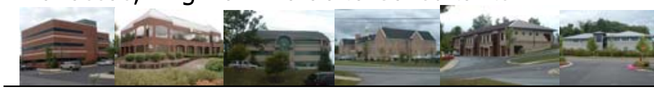


EXECUTIVE SUMMARY

The Signal Hill Professional Center is a four-story suburban office building that provides about 68,000 square feet of office space over an underground level of parking. Though it can be considered a traditional suburban low-rise office building, several unique features come to light:

- A composite steel structure to lessen the floor thickness and expand bay size and span;
- Varying beam elevations and angles in the driveway surface that seek to blend in with its surroundings which changes in elevation as much as 20'-0" from one side of the site to another;
- Lateral resistance from a combination of steel moment frames and a retaining wall in the underground parking area; and
- Larger moment requirements in the driveway surface from large fire engine live loads.

However, before considering the more complex sides of this building's structure, a simple analysis of the current Structural Concepts and Conditions proves to establish the current effectiveness of its structural design.

Key findings related to Structural Design considerations include:

- A design condition analysis shows that loads are mostly driven by the International Building Code of 2003 and its associated standards;
- Calculated loadings show that the 1.2D + 1.6L load combination controls for gravity loadings, that loads are especially large in the driveway area, and that the 1.2D + 1.6W + 0.5L load combination will control for lateral loadings;
- A model of the composite steel structure in RAMSteel reveals that the beams, girders, and columns are more than capable of resisting gravity loads, and that lateral loads may play a larger role;
- A portal frame analysis of a key moment frame to assess column and beam capacity to resist lateral loads in combination with gravity loads determined on the RAMSteel model shows that additional considerations for lateral load distribution needs to be considered for an appropriate lateral analysis; and
- Various detailed hand-calculations to assess beam, girder, and column ability to resist gravity loads at locations where irregularities interrupt the standard steel grid superstructure show that all irregularities are adjusted appropriately.

BASIC BUILDING STRUCTURE [*see Floorplans Appendix A*]

The Signal Hill Professional Center, designed to be a professional addition to the Manassas Town Center in Northern Virginia, is a 68,000 square foot, 4 story office building. The first floor features a pre-designed architectural and structural layout for a bank, while the upper three floors are more flexible, with open floor plans and expandable MEP and electrical systems. A key feature would be the one floor of underground parking, which extends beyond the 10,870 square-foot basic office building footprint to create a total footprint of 21,300 square feet. The interaction of these two footprints is shown in Figure 1. Since the height restrictions of nearby Washington, DC do not play as large a role in Northern Virginia, the structure of this building is mostly composite steel with prefabricated precast architectural wall panels.

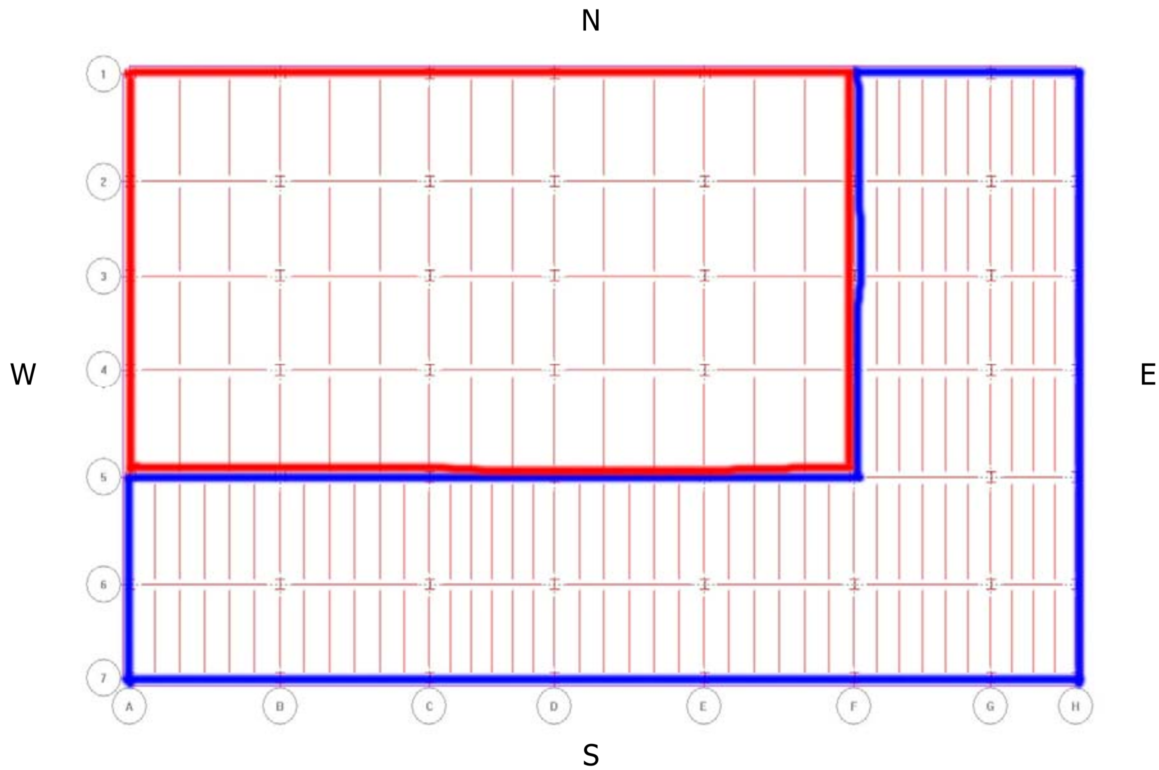


Figure 1. Basic Layout of First Floor; Office Building = Red, Driveway = Blue

Codes and Standards Used in Building Design. Designed within the last year, this building uses standard building codes and standards for its area:

- Virginia Uniform Statewide Building Code
- IBC 2000/2003
- AISC Specification, Third Edition, LRFD Methods for Steel Design
- ASCE-07/02
- ACI 318, Building Code Requirements for Reinforced Concrete
- AWS Standards for welded connections
- American Iron and Steel Institute (AISI) Standards for connection designs

Gravity Load Structural System. With the exception of the roof structure, the building uses primarily composite steel, which can support larger loads with smaller, shallower beams.

The main office structure is divided into steel grids that vary in bay size from 25'-0" x 17'-6" to 30'-0" x 20'-0". Featuring 3.5" thick, 4000 psi lightweight concrete floor slabs on a 3" deck designed using United Steel Deck specifications, these bays are supported by W10 composite beams, with two beams per bay, which are in turn supported by girders that range from W16 to W24. At junctures with features such as Stairwells, Elevators, and HVAC shafts, beams are rearranged to accommodate the openings.

With generally smaller loads on the roof surface, and a similar beam and girder layout non-composite beams are used, employing infill beams to support added dead loads from air handling units.

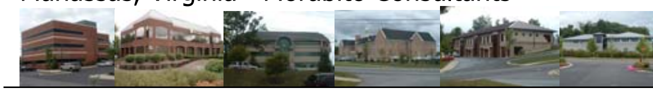
Since the driveway slab must be stronger to support fire engine loads, it features 4.5" thick, 4000 psi normal weight concrete floor slabs on a 2" deck designed using United Steel Deck specifications. Spanning bays ranging from 17'-0" x 17'-6" to 20'-0" x 27'-4", this slab rests on W10 composite beams spaced closer together than in the office building, resting on W16 to W24 girders.

Connections. One key unique structural feature in this building would be the driveway surface, which undulates to match the natural topography of the site that fluctuates as much as 20'-0" from one corner to another. Since driveway loads from every beam and girder are designed to transfer to the structure at the same elevation as the first floor, a system of coped flanges and welded W6 hangers is used. To accommodate large copings and varying connection heights, certain infill beams in the driveway and girders along the edge of the office building are upsized.

Lateral Load Structural System. Two moment frames and a shearwall provide lateral resistance for the Signal Hill Professional Center.

The two moment frames are located at the exterior east and west sides of the office building. Featuring moment connections, these walls extend for all four bays on the east and west side, and for all corner bays on the north and south side.

Figure 2. Layout of Moment Frames.



The moment frame to the west connects to a shear wall in the basement; therefore, member sizes are much larger to compensate in the east side moment frame. Columns in the west moment frame connect to the foundation via 24"x24" piers with 3000 psi concrete and eight #10 dowels attached to a base plate; the foundation is then upsized to a thickness of 18" to help resist shear.

Basement Structural System. Basic considerations for the basement design include supporting structural loads on an assumed 5000 psf soil bearing capacity and resisting lateral loads transferred from the west moment frame.

All concrete in the basement walls, slab, parking deck topping, and retaining walls is 4000 psi strength, and 3000 psi concrete is used in footings and piers connecting to the moment frame. Control joints, specified to be saw-cut 1 1/4" deep, are required every 20'-0".

Footings are generally 12" deep and extend 6" beyond wall edges, and feature 3000 psi concrete. Under retaining walls, footings range from 14" to 18" deep and 4'-0" to 9'-0" wide. Resting on footings is a 5" slab-on-grade with 6x6 – W2.9/W2.9 welded wire fabric over a 4" porous fill. Columns rest on footings that extend 3'-0" under the slab-on-grade with varying widths based on column load.

Architectural Precast Panels. These panels present a perimeter load onto floor beams, and more importantly, must be designed for wind and suction loads. These designs are the responsibility of the manufacturer; shop drawings and panel schedules must be provided to the designing engineer for approval.

Additional Considerations. Vertical shafts for stairwells and elevator shafts are generally masonry construction or shaft wall. Stairwells and elevator shafts extending form the parking level to the fourth floor feature 12" CMU with 1800 psi compressive strength, designed to support the stair structure. The additional stairwell and corridor walls feature Light Gage Metal Framing, with two layers of GWB around the stairwell to provide a two-hour fire rating.

LOADING [*see Calculations Appendix B*]

Loading was determined primarily based upon construction specifications. Live loads were confirmed with the IBC 2003 and dead weights of materials were confirmed through ASCE-07. The Wind and Seismic loads were determined using the Simplified Wind Load Procedure and the Equivalent Lateral Force Procedure, respectively, from the IBC 2003.

Office Building:
100 psf LL [IBC 2003]
60 psf DL from 3" deck with additional 3.5" lightweight concrete slab [from USD catalog]
10 psf DL from additional finishes and MEP [ASCE-07]

Roof:
2.5 psf DL from 2" deck [USD catalog]
7.5 psf DL from additional finishes and roof membrane [ASCE-07]
30 psf LL from snow, from IBC 2003 [Pf = 0.7CeCtIPg]
Pg = 30 psf (No. VA)
I = 1.0 (office building)
Ce = 1.0 (site class B)
Ct = 1.0 (heated building)
30 psf Uplift LL [specs]

Driveway/Parking:
250 psf LL (fire engine loading)
93 psf DL from 2" deck with additional 4.5" normal weight concrete slab and additional 4" asphalt topping [USD catalog, ASCE-07]
30 psf snow load [see Roof Load calculations]

Exterior Walls:
440 plf DL assuming 13'-4" height, 2" precast concrete on standard light-gage metal framed wall [ASCE-07]
220 plf DL from cornice, assuming half-height of normal walls

Stairwell Shaft Walls:
160 plf DL per floor from standard light-gage wall with two layers of GWB for 2-hour fire rating [ASCE-07]

Rooftop AHU:
10640 lb (from mechanical drawings)

Bank Vault:
55000 lb (from designer)

Wind Loads: Since the building is less than 60'-0" tall, the simplified wind load analysis [Iw = PgIλ] is acceptable per the IBC 2003, with the following loadings:

Zone	Lateral Pressure (psf)
A	**15.23**
B	-8.0
C	10.2
D	-4.8
E	-18.4
F	-10.5
G	-12.8
H	-8.1

Table 1. Wind Pressures on Various Building Surfaces (simplified method)

Of key importance here would be the 15.23 psf pressure on vertical (wall) surfaces. This assumes a mean roof height of 53'-0", a basic wind speed of 90 mph (northern Virginia), a Site Class B (urban/suburban location), and an Importance Factor of 1.0.

In addition, the loading on components and cladding, which would be key for designing connections to the precast exterior wall system, would be 17.37 / -18.80 psf on the wall surface and 17.37 / -23.21 psf on the corner wall surface.

Seismic: This analysis employs the Equivalent Lateral Force Procedure per the IBC 2003.

Using the values given in the Specifications and the IBC 2003:
Seismic Use Group I
Importance Factor = 1.0
Site Class "D"
Sds = 0.186 (from specifications)
Sd1 = 0.065 (from specifications)
R = 3.0 (Structural steel system not specifically designed for seismic resistance)
Ta = 0.60 = 0.028(building height)$^{0.80}$
Cs = Sds / (R/I) = 0.062 [largest, most critical]
W = weight of structure (total DL): Roof: 629 k
Floors 2-4: 1064 k
V (base shear) = CsW = 170 k [from specs]

RAM STEEL MODEL (GRAVITY LOAD ANALYSIS)

For a general analysis of the entire building structure, a simplified model of the main structural system was created in RAMSteel. This model only included the basic bays for each floor and assumed that all structural columns, though spliced at the same levels, continued down to the underground parking level. Therefore, sections around stairwells, under special air handling unit loads, and under irregular loading were later analyzed specifically. In addition, the slanted beams on the parking deck, designed to match the site's natural topography, were simplified into a flat surface.

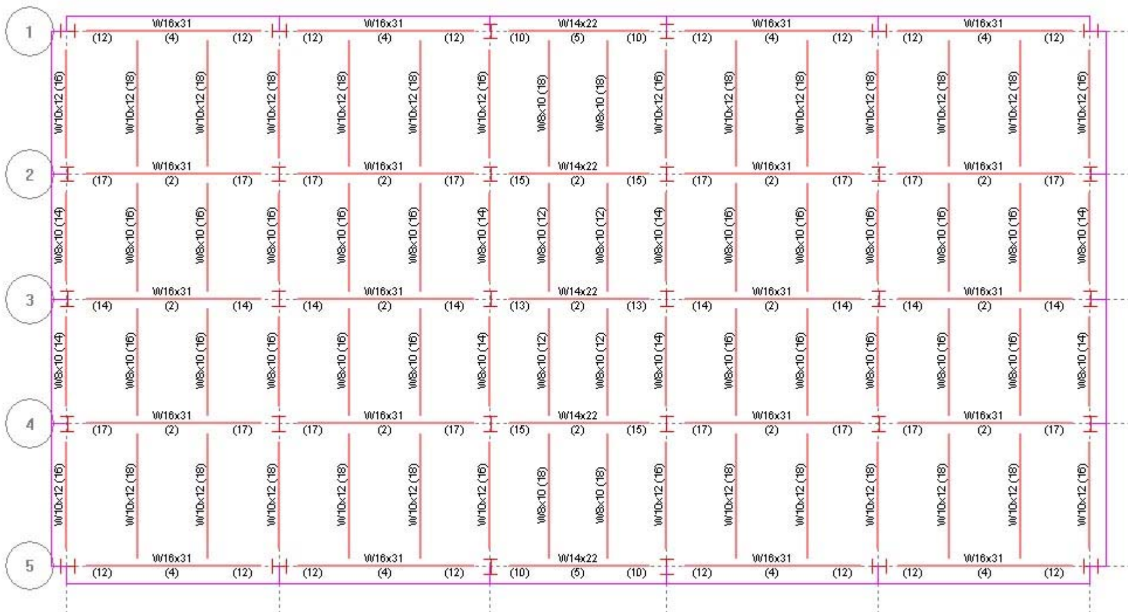


Figure 3. Basic Gravity Load Design of Standard Office Floor using RAMSteel.

General Results, Office Building. Using gravity loads as calculated above, a model of the typical floors for the office building were created, and the results coincide with the given plans:

- Infill beams were generally one step smaller than on the construction documents; the RAM designer sized W10x12 beams whereas W10x15s are found on the plan. This could simply be the designer's way of using a greater factor of safety.
- Girders were slightly smaller as well; the most critically smaller girders were along column lines 2 and 4, with W16x31s as calculated by RAM and W18x35s as shown on the plan. Larger depths would probably be needed to connect to the exterior walls along column lines A and F, which are used to resist lateral loads.
- Beams along the perimeter of the building, where loads from the precast exterior wall system would play the largest role, were much smaller as designed by RAM: W16x31 vs. W21x44 on the north and south sides, and W10x12 vs. W18x35 on the east and west sides. Considering that moment frames along the perimeter are the lateral force resisting system, it is not surprising that these beams would be upsized much more for lateral stability.

- Columns were generally also slightly smaller as designed by RAM; using a typical column supporting four bays at B-4, the RAM designer sized W10x49 columns that taper to W10x33 from the second floor upward while the plans feature W10x49 columns that taper to W10x39.

Figure 4. Basic Gravity Load Design of Parking/Driveway Surface using RAMSteel.

General Results, Parking Area. Combining the worst-case live load condition with a fire-engine load and a snow load, and simplifying the parking deck into one flat surface, the results as found on RAM also coincide closely with those found on the plans:

- Much like in the office building, designed infill beams were slightly smaller than those on the plans: the RAM designer sized W8x10, W10x12, and W12x14 infill beams while W10x15s and W10x19s are found on the plan. The shift to larger beams is most likely due to the need for larger cross sectional areas and flange thicknesses for the intricate connections between the parking deck, at varying elevations, with the first floor of the office building.
- Girders as designed by RAM were much the same as those found on the plans; the RAM designer sized a range of girders between W16x26 and W24x55 while girders ranging from W16x26 to W24x76 are found on the plans.
- Columns completely under the parking deck were also smaller on RAM: W10x39 vs. W10x49 per plan. W10x49s were most likely used to keep consistent with the W10x49 columns used at the basement level to support the office building.

LATERAL LOAD ANALYSIS [*see Calculations Appendix C*]

Wind and Seismic loads were analyzed over the tributary area of the wall to see which one controlled in this situation, and a portal analysis was carried out to determine loading on the members in the two moment frames on the east and west sides of the office building.

Portal Frame Analysis. Assuming that lateral forces were in the North-South direction, and using appropriate tributary areas along the north and south elevations of the structure, the following loading situation was established:

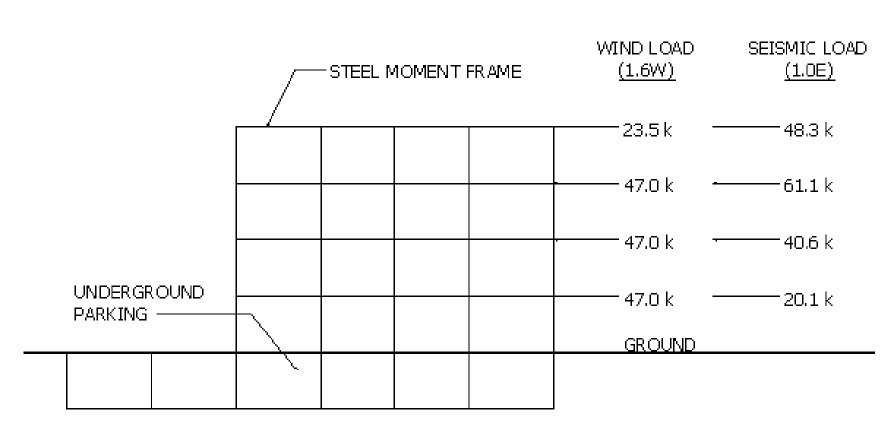


Figure 5. Lateral Point Loads based upon Tributary Area.

It should be noted that though the 1.2D + 1.6W + 0.5L loading situation will control, absolute maximum loads at each level are used for this analysis. Since the building is primarily braced by two moment frames on the east and west sides, it was assumed that each frame resists half the lateral load.

Assuming that all beams and columns have a moment of zero at midspan, the Portal Method was then used to determine maximum moments and axial loadings in each member from lateral forces. Column loads were distributed based upon relative moments of inertia, taking into account differing orientations. Key conditions were:

- A W21x44 beam on the second floor adjoining the building corner; and
- The supporting W12x79 column.

1. W21x44 Beam. Using the simplification that for the end moment:

$$M \text{ (negative)} = (1/24)wL^2$$

The 287 ft-k moment from wind loading was merely added to achieve a design condition of 364.7 ft-k negative moment and 29.5 k axial loading. Per the LRFD equation H1-1a, it was found that the moment capacity of the beam was exceeded. Most likely, this is due to inaccurate design moments from an oversimplification. Therefore, in a further analysis, a full moment distribution taking into account smaller loads and an uneven distribution of lateral forces between the shear walls and moment frame would be needed to determine more realistic design conditions.

2. Supporting W12x79 Column. Using moment and axial loads from the portal frame analysis, combined with a typical vertical gravity load from the RAM model, the column design conditions became 202 ft-k maximum moment and 306 k maximum axial load. Per the LRFD equation H1-1a, it was determined that the W12x79 column was acceptable for this condition.

Components and Cladding Analysis. Due to the unique nature of the precast exterior wall system, connections between the steel frame and the wall panels would need to be specially designed. Using the simplified wind pressure procedure ($w = PgI\lambda$) and the equivalent lateral force procedure ($w = 0.4SdsIwc$)

Wind	17.4 / -23.2 psf
Seismic	2.64 psf

Therefore, wind loads will control in this situation.

SPOT CHECKING KEY SECTIONS [*see Calculations Appendix D*]

Since the gravity load analysis from RAMSteel was a very simplified model, hand-calculations were performed at critical areas where a slight deviation from the regular steel grid was necessary. In turn, these calculations, which feature beams and girders under heightened loads, should confirm the results of the RAMSteel model.

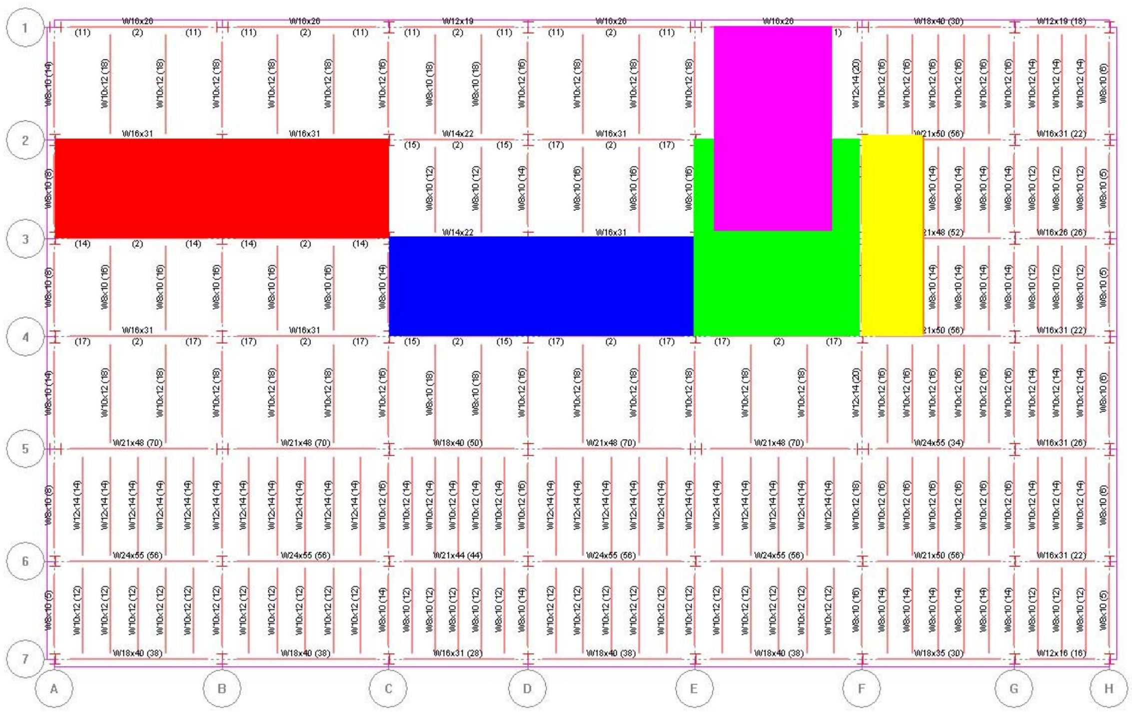


Figure 6. Locations of Spot-Checks.

Sections spot checked are:

- Beams and Girders supporting air handling unit loads on the roof;
- Beams and Girders surrounding elevator shafts;
- Beams and Girders surrounding and supporting stairwell and shaft walls;
- Infill Beams underneath heightened sidewalk loads; and
- Beams and Girders underneath the bank vault loads.

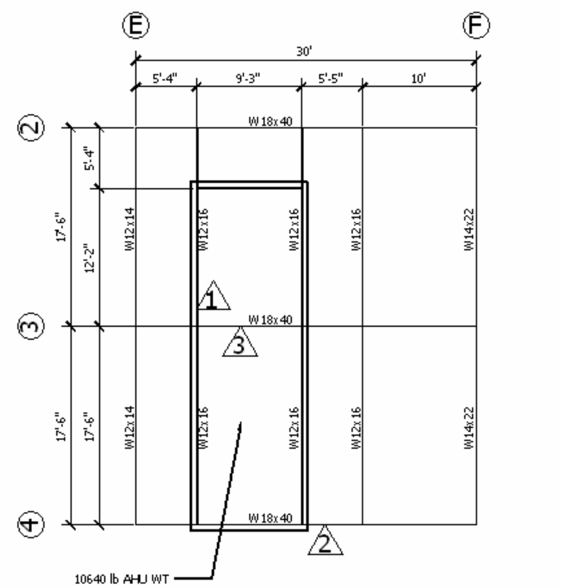


Figure 7. Location of Infill Beams and Air Handling Unit.

Testing Beams and Girders Under AHU Loads. The simplified RAM model did not take into account air handling unit loads in addition to standard live and dead loads; therefore, the existing design, using reactions derived from the RAM model will be used to test three key beams supporting the AHU:

- The W12x16 infill beams directly under each edge;
- The W18x40 girders supporting the infill beams on one side and standard loads from the RAM model on the other; and
- The W18x40 girder centrally under the middle of the AHU.

For this analysis, only the AHU on the right is considered; since it does not rest on any north-south girders and therefore relies entirely on infill beams for support, it is the more critical case. In addition, only beams 1, 2 and 4 were considered for strength since they had the maximum loading of all infill beams and girders involved.

For the HVAC roof loads spot-check, the bay concerned would be column lines E-F, and 2-4. At this point, a dead load of 10,640 lbs from the air handling unit would need to be distributed to infill beams that would then rest on the standard surrounding girders. For this analysis, it is assumed that:

- The load from the AHU is equally and completely divided between the left and right supporting beams;
- Though there is actually an opening below the AHU, a continuous slab will be assumed with typical live and dead loads; and

1. Infill Beams Framing AHU. Once the load of the AHU and the standard roof dead and live loads were distributed over the whole length of one 17’-6” long W12x16, it was determined that:

- 25.0 ft-k = $M_u < \Phi M_p$ = 51.3 ft-k, and
- 5.72 k = $V_u < \Phi V_n$ = 71.3 k.

2. Girder Supporting the AHU and a Regular Bay. Reactions from the AHU infill beams (1), an additional adjoining beam, and from a standard bay from the RAM model were loaded onto the W18x40 girder's 30'-0" length, determining that:

- 143.1 ft-k = Mu <ΦMp = 294 ft-k, and
- 15.4 k = Vu < ΦVn = 152 k.

3. Girder Centrally Under AHU Unit. Using the conservative estimate of four equal loadings from the tested infill beam (1), two from each side, in addition to loading from the additional beam (3), determined that:

- 156.2 ft-k = Mu <ΦMp = 294 ft-k, and
- 18.0 k = Vu < ΦVn = 152 k.

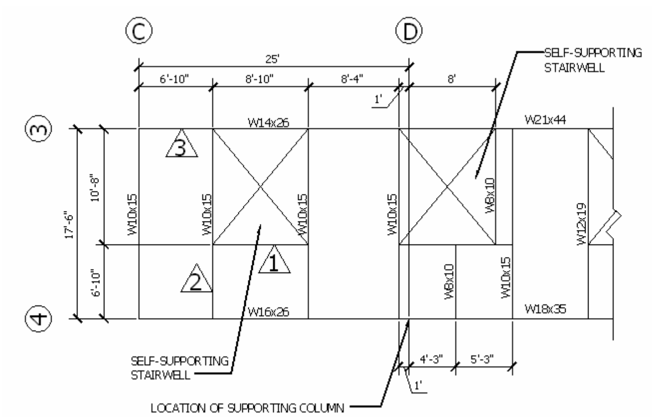


Figure 8. Location of Stairwell and Elevator Shaft.

Testing Beams and Girders Around Elevator Shafts. A concerning variance from the RAM model was the area around Columns D3 and D4, where the existence of a CMU elevator shaft wall forces a beam to connect to a girder rather than directly to the column. Therefore, the loads transferred from this beam may increase the total flexural and shear load on the girder. Using results from the RAM model and normal first floor loading, the following were analyzed:

- The beam immediately adjoining the elevator shaft; and
- The girder that supports this beam and another offset 1'-0" from the column.

For this analysis, the most critical beam with the largest tributary width around the elevator shaft was considered. Since this beam has unequal reactions at each side, due to the lack of loading from the elevator shaft, the girder on the south side, which must support the heaviest reaction in combination with the larger adjacent 20'-0" span was chosen for consideration.

Since these beams were most critical, they were determined to verify the design of all beams adjacent to the elevator shafts around column lines C-F and 3-4.

1. Beam Adjacent to Elevator Shaft Wall. Using the largest tributary width of 4'-2" adjacent to the elevator shaft and 8'-7" when supporting slabs on either side, it was determined that for the composite W10x15:
   - 55.0 ft-k = Mu <ΦMp = 143 ft-k, and
   - 15.1 k = Vu <ΦVn = 62.0 k.

2. Girder Supporting the Infill Beam (1) and a Regular Bay. Using the conservative assumption that the reaction from beam (1) is an approximation for the loading from all connection beams, and using reactions from the south bay given by the RAM model, it was determined that for the composite W16x26 girder:
   - 300.0 ft-k = Mu <ΦMp = 350.0 ft-k, and
   - 48.5 k = Vu < ΦVn = 106.0 k.

3. Girder Supporting the Infill Beam (1) and a Smaller Bay. Using the conservative assumption that the reaction from beam (1) is an approximation for the loading from all connection beams, and using reactions from the north bay given by the RAM model, it was determined that for the composite W14x26 girder:
   - 242.0 ft-k = Mu <ΦMp = 312.0 ft-k, and
   - 38.4 k = Vu < ΦVn = 95.7 k.

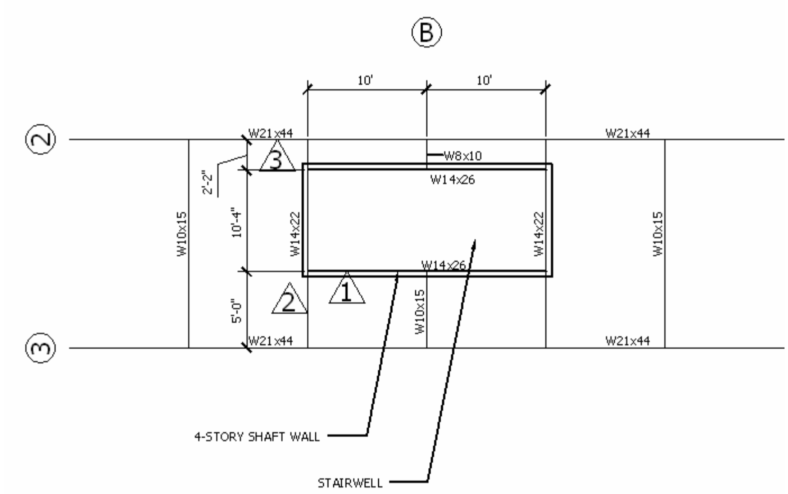


Figure 9. Location of Shaft Wall Stairwell and Loads.

Testing Beams and Girders Around Stairwells. The layout of the building varies again by the stairwell in the northwest corner, between column lines A-C and 2-3. Since this stairwell does not continue down to the underground parking and therefore is not supported by CMU walls, the stair weight is therefore transferred into the shaft wall which is then supported on each level. Using data related to stair weight and the RAM model, the following situations were analyzed:

- The beam immediately adjacent to the stairwell, which will have to carry the weight of the stairs for one floor and the shaft wall;
- The girder that supports this beam and normal floor loads; and
- The girder that in turn must support both stairwell loads and loads from a normal bay.

The most critical beam considered in this analysis was on the south side of the stairwell, which must carry additional normal floor load, and adjoining girders were chosen. Stairwell loads assume a normal live load of 100 psf and a somewhat reduced dead load of 45 psf to reflect a probable channel and smaller concrete deck construction. Though the weight of the stairs is not always supported at the same elevation as the floor, it is assumed that roughly one floor's loading from a stairwell makes it to each floor.

1. Beam Adjacent to Stair Well Wall. Using the beam adjacent to the south wall, which features a greater critical floor loading, it was determined that for the composite W14x26:

- 94.0 ft-k = $M_u < \Phi M_p$ = 312.0 ft-k, and
- 15.9 k = $V_u < \Phi V_n$ = 106.0 k.

2. Girder Supporting Beam (1). Using the conservative assumption that the reaction from beam (1) is an approximation for the loading from each beam adjacent to the stairwell, with normal floor loading, it was determined that for the composite W14x22:

- 117.2 ft-k = $M_u < \Phi M_p$ = 261.0 ft-k, and
- 32.5 k = $V_u < \Phi V_n$ = 85.1 k.

3. Girder Supporting the Girder (2) and a Normal Bay. Using the largest reaction from Girder (2) and reactions from typical beams found in the RAM model, it was determined that for the composite W21x44 girder:

- 534.0 ft-k = $M_u < \Phi M_p$ = 661.0 ft-k, and
- 53.4 k = $V_u < \Phi V_n$ = 196 k.

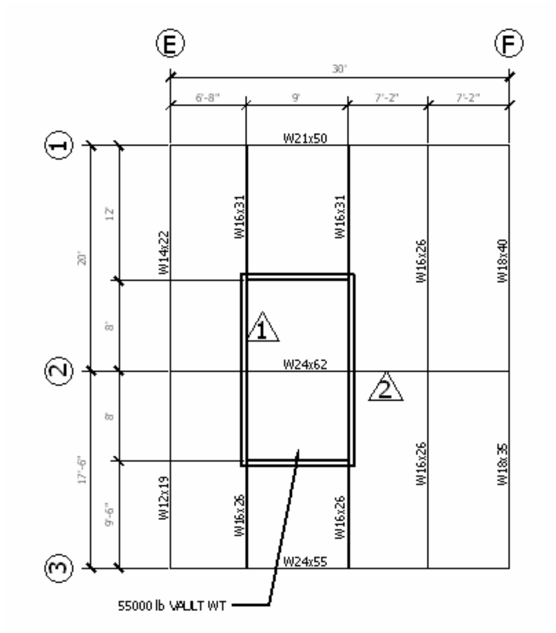


Figure 10. Location of Bank Vault and Infill Beams.

Testing Beams and Girders Under Vault Loads. Much like the AHU's on the roof, the added 55000 lb bank vault on the first floor was not included in the RAM model. Therefore, sections tested specifically under this load will include:

- The W16x31 infill beams directly under the vault edge; and
- The W24x62 girders directly underneath the vault supporting the infill beams and normal floor loads.

For the vault load spot-check, the bay concerned would be column lines E-F, and 1-3. At this point, a dead load of 55,000 lbs from the vault would need to be distributed to infill beams that would then rest on the standard surrounding girders. For this analysis, it is assumed that:

- The load from the vault is equally and completely divided between the left and right supporting beams; and
- Floor slab and live load will be considered continuous even inside the area of the vault.

1. Infill Beams Framing Vault. Once the load of the AHU and the standard roof dead and live loads were distributed over the whole length of one 17'-6" long W12x16, it was determined that:

- 127.2 ft-k = Mu <ΦMp = 383 ft-k, and
- 31.8 k = Vu < ΦVn = 118 k.

2. Girder Supporting the Infill Beam and Normal Floor Loads. Reactions from the infill beams (1), and from normal beams supporting floor loads were placed at concentrated points along the girder's length, determining that:

- 743.9 ft-k = Mu <ΦMp = 919 ft-k, and
- 84 k = Vu < ΦVn = 275 k.

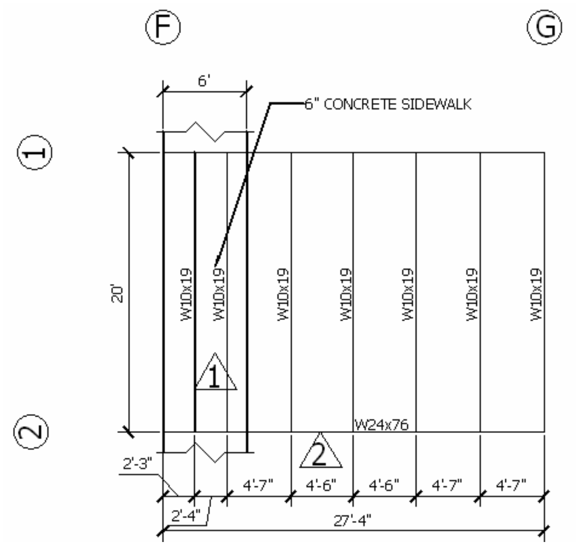


Figure 11. Location of Heightened Sidewalk Loads.

Testing Beams and Girders Under Sidewalk Loads. When calculating beam sizes for the standard 93 psf dead load, 250 psf live load, and 30 psf snow load on the driveway, the RAM model did not take into account the 6'-0" wide, 6" thick concrete sidewalk that adjoins the building. Therefore, the following situations were analyzed:

- The extra W10x19 beam directly under the sidewalk; and
- The W24x76 girders supporting the added 75 psf sidewalk dead load in addition to normal loads.

1. Extra Beam Under Sidewalk Load. Distributing the heightened loading over the W10x19 composite beam, it was determined that:

- 69.4 ft-k = Mu <ΦMp = 201 ft-k, and
- 13.9 k = Vu < ΦVn = 68.8 k.

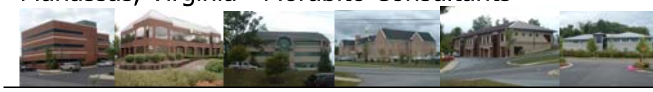
2. Girder Supporting Both Sidewalk and Normal Loads. Reactions from beam (1) and distributed normal driveway loads were placed on the 27'-4" length of the W24x76, determining that:

- 958 ft-k = Mu <ΦMp = 1230 ft-k, and
- 68.8 k = Vu < ΦVn = 284 k.

CONCLUSIONS

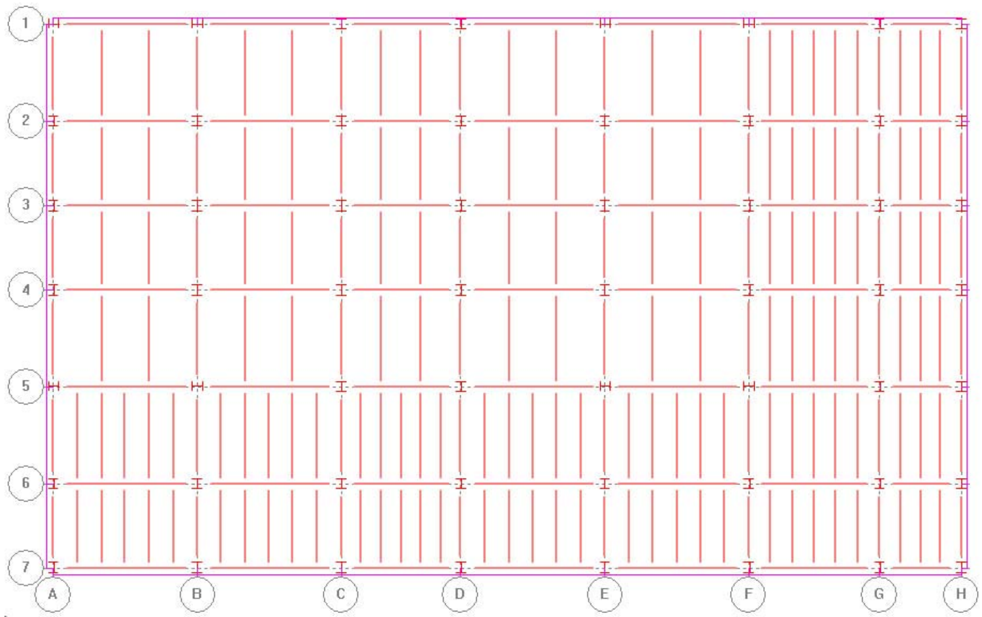
After analyzing code and standard requirements, loading patterns, gravity load resistance, lateral load resistance, and irregularities to the building structure, it was shown that:

- A basic, simplified analysis of gravity loads using a RAMSteel model revealed that the current composite beam and girder designs are more than satisfactory. Larger sizes are most likely due to the influence of lateral forces on two moment frames and added loads including air handling units, stairwells, and a bank vault.
- At first glance, the moment frames should be able to resist the applied wind loads. However, further considerations should establish the true gravity and lateral loadings through the 1.2D + 1.6W + 0.5L load combination as well as properly distribute lateral forces between the two moment frames and the supporting shear wall in the foundation.

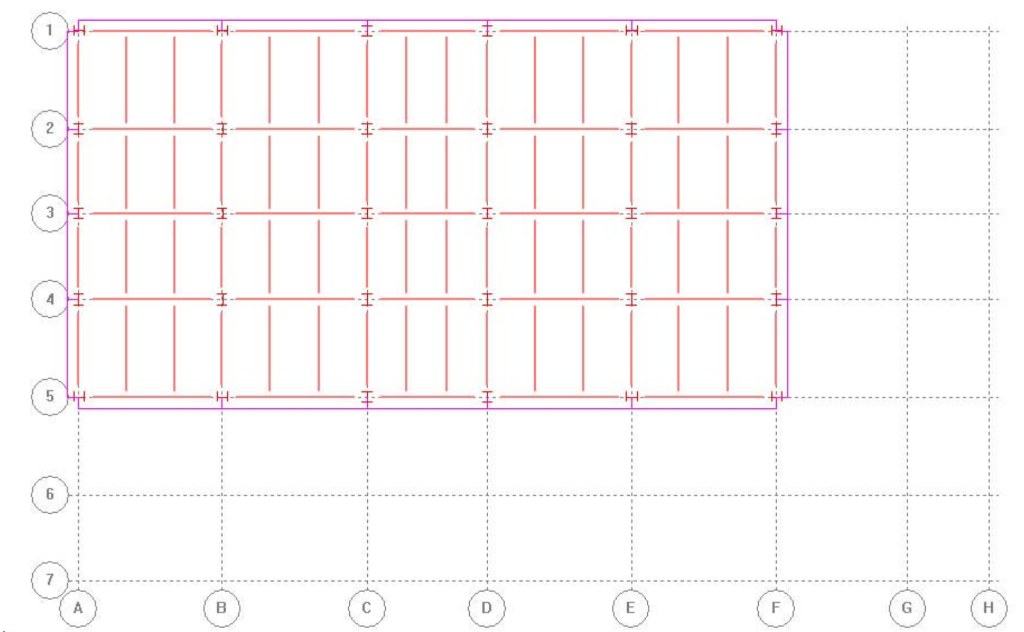


APPENDIX A: TYPICAL FLOOR PLANS

FIRST FLOOR PLAN, INCLUDING OFFICE AND DRIVEWAY



FLOORS 2-4, OFFICE FLOOR PLAN



APPENDIX B: CALCULATION OF SNOW, WIND, AND SEISMIC LOADS

SNOW LOADS

SNOW LOAD FROM [redacted] IBC 2003

Pg = 30 PSF (MANASSAS, VA)
TERRAIN CATEGORY "C" (SUBURBAN)
↳ PARTIALLY EXPOSED ROOF — Ce = 1.0 [1608.3.1]
THERMAL - NORMAL STRUCTURE — Ct = 1.0 [1608.3.2]
IMPORTANCE : NORMAL STRUCTURE — I = 1.0

SNOW LOAD = 30(1.0)(1.0)(1.0) = 30 PSF

WIND LOADS

WIND LOAD FROM IBC 2003

REGULAR SHAPED BUILDING, NORMAL SITE CONDITIONS
AVG. ROOF HEIGHT = 53'-0 < 60'-0 → SIMPLIFIED PROCEDURE OK.

V = 90 MPH [1609]
I = 1.0
λ = (EXPOSURE "B") = 1.19

ZONE	PRESSURE (ADJUSTED) PSF	
A	15.2	← PRIMARY CONCERN, WALL LOADING
B	-8.0	
C	10.1	
D	-4.8	
E	-18.4	
F	-10.5	
G	-12.7	
H	-8.1	

COMPONENTS AND CLADDING

ZONE	PRESSURE (ADJUSTED) PSF	
WALL	17.4	
	-18.8	
CORNER	17.4	
	-23.2	← MOST CRITICAL FOR PRECAST CONCRETE PANEL CONNECTIONS.

FLR	TRIBUTARY AREA		WIND LOAD (ADJUSTED ×1.6)
R	(145)(13.333)(0.5)	= 967 SQ. FT.	23.5
2-4	(145)(13.333)	= 1934 SQ FT	47.0

APPENDIX B, CONT'D

SEISMIC LOADS

SEISMIC LOADING. EQUIVALENT LATERAL FORCE PROCEDURE IBC 2003

SEISMIC USE GROUP I

$I = 1.0$

$S_{DS} = 0.186$

$S_{D1} = 0.065$

SITE CLASS "D"

$R = 3.0$

$V = C_s W$

STRUCTURE WT: DL + 10PSF FOR PARTITIONS
10875 SQ. FT. ROUGH FLOOR AREA, 440FT PERIMETER

ROOF: 40 PSF (10875) + 440 PLF (440) = 629 K
FLOORS 2-4 80 PSF (10875) + 440 PLF (440) = 1064 K

Σ 4883 K

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.186}{3} = 0.062 \leftarrow$$

$$\frac{S_{D1}}{T(R/I)} = \frac{0.065}{0.67(3)} = 0.032$$

$$0.044 S_{DS} I = 0.044(0.186) = 0.008$$

$T = C_t h_n^x$ $C_t = 0.028$, $x = 0.80$ [MOMENT RESISTING STEEL]
$h_N = 53'\text{-}0$

$T = 0.67s.$

$V = C_s W = 0.062(4883) = 302^K$ BASE SHEAR

$F_x = C_{vx} V$, USING 170^K BASE SHEAR IN SPECS.

$$C_{vx} = \frac{w_x h_x}{\Sigma w_x h_x}$$

FLOOR	$w_x h_x$
R	33337
4	42206
3	28019
2	13832
Σ	117394

$F_R = \cancel{85} \; 48.3^K$

$F_4 = 61.1^K$

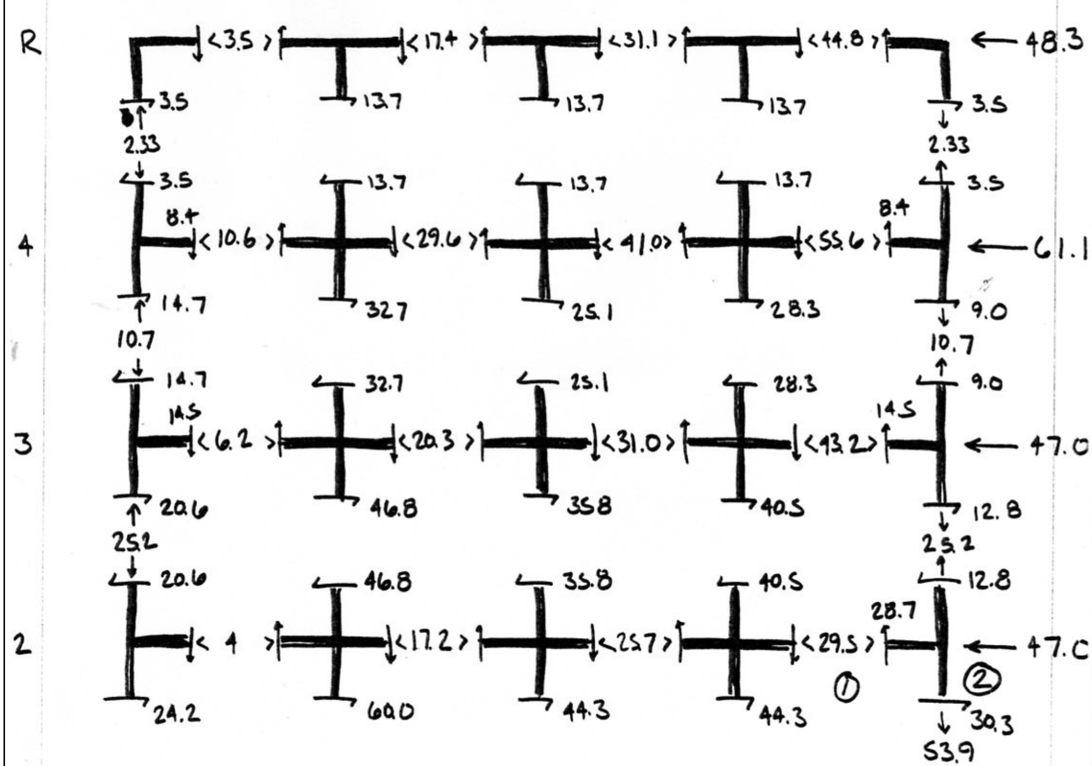
$F_3 = 40.6^K$

$F_2 = 20.1^K$

APPENDIX C. PORTAL FRAME ANALYSIS

PORTAL FRAME
ANALYSIS

USING LARGEST LATERAL LOAD PER FLOOR (1.6W or 1.0E)



(COLUMNS)

RELATIVE STIFFNESSES			FACTOR
R	W12x40	44.1	0.0734
	W10x35	171	0.284
3-4	W12x65	174	0.132
	W10x68	394	0.299
	W10x54	303	0.229
	W10x60	341	0.259
	W12x58	107	0.082
2	W12x79	216	0.119
	W10x88	534	0.295
	W10x68	394	0.218
	W12x96	270	0.149

APPENDIX C, CONT'D

PORTAL FRAME MEMBER ANALYSIS

PORTAL FRAME ANALYSIS,
CONT'D.

ANALYZING BEAM 1

LOADING FROM WIND: $(28.7)(10) = 287$ FT-K MOMENT
29.5 K AXIAL

GRAVITY LOADING: ASSUMING FIXED END ⊖ MOMENT $\frac{1}{12}wL^2 = -77.67$ K-FT
MIDSPAN ⊕ MOMENT $\frac{1}{24}wL^2 = +38.83$ K-FT

DESIGN. $M_U = -364.7$ FT-K
AXIAL $= 29.5$ K

W21x44 (COMPOSITE DESIGN PLAYS NO ROLE)

$\phi P_N = 0.85 F_y A = 0.85(50)(13.0) = 552.5^K$

$\frac{P_U}{\phi P_N} = 0.05 < 0.2$

$\frac{P_U}{2\phi P_N} + \left(\frac{M_{UX}}{\phi M_{NX}}\right) \leq 0$

$\phi M_{NX} = 359$ FT-K < 364.7 FT-K $\longrightarrow$ ∴ MOMENT DISTRIBUTION IS NECESSARY.

ANALYZING COLUMN 2

LOADING FROM WIND: $M = 30.3(6.6) = 202$ FT-K
AXIAL $= 53.9$ K

GRAVITY LOADING (FROM BEAM): $M = 0$
AXIAL $= 252.09$ K

$\phi P_N = 0.85(50)(23.2) = 986$ K

W12x79

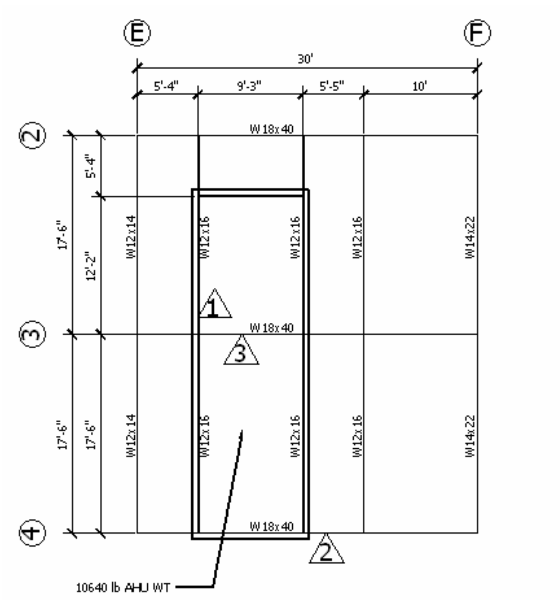
$\frac{P_U}{\phi P_N} = 0.31 > 0.2$

$\frac{P_U}{\phi P_N} + \frac{8}{9}\left(\frac{M_{UX}}{\phi M_{NX}}\right) \leq 0$

$\frac{306}{986} + \frac{8}{9}\left(\frac{202}{446}\right) = 0.713 < 0$ ✓OK

APPENDIX D: SPOT CHECK CALCULATIONS

TESTING BEAMS AND GIRDERS UNDER AHU LOADS



GIRDERS SUPPORTING
HVAC UNIT — SPOT CHECK #1

SEE IMAGE PREVIOUS PAGE

AHU DL = 10640 lb

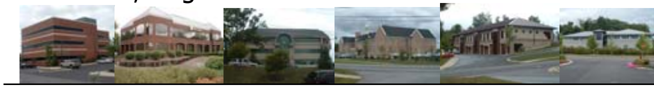
DIVIDES EVENLY BETWEEN EAST AND WEST BEAMS

① INFILL BEAMS DIRECTLY UNDER UNIT (WORST CASE)

10640 lb / 2 SIDES / 29.67' LENGTH = 179.3 PLF
ADD'L DL FROM SLAB: (10 PSF)(0.5(5.33' + 9.25' TRIB WIDTH)) = 72.9 PLF
LL = (30)(0.5(5.33' + 9.25' TRIB WIDTH)) = 218.7 PLF
* TL = 1.2D + 1.6L
= 0.653 KLF

$M_{MAX} = \frac{1}{8} wL^2 = 25$ FT-K
$V_{MAX} = \frac{1}{2} wL = 5.72^K$

W12x16 ASSUME BRACED. $\phi_b M_{px} = 51.3$ FT-K $> M_U$ ✓ OK
[LRFD CHARTS] $\phi_v V_N = 71.3^K > V_U$



APPENDIX D, CONT'D

TESTING BEAMS AND GIRDERS UNDER AHU LOADS, CONT'D

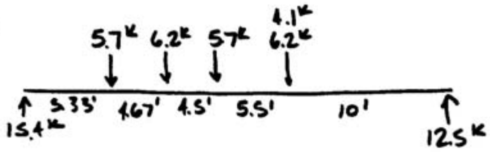
② GIRDER ON SOUTH END OF UNIT (WORST CASE)

LOADING FROM TYPICAL REACTIONS ON RIMSTEEL (FACTORED) : ~~1.14^K~~ 1.37 D @10', 20'
4.83 ~~3.00^K~~ L @ 10', 20'

LOADING FROM INFILL BEAM · 5.72^K TL @ 5'-4, 14'-7

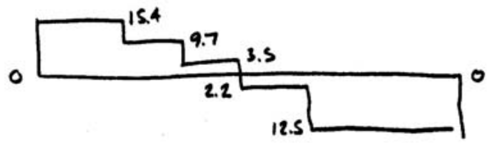
LOADING FROM BEAM ③ (TRIB · 0.5(5.5+10'))(1.2(10)+1.6(30)) · 4.1^K TL @ 20'

LOADING



V_{MAX} = 15.4^K
M_{MAX} = 143.1 FT-K

SHEAR



W18x40 ASSUME BRACED.
[LRFD CHARTS]

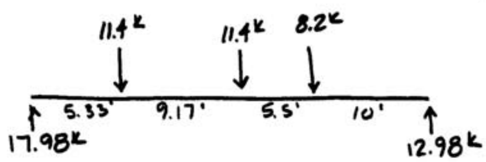
$\phi_b M_{px}$ = 294 FT-K > MU
$\phi_v V_N$ = 152 K > V_U ✓OK

HVAC UNIT CONT'D SPOT CHECK #1

③ CENTRAL GIRDER UNDER UNIT

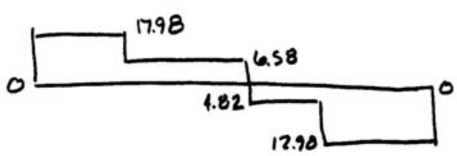
LOADING FROM INFILL ~~GIRDER~~ BEAM (WORST CASE) = 11.44^K TL @ 5'-4, 14'-7
LOADING FROM BEAM ③ (WORST CASE) = 8.2^K TL @ 20'

LOADING



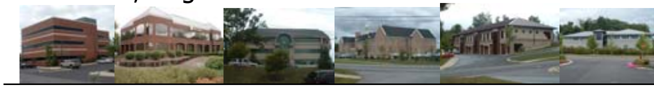
V_{MAX} = 18^K
M_{MAX} = 156.2 FT-K

SHEAR



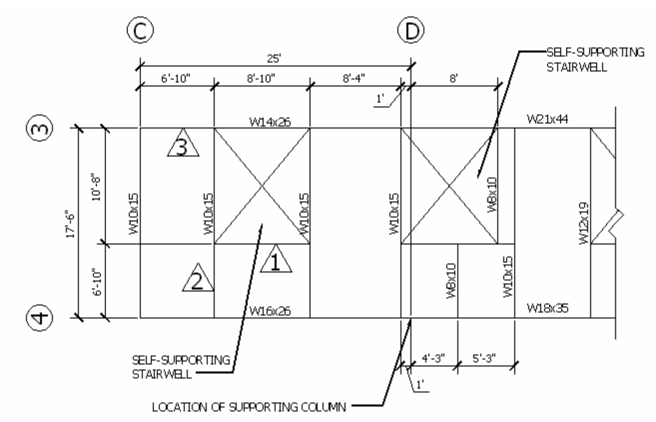
W18x40 ASSUME BRACED.
[LRFD CHARTS]

$\phi_b M_{px}$ = 294 FT-K >MU
$\phi_v V_N$ = 152^K >VU ✓OK



APPENDIX D, CONT'D.

TESTING BEAMS AND GIRDERS AROUND ELEVATOR SHAFTS



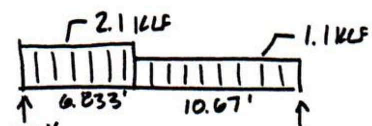
AROUND ELEVATORS   SPOT CHECK #2

SEE IMAGE PREVIOUS PAGE

① BEAM AROUND ELEVATOR SHAFT   LL = 100 PSF   DL = 70 PSF

LOADING. 0–6'10": (8.58' TRIB)(70PSF DL + 100PSF LL) · 2.1 KLF [FACTORED]
6'10–17'6: (4.167' TRIB)(70 PSF DL + 100 PSF LL) · 1.1 KLF

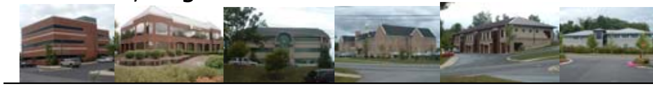
LOADING



$V_{MAX} = 15.1^K$
$M_{MAX} = 55$ FT-K

SHEAR

W10X15 COMPOSITE W/ [8 - 3/4" SHEAR STUDS]
$\Sigma Q_N = 8(\cancel{21.9}\ 19.6^K/\text{STUD}) = \cancel{175.2^K}\ 156.8^K$
ASSUME $Y1 = 0.135$ IN, $\Sigma Q_N = 167^K$
$b_{eff} = \frac{1}{8} l = 2.1875' = 26.25$ IN
$a = \frac{167}{0.85(4)(26.25)} = 1.87$ IN $\rightarrow Y2 = 6.5 - \frac{1}{2}a \approx 5.5$
[TABLE 5-14] $\phi_b M_p = 143$ FT-K $> M_U$ ✓OK
$\phi_V V_N = 152^K > V_U$



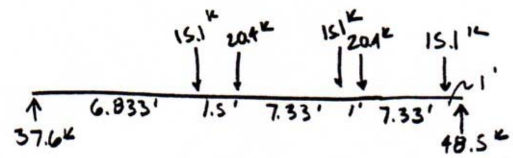
APPENDIX D, CONT'D

TESTING BEAMS AND GIRDERS AROUND ELEVATOR SHAFTS, CONT'D

② GIRDER SUPPORTING BEAMS AROUND ELEVATOR SHAFT.

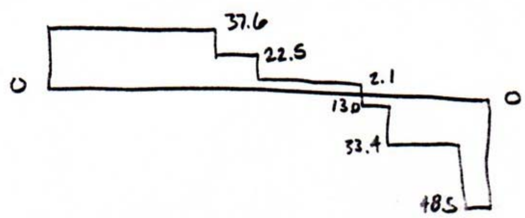
LOADING FROM TYPICAL RAMSTEEL REACTIONS : 20.4^k PL @ 8'-4, 16'-8
LOADING FROM BEAM ① : 15.1^k PL @ 6'-10, 15'-8, 24

LOADING



$V_{MAX} = 48.5^K$
$M_{MAX} = 300$ FT-K

SHEAR



AROUND ELEVATORS CONT'D — SPOT CHECK #2

② CONT'D.

W16x26 COMPOSITE W/[18 - $\frac{3}{4}$" SHEAR STUDS]

$\Sigma Q_N = 18(\cancel{21.9}\ 19.6) = \cancel{394.2^k}\ \cancel{384^k}\ 353^k$ → FULLY COMPOSITE

$b_{eff} = \frac{1}{8}(25') = 37.5"$ $Y1 = 0.086$

$a = \dfrac{\cancel{384}\ 353}{0.85(4)(37.5)} = \cancel{3.0}\ 2.76$ IN $Y2 = 6.5 - \frac{1}{2}a = 5$ IN

$\phi_b M_p = \cancel{360}\ 330$ FT-K $> M_U$ ✓OK
$\phi_v V_N = 106$ K $> V_U$

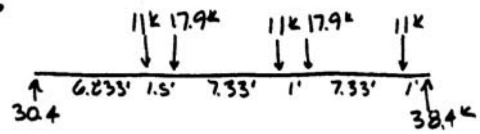
APPENDIX D, CONT'D

TESTING BEAMS AND GIRDERS AROUND ELEVATOR SHAFTS, CONT'D

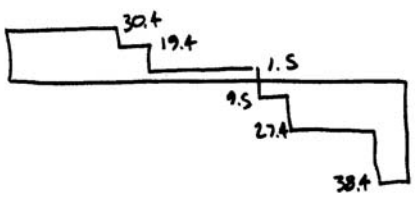
③ GIRDER SUPPORTING BEAMS AROUND ELEVATOR SHAFT.

LOADING FROM TYPICAL DIMSTEEL REACTIONS: 17.9^k TL @ 8'4, 16'8
LOADING FROM ~~GIRDER~~ BEAM① : 11^k TL @ 6'10, 15'8, 24'

LOADING



SHEAR



$V_{MAX} = 38.4^k$
$M_{MAX} = 242$ FT-K

W14x26 COMPOSITE W/ [25 · 3/4" SHEAR STUDS]
$\Sigma Q_N = 25(21.9) = 547.5^k \rightarrow 385^k \rightarrow$ FULLY COMPOSITE
$b_{eff} = \frac{1}{8}(25') = 37.5"$ Y1 = 0

$a = \frac{385}{0.85(4)(37.5)} = 3.0$ IN Y2 = 5 IN

$\phi_b M_p = 325$ FT-K > MU ✓OK
$\phi_v V_N = 95.7^k$ > Vu

APPENDIX D, CONT'D

TESTING BEAMS AND GIRDERS AROUND STAIRWELLS

FRAMING AROUND
STAIRCASE, CONT'D

SPOT CHECK # 3

② CONT'D

W14x22 COMPOSITE W/ [17 - 3/4" SHEAR STUDS]
$\Sigma Q_N = (19.6^k)(17) = 333.2^k > 325$ FULLY COMPOSITE
$b_{eff} = \frac{1}{8} l = 26.25 \text{ IN}$ $Y1 = 0$

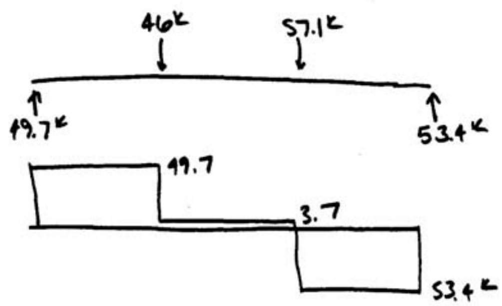
$a = \frac{325}{0.85(4)(26.25)} = 3.64 \text{ IN}$ $Y2 = 6.5 - \frac{1}{2}a = 4.7 \text{ IN}$

$\phi_b M_p = 261 \text{ FT-K}$
$\phi_v V_N = 85.1^k$ ✓OK

③ GIRDER SUPPORTING GIRDER ② AND ADJACENT BAY

LOADING FROM ② $= 325^k$ @ 20'
TYPICAL BM REACTIONS: $= 24.5^k$ @ 10', 20'
$= 21.5^k$ @ 10'

LOADING



$V_{MAX} = 53.4^k$
$M_{MAX} = 534 \text{ FT-K}$

SHEAR

W21x44 COMPOSITE W/ [35 - 3/4" SHEAR STUDS]
$\Sigma Q_N = (35)(19.6) = 686^k > 650^k$ FULLY COMPOSITE
$b_{eff} = \frac{1}{8} l = 45"$ $Y1 = 0$

$a = \frac{650}{0.85(4)(45)} = 4.25"$ $Y2 = 6.5 - \frac{1}{2}a = 4.38 \text{ IN}$

$\phi_b M_p = 661 \text{ FT-K} > M_U$
$\phi_v V_N = 196^k > V_U$ ✓OK

APPENDIX D, CONT'D

TESTING BEAMS AND GIRDERS AROUND STAIRWELLS, CONT'D

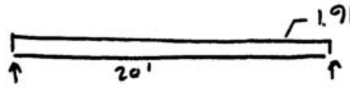
STAIRCASE SPOT CHECK # 3

SHAFTWALL WT: (12 PSF)(13.33' HIGH) = 0.160 KLF
STAIRWELL LOAD: 45 PSF DL
100 PSF LL
NORMAL FLOOR LOADS: 70 PSF DL
100 PSF LL

① BEAM ADJACENT TO STAIRWELL

APPROXIMATING DISTRIBUTED FLOOR LOAD: (2.5' TRIB)(1.2(70)+1.6(100)) = 0.610 KLF
LOADING FROM STAIRWELL: (5.167' TRIB)(1.2(45)+1.6(100)) = 1.1 KLF
LOADING FROM SHAFTWALL: 0.160(1.2) = 0.2 KLF

LOADING



$M_{MAX} = \frac{1}{8}wL^2 = 96$ FT-K
$V_{MAX} = \frac{1}{2}wL = 19.1^K$

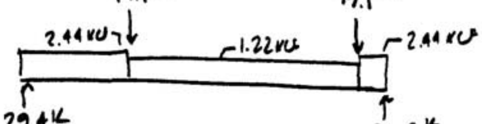
W14X26 COMPOSITE W/ [20 · ¾" SHEAR STUDS]
$\Sigma Q_N = 20(19.6^K) = 392^K > 385^K$ → FULLY COMPOSITE
$b_{eff} = \frac{1}{2}(60") = \frac{1}{8}(20') = 30"$ $Y1 = 0$
$a = \frac{385}{0.85(4)(30)} = 3.77$ IN $Y2 = 6.5 - \frac{1}{2}a = 4.6$ IN.

$\phi_b M_p = 312$ FT-K $> M_U$ ✓OK
$\phi_v V_N = 106^K > V_U$

② GIRDER SUPPORTING BEAM ①

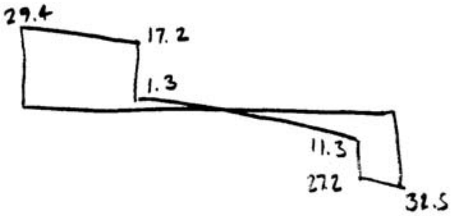
FLOOR LOADING. 0-5' : (10' TRIB)(1.2(70)+1.6(100)) = 2.44 KLF
5'-17'6": (5' TRIB)(") = 1.22 KLF
FROM INFILL BEAM : 19.1K @ 5', 15.33'

LOADING



$V_{MAX} = 32.5^K$
$M_{MAX} = 117.2$ FT-K

SHEAR



APPENDIX D, CONT'D

TESTING BEAMS AND GIRDERS UNDER VAULT LOADS

SUPPLEMENTAL BEAM UNDER VAULT

SPOT CHECK #4

VAULT LOAD: 55^K DL, ASSUMED TO BE DIVIDED EQUALLY BETWEEN EAST AND WEST INFILL BEAMS

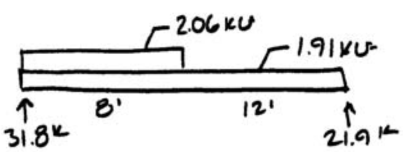
NORMAL FLOOR LOADS: 70 PSF DL
100 PSF LL

① INFILL BEAMS

VAULT LOADING: $55^K / 2 / 16'\text{-}0 = 1.72\text{ KLF} \times 1.2 = 2.06\text{ KLF}$

NORMAL LOADING: $(\text{TRIB } 0.5(6.67+9))(1.2(70)+1.6(100)) = 1.91\text{ KLF}$

LOADING



$V_{MAX} = 31.8^K$
$M_{MAX} = 127.2$ FT-K

SHEAR

W16x31 COMPOSITE W/[20 - 3/4" SHEAR STUDS]

$\Sigma Q_N = 20(19.6) = 392^K$ ASSUME $Y1 = 0.110$ IN.

$b_{eff} = \frac{1}{8}(20') = 30"$

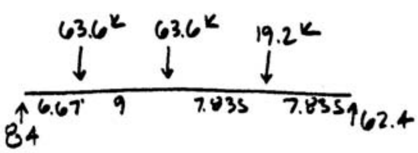
$a = \frac{392}{0.85(4)(30)} = 3.84\text{ IN}$ $Y2 = 6.5 - \frac{1}{2}a = 4.6\text{ IN}$

$\phi_b M_P = 383\text{ FT-K} > M_U$ ✓OK
$\phi_V V_N = 118^K > V_U$

② GIRDER SUPPORTING ① AND NORMAL LOADS

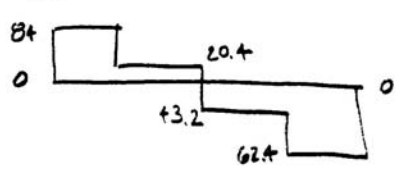
LOADING FROM ① : 63.6^K @ 6.67', 15.67'
LOADING FROM NORMAL LOADS
$\frac{1}{2}[(7.835'\text{ TRIB})(1.2(70)+1.6(100))](20')$ $19.2^K \rightarrow 38.4^K$ @ 22.8'

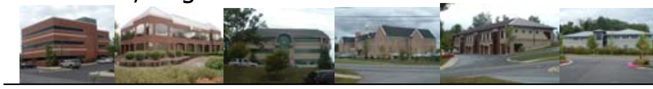
LOADING



$M_{MAX} = 743.9$ FT-K
$V_{MAX} = 84^K$

SHEAR





APPENDIX D, CONT'D

TESTING BEAMS AND GIRDERS UNDER SIDEWALK LOADS

UNDER SIDEWALK   SPOT CHECK # 5

NORMAL PARKING / DRIVEWAY LOADING: 93 PSF D
280 PSF L+S
SIDEWALK ADD'L : 75 PSF D

① CHECKING INFILL BEAM

LOADING. $(2.25' \text{ TRIB})(1.2(93+75)+1.6(280)) = 1.388$ KLF

$M_{MAX} = \frac{1}{8} wL^2 = 69.4$ K-FT
$V_{MAX} = \frac{1}{2} wL = 13.9^K$

W10x19 COMPOSITE W/ [22 - 3/4" SHEAR STUDS]
$\Sigma Q_N = (22)(19.6) = 431.2^K > 281$ FULLY COMPOSITE
$b_{eff} = 27$ IN. $Y1 = 0$
$a = \frac{281}{0.85(4)(27)} = 3.06$ IN. $Y2 = 6.5 - \frac{1}{2}a = 5''$
$\phi_b M_N = 201$ FT-K $> M_U$ ✓ OK
$\phi_v V_N = 68.8^K > V_U$

② CHECKING GIRDER UNDER ①

APPROX. DISTR. LOAD: $(18.75' \text{ TRIB})(1.2(93+75)+1.6(280)) + (18.75)(6)(1.2(75))/27.33 = 10.25$ KLF

$M_{MAX} = \frac{1}{8} wL^2 = 957.21$ K-FT
$V_{MAX} = \frac{1}{2} wL = 140^K$

W24x76 COMPOSITE W/ 45 [3/4" SHEAR STUDS]
$\Sigma Q_N = 45(19.6) = 882^K$ ASSUME $Y1 = 0.340$ IN.
$b_{eff} = \frac{1}{4} l = 82''$
$a = \frac{882}{0.85(4)(82)} = 3.17$ IN $Y2 = 6.5 - \frac{1}{2}a = 4.9$ IN.
$\phi_b M_N = 1230$ FT-K $> M_U$ ✓ OK.
$\phi_v V_N = 284^K > V_U$