REVIVING OLD TARBLE

Design of a Special Collections Library

Engineering 90 Senior Design Report

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1. INTRODUCTION

1.1. Topic Selection

We devised our project idea after speaking with Daniel M. Honig, P.E. who suggested we find a use for the Old Tarble space. Old Tarble is a building on the Swarthmore College campus that was badly damaged by a fire in 1983; it has remained largely unchanged in the years hence. The idea immediately appealed to us because Steve Huang is an aspiring architect and Kirk Ellison is an aspiring structural engineer. Not only would a renovation of Old Tarble allow us to gain experience in our respective fields, but it would allow us to work with an accessible structure with personal interest to us as Swarthmore College students.

From subsequent talks with Facilities Director, Larry Schall, and Associate Vice President of Facilities, Stuart Hain, we learned that although there were no current plans for that space, a number of ideas had surfaced since the 1983 fire. One such idea was to use the space to alleviate the shortage of shelving space currently being experienced by McCabe, Swarthmore College's main library.

We took this idea to College Librarian, Peggy Seiden, who informed us that a feasibility study had recently been conducted to expand McCabe Library. The college, however, had taken no further action with regard to this matter. As a result, Ms. Seiden was very enthusiastic about our project and suggested we convert Old Tarble into a special collections library. Since special collections are usually accessed by the public rather than students, they seemed like the most logical books to remove from the main collection. Do to the diminutive size of the existing structure, it was apparent that we would need to design a rather large addition to the existing space in order to house a sizeable portion of special collections.

1.2 Site History

Old Tarble as it exists today was originally constructed in 1928 as an addition to Carnegie Library, the college's first building used exclusively for storing books. The library was converted to and renamed as Tarble Social Center after the completion of McCabe Library in 1967. After this renovation, the building held the student café Paces, which served as the de facto student center for many years. The space provided students with a full kitchen for the café, as well as a small stage, pool tables, shuffleboard, publications offices and many lounge rooms. Plans and elevations for the first floor of the Old Tarble Social Center are provided in the appendix.

For many alumni, this space was the center of campus life and holds many fond memories. This is particularly important to us- it is this age group of alumni that would be the most likely to be donating money in the next few years, and getting their approval is of the utmost importance. A fire in 1983 destroyed the original section of Carnegie Library and left only the 1928 addition standing.

A number of tentative ideas have been proposed for the space during the decades since the fire, none of which have come to fruition. One idea was to recreate the old student center. Old Tarble was thus the logical choice. However, in the first round of site selection, prominent architect Robert Venturi argued for the use of Clothier Hall. Clothier Hall, like Old Tarble, was something of a lame duck. The college had built the structure largely out of vanity- it used 16th century French monastic architecture on a campus founded by American Quakers. It had no real use. Because the building envelope already existed, and was thus cheaper to use, Clothier Hall was renovated and Old Tarble was forgotten. According to Stuart Hain, the use of Old Tarble as a student space has not been revisited seriously since.

The building gained a revamped mechanical room, a concrete facing to replace the old fire wall, and a storage room, but was otherwise untouched. Since 1997, the building has been used as a classroom by the Art and Theater Departments. It should be noted, though, that only two classes are taught here. For the purposes of this project, those classes would be moved elsewhere along with secure storage.

Other major changes to the structure would include changes to the mechanical structure of the site. As seen in the appendix, the site has a major steam distribution hub located quite close to the structure. Any construction on the Old Tarble site would necessarily reroute these lines. In order to make any buils they are too close to avoid.

1.3 Project Scope

Our project was composed of three main stages: analysis of the existing structure including design renovations where necessary, three-dimensional architectural design of the new addition in accordance with our clients' demands, and structural design of the new addition using both concrete and steel. Although plumbing, HVAC and lighting considerations were taken into account, design of these systems was outside of the scope of this project.

Analysis of the existing structure could not take place before completion of architectural floor plans because loading of the floor structure was dependent on intended usage of the given area. This phase included checking the strength of all slabs, beams and columns on each of three stories for compliance with IBC 2003 General Building Code. This phase also required checking the strength of the roof trusses for all possible load combinations defined in ASCE 7-02. The rubble masonry shear walls were not checked for compliance, but were deemed appropriate for reasons explained in section 3.2.1.

Three-dimensional architectural design was performed using the computer program, Rhino 3D. The objective of this phase was to design an aesthetically pleasing and functionally adequate building that met the criteria defined by our librarian clients.   There were several realistic design constraints to be considered in this phase, including: ceiling heights and loading capacity of the existing structure, size of the HVAC system and mechanical room, requirements set forth in IBC 2003 and the Americans with Disabilities Act (ADA) codes.

The final phase, structural design, was the heart of our project. Given the architectural plans that we had already designed, we needed to come up with a feasible structural support system. After preliminary research on horizontal subsystems, we opted to divide this phase into three stages for educational purposes: design of a one-way steel joist/girder floor system with steel columns, design of a two-way concrete waffle slab with concrete columns, design of steel roof trusses, and design of a lateral-force resisting system composed of cross-bracing and a moment frame.

1.4 How Project Satisfies E90/ABET Requirements

The E90 senior design project requires seniors to demonstrate their competence in math, science, engineering and the liberal arts by pursuing projects that integrate materials from the courses they have taken. This project required that we call upon the skills that we learned in virtually all of our civil engineering classes. We used knowledge gained from Mechanics, Mechanics of Materials, Structural Theory and Design I and Structural Theory and Design II in the design of all connections and structural members. We also used knowledge learned in our Geotechnical Engineering course in order carry out design of the foundations and analysis of subsurface conditions.

The ABET criteria for a senior design project is that students must incorporate “engineering standards and realistic constraints that include most of the following considerations: economic, environmental, sustainability, manufacturability, ethical, health and safety, social, and political.” As we chose to tackle a common kind of real-world engineering problem, our project easily satisfies these criteria, as discussed subsequently.

Our project incorporates engineering standards in a literal sense in that every calculation and design consideration had to be checked for compliance with applicable codes. Likewise, realistic constraints had to be overcome during every phase of the project. This topic is expanded upon in the section 1.5.

Economic considerations were of primary importance in our design project. For this reason, loading on the existing structure was kept to a minimum such that few costly alterations would need to be made. The new addition was designed first using mostly steel because construction with this material generally results in the quickest and cheapest erection of a structure. In this manner, the concept of manufacturability is intermingled with economics. Steel is usually prefabricated off-site for quick assembly on-site. Thus money is saved by minimizing the amount of manpower required to construct the building.

Social considerations had to be taken into account as our library would affect several interested parties and influence the social dynamics between each. Another concern expressed by our clients was that a campus building devoted entirely to special collections would create a rift between the general public who use McCabe library to access primary historical documents and the students who use the library for more general purposes, such as accessing the general collection, using computers or just studying in a quiet place. Since students do not often access special collections, the librarians feared that students might not use the new building. Therefore, in addition to housing books, the librarians wanted the new structure to provide a pleasant atmosphere, including working space for students. Social interactions also had to be considered because dividing the collections of McCabe Library into two buildings also required that employees be divided as well. One final example of social considerations took place during topic selection when deciding whether or not Old Tarble was worth salvaging. We decided to restore the existing wing rather than replace it after considering the sentimental and historical value of the building for older faculty and alumni.

Consideration of health and safety was implicit in deciding to comply with national building standards. Allowable live loads due to usage of floor space and wind/snow/seismic loading, as well as the methods of egress and guidelines for fireproofing and construction were all included in the IBC 2003 code. Design of individual members was conducted using the AISC Load and Resistance Factor Design (LRFD) manual, which is accepted as a valid technique by IBC 2003.

Other considerations such as ethical, environmental and sustainability were not particularly relevant to this project since none of our plans threatened to cause problems in these areas.

1.5 Realistic Design Constraints

This project had tight restrictions on time, money and space. Time was perhaps the most critical constraint since an immovable deadline for project presentations existed on May 2nd. The feasibility of the project with respect to this time constraint was borne in mind throughout the proposal and at every stage during the design process. Furthermore, we were kept on track through weekly meetings with our advisor during which we had to show our progress.

Money constrained the project in two ways: first, a lack of it prevented us from utilizing many potential resources that could help us with design work; and second, it had to be considered in designing an economically viable option for relocation of the special collections library. The second monetary constraint was considered by designing the building with a steel frame. This is further discussed in the section 5.1.

The area of land to be built on is limited by the surrounding structures, walkways and embankments. Fortunately, the amount of space required for the special collections library is not significant enough to make this constraint a large problem. In fact, this constraint was considered and solved by deciding to use the Old Tarble space for special collections rather than some other purpose.

Most importantly, our project had to satisfy the demands of our clients, College Librarian Peggy Seiden and Friends Historical Library Curator Christopher Densmore, as determined by the 15 year projected needs assuming two percent growth per year. This topic is expanded upon in section 4.3.

2. DETERMINATION OF DESIGN LOADS

2.1 Load Combinations

IBC 2003 utilizes the live load combinations suggested by ASCE 7-02, Minimum Design Loads for Buildings and Other Structures. In determining live loads for strength design, engineers must design for the most extreme loading condition resulting from the following possible load combinations:

1. 1.4(D+F)
2. 1.2(D+F+T)+1.6(L+H)+0.5(L_r or S or R)
3. 1.2D+1.6(L_r or S or R)+(L or 0.8W)
4. 1.2D+1.6W+L+0.5(L_r or S or R)
5. 1.2D+1.0E+L+0.2S
6. 0.9D+1.6W+1.6H
7. 0.9D+1.0E+1.6H

Where: D = dead load; E = earthquake load; F = load due to fluids with well-defined pressures and maximum heights; H = load due to lateral earth pressure, ground water pressure, or pressure of bulk materials; L = live load; L_r = roof live load; R = rain load; S = snow load; W = wind load

2.2 Live Load on Floors

The following table of unfactored design live loads summarizes the relevant floor loads for our special collections library as listed in Table 4-1 of ASCE 7-02.

Table 1: Appropriate Live Loads

	Live Load	
Occupancy or Use	**Uniform**	**Concentrated**
Reading Rooms	60	1,000
Stack Rooms	150	1,000
Compact Shelving	300	2,000
Offices	50	2,000
Computer Room	100	2,000
1st floor corridors	100	---
Corridors above 1st floor	80	1,000
Stairs	100	---
Elevator Machine Room Grating	---	300

2.3 Roof Live Loads

2.3.1 Minimum Roof Live Load

Minimum roof live load, L_r, in lbs/ft^2 was determined according to the following equation:

$$L_r = 20R_1R_2 \text{ where } 12 \le L_r \le 20 \quad (1)$$

R_1 and R_2 in equation (1) are reduction factors found by equations (2) and (3) as follow:

$$R_1 = \begin{cases} 1 & A_t \le 200\,\text{ft}^2 \\ 1.2 - 0.001A_t & 200\,\text{ft}^2 \le A_t \le 600\,\text{ft}^2 \\ 0.6 & A_t \le 600\,\text{ft}^2 \end{cases} \quad (2)$$

Where: A_t = the tributary area supported by any structural member

$$R_2 = \begin{cases} 1 & F \le 4 \\ 1.2 - 0.05F & F \le A_t \le F \\ 0.6 & F \le F \end{cases} \quad (3)$$

Where: F = the number of inches of rise per ft.

Minimum roof live load was found to be 12 psf for both the new and existing structures.

2.3.2 Snow Load

The sloped roof snow load was calculated according to equation (4):

$$p_s = C_s p_f \tag{4}$$

Where: C_s = slope factor
p_f = snow load on flat roofs

The slope factor, C_s, was determined to be 0.6 for the existing structure and 0.7 for the new structure from Figure 7-2 in ASCE 7-02 for roof slope of 38 degrees and 22.5 degrees, respectively. The figure is reproduced as follows:

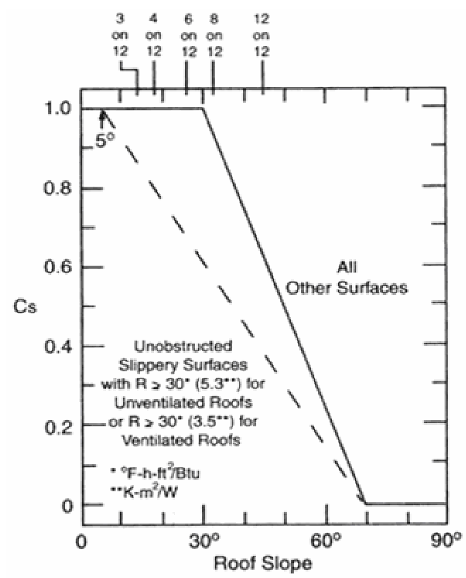

Figure 1. Graph for Determining Roof Slope Factor C

The snow load for flat roofs, p_f, was determined from equation (5):

$$p_f = 0.7 C_e C_t I p_g \tag{5}$$

Where: C_e = exposure factor (1.0 for partially exposed buildings in terrain category B)
C_t = thermal factor (1.0 for regularly heated structures)
I = importance factor (1.0 for category II buildings)
P_g=ground snow load (30 psf for southeastern Pennsylvania)

2.4 Wind Loads

Wind loads for the main windforce-resisting systems (MWFRS) were calculated in accordance with chapter 6 of ASCE 7-02. A detailed calculation for wind loading can be found in the calculations section of the appendix. Velocity pressures were calculated for a basic wind speed of 90 mph as determined from Figure 6-1 of ASCE 7-02 for southeastern Pennsylvania. The following equation was used to determine velocity pressure in lbs/ft^2 at a height, z:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I \qquad (6)$$

Where: K_z = velocity pressure exposure coefficient
K_{zt} = topographic factor (1.0 for no topographic effect)
K_d = wind directionality factor (for building MWFRS)
V = basic wind speed (90 mph for southeastern Pennsylvania)
I = importance factor (1.0 for category II buildings)

Values for velocity pressure were then plugged into the following general equation to determine design wind pressures for a number of scenarios with varying values for dynamic pressure coefficient, C_p, and internal pressure coefficient, GC_{pi}.

$$p = qGC_p - q_i(GC_{pi}) \qquad (7)$$

Where: q = q_z for windward walls
q = q_h for leeward walls, side walls, and roofs
q_i = q_z for positive internal pressure evaluation
G = gust effect factor (0.85 for exposure C)
C_p = dynamic pressure coefficient
GC_{pi} = internal pressure coefficient (±0.18 for enclosed buildings)

It should be noted that q_h is equivalent to q_z evaluated at the mean roof height. The dynamic pressure coefficient, C_p, and internal pressure coefficients, GC_{pi}, were determined from the tables in Figures 6-5 and 6-6 of ASCE 7-02. The coefficients used for this project are summarized in the following table:

Table 2: Wind Parameters

	Surface	Wind Direction		Existing Structure	New Structure
Internal Pressure Coefficient (GC_{pi})	All	All		±0.18	±0.18
Dynamic Pressure Coefficient (Cp)	Windward Wall	All		0.8	0.8
	Leeward Wall	Normal		-0.5	-0.5
		Parallel		-0.262	-0.395
	Side Wall	All		-0.7	-0.7
	Windward Roof	Normal	Case 1	-0.124	-0.416
			Case 2	0.238	0.03616
		Parallel	0 to h	-0.9	-0.9
			h to 2h	-0.5	-0.5
			>2h	-0.3	-0.3
	Leeward Roof	Normal		-0.6	-0.6
		Parallel	0 to h	-0.9	-0.9
			h to 1h	-0.5	-0.5
			>2h	-0.3	-0.3

2.5 Earthquake Loads

Earthquake loads were calculated using the equivalent lateral force method. By this method, the seismic forces are represented by horizontal point loads on a linear model of the structure, as described in chapter 9 of ASCE 7-02. The base of the building is assumed to be fixed and the seismic base shear is distributed among different levels of the structure as shown in Figure 2. Detailed calculations for seismic analysis of both the existing building and the new structure can be found in the appendix.

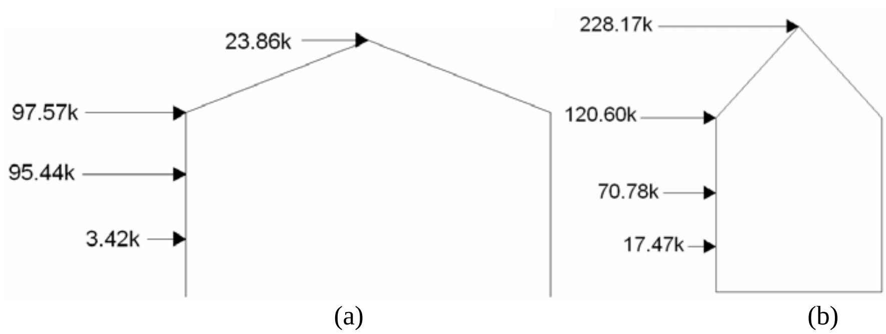

Figure 2. Equivalent force loadings for seismic analysis of (a) new and (b) existing structures.

The first equation to be considered by the equivalent lateral force method enables one to find the seismic response coefficient, Cs, from the following parameters: design spectra response acceleration in the short period range, S_{DS}, response modification factor, R, and the occupancy important factor, I. Each of these values were determined from tables or figures in ASCE 7-02, which depended on location, main seismic force resisting system and use of the structure, respectively.

$$Cs = \frac{S_{DS}}{R/I} \tag{8}$$

Where: Cs = seismic response coefficient
S_{DS} = design spectra response acceleration for short periods
R = response modification factor
I = occupancy importance factor

The seismic response coefficient was also subject to maximum and minimum constraints. If the value of the seismic response coefficient obtained in equation (8) did not fall within the range defined by equation (9), then the value of the nearest restraint was selected.

$$0.044 \times I \times S_{DS} \leq Cs \leq \frac{S_{D1}}{T(R/I)} \tag{9}$$

Where: I = occupancy importance factor
S_{DS} = design spectra response acceleration in the short period range
Cs = seismic response coefficient
S_{D1} = design spectral response acceleration at a period of 1s
T = fundamental period of the building
R = response modification factor

The fundamental period of the building, T, used in equation (9) was determined in the following manner:

$$T = Cu \times Ta = Cu \times (C_T \times h_n) \tag{10}$$

Where:T = fundamental period of the building
Cu = coefficient for upper limit on calculated period
Ta = approximate fundamental period
C_T = building period coefficient
h_n = height of the building

After determining the seismic response coefficient, the total design lateral shear at the base of the building could be found by the following equation:

$$V = Cs \times W \tag{11}$$

Where: V = total design lateral shear at the base of the building
Cs = seismic response coefficient
W = total gravity load of building assigned to a level

Finally, the portion of the seismic base shear, or horizontal point load, induced at a level could be found by multiplying the total design lateral shear at the base by a vertical distribution factor. The equation for determining each equivalent lateral force at a height, x, is displayed in equation (12).

$$Fx = Cvx \times V = \frac{Wx \times hx^k}{\sum Wi \times hi^k} \times V \qquad (12)$$

Where: Fx = the portion of the seismic base shear induced at a level
V = total design lateral force or shear at the base of the building
Wx,Wi = the portion of the total gravity load assigned to a level
hx,hi = height from the base level

3. ANALYSIS OF EXISTING BUILDING

3.1 Analysis of Existing Soil Conditions

The soil beneath the proposed site has silty clay and sandy silt with mica schist, according to borings performed beneath nearby Mertz dormitory. Standard Penetration tests show that, at some places, the soil may be highly susceptible to settlement in layers down to depths of 30 feet at some places. The problem is exacerbated by a relatively high water level at approximately 14 feet below the ground surface.

The ground in the Old Tarble region is highly irregular, due to the slope of the land. The land beneath McCabe is highly irregular, with bedrock occurring at a very shallow depth. Plans of the foundation show an uneven grade- the foundation was adapted to the rock and its footings appear at several different depths, indicating that they either lie on the rock, or lie in excavated sections of the rock.

As one gets further down the hill, the grade of the land flattens out considerably, and the depth of the rock drops quite a bit. The extent of this drop, however, is not known. Unfortunately, there have been no detailed geotechnical studies of the ground immediately under Old Tarble since the addition was made in 1926. There have been no geotechnical plans for the selective reuse of Old Tarble since then, not even it burned down.

In order to get around this limitation, we examined the footings of McCabe library, which is northwest of the site, and boring logs from Mertz, which is southeast of the site. The plans from McCabe showed representative columns and footings. We took several of the columns, and using information about the footing design and the axial load, backcalculated the allowable soil pressure, which was roughly 5.5 psf. This was in agreement with notes on the Old Tarble plans, which gave a very conservative allowable soil pressure of 6.0 psf. The boring logs from around Mertz were calculated similarly, and was also found to have an allowable soil pressure of 6 psf.

3.2. Analysis of Existing Structure

3.2.1 Overview of Main Structural Support System

The existing Old Tarble structure is composed of rubble masonry walls, monolithically poured beam/slab systems, a single column at the intersection of internal walls and double angle steel roof trusses spaced at 12 foot increments. Analysis of the structure focused mainly on the beam/slab systems, which were assumed to deflect only in one direction, and the roof trusses, which were assumed only to experience axial forces. The masonry walls were excluded from analysis because they were clearly over-designed. Standards for masonry wall thickness in the 1920s stated that the walls needed to be a certain width per foot of height. In compliance with that standard, the masonry walls in Old Tarble vary from 18-26", which is clearly a very conservative design.

3.2.2 Materials

Materials used routinely in the present day have notably different properties than those used in the 1920s. Therefore, the first step in analyzing the existing structure was to determine the relevant properties of the building materials in place. Notes on the existing structure called for use of 1:2:4 concrete, intermediate grade steel and Kahn bars. 1:2:4 concrete refers to the ratio of cement, sand and coarse aggregate in a concrete mix. It has a compressive strength of approximately 3300 psi according to a Concrete Reinforced Steel Institute website entitled *Evaluation of Reinforcing Bars in Old Concrete Structures*. For the purposes of this project, compressive strength, f'_c, was assumed to be 3000 psi. Intermediate grade steel in the 1920s had a yield strength, f_y, of 33,000 psi.

Kahn bars were reinforcing steel bars with a diamond-shaped cross-section and rectangular flanges. The flanges were stripped up at 45 degree angles to provide shear reinforcement, as shown in Figure 3. For the purposes of this project, the Kahn bar was assumed to have the same strength as the intermediate grade steel. Beams with Kahn bars were not checked for adequate shear strength due to an inability to determine the cross-sectional area of the flanges without damaging the existing structure. The assumption of adequate shear strength should not pose a problem because loadings on the existing structure have historically been more significant than those that are to be placed on the structure by the renovations proposed in this report.

Figure 3. Kahn Bar

3.2.3 Pan-joist floor systems

Pan-joist floor structures make up the main floor systems of the first floor and mezzanine. Figure 3 shows how the pan-joist system can be analyzed as a series of T-beams, rather than slabs resting on a series of rectangular beams.

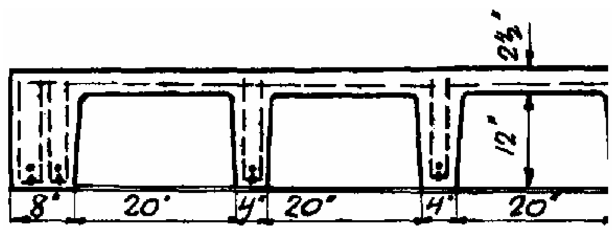

Figure 4. Pan-joist floor structure of existing mezzanine

The loads on the existing structure were factored in a manner compliant with older building code, but more conservative than necessary to comply with IBC 2003 or ACI 318-02. Loads were factored by the following equation:

$$w_u = 1.4DL + 1.7LL \tag{13}$$

Where: w_u = governing factored load
DL = dead load
LL = live load

The first step in checking the pan-joists for code compliance was to ensure that they had the minimal amount of necessary reinforcing, as determined by the following equation:

$$A_{s\min} = \frac{3\sqrt{f_c^{'}}}{f_y} \tag{14}$$

Where: A_s = cross-sectional area of steel reinforcement
f'_c = compressive strength of concrete
f_y = yield strength of steel

All sections were determined to be under-reinforced from equation (14) for steel strain. Therefore, steel strength governed the failure conditions. Since concrete strain is assumed to be 0.003, a section is said to be under-reinforced if steel strain, ε_t, exceeds 0.005.

$$\varepsilon_t = \frac{d-c}{c} f_c' = \frac{d-a/\beta}{a/\beta} f_c' \quad (15)$$

Where: ε_t = steel strain
d = depth to center of reinforcement
c = a/β
a = $\dfrac{A_s f_y}{0.85 f_c' b}$
β = 0.85 (for $f'_c \leq 4000$ psi)
f'_c = compressive strength of concrete

Since all T-beams were tension-controlled, the design moment, ΦM_n, was calculated for T-beams from the following series of equations:

$$\phi M_n = \phi Tz = \phi A_s f_y \left(d - \overline{y}\right) \quad (16)$$

Where: Φ = undercapacity factor (0.9 for tension-controlled section)
M_n = internal nominal moment
T = tensile force of steel reinforcement (=$A_s F_y$ for under-reinforced sections)
z = lever arm distance
A_s = cross-sectional area of steel reinforcement
f_y = yield strength of steel reinforcement
d = depth to center of reinforcement
$\overline{y}$ = distance from top of the flange to center of gravity of A_c

Checks on the shear strength of concrete were performed by comparing the critical shear, V_{cr}, to half the design shear strength of concrete, ΦV_c, using equation (16). The results showed that the concrete did not provide adequate resistance to shear by itself. Therefore, a proper analysis would have required that calculations be performed for the shear reinforcing provided by Kahn bars. This procedure was not carried out for reasons explained above.

$$\phi V_c = \phi 2\sqrt{f'c}\, b_w d \quad (17)$$

Where: Φ = undercapacity factor (0.75)
V_c = concrete shear strength
f'_c = compressive strength of concrete
b_w = width at the base
d = depth to center of reinforcement

3.2.4 Beams

The beams that support the pan-joist floor structure are part of the same monolithic pour; however, they were treated as if the one-way pan-joist system rested atop the beams. A cross-section of one such beam is shown in Figure 5. This image shows how the supporting beam actually intersected the pan-joists.

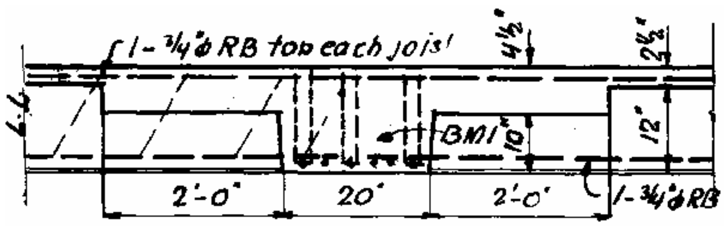

Figure 5. Cross-section of beam supporting pan-joist floor

Whereas the T-beam in figure 5 was subject to the same equations as the T-beams in the pan-joist floor structure described in the preceding section, other beams had rectangular cross-sections. These beams were analyzed in much the same manner, except that the design moment could be found more simply by equation (17):

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2} \right) \tag{18}$$

Where: Φ = undercapacity factor (0.9 for tension-controlled section)
M_n = internal nominal moment
A_s = cross-sectional area of steel reinforcement
f_y = yield strength of steel reinforcement
d = depth to center of reinforcement

$$a = \frac{A_s f_y}{0.85 f_c^{'} b}$$

3.2.5 One-Way Slabs

The one-way slabs were analyzed as one-foot wide strips. They were first checked for compliance with ACI 318-02 for minimum thickness, then for adequate shear strength and tensile reinforcement. Current code requires that one-way slabs be at least one-twentieth as thick as the span of the slab.

Ultimate shear was assumed to be experienced at midspan and was compared to ½ΦV_c in the same manner as for beams. Ultimate shear was determined by the following equation where the slab strip was modeled as a simply supported beam:

$$V_u = \frac{w_u l}{2} \tag{18}$$

Where: V_u = ultimate shear
w_u = factored uniform load per square foot
l = span length

The required amount of tensile reinforcement was determined by ultimate moment. For the simply supported beam model of the slab, this was determined by the following equation:

$$M_u = \frac{w_u l^2}{8} \tag{19}$$

Where: M_u = ultimate moment
w_u = factored uniform load per square foot
l = span length

The value for ultimate moment determined in equation (19) was plugged into equation (20), which could then be used to find the percentage of steel, ρ, from common tables.

$$\bar{k} = \frac{M_u}{\phi b d^2} \tag{20}$$

Where: M_u = ultimate moment
Φ = undercapacity factor (0.9 for tension-controlled section)
b = width of slab strip
d = depth to center of reinforcement

Analysis of the roof trusses was performed with help from the computer program, Multiframe. Loads were transferred in the model in three steps as exemplified in Figure 6. Step 1 transferred loads on the effective roof area to purlins by modeling the roof as a beam supported by pinned restraints at the locations of the purlins. Step 2 transferred loads from the purlins to joints of the truss by modeling a beam identical to that in step 1, which was loaded by the support reactions from step 1 and supported by pins at the locations of truss joints. Finally, the support reactions from step 2 were used to load the entire truss in step 3.

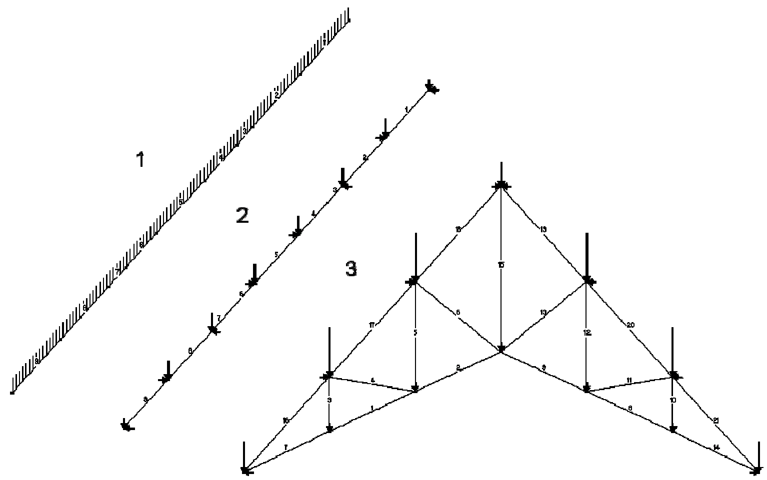

Figure 6. Dead loading example for Multiframe model of existing roof trusses

All unfactored dead and live loading cases were run through the model in order to determine compressive and tensile axial forces on each member. Loading of individual truss members was assumed to act only at the joints, rather than anywhere along the length of the member so that design would not have to consider moments. Once member forces were determined for each loading scenario, the forces were factored for every loading combination required by ASCE 7-02. The maximum tensile and compressive forces resulting from these equations were matched against the maximum design forces and checked for adequacy.

The capacity of the members in compression was determine using the Euler buckling formulas, where the critical compressive force is defined as:

$$F_{cr} = \frac{\pi^2 E}{12\left(1-\nu^2\right)}\left(\frac{t}{h}\right)^2 \tag{21}$$

Where: F_{cr}= compression at buckling
k = stiffness factor
E = modulus of elasticity
V= Poisson's ratio
t/h= slenderness ratio, or thickness/ height

The capacity of the members in tension was determined simply as:

$$F_{cr} = A_s F_y \tag{22}$$

Where: F_{cr}= tension at yielding
A_s= cross-sectional area of steel
F_{cr}= tension at yielding

4. PRELIMINARY/ ARCHITECTURAL

4.1 Discussion of Site

The Old Tarble location is a prime one on campus, having previously served as a student center in the 1960's through the early 1980's. It is a site that is loaded with meaning in the collective institutional memory of Swarthmore College. Since the fire, however, the site has been largely unused by the student population, and for the current generation of students, it has virtually no real meaning beyond its use as a repository for belongings during the summer months.

There have been talks of demolishing the existing building- this would solve a great deal of practical problems with building an addition. It would likely reduce costs as well, as the existing structure would not have to be updated at all, and the foundations of the new building would be much easier to design. However, the existing building is a very important one on campus. Built in a spate of construction in the early 1900's, the structure is part of a family of buildings that includes the old fraternities and sororities that are now being rennamed and reused as Worth, the Kitao Gallery, the Women's Resource Center, Delta Upsilon, Phi Sci, and Olde Club. This family of buildings were all of the same architectural style, and are as much a part of Swarthmore's history as Parrish itself. It would be a mistake to demolish any one of these.

An additional, more practical consideration is one of generating alumni interest in the project; possible construction on the site would simply not be possible without these donors. This generation of donors would have strong memories of the student center that used to occupy Old Tarble and its attendant snack bar and small theater. It is this generation of donors that would be most interested in reviving the structure to practical use.

Because of its very central location on campus, we view it as a great opportunity to build an architectural statement of the meaning of Swarthmore. As one walks up MacGill walk, one first sees the dormitories of Mertz and Alice Paul to the left, and Sharples and Clothier to the right. Parrish is directly ahead. The Old Tarble site will be the first opportunity for new and returning students to see an academic building on campus. The building should express the openness and earnestness of both students and faculty.

Physically the site provides some challenges, but solves others. Because it is on a five degree grade, drainage is simplified, but this same grade also means that entrances must be carefully coordinated so that they meet at the same elevation. Mapping of the site was accomplished with help from Woody Frazier over at Facilities, who provided topographic maps that were then extruded and lofted together in AutoCAD and Rhino.

4.2 Needs of the client

The function of student center, however attractive it may be to donors, however, is being met in Clothier Hall, and soon in the new Parrish space. Here faculty needs expressed by those in the library are more important. Space for new books is simply not available, and many library solutions have been proposed. Most recent is a feasability study conducted by Einhorn Yaffee Prescott.

A modern solution for increasing storage space is the use of basements with industrial-scale ceilings and oversized compact shelving. However, this solution is hard to implement at the McCabe site. It was suggested by the architects that the new basement could be sunk into the side of the hill. Because of the fairly shallow bedrock present there, however, this idea was not feasible.

A somewhat more temporary solution than a complete renovation of McCabe or an addition is the use of the Old Tarble space as an extension, a secondary library. Peggy Seiden briefly played around with the idea of moving Special Collections and the Treasury Room out to a secondary structure, proposing a bridge between McCabe and Old Tarble. The new structure would potentially hold special collections, conference rooms for collaborative student work, media production areas, and some office space. It is this idea that we found the most need for.

It must be stressed that the Old Tarble renovation is not meant to be a new library. Like Stanford University's Auxiliary Library (Planning Academic and Research Library Buildings, 44), the renovation is meant to alleviate the space needs of McCabe Library without building an entirely new library. The Special Collections includes both the Friends Historical Library and Peace Collections, and currently occupies a footprint of roughly 5300 ft^2. The Treasury Room occupies roughly 900 ft^2. If we project a two percent growth per year in the size of these collections, (Planning, 18) and 15 years between the time the library extension is built and a replacement for McCabe is begun, the collection will grow to 8300 ft^2, space that will need to be built in to the Tarble renovation. At a cost of $210 per square foot for the new addition and $195 per square foot of renovation, the projected cost of the project is $3.6 million

4.3 Architectural Design

A guiding principle of the design is an openness and transparency of community. Here the community space of the library, the reading room, is not stuffed in the center of the building, but is brought to the outside against a backdrop of a large partition wall. Inside the space are the utilitarian functions of the library, including the stacks on the first floor, and the circulation desk. The large partition wall serves several functions. It helps to unify the new and existing structures, being clad in a similar stone to the original building. It serves as a formal separation of academic and social spaces. It blocks some of the direct sunlight that would be shining on the books, helping to preserve the collection. Finally, the wall allows access to the inner building to be restricted at night. This is a very important function. The building is designed so that it might be used at all hours of the day. In order to allow this, access to the stacks and the special collections must be blocked at night.

The first floor of the existing structure will be used as a rotating display for some of the more interesting pieces available in the Friends Library. This is an important function, serving to remind Swarthmore of its Quaker roots by displaying some of its past history. This is a need that is not being met in the current space provided by McCabe.

The basement of both buildings will be used for compact shelving and storage of collections. The basement is the most appropriate place for these books; sunlight must be kept to a bare minimum, so windows must be minimized or eliminated. Compact shelving, required to increase the storage capacity of smaller libraries, also requires very high structural capacities that are harder to meet on upper floors. Access to such libraries by the general public must also be strictly controlled. All of these factors place the Friends Historical Library, the Peace Collections, and the Treasury Room in the basement. This combined space is estimated to have a holding capacity of 51,025 volumes.

The second floors of the new and existing structures will be used for collective meeting spaces and classrooms, providing students with blackboards and computers. The first floor is designed to be blocked off at night, but the second floor is not. As such, it can always be accessed. The circulation of the building is designed such that the stairs to the second floor are not closed at night.

Square footage required by certain features of the building were initially estimated using the library design manual, *Planning Academic and Research Library Buildings.* The initial estimates for space requirements were as follows:

Table 3: Usage Requirements

Existing

Area	sq. ft.
first	2312.5
inaccessible	108
second	947.3
inaccessible	72
basement	2312.5
total	5752.3

New

Area	sq. ft.
treasure room	1200
special	7200
bathrooms	270
offices	400
microfilm	150
stairs	200
elevators	200
circ. Desk	200
total	9820

Notes

Area	sq. ft.
mechanical rooms	(add 30-50%)
compact shelving	(reduce by 30%)

5. STRUCTURAL DESIGN

5.1 Joist and Girder Steel Design

In the new addition to Old Tarble, several structural systems were considered, including variations on steel and reinforced concrete designs. In talking to Dan Honig earlier in the semester, it was decided that the most cost effective designs would be rendered in, in order of diminishing cost effectiveness, steel girder and joist-construction, pre-fabricated concrete sections, and concrete requiring custom framework, including waffle slabs.

5.1.1 Joists

There are two choices of joists that we considered, K-joists and W-shapes. K joists are trusses made up of single and double angles. They are also called open-webbed joists. They are designed to be very efficient in bearing distributed loads, and are thus highly economical. They have the added advantage of reducing the total depth of the flooring system- standard joists sit at or below the level of the girders that support it. W-shapes must necessarily be stacked on top of the girders. In some cases, however, this stacking behavior is preferable, as it allows mechanicals to be run through alongside. Air ducts can not be run perpendicularly through either joist, and usually hung directly below the structural steel members. This increases the total height of the floor, but may be avoided if joists and beams are laid out with consideration to these needs.

Using previously drafted floor plans, live loads for each of the floors were calculated according to ASCE 7-02 as discussed previously. Initial dead load for the metal deck and concrete was estimated at 35 psf. Mechanicals were estimated at 10 psf, and the ceiling material was estimated at 5 psf. This created a total dead load of 50 psf. This was combined with information about live loads from the previously included table. The loads were factored as per LRFD, and the appropriate maximum moment per joist was calculated. The joists were modeled as simply supported beams with a distributed load. The moment was therefore calculated simply as in equation (19).

The spacing for the joists was approximated such that the maximum moments were kept fairly consistent from bay to bay. This was done to keep the section depths consistent. The American

Institute of Steel Joists' Standard Specification for Open Web Steel Joists, K-Series was then consulted. Here the table lists total safe uniformly distributed load-carrying capacities, in pounds per linear foot and live loads per linear foot of joist which will produce an approximate deflection of 1/360 of the span. Because the live loads that we find in this building were usually quite a bit higher than the dead loads, the deflection due to live load tended to govern when picking joists.

Another factor that was considered in picking joists was the depth of the joist itself. It is usually given as a rule of thumb that for every foot of span, an open-web joist should have one inch of depth. In our structure, the depths of the joists were limited to 14 inches on the first floor. Because clear spans were sometimes as large as 19'6", this depth was not ideal. However, because of architectural considerations, this was necessary.

The table was thus used for 14 inch depth K-joists, and each given span and allowable linear load. For the second floor, the depth was less critical, however, all joists were kept as shallow as possible. The results are tabulated below:

Table 4: Second Floor K-Joist Selections

Span	Joist Spacing c.c. (in.)	Dead Load (psf)	Live Load (psf)	Joist Selection
14.0	5.25	50	60	12K3
8	5.25	50	60	8K1
18	4.75	50	60	12K5
15.7	5.25	50	60	12K5
18.0	4.75	50	60	12K5
10.0	5.25	50	60	8K1

Table 5: First Floor K-Joist Selections

Span	Joist Spacing c.c. (in.)	Dead Load (psf)	Live Load (psf)	Joist Selection
14.0	3.75	50	100	14K1
19.5	3.75	50	100	14K6
16.0	3.75	50	100	14K3
15.4	3.75	50	100	14K3
14.3	3.75	50	100	14K1
14.3	3.75	42	100	14K1

For certain parts of the floor system, it was preferred to use W-shapes instead of K-joists. Using W-shapes allows the joists to be placed above the top of each beam, which allows clear space for air ducts and electricals that would otherwise have to be hung below the joists. Air ducts typically require at least 8 inches of clear space. The W-shapes were designed in Multiframe assuming that each joist would span only one bay, and would be simply supported at either end. The depth of each member was kept above 8 inches.

The W-shapes used are as shown in the following table:

Table 6: First Floor W-Section Selections

Span	Joist Spacing c.c. (in.)	Dead Load (psf)	Live Load (psf)	Joist Selection
19.5	2	50	150	W8x21
16.0	3	50	150	W8x18
15.4	3	50	150	W8x18
14.3	3	50	150	W8x15
14.3	3	50	150	W8x15

5.1.2 Decking

Steel decking with reinforced concrete slab is a very economical floor system. When pouring single slabs, plywood formwork is usually put in place and later removed when the slab has cured. Steel decking may be used instead as the formwork, and is simply left in place after the slab is cured. The remaining steel deck tends to act as tensile reinforcement. The decking is usually ribbed to increase its strength. The ribs run perpendicular to the joists and transfer their loads to these joists as a one-way slab would. Depending on the deck design, the decking is allowed to overlap at the seems. Separate sections of deck are simply welded together at these overlapping joints.

After the joists were selected, and the joist spacing was established, the deck was designed. The deck must be strong enough to carry the dead load of the wet concrete immediately after it is poured. To design the deck, we referenced the Vulcraft product catalog, which had several tables for each type of decking. The tables showed maximum construction clear spans allowed by the Steel Deck Institute, allowable uniform loads and reinforced concrete slab allowable loads. The first table looked at was the maximum construction clear spans between joists, which determined which thicknesses of decking could be used for a given clear span. After picking a suitable decking depth and gauge, those values were checked against the second table for allowable uniform load. Here, we assumed the decking would cross three spans and have a maximum allowable deflection of L/240 (which proved to be the most conservative of the design criteria). The decking type was also assumed to be non-composite, which is designated as the C-series of decking in the Vulcraft manual. For a given clear span, the maximum load is found under this table. The concrete slab required for this decking was found in the third table for reinforced concrete slab allowable loads, and the two-way wire reinforcement was picked. The results are tabulated below:

Table 7: Deckings Used in New Structure

Joist Spacing c.c. (in.)	Live Load (psf)	Deck Selection	Concrete (pcf)	Total Slab Depth (in.)	Wire Reinforcing
3.75	100	1.0C26	110	3	6x6-W2.1xW2.1
2.75	150	1.0C26	110	2.5	6x6-W2.9xW2.9
5.25	60	1.3C24	110	3.8	6x6-W2.1xW2.1

Earlier in the design process, the dead loads were assumed to be 35 psf for the dead load of the decking. This assumption was double checked, and it was found that all three decking choices were below 35 psf. Another check was made to ensure that the concrete met the minimum thickness requirements for fireproofing, which are provided for in IBC 2003. The slab thicknesses proved to be adequate.

Table 8: Fireproofing from IBC 2003, Table 720.1(3)

Floor or Roof Construction	Item No.	Ceiling Construction	Thickness of slab for 4 hour fire rating	Thickness of ceiling for 4 hour fire rating
Steel joists constructed with a reinforced concrete slab on top poured on a 1/2" deep steel deck	8-1.1	vermiculite gypsum plaster on metal lath attached to 3/4" cold rolled channels with 0.049" (No. 18 B.W. gage) wire ties spaced 6" on center	2.5"	3/4"

5.1.3 Beam and Column Design

The weight of the floor system can now be calculated. The weight of the joists are known, as are the true weight of the decking system. These dead loads are then transferred to the beams, and then to the columns. It is assumed that the deck and joists together act as a one-way slab. This means that either of the two beams supporting any given bay are required to carry half of the dead load of the slab as well as half of the live load.

The next step was laying out the girders themselves. Several factors were considered in this step. We tried to minimize cost by minimizing the amount of steel used and the number of connections that have to be made. For the seond floor, it was found that the unique shape of the floor could be built using either eight or ten beams. The ten beam designs were quickly scrapped for economic reasons. The remaining designs were vetted according to how conducive to the layout of HVAC systems they were. It was ultimately decided to hang the HVAC systems below the second floor instead of running them through the floor. Ultimately, however, the governing factor in the design was an architectural one- because about one quarter of the second floor is cantilevered, it must have enough continuous supporting girders to support it.

Other assumptions made were that columns and beams were spliced where appropriate. When a column meets with a beam, one of the two members has to be cut in two. We decided which member to cut in two by determining which member had the greater slenderness ratio, and thus, which member was more likely to buckle.

The girder layout was then loaded using the weights of the floor system which was previously determined. They are applied as local linear loads in accordance to one-way slab design. These loads were inserted into a 3d model in Multiframe where individual girders will be designed, again using the design function. The depth of the girders and floor joists will in turn determine the design of the columns. They are assumed to be linear elastic subject to only axial loads, and are designed in both Multiframe.

Wind loads, as previously written about, have been found using ASCE 7-02. These positive and negative loads exert lateral pressure on the building. It is assumed that all other joints are pinned and simply supported, and will therefore play no role in resisting moment in the frame. It is assumed that the lateral loads will be supported wholly by two braced frames at the end, and one moment frame in the middle. The braced frame is a rectangular truss where all joints are pinned. The moment frames are standard frames where the all of the joints are restrained relative to moment. All connections between footings and columns were assumed to be pinned, and not moment restrained. Multiframe Steel Designer was then used to pick the appropriate members. Where members were too deep, extra supports were added. After these joint connections were finalized, the frame looked as it does below. There were about nine different versions of the frame.

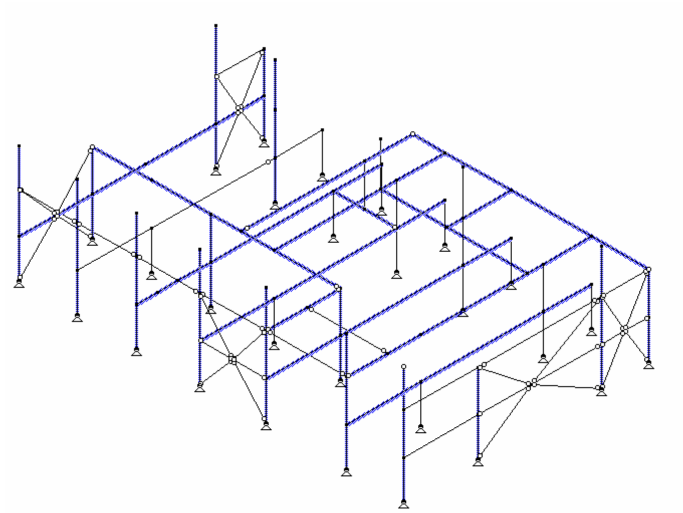

Figure 7. Multiframe diagram of frame with joint restraints

Another beam was also added across the top of all of the frames. This beam supports all eight of the roof trusses. It is pinned at the ends and at the middle. The other four supports are all rollers. The purpose of this beam is to distribute the lateral forces from wind and seismic loads from both the roof and the front and back façades across the frame into the moment and braced frames.

5.1.4 Footing Design

Footings were designed with reinforced concrete. They were modeled as two-way slabs, with checks for the over-turn ratio as well as checks for punching shear. The finalized designs are included with the footing schedule in the drawings section.

5.2 Roof Design

The roofing system in the new building is similar to that in the existing structure. The roof is supported by purlins that transfer loads to the roof trusses. The purlins rest upon the trusses and are kept in place by the use of sag rods, which also prevent biaxial bending of the purlins. Design of the individual roof truss members was based on axial forces determined by Multiframe in a manner identical to those in the existing building. See section 3.2.6 for discussion of the determination of axial design forces. The new roof trusses are supported by a pinned connection on one end and a roller at a joint 15 feet from the other end. This was necessary because no columns were to be constructed at the face of the glass wall.

The Multiframe model used to analyze the new roof trusses required only two steps rather than the three for the existing roof trusses. This is because the purlins in the new structure were designed such that they were supported at truss joints and at the centers of truss members. Step 2 described in section 3.2.6 could be omitted because load transmitted through the purlins was evenly dispersed between adjacent joints. An example of load transmittance in the Multiframe model for the roof trusses of the new structure is shown in Figure 7 below.

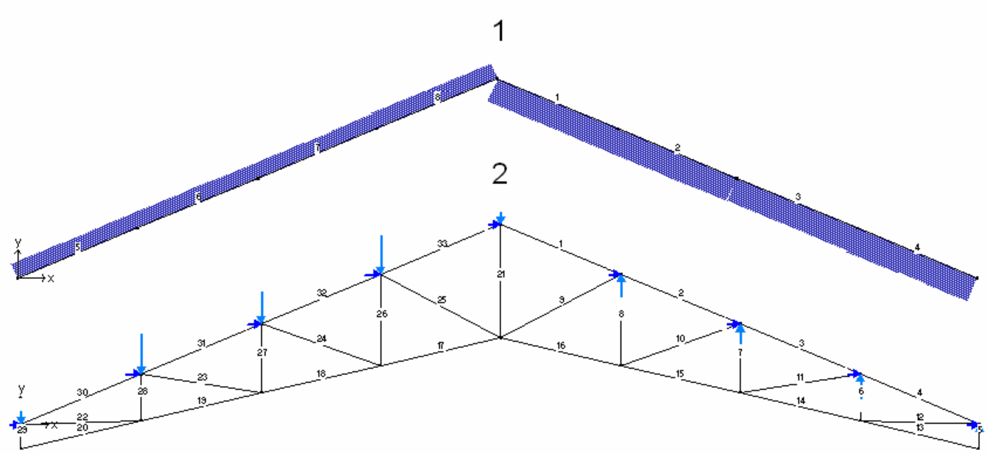

Figure 8. Wind loading example for Multiframe model of new roof trusses

5.3 Concrete Floor Design

A two-way flat slab floor system with drop panels was designed using the equivalent frame method. The direct design method was not applicable because some successive spans differed significantly from one another and thus the slab did not meet criteria set forth in ACI 318-02 for design by this method.

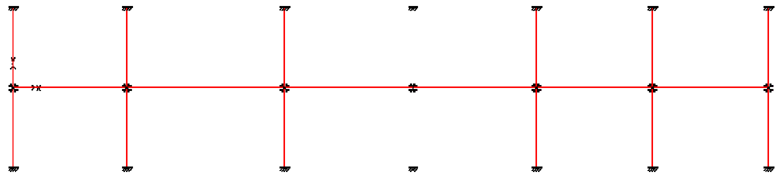

Figure 9. Equivalent Frame

In two-way slab design, the floor is divided into a series of strips in each direction. Column strips are located directly over the columns, and middle strips are located between columns. A design strip is composed of a column strip and the two half middle strips on either side.

The first step of the equivalent frame method is to determine slab thickness. For an interior panel of a two-way slab without beams, this is taken as some amount greater than one-thirtieth of the clear span. During this preliminary stage, shear strength is checked by considering a one foot wide strip at a distance d from the face of a support. Shear strength of concrete for a square interior column is determined by the following equation:

$$\phi V_c = \phi \sqrt{f_c^{'}} b_o d \tag{23}$$

Where: Φ = undercapacity factor (0.75)
f'_c = compressive strength
b_o = length of one edge of a square column
d = depth to center of reinforcement

As was necessary in this design, a drop panel can be added to increase shear strength at a column. In this instance, shear must be checked both in the drop panel and in the slab beyond the edge of the drop panel.

In the second stage of the equivalent frame method, members of the equivalent frame were defined in order to determine distribution factors and carry over factors for moment distribution. The carry over factor was determined from a table after determining the flexural stiffness, K_{sb} from equation (23) as follows:

$$K_{sb} = K_{NF}\frac{E_{cs}I_s}{l_1} \tag{24}$$

Where: K_{sb} = flexural stiffness
K_{NF} = stiffness factor
E_{cs} = modulus of elasticity of the concrete slab
I_s = moment of inertia of the slab
l_1 = panel span in the direction of the design strip

The slab-beam joint distribution factors were determined by dividing the flexural stiffness of the slab by the summation of flexural stiffnesses and equivalent column stiffnesses at each joint. The equivalent column stiffness was determined as follows:

$$K_{ec} = \frac{\sum K_c \times \sum K_t}{\sum K_c + \sum K_t} \tag{25}$$

Where: K_c = flexural stiffness of column members
K_t = torsional stiffness of torsional members

The torsional stiffness in equation (24) was determined from equation (25):

$$K_t = \frac{9E_{cs}C}{\left[l_2\left(1-c_2/_2\right)^3\right]} \tag{26}$$

Where: K_t = torsional stiffness of torsional members
E_{cs} = modulus of elasticity of concrete slab
C = torsional constant, $\sum(1 - 0.63\text{x/y})(\text{x3y/3})$
l_2 = panel span in the transverse direction of the design strip
c_2 = column width

In the third stage of the design of a two-way slab strip, moment distribution was performed for fixed end moments under a number of patterned loading scenarios. ACI 318 states that if a live load is very significant in proportion to the dead load, then a number of loading scenarios must be considered in addition to the full factored loading case. Maximum negative moment might be governed by loading 3/4ths of the full-factored live load on adjacent panels. Maximum positive moment might be obtained by loading 3/4ths of the full-factored live load on alternating panels.

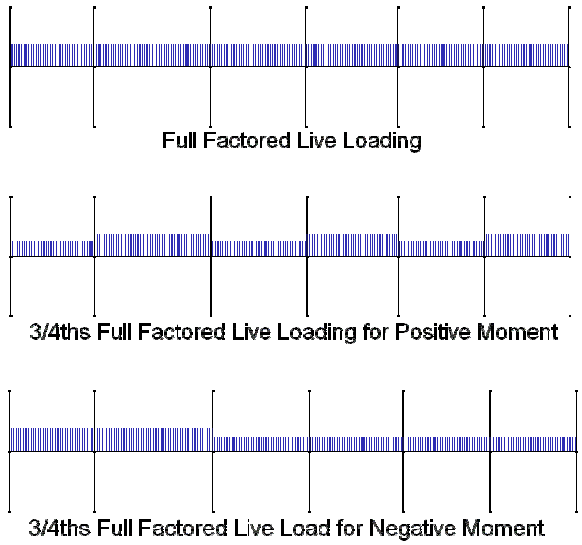

Figure 10. Factored Loading Scenarios for Moment Distribution in Two-Way Slab Design

In the fourth stage, design moments are determined by considering the moment distribution curves created in stage three. The maximum negative moments were taken at the edge of the drop panel.

Stage five took the design moments determined in stage four and distributed them across the slab-beam strip, according to the percentages in Table 9.

Table 9: Distribution of Factored Moments

	Column Strip		Middle Strip	
	Negative	Positive	Negative	Positive
Moment Distribution (%)	75	60	25	40

Finally, in stage 6, the design moments at the columns and at the midspans were used to design reinforcement and to check slab flexural and shear strength.

5.4 Concrete Slab on Grade Design

5.4.1 Obtaining a K-value

Boring logs from nearby Mertz dormitory and Alice Paul Hall suggest that the subsurface beneath Old Tarble is most likely composed of sandy clayey silt, sandy silt and silty fine sand at depths below nine feet, the depth of the basement floor. Borings performed for Alice Paul Hall suggest that the soil has a USCS classification of SM at a depth of 8 to 10 feet and CL from 3-5 feet. A conservative assumption for the corresponding standard modulus of soil reaction, K, from Figure 1 would be 65 lbs/in^3. However, since Alice Paul Hall is several hundred feet away and since no subsurface information is available for Old Tarble, we used a more conservative estimate of 50 lbs/in^3.

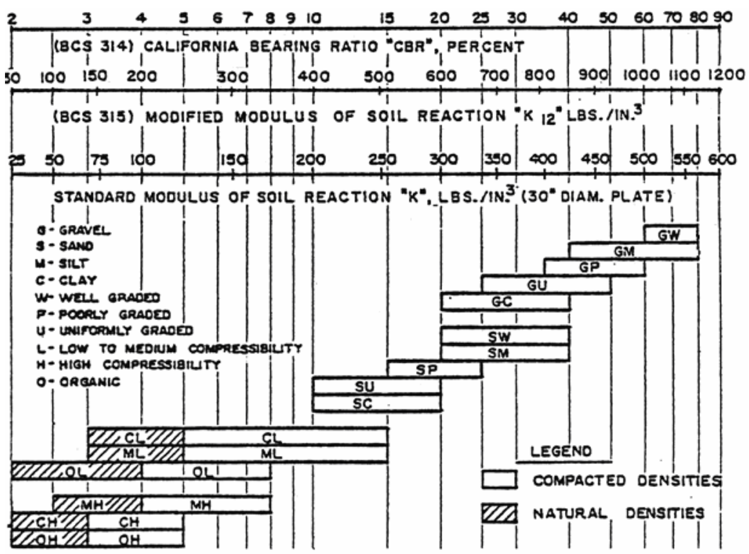

Figure 21. Modulus of soil reactions for USCS classifications

5.4.2 Determining Concrete Type and Slab Thickness

Slab thickness and concrete flexural strength were determined from Table 1 for a 160 psf load (150 psf live load plus 10 psf miscellaneous) with nonuniform loading and variable layout. It was clear from the table that a slab width of 5 inches and concrete flexural strength of 550 psi would be more than adequate, even after considering a safety factor of 1.5. Therefore, concrete with a compressive strength of 4000 psi was chosen.

Table 10. Allowable distributed loads, unjointed aisle (nonuniform loading, variable layout)

Slab Thickness, in	Subgrade k' pci	Allowable load, psf			
		Concrete flexural strength, psi			
		550	600	650	700
5	50	535	585	635	685
	100	760	830	900	985
	200	1075	1175	1270	1370
6	50	585	640	695	750
	100	830	905	980	1055
	200	1175	1280	1390	1495
8	50	680	740	800	865
	100	960	1045	1135	1220
	200	1355	1480	1603	1725
10	50	760	830	895	965
	100	1070	1170	1265	1365
	200	1515	1655	1790	1930
12	50	830	905	980	1055
	100	1175	1280	1390	1495
	200	1660	1810	1965	2115
14	50	895	980	1060	1140
	100	1270	1385	1500	1615
	200	1795	1960	2120	2285

5.4.3 Determining Joint Spacings

Isolation joints must be provided to separate the slab-on-grade from the interior walls and columns due to differential settlements that our bound to occur. Likewise, control joints must be provided to prevent cracking caused by concrete shrinkage. These control joints will be keyed and sawcut in order to allow some lateral expansion or shrinkage while preventing any vertical displacement. The maximum spacing for control joints was determined from Figure 2 to be approximately fifteen feet. Since the spacing between columns ranges from 14 to nearly 26 feet, the spacing between control joints will be taken as half the distance between any two columns.

5.4.4 Designing Steel Reinforcement

Steel reinforcement was chosen to be provided in the normal manner by use of welded wire fabric. The appropriate amount of steel reinforcing required was determined from Figure 3, which uses the following equation from subgrade friction drag theory:

$$A_s f_s = w\left(\frac{L}{2}\right)F \qquad (1)$$

Where A_s = cross-sectional area of the reinforcement (in^2) per foot of slab width
f_s = allowable stress in the reinforcement (assumed to be 30,000 psi for welded wire fabric)
w = weight of the slab (lbs/ft^2)
L = distance between joints (ft)
F = subgrade friction coefficient (assumed to be 1.5)

For a slab thickness of 5 inches and joint spacing of 13 feet, reinforcement was chosen to be 6 X 6 W1.4 X W1.4.

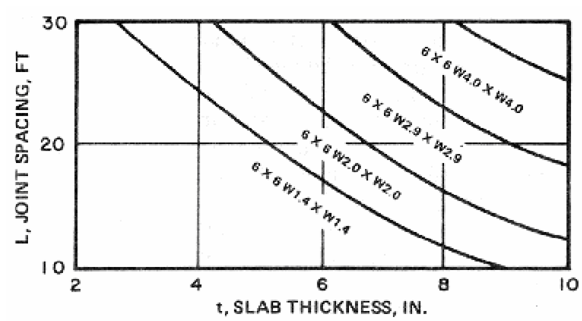

Figure 13. Minimum reinforcement for 30,000 psi allowable steel stress and 1.5 subgrade friction factor

6. CONCLUSIONS

The goals for our project were mostly met. Analysis of the existing structure and architectural design were completed, as well as most of the steel design and some of the concrete design. Failure to complete structural design of the addition was not an unexpected result, given the severity of time constraints that came into play. As a result, future work could be performed in order to improve and complete our structural designs in both steel and concrete.

Beams, decking, columns, footings, cross bracing and roof trusses were all designed in steel for the new structure. Elements not yet designed include: foundation walls, stairs, and joint connections. Concrete design was less complete. Time only permitted the design of one column strip using the equivalent frame method. The design of this strip assumed the use of identical columns at every slab-column interface, although a more complete design would consider different column types. As it stands, our design is very conservative.

Although we allotted space for heating, ventilation, and air conditioning, we did not design the actual system. More specific considerations of the system would be required if the structure were to be built. Libraries tend to require highly specialized systems with very close tolerances. Fireproofing was also ignored, but like heating, ventilation, and air conditioning, is beyond the scope of the project. Cost estimates were also largely ignored. There is still quite a bit more work to be done if this building were actually to be built.

We accomplished as mush as we hoped to. We went through the very earliest conceptualization with the clients, through the early site planning, through the structural design. We created a design from nothing.

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8. APPENDIX

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