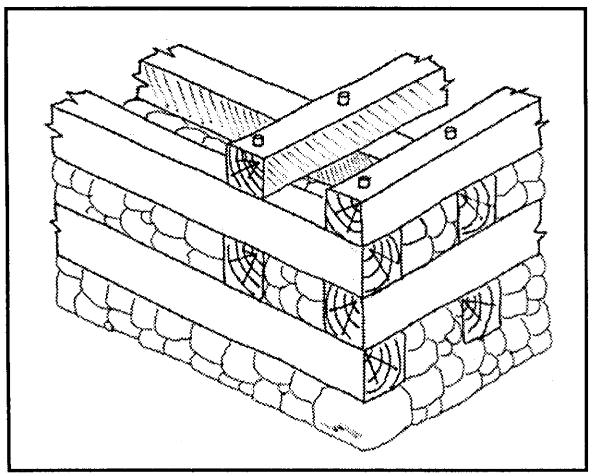

Figure 1.c Timber Cribbing System with Pegs

The Northern Areas, which includes the Hunza Valley, have traditionally been an agriculture and livestock based economy. The harsh climate, poor soil conditions, and difficult accessibility limited the prosperity of the general population. Settlement of the area began about 600 years ago, with the construction of the Baltit fort (Pirani, 1989), and a feudal society was established. Before 1947, the area was still under *Dogra* rule from Kashmir, and there was a strong local tribalism between the various clans in the region.

The living situation in most homes included an extended family of multiple generations, with one main room called the *ha*, usually about 16 to 20 feet square. This room was where most of the family life and living functions happened, including cooking, eating, and sleeping. Smaller rooms on the side of the house would be used for storage and/or livestock. Before the recent arrival of many catalysts for change, the general construction technology of houses in the town was similar to other mountain dwelling communities. Materials had to be gathered locally, since importing was logistically infeasible. Most building construction occurred on the slopes of the valley, to reserve the flatter valley basin lands for agriculture. Houses were relatively small and squarish, with a few small window openings. This layout made for easier heating. Entrances were often roof openings as opposed to doorways, for security purposes in the feudal era, and they also served as outlets for the smoke of indoor fires used in winter. It can still be seen that houses on slopes are located closely together, often sharing walls. This traditional clustering was probably a natural result of the strong clan/tribal mentality of the older times. Living close together as a tribe provided security and the benefits of community living. More closely packed housing had thermal benefits, and also helped preserve precious farmland.

The sloped valley land had to be terraced into usable sites for houses. Walls and foundations were made of local stone rubble, kept together with a silt-mud mortar. Walls had a thickness of 18 to 36 inches, for stability and insulation. Older houses had alternating courses of horizontal timber beams that could be locked together at the corners with hardwood pegs from apricot trees, one of the main regional agricultural products, or notched [**Figure 1.c**]. The roof was made from timber beams and joists spanning a series of interior columns and the walls. A rough closure would be made with timber planks, or twigs and branches in poorer houses. On top of this rested a layer of *halli* (birch bark) for water proofing. Then an 8-15 inch layer of earth/silt/twigs, called *gara*, was added for insulation. The finish on the roof could be a cow dung slurry, or soil and silt, possibly mixed with apricot juice to help with waterproofing. A particular type of roof system using this construction is the Gasirkum system [**Figure 1.d**], which specifically configures wood beams in a rising series of wood squares rotated 45° at each layer, and fit into the previous square. An alternative to this, called the Faskum system [**Figure 1.e**], simply spans the roof in one direction with beams resting on opposing walls which take most of the roof load.[1]

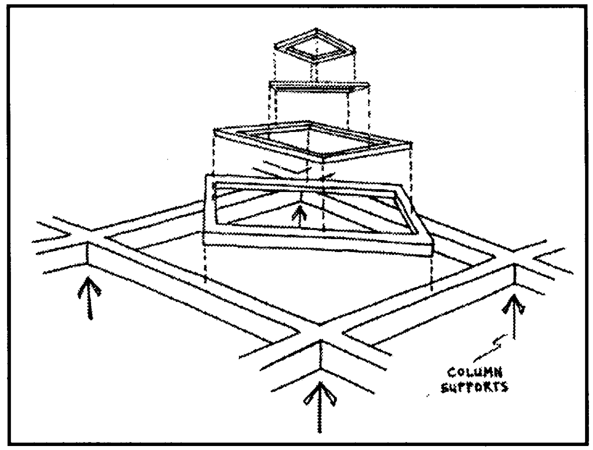

Figure 1.d Gasirkum Roof System

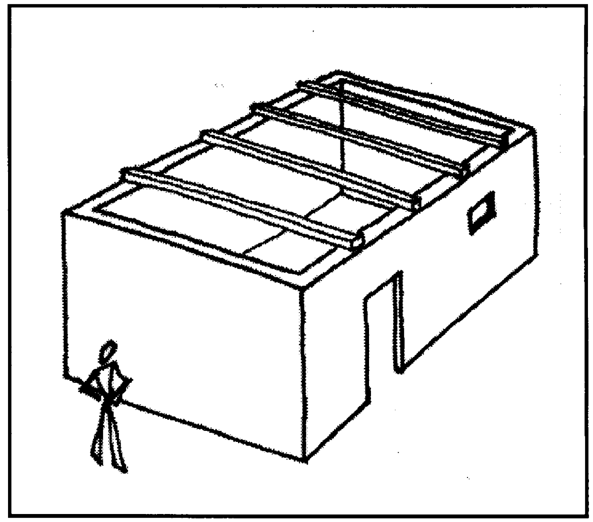

Figure 1.e Faskum Roof System (Beam System Exposed)

1. Much of this material is taken from Pirani, 1989.

Traditionally, the building of a house began by stockpiling materials years in advance, and then the whole community would participate in the building of an individual family's house. The materials would be collected, and mud mortar prepared. The timber came from the owner's farmland. The process is still similar for many people in Karimabad today.

In the broader cultural background, the major religion of the area historically, and presently, is Islam. The Aga Khan is a religious leader of a branch of Islam with a strong presence in Karimabad. In fact, the town name was relatively recently changed to Karimabad in honor of Prince Karim Aga Khan, when he made his first visit there in the 1960's.

Recent History 1.3

In the past 50 years, Karimabad and many similar villages in the Hunza Valley have experienced dramatic changes through external events. A life-style of strong self-sufficiency due to isolation was the norm, since the settlement of the town began. The dominant modes of transport through the difficult mountain terrain were horses, mules, or donkeys, and roads were not built to accommodate motor vehicles, further enforcing their isolation. The people gained their independence from the rule of Kashmir after an armed struggle in 1947, and soon after, the region became a part of Pakistan. However, the legislative framework of the Pakistani government wasn't implemented into the area until 1973. Local rule by the Mir of Hunza had been operational in the meantime. Transportation to and from the region remained very difficult until the early 1960's when the first jeepable roads were built into the region to slightly increase accessibility. During that time, the President of Pakistan made a visit, as did Prince Karim Aga Khan, and to commemorate that visit, the settlement around the Mir's residence was named Karimabad. Today, the town area has expanded to the edge of the Baltit fort settlements, and the distinction between Karimabad and the Baltit fort area has become blurred. Perhaps the strongest vehicle for change was the opening of the Karakoram Highway (KKH). In 1972 it was opened for local use, and became fully operative in 1979. The KKH extends from regions of western China all the way down to Rawalpindi, passing through the Hunza Valley along the way [**Figure 1.f**]. It is a major route of exchange for people, ideas, information, and goods. Along with this, China opened her frontier regions bordering Pakistan in

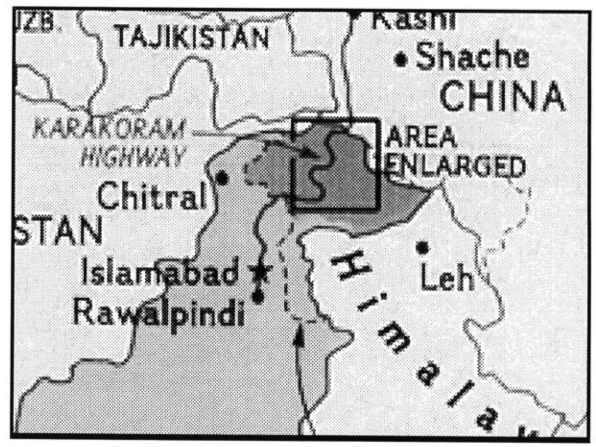

Figure 1.f Range of Karakoram Highway

1981 to non-Pakistani tourists, resulting in a flow of foreign tourists through the region traveling to Chinese Turkistan along the KKH. In 1991, nearly 40,000 foreign tourists traveled through the region. (McCarry, 1994)

Changes 1.4

Many cultural and societal changes have been brought on by these external events. Increased communication with developed parts of the country, such as Islamabad and Rawalpindi, via the KKH has made the local people aware of modern technologies and ideas. People desire to have modern things, such as flushing toilets, piped water, appliances, and new building technologies. In the past, self-sufficiency for a family was the dominant cultural attitude. To receive from one's neighbors, or to barter "was the ultimate disgrace," (Ali, 1993 p. 19) although the communal process of house construction with help from the extended family seems to have been an exception to this. The change from feudalism to capitalism, has shifted the local economy away from this highly self-sustained, independent agricultural base to a cash exchange system where goods and services are bought from outside the family. Surplus cash from tourism and increased commerce also contributes to this cultural change. Education is highly valued by the people, and many travel to outside cities for higher learning. The Aga Khan foundation has built good school facilities (primarily for women, because the state provides for male education) and health centers within the area as well. There is healthy mindset for progress and expansion, although care must be taken that progress isn't made at the expense of important and valuable tradition.

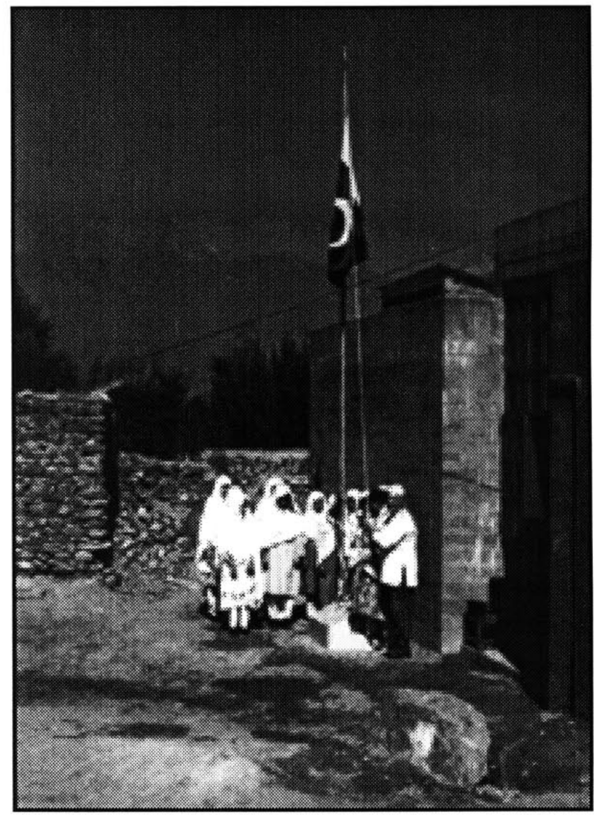

Figure 1.g Flag Raising Ceremony at Girls' School

The increased ability to travel and transport have also changed the way and quality of building. In recent years, wood has become less available due to deforestation, while new materials, particularly cement, are obtainable and perceived as desirable new technologies. In a 1992 survey of 60 buildings from various locations in Karimabad (50 residential, and 10 commercial) only 31% were built with assistance from neighbors. And for all the recently built houses, skilled laborers (masons and carpenters) were hired on a wage basis to do most of the construction, with the family providing unskilled labor. (Ali, 1993)

While many recent changes in Karimabad have been very good, such as better health standards and education, modernizing change brings many of its own problems. Within this situation, there is an increased demand for housing. It is anticipated that over the next 15 years, about 600 new houses will be built, due to rising prosperity, and the breaking of the extended/joint family system into nuclear families. In the 1993-94 school year, teams of architecture and engineering students and faculty collaborated to develop new housing designs for the town that would deal with what many perceive as problematic trends in the built environment of Karimabad.

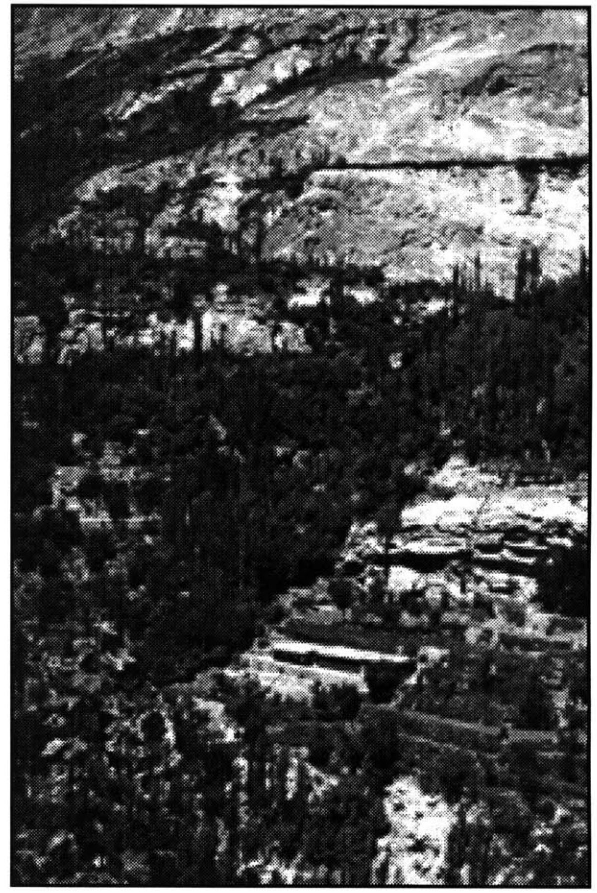

Figure 1.h View of Valley, with Houses and Greenery

The first of these trends is the switch from building houses on the slopes to siting them on the flat agricultural lands. People can afford to do this now because farming is not their only means of support. They desire to do this because, "they now require toilets; more rooms, since privacy is increasingly becoming a requirement; piped water to their houses; and proper access. They feel that in the existing conditions with small plot sizes, high densities and narrow pathways, these requirements cannot be fulfilled." (Ali, 1993 p.42) This is somewhat akin to "urban sprawl," on a smaller scale, and it jeopardizes the beauty of the agricultural and traditionally built environment.This beautiful environment is a major resource of the region because it attracts tourist commerce.

The compatibility of new materials with the traditional environment was also a concern. Many institutional buildings, including hotels, are "massive cement concrete structures that stand out in contrast to the traditional environment in terms of scale and texture. This creates severe environmental problems for the area as a whole." (Ali, 1993 p.54) New materials such as concrete (if reinforced and placed properly) have benefits such as seismic resistance and durability, and it would be inappropriate (and ineffective!) for outsiders to suggest that the people of Karimabad avoid building with any new materials merely to preserve their beautiful "look." However, as a design team familiar with the use of these new materials, and convinced of the merit of preserving Karimabad's agrarian beauty, we aspired to produce designs that would use a combination of new and traditional materials in an appropriate and progressive way.

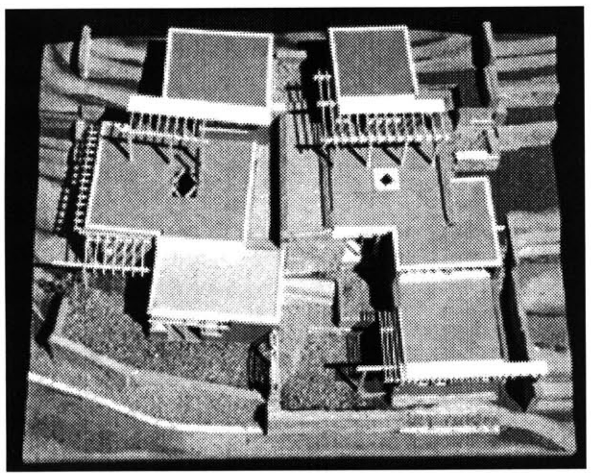

Figure 1.i Two Houses in a Clustered Arrangement (Model by Shaun Roth)

The design team promoted the concept of clustered housing [**Figure 1.i**], sited on the slopes, for its efficient use of space and other resources. Our intent was not to dictate the parameters of future construction, but to provide ideas for improvement through our illustrative designs, and aid the people as the region modernizes. There are hopeful signs that the citizens and leaders of Karimabad are becoming well informed to make their own good decisions to guide the town development. A less emphasized issue in the scope of this thesis, but nonetheless important to the overall project, was the cultural impact of changing spaces and layouts of the house. Designers had to understand the traditional centrality of the Ha, and design new layouts with kitchens, bedrooms, and openings etc. with sensitivity to the cultural context.

Engineering Involvement 1.6

The engineers involved with this project had a more specific role within the overall design objectives. *The goal of our involvement with the architectural designers was to provide them with technical knowledge and ideas that could be implemented in their designs in effective and responsible ways.* The search for appropriate technology was a burden every team member shared, however it was up to the engineering people to assess the technical effectiveness of the various ideas, and to make quantitative comparisons of alternatives.

Students and faculty with mechanical engineering skills investigated heating/cooling and insulation issues, which comprises a separate thesis in itself. On the structural engineering side we had to deal with the following issues of environment, usage, and safety:

Earthquake Loads
One of the first and most pressing issues results from the lack of timber in the region. Wood is expensive, and what can be afforded is mostly used to construct the roof spanning system. This leaves walls composed totally of stone and mud mortar, which has almost no ability to withstand tension forces. Because the Northern Areas are seismically active, these houses are potential death traps in the event of moderate to heavy earthquakes. One of the most recent earthquakes in the Northern Areas occurred in Patan on December 28, 1974. A UNESCO report states that nearly 700 people were killed, with about 4,000 injured. The most severe damage occurred in homes where tradi-

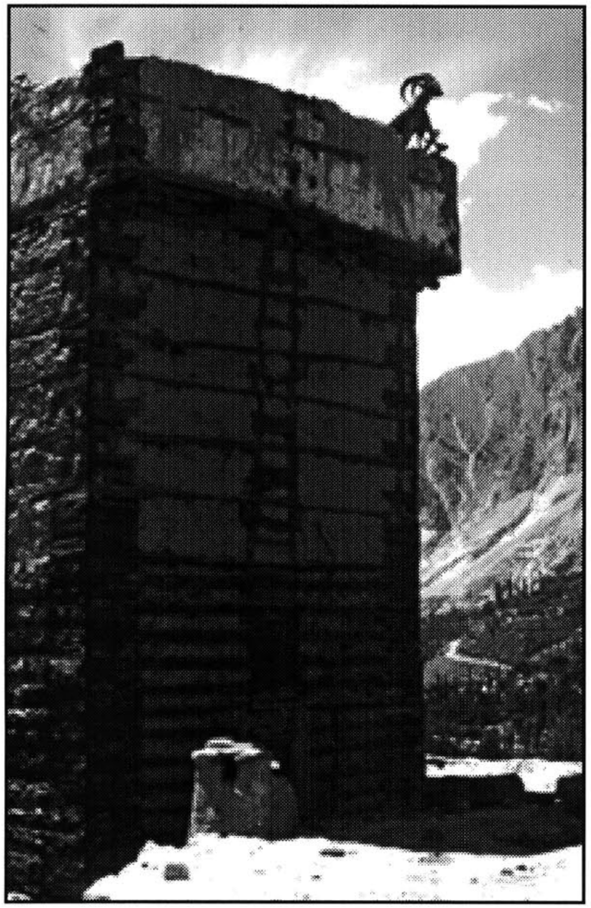

Figure 1.j Shingri Tower with Wood Cribbing Construction

tional stone and mud mortar masonry walls provided the primary roof support (i.e. the Faskum roof system). Houses where the walls were additionally supported by interior columns, as constructed in the Gasirkum system, suffered less damage, especially in terms of roof collapse. An important thing to note, is that the tall Shingri towers [**Figure 1.j**], that are usually parts of palaces or forts, which were built with the timber cribbing system up to heights of 45 feet did not suffer any noticeable damage. (Pirani, 1989 p.114-115) Therefore, the absence of wood cribbing in more recent construction removes the most effective means of earthquake load resistance traditionally available to buildings. Poplar trees were introduced into the region by the British decades ago, but the wood is still a precious commodity, and structural use of it must compete with the need for firewood.[1] With scarce timber resources, we had to look for an alternative way to take the tension forces developed in a wall due to lateral loads.

Problems with New Technology and Materials

There is less familiarity with the new techonologies of cement and block construction, but people still want to build with these materials because of their progressive appeal. Some recent structures of new materials may well be less safe than traditional constructions, especially the timber cribbing systems. It is ironic that reinforcing steel, which is a standard means of seismic resistance in most developed countries, is not as appreciated for two reasons: 1) an earthquake has not occurred in Karimabad within

1. Recently, a contact in Karimabad informed us that 24 hour electricity is now being provided through working turbine generators, and electric heaters are gaining use. This may make more timber available for construction in the future.

recent memory (though this doesn't guarantee anything about future events, see Section 2.1.5 concerning seismic assessment of the region) and 2) reinforcing steel is not a visible part of the structure, and doesn't contribute to a finished image of modernity. Walls are also being built thinner now, in the range of 12 to 15 inches, resulting in less seismic stability. Any construction technology we suggested had to be affordable and relatively simple, in order to facilitate an effective communication of knowledge, and especially to increase the chances of correct implementation.

Figure 1.k Recent Concrete Block Construction (photo taken Aug. 1994)

Structural engineers had to take into account some additional factors and how they would affect design. One major factor was *heavy roof loading*. Because most houses are built on the valley slope, people take advantage of the level space provided by flat roofs.Houses had to be able to support substantial roof live loads that could occur during family/community gatherings, or even from livestock. Dead loads are also substantial, due to the materials and techniques of construction. The *weathering* of construction was also considered, since the climate is very dry with the exception for a few days of “wet season” where the rain is intense. This cycle deteriorates construction through moisture damage. Methods to stop this cycle were searched out. Finally, issues of cutting and filling for retaining walls, stability, and *general soil conditions* were examined and understood to the extent possible.

Scope of Thesis 1.7

This thesis will focus on two related areas:

A report by this author of the proposed engineering design for houses in the region.
Various sources were researched concerning appropriate technologies, construction techniques, methods of analysis, etc. in order to reach an optimal design for houses in this region. The engineering of one particular house plan will be presented to illustrate how to analyze and structurally design houses built with an appropriate technology for the region, namely reinforced stone masonry. Effective construction techniques for realizing structural principles of seismic resistance will also be presented.

An examination and discussion of the role that civil/structural engineers played as part of the team involved in this project.
The effectiveness of interaction and communication with architectural designers to produce a strong, comprehensive design will comprise much of this section. The transfer of technical ideas across differing cultures is also discussed. The overall purpose of this section is offer suggestions and pathways for improving the teamwork between structural engineers and the other disciplines involved in building design, with an emphasis on architectural designers.

SECTION 2

TECHNICAL REPORT

"As for engineering, let us conclude once and for all that it is a blend of art, craft, and science." - Samuel Florman, <u>The Civilized Engineer</u>

Introduction 2.0

This section discusses the structural issues that had to be resolved in our housing designs for the region. One of the major issues was earthquake loading, and we had to develop an understanding of how earthquakes affected the masonry buildings typical of Karimabad. After taking stock of the structural challenges, appropriate materials and technologies were selected for the housing designs. Some of the major alternatives investigated are presented here, along with the reasoning behind our final choices. Procedures for analysis and design of structural elements, mainly walls, were also developed, and a final design example employing these procedures is included for illustration.

Structural Loads and Issues 2.1

Roof Loading 2.1.1

Because houses were traditionally built on slopes, where horizontal space is precious, families use the area provided by flat roofs. This must be taken into consideration when estimating live loads on the roof. Sometimes livestock may be on top of the house, but the more common problem is overloading during large festive gatherings, such as weddings. The structural design procedure outlined in this report allows for input of variable roof live and dead loads.

Water Damage 2.1.2

Although there is very little rainfall per year, it tends to be concentrated into a heavy rainfall that continues for a few days. This yearly cycle deteriorates the traditional method of dry construction. One researcher writes:

> A major problem found when surveying old Hunza houses is that of the soil/wood interaction particularly between the main roof beams and the soil membrane above... When subject to rain the soil moisture rapidly increases and then quickly evaporates but some is transferred to the wood. Upon drying out again the soil is able to help suck out moisture from the wood to below its natural level, so desiccating it. Because of this cyclic moisture regime and due to the nature of soil it becomes an ideal host for small animals and micro organisms. Under suitable temperature and moisture conditions these biological eco systems will attack the wood, a good food supply, and hence cause decay. The micro organisms can lie dormant in the soil or wood for many years, until the right conditions prevail. (Hughes, 1987)

Some practical construction guidelines to prevent water damage were found in the literature researched.

Soil Conditions 2.1.3

The local soil is a silt and gravel "murrain" which has good bearing strength when dry, but it becomes minimal when wet. Rains and even watering from the fields can cause slides and differential settlements. Foundation and retaining wall designs had to account for this issue.

Earthquakes 2.1.4

The Northern areas of Pakistan are close to the intersection of the Indo-Australian plate, and the Eurasian plate. The two collided an estimated 40 million years ago, and are currently still converging at a rate of about 50 mm/yr. The Himalaya, Pamir, and Hindukush mountains are a result of the crustal thickening due to this collision. The seismicity of the Northern areas is comparable to that of the Western U.S. and the Philippines, with the underthrusting of the Indo-Australian plate to the Eurasian plate being the primary source of earthquakes. (Ove Arup, 1991)

Seismic Risk in Karimabad 2.1.5

Assessing the seismicity for the specific area of Karimabad is a difficult task. Instrumental data of the general region only covers about 100 years, which is a short time compared to the return period of large events. It is held by some people that the town may not be susceptible to large earthquakes. Villagers interviewed in 1988 recall feeling tremors in July of the previous year, and also the year before. (Pirani, 1989) However, no serious damage is mentioned. However, human memory which spans only a couple of generations, is also short compared to the recurrence interval of large earthquakes. Ove Arup and Partners has documented the process they used to assess the seismicity of Chitral and Gilgit, two towns in the surrounding region, enclosed by a 67.5 - 77.5 °E and 33 - 39° N window. In their 1991 report they mention that two approaches could have been used:

The *deterministic approach* involves rationally choosing a "design earthquake" to occur at a given distance from the site. The ground motion parameters (acceleration, intensity etc.) are related to the distance from the site to define the on site ground motion. This alternative was not used because of the uncertainty in predicting the distance of an earthquake from any specific site in that region. Apparently, there are too many epicentral possibilities.

The *probabilistic approach* is based on statistical procedures which consider:

- Locations of seismic source zones relative to a site
- Source parameters: such as recurrence, maximum magnitude, and hypocentral depth. The Gutenberg-Richter recurrence relationship is considered in this.
- Ground motion attenuation - how site motion is affected by distance from the earthquake source
- Hazard curves of frequency of exceedence vs. ground motion levels

Using this approach, it was concluded that both Chitral and Gilgit have Expected Peak Accelerations (EPA) of 0.4 g. It was also suggested that this EPA is appropriate for the entire region bounded by 67.5 - 77.5 °E and 33 - 39° N, which includes Karimabad. (Ove Arup, 1991, 2.4)

In Coburn, 1981, a simpler approach was used to assess a similar region, bounded by 71-76°E and 33-38°N using seismic data from 1914 onward. A plot of the number of events (N) on a logarithmic scale vs. Richter magnitude roughly gave a straight line which suggested a relationship for predicting the recurrence interval of an earthquake of a given magnitude within the region. However, the distribution of earthquakes within the region is not uniform, and combined with the relative shortness of the data set, the Figures are very tentative. It was also noted that the bounded region is very large, and when the approximate radii of damage from prior earthquakes was considered, only 0.1 to 0.2% of the area bounded was predicted to experience damage by earthquake within a one year span. The return period for a damaging earthquake at any one particular site in the region fell in the order of 1000 years. For perceptible earthquakes, the return period was in the 3-5 year range.

Basic Seismic Resistance Concepts 2.2

Terminology 2.2.1

For a clearer understanding of the concepts to be presented, some basic terms are defined here:

- *lateral acceleration force*: force caused by horizontal ground acceleration during an earthquake
- *parallel wall*: a wall whose length is parallel to the direction of ground movement/lateral acceleration
- *perpendicular wall*: a wall whose length is perpendicular to the direction of ground movement/lateral acceleration
- *shear walls*: solid wall segments with the capacity to resist lateral forces when they are loaded as parallel walls
- *out of plane bending*: bending forces that occur in a wall when it is loaded as a perpendicular wall

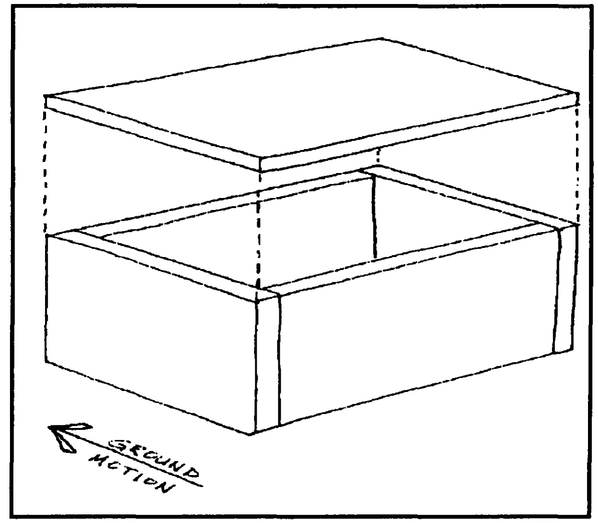

Figure 2.a Solid wall plane model

Overview of how Earthquakes Damage Houses 2.2.2

In order to examine and illustrate how earthquakes damage houses, it is useful to consider a very simple structure, such as a box [**Figure 2.a**], and see how lateral acceleration affects it. Assume that the box is made solid planes for the walls and ceiling and that the planes have substantial mass, and therefore inertia, much like the traditional stone rubble houses in Karimabad. When this box is subject to a lateral acceleration force (vertical motion is assumed to be negligible) the wall planes can react in several ways, depending on how they are connected to one another at the edges.

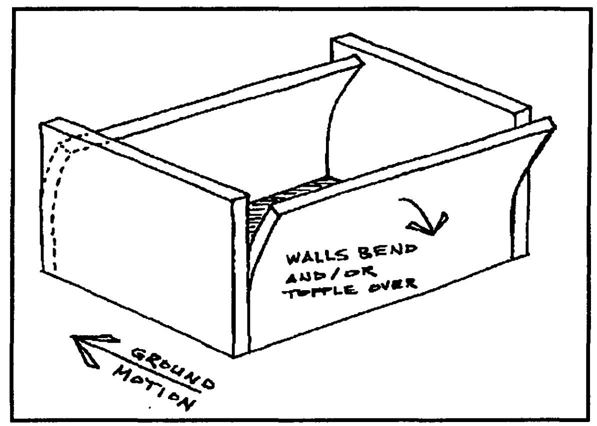

Figure 2.b Wall bending and toppling

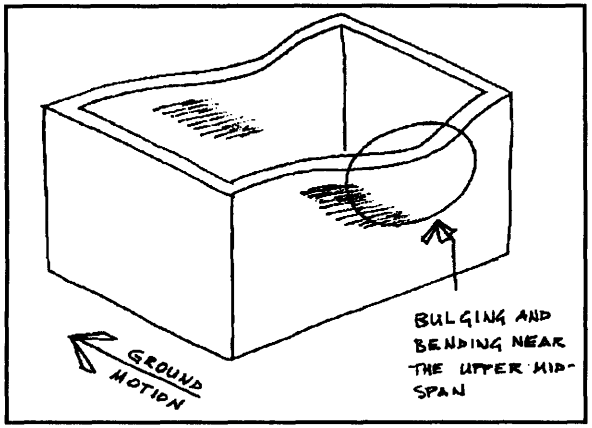

Figure 2.c Out of plane bending

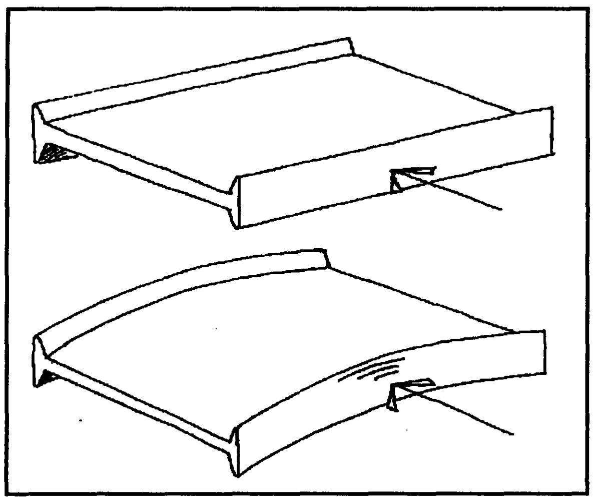
Figure 2.d Flexible roof deformation analogy

To begin simply, the roof plane will be detached. If the wall planes are not connected together, walls that are oriented perpendicular to the lateral acceleration will tend to fall over. Even if the walls are "fixed" into the ground with a foundation of some depth, stone walls have no capacity for the tension caused by out of plane bending [**Figure 2.b**].

Supposing that the walls are joined together at the edges, the parallel walls will help resist some of the overturning forces in the perpendicular walls by means of the edge connections and their (parallel walls) own shear resistance. The bending would look as shown in **Figure 2.c**.

When the roof is reattached in the model, it can either help resist out of plane bending or exacerbate the problem, depending on its rigidity.

A flexible roof plane or diaphragm will deform laterally because of its own inertia, during horizontal acceleration, like a large beam on its side [**Figure 2.d**]. For instance, a flexible diaphragm supported by two shear walls will deform as shown in **Figure 2.e**. (Perpendicular walls do relatively little to resist or affect diaphragm deformation)

A rigid roof diaphragm will not deform, and it transfers its inertial loads into the stiffer elements that support it against lateral movement (shear walls) [**Figure 2.f**]

When the actions of the roof and walls are considered together in the case of a flexible diaphragm, the weight of the roof is transferred to the top of the perpendicular walls and increases the out of plane bending force on them. **[Figure 2.g]**

A rigid diaphragm will hold the upper layer of the perpendicular walls in place and effectively transfer much of the out of plane forces into the parallel shear walls. This is the ideal structural behavior for buildings made with heavy plane elements.

Some observations can be made at this point, from the model examined:

- The edge connections between different planes should be strong and as continuous as possible. Corners and edge connections are places of high stress concentrations.
- Wall planes that are parallel to the lateral loading provide the primary resistance to lateral forces, so they must have the capacity to take shear forces. Because an earthquake's shaking direction is unpredictable, all structural wall planes must be able to take shear forces.
- Wall planes that are perpendicular to lateral acceleration must have some capability to sustain tension forces due to out of plane bending. Areas that are freer to deflect out of plane, such as midspan areas, and unconnected or less restrained edges are critical areas where tension forces will be greatest.

The behavior predicted by this model is evident in the earthquake damage patterns observed in actual unreinforced stone/adobe houses in areas with similar construction to Karimabad. Vertical cracks due to out of plane bending appear on the faces of walls perpendicular to the direction of ground motion **[Figure 2.h]**. Roofs are most likely flexible diaphragms, because construction of rigid roofs is either not understood or not affordable, and

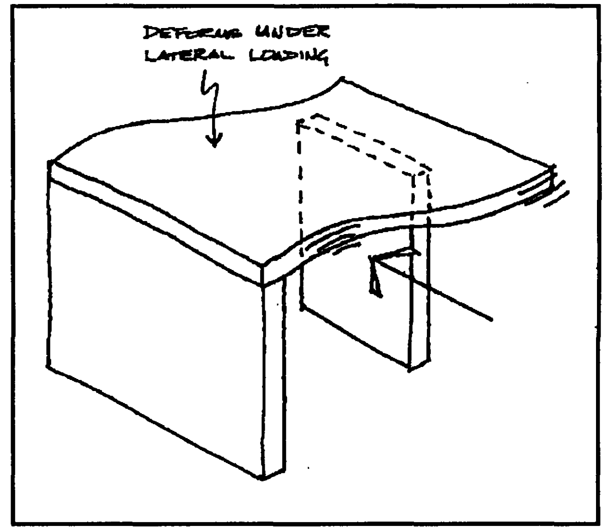

Figure 2.e Flexible roof diaphragm

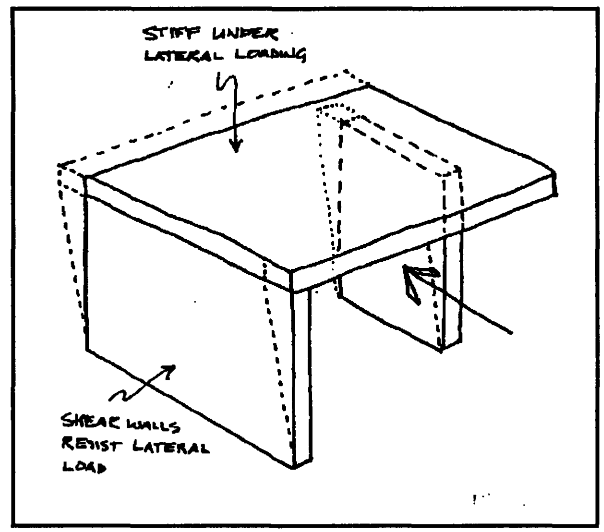

Figure 2.f Rigid diaphragm

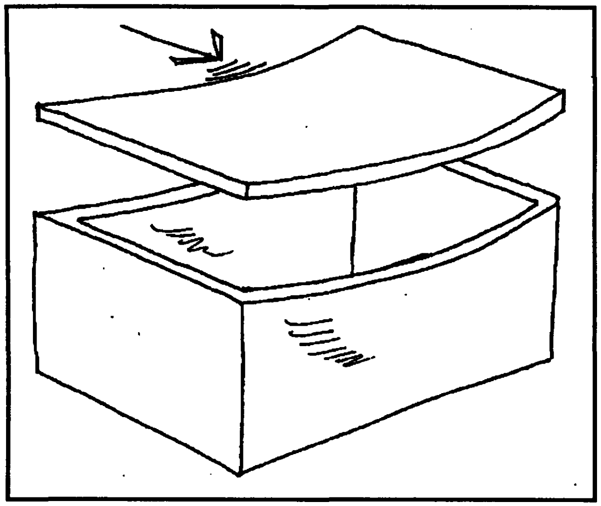

Figure 2.g Roof bending affects walls

the added lateral force from the roof often collapses the walls that are already loaded out of plane from self-weight. If the roof was being supported by these walls, partial or complete collapse of the roof follows [**Figure 2.i**]. Diagonal tension cracks also appear in walls loaded in shear. If shaking is persistent, stones or blocks begin to fall out and reduce the wall's capacity to resist shear. This allows more deformation and damage to happen to the building as a whole. Shear walls may also collapse after too much cracking and material loss occurs.

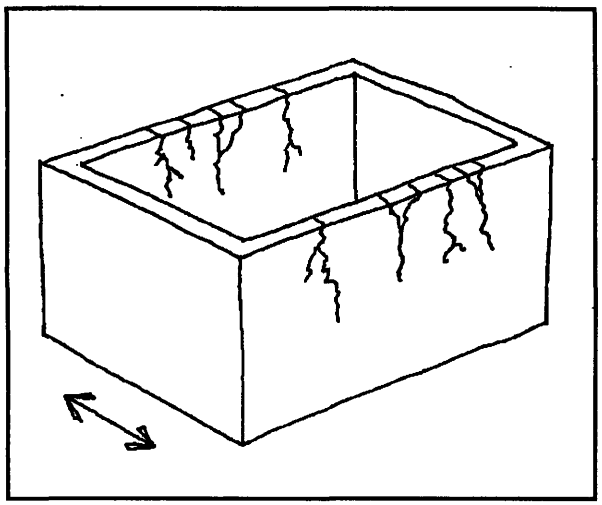

Figure 2.h Vertical cracking

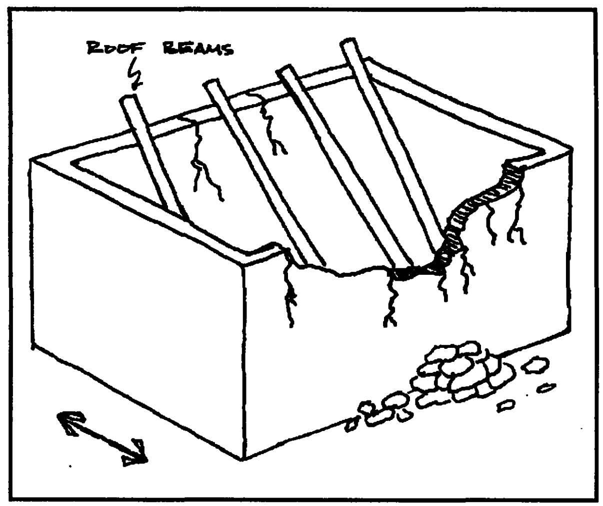

Figure 2.i Partial wall and roof collapse due to out of plane bending

Appropriate Materials & Technology 2.3

The concepts and decisions in this section developed over the span of a year and a half, with the input and effects of working together with a team of architects and other engineers. For an outline of project events, see Section 3.

Basic Materials Selection 2.3.1

Before any design calculations could even begin, we had to determine the best materials and technology for the area. There were 3 main criteria:

- *Health and safety for occupants* - This involves seismic resistance, and thermal insulation.
- *Compatibility with the existing built environment* - As mentioned in Section 1.5, building materials should be selected with consideration towards the traditional beauty of the town.
- *Affordability and constructibility* - It's a stark reality that not everyone will be able to afford houses that are even modestly built according to the guidelines described in this paper, but the aim was to make safe and better housing available to as many as possible. There was also an effort to select construction technologies that are related to existing techniques, and more likely to be understood and implemented by local builders.

The third criteria above narrowed down the range of choices considerably. There are many existing technologies for earthquake resistance and insulation, such as the common construction used in the U.S. of wood studs, dry wall, and batt insulation. However, for the base building materials (the materials which substantiate the walls and roofs), it was clear that locally available materials would be the best option for keeping costs reasonable. The cost of transporting materials from outside regions to Karimabad is still quite substantial, even with the Karakoram highway operational, so the base materials which make up the majority of a building by volume, should be found locally. (Locally available is defined here as within the area of Karimabad, or from neighboring towns.) Those materials were stone, either rubble or finished, concrete block which could be produced locally, silt/soil, and wood in limited quantities.

After taking stock of the locally available material we had to make a decision to use either stone or concrete blocks. Economically, it was difficult to determine which was really the more expensive option. Cost analyses done by other parties seemed to give conflicting results, but the relative expense really turns out to be based on the quality of products that are compared. (Pirani, 1989 p 129) For instance, fully hammer-dressed stone would be more expensive than concrete block, especially if little cement is used in the block mix. However, that would not be the case for rough cut stone vs. high quality concrete block. Some important factors made stone more favorable. The first is that stone matches the traditional built environment very well, and as mentioned the inappropriate use of concrete block construction is already deteriorating the appearance of Karimabad. Secondly, stone is the material most likely to be stockpiled by a family as it prepares to build a house (as mentioned in the introduction, stockpiling can be a process lasting for years) because it's readily available. And one more important consideration: most of the costs involved with stone production are in labor, almost all of the money paid for this material is circulated back into the local economy. The cost of cement is a large part of using concrete block, and is imported from farther regions such as China, so the money used to buy cement exits the local economy, which is a less preferable situation.

For roofing material, using the local timber had the more complicated issues of environmental preservation and higher cost associated with its use. However, for basic structural design, all that needed to be specified was the spanning system (main structural beams), and how it would connect to the walls. The fill or covering was a more important issue in terms of its self weight (and for insulation purposes) which was left as a variable input in the engineering design process. Major spanning systems could have been wood beams, concrete beams, or steel decking/joists. A concrete T-beam system was developed by Ove Arup for the Self-help School program, but that system would still be beyond the affordable and constructible range for most families. The cost of steel in general is high, and steel joists would definitely be out of range for the average Karimabad household. Steel itself is an expensive but necessary component for wall reinforcement in our designs, so money would be better spent for reinforcing steel as opposed to using it to span roofs.

The designs presented here utilize stone masonry as the basic material for the walls and foundations, and timber beams as the main spanning system for the roof. Suggestions are made for infill material for the roof, and for overall finished construction; however, further work should be done on roofing options.

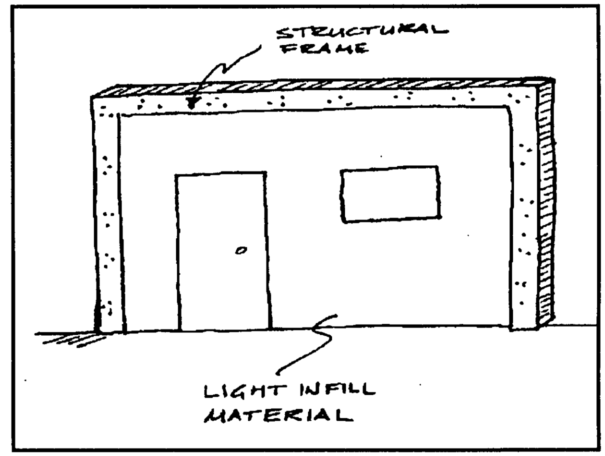

Figure 2.j Moment resistant frame system

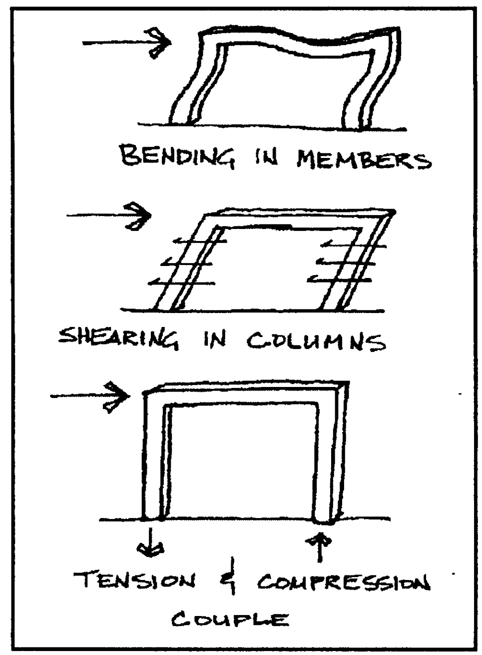

Figure 2.k Frame action

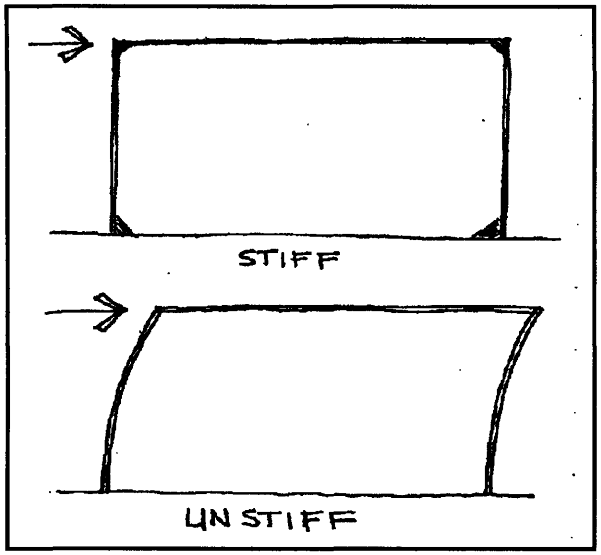

Figure 2.l Corner stiffness in frames

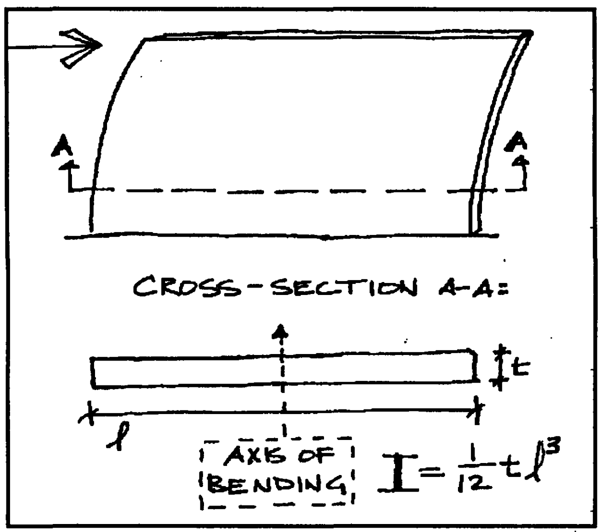

Figure 2.m Shear wall: deep beam action

Wall Systems 2.3.2

The most important function for walls structurally, is to resist the shear forces due to self weight, and those transferred from other parts of the building. A building relies on the stiffness of its walls in their lengthwise direction for support under lateral loads. Two basic conceptual systems were considered: *moment resistant frames*, and *shear walls*.

Moment resistant frames function as the name implies. Lateral stiffness is provided by an outer edge frame, and the rest of the wall is non-structural infill [**Figure 2.j**]. Frames resist lateral forces by a combination of shearing and bending forces in the walls, and a compressive-tensile axial force couple in the columns [**Figure 2.k**]. The rigidity of corner connections is crucial and should be as stiff as possible, otherwise the frame allows too much deformation [**Figure 2.l**]. Corners of frames can be made stiff by some sort of diagonal bracing, or adding material

Shear walls are solid planes which resist lateral forces through their own geometry. They behave like deep beams, taking advantage of the large moment of inertia when loaded parallel to their length [**Figure 2.m**]. Besides resisting lateral forces by shear and bending, the self weight of a shear wall also creates a moment resistive couple with the far edge of the wall [**Figure 2.n**].

Contrast and Comparison
Concerning frames:

- The frame can be infilled with almost anything desired. The big advantage is that one can use a very light, non-structural material to infill, and these materials can be selected for good

thermal insulation.
- They can allow walls to be easily removed for future expandability.
- There is more flexibility in the placement and amount of openings a wall can have.
- Moment resistant frames are more difficult to construct. Skills are needed for making formwork, mixing and casting concrete, and most importantly in the fabrication and placement of rebar for stiff joints.

Concerning shear walls:
- If the base building material for the walls will be stone, it would be efficient to take advantage of the natural ability of a solid stone wall to function as a shear wall.
- Construction for this type of wall is relatively simple compared to a moment resistant frame.
- They restrict the amount of openings a wall can have.

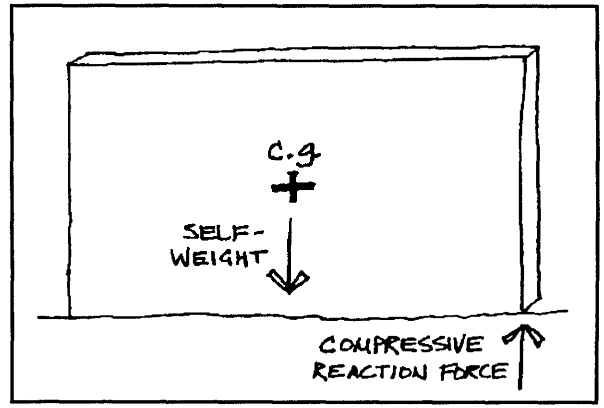

Figure 2.n Overturn resistive couple of a shear wall

Trial calculations were performed for one particular housing design created in the Fall 1993 workshop which employed a frame system. Moments in the frame were so high, that the cross sectional area of rebar required sometimes exceeded one square inch, which is the size of a #9 bar (see Appendix B). Tentative figures for necessary shear wall reinforcing steel at that time were considerably lower, and allowed for use of #4 or smaller rebar. Any sizes above #4 are difficult or impossible to bend and work without special tools.

Another major problem with using a frame system was the lack of suitable material in Karimabad to create light infill. Wood would have been an ideal choice for this; however, the cost is pro-

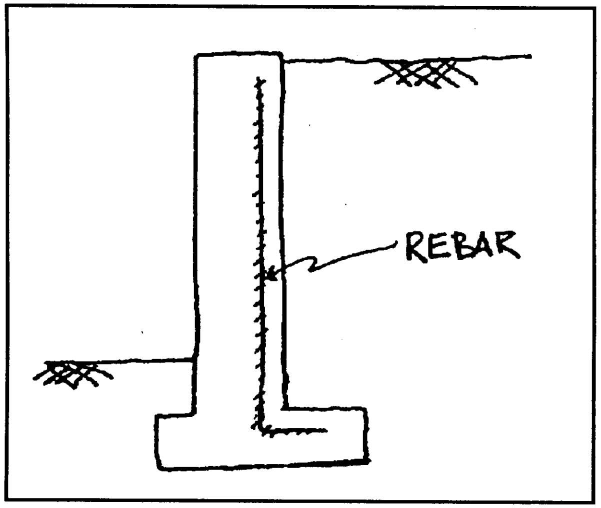

Figure 2.o Simple retaining wall (section)

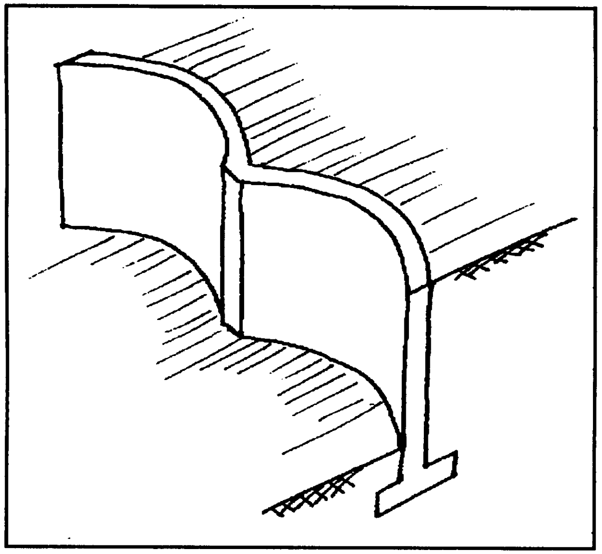

Figure 2.p Arched retaining wall

hibitive, especially when added on to the already expensive cost of cement, labor, and steel for the frame. Infill material would most likely have to be stone, or some sort of packed soil (which in itself can reach over 100 pcf). This infill is heavy, and must be reinforced against its own weight, and this brings the frame wall close to becoming a solid shear wall.

In traditional construction, openings were few and small, making the walls safer under seismic loads. However, it seems that many Karimabad residents would like more and larger openings in their new houses, which could pose seismic problems. After some study, we concluded that adding larger openings with reasonable placing would not cause problems in the shear strength of a wall.

It was decided that a shear wall system would be the most feasible and sensible for construction. Guidelines were researched for careful placement of openings, and efficient use of reinforcing steel within the shear wall.

Retaining Wall Systems 2.3.3

Designs ideas in the Spring 94 semester workshop required investigation into retaining wall systems.Houses sited on slopes can be built using the retaining wall as the house wall for one side. Some thermal benefit was found in this configuration, and possibilities for taking advantage of the retaining wall situation for seismic resistance were investigated.

Types of Walls Investigated

- Simple retaining walls [**Figure 2.o**] are the standard in many types of construction. They were the first and most obvious to consider.

- Arched retaining walls [**Figure 2.p**] use their geometry to provide the necessary stiffness to resist earth pressures. It was anticipated that steel would not be needed in this type of wall because all forces would be resolved in compression.
- Tie-back retaining walls [**Figure 2.q**] take advantage of the earth behind them to restrain the lateral movement of any elements well anchored into the earth (against lateral movement perpendicular to the retaining wall).

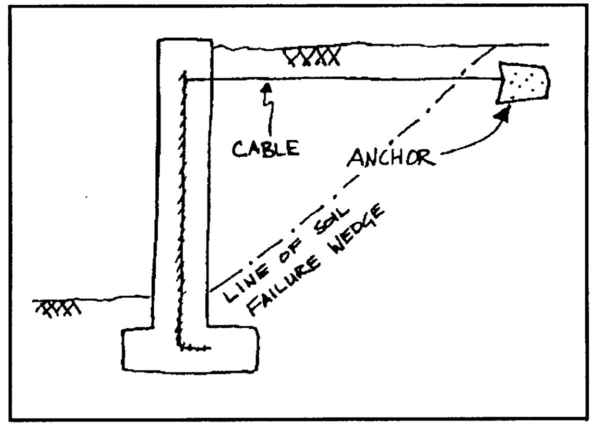

Figure 2.q Tie-back retaining wall (section)

Reinforcing requirements for each different retaining wall system were compared, using calculations based on the Mononobe-Okabe equations, which are standard equations used to design retaining walls against seismic loads. See Appendix C and Section 2.4.3 for a complete explanation of the processes.

Material Values

For comparison purposes, these material values were used for the calculations of all 3 types of retaining walls:

- unit weight of soil = 100 pcf
- weight of stone = 130 pcf
- steel yield strength = 36 ksi
- equivalent fluid weight of backfill soil = 42 pcf
- weight of steel = 490 pcf

For each system, the *test case wall* had dimensions of 2 ft. thickness, 10 ft. height, and 15 ft. length. Footing steel was not included in the comparison. Also, a common lateral acceleration of 0.2 g was used.

Table 1: Steel Required in 2 ft. Thick Simple Retaining Wall per 4 ft. of Wall Length

Height (ft.)	Area of steel (in2)	Rebar
1	0.00	-
2	0.00	-
3	0.00	-
4	0.00	-
5	0.04	1 X #3
6	0.08	1 X #3
7	0.20	1 X #4
8	0.32	2 X #4
9	0.48	3 X #4
10	0.68	4 X #4

Simple Retaining Walls

In these, the most straightforward of all retaining wall systems, the amount of reinforcing steel needed is mainly a function of height.

Some notes on **Table 1**:

- In quantifying the amount of steel needed, 4 ft. of linear wall length was used as just an arbitrary unit.
- Two measures of the steel required are given for each height - the cross sectional area, and the quantity and size of rebar. For instance, for a retaining wall 8 feet high, the cross sectional area of steel required is 0.32 in^2, which can be provided by two #4 bars (needed every 4 feet of wall length).

The total amount of steel needed in the test case wall was 100 lbs (or 15 #4 bars, each 10 ft. in length).

Arched Retaining Walls

These retaining walls take advantage of a special geometry to resist lateral earth forces. The arches must have a catenary shape, the shape that would result if a cable was hung between two supports. This ensures that all parts of the arch are in compression, and no internal moments exist.

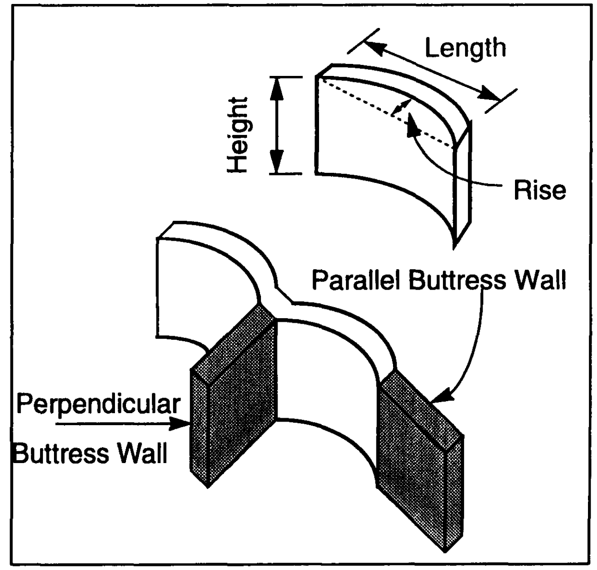

Figure 2.r Arched Retaining Wall Terms

The resistive capacity of an arched retaining walls depends on its geometry, and the presence of adequately sized buttress walls. Buttress walls resist the thrusts developed at the endpoints of the arches. **Figure 2.r** shows the geometric properties that must be considered, and defines the necessary perpendicular and parallel buttress walls. Note that, although not shown on the figure, perpendicular walls are needed at each end of an arch, and parallel walls must be provided at each end of a series of arches.

Some notes on **Table 2**:

- The test case wall dimensions are used. Height = 10 ft. Length = 15 ft. Thickness = 2 ft. Rise = 3 ft.
- It seems that an arched retaining wall that is 7 ft. or shorter in height requires no buttressing walls, which is counter intuitive because one would think the arches would always need parallel walls to counter lateral thrust. Due to self weight and geometry, the arched wall has a very high resistance to overturning. Buttress walls were not calculated until the overturning moment due to lateral acceleration became greater than the resistive moment due to self weight. See Appendix C for the full calculation process. It is suggested that if arched retaining walls *shorter* than 7 ft. are used, some parallel buttressing should still be provided because it is not clear how well the resistive moment due to self weight could

Table 2: Length of Buttress Walls at Each End (L = 15 ft. H = 10 ft., Rise = 3 ft.)

Height (ft.)	Perpendicular Wall (ft.)	Parallel Wall (ft.)
6	0.00	0.00
7	0.00	0.00
8	4.75	4.25
9	7.75	7.00
10	10.0	9.00

resist the lateral thrusts that should be taken by parallel buttresses.

- At a point where two arches come together, as shown in **Figure 2.r**, the required perpendicular buttress wall would be twice as long, because two arches are now exerting thrust forces.
- The requirements for the test case wall were a perpendicular buttress 10 ft. long, and a parallel buttress 9 ft. long (at each end of an arch section).

Tie-back Retaining Walls

The advantage of this system is the potential for using the earth behind the wall as a stable mass, and embed an anchor into it. The anchor system is the tie-back cable, attached to a fixed mass placed outside the soil failure wedge [see **Figure 2.q**]. In the event of lateral acceleration in a direction perpendicular to the face of the retaining wall, this tie-back could greatly reduce the swaying movement of the structure, which is the main cause of damage in an earthquake.

Because the calculations for a tie-back retaining wall are more involved, only the test case was quantified, unlike the other systems where a spreadsheet was easily developed to calculate the steel for a range of heights. For this calculation, the assumed failure angle of the soil was 30°.

The required amount of reinforcing steel for the test case wall, including the steel for the tie-back cable, was 107 lbs. It should be noted for this wall system that one #4 bar would be required per linear foot, the same requirement as the simple retaining wall. This also requires that a tie-back be constructed at regular intervals along the wall, which could be costly and time consuming.

Comparison

Table 3 gives a summary of results for comparison. Again these results apply to the test case dimensions mentioned earlier. (One should note that for retaining walls shorter than 7 ft. in height, the arched system may be a very good way of saving steel.)

A "best" option for the type of retaining wall that should be used was not as clear as it was in the case of wall systems. A general decision that was made for the design example in this thesis, was to use the simple retaining wall system, for the following reasons:

- The simple system is the more easily constructed than the tie-back system, and uses about the same amount of steel.
- The main benefit of the tie-back system, which would be the bracing of roof loads against lateral motion, would really work in only one direction of lateral movement.
- Arched retaining walls do save steel, but they are restrictive on the layout of the house, and by the time all the work was completed on the retaining wall investigation, most designs had pretty well established layouts, and it would have set back the design process to have the architects retrofit everything to allow for buttressing walls. In addition, the buttress walls themselves have to be reinforced against earthquake loads.

The retaining wall decision was not as easily and concisely decided as implied by this section. Please refer to Section 3, where a more complete record of the non-technical factors affecting this decision is given.

Table 3: Requirements for Different Retaining Wall Types (Test Case)

Type	Requirements
Simple	100 lbs. of steel
Arched	Perpendicular buttress wall: 10 ft. Parallel buttress wall: 9 ft. (at each end of an arch section)
Tie-back	107 lbs. of steel

Before actual construction of these houses begins, a final look should be given to the feasibility of the alternative retaining wall systems from a local standpoint to more clearly assess the relative constructibility. In particular, there is a possibility that the height of earth retained will be lower on the actual sites used than in the example case here, and in that case the arched retaining walls may be a better option.

Roofing 2.3.4

The main concern for roofs was to find a rigid system that was well connected to walls in order to transfer forces to the shear walls. This is the least specifically developed of all the structural elements in this report because the main focus of our initial engineering design efforts was the walls. The roofing system did not have to be fully specified for the seismic design of the walls.

Two Traditional Systems Examined
The Gasirkum system employs sets of main beams spanning in two directions, supported by interior columns at the beam intersections. A skylight in the center of the roof is created by a 45° rotation of successively smaller squares made from timber [see **Figure 1.d**]. This system has worked better than the alternative Faskum system because the internal columns provide the roof with extra support, and in the case of partial wall collapse during earthquakes, roofs were still standing. (Coburn,1981, p. 242-243)

The Faskum system [see **Figure 1.e**] is a set of beams spanning between walls in only one direction. If the lateral acceleration is in the same direction as the beam alignment, this system is more dangerous under lateral loads because the lateral force due to the roof self weight is almost totally transferred into the perpendicular walls to cause out of plane bending.

Vaulted Roofs
In the fall of 1993, Mike McCormick, a fellow Building Technology/Structures student investigated the possibility of using vaulted roofs. The appeal of this roof system was the possibility of saving wood, because only one set of main wood beams arranged to form large squares would be needed. The smaller wooden joist necessary in the traditional systems would be replaced by vaults, made of masonry or concrete, which act essentially like shallow arch domes, resolving all of their loads in compression. Mike's original work for this is included in Appendix D.

Roof Isolation
An altogether different approach could be used, involving the principle of roof isolation. The concept is to allow the roof to move laterally in a more independent manner from the walls [**Figure 2.r**]. Movement of the wall is limited to a reasonable distance, and as the entire roof is allowed to displace as a unit, very little inertia force from self weight is transferred to the walls, and the movement itself serves to dissipate energy. This idea was discussed only recently by the students working in the Spring 1995 workshop, and it is hoped that they will be able to develop this possibility. It remains outside the scope of the thesis.

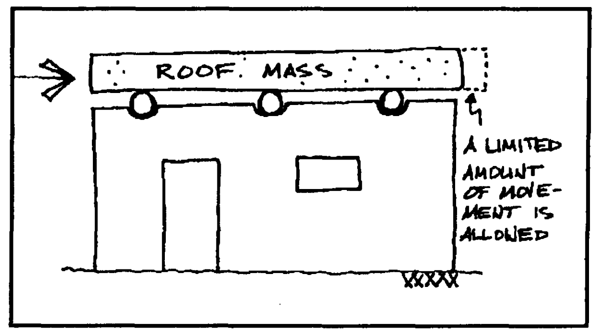

Figure 2.s Roof isolation concept

A final decision for the complete roofing system to be used was not crucial for the analysis and design of the houses overall, because the most important factors from a roof are its weight, rigidity, the type of main spanning system it employs, and how it connects to the walls. The enclosure system (joists, packed earth)

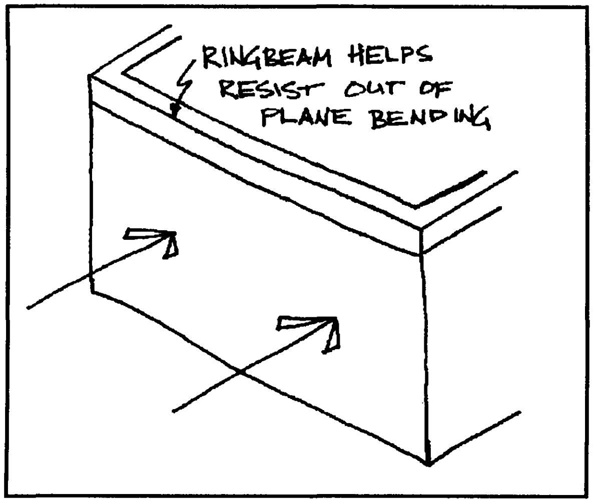

Figure 2.t Ring beam action

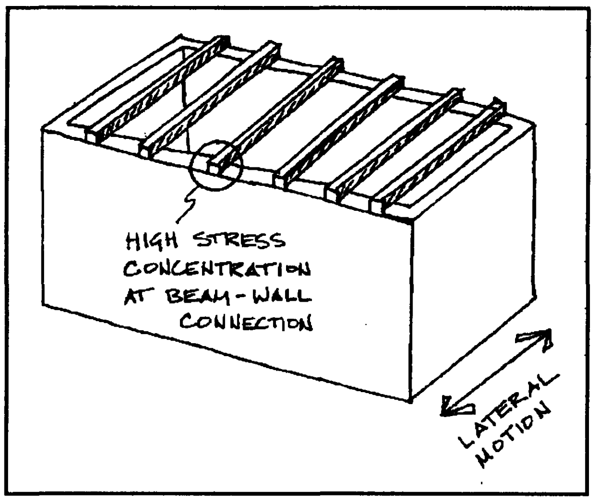

Figure 2.u Roof beam - wall connection

does not have a great effect on the system of steel reinforcing used in the walls, except for the dead loads contributed.

It was decided that the basic Gasirkum system would be best for the housing designs, meaning the use of a spanning system of main beams in both axial directions. The skylight or closure system could be made in the traditional way, or any other effective way available to span the main beams. Section 2.5.2 gives more a more complete description of roof construction.

Ring Beams 2.3.5

One of the most important and effective first measures toward earthquake resistance is the concept of a ring beam. This is basically a continuous horizontal layer through the perimeter walls which has the capacity to take tensile forces, and effectively holds the walls together. Ring beams also help to counter tension induced by out of plane bending [**Figure 2.t**]. Ring beams are very effective because they directly counter the most destructive kinds of forces that result from lateral acceleration on a traditional style house. The most important place for a ring beam is at or near the top of the walls, because this is where out of plane bending forces are usually the greatest, and where the roof to wall connections cause concentrated forces [**Figure 2.u**]. However, it is beneficial to place 2 or 3 more throughout the height of the wall if possible, including one at ground level.

Ring beams must be made in such a way that they are continuous over a horizontal layer, and are able to take tension and bending forces. Wooden ring beams, called hatils, are effective, and another system using concrete U-blocks and steel rebar is even better [**Figure 2.v**], creating a continuous reinforced concrete beam. The latter system was used in the Ove Arup design for Self-help schools.

A Final Note:

The end product of all the "appropriate technology" decisions was to go with something very close to traditional construction, with the major changes being the addition of steel for reinforcement, and ringbeams. Specific construction guidelines are given in Section 2.5.

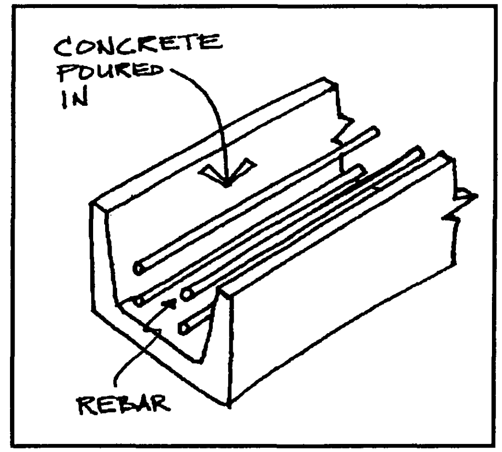

Figure 2.v U-block ringbeam

Selection of Design Loads 2.4.1

Level of Earthquake Lateral Loading

It is not only difficult to predict the lateral acceleration caused by an earthquake, but further thought must also be put into selecting the level of resistance a building will have. As mentioned in Section 2.1.5 the expected level of lateral acceleration for the town of Karimabad is predicted with some uncertainty. If we designed conservatively, and selected the highest possible earthquake level for the area as the loading condition on the houses to be engineered, our final product may be over-designed, and unaffordable for the majority of the people. Of course, erring in the other direction, and risking a design level that is too low, has the potential for a disastrous outcome.

In order to achieve a reasonable level of safety, it was necessary to define an appropriate level of earthquake performance for the houses. California policy makers have drawn up some defining guidelines for "acceptable risk," which reflect a mindset of safety and practicality. (Coburn, 1992 p. 232)

The objectives are:

- To resist minor earthquakes without damage.
- To resist moderate earthquakes without significant structural damage, but with some non-structural damage.
- To resist major or severe earthquakes without major failure of the structural framework, of the building or its component members and equipment, and to maintain life safety.

These guidelines seemed appropriate for our use, and they were condensed into one controlling principle: to maintain life safety, and allow people a reasonable amount of time to exit their houses before any major structural failure could occur. It is true that some earthquakes strike with a quickness and violence that makes it difficult for people to reach safety. Our designs were aimed to provide a degree of energy absorption, and redundancy in structural support, so that in the event of an earthquake strong enough to cause failure, the failure would not be sudden, but gradual.

The final decision process for selecting the level of lateral acceleration for design was based on existing reports and a program to allow people to build as safely as they could afford. Data from the U.S. Geological Survey for an area somewhat smaller than the areas examined in Section 2.1.5 was available, but deriving an expected level of acceleration would have involved a long and uncertain process of statistical analysis. It seemed better to save the time, and use the analyses already carried out. The analysis process used by Ove Arup for the 67.5-77.5 °E and 33-39 °N window was the most comprehensive, so the resulting design level of 0.4 g is suggested for maximum reasonable safety. In this report, an alternate design level of 0.2 g is also included in the example design calculations for comparison, and for a more affordable alternative. It is believed that a house constructed with the lower design level would still perform much better towards preserving life, than the standard unreinforced stone construction prevalently used.

Roof Loading
The traditional roof covering uses 8-12 inches of silt/soil over the wood beams and planks. With low compactive effort, the dry density of this soil roofing is less than 1400kg/m^3 (about 87 pcf). Roof dead loads for the design calculations here are set at 100 psf, and this includes the weight of the wood beams and rafters.

Live loads are a bit more difficult to set. A standard live load for residential floors is 40 psf in the U.S., and this would seem applicable to the roofs in Karimabad, which are also part of residential use. However, in the event of large gatherings, this live load may increase. The calculations done in this thesis were for the quantification of reinforcing steel in the walls necessary to resist lateral earthquake loads, and it is assumed that the probability of an earthquake and simultaneous large gathering on the roof is small. A standard live load of 40 psf was used in the seismic design of houses. Ove Arup estimated a potential 10 psf for snow load also; however, in the cold winters, most people stay indoors, leaving the roofs unoccupied, so these two loads were not combined. *Further, the 1994 UBC requires the use of combined live and dead roof loads in seismic design*, but snow loads of less than 30 psf need not be included.

Material Properties and Loading Magnitudes 2.4.2

- weight of stone = 130 pcf; Stone was estimated to be 1700-2400 kg/m^3 (106-150 pcf) by a site engineer at the Baltit Fort renovation project.
- weight of backfill soil = 100 pcf
- equivalent fluid weight of backfill soil = 42 pcf; This value is

necessary for computing active earth pressures, and was based on data from Hughes (Conference Proceedings, 1986) and Unified Soil Classification charts.

- mortar strength = 750 psi in compression
- yield strength of steel = 36 ksi
- roof live load = 40 psf
- roof dead load = 100 psf
- lateral acceleration = 0.2 g and 0.4 g; Separate calculations were made for these two levels of acceleration.

Foundations

Good foundations are important in seismic areas for more than the prevention of uneven settling and resulting cracking. Walls can be tied down to strong foundations to prevent overturning. Footings provide a more stable base and distribute the relatively concentrated forces along the bottom edge of a wall over a broader, and more stable area.

Foundations and footings for retaining walls were not examined in full detail for this report. Reports researched gave similar guidelines for simple, strip footing foundations. Continuous concrete footings were recommended because placing the block material for walls (stone or concrete block) directly on the ground can result in uneven settling, cracking and increased air infiltration (Intertect, 1981). The Ove Arup 1991 Self-Help Schools report, section 4.2 recommends an excavation depth of 2'-6" to 3' to prevent frost heave. The dead loads and other values used in the Ove Arup report were similar to the values used in the design calculations of this report, so the detail is presented in Appendix E as a rough guideline. An important difference must be noted: the Self-Help School walls were designed with 8 inch thick CMUs whereas the house designs presented here use 12 inch thick stone walls. Consequently, a wider footing is probably appropriate. Although a complete footing design is outside the scope of this thesis, a footing width of roughly 30 inches (2 1/2 times 12 inches) is recommended.

Retaining Walls 2.4.3

Retaining walls have to be designed to withstand lateral forces from the earth pressure behind them. The basic equations for determining active earth pressure forces were derived by Coulomb. Due to self-weight, retaining walls can resist earth pressure forces up to a certain point, but when the backfill pressure is too great, the retaining wall can topple over. Reinforcing steel must be placed on the side of the wall near the backfill, where tension forces develop.

Overview of the Mononobe-Okabe Equations and Their Usage
In an earthquake, if lateral acceleration is perpendicular to the retaining wall, an extra component of lateral force due to the acceleration of the backfill must be added into the calculations. The Mononobe-Okabe equations are a modified version of Coulomb's active earth pressure equations for a dry cohesionless backfill on a retaining wall, and account for any vertical or horizontal accelerations in the backfill. The Mononobe-Okabe equations (taken from Das, 1983 p. 305) are as follows:

$$P_{ae} = \frac{H^2\gamma}{2}(1-\kappa_v)K_{ae}$$ **Equation 2.a**

Equation 2.b

$$K_{ae} = \frac{\cos^2(\phi - \psi - \beta)}{\cos\psi \cos^2\beta \cos(\phi_w + \beta + \psi)\left[1 + \sqrt{\dfrac{\sin(\phi + \phi_w)\sin(\phi - \psi - i)}{\cos(\phi_w + \beta + \psi)\cos(i - \beta)}}\right]^2}$$

Where:

P_{ae} = the dynamic active earth pressure during an earthquake; the resultant force (lbs) exerted on a unit length, vertical "slice" of wall

H = wall height

γ = unit weight of backfill soil *(100 lb / ft3)*

β = angle between inner face of wall and backfill soil *(0°)*

ψ = $\tan^{-1}(\kappa_h / (1 - \kappa_v)$

ϕ = friction angle of backfill soil *(30°)*

ϕ_w = friction angle between inner face of the wall and backfill soil *(30°)*

i = backfill slope angle *(0°)*

κ_h = coefficient of horizontal acceleration of soil wedge *(*usually taken as 1/2 or 1/3 of the peak expected acceleration, so it varies between *0.2 g or 0.1 g* depending on which acceleration level is used*)*

κ_v = coefficient of vertical acceleration of soil wedge *(0.0 g)*

Italicized texts indicate assumed values where appropriate. The following figure illustrates the physical meaning and placement of these variables.

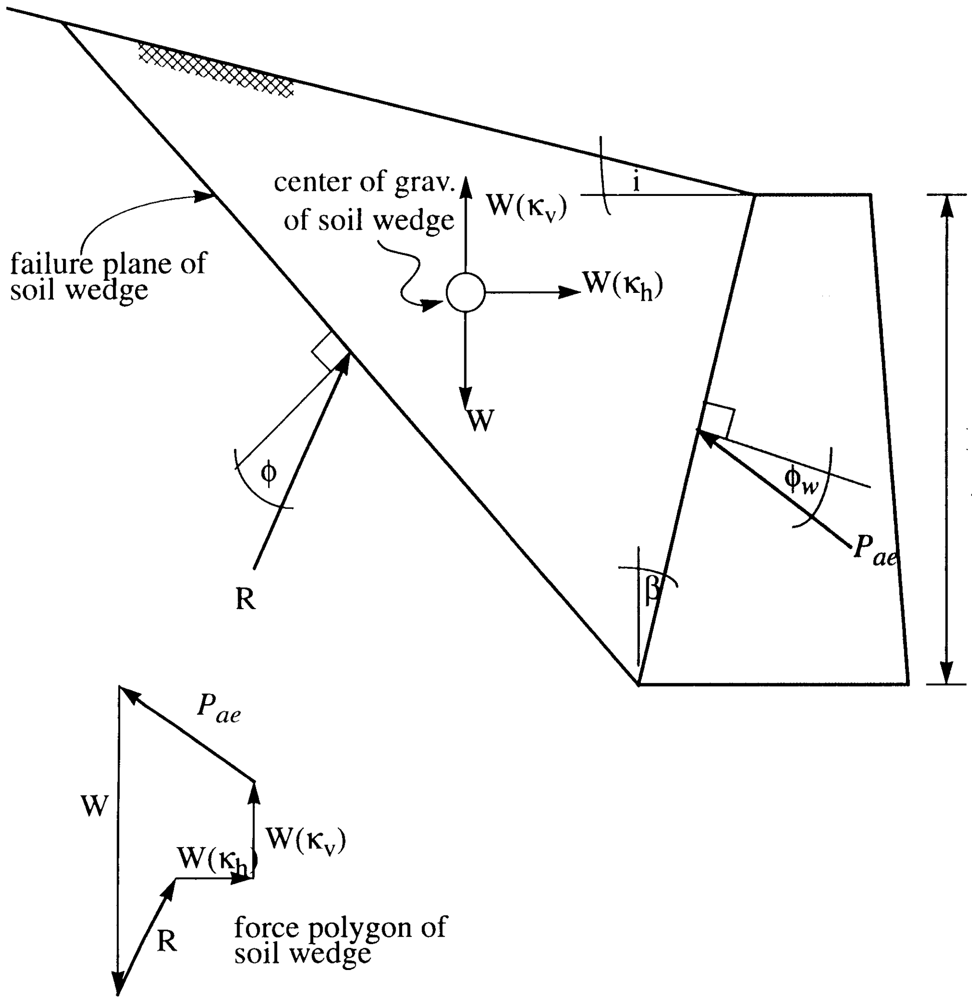

Figure 2.w (above) Illustration of Mononobe-Okabe equation variables

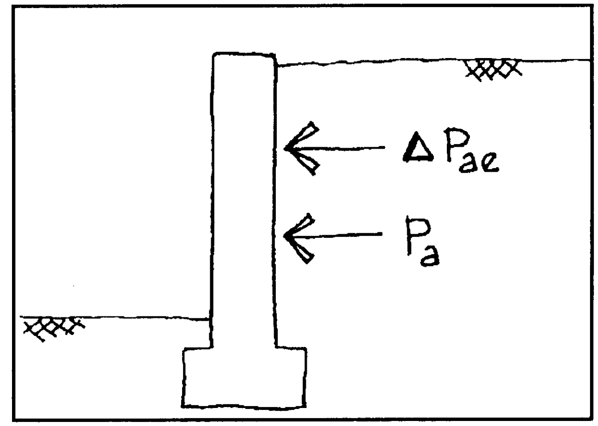

Figure 2.x Dynamic and static earth pressure forces

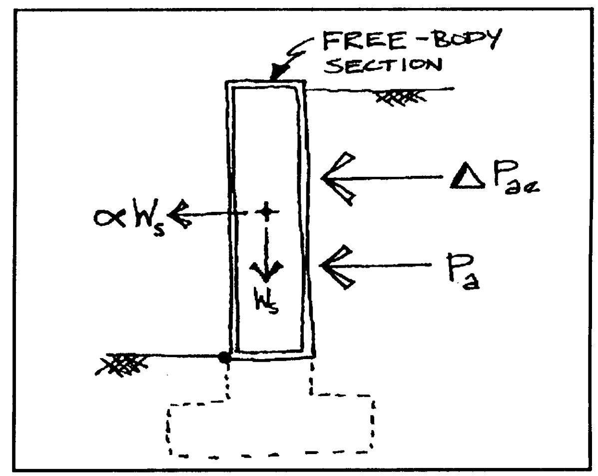

Figure 2.y Acceleration forces acting on retaining wall

During an earthquake, the forces that act on a retaining wall can be divided into two components [**Figure 2.x**]. P_a is the static active earth pressure, and is calculated in the same way, whether or not there is lateral acceleration, using the equivalent fluid method:

Equation 2.c

$$P_a = \frac{1}{2}\gamma_e H^2$$

γ_e = the equivalent fluid weight of the backfill soil, and is applied at H/3 from the base (42 pcf).

ΔP_{ae} is the difference between the static and dynamic active earth pressures:

Equation 2.d

$$\Delta P_{ae} = P_{ae} - P_a$$

ΔP_{ae} is applied at approximately $2H/3$ from the base.

The self weight of the wall (W_s) affects the overall equilibrium in two ways. W_s and its downward orientation will produce a moment which resists overturning about the base. However αW_s, which is lateral acceleration times self-weight, will cause an overturning moment. **Figure 2.y** shows the forces acting on a free body diagram of the retaining wall. Recalling that t is the thickness of the retaining wall, the total overturning moment (M_o)at the base is:

Equation 2.e

$$M_o = \frac{2}{3}H(\Delta P_{ae}) + \frac{1}{3}H(P_a) + \frac{1}{2}H(\alpha W_s) - \frac{1}{2}t(W_s)$$

If M_o is positive, then the wall will topple unless reinforcing is provided.

Assuming a 3 inch cover for reinforcing steel, the cross-sectional area of rebar needed per linear ft. of the retaining wall can now be calculated in a fashion similar to ultimate strength concrete beam design. **Figure 2.yy** gives the assumptions for steel placement.

$$M_o = \phi M_n$$

Equation 2.f

ϕ is a safety factor taken from the LRFD method of concrete beam design, and is equal to 0.9.

$$M_n = A_s f_y jd$$

Equation 2.g

where:

A_s = cross sectional area of steel rebar required

f_y = yield stress of steel (*36 ksi*)

j = estimation coefficient for steel moment arm (*usually taken as 0.9*)

d = distance between rebar and compressive edge of wall

Equations 2.f and 2.g can be used to solve for A_s:

$$A_s = \frac{M_o}{0.9^2 f_y d}$$

Equation 2.h

A spreadsheet was developed to find the required A_s, with variable inputs allowed. A printout of the spreadsheet is included in Appendix C.3 with some notes and explanations. Section 2.6.2 applies the spreadsheet and procedures explained here.

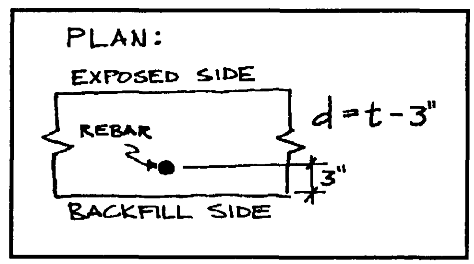

Figure 2.yy Assumptions for steel placement

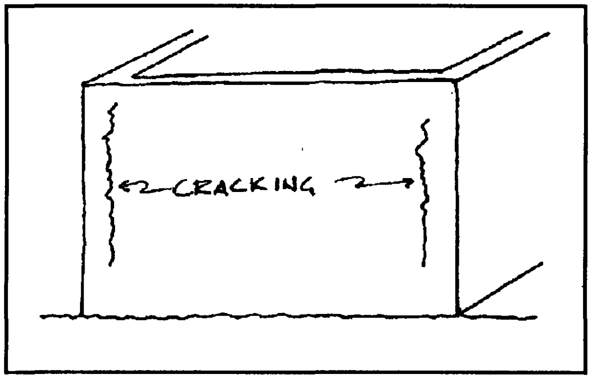

Figure 2.z

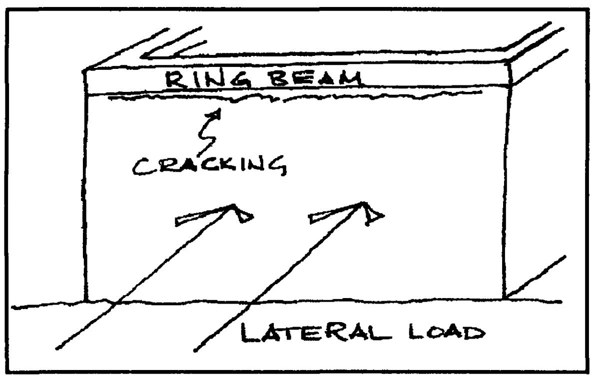

Figure 2.aa

Walls 2.4.4

The primary question in the engineering design of the walls was, "Where should reinforcing steel be placed to give the most efficient and effective seismic resistance?" Some formulas and derivations were performed on a theoretical level, and existing codes and research conducted on physical models were examined.

Based on general reading on earthquake resistant construction, and the solid plane box model examined in Section 2.2.2 there was already a general idea that a ring beam should be placed on the upper layer of the walls. Ideally, a well reinforced wall would have steel rebar running at regular intervals both horizontally and vertically, to counter out of plane bending and shear forces. However, for efficiency it had to be determined:

- At what spacing/interval should the layers be placed?
- Which layers are more important? Which are redundant and expendable?
- What is the difference in function between horizontal and vertical layers? Which should be the primary orientation (relates to the second question)?

Theoretical Analysis

A comparative analysis was carried out for two wall planes of the same dimensions, but with different primary steel orientations. (The analysis uses the standard methods of tributary areas and reinforced concrete beam design utilized throughout this thesis.) The goal was to see if one orientation was more efficient than the other. Assumptions had to be made for this analysis, such as fixity conditions, and the number of layers of reinforcing. In the end, derivations showed that either orientation could be more efficient, depending on the dimensions of the walls and fixity conditions assumed. What was most helpful from this analysis was that it showed that one primary orientation for steel is *not grossly inefficient* when compared to the other. It seems that one can choose the orientation which is best for constructibility, opening configuration, and other pragmatic factors, as opposed to trying to gear all design and construction plans for one primary orientation. But added to this theoretical analysis were observations taken from shaking table and lateral loading tests conducted on masonry/stone houses or scale models.

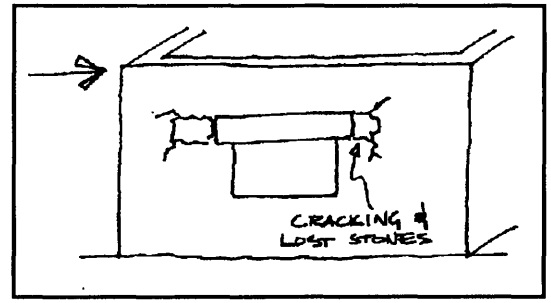

Figure 2.ab

Experimental Observations in Published Papers
Papers and articles from physical tests were surveyed for common points of failure in walls. Observed failures included:

- Shear cracks on walls parallel to lateral loading
- Cracks appearing at the edges of walls due to the connections with perpendicular walls. This could be due to shear and/or out of plane bending. [**Figure 2.z**]
- Cracking between the ringbeam layer and adjacent stones. [**Figure 2.aa**]
- Stones horizontally adjacent to lintels cracking loose and falling out [**Figure 2.ab**].
- Out of plane bending cracks along the edges and bottoms of walls [**Figure 2.ac**]
- Shear cracks along the bottoms of walls [**Figure 2.ad**]
- Vertical cracks from out of plane bending [**Figure 2.ae**]

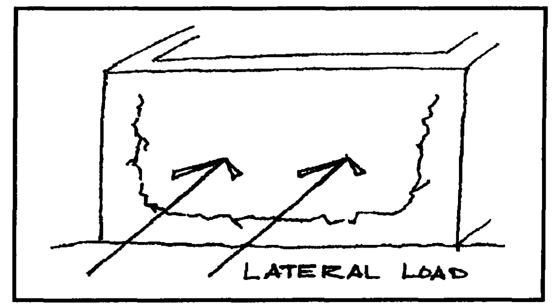

Figure 2.ac

Post earthquake damage reports cite the shear cracking and out of plane bending failures as the major modes of failure. Cracks often initiate from corners of openings and junctions of walls because these are the places of highest stress. Shear cracks then propagate to such an extent that material begins to fall out, and

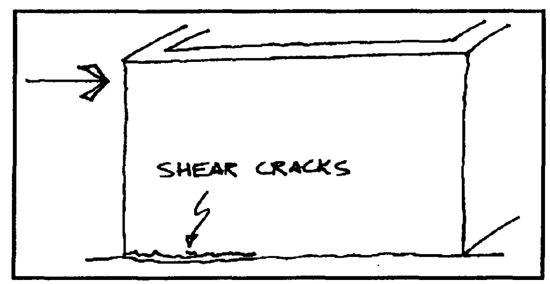

Figure 2.ad

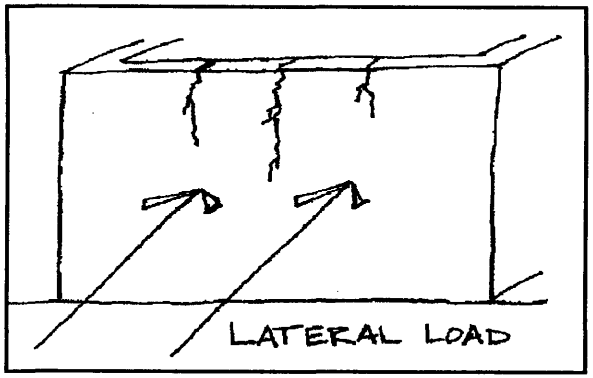

Figure 2.ae

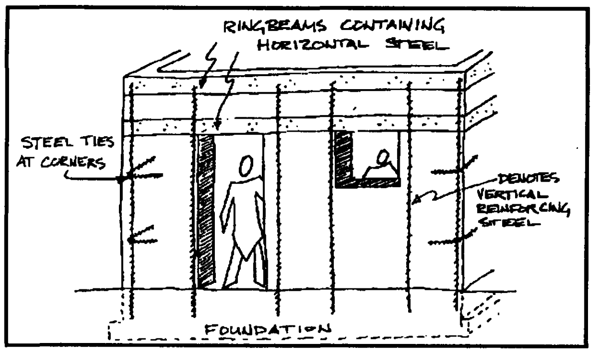

Figure 2.af General scheme of wall steel placement

lateral deformations become larger in the house as a whole. The shear walls may also collapse due to the loss of too much material. Out of plane bending will simply topple over a wall, or dislodge stones that have been already loosened.[1]

Codes and guidelines for the prevention of earthquake damage have been drawn up from experience, and the information researched agree on many common points. Intertect (1981) guidelines are representative of the types of suggestions given:

- All openings should have vertical reinforcing on either side.
- Vertical rebar should be spaced to roughly form a square shape with the ring beam.
- Walls and piers anchored to foundation not more that 2 m apart with 15mm rods.
- Corners and intersections should be made continuous with neighboring walls through some sort of horizontal tie

Final Steel Placement and Orientation Scheme

After considering the information found in codes, physical tests, earthquake damage reports, and my own theoretical analysis, a final placement scheme for reinforcing steel in the walls was set. [**Figure 2.af**] It was assumed that cement would be available to make a decent quality mortar. The steel placement is explained:

- A ring beam is needed at top level where the roof joins the wall. This primarily counters the out of plane bending stresses caused by the roof weight, and helps to distribute the concentrated forces of roof to wall connections (i.e. beams) more evenly over the wall in the event of shear loading.
- A lintel level ring beam is included to add an additional layer

1. Most examples taken from Conference Proceedings, 1986.

of horizontal connectivity, and to help counter out of plane bending in the top part of the wall, where those forces will be highest. This also conveniently doubles as a lintel, and saves on the heavy wood pieces often used to make the lintels.

- Vertical steel, from upper ring beam down into the foundation, is placed next to any opening, and according to Intertect guidelines, at least on intervals no greater than the height of the wall itself, to prevent shear cracks. UBC requires a spacing of no greater than 4 ft. This also prevents shear forces from overturning sections of wall which may be tall and narrow, as often is the case next to doorways.
- Short pieces of steel, bent into "L" shapes (stirrups), should connect each wall at the corners as often as possible, in order to join walls together more strongly. Stirrups can be overlapped with the horizontal steel in ringbeams (if there is a ringbeam on the same level) or bent around 3 vertical rebars at corners [see **Figure 2.as**].
- Another ringbeam placed at the foundation level is recommended if possible, to maintain a more continuous base of support, especially in the event of vertical cracks. If a long vertical crack propagates from somewhere in the wall and down through the foundation, parts of the wall in either side of the crack are more free to move apart, especially if loaded in shear.

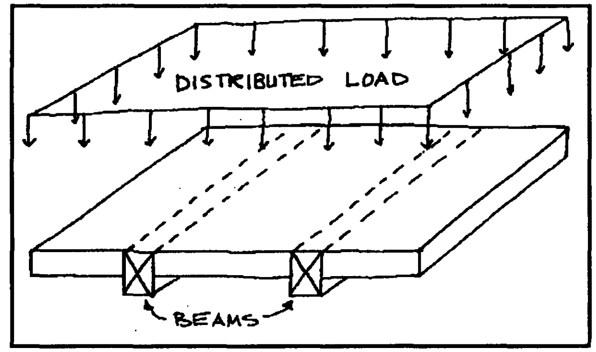

Figure 2.ag Floor beam analogy for wall tributary area

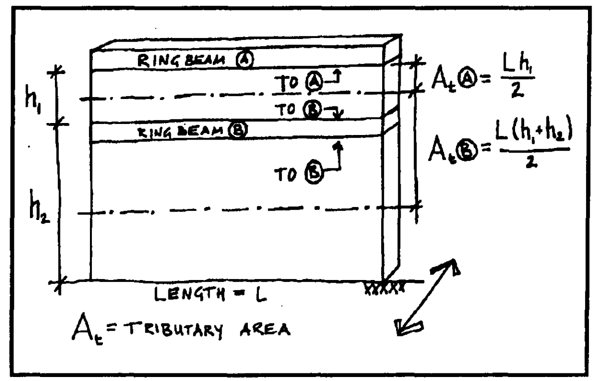

Figure 2.ah Ringbeam tributary areas

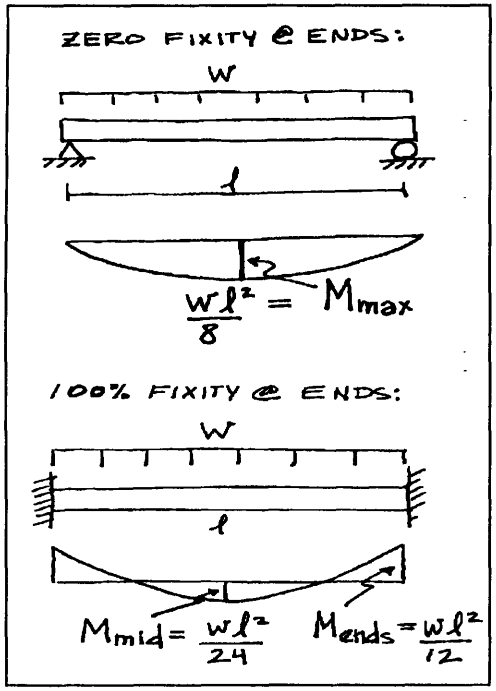

Figure 2.ai Moment diagrams for different fixities

Design Process for walls

There are two major steps in the sizing of steel reinforcement within walls. One is to quantify the steel necessary for out of plane bending (OOPB) and the other is quantifying the shear steel.

Out of Plane Bending Steel

The principal resistance to out of plane bending will be provided by the ringbeams at eave (just under the roof) and lintel level. As a perpendicular wall is loaded and undergoes OOPB, it behaves similarly to a floor slab under a two dimensional distributed load [**Figure 2.ag**]. The ringbeams act like floorbeams, and are loaded according to their tributary areas. The tributary areas on ringbeams extend 1/2 the distance from the beam itself to the next element that offers lateral resistance (another ringbeam, or the point where the wall meets the ground). [**Figure 2.ah**]

The magnitude of moment caused by the lateral load from tributary areas also depends on the fixity conditions at the wall corner connections. There are two extreme cases. **Figure 2.ai** shows two walls in plan, simplified into structural diagrams. The top half represents a wall with zero fixity at the ends, which is essentially like a simply supported beam. the maximum moment at midspan is $wl^2/8$, and moment is zero at the ends. In the lower part of **Figure 2.ai**, the ends are completely fixed, resulting in a moment of $wl^2/12$ at the ends and a midspan moment of $wl^2/24$. The spreadsheet developed for calculating steel quantities allows for a variable input of fixity at the ends. For example, if e (the end fixity) is set at 25% (e = 1/4), that would mean that the fixity of the wall ends is a about 1/4 of the way between zero fixity and full fixity, and the end moment (M_e) would take on a corresponding value:

$$M_e = \frac{wl^2}{24}e = \frac{wl^2}{96}$$ **Equation 2.i**

and the midspan moment (M_m) would be:

$$M_m = \frac{wl^2}{8} - M_e = \frac{11wl^2}{96}$$ **Equation 2.j**

The size of steel can be calculated once M_e and M_m are known for each wall plane through a process similar to that used in sizing the retaining wall steel.

$$\phi M_n = M_m \text{ or } M_e$$ **Equation 2.k**

$$M_n = A_s f_y jd$$ **Equation 2.l**

where ϕ, A_s, f_y, j, and d are defined as for equation 2.g. Isolating A_s, and substituting in the full expressions for M_e and M_m gives:

- at wall ends ($M_u = M_e$)

$$A_s = \frac{wl^2}{12}e \times \frac{1}{\phi f_y jd}$$ **Equation 2.m**

- at wall mid-span ($M_u = M_m$)

Equation 2.n

$$A_s = \left[\frac{wl^2}{8} - M_e\right] \times \frac{1}{\phi f_y jd}$$

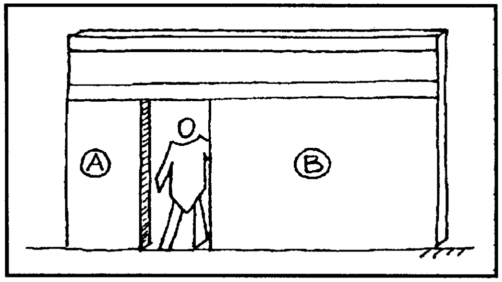

Figure 2.aj

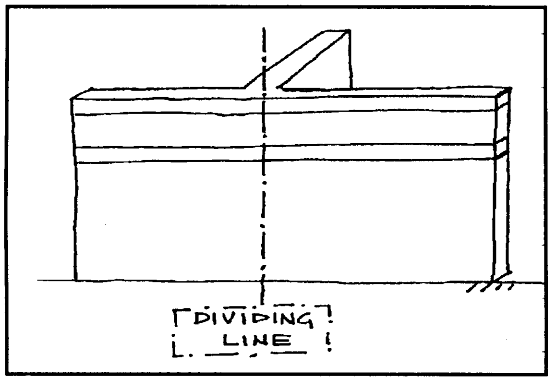

Figure 2.ak Intermediate wall

A couple of other notes on the process which was used:

- If there are openings in the wall plane, as shown in **Figure 2.aj**, the area of sections A and B are combined into a total tributary area, and is evenly distributed over the neighboring bending elements.
- If there is an intermediate perpendicular wall, as in **Figure 2.ak**, the wall is divided into two sections over the intersection point. Each side is analyzed as shown previously. The principle of superposition is used to find the bending moment over the intersection, which is the sum of the corresponding end moments of the two adjacent sections. (This assumes full fixity of the wall over the intersection.)

Roof Loads Exerted onto the Eave Level Ring Beam

Loads from the roof, both live and dead, will cause out of plane bending in the eave level ring beam, and steel must be sized to accommodate the added bending moment.

The adjacent area (A_{ar}) of the roof to the ring beam is defined in **Figure 2.am** (denoted A_{adj} in the figure). It is assumed that opposing perpendicular walls are well enough connected to the roof that they can each take 1/2 of the lateral load from the total roof area spanning between them. (This is a big assumption, and a strong connection between roof beams and eave level ringbeams should be designed so that lateral load is shared as much as possible by opposing walls.)

The roof rigidity (R_r) is a measure of roof stiffness under lateral loads. Referring to **Figure 2.al**, if the R_r is zero under the given direction of lateral loading, then the perpendicular walls will take 100% of the roof lateral loads. If R_r is 1 then the perpendicular walls take none of the lateral roof load, while the parallel (shear) walls take all of it. If R_r is somewhere in between, 0.2 for example, in this case the perpendicular walls will take 80% of the roof load, while the parallel walls take 20%.[1]

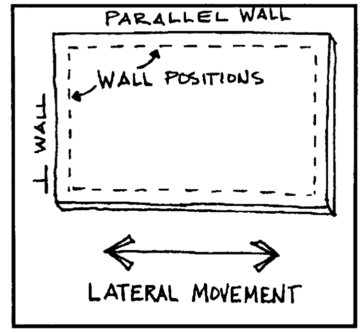

Figure 2.al

The lateral load from the roof (W_{ar}) onto the ring beam during lateral acceleration is:

$$W_{ar} = \frac{[A_{ar}(DL+LL)]\,(1-R_r)\,\alpha}{l}$$

Equation 2.o

where:

- DL = dead load of roof (psf)
- LL = live load of roof (psf)
- α = lateral acceleration (a fraction of g)
- l = the length of the ring beam in contact with the A_{ar}

The eave level ring beam must resist these moments plus the moments due to self weight (described previously):

- for M_e, this becomes $W_{ar}\, l^2e/12$
- for M_m, this becomes $W_{ar}\, l^2/8$ - $W_{ar}\, l^2e/12$

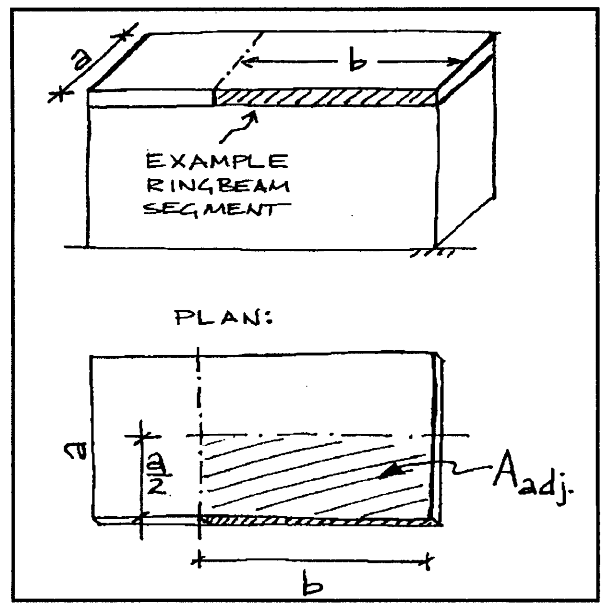

Figure 2.am Adjacent roof area defined

1. Roof rigidity affects the load path, but ultimately all lateral load is carried by shear in the parallel walls.

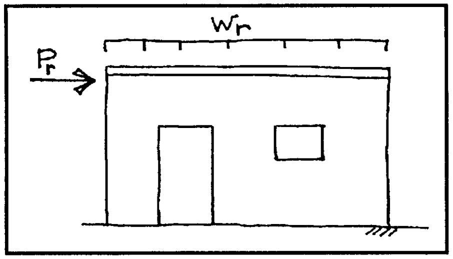

Figure 2.an Forces on shear walls

Shear Steel

To quantify the necessary shear steel, the loads which must be resisted are first identified [**Figure 2.an**].

Determining Load P_r

P_r is the lateral force due to the mass of the roof distributed onto the parallel (shear) walls. Regardless of the roof rigidity, the parallel walls ultimately resist the lateral forces formes roof.[1]

Equation 2.p

$$P_r = A_{ar}(DL + LL)\alpha$$

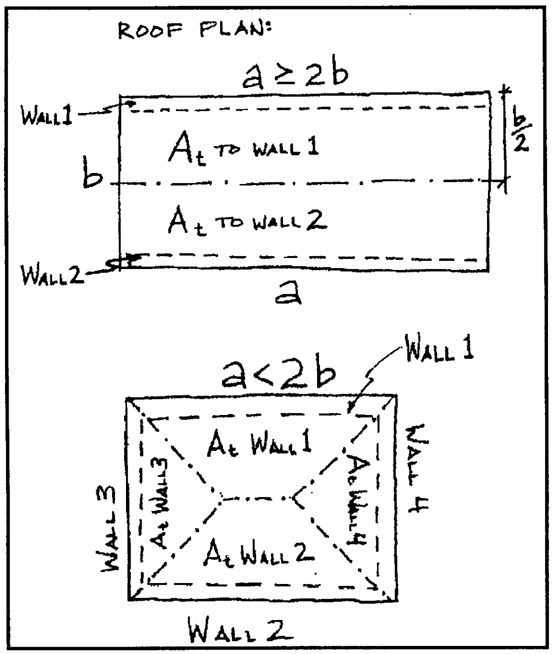

Figure 2.ao

Determining Load W_r

W_r is the static, load of the roof onto the wall due to gravity, not lateral acceleration, and the tributary areas used to determine it are not the same as adjacent tributary areas (A_{ar}). It is determined using a the tributary area process illustrated in **Figure 2.ao**.

Referring to **Figure 2.ao**, regardless of the direction of lateral acceleration, if $a \geq 2b$, then treat the roof as a one-way slab, and distribute all of the weight onto the longer walls. If $a < 2b$ then treat it as a two-way slab and distribute onto all four walls as shown. This procedure assumes that there are frequent enough connections between the roof and wall planes to approximate a distributed load.

1. A stiff roof just ensures that the load path of lateral force from the roof will not travel through the perpendicular walls and then be transferred to the parallel walls at the corners. In a stiff roof, the lateral force from the roof goes directly in to the parallel walls.

Pier Divisions

If a wall has openings, it must be divided into piers. The wall in **Figure 2.ap** has 5 piers, and each must be analyzed in shear.

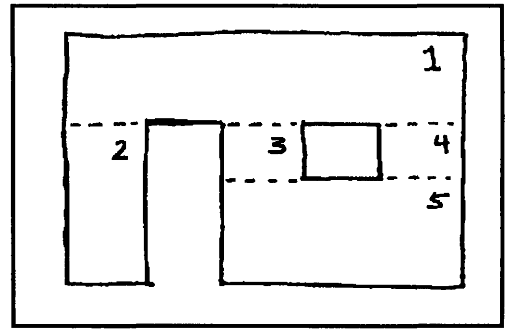

Figure 2.ap Pier divisions

Taking pier 3 as an example, among the loads it resists is the dead load of the stone section above it (tributary area on 3 = A_{t3}). And the amount of W_r that is exerted on pier 3 is directly proportional to its horizontal length in comparison to piers 2 and 4.

Pier 1 exerts a lateral force of P_{s3} onto pier 3 (a portion of its self weight times lateral acceleration) as well as P_{r3} (a portion of P_r). The combined forces of P_s and P_r are distributed over piers 2, 3, and 4 as detailed in Appendix F. **Figure 2.aq** shows an expanded view of pier 3, and the specific forces exerted on it.

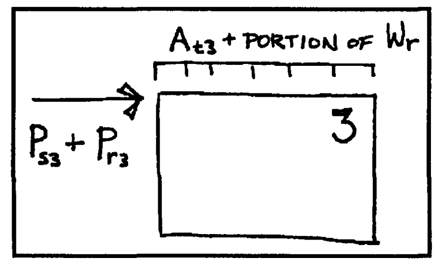

Figure 2.aq Forces on pier 3

All piers can be isolated and diagrammed as pier 3. Continuing to use pier 3 as an example, four values should be calculated and checked:

σ_{crush} = the stress on the right side base of the pier (according to **Figure 2.ar**)

σ_{pull} = the stress on the left side base

τ_b = shear stress along the base

τ = maximum shearing stress (This value would be found from an analysis using Mohr's circle. This quantity is not calculated in the spreadsheet or design example. Final designs should consider this quantity. One should note however that the shear strength of masonry alone is usually not enough to resist lateral forces, and it is the spacing/placement of reinforcing steel which really resists shear cracking.)

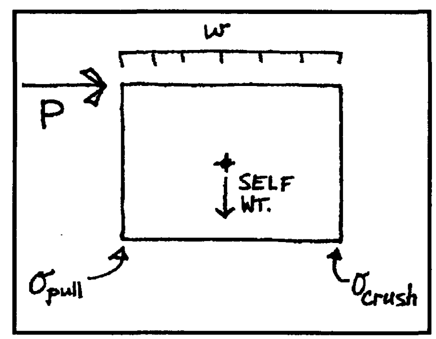

Figure 2.ar

Equations 2.q

$$\sigma_{crush} = \sigma_{base} + \frac{6Ph}{tl^2}$$

$$\sigma_{pull} = \sigma_{base} - \frac{6Ph}{tl^2}$$

$$\tau_b = \frac{P}{tl}$$

where:

P = all lateral forces on the pier (in this example, P_{s3} + P_{r3})

t = pier thickness

l = pier horizontal length

- $6Ph/(tl^2)$ is derived from the bending stress equation for beams: stress = *Moment / Section Modulus*
- Tension occurs along the side of the wall if σ_{pull} is negative. In that case, appropriate reinforcing is required on that side.

A spreadsheet, shown and explained in Appendix G, was developed with variable inputs of dimensions and loads, to do all these out of plane bending and shear steel calculations.

Walls 2.5.1

Good construction techniques must be combined with the engineering analysis in order for safe walls to be built. Many of the important safety measures are not engineered details such as rebar sizes, but construction techniques that make good use of what is available. Several suggestions were found in literature researched:

- Stone laying: Through-stones should be used at regular intervals to make the walls more continuous through their thickness. "Keystones" at corners help make wall to wall connections at corners more continuous (KPSS, 1993), which is an important principle mentioned in Section 2.2.2. However, these stones must be laid outside of the path of rebars within the wall.
- If not enough cement is available for mortar, it is suggested that the walls be made with traditional stone and mud mortar, with occasional courses of cement mortar. Horizontal cement mortar courses should be laid as often as possible.
- It is necessary for reinforcing steel to develop a strong bond with its surrounding material, in order to be effective. Therefore, quality mortar must be provided in the areas where steel is placed, and the horizontal layers of mortar are a proper place for steel bars.[1] Where vertical reinforcement is used in walls, a continuous space can be left around the vertical bars, and good mortar grout should then be poured into the spaces. (The concept of using intermittent bands of higher quality materials with traditional construction yields results similar

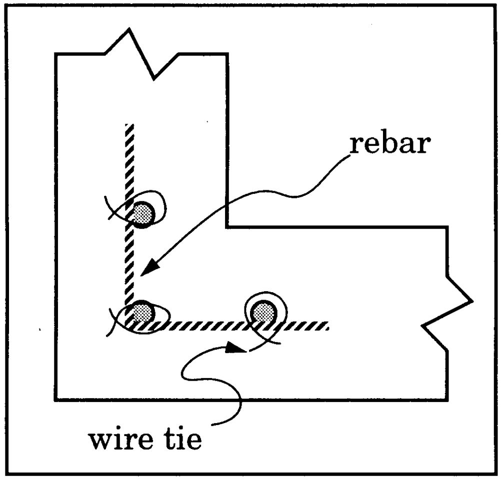

Figure 2.as Corner reinforcement detail

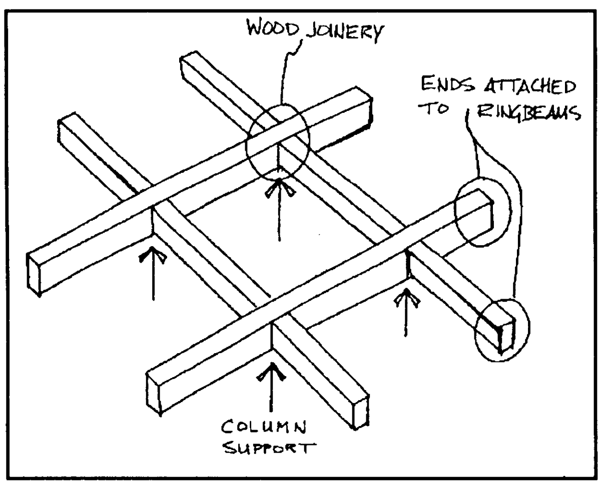

Figure 2.at Roof support system

to the timber cribbing or hatils of early construction.) If rebar is too expensive, some sort of wire mesh, like chicken wire could be laid in the horizontal courses (50-75 mm thick). Coburn's article recommends spacing the layers an distance no more than the thickness of the wall.

- Mortar mixes should ideally be in the 1:4 or 1:5 cement/sand ratio, possibly with local soil/silt. But anything leaner than 1:6 would yield very unpredictable strengths, and be very difficult to mix effectively.
- Vertical reinforcement is not very effective without mortar of adequate strength, an upper level ringbeam, and (some literature even suggests) a ground level ringbeam. At any rate, priority should be placed on constructing an eave level ringbeam, and placing vertical steel around openings and at corners.
- Stones should be wetted before laying. This prevents dry stones from absorbing the moisture out of the mortar before it can cure properly.
- Corner joining of walls can be achieved with several details, but the concept is the same: to bond perpendicular walls as continuously to each other as possible. Extending a piece of bent rebar a small distance into each wall at the corners is a good way to achieve this. **Figure 2.as** illustrates in plan, a simple detail using 3 vertical bars at the corners connect to piece of horizontal corner reinforcement by wire ties.

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1. Many of these concepts were taken from Andrew Coburn's article "Analysis of Earthquake Damage and Proposals for Strengthening Stone Masonry Buildings in Eastern Anatolia" which can be found in the Conference Proceedings of the Middle East and Mediterranean Regional Conference on Earthen and Low-strength Masonry Buildings in Seismic areas (1986).

Roofs 2.5.2

Roof were not designed in detail; however, a structural system and some construction guidelines were found and/or developed to provide a relatively rigid diaphragm.

Figure 2.at shows the basic structure supporting the roof. This system provides connected beams in two directions and beams that are perpendicular to the lateral acceleration act to transfer the lateral force from the roof, to the parallel shear walls [**Figure 2.au**].

In order to create an even surface for the roof closure system, some joinery must be done where the beams intersect [**Figure 2.av**]. And column capitals are necessary in order to prevent local failures at the joinery cuts [**Figure 2.ax**].

In addition:

- ⇨ Bases of columns could be sunk into the ground a bit, and set on a flat stone base, to give more stability.
- ⇨ Metal brackets to attach beam to column elements would improve lateral stability. (Coburn,1981 p. 247)
- ⇨ If a concrete ringbeam cannot be obtained, a ringbeam made of wood can be substituted. A construction detail for this wood ringbeam, or wall plate system, can be found in KPSS, 1993 Section B.1.1.6.

Protection Against Water Damage

Cracks are likely to initiate and propagate from construction flaws and weak spots. The variability in material and construction quality of traditionally built houses in Karimabad make these kinds of weaknesses more likely, especially at the places

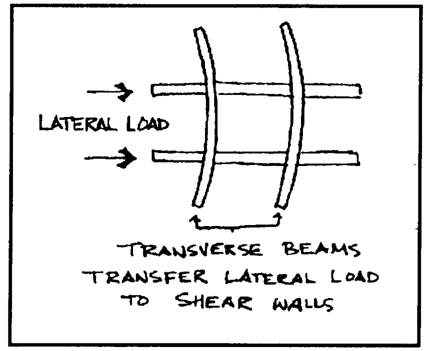


Figure 2.au

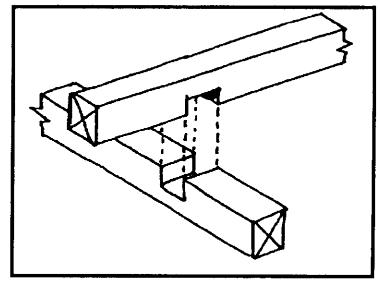


Figure 2.av Wood joinery

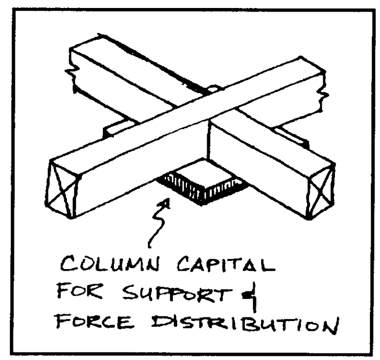


Figure 2.ax

where water damage can occur. The roof to wall junction is particularly vulnerable to defects due to water damage and construction flaws (some examples are illustrated in Coburn 1992 p. 236). KPSS (1993) suggests some ways to prevent water damage through some good roof construction practices which are drawn from traditional ideas:

- Use of halli as a waterproofing membrane is good, if halli is available.
- Projecting the roof face beyond the boundaries of the wall, and creating a slight overhang will help keep the walls and connection areas dry. Keep in mind however, this will add weight to the roof, and any material which hangs over exits is a potential danger during an earthquake.
- Making baldis or verandas on the roof offers an extra precipitation barrier.
- Putting a slight pitch in the roofs is important for preventing pools from forming and seeping through the roofs.

Some sources propose that use of cement stabilized soil would be a good water proofing barrier for the top layer of a roof. Hughes (Conference Proceedings 1986) suggests that this would *not* be a good idea because a soil/cement mix cannot be reworked if it cracks or crumbles, or made to "self-seal" by local remolding. These cracks are likely to happen through temperature changes and loading by general use, and quickly compromise the water resisting ability of a roof.

Layout 2.5.3

The layout of a house itself was found to be an important factor in seismic resistant design, even though it is not something that can be quantified like reinforcing steel. Intertect (1981) gives good general guidelines:

- Symmetry over the primary axes of the wall layout helps balance the distribution of seismic loads evenly. Keeping the openings on opposite walls as balanced as possible will also give some benefit.
- An "L" shaped layout should not be used, without the provision of some type of "crush" zone made from lighter materials to join the two main sections. [**Figure 2.ay**]
- Doors and windows should be placed at least one meter from the ends of walls and from each other.
- Keep doors and exit areas clear of overhanging structure or decor that could fall on people as they emerge.
- Doors to the outside should open outward (though this may not be desirable for other reasons) and interior doors should open in the direction of the nearest exit.

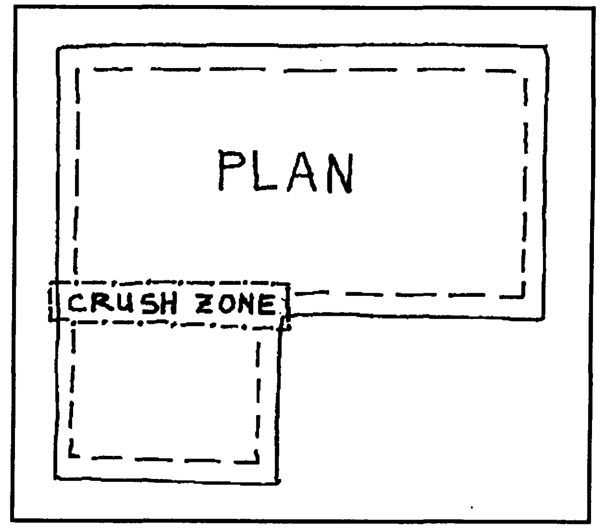


Figure 2.ay "L" plan with crush zone

Engineering of Walls 2.6.1

A typical wall from one of the house designs by Rukiye Devres is engineered in this section to illustrate the processes detailed in Section 2.3. This particular house design is included in Appendix H with all walls labeled with letters for identification.

Parameters

- wall thickness (t) = 1 ft.
- density of wall material (γ_w) = 0.13 kips/ft.^{3}
- end fixity of walls (e) =.75; This results in equal maximum end and mid-span moments.
- lateral acceleration (α) = 0.2 g
- steel cover (d) =.5 ft.; This places the reinforcing steel in the middle of the wall thickness.
- coefficient of rebar moment arm (j) = 0.9
- roof rigidity (R_r) = 0.7
- roof dead load (DL) = 0.1 ksf
- roof live load (LL) = 0.04 ksf
- beam bending load resistance factor (ϕ) = 0.9

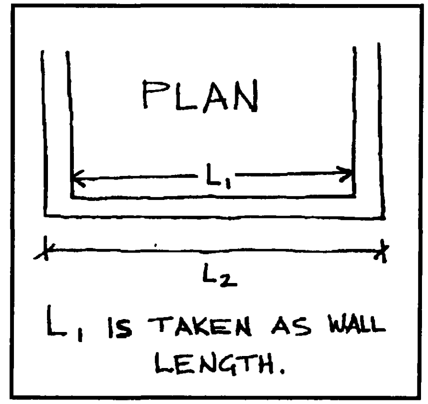


Figure 2.az Horizontal length

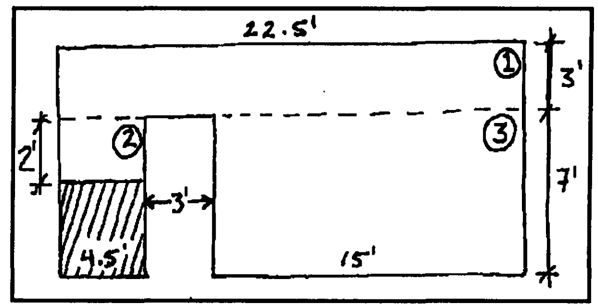


Figure 2.ba Wall C dimensions

Step 1: Inventory of Walls and Take-off of Dimensions

Each wall was labeled and dimensions, included those of openings, were taken. **Figure 2.az** shows the assumption used in taking the horizontal length. It is acknowledged here that the assumption is a bit unconservative.

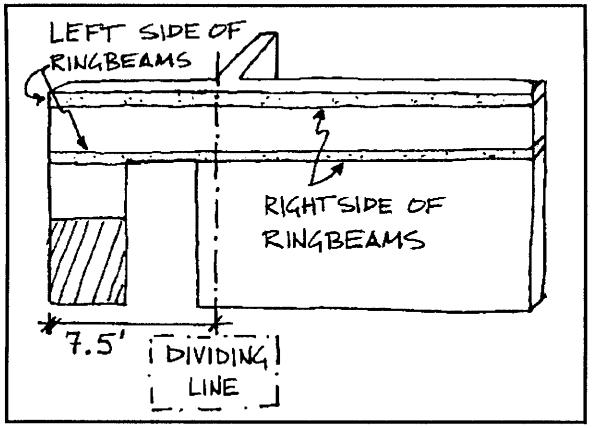


Figure 2.bb Left and right side ringbeam sections

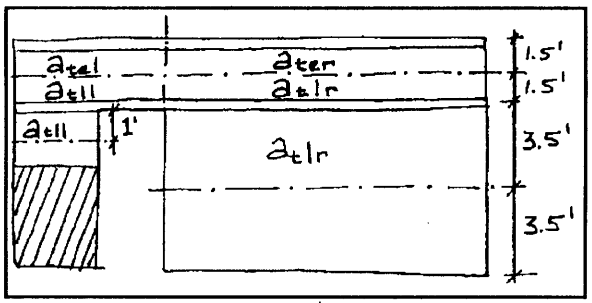


Figure 2.bc Division of tributary areas over left and right ringbeam sections

Step 2: Division of Roof Tributary Loads
Appendix H shows how the static roof loads were distributed over each of the walls. Static roof load distribution follows the procedure explained in Section 2.4.4.

From this point forward, the calculations for Wall C will be shown. All other walls were engineered in a similar fashion.

Step 3: Distribution of Wall Self-Weight in Out of Plane Bending
The dimensions of Wall C are shown in **Figure 2.ba**. Note that under pier 2, the shaded section is where the wall intersects with a stepped retaining wall running perpendicular to it. (See house plans in Appendix H for clarification.) This section is essentially fixed against out of plane bending and shear at this particular stage of calculation. The retaining wall is engineered separately.

Figure 2.bb shows where a perpendicular wall intersects with Wall C, and how the ringbeams are split into left and right sides. The tributary areas of Wall C onto the eave and lintel level ringbeams are shown and denoted in **Figure 2.bc**:

A_{tel} = tributary area of the eave level ringbeam, left side (11.25 ft.^{2})

A_{ter} = tributary area of the eave level ringbeam, right side (22.5 ft.^{2})

A_{tll} = tributary area of the lintel level ringbeam, left side (15.75 ft.^{2})

A_{tlr} = tributary area of the lintel level ringbeam, right side (75 ft.^{2})

Defining l_l to be the length of the left side of the ringbeams, and l_r for the right:

- ⇨ W_{tel} = distributed load on left side, eave level ringbeam from A_{tel}

$$W_{tel} = \frac{A_{tel} t \gamma_w \alpha}{l_l} = \frac{(11.25\text{ft}^2)\ (1\text{ft})\ (0.13\frac{\text{k}}{\text{ft}^3})\ (0.2\frac{\text{g}}{\text{g}})}{7.5\text{ft}} = 0.039\frac{\text{k}}{\text{ft}}$$

In similar fashion:

- ⇨ W_{ter} = 0.039 k/ft
- ⇨ W_{tll} = 0.055 k/ft
- ⇨ W_{tlr}=0.13 k/ft

Step 4: Determining Adjacent Roof Areas

Figure 2.bd shows Wall C in plan, and the adjacent roof area segments. These segment areas are combined into one value (A_{ar} = 255 ft.^{2}) which is divided between the left (A_{arl}) and right (A_{arr}) sides in proportion to their horizontal length.

A_{arl} = (A_{ar})(7.5/22.5) = 85 ft.^{2}
A_{arr} = (A_{ar})(15/22.5) = 170 ft.^{2}

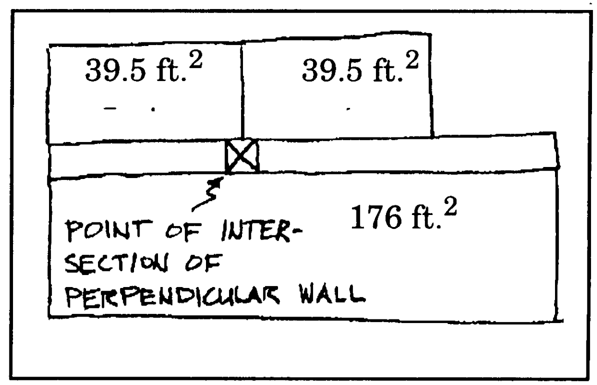


Figure 2.bd Plan view of Wall C and adjacent tributary roof areas

Using Equation 2.o we can find W_{ar} for the left (W_{arl}) and right (W_{arr}) sides.

$$W_{arl} = \frac{85\text{ft}^2\,(0.1+0.04)\,\frac{\text{k}}{\text{ft}^2}\,(1-0.7)\ (0.2\frac{\text{g}}{\text{g}})}{7.5\text{ft}} = 0.095\frac{\text{k}}{\text{ft}}$$

similarly:

W_{arr} = 0.095 k/ft

Step 5: Calculating Mm and Me for all Ringbeam Sections

⇨For the left side, lintel level ringbeam

$M_e = W_{tll}l^2e/12 = 0.19$ kft

$M_m = (W_{tll}l^2/8) - M_e = 0.19$ kft

⇨ For the right side, lintel level ringbeam

$M_e = W_{tlr}l^2e/12 = 1.83$ kft

$M_m = (W_{tlr}l^2/8) - M_e = 1.83$ kft

For the eave level ringbeam, W_{te} and W_{ar} must be combined into a total distributed load (W_{tot}) for each side.

$W_{totl} = W_{tel} + W_{arl} = 0.13$ k/ft

$W_{totr} = W_{ter} + W_{arr} = 0.13$ k/ft

⇨ For the left side, eave level ringbeam

$M_e = W_{totl}l^2e/12 = 0.47$ kft

$M_m = (W_{totl}l^2/8) - M_e = 0.47$ kft

⇨ For the right side, eave level ringbeam

$M_e = W_{totr}l^2e/12 = 1.83$ kft

$M_m = (W_{totr}l^2/8) - M_e = 1.83$ kft

Step 6: Calculating Cross-sectional Areas of Reinforcing Steel

From Equations 2.m and 2.n

$$A_s = (M_e \text{ or } M_m) / (\phi f_y jd)$$

In this example, M_e and M_m are equal within each ringbeam section, and the A_s required for each side can be found in one calculation:

- For the left side, lintel level ringbeam
  $A_s = 0.19 \text{ kft} \times [(0.9)(36 \text{ ksi})(0.9)(.5)]^{-1} = 0.013 \text{ in}^2$
- For the right side, lintel level ringbeam
  $A_s = 0.126 \text{ in}^2$
- For the left side, eave level ringbeam
  $A_s = 0.032 \text{ in}^2$
- For the right side, eave level ringbeam
  $A_s = 0.126 \text{ in}^2$

These cross-sectional areas were multiplied by the appropriate lengths in order to quantify the amount of steel needed.

Step 7: Dividing Wall C into Piers for Shear Steel Calculations
Figure 2.ba also shows Wall C divided into piers. P_1, P_2, and P_3 are the total lateral forces on each of the piers, which can only be determined after the roof loads and wall self-weight loads are found for each pier.

Step 8: Calculations for Pier 1
Because Pier 1 is at the very top, and is a single unit, forces and resulting stresses for it can be calculated directly, without any considerations of stiffness distributions (which must be done for Piers 2 and 3).

Tributary area for static roof load (A_{tr}) = 226 ft.^{2} (Appendix H gives the breakdown for the roof tributary areas, and Section 2.4.4 under *Shear Steel* explains the process.)

Roof static load on Pier 1 = $A_{tr}(DL + LL)$
= 31.6 k

Self weight of Pier 1 = (area)$t\gamma_w$
= (22.5 x 3) ft.^{2}(1 ft.)(0.13 k/ft^3)
= 8.8 k

Total stress at base (σ_{base})of Pier 1 = (31.6k + 8.8k)/(22.5 x 1 x 144) in^2
= 0.013 ksi

Adjacent roof area, for dynamic roof loading (A_{ar}) = 255 ft.^{2}

Lateral force $P_1 = A_{ar}(DL + LL)\alpha$
= 255 ft.^{2}(0.14 k/ft^2)(0.2g/g)
= 7.1 k

Moment caused by lateral force P_1 = (height of Pier 1)(P_1)
= 3 ft. x 7.1 k
= 21.4 kft

σ_{crush} (from Equations 2.q)
= σ_{base} + [(21.4kft) (12in/ft)(135in) ÷ 2.95x10^{7}in^4]
= 0.014 ksi
σ_{pull} (from Equations 2.q)
= σ_{base} - [(21.4kft) (12in/ft)(135in) ÷ 2.95x10^{7}in^4]
= 0.012 ksi

Checking shear at base:

Lateral force cause shear on Pier 1 = 7.1 k

Base shear resistance of pier = A_s x F_{vs} + $Base\ Area(0.9)F_{vm}$

Where: As = area of vertical steel in pier (using #3 bar/4 ft.)

F_{vs} = shear strength of steel (taken as $0.58F_y$)

0.9 is a reduction factor for cross-sectional area at the base of the pier which accounts for mortar joints.

F_{vm} = shear strength of mortar

= (compressive str. of mortar in psi)$^{1/2}$ ÷ 3

3 is an assumed factor of safety

Base shear resistance of Pier 1

= (0.55in^2 x 20.9ksi)+(22.5ft x 1ft x 144in^2/ft.^{2} x 0.9 x 0.009ksi)

= 37.7 k

Pier 1 is adequate in shear because the shear resistance (37.7k) is greater than the lateral force causing shear (7.1k).

All similar forces and stresses for other piers were found in this way.

Step 9: Distributing Lateral Forces between Piers 2 and 3

Following a process from Schneider, 1987 (outlined in Appendix F), the relevant parameters of Piers 2 and 3 are:

- Pier 2

  $h/d = 0.44$

  $\Delta_{f2} = 0.1171$

  $k_2 = 8.54$

- Pier 3

  $h/d = 0.47$

  $\Delta_{f3} = 0.126$

  $k_3 = 7.93$

where Δ_{fx} is a stiffness value from the equation:

$\Delta_{fx} = 0.0833(h/d)^3 + 0.25(h/d)$

and k_x is the reciprocal of Δ_{fx}, used in the calculation of relative stiffnesses.

The total lateral force on Piers 2 and 3 (P_{tot}) is defined here as:

$P_{tot} = P_1 + \alpha$(self weight Pier 1)
= 7.1 k + (0.2 g/g x 8.8 k)
= 8.9 k

⇨ P_{tot} distributed to Pier 2 (P_2) = $P_{tot}k_2 / (k_2 + k_3)$
= (8.9 k x 8.54) / (16.5)
= 4.6 k

⇨ P_{tot} distributed to Pier 3 (P_3) = 4.3 k

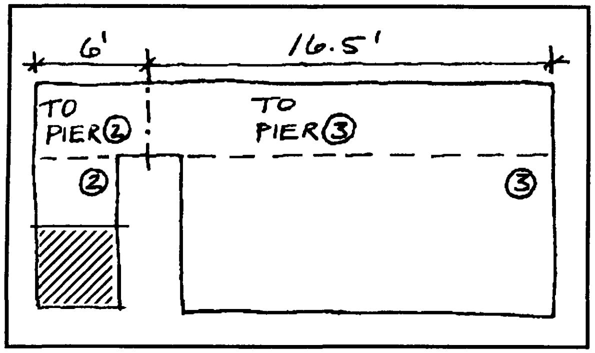


Figure 2.be Division of Pier 1 self weight onto Piers 2 and 3

Step 10: Finding Base Stresses of Pier 2 and 3

The forces from the weight of Pier 1 will also be transferred onto Piers 2 and 3. Shown in **Figure 2.be,** Pier 1 is split evenly over the door opening, and the two sections are distributed over 2 and 3.

Weight to Pier 2 = 2.34 k
Weight to Pier 3 = 6.44 k

The static load from the roof tributary area is distributed over 2 and 3 in proportion to their relative lengths:

Static load from roof = 31.6 k
Static roof load to Pier 2
= (Pier 2 length / Combined length 2 and 3) x (31.6 k)
= 8.6 k
Static roof load to Pier 3 = 23.1 k

The base stresses of 2 and 3 are the combined weights from Pier 1 and the static roof load, and self weight, divided by their own base area.

⇨ Pier 2 σ_{base} = (2.34k + 8.6k + 1.17k) / [(4.5ft. x 1ft)(144in^2/ft.)]
= 0.019 ksi

⇨ Pier 3 σ_{base} = (6.44k + 23.1k + 13.7k) / [(15ft. x 1ft)(144in^2 / ft.)]
= 0.020 ksi

And finally, from Equations 2.q:
⇨ σ_{crush2} = 0.019 ksi + [(9.18 kft)(12 in/ft)(27 in) / (1.57x10^5 in^4)]
= 0.038 ksi
⇨ σ_{pull2} = 0.019 ksi - [(9.18 kft)(12 in/ft)(27 in) / (1.57x10^5 in^4)]
= 0.000 ksi
⇨ σ_{crush3} = 0.020 ksi + [(30.4kft)(12in/ft)(90in) / (5.83x10^6 in^4)]
= 0.026 ksi
⇨ σ_{pull3} = 0.020 ksi - [(30.4kft)(12in/ft)(90in) / (5.83x10^6 in^4)]
= 0.014 ksi

This particular design employed a stepped retaining wall system. This resulted from an early idea that a series of smaller walls would ultimately save steel over one wall of full height.

In order to use a stepped retaining wall system the base of the upper retaining wall must be outside the failure wedge of the lower wall. The failure angle of the soil (θ) was found using an iterative method with the following equation (Truitt, 1983):

Equation 2.r

$$F_w = W_s \frac{(\sin\theta - \phi)}{\sin(\beta_2 + \phi + \delta - \theta)}$$

Where:

Figure 2.bf illustrates the variables.

F_w = resultant force of the sliding soil wedge
W = weight of the soil wedge (soil taken as 100 pcf)
ϕ = soil friction angle (*30°*)
β_2 = angle from horizontal to backfill face of wall (*90°*)
δ = soil/wall friction angle (*15°*)

Trial values for θ were selected to find the maximum value of F_w, which corresponds to the most probable soil failure angle. Using a spreadsheet, $\theta = 54°$ was determined.

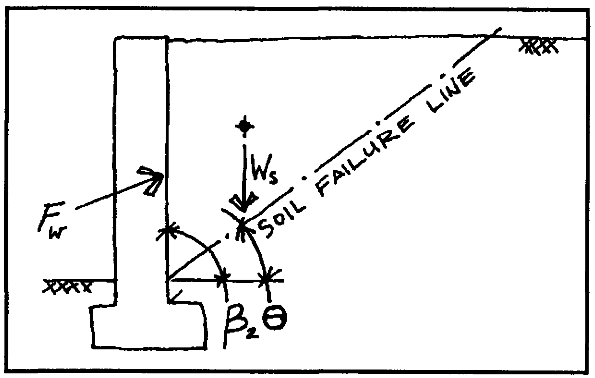


Figure 2.bf Variables for finding soil failure angle

From this point, the necessary geometry for using a stepped system was drawn [**Figure 2.bg**]. In the original house design the ledge distance is only 4 feet. However, if the ledge distance can be increased to >5.1 ft., then stepped retaining walls can be used. The area of steel required for this stepped system is 0.02 in^2 per linear ft. (a #3 bar every 4 feet would be sufficient).

However, if the ledge distance cannot be increased, the upper retaining wall can be extended far enough down into the ground, that its foundation is outside the lower wall's failure wedge. This necessitates designing the upper to a higher height. The geometry shown in **Figure 2.bh** is sufficient for this purpose, and the upper wall was designed as a 7 ft. retaining wall, requiring 0.08 in^2 of steel per linear foot, equaling one #3 bar at every foot. (For comparison, a retaining wall 1.5 ft. in thickness, and 10 ft. high requires 0.17 in^2 of steel per linear ft.)

Some important issues that still should be investigated:
- How does lateral acceleration affect the soil failure angle?
- Is the failure angle affected by the surcharge that the upper step of soil places on the earth directly behind lower wall?

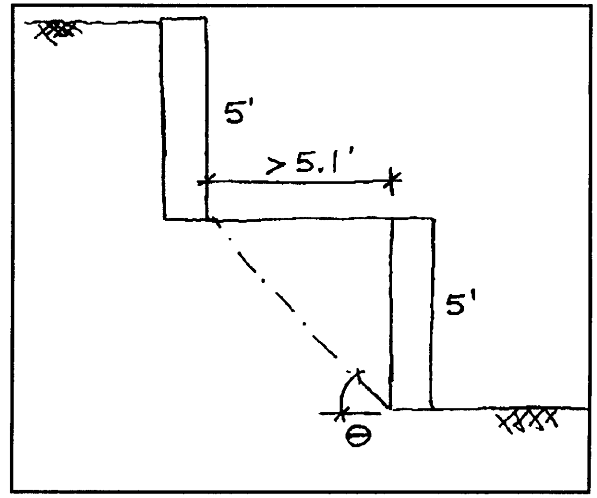


Figure 2.bg Stepped retaining wall, upper wall outside of lower failure wedge

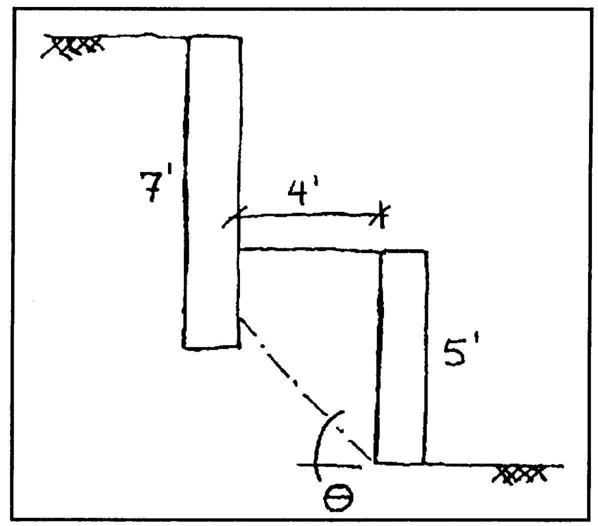


Figure 2.bh Stepped retaining wall, upper wall inside of lower failure wedge

Table 4: Steel required for reinforcement of different design configurations

Retaining Wall Type	0.2 g	0.4 g
5' + 5' Stepped	870 lb	1420 lb
7' + 5' Stepped	930 lb	1550 lb

Final Steel Quantities 2.6.3

The process so far described in Section 2.5 was used to quantify the amount of steel needed for reinforcement in the design example house, shown in **Table 4**. The cross-sectional areas of steel combined with wall dimensions from the plans allowed for a calculation of steel volume. As mentioned before, two different levels of earthquake acceleration were used. Some notes on the steel quantification process and **Table 4**:

- In the design example, #3 bars (0.11 in^2) were found to be sufficient for vertical reinforcement, and the UBC minimum requirement of 1 #3 bar placed every 4 feet in length was applied.
- At corners, 3 verticals are used, as suggested by various guidelines. (Using 1 vertical per corner would be sufficient, but saves only about 25 lb of steel.)
- The numbers presented are only *estimates*, because details such as splices were not included in the calculations.
- All vertical rebars are assumed to run from the top of the walls (including retaining walls) into the foundation a depth of 1 ft.
- Steel was assumed to have a unit weight of 490 $lb/ft.^3$.
- Various combinations are shown for the combinations of 1 or 3 vertical bars at each corner, and whether system of stepped retaining walls each 5 ft. in height is used, or if the 7 ft. and 5 ft. retaining wall system is used.

SECTION 3

ARCHITECT / ENGINEER INTERACTION

"One of the pleasures of engineering is the non-ambiguous nature of most scientific facts, but one of the greatest challenges of engineering is coming up with a design or an answer when no clear-cut solution exists." - Samuel Florman, <u>The Civilized Engineer</u>

General Description 3.1

This project was distinctive from most other design projects undertaken in architectural education because of the consistent involvement of engineers, from start to finish. Design students and faculty formulated questions for engineers throughout the design process, and tried to incorporate the information received into the plans. It was also different from the type of projects given in engineering education because the formulation of solutions and the solutions themselves were very open-ended. A very "real-time" consulting experience was given to the engineers as we received questions from the architectural designers and dealt with issues we also perceived necessary. The course of our work followed no set curriculum nor preconceived outline of topics, as in a standard academic class, but we had to formulate our own plan of investigation and react to issues as they arose. The fact that the two different specialties of engineering and architecture were co-dependent and could both affect the course of the housing designs made this a good opportunity to observe and evaluate the interaction between people educated in these two fields.

The final plans, and the information presented in the technical report may seem very straightforward to an outside observer, but this section will give a "behind the scenes" look at how our ideas were formulated, and how the final designs came about. Besides the evaluation of architecture and engineering interaction given in this report, I hope the reader can evaluate and learn for him/herself through reading and thinking about the synopsis of events presented.

Two main questions will guide this discussion:

- How has this design project benefitted from our interaction as engineers and architects?
- What have we learned from our interaction which could improve the teamwork among members of our respective fields?

It is unavoidable that this section presents issues more strongly from an engineer's point of view, and with that in mind, a heavier emphasis is placed on the things that an engineer can do to work effectively with architects.

Overview of the Interaction/Relationship 3.2

Historical Perspective 3.2.1

One can better understand the current issues in the relationship between architects and engineers with some knowledge of how the two professions developed with respect to building and construction. (Much of this section is taken from part one of Bridging the Gap, called "Schism.")

The greek root for architect is "architekton" meaning "master builder." This meaning well captures the role of the architect before engineering arose as a separate discipline. The classic master builder/architect of the pre-Eighteenth century understood the engineering concepts of the time, which were less quantitative than the present body of engineering knowledge. Along with carrying the responsibilities of design engineer, the architect naturally executed the overall design, and determined the method of construction, most often overseeing the building process. The architect would sometimes look to other people for their expertise in a particular area (i.e. stone masons), but overall the architect well understood all aspects from design to the finished product.

There are differing perspectives of how architects and engineers developed in relationship to each other. A common view is that the two professions were once unified and then split apart for various reasons, and developed into the situation we see today. This view is a bit too simplistic, and misses one very important point: modern engineering is relatively young compared to architecture - a newer development originating around the mid-Eighteenth and Nineteenth centuries. In this time, schools such as the École des Ponts et Chausées in Paris began to train students with a more technical focus, altering the old apprenticeship system. The industrial revolution of the nineteenth century distinguished the field of engineering perhaps more than it ever had been before. New materials, mainly iron, steel, and reinforced concrete brought about a need for people to be trained to design and build with them. Early, accomplished structural engineers such as Telford, Roebling, and Eiffel still played the role of "Master Builder" on projects like the Craigellachie Bridge, Niagra River Bridge, and Eiffel tower because of their understanding of new materials and engineering mechanics [**Figures 3.a** and **3.b** - both images taken from Bridging the Gap]. In a nutshell, the amount of knowledge and the complexity of concepts involved in building increased and spawned the specialized discipline of structural engineering. Due to this, the distinction between engineering and architecture became sharper, as it became increasingly necessary for an individual to focus their learning on one or the other field in order to understand it on a competent, professional level.

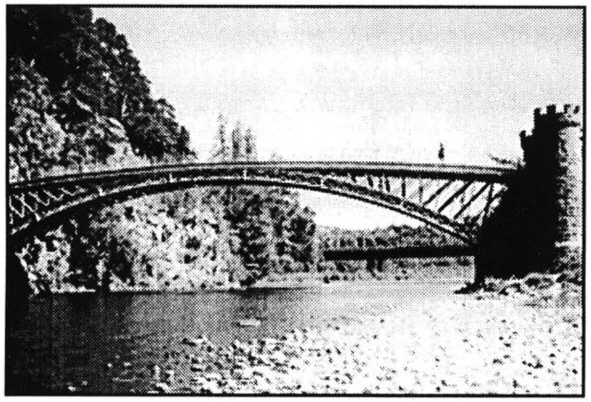


Figure 3.a Craigellachie Bridge, by Thomas Telford

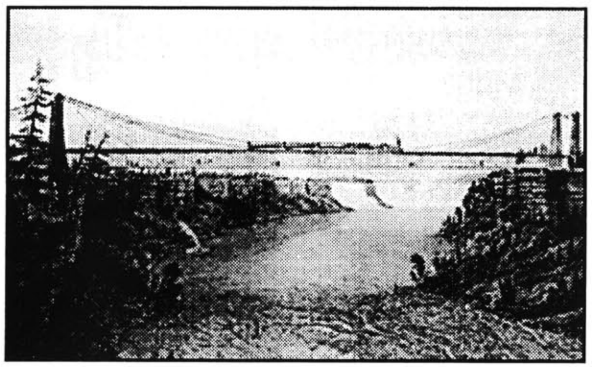


Figure 3.b Niagra River Bridge, by John A. Roebling

Today 3.2.2

In current practice, architecture deals with the more qualitative aspects of a building, such as it's form, spatial arrangement, and context in history, theory, and society. Structural engineering seeks to specifically quantify the forces and structural elements, and arrange them in efficient and effective ways. The nature of these roles can lead to a very confined interplay where the architect "decides how the building is going to look," and the engineer "figures out how to make it stand." This statement doesn't do any great injustice to the essential functions of the two disciplines, except maybe in oversimplification. The main problem with the statement is that it carries an attitude of separatism, suggesting that each field stays within its own boundaries. Letting each specialist do what s/he is best at may sound like a good idea, but in the case of buildings, the form a structure deeply affects how it functions, so decisions made by architects will have impact on the substance that engineers handle, and vice versa.

In the professions of architecture and structural engineering today, project designs are developed in a spectrum going from isolationism to interactive synthesis. Despite the variances in practice, there seems to be a general consensus among professionals and educators that effort must be made to promote and improve the teamwork between the two disciplines.

There are several principles that could be followed to promote good interaction between architects and engineers. Some are obvious to anyone who has thought about teamwork in general, and some come from people who have worked and reflected on the fields. These principles include:

- Having an understanding of the current situation, and the nature of the two professions: Architecture and engineering are not independent, yet they are also distinct disciplines for legitimate reasons. The body of knowledge necessary to perform the functions of both fields, especially in larger projects, is too vast for one person to master. A healthy, positive acceptance of collaboration, and respect for the other discipline, will help overcome many of the personal barriers to effective teamwork.
- Communication: This is the essential element in any collaborative effort. One can see, as the process of design in this project is reviewed, that clear and effective communication between architects and engineers is more than a matter of just meeting in the same room and talking. (Although that in itself is an essential first step.)
- Keeping an open mind and being flexible: People in both fields must acknowledge the validity of ideas and purposes of those outside their professions. This applies particularly to engineers, who must be willing to examine alternative ways to make things work, as opposed to vigorously promoting the most obvious scheme without giving fair effort in evaluation of others. Engineers should also acknowledge that non-quantifiable issues can be valid constraints.

Basic Organization of the Fall 1993 Teams 3.4.1

In the fall of 1993, a team of engineers and architecture students were enrolled in a class which ran parallel to a design studio focusing on Karimabad. The technical team included two professors from Building Technology, one of whom focused on structural issues, and the other on energy/heating issues. There were also 5 students from Building Technology, 3 focusing on energy/heating, and 2 (including myself) working on structural engineering. In addition to this, a few architecture students were in the technical team also, to provide design insight, and to help with information gathering. The design studio met in a separate building, and there were no regularly scheduled meetings between the two classes. The professors from the technical team would often attend the studio reviews, and communicate ideas and suggestions from the technical work. And once or twice, students from our team would go to the studio and present our work. As structural engineering students, we visited once, to present and discuss some basic concepts of earthquake resistance, and addressed the issues of shear walls and frame systems. The rather separated organization of the technical and architectural design teams was probably due more to natural precedent and school culture as opposed to any direct intention of segregation. At the time, architecture studios generally met in building N52, while Building Technology classes were usually held on the main campus. The inclusion of a technical team, even though separate in location and meeting times, was seen as a positive step forward, since most studio classes generally don't have a supplementary technical team.

Events in Fall 1993 3.4.2

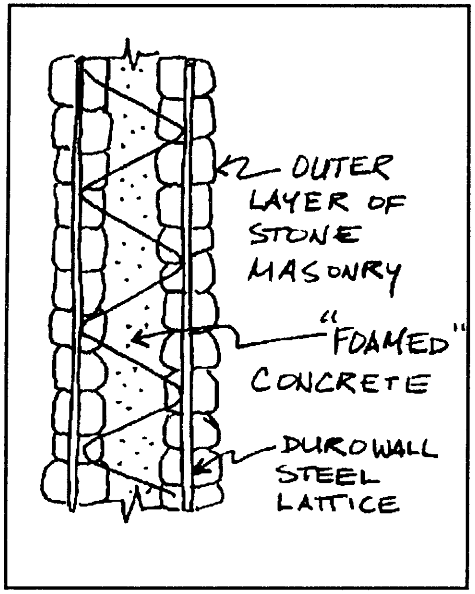


Figure 3.c Sandwich wall idea, cross-section

Much of the background and cultural information was passed on to us in the beginning from architecture faculty who were already familiar with the town, and the changes taking place. We got a sense of the culture, societal changes, land use issues, and traditional ways of building that were discussed in the introduction of this thesis. The first information searches were geared towards material availabilities and environmental factors such as seasonal temperatures and regional seismicity. The people working separately on the issues of thermal insulation, and structural design worked very closely together at this early stage. One of the earliest innovative ideas was suggested by the professors, involving the use of foamed concrete sandwiched between two layers of stone, and reinforced by a steel lattice system (a commercial product called "Durowall" was considered). [**Figure 3.c**] The concept was to efficiently resolve the structural and heating issues in one system. Meanwhile, other appropriate technologies were researched because there still wasn't a definite sense of what would be feasible in this region.

My main goal throughout the semester mainly involved finding ways to analyze the structural systems we were likely to be using. Coming straight from an undergraduate education, I was very unsure of how to proceed with any kind of design work before a suitable method of analysis was found. In retrospect, this was probably a backwards approach. What happened was that in my investigation of alternative materials and systems, undue priority may have been given to familiar materials used in familiar ways simply because I could understand their analysis. More emphasis should have been placed on first finding appropriate systems of construction based on cultural and environmental considerations. It's interesting to note that the push for analysis tools caused us to develop a spreadsheet that could analyze shear walls according to UBC guidelines, even as we began to gain only a very basic understanding of seismic behavior.

As we reached the midpoint of the semester, there had been little contact with the design studio, and the technical students didn't have a good idea of what was happening in the studio. Some designs were underway that were multi-storied, and relied on moment resistant framing systems. These designs facilitated incremental construction for expandibility, and a flexibility in wall openings, which were two goals of the design studio. However, as the engineers began to understand the implications of shear wall systems versus moment resistant frames, we suggested that shear walls would probably be a better alternative. This was one of the difficult points of communication. Because the studio designs were already well underway, and the amount of time it took for the engineers to formulate quantitative answers about the two alternatives, most architectural designs proceeded with whichever system they had started using. Two of the studio designs relied heavily on moment resistant frames.

In the rest of the semester, the structural engineering group looked at roofing alternatives, particularly the idea of vaulted arched roofs. An analysis technique for a shear wall with openings was learned, and steel reinforcement in walls for out of plane bending was calculated. Reinforcing steel for out of plane bending relied on an assumption of a fully rigid roof, and used a primarily vertical scheme of placement. (See Section 2 and Appendices for a full explanation of these concepts.)

Along the way, it was decided that the foamed concrete alternative would probably not be feasible because of its complexity, and we also discovered that as the density of foamed concrete was lowered to provide adequate insulation, it became extremely weak structurally. Although it wasn't necessary to use it structurally in the walls, some compressive strength would be necessary if it was to be used in the roofing.

At the end of the semester a final presentation was given where students highlighted their work and final products. The structural engineering students presented the basic concepts of seismic resistance, and quantified reasoning for using shear walls over moment resistant frames. It was also shown that shear walls were not as restrictive on openings as was originally thought.

Spring 1994 Workshop 3.4.3

A workshop was conducted in the following semester to continue with the same design issues. However, a new group of architecture students was brought in, and designs started from almost ground zero. This time, only two building technology students were involved, Greg Sullivan with heating/insulation issues, and myself continuing with structural concepts. The workshop met every week, with engineering and architectural design people all present. This was a fundamental difference in organization from the fall semester.

One of the first issues discussed, was the feasibility of one versus two story houses. By this time we had a better idea of the limitations of stone construction, and the technology available in Karimabad, so I suggested one story homes, with functional roof space, much like the traditional construction. This suggestion was well taken, after I presented a few simple calculations showing lateral force as a function of the building height.

The usage of a shear wall system was generally agreed upon, and some sort of reinforcement would be decided upon for tensile capacity. We looked into the possibility of using bamboo as reinforcing, and the idea seemed quite promising at the time. For much of the early semester Greg Sullivan and I investigated the growth and culturing of bamboo, and structural usage respectively. The information was presented during the midterm review. This technology really didn't affect the design decisions of the architects, since we were dealing with alternative types of reinforcement within a shear wall system. Walls would basically function the same way, and follow the same guidelines of form, no matter what kind of reinforcing was used. This alternative proposal was eventually dropped because an adequate bond between bamboo and concrete is difficult to achieve, and it is uncertain how long bamboo will last when cast inside concrete.

From late February, Greg progressed into heating and insulation analyses, and I was faced with the task that would take up the majority of the semester - analyzing and understanding retaining walls. Overall, there seemed to be good reason for building the homes with one wall directly up against the earth. First, it seemed more economical to use the required or existing retaining wall as the part of the home. There was also thermal benefit to this placement. A study was done by the architectural designers which demonstrated the area of land savings that could be achieved with such a system. Finally, there was a hope that somehow the mass of the house could be fixed to the earth behind it, and in that way lateral loads from seismic acceleration might be reduced. Various types of walls were investigated, and the necessary reinforcement schemes were compared. The behavior and design of retaining walls under seismic loads was completely new to me, and much time and effort was spent in understanding the Mononobe-Okabe design equations used for that purpose.

During the semester, it was necessary to answer the questions about general wall layout posed by the architects. Most designs followed the general layout guidelines mentioned in Section 2.5.3 and a good balance of shear walls was available in the two axes of the homes. The architects understandably had to proceed with designs even though no solid answer regarding the best retaining wall system was available for most of the semester. In fact the whole issue of retaining walls seemed to be open-ended, and the "best solution" changed depending on what priorities one had for the design.

At the end of the spring semester, most designs had very sensible shear wall layouts, and although not enough time or tools were available to engineer all the designs, the plans seemed very feasible for construction. They utilized many traditional methods of construction, and the steel placement was relatively simple. In fact, when we traveled to Karimabad in August as a design team, our models and plans were favorably received. The proposed houses were attractive to the people and leaders who interacted with us, and also seemed very accessible.

And so on... 3.4.4

In the following summer and next school semesters, I had the chance to finalize the design and analysis procedures used to quantify the wall steel (presented previously in this thesis). One should note that it really wasn't until the summer of 1994 that I had a clear and correct design process for retaining walls. And my initial assumptions in the Fall 1993 semester for calculating the reinforcing steel in general walls weren't as valid as the conditions and assumptions of behavior that I learned about in the IAP session of January 1995, when I finalized the analysis process for sizing the reinforcing steel in walls.

We also made a trip to Karimabad itself in the August 1994, and it gave us a first hand view of the possibilities and limits of the region. In my interaction with architects in times following the summer of 1994, I could give answers about general form and point out possible areas of concern with reasonable confidence, because of my relatively developed understanding of the seismic behavior of heavy wall structures. Giving solid numbers was still a difficult and time consuming process, and often my suggestions were given based on intuition and experience rather than calculation. (Intuition that was based on past calculations of similar structural elements.) I became less rigorous in trying to base my recommendations on calculations because I realized that many design decisions could not wait for a thorough analysis of alternatives, and I also had a sense that as long as the basic design was reasonable in form, it would be possible to perform calculations to specify the details for adequate strength at a later stage. This realization reflects a fundamentally important part of the architect/engineer collaboration. The engineer gives guidance based on broad principals, the architect has reasonable creative latitude, and design development iterates toward final design and specification.

Ideals of teamwork and interaction are important things to strive for, and that is what we attempted to do in this project. However, in many things that involve human interaction and teamwork, things are easier said than done. What have we learned from our experiences together as architects and engineers, through the course of this project? Here are some conclusions and suggestions:

A clear understanding of the overall program, and goals of the project will help the working relationship greatly. One of the main reasons teamwork was so well accepted from the architecture side of this project was the clear understanding that we were trying to design homes that would perform more safely and effectively than what has previously been achieved under the many technological restrictions and environmental conditions in Karimabad. The architects realized that the engineers would need to affect the form of the design in order to use the available materials and technologies to their fullest measure. On the other hand, engineers were brought in from the very beginning to hear and understand the cultural and societal issues involved with new construction in Karimabad. This made the engineers respectful of the design goals relating to tradition, arrangement of spaces, openings in walls etc. that would affect the structural form, and so we sought to accommodate forms that would achieve those goals.

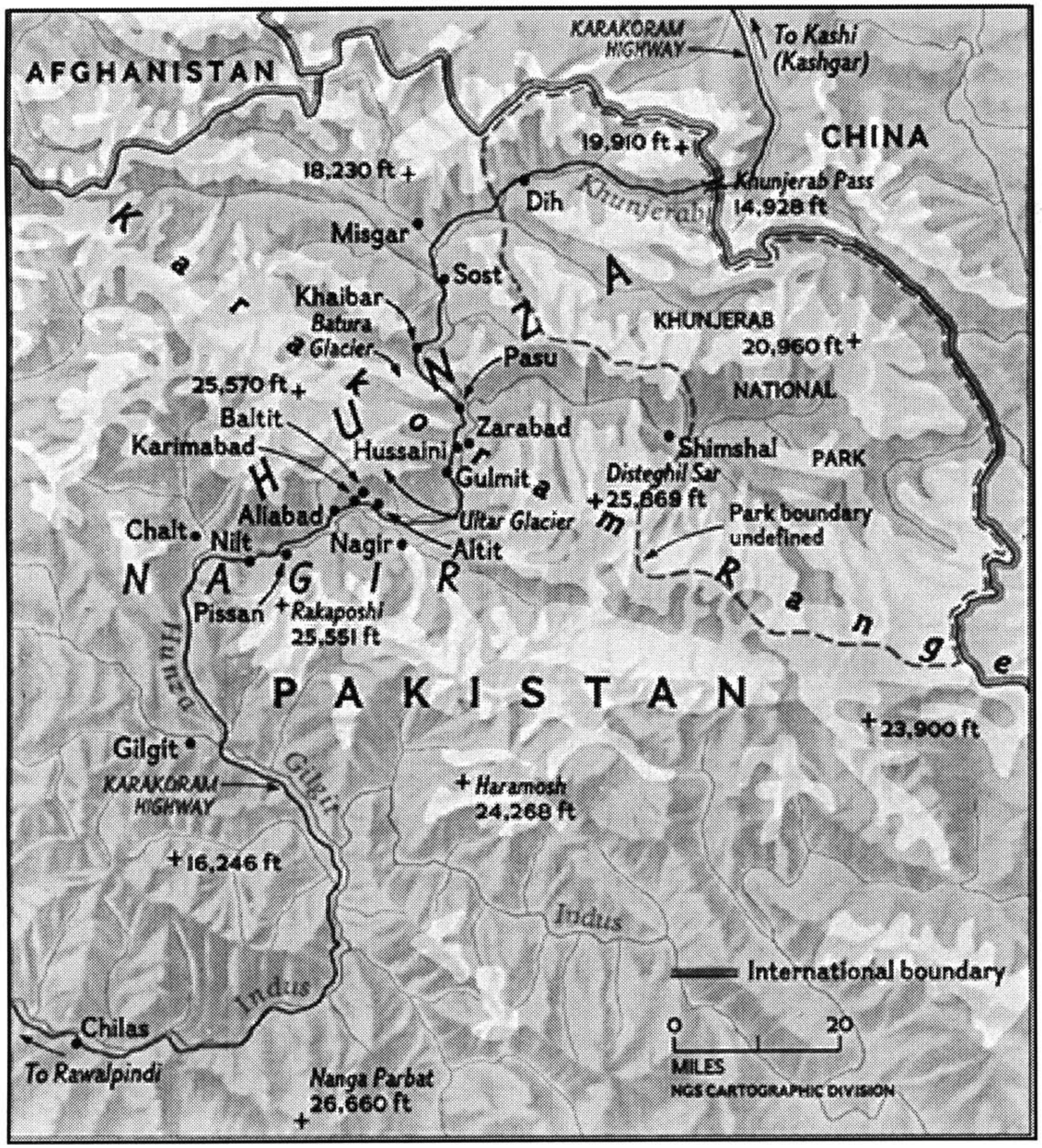
It's interesting to note that in the Spring 1994 workshop, although the interaction and communication was much better than in the previous semester, it seems that a smaller quantity of things were done by the structural engineering side. There are several incidental reasons for this. In the first semester, there were two students working on the structural issues, whereas in the next, I was on my own. My course load in the second semester was also heavier. But the more interesting reason, was ironically due to the increase in interaction between architects and engineers. When people operate in a vacuum, without the questioning and input of other groups, they can often get more apparent work done. But the work done in a vacuum is less likely to be well incorporated into the group project, and may even be so off base as to be useless. Responding to the questions of architects, or any outside group for that matter, will often take one's own work in unexpected directions, and an overall sense of progress may be more difficult to maintain. But that is a necessary factor in collaborative efforts, and in the end, a much better product was made.

To get beyond just mutual respect of each other's professions, and achieve a truly effective working relationship of synthesis, there must be TRUST between architects and engineers. It is unfortunate for projects with tight deadlines involving people who are strangers to one another that trust takes time to build. I would say that the working rapport developed between myself and the architects in this project took the better part of a year. However, personal knowledge and experience can help develop this relationship of synthesis, even if a time span is short.

Regularly scheduled meetings are very important. Informal, on the fly collaboration, like that in the Fall 1993 studio was not as effective because people did not meet and discuss the progress and thinking going on in both sides. When people touch base very infrequently, each side progresses independently, and most likely in different directions, so the work done by one group is not applicable or easily incorporated into the other.

Identify the work that is important to design decisions, and get it done as quickly as possible. This lesson stands out the most to me, as I look back on the project. Much time was spent on things that were not as important to the early design decisions, at the beginning of each semester. By the time structural engineers had formulated definite suggestions regarding those issues, many of the basic design decisions had already had to be made by the architects, and really, if those decisions were later shown to be wrong, no one could blame them. Architects could not just sit around for weeks waiting for an answer from engineers before they could start designing. And in the reverse situation, it's equally inefficient for engineers to remain idle while architectural decisions are pending. If the important work can be accurately determined early in a project, there will be less "lag" time and/or progress by one side lacking the collaboration of the other.

APPENDICES



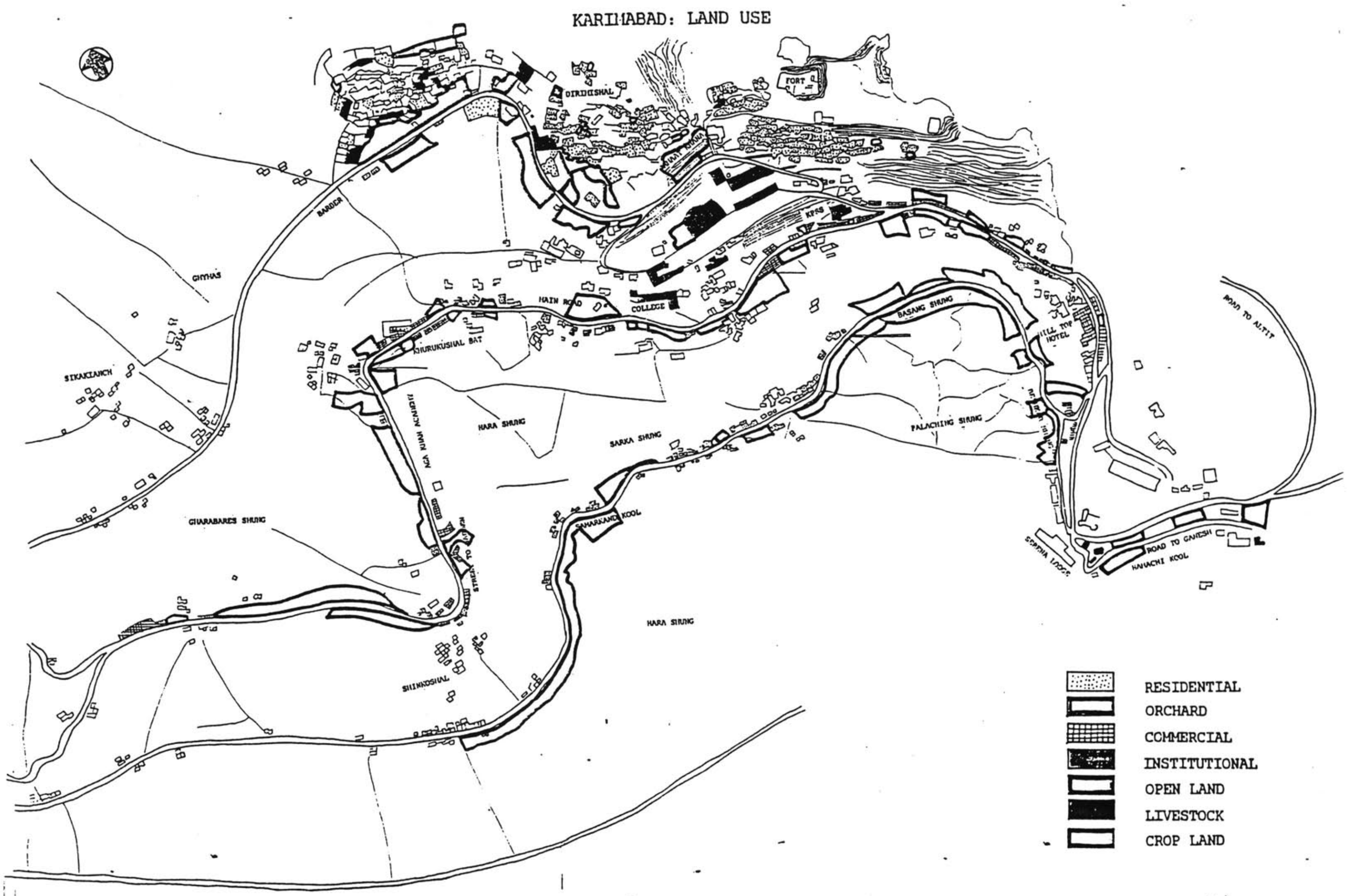
Map 1

Karimabad and Surrounding Region (McCarry, 1994)

Map 2

Karimabad: village layout (Ali, 1993)

In general, the valley site of Karimabad is much like a bowl. Unfortunately, this map does not convey topography very well. The large blank area to the left of the map legend can be thought of as the bottom of the bowl, with the valley and town rising around it.



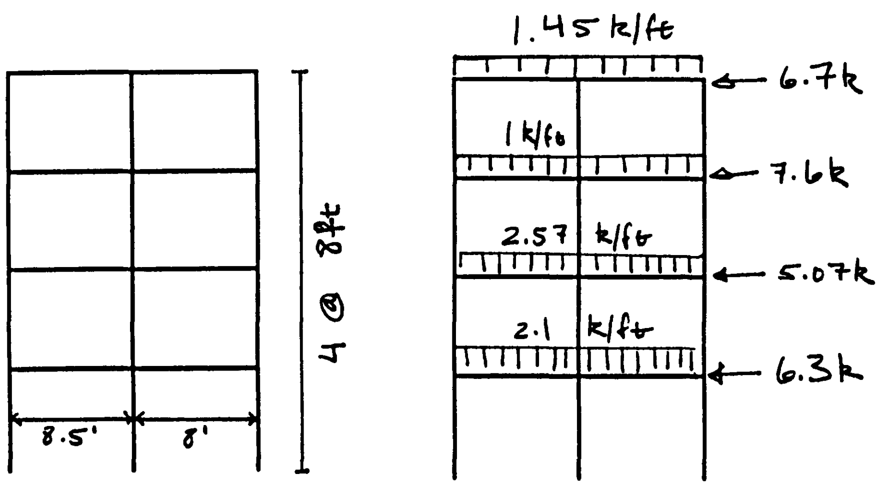
Appendix B Analysis of Fall 1993 Studio Designs

The following analyses and column interaction diagrams were used to determine the necessary steel reinforcing for the moment resistant frame designs of the 1993 Fall studio. A standard method, taken from Nilson and Winter Design of Concrete Structures (1991) was used.

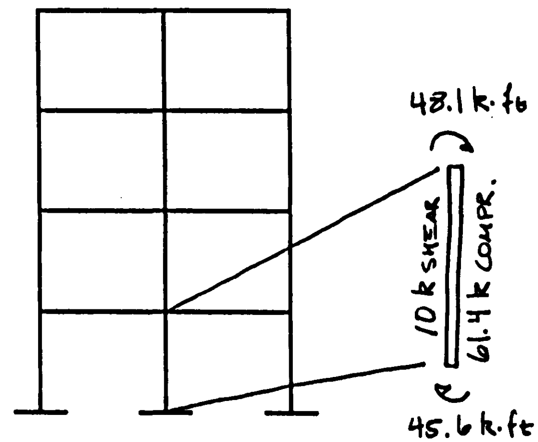
Frame System Example

One particular design involved multi-story sections (>3) and complex floor plans making it appropriate for a reinforced concrete frame system.The member taking the highest loads in this frame system was designed as follows:

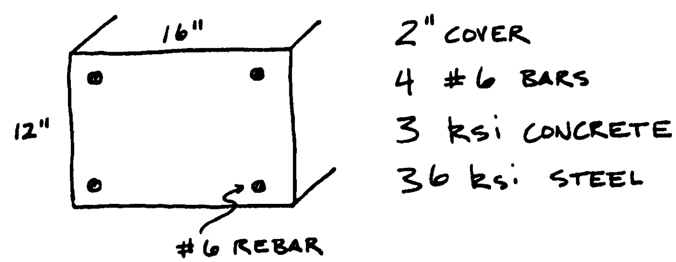
- A column - beam system that would fit the layout of the design was determined floor by floor.
- The frame segment that took the most loads was determined.
- The Layout Analysis Spreadsheet was used to redistribute the lateral loads on each floor as described in the UBC. The frame segment analyzed was this one:



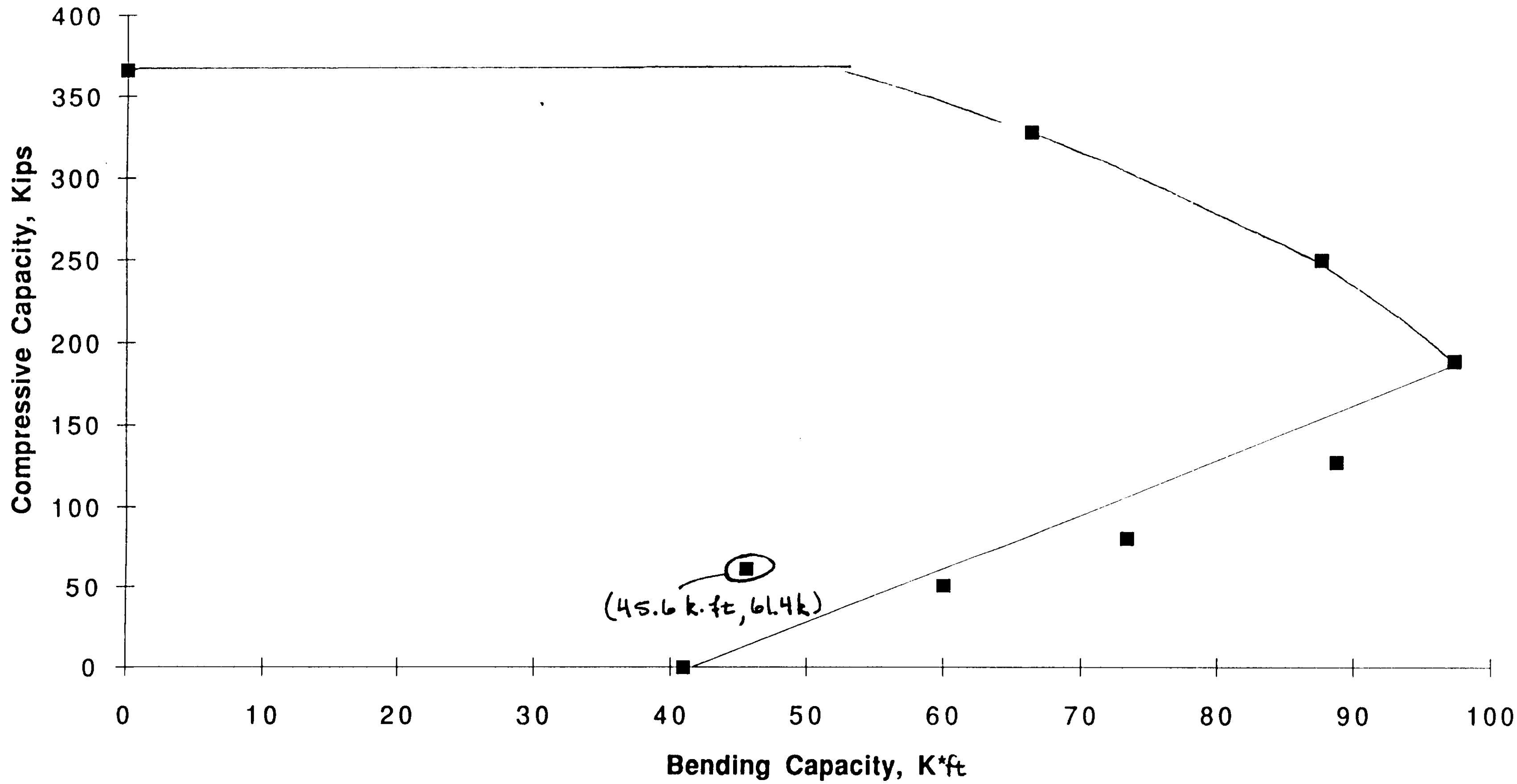
• Using GrowlTiger, a structural analysis program available on Athena, the most heavily loaded column turned out to be this one.



• After testing various column sections with the Interaction Diagram Spreadsheet, this section was found to be adequate:



Interaction diagram for frame system example

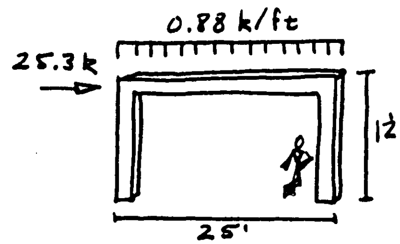


Hybrid (Frame and Shear Wall combined) Example

One studio designs was made of 3 solid walls along the north, east, and west sides, with large openings in the south wall. This design was suitable for a combined shear wall and frame system.

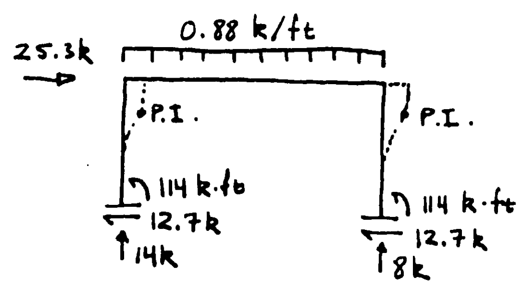
A preliminary design for the columns of the frame component was performed as follows :

• General dimensions of the frame component (from Albert's design)

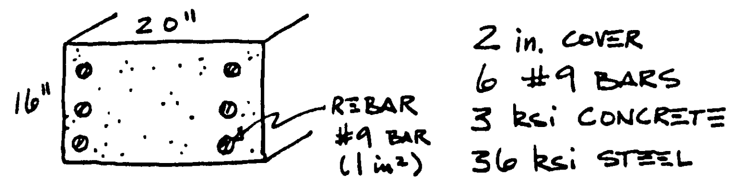


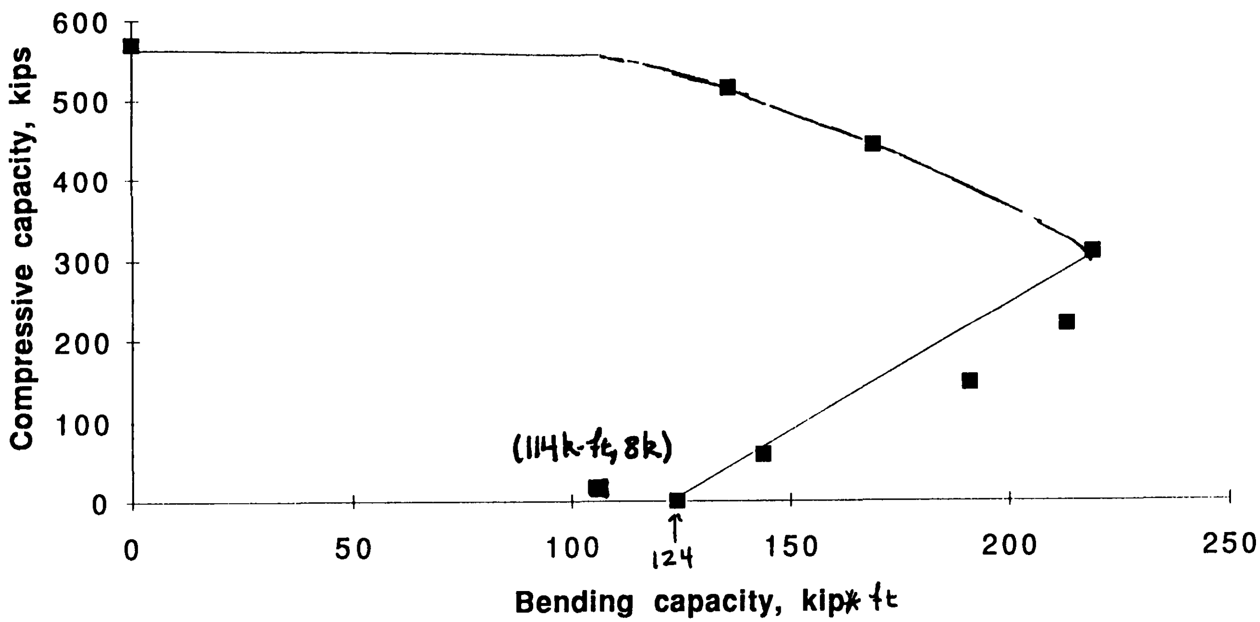
• Putting the building through the Lateral Force Distribution Spreadsheet gave the lateral load shown above. (25.3 kips)

• The frame was analyzed by hand, assuming 1/2 full frame action - meaning the frame was assumed to be somewhere between fully rigid and pin connected at the joints. Reactions are as shown:



• Various column cross sections were tested on the interaction diagram, and a 20 in x 16 in section, with six #9 bars was determined adequate.





Arched Retaining Walls C.1

Arched retaining walls were designed in a similar fashion to simple retaining walls. The earth pressure and overturning forces are calculated from the Mononobe-Okabe equations as shown in Section 2.4.3. The main difference for this type of wall is in the approximation of the resistive moment to overturn (M_r).

$M_r = W_s \times 1/2(t + rise)$
W_s = self wt. approximated by $lth\gamma_{stone}$

Figure C.a illustrates the variables.

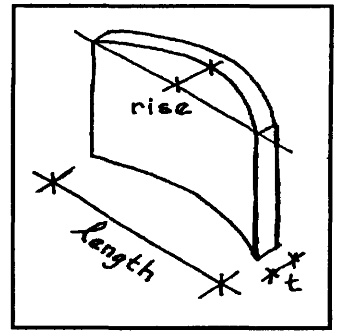


Figure C.a

When the overturning moment (M_o) exceeds M_r, then the excess Mo is converted back to a load/horizontal length value and the arch is analyzed as shown in **Figure C.b.**

T_h = horizontal thrust = $wl^2/(8rise)$
V_h = vertical thrust = $wl/2$

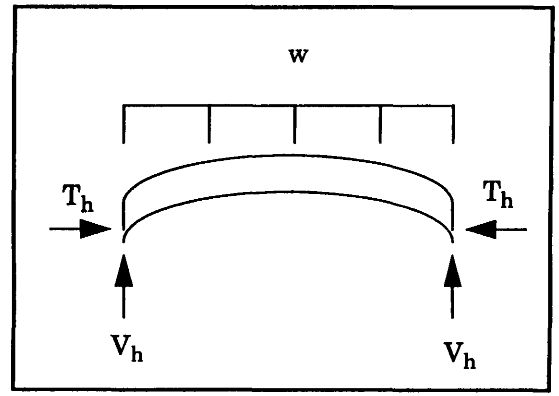


Figure C.b

To determine the dimensions of parallel and perpendicular buttress walls, the following equation is used:

Resistive moment of buttress wall[1] (M_{br}) = $lW_s/6$

1. Note: $M_{br} = lW_s/2$ was not used because at the time this procedure was developed, the system of reinforcment had not yet been decided. $lW_s/6$ was used in order to assure that no tension would develop within the buttress walls.

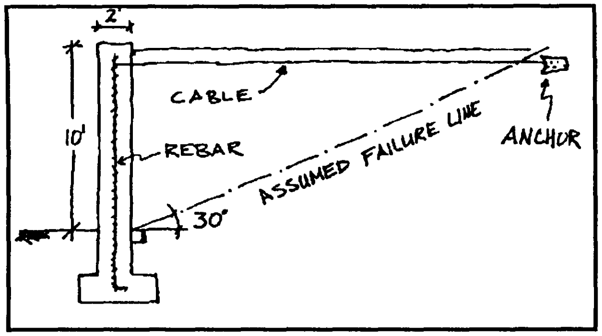


Figure C.c Tie-back retaining wall configuration

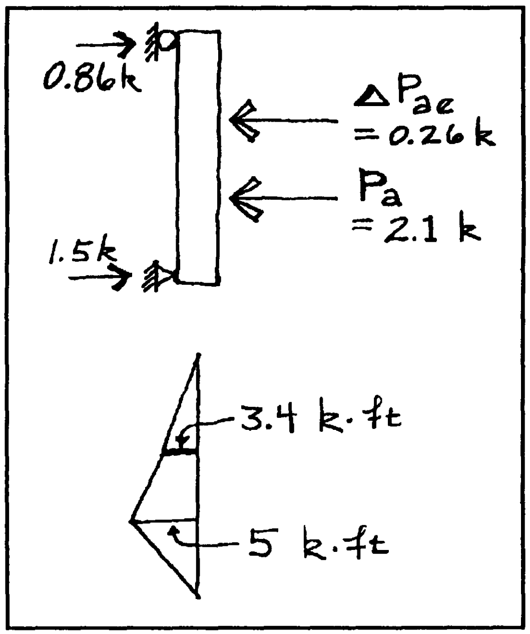


Figure C.d Model of tie-back retaining wall

where l is the horizontal length of the wall, and W_s is the self weight.

Tie-back Retaining Walls C.2

Tie-back retaining walls were engineered for with the configuration shown in **Figure C.c**, and the following material values:

$\gamma soil =$ 100 pcf
$\gamma_{stone} =$ 130 pcf
γ_{eq} = equivalent fluid wt. of soil, 42 pcf

The retaining wall was modeled as a simply supported beam[1], as shown in **figure C.d**. with the resulting moment diagram (per linear ft. of wall).

$\Delta P_{ae} =$ 0.26 k applied at 0.6 H (dynamic component)
P_a = 2.1 k applied at 1/3 H (static earth pressure)

$M_{max} =$ 5 kft at 1/3 H

Using equation 2.h, and substituting M_{max} for M_o, gives a required steel cross section of 0.17 in^2.

1. The lower support of the retaining wall is at least partially fixed, and it is possible for tension to occur on either side of the wall. The rebar is placed in the middle of the wall thickness so that it can resist tension from either side of the wall.

The required cable steel cross sectional area is: 0.86 k/ 36 ksi which equals 0.024 in^2. (This is about a 1/6 in. diameter.) And a cable length of about 18 ft. would be required for keeping the anchor outside of the soil failure wedge.

The total volume of steel needed per linear ft. of wall = 25.3 in^3, which translates into 107 lbs of steel for a 15 ft. long, 10 ft. high wall.

Spreadsheet for Mononobe-Okabe Calculations C.3

The following pages show the spreadsheet incorporating the Mononobe-Okabe equations used to calculate steel sizes for simple retaining walls and buttress sizes for arched retaining walls.

All values that are inside the double-lined boxes are input parameters that can be changed (i.e. unit weight of stone), and the arched retaining wall section of the spreadsheet uses the same parameters, with some additional inputs, such as rise and thickness of buttress walls. (Note: the values marked A, B, and C are just some intermediate cells necessary for holding different parts of the large Mononobe-Okabe equation calculations. The important results are given in column format.)

Spreadsheet section for simple retaining walls

Inputs

		Theta =	11.30993247				
unit wt soil (pcf) =	100	steel str ksi =	36	equivalent fluid wt of soil(pcf)=		42	
soil friction angle =	30	rad to deg	0.017453293				
backfill slope =	0						
wall/soil frict. angle =	30	*A*	*B*	*C*			
Beta =	0	0.89731258	0.736562888	2.585113917			
Kv =	0						
Kh =	0.2						
Wall mat'l selfwt pcf =	130						
wall thickness (ft) =	2						

Per linear foot of wall length

Height	Kae	Pae (lb)	dynamic Mo (k*ft)	self wt Mr (k*ft)	In^2 Area steel	(Inertia) Mom. due to selfwt(k*ft)	Pa (lb) static force	(kft) Mom. due to static comp.
1	0.4713	24	0.00	0.26	0.00	0.03	21	0.007
2	0.4713	94	0.01	0.52	0.00	0.10	84	0.056
3	0.4713	212	0.04	0.78	0.00	0.23	189	0.189
4	0.4713	377	0.10	1.04	0.00	0.42	336	0.448
5	0.4713	589	0.19	1.30	0.01	0.65	525	0.875
6	0.4713	848	0.33	1.56	0.02	0.94	756	1.512
7	0.4713	1155	0.53	1.82	0.05	1.27	1029	2.401
8	0.4713	1508	0.79	2.08	0.08	1.66	1344	3.584
9	0.4713	1909	1.12	2.34	0.12	2.11	1701	5.103
10	0.4713	2356	1.54	2.60	0.17	2.60	2100	7.000

Spreadsheet section for arched retaining walls

Arched retaining walls:

length (ft)=	15	thickness of	
rise (ft) =	3	buttress walls (ft) =	2
center of gravity at (ft)	2.5		

height	self wt. wall (kips)	overturn resist(kft)	Tot. dynamic and static overturn mo.	Inertia overturn mom. (kft)	overturn mom. to resist (kft)	residual lateral thrust (k/ft)
1	3.9	9.8	0.13	0.39	0.00	0.00
2	7.8	19.5	1.02	1.56	0.00	0.00
3	11.7	29.3	3.46	3.51	0.00	0.00
4	15.6	39.0	8.20	6.24	0.00	0.00
5	19.5	48.8	16.01	9.75	0.00	0.00
6	23.4	58.5	27.66	14.04	0.00	0.00
7	27.3	68.3	43.93	19.11	0.00	0.00
8	31.2	78.0	65.57	24.96	12.53	0.17
9	35.1	87.8	93.36	31.59	37.20	0.46
10	39	97.5	128.06	39.00	69.56	0.77

		perpendicualar thrust (k)		w/o steel	length of perp. walls (ft)	
height	parallel thrust (k)	at end of one arch	at junction of two arches	length (ft) of parallel walls	at end of one arch	at junction of two arches
1	0.00	0.00	0.00	0.00	0.00	0.00
2	0.00	0.00	0.00	0.00	0.00	0.00
3	0.00	0.00	0.00	0.00	0.00	0.00
4	0.00	0.00	0.00	0.00	0.00	0.00
5	0.00	0.00	0.00	0.00	0.00	0.00
6	0.00	0.00	0.00	0.00	0.00	0.00
7	0.00	0.00	0.00	0.00	0.00	0.00
8	1.63	1.31	2.61	4.75	4.25	6.01
9	4.31	3.44	6.89	7.72	6.91	9.77
10	7.25	5.80	11.59	10.02	8.96	12.67

This section includes Mike McCormick's original report section on vaulted roof sections. Only general information and results are included, refer to the original report (Structural Strategies for Northern Pakistan, Fall 1993) for a more complete treatment.

Types of Roofs and Floors

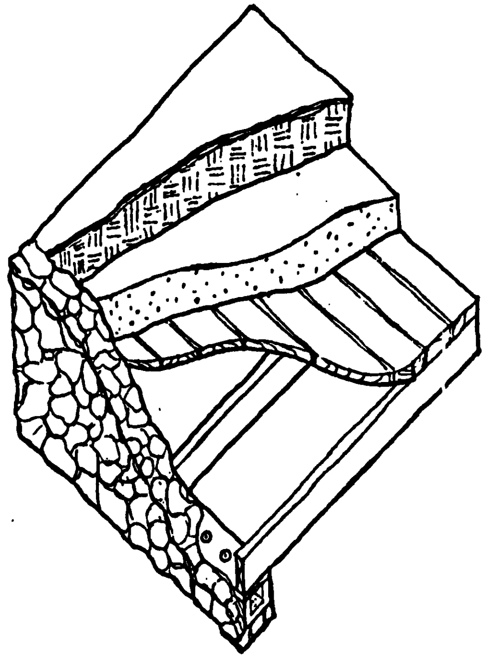
In examining how the roofs and floors should be constructed in Karimibad, many options were considered. Each was evaluated on the basis of its *viability*. This means that each of the following was considered as being important to the success of the system:

- Weight of the individual pieces (for ease of assembly)
- The skill level required to assemble the pieces
- The equipment and machinery required to manufacture and assemble the pieces
- The simplicity and the strength of the connections
- The availability of materials
- The cultural appropriateness

The systems evaluated include:

- Wood planks on wood joists (nailed together)
- Pre-cast concrete planks on wood joists
- Wood planks on steel joists (screwed together)
- Pre-cast concrete T-beams
- Poured-in-place concrete waffle slabs
- Poured-in-place ribbed concrete slabs
- Doubly curved vaults (made of bricks)
- Square vaults on wood beams

Of these systems, most were disregarded at an early stage because of constructability concerns. We also found ourselves faced with a difficult problem. Karimibad's limited wood resources and the complete lack of steel seemed to discourage the use of any system where elements were loaded in bending, as tension would be induced within the members. However, historically, the floors and roofs were almost entirely made out of wood. These considerations caused us to focus on three schemes for further analysis: wood planks on wood joists, precast concrete planks on wood joists, doubly curved vaults, and square vaults.

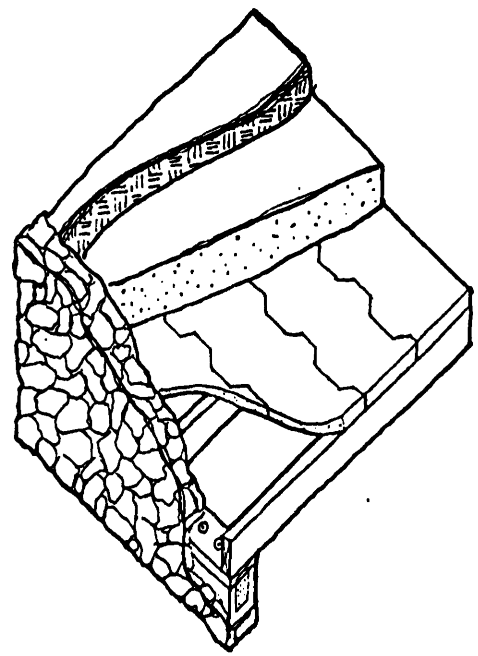


<u>Wood Planks on Wood Joists</u>

This system is obviously very simple and easy to construct. Good adequate connections are relatively easy to make, ensuring proper diaphragm action. It is also the most light weight system, thus reducing the inertial loads that would develop during an earthquake. However, this system is the most wood intensive of the options. The following figure shows a typical application with a foamed insulation on top of the wood planks and a soil/silt mixture on top of this to provide waterproofing.

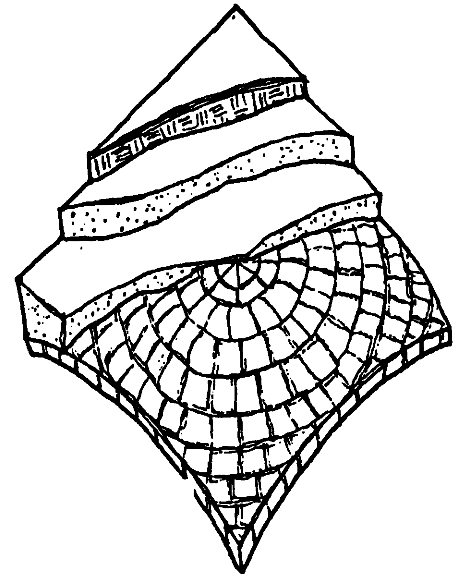
<u>Pre-cast Concrete Planks on Wood Joists</u>

This system is similar to the previous one except concrete planks are used instead of wood. This takes advantage of wood's strength to weight ratio in the long members, a benefit to those lifting the joist and beams into place, but it uses a different material for the planking. The drawback to this system is that the connections involved can be much more difficult. Attaching the concrete planks to the joists is difficult enough, but providing an adequate plank-to-plank connection to prevent in-plane shearing would be the biggest challenge. The following figure showing pre-cast concrete planks with"teeth" represents one possibility in strengthening this connection.

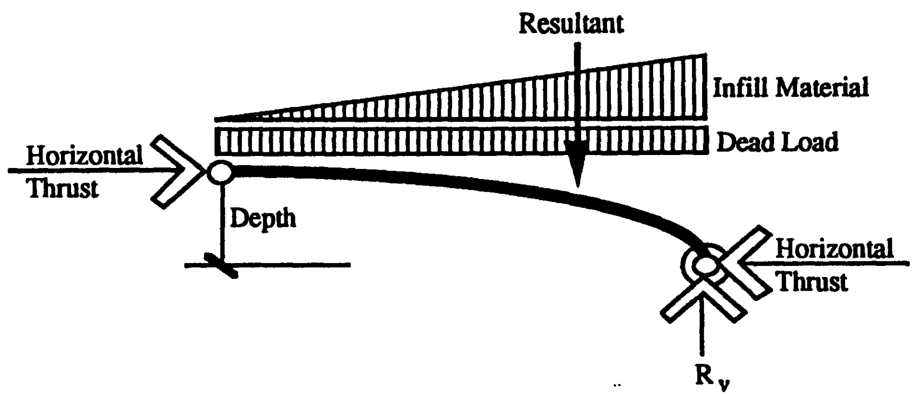


Doubly Curved Vaults

The primary advantage of this system is that *all* loads are resisted by compression. The vaults could be formed with brick, filled in with silt to form a level surface and topped with insulation and a soil/silt mixture. At each of the four corners a column would transfer the load directly to the ground.



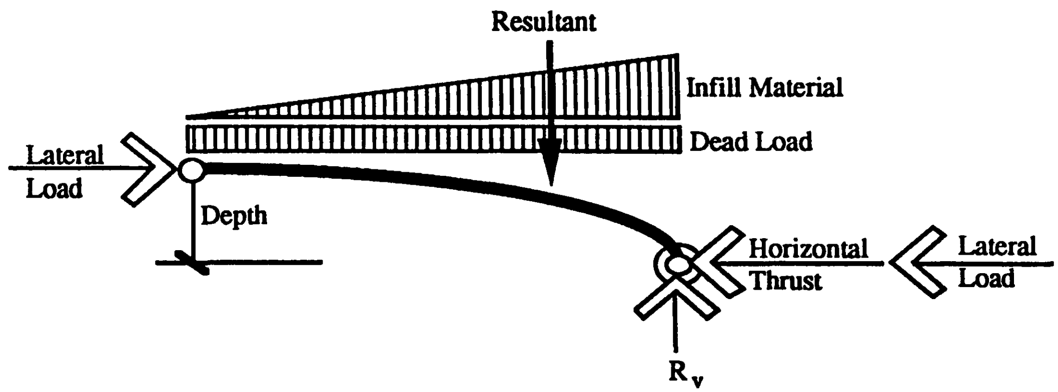
In order to fully understand the implications of this system, we found it necessary to conduct further analysis. In following free body diagram, if moments are summed about the support point, it is apparent that a horizontal thrust is required to satisfy equilibrium. This thrust must then be resisted at the support point. As one can see, by increasing the depth of the vault (the moment arm), the horizontal thrust decreases.



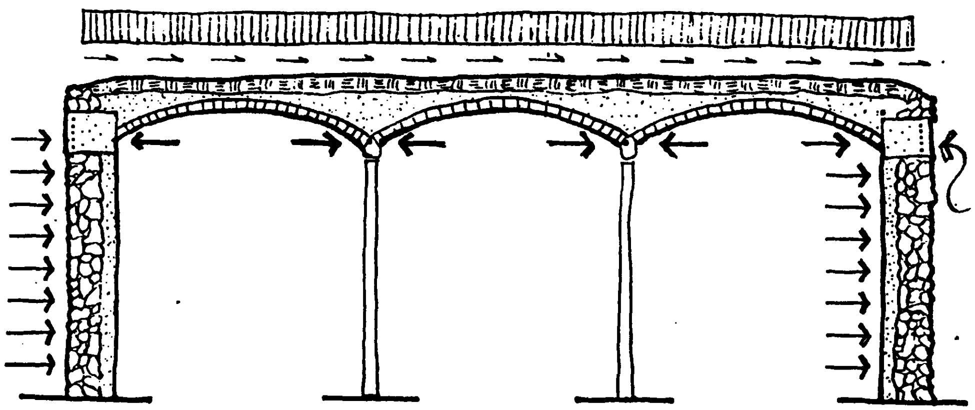
However, as a lateral load is applied the situation changes. At the top, the thrust is essentially provided, at least in part, by the lateral load. If, on the other hand, the horizontal thrust has been reduced to the point where it is actually less than the lateral load, the gravity load will not be adequate to maintain equilibrium, thus causing failure. The optimum span-to-depth ratio of the vaults comes when the horizontal thrust is the same as the total lateral load of the roof. However, this depends on several variables, including:

- The combined earthquake factors
- The number of vaults
- The dead load
- The live load
- The weight of the infill material

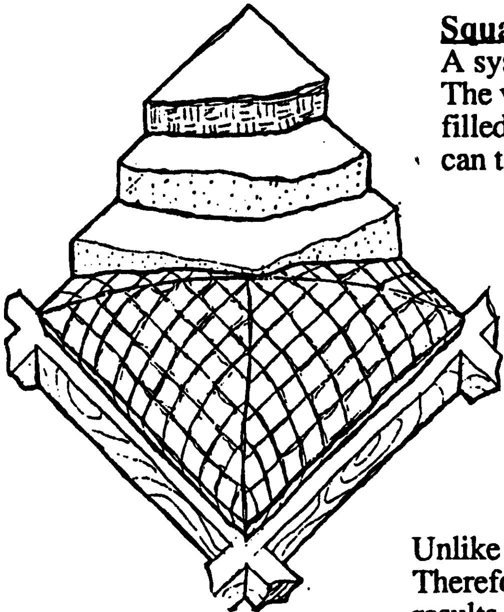
If equilibrium is maintained, as is usually the case, one can see that at the point of support, the thrust plus the total lateral lateral load must be resisted.



If we look at a typical layout that is 24' x 24' with nine 8'-0" square bay, the result can be seen in the following diagram. As the gravity load is applied from above, horizontal thrust is developed in each bay. However, at the two central columns this thrust is simply resisted by the other vault. During an earthquake, the inertial load from the first wall will be used to resist the horizontal thrust of the first vault. Then, the inertial load from the entire roof be resisted by the ring beam on top of the second wall. At this point the beam must also resist the outward thrust of the last vault and the inertial load of the wall. The end result is an extremely high outward force on the perimeter beam. Furthermore, if there are no exterior buttresses, the span of this beam will be 24' long. The combination of the high horizontal load and the lack of interior ties would require a concrete perimeter ring beam that is 12" x 20" with five #4 bars.

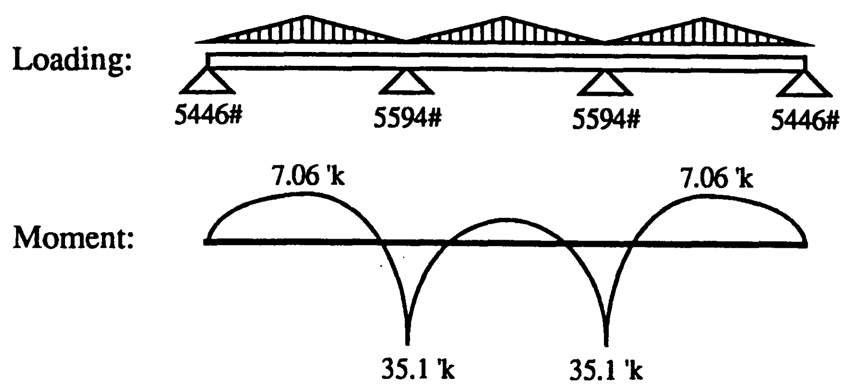


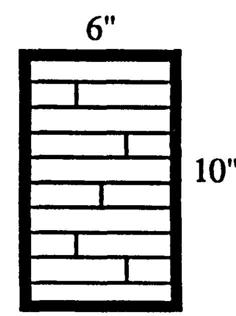
A 12" x 20" concrete ring beam with five #4 bars in it is required to resist horizontal thrust.



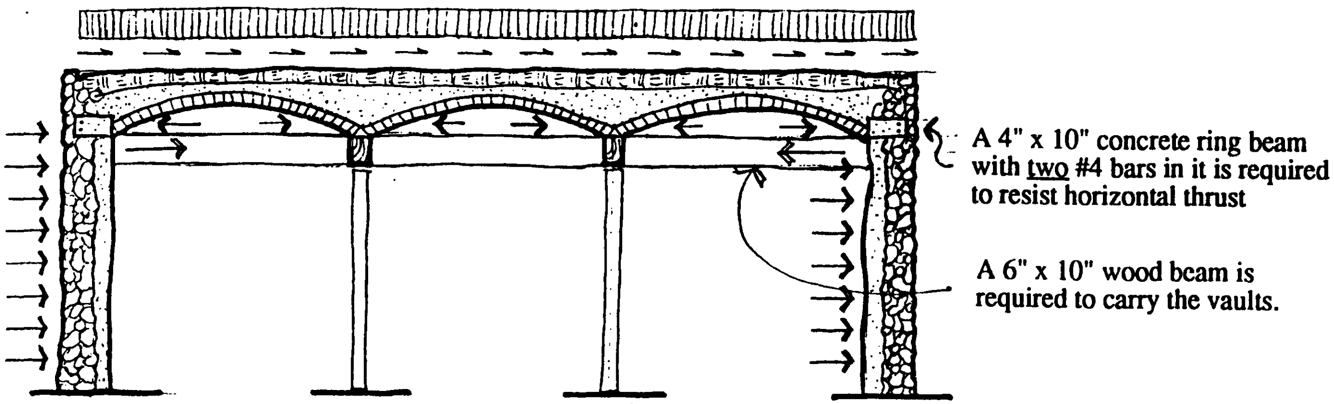
Square Vaults
A system of square vaults differs from the previous system in that it is not doubly curved. The vaults are therefore not free standing. They can, however, still be formed with brick and filled in with silt, but the vaults will sit on top of wood beams. Insulation and waterproofing can then be applied.

Unlike doubly curved vaults, square vaults can only transfer loads in one direction. Therefore, the beams are being loaded based on their triangular shaped tributary area. This results in a triangular load distribution with essentially no load at the corners. The following diagram illustrates the loading on one of the wood beams and the resultant moment diagram. For the 24' x 24' example considered here these loads would require a 6" x 10" continuous wood beam, which could easily be made from multiple laminations of much smaller pieces of wood.



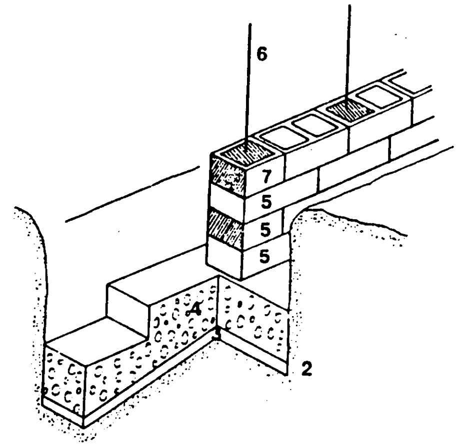


Square vaults develop horizontal thrusts in the same way as the doubly curved vaults, but because the square vaults transfer loads in one direction, the thrust also decreases to zero at the corners. In addition, the wood beams can function as ties across the roof, linking one wall to the other. This effectively reduces the span of the perimeter ring beam to 8', the size of one bay. The result is a much smaller moment being developed in the ring beam and consequently a much smaller beam being required, namely a 4' x 12" concrete beam with two #4 bars. See Appendix 8 for calculations.



Appendix E Sample Foundation Design

A copy of the foundation detail used in the Self-help School designs by Ove Arup (1991) is shown here:



1 Set out trenches to Drawing No. . Establish finished ground floor level , about 9" above surrounding ground level.

2 Excavate trench to at least (2'6" / 3') below existing ground level. Ensure suitable founding soil:CLAY - difficult to penetrate with thumb.
SAND - unable to push spade in easily.

3 Blind base of trench with 2" minimum thickness of concrete (1:4:8)

4 Establish surface level of foundation concrete which must be a multiple of 6" (ie. 6",1',1'6" etc.) below the finished ground floor level and allow at least an 8" thickness of foundation concrete. Place foundation concrete (1:3:6).

If the base of the trench is sloping , step the surface of the foundation concrete by 6" (see A).

5 Set out blockwork walls and place/mortar the 3 lowest levels of blocks.

6 Position wall vertical steel bars and concrete into the 3 rows of blocks.

7 Continue blockwork to 12" below finished ground floor level. Concrete vertical steel bars in each lift of blockwork.

Appendix F — Distribution of Shear Forces

The process outlined in this section is taken from Schneider, (1987).

When the overall lateral force at the top of a wall due to roof loads is known, the following method can be used to find the lateral forces distributed to each of the piers in a wall.

First, a wall is isolated and broken into component piers [**Figure F.a**].

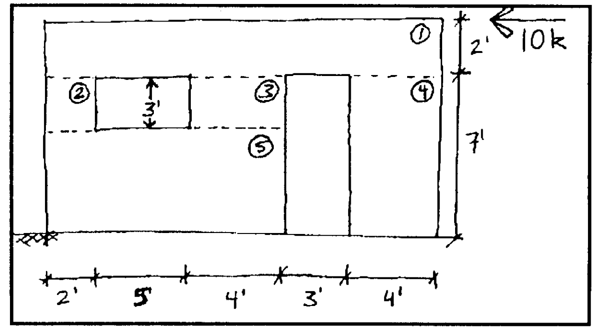


Figure F.a Pier divisions

Calculate the h/d ratio, Δfixed, and k of each pier [**Table F.1**]:

- $\Delta_{fixed} = 0.0833(h/d)^3 + 0.25(h/d)$
- $k = 1/\Delta_{fixed}$

  Δ_{fixed} is a value related to the lateral stiffness of the wall, and the reciprocal k is the form of that value used to find the relative stiffnesses of different sections.

Table F1: Pier Properties

Pier #	h/d	Δ_{fixed}	k
1	0.11	0.025	40
2	1.5	0.6	1.67
3	0.75	0.203	4.94
4	1.75	0.87	1.15
5	0.36	0.088	11.43

Proceeding from the first level where the tops of the windows and/or doors begin to break up the wall, divide the wall into units, as shown in **Figure F.b**.

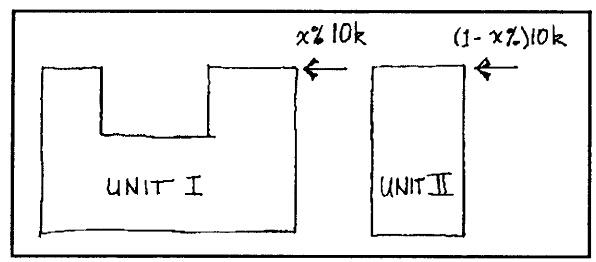


Figure F.b Unit divisions

The horizontal force (P) from the roof lateral loads and the pier(s) above must be distributed over each unit relative to its stiffness.

The relative stiffness of each unit is found by: $k_i/\Sigma\, k_i$

In order to find the k_i for each unit, Δ for each unit must be found.

For Unit I:

⇒ $\Delta_{unit\ I} = \Delta_5$ + *the combined* Δ *of piers 2 and 3*

[combined Δ of piers 2 and 3 = $1 / (1/\Delta_2 + 1/\Delta_3)$]

= 0.088 + 1 / (1/0.6 + 1/0.203)

= 0.24

⇒ $k_{unit\ I}$ = 1/0.24 = 4.18

For Unit II:

⇒ $\Delta_{unit\ II} = \Delta_4$ = 0.87

⇒ $k_{unit\ II}$ = 1/0.87 = 1.15

= 0.87

Using relative stiffness of a unit = $k_i / \Sigma\, k_i$

⇒ Rel. stiffness of unit I = 4.18/(4.18 + 1.15) = 78.4%

⇒ Rel. stiffness of unit II = 1.15/(4.18 + 1.15) = 21.6%

Distribute *P* over units I and II according to their relative stiffness. Assuming P = 10 k:

⇒ Force on unit I = 78.4% of 10 k = 7.84 k

⇒ Force on unit II = 21.6% of 10 k = 2.16 k

The 7.84 k force on unit I must also be distributed over piers 2 and 3 in a similar way, and so forth for all remaining piers.

Appendix G Wall Reinforcement Spreadsheet

Some sample sections from the spreadsheet used to quantify the amount of steel for wall reinforcement are included here. The input parameters are listed at the top of the spreadsheet. For each wall, the necessary individual inputs, such as dimensions and roof tributary areas, are contained within the double-lined boxes. Walls A and C are included as examples. *Important note*: When inputting the adjacent tributary areas of roof (A_a: which is denoted "adj. area of roof: ft^2" in the spreadsheet) the total area of the roof between opposing walls should be used. In others words, twice the area used in the process outlined in Section 2.4.4 *Roof Loads Exerted onto the Eave Level Ringbeam.*

GENERAL INPUT PARAMETERS

(t)wall thickness: ft	1
stone density: kcf	0.13
end fixity	0.75
lateral acceleration: g	0.2
steel cover: fraction of t	0.5
steel cover fraction	0.9
ult. str of mortar: ksi	
yield str of steel: ksi	36
ϕ	0.9
roof dead load: ksf	0.1
roof live load: ksf	0.04
roof rigidity	0.7

WALL A

layer	lintel	layer	eave
parallel length: ft	22.5	parallel length: ft	22.5
total tributary area: ft^2	112.5	total tributary area: ft^2	33.75
		adj. area of roof: ft^2	352
moment at ends: k*ft	4.1132813	moment at ends: k*ft	3.3129844
moment at midspan: k*ft	4.1132813	moment at midspan: k*ft	3.3129844
		dist. load of roof: k/ft	0.0657067
area of steel at ends: in^2	0.2821181	area of steel at ends: in^2	0.227228
area of steel at midspan: in^2	0.2821181	area of steel at midspan: in^2	0.227228

	1
pier length: ft	22.5
pier height: ft	10
self wt: k	29.25
overburden roof wt: k	24.64
overburden stone wt: k	0
lateral force: k	4.928
base stress: ksi	0.0166327
lateral force mom.: k*ft	49.28
crushing stress: ksi	0.0206887
pulling stress: ksi	0.0125767
roof tributary area: ft^2	176
total roof area: ft^2	352
area of stone above pier: ft^2	0

length at lintel ends: ft	10	
area of rebar: in^2	0.31	
length at lintel mid: ft	10	
area of rebar: in^2	0.31	
length at eave ends: ft	10	
area of rebar: in^2	0.31	
length at eave mid: ft	10	
area of rebar: in^2	0.31	
	total steel: ft^3	0.08611111

WALL C

layer	lintel left	layer	eave left			
parallel length: ft	7.5	parallel length: ft	7.5	length at lintel ends: ft	10	
total tributary area: ft^2	15.75	total tributary area: ft^2	11.25	area of rebar: in^2	0.11	
		adj. area of roof: ft^2	171.1	length at lintel mid: ft	10	
moment at ends: k*ft	0.1919531	moment at ends: k*ft	0.4739625	area of rebar: in^2	0.11	
moment at midspan: k*ft	0.1919531	moment at midspan: k*ft	0.4739625	length at eave ends: ft	10	
		dist. load of roof: k/ft	0.095816	area of rebar: in^2	0.11	
area of steel at ends: in^2	0.0131655	area of steel at ends: in^2	0.0325077	length at eave mid: ft	10	
area of steel at midspan: in^2	0.0131655	area of steel at midspan: in^2	0.0325077	area of rebar: in^2	0.11	
					total steel: ft^3	0.03055556

layer	lintel right	layer	eave right			
parallel length: ft	15	parallel length: ft	15	length at lintel ends: ft	10	
total tributary area: ft^2	75	total tributary area: ft^2	22.5	area of rebar: in^2	0.2	
		adj. area of roof: ft^2	342	length at lintel mid: ft	10	
moment at ends: k*ft	1.828125	moment at ends: k*ft	1.8950625	area of rebar: in^2	0.2	
moment at midspan: k*ft	1.828125	moment at midspan: k*ft	1.8950625	length at eave ends: ft	10	
		dist. load of roof: k/ft	0.09576	area of rebar: in^2	0.2	
area of steel at ends: in^2	0.1253858	area of steel at ends: in^2	0.1299769	length at eave mid: ft	10	
area of steel at midspan: in^2	0.1253858	area of steel at midspan: in^2	0.1299769	area of rebar: in^2	0.2	
					total steel: ft^3	0.05555556

	1
pier length: ft	22.5
pier height: ft	3
self wt: k	8.775
overburden roof wt: k	35.7
overburden stone wt: k	0
lateral force: k	7.182
base stress: ksi	0.0137269
lateral force mom.: k*ft	21.546
crushing stress: ksi	0.0155002
pulling stress: ksi	0.0119535
roof tributary area: ft^2	255
total roof area: ft^2	513
area of stone above pier: ft^2	0

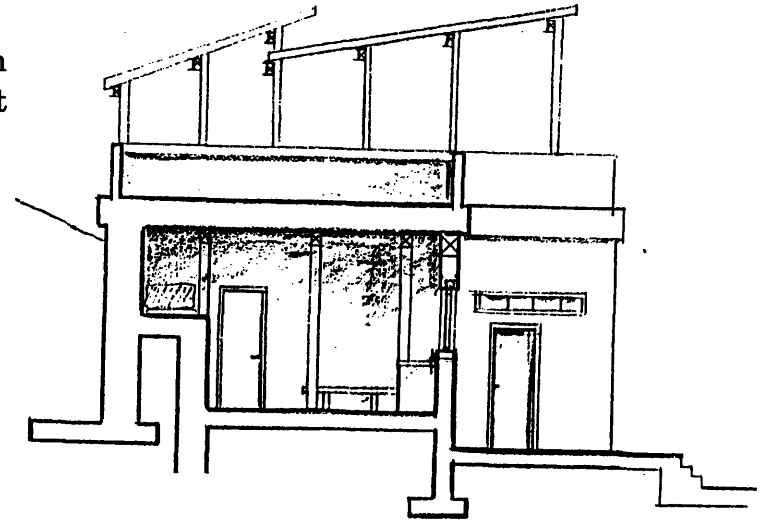
stiffness distribution of		C 2 and 3
delta	2	0.1184241
delta	3	0.1251324
k of	2	8.4442241
k of	3	7.9915344
relative stiffness of	2	0.5137715
relative stiffness of	3	0.4862285

	2		3
pier length: ft	4.5	pier length: ft	15
pier height: ft	2	pier height: ft	7
self wt: k	1.17	self wt: k	13.65
overburden roof wt: k	8.4	overburden roof wt: k	23.24
overburden stone wt: k	2.34	overburden stone wt: k	6.435
lateral force: k	4.5915758	lateral force: k	4.3454242
base stress: ksi	0.0183796	base stress: ksi	0.0200579
lateral force mom.: k*ft	9.1831516	lateral force mom.: k*ft	30.41797
crushing stress: ksi	0.037275	crushing stress: ksi	0.0256908
pulling stress: ksi	-0.0005157	pulling stress: ksi	0.0144249
roof tributary area: ft^2	60	roof tributary area: ft^2	166
total roof area: ft^2	513	total roof area: ft^2	513
area of stone above pier: ft^2	18	area of stone above pier: ft^2	49.5

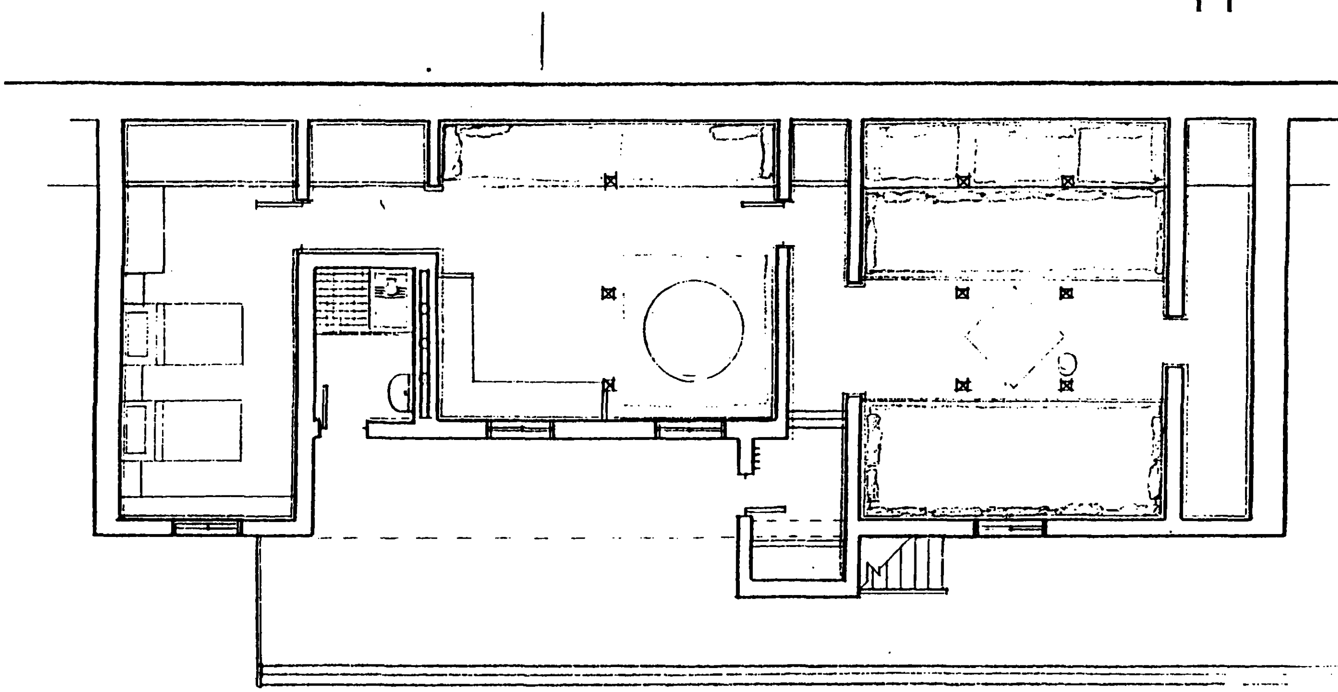
Appendix H Design Example: Plans and Take-offs

Drawings by Rukiye Devres of the design example house plan are included here, along with the breakdowns and take-offs that were necessary for engineering calculations.

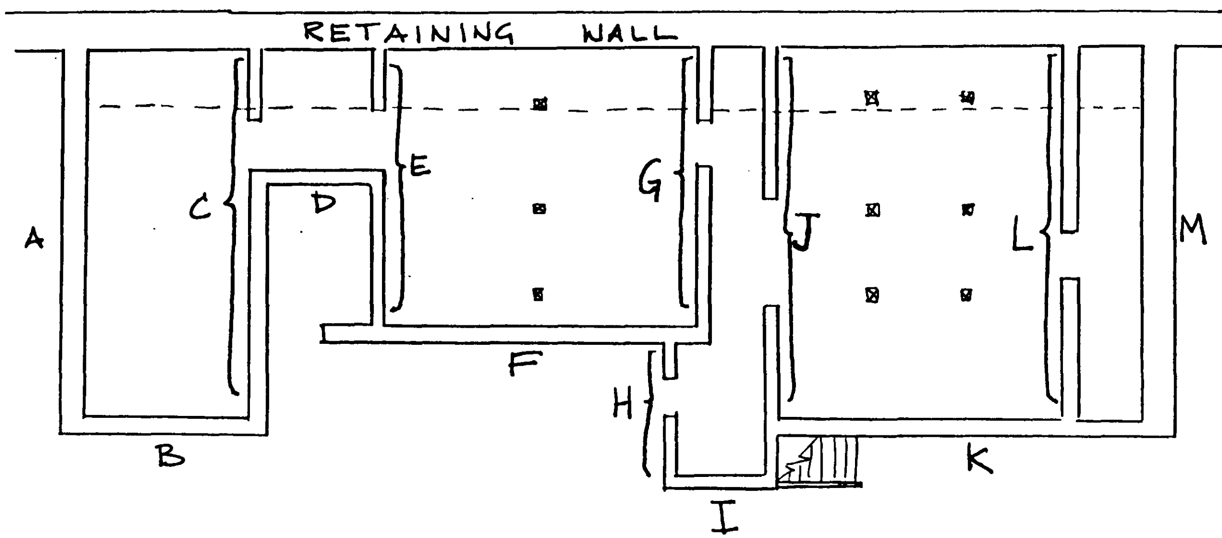
Section:



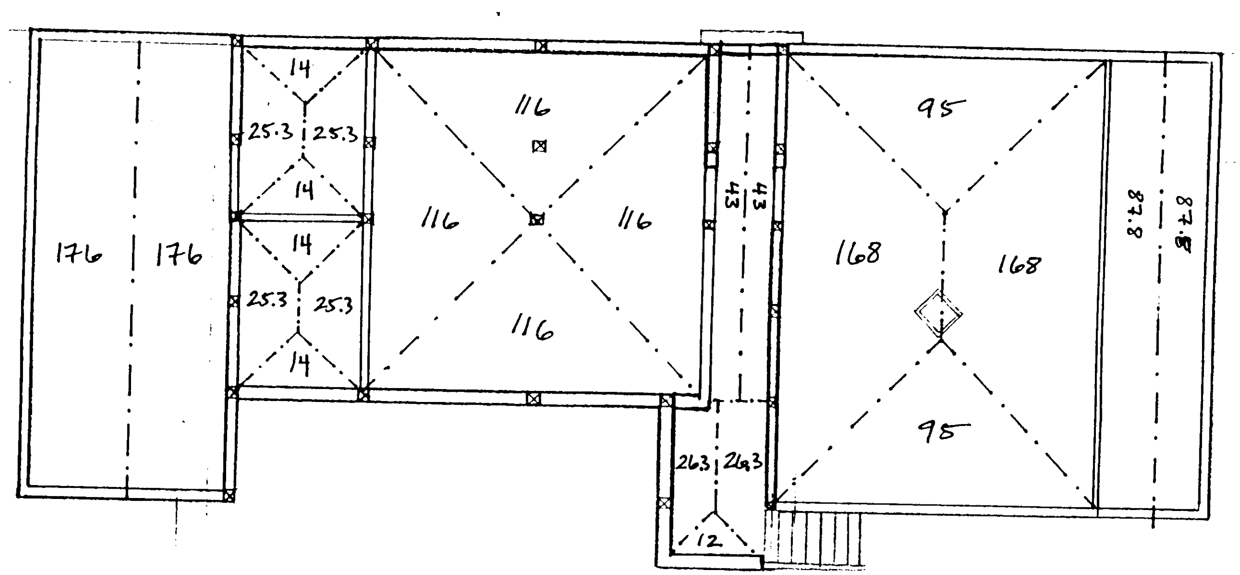
Plan:



Notation of walls:



Roof tributary area breakdown:



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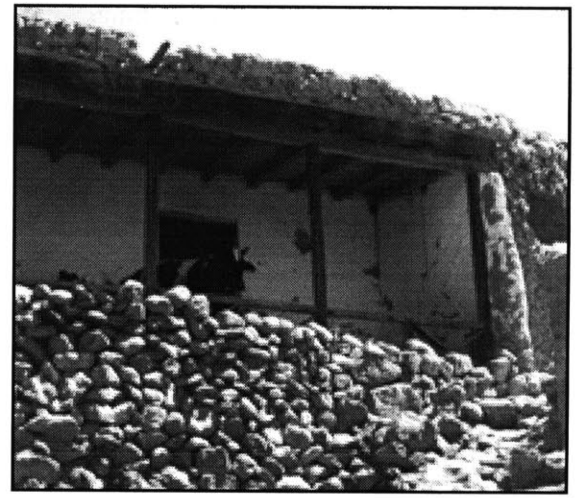

A cow with a view