NORTHBROOK CORPORATE CENTER

DIMITRY A. REZNIK

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

FIRE PROTECTION:

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

***MECHANICAL*:**

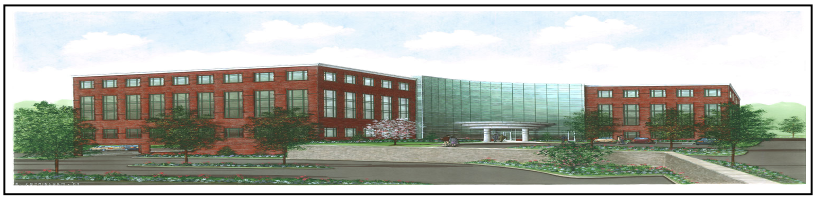
Heating/cooling by fan powered air volume system.

***STRUCTURAL / ARCHITECTURAL*:**

Steel columns-girders-joists-decking

Insulated flat roof- decking on steel joists

Exterior materials: brick, stone, glass



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ARCHITECTURAL REDESIGN

DIMITRY A. REZNIK

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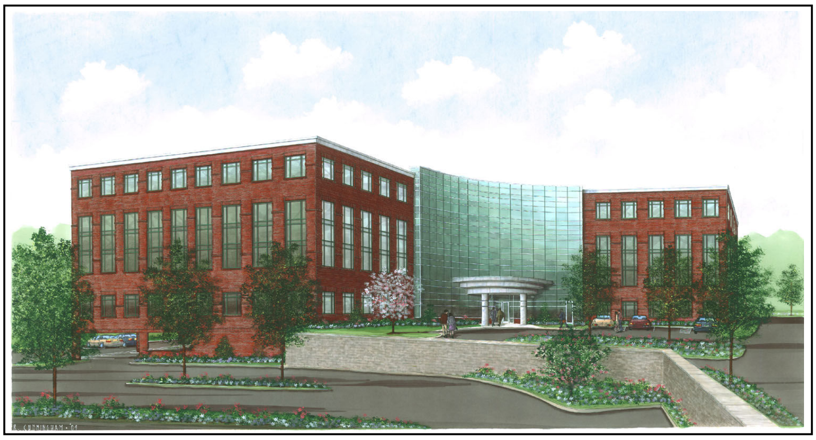
ADVISOR: DR. MEMARI
NORTHBROOK CORPORATE
CENTER

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. .

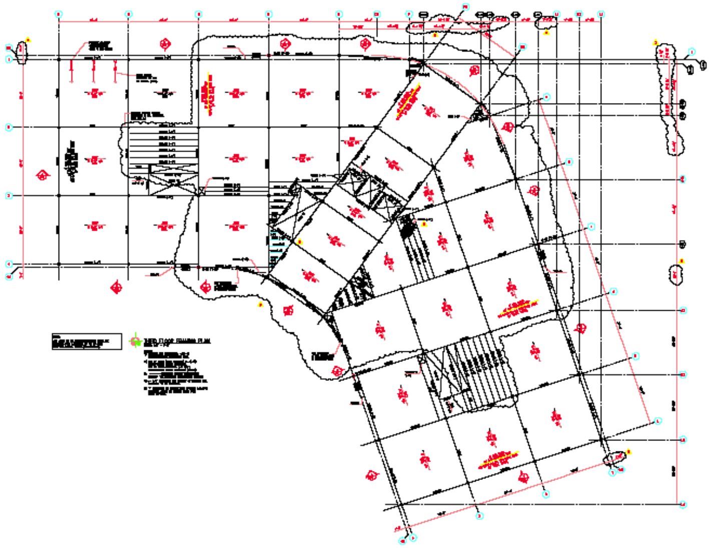
Transportation:
There are two entrances into a development in which the building is constructed. There is only one entrance from the Northbrook Drive into the building's drive way, which later subdivides and leads to parking lots. Northbrook Corporate Center has a main entrance into the building, and 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

$S_S = 0.335$ (MAP 9.4.1.1a)

$S_1 = 0.085$ (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

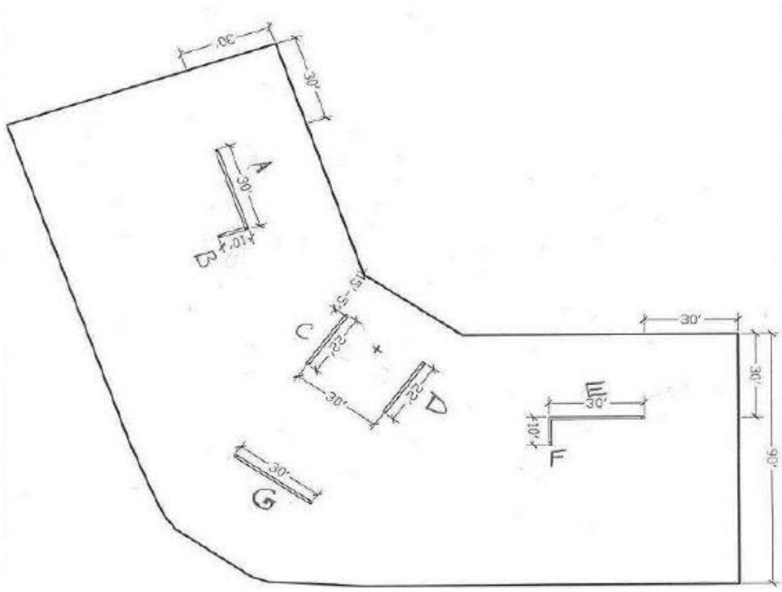
Wind k=1.005	Lev	h	Seismic (plf)	Wind (plf)
Roof	5	7	6.9	400
Office	4	5	31	421
Office	3	4	23.3	386
Office	2	2	15.5	350
Office	1	1	7.7	328
Total			84.4	1885

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



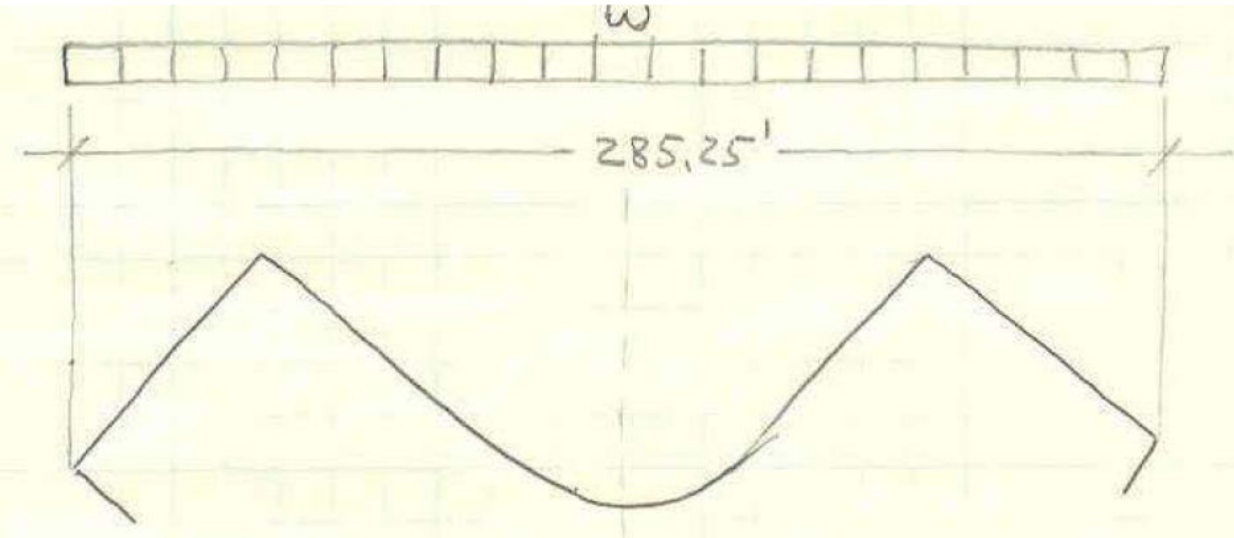
To preserve the flexibility of the moment frame system, the braced frames are designed into the frames with permanent walls. On the very far end of the building, as shown in the drawing, two braced frames wrap around the star well. The shortest wall of the stair well does not span the full length between 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.

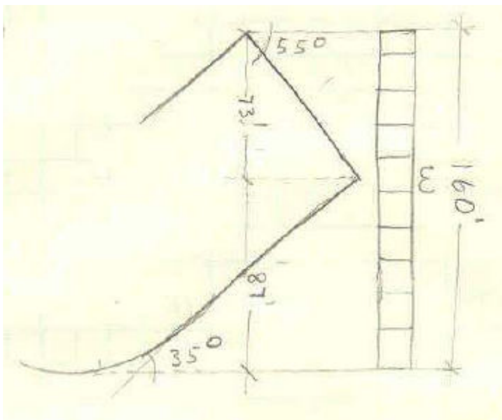
Wind Load Distribution

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:

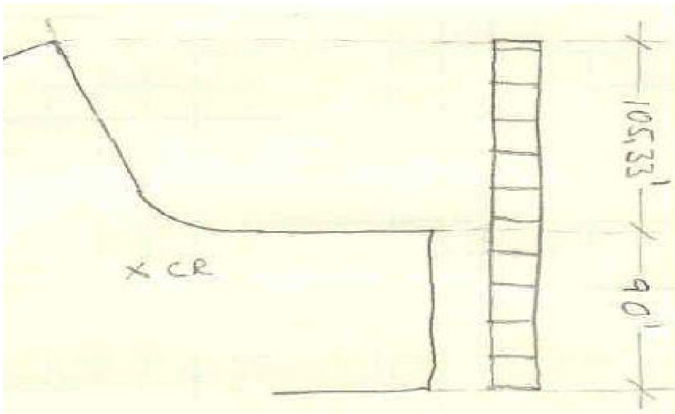
Case 1: wind parallel to y-axis



Case 2: wind parallel to x-axis



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

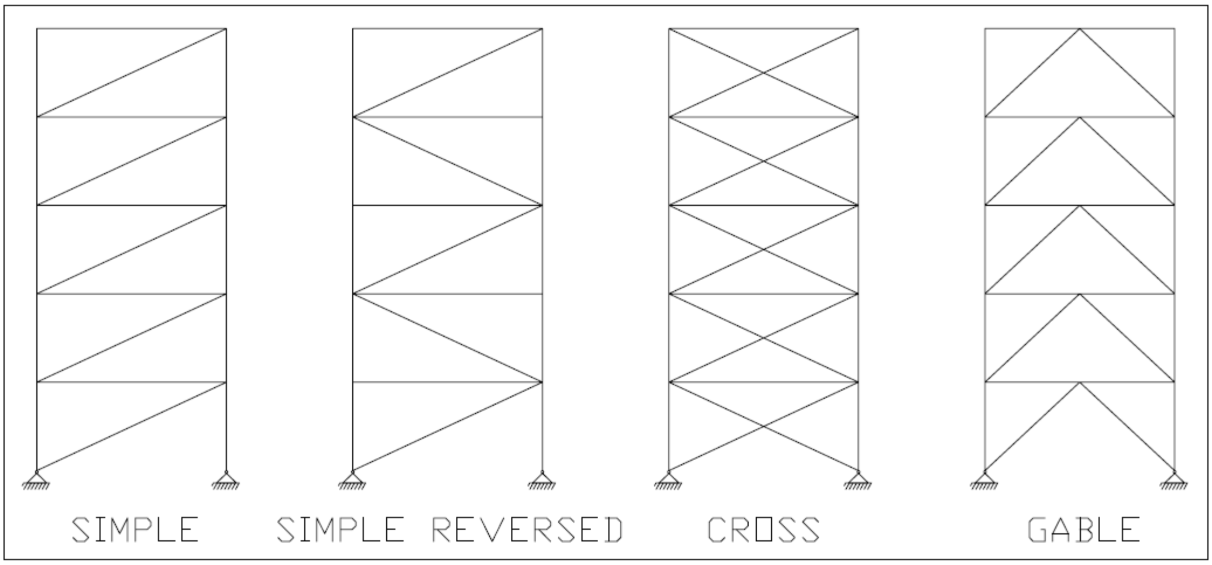


Due to the symmetry of the building, the wind in the y-axis direction does not produce torsional forces in the building. The building is not symmetrical in the x-axis direction; therefore, any wind case not parallel to the y-axis produces the torsional forces in the building. The center of the rigidity 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.

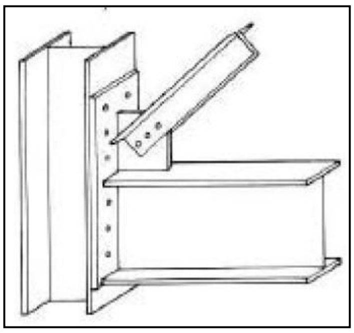
Because the building's x-axis dimension is the largest dimension, case 1 is expected to be the controlling case. The case comparison tables shown below prove the expectation to be correct. The braced frame "G", however, is governed by case 2. In case 1, the "G" frame is positioned perpendicularly to 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.

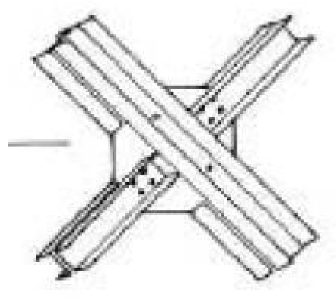
The simple diagonally braced frame (first from the left) incorporates a total of thirteen connections. Nine of these connections are shear connections that service both the beam and the diagonal brace and could look something like the one shown on the right (drawing is from www. ocw.mit.edu). One connection 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

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.

<u>REDESIGN OF BEAMS</u>

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

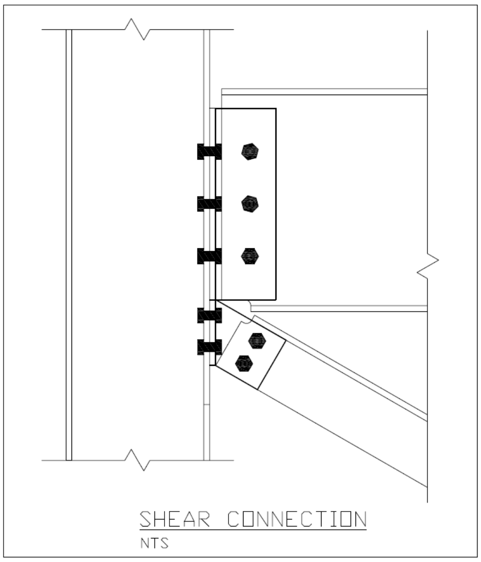
Lateral Loads:

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.

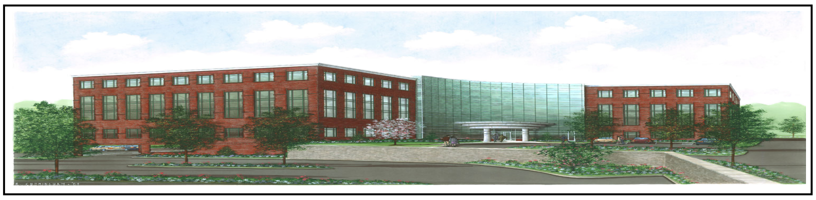
STORY DRIFT (inches)						
Frame	Level 1	Level 2	Level 3	Level 4	**Roof**	**Allowable**
Frame A	0.1573	0.4799	0.9158	1.327	**1.579**	**2.333**
Frame B	0.1253	0.4541	0.9167	1.401	**1.709**	**2.333**
Frame C	0.1128	0.3905	0.7031	1.021	**1.186**	**2.333**
Frame D	0.1128	0.3905	0.7031	1.021	**1.186**	**2.333**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

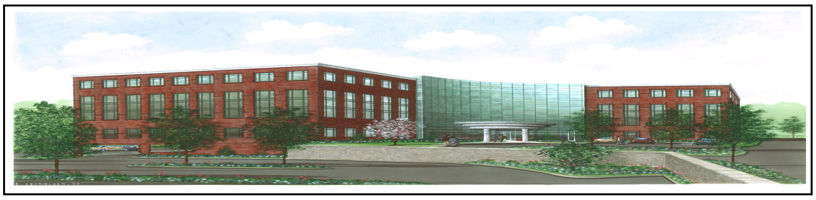
$R = 8$ *R should be 3* (TABLE 9.5.2.2)

I

$I = 1.0$ (TABLE 9.1.4)

T PERIOD

$T_a = 0.57$ (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.

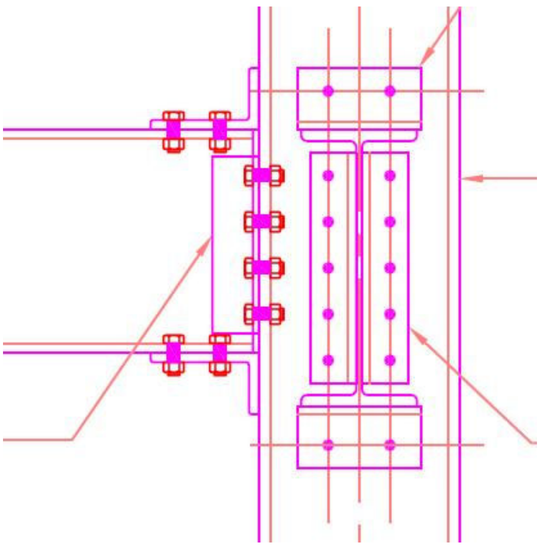
This breadth study will evaluate the cost of:

- 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:

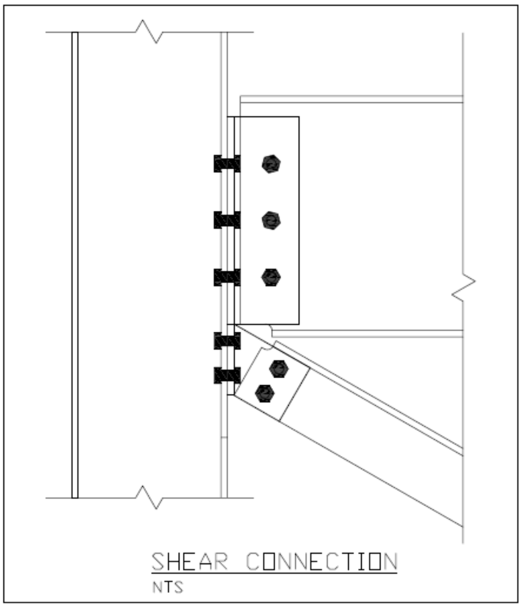
• Bolts	$110	
• Weld	$96.5	
• Angle	<u>$48.55</u>	
TOTAL:	$255	($97.45 of which is for labor)

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	<u>$4.25 /cf</u>
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	<u>$4,550</u>
TOTAL	**$103,976**

<u>COST ESTIMATE OF THE REDESIGNED AND NEW FOOTINGS</u>

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

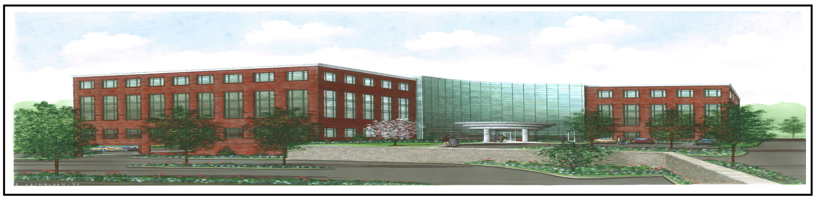
COST COMPARISON BETWEEN TWO SYSTEMS

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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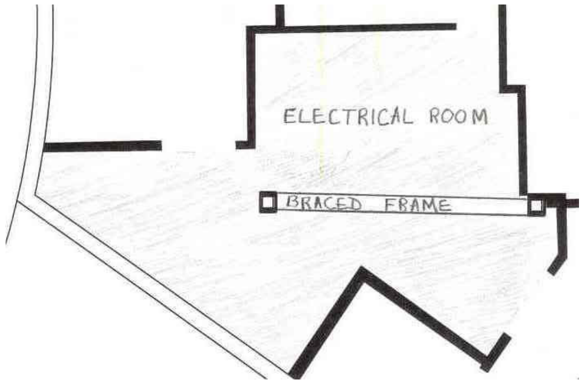
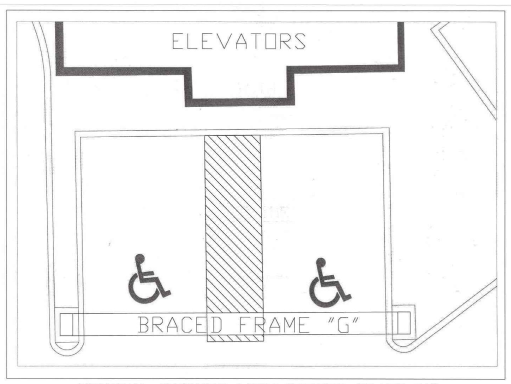
INTERIOR WALL REDISIGN

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.

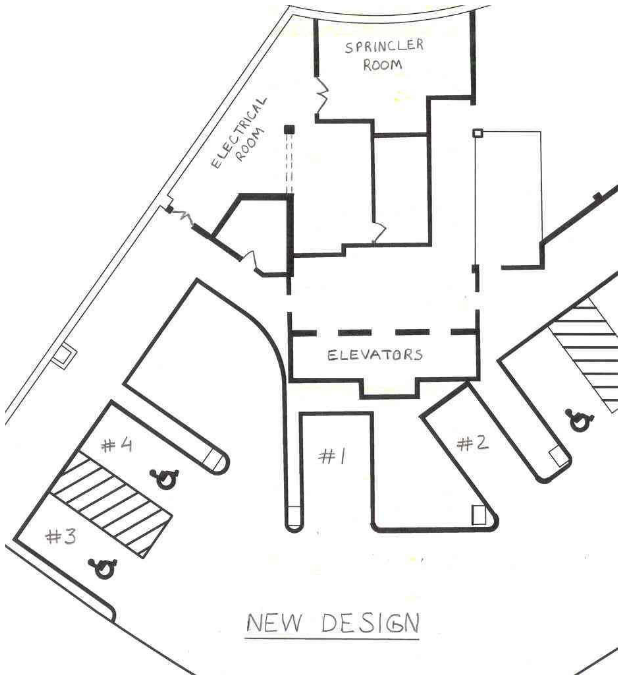


ORIGINAL PARKING WITH BRACED FRAME "G"
NTS

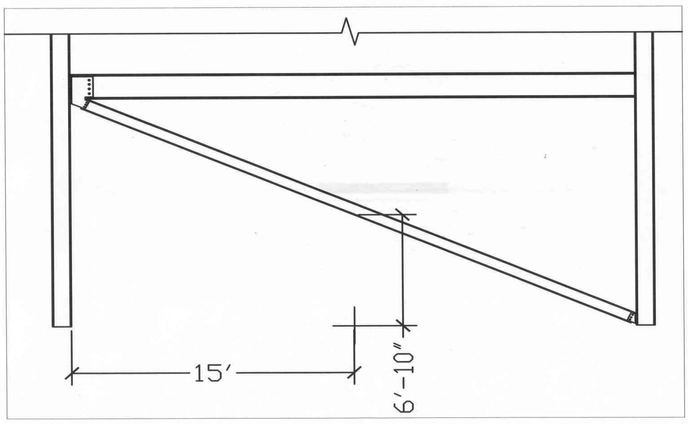
Several issues were considered prior to redesigning the layout:

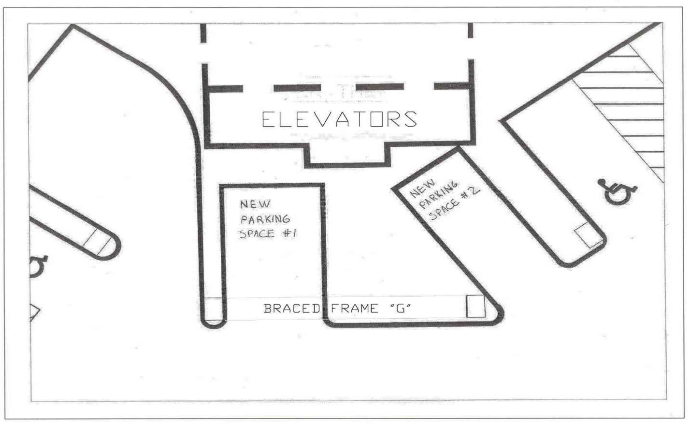
1. The number of parking spaces must be preserved.
2. New handicap parking spaces must have a loading area.
3. The distance between new walls and the existing column should not be less than the width of the main electrical room doors (56").
4. The redesign must not intervene with the current electrical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

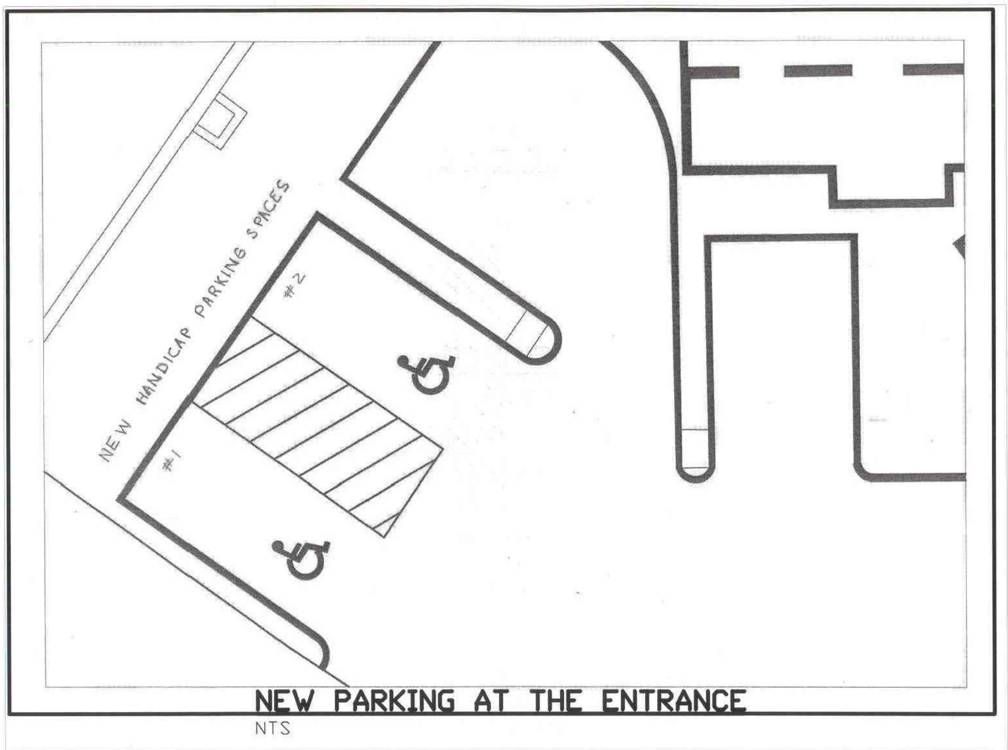


In the garage, the two handicap parking spaces have replaced the three parking spaces at the exit of the garage, as shown below in the drawing on the left. The diagonal member in the “G” frame is sloping upward from right to left. Hence there is enough room for one parking space on the left, where the 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 HANDICAP PARKING SPACES
#2
#1
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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IBC 2003: "International Building Code."

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estudio - eResources
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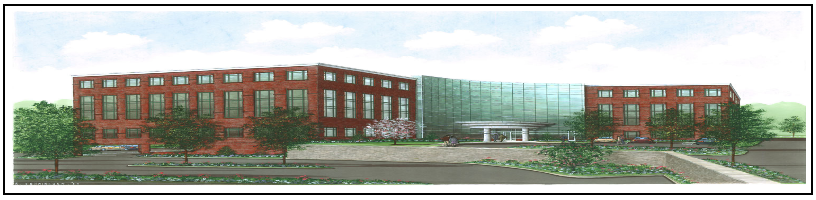
American Institute of Steel Construction



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Special Thanks to:

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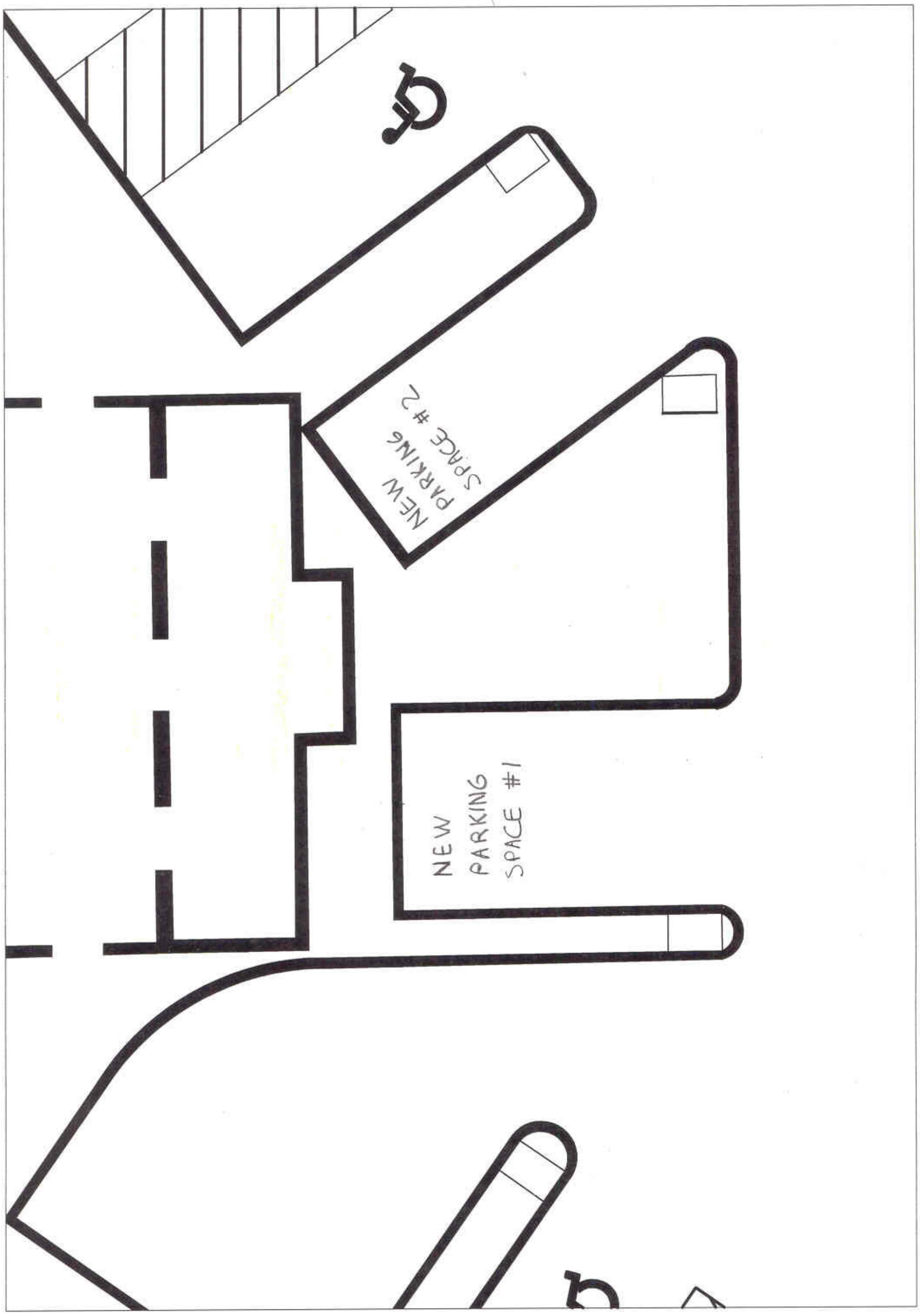
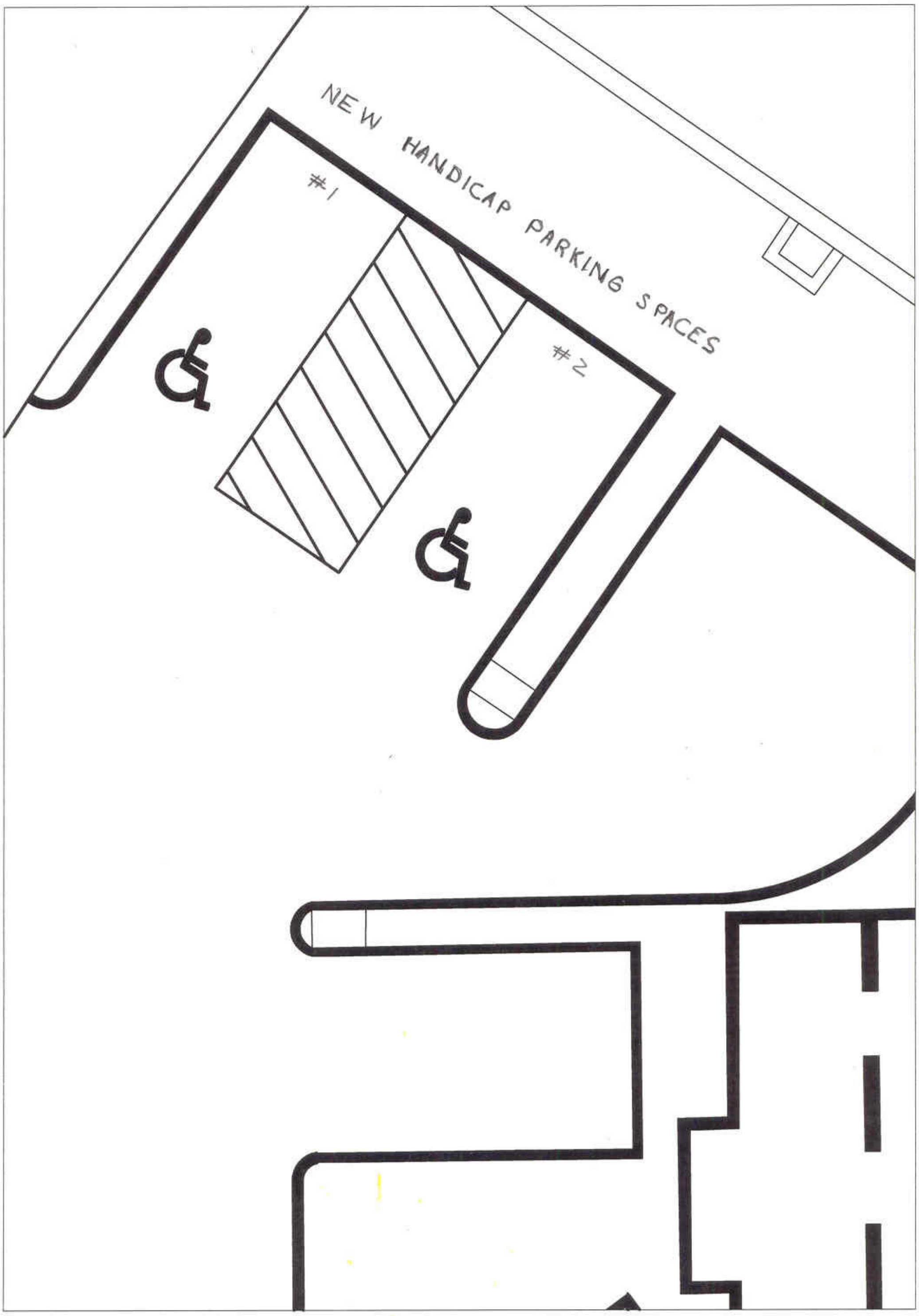
M. Kevin Parfitt
Dr. Louis F. Geschwinder
Dr. Linda Hanagan

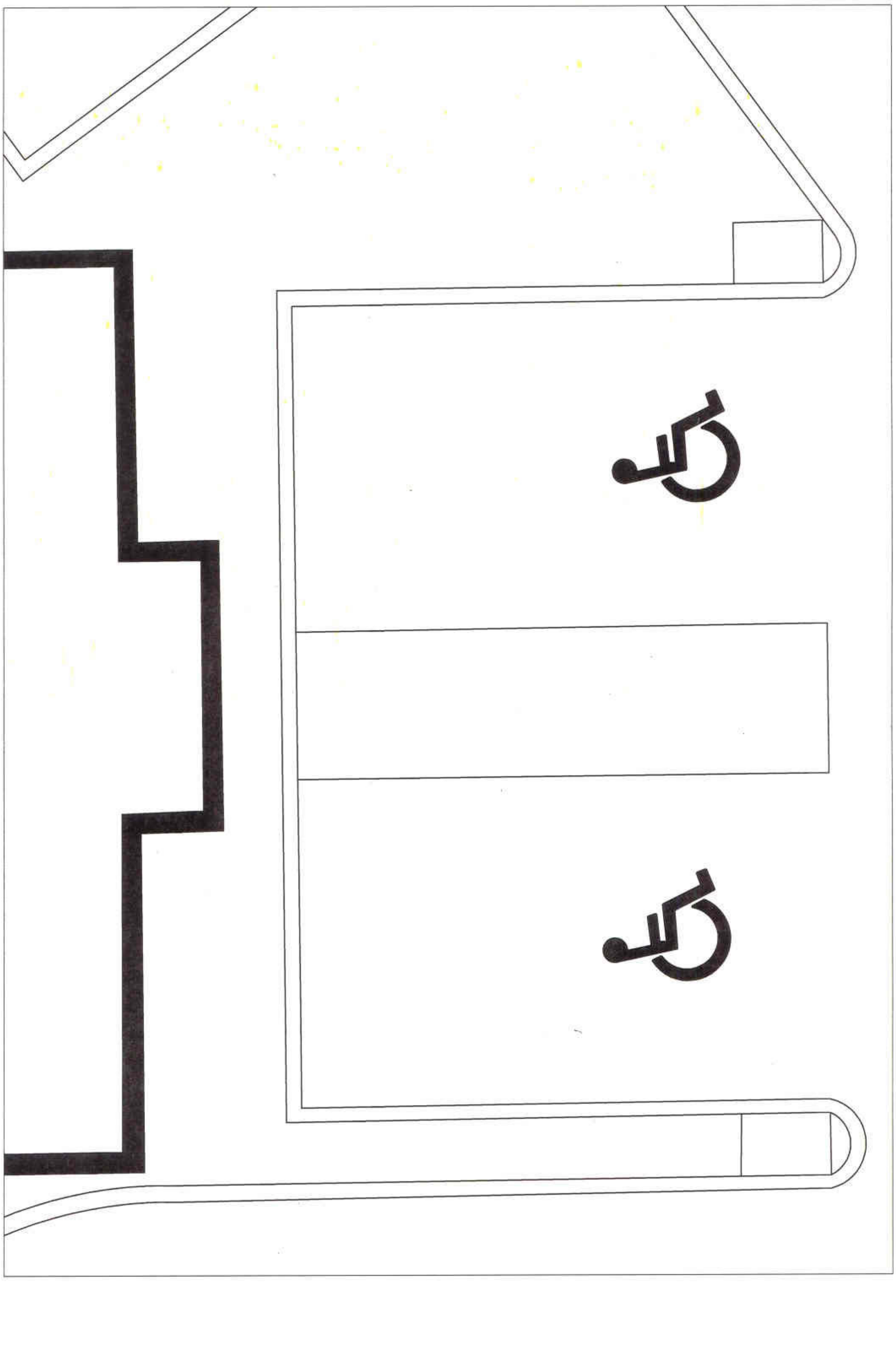
Thanks to my family and friends who supported me at all the times.

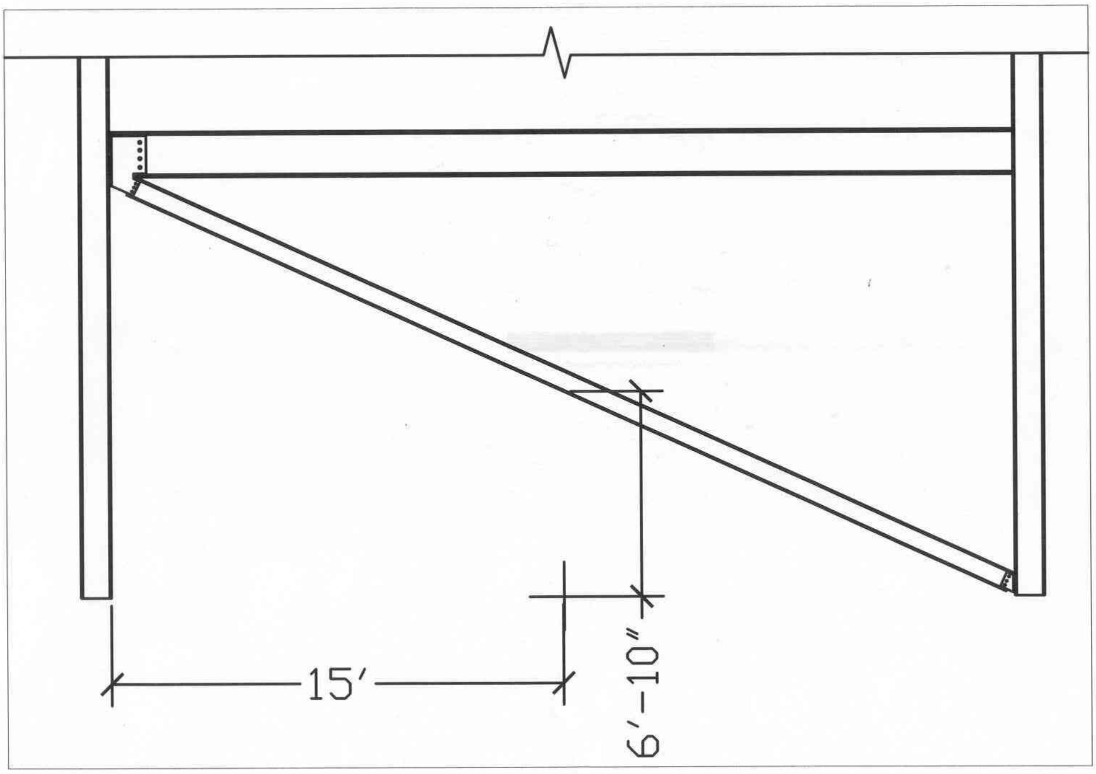
THE APPENDIX

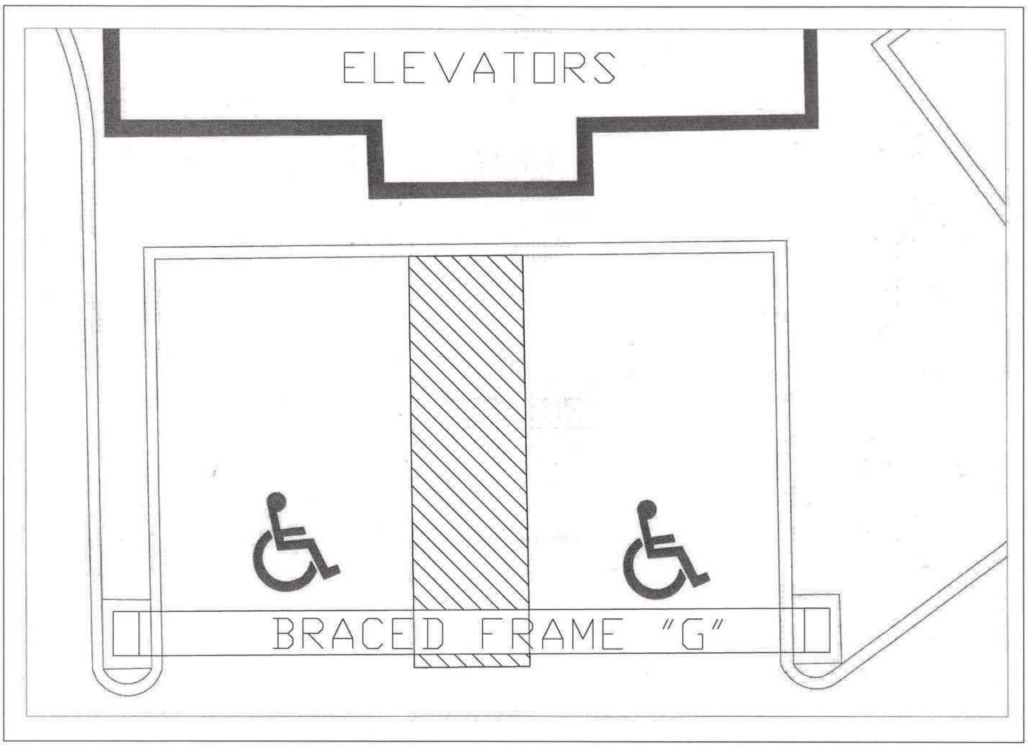
Architectural Redesign


NEW PARKING SPACE #2
NEW PARKING SPACE #1


NEW HANDICAP PARKING SPACES
#1
#2

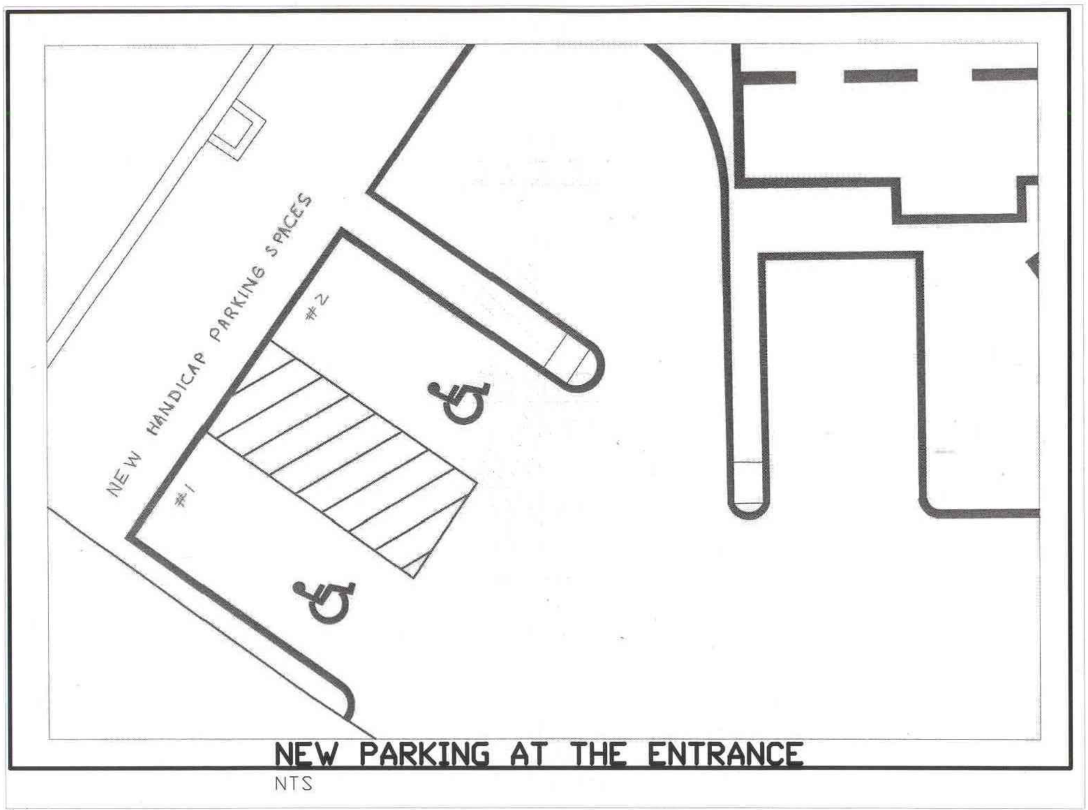


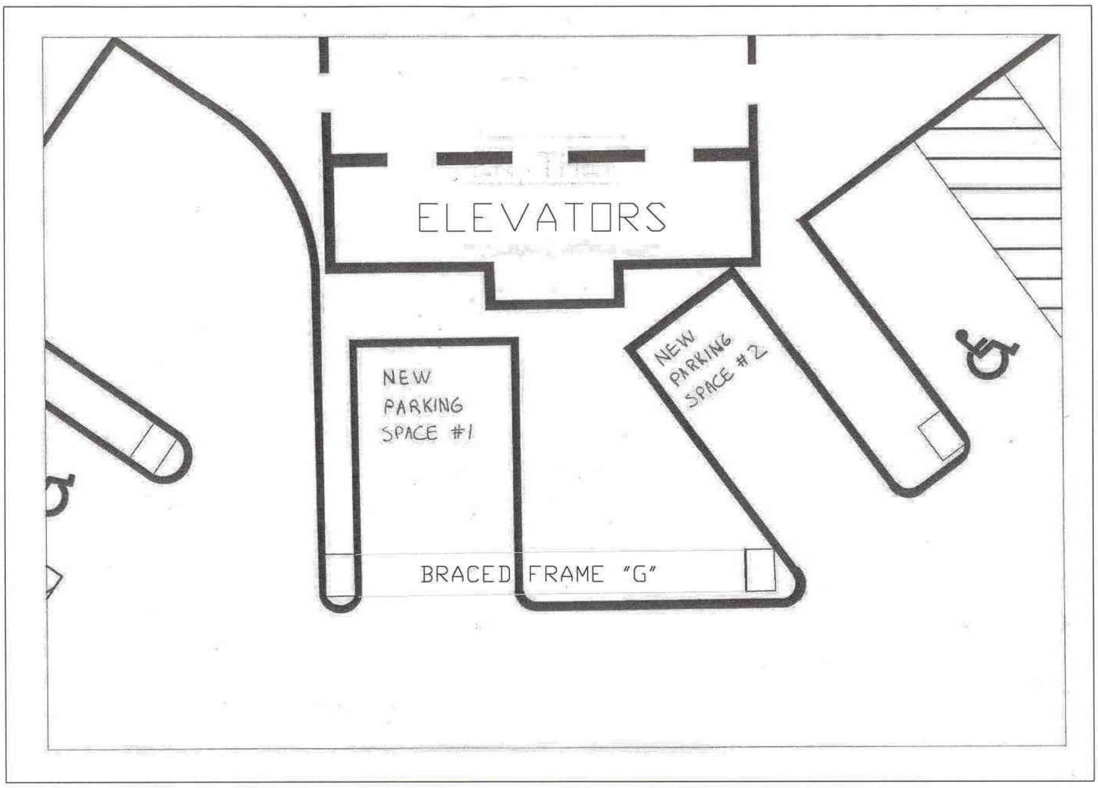




ORIGINAL PARKING WITH BRACED FRAME "G"

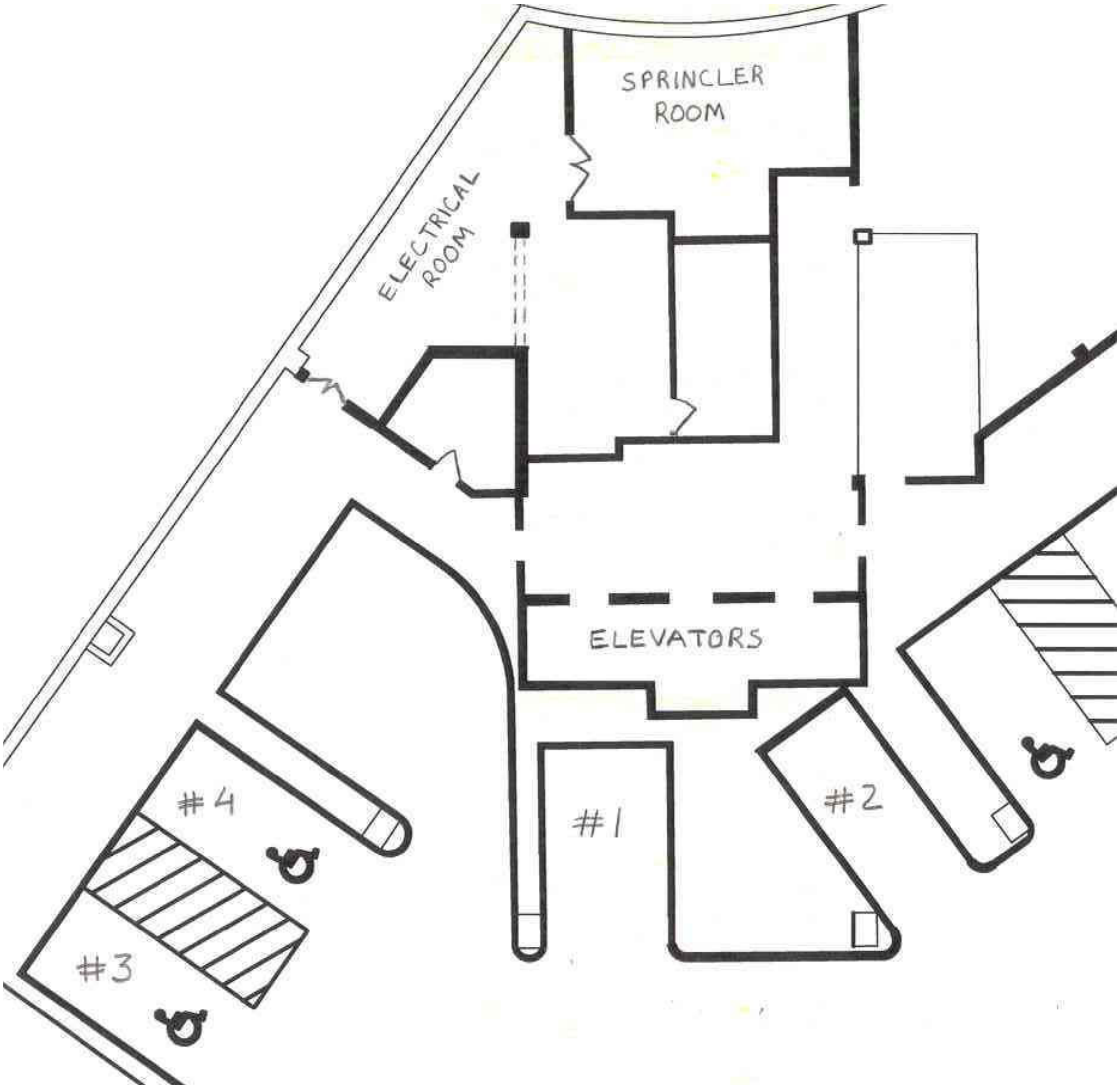
NTS


NEW HANDICAP PARKING SPACES
#1
#2
NEW PARKING AT THE ENTRANCE
NTS

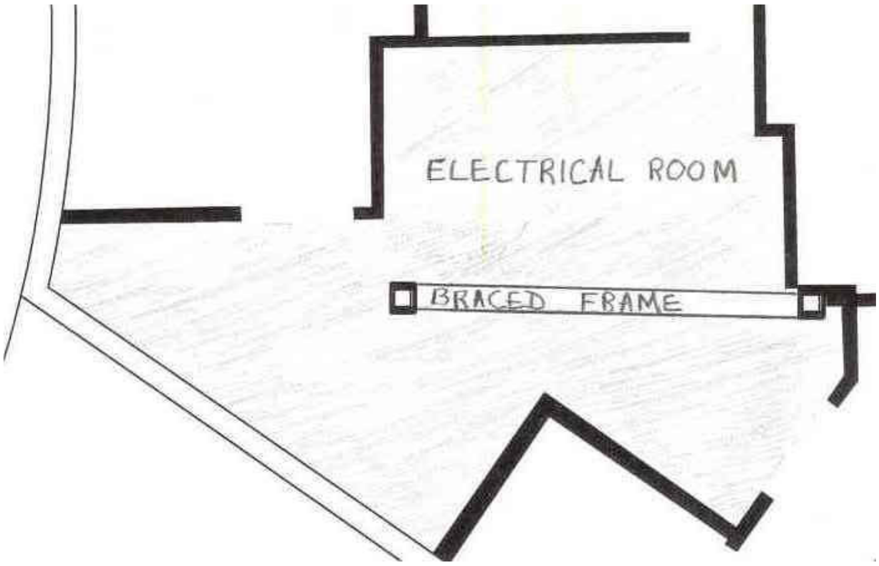


NEW PARKING UNDER THE BRACED FRAME 'G'

NTS



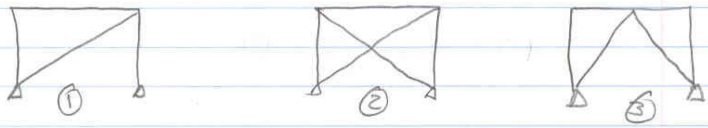
NEW DESIGN


ELECTRICAL ROOM
BRACED FRAME

Braced Frame Analysis

Brased frame Analysis (1)

3 Possibilities:

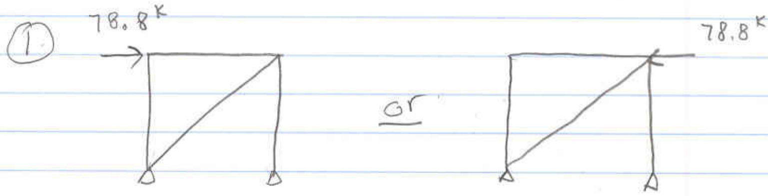


Design for Compressive strength	Design for tension strength or Combination of tension and compression	Design for compression strength.

Resuts

weight of Diagonals	1970 lbs or 1257 lbs	540 lbs	1373.5 lbs
# of Additional connections	2	4	3
# of diagonal members	1	2	2
Least Labor Intensive 1-3	1	3	2

Braced frame Analysis: (2)



Calculated on STAAD 2004

Diagonal Axial Force: 87^k (T)

Diagonal Axial Force: $\boxed{87^k \text{ C}}$

controls

USE:

$\boxed{\text{Result: W 8 x 58}}$

• Total weight: (33.1 FT) • 58 lb = 1920 lb

Also Try WT 9 x 38

$\phi P_{nx} = 92.6$ OK

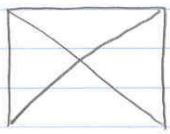
$\phi P_{ny} = 96.1$ OK

$\frac{Kl}{r} < 200$ OK

Tot weight = 33.1 (38) = $\boxed{1257.8}$ lbs

Braced Frame Analysis: (3)

(2)



A: Design for Tension; diagonals work one of the time.

B: Design for a combination both diagonals work simultaneously.

A. Diagonal Axial Force = 87^k Tension

Assume the design is control by tension yielding:

$\phi_t P_n = \phi F_y A_g$

$87^k = .9(50)(A)$

$A = 1.93 \text{ in}^2$

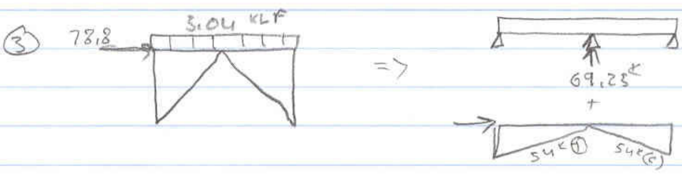
$\boxed{\text{USE } 2L\, 4 \times 4 \times \tfrac{5}{16}}$ (2 Angles)

Total weight = $2(8.16 \frac{lb}{ft})(33.1 \text{ ft})$

$= \boxed{540.2 \text{ lbs}}$

B. Compressive strength of small, long members is very small; the tension-based design is more economical.

Brose Frome Analysis (4)



Compression controls

=> Diogonal Axiol Force = $54^K + 69.25^K = 123.25^K$

TRY WT 8 x 33.5

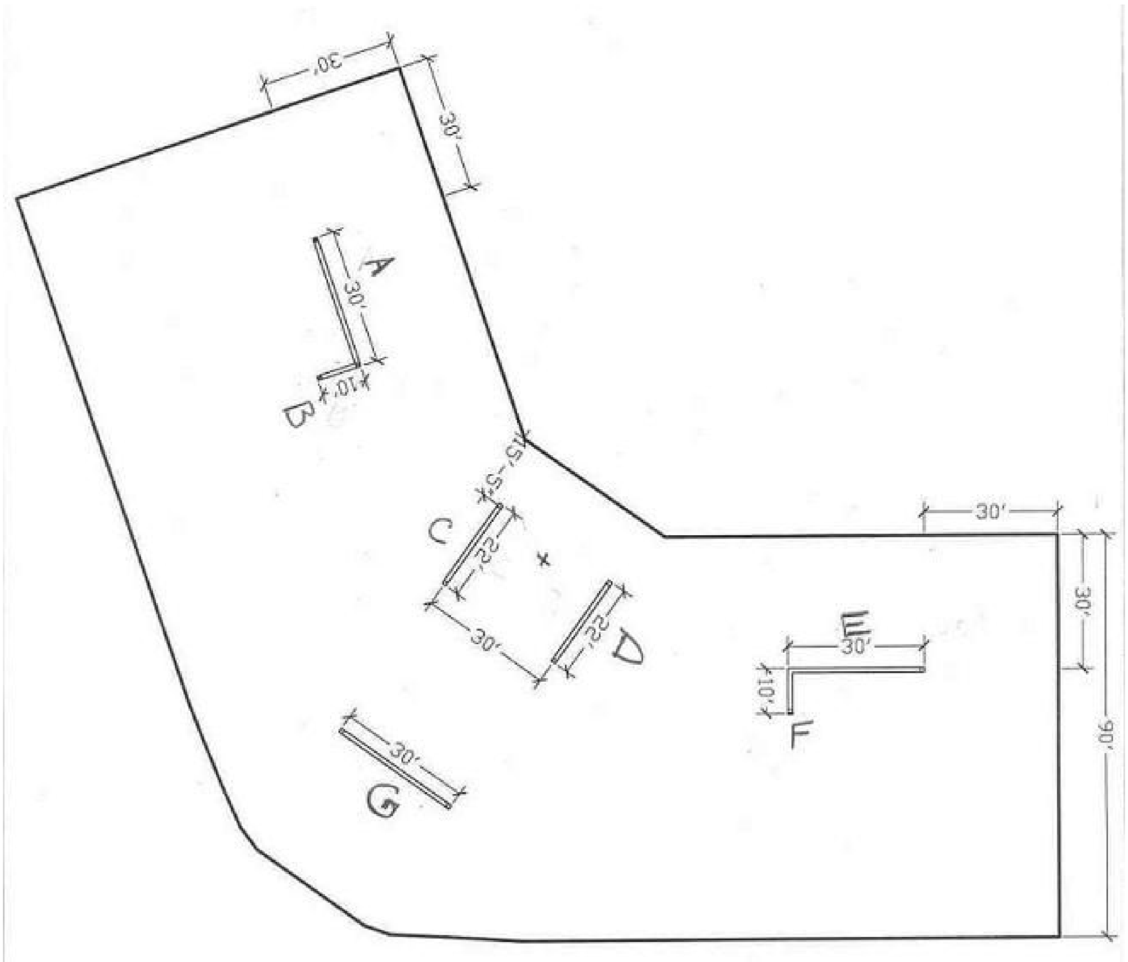
$\phi P_{n\,x} = 174^K$ OK

$\phi P_{n\,y} = 193^K$ OK

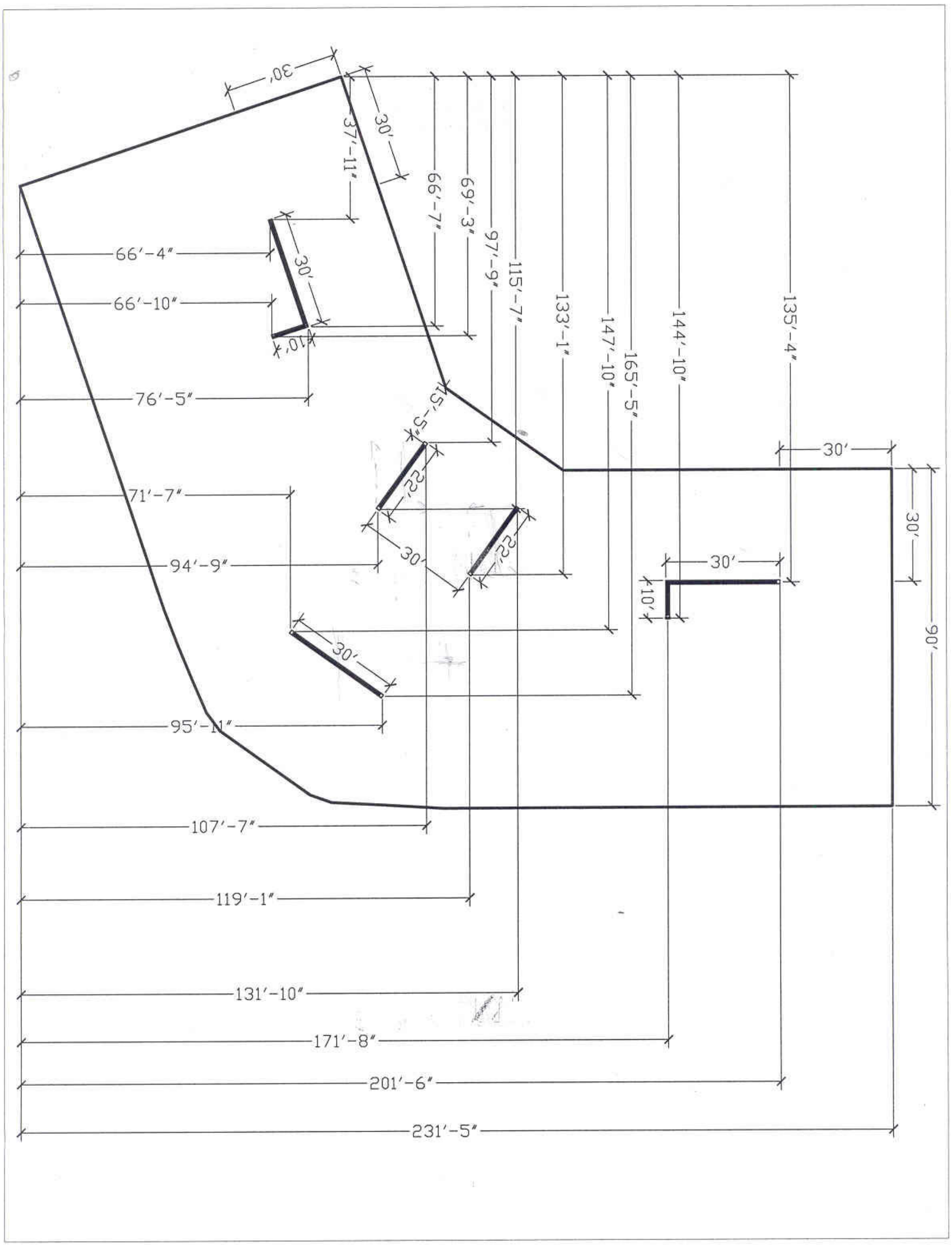
$\frac{Kl}{r} < 200$ OK

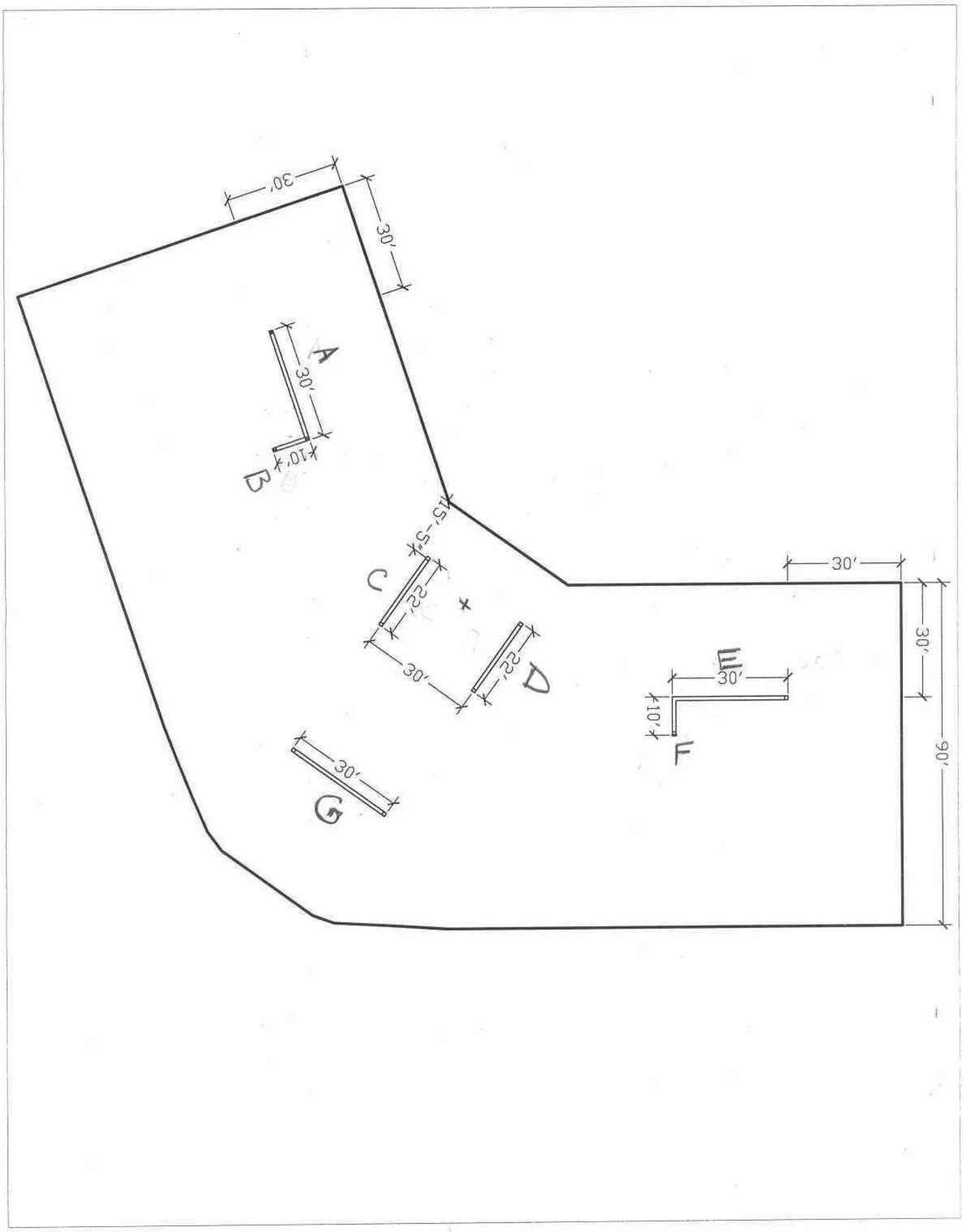
USE $\boxed{2\ \text{WT } 8 \times 33.5}$

TOT WEIGHT = 2(20.5')(33.5) = $\boxed{1373.5 \text{ lbs}}$

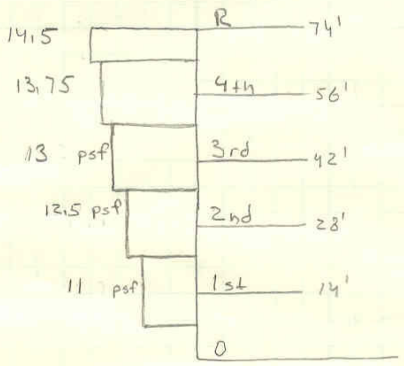


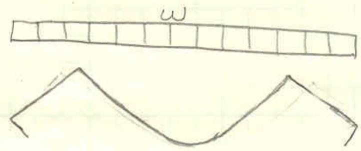
A
B
C
D
E
F
G
30'
10'
22'
90'


30'
30'
37'-11"
66'-7"
69'-3"
97'-9"
115'-7"
133'-1"
147'-10"
165'-5"
144'-10"
135'-4"
66'-4"
30'
66'-10"
10'
76'-5"
15'-5"
22'
71'-7"
22'
30'
94'-9"
30'
30'
30'
10'
30'
90'
95'-11"
107'-7"
119'-1"
131'-10"
171'-8"
201'-6"
231'-5"


A
B
C
D
E
F
G
30'
10'
22'
90'
15'-5"

Load Distribution (1)





Total distributed wind load at each story height

$w_{ROOF} = 10.5'(14.5 \text{ psf}) = 152.25 \text{ plf}$

$w_4 = 152.25 + 14'(13.75 \text{ psf}) = 344.75 \text{ plf}$

$w_3 = 344.75 \text{ plf} + 14'(13 \text{ psf}) = 526.75 \text{ plf}$

$w_2 = 526.75 \text{ plf} + 14 (12.5 \text{ psf}) = 701.75 \text{ plf}$

$w_1 = 701.75 \text{ plf} + 14 (11 \text{ psf}) = 855.75 \text{ plf}$

$w_0 = 855.75 \text{ plf} + 7.5 (11 \text{ psf}) = 938.25 \text{ plf}$

Load Distribution (2)

$W_R = 152.25$ plf

$W_4 = 344.75$ plf

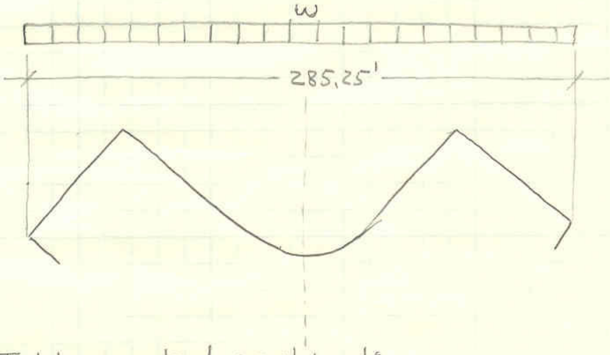
$W_3 = 526.75$ plf

$W_2 = 701.75$ plf

$W_1 = 855.75$ plf

$W_0 = 938.25$ plf

UNIFORMLY DISTRIBUTED WIND LOAD AT EACH STORY



Total equivalent point Load:

$W_R = 152.25(285.25) = 43.4^K$

$W_4 = 344.75(285.25) = 98.3^K$

$W_3 = 526.75(285.25) = 150.3^K$

$W_2 = 701.75(285.25) = 200.2^K$

$W_1 = 855.75(285.25) = 244.1^K$

Load Distribution (3)

THE CENTER OF RIGIDITY

$$\bar{x} = \frac{15'(22') - 15'(22')}{44} = 0$$

$$\bar{y} = \frac{32.5'(30') + 97.25'(5.88)(2) + 109.58(24.58)(2)}{30 + 2(5.88) + 2(24.58)} = 82.6'$$

POLAR MOMENT OF INERTIA

$I_p = I_{xx} + I_{yy}$

$I_{xx} = \sum \ell_x y^2$

$= 30'(50.17)^2 + 24.58'(27.\ )^2(2) + 5.88'(14.67)^2(2)$

$= 113{,}879.36\ ft^3$

$I_{yy} = \sum \ell_y x^2 = 22'(15^2)2 + 17.58(71.08)^2(2) + 8(61.75)^2(2)$

$= 248550.20\ ft^3$

$I_p = 113879.36 + 248550.20 = 362429.56\ ft^3$

Load Distribution (4)

Wind parallel to the $\bar{y}$ axis

Roof

$W_R = 43.4^k$ Wall Force $= \frac{W\ell_i}{\Sigma\ell} + \frac{M_T \times \ell}{I_P}$

$M_T = 0$

Wall $A_y = \frac{43.4^k(17.59')}{2(17.59)+2(22)+2(8)} + 0 = 8.02^k$

Wall $B_y = \frac{43.4(8)}{95.18} + 0 = 3.65^k$

$\boxed{\text{Wall } C = \frac{43.4(22)}{95.18} \qquad \text{Wall } D = \text{Wall } C}$

Wall $C = 8.67^k$

Wall $D = 8.67^k$

Wall E_y = Wall A $= 8.02^k$

Wall F_y = Wall B $= 3.65^k$

$\boxed{36.54} \approx 36.7$ OK

Wall A $= \frac{8.02}{\cos 55} = 13.98^k$

Wall B $\frac{3.65}{\cos 35} = 4.46^k$

Wall E = Wall A $= 13.98^k$

Wall F = Wall B $= 4.46^k$

Load Distribution (5)

Wind Parallel to Y-axis

4th story

$W_4 = 98.3^k$ $M_T = 0$

	(k)
Wall A	= 31.67
Wall B	= 10.092
Wall C	= 22.72
Wall D	= 22.72
Wall E	= 31.67
Wall F	= 10.09

3rd story

$W_3 = 150.3$ $M_T = 0$

	(k)
Wall A	= 48.43
Wall B	= 15.42
Wall C	= 34.74
Wall D	= 34.74
Wall E	= 48.43
Wall F	= 15.42

2nd story

$W_2 = 200.2^K$ $M_T = 0$

Wall	Force (k)
A	64.51
B	20.55
C	46.27
D	46.27
E	64.51
F	20.51

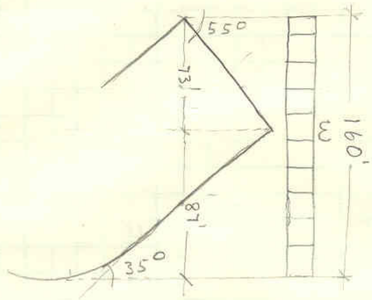
1st story

$W = 244.1^K$ $M_T = 0$

Wall	Force (k)
A	78.65
B	25.05
C	56.42
D	56.42
E	78.65
F	25.05

Load Distribution (6)

Wind Parallel the $\overline{X}$ - axis



(K)

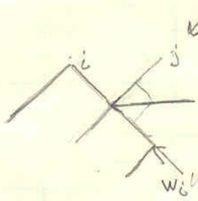
$W_R = 152.25(160) = 24.36$

$W_4 = 344.75(160) = 55.16$

$W_3 = 526.75(160) = 84.28$

$W_2 = 701.75(160) = 112.28$

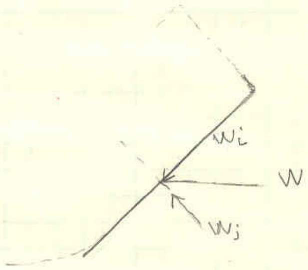
$W_1 = 855.75(160) = 136.92$



$W = 73(152.25) = \boxed{11.114^K}$

$W_i = 11.11 \cos 55 = 6.37^K$

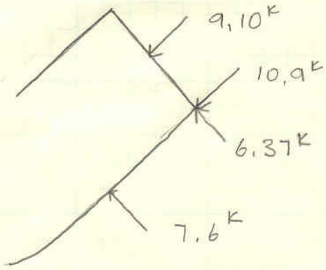
$W_j = 11.114 \sin 55 = 9.10^K$



$W = 87(152.25) = \boxed{13.25^K}$

$W_i = 13.25 \cos 35 = 10.9^K$

$W_j = 13.25 \sin 35 = 7.6^K$



Final Loads

Load DISTRIBUTION (7)

Roof

$M_T = 11.114^K(41') + 13.25(39.17) = 63.3^{'K}$

$W. = 24.36^K$

Direct Forces

$\text{Wall } A = \dfrac{24.36(24.583)}{2(24.583)+2(5.875)+30} / \cos 55 = 11.60^K$

$\text{Wall } B = \dfrac{5.875(24.36)}{90.916} / \cos 35 = 1.92^K$

$\text{Wall } E = 11.60 = 11.60$

$\text{Wall } F = \quad = 1.92$

$\text{Wall } G = \dfrac{30(24.36)}{90.916} = 8.04^K$

Forces due to Torsion

$\text{Wall } A_T = \dfrac{(63.3^{'K})(19.08)(30')}{362429.56\ ft^3} = 0.1^K$ add

$\text{Wall } B_T = \dfrac{(63.3)(47.25)(10')}{362429.56} = 0.08^K$ subtract

$\text{Wall } C = \dfrac{(63.3)(15)(22)}{362429.56} = 0.06^K$ } oposite directions

$\text{Wall } D = \quad 0.06^K$

$\text{Wall } E = \quad 0.1^K$ add

$\text{Wall } F = \quad 0.08^K$ subtract

$\text{Wall } G = \dfrac{63.3(50.17)(30)}{362429.56} = 0.26^K$ add

Total Forces in each wall:

Wall	Force (k)	Wall	Force (k)
A	11.7	E	11.7
B	1.84	F	1.84
C	0.06	G	8.36
D	-0.06		

wind parallel to the $\bar{x}$-axis

The distribution of a lateral Load on the remaining stories are proportional to the calculated distribution of forces at the roof level.

As shown in a chart below, the wind load case parallel to $\bar{y}$-axis controls the system, with the exeption of the wall G

Distribution of forces at roof level)			
Comparison Table 1			
X-axis		Y-axis	
wall	Force (K)	wall	Force (K)
A	11.7	A	13.98
B	1.84	B	4.46
C	0.06	C	8.67
D	0.06	D	8.67
E	11.7	E	13.98
F	1.84	F	4.46
G	8.30	G	0

Load Distribution (9)

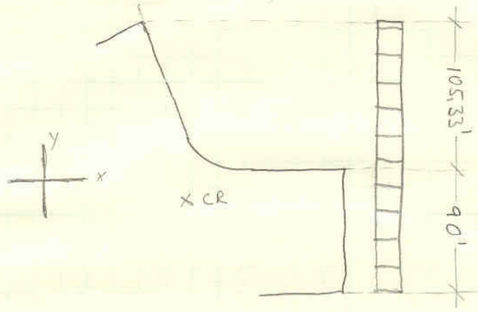
wind Parallel to the $\bar{x}$ axis

Wall G	
Story	Force (k)
R	8.30
4	18.80
3	28.72
2	38.26
1	46.65

Load Distribution (10) NE - SW

Wind in NE - SW direction

Roof



$W_1 = 105.33(152.25 \text{ plf})$
$= 16.04^K$

$W_2 = 90.00(152.25)$
$= 13.70^K$

Moment arms:

W_1: 63.25'
W_2: 34.42'

$M_T = 16.04(63.25) - 13.70(34.42)$
$= \boxed{542.98^{'K}}$

Wall	Length x	Length y
A	9.58'	28.83'
B	9.5'	3.08'
C	12.83	17.83
D	12.83	17.83
E	30'	0
F	0	10'
G	24.33'	17.58'

Wind in the NE-SW direction

$$\text{Walls} = \frac{W\ell}{\Sigma\ell} + \frac{M_T \times \ell}{I_P}$$

Final Distributed Forces in each wall

Wall	Direct shear Force	M_T Force	Total
A	10.377 K	− 0.863	9.51
B	3.00 K	+ 0.872	3.87
C	6.603 K	+ 0.49	7.10
D	6.603 K	− 0.49	6.11
E	9.00 K	− 0.874	8.13
F	0 K	0.877	0.88
G	9.01 K	− 2.25	6.67

Roof Level

Comparison Table 2

Wall	Load ∥ $\bar{X}$ (K)	Load ∥ $\bar{Y}$ (K)	Load ∥ NE-SW (K)
A	11.7	13.98	9.51
B	1.84	4.46	3.87
C	0.06	8.67	7.10
D	0.06	8.67	6.11
E	11.7	13.98	8.13
F	1.84	4.46	0.88
G	8.30	0	6.67

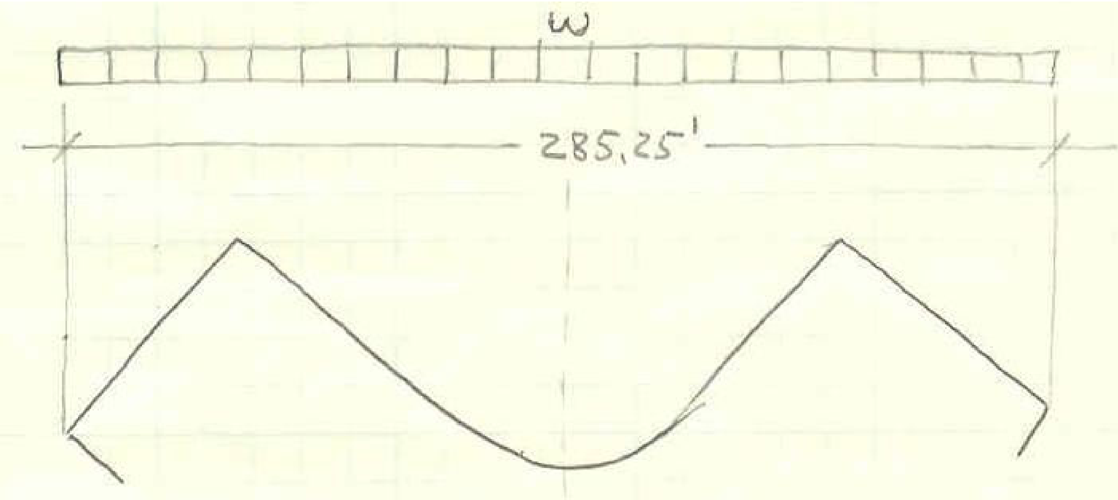
* Underlined are the controlling forces.

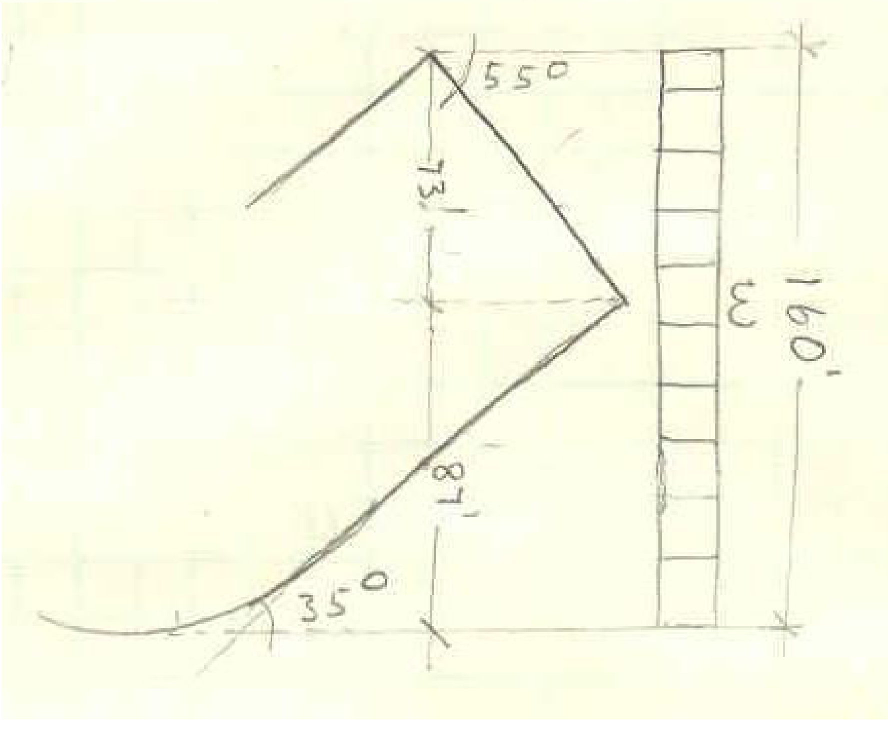
Load Distribution (12) NE-SW

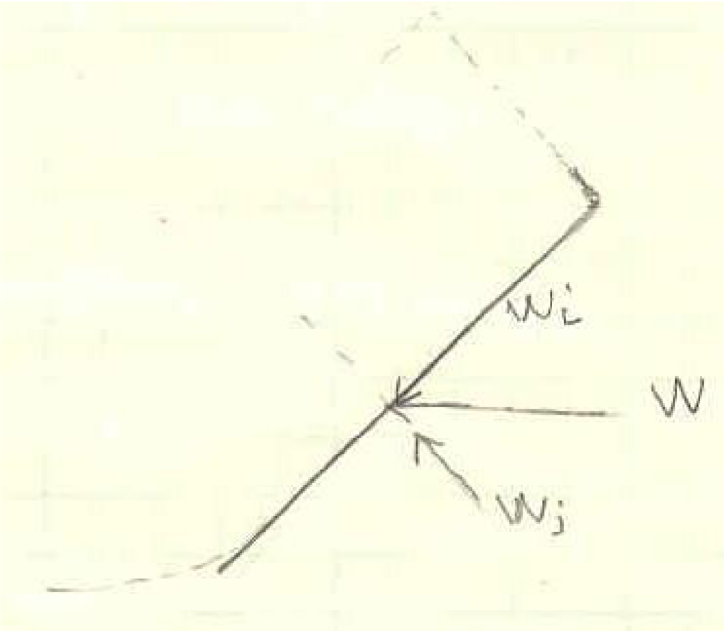
Wind parallel with the NE-SW axis

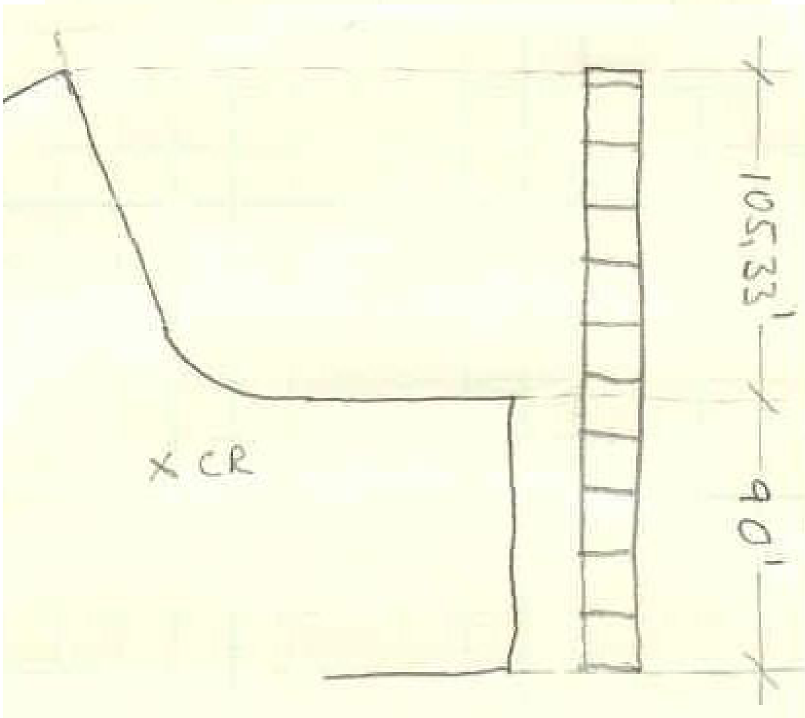
Wall G

Level	W_p (k)	Total Force in wall (k)
R	29.7	6.67
4	67.3	15.30
3	102.9	23.39
2	137.1	31.17
1	167.2	38.01


ω
285.25'


55°
73'
87'
35°
ω
160'


Wi
W
Wj


× CR
105,33'
90'

LOAD DISTRIBUTION TABLES

CASE 1

Resulting Distributed Forces at the top of the frame in kips

Frame	Garage	Level 1	Level 2	Level 3	Level 4
A	*78.65*	*64.51*	*48.43*	*31.67*	*13.98*
B	*25.05*	*20.55*	*15.42*	*10.09*	*4.46*
C	*56.42*	*46.27*	*34.74*	*22.72*	*8.67*
D	*56.42*	*46.27*	*34.74*	*22.72*	*8.67*
E	*78.65*	*64.51*	*48.43*	*31.67*	*13.98*
F	*25.05*	*20.55*	*15.42*	*10.09*	*4.46*
G	0	0	0	0	0

CASE 2

Resulting Distributed Forces at the top of the frame in kips

Frame	Garage	Level 1	Level 2	Level 3	Level 4
A	N/A	N/A	N/A	N/A	11.7
B	N/A	N/A	N/A	N/A	1.84
C	N/A	N/A	N/A	N/A	0.06
D	N/A	N/A	N/A	N/A	0.06
E	N/A	N/A	N/A	N/A	11.7
F	N/A	N/A	N/A	N/A	1.84
G	*46.65*	*38.26*	*28.72*	*18.8*	*8.3*

CASE 3

Resulting Distributed Forces at the top of the frame in kips

Frame	Garage	Level 1	Level 2	Level 3	Level 4
A	N/A	N/A	N/A	N/A	9.51
B	N/A	N/A	N/A	N/A	3.87
C	N/A	N/A	N/A	N/A	7.1
D	N/A	N/A	N/A	N/A	7.1
E	N/A	N/A	N/A	N/A	9.51
F	N/A	N/A	N/A	N/A	3.87
G	N/A	N/A	N/A	N/A	6.67

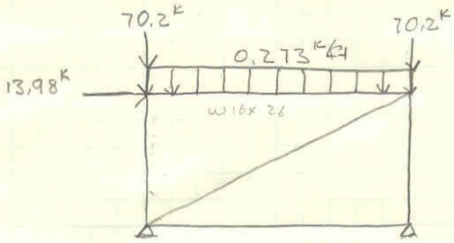
NOTE: N/A - values that do not control the design

Italicized are the values that control the design

	Steel Memeber design Summary				
		Braced Frame A Size			
Frame	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	W12x106	W12x106	W12x106	W12x50	W12x50
BC	W18x35	W18x35	W18x35	W18x35	W16x26
CD	W12x106	W12x106	W12x106	W12x50	W12x50
AC	W8x58	W8x48	W8x35	W8x31	W8x21
		Braced Frame B Size			
Frame	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	W12x65	W12x65	W12x65	W12x35	W12x35
BC	W18x35	W18x35	W18x35	W18x35	W21x50
CD	W12x65	W12x65	W12x65	W12x35	W12x35
AC	W8x35	W8x31	W8x28	W8x24	W8x21
		Braced Frame C Size			
Frame	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	W12x96	W12x96	W12x96	W12x40	W12x40
BC	W18x40	W18x40	W18x40	W18x40	W18x35
CD	W12x96	W12x96	W12x96	W12x40	W12x40
AC	W8x48	W8x35	W8x31	W8x28	W8x21
		Braced Frame D Size			
Frame	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	W12x96	W12x96	W12x96	W12x40	W12x40
BC	W18x40	W18x40	W18x40	W18x40	W18x35
CD	W12x96	W12x96	W12x96	W12x40	W12x40
AC	W8x48	W8x35	W8x31	W8x28	W8x21
		Braced Frame E Size			
Frame	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	W12x106	W12x106	W12x106	W12x50	W12x50
BC	W18x35	W18x35	W18x35	W18x35	W16x26
CD	W12x106	W12x106	W12x106	W12x50	W12x50
AC	W8x58	W8x48	W8x35	W8x31	W8x21
		Braced Frame F Size			
Frame	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	W12x65	W12x65	W12x65	W12x35	W12x35
BC	W18x35	W18x35	W18x35	W18x35	W21x50
CD	W12x65	W12x65	W12x65	W12x35	W12x35
AC	W8x35	W8x31	W8x28	W8x24	W8x21
		Braced Frame G Size			
Frame	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	W12x96	W12x96	W12x96	W12x40	W12x40
BC	W16x26	W16x26	W16x26	W16x26	W16x26
CD	W12x96	W12x96	W12x96	W12x40	W12x40
AC	W8x35	W8x31	W8x31	W8x24	W8x21

Braced Frame — Wall A

Level: 4 – Roof



Trib Area $= (30')(3.5') = 105\ ft^2$

LL = 30 psf P_u = 78 psf

DL = 25 psf

$W_u = 1.2(25)(3.5) + 1.6(30)(3.5)$

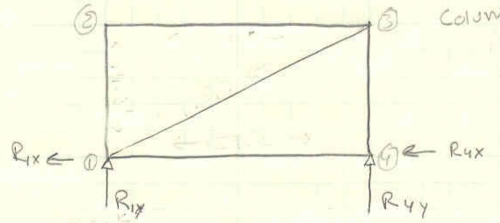
$= 273\ lb/ft$

$= 0.273\ K/ft$

Column $= (900\ ft^2)(78\ psf)$

$= 70.2^K$

← C →
→ T ←



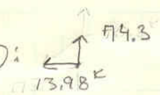
$\Sigma M_{①} = 0 = 14(13.98) + (8.19)(15) + 70.2(30) - 30 R_{4y}$

$R_{4y} = 80.8^K$

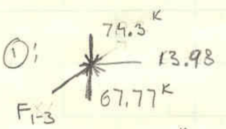
$R_{1y} = 2(70.2) + 8.19 - 80.8 = 67.77^K$

$R_{1x} = R_{4x} = \frac{13.98}{2} = 6.99^K$

②:

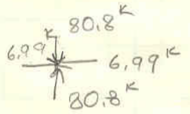


①:



$\frac{74.3 - 67.77}{F_{1-3}} = \frac{14}{33.1}$

④:



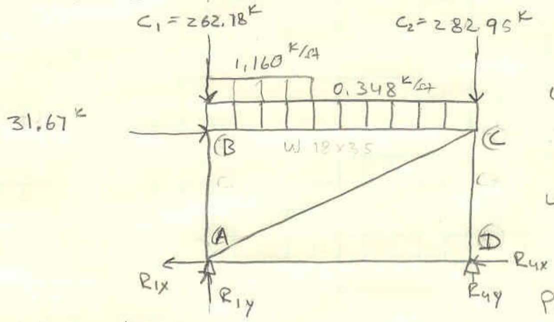
$\boxed{F_{1-3} = 15.44^K}$

$F_{1-2} = 74.3^K$
$F_{2-3} = 13.98^K$
$F_{3-4} = 80.8^K$
$F_{1-4} = 6.99^K$

Broced Frome Design (2) 3→4

Broced Frome - Woll A

Level 3



$\omega_{u_1} = [1.2(60) + 1.6(100)]\,1.5'$

$= 0.348^K/ft$

$\omega_{u_2} = [1.2(60 + 1.6(100)]\,5'$

$= 1.160^K/ft$

$P_u = 1.2(60) + 1.6(100)$

$= 232$ psf

Column Loads:

$C_1 = \dfrac{(900\ ft^2)(232\ psf)}{1000} - 14.89^K + 87.77 + 0.5 = 262.18^K$

$C_2 = \dfrac{(900)(232)}{1000} - 7.15^K + 80.8 + 0.5 = 282.95^K$

$C_1 = (A_T)(P_u)$ - Uniformly Dist loods + column loods from L4

$\Sigma M_{①} = 0 = 14(31.67^K) + 1.16(10')(5') + 0.348(30)(15) + 282.95(30) - 30\,R_{Dy}$

$R_{Dy} = 304.88^K$

$R_{Ay} = 1.16(10) + .348(30) + 282.95 + 262.18 - 304.88$

$= 262.29^K$

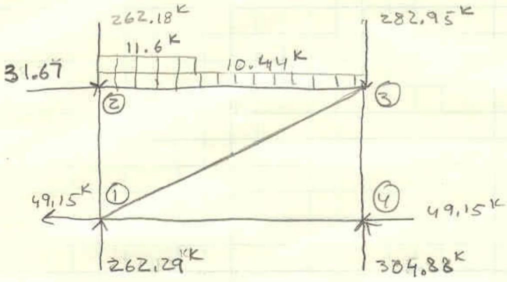
$R_{Ax} = \dfrac{98.3}{2} = 49.15^K$

$R_{4x} = \dfrac{98.3}{2} = 49.15^K$

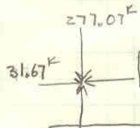
Braced Frame Design (3) Level 3-4

Braced Frame - Wall A

Level 3



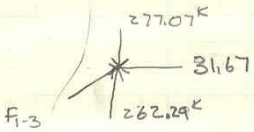
2:



$\boxed{F_{2-3} = 31.67^K}$ (C)

$\boxed{F_{12} = 277.07^K}$ (C)

1



$$\frac{277.07 - 262.29}{F_{1-3}} = \frac{14}{33.1}$$

$\boxed{F_{1-3} = 34.9^K}$ (T)

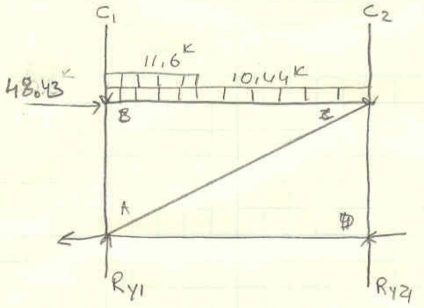
4 $\boxed{F_{3-4} = 304.88^K}$ C

MEMBER	Force (K)	T or C
F_{1-2}	277.07	C
F_{1-3}	34.9	T
F_{2-3}	31.67	C
F_{3-4}	304.88K	C

Braced Frame Design (4) Level 2-3

Braced Frame - Wall A

Level 2



$C_1 = 456.7^K$

$C_2 = 507.03^K$

$R_{YA} = 449.00^K$

$R_{YD} = 537.05$

$R_{xA} = 24.2^K$

$R_{xD} = 24.2^K$

MEMBER	axial Force (K)	Direction
A B	472	C
B C	48	C
C D	537	C
A C	53	T

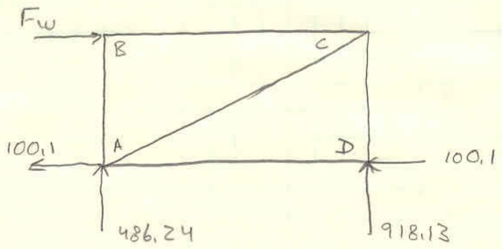
Braced Frame Design (5) Level 1-2

Level 1

$C_1 = 643.41^K$ $R_{Ay} = 628.2^K$

$C_2 = 739.20^K$ $R_{Dy} = 776.5^K$

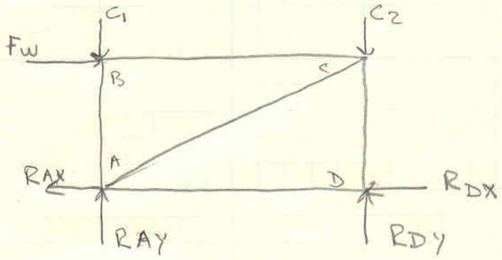
$F_{WIND} = 64.51^K$ $R_{Ax} = R_{Dx} = 32.25^K$



MEMBER	AXIAL Force (K)	Direction
AB	658	C
BC	64.5	C
CD	776	C
AC	71.2	T

Broced Frome Design (6) Level 0-1

Level 0



$F_W = 78.65^K$

$C_1 = 822.61^K$

$C_2 = 978.65^K$

$R_{Ax} = 39.3^K$

$R_{Ay} = 800.8^K$

$R_{Dx} = 39.3^K$

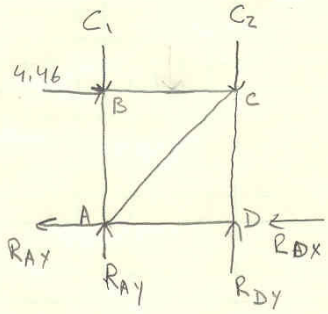
$R_{Dy} = 1022.5^K$

MEMBER	Axioc Force	Dirrection
AB	837	C
BC	78.7	C
CD	1023	C
AC	86.8	T

Braced Frame Design (7) Wall B - 14 - Roof

Wall B

Level: 14



$C_1 = (5+15')(30')(78\ psf) = 46.8^K$

$C_2 = (15')(30')(78\ psf) = 35.1^K$

$F_w = 4.46^K$

$R_{AX} = 2.23^K$

$R_{Ay} = 40.56^K$

$R_{DX} = 2.23^K$

$R_{Dy} = 41.34^K$

MEMBER	AXIAL FORCE (K)	DIRECTION
AB	46.8	C
BC	4.46	C
CD	41.34	C
AC	7.67	T

Braced Frame Design (8) Wall B - 3

Level 3

$C_1 = (20')(30')(232\,psf) + 40.56^K + 0.5 = 180.26^K$

$C_2 = (15')(30')(232\,psf) + 41.34^K + 0.5 = 146.24^K$

$F_w = 10.09^K$

MEMBER	AXIAL FORCE (K)	Direction
AB	180.26	C
BC	10.9	C
CD	161.5	C
AC	18.75	T

Braced Frame Design (9) Wall B, Levels 0 - 2

WALL B

MEMBER	Level 2 AXIAL Force (K)	Level 1 AXIAL Force (K)	Level 0 AXIAL Force (K)	Direction
AB	304.7	422.8	533.7	C
BC	15.42	20.55	25.05	C
CD	288	421.662	361.6	C
AC	26.5	35.346	43.1	T
C1	304.7	422.8	533.7	
C2	266.4	392.9	526.7	

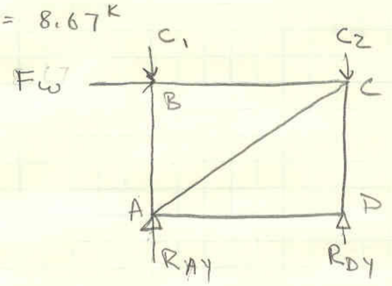
WALL C

Level 4

$C_1 = (10+11)(15+5)(78) = 32.76^K$

$C_2 = (11+20)(15+10)(78) = 60.45^K$

$F_w = 8.67^K$



Level 3

$C_1 = R_{AY} + (420\ ft^2)(.232\ ksf) + 0.5 =$

$C_2 = R_{DY} + (775\ ft^2)(.232\ ksf) + 0.5 =$

$F_w = 22.72^K$

$\boxed{C}$

WALL C Force Distribution TABLE

MEMBER	Level 4 AXIAL F (K)	Level 3 axial F (K)	Level 2 axial F (K)	Level 1 axial F (K)	Level 0 axial F (K)	Direction
AB	32.76	125.18	208.6	284.5	353	C
BC	8.67	22.72	34.74	46.27	56.42	C
CD	66.0	260.73	463.1	672.9	889.1	C
AC	10.28	26.9	41.2	54.8	66.88	T
C1	32.76	125.2	208.7	284.5	353.0	
C2	60.45	246.3	441.0	643.4	853.2	
Fw	8.67	22.72	34.74	46.27	56.42	

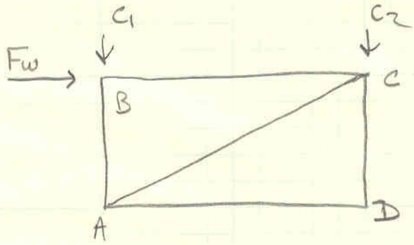
Braced Frame Distribution (11) Wall G – 4-Roof

$A_{TC1} = (20+17)(15) + \frac{(15)(15)}{2} + 7(20) = 807\ ft^2$

$A_{TC2} = 807\ ft^2$

$C_1 = C_2 = (807)(.232) = 187.2^K$ floors

$C_1 = C_2 = (807)(.78) = 62.9^K$ roof



WALL G FORCE DISTRIBUTION TABLE – Axial Force

Member	Level 4 (K)	Level 3 (K)	Level 2 (K)	Level 1 (K)	Level 0 (K)	Direction
AB	62.9	246.7	422.5	591.9	755.2	C
BC	8.3	18.8	28.72	38.26	46.65	C
CD	66.78	266.4	472.4	684.5	901.8	C
AC	9.16	22.28	34.04	45.35	55.29	T
C_1	62.9	246.7	422.5	591.9	755.2	
C_2	62.9	254.4	454.1	660.1	872.2	
Fw	8.3	18.8	28.72	38.26	46.65	

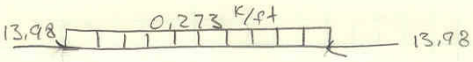
Beam Design

Beam Design (1) Wall A

Wall A

Level: 14 story

Member B-C



$M_{max} = \frac{w\ell^2}{8} = \frac{0.273(30^2)}{8} = 30.71^{1K}$

$P_u = 13.98^K$

$KL_x = 30'$ $B_{1x} = 1.0$

$M_{ux} = (1.0)(30.71) = 30.71^{1K}$

TRY W16×26 $\phi M_p = 166^{1K} \gg 30.71^{1K}$

Tensile Force is too small to be considered.

$I_x = 301 \text{ in}^4$ $E = 29000$ ksi

$\Delta_{15'} = \frac{5(.273)(30^4)(1728)}{384(29000)(301)} = 0.57 \text{ in} < \frac{30(12)}{360} = 1''$

OK

use $\boxed{W16\times26}$

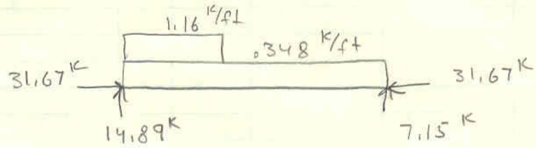
W18x35

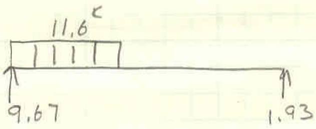
Beam Design (2) Wall A

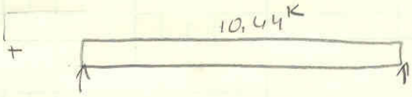
Level 3

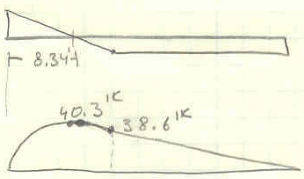
Wall A

Beam BC









$M_c = \frac{.348(30^2)}{8} = 39.15'^K$

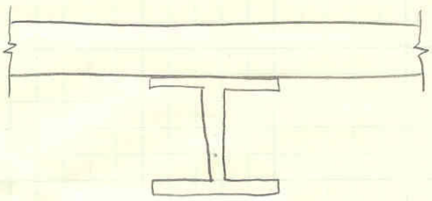
$M_{x=10} = 34.3'^K$

$M_{max} = 38.6 + 34.3 = 73'^K$

EI

TRY W18x35

Nominal Flexural Strength Mn



$b_{eff} = \frac{30}{4}$ or $2'$

$C_{conc} = (.85)(5)(2'')(12)(4'') = 408^K$

$T_{st} = (.50)(10.3) = 515$

$T_{st} > C_{con} \Rightarrow 408 = 50(10.3 - x6)$

PNA $= x = 0.357''$ from TOF

$M_p = 408(2) + 50(6)(.357)\left(\frac{.352}{2}\right) + 50(10.3 - 6(.357))(\approx 10)$

$= 406.3'^K$

Beam Design 3 Wall A

Level 3 cont...

$\lambda_c = \frac{1.0(30)}{7.04\pi}\sqrt{\frac{50}{29000}} = 0.0563$

$\phi P_n = 0.85(0.658^{(0.0563)^2})(50)(10.3) = 437.17$

$\frac{P_u}{\phi P_n} = \frac{31.67}{437.17} = 0.07 < 0.2 \quad \Rightarrow$ H1-1b

$\frac{31.67^k}{2(437.17)} + \left(\frac{73^{1k}}{406.3^{1k}(.85)}\right) = 0.248 \leq 1.0$ OK

$\Delta = 0.428 < \frac{1}{360}(12) = 1$ OK

TRY

W16x26

$C = 0.85(5)(2)(12)(4) = 408^k$

$T_s = (50)(7.68) = 384$

$a = \frac{384}{0.85(5)(24)}$ $3.76''$

$M_p = 50(7.68)(7.85) + 384(2.118) = 3827.6$
$= 319.0^{1k}$

$\phi M_n = 271.1^{1k}$

$\phi P_n = 325.9$

$\frac{P_u}{\phi P_n} = 0.097 < 0.2 \quad \Rightarrow$ H1-1b

H1-1b $\Rightarrow 0.3179 \leq 1.0$ OK

$\Delta =$ NG

USE $\boxed{\text{W18X35}}$

Summary of Wall A - Beams B-C

4	W16x26	0 W18x35
3	W18x35	
2	W18x35	
1	W18x35	

Beam Design | 4 | Wall A

Level 2

TRY $\boxed{W18\times35 \quad 4'' \text{ conc}}$

$\phi M_A = 345.4^{'K}$

$\phi P_n = 437.17^K$

$\frac{P_u}{\phi P_n} = \frac{48.43^K}{437.17^K} = 0.11 < 0.2 \Rightarrow H1\text{-}1b$

$H1\text{-}1b = 0.266 \leq 1.0$ OK

Δ ok

Level 1

$\boxed{W18\times35 \quad 4'' \text{ conc}}$

$\phi M_n = 345.4^{'K}$

$\phi P_n = 437.17^K$

$\frac{P_u}{\phi P_n} = \frac{64.51}{437.17} = 0.15 < .2 \Rightarrow H1\text{-}1b$

$H1\text{-}1b = 0.285 < 1.0$ OK

Δ OK

Level 0

$\boxed{W18\times35 \quad 4'' \text{ conc}}$

$P_u = 78.65$

$\frac{P_u}{\phi P_n} = 0.18 < .2$ use H1-1b

$H1\text{-}1b \Rightarrow 0.3 < 1.0$ OK

Δ OK

Beom Design | 5 | Woll B

Level 4

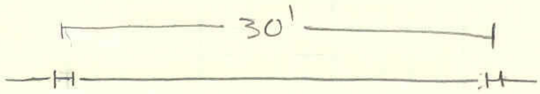
$M_u = 29.25^{1k}$

$P_u = 4.46^{k}$

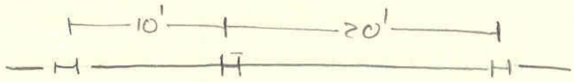
W 21x50

use orignol size

Beom Got stronger with the addition of a column



original
Design



New Design

Beam Design (6) Wall B

Level 3 → Level 0

use W 18 x 35

$w_u = (0.232)(30) = 6.96^{k/ft}$

$M_u = \frac{6.96(10^2)}{8} = 87^{1k}$

$P_u = 10.09^k$

- Nominal Flex. Strength

$b_{eff} = \frac{10'}{4}$ or $30'$

$= 30''$

$C_{conc} = 0.85(5)(30)(4) = 510^k$

$T_{st} = 515^k$

$a = \frac{-510 + 50(10.3)}{50(6)} = 0.02''$ from TOF

$M_n = 510(2) - 50(6)(.02)\left(\frac{.02}{2}\right) + 50(10.3 - 6(.02))(8.9)$

$= 462.5^{1k}$

$\phi M_n = 393.1^{1k}$

$\phi P_n = 437.7^k$

$\frac{P_u}{\phi P_n} = 0.023 < 0.2 \Rightarrow H1-1b$

$H1-1b = 0.233 < 1.0$ ok

$\Delta \Rightarrow$ ok

use $\boxed{18 \times 35}$

Beam Design (7) Wall B

Levels 2, 1, 0

W18x35

Level 2

$P_u = 15.42^k$

$\frac{P_u}{\phi P_n} = 0.035 < .2 \Rightarrow H1-1b = 0.239 < 1.0$ ok

Δ = OK

Levels	Beam B-C size
4	W21x50
3	W18x35
2	
1	
0	W18x35

Wall C

$b_{eff} = 66''$

$a = \frac{515}{.85(5)(66)} = 2.8136$

$M_n = 590(2.95) + 590(8.95) = 585^{1K}$

$\phi M_n = 497.3^{1K}$

$\phi P_n = 501.2$

$M_{p4} = 117.975^{1K}$

$M_{p0\text{-}3} = 350.9^{1K}$

$I_{LB} = 1450 \text{ in}^4$

$\Delta = \frac{5 w \ell^4}{384 E I_{LB}}$

$\frac{\ell}{360} = \frac{22(12)}{360} = 0.733$

Level	Axial Force	Mp	Size	Δ
4	8.67	118	W 18x35	OK
3	22.72	351	W 18x40	0.72
2	34.74	351	W 18x40	0.72
1	46.27	351	W 18x40	0.72
0	56.42	351	W 18x40	0.72

Wall G

w_E 0.468 k/ft
w = 0.696 k/ft
b_{eff} = 36"
W 16 x 26
ϕM_n = 290 'k
ϕP_n = 325 k
I_{LB} = 730 in^4
Max Δ = 1"

Roof M_p = 52.65 'k
Floor M_p = 78.3 'k

Eq H1-1a governs

Level	Axial Force	Mp	size	Deflection (in)
4	8.3	52.65	W16x26*	.977
3	18.8	78.3	W16x26	0.6
2	28.72	78.3	W16x26	0.6
1	38.26	78.3	W16x26	0.6
0	46.05	78.3	W16x26	0.6

* Noncomposite Beam

NEW 1

Typical Girder design.

Loads:

LL = 100 psf

DL = 60 psf

LL Reduction

$0.25 + \frac{15}{\sqrt{2(30)^2}} = 0.6 > 0.4$ OK

LL = 100(0.6) = 60 psf

Factored Loads

$1.2(60)(30') + 1.6(60)30' = 5040$ pLF = $\boxed{5.04 \text{ KLF}}$

Max Moments & shear

$M = \frac{wL^2}{8} = \frac{5.04(30^2)}{8} = \boxed{567^{1K}}$

$V_u = \frac{wL}{2} = \frac{5.04(30)}{2} = \boxed{75.6^K}$

Composite Beam:

Assume a = 2 in

$Y_2 = Y_{con} - \frac{a}{2} = 4 - \frac{2}{2} = 3$ in

TRY: W 24x62 $\phi_b M_n = 962^{1K}$ @ TFL, $\Sigma Q_n = 915^K$

Typical Girder Design 2

check Y_2:

$$b \le \begin{cases} 2 \times \frac{30}{8} = 7.5' & \text{- controls} \\ 30' \end{cases}$$

$b = 90''$

$$a_{req} = \frac{\Sigma Q_n}{.85 f'c\, b} = \frac{915^k}{.85(3)(90'')} = 3.99''$$

$$Y_2 = 4 - \frac{3.99}{2} = 2 < 3 \quad \boxed{OK}$$

$I_{LB} = 3580$ [LRFD T 5-15]

$$\Delta = \frac{5wL^4}{384EI} = \frac{5(5.04)(30^4)(12^3)}{384(29000)(3580)} = 0.885''$$

$$\frac{30(12)}{360} = 1'' > 0.885'' \quad OK$$

Shear check

$\phi V_n = 275^k > 75.6^k$ OK

Check UNSHORED!

Assume LL = 20 psf

(1) $1.4(60)(30) = 2.52$ KLF
(2) $1.2(60)(30) + 1.6(20)(30) = 3.12$ KLF ← controls

$$M_u = \frac{3.12(30)^2}{8} = 351^{'k} < 578^{'k} \quad OK$$

$$V_u = \frac{3.12(30)}{2} = 46.8^k < 275^k \quad OK$$

Typical Girder design

Unshored Girder (W24x62) I = 1560

Deflect. check

$$\Delta = \frac{5wL^4}{384EI} = \frac{5(3.12)(30^4)(12^3)}{384(29000)(1560)} = 1.257''$$

$$\frac{30(12)}{360} = 1 < 1.257'' \quad NG$$

TRY W24x68

Shear & Moment capacities are OK
Composite $\Delta < 1''$ OK

UNShored Moments & Shears are OK

$$\Delta = \frac{5wL^4}{384(EI)} = \frac{5(3.14)(30^4)(12^3)}{384(29000)(1830)} = 1 \quad OK$$

$\boxed{\text{USE W24x68}}$

NO changes with respect to ORiginal Design.

Column Design

WALL A

W 12x22

Not slender

$\frac{Kl_x}{r_x} = \frac{1(14)(12)}{5.31} = 31.638$

$\lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}}$

$\frac{Kl_y}{r_y} = \frac{1(14)(12)}{3.04} = 55.21$

Buckling about Y-Y axis controls

$\phi_c P_n = \phi_c F_{cr} A_g$

$= 0.85(.658^{\lambda_c^2})(50)(21.1) = 717.3^K$

W 12x79

$\phi P_n = 789.8$

W12x106 $\phi P_n = 1071^K > 1022$ ok

W12x50 $\phi P_n = 362.6^K > 305$ ok

Columns

use $\boxed{W12x106}$ column spaning from 0 to 3

use $\boxed{W12x50}$ from 3 to roof

Colum Design (2) Wall B, C, D, E, F, G

WALL B

TRY W 12x35 $\phi P_n = 183.4^k > 180^k$ ok

TRY W 12x65 $\phi P_n = 647.4 > 561.6$ ok

use:

W12x35 for levels 3 & up

W12x65 for levels 0 to 3

WALL C

W12x40 ($\phi P_n = 287 > 260.7$) Levels 3 & up

W12x96 ($\phi P_n = 965 > 889.1$) Levels 0 to 3

WALL G

W12x40 ($\phi P_n = 287 > 266.4$) Levels 3 & up

W12x96 ($\phi P_n = 965 > 901.8$) Levels 0 to 3

WALL D

W12x40 3+

W12x96 0-3

WALL E

W12x50 3+

W12x106 0-3

WALL F

W12x35 3+

W12x65 0-3

Nonbraced Frame Column Design. (1)

30'x30' Bay

DL = 60 psf — Floor
LL = 100 psf
SL = 21 psf
DL = 25 psf — Roof
LL = 30 psf — Roof

LL Reduction:

- FLOOR

$$\frac{LL}{L_o} = 0.25 + \frac{15}{\sqrt{A_I}}$$

$$= 0.25 + \frac{15}{\sqrt{60^2 \times 4}} = 0.375 < \boxed{0.4} \text{ controls}$$

- ROOF

$L_r = 30 R_1 R_2$ $(12 \leq L_r \leq 20)$

$R_1 = 0.6$ $(900\ ft^2)$

$R_2 = 1$ (Flat Roof)

$L_r = 20(0.6)(1) = 18$ psf

Loads:

$DL = 60(4)(30^2) + 25(30^2) = 238.5^K$

$LL = 0.4(100)(30^2)4 + 18(30^2) = 160.2^K$

$SL = 21(30^2) = 18.9^K$

Load Combinations.

$1.4(D)$

$1.2 D + 1.6 L + 0.5 S = \boxed{551.65^K}$ ← controls

Nonbraced Frame Column Design (2)

$P_u = 551.65$

TRY W12x58 [$r_x = 5.28$, $r_y = 2.51$, $A_g = 17.0 \text{ in}^2$]

Bucling about y-y axis

$\phi_c P_n = \phi_c F_{cr} A_g$

$$\lambda_c = \frac{KL}{r\pi}\sqrt{\frac{F_y}{E}} = \frac{1(14)(12)}{(2.51)(\pi)}\sqrt{\frac{50}{29000}} = 0.88$$

$\phi_c P_n = 0.85(.658^{.88^2})(50)(17.0) = 522.5^K$ NG

TRY W12x65

$\phi_c P_n = 647^K > 551.65^K$ OK (LRFD T 4-2)

$\boxed{\text{Use W 12x65}}$

* Note: Compare to W12x72 in original design

Aproximately 10% reduction strength wise

Aproximately 10% reduction in size

Reason: Elimination of:

① Wind Moments

* Note: Moment connections are only 40'K resistive, and do not resist Beams FEM. Beams were designed as pin connected.

Foundation Design

Foundation Design - Typical Non-Braced Frame.

$P_D = 238.5^k$
$P_L = 157.5^k$

$P = P_D + P_L + P_S$ 16"

$B \geq \sqrt{\frac{414}{4}} = 10.25$ FT use 11'x11'

$P_u = 1.2(238.5) + 1.6(157.5) + .5(18) = 547$

$q = \frac{547}{11^2} = 4.52$ ksf $= 31.4$ psi

$v_c = 0.75(4)(\sqrt{3000}) = 164$ psi

$d^2(164 + \frac{31.45}{4}) + d(164 + \frac{31.45}{2})24 = \frac{31.45}{4}(132^2 - 24^2)$

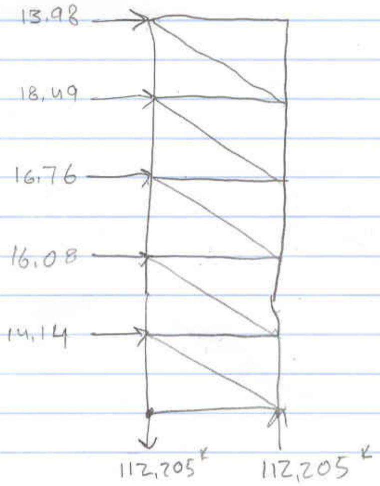
$171.85 d^2 + 4312.8 d - 132257 = 0$

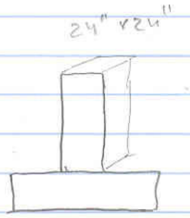
$d = 17.90''$
$h = 17.90 + 3 + 0.625 = 21.5''$ use 22"

Footing: $\boxed{10' \times 10' \times 22''}$

Foundation Design Frame A, E 2

(1)





144000

$\frac{L}{L_0} = .1\left(.25 + \frac{15}{\sqrt{4(3600)}}\right) \le .4$ use .4

$L = (100\ psf)(.4) = 40\ psf$

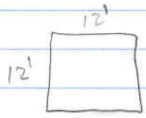
$LL = 40 psf(30^2)4 + .5(30)(30^2) = 157,500 = 157.5^k$

$DL = 60(30^2)4 + 25(30^2) = 238.5^k$

$P_T = P_D + P_L + P_W = 238.5 + 157.5 + 112.205 = 508.205^k$

$q_a = 4000\ psf = 4\ ksf$

$B \ge \sqrt{\frac{508.2}{4}} = 11.27'$ use 12'



(2) $1.2(238.5) + 0.5(157.5) + 1.6(112.205) = 544.478^{k}$

~~1.4(238.5)~~ =

$1.2(238.5) + 1.6(157.5) + 0.5(21 \times 30^2) = 547.65$ ← controls

$q = \frac{547.65}{144} = 3.8 \text{ ksf} = 26.39 \text{ psi}$

$v_c = .75(4)1\sqrt{3000} = 164 \text{ psi}$

$d^2\left(164 + \frac{26.39}{4}\right) + d\left(164 + \frac{26.39}{2}\right)(24) = \frac{26.39}{4}\left(144^2 - 24^2\right)$

$170.6\, d^2 + 4252.68\, d - 133005.6 = 0$

$d = 18.11''$

$h = 18.11 + 3 + 0.625 = \boxed{21.7''}$

USE $h = 22''$

$d = 22 - 3 - .625 = 18.375''$

$\ell = \frac{B - 24''}{2} = \frac{12' - 2'}{2} = 5'$

$M_u = \frac{3.8(5)^2}{2} = 47.5^{1k}$

$a = \frac{A_s(60 \text{ ksi})}{.85(3\text{ksi})(12'')} = 1.96 A_s$

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

$47.5(12) = 0.9 A_s(60\text{ksi})\left(18.375 - \frac{1.96 A_s}{2}\right)$

$10.56 = 18.375 A_s - \frac{1.96 A_s^2}{2}$

$0.98 A_s^2 - 18.375 A_s + 10.56 = 0$

$A_s = 0.59 \approx \boxed{0.60 \text{ in}^2}$

4

③ use #7 @ 12" o.c , $A_s = 0.60\ in^2$

$\rho = \frac{A_s}{bh} = \frac{0.60}{(12")(22")} = 0.00227 \geq .0018$ OK

$a) = 1.96(A_s) = 1.96(.6) = 1.176"$

$c = \frac{a}{.85} = 1.568"$

$\varepsilon_s = \frac{0.003}{c}(d-c) = \frac{0.003}{1.568}(18.375 - 1.568) = 0.0322 \frac{in}{in} \geq 0.005 \frac{in}{in}$

$\Rightarrow \phi = 0.9$

Spacing ok by inspection

$\boxed{\text{USE (12)\#7 each way}}$

$\phi B_n = 0.65(0.85)(3\ ksi)(24")^2 = 955^k$

$\phi B_n = 955k \geq P_u = 508.2^k$ OK

$A_{smin} = 0.005\ A_{col}$

$= 0.005(24")^2 = 2.88\ in^2$ → does not control

Final: USE 12#7 each way

Footing SIZE 12' x 12' x 22"

5

FOUNDATION DESIGN

Typical BAY, NON BASED FRAME

Using Gravity Loads obtained earlier in Braced frame foundation design.

$LL = 157.5^k$ (Reduced)

$DL = 238.5^k$

$SL = 21(30^2) = 18.9^k$

$1.2(238.5^k) + 1.6(157.5) + 0.5(18.9) = 547.65$

No moments

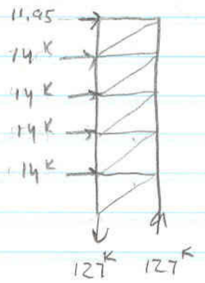
$1.2D + 1.6L + 0.5S$ Load combination controls

So: design is the same as shown in Braced frame foundation design.

USE	(12) #7 each way
size	12' x 12' x 22"

6

Foundation Design   Frame C, D



$A_T = 512.5 \ ft^2$

$A_I = 2460 \ ft^2$

DL = 60 psf

DL_{Roof} = 25 psf

LL = R · 100 psf

SL = 21 psf

$$\frac{LL}{Lo} = .25 + \frac{15}{\sqrt{4(2460)}} = 0.401 \leq 0.4 \quad OK$$

$$LL = 100(.4) = 40 \ psf$$

$$P_D = 60(512.5)4 + 25(512.5) = 135.8^K$$

$$P_L = 40(512.5)4 + 30(512.5) = 97.4^K$$

$$P_W = 127^K$$

$$P = P_D + P_L + P_W = 360.2^K$$

$$B = \sqrt{\frac{360.2}{4}} = 9.5' \Rightarrow 10'$$

$$P_U = 1.2(135.8) + 1.6(97.4) + 0.5(21)\left(\frac{512.5}{1000}\right) = 324.2^K$$

$$\longrightarrow \quad P_U = 1.2(135.8) + 0.5(97.4) + 1.6(127) = 414.9^K$$

$$q = \frac{414.9}{10^2} = 4.14 \ ksf = 28.8 \ psi$$

$$V_c = 164 \ psi$$

$$d^2\left(164 + \frac{28.8}{4}\right) + d\left(164 + \frac{28.8}{2}\right)24 - \frac{28.8}{4}\left(170^2 - 24^2\right) = 0$$

$$171.2\,d^2 + 4281.6\,d - 99532.8 = 0$$

$d = 14.7$

$h = 14.7 + 3 + .625 = 18.3'' \Rightarrow 20''$

Footing: $\boxed{10' \times 10' \times 20''}$

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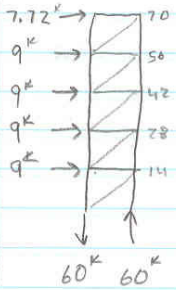
Foundation Design - Frame ⑤



$A_T = 835\ ft^2$

$A_I = 3015\ ft^2$

$DL = 60$ psf

$DL_{Roof} = 25$ psf

$LL = R100$ psf

$SL = 21$ psf

$LL_R = 30$ psf

$$\frac{LL}{L_0} = .25 + \frac{15}{\sqrt[4]{3015}} = .39 \Rightarrow 0.4$$

$LL = 0.4(100) = 40$ psf.

$P_D = 835(60)(4) + 835(25) = 221.3^K$

$P_L = 835(40)(4) + 835(30) = 158.7^K$

$P_W = 60^K \qquad = 60^K$

$P_S = 21(835) = \qquad = 17.5^K$

$P = 440^K$

$B = \sqrt{\frac{440}{4}} = 10.4' \Rightarrow 11'$

$P_U = 1.2(221.3) + 1.6(158.7) + 0.5(17.5) = \boxed{528.23^K}$

$P_U = 1.2(221.3) + 0.5(158.7) + 1.6(60) = 440.9^K$

$q = \frac{528.23}{11^2} = 4.3\ ksf = 30.3$ psi

$V_c = 164$ psi

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Foundation Design Frame 6

$$d^2\left(164+\frac{30.3}{4}\right)+d\left(164+\frac{30.3}{2}\right)24-\frac{30.3}{4}\left(132^2-24^2\right)=0$$

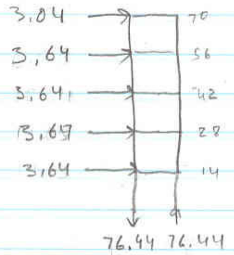
$$171.6\,d^2+4299.6\,d-127623.6=0$$

$$d=17.5''$$

$$h=17.5+3+.625=21.1 \Rightarrow 22''$$

Footing: $\boxed{11' \times 11' \times 22''}$

Foundation Design Frome B (1)
Frome F



$A_T = (15)(30) = 450\ ft^2$

$A_I = (30)(60) = 1800\ ft^2$

$DL = 60\ psf$

$R_{DL} = 25\ psf$

$LL = 100\ psf\ (R)$

$\frac{LL}{L_0} = \left(.25 + \frac{15}{\sqrt{4(1800)}}\right) = 0.42 > 0.4\ ok$

$LL = 100\,(.427) = 42.7\ psf$

$P_D = 60(450)(4) + 25(450) = 119.25^k$

$P_L = 42.7(450)(4) + 30(450) = 90.36^k$

$P_W = 76.44$

$P = P_D + P_L + P_W = 119.25 + 90.36 + 76.4 = 286^k$

$B \geq \sqrt{\frac{286}{4}} = 8.5'$ use $9'$

$P_U = 1.2(119.25) + 1.6(90.36) + 0.5\frac{(21)(450)}{1000} = 292^k$

$P_U = 1.2\,(119.25) + 0.5(90.36) + 1.6(76.4) = 310.5^k$

$q = \frac{310.5}{9^2} = 3.83\ ksf = 26.62\ psi$

$v_c = .75(4)(\sqrt{3000}) = 164\ psi$

11

Foundation Design Frome B (2)

$$d^2\left(164+\frac{26.6}{4}\right)+d\left(164+\frac{26.6}{2}\right)24-\frac{26.6}{4}\left(108^2-24^2\right)=0$$

$$170.7\,d^2+4255.2\,d-73735.2=0$$

$d = 11.77''$

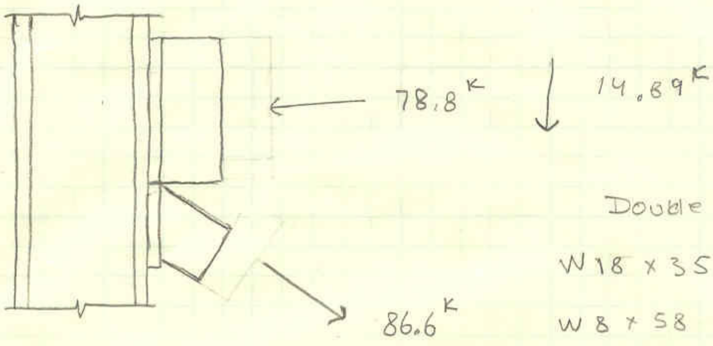
$h = 3 + 11.77 + .625 = 15.4$ use $16''$

Footing: $10' \times 10' \times 16''$

Pier : $24'' \times 24'' \times 20''$

Connection Design

CONNECTION DESIGN (1)



Double Plate

W18 x 35 : $T = 15\frac{1}{2}$, $t_w = 0.30''$

W8 x 58 : $T = 5\frac{3}{4}$, $t_w = .510''$

W12 x 106 : $t_w = .610''$

FRAME A

FLOOR OF LEVEL 1

Connection consists of 2 separate connections: one supports the beam, another connects diagonal brace to the column.

— Beam Connection

Shear Stress

FACTORED REACTION $R_{u_y} = 14.89^k$ $R_{u_x} = 78.8$ $R_u = 80.2^k$

TRY 3 Rows of Bolts (TABLE 10-1 LRFD)

- $\phi R_n = 94.9^k > 14.89^k$ OK (STD, 5/16'' 3/4'' Bolts)(2'' Long)
- ϕR_n for uncoped 0.300 in web $= 344(.300) = 103.2 > 14.89$ OK
- $\phi R_{n_{sup.col}} = 2(344)(.610) = 419.68 > 14.89$ OK (W12x106)

$L = 1\frac{1}{4} + 1\frac{1}{4} + 2(3) = 8\frac{1}{2}'' < 15\frac{1}{2}''$ OK

USE (2) $\angle 5 \times 3 \times \frac{5}{16}$ Angle connection with 3 rows of $\frac{3}{4}''$ Bolts

— Diagonal Connection (@ the column) (T 7-10 LRFD)

- Shear stress: on the column $R = 86.6 \sin(25°) = 36.62^k$

TRY 2 3/4'' A325 BOLTS $\Rightarrow f_v = \frac{36.62}{2(2)(0.442)} = 20.72$ ksi

20.72 ksi < 45 ksi OK

Connection Design

2

Tension stress: @ column

$$f_t = \frac{T_u}{N_b\, A_b} = \frac{86.6(\cos 25)}{4\,(0.442)} = 44.4 \text{ ksi} < 45 \text{ ksi OK}$$

Limiting tension:

$$\text{stress } \phi F_t = 0.75(117 - 2(20.72) \le 90] = 56.67 > 44.4$$

OK

Shear stress on the beam

$$f_v = \frac{86.6}{2(3)(.442)} = 32.65 \text{ ksi} < 45 \text{ ksi} \quad \text{OK}$$

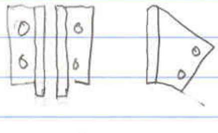
$$L = 1\tfrac{1}{4} + 1\tfrac{1}{4} + 2(3) = 8.5 > T = 5.75 \quad \text{N G.}$$

TRY 2 · 7/8" Bolts

$$f_v = \frac{86.6}{2(2)(.601)} = 36.02$$

$$L = 5.5 < 5.75 \quad \text{OK}$$

USE Double Angles: 2 3/4 A325 Bolts/angle to connect to colum
& 2 7/8 A325 Bolts to connect to beam.



Connection Design 3

Summery:

Typical Beam connection

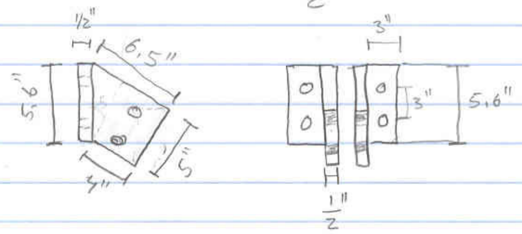
(2) $L5 \times 3 \times \frac{5}{16} \times 10''$ 3 rows of $\frac{3}{4}''$ A325 Bolts

Total weight of $L = 8.19\left(\frac{10}{12}\right)(2) = \boxed{13.65 \text{ lbs}}$

Diagonal connection

Frame A, Level 0.

(2) sloped Angles: $\frac{1}{2}''$ thick



Total weight of Angles $= \left[2(3.5)(5.6)\left(\tfrac{1}{2}\right) + 2(5)(4)\left(\tfrac{1}{2}\right) + 2.5(5)\left(\tfrac{1}{2}\right) - 2.4\right] \times .2835$

$= \boxed{12.32 \text{ lbs}}$

Note: ρ of steel = $\frac{0.2835 \text{ lbs}}{in^3}$

Computer Analysis

Computer Analysis (1)

K - values:

$K = \frac{F}{\Delta}$ (Δ were obtained from Stood 2004)

- Frame A:

  $K = \frac{1}{0.017} = 58.82 \ \frac{K}{in}$ (48.193, 33.72)

- Frame B:

  $K = \frac{1}{0.069} = 14.49 \ \frac{K}{in}$ (8.52, 11.592)

- Frame C

  $K = \frac{1}{0.018} = 55.56 \ \frac{K}{in}$ (0, 55.56)

- Frame D

  $K = \frac{1}{0.018} = 55.56 \ \frac{K}{in}$ (0, 55.56)

- Frame E

  $K = \frac{1}{0.017} = 58.82 \ \frac{K}{in}$ (48.193, 33.72)

- Frame F (8.52, 11.592)

  $K = \frac{1}{0.069} = 14.49 \ \frac{K}{in}$

- Frame G (47.62, 0)

  $K = \frac{1}{0.021} = 47.62 \ \frac{K}{in}$

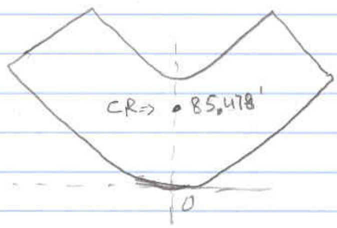
Computer Analysis (2)

The center of Rigidity

$\bar{x} = \boxed{0}$ (symmetry)

$$\bar{y} = \frac{32.5'(47.62) + 97.25'(0.588)(14.49)(2) + 109.58(.819)(58.82)2}{47.62 + (2)(.588)(14.49) + 2(.819)(58.82)}$$

$\bar{y} = \boxed{85.478 \text{ ft}}$



$I_{xx} = 47.62(50.17)^2 + 48.17(27)^2(2) + 8.52(14.67^2)(2)$

$= 193759.9 \ \frac{k}{in} \cdot ft^2$

$I_{yy} = 55.56(15^2)(2) + 34.47(71.08^2)(2) + 11.592(61.75^2)(2)$

$= 461714 \ \frac{k}{in} \cdot ft^2$

$I_p = 193759.9 + 461714 = \boxed{655474.08} \ \frac{k\,ft^2}{in}$

Computer Analysis (3)

Direct Forces: $F_D = \dfrac{K_{iy} P_y}{\Sigma K_{iy}}$

Forces due to Torsion: $F_T = \dfrac{K_i d_i}{\Sigma(K_i d_i^2)} M = \dfrac{K_i d_i M}{I_P}$

Resulting Controling forces: in K

	Level				
Frame	Roof	4th	3rd	2nd	1st
A	12.64	28.62	43.76	58.29	71.07
B	3.04	6.90	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	6.90	10.54	14.05	17.13
G	7.718	17.48	26.7	35.57	43.38

Software licensed to PENN STATE UNIVERSITY	Job No	Sheet No 1	Rev
	Part		
Job Title	Ref		
	By	Date 22-Nov-06	Chd
Client	File Braced Frame Evaluatio	Date/Time 22-Nov-2006 16:22	

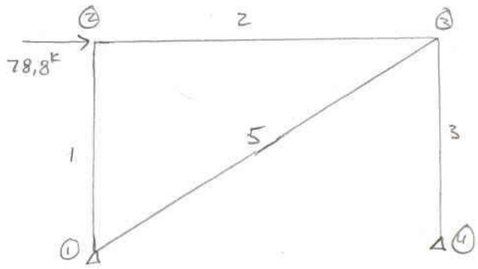
Job Information

	Engineer	Checked	Approved
Name:			
Date:	22-Nov-06		

Structure Type	SPACE FRAME

Number of Nodes	4	Highest Node	4
Number of Elements	4	Highest Beam	5

Number of Basic Load Cases	1
Number of Combination Load Cases	0



Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	WIND LOAD

Beam End Forces

Sign convention is as the action of the joint on the beam.

			Axial	Shear		Torsion	Bending	
Beam	**Node**	**L/C**	**Fx** (kip)	**Fy** (kip)	**Fz** (kip)	**Mx** (kip-in)	**My** (kip-in)	**Mz** (kip-in)
1	1	1:WIND LOAD	0.000	0.000	0.000	0.000	0.000	0.000
	2	1:WIND LOAD	-0.000	-0.000	0.000	0.000	0.000	0.000
2	2	1:WIND LOAD	78.800	0.000	0.000	0.000	0.000	0.000
	3	1:WIND LOAD	-78.800	-0.000	0.000	0.000	0.000	0.000
3	3	1:WIND LOAD	36.773	0.000	0.000	0.000	0.000	0.000
	4	1:WIND LOAD	-36.773	-0.000	0.000	0.000	0.000	0.000
5	1	1:WIND LOAD	-86.958	0.000	0.000	0.000	0.000	0.000
	3	1:WIND LOAD	86.958	-0.000	0.000	0.000	0.000	0.000

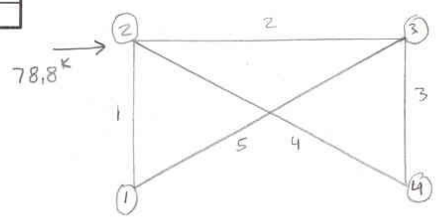
Reactions

		Horizontal	Vertical	Horizontal	Moment		
Node	**L/C**	**FX** (kip)	**FY** (kip)	**FZ** (kip)	**MX** (kip-in)	**MY** (kip-in)	**MZ** (kip-in)
1	1:WIND LOAD	-78.800	-36.773	0.000	0.000	0.000	0.000
4	1:WIND LOAD	-0.000	36.773	0.000	0.000	0.000	0.000

Software licensed to PENN STATE UNIVERSITY	Job No	Sheet No 1	Rev
	Part		
Job Title: Braced Frame Force Distribution	Ref		
	By	Date 22-Nov-06	Chd
Client	File Braced Frame Evaluatio	Date/Time 22-Nov-2006 16:05	

Job Information

	Engineer	Checked	Approved
Name:			
Date:	22-Nov-06		



Structure Type	SPACE FRAME

Number of Nodes	4	Highest Node	4
Number of Elements	5	Highest Beam	5

Number of Basic Load Cases	1
Number of Combination Load Cases	0

Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	Wind Load

Beam End Forces

Sign convention is as the action of the joint on the beam.

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
1	1	1:Wind Load	-25.232	0.000	0.000	0.000	0.000	0.000
	2	1:Wind Load	25.232	-0.000	0.000	0.000	0.000	0.000
2	2	1:Wind Load	24.731	0.000	0.000	0.000	0.000	0.000
	3	1:Wind Load	-24.731	-0.000	0.000	0.000	0.000	0.000
3	3	1:Wind Load	11.541	0.000	0.000	0.000	0.000	0.000
	4	1:Wind Load	-11.541	-0.000	0.000	0.000	0.000	0.000
4	4	1:Wind Load	59.666	-0.000	0.000	0.000	0.000	0.000
	2	1:Wind Load	-59.666	0.000	0.000	0.000	0.000	0.000
5	1	1:Wind Load	-27.292	0.000	0.000	0.000	0.000	0.000
	3	1:Wind Load	27.292	-0.000	0.000	0.000	0.000	0.000

Reactions

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-in)	MY (kip-in)	MZ (kip-in)
1	1:Wind Load	-24.731	-36.773	0.000	0.000	0.000	0.000
4	1:Wind Load	-54.069	36.773	0.000	0.000	0.000	0.000

	Job No	Sheet No 1	Rev
Software licensed to PENN STATE UNIVERSITY	Part		
Job Title	Ref		
	By	Date 22-Nov-06	Chd
Client	File Braced Frame Evaluatio	Date/Time 22-Nov-2006 17:00	

Job Information

	Engineer	Checked	Approved
Name:			
Date:	22-Nov-06		

Structure Type	SPACE FRAME

Number of Nodes	5	Highest Node	5
Number of Elements	6	Highest Beam	6

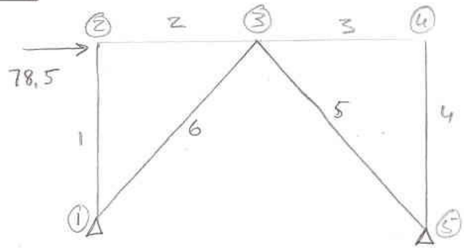
Number of Basic Load Cases	1
Number of Combination Load Cases	0

Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	Wind Load



Beam End Forces

Sign convention is as the action of the joint on the beam.

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
1	1	1:Wind Load	0.000	0.000	0.000	0.000	0.000	-0.000
	2	1:Wind Load	0.000	-0.000	0.000	0.000	0.000	0.000
2	2	1:Wind Load	78.800	0.000	0.000	0.000	0.000	0.000
	3	1:Wind Load	-78.800	0.000	0.000	0.000	0.000	0.000
3	3	1:Wind Load	0.000	0.000	0.000	0.000	0.000	0.000
	4	1:Wind Load	-0.000	0.000	0.000	0.000	0.000	0.000
4	4	1:Wind Load	0.000	0.000	0.000	0.000	0.000	0.000
	5	1:Wind Load	0.000	-0.000	0.000	0.000	0.000	-0.000
5	1	1:Wind Load	-53.895	0.000	0.000	0.000	0.000	0.000
	3	1:Wind Load	53.895	-0.000	0.000	0.000	0.000	0.000
6	3	1:Wind Load	53.895	0.000	0.000	0.000	0.000	0.000
	5	1:Wind Load	-53.895	-0.000	0.000	0.000	0.000	0.000

Reactions

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-in)	MY (kip-in)	MZ (kip-in)
1	1:Wind Load	-118.200	-36.773	0.000	0.000	0.000	0.000
5	1:Wind Load	-39.400	36.773	0.000	0.000	0.000	0.000

Cost Comparison

Connection Cost Estimate:

Moment Connection

- (20) 3/4" x 2" Bolts
- 5' 5/16" 0.4#/LF weld
- 52" 5"x3"x 3/8" Angle

Cost Includes Material, Labor, Eq. & O&P

① Bolts = 5.50 $/Bolt

② Weld = 19.30 $/LF

③ Angle = 1.368 $/LB

Time:

① Bolts = 0.067 h/Bolt

② Weld = 0.211 h/LF

Total Cost

Bolts = 20(5.50) = $110

Weld = 19.30(5') = $96.5

Angle = 1.368(8.19)(52/12) = $48.55

$\boxed{\text{Total} = \$255}$ ($97.45 for labor)

Total Time: 0.067(20) + 0.211(5) = $\boxed{2.40 \text{ hours}}$

Connection Cost Estimate:

Shear Connection:

# of Bolts = 9 + 6 = 15 Bolts/connection

Cost = $3.50/Bolt

Time = 0.067 hours/Bolt

Two Angles = 13.65 lbs
Two Plates = 12.32 lbs

- Cost of Bolts = (15)(3.50) = $52.5
- Total cost of Bolts including Mat, labor, Eq, O&P
  (15)(5.55) = $ 83.25
- Total cost of angles = $1.35 \frac{\$}{lb} \times 13.65\,lb = \$18.5$
- Total cost of plates = $\frac{21.50\$}{SF} \cdot \frac{SF}{20.4\,lbs} \cdot 12.32\,lbs$

  = $13.0 / 2 plates

Total Cost = $ 83.25 + $18.5 + $ 13.0 = $\boxed{\$114.75}$
(39.9 $ for labor)
per connection

Total time = 0.067(15) = $\boxed{1.005 \text{ hours}}$

3

Connection cost Estimate

Shear connection - Non. Braced Frame.

\# of Bolts = 9
Two angles = 13.65 lbs

Cost of Bolts = 5.55 $/Bolt

Cost of Angles = 1.35 $/lb

Total cost = 9(5.55) + 1.35(13.65) = $\boxed{\$68.38}$

Total time = 0.067(9) = $\boxed{0.603 \text{ hrs}}$

Estimate of cost difference between original & new columns.

Labor, Equipment, and O&P, costs difference between the original and new columns are neglegable. The only variable is the material cost.

- Material cost of original = 1.04(72) = 74.88 #/LF
- Material cost of new = 1.04(65) = 67.60 #/LF

Difference = $ 7.28 /LF => ≈10% increase

Weight difference: ≈ 36(72)(42) − 36(65)(42) + 36(53)(28) − 36(50)(28) ≈ 13,608 lbs

$ = 1.04(13608) = $ 14152.32

Total Difference in column cost is $\boxed{\$\ 14152.32}$

Original Moment Frame Columns: $ 168,779.52

Redesigned Columns : $ 154,627.20

5

Diagonals Cost Estimate

WT9 x 38 => 132.40 ft
WT8 x 33.5 => 796.91 ft

Material = 1.04 \$/lb
Labor = 2.02 \$/LF
Equip = 1.32 \$/LF
O&P = 11%

$$WT9 \times 38 = \left(1.04 \frac{\$}{lb} \times \frac{38\ lb}{LF} + 2.02 + 1.32\right) 132.4$$

$= \$5674.66$

$+$ O&P

$= 5,674.66 + 5,674.66(.11) = \underline{\$6,298.88}$

$WT8 \times 33.5 = (1.04(33.5) + 2.02 + 1.32)796.91$

$= \$31222.93$

$+$ O&P

$= 31222.93 + 31222.93(.11) = \$34657.46$

Total Cost = $\boxed{\$40,956.}$

Time: $0.057(132.4 + 796.91) = 53$ hrs

= 371 hours for 1 person

6

Cost Estimate of Additional Columns

W12x65 => 84'
W12x35 => 56'

Material : 1.04 \$/lb
Labor : 2.15 \$/LF
Equip : 1.38 \$/LF
O+P : 11%

W12x65 => $[1.04(65) + 2.15 + 1.38](84) = 5974.92$

$5974.92 + .11(5974.92) = 6632.16$ \$

W12x35 => $[1.04(35) + 2.15 + 1.38]\, 56 = 2236.08$

$2236.08 + .11(2236.08) = 2482.05$

Total Cost = $\boxed{\$9,114.2}$

Time = $0.057(84+56) = 7.98$ (7 people)

≈ 55.86 hrs / 1 person

7

Footing Cost Estimate

Using the Interpolation:

Material = 5.50 $/CF

Labor = 4.25 $/CF

Total 9.75 $/CF

FRAME A: $12 \times 12 \times \frac{22}{12} = 264$ CF => $ 2,574

B: $10 \times 10 \times \frac{16}{12} = 133.3$ CF => $ 1,300

:Note Additional Footing + Pier => $ 1,365

C: $10' \times 10' \times \frac{20'}{12} = 166.7$ cf => $ 1,625

D: $ 1,625

E: $ 2,574

F: $ 1,300

G: $11' \times 11' \times \frac{22'}{12} = 221.8$ CF => $ 2,163

Typical Non Braced Frame:

$11' \times 11' \times 22'' = 183$ CF $ 2163

Typical Original: $11' \times 11' \times 28'' = 283.33$ CF $ 2,753

8

Footing Cost Estimate + Comparison

Original @ Frame B => 10x10x24 => $1950

Original @ Frame C => 8'x8'x28 => $1456

Original @ Frame G => 10'x10'x28" => $ 2,275

Original @ Frame A => 11'x11'x28 => $ 2,753

Original System:

≈ 34	Typical	=> $ 93602
4	@ C+D	=> $ 5824
2	@ G	=> $ 4550
Total		$ 103976

New System:

	≈ 30	Typical	=> $ 64890
	4	A+E	=> $ 10296
New	* 2	B+F	=> $ 2730
	4	C+D	=> $ 6500
	2	G	=> $ 4326
	Total		$ 88962

Difference: ≈ $ $\boxed{15,000}$

9

Total Cost Comparison,

Cost Difference due to change or addition:

Footing	=> $ 15000	(+)
Connections	=> $ 136035	(+)
Column size	=> $ 14153	(+)
Diagonals	-> $ 40,956	(-)
New Columns	=> $ 9,114	(-)
TOTAL	$\boxed{\$\ 86{,}812}$	

Connections

Moment Connections : $ 191250
Shear Connections s : $ 55,215

New Design is $\boxed{\$86{,}812}$ less expensive

Impact on schedule,

Original system:

750 connections @ 2.40 hrs/connection

750 (2.4) = 1800 hrs

New system:

- 690 connections @ 0.63 hrs/conn

- Diagonals 371 hours (1 person)
- New columns 56 hours
- New footing 40 hours (1 person)

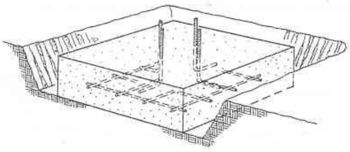
- Δ footing size $\approx$ 0 hours

Total Time = 690(.63) + 371 + 56 + 40

= 901.7 hrs

Difference $\approx$ 900 hours (1 person

A1010 Standard Foundations



The Spread Footing System includes: excavation; backfill; forms (four uses); all reinforcement; 3,000 p.s.i. concrete (chute placed); and screed finish.

Footing systems are priced per individual unit. The Expanded System Listing at the bottom shows footings that range from 3′ square x 12″ deep, to 18′ square x 52″ deep. It is assumed that excavation is done by a truck mounted hydraulic excavator with an operator and oiler.

Backfill is with a dozer, and compaction by air tamp. The excavation and backfill equipment is assumed to operate at 30 C.Y. per hour.

Please see the reference section for further design and cost information.

System Components	QUANTITY	UNIT	COST EACH MAT.	COST EACH INST.	COST EACH TOTAL
SYSTEM A1010 210 7100					
SPREAD FOOTINGS, LOAD 25K, SOIL CAPACITY 3 KSF, 3′ SQ X 12″ DEEP					
Bulk excavation	.590	C.Y.		4.51	4.51
Hand trim	9.000	S.F.		6.84	6.84
Compacted backfill	.260	C.Y.		.79	.79
Formwork, 4 uses	12.000	S.F.	7.80	50.16	57.96
Reinforcing, fy = 60,000 psi	.006	Ton	5.61	6.15	11.76
Dowel or anchor bolt templates	6.000	L.F.	5.58	20.76	26.34
Concrete, f′c = 3,000 psi	.330	C.Y.	37.62		37.62
Place concrete, direct chute	.330	C.Y.		6.27	6.27
Screed finish	9.000	S.F.		4.14	4.14
TOTAL			56.61	99.62	156.23

A1010 210	Spread Footings	COST EACH MAT.	COST EACH INST.	COST EACH TOTAL
7090	Spread footings, 3000 psi concrete, chute delivered			
7100	Load 25K, soil capacity 3 KSF, 3′-0″ sq. x 12″ deep	56.50	99.50	156
7150	Load 50K, soil capacity 3 KSF, 4′-6″ sq. x 12″ deep RA1010 -120	121	172	293
7200	Load 50K, soil capacity 6 KSF, 3′-0″ sq. x 12″ deep	56.50	99.50	156
7250	Load 75K, soil capacity 3 KSF, 5′-6″ sq. x 13″ deep	193	244	437
7300	Load 75K, soil capacity 6 KSF, 4′-0″ sq. x 12″ deep	98	147	245
7350	Load 100K, soil capacity 3 KSF, 6′-0″ sq. x 14″ deep	245	292	537
7410	Load 100K, soil capacity 6 KSF, 4′-6″ sq. x 15″ deep	150	203	353
7450	Load 125K, soil capacity 3 KSF, 7′-0″ sq. x 17″ deep	390	420	810
7500	Load 125K, soil capacity 6 KSF, 5′-0″ sq. x 16″ deep	193	244	437
7550	Load 150K, soil capacity 3 KSF 7′-6″ sq. x 18″ deep	470	490	960
7610	Load 150K, soil capacity 6 KSF, 5′-6″ sq. x 18″ deep	258	305	563
7650	Load 200K, soil capacity 3 KSF, 8′-6″ sq. x 20″ deep	670	650	1,320
7700	Load 200K, soil capacity 6 KSF, 6′-0″ sq. x 20″ deep	340	380	720
7750	Load 300K, soil capacity 3 KSF, 10′-6″ sq. x 25″ deep	1,250	1,050	2,300
7810	Load 300K, soil capacity 6 KSF, 7′-6″ sq. x 25″ deep	645	635	1,280
7850	Load 400K, soil capacity 3 KSF, 12′-6″ sq. x 28″ deep	1,975	1,550	3,525
7900	Load 400K, soil capacity 6 KSF, 8′-6″ sq. x 27″ deep	895	825	1,720
7950	Load 500K, soil capacity 3 KSF, 14′-0″ sq. x 31″ deep	2,725	2,050	4,775
8010	Load 500K, soil capacity 6 KSF, 9′-6″ sq. x 30″ deep	1,225	1,075	2,300

05 05 Common Work Results for Metals

05 05 23 – Metal Fastenings

05 05 23.05 Anchor Bolts		Crew	Daily Output	Labor-Hours	Unit	2007 Bare Costs Material	Labor	Equipment	Total
0690	42″ long	2 Carp	11	1.455	Ea.	28.50	53.50		82
0700	48″ long		10	1.600		32.50	58.50		91
0710	54″ long		9	1.778		38.50	65		103.50
0720	60″ long		8	2		41.50	73.50		115
0730	66″ long		7	2.286		44.50	84		128.50
0740	72″ long		6	2.667		49	98		147
0990	For galvanized, add					75%			

05 05 23.10 Bolts and Hex Nuts		Crew	Daily Output	Labor-Hours	Unit	Material	Labor	Equipment	Total
0010	**BOLTS & HEX NUTS**, Steel, A307								
0100	1/4″ diameter, 1/2″ long	1 Sswk	140	.057	Ea.	.07	2.36		2.43
0200	1″ long		140	.057		.08	2.36		2.44
0300	2″ long		130	.062		.11	2.54		2.65
0400	3″ long		130	.062		.16	2.54		2.70
0500	4″ long		120	.067		.18	2.76		2.94
0600	3/8″ diameter, 1″ long		130	.062		.12	2.54		2.66
0700	2″ long		130	.062		.15	2.54		2.69
0800	3″ long		120	.067		.20	2.76		2.96
0900	4″ long		120	.067		.25	2.76		3.01
1000	5″ long		115	.070		.31	2.88		3.19
1100	1/2″ diameter, 1-1/2″ long		120	.067		.24	2.76		3
1200	2″ long		120	.067		.27	2.76		3.03
1300	4″ long		115	.070		.41	2.88		3.29
1400	6″ long		110	.073		.56	3.01		3.57
1500	8″ long		105	.076		.73	3.15		3.88
1600	5/8″ diameter, 1-1/2″ long		120	.067		.47	2.76		3.23
1700	2″ long		120	.067		.51	2.76		3.27
1800	4″ long		115	.070		.71	2.88		3.59
1900	6″ long		110	.073		.89	3.01		3.90
2000	8″ long		105	.076		1.29	3.15		4.44
2100	10″ long		100	.080		1.60	3.31		4.91
2200	3/4″ diameter, 2″ long		120	.067		.74	2.76		3.50
2300	4″ long		110	.073		1.02	3.01		4.03
2400	6″ long		105	.076		1.30	3.15		4.45
2500	8″ long		95	.084		1.92	3.48		5.40
2600	10″ long		85	.094		2.50	3.89		6.39
2700	12″ long		80	.100		2.91	4.14		7.05
2800	1″ diameter, 3″ long		105	.076		1.90	3.15		5.05
2900	6″ long		90	.089		2.93	3.68		6.61
3000	12″ long		75	.107		5.50	4.41		9.91
3100	For galvanized, add					75%			
3200	For stainless, add					350%			

05 05 23.15 Chemical Anchors		Crew	Daily Output	Labor-Hours	Unit	Material	Labor	Equipment	Total
0010	**CHEMICAL ANCHORS**								
0020	Includes layout & drilling								
1430	Chemical anchor, w/rod & epoxy cartridge, 3/4″ diam. x 9-1/2″ long	B-89A	27	.593	Ea.	13.70	19.80	3.89	37.39
1435	1″ diameter x 11-3/4″ long		24	.667		26.50	22.50	4.37	53.37
1440	1-1/4″ diameter x 14″ long		21	.762		50.50	25.50	5	81
1445	1-3/4″ diameter x 15″ long		20	.800		95	26.50	5.25	126.75
1450	18″ long		17	.941		114	31.50	6.15	151.65
1455	2″ diameter x 18″ long		16	1		145	33.50	6.55	185.05
1460	24″ long		15	1.067		190	35.50	7	232.50

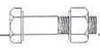
05100 | Structural Metal Framing

05120 Structural Steel		CREW	DAILY OUTPUT	LABOR-HOURS	UNIT	2006 BARE COSTS MAT.	LABOR	EQUIP.	TOTAL	TOTAL INCL O&P	
0010	COLUMNS, LIGHTWEIGHT										250
1000	Lightweight units (lally), 3-1/2" diameter	E-2	780	.072	L.F.	2.99	2.79	1.83	7.61	10.10	
1050	4" diameter	"	900	.062	"	4.40	2.42	1.59	8.41	10.75	
5800	Adjustable jack post, 8' maximum height, 2-3/4" diameter				Ea.	32			32	35	
5850	4" diameter				"	51			51	56	
0010	COLUMNS, STRUCTURAL R051223-10										260
0020	Shop fab'd for 100-ton, 1-2 story project, bolted conn's.										
0800	Steel, concrete filled, extra strong pipe, 3-1/2" diameter	E-2	660	.085	L.F.	31.50	3.30	2.16	36.96	42.50	
0830	4" diameter		780	.072		35	2.79	1.83	39.62	45.50	
0890	5" diameter		1020	.055		41.50	2.14	1.40	45.04	51	
0930	6" diameter		1200	.047		55	1.82	1.19	58.01	65	
0940	8" diameter		1100	.051		55	1.98	1.30	58.28	65.50	
1100	For galvanizing, add				Lb.	.22			.22	.25	
1300	For web ties, angles, etc., add per added lb.	1 Sswk	945	.008		.95	.34		1.29	1.66	
1500	Steel pipe, extra strong, no concrete, 3" to 5" diameter	E-2	16000	.004		.95	.14	.09	1.18	1.38	
1600	6" to 12" diameter		14000	.004		.95	.16	.10	1.21	1.43	
1700	Steel pipe, extra strong, no concrete, 3" diameter x 12'-0"		60	.933	Ea.	117	36.50	24	177.50	218	
1750	4" diameter x 12'-0"		58	.966		171	37.50	24.50	233	280	
1800	6" diameter x 12'-0"		54	1.037		325	40.50	26.50	392	460	
1850	8" diameter x 14'-0"		50	1.120		575	43.50	28.50	647	740	
1900	10" diameter x 16'-0"		48	1.167		830	45.50	29.50	905	1,025	
1950	12" diameter x 18'-0"		45	1.244		1,125	48.50	31.50	1,205	1,350	
3300	Structural tubing, square, A500GrB, 4" to 6" square, light section		11270	.005	Lb.	.95	.19	.13	1.27	1.52	
3600	Heavy section		32000	.002	"	.95	.07	.04	1.06	1.22	
4000	Concrete filled, add				L.F.	3.47			3.47	3.81	
4500	Structural tubing, sq, 4" x 4" x 1/4" x 12'-0"	E-2	58	.966	Ea.	157	37.50	24.50	219	264	
4550	6" x 6" x 1/4" x 12'-0"		54	1.037		257	40.50	26.50	324	380	
4600	8" x 8" x 3/8" x 14'-0"		50	1.120		555	43.50	28.50	627	715	
4650	10" x 10" x 1/2" x 16'-0"		48	1.167		1,025	45.50	29.50	1,100	1,225	
5100	Structural tubing, rect, 5" to 6" wide, light section		8000	.007	Lb.	.95	.27	.18	1.40	1.72	
5200	Heavy section		12000	.005		.95	.18	.12	1.25	1.49	
5300	7" to 10" wide, light section		15000	.004		.95	.15	.10	1.20	1.40	
5400	Heavy section		18000	.003		.95	.12	.08	1.15	1.35	
5500	Structural tubing, rect, 5" x 3" x 1/4" x 12'-0"		58	.966	Ea.	152	37.50	24.50	214	259	
5550	6" x 4" x 5/16" x 12'-0"		54	1.037		238	40.50	26.50	305	360	
5600	8" x 4" x 3/8" x 12'-0"		54	1.037		345	40.50	26.50	412	480	
5650	10" x 6" x 3/8" x 14'-0"		50	1.120		555	43.50	28.50	627	715	
5700	12" x 8" x 1/2" x 16'-0"		48	1.167		1,025	45.50	29.50	1,100	1,225	
6800	W Shape, A992 steel, 2 tier, W8 x 24		1080	.052	L.F.	25	2.02	1.32	28.34	32.50	
6850	W8 x 31		1080	.052		32.50	2.02	1.32	35.84	40.50	
6900	W8 x 48		1032	.054		50	2.11	1.38	53.49	60	
6950	W8 x 67		984	.057		70	2.21	1.45	73.66	82.50	
7000	W10 x 45		1032	.054		47	2.11	1.38	50.49	56.50	
7050	W10 x 68		984	.057		71	2.21	1.45	74.66	83.50	
7100	W10 x 112		960	.058		117	2.27	1.49	120.76	135	
7150	W12 x 50		1032	.054		52.50	2.11	1.38	55.99	62.50	
7200	W12 x 87		984	.057		91	2.21	1.45	94.66	105	
7250	W12 x 120		960	.058		125	2.27	1.49	128.76	144	
7300	W12 x 190		912	.061		199	2.39	1.56	202.95	224	
7350	W14 x 74		984	.057		77.50	2.21	1.45	81.16	90.50	
7400	W14 x 120		960	.058		125	2.27	1.49	128.76	144	
7450	W14 x 176		912	.061		184	2.39	1.56	187.95	208	
8090	For projects 75 to 99 tons, add				All	10%					
8092	50 to 74 tons, add					20%					
8094	25 to 49 tons, add					30%	10%				

METALS 5

05050 | Basic Metal Materials & Methods

05090 | Metal Fastenings

		Description	Crew	Daily Output	Labor-Hours	Unit	2006 Bare Costs Mat.	2006 Bare Costs Labor	2006 Bare Costs Equip.	2006 Bare Costs Total	Total Incl O&P	
080	0480	54" long	2 Carp	12	1.333	Ea.	13.35	47.50		60.85	88.50	080
	0490	60" long		10	1.600		14.65	57		71.65	105	
	0500	1-1/2" diameter x 18" long		22	.727		8.65	26		34.65	50	
	0510	24" long		19	.842		10.10	30		40.10	57.50	
	0520	30" long		17	.941		11.40	33.50		44.90	64.50	
	0530	36" long		16	1		13.10	35.50		48.60	70	
	0540	42" long		15	1.067		14.90	38		52.90	75.50	
	0550	48" long		13	1.231		16.75	44		60.75	86.50	
	0560	54" long		11	1.455		20.50	51.50		72	103	
	0570	60" long		9	1.778		22.50	63		85.50	123	
	0580	1-3/4" diameter x 18" long		20	.800		13.05	28.50		41.55	59	
	0590	24" long		18	.889		15.30	31.50		46.80	66	
	0600	30" long		17	.941		17.80	33.50		51.30	71.50	
	0610	36" long		16	1		20.50	35.50		56	78	
	0620	42" long		14	1.143		23	40.50		63.50	88.50	
	0630	48" long		12	1.333		25	47.50		72.50	102	
	0640	54" long		10	1.600		31	57		88	123	
	0650	60" long		8	2		33.50	71		104.50	148	
	0660	2" diameter x 24" long		17	.941		19.50	33.50		53	73.50	
	0670	30" long		15	1.067		22	38		60	83	
	0680	36" long		13	1.231		24	44		68	94.50	
	0690	42" long		11	1.455		27	51.50		78.50	110	
	0700	48" long		10	1.600		31	57		88	123	
	0710	54" long		9	1.778		37	63		100	139	
	0720	60" long		8	2		39.50	71		110.50	155	
	0730	66" long		7	2.286		42.50	81.50		124	174	
	0740	72" long	▼	6	2.667		46.50	95		141.50	199	
	0990	For galvanized, add				▼	75%					
150	0010	**BOLTS & HEX NUTS** Steel, A307										150
	0100	1/4" diameter, 1/2" long	1 Sswk	140	.057	Ea.	.07	2.28		2.35	4.19	
	0200	1" long		140	.057		.08	2.28		2.36	4.20	
	0300	2" long 		130	.062		.11	2.46		2.57	4.54	
	0400	3" long		130	.062		.16	2.46		2.62	4.60	
	0500	4" long		120	.067		.18	2.66		2.84	4.98	
	0600	3/8" diameter, 1" long		130	.062		.12	2.46		2.58	4.55	
	0700	2" long		130	.062		.15	2.46		2.61	4.59	
	0800	3" long		120	.067		.20	2.66		2.86	5	
	0900	4" long		120	.067		.25	2.66		2.91	5.05	
	1000	5" long		115	.070		.31	2.78		3.09	5.35	
	1100	1/2" diameter, 1-1/2" long		120	.067		.22	2.66		2.88	5.05	
	1200	2" long		120	.067		.25	2.66		2.91	5.05	
	1300	4" long		115	.070		.38	2.78		3.16	5.40	
	1400	6" long		110	.073		.52	2.91		3.43	5.80	
	1500	8" long		105	.076		.68	3.04		3.72	6.25	
	1600	5/8" diameter, 1-1/2" long		120	.067		.45	2.66		3.11	5.30	
	1700	2" long		120	.067		.49	2.66		3.15	5.35	
	1800	4" long		115	.070		.68	2.78		3.46	5.75	
	1900	6" long		110	.073		.85	2.91		3.76	6.20	
	2000	8" long		105	.076		1.23	3.04		4.27	6.85	
	2100	10" long		100	.080		1.53	3.20		4.73	7.45	
	2200	3/4" diameter, 2" long		120	.067		.70	2.66		3.36	5.55	
	2300	4" long		110	.073		.97	2.91		3.88	6.30	
	2400	6" long		105	.076		1.23	3.04		4.27	6.85	
	2500	8" long		95	.084		1.83	3.36		5.19	8.05	
	2600	10" long		85	.094		2.38	3.76		6.14	9.35	
	2700	12" long	▼	80	.100	▼	2.78	4		6.78	10.25	

5 METALS

05090 | Metal Fastenings

		05090 Metal Fastenings	Crew	Daily Output	Labor-Hours	Unit	2006 Bare Costs Mat.	Labor	Equip.	Total	Total Incl O&P
880	0620	Steel, type 6010, 1/8" dia., less than 500#				Lb.	2.09			2.09	[illegible]
	0630	500# to 2,000#					1.88			1.88	[illegible]
	0640	2,000# to 5,000#					1.77			1.77	[illegible]
	0650	Steel, type 7018 Low Hydrogen, 1/8" dia., less than 500#					1.79			1.79	[illegible]
	0660	500# to 2,000#					1.61			1.61	[illegible]
	0670	2,000# to 5,000#					1.51			1.51	[illegible]
	0700	Steel, type 7024 Jet Weld, 1/8" dia., less than 500#					1.89			1.89	[illegible]
	0710	500# to 2,000#					1.70			1.70	[illegible]
	0720	2,000# to 5,000#					1.60			1.60	[illegible]
	1550	Aluminum, type 4043 TIG, 1/8" dia., less than 10#					6.55			6.55	[illegible]
	1560	10# to 60#					5.90			5.90	[illegible]
	1570	Over 60#					5.55			5.55	[illegible]
	1600	Aluminum, type 5356 TIG, 1/8" dia., less than 10#					7.25			7.25	[illegible]
	1610	10# to 60#					6.55			6.55	[illegible]
	1620	Over 60#					6.15			6.15	[illegible]
	1900	Cast iron, type 8 Nickel, 1/8" dia., less than 500#					22.50			22.50	[illegible]
	1910	500# to 1,000#					20			20	22
	1920	Over 1,000#					19			19	21
	2000	Stainless steel, type 316/316L, 1/8" dia., less than 500#					10.30			10.30	[illegible]
	2100	500# to 1000#					9.30			9.30	[illegible]
	2220	Over 1000#					8.75			8.75	[illegible]
900	0010	**WELDING STRUCTURAL**									
	0020	Field welding, 1/8" E6011, cost per welder, no oper. engr R050521-20	E-14	8	1	Hr.	3.80	42	11.15	56.95	92
	0200	With 1/2 operating engineer	E-13	8	1.500		3.80	59.50	11.15	74.45	118
	0300	With 1 operating engineer	E-12	8	2		3.80	77	11.15	91.95	145
	0500	With no operating engineer, 2# weld rod per ton	E-14	8	1	Ton	3.80	42	11.15	56.95	92
	0600	8# E6011 per ton	"	2	4		15.20	168	44.50	227.70	365
	0800	With one operating engineer per welder, 2# E6011 per ton	E-12	8	2		3.80	77	11.15	91.95	145
	0900	8# E6011 per ton	"	2	8		15.20	310	44.50	369.70	580
	1200	Continuous fillet, stick welding, incl. equipment									
	1300	Single pass, 1/8" thick, 0.1#/L.F.	E-14	150	.053	L.F.	.19	2.24	.59	3.02	4.89
	1400	3/16" thick, 0.2#/L.F.		75	.107		.38	4.47	1.19	6.04	9.80
	1500	1/4" thick, 0.3#/L.F.		50	.160		.57	6.70	1.78	9.05	14.70
	1610	5/16" thick, 0.4#/L.F.		38	.211		.76	8.85	2.35	11.96	19.30
	1800	3 passes, 3/8" thick, 0.5#/L.F.		30	.267		.95	11.20	2.97	15.12	24.50
	2010	4 passes, 1/2" thick, 0.7#/L.F.		22	.364		1.33	15.25	4.05	20.63	33.50
	2200	5 to 6 passes, 3/4" thick, 1.3#/L.F.		12	.667		2.47	28	7.45	37.92	61.50
	2400	8 to 11 passes, 1" thick, 2.4#/L.F.		6	1.333		4.56	56	14.85	75.41	122
	2600	For all position welding, add, minimum						20%			
	2700	Maximum						300%			
	2900	For semi-automatic welding, deduct, minimum						5%			
	3000	Maximum						15%			
	4000	Cleaning and welding plates, bars, or rods									
	4010	to existing beams, columns, or trusses	E-14	12	.667	L.F.	.95	28	7.45	36.40	59.50
920	0010	**STEEL CUTTING**									
	0020	Hand burning, incl. preparation, torch cutting & grinding, no staging									
	0100	Steel to 1/2" thick	E-25	320	.025	L.F.		1.05	.25	1.30	2.17
	0150	3/4" thick		260	.031			1.29	.31	1.60	2.66
	0200	1" thick		200	.040			1.68	.41	2.09	3.47

05120 | St

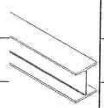
250	0010	COLUMNS, LI
	1000	Lightw
	1050	4"
	5800	Adjustable
	5850	4" diam
260	0010	COLUMNS, ST
	0020	Shop fab'd
	0800	Steel, conc
	0830	4"
	0890	5"
	0930	6"
	0940	8"
	1100	For galv
	1300	For wet
	1500	Steel pipe,
	1600	6" to 1
	1700	Steel pipe,
	1750	
	1800	
	1850	
	1900	
	1950	
	3300	Structural tu
	3600	Hea
	4000	Concret
	4500	Structural tu
	4550	6" x
	4600	8" x
	4650	10"
	5100	Structural tu
	5200	Hea
	5300	7" to 10
	5400	Hea
	5500	Structural tu
	5550	6" x
	5600	8" x
	5650	10"
	5700	12"
	6800	W Shape, A9
	6850	
	6900	
	6950	
	7000	
	7050	
	7100	
	7150	
	7200	
	7250	
	7300	
	7350	
	7400	
	7450	
	8090	For projects
	8092	50 to 74
	8094	25 to 49

05100 | Structural Metal Framing

05120		Structural Steel	Crew	Daily Output	Labor-Hours	Unit	2006 Bare Costs Mat.	Labor	Equip.	Total
260	8096	10 to 24 tons, add (R051223 -10)				All	50%	25%		
	8098	2 to 9 tons, add					75%	50%		
	8099	Less than 2 tons, add					100%	100%		
300	0010	**CURB EDGING**								
	0020	Steel angle w/anchors, on forms, 1" x 1", 0.8#/L.F.	E-4	350	.091	L.F.	1.61	3.70	.26	5.57
	0100	2" x 2" angles, 3.92#/L.F.		330	.097		5.15	3.92	.27	9.34
	0200	3" x 3" angles, 6.1#/L.F.		300	.107		8.15	4.31	.30	12.76
	0300	4" x 4" angles, 8.2#/L.F.		275	.116		10.55	4.71	.32	15.58
	1000	6" x 4" angles, 12.3#/L.F.		250	.128		15.25	5.20	.36	20.81
	1050	Steel channels with anchors, on forms, 3" channel, 5#/L.F.		290	.110		6.40	4.46	.31	11.17
	1100	4" channel, 5.4#/L.F.		270	.119		6.85	4.79	.33	11.97
	1200	6" channel, 8.2#/L.F.		255	.125		10.55	5.10	.35	16
	1300	8" channel, 11.5#/L.F.		225	.142		14.35	5.75	.40	20.50
	1400	10" channel, 15.3#/L.F.		180	.178		18.65	7.20	.50	26.35
	1500	12" channel, 20.7#/L.F.		140	.229		25	9.25	.64	34.89
	2000	For curved edging, add					35%	10%		
440	0010	**LIGHTWEIGHT FRAMING** (R051223 -35)								
	0400	Angle framing, field fabricated, 4" and larger	E-3	440	.055	Lb.	.55	2.22	.20	2.97
	0450	Less than 4" angles (R051223 -45)		265	.091		.57	3.68	.34	4.59
	0600	Channel framing, field fabricated, 8" and larger		500	.048		.57	1.95	.18	2.70
	0650	Less than 8" channels		335	.072		.57	2.91	.27	3.75
	1000	Continuous slotted channel framing system, shop fab, min.	2 Sswk	2400	.007		2.95	.27		3.22
	1200	Maximum	″	1600	.010		3.33	.40		3.73
	1250	Plate & bar stock for reinforcing beams and trusses					1.05			1.05
	1300	Cross bracing, rods, shop fabricated, 3/4" diameter	E-3	700	.034		1.14	1.39	.13	2.66
	1310	7/8" diameter		850	.028		1.14	1.15	.11	2.40
	1320	1" diameter		1000	.024		1.14	.97	.09	2.20
	1330	Angle, 5" x 5" x 3/8"		2800	.009		1.14	.35	.03	1.52
	1350	Hanging lintels, shop fabricated, average		850	.028		1.14	1.15	.11	2.40
	1380	Roof frames, shop fabricated, 3'-0" square, 5' span	E-2	4200	.013		1.14	.52	.34	2
	1400	Tie rod, not upset, 1-1/2" to 4" diameter, with turnbuckle	2 Sswk	800	.020		1.24	.80		2.04
	1420	No turnbuckle		700	.023		1.19	.91		2.10
	1500	Upset, 1-3/4" to 4" diameter, with turnbuckle		800	.020		1.24	.80		2.04
	1520	No turnbuckle		700	.023		1.19	.91		2.10
480	0010	**LINTELS**								
	0020	Plain steel angles, under 500 lb.	1 Bric	550	.015	Lb.	.73	.53		1.26
	0100	500 to 1000 lb.		640	.013		.71	.46		1.17
	0200	1,000 to 2,000 lb.		640	.013		.69	.46		1.15
	0300	2,000 to 4,000 lb.		640	.013		.67	.46		1.13
	0500	For built-up angles and plates, add to above					.24			.24
	0700	For engineering, add to above					.10			.10
	0900	For galvanizing, add to above, under 500 lb.					.30			.30
	0950	500 to 2,000 lb.					.27			.27
	1000	Over 2,000 lb.					.22			.22
520	0010	**PIPE SUPPORT FRAMING**								
	0020	Under 10#/L.F.	E-4	3900	.008	Lb.	1.27	.33	.02	1.62
	0200	10.1 to 15#/L.F.		4300	.007		1.25	.30	.02	1.57
	0400	15.1 to 20#/L.F.		4800	.007		1.24	.27	.02	1.53
	0600	Over 20#/L.F.		5400	.006		1.22	.24	.02	1.48
560	0010	**PLATES** (R051223 -80)								
	0020	For connections & stiffener plates, shop fabricated								
	0050	1/8" thick (5.1 Lb./S.F.)				S.F.	4.85			4.85
	0100	1/4" thick (10.2 Lb./S.F.)					9.70			9.70

05100 | Structural Metal Framing

05120 Structural Steel			CREW	DAILY OUTPUT	LABOR-HOURS	UNIT	2006 BARE COSTS MAT.	LABOR	EQUIP.	TOTAL	TOTAL INCL O&P	
560	0300	3/8" thick (15.3 Lb./S.F.) R051223-80				S.F.	14.55			14.55	16	560
	0400	1/2" thick (20.4 Lb./S.F.)					19.40			19.40	21.50	
	0450	3/4" thick (30.6 Lb./S.F.)					29			29	32	
	0500	1" thick (40.8 Lb.S.F.)					39			39	42.50	
600	0010	STRESSED SKIN ROOF & CEILING SYSTEM										600
	0020	Double panel flat roof, spans to 100'	E-2	1150	.049	S.F.	7.60	1.89	1.24	10.73	13	
	0100	Double panel convex roof, spans to 200'		960	.058		12.35	2.27	1.49	16.11	19.15	
	0200	Double panel arched roof, spans to 300'		760	.074		19	2.87	1.88	23.75	28	
640	0010	STRUCTURAL STEEL MEMBERS R051223-10										640
	0020	Shop fab'd for 100-ton, 1-2 story project, bolted conn's.										
	0100	W 6 x 9	E-2	600	.093	L.F.	9.40	3.63	2.38	15.41	19.20	
	0120	x 16		600	.093		16.70	3.63	2.38	22.71	27.50	
	0140	x 20		600	.093		21	3.63	2.38	27.01	32	
	0300	W 8 x 10		600	.093		10.45	3.63	2.38	16.46	20.50	
	0320	x 15		600	.093		15.70	3.63	2.38	21.71	26	
	0350	x 21		600	.093		22	3.63	2.38	28.01	33	
	0360	x 24		550	.102		25	3.96	2.59	31.55	37	
	0370	x 28		550	.102		29.50	3.96	2.59	36.05	41.50	
	0500	x 31		550	.102		32.50	3.96	2.59	39.05	45	
	0520	x 35		550	.102		36.50	3.96	2.59	43.05	49.50	
	0540	x 48		550	.102		50	3.96	2.59	56.55	64.50	
	0600	W 10 x 12		600	.093		12.55	3.63	2.38	18.56	22.50	
	0620	x 15		600	.093		15.70	3.63	2.38	21.71	26	
	0700	x 22		600	.093		23	3.63	2.38	29.01	34.50	
	0720	x 26		600	.093		27	3.63	2.38	33.01	39	
	0740	x 33		550	.102		34.50	3.96	2.59	41.05	47.50	
	0900	x 49		550	.102		51	3.96	2.59	57.55	66	
	1100	W 12 x 14		880	.064		14.65	2.48	1.62	18.75	22	
	1300	x 22		880	.064		23	2.48	1.62	27.10	31.50	
	1500	x 26		880	.064		27	2.48	1.62	31.10	36	
	1520	x 35		810	.069		36.50	2.69	1.76	40.95	46.50	
	1560	x 50		750	.075		52.50	2.90	1.90	57.30	64.50	
	1580	x 58		750	.075		60.50	2.90	1.90	65.30	73.50	
	1700	x 72		640	.088		75	3.40	2.23	80.63	91.50	
	1740	x 87		640	.088		91	3.40	2.23	96.63	108	
	1900	W 14 x 26		990	.057		27	2.20	1.44	30.64	35.50	
	2100	x 30		900	.062		31.50	2.42	1.59	35.51	40.50	
	2300	x 34		810	.069		35.50	2.69	1.76	39.95	45.50	
	2320	x 43		810	.069		45	2.69	1.76	49.45	56	
	2340	x 53		800	.070		55.50	2.72	1.78	60	67.50	
	2360	x 74		760	.074		77.50	2.87	1.88	82.25	92	
	2380	x 90		740	.076		94	2.94	1.93	98.87	110	
	2500	x 120		720	.078		125	3.03	1.98	130.01	145	
	2700	W 16 x 26		1000	.056		27	2.18	1.43	30.61	35.50	
	2900	x 31		900	.062		32.50	2.42	1.59	36.51	41.50	
	3100	x 40		800	.070		42	2.72	1.78	46.50	52.50	
	3120	x 50		800	.070		52.50	2.72	1.78	57	64	
	3140	x 67		760	.074		70	2.87	1.88	74.75	84	
	3300	W 18 x 35	E-5	960	.083		36.50	3.28	1.58	41.36	47.50	
	3500	x 40		960	.083		42	3.28	1.58	46.86	53.50	
	3520	x 46		960	.083		48	3.28	1.58	52.86	60.50	
	3700	x 50		912	.088		52.50	3.46	1.66	57.62	65.50	
	3900	x 55		912	.088		57.50	3.46	1.66	62.62	71	
	3920	x 65		900	.089		68	3.50	1.68	73.18	82.50	
	3940	x 76		900	.089		79.50	3.50	1.68	84.68	95.50	
	3960	x 86		900	.089		90	3.50	1.68	95.18	107	



METALS 5