Michael Steehler

Senior Thesis Final Report

Structural Emphasis

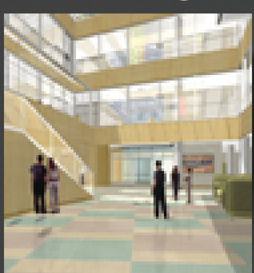
Spring 2007

University Of Rochester
BME/Optics Building
Rochester, NY

University of Rochester BME/Optics Building

Overview/Architecture

- 100,000 sq ft facility for acclaimed Biomedical Engineering and Optics Departments at U of R
- Total Project Cost: $37.7 million
- Construction: Apr. 2005 – Dec. 2006 (expected)
- Design – Bid – Build approach
- 5 stories above grade, plus mechanical penthouse and partial basement
- Includes laboratories, classrooms, offices, and a lecture hall
- Functionally connected to Wilmot Hall, although structurally independent



- Pedestrian bridge on 2nd floor to nearby CSB Building
- 80 foot atrium inside main entrance, lit by skylights
- Limestone and brick façade, with channel glass at stairwells and glass curtain wall at entrances

Mechanical

- Existing utility tunnel provides high pressure steam, pumped condensate, cogeneration water supply/return, and chilled water supply/return
- Air handling units in mechanical penthouse utilize glycol preheat, reheat coils, chilled water coils, and humidifiers
- Special consideration to pressurization and ventilation of laboratories
- Unique plumbing needs in laboratories

Project Team

Owner:	University of Rochester
Architect:	Perkins & Will
Structural Design:	LeMessurier Consultants
Associate Architect/ Struct. Documentation:	SWBR Architects
MEP:	M/E Engineering, P.C.
General Contractor:	LeChase Construction

Structural

- Foundation consists of piles/pile caps with grade beams at exterior walls and above utility tunnel
- Steel columns support composite steel beams and girders
- 4 ½ " typical floor slab on 3" composite deck
- Chevron bracing provides primary lateral support

Lighting/Electrical

- Lighting: Surface mounted fluorescent in laboratory and mechanical space, Recessed downlights in corridors, lounge areas, and lecture hall, Recessed fluorescent in office space
- 277/480V substation powered by 1000/1500 kVA 3 Φ cast coil transformer
- 120/208V substation powered by 750/1125 kVA 3 Φ cast coil transformer
- 00kW diesel emergency generator with bypasses for life safety and critical equipment

MIKE STEEHLER
MJS587@PSU.EDU
ARCHITECTURAL ENGINEERING / STRUCTURAL OPTION
THE PENNSYLVANIA STATE UNIVERSITY
WWW.ARCHE.PSU.EDU/THESIS/EPORTFOLIOS/2007/PORTFOLIOS/MJS587

Executive Summary

This report summarizes and describes a redesign of BMEO as a concrete structure. The redesign consists of a 10" thick concrete flat slab for typical floors, and a similar 12" thick system at the mechanical penthouse/low roof level. Drop panels were used at the western face of the building, where the longer spans and high loads induced high shear forces at columns. The floor system design procedure used finite element analysis computer software to more accurately predict the slab behavior for this uniquely shaped building. The equivalent frame procedure, a more traditional design method, was also completed for comparison. Reinforced concrete shear walls provide lateral stability for the new structural system, located in areas of the building that minimized architectural impact. A few design challenges, such as transfer girders at the second floor, made this concrete redesign difficult. A few unique areas of the building are pointed out where the new concrete system has clear advantages over the original steel system.

In addition to structural work, two other breadth topics are explored. Construction management issues of cost and scheduling are analyzed to provide a comparison between to the original steel design and the concrete redesign. Also, research on BMEO as a green building is described. Rather than listing methods of achieving LEED points for certification, this section analyzes one hypothetical method of an environmentally friendly and energy-saving design, an ETFE foil cushion roof over the atrium.

Upon completion of this project, the following conclusions were made:

- Recent technologies such as finite element analysis make concrete design for more complex structures possible and more efficient

- Although more labor intensive, cast-in-place concrete can be very economical in buildings such as BMEO, that would otherwise require a large quantity of steel

- Technological advancements such as EFTE foil cushion membranes can provide architecturally unique, environmentally friendly, and energy-saving building solutions

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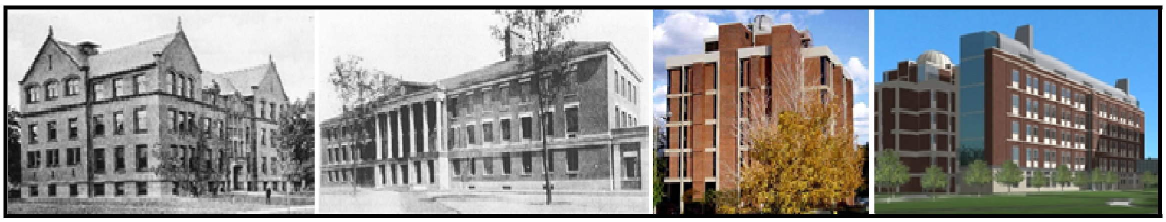
Building Background

The Institute of Optics at the University of Rochester was founded in 1929, making it the first optics education program in the United States. Since its beginning, the Institute has expanded into a diverse spectrum of studies, while remaining recognized as one of the top programs in the country. It has had several homes, including the Eastman Building, Bausch and Lomb Hall, and Wilmot Hall, as shown below.



Eastman Building (1929-31) Bausch & Lomb Hall (1931-77) Wilmot Hall (1977) Wilmot Hall / Goergen Hall

Institute of Optics Facilities

The Institute of Optics currently has about 100 undergraduate students, 100 graduate students, 12 research scientists, and 24 professors. In order to meet the growing demands of the department, a new facility was recently proposed. The Institute of Optics, along with the University of Rochester's newest program, Biomedical Engineering, will be occupying a new $38 million, 100,000 square foot building. This structure will not simply replace Wilmot Hall, the current Optics facility. Instead, it is being built adjacent to Wilmot Hall, to form a unique complex that will meet the growing demands of the University's top programs. Construction began in January of 2005, and is nearing completion at the time of this report.

The University of Rochester has officially named this building Goergen Hall for the generous donations and continued support to the University by Robert B. Goergen. For consistency in this thesis project, however, the building will continue to be referred to as the "BME/Optics Building" or "BMEO" for short, as designated in previous technical reports and as referred to by the building's original designers.

Key Players

The design and management process of BMEO was a bit unique. Perkins & Will (Boston, MA) was the architect of record, with LeMessurier Consultants (Cambridge, MA) as the structural engineer. They carried the design through the design development phase, where it was then handed over to SWBR Architects & Engineers in Rochester, NY. SWBR completed both the architectural and structural design, and prepared construction documents.
The full project team is as follows:

Owner
University of Rochester
Rochester, NY
www.rochester.edu

General Contractor
LeChase Construction, LLC
Rochester, NY
www.lechase.com

Architect
Perkins & Will
Boston, MA
www.perkinswill.com

Associate Architect
SWBR Architects
Rochester, NY
www.swbr.com

Structural Engineering: Design
LeMessurier Consultants
Cambridge, MA
www.lemessurier.com

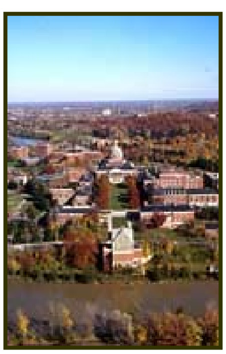
Structural Engineering: Documentation
SWBR Architects
Rochester, NY
www.swbr.com

Civil Engineering / Survey
Parrone Engineering
East Rochester, NY
www.parroneeng.com

MEP / Fire Protection
M/E Engineering, P.C.
Rochester, NY
www.meengineering.com

Architecture

The new BME / Optics Building is a 100,000 square foot structure, consisting of five stories plus a mechanical penthouse and partial basement. It includes several teaching laboratories, research facilities, and offices, as well as a large lecture hall on the first floor.



BMEO is strategically located on the University of Rochester's River Campus, across the street from the Medical Center. As mentioned earlier, it is built adjacent to the existing Wilmot Hall on two sides, with pedestrian access at the first and fourth floors. In addition, there is a pedestrian bridge linking BMEO with the nearby CSB Building, to provide easy access to computer lab and library services. The new facility is intended, both symbolically and functionally, to be a link between optics and medicine, two of the featured programs at the University of Rochester.

Although the red brick exterior and rectangular shape is typical for buildings at U of R and many other college campuses, there are a few architectural characteristics that set this building apart. The main feature of the building is the 80+ foot tall atrium inside the main entrance, lit by skylights. Stairs are cantilevered into the atrium from the floors that provide a “floating” effect. Interior glazing at corridors on every floor provide views into the atrium. Also, cantilevered canopies over entrances continue the “floating” idea.

Envelope

The building envelope varies, but consists entirely of non-load bearing façade walls. The primary exterior wall is 3” limestone veneer up to the second floor and 4” standard red brick above, with metal stud backup. Window openings at these sections of the building are 6’-4” wide and vary in depth, not exceeding 10’-8”. This type of red brick façade is common on the U of R campus, and allows BMEO to fit in with the surrounding architecture.

To make this building stand out architecturally, there are a few sections of the façade that differ from the traditional brick and stone system. These include channel glass at stairwells, and a few sections of an aluminum framed glass curtain wall system, most notably at the main entrance. The use of glass on the building’s façade allows natural light into the atrium space, allows the building’s interior lighting to make the building glow at night, and architecturally leads the eye to the entrances of the building.

Roof

The main roofing system is a cold process built-up roof using 3” metal deck. It is at ¼” slope with primary and auxiliary roof drains and a parapet wall. Also, there are 3 large skylights above the atrium at the roof level.

The high roof covering the mechanical penthouse is a metal clad, curved slope roof. This system uses 3” roof deck at a radius sitting on rolled W8 steel shapes. It is described in more detail in the existing structural design section of this report.

Mechanical

An existing utility tunnel that runs under the footprint of the building supplies the mechanical system of BMEO. This tunnel is used by several buildings on campus, and provides high-pressure steam, pumped condensate, cogeneration water, and chilled water to the building. A large mechanical penthouse contains air handling units that utilize glycol preheat, reheat coils, chilled water coils, and humidifiers to supply heating and cooling needs to the building.

The laboratory spaces of BMEO required special mechanical consideration. Each laboratory has special requirements for pressurization and ventilation, as well as plumbing needs including nitrogen, deionized water, and acid waste removal, to name a few.

Lighting / Electrical

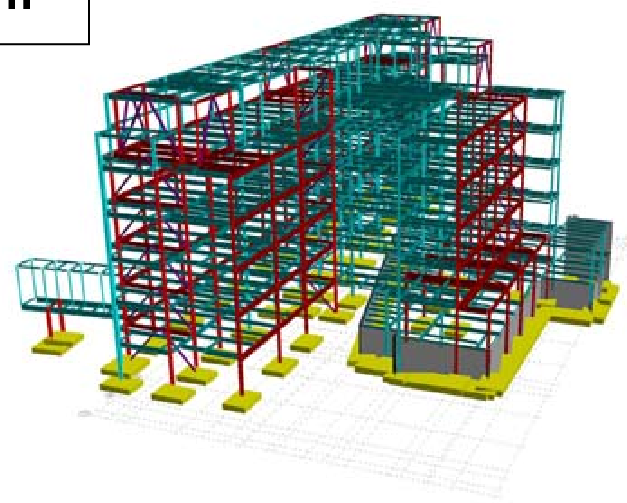
The lighting methods in this building vary based on usage. Laboratory and mechanical spaces use surface mounted fluorescent lighting, while recessed fluorescent lighting is used in offices. Also, recessed downlights illuminate corridors, lounge areas, and the lecture hall.

The electrical system uses a 277/480V substation powered by a 1000/1500 kVA 3 Φ cast coil transformer and a 120/208V substation powered by 750/1125 kVA, 3 Φ cast coil transformer. There is also an 800kW diesel emergency generator with bypasses for life safety and critical equipment.

Construction

The construction of BMEO used a design-bid-build approach. The total project cost was $37.7 million, with construction beginning in January 2005 and a scheduled completion of December 2006. Originally, BMEO was set to be open for the Spring 2007 semester. Unfortunately, construction was not completed as scheduled due to several setbacks, and the building will not be open until April/May 2007.

Existing Structural System



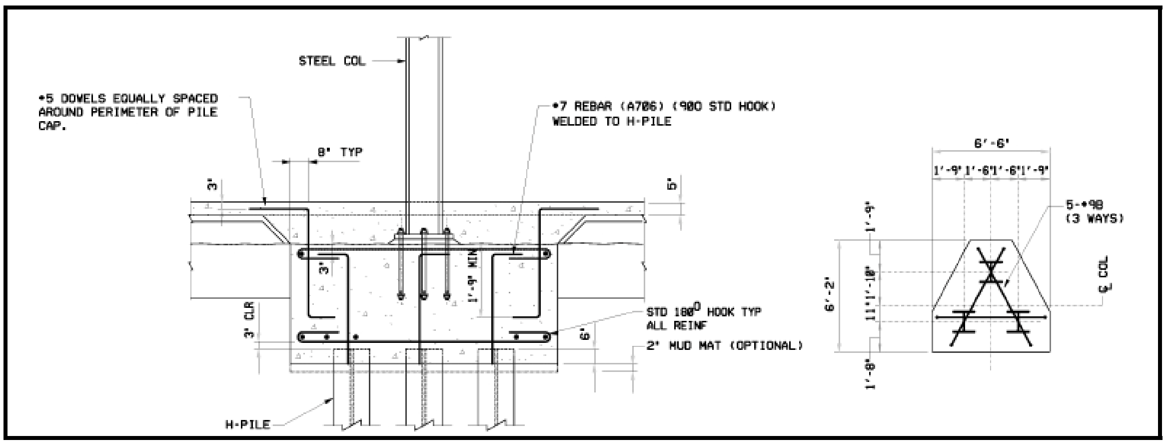
Foundation: Steel H-Piles

The foundation system used in this building consists of pile caps supported by 50 ksi steel H-piles bearing on bedrock. There are several different pile configurations, but they all use HP8x36 (Design capacity = 66.3 tons), HP10x42 (77.5 tons / 13 kips design uplift), and/or HP10x57 (105 tons). The pile caps have a design lateral load capacity of 4 kips each.

The foundation system also uses grade beams at different sections of the building. All exterior walls are supported by grade beams, typically 16"x 48", with some variations in size. Also, the existing steam/utility tunnel running under the footprint of the building is framed around by grade beams 24"x 54" and 18"x 24". Since this tunnel supplies several buildings on this section of campus, its complete functionality throughout construction of BMEO was an important design consideration.

Finally, the basement area, which is only a small portion of the building's footprint, has reinforced concrete walls, typically 16" thick. These walls are supported by grade beams 16' below grade that frame into pile caps.

Concrete for pile caps and grade beams is normal weight with design strength of 4000 psi. All steel reinforcing conforms to ASTM A615 Grade 60.



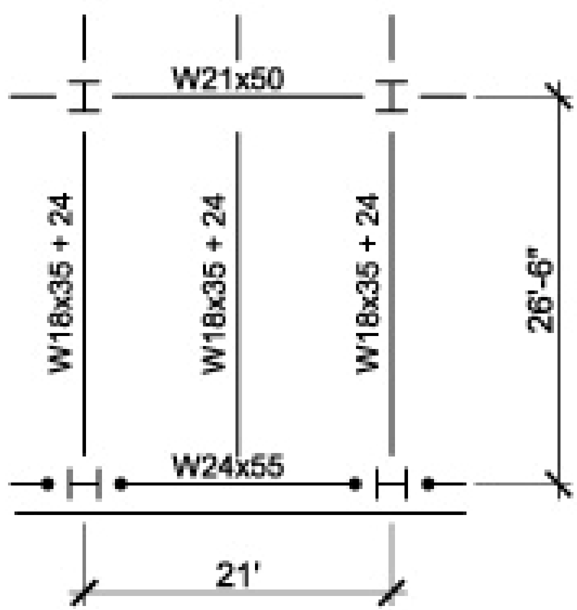
Pile Cap Plan & Section

Framing: Composite Steel

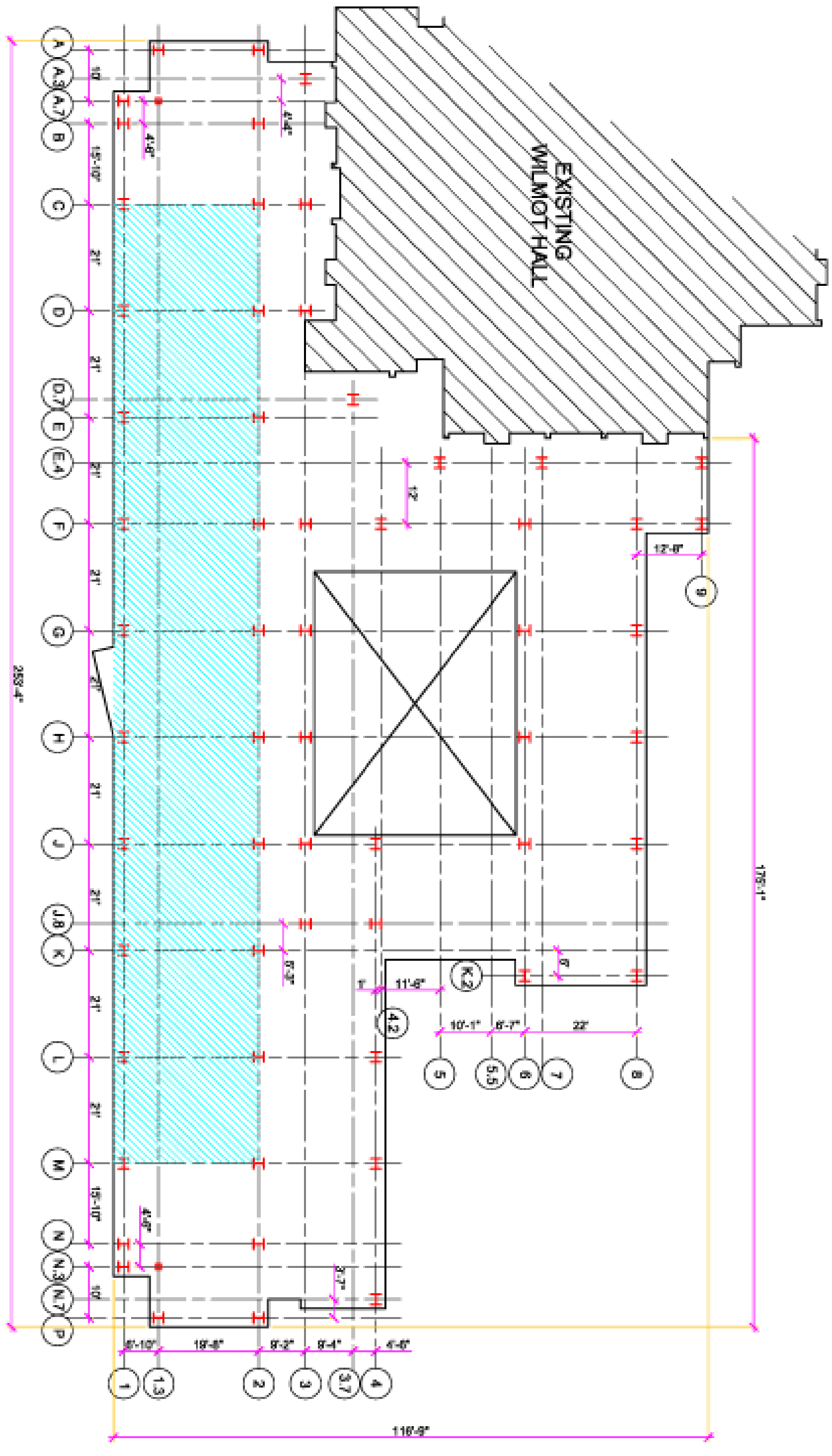
BMEO was designed as a composite steel structure. By using ¾" diameter shear studs on the beams and girders, the framing system in BMEO can utilize the compressive strength of the concrete slabs through composite action, thus reducing steel member sizes and limiting floor deflections and vibrations. In order to achieve this, a 4 ½" concrete slab on 3" composite metal deck (7½" total depth) was used on all floors.

Although the loads are relatively constant throughout the building, the steel beams and girders vary in size due to varying spans. This is because of the irregular shape of the building and column layout designed to meet its architectural and spatial challenges. Thus, there is no "typical bay" redundancy in BMEO that summarizes the floor system as a whole. However, the nine bays along the west face of the building are the same, and can be considered the critical condition, as they have the longest spans.

Similarly, the columns in BMEO also vary significantly throughout the building. They are primarily W12 shapes, but vary in weight from 40 lb/ft supporting the roof to 120 lb/ft at moment frames. Column splices occur at the fourth floor only.



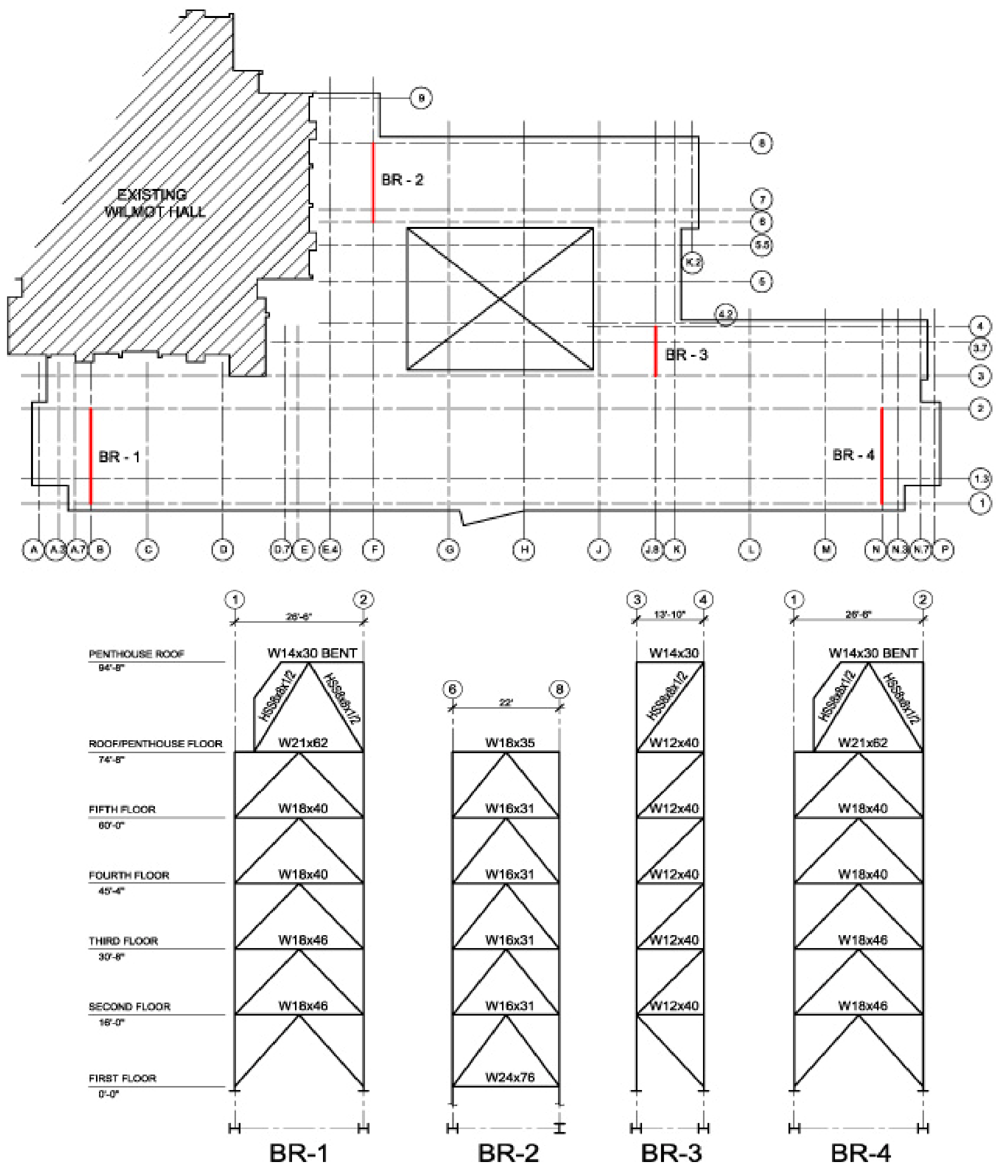
Critical Bay



Typical Column Layout, Floors 2-5, Critical Bays Shaded

Lateral Stability: Concentrically Braced Frames (East-West)

The structural system designed for the University of Rochester BME/Optics Building consists of four concentrically braced frames to support lateral forces in the East-West direction. These braced frames are strategically placed near large mechanical penetrations in order to minimize their effect on spatial layouts of the floors. All members use HSS 7x7x1/2 bracing members up to the low roof/penthouse floor level. Three of the main braced frames use chevron bracing, while the fourth uses diagonal bracing due to its narrower dimension. Lateral load is transferred to the frames from the concrete floor slab by ¾" diameter shear studs, with a minimum of one stud per foot. The locations and elevations of the frames are shown below.



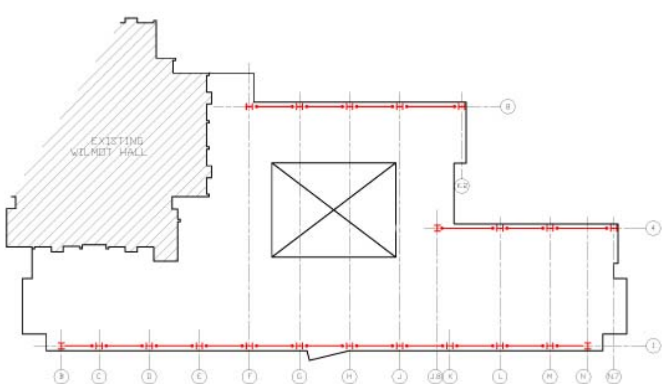
Braced Frame Plan & Elevations

Lateral Stability: Steel Moment Frames (North-South)

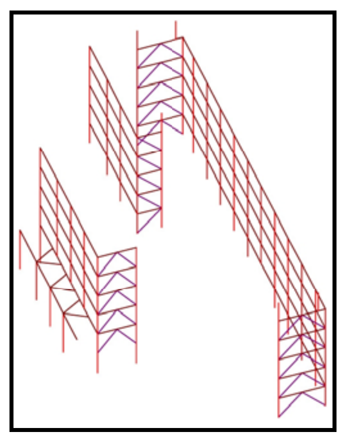
In the longer, narrower direction of the BME/Optics Building, ordinary steel moment frames are utilized to provide lateral stability. Although moment frames tend to be expensive due to increased member sizes and the high cost of connections, they were deemed necessary for this building. There were no logical locations for braced frames large enough to resist the lateral forces in this direction. Instead, moment frames were used along building faces so window openings would not be interfered with.

In all, there are four moment frames, with the largest at the west face of BMEO spanning almost the entire length of the building. Locations and elevations of these frames are shown in Figure 9.

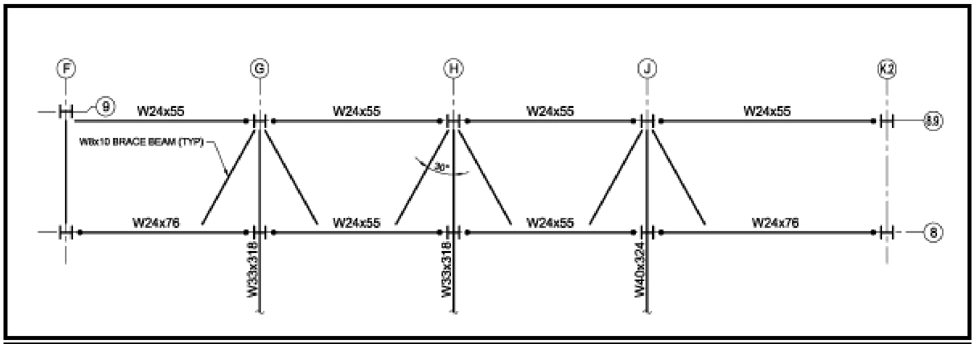
The most interesting of these frames is located at the east face of the building. The columns of this frame are not continuous. The lecture hall at the first floor, which requires a larger column free space, has columns set wider than the floors above, with large (W33x318) transfer girders to transfer the gravity load from the columns above. Since the columns above are part of a moment frame, additional consideration was needed to transfer lateral loads and maintain continuity of the frame. To achieve this, W8x10 diagonal brace beams form a sort of horizontal truss. This structural aspect of the building is one of many design challenges that required unique engineering solutions.



Location of Moment Frames, N-S Direction
Columns not part of moment frames have been omitted for clarity



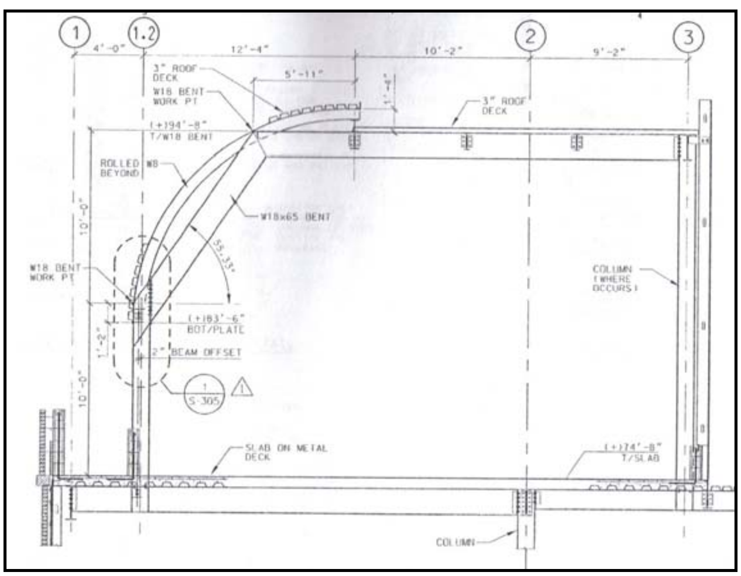
Location of Main Lateral Force Resisting Elements



Horizontal Truss Plan (2nd Floor)

Lateral Stability: Mechanical Penthouse

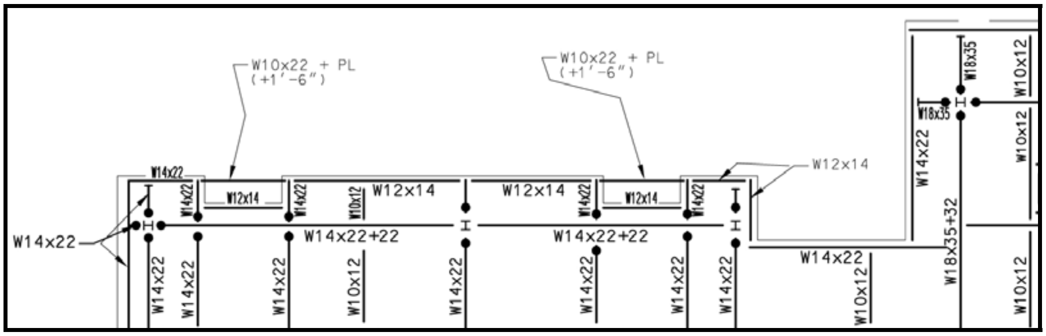
The lateral stability of the mechanical penthouse differs from the rest of the building. Three of the four braced frames in the East-West direction continue to the high roof to support the penthouse. In addition, moment frames, as shown below, are regularly spaced to help resist lateral load in this direction. They are needed because there is no concrete slab at the high roof to act as a diaphragm in transferring lateral loads to the braced frames, which are spaced very far apart. In the North-South direction, six concentrically braced frames support the penthouse.



Typical Moment Frame at Mechanical Penthouse, Elevation

Design Challenges

As mentioned earlier, the BME/Optics Building is a very unique structure with several design challenges. A few of them are outlined below:



- *Adjacent Building*

  BMEO is being built adjacent to Wilmot Hall on two sides, as can be seen in several of the diagrams earlier in this section of the report. Although BMEO is structurally independent from this building, it still presented a challenge to the building's designers. Isolation joints separate the two buildings, increasing in size at higher floors. These are formed by framing out from the columns, cantilevering the slab to the required distance. In addition to the moment connections and the complex framing necessary, lateral drift became a challenge for BMEO as well. Although the joints seem large enough to allow significant drift, thought must be considered in seismic design. Since the two buildings inevitably have different fundamental periods, they could actually converge on one another in the event of seismic forces. Therefore, lateral drift in BMEO had to be carefully considered.

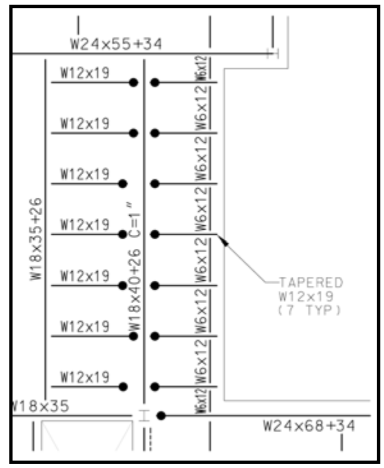
- *First Floor Lecture Hall*

  The architectural design of BMEO called for a large lecture hall on the first floor. This required column free space for sight line and seating considerations. However, the location of the lecture hall did not fit the column layout of the floors above. Large W33 and W40 transfer girders along three column lines were required to transfer gravity forces from the columns above to columns set wider, thus providing open space for the lecture hall. Also, the columns above were part of a moment frame for lateral stability. Diagonal bracing members were needed to form a sort of horizontal truss to transfer lateral loads, as described earlier.

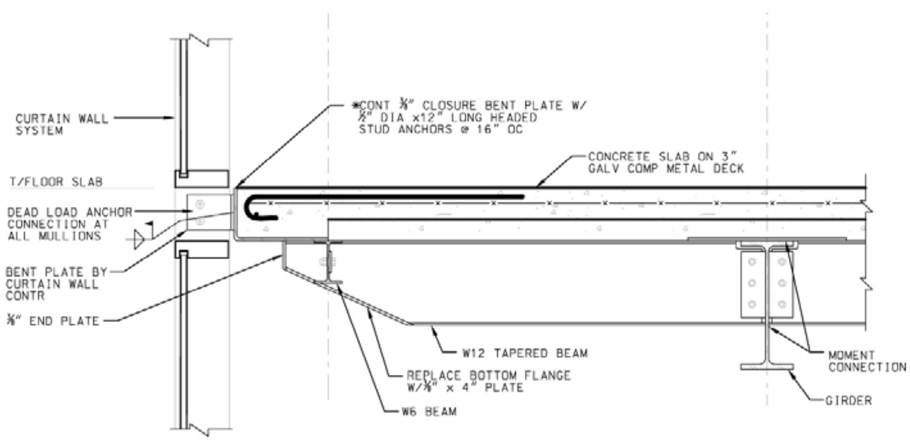
- *Mechanical Penthouse*

  Similarly, many of the columns of the mechanical penthouse do not line up with the columns below. However, the loads from these columns were a lot less, only coming from roof loads, self weight of the high roof, and lateral load reactions as opposed to five floors worth of full loading. Transfer girders at the penthouse floor level, typically W21x62, transfer these loads to the columns below.

- *Glass Curtain Wall*



  Another structural challenge came at the glass façade at the North entrance of the building. Had there been no glass façade, a rectangular bay would have been sufficient, with a W21 girder running along the face of the building. Since the façade was in fact glass, this was not satisfactory. With a total slab depth of 7 ½“ and a 21” deep beam, an unsightly condition would occur at the façade. Instead, the building's designers moved the girder in 5' from the face of the building, and framed out to the edge using tapered W12x19 beams connected rigidly. This reduced the total depth of about 1' at the face, which was more aesthetically pleasing, especially with the use of end plates. Although this was costly, it successively achieved the architectural goals for the building.



Summary

This new Biomedical Engineering and Optics Building will be a state of the art facility for years to come, and will serve as a symbol of excellence for the University of Rochester. In three previous technical reports, the existing structural system was investigated in detail and found to be well designed and well suited to meet the needs of the building. As discussed earlier, this building is far from ordinary, with several design challenges that required unique engineering solutions.

Because of these challenges, there seems to be a large amount of steel members in this building compared to its size, in addition to over 400 moment connections and the additional design details noted in this section. These factors drove the cost of the structure up significantly. Although the existing design works extremely well with the building, alternative structural designs will be considered, for comparison purposes as well as to gain a better understanding of structural design for more complex structures.

Problem Statement / Proposed Solution

The existing composite steel design for the new University of Rochester BME/Optics Building was found to be well designed to meet the structural challenges and architectural requirements of this unique building.

The nature of steel framing allows for complicated geometries and conditions to be designed for gravity loads relatively easily, as compared to concrete structures. Once the layout of the framing is decided, the beams must be designed for flexure and shear, and the columns for axial force and slenderness effects. Because of the nature of common shear connections, beams do not transfer any significant moments to girders or columns. Therefore, member sizes can be interchanged until they are strong enough to resist design loads without significant changes to other members of the structure. Software programs such as RAM Structural System make steel design process much more efficient, as the entire building can be modeled in order to size all members to resist design loads. Also, steel buildings are lighter than concrete structures, and are usually less labor intensive. Because of these factors, steel design is often preferred by designers, especially in buildings such as BMEO that would make a concrete design complicated.

However, the existing design of BMEO uses about 1300 steel beams, amounting to almost 300 tons, with over 6000 shear studs and over 400 moment connections. This seems to be extensive for a five-story structure. That is not to say it was poorly designed or uneconomical. Rather, the structural design was driven by the architecture of the building, dictating an expensive system.

Cast-in-place concrete structures, well more complex to design, can have significant advantages for a building of this nature. Most notably, they have inherent moment capacity, due to the columns being poured monolithically with the floor system. In BMEO, this is significant, especially in the cantilevered areas of the building that required additional framing members and a large number of moment connections, which tend to be quite expensive. Other advantages to concrete structures include inherent fire rating, reduced floor system depth, and durability, to name a few. However, concrete buildings tend to be labor intensive, especially when large amounts of formwork and reinforcement is needed.

While the existing steel structure works extremely well for this building, a structural redesign in cast-in-place concrete is being proposed. This system will be designed as a flat slab to resist gravity loads determined from ASCE 7-05, using the guidelines of ACI 318-02. Cast-in-place shear walls will be designed for lateral stability to resist design wind and seismic loads. The redesign of BMEO as a concrete structure will achieve the following goals:

- To gain a better understanding of the design process for concrete structures
- To design a complete, economical, and structurally sound concrete system
- To compare a concrete redesign with the existing steel design for this unique building
- To determine the effects of a concrete design on cost, scheduling, quality, and feasibility
- To analyze the new BME/Optics Building as a green, environmentally friendly building

Structural Redesign: *Cast-In Place Concrete*

Steel Considerations

In addition to a redesign of BMEO as a concrete structure, redesign in the steel system was also originally proposed based on some preliminary investigations in a previous technical report. The idea was to determine if advantages could be made, as a basis for comparison with the original design as well as the concrete redesign. However, upon further investigation, none of the ideas for the steel structure were found to have significant advantages over the existing design. Though each idea was considered thoroughly, a full steel redesign would not be worthwile. The following steel redesign ideas were considered:

- *Column Layout*
  The existing column layout follows the architectural layout of the laboratory and office space. Spans are relatively short, so areas where spans could be increased or bays doubled were considered. Along the west face of the building, where the critical bays are located, was the main area of consideration. Doubling these bays would mean fewer members and may have caused some economic advantage. However, the girders at the west face of the building form a moment frame to resist lateral load. Doubling the bay size meant these frame members became twice as long. In order to maintain stiffness of the moment frames, these girders became excessively large, negating any advantages of the increased bay size.

- *Steel Joist Floor System*
  Also considered was a steel joist system, as opposed to the existing composite beam design. Because of the relatively high loads, joists for the 26.5' span would have to be spaced at 1.5', meaning a very large number of joists. More importantly, joists are not easily fireproofed and perform poorly in regards to vibration damping. Though usually an inexpensive floor system, steel joists are not suited for BMEO.

- *Lateral System*
  The lateral system of BMEO, as described earlier, was dictated by the architecture of the building. In one direction, four concentrically braced frames are located adjacent to mechanical openings. In the other direction, moment frames at the faces of the building were necessary to allow for window openings. Though moment frames tend to be expensive, there were no logical places to put braced frames large enough to economically resist the lateral loads without dramatically affecting the architecture.

As stated, BMEO's existing steel design was dictated by its architecture. Though expensive, it is well suited to meet the needs of the building. No significant advantages in the steel system could be found that would justify a full steel redesign.

Floor System: Concrete Flat Slab

Design Loads

The current code adopted by local ordinance is the Building Code of New York State (2002), which references IBC 2000 and ASCE 7-98. For this concrete redesign, gravity loads were determined from ASCE 7-05 provisions as an update. Superimposed loads from the original steel design were compared with the newer code and used where applicable. The load combinations for strength design were 1.4D and 1.2D + 1.6L + 0.5S, with the former controlling in most cases. Specific data for mechanical equipment weights were not available, so the 150 psf allowance from the original bid documents was used in the concrete redesign. This is desirable, as it allows for changes in mechanical systems over the life of the building. Design gravity loads are as follows:

Live Loads

Laboratory/Office Space	80 psf
Main Lobby/Stairs	100 psf
Mechanical Room	150 psf (or equip weight)

Dead Loads

Typical Floor	
Slab Self-Weight (150 pcf)	as req'd
Superimposed (Flooring, MEP)	20 psf
Brick Façade w/ Metal Stud Backup	750 plf
Low Roof	
Slab Self-Weight (150 pcf)	as req'd
Built-Up Roof	10 psf
Parapet Walls	300 plf

Two-Way Slab Design: Traditional Methods

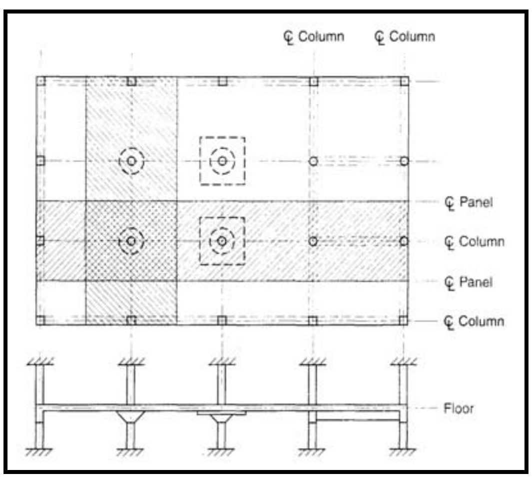
Column supported slabs tend to act flexurally in two orthogonal directions, with negative moments at the support faces and positive moments between them. Since the columns are cast monolithically with the floor slab, the moments are also distributed to the columns. Traditionally, two-way concrete slabs are designed independently in each direction by approximating a region of the slab as a frame and analyzing the distribution of moments. Steel reinforcing of the slab is determined from the moment distribution, and concentrated at column lines. There are a variety of methods for analyzing slabs, including the direct design method, equivalent frame method, yield line analysis, and strip method. Each has limitations, as well as situations to which they are best suited.

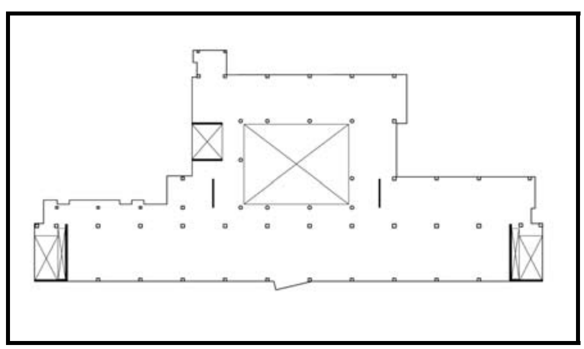
Direct Design Method

The Direct Design Method is a simplified approach to designing two-way slabs. Column lines in each direction are considered as frames with design loads applied to them. The total static moment is found using a simple formula, and is distributed to different portions of the slab based on distribution factors given in a chart. Steel reinforcing is laid out in column strips and middle strips, as defined in the ACI code. This method, although approximate, makes slab design quite simple. Naturally, there are limitations to the use of the Direct Design Method given in ACI section 13.6.1. Most notably, there needs to be three continuous spans in each direction, making this method unacceptable for BMEO.

Equivalent Frame Method

The Equivalent Frame Method for slab design is similar in approach to Direct Design, but is more complicated, and therefore has fewer limitations. This method involves representing the slab system as a series of two-dimensional frames in each direction, consisting of columns and slab-beams. A series of formulas and charts help determine equivalent stiffnesses of slab-beams and columns based on their dimensions. These stiffnesses are used to determine positive and negative design moments on members via moment distribution for different loading configurations. Similar to Direct Design, the factored moments are then used to lay out reinforcing for the column strips and middle strips based on given distribution factors. Though tedious, this method is the most widely used for two-way slab design. The computer program PCA Slab (formerly ADOSS) is a straightforward application of the Equivalent Frame Method, and makes the calculation process much quicker, and often more accurate than hand calculations.





Building Idealization for Equivalent Frame Method vs. BMEO's Geometry

Two-Way Slab Design: Finite Element Analysis

The BME/Optics Building is quite different from the "typical" building used in traditional design methods. Since some column lines in this building are discontinuous and some areas of the building have unique geometric layouts, the Equivalent Frame Method becomes complicated.

When dealing with more complex load scenarios and irregular geometries, the approximation of the system into frames can be quite inaccurate. Today, finite element computer programs such as RAM Concept can more accurately predict the elastic behavior of concrete slab systems, regardless of the structures' irregularities.

Specifically, RAM Concept allows the designer to model and analyze an entire floor rather than a single, two-dimensional frame. Column and slab geometries can be accurately modeled rather than approximated. The program also allows for user-inputted pattern loading to ensure maximum design moments are found.

The Finite Element Analysis involves generating a mesh. This involves dividing the structure into countless three-dimensional sections that are similar in size and geometry. The software analyzes each finite element based on design forces, producing extremely accurate analysis of force and moment distribution in the structure.

RAM Concept uniquely combines the use of finite element analysis with the reinforcing layout procedures of traditional slab design methods. It achieves this by using design strips. These strips are inputted by the user, allowing the program to determine required steel reinforcing based on the calculated finite element forces. It has the capabilities to distribute the reinforcing to column and middle strips based on ACI guidelines.

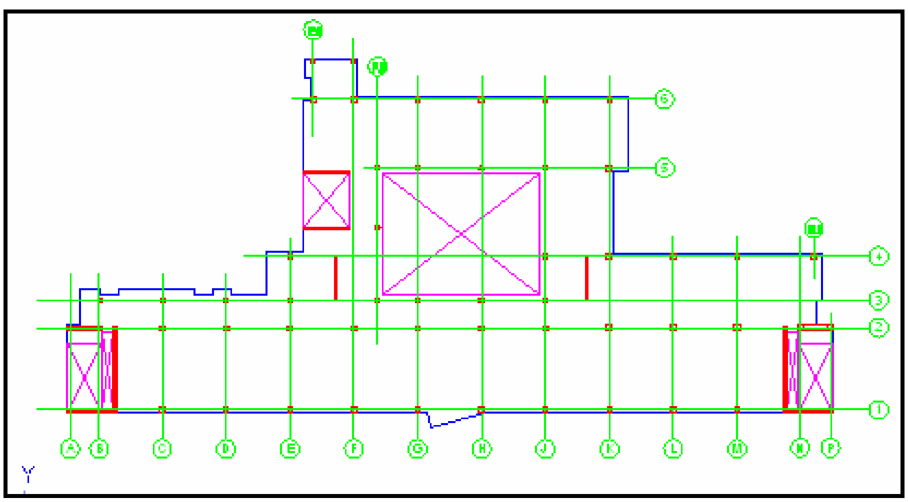
Finite Element software such as RAM Concept can be extremely powerful tools, especially for a structure like BMEO. However, the programs need to be fully understood in order to produce accurate, usable results. Responsible engineering practice would dictate checks of the design using traditional design methods such as the Equivalent Frame Method for critical span conditions and column moments.

Column Layout

The most challenging aspect in designing the BME/Optics Building as a concrete structure was its geometric layout. The footprint of the building and the challenges associated with its architectural features such as the atrium made the column layout of BMEO difficult. In designing cast-in-place concrete structures, the columns have to be laid out so that the two-way behavior of the floor slab is predictable. As discussed earlier, the nature of steel structures provides for an advantage in the freedom of column layouts. As long as framing members can transfer gravity loads to the columns, their layout does not matter.

For the concrete redesign of BMEO, several column layouts were considered. The final layout turned out to be the same as the original steel structure in the areas of rectangular bays. The only differences in column layout occurred around the atrium and shear walls. These changes were necessary to make the slab behavior more predictable and to limit the use of cantilevered slabs.

The reason for circular columns surrounding the atrium is for architectural considerations. These areas are corridors with glazing allowing views to and from the atrium. In the existing design, these columns are W12 shapes with circular casing.



Floor Slab

Minimum Thickness

The minimum thickness for two-way concrete floor slabs is given in ACI section 9.5.3. This guideline is intended to limit deflections when designing slabs using the Direct Design Method or the Equivalent Frame Method. The thickness limits are given as a ratio of the clear span between columns.

For BMEO, the minimum thickness was found to be 10", with the controlling condition being $l_n/30$ for 60 ksi reinforcing steel in an exterior panel without drop panels or edge beams.

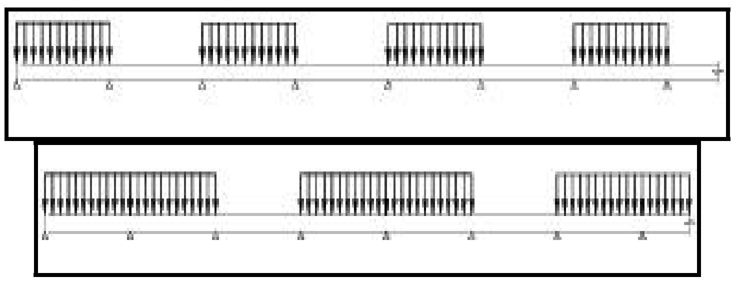
The ACI provisions state that this minimum thickness is to limit deflections when designing slabs using traditional methods. Thicknesses less than the specified value are permitted when calculated deflections are within acceptable limits. Although deflections with a 10" slab were minimal, the slab thickness was not reduced because of punching shear and reinforcing considerations, which will be discussed later.

Design Strips

To calculate required reinforcing for the floor slabs, design strips were set up in RAM Concept. These design strips help the program to lay out and distribute steel reinforcing correctly to resist the forces analyzed in the finite elements. The design strips calculate the width of column strips and middle strips according to ACI guidelines. They are an important tool, but must be accurately set up to reflect the behavior of the slab and reinforcing layout. In an ideal concrete building, design strips are easy to set up, with spans running orthogonally between columns. In more complex structures, where RAM Concept has a significant advantage, careful consideration must be given to design strips in order to produce accurate reinforcing plans.

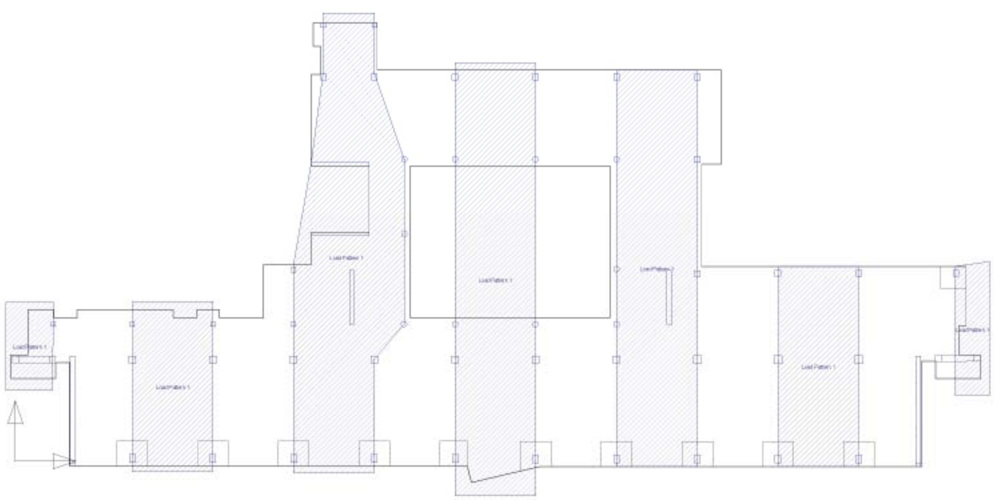
Pattern Loading

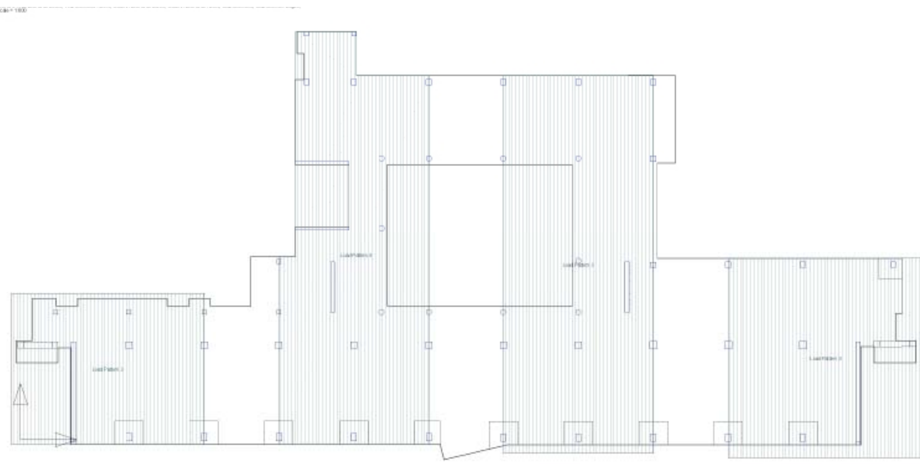
The critical load combination for this structure is 1.2D + 1.6L + 0.5S. However, in order to get accurate design moments in concrete structures, live load patterns must also be considered. The theory behind pattern loading is that a uniformly distributed load over the whole structure does not produce the maximum possible moments. In a given span, the positive moment is reduced when adjacent spans are loaded. Therefore, the maximum positive moment for a given spans occurs when alternating spans are loaded. The maximum negative moment at the column face occurs when the two adjacent spans are loaded, and a similar on and off pattern of loading is continued for the rest of the spans in the given frame. Since the actual occurrence of a perfect pattern loading is unlikely, only 75% of the design live load is used in pattern loading. Dead loads, which tend to be relatively constant throughout the structure are not analyzed in patterns, but are included in every span for pattern loading.



Pattern Live Loading for Maximum Positive Moments (Top) and Maximum Negative Moments (Bottom)

When using the Equivalent Frame Method, each load pattern must be considered separately for each frame. RAM Concept, on the other hand, allowed for layout of 10 live load patterns (5 in each direction) that were analyzed simultaneously. Pattern loading examples for BMEO are shown below.

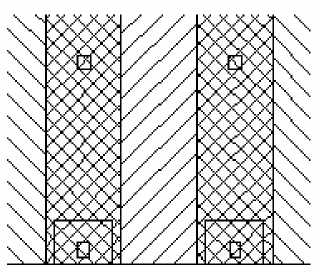




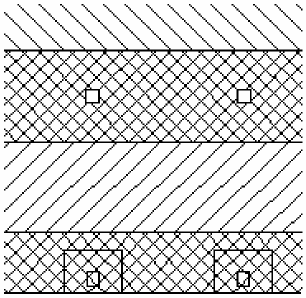
Reinforcing

The final design of the typical floor for BMEO consists of a 10" thick, 4000 psi slab. The steel reinforcing is 60 ksi. The reinforcing was designed using RAM Concept, and spot checked using the Equivalent Frame Method at critical spans. These spot checks were performed both by hand using moment distribution, and with the computer program PCA Slab.

As previously discussed, the spans and conditions vary throughout the building. For the purposes of this report, only the critical bay reinforcing at the west face of the building will be discussed in detail. Full reinforcing plans would be difficult to read at a scale that would fit in a report format, but are available upon request.



	Top, Ext Col	Top, Int Col	Bottom
Column Strip	(14) #5	(20) #5	(12) #4
1/2 Middle Strip	(4) #5	(4) #5	(8) #4

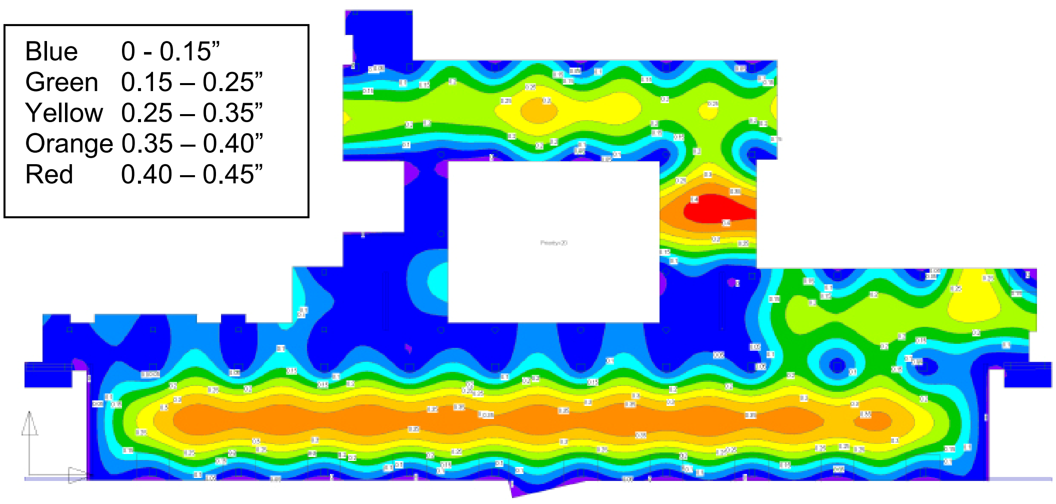


	Top, Ext Col	Top, Int Col	Bottom, Ext	Bottom, Int
Column Strip	(9) #5	(17) #5	(8) #4	(12) #4
1/2 Middle Strip	(6) #5	(6) #5	(9) #4	(9) #4

Deflection

Deflection limits, according to ACI code are L/360 for live load and L/240 for total load, as long as no attached or supported elements are likely to be damaged from deflections. These criteria matches the deflection limits listed in the general notes for the existing steel design.

Deflection of the floor slabs, however, were not the controlling factor in design. Like many flat slab and flat plate systems, punching shear dictated the slab thickness. Maximum deflections for the typical floor were calculated at about 0.4", and at 0.3" for the penthouse floor, which has a thicker slab and higher loads.



Deflection Plan, Typical Floor

Column Design

Since shear walls for BMEO were designed as the main lateral force resisting system, the columns of the building were designed for gravity forces and moments only.

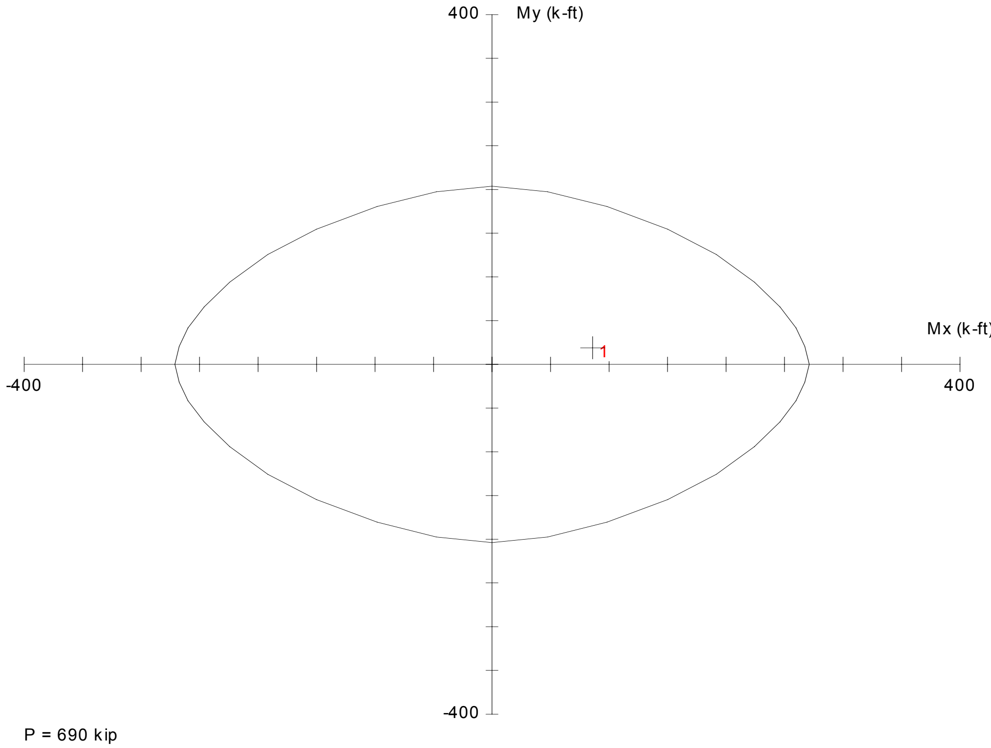
The design of concrete columns was iterative. Initial sizes were estimated based on axial loads and column locations. After analysis and distribution of moments, the design column forces and moments could be tabulated. Based on axial compression and biaxial bending, the reinforcement for each column were then designed using PCA Column. Upon investigation, the initial sizes needed to be adjusted to determine more economical column designs. Because of the inherent nature of cast-in-place concrete structures, larger columns take more of the moment. Therefore, whenever column sizes changed, the floor system needed to be reanalyzed for maximum design moments, and the column reinforcement needed to be adjusted accordingly. Several iterations were necessary to determine adequate column designs.

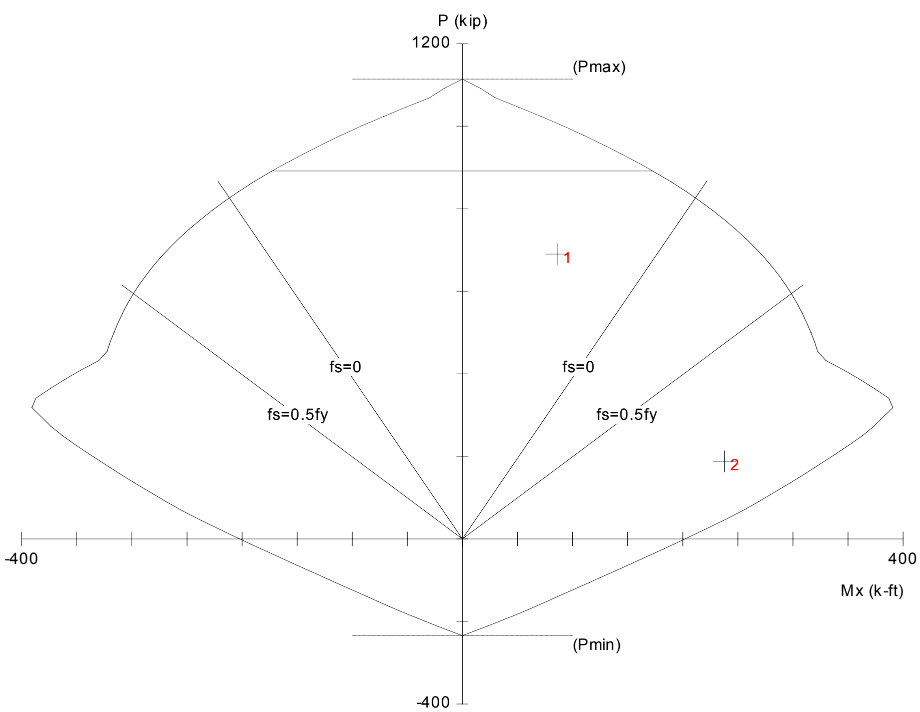
Slenderness Effects

If a column's cross-sectional dimensions are small compared with its length, slenderness effects may need to be considered. Slender columns have a significantly lower capacity due to buckling failure and P-delta effects. Fortunately, slenderness effects could be neglected, even for the smallest (14"x14") columns based on chapter 10 provisions of the ACI code.

Interaction Diagrams

The capacity of a concrete column is based on the interaction between axial compression and bending moment about a given axis. This interaction is represented as a curve, with a given column being sufficient if the compression force, P and bending moment, M falls inside the curve. Biaxial bending produces a three-dimensional curve, and the interaction can be shown by taking a horizontal section of the interaction diagram at a given P value.



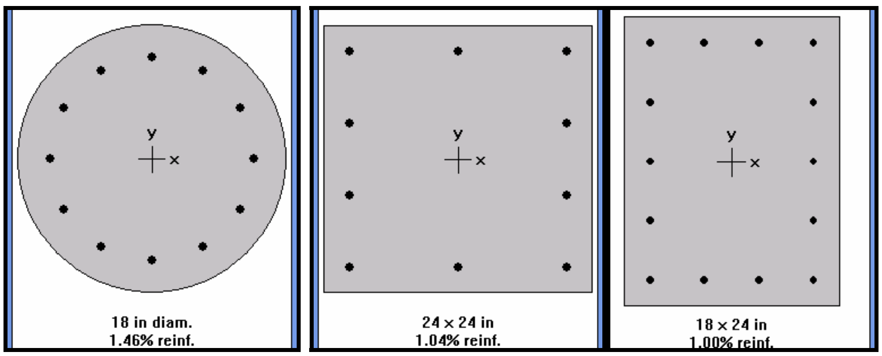


Examples of Interaction Diagrams Used in Column Design (PCA Column)

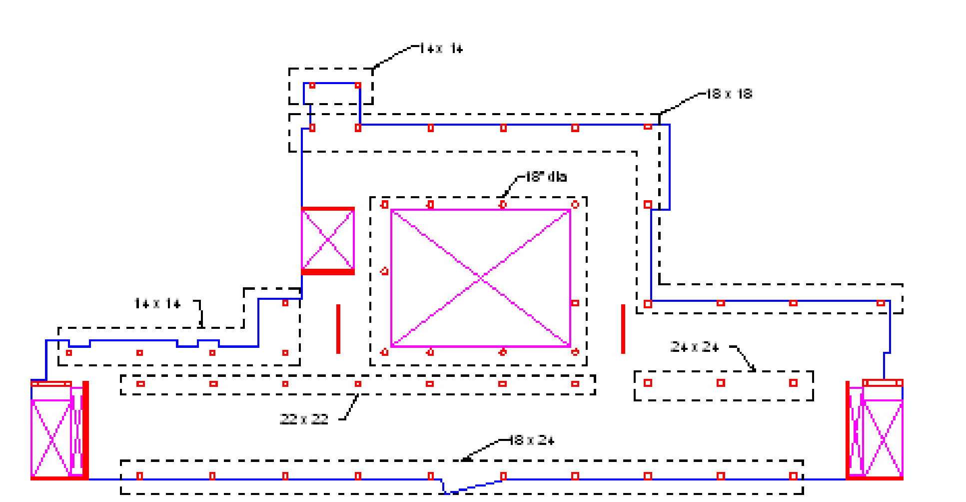
Column Sizes & Reinforcing

For ease of constructability, 6 different column designs were used, and column dimensions are continuous from the base to the top of the structure. #3 ties are used in all columns. The column designs are as follows:

Size	Reinforcing
24"x24"	(10) # 7
22"x22"	(16) # 5
18"x24"	(14) # 5
18"x18"	(8) # 6
14"x14"	(8) # 5
18" Φ	(12) # 5



The reason for rectangular columns at the west face of the building is because the moment was significantly greater in one direction than the other. By using a rectangular column, the distance between the compression face of the column and the tension steel is increased without increasing the concrete area, thus making a more effective section.



Column Sizes

Punching Shear

The most common type of catastrophic failure for concrete structures is from punching shear. Punching shear occurs in concrete slabs when the high shear forces around a column exceed the shear capacity of the slab, causing the slab to tear a hole around the column, often leading to progressive collapse as one floor falls onto another.

The critical section for punching shear occurs at a distance d/2 from the column face, where d is the distance from the compression face of the slab to the center of the tension reinforcement. Because of this, punching shear tends to be most critical at edge and corner columns, where the critical section cannot continue around all sides of the column.

Because of the critical nature of punching shear, all columns were checked by hand in addition to the punching shear checks performed by RAM Concept. At typical floors, all columns along the west face of building, as well as the corner column N4 were found to need shear reinforcement to resist punching shear. At the mechanical penthouse floor level, the slab also failed in punching shear initially due to higher design loads, as well as column reactions from the high roof that do not line up with columns below.

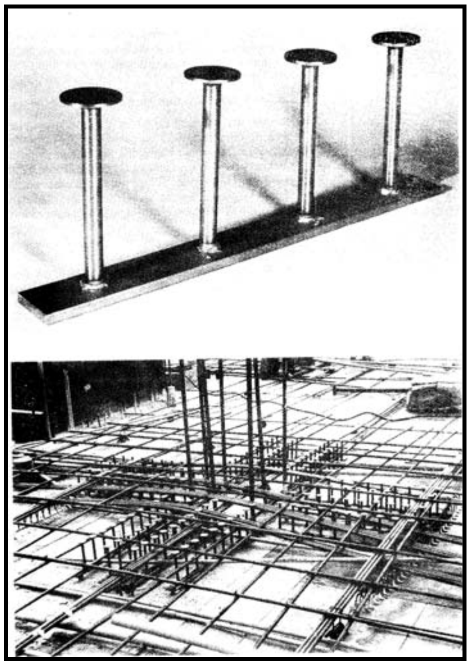
There are several ways to resist punching shear. These include using concrete with higher design strengths, increasing slab thickness or column dimensions, use of drop panels around columns, and a variety of steel reinforcing methods. Many of the steel methods, such as shearheads and integral beams with vertical stirrups, use a large area of steel and are often difficult to place with the reinforcing steel needed for design moments.

All of these methods create significant cost increases for the building. Consideration of several methods is often necessary to determine the most economic choice for the situation.

For BMEO, increasing the slab thickness was not reasonable because only a few of the columns were found to be critical. Also, increasing column dimensions would be uneconomical, as a large amount of additional steel reinforcing would be necessary and the columns would tend to push the limits for the architectural layout.

To resist punching shear, two methods were considered. Drop panels, extending 4” below the bottom of the slab were found to be sufficient in all cases. When analyzing drop panels, it was also necessary to consider the critical section at the edge of the panel where the concrete slab thickness changes.

Also, shear stud reinforcing strips (SSR) were considered. These consist of a series of shear studs welded to a steel plate. Typically, several strips are arranged in the two orthogonal directions. Shear stud reinforcing tends to be more efficient and easily constructible than other steel reinforcing methods for shear. ½” diameter SSR was designed for the critical columns, with the strips ranging from 6 studs @ 3.75” spacing to 12 studs @ 3.25” spacing.



After considering the two choices, drop panels were chosen as the final design. Although additional formwork is necessary, it was estimated that it would still be more economical than SSR for this building.

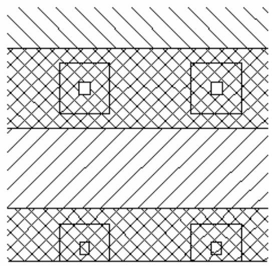
The drop panels work twofold. In addition to resisting punching shear, this additional area of concrete increases the effective stiffness of the columns, thus reducing the required reinforcement in the slabs. Because columns were compression controlled, the additional moments distributed to them from increased stiffness did not change their design.

Design Challenges

Mechanical Penthouse Floor Slab

The mechanical penthouse floor had significantly higher loads than the typical floor. In addition, many of the columns supporting the penthouse roof do not line up with columns below. To account for these loads, the floor slab at this level had to be increased to 12".

Column reactions from the high roof were relatively small. Though some did not line up with columns below, all were at least located in the column strips, where steel reinforcing is concentrated. Because of this, the flexural capacity of the slab was sufficient, with the reinforcing noted below.



	Top, Ext Col	Top, Int Col	Bottom
Column Strip	(14) #5	(20) #5	(12) #4
1/2 Middle Strip	(4) #5	(4) #5	(8) #4

While the point loads could be resisted in flexure, they caused large shear forces at the columns. As mentioned earlier, the final gravity design called for drop panels extending 4" below the slab (16" total depth for the penthouse level), which were sufficient to resist these high shear forces.

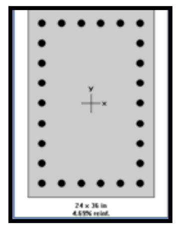
Transfer Girders: Lecture Hall

At the first floor of BMEO, there is a large lecture hall at the east side of the building. This lecture hall required an open, column free space so as not to interrupt sight lines from the seating areas. This required a wider column layout at the first floor than the other floors. Therefore, transfer girders were needed to transfer the high forces from the five floors above to the first floor columns. The columns and girders were analyzed as simple concrete moment with column reactions (forces and moments) applied. RAM Advance was used to simplify the moment distribution. As with all concrete design, this process was iterative, with initial member sizes estimated and adjusted accordingly.

For the lecture hall, three transfer girder frames were needed. Two were identical, while third (Col. Line J) having much larger forces based on geometry. The final designs are as follows:

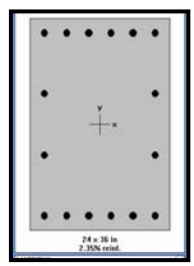
Col. Line J

Beam	Design Positive Moment	1530	ft-k
	Design Negative Moment	1750	ft-k
	Size	***24x36***	d = 32.5"
	Positive Reinf	***(11) #10***	Top
	Negative Reinf	***(10) #10***	Bottom
Columns	Design Moment	2660	ft-k
	Design Compression	675	k
	Size	**24x36**	
	Reinf	**(26) #11**	



Col. Lines G & H

Beam	Design Positive Moment	1210	ft-k
	Design Negative Moment	1300	ft-k
	Size	***24x36***	d = 32.5"
	Positive Reinf	***(10) #9***	Top
	Negative Reinf	***(10) #9***	Bottom
Columns	Design Moment	1722	ft-k
	Design Compression	270	k
	Size	**24x36**	
	Reinf	**(16) #10**	

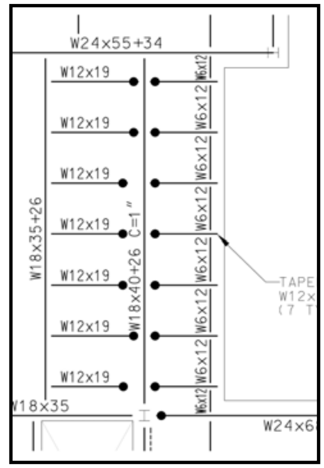


These members, as expected, were large in size and required a large amount of steel reinforcing. In concrete, moments are transferred from beams to columns, causing the columns to be much bigger than they would need to be for the compressive forces alone. The transfer girders at the lecture hall is an area where the original steel system may have an advantage. Large concrete members of this nature are much more labor intensive than steel girders because of the large amount of formwork and reinforcing steel, as well as detailing requirements.

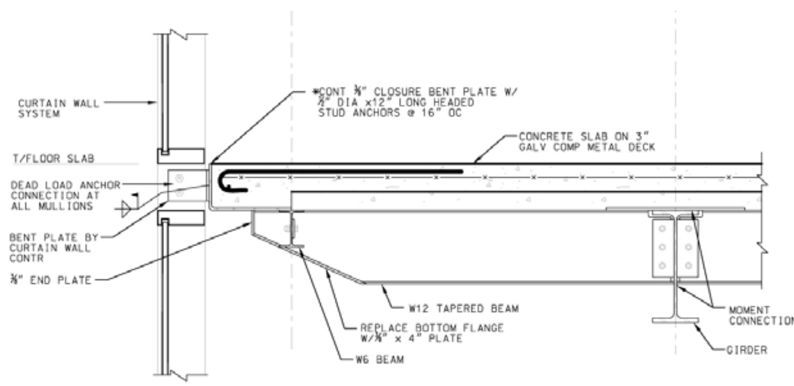
Concrete Design Advantages

There were several critical areas in the existing steel system in which the concrete system seemed to have significant advantages. Two of them are outlined below:

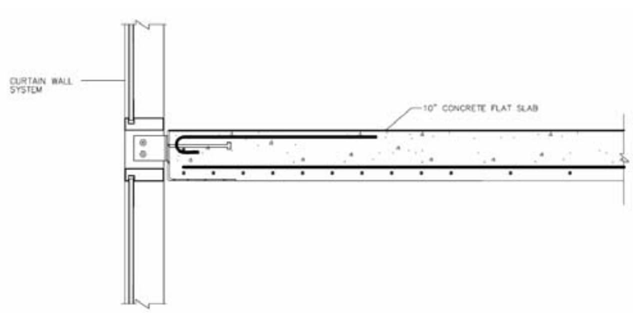
Glass Curtain Wall



The main entrance of the building uses a glass curtain wall façade. The steel framing of the upper floors would have been a simple bay design, with a girder at the face of the building. However, the large depth of the system at the building face (7.5" slab plus 18" girder) would not be aesthetically pleasing through the glass façade. Instead, the building's designers moved the girder 5' inward, and framed out the floor to the façade using tapered W12 beams with moment connections. This worked well, but necessitated the use of additional steel: (14) W12 members, (8) W6 members, (14) moment connections, and (14) steel plates welded to members *at every floor*. In addition, the beams needed to be tapered and painted, further increasing costs.



The concrete redesign, on the other hand, required no additional considerations at the curtain wall. The slab is 10" thick, easily masked by the aluminum framing of the façade.



Isolation Joint with Wilmot Hall

Since BMEO is adjacent to Wilmot Hall on two sides, isolation joints were needed to separate the two buildings, for both fire rating and lateral drift. These were formed by cantilevering the framing in BMEO out from the columns to the required distance at each floor. Similar to the curtain wall issue, this required several additional steel members and moment connections at every floor.

In a concrete system, there is inherent moment capacity between the slab and columns. This allows reasonably small cantilevers to be possible without additional considerations other than reinforcement detailing. For the isolation joints, extra diagonal reinforcing was required at reentrant corners, and bent bars were needed at the end of cantilevers. The effects of these details are minimal compared to the amount of additional steel needed in the existing system.

Lateral System: Concrete Shear Walls

When redesigning BMEO as a concrete structure, the existing design for lateral stability needed to be reconsidered. The redesign utilizes reinforced concrete shear walls in both directions, as opposed to steel moment frames in the North-South direction and concentrically braced frames in the East-West direction. The new shear wall design follows the guidelines set forth in ASCE 7-05 and ACI 318-02. The building's location in Rochester, NY dictates a relatively straightforward procedure for determining lateral forces.

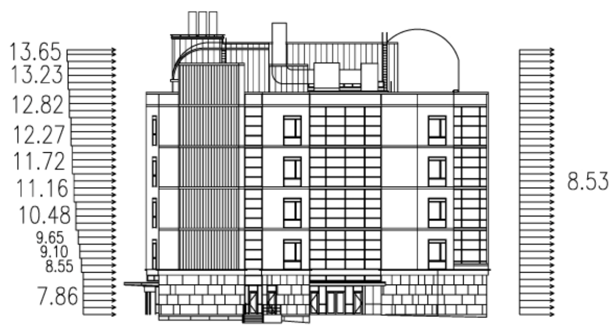
It should be noted that in concrete structures, the columns are cast monolithically with the floor slab. This means that the columns carry moment, and the floor system has potential of acting as a series of moment frames to resist lateral load. For this redesign, however, column/beam-slab frames were not designed to resist lateral forces. Thus, shear walls act as the main lateral system and were designed to resist the entire lateral load in each direction. This practice is common in many concrete structures where lateral loads are reasonable. In high seismic regions or extremely tall buildings where wind loads are excessive, concrete structures can be designed as dual systems, with the concrete frames and shear walls acting together to limit drift.

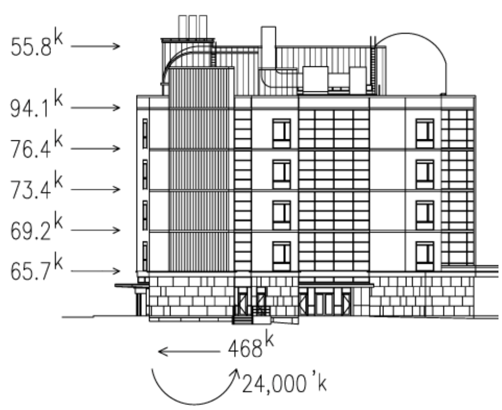
Wind Load

The wind forces on BMEO were calculated using the analytical method in Chapter 6 of ASCE 7. They are based on a design wind speed of 90 mph in Exposure Category B. Wind forces were found in control in the East-West direction of the building, where the long building dimension provides a large surface area for wind forces.

<u>Wind Load</u> *(Analytical Method, ASCE 7-05)*

Basic Wind Speed	**V = 90 mph**
Importance Factor	**I = 1.15**
Exposure Category	**B**
Building Height	**h = 95'**
Building Classification	**Rigid, Enclosed**
Directionality Factor	**K_d = 0.85**
Gust Effect Factor	**G = 0.85** (approximated)
Internal Pressure Coeff.	**GC_{pi} = ± 0.18**
External Pressure Coeff.	**C_p = 0.8** Windward
	C_p = - 0.5 Leeward (E-W)





Seismic Load

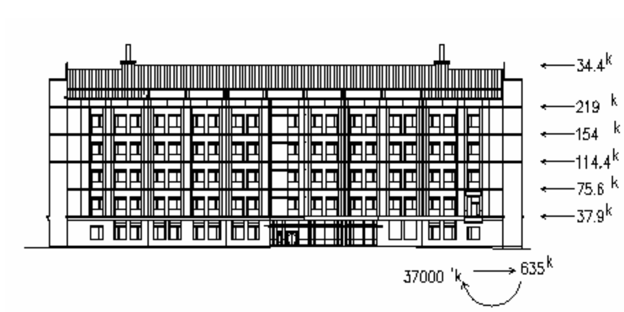
Seismic loads on BMEO were calculated using the Equivalent Lateral Force Procedure in Ch. 12 of ASCE 7-05. It is important to note that this building is in Seismic Design Category B, which allows for several design simplifications in the code. Structural irregularities, torsional amplification, and redundancy can all be neglected for this low seismic region, to name a few. Ordinary reinforced concrete shear walls are allowed, with no height restriction.

While wind loads remained constant when changing to a concrete structure, seismic loads changed considerably. Generally, seismic effects are greater for heavier structures. The redesign of BMEO as a concrete structure increased its equivalent seismic weight by 33%. However, the seismic load on this building actually *decreased* significantly for the North-South direction despite the increase in weight.

Seismic effects on a building are a result of ground accelerations and vibrations. In reality, this creates a dynamic response of the building, which is difficult to predict and design for. The provisions of ASCE 7 permit the use of the Equivalent Lateral Force Procedure, which simplifies the effects of ground motion as a series of forces acting at each story of the building. The equivalent base shear for a building using this method is the product of the equivalent seismic weight, W and a seismic response coefficient, C_s . Because of the use of shear walls, this coefficient decreased dramatically as a result of an increased fundamental period and response modification factor, as shown below.

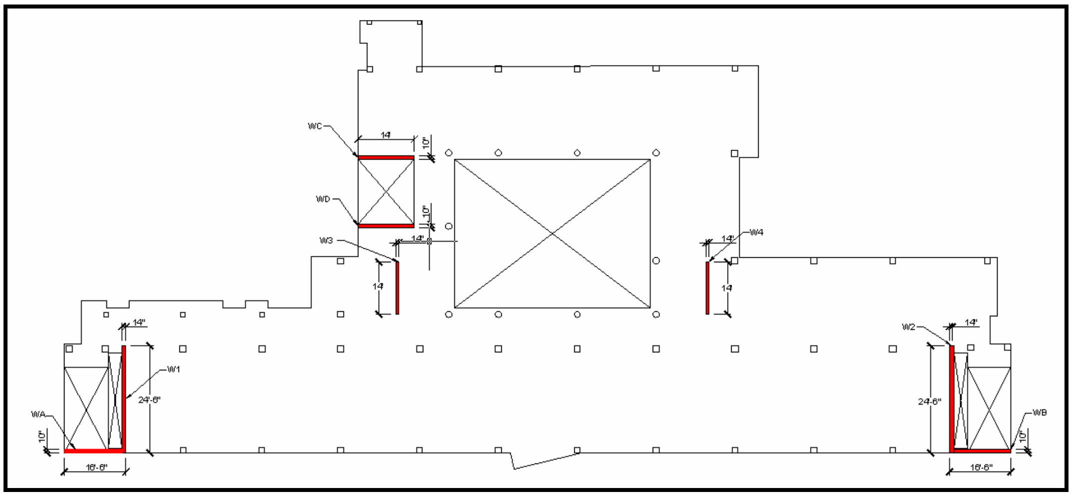
Steel Moment Frames	**Reinforced Concrete Shear Walls**	**Code Reference**
R = 3.5	R = 5	Table 12.2-1
Ta = 0.61	Ta = 1.21	12.8.2.1
Cs = 0.066	*Cs = 0.02*	12.8.1.1
W = 12,000 kips	*W = 16,300 kips*	12.7.2
V = 800 kips	**V = 326 kips**	12.8.1

Despite this dramatic decrease, seismic forces still controlled in the North-South direction of the building. Story forces are shown below.



Shear Wall Layout

The layout of the shear walls was a bit challenging for this unique structure. Four shear walls, 10” thick, were placed at the elevator shaft and at the outer face of stairwells for the North-South direction. In the other direction, two shear walls were placed at large mechanical penetrations at the buildings extremities, with the other two next to mechanical/electrical spaces closer to the atrium space. These four shear walls are 14” thick. All concrete was designed as 4000 psi, with 60 ksi reinforcing steel. This shear wall layout worked well with the building, limiting torsion while leaving the architecture uninterrupted.



Lateral Force Distribution to Shear Walls

After the design story forces were determined, they were distributed to the shear walls based on relative rigidities. Extremely detailed lateral force distributions for buildings can be found with computer programs such as ETABS. They are powerful tools that allow designers to determine accurate distributions with complicated lateral loading.

However, the BME/Optics Building is located in a low Seismic Design Category with no special considerations for wind loading. Additionally, the shear wall geometries are relatively simple, with all walls in a given direction having the same thickness. Because of the straightforward application of static forces to approximate seismic loading and the simple geometry of the shear walls, hand calculations to determine lateral distribution was deemed acceptable.

Rigidity is defined as the inverse of deflection. The relative rigidities of the walls were found at a given story by assuming a fixity at each end of the wall. The deflection of a fixed wall is given as:

$$\Delta = \frac{V}{bE} \times \left[\left(\frac{h}{d} \right)^3 + 3\left(\frac{h}{d} \right) \right]$$

Taking the inverse of the deflection of each wall under a set point load of V = 1 kip provided a relative rigidity for each wall, which was used in calculating the percentage of the total force acting on each wall.

	h	d	h/d	(h/d)^3	3(h/d)	b	Delta	k	% Load
1	14.67	26.5	0.55	0.17	1.66	14	0.131	**7.65**	***35.06***
2	14.67	26.5	0.55	0.17	1.66	14	0.131	**7.65**	***35.06***
3	14.67	14	1.05	1.15	3.14	14	0.307	**3.26**	***14.94***
4	14.67	14	1.05	1.15	3.14	14	0.307	**3.26**	***14.94***
A	14.67	16.5	0.89	0.70	2.67	10	0.337	**2.97**	***28.01***
B	14.67	16.5	0.89	0.70	2.67	10	0.337	**2.97**	***28.01***
C	14.67	14	1.05	1.15	3.14	10	0.429	**2.33**	***21.99***
D	14.67	14	1.05	1.15	3.14	10	0.429	**2.33**	***21.99***

Torsion

After determining the direct shear on each wall at each story level, torsional effects were considered. Torsion occurs when the resultant of a lateral force acts eccentrically from the center of rigidity of the structure. The twisting of the building caused by torsion increases the forces on the lateral force resisting elements.

The layout of the shear walls provided a center of rigidity that was very close to the center of mass. (e_x = 2.8’, e_y = 8.2’). For seismic forces, 5% of the overall building dimension was added to the eccentricity based on ASCE 7 criteria.

When looking at wind forces, there was a little more eccentricity between the wind force resultants and the center of rigidity. (e_x = 3.1’, e_y = 19.7’). This dictated calculations for wind torsion as well.

Because neither load case was dramatically greater than the other for a given direction, torsional forces were added to direct shear for each wall for both wind and seismic loading in both building directions. After analyzing the data and designing walls, it was confirmed that seismic loads control in the North-South direction and wind loads control in the East-West direction, as originally expected.

Shear Wall Design

Shear wall design, like most other aspects of structural design, is an iterative process. Estimations for wall layouts and geometries, as well as concrete strengths, were used initially. After analysis, the walls could be adjusted, causing changes in rigidities and thus design forces.

To determine if a shear wall will work, its design shear capacity is compared to the base shear from lateral loads. ACI code provides this capacity as:

$$V_c = 2\sqrt{f'_c}hd$$

Although horizontal shear steel reinforcement contributes the design shear capacity, the walls for this building were designed to have adequate shear capacity in the concrete alone.

The minimum reinforcing ratio for shear walls for both horizontal and vertical bars is 0.0025, dictating 2 layers of #5 bars @ 18" spacing each way.

Drift / Deflection

The total deflection at the top of a concrete wall under lateral loads is a combination of shear and bending deflection. Generally speaking, the more tall and slender a shear wall is, the greater the percentage of the total deflection due to bending. Based on virtual work, equations can be generated for the shear and bending deflections at the top of a shear wall. The total deflection, naturally, is the sum of the two.

$$\Lambda_{SHEAR} = \frac{6}{5} \times \frac{1}{2} \times \frac{VH}{A_w G} \qquad \Delta_{BENDING} = \frac{wH^4}{8EI_w}$$

It should be noted that these equations are approximate, and can be inaccurate in bending moment deflection when loads are not very evenly distributed throughout the height of the wall.

As a rough check, the shear wall deflections were calculated using these equations. The deflections calculated were less than 1" in both directions, well within the h/400 ratio commonly used in practice as an acceptable limitation.

Concrete Redesign Summary

Floor System

Floor System:	Two-Way Flat Slab
Design Procedure:	Finite Element – RAM Concept Equivalent Frame – PCA Slab & Hand calculations
Concrete:	4000 psi, Normal weight
Steel:	60 ksi
Slab Thickness:	10" Typical Floor 12" Penthouse Floor
Reinforcing:	Varies Typically #5 top, #4 bottom Distributed in column and middle strips per ACI code.
Drop panels:	Col Line 1 (West face of building) & Col N.5-4 (Corner) Add'l panels @ penthouse floor, Col Line 2 4" projection below floor slab
Deflection:	< 1"

Columns

Type:	Reinforced Concrete, Gravity only	
Design Procedure:	Compression + Biaxial bending PCA Column	
Concrete:	4000 psi, Normal weight	
Steel:	60 ksi	
Slenderness Effects:	Neglected per ACI code	
Sizes & Reinforcing:	24"x24"	(10) # 7
	22"x22"	(16) # 5
	18"x24"	(14) # 5
	18"x18"	(8) # 6
	14"x14"	(8) # 5
	18" Φ	(12) # 5
	#3 Ties	

Transfer Girders

Location:	(3) @ Second floor, above lecture hall	
Design Procedure:	Hand calculations	
Concrete:	4000 psi, Normal weight	
Steel:	60 ksi	
Girder Sizes & Reinforcing:	(1) 24 x 36	(11) #10 Top (10) #10 Bottom
	(2) 24 x 36	(10) #9 Top & Bottom
Column Size & Reinforcing:	(2) 24 x 36	(26) # 11
	(4) 24 x 36	(16) # 10

Lateral System

Type:	Reinforced Concrete Shear Walls
Design Procedure:	Hand Calculations Force Distribution: Relative rigidities Deflection: Virtual work equations
Concrete:	4000 psi, Normal weight
Steel:	60 ksi
Sizes:	(4) 14" thick, East-West (4) 10" thick, North-South
Minimum Reinforcement:	(2) #5 @ 12" each way
Deflection:	<1"

Breadth #1: Cost & Scheduling Implications

Cost Comparison

The new Biomedical Engineering and Optics Building at the University of Rochester will be a state of the art facility, serving as an architectural representation of the world class Institute of Optics and up-and-coming Biomedical Engineering Department. It is a unique building, with several distinguishing architectural features.

In buildings of this nature, bottom lines and cost-cutting techniques are not as critical as in office buildings or apartment buildings designed to create revenue from rental spaces. Campus buildings are funded by donations and university budget rather than private owners. The main design goal is to create a structure of the best quality that will last for decades to come. That being said, it is still the engineers' responsibility to keep a balance between quality, constructability, and economy.

The existing steel design efficiently meets all of the structural challenges of the building. As outlined earlier, no advantages in design could be found that would significantly reduce the cost of the building as a steel structure. However, there is a large amount of steel in this building for this size. The five-story building has almost 1300 framing members weighing almost 300 tons, with over 6000 shear studs and over 400 moment connections.

A few sections of the building were mentioned earlier in this report that would economically favor a concrete structure. A cost estimate of the concrete superstructure was performed to compare to the redesign with the original steel structure.

Estimates include both cost and labor using RS Means, and actual obtained cost data where applicable. The two designs will be compared in 2005 dollars, the year the original design was completed.

<u>Original Composite Steel Design</u>

Structural Steel (Obtained from LeChase Construction)	$2,403,000
Concrete Slab on Deck w/ WWF Reinforcing (Estimated)	$450,000

Total Superstructure Cost: $2.85 Million

Cast-in-Place Concrete Redesign

Formwork		$785,000
	Floor Slabs - 75,000 sq ft	
	Columns – 16,600 sq ft	
	Shear Walls – 22,500 sq ft	
Concrete		$372,000
	Floor slabs – 2500 cu yds	
	Columns – 243 cu yds	
	Shear Walls – 404 cu yds	
Reinforcing Steel		$242,000
	132 Tons	
	(w/ 20% increase to account for ties, hooked and bent bars, etc.)	
	Total:	$1.4 Million

After adjustments for city index,
Total Concrete Superstructure Cost: $1.5 Million

The difference is cost between the two structures n this estimate is significant, over $1 million. This amounts to almost 4 % of the total project cost of $37.7 million.

It should be noted that this is just a rough estimate, and the actual savings would not be this dramatic. The cost data for the steel system is based on actual information, while the concrete cost is estimated from RS Means. Therefore, the steel cost information includes all detailing requirements, including façade anchorage, canopy framing, mechanical penthouse roof deck, etc. Also, the concrete system is heavier, necessitating additional steel piles. This estimate may not represent cost savings with precise accuracy, but still shows the general idea. It is difficult to justify a savings of $1 million dollars, but the concrete system would definitely result in significant cost savings.

Scheduling Issues

Similar to cost, the duration of construction is not as big of an issue for this type of building than for office or apartment buildings. In those types of buildings, construction is often fast-tracked to finish the project as quickly as possible. The goal of a speedy construction is to open the building sooner so that rental space can begin to generate revenue.

For BMEO, speed of construction was not the main priority, as the purpose of the building is not for the economic gains of the owner. However, there was an original goal of completing the project in January of 2007 so that it could be open for use for the spring semester. However, several setbacks pushed back the construction completion date, and the building will not be used extensively until the fall semester.

A concrete redesign would inevitably increase the construction time of this project due to formwork labor, large amounts of steel reinforcing and the special needs for cold weather construction. However, the long lead time for structural steel is reduced, making it possible to start construction earlier.

As stated, schedule increases would not affect the use of the building unless it proved to take about five months longer to build the steel system such that the building could not open for the fall 2007 semester.

The implications of a schedule increase in the structural system is difficult to quantify in terms of the overall building schedule. Efficiency in construction of a building is a result of collaboration between different trades, good management, restrictions of the site, and availability of materials and laborers. For investigation, a crew size was selected from RS Means data to give a rough idea of how long construction of the concrete structure would take. To limit site congestion, only one of each crew type was considered for each task

	Crew	Daily Output	Labor Hours	Duration
Columns	C-14A	16 CY	2900	18
Shear Walls	C-14A	20 CY	4000	20
Slab	C-14B	50 Cy	10000	50
				88

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Crew C-14A

- 1 Carpenter Foreman
- 16 Carpenters
- 4 Rodmen
- 2 Laborers
- 1 Cement Finisher
- 1 Equip Operator
- 1 Gas Engine Vibrator
- 1 Concrete Pump

Crew C-14B similar, with 1 extra cement finisher.

The concrete structure would take roughly 18 weeks to complete with the crew sizes selected. This would result in about 17,000 total labor hours. Construction could take significantly longer than this if curing time and cold weather dictate a lot of off-time. As stated, the effects of this on the total project construction, which lasted from March 2005 until April 2007.

Breadth Topic #2: Green Building Design (ETFE Foil Cushion Roof)

Building Green

In recent history, there has been a significant push toward "green" technology. As pollution continues, costs of fossil fuels increase due to declining supply, and environmental threats such as global warming have been studied, there has been a steadily increasing demand for environmentally friendly technology. Recent advances in renewable energy sources and technologies such as hybrid cars aim to lessen human impact on the environment.

Similarly, there has been an increasing demand for environmentally friendly and sustainable architecture. Though the perception of the term "green building" varies, four main goals can be used embody the concept of green architecture:

- *Reduction in Energy Use*
  Ex: Natural ventilation, low energy electrical appliances, use of solar energy
- *Minimizing Environmental Impact*
  Ex: Recycling storm and waste water, minimizing pollutants
- *Reducing Embodied Energy and Resource Depletion*
  Ex: Local, recycled, or sustainable materials
- *Minimizing Internal Pollution and Health Risks*
  Ex: Adequate ventilation, non-toxic and low-emission materials

It is important to realize that the concept of green building design should be a holistic approach. Green buildings, in the truest sense, are designed to be environmentally friendly from the initial design phases. The impact of every aspect of the building's design, from the site layout to the materials, should be considered.

Several aspects of the BME/Optics Building show an environmentally friendly aim in design. For the purposes of this analysis, BMEO will not be completely redesigned as a green building. Instead, one innovative technology, an ETFE foil cushion roof, will be analyzed and applied.

LEED Rating

Recently, the U.S. Green Building Council (USGBC) created a rating system to quantify environmentally friendly buildings. This system, called LEED (Leadership in Energy and Environmental Design) is a point system based on a variety of building aspects including materials, site considerations, and any innovative designs. When a building applies for a LEED rating, it is given a score that relates to different categories- LEED Certified, Silver, Gold, or Platinum. This rating system is widely accepted as the measure of green building design.

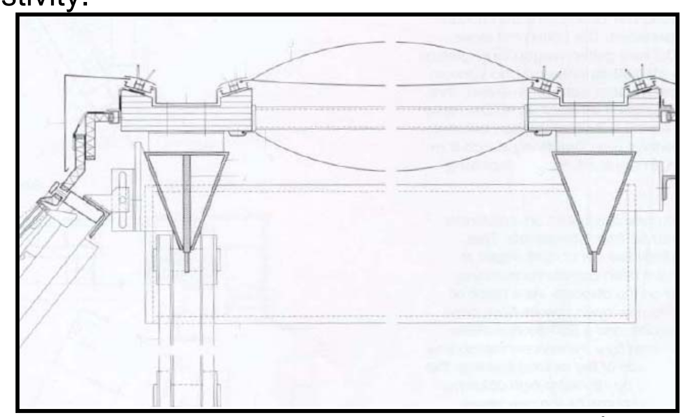
The University of Rochester always aims to apply environmentally friendly design practice to their new building designs where they can. However, the university does not currently apply for LEED rating.

Rather than listing the different ways to achieve LEED points, the focus of this section will be to describe one innovative technology, EFTE foil cushion roofing, that embodies the idea of green architecture. It should be noted that this roof system has not been used widely in the United States, and probably would not have been feasible at the time of BMEO's design due to availability and constructability. This report will present the use of foil cushion roofing in BMEO as an example of a recent technology that may have an impact in the United States in the future.

ETFE Foil Cushion Roofing

Ethylenetetraflouroethylene, or EFTE, is a transparent polymer with a high corrosion resistance and a high strength over a broad temperature range. Extensive testing shows that it is unaffected by UV rays, pollutants, and weathering. Only a few companies worldwide currently manufacture EFTE, under the brand names Tefzel, Fluon, and Texlon.

To form the foil cushion roofing system, thin layers of EFTE foil are extruded into an aluminum perimeter supported by the buildings frame. The EFTE is inflated pneumatically to about 0.03 psi, forming cushions that give the system strength and thermal resistivity.



www.architen.com

Section of Foil Cushion – Note the three layers of EFTE foil

EFTE foil cushion systems have been used in several projects for a variety of applications. In Munich, the Allianz Arena utilizes the transparency of the cushion system at night, when the entire stadium lights up with team colors. The largest application of EFTE is in Cornwall, England where two giant geodesic domes serve as an environmental facility called the Eden Project. In more practical applications, foil cushion roofs are used for entrance canopies and roofs where natural light is desired.

en.wikipedia.org/wiki/Allianz_Arena

www.gooseham-barton.com

www.les-stclair.co.uk

http://www.covertex.de

Examples of EFTE Foil Cushion Applications

Similar Case Study: Munich Office Building, Atrium Roof

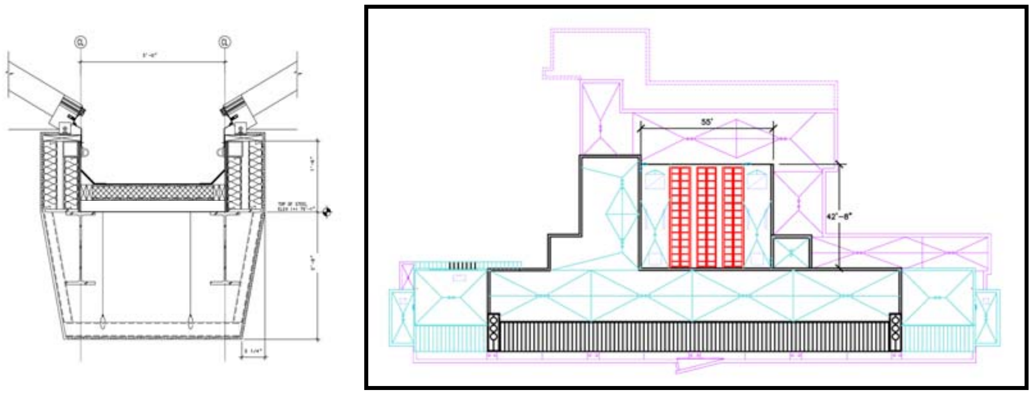
Through research, a building was found that has similar conditions to those in BMEO. An administration building in Munich, Germany, constructed in 2002, was investigated as an example of ETFE foil cushion application for an atrium roof. The atrium in this building is approximately 1300 sq ft. It used five sections of arched roof, each consisting of three-layered, air-supported foil cushions. Small rolled steel shapes in the shape of the desired arch support these cushions. It should be noted that the calculated area dead weight is less than 0.2 lb/sq ft.

One of the five roof panels for this building was designed as a movable element, shown below. Thus, the roof system can also provide ventilation to release heat or smoke in the event of a fire. At the peak of the arch, the two outer layers of foil connect to the top of the steel member, while the inner layer connects to the bottom. This 8” separation between foil layers provides for more air in the cushion, increasing its insulating qualities by about 20%.

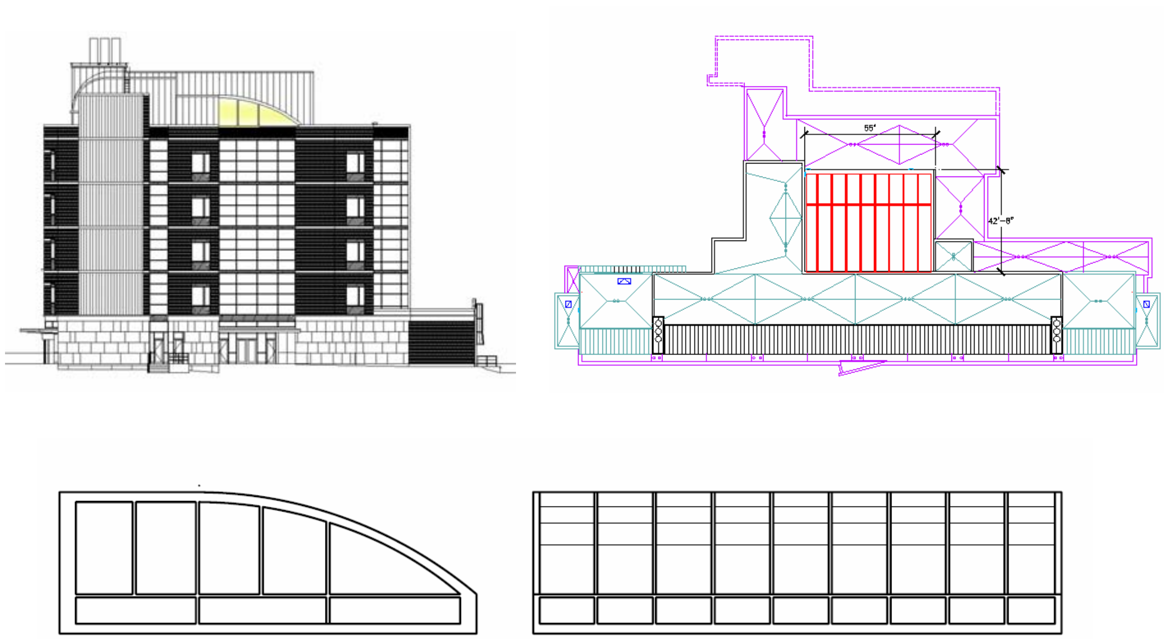
http://www.covertex.de

Design for BMEO

A possible design for an EFTE cushion foil roof for the atrium of BMEO was created for visual comparison. A completely arched roof, like that used in the Munich office building would not be desirable, because mechanical penthouse walls exist on two sides of the atrium roof, as shown below. An arched roof would create a space for snow loads to accumulate. Instead an EFTE structure which is partially flat and the rest arched is proposed as a possibility. Rough sketches of this idea are shown, as well as an elevation representing what the new atrium roof may look like from the front entrance when illuminated.



Original Roof with Skylights – Plan and Detail



New ETFE Foil Curtain Roof Idea

Advantages

- Insulation

    The three layer EFTE cushion design for BMEO has a U value of 1.96 w/m^2 K, significantly higher than the glazing in the current skylights. This helps to reduce heat loss in the cold Rochester months and thus reduce energy usage.

- Natural Lighting

    EFTE is extremely transparent, transmitting 94-97% of total light. It is transparent across the entire visible light region, leading to excellent color rendering. Also, the structural capacity allows the EFTE roof to span over the entire atrium, greatly increasing the amount of natural lighting in the atrium. This natural light reduces electricity consumption needs from artificial lighting, improves the indoor atmosphere, and can also help to heat the building.

- Weight

    The weight of the foil roof system is extremely light, almost negligible. The only loads that need to be considered on the roof system are snow and wind uplift.

- Durability and Maintenance

    Extensive testing has shown that the foil is not affected by excessive sunlight, pollution, or weathering. In addition, the material is extremely smooth. It does not attract dirt, and is self-cleaning in the rain. The exterior never needs to be cleaned or accessed

- Environmental Impact

    The amount of energy used to produce a material is often described as its embodied energy. The embodied energy rating for EFTE is about 27 MJ/m^2 , compared to glass at about 300 MJ/m^2 due to the high heat needed in the manufacturing process. Also, due to its low density, the energy required to transport it is about one tenth that of glass. There are no dangerous byproducts in the manufacturing of EFTE, as it is a copolymer of ethylene and Teflon, with a low softening heat for manufacturing.

Conclusions

The EFTE foil cushion system is an innovative new technology that can be used in a variety of applications, and follows the principles of green building design. The advantages of these systems are countless, having an impact on the interior atmosphere of buildings, energy savings, unique architectural features, and thermal insulation. Though not widely used in the United States, EFTE may change the way roof and façade systems are designed in the future.

Recommendation

A complete redesign and analysis of a cast-in-place structural system for the new Biomedical Engineering and Optics Building at the University of Rochester has shown that this system is extremely economical and efficient, while meeting structural challenges, and unique conditions provided by the architectural and spatial layout of BMEO. In addition to cost savings, the benefits of a concrete structure include:

- Durability
- Inherent Fire Protection
- Limited Deflections
- Vibration Damping
- Quality Control

Although the construction process for a concrete structure is significantly longer, this issue is not critical due to the nature and usage of the building. Therefore, the cast-in-place concrete flat slab structural is system described in this report is recommended for the University of Rochester Biomedical Engineering / Optics Building.

Conclusions

After completing this project, the following conclusions can be made:

- Recent technologies such as finite element analysis make concrete design for more complex structures possible and more efficient

- Although more labor intensive, cast-in-place concrete can be very economical in buildings such as BMEO, that would otherwise require a large quantity of steel

- Technological advancements such as EFTE foil cushion membranes can provide architecturally unique, environmentally friendly, and energy-saving building solutions

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Calculations

For brevity, calculations have been omitted from this report. The following are available upon request:

- Design Load Calculations
- Minimum Slab Thickness Calculations
- Computer Program Outputs
  - RAM Concept
  - PCA Slab
  - PCA Column
- Full Reinforcing Plans
- Slab Spot Checks: Equivalent Frame Procedure, Moment Dist.
- Column Design Forces and Moments
- Transfer Girder Hand Calculations
- Shear Wall Design & Drift Calculations
- Takedowns for Cost Estimation

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Wayne Goodman and the University of Rochester

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