RACE STREET DORMITORY

PHILADELPHIA, PA

STRUCTURAL THESIS
SPRING 2007

ADVISOR: PROF PARFITT

DOUGLAS TOWER

Executive Summary

The Race Street Dormitory is a recently constructed residence hall housing students in twelve stories of 4 person suites, 12 suites per floor. Architecturally, it is a sharply angled building with two nearly perpendicular wings. Bays in the building are very typical from the 2nd floor up, as they are predominately identical suites. It is a steel framed building with a hollow core plank floor system and brace frame lateral load resisting system. The structural system was mainly chosen for speed of construction and cost efficiency. Other traditional alternative systems do not compare in speed and cost. This report is an attempt to completely redesign the building in concrete with a system that meets these goals.

The alternative that will be studied is a structural system based around room tunnel form construction. Tunnel form construction is a method of using pre-built forms to form the sides of walls and flat slab together as one. Essentially, the structure becomes a complex shear wall and slab system since wall and slab elements are securely fixed to each other. The system is very economic for uniform bays and very rapidly constructed, as forms are poured one per day, then taken out the next to form another part of the structure. Simple designs are economic, variations cost extra, but blocking off sections of the walls is common practice for all sorts of openings. Problematic variations tend to be different bay sizes, and non rectilinear shapes.

The development of a redesign began with simple layout based on a typical bay within a suite. Problems arose because the building was not as uniform as it seemed- different forms were required, as well as aspects poured outside of the system, lateral resistance became a problem because of the number of walls and reentrant corner. It became apparent that the system was so stringent it could not be applied to design, but designed from scratch. Furthermore, the building would be designed as two structures.

Once a greater understanding the system developed, a uniform center line to center line 10’ high by 10’ wide bay was developed. This 6” slab and 7” walls were designed as well as beams and wall sections to accommodate major cutouts in the wall system on the first floor. Thin wall segments under high gravity loading were checked for capacity using PCA column, slab was reinforced using ADOSS. Wind was found to control in on both directions on one structure, and on one in the other. Walls were reinforced as shear walls based on the lateral load each resists due to its relative stiffness. The weak axis moment induced in the walls due to uneven live loading was found through moment distribution and the shear wall reinforcement was checked for sufficient capacity in this direction.

An architecture layout was performed based on adding the elements of the existing building into the redesign. Floors 2-11 had become more open and spacious due to a larger plan area needed to accommodate the forms. The first floor became clustered as it had been designed for more large open spaces which require deep beams to carry the walls above. A construction timeline was developed to find the time for construction in order to evaluate cost. The estimated time saving is 24 days, not accounting for an elimination of time constructing partition walls. Cost is questionably low, but likely to be cheaper than the existing structure.

Overall the system was effective in achieving design goals of speed and cost, but it came at a high price, and has impacted the entire building considerably. Most of this impact has been negative.

DOUG TOWER
STRUCTURAL

DMJ166@PSU.EDU

WWW.ARCHE.PSU.EDU/THESIS/EPORTFOLIO/2007/PORTFOLIOS/DMJ166/

RACE STREET DORMITORY

Drexel University

3300 RACE STREET
PHILADELPHIA, PA

GENERAL

- -INTECH CONSTRUCTION, GENERAL CONTRACTOR AND CONSTRUCTION MANAGER
- -ERDY MCHENRY ACHITECTURE, LLC, ARCHITECT
- -INDOOR QUALITY SOLUTIONS, MECHANICAL ENGINEER
- -CAGLEY HARMON & ASSOCIATES, INC. STRUCTURAL ENGINEER
- -PENNONI ASSOCIATES, INC. CIVIL ENGINEERS
- -132,800 S.F
- -$22M CONSTRUCTION COST
- -DESIGN-BUILD DELIVERY METHOD
- -SCHEDULED COMPLETION FEBRUARY 2007

ENVELOPE

- -FLOOR TO CEILING GLAZED CURTAIN WALLS
- -COMPLEX EXTERIOR MEMBRANE MADE FROM RIBBED, SMOOTH AND CORRUGATED ALUMINUM , STEEL , AND ALLOY-COATED STEEL
- -COLD FORMED STEEL STUDS WITH FIBERGLASS INSULATION
- -EPDM SINGLE-PLY MEMBRANE ROOFING OVER RIGID INSULATION
- -SLAB ON GRADE
- -EXPOSED CONCRETE WALLS AT GROUND LEVEL

STRUCTURE

- -CONCRETE PIER AND FOOTING FOUNDATION
- -DIAGONALLY BRACED STEEL FRAME AND MOMENT CONNECTIONS
- -PRECAST CONCRETE HOLLOW CORE PLANK FLOORING
- -6' SPACED BEAMS AND STEEL DECK TO SUPPORT ROOFING

ARCHITECTURE

- -"L" SHAPED BUILDING
- -12 STORIES ABOVE GRADE-SECURITY DESK, MAIL ROOM, MULTIPURPOSE ROOM ON FIRST FLOOR
- -EXERCISE ROOM ON SECOND FLOOR
- -130 STUDENT SUITES FLOOR TWO THROUGH ELEVEN, 490 STUDENTS
- -TWO BEDROOMS, LIVING , BATH, KITCHEN PER SUITE
- -TWO COMMON ROOMS PER FLOOR
- -3 ELEVATORS, 2 STAIRWELLS

LIGHTING/ELECTRICAL

- -BOTH 480/277 V AND 208/120 V
- -3 PHASE, 4 WIRE SYSTEMS
- -EMERGENCY DIESEL GENERATOR AT GROUND LEVEL

TYPICAL KITCHEN UNIT-
    1'X4' SURFACE CLOUD
TYPICAL BATH UNIT-
    4"X2' WALL CLOUD
TYPICAL BED UNIT-
    14" DIAMETER WALL SCONCE
TYPICAL CORRIDOR UNIT-
    2'X2' PRYSMATIC LAYIN

MECHANICAL

- -FAN DRIVEN AIR VOLUME SYSTEM
- -EMERGENCY STAIR PRESSURIZATION FANS

Table of Contents

I. Acknowledgements

I would like to thank all those who helped me with this project this year. Chas Ricciardi at INTECH Construction helped me initially chose a building, has been there every step of the way, and has been a faithful servant of information in regards to the project. Dave Gust, a general superintendent at Centex Construction in Florida was wonderful with helping me decipher tunnel form construction. Tom Belace, of Alliance Structural Engineers in Houston TX gave me a great starting point early in the semester and helped a lot. DJ Cramer, an employee at Symons who specializes in tunnel forms was also an invaluable asset. Thank you all.

II. Introduction and Proposal

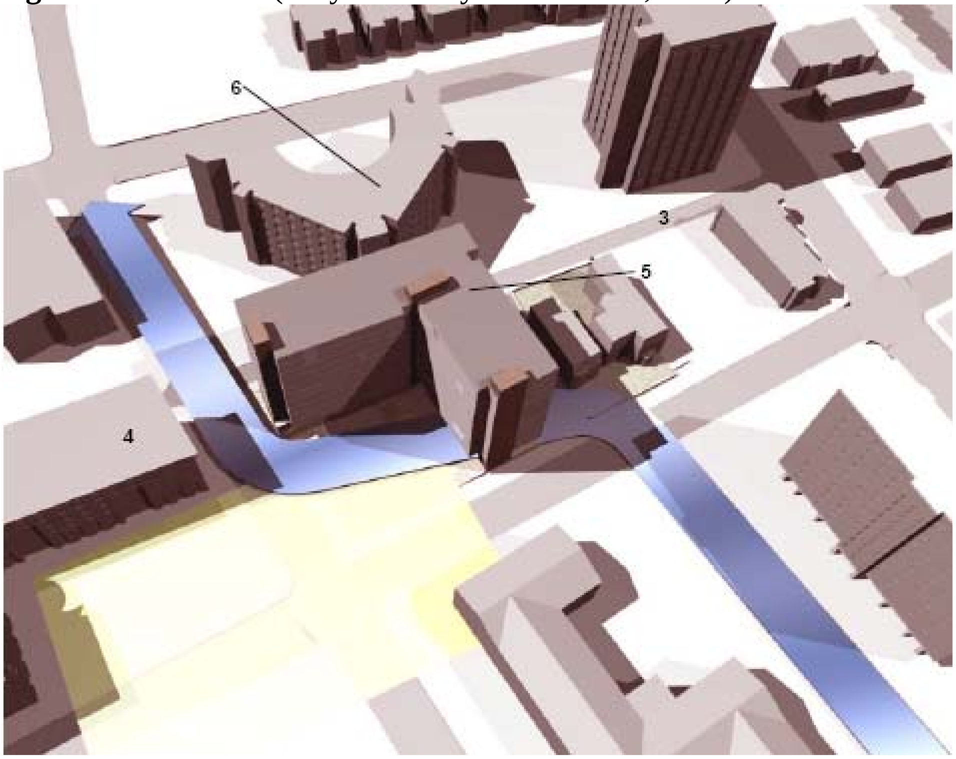
Recently, Drexel University began construction of a new dormitory on Race Street in Philadelphia, PA. The structural and architectural components of this residence hall are well designed in terms of quality, cost, and construction and use of space. This final report is a summary of the design of that structure and a further exploration of an alternative structural and architectural design. It is an attempt to design an alternative structure with the goals of the original designers in mind. Based on research, a very possible alternative is a reinforced concrete building utilizing a room tunnel form construction method. This report will give first give a background into the existing design of the Race Street Dormitory, next a brief introduction to tunnel forms, and finally a redesign with structural, architectural, and construction analysis of this alternative design. For the purposes of this report, the actual constructed dormitory will be referred to as the “existing design” or “existing structure”, while the alternative design discussed in this report will be referred to as the “new design” or simply, “redesign”.

III. Background

i. Design

This Race Street Dormitory was designed around Drexel University’s desire for a fast tracked design-build project. The university requested a residence hall with 10 stories of suites with 11 to 13 suites per floor and 4 students per suite. The building also needed to accommodate Resident Assistants and other space needs, including a lobby, mail room, etc. The original architect designed the building in an L-shape in order to architecturally respond to another residence hall, North Hall, and accommodate future circulation patterns of the university. A major factor that influenced the structural design was speed, and fast tracking and prefabricated members were used. Floor to floor heights were especially critical to keep building height to a minimum for cost considerations. See figure below in figure 1b:

Figure 1b: *Site Plan* (Erdy McHenry Architecture, 2006)



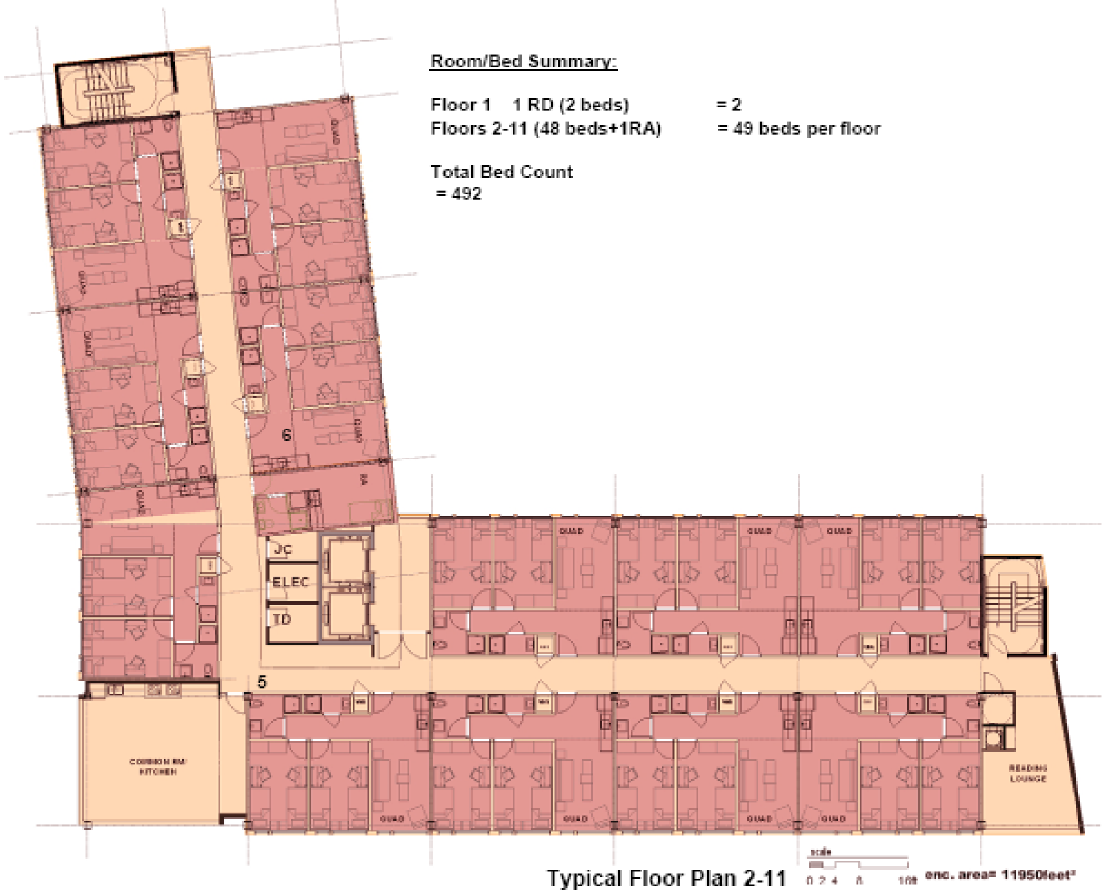
Legend:
North Hall = #4, Race Street Dormitory = #5, Blue line is future circulation pattern.

ii. Existing Building

The Race Street Dormitory is a twelve story, 120 ft high steel framed building with hollow core plank decking. As discussed earlier, the dormitory is an 'L' shaped building with legs roughly 116 ft and 165 ft long that veer 4 degrees off a right angle at one point. At its lowest level above grade, the building consists of only part of one leg of the 'L' shape- a roughly rectangular length running east-west. This ground level consists of mechanical rooms, an electrical room, and maintenance rooms as well as a shop and bicycle room. This floor is abutted against a higher grade (one story higher) on which sits the shorter wing of the building on free standing columns. Figure 1 shows some of the flooring at this level and the piers for the free standing columns. An enclosed first floor lies on the footprint of the ground floor and contains the main entrance lobby, a security entrance, a mailroom, a Resident Assistant suite, and a large common room. The second floor and consecutive floors form the main "L" shape of the building. These floors have a central hallway with rows of suites on either side. Suites have two bedrooms, common room, two showers, two baths, and kitchenette. There are three elevators at the south-east corner (bend) in the building, one of which begins at the ground story level. There are two stairways at the far north and east ends of the building. (See figures below)

Figure 2b: *First Floor Early Architectural Plan (changed slightly)* (Erdy McHenry Architecture, 2006)

Figure: *Typical Architectural Floor Plan (changed slightly)* (Erdy McHenry Architecture, 2006)



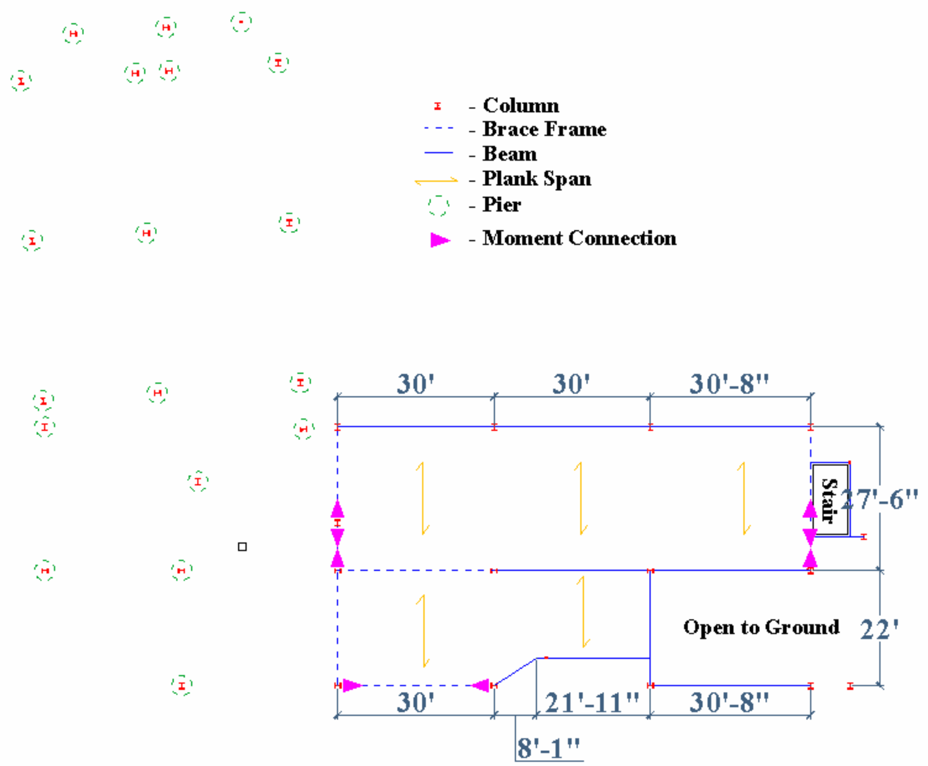
iii. Existing Structural System

The residence hall is mainly a steel W-shaped column and beam frame with moment connections, moment frames, braced frames. The floor to floor heights are 9’4” for floors two through eleven, 14’ for level one, and 10’ for ground level. (See figure 5 for height layout) Beams run predominately longitudinally along the building, as floor planks span two horizontal bays. (See figures 1-3) Beam sizes are mainly W12 or W18, and span up to 30’8”. The third through eleventh floors have identical beam systems, while the beams at the first and second floors are unique and generally larger.

The roof is flat and consists of mainly W12 purlins spaced 6’ on center and Grade 33 structural galvanized steel decking supporting EPDM single-ply membrane roofing over rigid insulation.

Each floor consists of pre-stressed pre-cast hollow core concrete planks 8” deep, typically 8’ wide with 2” cast-in-place concrete topping. (See figures 1-3) The planks are typically 22’8 or 28’2” long (8” overhang typical). The maximum depth of the floors is about 28” (roughly 18” beams, 8” decking, and 2” leveling slab), but, as noted before, beams do not frame each bay of the system, and are not intermediately placed within bays. This allows for up to 90’ expanses in length of 10” deep flooring uninterrupted by beams (see fig. 3).

Figure 1 - First Floor Framing Plan



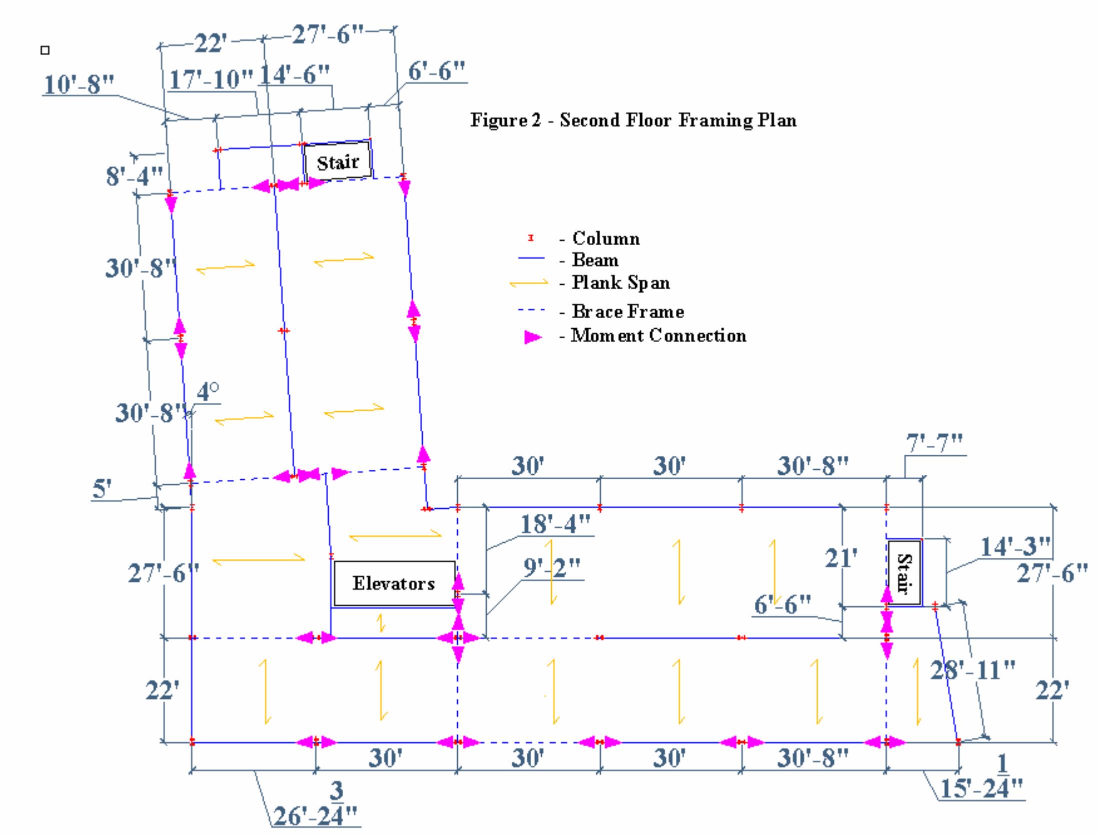


Figure 2 - Second Floor Framing Plan

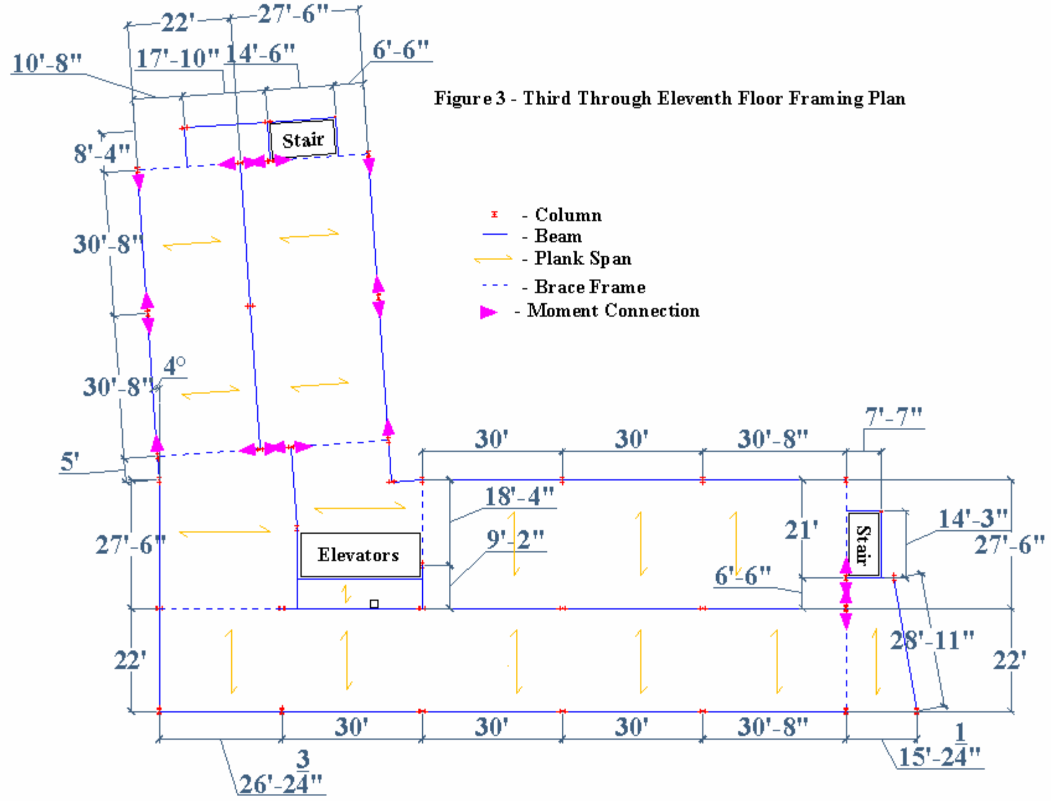


Figure 3 - Third Through Eleventh Floor Framing Plan

iv. Lateral Load Resisting System

The lateral load resisting system is a series of diagonally braced frames with moment connections and moment frames. The two wings of the building were designed with separate lateral systems. The moment frames were used where brace frames would not work for architectural reasons (along exterior walls). Brace frames were designed to accommodate hallways down the center of the building, and lobbies and other open spaces. (See figures 4-9)

Figure 4 - Brace Frames 1 and 2 with Wind Loading

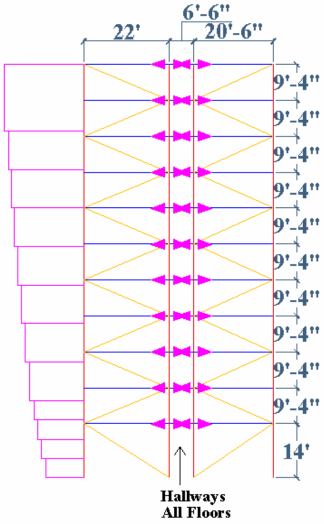


Figure 5 - Brace Frame 3 with Wind Loading

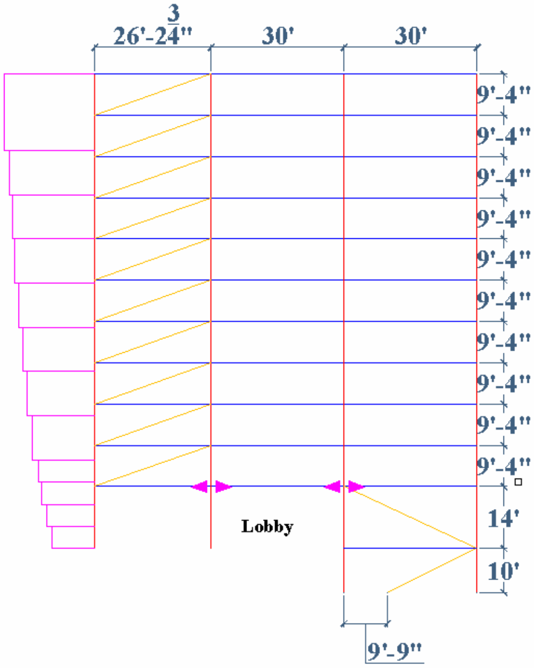


Figure 9 - Moment Frames 1 and 2

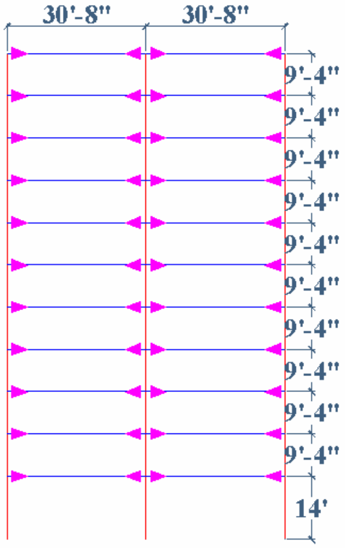


Figure 6 - Brac

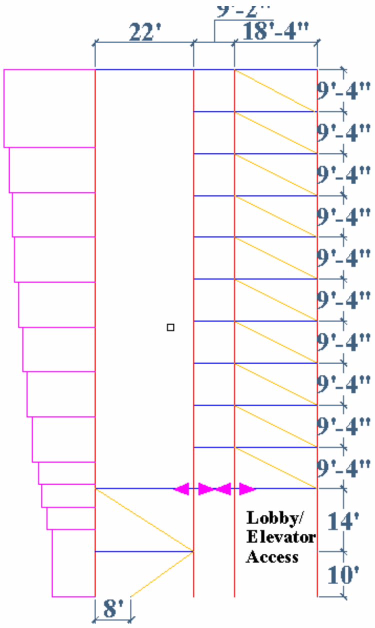


Figure 7 - Brace Frame 5 with Wind Loading

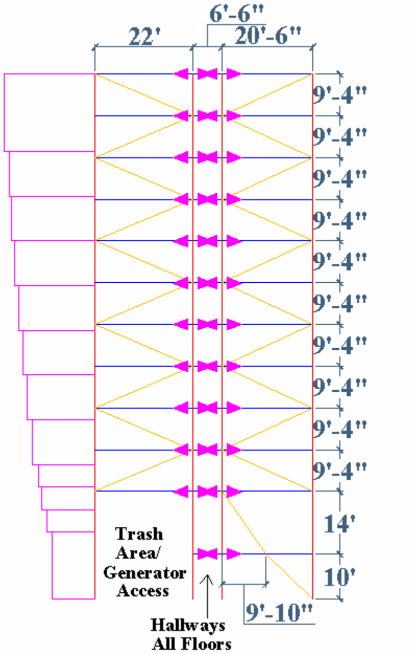
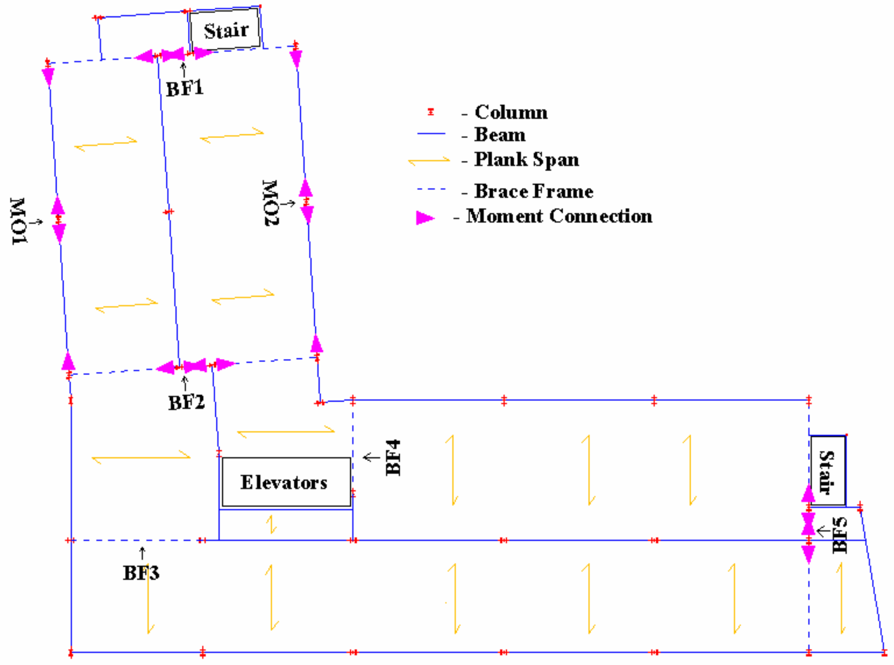


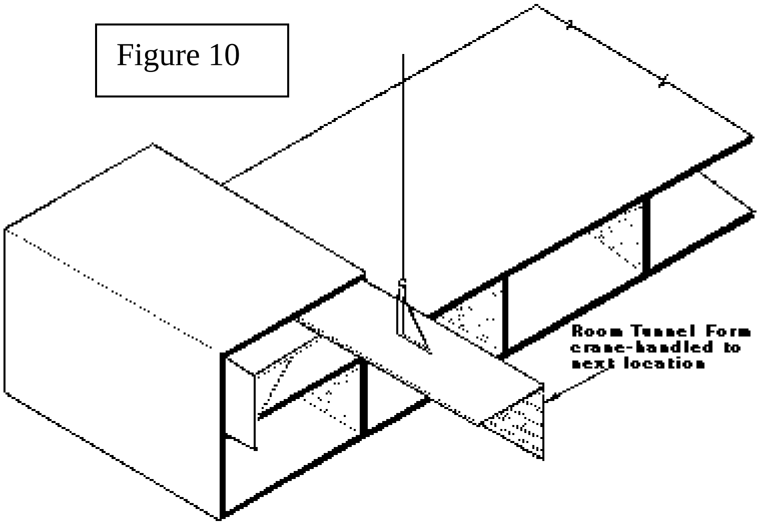
Figure 9 - Third Through Eleventh Floor Braced Frames



IV. Tunnel Form System Overview

Tunnel form construction is a simple, viable method of building a concrete structure in a timely manner, and without the large columns associated with two way slabs. The method uses large, pre-built, angled forms to form a load bearing wall-slab structural system. Forms are steel and braced upright with wheels (figure 12). They are slid in from the exterior by crane, two per bay, forming the walls and floor above as shown in figure 10. Floors are separated into sections which are poured all at once in one day with the use of high early strength concrete (>1000 psi in 24hr) often cured with the help of heaters and tarp coverings in the night which can increase the strength to up to 1600 psi in 24 hr. Once a floor section is poured, it is left to cure overnight before the forms are removed. As the forms are taken out, temporary pole shoring is installed under the slab until the concrete reaches a 3000 psi compressive strength in order to insure no creep deformation. Typically shoring is placed at 8' spacing along the length of a tunnel. Once removed, forms can be easily lifted into another position in the structure and form another series of walls and sections. Block outs within the forms are possible in order to open up larger spaces or create openings for door/window openings or plumbing. Typically, the floor sections are reinforced on day, formed the next, poured the following, and forms are removed the next.

Figure 10

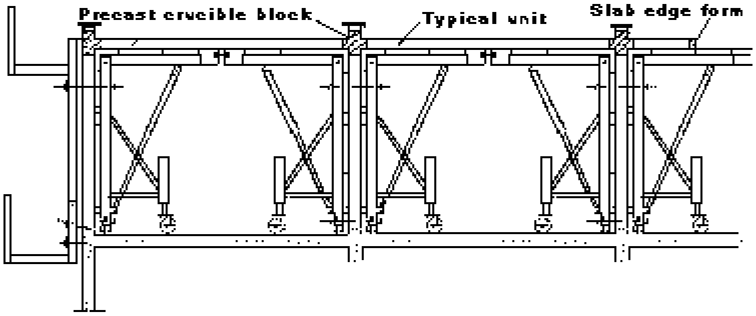


There are many advantages to a properly designed tunnel form system. The method is highly efficient because of the modular dimensions and simplicity involved. Unlike typical concrete formwork, forms are pre-built, reused, and easily moved. Dimensional accuracy and quality are also strong due to uniformity, which enables a direct paint finish. A building dominated by concrete walls also has many positive properties such as good fire protection (and therefore lower insurance rates), acoustical isolation (important in a loud dormitory!), durability and reduced maintenance costs, and strong insulating properties creating energy savings. Of course a uniform plan layout is key in keeping costs down and construction speed up. Non-uniform bays, non-uniform tunnels, and a layout not capable of being built in carefully scripted sequence all raise costs. Forms can be purchased (cost effective if one plans on more than one similar building) or rented, where purchased forms clearly need to be identical in order to accommodate later projects. For projects requiring a very different first floor layout, columns and a thick distributing slab are used at the first story, but this system is not as cost effective as carrying the formwork down to the ground. In the case of this redesign, the structural system will consist of tunnel formed walls from ground to roof; partly because of the extra cost of the distributing slab, and party to explore the architectural challenges and possibilities of using these forms on floors with less uniformity.

Figure 11 – an example of the results of blocked out tunnel form construction


copyright Highrise Concrete Systems, Inc.

Figure 12 - cross section of poured tunnel forms (note: pre-cast blocks will not be used.)



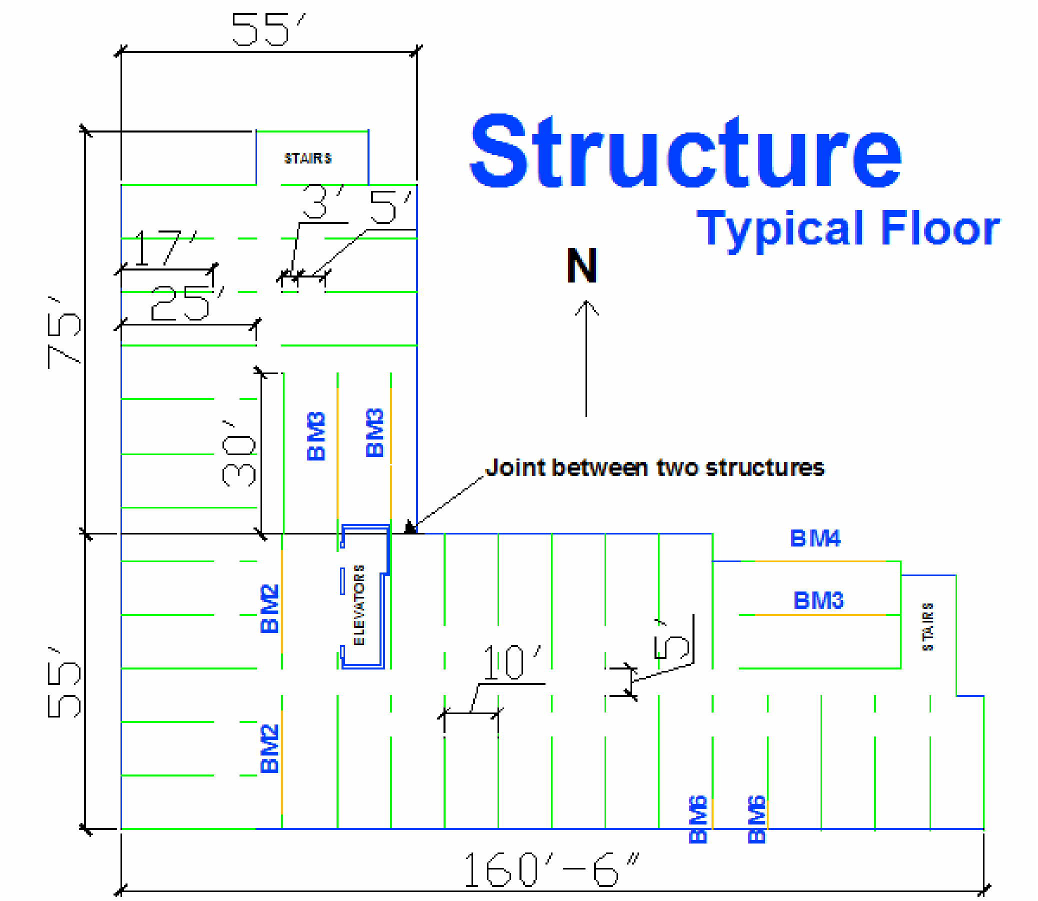
V. Proposed Alternative Structural System

The proposed structural system for the Race Street Dormitory is two independent structures of load bearing concrete walls and flat slab connected. A more strict “L” shape reminiscent of the existing building has been developed and dimensions have been increased slightly from the existing building to accommodate forms, while still within the confines of zoning requirements. The original considerations for design, such as economy and constructability were considered in the redesign as per tunnel form construction, as were minimal architectural impact and lateral load resistance. All forms are one size for ease in programming the construction process and making the system more attractive to future investment through purchase of the forms. These building changes caused spacing, floor height, building height, and wall thickness challenges.

It must be noted that the building was actually completely redesigned three times. Earlier designs are shown in the appendix. The first design was a single “L” shaped structure, but this led to lateral load resistance problems as discussed later. The second system was designed for the least interruption of the dimensions and shape of the existing building without the understanding that tunnel forms cannot be ‘applied’ to a space, instead the space is a result of the form shapes and construction technique. What initially seemed a good design turned quite ill with this mindset. Clearly this is anything but a flattery of tunnel form systems. As the second design ensued, it was apparent that shape, site, and required spatial programming restrictions were negative impacts on all aspects of the design. This not only impacted the architecture of the building, but complicated the structural system. It was a crippling realization to completely design a structure without promising speed, cost effectiveness, and constructability basically due to poor tunnel form compatibility. After further discussing designs with professional construction managers and form manufacturers in the tunnel form business, hope was renewed. A final design was attempted by designing “outside of the box,” or creating a structure outside many confines of the existing one. For this final redesign, floor to floor heights are a consistent 10’ for all floors, reducing the first to second floor height 4’ and increasing all other floor heights, except the ground to first, by 6”. The lowering of the height between floors one and two by nearly 30% was a harsh compromise to the tunnel form system in order to not increase the overall height of the building significantly. For the purposes of structural analysis, loading for gravity analysis was based on 1.2D+1.6L load factors, and loading including lateral loads was based on 0.9D+1.0E or 1.6W, and 1.2D+1.0E or 1.6W+L.0.5S.

General structural floor plans are shown in Figures 13, 14, 15

Figure 13



The structure of the typical floor plan evolved around arrangement of tunnel forms for construction. The N-S structure required longitudinal stiffness so a form layout was devised to form wall in that direction.

Figure 14

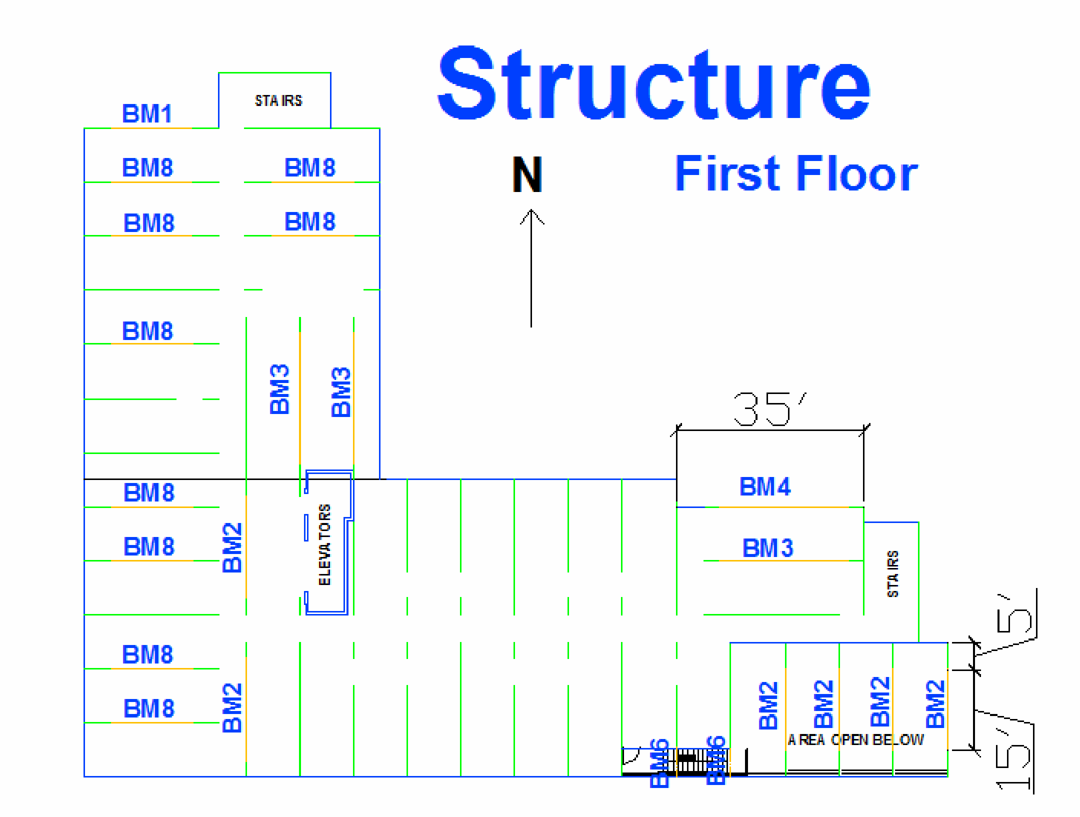
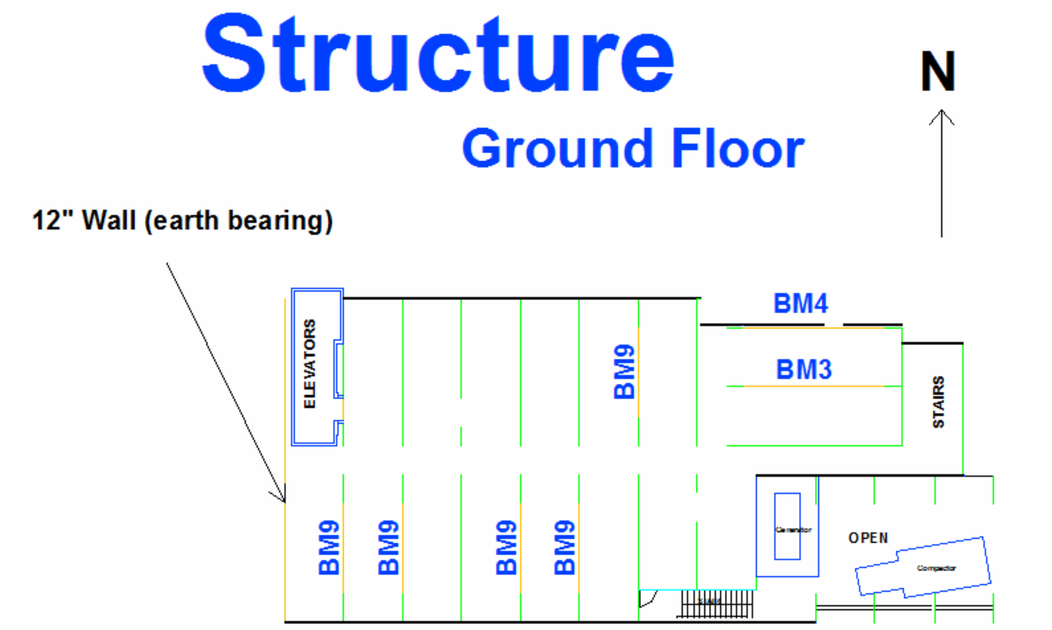


Figure 15

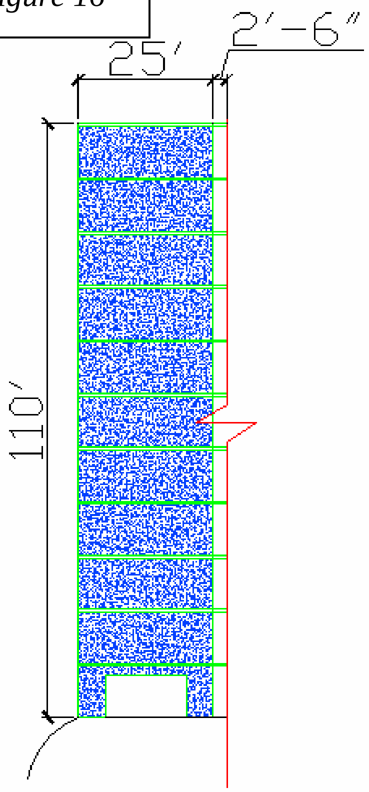


i. Wall System

The wall layout has evolved through many stages of development to finally yield the uniformity and spatial requirements shown above. All walls are 7” thick normal weight concrete. The wall spacing has been derived from the 10’ C.L. to C.L. spacing of suite partition walls in the existing building to eliminates the cost of construction of steel stud walls and is a part of the idea behind an economic tunnel form solution (and also considering cost of larger slab thickness of larger spans). Pipes and other systems that utilized these walls in the existing building can be diverted a short distance into the partition walls lining the hallway to avoid form block out. Common walls are 3’ sections in line with 17’ sections separating bedrooms in suites, and 25’ sections separating suites themselves. There are various wall cutouts shown in plan and elevation, creating a variety of cantilever beams and special reinforcing discussed in the beam section.

Compression analysis was based on a simple hand calculation of the axial force in the wall as per ACI 318 Empirical Design of walls. In order to open up spaces on the first and ground floors for communal areas, wall areas were cut out at this level at the cost of extra reinforcement and time. These sections created critical compression situations, as they are required to carry the entire mass of the wall above (figure 16- full wall, figure 17, partial wall) as well as the loads on the slab within the wall section’s tributary width at each floor above. These wall sections were designed as columns using PCA column and were necessarily 5’ long and could only be a maximum height of 15’. Those 10’ high sections (figures 16 and 17) could be shortened to 4’ long, but this would require a deeper beam closing the space. The tension steel for bending under lateral loads also increases this reinforcement area and is discussed later. A further design challenge was the opening for the trash compactor access. The vertical open space in the existing building is clear from the ground to second floor, a clearance of 24’. This height dropped to 20’ in the redesign, but, as noted before, with slenderness effects the 7” wall section must be 5’ long and only up to 15’ high (figure 18). Walls were also checked for weak axis bending. After uneven live loading and moment distribution, a 7.7 in-k/ft moment is induced at the top of walls in a in the critical section of a typical bay (see calculation tables). With minimum vertical reinforcement for shear wall action, the moment capacity is 48 in-k/ft, a well sufficient capacity.

Figure 16



ii. Slab System

The slab transfers a live load, superimposed dead load, and the slab self weight to walls below based on its effective tributary area. The structural floor of the dormitory is flat reinforced concrete slab 6” thick. The nature of the wall layout in the dormitory makes analyzing the floor slab particularly complex. The slab thickness was initially determined based on a deflection limitation of Span/24 (ACI 9.5.2) for a typical one-way span with the exterior end pinned (120” typically longest span/24 = 5”). Deflection results from an assumed two-way action in the corridor based on ADOSS calculations required a slab thickness of 5.5”. A 5.5” slab was insufficient, however, to limit deflections of the 5’ cantilever section by the isolation joint, a 6 in slab is required. ADOSS reinforcement is shown in figure 19. Live loading and superimposed dead loading was used for calculations based on Table 1.

Figure 17

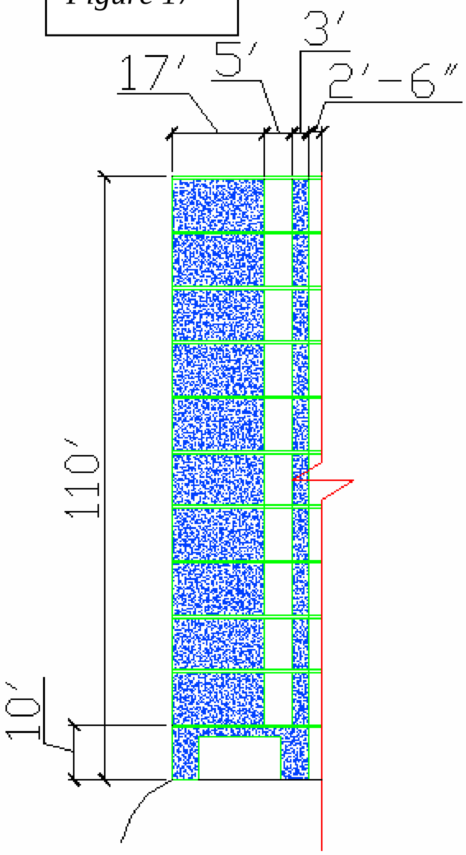
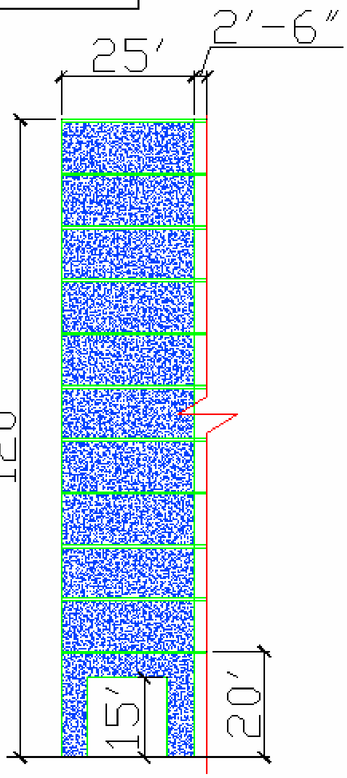


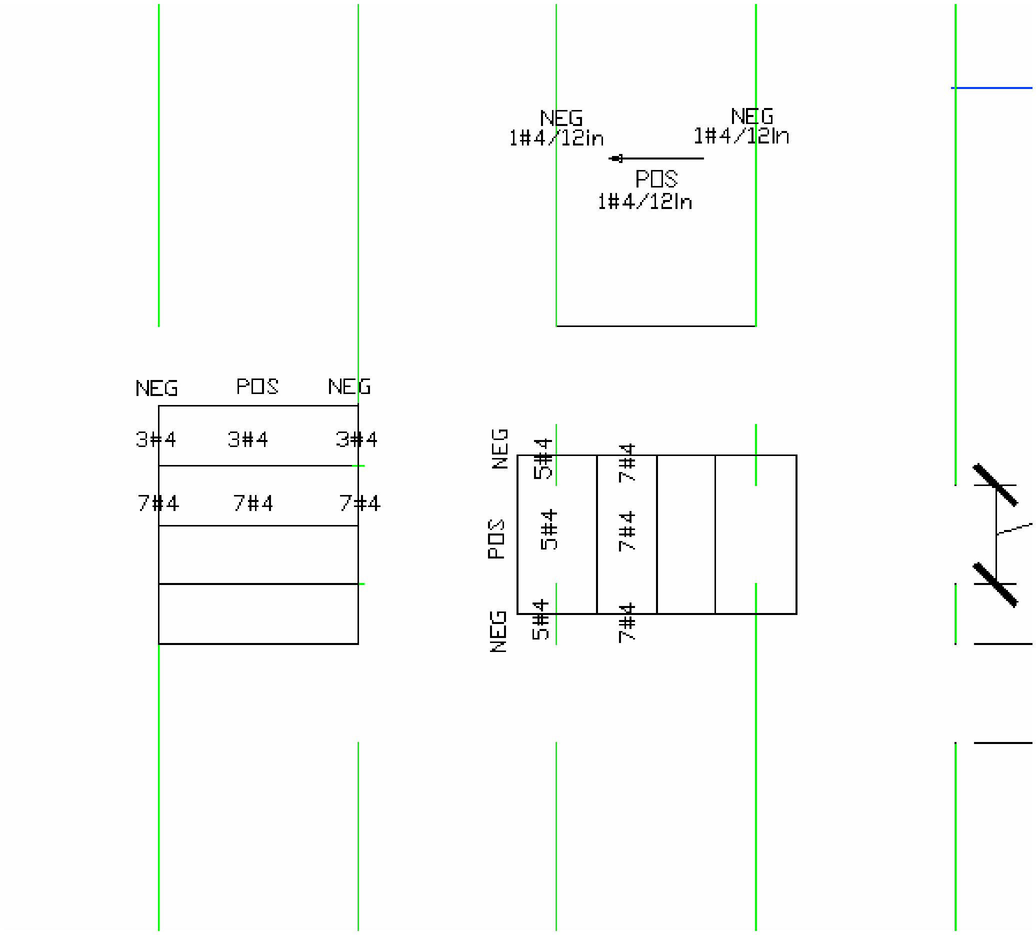
Figure 18



The majority of floor spans are 10’ one-way load distribution and were initially analyzed by hand using the moment coefficients for worst case loading (ACI 8.3.3). ADOSS calculations were based on assumed simplified two-way and one-way action of the slab and slab beams in various regions of the floor system. The one way results were checked by hand. A RAM Concept Model was made punching shear failure was not found to control.

Table 1 - Loading	**Existing Design**	**Thesis Design (IBC 2003)**
Service Level Live Loads (psf)		
All floors	40 psf	40 psf
Roof	20 psf	20 psf
Dead Loads (psf)		
Partitions	15 psf	15 psf
Curtin Wall	Not noted	120 pLf
Concrete Slab Weight	Not noted	12.5 lb/ in depth

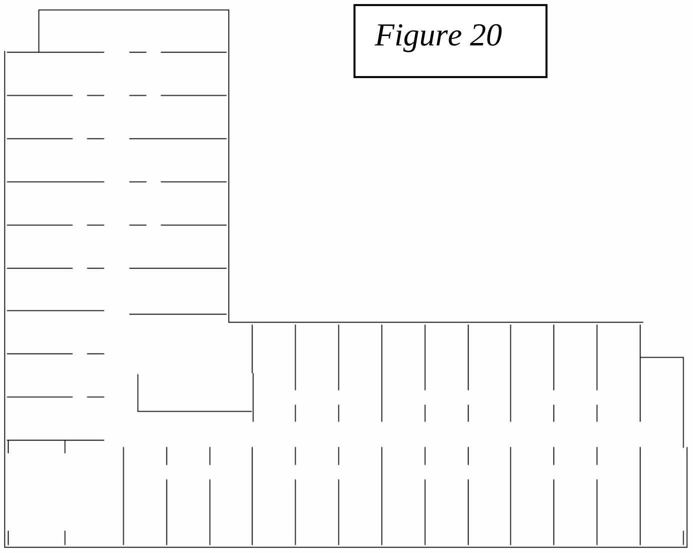
Figure 19- ADOSS calculations for slab reinforcement



iii. Beams Supports

Beams have been designed to carry load between walls were the slab is of insufficient depth. Simple hand calculations were used based on general equations from the AISC Manual of Steel Construction, Third Edition, for fixed beams under their respective loadings. Transfer beams have been designed to transfer wall load over cutouts in the walls. Where a wall is cut out into a 3' section on either end, the bottom of the beam runs flush with bottom of the slab, keeping it from imposing on the area below and requiring extra detailing. Beams carrying walls were reinforced to accommodate concentrated shear forces induced in their interface with the smaller area cross section of the walls below them. Simple flexural calculations are shown in tables the appendix corresponding to the beam labels in the structural plans, figures 13 through 15.

VI. Lateral Loads



Original Typical Floor Structure (no Joint)

Figure 20

Tunnel formed walls and slab poured together inherently creates a very stiff structure. The strong fixity of the two elements requires extra detailing to prevent cracking under flexure and shear. All walls in the structure inherently resist both loadings because of their strong tie to the structure, and therefore must each be designed as shear walls. At first, it seemed only natural that in an “L” shaped building with walls poured at perpendicular to the longitudinal axis of each wing would have lateral resistance in both orthogonal directions with some extra torsional shear force. Because of this, the first building design was a single “L shaped structure. It soon became apparent, however, that it was nearly impossible to match the longitudinal shear wall stiffness in a given building wing with the combined stiffness of the many parallel walls in the other wing. With the original tunnel form layout as shown in figure 17, there was little or no longitudinal stiffness along each wing except at the connection between the two. Preliminary stiffness analysis confirmed that the two wings would collide or tear transversely or longitudinally apart from each other as they drift and vibrate differently under lateral loading. To account for this, the buildings were designed as two independent structures separated by a simple isolation joint at the base of the shorter leg (as shown in figures). Aptly named the “N-S Structure” and “E-W Structure”, their titles describe the major axis (longitudinal direction) of each structure. Clearly, with this separation, there is no wall running along the length of the N-S structure for longitudinal stiffness in this design. As shown in the appendix, the second design attempted to add a wall along that direction along the N-S, formed outside the tunnel form system. This is part of what prompted a rethinking of the entire structural layout.

i. Wind Analysis

Wind loadings on the building were determined in accordance with ASCE 7-05 and their factored loads (1.6W) were found to control over seismic loads in both directions of the N-S structure and in the North-South direction of the E-W structure. Overall coefficients of 1.8 were used to incorporate pressure and suction based on the principles of the earlier technical reports. Wind analysis (controlling values) can be found in the appendix.

ii. Seismic Analysis

Seismic loads were determined by hand in accordance with ASCE 7-05 using the Equivalent Lateral Force Method due to Seismic Design Category B requiring no further analysis. A factor of 1.7 was used to find the period of the structure in order to further calculate Cs. Because of the high stiffness of this structure, a 3D computer model would have been a more accurate method of calculating the period of the structure, but was not performed due to ETABs Nonlinear v.8 model problems. Seismic analysis (controlling values) can be found in the appendix.

iii. Analysis Strategy and Results

Due to the fixity of this wall-slab structural system and hence its tendency to crack, a full building computer analysis would be an effective addition to design in order to not only determine the true action of the slab, but also confirm proper reinforcement detailing to prevent cracking. A computer model was not performed due to design changes and time constraints. For the purpose of education, the following procedures were taken to adequately design for and address lateral loading in the force resisting elements. For purposes of analysis, concrete slab flooring was assumed to act as a rigid diaphragm and carry lateral loads to wall elements based on their relative rigidity to each other through deep beam action. Most importantly, all walls were reinforced as shear walls as per ACI 318 in order to make them compatible with deformations of the structure as a whole. SAP2000 v9 was used to find the stiffness of walls based on the equality: [Stiffness] = [1 kip lateral load at the top] / [Drift]. Shear load to each wall (percentage of base shear based on relative stiffness and the maximum of 1.0E or 1.6W) was compared to nominal shear capacity of the concrete section and horizontal steel was added in the cases where additional capacity was required. Minimum horizontal and vertical reinforcing (reinforcement ratio of 0.0025 and maximum spacing of 18") was used where shear values were within their limits. Walls with an extreme fiber tensile stress under gravity load and lateral load overturning moment (load combinations 0.9D+1.0E or 1.6W, and 1.2D+1.0E or 1.6W+L.0.5S) were designed with extra longitudinal steel at the end of the wall as required. Boundary elements, or extra reinforcement and ties at the ends of the walls, were determined based on the maximum compressive stress in the extreme fiber of the wall and the minimum depth of the neutral axis under gravity and lateral load combinations.

- A typical shear wall calculation is shown in the appendix.
- Relative stiffness evaluation of walls, labeled based on a "wall line" described in the table and corresponding figure in the appendix.

Torsion was originally considered before the building was broken into two structures using Microsoft Excel spreadsheets two find the center of mass, center of rigidity and eccentricity of walls, and subsequent torsional shear in each wall. The relatively low shear values in that case (total torsion induced in walls around 2 kips), the general symmetry of walls under a two structure system, make torsion in this case negligible and was assumed to be zero.

iv. Drift

Drift has been calculated based on the overall stiffness of the structures in either orthogonal direction. The maximum drift based on H/400 is 3.6” for the E-W structure and 3.3” for the N-S structure. Critical drift is that of the combination of the drift of each structure in the N-S direction because opposing drifts could cause the buildings to crash into one another. The resulting drift in this N-S direction from controlling design wind loads was calculated at 1.42” for the N-S structure and 0.90” for the E-W structure, for a combined “worse case scenario” of 1.9”. Wind does control, and typically gusts would not come from both directions at the same time, but in order to separate them safely a 2” isolation joint was designed between the structures. The drift in the N-S structure was actually designed to be this low in order to accommodate as small a joint as possible. This joint, however, does cut directly through a room and cantilever the floor out from the structure. A possible solution to this joint interrupting the space is a metal angle that covers the top of it while not connecting the two buildings.

VII. Note on Foundation

The foundation for the existing dormitory is a series of drilled piers supporting column loads directly, as well as grade beams. The drilled piers are very deep suggesting pour soil quality if not simply very high axial loads. The nature of this redesign is very different. Loads reach the ground spread out along walls. The walls tend to be spaced 10’, which is narrow compared to the column spacing of 22’-30’, which also means better dispersion of the load to the ground. Tunnel form walls are usually built up from a flat slab and rely on load spreading rather than increased depth for support. The foundation of the dormitory was not considered part of this redesign, but it could be found quite simply because it uses a much larger load spreading system to a shallower support surface. Overturning would also have to be considered because of high bending moments induced by lateral loads.

VIII. Architecture Breadth Study

The architecture breadth study was an attempt to accommodate the existing architecture to the new structure as well as the new structure to the architecture, while maintaining the same general framework as the existing building. Clearly important requirements such as offices and RD and RA suites and most of the spaces on the first floor were carefully fit into the new structure.

Figure 21 – Existing Building Typical Suite Unit

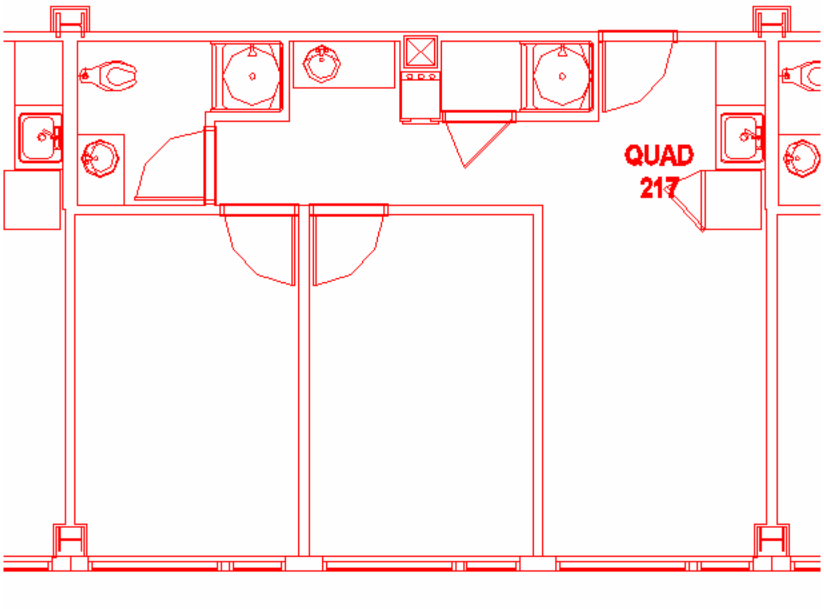


Figure 21

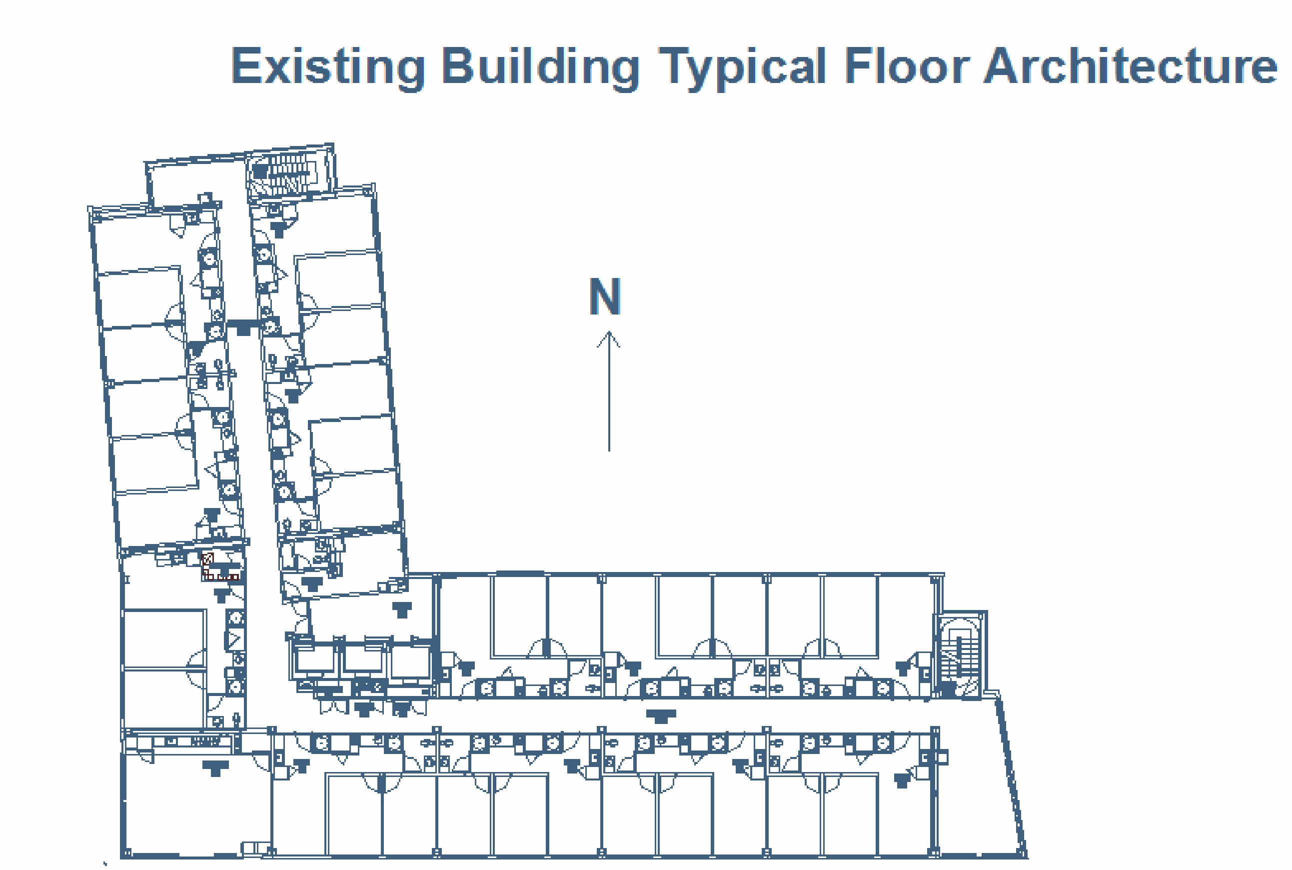


Figure 22– Redesign Typical Suite Unit

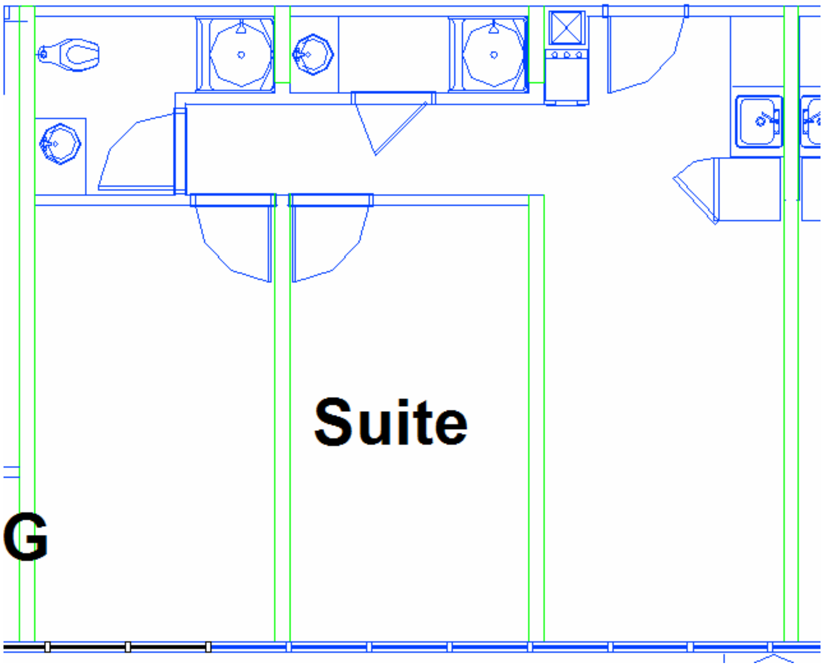
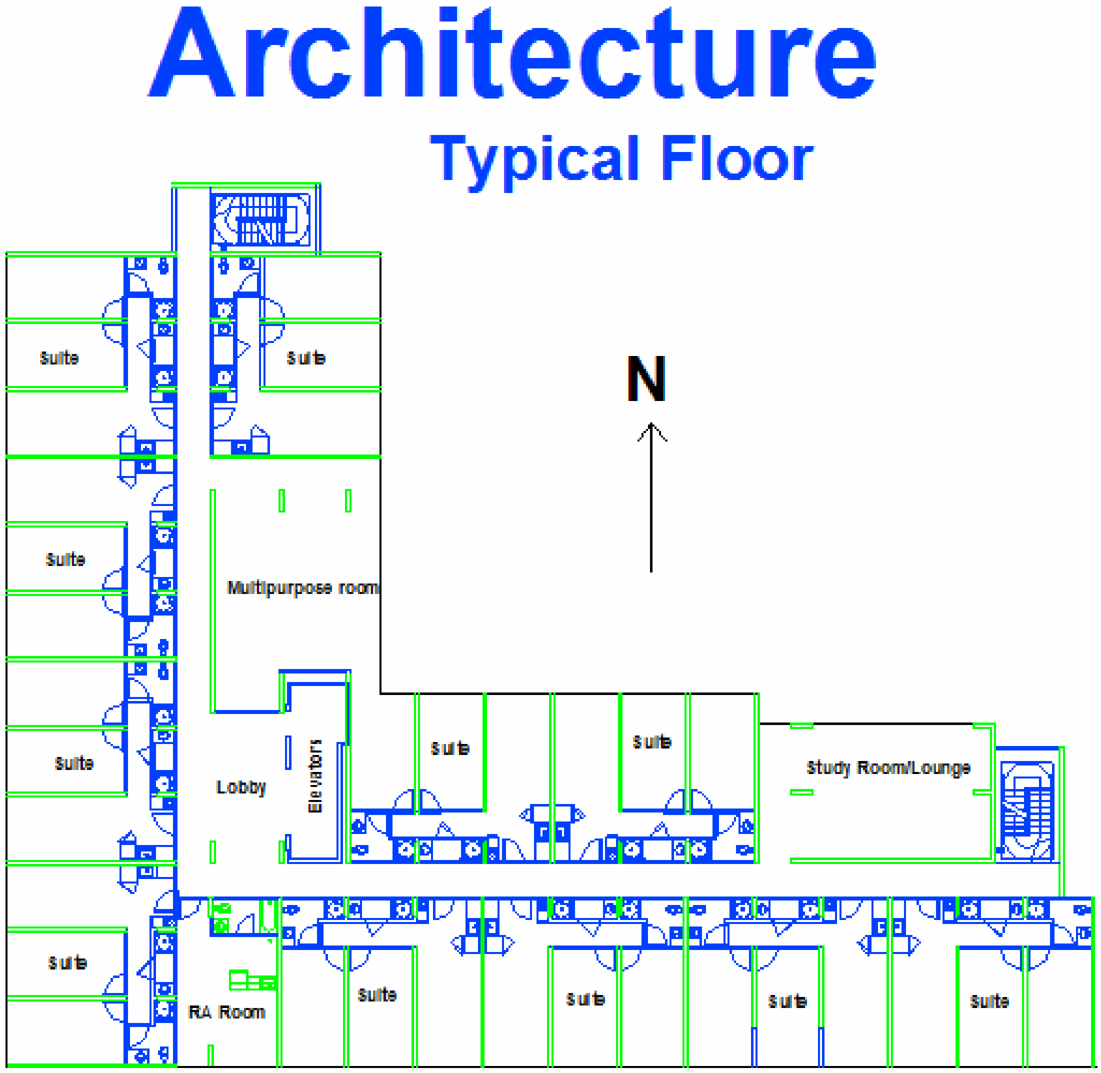


Figure 23– Redesign Typical Floor Architecture



Typical floors were not designed with the same number of suites as the existing building. Each floor now has one less suite per floor because of the awkward space created by longitudinal form layouts. This does still meet the requirements of Drexel, which was a minimum of 11 suites per floor, but it is a net loss of 8 suites and 32 students giving income to the university. The multipurpose room could be converted to an awkwardly organized suite or double unit at the discretion of the University.

Figure 24 – Existing Building Typical Floor Elevator/RA Unit Area

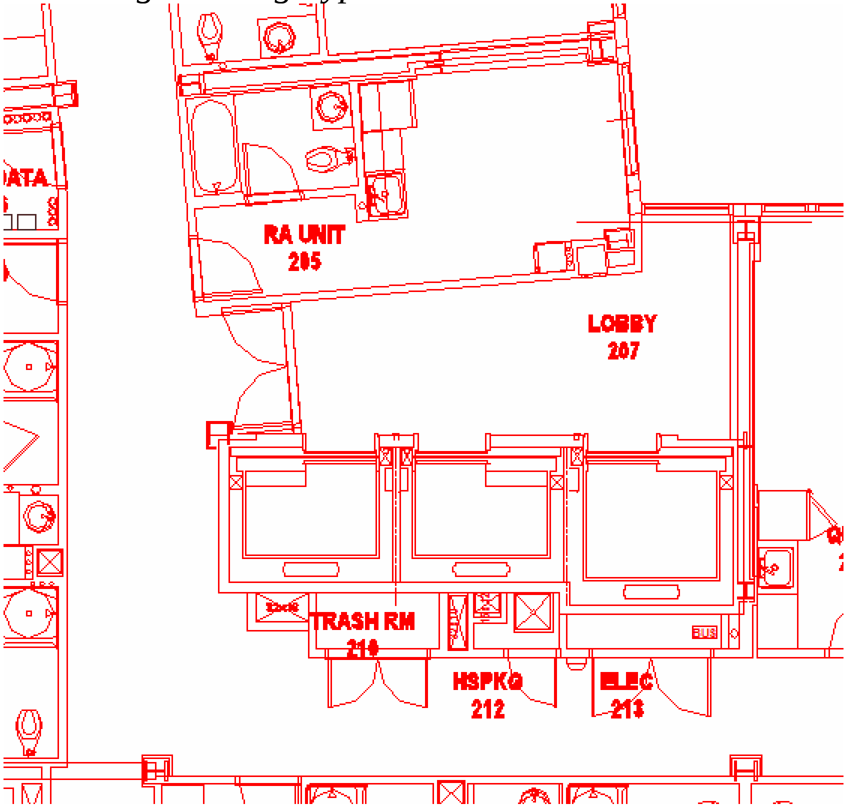


Figure 25– Redesign Typical Floor Elevator/RA Unit Area

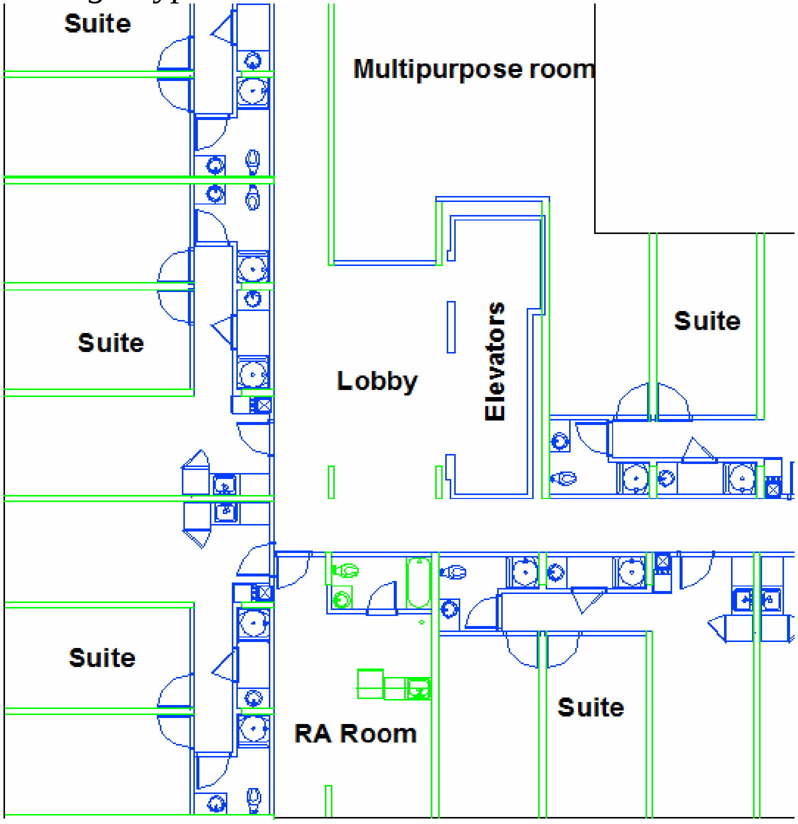


Figure 26

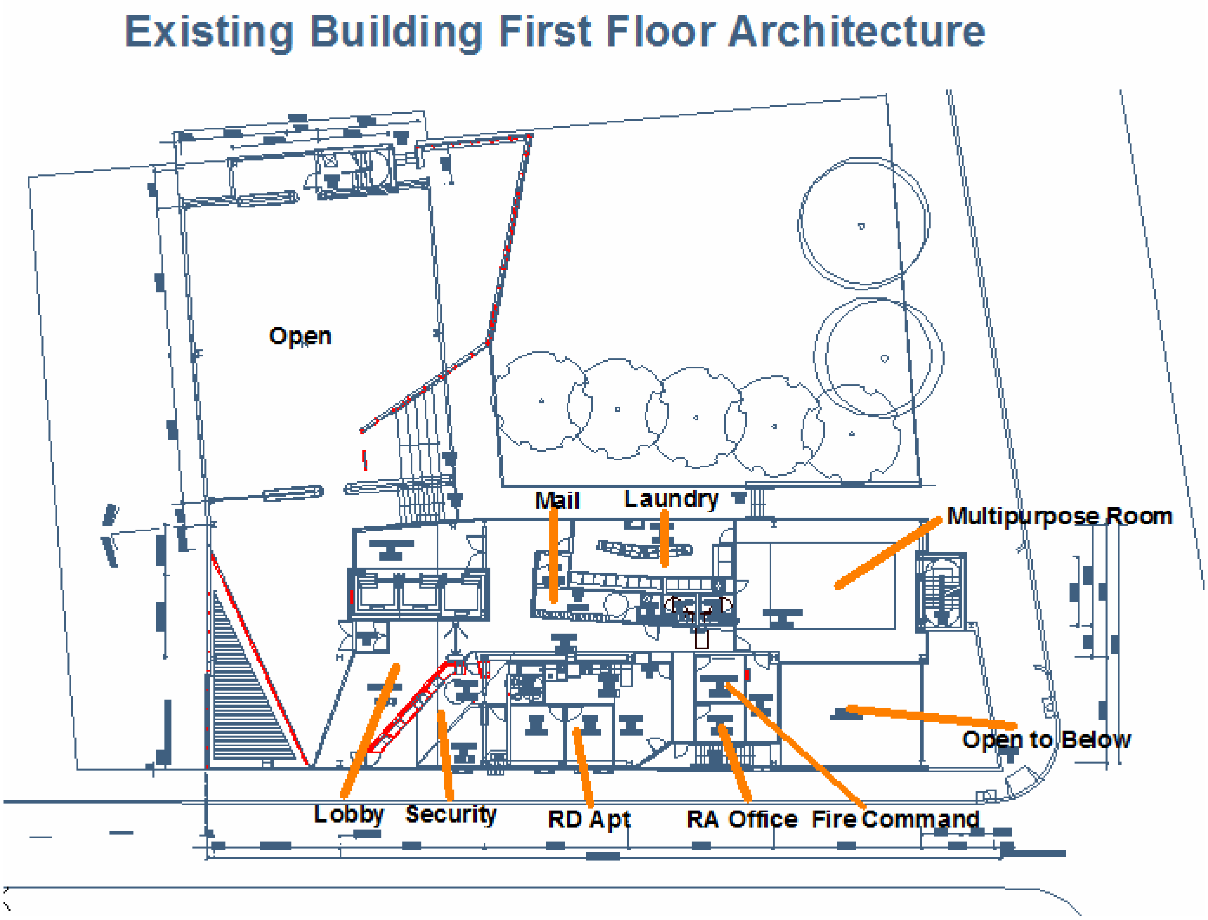
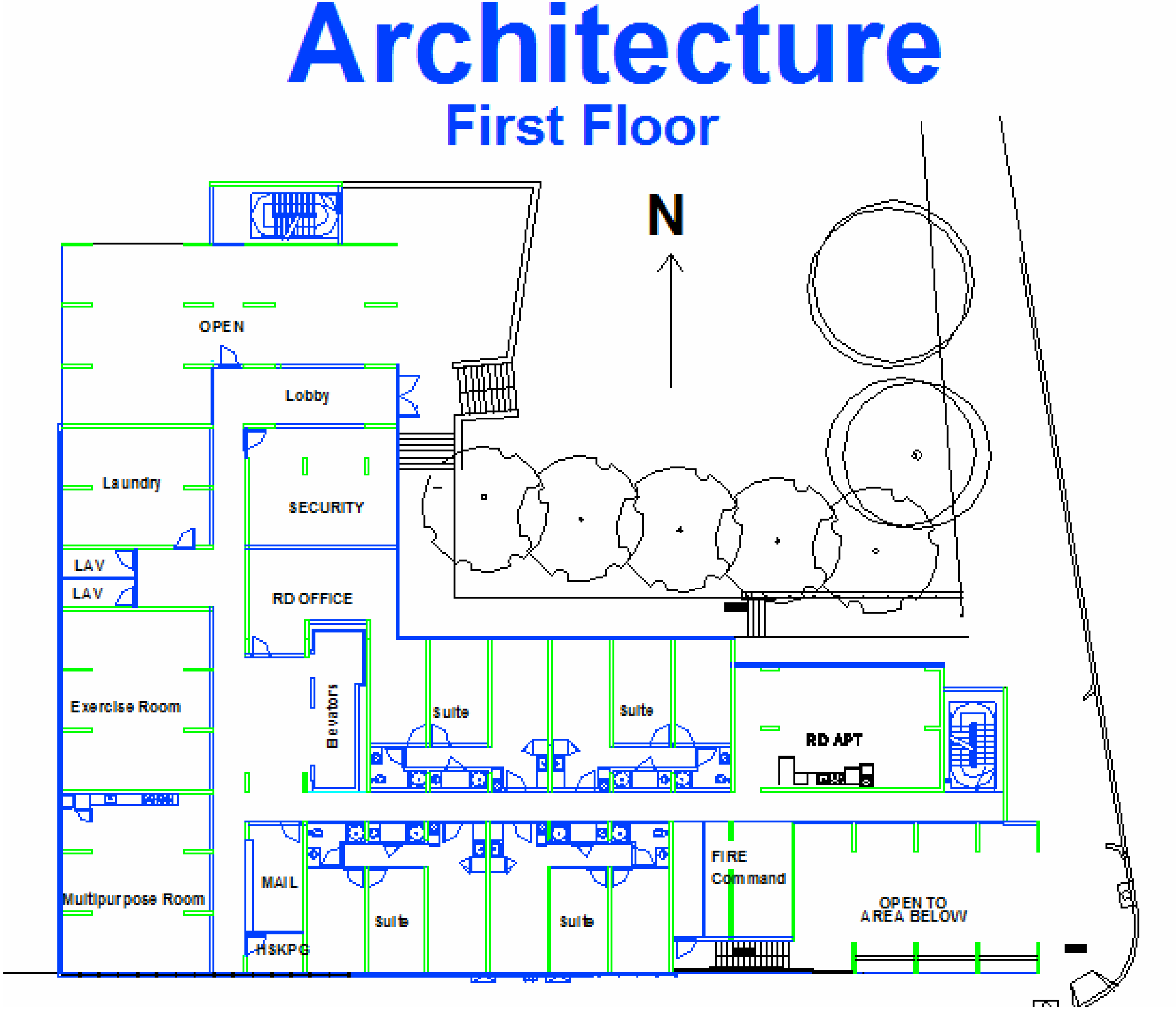

Existing Building First Floor Architecture
Open
Mail
Laundry
Multipurpose Room
Open to Below
Lobby
Security
RD Apt
RA Office
Fire Command

Figure 27



The first floor offered many challenges in rearrangement in order to fit each of the spaces from the existing building into a roughly equally sized space in the new building. Four more suite units and the exercise room were added to this floor, increasing the capacity of the building to 12 more students. The wall cutout supports, although 5' long and protruding into spaces will probably not be large problem in multipurpose, exercise, and laundry rooms where they are most significant. The drop in floor to floor height, however, destroys the character of the space, as one would imagine the low head room under a significant number of beams in larger spaces would feel ominous, one might feel claustrophobic. Because of these poor aesthetics and the extra cost of blocking out walls on the first floor into 5' lengths spanned by 15' beams up to 27" deep (less than 8' clearance), the first floor interior area has been extended into the open area patio.

Figure 28 – Existing Building Ground Floor Plan

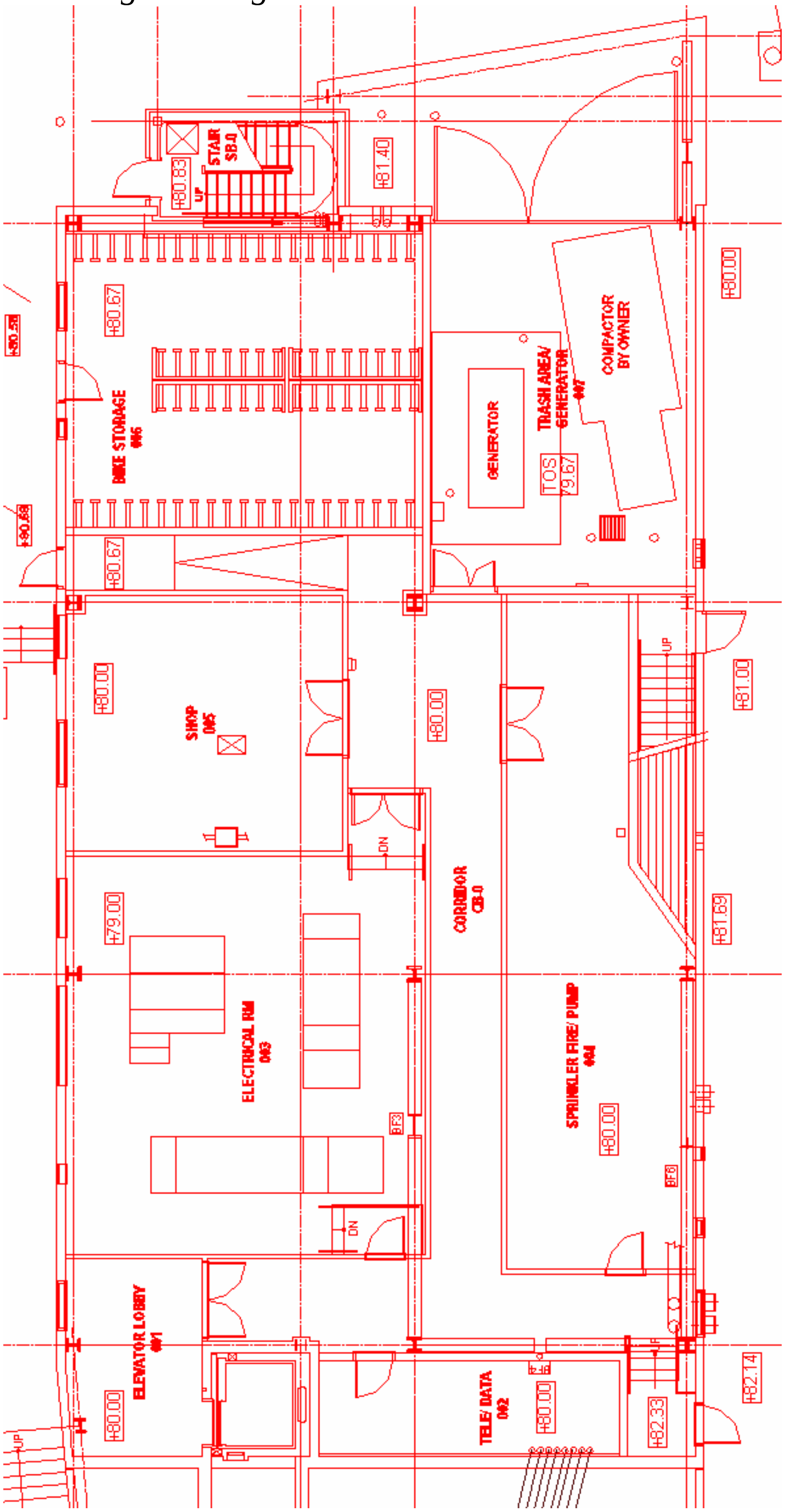
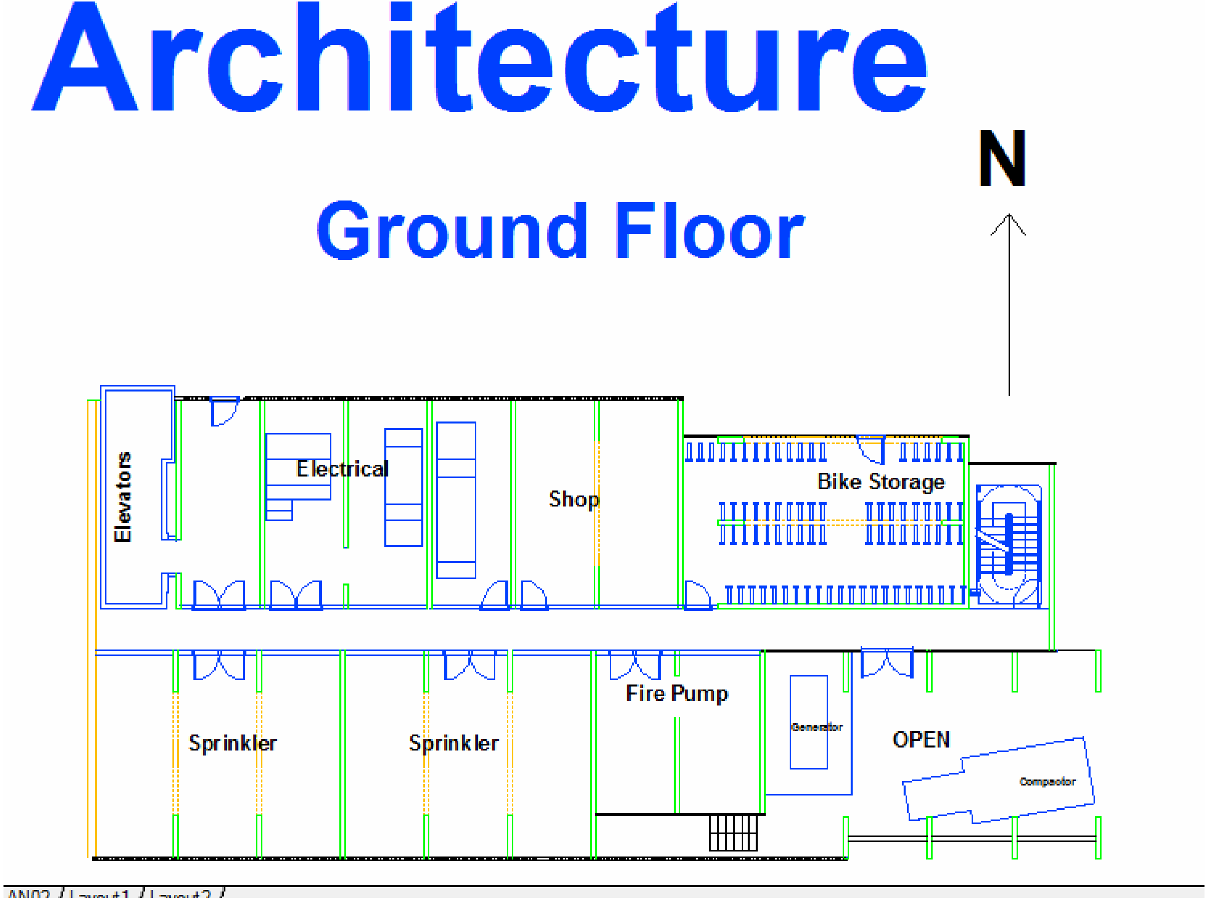


Figure 29

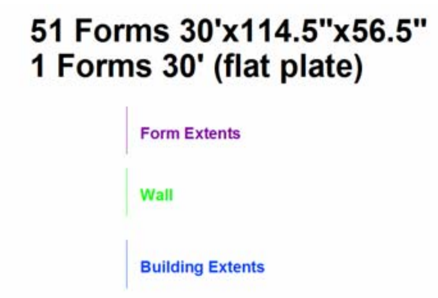


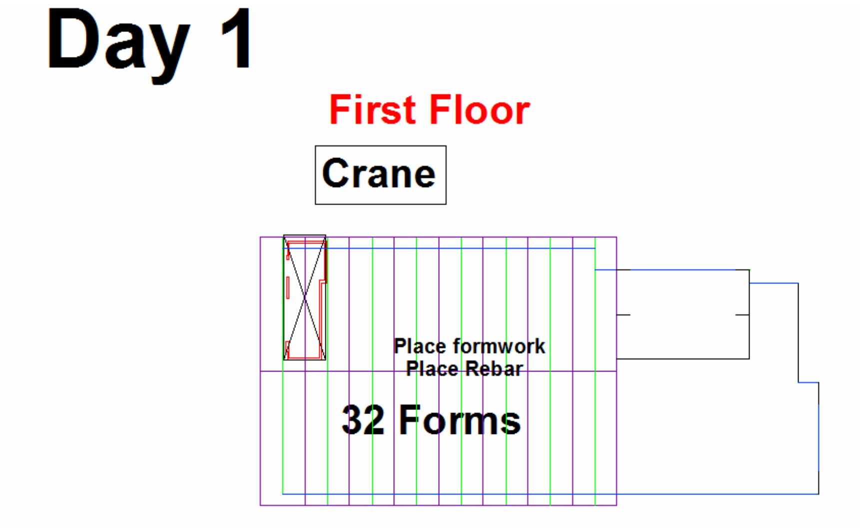
The ground floor was rearranged to fit around support walls with as few transfer beams as possible, although several were required. This arrangement is quite simple as the spaces are not particularly important architecturally. A strong effort was made to keep outdoor trash area accessible even with the thin wall supports as discussed before and shown in figure 18. 5' by 7" wall sections were able to create a 15' by 15' access opening. (Note that the pre-cast concrete panels of the existing building's envelope at the ground floor now give lateral bracing to the bottom 10' of these walls).

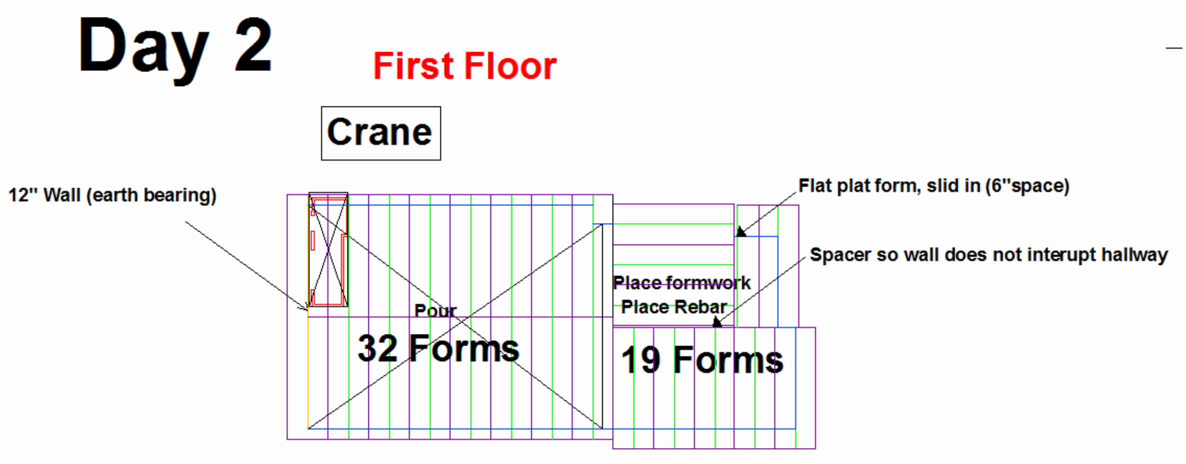
In general, the typical floor is more comfortable than that in the existing building- it is more spacious and has a 6" higher ceiling- but that comes with a cost. The first floor really shows how tunnel forms do not react well to a soft story because of the need to carry large wall loads on slender beams and end walls.

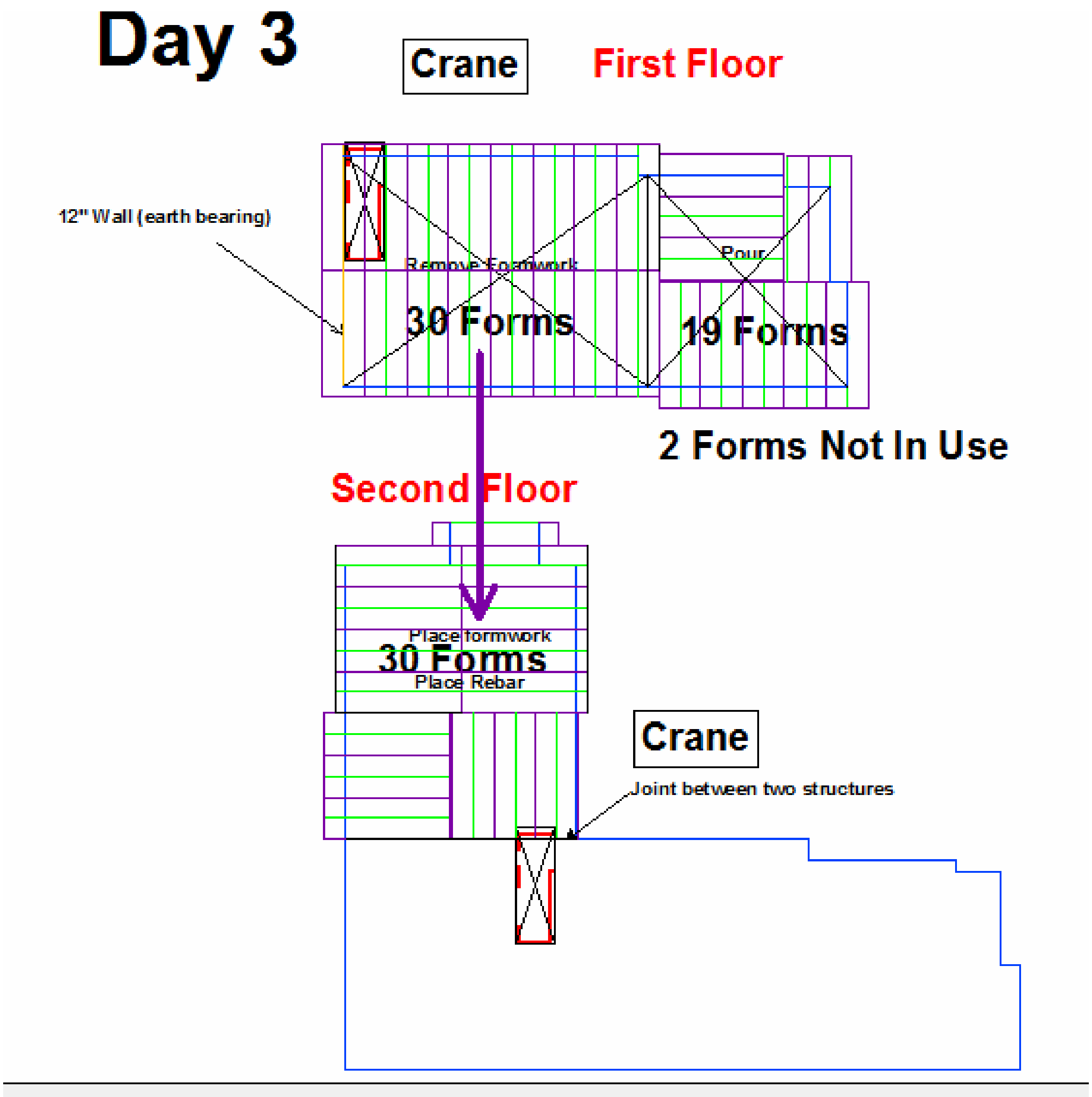
IX. Construction Breadth Study

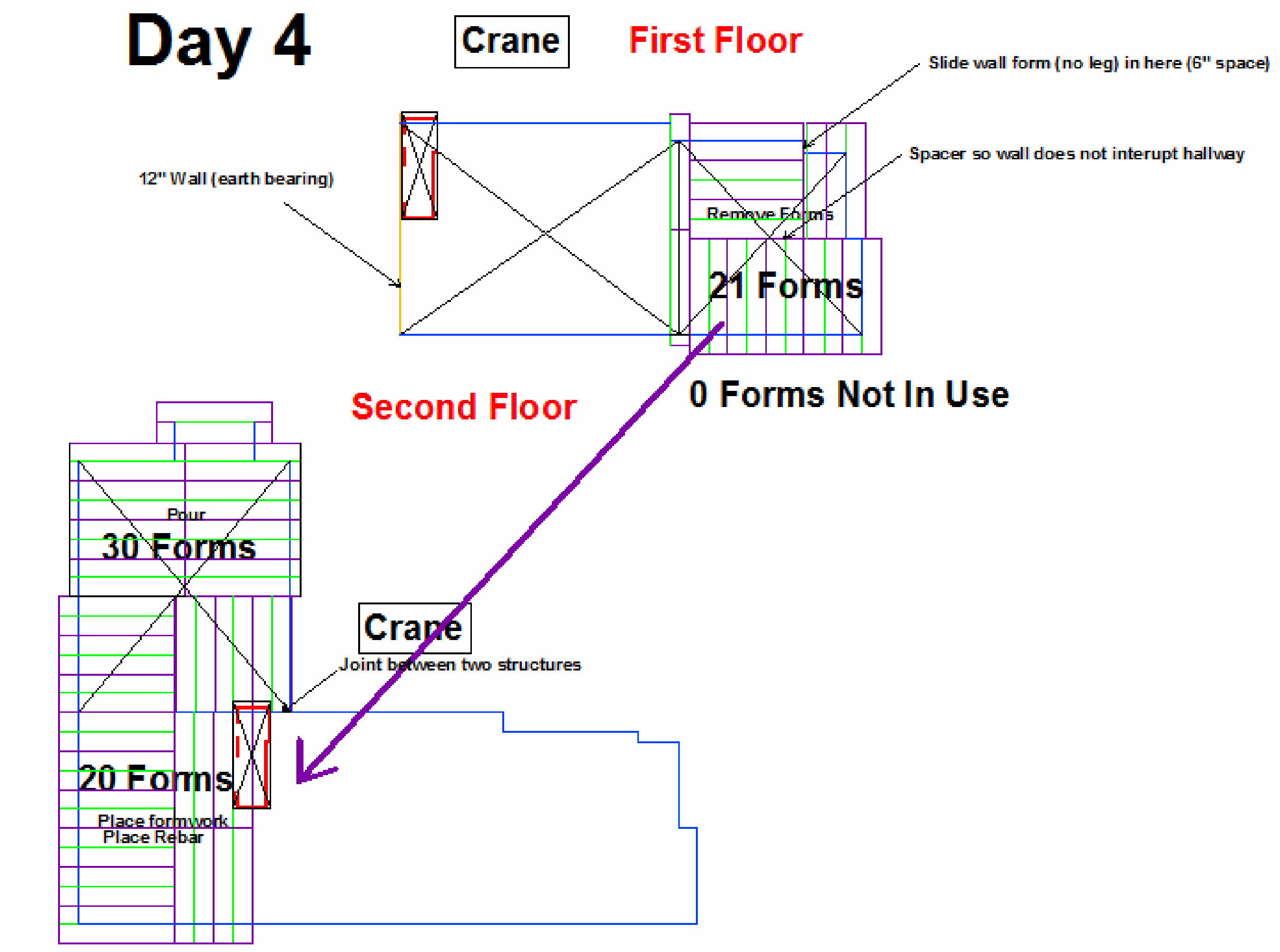
The proposed building will be built in stages lasting a total of in 65 days including weekends. The first floor is poured in 3 days and consecutive floors are poured in 4. Careful management and fitting of forms was considered based for optimal flow of formwork. Essentially there are two elements- each day formwork is removed from a section cast the day before and placed in the next section where rebar is installed. Also each day, another section is being poured; creating a daily cycle of formwork being removed, installed, and poured. The sequence is shown in the following figures from day one, where blue lines represent the building extents, green lines the walls being poured (not including block outs) and the purple lines represent the edges of formwork not touching the walls. The forms are intentionally slightly longer than they need to be in most cases in order to allow workmen to work around their edge. Particularly challenging was to laying out the forms in reasonable sections for daily work. The end section poured on days 3 and 7 was also particularly challenging to layout because of the variation in wall direction. A flat plate form is a form of required length and height but no angle for slab support. Often they are used on the outside of edge walls, but angled forms will be used here to support workers walking. A flat plate form is required to fit into an area in the aforementioned section in order to form a tight wall. The wall was jutted out 6” to accommodate the form system. All forms have 3/16” thick steel skin with bracing and 10” diameter wheels for easy movement into place. Note the pour zone always ends at the quarter point between bays for reinforcement. The form that only halfway covered for this reason stays behind as the next section is poured.

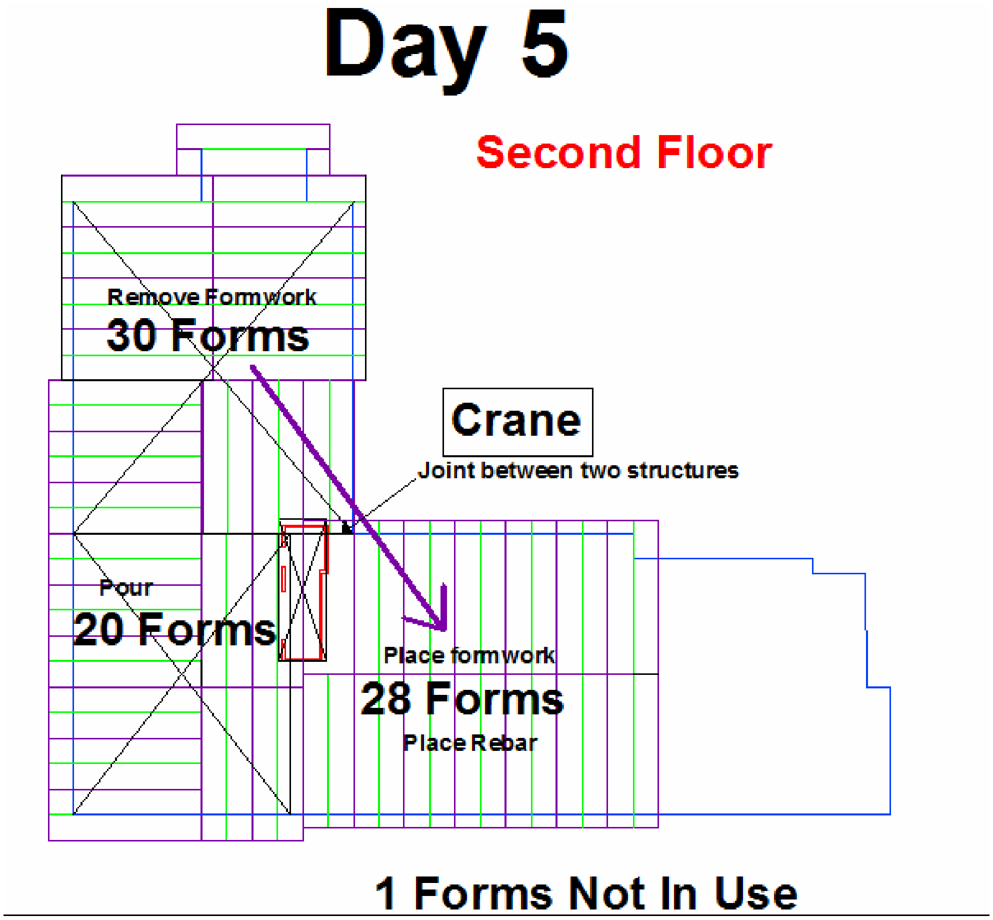


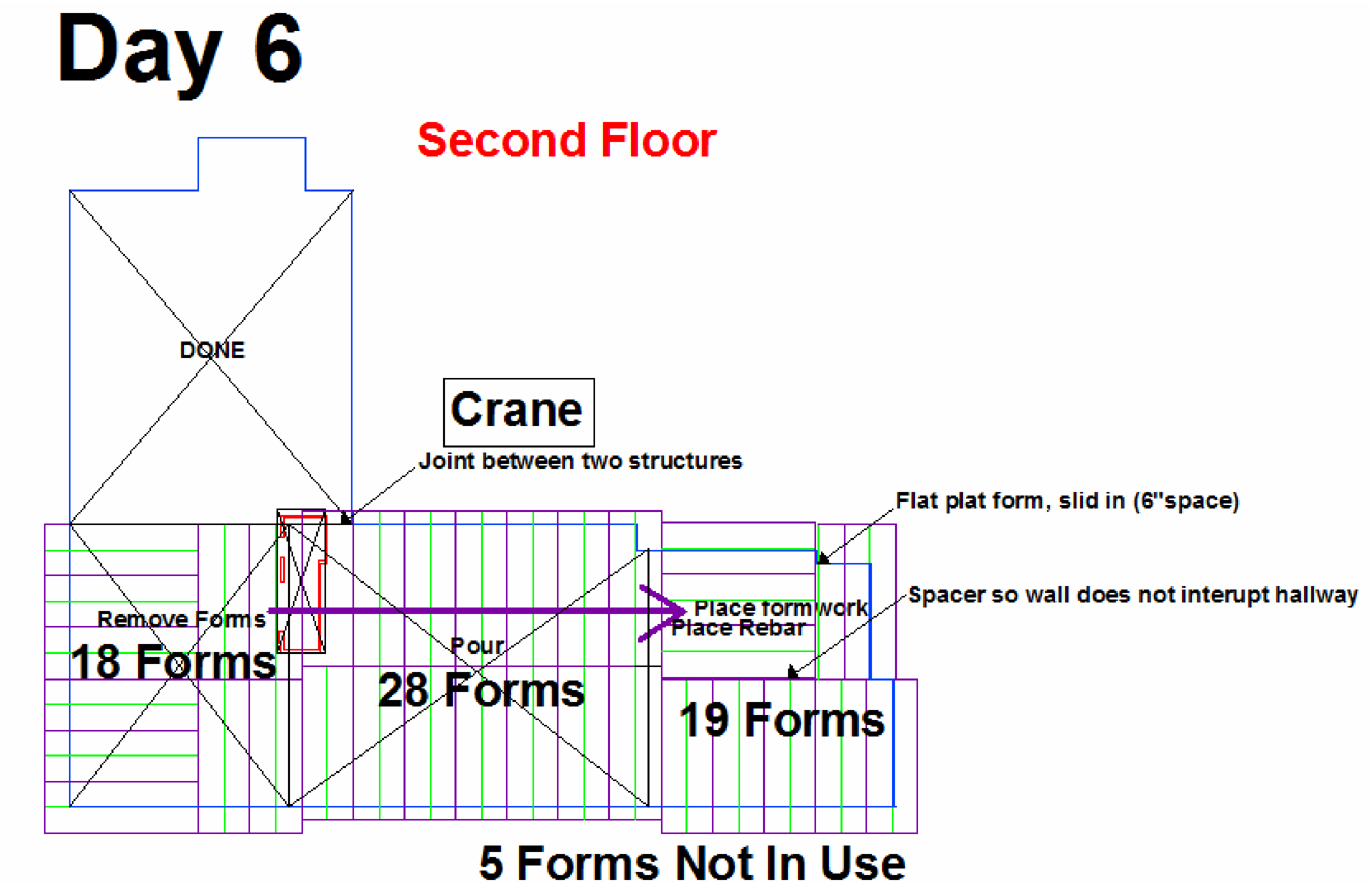

Day 1
First Floor
Crane
Place formwork
Place Rebar
32 Forms


Day 2
First Floor
Crane
12" Wall (earth bearing)
Flat plat form, slid in (6"space)
Spacer so wall does not interupt hallway
Pour
32 Forms
Place formwork
Place Rebar
19 Forms

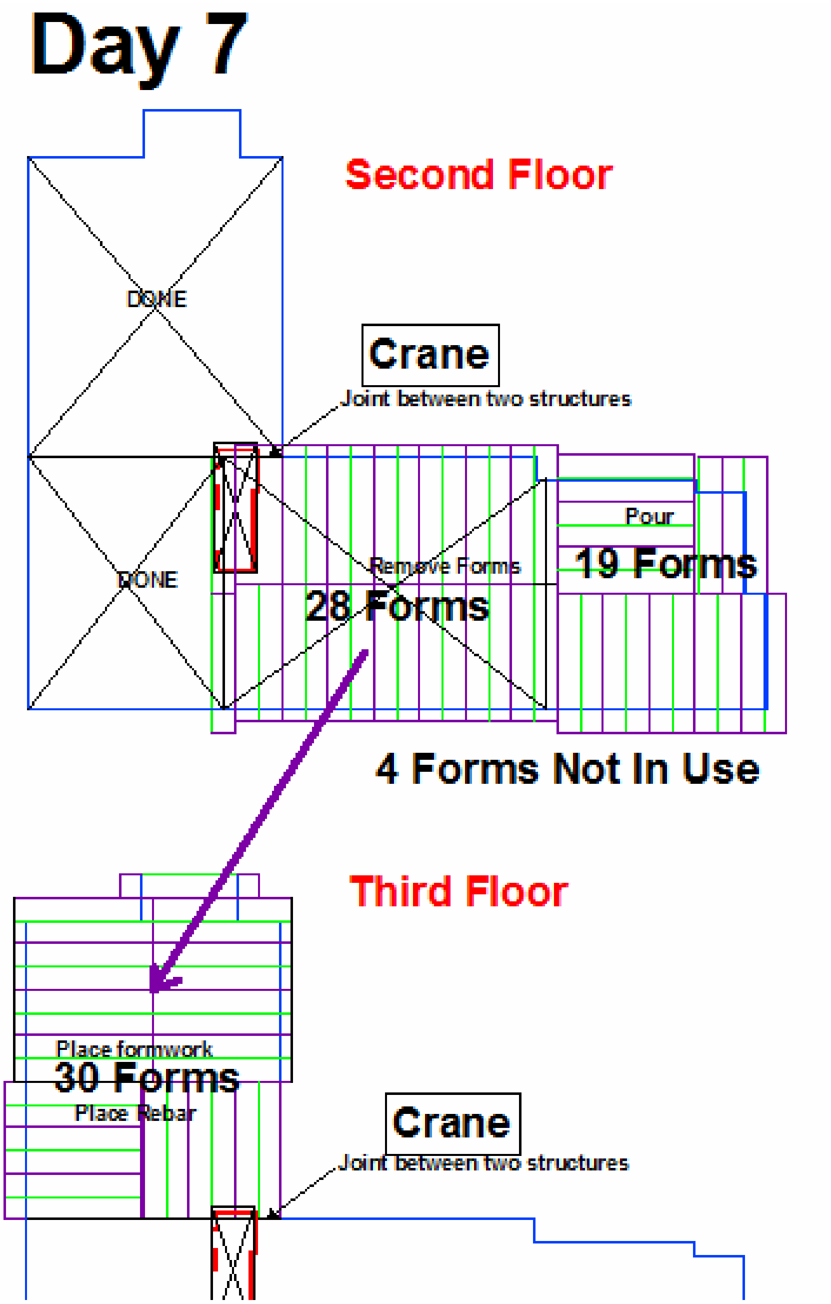

Day 3
Crane
First Floor
12" Wall (earth bearing)
Remove Formwork
30 Forms
Pour
19 Forms
2 Forms Not In Use
Second Floor
Place formwork
30 Forms
Place Rebar
Crane
Joint between two structures


Day 4
Crane
First Floor
Slide wall form (no leg) in here (6" space)
Spacer so wall does not interupt hallway
12" Wall (earth bearing)
Remove Forms
21 Forms
0 Forms Not In Use
Second Floor
Pour
30 Forms
Crane
Joint between two structures
20 Forms
Place formwork
Place Rebar


Day 5
Second Floor
Remove Formwork
30 Forms
Crane
Joint between two structures
Pour
20 Forms
Place formwork
28 Forms
Place Rebar
1 Forms Not In Use



Day 6
Second Floor
DONE
Crane
Joint between two structures
Flat plat form, slid in (6"space)
Spacer so wall does not interupt hallway
Remove Forms
18 Forms
Pour
28 Forms
Place form work
Place Rebar
19 Forms
5 Forms Not In Use



The existing structure of steel and plank was built and topped in 88 days and cost $3,554,000.00. This is a significant increase in time, a full 24 days. Furthermore, another 25 days total was required to partition the entire building. The redesign has an estimated 2/3 of the partitions in the existing building and therefore cuts this time down to 17 days. According to RS Means 2005, and some information received from professionals, the cost for the structural system is a mere $1.5 million based on a crew of 14, a crane, and $773,000 for form rental. Based on ballpark figures given on a leading tunnel form contractor’s website, values should be between $18 and $22 per square foot, or $2.52 million and $3.08 million, so this value is probably wrong. The maximum estimated value of $3.08 million is still %13 less than the cost of the steel and plank building.

X. Advantages, Disadvantages and Conclusion

In the end, the goals of designing the Race Street dormitory did work out fine in the end, with the exception of the first floor headroom. In initial design, the tunnel form system seemed to make a structural design more complex because longitudinal stiffness was at a premium, and spatial restrictions were created everywhere. The system clearly has many advantages for some applications, those which are impersonal block style rooms such as hotels (especially with its strong fire rating lowering insurance rates). At its cheapest, of course, it is also the most boring. Architectural variations and style are not very compatible with tunnel forms, as from the beginning of this design the building needed to be cut into a sharp right angle from the free, expressive angle of the existing building. The system was constantly controlling liberal division of space and was not very forgiving. The flair of the exiting dormitory, with its "suite" rooms, stylish façade, angles, and varying story heights can not be restructured with tunnel forms. Through the early designs, their failure, and the final design, it seems an entirely new building must be designed and spatial interests forced into it. The system, however, does have the advantages of potential low cost and a very high rate of construction. The spaces required for the building, however, were not particularly easy to design for with a tunnel form system. A transfer slab would definitely be something to explore further to make this design more attractive and viable. Overall, redesign of structure, architecture, and construction facets oft the Race Street Dormitory has shown that the tunnel form system is a fast way to design structures and is structurally simple if designed economically, but that it is not particularly feasible for this building, unless Drexel is willing to give up a lot for speed and cost.

XI. References

1) Tower, Douglas (2006), *'Drexel University Race Street Dormitory,'*
Technical Reports 1-3, Thesis Proposal.

2) Drexel University (2005), *'Project Summary and Scope Components'*
RFP. June 6

3) Erdy McHenry Architecture (2006), *'Drexel University New Residence Hall,'*
presentation by Chris Boskey, Nov. 29

4) Highrise Concrete Systems, Inc.
http://www.highriseconcrete.com/apts.htm)

5) Dave Gust, General Superintendent, Centex Construction, FLA

6) Tom Belace, Alliance Structural Engineers, Houston, TX

7) Chas Ricciardi, Project Manager, INTECH Construction Inc., Philadelphia, PA

8) DJ Cramer, Symons by Dayton Superior

9) Symons website, www.symons.com

XI. Appendix

BM1-Beam fully Loaded by Wall				(fixed)		
first floor level					End Mu =	wL^2/16
					Mid Mu =	wL^2/12
Thickness (in)	Span (ft)	Load lb/ft	Mid Mu (lb-ft)	End Mu (lb-ft)		p
7	15	15078.65	282724.622	212043.5		0.0181
Size						
d (in)	h (in)					
24.29939371	27					
Pos Rein			Neg Rein			
As	4#8	As	a	As	4#7	As
3.078733183		3.16	7.4	2.287483		2.4
			5.76676476			

BM2-Beam Carrying Slab						
Wt	Load (psf)					
7.5	172					
Thickness (in)	Span	Load (plf)	Mid Mu (lb-ft)	End Mu (lb-ft)		p
7	19	1290	38807.5	29105.63		0.0181
Size						
d (in)	h (in)					
9.002672572	12					
Pos Rein			Neg Rein			
As	4#4	As	a	As	4#4	As
1.140638615		1.24	2.6	0.839698		1.24
			2.11688503			

BM3- Beam Carrying Slab

Wt	Load (psf)					
10	172					
Thickness (in)	Span	Load (plf)	Mid Mu (lb-ft)	End Mu (lb-ft)		p
7	24	1720	82560	61920		0.0181
Size						
d (in)	h (in)					
13.13102004	16					
Pos Rein			Neg Rein			
As	4#6	As	a	As	4#4	As
1.663700239		1.76	2.6	1.163044		1.24
			2.93204436			

BM3- Beam Carrying Slab

Wt	Load (psf)					
5	172					
Thickness (in)	Span	Load (plf)	Mid Mu (lb-ft)	End Mu (lb-ft)		p
7	24	860	41280	30960		0.0181
Size						
d (in)	h (in)					
9.285033314	12					
Pos Rein			Neg Rein			
As	4#4	As	a	As	4#4	As
1.176413721		1.24	2.6	0.861612		1.24
			2.17213093			

BM8-Beam partially Loaded by Wall

					(fixed)	
first floor level		estimated based on fraction of			End Mu =	wL^2/16
		fully loaded condition			Mid Mu =	wL^2/12
Thickness (in)	Span (ft)	Load lb/ft	Mid Mu (lb-ft)	End Mu (lb-ft)		p
7	15	12966.18	243115.8	182336.8714		0.0181
Size						
d (in)	h (in)					
22.53306	25					
Pos Rein			Neg Rein			
As	4#8	As	a	As	4#7	As
2.854939		3.16	7.4	2.15149834		2.4
			5.423945			

BM6-Cantilever Beam carrying Slab

Wt	Load (psf)					
10	172					
Thickness (in)	Span	Load (plf)		End Mu (lb-ft)	Delta	p
7	5	1720		21500	0.08215	0.0181
Size						
d (in)	h (in)					
6.700896	9					
Pos Rein			Neg Rein			
As	3#5	As	a	As	4#4	As
0.849003		0.93	2.6	0.884626945		1.24
			2.230152			

BM9-Beam partially Loaded by Wall						
ground floor level					(fixed) End Mu =	wL^2/16
					Mid Mu =	wL^2/12
Thickness (in)	Span (ft)	Load lb/ft	Mid Mu (lb-ft)	End Mu (lb-ft)		p
7	15	12966.18	243115.8	182336.8714		0.0181
Size						
d (in)	h (in)					
22.53306	25					
Pos Rein			Neg Rein			
As	4#8	As	a	As	4#7	As
2.854939		3.16	7.4	2.15149834		2.4
			5.423945			

Table 7 - Wind Load Criteria		
Basic Wind Speed (3s Gust)	90 mph	90 mph
Building Category	II	II
Wind Importance Factor, Iw	1	1
Wind Exposure	B	B
Internal Pressure Coefficient	0.18	0.18

Table 8 - Wall Wind Loading

Height above Ground, z (ft)	Velocity Pressure Exposure Coefficient, Kz	qz	Uniform Wind Load p (psf) (see figure 8)
0-15	0.57	10.05	9.76
20	0.62	10.93	10.91
25	0.66	11.63	11.82
30	0.7	12.34	12.74
40	0.76	13.40	14.11
50	0.81	14.28	15.26
60	0.85	14.98	16.18
70	0.89	15.69	17.09
80	0.93	16.39	18.01
90	0.96	16.92	18.70
100	0.99	17.45	19.38

120	1.04	18.33	20.53

Table 9 - Seismic Load Criteria			
SDC	B	**Td**	0.068
Importance	1	**Ts**	0.339
R	4	**Ta**	0.698
Overstrength	2.5	**Sa**	0.117
Cd	4	**Cs**	0.0173
Ss	0.33		
S1	0.082		
Fa	1.1		
Fv	1.5		
Sms	0.363		
Sm1	0.123		
Sds	0.242		
Sd1	0.082		

EW Stucture

E-W Direction							
Floor	hx	wh^k	Cvx	Direct Floor Shear, Fx (kips)	Overturning Moment	Drift (in)
G	0	N/A	N/A	N/A		0.74
1	10	N/A	N/A	N/A		
2	20	27495866.02	0.02	4.62	92.42	
3	30	42950456.87	0.04	7.22	216.54	
4	40	58938679.06	0.05	9.91	396.20	
5	30	42950456.87	0.04	7.22	216.54	
6	60	92066319.76	0.08	15.47	928.34	
7	70	109079277.3	0.10	18.33	1283.21	
8	80	126337824.3	0.11	21.23	1698.56	
9	90	143814000.8	0.13	24.17	2175.21	
10	100	161485825.4	0.14	27.14	2713.88	
11	110	179335538.6	0.16	30.14	3315.25	
R	120	137498714.1	0.12	23.11	2772.92	
				Total (k)	Total (ft-k)	
			Controls	188.55	15809.07	

EW Structure

N-S Direction										
Floor	Percent Shear	Distributed Wind Load (psf)	Vertical Tributary Area in previous distributed load region (ft)	Distributed wind load (psf)	Vertical Tributary Area in previous distributed load region (ft)	Total Load at Floor Level (kips)	Design Vu (1.6)Vu (k)	hx	Overturning Moment	
G	1.00	9.76	5.00			7.83	12.53	0.00	0.00	
1.00	1.00	9.76	5.00			7.83	12.53	10.00	125.33	
2.00	1.00	10.91	5.00	11.82	5.00	18.24	29.19	20.00	583.71	
3.00	1.00	12.34	5.00	13.40	5.00	20.65	33.04	30.00	991.25	
4.00	1.00	13.40	5.00	14.28	5.00	22.21	35.53	40.00	1421.24	
5.00	1.00	14.28	5.00	14.98	5.00	23.48	37.57	50.00	1878.40	
6.00	1.00	14.98	5.00	15.69	5.00	24.61	39.38	60.00	2362.70	
7.00	1.00	15.69	5.00	16.39	5.00	25.74	41.19	70.00	2883.22	
8.00	1.00	16.39	5.00	16.92	5.00	26.73	42.77	80.00	3421.85	
9.00	1.00	16.92	5.00	17.45	5.00	27.58	44.13	90.00	3971.79	
10.00	1.00	17.45	5.00	18.33	5.00	28.71	45.94	100.00	4594.15	
11.00	1.00	18.33	10.00			29.42	47.07	110.00	5178.03	
R	1.00	18.33	5.00			14.71	23.54	120.00	2824.38	
						Total (k)	Total (k)		Total (ft-k)	Drift
						277.76	444.42		30110.73	0.72

NS Structure

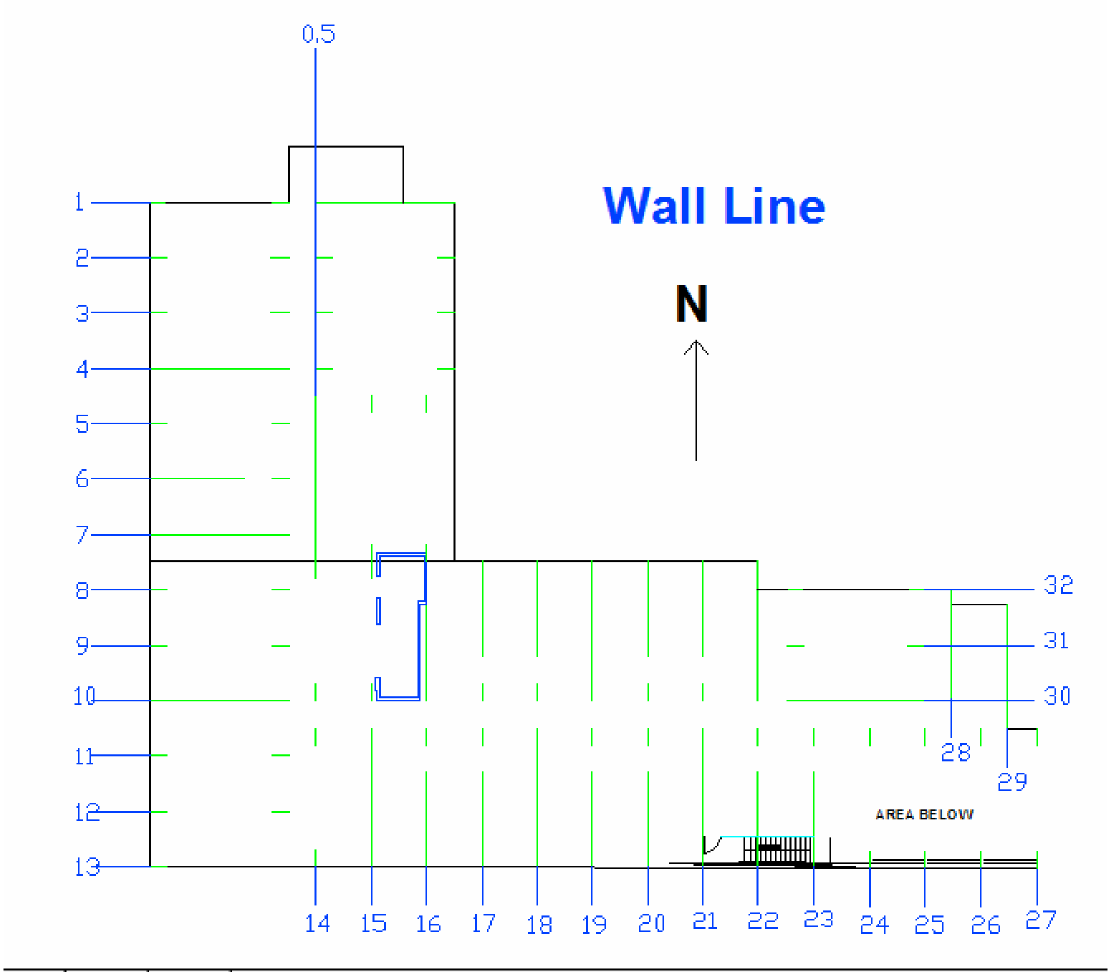
E-W Direction

Floor	Percent Shear	Distributed Wind Load (psf)	Vertical Tributary Area in previous distributed load region (ft)	Distributed wind load (psf)	Vertical Tributary Area in previous distributed load region (ft)	Total Load at Floor Level (kips)	Design Vu (1.6)Vu (k)	hx	Overturning Moment	
G	1.00							0		
1	1.00	9.76	5			3.66	5.85670176	10	58.5670176	
2	1.00	10.91	5	11.82	5	8.52	13.63811904	20	272.7623808	
3	1.00	12.34	5	13.40	5	9.65	15.4400256	30	463.200768	
4	1.00	13.40	5	14.28	5	10.38	16.6033152	40	664.132608	
5	1.00	14.28	5	14.98	5	10.97	17.5550976	50	877.75488	
6	1.00	14.98	5	15.69	5	11.50	18.4011264	60	1104.067584	
7	1.00	15.69	5	16.39	5	12.03	19.2471552	70	1347.300864	
8	1.00	16.39	5	16.92	5	12.49	19.9874304	80	1598.994432	
9	1.00	16.92	5	17.45	5	12.89	20.621952	90	1855.97568	
10	1.00	17.45	5	18.33	5	13.42	21.4679808	100	2146.79808	
11	1.00	18.33	10			13.75	21.9967488	110	2419.642368	
R	1.00	18.33	5			6.87	10.9983744	120	1319.804928	
						Total (k)	Total (k)		Total (ft-k)	Drift
						126.13	201.81		14070.43	0.55

NS Structure

S-N Direction

Floor	Percent Shear	Distributed Wind Load (psf)	Vertical Tributary Area in previous distributed load region (ft)	Distributed wind load (psf)	Vertical Tributary Area in previous distributed load region (ft)	Total Load at Floor Level (kips)	Design Vu (1.6)Vu (k)	hx	Overturning Moment	
G						0.00		0		
1	1.00	9.76	5			2.68	4.294914624	10	42.94914624	
2	1.00	10.91	5	11.82	5	6.25	10.0012873	20	200.0257459	
3	1.00	12.34	5	13.40	5	7.08	11.32268544	30	339.6805632	
4	1.00	13.40	5	14.28	5	7.61	12.17576448	40	487.0305792	
5	1.00	14.28	5	14.98	5	8.05	12.87373824	50	643.686912	
6	1.00	14.98	5	15.69	5	8.43	13.49415936	60	809.6495616	
7	1.00	15.69	5	16.39	5	8.82	14.11458048	70	988.0206336	
8	1.00	16.39	5	16.92	5	9.16	14.65744896	80	1172.595917	
9	1.00	16.92	5	17.45	5	9.45	15.1227648	90	1361.048832	
10	1.00	17.45	5	18.33	5	9.84	15.74318592	100	1574.318592	
11	1.00	18.33	10			10.08	16.13094912	110	1774.404403	
R	1.00	18.33	5			5.04	8.06547456	120	967.8569472	
						Total (k)	Total (k)		Total (ft-k)	Drift
						92.50	148.00		10318.32	1.261


Wall Line
N
0.5
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
31
32
AREA BELOW

North South Structure

	Wall Line	K (SAP)	%Shear		
N-S	line 0.5	117.37	100.00	Drift	Combined N-S Drift
E-W	line 1	65.49	32.28	1.260934	1.9809759
	line 1	45.43	22.40	Drift	
	line 2	5.48	2.70	0.5504668	
	line 2	5.48	2.70		
	line 3	5.48	2.70	*Relative Stiffness of 3' Sections*	
	line 3	5.48	2.70		*0.1279499*
	line 4	70.03	34.52		
	line 4	65.49	32.28		
	line 5	5.48	2.70		
	line 6	22.76	11.22		
	line 7	70.03	34.52		

East West Structure

	Wall Line	K (SAP)	%Shear				
E-W	line 8	5.48	2.16	line 30	91.83	36.18	Drift
	line 9	5.48	2.16	line 31	0.00	0.00	0.742919938
	line 10	70.03	27.59	line 32	0.00	0.00	
	line 11	5.48	2.16	line 19	2.00	0.54	
	line 12	5.48	2.16	line 20	2.00	0.55	
	line 13	70.03	27.59	line 20	45.43	12.47	
N-S	line 14	0.00	0.00	line 21	45.43	14.24	Drift
	line 14	0.00	0.00	line 21	0.00	0.00	0.720041879
	line 15	45.43	7.36	line 22	45.43	16.61	
	line 15	0.00	0.00	line 22	14.00	6.14	
	line 16	2.00	0.35	line 23	20.00	9.34	
	line 16	45.43	7.97	line 24	47.42	24.42	
	line 17	2.00	0.38	line 25	7.91	5.39	
	line 17	45.43	8.70	line 26	7.91	5.70	
	line 18	17.60	3.69	line 27	47.42	36.22	
	line 18	45.43	9.89	line 28	40.00	47.92	
	line 19	45.43	10.98	line 29	43.48	100.00	

Sample Wall Reinforcement Calculations

Load Combination= 0.9D+1.6W

Wall Line 1a, 1b		Tributary Width (in) =	60	No. Stories Slab=	11		
Height (in)	Thickness	Length	A	I	Pu (lb)	Pu/A (psi)	MuY/I (psi)
1248	7.00	526.08	3682.56	84932159.86	537802.07	146.04	225.11
				H/L	alpha	Vu=	58.24
Assume d=0.8L	Assume z=0.6L			2.37	2		0.75 Vc/2 (k)
c=	346.95	>?	350.72	for Design Displacement H/400	**No Boundary Elements**	Vu<	139.74
	0.2f'c (psi)		Pu/A+Muy/I	Pu/A-Muy/I		**Min Shear Rein**	Acr*Sqrt(f'c)
	800.00	>?	371.15	-75.39		Vu<	232.92
	Compression Mu/z+Pu (lb)		Tension Mu/z (lb)				
	768076.165		230274.0925				

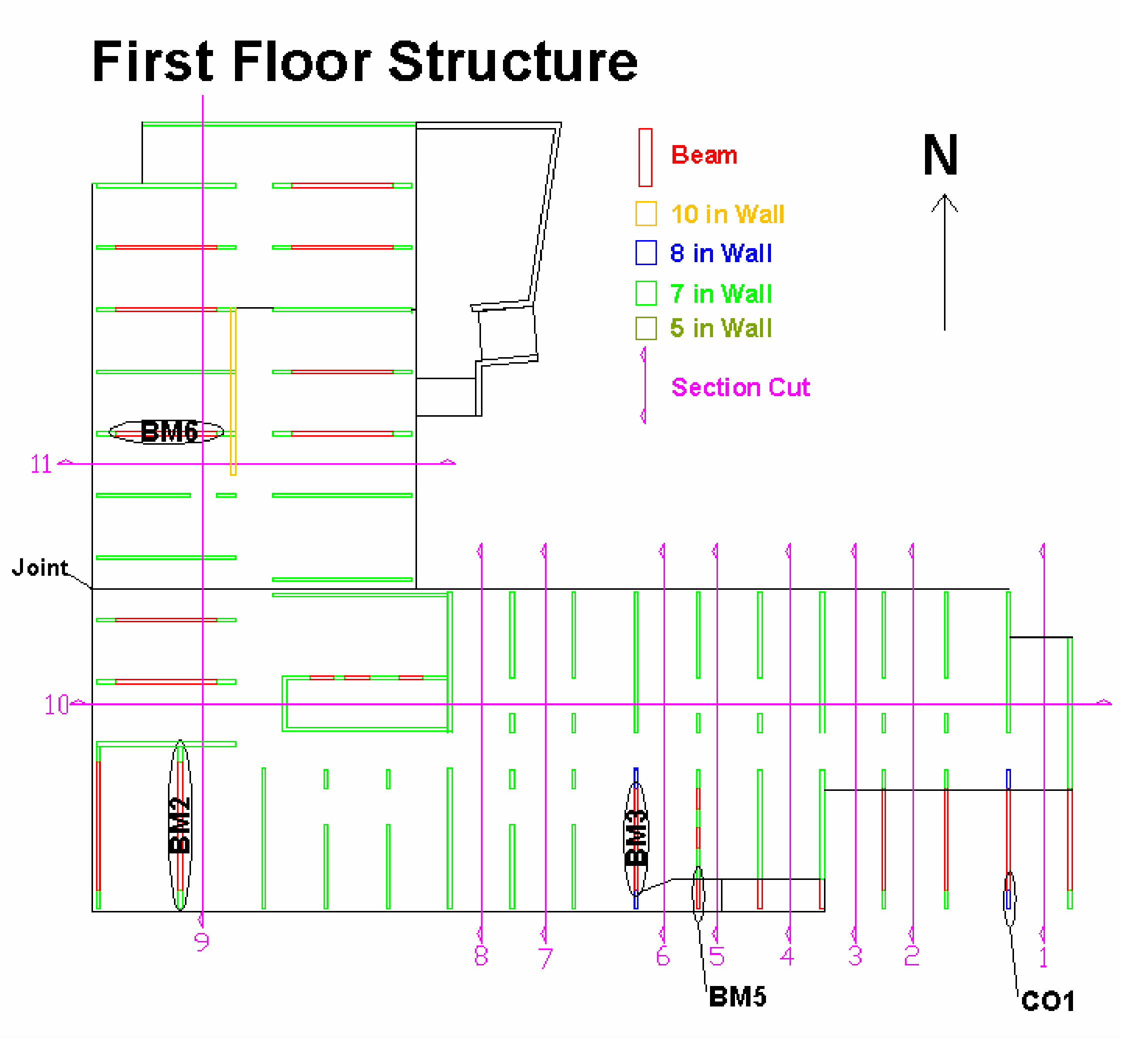
Sample Wall Reinforcement Calculations (cont.)

Load Combination= 1.2D+1.6W+L+0.5S

Wall Line 1a, 1b		Tributary Width (in) =	60	No. Stories Slab=	11		
Height (in)	Thickness	Length	A	I	Pu (lb)	Pu/A (psi)	MuY/I (psi)
1248	7.00	526.08	3682.56	84932159.86	789625.79	214.42	225.11
				H/L	alpha	Vu=	98.87
Assume d=0.8L	Assume z=0.6L			2.37	2		0.75 Vc/2 (k)
c=	410.87	>?	350.72	for Design Displacement H/400	**Boundary Elements (c>)**	Vu<	139.74
	0.2f'c (psi)		Pu/A+Muy/I	Pu/A-Muy/I		**Min Shear Rein**	Acr*Sqrt(f'c)
	800.00	>?	439.53	-10.69		Vu<	232.92
	Compression Mu/z+Pu (lb)		Tension Mu/z (lb)				
	1213376.975		407861.7971				

Horizontal Steel	rho horizontal	vertical section	As Req.	Max Spacing	Spacing	No. Bars	Area Req./Bar	Bar	Area/Bar	As	Actual rho
min 0.0025	0.0025	8736	21.84	18	16	79	0.28	#5	0.31	24.49	0.002803
Vertical Steel	rho vertical	horizontal section	As Req.	Max Spacing	Spacing		Area Req./Bar	Bar	Area/Bar	As	
	0.0025	3682.56	9.2064	18	18	70	0.13	#4	0.2	14.07	0.003820
			Tension Steel	Area	# perp rows	# long rows	Bar	Area (in^2)			
				3.837901542	4	2	#9	4			

Earlier Design



Earlier Design