NORTHBROOK CORPORATE CENTER

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STRUCTURAL
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Northbrook Corporate Center

BUILDING NAME: Northbrook Corporate Center

BUILDING LOCATION: 1150 Northbrook Dr., Philadelphia PA

SIZE: 109,000 SF

NUMBER OF STORIES: Four (total height: 74')

OCUPANCY: Office Space

OWNER: Acorn Development Corporation

ARCHETECT: RHJ Associates

MECHANICAL/ &LIGHTING ENGINEERS: N.E. Fisher & Associates

STRUCTURAL ENGINEERS: O'Donnel & Naccarato, Inc.

CONSTRUCTION MANAGER: Norwood Company

ELECTRICAL / LIGHTING:

[2] 480/277 V, 3 phase - 4 wire systems

Mostly 2x4 Deepcell Parabolic fixtures

MECHANICAL:

Heating/cooling by fan powered air volume system.

FIRE PROTECTION:

Sprinkler System, photoelectric fire alarm system, and rescue assistance call station

STRUCTURAL / ARCHITECTURAL:

Steel columns-girders-joists-decking

Insulated flat roof- decking on steel joists

Exterior materials: brick, stone, glass

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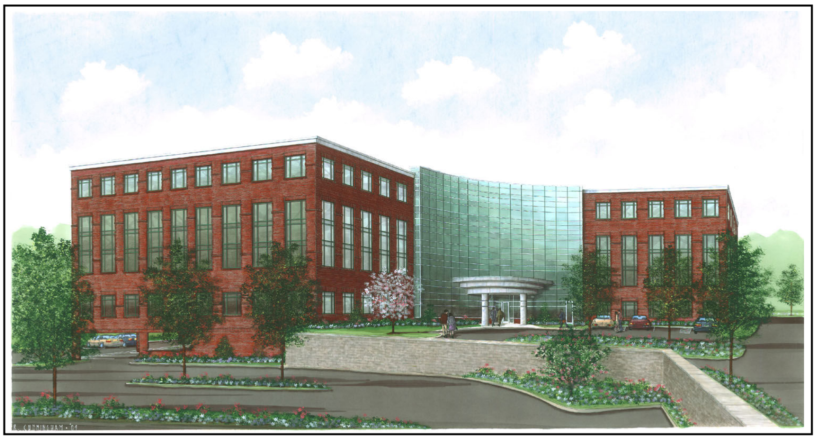
STRUCTURAL SYSTEM REDISIGN

COST ESTIMATE
ARCHITECTURAL REDESIGN

DIMITRY A. REZNIK

STRUCTURAL OPTION
ADVISOR: DR. MEMARI
NORTHBROOK CORPORATE
CENTER

12/15/2006
AE 482



EXECUTIVE SUMMARY

The Northbrook Corporate Center is a 5 story building located on 1150 Northbrook Drive, Philadelphia, PA. After its completion in the spring of 2006, the building provides roughly 104,000 square foot of usable office space. With each story being 14 feet high, the total height of the building is 74 feet. The building provides a parking garage on its lowest level.

The structural system of the building consists of steel columns, composite steel girders, and composite steel joists. Steel joist support a 4 inch concrete slab on metal deck; joists are spaced at 3 feet o.c., and span 30 feet between the girders. Steel girders, typically W24x68, are connected to steel columns, typically W12x72, with a moment resisting connection in order to resist the lateral loads.

This report provides a background description of the Northbrook Corporate Center, and provides a detailed description of the building's lateral load resisting system. In the report it is proposed that a braced frame system can be a more feasible system for the building. The redesign of the lateral resisting system is motivated by the high costs of the currently used moment frame system.

The main point of interest of this report is the redesign of the lateral force resisting system. The analysis of a load development and distribution, placement of braced frames, and the design of each individual member of the braced frame system are included in this report. This report also includes the design of additional columns, additional footings, and the redesign of affected columns, connections, and footings. Also a detailed cost estimate of both, the moment frame and the braced frame systems, and their cost comparison calculations are performed as a breadth study of this report. The results of these estimates show that the braced frame system is less expensive than the moment frame system by about $90,000. This advantage, however, is counterbalanced by the unfavorable impact the redesigned system has on the layout of the interior space. Two of the braced frames have blocked the access to the two handicap parking spaces and the main traffic path in the electrical room. To correct this problem an interior space layout of the garage level was

redesigned as a part of the architectural breadth study. The redesign of the electrical room was successful; however, one parking space was lost in the redesign of the garage layout.

There are several problems with the redesigned system. First the overturning moment of braced frames C and D is questionable. Second, the flexibility of the interior design is slightly altered. And thirdly there is a mistake in the seismic load development section of this report, which leads to more uncertainty of the accuracy of the overall design.

Because of the stated problems, this report concludes that the braced frame system is not a more feasible lateral load resisting system for the Northbrook Corporate Center.

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BILDING STATISTICS

GENERAL BUILDING DATA

Building Name: Northbrook Corporate Center
Location: 1150 Northbrook Dr., Philadelphia, PA
Occupancy Type: Office Space
Size: 109,000 square feet, not including the garage
Number of Stories above grade: 4 stories in the front, and 5 stories in the back - due to the sloping ground.
Total Height: 74 feet from garage floor to the top of the building.
Dates of Construction: Fall 2005 - Spring 2006
Project Delivery Method: Design-Bid-Build

PRIMARY PROJECT TEAM

Owner/ Developer: Acorn Development Corporation
Construction Manager: Norwood Company
Architect: RHJ Associates
Mechanical and Electrical Engineer: O'Donnel & Naccorato, Inc.
Structural Engineer: N.E. Fisher & Associates

Codes: IBC 2003

Building Envelope:
North Brook Corporate Center's exterior is composed of three different materials. All walls, with the exception of curved, main entrance wall, are decorated with red brick. The facade is defined by a curved - glass curtain wall visually supported by a row of stone wall tile at the ground level. Building has a flat roof.

Electrical and Lighting:
Northwood Corporate Center is powered by two 480/277 volt, 3 phase - 4 wire voltage systems. Typical light fixtures include 2x4 Deep cell parabolic fixtures.

Mechanical:
The building is heated and cooled by a fan powered air volume system. Air is circulated through a system of vertical and horizontal duct work.

Structural:
Frame of the building is composed of steel columns, steel girders, composite steel beams, composite steel joists, and 4 inch concrete slab on metal decking. Typical columns sizes are W12x60, W12x65 and W12x72. In most cases each column extends from the garage floor to middle of third floor where it is connects to and continued by smaller, lighter column. Typical joist (26k7) is supported by a steel girder, typically W24x68. All girder/joist to column connections are designed for a moment of 40 ft-kips to resist wind and seismic loads. All lateral loads are resisted by moment connections. The loads are transferred in this order: A four inch thick concrete slab on metal decking is connected to and held in place by steel joists; steel joists are connected to steel girders, girders are supported by steel columns, and columns stand on shallow concrete footings.

Fire Protection:
Northbrook Corporate Center is protected by sprinkler system and photoelectric smoke fire alarm system. Building also has area of rescue assistance call station located in fire protected stairwells. .

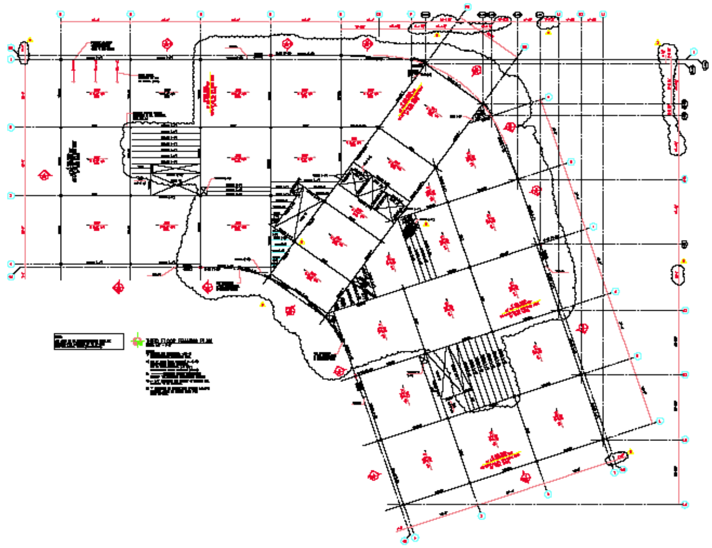
Transportation:
There are two entrances into a development in which the building is constructed. There is only one entrance from the Northbrook Drive into the building's drive way, which later subdivides and leads to parking lots. Northbrook Corporate Center has a main entrance into the building, and two large entrances into a garage. There are also several handicap side entrances on both garage and first floor level. Movement inside the building is facilitated through a system of corridors, stairwells, and elevators that connect all tenant spaces to main lobby.

Telecommunication:
Each office unit in the building is equipped with basic data jacks, cable, and telephone outlets.

EXISTING STRUCTURAL SYSTEM

LATERAL FORCE RESISTING SYSTEM

The Northbrook Corporate Center's lateral force resisting system consists entirely of moment connections. Because the building is only 74 feet high, the accumulation of wind forces is small enough for moment connections to resist. Most columns are spread 30 feet apart in each direction and rest on shallow concrete foundations. Typical column size is W12x72. Almost all columns span from the garage floor to the third story, where they are connected to and continued by a smaller column, typically W12x53. This connection is made 4 feet above the floor of the third story. Typical girder size is W24x68. The girders are connected to the columns through a moment connection, capacity of which ranges from 40 ft-k to 15 ft-k. 40 ft-k moment resistive connections are found on the first and second floors, 30 ft-k moment connections are found on the third floor, and 15 ft-k moment connections are found on the fourth floor. The girder size is satisfactory to carry all gravity loads without relying on moment connections, thus, the moment connections are used to resist only the lateral loads.



The steel joists rest on girders, and support the concrete slab on metal deck. The joists do not contribute to the lateral force resisting system, with the exception of the joists that are connected directly to a column.

The Northbrook Corporate Center is not a perfectly rectangular building; in fact, the design incorporates curved exterior walls and inconvenient angles, as shown in the drawing above. Hence, the structural layout is not entirely uniform. The bays located in the center region of the building vary in size and proportion. In some instances 'W' shaped beams are used instead of steel joists (see the drawing).

STRUCTURAL FLOOR SYSTEM

Floor systems of all the stories are almost identical. Because the basement does not take up the whole building's floor area, the first floor system is not uniform through out. First floor design incorporates four inch concrete slab on grade system in areas where the ground is not excavated. Second, third and fourth floor systems are very similar in design. The floor area is composed of composite steel joist system, where a 4 inch concrete slab on metal decking is held in place by 26K7 composite steel joists. Joists are spaced 3 feet apart center to center, and are held from both sides by composite W24X68 steel girders. Concrete is poured on 9/16" – 26 GA. UFS form deck, and is reinforced with 6x6 – W2.9xW2.9 WWF; thus, the total slab thickness is 4 inches.

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PROPOSAL

Problem Statement

The North Brook Corporate Center resists the lateral loads through the use of moment connections at every point where the joists or beam meets the column (there are exceptions). The capacities of moment connections vary from floor to floor. The story shear force of the higher floor is lower than the story shear force of the lower floors, so the capacity of moment connections changes accordingly. The moment frame lateral load resisting system creates the flexibility for the design of the interior space. By eliminating obstacles such as braced frames or shear walls, interior walls can be moved around at anytime to accommodate the needs of an occupant. This flexibility, however, is achieved at high cost. The stiffness of moment frames is extremely low compared to the stiffness of braced frames or shear walls, consequently, a large number of moment connections is needed to resist the lateral load applied on the building. The deflection of the moment frame is engineer's biggest concern. The excess movement of the moment frame will result in the cracking of the wall, floor, and ceiling finishes, and will lead to other damages in the future. To successfully resist the lateral load, the Northbrook Corporate Center incorporates 150 moment connections per story, that is a total of 750 moment connections in the building. With respect to a shear connection, a moment connection uses more steel in form of angles and bolts, and takes more time to install. These differences accumulate into a difference in the overall cost. Large number of moment connections will increase the overall cost of the building and can potentially have an impact on the schedule.

Proposed Solution

To lower the cost of lateral load resisting system the braced frame system will be designed. All moment connections will be replaced by simple shear connections and that way moment frames will be completely eliminated. Because the flexibility of the interior space design is an important feature of the Northbrook Corporate Center, braced frames will be located in the frames where walls are permanent by the architectural design. Such walls include bathroom walls, elevator shafts, and stair wells. Because of the mechanical and electrical systems that deal with these spaces, their walls are not expected to be portable. The stiffness of the braced frame is much greater than the stiffness of the moment frame; hence, there is no need for large number of braced frames. The braced frames cannot be located in the exterior wall because of the window openings, yet, when possible, they will be placed at the far ends of the building to resist the torsional forces with more ease. Without the moment connections the columns outside of braced frames are expected to decrease in size; the amount of steel, however, is not the biggest cost factor of the lateral force resisting system. In fact this change is not expected to play a big role in the money-saving. The decrease in the cost of the lateral system will be achieved by eliminating the labor costs of the moment connections. The design of the new lateral load resisting system can result in more steel because of the diagonal members in the braced frame, and/or new column lines; however, because the labor costs dominate over material costs, the net cost of the new system is expected to be much less than the cost of the original system. It is the proposal of this report, that the braced frame lateral load resisting system can successfully replace the more expensive moment frame system without eliminating the feature of flexibility of the original system.

The redesign of the lateral load resisting system will have an impact on many parts of the system. Once the moments are removed from the equation, the column sizes are expected to decrease. The new design will introduce new members such as diagonal brace members and new columns. These new members will require more connections to service beams, diagonals, and new columns. The foundation will be affected as well. Once the moments are removed from the column base, the footing size is expected to decrease. However, it is yet uncertain what kind of change will be required for the braced frame footings. New columns will also require new footings. All this will have an impact on the cost and schedule of the construction. It should be noted that the size of the girders are not expected to change because they were originally designed as simple beams. This was done so because the gravity loads were not distributed to the columns through the moment connections. The moment connections were there for the lateral load purposes only.

The overall comparison and evaluation of the two systems will be based on the cost, schedule, and flexibility advantages of the systems. In order to do this more accurately this report will study and evaluate the changes made to the columns, connections, column footings, interior layout, and changes that will result from the introduction of new members such as columns, footings, and diagonals.

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LATERAL LOAD DEVELOMPMENT

Wind and Seismic Loads

SEISMIC

Seismic design is based on ASCE7-02, Section 9.

These are the major factors used in the calculation of seismic loads:

OCUPANCY CATEGORY	II	[TABLE 1-1]
SEISMIC GROUP	I	[TABLE 9.1.3]
SITE CLASSIFICATION	D	TABLE 9.4.1.2

ACCELERATION

$S_L = 0.335$ (MAP 9.4.1.1a)

$S_2 = 0.085$ (MAP 9.4.1.1b)

SITE CLASS ADJUSTMENT

$F_A = 1.0$ (TABLE 9.4.1.2.4a)

$F_V = 1.0$ (TABLE 9.4.1.2.4b)

$S_{ms} = 1.0(0.335) = 0.335$

$S_{m1} = 1.0(0.085) = 0.085$

DESIGN SPECTURAL RESPONSE

$S_{DS} = 0.667(0.335) = 0.223$

$S_{D1} = 0.667(0.085) = 0.057$

SEISMIC DESIGN CATEGORY

B (TABLE 9.4.2.1a)

B (TABLE 9.4.2.1b)

Using the factors shown above, the seismic loads were calculated to be the following:

k=1.0	Lev	Weight	h	$w_x h_x^k$	C_{vx}	V	F_x(k)	N-S
Roof	5	650	7	49385.2	0.17628	7.67	1.3520	6.9110
Office	4	1560	5	92473.9	0.33008	18.4	6.0762	31.058
Office	3	1560	4	69255.7	0.24720	18.4	4.5506	23.260
Office	2	1560	2	46076.9	0.16447	18.4	3.0276	15.475
Office	1	1560	1	22958.7	0.08195	18.4	1.5085	7.7109
Total		6890		280150.	1	81.3	81.302	84.415

WIND

z (ft)	Kz (T 6-3)	q	P	P total
0-15	0.57	10.04	6.5862	10.51287
20	0.62	10.93	7.1700	11.09633
25	0.66	11.64	7.6358	11.56178
30	0.7	12.34	8.0950	12.02068
40	0.76	13.4	8.7904	12.71559
50	0.81	14.28	9.3676	13.29249
60	0.85	14.99	9.8334	13.75794
70	0.89	15.69	10.292	14.21684
80	0.93	16.39	10.751	14.67574

Wind design was based on ASCE7-02, Section 6. Analytical procedure outlined for method 2 was used to calculate the wind loads.

The following factors were used in the wind load calculations:

ASCE7-02 WIND

Method 2: Analytical Approach

Location data	**Value**	**Reference**	**Ch/Fg/T**
Occupancy Type	2	1.5.1	T 1-1
Importance Factor	1	6.5.5	T 6-1
Surface Roughness	B	6.5.6.2	NA
Exposure Factor	B	6.5.6.3	NA
Topographic	1	6.5.7.2	Fg 6-4
Rigid Structure: natural frequency			

Building Dimensions (ft)	**Value**	**Reference**
Height Above Base	74.33	9.5.5.3

Height Above Ground	74.33	6.3	
Horiz. Length Parallel	232.146	6.3	
Horizontal Dimension Ratio	1.187	F 6-6	
Horiz. Length Perpendicular.	195.64	6.3	

Wind Velocity (mph)	**Value**	**Reference**	**Ch/Fg/T**
Basic Wind Speed	90	6.5.4	F 6.1
Wind Directionality	0.85	6.5.4.4	T 6-4
3-sec Gust Power Law	7	6.3	T 6-2
Mean Wind Speed Factor	0.25	6.5.8.2	T 6-2
Wind Coefficient (b)	0.45	6.5.8.2	T 6-2
Wind Coefficient (z)	44.6	6.5.8.2	T 6-2
Mean Hourly Wind Speed	64.05	6.5.8.2	Eq 6-14
Height atm Boundary	1200	6.3	T 6-2
Velocity Pressure Exp.	0.90332	6.5.6.6	T 6-3

Integral Length Scale	**Value**	**Reference**	**Ch/Fg/T**
Integral Length Scale Factor	320	6.5.8.1	T 6-2
Integral Length Scale Exp.	0.333	6.5.8.1	T 6-2
Integral Length Scale, Turb.	353.796	6.5.8.1	Eq 6-7
Turbulence Intensity Factor	0.3	6.3	T 6-2
Intensity of Turbulence	0.392	6.5.8.1	Eq 6-5

Fundamental Period	**Value**	**Reference**	**Ch/Fg/T**
Period Coefficient	0.02	9.5.3.2	T
Approx. Fund. Period	0.51	9.5.3.2	Eq
Natural Frequency	1.96	6.5.8.2	1/T

Gust Effect Factor	**Value**	**Reference**	**Ch/Fg/T**
Gust Coefficient	3.4	6.5.8.2	N/A
Gust Coefficient	3.4	6.5.8.2	N/A
Background response	0.80821	6.5.8.1	Eq 6-6
Gust Factor	0.81946	6.5.8.2	Eq 6-4

Wind Pressure	**Value**	**Reference**	**Ch/Fg/T**
Velocity Pressure	17.6256	6.5.10	Eq 6-15
Volocity Pressure at z	15.99	6.5.12.2	T 6-3

External Pressure Coefficient	**Value**	**Reference**	**Ch/Fg/T**
Windward Side	0.8	6.5.11.2	F 6-6
Leeward Side	-0.3	6.5.11.2	F 6-6
Final Pressure (psf)	15.4857	P = qGCp - qGCp	

Wind and Seismic Comparison Chart

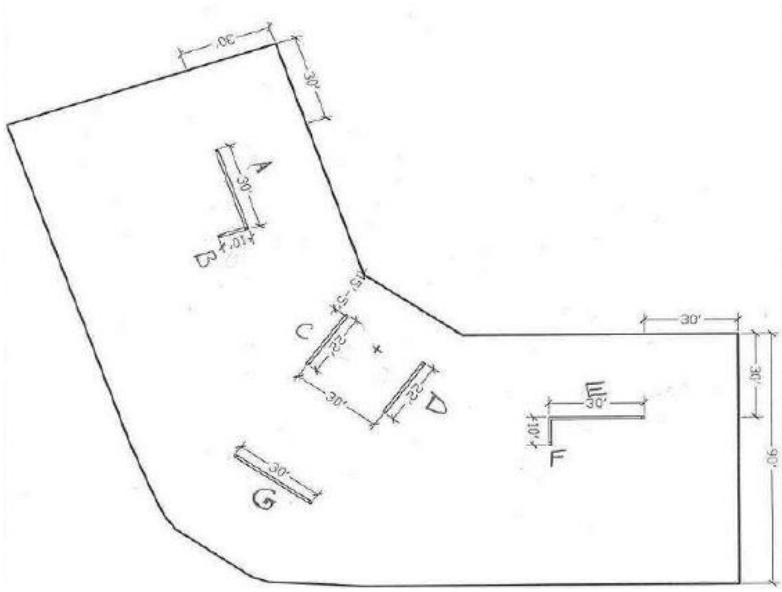
Wind k=1.005	Lev	h	Seismic (plf)	Wind (plf)
Roof	5	7	6.9	400
Office	4	5	31	421
Office	3	4	23.3	386
Office	2	2	15.5	350
Office	1	1	7.7	328
Total			84.4	1885

Wind controls the design.

NOTE: See the Lateral Load Development Correction section of this report!

BRACED FRAME LOAD DISTRIBUTION AND DESIGN

Location of Braced Frames



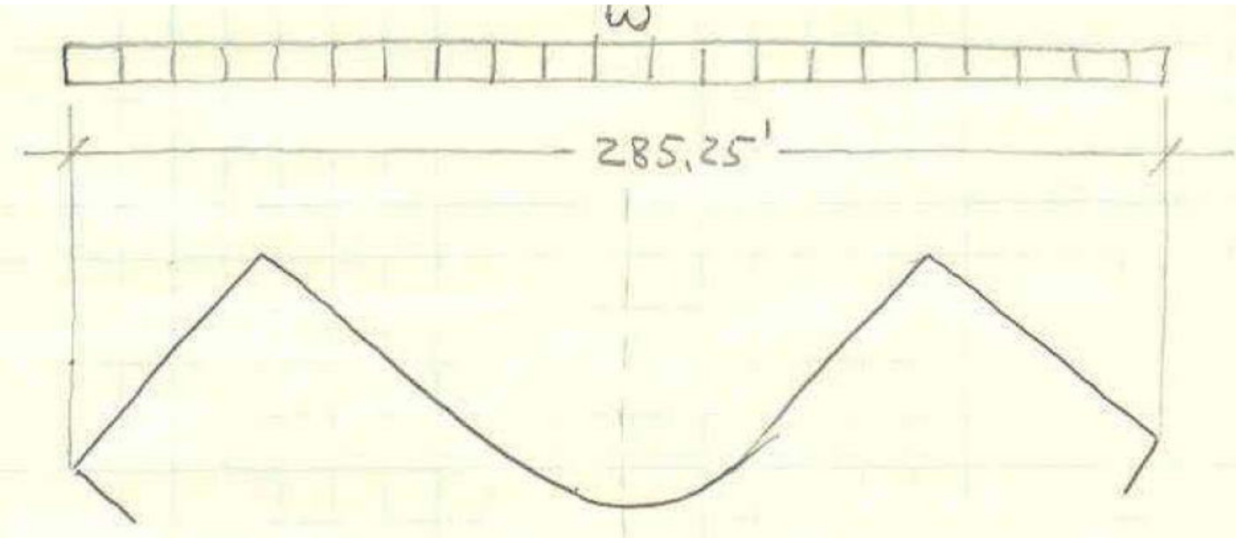
To preserve the flexibility of the moment frame system, the braced frames are designed into the frames with permanent walls. On the very far end of the building, as shown in the drawing, two braced frames wrap around the star well. The shortest wall of the stair well does not span the full length between the columns; in fact, it is only 10 feet long. In order to avoid redesigning of the interior space, and thus taking away from the flexibility of the original system, a new column is placed at the corner of the stair well - 10 feet away from the existing column. By doing so the braced frame in this direction is designed within the limits of the short wall's dimension. The longest wall of the stair well does not span the full length between the columns as well, however, the distance between the end of the wall and the following column is only 10 ft. In this case the braced frame will extend beyond the limits of the long wall to avoid the need of the additional column. This change does not substantially impact the flexibility of the interior space because the column is to close to the existing wall as it is. Due to the symmetry of the building, the design of braced frames of the both far ends of the building is identical.

The remaining braces are designed into existing frames, without the need for additional columns. The steel joists of braced frame labeled "G" are replaced by steel beams. Joists are designed to withstand flexural loads only, and thus are not practical in braced frames.

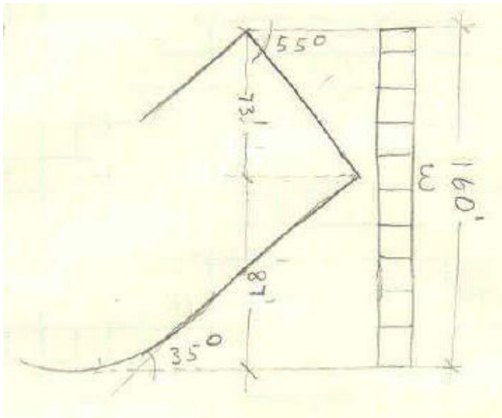
Wind Load Distribution

As shown in the lateral load calculation section of this report, the wind load is the controlling lateral load. In order to find the worst case scenario, three cases of wind loads are analyzed:

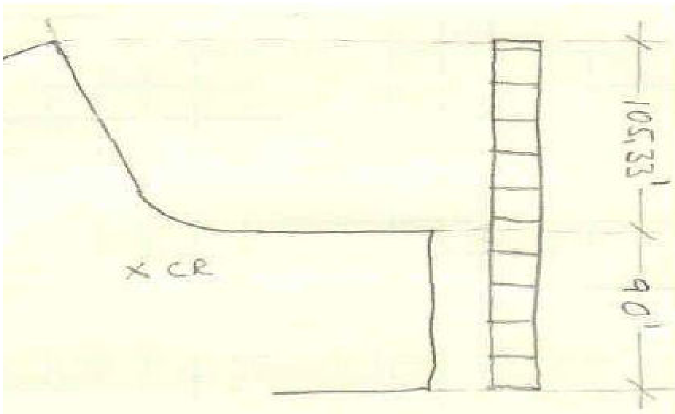
Case 1: wind parallel to y-axis



Case 2: wind parallel to x-axis



Case 3: wind at the 45 degree angle with the x-axis.

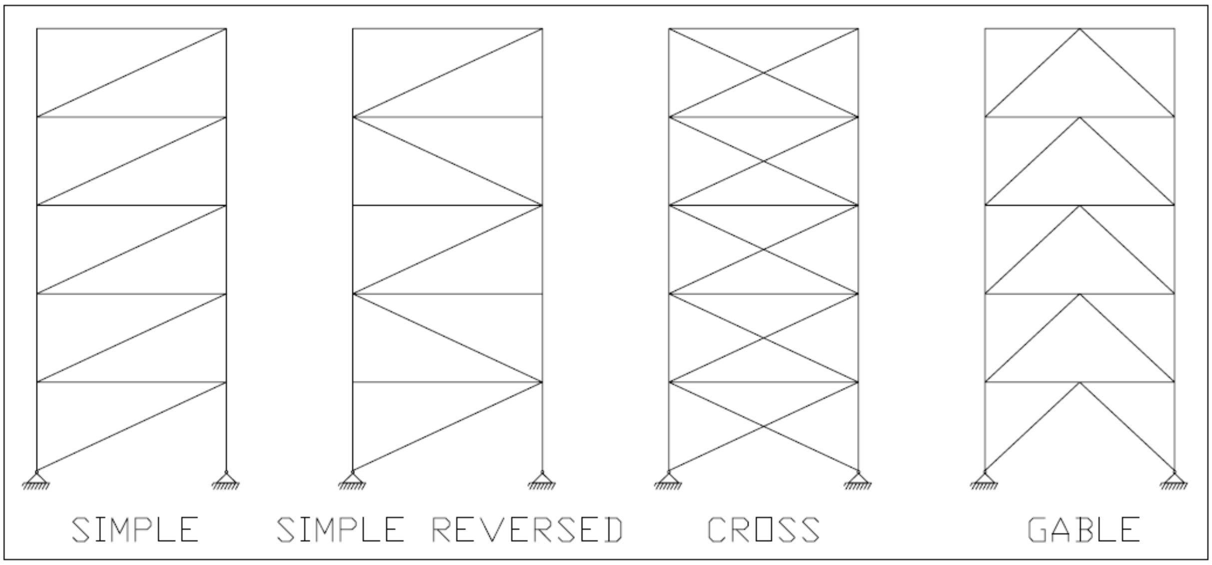


Due to the symmetry of the building, the wind in the y-axis direction does not produce torsional forces in the building. The building is not symmetrical in the x-axis direction; therefore, any wind case not parallel to the y-axis produces the torsional forces in the building. The center of the rigidity is determined using the relative stiffness analysis as shown in the appendix of this report, and torsional forces are distributed using the polar moment of inertia.

While calculating the torsional forces in the building, it was unclear weather to apply the factors of the sloping wall. After a brief analysis, it was decided that the difference between the slope-adjusted loads and the averaged loads is not significant for a five story building. The elimination of the slope-adjusting procedure does not create a risk, in fact, the magnitude of the adjusted loads is smaller than the magnitude of the unadjusted loads; therefore, it is more conservative to use the unadjusted loads.

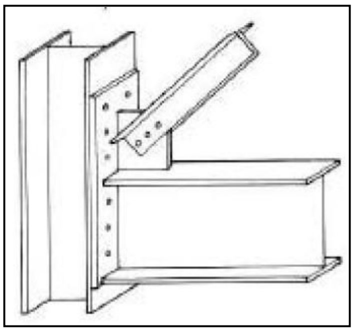
Because the building's x-axis dimension is the largest dimension, case 1 is expected to be the controlling case. The case comparison tables shown below prove the expectation to be correct. The braced frame "G", however, is governed by case 2. In case 1, the "G" frame is positioned perpendicularly to the wind load direction, and can resist only the torsional forces; due to the symmetry of the building, however, these forces are absent. The design of the "G" braced frame will be governed by the case 2, while the rest of the frames will be designed for the loads of case 1.

Braced Frame Selection



There are several ways to design a braced frame. Each design or layout has its advantages and disadvantages. I have analyzed four different variations of braced frames. The following factors were considered in this evaluation: economy of the design, number of connections, the complexity of each connection, the amount of steel, the ease of installation, advantages and disadvantages of each variation, and the amount of usable space under the brace.

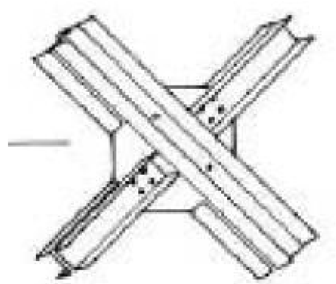
The simple diagonally braced frame (first from the left) incorporates a total of thirteen connections. Nine of these connections are shear connections that service both the beam and the diagonal brace and could look something like the one shown on the right (drawing is from www. ocw.mit.edu). One connection supports only the roof beam, two supports only the column, and one supports only the diagonal brace. All together there are tree simple shear connections and nine more complex shear connections. In this case the diagonal member must resist both, the compression forces and the tension forces, depending on the direction of the wind. Because of the proportion, the diagonal can resist large tensile forces and much smaller compression forces. The long members resist smaller compression loads and fail mostly due to buckling about its weakest axis. Thus the compression strength of the member is the limiting factor of this design.



The simple – reversed braced frame requires a total of thirteen connections, eight of which are simple shear connections that service only one member. One of the thirteen connections is the connection that services both the beam and the diagonal, and four connections are more complex shear connections that service the beam and two diagonal braces. In all of the wind cases the direction of the diagonal forces will be reversed from one story to another. For example three diagonals will be under the compressive forces while the remaining two will be the under tensile forces, and when the wind direction reverses, two will be in compression and three in tension. Nevertheless, just like in the simple braced frame, all diagonals must be designed to accommodate tension and compression forces, and the compressive strength of the member will again control the design. The sizes of the diagonal members are identical in both simple and simple-reversed braced frame layout. The difference between them is in the types of the connection. The simple-reversed system has more simple connections but it has four connections that are more complex than the simple system.

The cross braced frame system (third from the left) requires fourteen connections. Four of these connections are simple shear connections that support only one member. Two of the connections support both the beam and the diagonal, and eight of the remaining connections are complex shear connections that support the beam and two diagonals. This design will work at its most high efficiency if all members are very small and resist only the tensional forces. Should they be required to resist any compressive forces, their size will be increased dramatically and they will have no chance in competing with other braced frame systems. This is so because this layout requires the largest number of complex connections, and has the largest numbers of diagonal members. In order to compete with other systems the overall amount of steel needs to stay as small as possible. We can strengthen the weakest axis of the diagonals by connecting the two diagonals at the point of their intersection. However, this will only be desired if diagonal members resist compressive forces as well. If the diagonals resist only the tensional forces then the connections at the intersection will not be needed since their failure will not be the buckling about any axis but will be due to yielding (or rapture) of the member (considering only the member, not bolts, weld, or connection).

There is a variation of the cross braced frame system. The size of the diagonals can be different, so that the bigger diagonal resists all the lateral loads when it is in tension (wind from right to left on the drawing – www. ocw.mit.edu). When the direction of the lateral load is reversed (left to right), both members share the load: smaller brace is in tension and the larger brace is in compression. Should the smaller brace be removed, the remaining diagonal would have to be increased to carry all the lateral loads. This system, however, requires more steel, and possibly more connections (depending on the thickness of the walls and required size of the diagonals).



The gable braced frame incorporates nineteen connections, six of which are simple shear connections, and thirteen connections that service two members each. Each of the diagonal members resists the lateral load at the same time. Regardless of the direction of the lateral load, both members carry equal but opposite forces: one member is in compression one is in tension.

The compression and tension distribution reverses with the reversal of the lateral load direction. Consequently, both diagonals need to be designed to resist lateral loads in compression. The biggest advantage of this system is that the braces carry parts of the beam loads, which allows for the beam size to be smaller. This system, however, is most desired when the span of the columns is large. In which case, the beam size can be decreased by sharing its load with the diagonal braces. The biggest disadvantage is the number of connections and diagonal members. The cost of the material is not always the governing factor. In the specific case of the North Brock Corporate Center it is more economical to keep the beam sizes uniformly constant. When the gable braced frame system is evaluated, the beam size is not redesigned so that the beams are uniform throughout.

To compare the size and the amount (weight) of the diagonals, one single story frame was evaluated. A point load of 78.8 kips was applied at the upper node of each different single frame system. STAAD.Pro 2004 was used to distribute forces to each member. The table bellow shows the resulting axial forces in each diagonal of each frame. It also shows the size, total weight, diagonal member material cost, and number of simple, two member, and three member connections needed in each frame. The weight and prices are shown for one single story frame, while the number of connections is shown for the full four story frame. RSMeans was used to estimate these costs.

FRAME COMPARISON CHART

Frame System	Force (k)	Direction	Size	Weight (lbs) Total	Diagonal cost ($)	# 1 conn	# 2 conn	# 3 conn
Simple	86.958	C	WT8x33.5	1108.85	1155.05	4	9	0
Simple - reversed	86.958	C	WT8x33.5	1108.85	1155.05	8	1	4
Cross	86.958	T	L3x3x1/2	618.97	648.91	4	2	8
Gable	53.895	C	WT8x22.5	923.32	960.25	6	13	0

The cross braced frame diagonals are the least expansive; however they have the largest number of complex shear (3 members) connections. As shown in the cost comparison section of this report each simple shear connection costs approximately $68.00, each two member connection costs approximately $114.00. Each three member connection is estimated to be $160.

The total cost of connections of the simple and simple-reversed systems is identical: $1298.00.

The total cost of connections of the cross system is $1780.

The total cost of connections of the gable system is $1890.

At this point the gable system is out of the question. The diagonal cost is nearly similar, but the connection cost of the system is relatively large. Since the simple and simple-reversed systems are identical, we will use simple system for further comparison. The total cost difference between the simple and cross braced systems for the full five story frame is estimated to be $1200, where simple system is more expansive. However, simple system takes 4 hours less to install all of the connections. This four-hour difference does not include the installation of the actual members.

The time difference will be increased even farther due to the fact that the cross system has twice as many diagonal members as the simple system does.

Simple braced system is almost as expansive as the cross braced system, but it takes less time to assemble. The simple braced system does not incorporate three member connections, uses much less diagonals, and it is easier to assemble. All these factors make simple braced system more favorable and more feasible system for this particular project.

Simple braced frame system will be used for further calculations in this report.

Distribution of Load to Each Member

The loads shown in the table are applied at the top of the braced frame. Further distribution of the load to each individual steel member is administered using the frame equilibrium method. All connections are shear connections, or pin connections for the sake of this analysis, thus the sum of forces in each direction must equal zero. Braced frames incorporate point, or shear connections, thus there are no moments present in this analysis. The result of frame equilibrium analysis shows only the axial forces in each member. Farther calculations are needed to adjust for the flexural forces in the beam, as shown in the *beam design* section of this report. The table below shows a complete list of all the braced frame members of the building with their corresponding axial forces. Detailed calculations of this analysis are provided in the appendix of this report.

	Summary of Axial Forces				
		Braced Frame A Size			
Member	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	837	658	474	277.1	74.3
BC	78.8	64.5	48	34.9	13.98
CD	1023	776	537	31.67	80.8
AC	86.6	71.2	53	304.9	6.99
		Braced Frame B Size			
Member	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	533.7	422.8	304.7	180.3	46.8
BC	25.05	20.55	15.42	10.9	4.46
CD	661.6	421.7	288	161.5	41.34
AC	43.1	35.35	26.5	18.75	7.67
		Braced Frame C Size			
Member	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	353	284.5	208.6	125.2	32.76
BC	56.42	46.27	34.74	22.72	8.67
CD	889.1	672.9	463.1	260.7	66
AC	66.88	54.8	41.2	26.9	10.28
		Braced Frame D Size			
Member	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	353	284.5	208.6	125.2	32.76
BC	56.42	46.27	34.74	22.72	8.67
CD	889.1	672.9	463.1	260.7	66
AC	66.88	54.8	41.2	26.9	10.28
		Braced Frame E Size			
Member	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	837	658	474	277.1	74.3
BC	78.8	64.5	48	34.9	13.98
CD	1023	776	537	31.67	80.8
AC	86.6	71.2	53	304.9	6.99
		Braced Frame F Size			
Member	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	533.7	422.8	304.7	180.3	46.8
BC	25.05	20.55	15.42	10.9	4.46
CD	661.6	421.7	288	161.5	41.34
AC	43.1	35.35	26.5	18.75	7.67
		Braced Frame G Size			
Member	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	755.2	591.9	422.5	246.7	62.9
BC	46.65	38.26	28.72	18.8	8.3
CD	901.8	684.5	472.4	266.4	66.78
AC	55.29	45.35	34.04	22.28	9.16

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REDISIGN OF COLUMN, BEAM, FOOTINGS, CONNECTIONS, AND DIAGONAL MEMBERS

REDESIGN OF COLUMNS

The braced frame columns do not carry moment loads, and are design as compression members. Even though most columns span three stories high, they are braced at every 14 feet interval. The columns were designed by hand, using the standard design procedure for members under compression. The original column sizes of a typical column range from W12x65 to W12x79. The new columns must be W12 columns in order not to intervene with wall thickness.

During the design process, it was convenient to import certain equations into the spreadsheet to ease the process of finding the smallest W12 size column. The redesign procedure of the columns used in braced frames is outlined in the Section E-2 in the Specification of the LRFD manual of steel construction, third edition. In all of the redesigned columns the controlling failure was due to the buckling about the y-y axis. In the braced frame system, the entire lateral load is resisted by only few braced frames, therefore, each individual braced frame member must resist a large axial load.

Because each column span from the footing to the third floor, the design of the columns was governed by the compression forces applied to the column at the lowest level of the building. The redesigned system did not affect all of the columns. The only columns were affected by the redesign are the columns that were part of the moment frame resisting system, and the columns that became part of the braced frame resisting system. Most moment carrying columns with the exception of some at inconvenient locations were of the same size: W12x72. After the redesign this size changed to W12x65. Overall the size of all other columns that used to carry moment loads decreased by approximately 11%, while the size of the braced columns increased by up to 150%. The layout of the braced frames requires an addition of two new columns to be part of braced frames B and F. The total difference in steel was calculated to be 13,608 pounds. This number includes all new columns, diagonals, the change in braced frame columns, and the change in all other columns that were part of the moment frame system.

REDESIGN OF BEAMS

All beam calculations were performed by hand. With the exception of the roof beam, all braced frame beams are composite beams. All composite beams were designed according to the procedure outlined in the Combined Compression and Flexure (composite beams, section I4) section in the Specification of the LRFD manual. As in the column design calculations, spreadsheet was used because of the repetition of the design process. Plastic moment capacity was found using both the LRFD Manual tables, and/or hand calculations. Equations H1-1b and H1-1a were used to determine the combined, flexural and compression strength of the composite beam. The deflection was calculated using the lower bound elastic moment of Inertia found in table 5-15 in the LRFD manual. The axial forces in the beams were generally small, and did not have much impact on the beam. Because the axial forces, and the fact that the beams no longer have moment connections on the ends, it was expected that the redesigned beams would increase in size. However, after all calculations, the original size of the beams was found to be satisfactory for the use in the braced frame. This lead to further study of the original moment frames. Original structural drawings indicated that moment connections resist only lateral loads. After calculations I was able to verify that all beams that were part of the moment frame were designed as simple beam with pin (shear) connections. They are sufficient to carry all gravity loads without redistributing their internal moments to columns. The lateral load induced only 40 ft-k at each end of the beam (+/-), and the beams could sufficiently carry the gravity loads and resist moments from lateral loads. Consequently, the beams were not redesigned but only checked for strength (moment and shear) and deflection.

The maximum moment due to gravity (with or without the lateral loads) were calculated to be 567 ft-k, and the deflection at the center is 0.885 inches (30' beam).

DL = 60 psf
LL =100 psf (not reduced)
LL Reduction factor = 0.6
LL = 60 psf (reduced)

Typical Beam: W24x68

The beam must be sufficient to carry all gravity loads until the concrete cures. It is assumed that the live construction load is 20 psf.

LL = 20 psf
DL = 60 psf

The governing factored uniformly distributed load is calculated to be 3.12 klf. This produces a maximum moment of 351 ft-k at the center of the beam. The maximum moment (578 ft-k) and maximum shear (275k) of W24x68 proves to be strong enough to carry the required gravity loads before the concrete cures. The deflection of the beam before the curing of the concrete is calculated to be 1 inch which is exactly the limit for this span. In this case the stage at which the concrete is in the process of curing is the controlling case. If the beam was shored, the steel beam W24x62 would be sufficient to carry all the gravity loads after the concrete cures.

All detailed calculations are provided in the appendix of this report.

REDESIGN OF DIAGONAL MEMBERS

The diagonal members in the braced frame resist only the axial forces. There is only one diagonal member per braced frame. These diagonals must resist tensile forces as well as compressive forces. At the beginning of the analysis, the angles were considered to be used as the diagonal member; however, due to long spans and low moment of inertia most angles would experience a flexural buckling about either one axis. WT- shaped steel members, on the other hand, behave well under both the compressive and the tensile forces. Because the tensile strength of the WT-shaped member is much greater then its compressive strength, the diagonal steel members were analyzed for the compressive strength only. In all of the cases the design was controlled by the flexural buckling about the y-y axis. With the exception of the beam, there is no moment present in any one of the braced frame members. The design procedure of the diagonal members was identical to the design procedure of the column.

It must be noted that the flexural forces due to self weight of the diagonal are present in the member. These forces are insignificant when compared to the overall loads on the member, and thus were ignored in the analysis of the diagonal member design.

On the right is the table with the redesigned beam, column, and diagonal sizes. All detailed calculations for obtaining these sizes are provided in the appendix of this report.

Steel Member Design Summary					
		Braced Frame A Size			
Member	**Level 0**	**Level 1**	**Level 2**	**Level 3**	**Level 4**
AB	W12x106	W12x106	W12x106	W12x50	W12x50
BC	W18x35	W18x35	W18x35	W18x35	W16x26
CD	W12x106	W12x106	W12x106	W12x50	W12x50
AC	WT9x38	WT9x38	WT8x33.5	WT8x33.5	WT8x33.5
		Braced Frame B Size			
Member	**Level 0**	**Level 1**	**Level 2**	**Level 3**	**Level 4**
AB	W12x65	W12x65	W12x65	W12x35	W12x35
BC	W18x35	W18x35	W18x35	W18x35	W21x50
CD	W12x106	W12x106	W12x106	W12x50	W12x50
AC	WT8x33.5	WT8x33.5	WT8x33.5	WT8x33.5	WT8x33.5
		Braced Frame C Size			
Member	**Level 0**	**Level 1**	**Level 2**	**Level 3**	**Level 4**
AB	W12x96	W12x96	W12x96	W12x40	W12x40
BC	W18x40	W18x40	W18x40	W18x40	W18x35
CD	W12x96	W12x96	W12x96	W12x40	W12x40
AC	WT8x33.5	WT8x33.5	WT8x33.5	WT8x33.5	WT8x33.5
		Braced Frame D Size			
Member	**Level 0**	**Level 1**	**Level 2**	**Level 3**	**Level 4**
AB	W12x96	W12x96	W12x96	W12x40	W12x40
BC	W18x40	W18x40	W18x40	W18x40	W18x35
CD	W12x96	W12x96	W12x96	W12x40	W12x40
AC	WT8x33.5	WT8x33.5	WT8x33.5	WT8x33.5	WT8x33.5
		Braced Frame E Size			
Member	**Level 0**	**Level 1**	**Level 2**	**Level 3**	**Level 4**
AB	W12x106	W12x106	W12x106	W12x50	W12x50
BC	W18x35	W18x35	W18x35	W18x35	W16x26
CD	W12x106	W12x106	W12x106	W12x50	W12x50
AC	WT9x38	WT9x38	WT8x33.5	WT8x33.5	WT8x33.5
		Braced Frame F Size			
Member	**Level 0**	**Level 1**	**Level 2**	**Level 3**	**Level 4**
AB	W12x65	W12x65	W12x65	W12x35	W12x35
BC	W18x35	W18x35	W18x35	W18x35	W21x50
CD	W12x106	W12x106	W12x106	W12x50	W12x50
AC	WT8x33.5	WT8x33.5	WT8x33.5	WT8x33.5	WT8x33.5
		Braced Frame G Size			
Member	**Level 0**	**Level 1**	**Level 2**	**Level 3**	**Level 4**
AB	W12x96	W12x96	W12x96	W12x40	W12x40
BC	W16x26	W16x26	W16x26	W16x26	W16x26
CD	W12x96	W12x96	W12x96	W12x40	W12x40
AC	WT8x33.5	WT8x33.5	WT8x33.5	WT8x33.5	WT8x33.5

REDESIGN OF THE FOOTINGS

In original system all footings that were part of the moment frame resisting system were designed to carry the moment of up to 25 ft-k. The presence of the moment mostly affects the depth or height of the footing design, and plays a minimal role in the design of footings surface area. It also affects the reinforcing of the pier. After the redesign, all moments were removed from the footings. This called for a redesign of all the footings that were part of the moment frame system. The footings that support braced frames will carry wind loads in addition to all gravity loads. These loads will be applied as normal forces on the pier and footing. To accurately evaluate what impact the redesign has on the foundation system, two types of footings will be redesigned: new typical footing, and all braced frame footings. In order to proceed with the calculations it was assumed that the ground allowable stress is 4000 psi.

The columns of any braced frame have different axial forces. These forces are reversible depending on the direction of the wind, and so the largest axial forces are used to design both footings of the braced frame.

Gravity loads:

LL = 40 psf (reduced; reduction factor varies depending on the influence area)
DL = 60 psf
SL = 21 psf

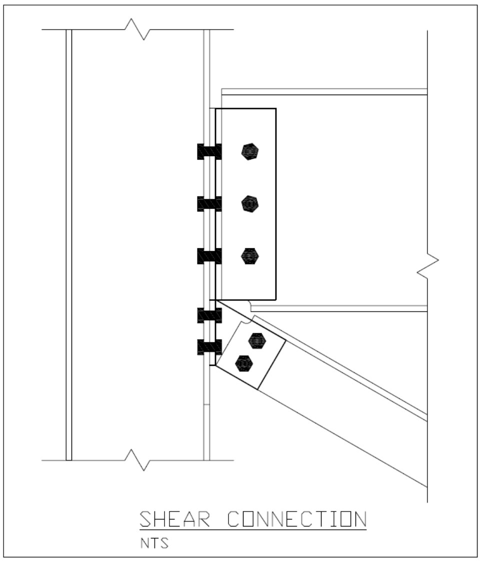
Lateral Loads:

WL = vary from 60 kips to 127 kips

The controlling combination of factored loads on footing of frame A was 547.65 kips (1.2DL+1.6LL+0.5SL). In this case the combination with the wind load did not govern the design. The size of the footing was calculated to be12'x12'x22" with (12) #7 reinforcing bars each way. The cost of the footing is determined mostly from the footing's size. The difference in reinforcing is not significant with respect to the overall cost of the footing. The size of the piers is left the same to provide sufficient force distribution to the footing. Bellow is the table showing the original and redesigned footing size as well as their cost.

FOOTING SIZE AND COST					
Footing	Original	Cost ($)	Redesigned	Cost ($)	Difference ($)
Typical	11'x11'x28"	2753	10'x10'x22"	2163	590
Frame A	11'x11'x28"	2753	12'x12'x22"	2574	179
Frame B	10'x10'x24"	1950	10'x10'x16"	1300	650
Frame C	8'x8'x28"	1456	10'x10'x20"	1625	-169
Frame D	8'x8'x28"	1456	10'x10'x20"	1625	-169
Frame E	11'x11'x28"	2753	12'x12'x22"	2574	179
Frame F	10'x10'x24"	1950	10'x10'x16"	1300	650
Frame G	10'x10'x28"	2275	11'x11'x22"	2163	112

DESINGN OF SHEAR CONNECTIONS



All moment connections are replaced by simple shear connections. The redesign procedure of simple shear connections is outlined in the Section J-2 in the Specification of the LRFD manual of steel construction, third edition. All selections were based on the table 10-1 of that same manual.

An example of a simple shear connection is shown on the right. The beam connection does not resist any moments or tensile forces and provides the beam with only the shear support. The typical beam connection incorporates two angles – one angle at each side of the beam, and will use 3 rows of ¾" A325 bolts at each being 2" long. The thickness of the angle was calculated to be 5/16". The distance between the bolts is more than 3 inches o.c., and the distance between the top and bottom edge of the connection and the center of the bolt is more that 1 ¼". The distance from the edge of the column flange to the center of the holes at the beam is 2 ¼". The total weight of the angles is estimated to be 15.65 pounds per connection ((2) L5x3x5/15"x10").

The connection that supports a diagonal member is resisting both the shear and the tensile forces. This connection will consist of two angular plates both having the thickness of ½". The total weight of the angles is estimated to be 12.32 pounds. The connection is fastened to a column by 2 rows of ¾" A325 bolts, and fastened to a beam by two rows of 7/8" A325 bolts. All connections use standard holes.

All detailed calculations of both connections are provided in the appendix of this report.

STORY DRIFT SUMMARY

Story drift was checked using STAAD.Pro 2004. Table bellow shows the drift at each story. The total drift (at the roof) is then compared to the allowable drift on the basis of L/360.

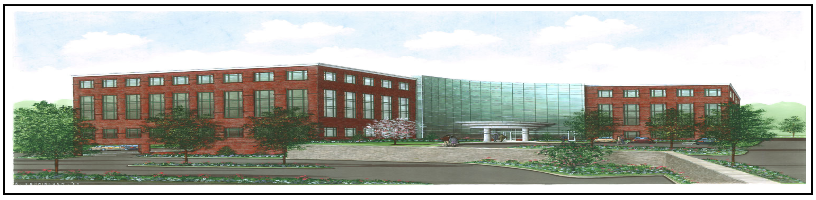
STORY DRIFT (inches)						
Frame	Level 1	Level 2	Level 3	Level 4	**Roof**	**Allowable**
Frame A	0.1573	0.4799	0.9158	1.327	**1.579**	**2.333**
Frame B	0.1253	0.4541	0.9167	1.401	**1.709**	**2.333**
Frame C	0.1128	0.3905	0.7031	1.021	**1.186**	**2.333**
Frame D	0.1128	0.3905	0.7031	1.021	**1.186**	**2.333**
Frame E	0.1573	0.4799	0.9158	1.327	**1.579**	**2.333**
Frame F	0.1253	0.4541	0.9167	1.401	**1.709**	**2.333**
Frame G	0.14	0.4078	0.7524	1.072	**1.263**	**2.333**

OVERTURNING MOMENTS

The wind loads are transferred to the ground trough beams, diagonals, and columns. Gravity forces act on the braced frame to counteract the vertical reaction from the wind forces. Bellow is the summary of all braced frame reactions, and the summary of the gravity loads.

	Wind Load (k)		Dead load (k)		
Frame	R1	R2	R1	R2	Diff Factor
A	112.2	-112.2	238	238	2.121212
B	76.44	-76.44	119.25	119.25	1.560047
C	127	-127	135.8	135.8	1.069291
D	127	-127	135.8	135.8	1.069291
E	112.2	-112.2	238	238	2.121212
F	76.44	-76.44	119.25	119.25	1.560047
G	60	-60	221.3	221.3	3.688333

The gravity loads at the support are larger than the resulting uplift forces on the support. At the braced frames C and D, this difference is small, and could potentially be problematic. The overturning moment at each of the frames D and C is only 193.6 ft-k. This problem would be eliminated if the columns of each frame were moved farther apart.



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COMPUTER ANALYSIS

All of the above calculations were done by hand. STAAD.Pro 2004 was used to check these calculations. The 3D computer analysis is time consuming and in this case not necessary. As stated before, the moment connections in the original system resist only lateral loads, and do not distribute beam gravity moments to the columns. The beams were designed originally as if simply supported. The only structural steel members that need to be redesigned or designed are the columns and diagonal braces. The purpose of this computer analysis is to verify that provided work is true and accurate.

Section Properties

Prop	Section	Area (in^2)	I_{yy} (in^4)	I_{zz} (in^4)	J (in^4)	Material
1	W8X58	17.100	75.100	228.000	3.228	STEEL
2	W8X48	14.100	60.900	184.000	1.890	STEEL
3	W8X35	10.300	42.600	127.000	0.719	STEEL
4	W8X31	9.130	37.100	110.000	0.494	STEEL
5	W8X21	6.160	9.800	75.300	0.264	STEEL
6	W12X106	31.200	301.000	933.000	8.730	STEEL
7	W12X50	14.700	56.300	394.000	1.596	STEEL
8	W18X35	10.300	15.300	510.000	0.459	STEEL
9	W16X26	7.680	9.600	301.000	0.229	STEEL

Node Displacements

Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
1	1:1k Load	0.000	0.000	0.000	0.000	0.000	0.000	0.000
2	1:1k Load	0.002	0.000	0.000	0.002	0.000	0.000	0.000
3	1:1k Load	0.005	0.001	0.000	0.005	0.000	0.000	0.000
4	1:1k Load	0.009	0.001	0.000	0.009	0.000	0.000	0.000
5	1:1k Load	0.013	0.001	0.000	0.013	0.000	0.000	0.000
6	1:1k Load	0.018	0.001	0.000	0.018	0.000	0.000	0.000
7	1:1k Load	0.017	-0.002	0.000	0.017	0.000	0.000	0.000
8	1:1k Load	0.012	-0.001	0.000	0.012	0.000	0.000	0.000
9	1:1k Load	0.008	-0.001	0.000	0.008	0.000	0.000	0.000
10	1:1k Load	0.004	-0.001	0.000	0.004	0.000	0.000	0.000
11	1:1k Load	0.001	-0.000	0.000	0.001	0.000	0.000	0.000
12	1:1k Load	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Each frame was analyzed in STAAD.Pro individually. After all the properties and specifications were entered, a one kip load was applied at the roof level of each frame. STAAD then calculated the drift of each node. Example of the analysis of frame A is shown on the left. All of these calculations are printed in the form of the report, and are included in the appendix of this report. After obtaining the total drift of the frame, I was able to calculate the stiffness factor of the frame using the following equation:

$k=F/\Delta$.

- Frame A k=58.82 (k/in)
- Frame B k=14.49 (k/in)
- Frame C k=55.56 (k/in)

- Frame D k=55.56 (k/in)
- Frame E k=58.82 (k/in)
- Frame F k=14.49 (k/in)
- Frame G k=47.62 (k/in)

These relative stiffness factors determined the center of rigidity to be (0, 85.5) ft. This is only 2.9 ft away from the point of rigidity that was calculated by hand. The direct and torsional forces were distributed to each frame according to its relative stiffness and the distance of the frame from the center of rigidity. Shown bellow is the table with the resulting forces at the top node of each frame at each story level.

Distribution of Lateral Loads (k) Computer Analysis					
	Level				
Frame	Roof	4th	3rd	2nd	1st
A	12.64	28.62	43.76	58.29	71.07
B	3.04	6.9	10.54	14.05	17.13
C	11.95	27.07	41.39	55.14	67.23
D	11.95	27.07	41.39	55.14	67.23
E	12.64	28.62	43.76	58.29	71.07
F	3.04	6.9	10.54	14.05	17.13
G	7.718	17.48	26.7	35.57	43.38

Computer analysis shows that more load resisted directly by frames C and D. In all the other cases, the load on the frame has decreased by little more than one kip. The resulting distribution of the axial forces in the diagonals of both frames is shown bellow.

- Roof 14.16 k
- 4^{th} 32.09 k
- 3^{rd} 49.06 k
- 2^{nd} 65.36 k
- 1^{st} 79.69 k

These changes do not affect the overall design. The diagonal size that was picked by hand calculations is satisfactory to carry these loads as well. This analysis does not confirm the accuracy of the lateral load development calculations, but only confirms the accuracy of the overall wind load distribution and the accuracy of the size selection.

Additional computer analysis material is provided in the appendix of this report.

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LATERAL LOAD DEVELOPMENT – CORRECTION

The lateral force resisting system of the Northbrook Corporate Center was redesigned considering wind loads to be the controlling lateral load. After the completion of this project I looked over the seismic load development section of this report and found an error in the calculations. The response modification factor was mistakenly given as 8 when in reality it should be 3. This changes the seismic shear coefficient to approximately 0.08. To compare seismic load to wind loads the 4th floor will be evaluated in this section of the report. Seismic shear equals the weight of the building multiplied by the seismic shear coefficient.

The total weight of the roof = (25 psf)(26000 sf) = 650000 lbs

Lateral Load Parallel to the X- axis:

- Seismic shear = (650000lbs)(0.08)/160ft = 325 plf
- The total wind load at roof level = 228 plf
- The seismic load will govern the design parallel to the x – axis of the building.

Lateral Load Parallel to the Y – axis at roof level:

- Seismic shear = 182.3 plf
- Wind load = 228 plf
- Wind load controls the design in this direction.

Lateral Load Parallel at 45 degrees with the Y or X axis:

- Seismic shear = 266.2 plf
- Wind load = 228 plf
- Seismic shear controls the design in this direction.

The seismic load governs two of the 3 evaluated directions. This means that braced frames A through F are governed by the wind loads and were designed correctly. Braced frame G however is governed by the seismic load. The redesign of the G frame might change the stiffness factor of the frame. As a consequence the over all distribution of forces to each frame might be slightly altered. At this point it is too late to act on these changes, but it should be noted that the accuracy of this redesign is now slightly impaired.

Shown bellow is the procedure for obtaining the seismic shear coefficient.

SEISMIC LOAD CALCULATIONS ASCE 7-02 CH 9

OCUPANCY CATEGORY	II	[TABLE 1-1]
SEISMIC GROUP	I	[TABLE 9.1.3]
SITE CLASSIFICATION	C	ASSUME

ACCELERATION

$S_S = 0.335$ (MAP 9.4.1.1a)

$S_1 = 0.085$ (MAP 9.4.1.1b)

SITE CLASS ADJUSTMENT

$F_A = 1.2$ (TABLE 9.4.1.2.4a)

$F_V = 1.7$ (TABLE 9.4.1.2.4b)

$S_{MS} = 1.2(0.335) = 0.402$

$S_{M1} = 1.7(0.085) = 0.145$

DESIGN SPECTURAL RESPONSE

$S_{DS} = 0.667(0.402) = 0.268$

$S_{D1} = 0.667(0.145) = 0.096$

SEISMIC DESIGN CATEGORY

B (TABLE 9.4.2.1a)

B (TABLE 9.4.2.1b)

SEISMIC BASE SHEAR

$V = C_S W$

$C_S = \frac{S_{DS} \cdot I}{R} < C_S = \frac{S_{D1} \cdot I}{T \cdot R}$

$< C_S = 0.044 I S_{DS}$

RESPONSE MODIFICATION FACTOR

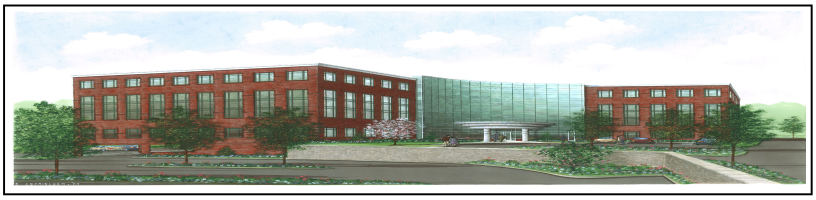
$R = 8$ *R should be 3* (TABLE 9.5.2.2)

I

$I = 1.0$ (TABLE 9.1.4)

T PERIOD

$T_a = 0.51$ (EQ. 9.5.3.3-1)



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BRACED FRAME VS MOMENT FRAME COST COMPARISON

Breadth Study

The feasibility of the new design greatly depends on the overall cost advantage. In order to compare the cost of the two systems more accurately several parts of the system have to be evaluated. As stated earlier in this report, many areas of the structural system were affected by the redesign. All moment connections were redesigned to simple shear connections. Most of the typical footings had to be redesigned as well. Braced frames introduced new columns, footings, and steel diagonal braces. All these changes have an impact on both the cost of the building and the schedule of the construction of the building.

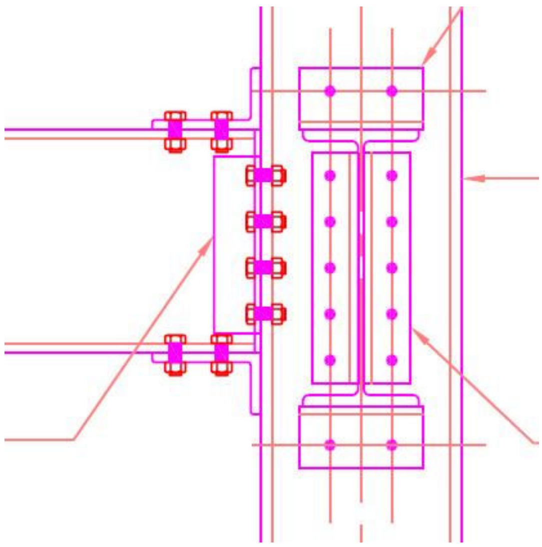
This breadth study will evaluate the cost of:

- Moment connections
- Shear connections (all of the introduced kinds)
- Diagonal braces
- Moment frame columns
- Redesigned columns
- New columns
- Original footings
- New and redesigned footings

This breadth study will also evaluate what kind of impact these changes have on the schedule of the construction of the Northbrook Corporate Center.

MOMENT CONNECTION COST ESTIMATE

In the North Brook Corporate Center's original design moment connections are both welded and bolted. Using the RS Means, a detailed unit price method was used to estimate the cost of one typical moment connection. A typical moment connection has one angle on each side of the beam. These two angles are bolted together compressing the beam in between them. The column ends of the angles are welded to the column.



A typical moment connection has 20 bolts and 60 inches of 5/16 inch thick welding. The material, labor, equipment, and O&P cost of angles, bolts, and welding are shown bellow.

- 5/16" 0.4#/LF weld $19.30 /LF
- ¾" A325 2" long bolt $5.50 each
- L5x3x3/8" $1.37 /LF

It is difficult to estimate the cost of every single variation of the moment connection in the building. To simplify the procedure all connections are assumed to be similar all through out the building. Same assumption will apply to the cost estimate of the redesigned connections. The moment connections become smaller from the bottom of the building to the top, while the typical redesigned connections remain constant throughout. As a result, the difference between the cost of the moment and shear connections will be slightly different than the true difference. This error will be considered at the end of the systems' comparison section of this breadth study.

As stated above, each moment connection incorporates:

- ¾" A325 2" long bolts 20
- 5/16" 0.4#/LF weld 5 ft
- L5x3x3/8" 4.3 ft

The cost of each moment connection is calculated to be:

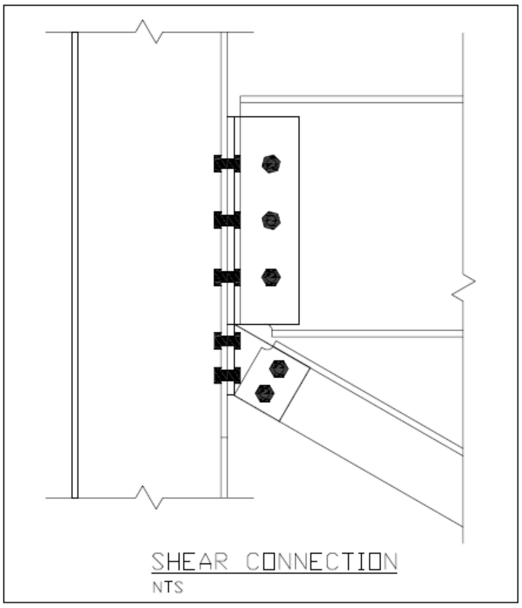
• Bolts	$110	
• Weld	$96.5	
• Angle	<u>$48.55</u>	
TOTAL:	$255	($97.45 of which is for labor)

It takes 0.067 hours for one person to install one bolt, and 0.211 hours for one person to do one linear foot of weld. As a result it takes 2.40 hours to complete one moment connection.

The Northbrook Corporate Center has a total of 750 moment connections.

TOTAL COST **$191,250**
TOTAL TIME **6.5 weeks for 7 people crew**

SHEAR CONNECTION COST ESTIMATE



There are two types of shear connections: beam to column and beam and diagonal to column. An example of beam and diagonal to column is shown on the right. A typical simple beam to column connection size stays constant through out the building, while the size of the diagonal connection varies. However, as was done in the moment connection cost estimate, these variations will be considered minimum, and the size will be assumed constant all through out the building.

Each typical beam to column connection incorporates:

- ¾" A325 2" long bolts 9 bolts
- L5x3x5/16" 10 in

Each typical beam and diagonal to column incorporates:

- A325 2" long bolts 15 bolts
- (2) L5x3x5/16" 10 in – 13.65 lbs
- (2) angular plates 12.32 lbs

The cost of each ¾" A325 2" long bolt is $5.55 including material, labor, equipment, and O&P costs. The cost of the angles is $1.35 /lbs. It takes 0.067 hours to install one bolt.

The total cost of each beam to column connection is $68.38 (0.603 hours)
The total cost of each beam and diagonal to column connection is $114.75 (1.005 hours)

In the entire building there are:

- 690 typical beam to column connections
- 63 beam and diagonal to column connections
- 7 diagonal to column connections

TOTAL COST = $54,736
TOTAL TIME = 8.6 days

Note: RSMeans was used to estimate the given costs.

DIAGONAL BRACES COST ESTIMATE

A detailed cost estimate was used to estimate the cost of the diagonal braces. By interpolating between the given steel sizes, the following prices were obtained:

- Material $1.04 /lb
- Labor $2.02 /LF
- Equipment $1.32 /LF
- O&P 11% of total cost

Including the installation of connections, it takes a crew of 7 people 0.057 hours to install 1 linear foot of the diagonal member.

There are a total of 132.4 ft of WTx38 steel members.
There are a total of 796.91 ft of WT8x33.5 steel members.

TOTAL COST = $40,956
TOTAL TIME = 371 HRS (for one person)

COST ESTIMATE OF THE MOMENT FRAME COLUMNS

There are approximately 36 moment frame columns in the building. The size of each column is uniform throughout the first three stories, and then changes to a smaller size. The smaller size columns are uniform throughout the upper two stories. The total weight of all these columns is 162288 lbs. According to the RSMeans, the material cost of this size steel is $1.04 per pound. It takes same amount of time to install the redesigned columns as it takes to install the original columns. The difference is adjusted for in the time it takes to install shear connection vs. moment connections, and will be shown at the end of this breadth analysis.

TOTAL COST = $168,779.52

COST ESTIMATE OF THE REDESIGNED COLUMNS

The columns are redesigned in the similar distribution. That is, the size of each column is uniform throughout the first three stories, and then changes to a smaller size. And just like in the original design, the smaller size columns are uniform throughout the upper two stories. The weight of the 36 redesigned columns is 148680 lbs.

TOTAL COST = 154,627.20

COST ESTIMATE OF NEW COLUMNS (Frame B and F)

New design incorporates new columns, each of which consist of two different sizes: W12x65 and W12x35.

The total length of each size is:

- W12x65   84 ft
- W12x35   56 ft

The prices of material, labor, equipment, and O&P are as follows:

- Material $1.04 /lb
- Labor $2.15 /lb
- Equipment $1.38 /lb
- O&P   11% of the total cost

(Note: some costs were interpolated between the smaller and larger sizes of the actual column)

It takes a crew of 7 people 0.057 hours to install one linear foot of this size range of structural steel.

The total cost of W12x65 column is $6,632.16
The total cost of W12x35 column is $2,236.08

TOTAL COST = $9,114.20
TOTAL TIME = 56 HRS (for one person)

COST ESTIMATE OF THE ORIGINAL MOMENT FRAME FOOTINGS

The footing cost estimate includes the cost of the following:

- Bulk excavation
- Hand Trim
- Compacted backfill
- Formwork, 4uses
- Reinforcing, Fy = 60 ksi
- Dowel or anchor bolt templates
- Concrete, f'c= 3000 psi
- Place concrete, direct chute
- Screed finish

To calculate the cost more accurately the values were interpolated between the smaller and larger sizes of the footing. Then these values were changed to dollar amount per cubic foot.

- Material $5.50 /cf
- Labor $4.25 /cf

  TOTAL $9.75 /cf

The changes were made to all braced frame footings and all typical footings. To calculate the cost of each footing the volume of the footing was calculated and multiplied by $9.75. Bellow is the total cost of each size of the footing.

• 34 typical footings (11'x11'x28")	$93,602
• 4 footings at frames C and D (8'x8'x28")	$5,824
• 2 footings at frame G	$4,550
TOTAL	**$103,976**

COST ESTIMATE OF THE REDESIGNED AND NEW FOOTINGS

The footing cost estimate includes the cost of the following:

- Bulk excavation
- Hand Trim
- Compacted backfill
- Formwork, 4uses
- Reinforcing, Fy = 60 ksi
- Dowel or anchor bolt templates
- Concrete, f'c= 3000 psi
- Place concrete, direct chute
- Screed finish

There redesign incorporates 30 typical footings. Each of the braced frame footing was redesigned. Bellow is the summary of all the redesigned footings along with the total cost of each type of footing.

• 30 typical footings	10'x10'x22"	$64,890
• 4 frame A and E footings	12'x12'x22"	$10,296
• 2 NEW footings at frames B and F	10'x10'x16"	$2,730
• 4 footings at frames C and D	10'x10'x20"	$6,500
• 2 footings at frame G	11'x11'x22"	$4,326
TOTAL		**$88,962**

Shown bellow is the table with original and redesigned sizes, their individual costs, and the difference of the original and redesigned footing.

FOOTING SIZE AND COST					
Footing	Original	Cost ($)	Redesigned	Cost ($)	Difference ($)
Typical	11'x11'x28"	2753	10'x10'x22"	2163	590
Frame A	11'x11'x28"	2753	12'x12'x22"	2574	179
Frame B	10'x10'x24"	1950	10'x10'x16"	1300	650
Frame C	8'x8'x28"	1456	10'x10'x20"	1625	-169
Frame D	8'x8'x28"	1456	10'x10'x20"	1625	-169
Frame E	11'x11'x28"	2753	12'x12'x22"	2574	179
Frame F	10'x10'x24"	1950	10'x10'x16"	1300	650
Frame G	10'x10'x28"	2275	11'x11'x22"	2163	112

COST COMPARISON BETWEEN TWO SYSTEMS

ORIGINAL SYSTEM			REDESIGNED SYSTEM		
Structural component	Total Cost ($)	Time (weeks)	Structural Component	Total Cost ($)	Time (weeks)
Moment Connections	191250	6.43	Shear Connections	54736	1.55
Moment Frame Columns	168,779.00	NA	Diagonal Braces	40956	1.34
Original Footings	103976	NA	Redesigned Columns	154627	NA
			New Columns	9114	0.2
			New and Redesigned Footings	88962	0.143
TOTAL COST	**464005**	**$**			
TOTAL TIME (7 people)	**6.43**	**weeks**	**TOTAL COST**	**348395**	**$**
			TOTAL TIME (7 people)	**3.233**	**weeks**

The values shown above are the values that were affected by the redesign of the lateral force resisting system. The total cost of the affected original system is $464,005. The total cost of the equivalent redesigned system is $348,395. According to these calculations the original system is more expansive by $115,610. There were several assumptions made prior to these calculations. The difference should be reduced by 25% to accommodate for the error from the earlier stated assumptions. Thus the final difference in cost is **$92,488.**

IMPACT ON THE CONSTRUCTION SCHEDULE

Like most modern commercial constructions, the schedule of the Northbrook Corporate Center is overlapping. It means that structural steel frame is already being erected while the footings are still being poured on the other side of the building. This implies that the 3 week difference between the two systems does not mean that the original design will take 3 weeks longer to build. Because different jobs overlap one another the 3 week difference does not have a significant impact on the schedule. In this comparison the cost is the only significant factor.

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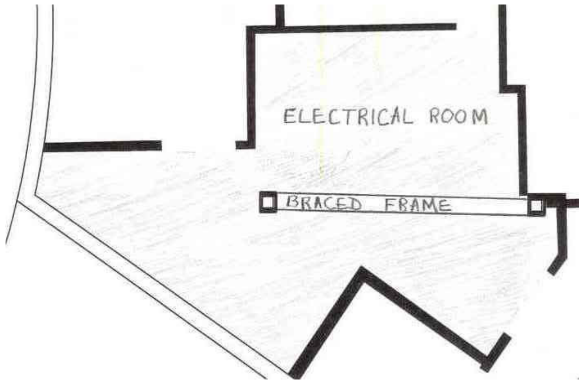
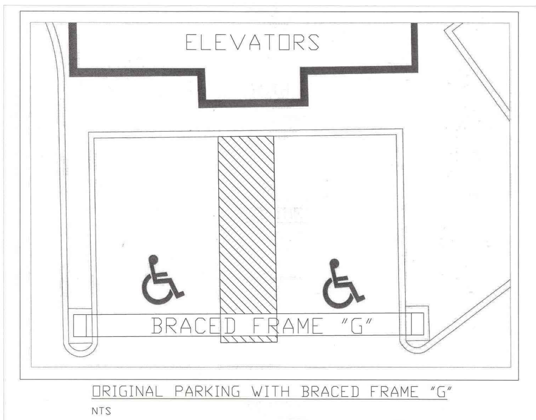
INTERIOR WALL REDISIGN

Breadth Study - Architectural Redesign



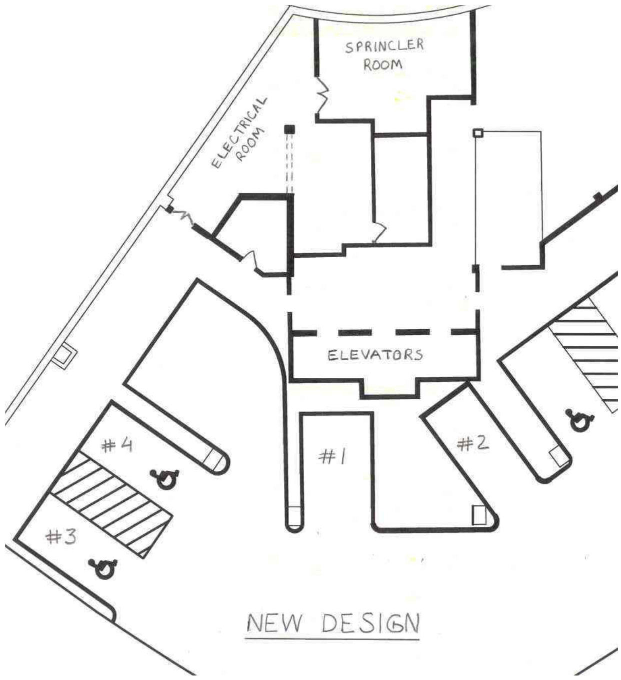
The similar layout of the building allows the braced frames to span through all four stories without becoming an obstacle to the interior space. When the braced frame systems enter the garage level, however, its integration with the layout becomes a problem. In the electrical room, a braced frame intersects with the main walk path of the interior space (see the top drawing on the right). Even though it is not a 'living space' the diagonal member of the frame can become an obstacle at the time of the installation of the large electrical equipment. The code requires a certain number handicap parking spaces with an area for loading the wheal chair. Unfortunately, the braced frame labeled "G" is positioned at the entrance of the two handicap parking spaces (see the bottom drawing on the right). Due to these complications the interior layout of the garage floor will be modified as a part of the breadth study analysis.



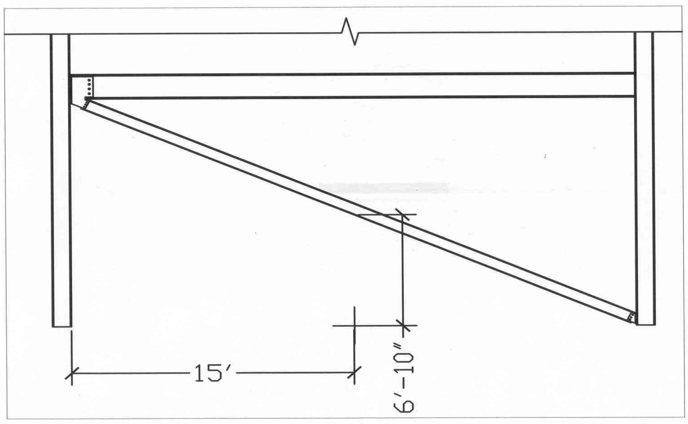
Several issues were considered prior to redesigning the layout:

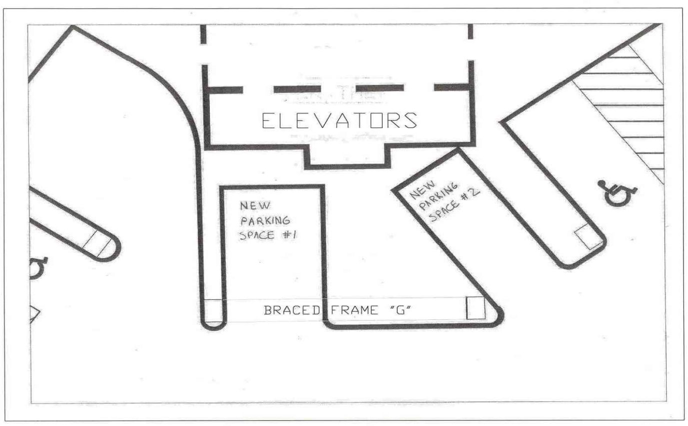
1. The number of parking spaces must be preserved.
2. New handicap parking spaces must have a loading area.
3. The distance between new walls and the existing column should not be less than the width of the main electrical room doors (56").
4. The redesign must not intervene with the current electrical and mechanical systems.
5. The area of the redesigned space must be preserved approximately.
7. The walkways and the redesigned space must be well integrated.

The telephone room's location has no significant importance as long as it is located close to the main traffic, close to the elevators, and can be accessed from the garage area. In the redesign the telephone room was moved closer to the elevators. The walls of the room do not extend beyond the main walls, and do not create a problem for the elevator traffic. Because the room was moved only about 8 feet, the relocation creates no problem to the mechanical or electrical design. As a result of the relocation, the interior space of the electrical room has changed favorably. The entrance to the electrical room has been moved to where the telephone room was located initially. Half of the braced frame wall is shared with the telephone room, while the other half continues into the electrical room. This second half, however, is no longer an obstacle because the diagonal at this length of the frame has sloped to 7 feet high and continues to slope upward until it meets the column. The area where all of the main electrical equipment is to be placed has been left untouched.

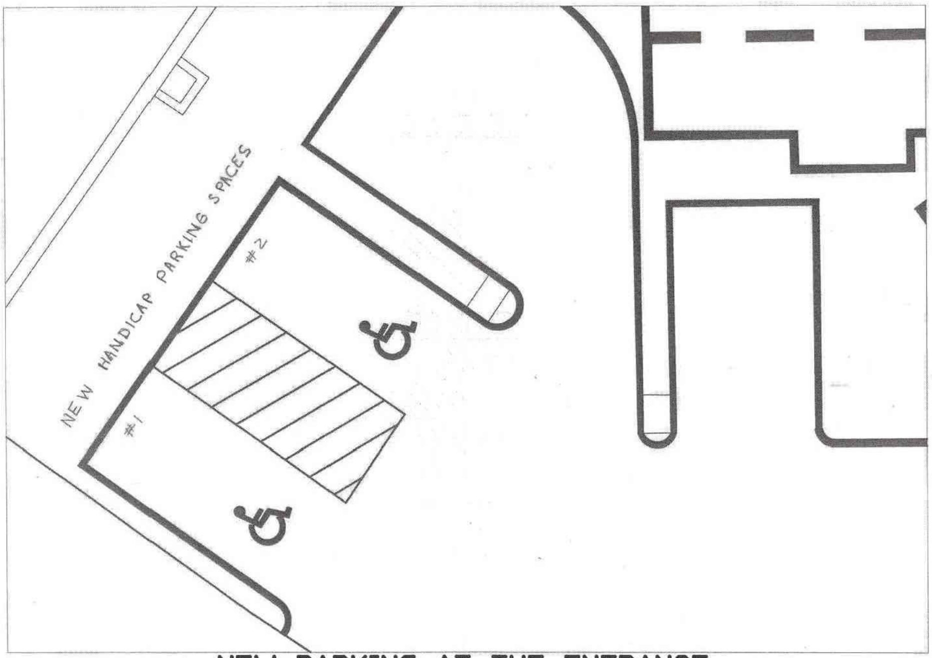


In the garage, the two handicap parking spaces have replaced the three parking spaces at the exit of the garage, as shown below in the drawing on the left. The diagonal member in the “G” frame is sloping upward from right to left. Hence there is enough room for one parking space on the left, where the original handicap space was located prior to redesign. As shown in the drawing on the right , the vertical distance from the ground to the diagonal member at the midspan of the frame is almost 7 feet, and approximately 8 feet where the new parking space begins. That is more than enough for most vans and trucks. Another parking space was placed in between the two existing columns (see the drawing), and appears to continue the line of existing parking spaces. Unfortunately, the redesigned garage space has lost one parking space. The intent to preserve the total number of the parking spaces proved to be unsuccessful.





NEW PARKING UNDER THE BRACED FRAME 'G'
NTS



NEW PARKING AT THE ENTRANCE

NTS

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SUMMARY AND CONCLUSION

Northbrook Corporate Center, a five story building, incorporates moment frames to resist the lateral forces. To successfully withstand the wind forces the structural design of the lateral system uses moment connections at every joist/beam to column connection. The building uses a total of 750 connections of different moment capacities. The moment frames are generally very expensive. In this case the price of moment connections alone was calculated to be $190,000. Because of the costly nature of the moment frame lateral force resisting system, the system was redesigned. The new system was designed of braced frames with shear connections. The frames were placed into the permanent walls of the building to preserve the flexibility of the interior space of the original system. Because braced frames are much more rigid than the moment frames, the new design incorporated only 63 braced frame shear connections. The remaining shear connections were of less complexity and, hence, less cost. The total cost of the redesigned system included all connections, new columns, diagonals, and new footings and was calculated to be $348,395. The cost of the affected original system is calculated to be $464,005. The difference between the moment frame system and the braced frame system was calculated to be $92,488. The redesigned system had very minimal impact on the schedule. The difference was calculated to be 3 weeks, which is insignificant when the jobs are overlapped.

There were several problems with the redesigned system. First of all, the redesign of the lateral load resisting system became an obstacle to the interior space at the garage level of the building. This fact created a need to redesign the layout of the interior space of this floor level. The telephone room was moved closer to the elevators, and two handicap parking spaces were moved to a different location. Unfortunately, one parking space was lost in the process. Secondly, there was an error in the seismic load development section of this report which introduced a level uncertainty about the accuracy of the redesign and its consequences. And lastly the overturning moments of frames C and D are problematic. This design became more of the uncertainty when the error in the seismic design was discovered.

The detailed study of this report has accented on the redesign of the lateral load resisting system. The basis for the redesign was motivated by the costly nature of the original system. The redesigned proved to be less expansive than the original design. The redesign introduced several unknowns, however. The braced frame design created several obstacles to the overall interior layout. The span of the columns of the braced frame C and D creates potential problem with the overturning moment. The errors in the seismic design only add to this growing level of uncertainty. With these unknowns, the cost comparison alone is not sufficient enough to conclude that the redesigned, braced frame system is more feasible system for the building. At this point, the braced frame lateral load resisting system needs to be evaluated further in order to compete with the existing system. Because there is not enough evidence that the redesigned system is satisfactory, this report concludes that the braced frame lateral force resisting system is not a more feasible system for the Northbrook Corporate Center.

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estudio - eResources
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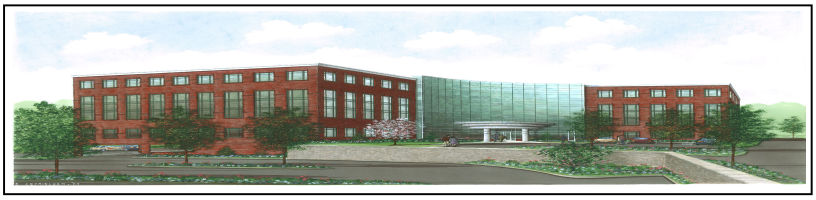

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CREDITS AND AKNOWLEDGEMENTS

I would like to express my appreciation to the entire architectural engineering faculty for the direct or indirect contribution of each one during my five year experience at Penn State University.

Special Thanks to:

Dr. Ali Memari
M. Kevin Parfitt
Dr. Louis F. Geschwinder
Dr. Linda Hanagan

Thanks to my family and friends who supported me at all the times.