Executive Summary:

The structural design of 2941 Fairview Park was done with consideration given to many different aspects, most importantly cost. Factors including future retrofit and serviceability have been shown to play a large role in determining what rent the owner may charge the tenants. A composite steel floor system was chosen because it was the most affordable solution and allowed for tenant retrofit with ease. This system is very efficient and fit in well with the exterior architectural concept.

This report was compiled in an effort to analyze the benefits and drawbacks of various structural systems, including structural steel, cast-in-place, and precast systems. Structural steel systems, including castellated beams and open-web joists were considered and found to be very efficient, but with possible vibration problems, reduced plenum space, and long lead times. Site-cast solutions, including pan joists, PT slabs, and PT beams, were also considered. Aspects including fire rating and acoustics were beneficial, but also exhibited significantly increased dead weight (and therefore foundation costs) as well as significant increases in labor costs and construction duration. Precast hollow-core planks were also considered, even though they have a long-lead time. Benefits of increased plenum depth, fire rating, and acoustics were available, but without the increased labor cost and gain in dead weight.

Tenant retrofit was simplest for the steel solutions, and increasingly more difficult with the more technical concrete solutions. Other serviceability issues favored the concrete solutions because of their greater mass. Overall, results have yet to either eliminate or favor any one particular system. In my opinion, the hollow-plank systems, spanning between either steel girders or PT girders (that doubled as part of the façade) seem like they merit a more detailed consideration. Further analysis, including a detailed cost estimate will need to be performed. At that time, it may also be necessary to weight the importance of different serviceability requirements versus cost.
Composite Steel Frame:

The current structural system being employed is a composite steel frame. Composite steel girders form 20'-0” bays from the central core to the exterior supports. Composite beams, typically three in number, span between the girders and support the deck. All steel members are grade ASTM A572 steel. Steel deck is 19 gage with 2” flutes. A 3 ¼” lightweight concrete slab covers the steel deck (5 ¼” total depth). ¾” A108 shear studs are placed along the beam and resist longitudinal shearing forces, thus enabling composite action between the beam and slab. Composite action allows the steel beam and concrete slab to work together as one section to resist bending moments. Roof deck is 1 ½” 20 gage steel deck.

Composite steel was chosen as the structural system for several reasons, including both economic and serviceability concerns. Structural steel is perhaps one of the quickest systems to erect, and therefore cuts down dramatically on construction costs. Pieces are delivered to site prefabricated with erection markings designating their location. They are simply placed in position, and an erection crew installs the minimum number of connector necessary to keep the member in place. Approximately thirty members can be erected per day by a typical crane. A different crew, at a later time, will pass through the building, completing all connections as per the structural drawings.

The dead weight of a steel structure is relatively small. This is especially true for composite steel, which requires less steel than a comparable non-composite structure. Foundations are minimized.
in size and complexity, thereby reducing both foundation and excavation costs. In this case, however, buried foundations were required to resist the wind uplift forces.

Of all the structural systems possible, composite steel is perhaps the easiest to retrofit to meet future tenants’ needs. Small openings can be cut in beams to run additional pipes and conduits. Larger openings, such as for stairs, can be saw-cut in the slab. Additional beams can be inserted to frame out the opening, and a new edge of slab can be poured. This procedure is fairly simply and cost efficient.

With all the advantages of structural steel, there are a few drawbacks. The fabrication time necessary to acquire structural steel is perhaps the largest negative. The time to roll steel shapes is approximately 6 to 8 months in the DC area. This long lead time requires designers to chose shapes and quantities early in the design process, when not all loads (such as mechanical equipment) are known precisely. Changes to the design, such as member size changes and architectural dimension changes are fairly difficult to implement because of the fabrication process. Some common shapes are available from warehouses, but come at a premium price.

Vibration can be an issue when designing either long spans or structures without a lot of mass. A RAM Steel model was constructed and typical members were tested for vibration during the design of this structure. It would be worthwhile to check Design Guide 11 for additional verification.

Steel construction can be difficult at times. Iron workers are unwilling to work in wet conditions due to the hazards presented by possible slipping. This can make construction difficult during rainy periods or during the winter. Steel superstructure erection did occur during the winter and early spring months on this project.

In some cases, composite steel construction requires the use of shoring because the gravity members have not gained sufficient strength or stiffness to resist construction loads to a serviceable degree. Shoring is a very expensive and time consuming procedure, and is best avoided whenever possible. Analysis of the existing structure showed sufficient strength and stiffness in the members to resist construction loads without shoring.

The depth of the structural members can possibly present limitations on the other designers. The high floor-to-floor heights associated with structural steel framing increase curtain wall costs. The overall exterior building façade is designed for a relatively high floor-to-floor height, so this does not
present a problem for this particular building. The plenum depth, however, is not that large. Distance between finished ceiling and finished floor is 4'-8". Typically, 6" is occupied by deck & slab, 1'-9" of this is occupied by steel beams (6" more at moment frames), leaving only 2'-5" for plenum depth. Other trades, such as MEP and telecommunications might benefit from a larger plenum space.

A composite steel frame with castellated beams is a slight variation to the typical composite steel frame that could be considered. The difference is the use of castellated beams instead of typical W-shapes for the beams and girders. Castellated beams are deeper than typical steel beams without the added mass of the larger steel shapes. This is accomplished in the mill by cutting and rearranging the beams so as to create openings near the neutral axis of the beam. The result is a lighter, stiffer beam that is suitable for long spans.

The main advantage of castellated beams over typical steel members results from the increased stiffness. The use of shear studs may be either reduced, or eliminated completely on these members. Additionally, lighter members may be used, thereby reducing dead weight. This decreases the cost of most foundation work, as well as other members within the structure.

Some MEP runs, such as pipes, conduits, and possibly ductwork, can be run through the holes in the beams. This would have the effect of un-cluttering the plenum space, or possible even reducing its depth (if ducts can be run through the holes). Most trades do not like to run their equipment through castellated beams, however, because it is more difficult than through a plenum space. Most likely, wire and pipes would be run through the castellations, but ducts would be run through the plenum space.

The depth of the castellated beams is greater than the depth of their corresponding W-shapes. If this is not compensated for by running MEP and telecommunications items through the holes in the beams, then the floor-to-floor height may need to be increased. If floor-to-floor height is kept constant, then the plenum spaced needs to be reduced. Even if pipes and electrical conduits are allowed to run through the holes, ducts are left to run through the plenum. With the reduced plenum depth, the ducts must be flatter, which creates more friction head for the HVAC system to overcome.

The decreased weight of the members increases the net uplift force acting on the buried mat foundations. This could be compensated for by increasing member weight elsewhere, such as encasing lower-level columns or using a heavier slab on the lower levels. The decreased weight of the members also leads to a greater possibility of encountering vibration problems.
Open Web Steel Joists:

Another steel structure that should be considered uses open-web steel joists as a floor system, while maintaining the same braced and moment frame lateral system. Steel joists, 24’-30’ in depth, would be spaced at 2’-0” on center and span 39’-10” from the core to the exterior of the building. Steel girders at the core and edge of the building should remain unchanged because the tributary width has not changed. Some steel beams may need to remain in the floor system to resist lateral forces. Joist girders may be an option for these beams. This is pending further analysis and design consideration. Steel beams in the core will remain to simplify future slab openings.

The reduction in spacing between the joists allows for a decrease in steel deck costs. A 22 gage can be used, as well as a reduced slab depth of 1 ½” (2 ½” to maintain a 2 hour fire rating). This further decreases dead weight, even if normal weight concrete is used. Open-web steel joists, like castellated beams, present the opportunity for conduits and pipes to be run through them. A 28” joist can accommodate ducts up to 18” in diameter (or a 13” square duct). Just as with castellated beams, however, the remaining plenum space is reduced in depth, which may cause problems for ductwork and other HVAC equipment.

A joist floor system creates more problems if a tenant retrofit is desired. Joists are not designed to take concentrated loads along their spans. Framing a large opening, such as a staircase, would require extensive framing with steel beams running both from core to exterior and as framing members. This
would be possible, but at a greater cost than would be necessary with a composite steel frame. The use of joist girders in areas likely to be altered would greatly simplify retrofit, but at a greater initial expense.

Lateral stability of joists is always an issue during construction. Until the decking has been placed and secured to them, they are very susceptible to lateral buckling failures. Three rows of diagonal bridging will be required for lateral stability.

With the reduced size of the deck and slab, the dead weight of the structure is significantly reduced. This may likely lead to vibration problems due to insufficient structural dampening. The application of normal weight concrete might be preferable to lightweight concrete in this scenario because of the increased mass.

Fire rating is a large issue that needs to be addressed when using steel joists. Due to their size, the joists heat up easily and create a potentially unstable condition in the case of a fire. Joists need to be heavily protected from heat, which adds greatly to the cost of the ceiling system.

Custom steel trusses would be a slight variation to the open-web steel joist system. In this case, trusses would consist of larger members and be spaced further apart. Because of their increased stability, these trusses might be able to transmit lateral forces and bending moments, as well as simplify renovation work. Due to their custom nature, they would be more expensive than typical joists. The trade-off between individual member expense and decreased system expense would need to be considered.
Pan Joists:

A structural cast-in-place concrete floor system would most likely include either regularly reinforced concrete pan joists, or some type of post-tensioning. Skip joists, a pan joist system where every other stem is blocked out, would span 39’-10” between reinforced concrete girders at the core and exterior of the building. Member depth, including the slab, might be 24”, as confirmed by rough calculations that can be found in the appendix. Girders would be roughly 18” wide by 24” deep to match the depth of the pan joists. Deflection within the skip joists was estimated to fall just within acceptable bounds for immediate loads, and just over the criteria for long term deflection. These calculations were very rough and a slight redesign of the joists would allow for these values to fall into acceptable ranges. Deflection should be less of an issue with the girders, which span only 20’-0”. Lightweight concrete with $f'_{c} = 4000$ psi should be used to minimize dead load as well as cost. Shear and torsion reinforcement in the girders will be excessive because the girders are end-beams and carry a large load over a short span.

A concrete system increases dead weight considerably over a steel structure. This would eliminate the need for the buried mat foundations, as there would no longer be a net uplift force to consider. The geotechnical engineer’s report indicates that for the most part, the soil would be able to carry the additional load without massive foundations. Foundations, however, would need to increase
dramatically in size to carry the additional dead weight. Other advantages of the increased dead weight include greater sound isolation, greater fire protection, and less susceptibility to vibration issues.

There is no virtually no lead time for site cast concrete. Rebar need only to be fabricated to length and bent as necessary. This short lead time allows the designers greater flexibility for revisions because substitutions and additional fabrications are relatively easy. Furthermore, the greater duration of concrete construction allows other trades to work simultaneously below them.

Site cast concrete is significantly slower during construction than steel and precast concrete. Adding to this slower rate is that concrete contractors in the DC area typically don’t like to work on tall buildings.

Winter construction would also be difficult because the concrete would need to be heated to achieve proper curing. This might require that either the curtain wall was installed shortly after the concrete reached sufficient strength, or that temporary enclosures were made to minimize heat loss. Either method causes construction complications and incurs additional expenses.

An architectural disadvantage to a concrete floor system arises when the gravity system as a whole is considered. Concrete columns would be used in this system. These occupy a much larger area than steel columns. Current column wraps are 2’-0” by 3’-0”. It is unknown as of now what the architectural effect of using concrete columns would be.

A concrete structure requires a significantly different lateral system than the braced frame system that is being employed now. The increase in dead weight would increase seismic loading dramatically. A combination of shear walls and moment frames seems the most logical lateral system for this floor system. Joints would need to be detailed in such a way as to allow moment transfer between the columns and girders. Additionally, floor systems need to be designed to both resist and transmit lateral loads. Shear walls, provided in the core of the building, might pose problems in elevator shafts where elevator operators benefit from additional room provided between braced frame diagonal members.
**Post-Tensioned Flat Plate:**

A post-tensioned flat plate with either banded or typically arranged tendons is another cast-in-place concrete solution. This system consists of a slab of constant depth throughout the floor with post-tensioned tendons providing the necessary compressive forces in the slab (to counteract tensile force areas). Slab depths of similar spans and loading was observed at Cagley & Associates to be 9”. It is assumed that a slab of no more than 10” would be necessary.

Since there are no beams to be formed or reinforced, this system is very easy to construct. Sometimes drop panels, however, are necessary to control deflections or resist punching shear. Benefits, such as fire rating and acoustic separation, typical to heavy structural systems are present with this system.

With proper detailing, ‘lift’ slabs may be employed. These allow for forming, reinforcing, pouring, curing, and jacking of the slab to occur on the floor below. Once sufficient strength has been obtained, the slab can simply be lifted into place and secured at the proper elevation. This reduces labor by eliminating shoring and other items that are necessary to form members above surface.

Post tensioned slabs can have very little deflection in them due to the equivalent upwards force provided by the stressing tendons. Further calculations are necessary to determine actual deflections.

Flat plate structures can have the lowest floor-to-floor height. While this may not fit in with the exterior architectural scheme of this project, this system would allow for the greatest plenum depth. The
increased flexibility could also allow for a raised floor space where conduit and telecommunications equipment could be run.

This structure could provide a nearly finished surface for either the floor or the ceiling (depends upon finishing techniques and tolerances). Simply adding a carpet or a ceiling finish material might be acceptable, and would certainly save a lot of money.

Using a post-tensioned flat plate system, it might be possible to eliminate several columns, thus creating bays that were roughly 40’-0” square. Reducing the number of columns frees up rentable space as well as eliminates members. The remaining columns, however, would increase in size to account for the additional load placed upon them, and may cause architectural problems.

Another problem that can be encountered with post-tensioned tendons is rust. It is not uncommon for the PT contractor to either use a poor grout mixture, or to incorrectly add grout to the tendon sleeve, causing areas where the tendons are not protected from water by the grout. This is especially easy to do when tendons have parabolic profiles with many drape points.

Site cast concrete is significantly slower during construction that steel and precast concrete. Adding to this slower rate is that concrete contractors in the DC area typically don’t like to work on tall buildings. Furthermore, the additional construction step of jacking adds to the difficulty and length of construction. Post-tension contractors in the area should, however, be able to work above grade because many buildings downtown have PT-slabs. Other trades may take advantage of this longer duration.

Post-tensioned members are typically more efficient when higher strength concretes are used. Higher strength concrete mixes require a higher cement ratio, lower water ratio, and that the contractor exercise greater control over mixing procedures. This adds significantly to the cost of both labor and materials.

A concrete structure requires a significantly different lateral system than the braced frame system that is being employed now. Shear walls, provided in the core of the building, would probably be the optimal lateral system.

Another site-cast solution is to use post-tensioned beams spanning between the building core and the exterior of the building. Beam spacing might be something like 16’-0” O.C. Typically reinforced one-way slabs would span between the beams. Deflection of the post-tensioned beams should not be an issue due to the tendons effective upwards force.
Hollow-Core Planks:

Although concrete was originally determined to be too expensive of an option for detailed consideration, only site-cast concrete options were evaluated. As system including steel framing and precast hollow-core planks was not considered. This system takes advantage of the benefits of both structural steel and precast concrete.

Pre-tensioned, precast hollow-core planks would span 39’-10” between steel girders at the building core and exterior. The planks will bear upon the interior girders on bearing angles that are welded to the beams in the fabrication plant. They will bear upon the top beam flange on the exterior girders. Exterior steel girders are placed so that their top of steel corresponds to the height of the bearing angles on the interior girders (done for ease of construction). The planks are post-tensioned together in the transverse direction so that they act as a diaphragm. They will also have to be detailed so as to allow them to transmit lateral forces to the main lateral force resisting system. The core would remain composite steel to allow for future slab penetrations to be created with minimal effort.

Structural depth of the members would be approximately 10”, as determined from the Hollow-Core Manual. Assuming a 2” topping slab is added, the total depth left over for plenum space is 3’-8”. This is quite large, and allows for maximum efficiency in duct, pipe, and conduit run. Circular ducts, which are most efficient, could most likely be used. Additionally, this would present the possibility of
decreasing the floor-to-floor height. While decreases would not be too significant, this would slightly reduce the cost and weight of the columns and precast curtain wall elements, as well slightly reduce lateral forces.

Precast members can be constructed in near optimal conditions with more precise and accurate formwork and rebar placing. Precast members are often steam cured for optimal performance. Furthermore, bonded pre-tensioning becomes an option in addition to unbonded post-tensioning. Finally, because of there greater control employed over concrete mixtures, concrete strengths of 8000 psi are common, and do not come at such a premium as they do with site-cast concrete mixes.

Precast hollow-core planks offer superior acoustical isolation and fire rating. Only steel girders and columns would need to be fireproofed. Fire rating can be determined through the use of effective plank depths (equal to the average thickness of concrete). A 3 ½ hour fire rating would be typical for a 10” plank made with lightweight concrete. An STC value of 50 is conservative for all assemblies using an 8” hollow-core plank.

This system is highly prefabricated, and does not allow for easy retrofit to meet tenants’ needs. Pre-tensioning strands would need to be cut within the plank that is to be cut. Cuts would have to be made around the unbonded post-tensioned tendons and new members with sufficient strength and stiffness would have to be inserted. This would keep the required stress in these tendons. Then, the remainder of the plank would have to be removed. Finally, a framing scheme using structural steel would have to be employed. Needless to say, retrofit would not be much of an option.

One of two things will need to happen to the lateral system. First, the planks will need to be detailed in such a way as to allow them to transmit lateral forces and bending moments so as to act as a moment frame.

The other possibility is a total redesign of the lateral system. Keeping with the quick erection times of structural steel and pre-cast concrete, I would look into a pre-tensioned, pre-cast shear wall. The disadvantage with this is that it takes away from extra space in the core that could be used by elevator operators. Additionally, the large precast members may require the use of larger cranes. The use of external CFCC post-tensioning tendons might be something to look into. Maybe an active drift control system could be employed. A passive system of typical reinforcing and pre-tensioning controls story drift and strength. An active system of external post-tensioning tendons controls building drift.
Precast members allow for elevator operator openings to be cast into the form and detailed properly without construction error.

A similar solution, also using pre-tensioned hollow-core planks as the main structural members, uses pre-tensioned precast girders for interior and exterior members. The exterior girders can double as the precast curtain wall elements, thus reducing the number of members and simplifying curtain wall connection details. Member depth decreases from steel to post-tensioned concrete girders.

With the use of post-tensioned girders, it may be possible to remove some columns. As stated earlier, post-tensioned members can be designed to yield minimal deflections. A 40'-0” span along the exterior of the building should be feasible. Precast shop drawings should be reviewed to determine possible exterior girder sizes.
Composite Steel between Steel Girders
MATERIALS:
STEEL         ASTM A992
CONCRETE      f'c = 4000 psi
REINFORCING BAR fy = 60 ksi
STEEL DECK    2", 22 gage

Steel Joists between Steel Girders
Concrete Pan Joist between Concrete Girders
MATERIALS:

CONCRETE
- $f'c = 5000$ psi
- 145 pcf

REINFORCING BAR
- $fy = 60$ ksi

STRESSED TENDONS
- ASTM A416
- Grade 270

PT Slab
Precast Hollow-Core Plank between Steel Girders