T.C. WILLIAMS HIGH SCHOOL

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STRUCTURAL OPTION

STRUCTURAL DEPTH

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STRUCTURAL DEPTH

PROBLEM STATEMENT

Due to the large budget, the structural system was designed using fairly conservative sizes, and a simple design. Had the owner felt the need for a more valued engineering approach, the structural system would most likely need to be optimized. A major problem with the building site was its poor soil quality, which led to complicated foundations. If two more stories were added to the top of the classroom wings, and the overall square footage of the wings remained the same, then there may be some savings in the overall cost of the structure. With an addition to a reduction in just foundation costs, it will also be beneficial to examine the possibility of a more economical building design. Had the owner felt a need for additional stories at the time more solutions may have been explored by the design engineer. I intend to propose a value engineered solution that will decrease construction costs, project duration, and material usage, while accounting for two additional stories in exchange for a smaller building footprint. The overall building square footage will remain approximately the same (108,000 SF / Classroom Wing). To accomplish this I will use code requirements from IBC 2006, and ASCE 7-05.

STRUCTURAL REDESIGN ELEMENTS

- REDESIGN OF ROOFING SYSTEM
- REDESIGN OF FLOOR SYSTEM
- REDESIGN OF COLUMNS
- REDESIGN OF LATERAL FORCE RESISTING SYSTEM
    - WIND
    - SEISMIC
    - DISTRIBUTION
- REDESIGN OF EXTERIOR WALLS
- REDESIGN OF FOUNDATIONS

REDESIGN OF ROOFING SYSTEM

Originally there was nothing wrong with the roofing system. It was both economically efficient, and aesthetically pleasing. However with the adding of two additional stories, and the thinning of the building, the mechanical systems on the roof which were once hidden from sight may now be seen.

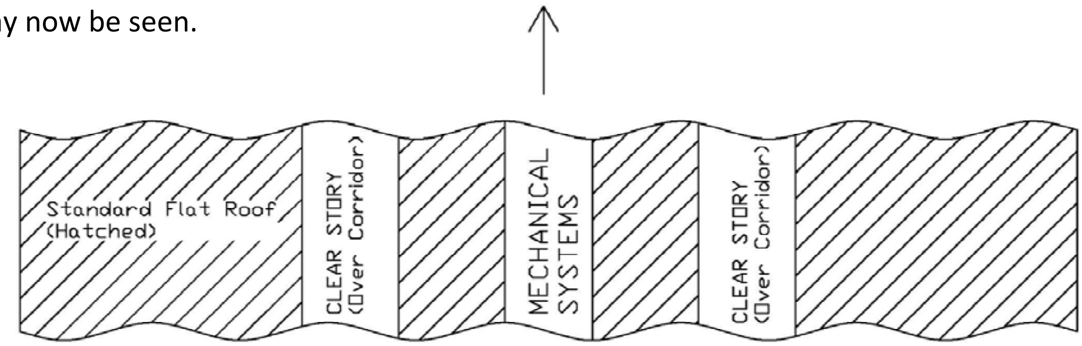


Figure 5 - Existing Roof Plan Strip

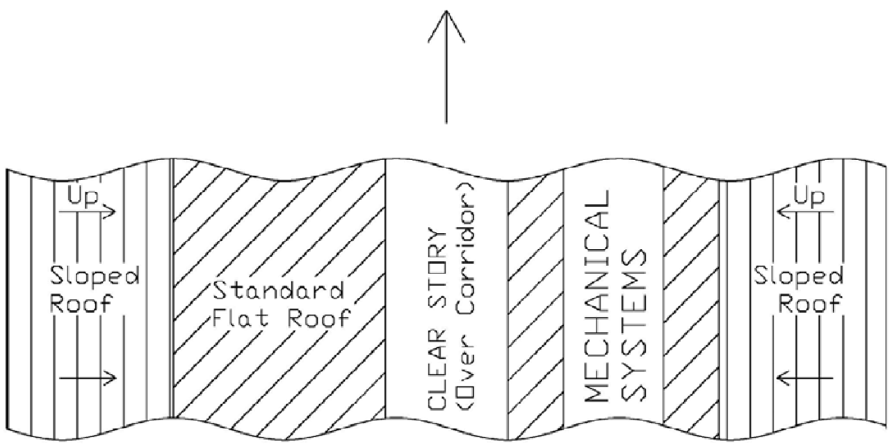


Figure 6 - New Designed Roof Plan Strip

The existing roof was made up of 24K6 open web steel joists spanning a maximum 34 feet and spaced no more than 5 feet on center. Supporting the two clearstories are large trusses, used additionally as an architectural feature, and are spaced 23 feet on center. 24KCS5 steel joists are found under the mechanical systems, because they are able to better resist an unbalanced load, where a k-series steel joist is more suitable for resisting an evenly distributed load. The KCS joists are typically spaced no more than 3-4 feet apart. W21x44 girders were used to transfer the loads from the joists to the columns.

In the redesign of the roof a more complicated system was needed. 24KCS3 joists and 1.5” 18GA decking support the mechanical system. These joists are spaced no more than 3 feet on center, and span 24.5 feet. 18K3 joists spaced 5 feet on center support the same area as the KCS joists, but only when mechanical systems are not present. Where the roof slopes, 10K1 joists on 1.5” 22GA deck are used to resist the 11.5 foot span. W18x35 girders were designed to transfer the loads from the joists to the columns. The steel truss supporting the clearstory remained the same. All roof joists are subject to meet an L/240 total load deflection, and an L/360 life load deflection.

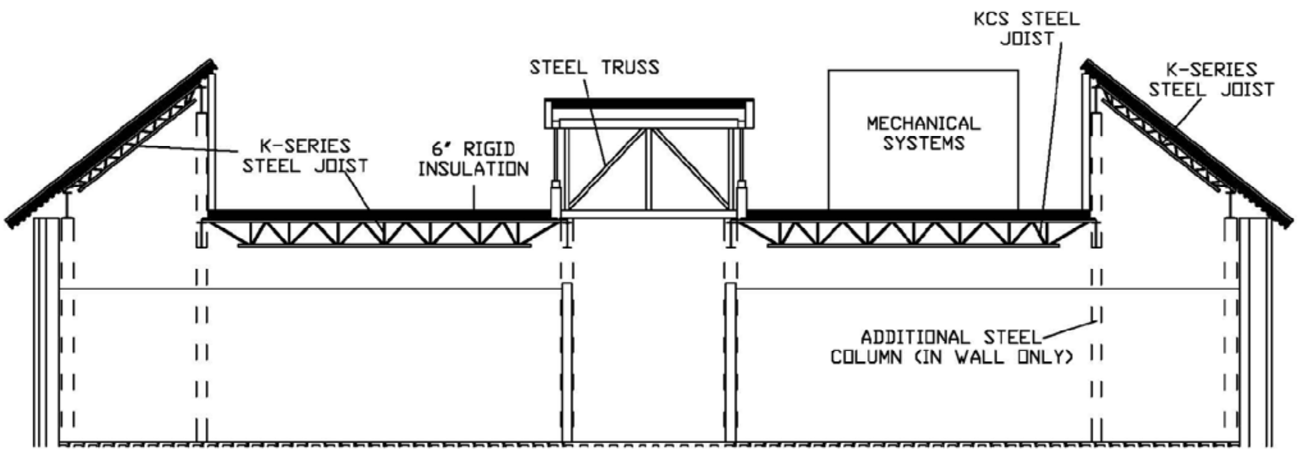


Figure 7 - New Designed Roof Section Cut

As seen in the figure above, a column comes up from the floor below and supports the two girders at mid span. This column starts at the 5th floor and ends at the roof. It is supported by a steel beam on the floor below, and braced at the top, in two positions by steel girders. This column is only located inside masonry partitions, and are therefore typically spaced 23 feet on center.

REDESIGN OF FLOOR SYSTEM

Originally the floor system was composed of W18x35 composite steel beams on 1.5” 18GA composite deck. ¾” thick 3.5” long shear studs with a 3” NWC slab, resulting in an effective slab depth of 3.75”, and a total slab depth of 4.5” were used to transfer the composite action. The beams span a maximum of 34 feet and are staggered spaced in a 23 foot bay at 8’-

1¼", 6'-9½", and 8'1 ¼". The W21x50 girders supporting the beams are also composite, and typically span 23 feet. In all instances the steel studs are spaced 1 foot on center. In addition, none of the beams are cambered.

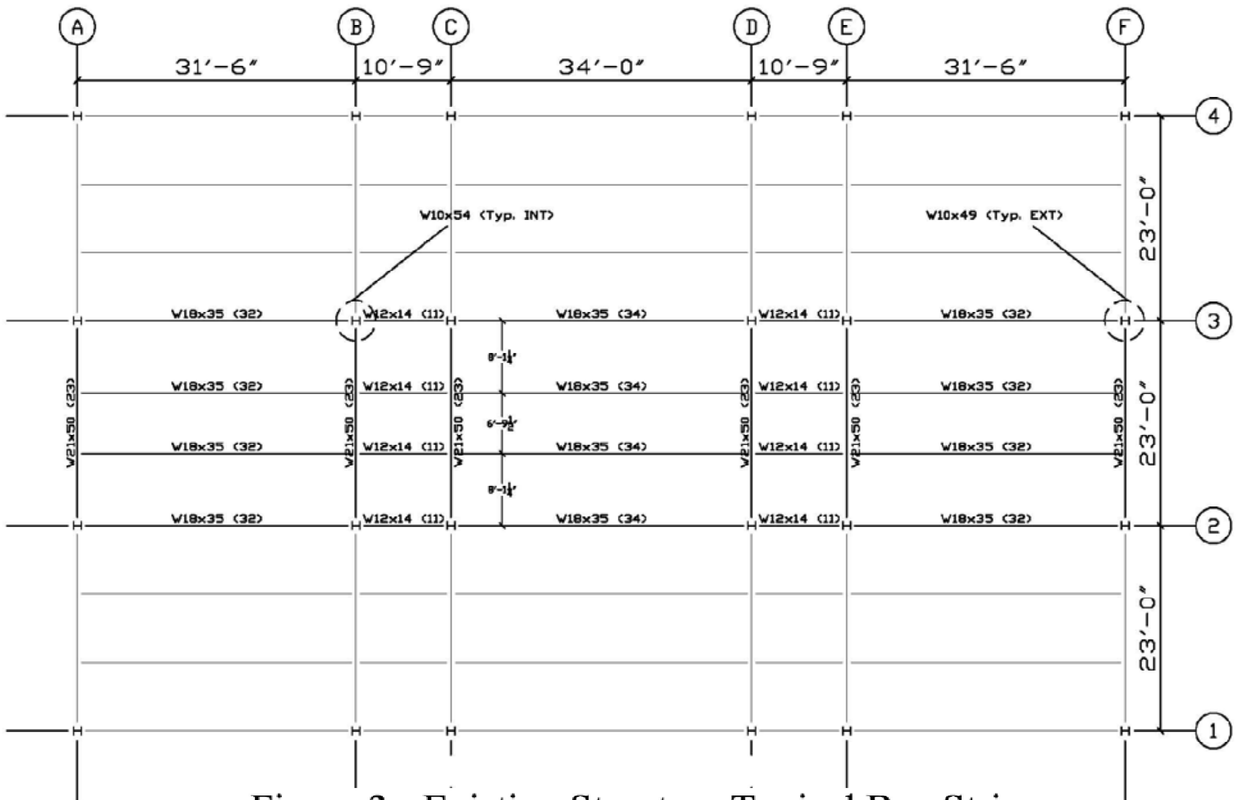


Figure 3 - Existing Structure Typical Bay Strip

STRENGTH DESIGN

EVALUATION CRITERIA

- DL = 50 PSF
- LL (reduced) = 46 PSF
- Δ = L / 600 (Live for Masonry Walls)
- Span = 34'
- Spacing = 7.67'

In the floor redesign an attempt was made to reduce the slab thickness by switching to a steel joist system. In addition to a thinner slab, consideration was also made to keep the cost of fireproofing of joists down. By reducing the number of joists and increasing their spacing to that of the composite system, it significantly reduces the cost of fireproofing. Normal K-Series steel joists normally will only be efficient when spaced 24"-30" on center. Therefore a composite joist system was designed to meet the required criteria. Using the Vulcraft catalog for composite steel joists, for strength and deflection design the minimum required size was found to be a 20VC1200 (weighing 21 PLF). To meet the required spans a 2 inch deck with 2.5 inch topping is required. It would also be most efficient to place the shear studs as shown, in the strong position. This will slightly increase the strength capacity of the composite joist. Additionally a worst case design was used to design the joist that would place a joist under a masonry wall partition, and limit its live load deflection to L/600. An L/600 deflection is chosen to prevent the masonry wall from cracking.

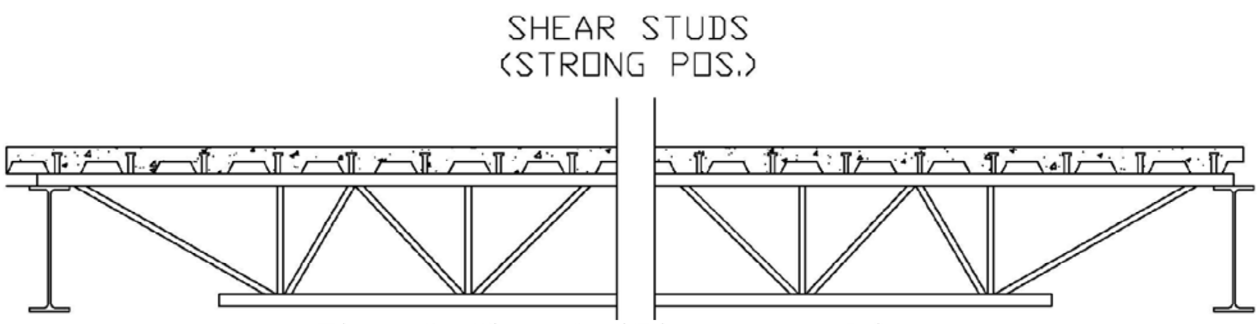


Figure 8 - Shear Stud Placement on Joist

CORRIDOR

Supporting the corridor is a 10k1 K-series steel joist. Normally a form deck would be the most appropriate for the non composite joist system, but since most of the floor is using composite decking it would make more sense to just stay consistent than to change the decking in the middle of the floor.

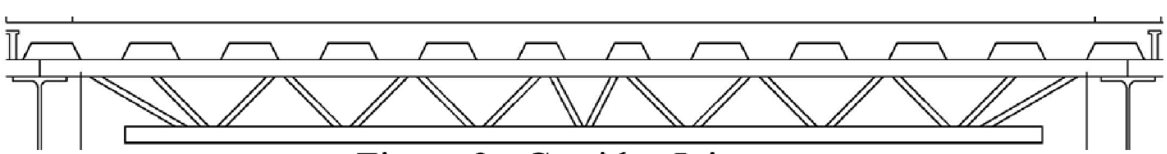


Figure 9 - Corridor Joist

VIBRATION

EVALUATION CRITERIA

- $P_o = 65\text{lb}$
- $\beta = 0.03$
- $\alpha_o / g = 0.5\%g = 0.005g$
- $a_p / g = P_o\, e^{(-0.35fn)} / \beta W$
- $f_n = 4.9$ Hz
- $W = 78$ kips

When vibration analysis was calculated using Design Guide 11, both the joist and girder sizes had to be bumped up to meet criteria for an office building design. While an office building definitely is not the same as a school building, the assumption this building is an office building should be fairly conservative. A live load of 11 PSF, and a dead load of 4 PSF was assumed in the calculations, because these are the design loads for a paper office building. A β value of 0.03 was chosen since masonry walls surround the exterior of the bay. If a masonry wall is inside the bay a β value may be used of 0.04, but to be conservative all bays were designed using β = 0.03. The joist was sized up to a 22VC1600 (weighing 24 PFL). This joist is able to support an additional 400 pounds per foot, and needed to be 2 inches deeper than the joist designed just for strength and deflection purposes. The deck and slab remained the same as the strength design. The final slab properties are 2” decking with a 2.5” topping equating to an effective slab depth of 3.5”, and a total slab depth of 4.5”.

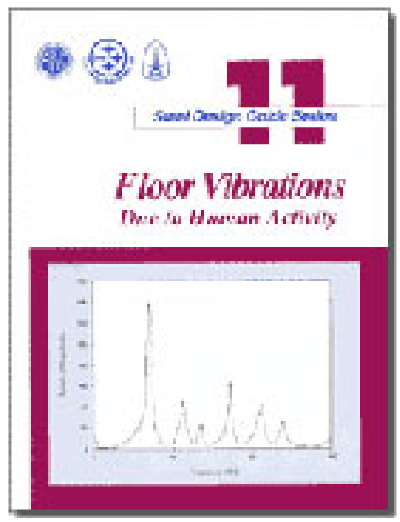


Figure 10 - DG 11 Floor Vibrations

FINAL COMPOSITE JOIST DESIGN

A more efficient composite joist could have been chosen, but it would have had to have been a deeper joist, and more coordination with the mechanical engineer would need to have been taken into account. With all things considered it most likely would be less economical to increase the depth of the joist any further, as it would cause a change to the mechanical system. As it is now the joist is almost even with the 21 inch girders. Considering the joist seat is 2 inches deep, the bottom of the joist will still be able to increase one inch in depth before it reaches even with the girder.

The Vulcraft design guide of composite steel joists requires a 22VC1600 joist to contain 24 ¾" thick shear studs, for the size required by the vibration analysis. However for strength only 18 ¾" thick shear studs are required. Since shear studs do not effect vibration at all, since all members assume composite action with vibration analysis, it will be sufficient to only use 18 ¾" thick shear studs as required by the strength and deflection design. This is approximately 1 stud every 2 feet.

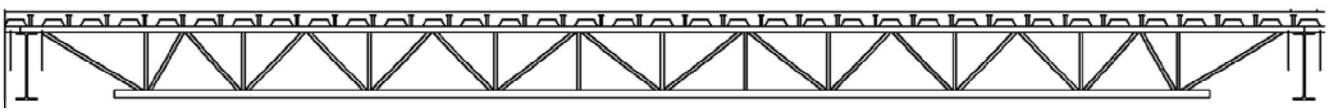


Figure 11 - Typical Composite Joist

Without driving the cost of fireproofing up, the slab was only able to be ¼" thinner than originally designed, which is very minimal savings, and will only result in about a $10,000 savings for each classroom wing. However, approximately 50% less shear studs will be required with this design, which should add up to more sufficient savings. But, considering joists cost more to make than a beam it will be interesting to see if this system is actually cheaper than the composite steel beam system.

REDESIGN OF COLUMNS

The columns used to support the previous floor system of the classroom wings were all steel and ranged from a W10x49 to W10x54 Grade 50 ASTM A 992 members. None of the columns needed to be spliced in the previous design, as they all spanned the full 45 feet or 3 stories.

With the building increasing in size, to 75 feet in height and 5 stories tall, it is now necessary to splice the columns. Typically columns are spliced at either every 2^{nd} or every 4^{th} floor because it is the most economical. However the reasoning behind this is for construction purposes with the different trades as you go up the building. Since a splice is also equivalent to 500 pounds of steel, it will be necessary to reduce the number of splices. For this building of 5 stories it will be most economical to splice the column after 3 stories. This will keep the size of the columns to a manageable size, and the effect of a 4:1 splice, or a 3:2 splice would be the same when considering construction trades. The splice will be taken a couple feet above floor level where it will be most manageable.

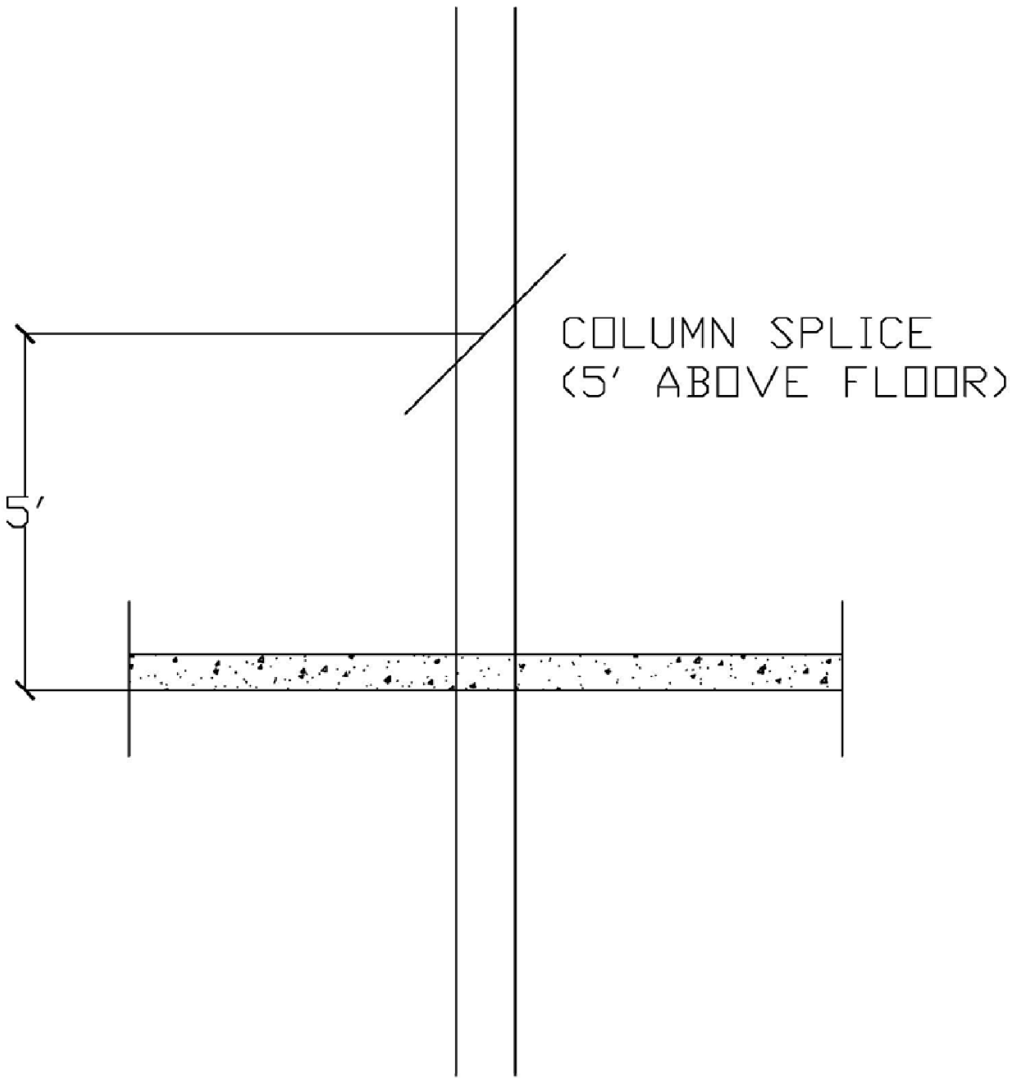


Figure 12 - Column Splice

The typical column with the new design will be a W10x54, similar to the worst case design of the original building. This may be due to the fact of a lighter floor system and a change in codes from ASD to LRFD. Some of the columns actually turned out to be slightly smaller, but it was decided to standardize them at W10x54 to lower the overall costs.

REDESIGN OF LATERAL SYSTEM

WIND

Wind originally wasn't the controlling load case with the existing building. Due to the buildings short and wide shape, along with poor soil conditions, seismic controlled in each direction. When the building gained height and reduced in thickness, the original controlling cases had to be rechecked. Wind was originally designed to give a base shear of 332 kips in for wind acting towards the long direction and 120 kips for wind acting towards the short direction, both of which turned out to be less than the seismic base shear. With the change in shape of the building, the loads where recalculated. The new wind forces were calculated to be 410 kips for wind acting in the long direction, and 157 kips for wind acting towards the short direction. The differences in these numbers are related to the amount of square footage of building façade the wind has to act on.

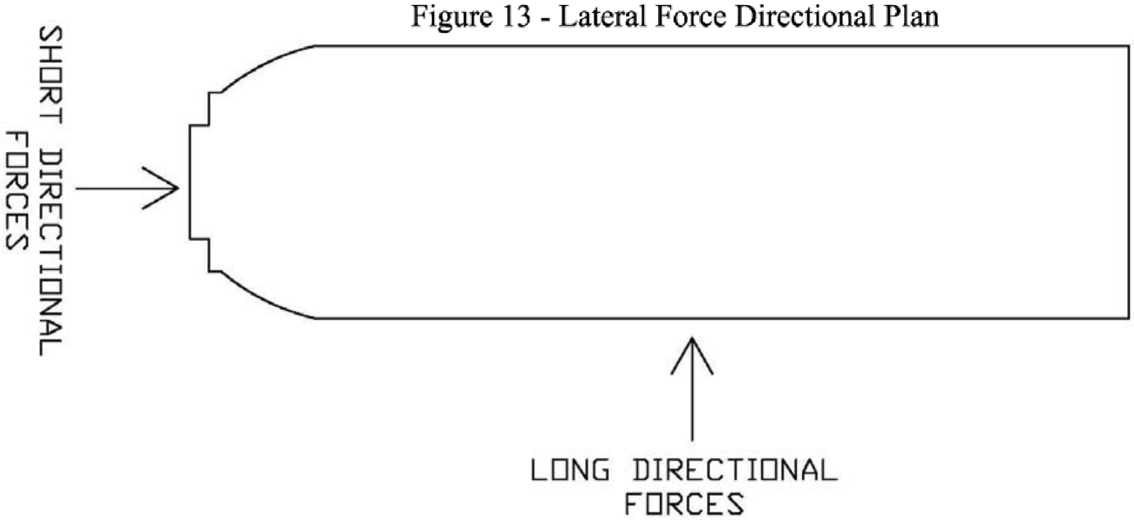
SEISMIC

EVALUATION CRITERIA

- Site Class D
- S_{DS} = 0.1632, S_{D1} = 0.080
- Seismic Design Cat: B
- R = 4.0
- T = 0.867 seconds
- C_S = 0.029
- W_F = 2200 kips / Floor
- W_{RF} = 1620 kips / Roof
- V = 302 kips

Seismic originally easily controlled in both directions with a base shear of 488 kips. With the buildings change in size to a thinner and taller building, along with the slight reduction in floor weight, and change in lateral resisting system, the new calculated base shear was 302 kips. This is significantly smaller than the previous shear force. With the new building design seismic no longer controls for forces acting in the long direction, but instead only controls the forces acting toward the short direction.

Figure 13 - Lateral Force Directional Plan



LATERAL FORCE COMPARISON

Both wind and seismic will have an effect on the buildings lateral system. Wind will be the controlling long directional force, and seismic will be the controlling short directional force. The reasoning for this can be related to the amount of square footage the wind force has to act on. The short direction is only 80 feet in width, compared to the long direction which is 270 feet in length. The total seismic base shear will ignore the dimensions of the building, and is strictly related to the buildings weight, which allows seismic forces to govern the design of the lateral resisting system resisting loads from the short directional forces. The difference in the pound per square foot wind forces is from the difference in effects from leeward wind forces. The leeward wind force grows with the length of the building in the respective directions.

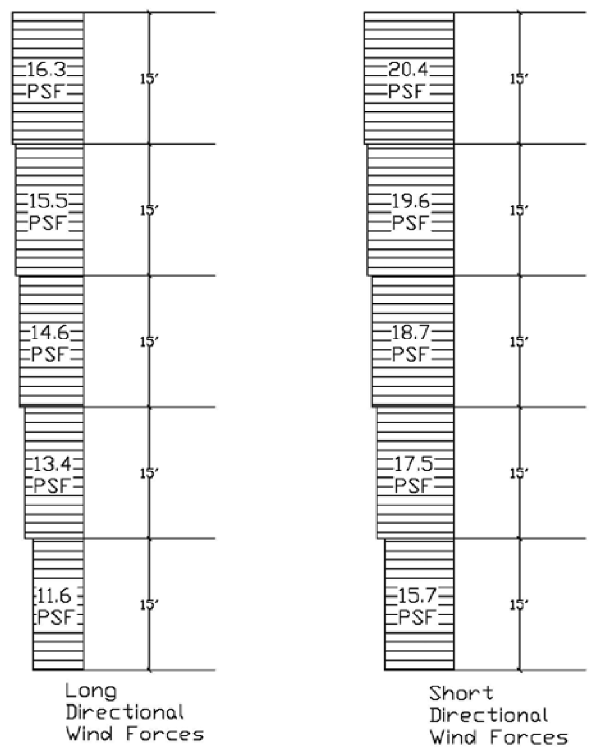


Figure 14 - Wind Force Diagram

LATERAL FORCE DISTRIBUTION

For wind design, the forces distributed to each floor are a combination of the force on the wall below and above the floor level. These are taken from the midpoint of the wall. For seismic the forces are taken from a combination of floor height about ground level, and the weight of the floor. After these loads are distributed to the floor level, they are then distributed to the ordinary reinforced masonry sheer walls based on stiffness of the respective walls. This is due to the concrete slab which lets the floor act like a rigid diaphragm. At the roof level the forces are distributed through tributary area, because it is assumed to act as a flexible diaphragm, since it is just steel decking. Along with direct shear forces, walls also receive a torsional shear force that can either raise or lower the total shear force in the wall, depending on the location of the sheer walls with respect to each other and the center of mass.

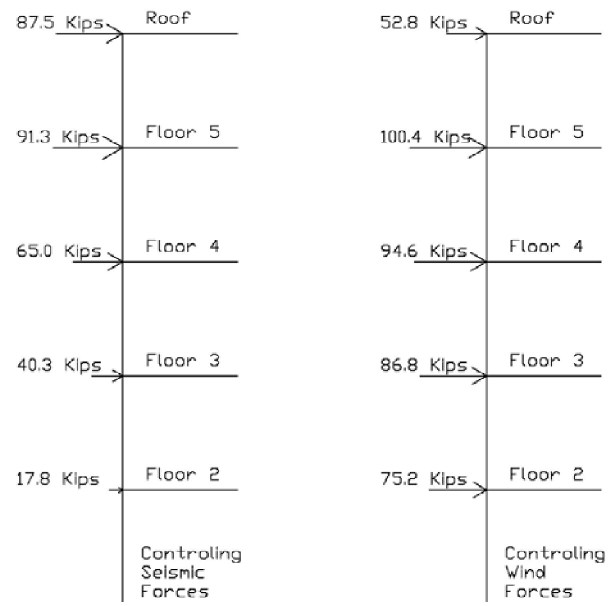


Figure 15 - Controling Lateral Force Floor Distributions

LATERAL RESISTING SYSTEM REDESIGN

The existing lateral force resisting system in the classroom wings was ordinary braced frames. The braced frames had a response modification coefficient, also known as an R-Value of 3.25. A switch to ordinary reinforced masonry shear walls will result in an R-Value of 4.0, which in turn will allow the seismic force taken on the building to be reduced. An ordinary reinforced masonry shear wall requires a minimum reinforcement spacing of 48 inches.

Main reasoning between the switch to masonry shear walls was an attempt to save money by using materials already present in the building. 8 inch CMU partitions are already found in the classroom wings, and it would be cost effective to take advantage of that. However, existing partitions are not full height, and generally stop approximately 4.5 feet below the next floor. Therefore, the only extra materials that will be needed are a few extra courses of CMU blocks, grout, and reinforcing steel. By extending the wall the full height, it also creates the ability to provide additional gravity load on the wall, which will in turn help resist the applied lateral forces.

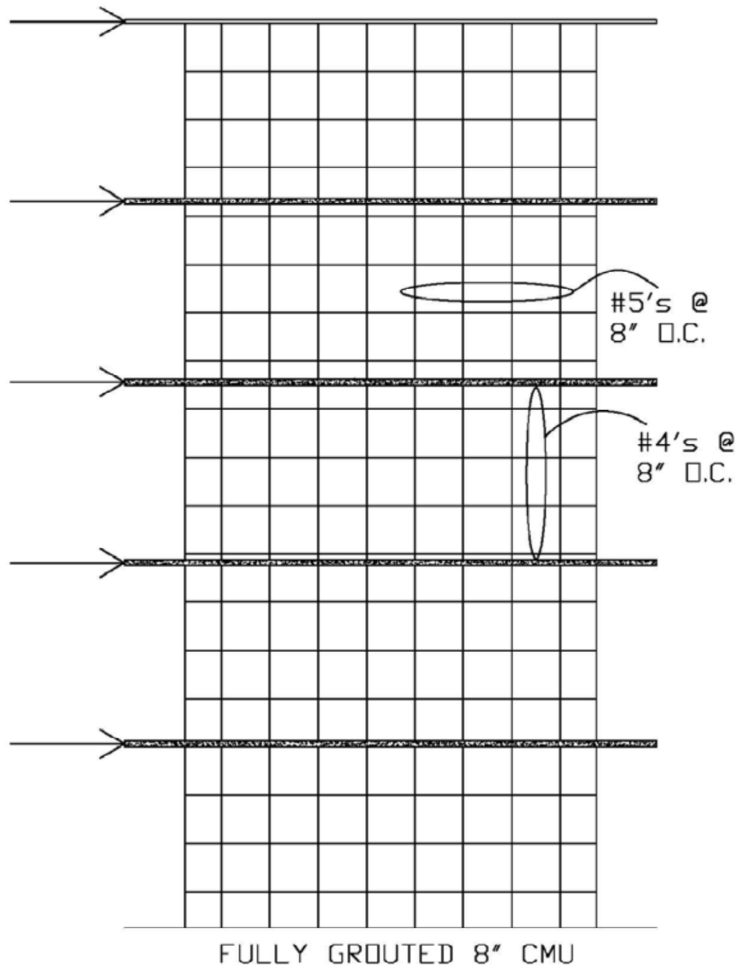


Figure 16 - Masonry Shear Wall

A spread sheet was created to make it possible to find the amount of reinforcing steel, and the resulting moment capacity of the 34 foot long shear wall. It was determined that when distributing the lateral load over 2 shear walls, while accounting for torsion, a fully grouted and reinforced shear wall would be required. Number 5 reinforcing bars where chosen to allow sufficient room to splice. Anything larger would cause problems for the contractor. The depth of the neutral axis, 'c', was calculated to be 73.024 inches. With (#5) vertical reinforcement, spaced 8 inches on center, the resulting moment capacity, ϕMn was calculated to be 121,120 inch kips. This was greater than the calculated moment load Mu of 107,838 inch kips. A calculated factored base shear of 205 kips was also calculated from the applied loading. To resist the base shear, (#4) reinforcing bars where chosen. When calculating the minimum required spacing, the effect of gravity was ignored. This is a conservative approach, and should generally be used when designing with masonry shear walls, as they have the tendency to crack. Minimum spacing was calculated to be 11.83 inches. However, since actual spacing of reinforcement in masonry walls must be in multiples of 8 inches, it was decided that #4 reinforcing bars spaced at 8 inches on center was acceptable to resist the shear loads.

The final shear wall design is fully grouted 8 inch CMU with #5 vertical reinforcement spaced at 8 inches on center, and #4 horizontal reinforcement spaced 8 inches on center. A total of two masonry shear walls in each direction are required to resist the lateral loads.

DRIFT CHECK

EVALUATION CRITERIA

- ΔShear = V*h / A*G
- ΔBending = $w*h^4 / 8E_mI_w$
- ΔTotal = ΔShear + ΔBending
- ΔAllowable = L / 400 = 2.25"

Drift was checked using a combination of shear and bending deflections. The total drift is the sum of the two. Each deflection was computed using fixed – fixed criteria. Area of the wall was taken as the walls length, multiplied by the walls width, because it is fully grouted. Moment of inertia of the wall, I_w was computed by taken by the equation $bh^3 / 12$, where b is the wall thickness and h is the wall length. The sum of the two deflections was 0.55 inches, which is far less than the required 2.25 inches.

REDESIGN OF EXTERIOR WALLS

The existing exterior curtain wall consisted oscillating 'window sections', and a 'column sections'. What are referred to as window sections, is a 14 foot span of exterior wall that contains the large windows. Above the windows are steel lintels that transfer the weight to the 'column sections'. The column sections consist of 8 inch CMU backing up 14 inch CMU, that then backs up the 4 inch face brick. Column sections are approximately 5 feet in length. A redesign was attempted to possibly make this exterior wall load bearing. Given the increase in height this would create a 75 foot high load bearing wall. As expected the wall fails to be able to resist any extra load bearing forces, and where just redesigned to resist components and cladding wind forces, and self weights.

The 14” CMU was able to remain un-grouted with the use of fully bedded masonry. However, the 8” CMU was required to be fully grouted with reinforcement spaced 16” on center. This is up from 8” CMU grouted and reinforced at 48” on center.

REDESIGN OF TYPICAL FOUNDATIONS

Since the change in height on the building related to a change in loads on the footings, they required a structural redesign. The Geopier system to increase the poor soil conditions was left unchanged, as it is the most efficient system for this job.

GEOPIER SOIL REINFORCEMENT

THE GEOPIER RAMMED AGGREGATE PIER SOIL REINFORCEMENT SYSTEM

- Drill a cavity to depths ranging from 7 to 30 feet deep.
- Place a 12-inch layer of open-graded aggregate at the bottom of the cavity.
- Compact the aggregate using a tamper that delivers a high-energy impact ramming action.
- The ramming action compacts the aggregate and pre-stresses the surrounding soil. Successive lifts of well-graded aggregate are then rammed in place.

Geopier ‘RAP’ Systems are intermediate foundation systems, constructed by densely compacting successive thin lifts of high quality crushed rock in a 2 to 3 foot cavity of varying depth using ramming equipment. The vertical ramming action increases the lateral stress and improves the soils surrounding the cavity, which results in foundation settlement control and greater bearing pressures for design.

RAP Systems can be installed using replacement or displacement methods, depending on site requirements. The installation process utilizes vertical impact ramming energy, resulting in extra strength and stiffness. RAP Systems are used to reinforce good to poor soils, including soft to stiff clay and silt, loose to dense sand, organic silt and peat, variable, uncontrolled fill and soils below the ground water table.

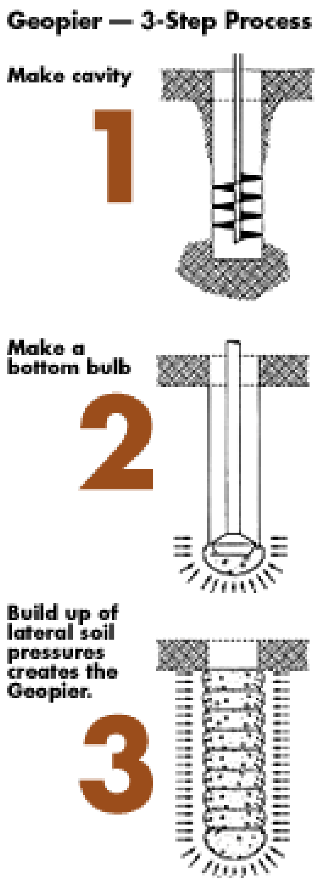


Figure 17 - Geopier Reinforcement

Geopier 'RAP' soil reinforcing is still the suggested form of reinforcement. The old TC Williams High school was still holding classes while the new school was under construction. The amount of noise and vibrations caused from installation would be critical to control. This Geopier process is excellent at creating minimal vibrations and noise, and therefore perfect for this environment. Additionally poor soil conditions, consisted of fill including clay, silt and gravel to depths ranging from 2 to 29 feet below the ground surface. The fill was underlain by native gravel, sand sandy silt, and clay. Groundwater was then encountered at 15 feet below the surface.

Deep foundations such as auger cast piles, pre-cast concrete piles and timber piles were initially considered, but the Geopier 'RAP' system was selected because it offered the most cost-efficient solution without compromising integrity. The savings were estimated at around 20%. More than 1,700 'RAP's were installed to reinforce the existing fill and support the shallow foundations.

FOOTINGS

Foundations were redesigned to account for the addition loads from the columns, exterior walls, and lateral resisting shear walls. Original spread footings for the columns were sized 6 feet by 6 feet and 7 feet by 7 feet by 1 foot 4 inches thick. They are reinforced by #6 and #7 bars respectively, at 12 inches on center. Typical strip footings were designed at 5 feet wide by 1 foot 4 inches deep. They are reinforced with #6 bars at 12 inches on center. All footings are 3,000 psi normal weight concrete.

The typical spread footing was redesigned to be 8 feet by 8 feet and 18 inches deep, with #5 reinforcing bars spaced 8 inches on center in each direction. An allowable soil bearing capacity of 6,000 PSF was used, along with 3,000 psi normal weight concrete. A 24 inch by 24 inch steel base plate was chosen to connect the column to the footing. The factored load from the column was calculated to be 406 kips. Wide beam shear was the controlling factor in the design of the spread footings.

The typical strip footing was redesigned to be 5 feet wide and 12 inches deep. The loads on the strip footings are just the self weight of the wall, which is approximately 14.325 KLF, factored. With an allowable soil bearing capacity of 6000 PSF, the minimum required width was 2.4 feet, but in order to keep the load of the oscillating wall in the kern, a width of 5 feet was chosen. Wide beam shear was found to control in the design, and the minimum reinforcement was found to be #5 bars spaced 12 inches on center. This also satisfies the minimum shrinkage and temperature reinforcement.

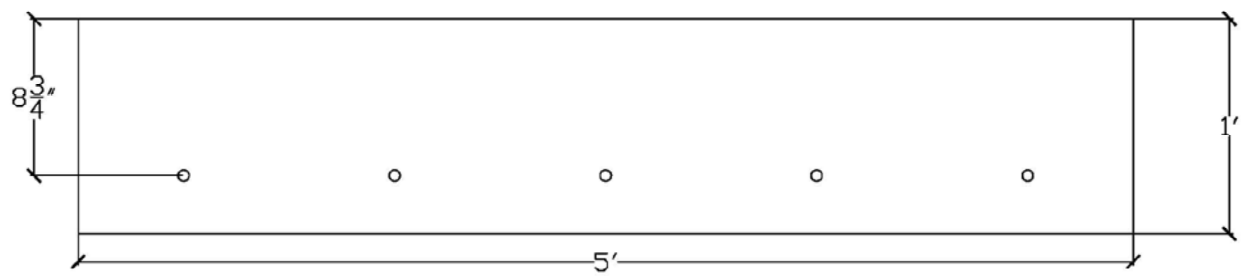


Figure 18 - Strip Footing

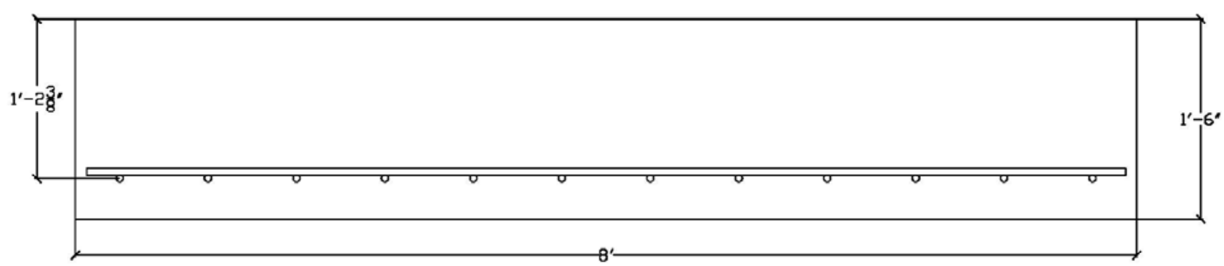


Figure 19 - Spread Footing

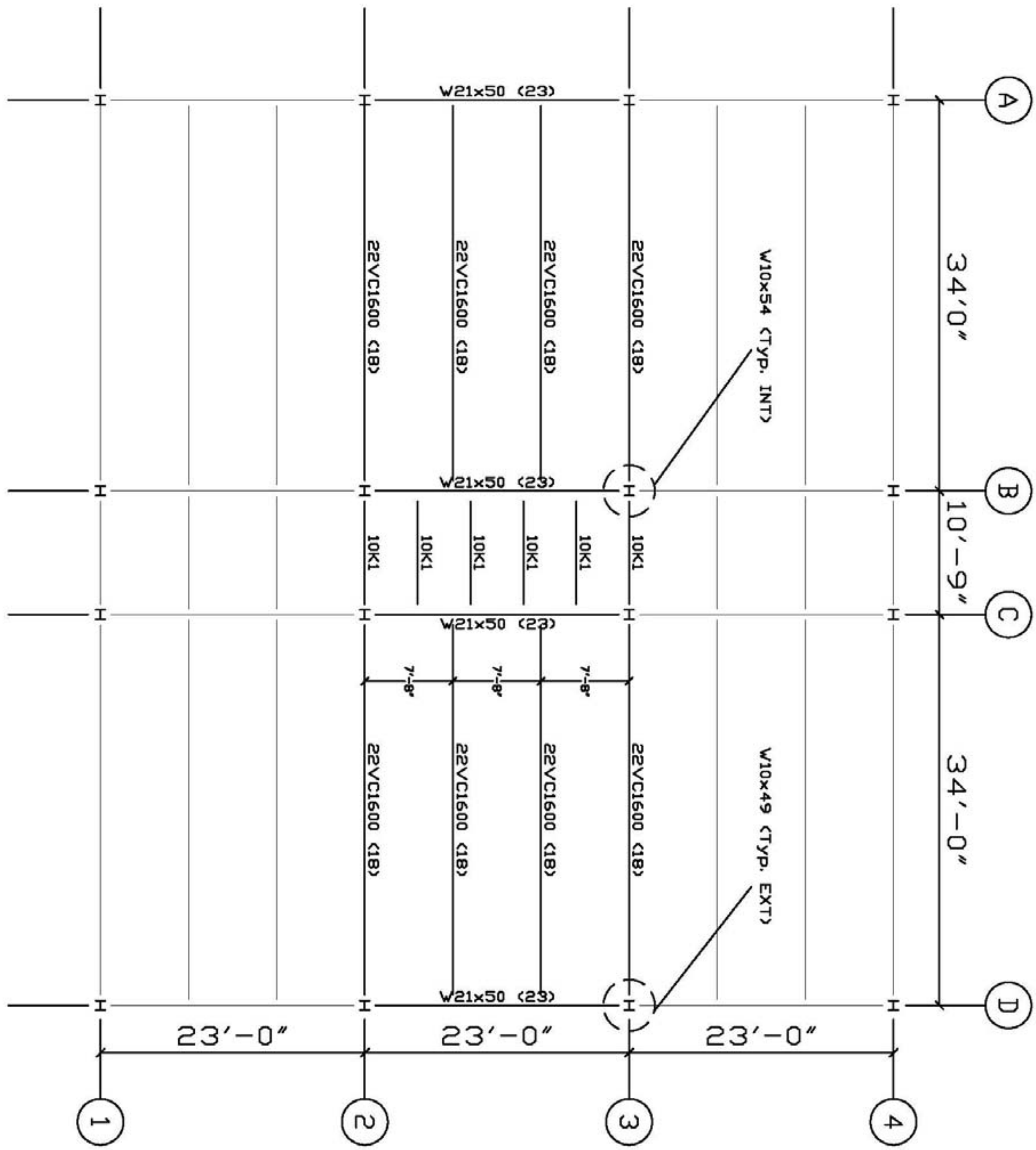


Figure 20 - New Structure Typical Bay Plan

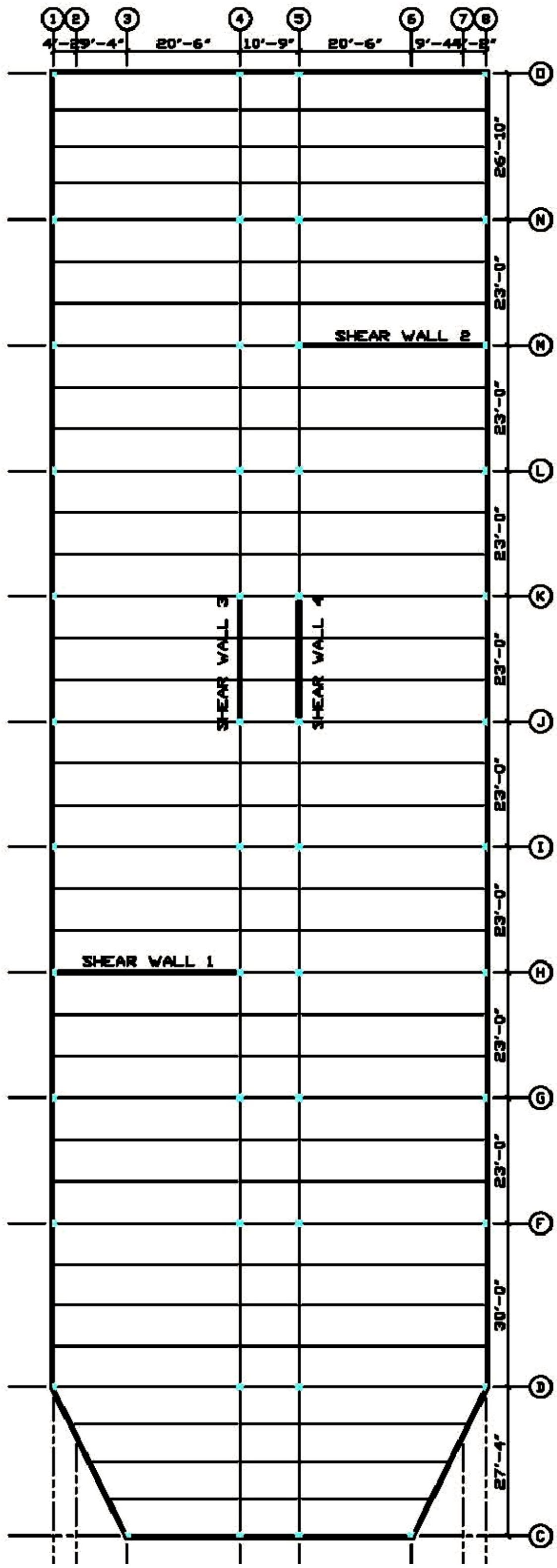


Figure 21 - New Structure Typical Floor Plan