

David N. Patariu
Resident and homeowner in the Town of Vienna
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August 15, 2018

Town Council
The Town of Vienna
Town Hall
127 Center Street S
Vienna, VA 22180
E-mail: counsel@viennava.gov

Re: Comments on the 444 Maple Avenue development

Dear Town Council,

I am a homeowner and resident of the Town of Vienna, and am writing to share my opinions¹ regarding the proposed development at 444 Maple Avenue. The Town Council should deny the 444 Maple Avenue development application for the following reasons:

1) The 444 Maple Avenue traffic impact analysis review is deficient.

The Kimley-Horn 444 Maple Avenue Multimodal Transportation Impact Analysis (TIA) Review (Exh. A) only considers the impact of the 444 Maple Avenue's addition of 160 multifamily dwelling units and 20,000 gross square foot (GSF) of retail use. It does not consider the impact of any of the several other planned developments along Maple Avenue (i.e., Vienna Market, 380 Maple Ave, Center and Maple, Rite Aid, SunTrust, and Giant Shopping Center), or their collective impact on Maple Avenue. For at least this reason, the traffic impact analysis is deficient and should be amended to consider the impact of *all* of the Maple Avenue developments on Vienna traffic patterns.

The implicit conclusion of the TIA report(s), that adding 160 household units and 20,000 ft. of retail use at 444 Maple creates less traffic than just adding retail without the households, is also odd and needs further scrutiny.

Additionally, it is important to ensure that the traffic impact analysis is performed by an independent organization chosen by the Town of Vienna. Recently Loudoun County supervisors began exploring options to take control over traffic studies that are prepared for proposed developments by developers² (emphasis added).

¹ The opinions in this letter are my own, as a Vienna resident and homeowner, and do not reflect the opinions of my employer or clients.

² "County Government Moves to Take Over Development Traffic Studies," LoudounNow, June 6, 2018.

Currently, when developers propose large projects to the Board of Supervisors, they are required to conduct studies on the expected impact of that development on traffic. Developers typically hire their own consultants to conduct those studies, and Supervisor Matthew F. Letourneau (R-Dulles), who proposed the change with Supervisor Tony R. Buffington (R-Blue Ridge), said “increasingly there have been strains on credibility” in the results.

Commenting on the need to takeover traffic studies, one Loudoun board member stated developers won’t “volunteer to add roads to the study,” that “I don’t want to call it the fox guarding the chicken coop, but in a way I think having it done independently presents a much more fair image to the public.”

Similarly, Vienna must utilize independent traffic studies which consider all of the factors that can impact our community, to avoid the “fox guarding the chicken coop” issue observed by the Loudoun County board. The Town of Vienna application approval process should not rely upon a developer conducted traffic impact analysis. For this reason alone, the 444 Maple Avenue project application should be denied.

2) Independent analysis by a licensed geotechnical engineer regarding the impact of the current 444 Maple Avenue West project is needed.

The Orr Partners letter dated August 8, 2018, evaluating concerns expressed by the Vienna community about how the proposed development at 444 Maple West might impact the water table and potentially damage, is incomplete (attached as Exh. B). On page 2 of the letter, Orr Partners concludes “[w]e do not expect impacts to the surrounding properties due to the permanent underslab drainage,” but then issues a disclaimer, that “[w]hile we have recent experience with similar projects, *we are not geotechnical engineers or geologist* and our opinions are based on our experience as developers and construction managers surrounding properties” (emphasis added). As Orr Partners tacitly points out through their disclaimer, independent analysis by geotechnical engineers or a geologist on the impact of the current 444 Maple Avenue project is needed. The Orr Partners evaluation lacks this critical input, and thus its analysis is incomplete.

Further, it appears that the author of the Orr Partners evaluation letter, Mr. Dan Strotman, is not a licensed professional engineer in the State of Virginia. Every state regulates the practice of engineering to ensure public safety by granting only Professional Engineers (PEs) the authority to sign and seal engineering plans and offer their services to the public. Only a licensed engineer may prepare, sign and seal, and submit engineering plans and drawings to a public authority for approval, or seal engineering work for public and private clients.³ On the Virginia professional license lookup page,⁴ only three

³ See <https://www.nspe.org/resources/licensure/what-pe> (last visited Aug. 15, 2018).

⁴ Virginia Department of Professional and Occupational Regulation, License Lookup (<http://www.dpor.virginia.gov/LicenseLookup/>) (last visited Aug. 15, 2018).

licensed individuals with the last name of “Strotman” appear, none of which is “Dan Strotman” from Orr Partners:

LICENSE LOOKUP

License Search Advanced License Search Disciplinary Action Search

License Number	Name	Address	License Type	Board
0402045413	STROTMAN, SHANNON D	WINDBER, PA 15963	Professional Engineer License	Board for APELSCIDLA
0632007741	STROTMAN, RYAN JAMES	GLEN ALLEN, VA 23059	Fair Housing Certificate	Fair Housing Board
2705048891	KEVIN RAY STROTMAN	9701 JEFFERSON DAVIS HGW LOT 7, RICHMOND, VA 23237	Contractor	Board for Contractors

Showing 1 to 3 of 3 entries

Mr. Strotman’s web page/bio⁵ indicates he is a construction project manager, whose “primary function as VP-Project Executive is to oversee total projects from programming to project close-out, ensuring that the schedule is on track and budget constraints are met.” Mr. Strotman’s web bio does not state that he is a licensed professional engineer, nor does it state that he has specialized experience evaluating geotechnical or geologic issues. The Town of Vienna should solicit guidance from independent, licensed engineers that specialize in geotechnical or geologic evaluations of construction projects before allowing the 444 Maple Avenue West project to proceed.

3) An updated, independent, geotechnical engineering report study is needed.

The 444 Maple Avenue project application appears to rely on a geotechnical report by D.W. Kozera, Inc. dated November 17, 2014. Section 10.4 of this geotechnical report (Exh. C) states “[a]ny deviation to the project design subsequent to the date of this report, such as changes in floor grades, building loads and building location, should be brought to our attention to determine if our recommendations contained herein remain valid.” As the 444 Maple Avenue projects plans have likely changed since 2014, an updated, independent, geotechnical engineering report study should be commissioned to ensure the recommendations D.W. Kozera, Inc. are still valid. Until that happens, the 444 Maple Avenue project application should be tabled.

⁵ Dan Strotman, Vice President, Orr Partners (<http://orrpartners.com/team-strotman.html>) (last visited Aug. 15, 2018).

4) If all of the slated projects under the MAC were allowed to proceed, the number of households in Vienna would increase by ~1000 households, from ~5,500 households to ~6,500 households.

The 2010 census indicates that the Town of Vienna had 5,528 households, and 4,215 families residing in the town. The MAC ad hoc committee’s Analysis of Current, Proposed and Potential MAC Developments chart (Exh. D, shown below) indicates that if development at the 60 dwelling units per acre amount is allowed, the number of households in Vienna would increase by ~1123 households in a very short period of time (1-3 years).

Analysis of Current, Proposed and Potential MAC Developments - Dwelling Units per Acre (DU/A)

Development/Site	Total Acres	Actual MAC Projects DU/A	Range of Dwelling Units per Acre*												
			Applied to Approved, Proposed and Potential MAC Developments												
			15 DU/A	20 DU/A	25 DU/A	30 DU/A	35 DU/A	40 DU/A	45 DU/A	50 DU/A	55 DU/A	60 DU/A	65 DU/A	70 DU/A	75 DU/A
Vienna Market (approved; 44 townhouses)	2.0	22	30	40	50	60	70	80	90	100	110	120	130	140	150
444 Maple Ave (proposed; 160 apartments)	2.8	57	42	56	70	84	98	112	126	140	154	168	182	196	210
380 Maple Ave W (potential; 45 condominiums)	.9	50	14	18	23	27	32	36	41	45	50	54	59	63	68
Center and Maple	.7	--	11	14	18	21	25	28	32	35	39	42	46	49	53
Rite Aid	1.0	--	15	20	25	30	35	40	45	50	55	60	65	70	75
SunTrust	1.6	--	24	32	40	48	56	64	72	80	88	96	104	112	120
Giant Shopping Center	10.4	--	156	208	260	312	364	416	468	520	572	624	676	728	780

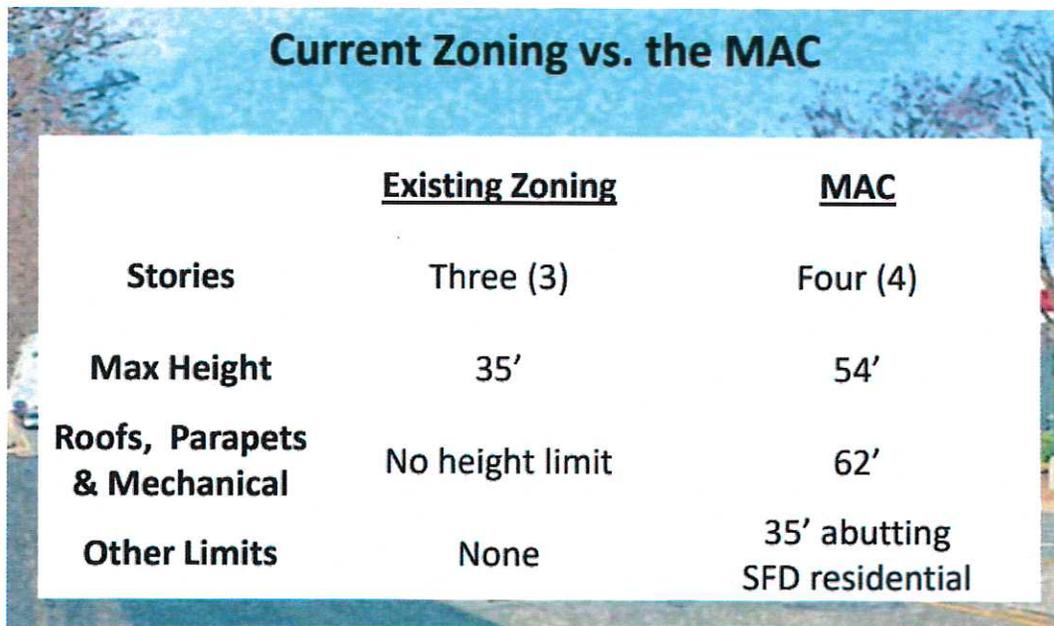
This scale of development—adding 1000 households in 1-3 years along Maple Avenue—is ill advised in light of the current heavy traffic problems along Maple Avenue. Even at the lowest 15 DU/A amount, 455 households would be added along Maple Avenue. These high numbers of additional households concentrated along Maple Avenue indicate a fundamental problem with the MAC—it fails to address town infrastructure and services demands of the rapid development allowed under the MAC.

Additionally, our schools, police, firefighting, and town services are probably not prepared for a concentrated 20-25% increase in the number of households in the Town of Vienna in such a short period of time. For this reason alone, the 444 Maple Avenue project should be delayed until the planning board can collectively consider these developments and their adverse impact on our community, and other neighboring communities like Oakton whose residents depend on egress via Maple Avenue. For this reason alone, the 444 Maple Avenue project application should be denied.

5) The 444 Maple Avenue project does not accord with the small town goals of the Maple Avenue Corridor plan.

The Town’s web site describing the MAC zone states the MAC “establishes that vision and the framework for progress by encouraging compact, pedestrian-oriented development and high-quality, eclectic architecture that is *consistent with Vienna’s small-town identity*” (emphasis added).

Historically development in Vienna has been limited to 2-3 stories (~35 ft.), which has prevented the blight of Mosaic or Tyson’s-like developments in Vienna (see Figures 1 and 2 below). The proposed 444 Maple Avenue project, whose building’s maximum height would be 62 feet, does not align with Vienna’s small-town identity.



	<u>Existing Zoning</u>	<u>MAC</u>
Stories	Three (3)	Four (4)
Max Height	35'	54'
Roofs, Parapets & Mechanical	No height limit	62'
Other Limits	None	35' abutting SFD residential

Figure 1. Slide from PC18-045 - Planning Commission - 7-30-18, page 29/44.

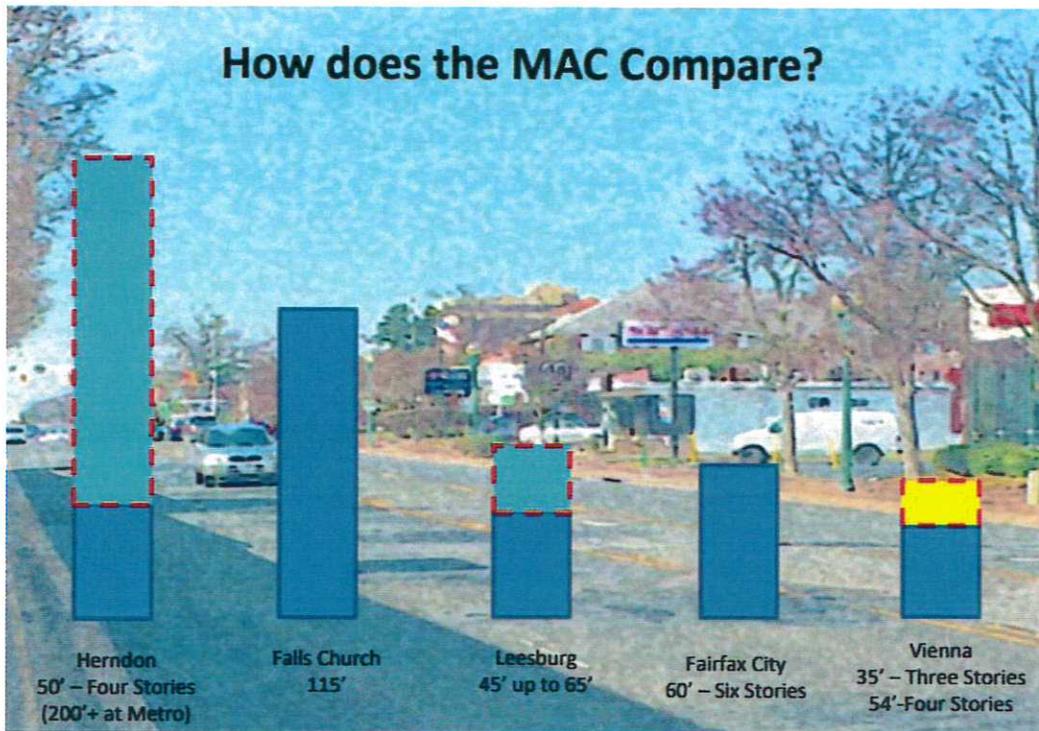


Figure 2. Slide from PC18-045 - Planning Commission - 7-30-18, page 30/44.

That a 62 foot tall building is acceptable under the MAC is inconsistent with Vienna's "small-town" identity, and indicates a serious flaw with the MAC that should be corrected. The building height analysis in Fig. 2 also shows why neighboring communities like Falls Church—which allows 115 ft. tall buildings—are having problems with over-development and deficient infrastructure. For this reason alone, the 444 Maple Avenue project application should be tabled or denied until the MAC is corrected to align with its stated "small-town" intent.

- 6) The Maple Avenue Corridor plan is currently under revision; further development should be halted until the plan is revised to reflect the current concerns of the Vienna community.**

The Maple Avenue Commercial (MAC) zone was approved by Town Council in October 2014, amended in January 2017, and is yet again undergoing additional amendments. Until the amendment process is complete, development under the plan should be postponed.

- 7) A majority of residents do not support the 444 Maple Avenue development.**

Since its announcement, the 444 Maple Avenue West development plan lacks community support, and if anything has been met with great disdain by Vienna residents.⁶ Residents have "expressed fears about an influx of traffic, vehicles cutting

⁶ "Vienna residents lambaste mixed-use proposal," Sun Gazette Newspapers (July 11, 2018).

through their neighborhoods to avoid backups, higher taxes to meet infrastructure demands, problems from the development's planned incursion into the water table and overburdening of local schools, many of which already are stretched to capacity." As noted by Council member Howard Springsteen "[a]t the Town Council meeting last night, not a single voice from any neighborhood in Vienna was raised in the support of this proposed [444 Maple Avenue] project."

Yet another indicator of the lack of support for the 444 Maple Avenue project is a petition organized by Vienna resident Steven Potter collecting signatures requesting that the Town of Vienna turn down the 444 Maple Avenue application and "send this poorly conceived design back to the drawing board." As of Aug. 15, 2018 at 10 a.m., 944 Vienna residents have signed the "turn down" petition.⁷ The competing "Support the 444 Maple Ave redevelopment effort!" petition⁸ has only received eight supporting signatures of Vienna residents as of Aug. 15, 2018 at 10 a.m.

8) The MAC needs realignment to address the concerns of the Vienna residents.

Finally, the lack of community support for development like the 444 Maple Avenue plan and its predecessors (e.g., 2016 proposal at Maple and Pleasant) should trigger a re-evaluation of the Maple Avenue development plan. The two major problems with the current plan are the 54-foot+ maximum building height allowed under the current MAC zoning ordinance (the 444 Maple Avenue West project is building's maximum height would be 62 feet), and the large number of households/dwelling units per acre allowed under the MAC/current Vienna zoning regulations.

The Maple Avenue Development Plan needs to be revised to reflect what has kept Vienna's small town feel, a maximum height allowance of 2-3 stories or 35 ft. (including rooftop mechanical devices) (see Fig 1 and 2 above), and a reduced dwelling units per acre allowed (i.e., 5-10, not the current 15-75) so that Vienna's Maple Avenue's traffic and infrastructure problems are not further exacerbated.

I appreciate your time and consideration, and thank the Town of Vienna and the Town Counsel for the opportunity to comment on the 444 Maple Avenue application and the Maple Avenue Corridor (MAC) plan.

Sincerely,



David N. Patariu

Attachments

⁷ Petition to "Turn down the 444 Maple Avenue application and send it back to the drawing board, Now!" (<https://www.thepetitionsite.com/573/034/838/turn-down-the-444-maple-avenue-application-and-send-it-back-to-the-drawing-board-now/>) (last visited Aug. 15, 2018).

⁸ Petition to "Support the 444 Maple Ave redevelopment effort!" (<https://www.thepetitionsite.com/473/986/194/support-the-444-maple-ave-redevelopment-effort/>) (last visited Aug. 15, 2018).

Exhibit A



Memorandum

To: Cindy Petkac, AICP
Director of Planning and Zoning
Town of Vienna
127 Center Street S
Vienna, VA 22180

From: David B. Samba, P.E., PTOE
Kimley-Horn

Date: August 2, 2018

Subject: 444 Maple Avenue Multimodal Transportation Impact Analysis
TIA Review

This memorandum summarizes Kimley-Horn's review of the transportation impact analysis (TIA) prepared by Wells & Associates for the 444 Maple Avenue development located in the southeast quadrant of intersection of Nutley Street NW and Maple Avenue NW. The 444 Maple Avenue TIA is dated December 6, 2017 with revisions dated February 16, 2018.

Overview

The Applicant is seeking a rezoning application for a proposed redevelopment in the Town of Vienna, Virginia. The project would redevelop the subject site with a mixed-use development including approximately 160 multifamily dwelling units and 20,000 gross square foot (GSF) of retail uses. The proposed uses would replace a 119-room hotel and a 3,500 GSF sit-down restaurant. The proposed rezoning application would convert the property from C-1 (local commercial) to MAC (Maple Avenue Commercial).

Access to the site is currently provided via a right-in/right-out driveway on Nutley Street, a right-in/right-out driveway on Maple Avenue, and a full-movement driveway at an existing, unsignalized median break on Maple Avenue. The right-in/right-out along Maple Avenue would be removed with this application.

A TIA scoping meeting was held on April 19, 2016 with representatives of the Virginia Department of Transportation (VDOT) and the Town of Vienna to discuss the parameters for the study area.

TIA Review Comments

1. The TIA appears to be consistent with industry standard procedures for conducting an analysis of the transportation impacts associated with development.
2. The TIA is generally in conformance with the scoping document/agreements contained in Appendix B. Exceptions include:

- a. Scoping document references a development program that includes 152 multifamily dwelling units, 51,190 GSF of grocery, and 5,482 GSF of additional retail.

It is noted that it is not uncommon for the development plan to change between scoping and the preparation of the TIA. It is assumed that these changes have been coordinated with the appropriate Town staff.

- b. Scoping document states that turn lane warrants would be conducted at site entrances. No turn lane warrant analyses are described in the TIA.
 - c. Scoping document states that trip generation analysis would include both the “peak hour of generator” and the “peak hour of adjacent street”. The trip generation table in Chapter 5 of the TIA does not specifically identify the trips associated with the “peak hour of generator” and the trips associated with the “peak hour of adjacent street.”
3. Trip generation was based on the application of the rates/equation from the 9th edition of the Institute of Transportation Engineers (ITE) Trip Generation Manual.

It is noted that the 10th Edition of the Trip Generation Manual was released in August/September 2017. Because the study was started in 2016, it is appropriate that 9th Edition was used for the purposes of trip generation.

For context, however, it is noted that applying the 10th Edition land use code, rates, and equations would result in differences in trip generation due to broad changes that occurred between the 9th and 10th Edition (i.e. addition/removal of specific land use codes, addition of context-sensitive trip generation rates and equations, and removal of data that originated in the 1960s and 1970s).

Key differences in the ITE Editions that are relevant to this analysis include:

- Land Use Code 220 (Apartment) has been removed from the 10th Edition. Users must now must choose between more specific land uses such as new land use code 220 (multifamily housing, low-rise), land use code 221 (multifamily housing, mid-rise), and land use code 222 (multifamily housing, high-rise), among others.
- Land Use Code 826 (Specialty Retail) has been removed from the 10th Edition. ITE recommendation is for users to apply Land Use Code 820 (shopping center) unless the specific retail use is known and that specific retail use has an applicable land use code in the 10th Edition.
- Land Use Code 820 is generally a higher peak hour trip generator than Land Use Code 826 – this means that had the applicant used ITE 10th Edition for the purposes of trip generation, the PM peak hour retail trips would have increased by about 40 percent. It is noted that this increase could be offset by a reduction in the trip generation associated with the residential portion of the development; generally, the more specific multifamily land uses of ITE 10th Edition (low-rise, mid-rise, etc.) result in less peak hour trips than the more generic “apartment” land use of ITE 9th Edition.

Based on the above, we believe the net trip generation differences to be insignificant for this particular mix of land uses and as such the use of the ITE 9th Edition is appropriate.

4. Analyses were conducted using Synchro Version 9.1. Synchro Version 10 was released in January 2017.

Because the study was started in 2016, it is appropriate that Synchro Version 9.1 was used for analysis. It is also noted that, typically, TIAs are prepared using the analysis platform that is also in use by staff; it is our experience that most Northern Virginia jurisdictions have not yet required studies to be conducted in Synchro Version 10.

Based on a review of release notes, the most significant change of Synchro Version 10 was the addition of functionality to run calculations using the methodologies outlined in Highway Capacity Manual 6 (HCM 6).

It is noted that VDOT's Traffic Operations and Safety Analysis Manual (TOSAM,) from which this study's analytical parameters are drawn, has not yet been updated to consider HCM 6. TOSAM currently suggests that studies be prepared per HCM 2000 or HCM 2010 methodologies. Both of these methodologies are contained in Synchro Version 9.1.

As such we find the use of Synchro Version 9.1 to be acceptable.

5. Generally, VDOT's TOSAM suggests that studies be conducted using HCM 2010 unless there are specific conditions that warrant the use of HCM 2000 methodologies. The applicant should state within the TIA why HCM 2000 methodologies were used.
6. It is noted that while new traffic counts were collected in March 2016 and April 2016 for most intersections, counts from January 2015 were used for the intersection of Maple Avenue and James Madison Drive.

It is noted that some jurisdictions do not accept traffic counts that are more than one year old at the time of study commencement. By contrast, VDOT is willing to accept traffic counts up to two years old.

Since the impacts of the proposed development are negligible at this intersection, this may not be a significant variance.

7. We are unable to reproduce some of the trip generation results shown in Table 5-1. The applicant should specifically state how the trips were derived (i.e., using commuter peak hour versus using the peak hour of generator, and using rates versus using equations).

Generally, we would suggest that the use of the commuter peak hour is appropriate and that the use of either the rate or equation should be determined based on ITE guidance.

8. A by-right trip generation calculation was included in Table 5-1. It demonstrates that a by-right development of a 62,780 GSF supermarket would generate more trips than the proposed program of development. In context, this means that the traffic impacts of what the applicant could achieve by-right would be more significant than the traffic impacts associated with the proposed development.

9. The level of service (LOS) results presented within the TIA are oriented around the overall intersection LOS. Minimal discussion is provided regarding the LOS of individual movements and approaches. Given that the intersection of Maple Avenue and Nutley Street already operates at LOS E, it may have been prudent to discuss specific movements or approaches to indicate how the proposed development affects the local area.
 - a. Generally, all intersections operate at the same overall LOS under 2022 conditions without the development as under 2022 conditions with the development.
 - b. The proposed development increases the overall delay at the intersection of Maple Avenue and Nutley Street by 5.0 seconds during the AM peak hour, by 2.4 seconds under the PM peak hour, and by 12.7 seconds during the Saturday midday peak hour. Because the intersection generally operates at high delays under existing conditions (and is projected to operate under higher delays in the 2022 without development scenario), the increases in delay associated with the proposed development are unlikely to greatly alter the travel patterns or perceptions of the average traveler.
 - c. Specific movements at the intersection of Maple Avenue and Nutley Street operate at worse LOS under 2022 conditions with development (westbound left during the AM and Saturday peak hours, northbound through-left during the Saturday peak hour).
10. The queuing analysis indicates that the 95th percentile queuing would exceed the available turning lengths at multiple locations under every scenario (existing, no-build, and total future).
11. The conclusions of the TIA do not present specific mitigation measures for the site's traffic impacts.

While it is recognized that overall intersection LOS is unchanged at intersections in close proximity to the site, it is common practice to identify some mitigation strategies that would address the additional delay and queuing generated by the development.

At a minimum, since the TIA identifies transportation demand management (TDM) as a mitigation strategy, some estimate of the potential trip reductions that would result from the TDM strategies should be described and backed with supporting data.

In summary, we offer the following recommendations to the Town:

1. Request that the applicant review, further expand on, and, if applicable, correct the trip generation shown in Table 5-1 (using either ITE 9th or 10th generation data).
2. Verify that the use of 2015 traffic count data is acceptable.
3. Request that the applicant provide the basis for using HCM 2000 methodologies or otherwise update the analysis to demonstrate the results under HCM 2010 methodologies.
4. Request that the applicant conduct turn lane warrant analyses at the site entrances. While we note that right turn lanes may not be desired along these streets and may be inconsistent with the Maple Avenue Commercial corridor, conducting turn lane warrant analyses could reveal potential queuing/site access issues that may need to be addressed in the future.

5. Request that the applicant provide additional details regarding traffic impact mitigation options. For example, the Town may consider requiring the applicant to participate in traffic signal or intersection configuration improvements at the intersection of Maple Avenue and Nutley Street.

Thank you for the opportunity to review the 444 Maple Avenue TIA. Please feel free to contact us at 703-674-1300 should you have any questions.

Exhibit B

August 8, 2018

Michael J. Gallagher
Director of Public Works
Town of Vienna
127 Center St. S.
Vienna, VA 22180

Re: 444 Maple Avenue, West Vienna Virginia
Letter form D. W Kozera, Inc dated February 19, 2018 P

Dear Mike,

Per your request, we have reviewed the above referenced letter. We understand that there has been concern expressed by the community about how the proposed development at 444 Maple West might impact the water table and potentially damage surrounding properties.

We have reviewed the following documents:

- 1) 444 West Maple Avenue Rezoning Plan, dated June 25, 2018,
- 2) 2) Letter prepared by D.W. Kozera Inc dated February 19. 2018.
- 3) Geotechnical Report and Logs provided on 8/1/18.

The proposed development consists of a mixed use building consisting of:

- Underground parking with a finished floor elevation of 375'-0"
- Ground level consisting of parking on structure, retail space, residential lobby and leasing, and service areas
- Three levels of residential units

Recent geotechnical investigation has noted the water table at between elevation 372.8' and 379.5'. This would place the water table slightly above the proposed floor lowest floor elevation. As Mr. Kozera notes in his letter, this will require the installation of a permeant underslab drainage system to prevent water infiltration into the underground parking level. Mr. Kozera has calculated that this underslab drainage system can expect to collect up to 5 gallons per minute of groundwater. Mr. Kozera has shown calculations that indicate that the "draw down radius" is estimated to be less then 20 feet outside the building envelope.

We do question the formula used in the calculation of the draw down radius. The "figure 5.3" included in the letter and the accompanying formula are based on a single pumping well that extends considerably below the water table. The underslab drainage system will be distributed around the entire perimeter of the building and will be relatively close to the elevation of the water table. It would be prudent to address this inconsistency with the Developer. Based on our previous experience on similar projects, it is unlikely that the permanent underslab drainage will have any noticeable effect on the existing water table.

Encl

Mr. Kozera has not, however, addressed the conditions during the construction of the building foundations. The spread footings of the building will be between 4 to 5 feet below the lowest floor elevations. Some areas, such as elevator pits, will require even deeper excavations. It will be essential to control groundwater coming into these deeper excavations during the placement of footings. **We note that there are state and local permit requirements for the discharge of storm water (which includes groundwater) during construction. The Developer needs to comply with all submissions and permits.**

There are two ways to control groundwater during construction, active well points or passive sumps.

Active well points consist of multiple wells drilled outside the perimeter of the excavation. Pumps are installed at each well and are typically piped together to a common discharge. The purpose of the wells is to lower the ground water level around the excavation to a level below the bottom of the footings. To achieve this, we would expect the amount of water removed by a well point system to be considerably higher than the 5 GPM estimated. The well points are removed once the foundations are in place and the slab on grade has been poured in the lowest level. At that point the underslab drainage system would be in place to control the water and we would expect the water table to return to the original level.

Active well points will likely lower the water table on surrounding properties, including Nutley Street and Maple Avenue, during the period the system is in use. Extraction of water from soil can cause consolidation of material that may result in settlement of adjacent structures or surface areas. While the risk of damage is slight, we would recommend that the Developer be required to provide detailed plans and calculations if an active well point system is planned.

Passive sumps are essentially large holes, either dug or drilled, strategically placed inside excavation. The bottom of the sump is lower than the adjacent foundation work. A perforated pipe is installed vertically and gravel or other permeable material is placed around the pipe to encourage water to flow to the sump. A submersible pump then moves the water to a discharge outside the excavation. In addition to the sumps there are generally separate pumps placed in foundation excavations to help control any water that does not flow to the sumps.

Because passive sumps are only gathering the water that is flowing into the excavation, there is very little impact to the surrounding properties. If a passive system is used, we would not expect any adverse impacts on the surrounding properties.

Conclusions

- Stormwater discharge permitting requirements must be followed.
- If the Developer intends to install an active dewatering system for use during construction, we recommend the Town require a more definitive submission for review.
- No further study is necessary if passive dewatering is used.
- We do not expect impacts to the surrounding properties due to the permanent underslab drainage.

While we have recent experience with similar projects, we are not geotechnical engineers or geologist and our opinions are based on our experience as developers and construction managers. Please let us know if you have any questions.

Sincerely,

ORR PARTNERS



Dan Strotman
Vice President

Exhibit C

**Geotechnical Engineering Study,
444 Maple Ave West, Vienna,
Virginia (DWK Contract Number
14107.D)**

D.W. KOZERA, INC.
PROFESSIONAL ENGINEERS & GEOLOGISTS

November 17, 2014

Vienna Development Associates, LLC
c/o Hekemian & Co., Inc.
505 Main Street
Annapolis, MD 21403

Attn: Mr. Christopher P. Bell, Senior Vice President
(cbell@hekemian.com)

Subject: Geotechnical Engineering Study, 444 Maple Avenue West, Vienna,
Virginia (DWK Contract Number 14107.D)

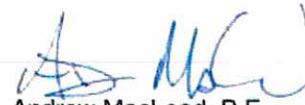
Dear Mr. Bell,

This report presents the results of our geotechnical study prepared for the proposed building to be constructed at 444 Maple Avenue in Vienna, Virginia. This report was prepared in accordance with our contract dated July 16, 2014.

We appreciate the opportunity to be of service to you and the project team. Please contact us to answer questions related to this study, should they arise.

I hereby certify that this document was prepared or approved by me, and that I am a duly licensed professional engineer under the laws of the Commonwealth of Virginia.

Very truly yours,
D.W. KOZERA, INC.



Andrew MacLeod, P.E.
Geotechnical Engineer

David W. Kozera, P.E.
Commonwealth of Virginia No. 011857
Expiration: 06-30-2015

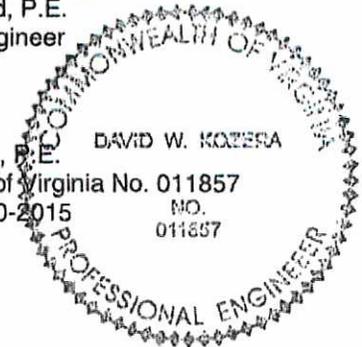


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EXECUTIVE SUMMARY

The following indicates a summary of the geotechnical findings and recommendations contained within this report.

Proposed Construction- A new seven story mixed use building is planned for the site at 444 Maple Avenue West in Vienna, Virginia. Construction is to consist of a four level concrete podium structure with three levels of residential units on top. Two of the four concrete levels are to be below grade with a lower floor elevation of EL 363. Maximum column and wall loads of 1200 kips and 9 klf, respectively, were provided by TCE & Associates, Inc.

Subsurface Conditions- The site is underlain by a thin layer of existing fill soils, which sit on top of natural residual soils formed by the weathering of the underlying bedrock.

-The groundwater elevation is estimated to be at EL 381±, which is 18 feet above the lower floor elevation.

Seismic Site Class- The site is considered a Site Class D, per IBC 2012.

Earthwork- After demolition of the existing structures, conventional earthmoving equipment is expected to be feasible for the cut to fill operations. Cuts of up to 30 feet are required to reach lower floor grades.

-The on-site soils are expected to be suitable for reuse as compacted structural fill.

-The on-site residual soils are expected to become unstable when exposed to construction activity and wet weather; therefore, an allowance should be included in the construction budget for subgrade repair.

Construction Dewatering- Extensive dewatering is required during construction to drawdown the water table beneath excavation grades. A series of perimeter ejector wells with two lines of interior wells is considered a feasible option. In addition, the installation of localized temporary sumps adjacent to footings will be required during footing excavation.

Foundation System- The project is expected to be supported on a shallow foundation system of spread footings. Foundations are to be founded on suitable natural residual soils of Stratum B. New foundations may be designed for an allowable bearing pressure of 6.0 ksf.

-Temporary sumps are expected to be required adjacent to footings during footing excavation. In addition, sumps within the footprint of the footing may also be required, depending on water flow into the excavation during construction. If neither of these are successful, individual shoring of each footing excavation may be required.

Floor Slab- The floor slab will be supported on natural soils, and may be designed using a modulus of subgrade reaction (k) of 90 pci.

-An underfloor subdrainage system is considered necessary for the floor slab of the building.

Pavements- The on-site soils have a low CBR value, and a subbase layer with a minimum thickness of 24 inches is required for the paving sections.

Construction Considerations- Geotechnical observations and testing of foundation installation and earthwork construction should be provided by D. W. Kozera, Inc., in order to verify that the assumptions and recommendations contained in this report require modifications due to unforeseen changes in the subsurface conditions.

1.0 INTRODUCTION

1.1 Purpose and Scope

This report contains the results of our geotechnical investigation and analysis performed for the proposed mixed-use building to be constructed at 444 Maple Avenue West in Vienna, Virginia. This study was conducted to characterize the subsurface conditions, and to establish engineering properties of the underlying materials in order to develop recommendations for foundation and slab design, as well as earthwork, and constructability issues related to the foundation and site.

The geotechnical investigation was performed in accordance with our proposal dated July 16, 2014 and included:

- Review of available geotechnical and geological data from our previous studies.
- Performance of 21 test borings.
- Evaluation and analysis of the subsurface data.
- Development of preliminary foundation design and construction recommendations.

Refer to our proposal for a list of specific scope items that were included in this work.

1.2 Limitations

This geotechnical study has been prepared in accordance with generally accepted geotechnical engineering practices. It is intended for the exclusive use of Vienna Development Associates, LLC for the design and construction of the building as described herein. This report includes both factual and interpreted information. Factual information is defined as objective data based on direct observations, such as soil samples and laboratory testing results. Interpreted information or geotechnical engineering interpretation is based on the engineering judgment, correlation, or extrapolation from factual information.

This report is based on information for the proposed structure that was made available to us at the time of the writing of this report. No warranties, express or implied, are intended or should be assumed. D.W. Kozera, Inc. should be allowed to review the project drawings and specifications as a continuation of our design recommendations and as a precursor to our providing geotechnical engineering services during construction. In the event that any changes in the floor grades, building loads, or structure location as described in this report are planned, the conclusions and recommendations contained herein shall not be considered valid unless D.W. Kozera, Inc. reviews the changes, and either verifies or modifies the conclusions of this report in writing.

Information contained in this report is based on data obtained from limited subsurface exploration that represents the soil conditions only at the specific location and time investigated, and only to the depth penetrated. Subsurface conditions and groundwater levels at other locations or depths may differ from conditions occurring at the investigated locations. An attempt has been made to provide for normal contingencies, but the possibility remains that unexpected conditions may be encountered during construction.

D.W. Kozera, Inc. considers construction observations and testing of the foundations and earthwork an integral part of the geotechnical design, and therefore, these services should be provided by the geotechnical engineer of record. This is necessary so that we may modify our assumptions and recommendations based on actual conditions that are exposed during construction and observed by us. We cannot assume responsibility or liability for the adequacy of our foundation recommendations if we do not observe the construction.

1.3 Site Description

The project site is located at 444 Maple Avenue West in Vienna, Virginia. The site is bordered by Maple Avenue to the north, Nutley Street to the West, residential properties to the south, and a neighboring commercial property to the east. The site is currently occupied by a hotel on the eastern portion and a restaurant on the western portion. The existing buildings are surrounded by asphalt drives and parking areas. The ground surface of the site slopes gently down from the northeast to southwest with a change in ground surface elevation from EL 394± to EL 384±.

1.4 Proposed Construction

Proposed for this project is a seven story mixed-use building which will consist of four levels of concrete podium construction and three wood-framed levels. The two lower levels will be below grade parking, with a lower floor finished floor elevation of EL 363.0. As the ground surface varies from EL 384± to EL 394±, the basement varies from approximately 20 to 30 feet below existing site grades. The maximum column loads of 1200 kips were provided by TCE & Associates, Inc. Two residential courtyards are planned above the concrete podium. Two asphalt entrance drives are also planned: one entering from Nutley Street on the south side of the building, and one entering from Maple Avenue on the east side of the building.

1.5 Adjacent Construction/Site History

The proposed building is located approximately 60 feet from adjacent buildings, and this appears to be sufficient distance such that loads from the new building will not adversely affect any adjacent structures. We have no knowledge of the site's development prior to its current use.

1.6 Regional Geology

The site is located within the Piedmont Physiographic Province. The soils below the site consist of the residual material formed by the weathering of the underlying bed rock. The bedrock is mapped as the Peters Creek Schist formation. A layer of existing fill soils, up to 7 feet thick was encountered at the surface of the soil profile. These fill soils are believed to be due to previous episodes of construction on the site.

2.0 SUBSURFACE INVESTIGATION

2.1 Test Boring Investigation

The subsurface investigation for this project was performed from September 22 to October 2, 2014. This investigation consisted of 21 test borings using hollow stem auger drilling techniques. Standard Penetration Tests (SPT) were conducted, in accordance with ASTM D-1586, at changes in strata or at intervals not exceeding 5 feet. The boring location plan and the test boring logs are included in Appendix A of this report.

2.2 Groundwater Conditions

Water was observed in the test borings during and after completion of the drilling. Temporary standpipes were set in several borings, and wells were installed at 4 locations for long term groundwater readings. Based on measurements at the test borings, temporary standpipes, and wells, the groundwater surface is estimated to lie as high as EL 381±, which is approximately 18 feet above the proposed lower floor elevation. Groundwater levels will fluctuate due to seasonal changes, precipitation, and construction activity.

2.3 Soil Laboratory Testing

Laboratory tests were conducted on a limited number of the collected soil samples. These tests consisted of:

Natural Moisture Content Tests	ASTM D2216
Atterberg Limits Test	ASTM D4318
Particle Size Analysis	ASTM D422
Moisture v. Density	VTM-1
California Bearing Ratio	VTM-8

Nine samples from the project area were tested for classification purposes, per the appropriate ASTM standards. The soils were found to classify SILT (ML). One sample was tested for Moisture v. Density and California Bearing Ratio per the appropriate Virginia Test Methods. The results of the soils laboratory testing are included in Appendix B.

2.4 In-Situ Testing

Dilatometer testing was performed at five locations on the site on August 6, 2014. The dilatometer testing was performed at an offset of approximately 5 feet from test borings B-1, B-5, B-8, B-10, and B-15. This testing was performed to probe refusal, which was encountered from depths of 40 to 50 ft beneath the ground surface. Soil modulus values resulting from these tests were used to estimate settlements of the proposed shallow foundations. The results of the dilatometer testing are included in Appendix D.

Wells were installed in four of the test borings to allow for the performance of rising head "slug" tests. The Bouwer and Rice method was utilized to determine the hydraulic conductivity of the in-situ soils. Wells were installed to the bottom depth of the test borings. Wells EX-1 and EX-4 were screened to 10 feet above the bottom and a bentonite seal was installed 1 foot above the top of screen. Wells EX-3 and EX-6 were screened to 20 feet above the bottom and no seal was installed. Table 2-1 presents the hydraulic conductivity determined from these tests. The results of the slug tests are included in Appendix E.

Well	Bottom of Well (EL)	Top of Screen (EL)	Hydraulic Conductivity (gpd/ft ²)
EX-1	360	370	4.0
EX-3	362	382	16.9
EX-4	357	367	6.4
EX-6	360	380	11.5

3.0 SUBSURFACE CONDITIONS

3.1 Stratification

The test boring data, visual and laboratory classification of the sampled soils, and our knowledge of local geology were used to separate the soils into three distinct strata, having the following generalized properties.

3.1.1 Stratum A: Man-Placed Fill

Man-placed fill was encountered in all of the test borings to depths of 1 to 7 feet below the ground surface. The fill is variable in composition and consistency, and generally consisted of silt and sandy silt. The penetration resistance in the man-placed fill indicated a generally loose to medium density, with SPT "N" values ranging from 3 to 21 blows per foot (bpf).

3.1.2 Stratum B: Residual Soils

Residual soils were encountered below the man-placed fill soils. The degree of weathering within the soil profile appears varied with the mineralogy and jointing within the profile. The residual soils are interlayered with the denser disintegrated rock of Stratum C. Although these materials generally become denser with depth, grading into disintegrated rock, these residual soils were encountered to the bottom depths of some borings, 60 feet beneath the ground surface. The residual soils were predominantly classified as SILT (ML) and SANDY SILT (ML). The density of these soils varied significantly due to degree of weathering within the profile, with SPT "N" values of 4 to 52 bpf.

3.1.3 Stratum C: Disintegrated Rock

Disintegrated rock is defined as residual material with SPT "N" values greater than 60 blows per foot. This rock like material was encountered to maximum depths of the test borings, approximately 60 feet beneath the ground surface. The disintegrated rock is interlayered within the soils of Stratum B in some of the test borings and was encountered as shallow as 30 feet beneath the ground surface.

4.0 SEISMIC CONSIDERATIONS AND SITE CLASS

This section presents the testing and analysis conducted to evaluate the liquefaction potential of the soils and to determine the 2012 International Building Code (IBC) seismic site class for this project site.

4.1 Liquefaction Potential

Liquefaction typically occurs in loose cohesionless sands located below the water table. The test borings did not encounter these conditions, and therefore a liquefaction analysis is not warranted.

4.2 IBC 2012 Seismic Site Class and Design Parameters

Seismic design parameters were determined in accordance with the 2012 International Building Code (IBC). The "U.S. Seismic Design Map Web Application" available through the USGS website provides hazard curves, uniform hazard response spectra, and design parameters for sites in the 50 states of the United States, Puerto Rico, and the U.S. Virgin Islands. These parameters were developed using two-percent probability of exceedence (PE) in 50 years. Following are the mapped spectral response acceleration values for the project site at Latitude (38.8949) and Longitude (-77.2731).

Description	Period (Sec)	S _a
Mapped Short Period Spectral Response Acceleration (S _S)	0.2	0.121 g
Mapped 1-Second Period Spectral Response Acceleration (S ₁)	1.0	0.052 g

The Seismic Site Classification influences the determination of the Site Coefficients, the Design Spectral Response Acceleration values, and ultimately the Seismic Design Category. Note that the Seismic Site Classification is based on the characteristics of the upper 100 feet of soils and rock below the site. The IBC requires the use of Standard Penetration Test Resistance (test borings), Shear Wave Velocity (geophysical methods), and/or Undrained Shear Strength (soil laboratory testing) to categorize the Seismic Site Classification.

The Seismic Site Classification was determined to be Site Class D based on the test borings performed for this study. For a Site Class D, with the above-indicated mapped spectral acceleration values, the following are the calculated Site Coefficient values and the Maximum and Design Spectral Response Acceleration values, per IBC Section 1613.5.

Site Class	D
Soil Profile	Stiff Soil Profile
Site Coefficient (F _a)	1.6
Site Coefficient (F _v)	2.4
Short Period, Maximum Spectral Response Acceleration (S _{MS})	0.193 g
1.0 Second Period, Maximum Spectral Response Acceleration (S _{M1})	0.124 g
Short Period, Design Spectral Response Acceleration (S _{DS})	0.129 g
1.0 Second Period, Design Spectral Response Acceleration (S _{D1})	0.083 g

The Design Spectral Response Acceleration values are to be used with the Occupancy Category (IBC 2012, Table 1604.5) of the building or structure to determine the Seismic Design Category. Complete results of Spectral Acceleration with varying period are given in Appendix C.

5.0 EARTHWORK

5.1 Discussion

Cuts and fills of up to 30 feet are anticipated to reach proposed lower level floor grades. Excavation will extend beneath the groundwater table and an extensive dewatering system is required for construction of the lower levels. The southern portion of the site is expected to be partially laid back for construction of the lower levels; therefore, significant backfilling along the southern portion of the site is expected. Minimal cuts and fills for grading purposes are expected around the other three sides of the proposed building. The majority of on-site soils are expected to be suitable for re-use as backfill; however, it is expected that there will be limited space for stockpiling, and offsite borrow may be required at the time of backfilling.

Careful subgrade preparation, including stripping of existing asphalt, organic layers, and/or soft surface soils, is required to prepare a suitable fill subgrade. Earthwork is recommended to take place in the warmer, drier months between May and October.

5.2 Excavation Characteristics

After demolition of the existing buildings, the excavation of this site is expected to be performed using conventional earthmoving equipment. Careful preparation of fill subgrades, proper placement and compaction of structural fill and backfill are both necessary to prepare a suitable site for the support of the proposed structures. Details of these requirements are included in the following sections.

5.3 Fill Subgrade Preparation

The fill subgrades should be proofrolled to assure that all unsuitable, soft and loose soils have been removed from below the building and pavement areas. During proofrolling, the subgrades should be observed by the geotechnical engineer of record. Any unsuitable soils that are observed to be excessively settling or pumping during proofrolling, should be removed down to firm soils and then replaced with satisfactory soil materials compacted in accordance with the project specifications.

Care must be taken to protect fill subgrades during construction. The on-site, residual soils will become unstable in wet weather and under construction traffic. Undercutting of floor, fill, and pavement subgrades should be expected if the subgrades are exposed to the above events. The Project Specifications should require the contractor be responsible for protecting the subgrades from weather and equipment damage. An allowance should be included in the construction budget for repair of subgrades.

5.4 Compacted Structural Fill

Compacted structural fill and backfill should consist of satisfactory soils classified as ML or better in accordance with the Unified Soil Classification System, ASTM D2487. Soils meeting this requirement are classified as ML, SM, SP, SW, GM, GP, and GW. The majority of on-site soils are expected to meet this requirement and may be used in fills. Unsatisfactory soils are those classified as CL, OL, OH, MH, and CH. SC and GC materials may be suitable depending on plasticity. Suitable material must have a Plasticity Index (PI) of less than 10.

Soils used for compacted fill should be free of unsuitable materials such as topsoil, other organics, rubble, and rocks larger than three inches in diameter. The in-place moisture content of the satisfactory soils material shall be adjusted by the contractor, through wetting or drying, to within three percent of the optimum moisture content. Additives such as quick lime, kiln dust, fly ash, or Portland cement may be useful in expediting fill operations.

Compacted fill should be placed on subgrades which have first been stripped of vegetation, existing asphalt, topsoil, and unsuitable soft areas. Compacted structural fill should be placed in approximately horizontal layers, each layer having a loose thickness of not more than eight inches. All structural fill in the

building and pavement areas should be compacted to 95 percent of the maximum dry density in accordance with VTM-1. Structural fill placed in the top 24 inches beneath pavements should be compacted to 100 percent of this same standard, while compacted structural fill in lawns and unpaved areas may be compacted to 90 percent of the same standard, when placed as indicated above. The contractor should select appropriate compaction equipment to achieve the required compaction.

5.5 Dewatering During Construction

Depending upon the depths of the foundations, the groundwater table is expected to be 20 to 25 feet above the bottom of the excavation required for foundation construction. A comprehensive dewatering system will be required to keep the groundwater level below the bottom of footing elevations during construction. A series of perimeter ejector wells with two lines of interior ejector wells are expected to be suitable for dewatering to maintain the water surface below the excavation subgrade. Due to the relatively low hydraulic conductivity of the soils beneath this site, ejector well spacing of approximately 5 to 10 feet is anticipated. A wellpoint system may also be considered, but a two-stage wellpoint system with wellpoint spacing of 3 to 5 feet will likely be required if well points in lieu of ejector wells are used. If wells cannot be installed outside of the excavation due to property line constraints, battered wellpoints may be installed within the excavation. The dewatering system is expected to take approximately four weeks to reach the required drawdown of 25 feet.

Even with the extensive dewatering system in place, the excavation is expected to weep. Individual footing excavations will require their own temporary sumps placed lower than the foundation subgrade in order to provide a stable subgrade suitable for footing placement. Additional temporary sumps are also expected in other areas throughout the excavation where the bottom of excavation is observed to be soft or weeping during construction. The dewatering contract shall provide the design for the dewatering system. The contractor shall determine the number of ejector wells, wellpoint stages, sumps, and spacing required to achieve a sufficient level of dewatering.

The dewatering contractor shall also determine the drawdown near adjacent structures. If the drawdown beneath adjacent structures exceeds that which would cause intolerable settlements of the structures, the need for a cut-off wall around the site should be evaluated. The nearest existing building appears to be approximately 60 feet from the proposed building, and from our preliminary analysis, it appears that drawdown beneath this structure would be less than 5 feet. Settlements due to this amount of drawdown are expected to be less than 0.25 inches; however, this will depend on the final design of the dewatering system. The dewatering contractor may install a monitoring well adjacent to this structure to measure the drawdown at this point during the dewatering operation.

6.0 FOUNDATION DESIGN

6.1 Summary

We have conducted our geotechnical analysis for foundation design based on the results of the test borings, dilatometer probes, laboratory tests, and our experience with similar geologic conditions. The maximum column and wall loads for the building are expected to be 1200 kips and 9 klf, respectively. Typical interior column loads are expected to range from 850 to 1000 kips, while the typical exterior column load is 500 kips. The structural loading information was provided by TCE & Associates, Inc. Conventional spread footings founded on suitable residual soils are feasible to support these loads.

6.2 Spread Footings on Natural Soil

An allowable soil bearing pressure of 6.0 ksf is recommended for footings founded on suitable natural soils of Stratum B. Based on the test borings performed in the building area and a proposed finished floor elevation of EL 363, these suitable soils are expected to be encountered at foundation grades. Extensive dewatering is required to drawdown the water table beneath foundation grades. It is expected that temporary sumps will be required adjacent to individual footings to sufficiently dewater the footing excavation. Depending on the size of the footing and the rate of water inflow, additional sumps may be required within the footprint of the footing. It is expected that these sumps will be 12 inches below bottom of footing elevation. All sumps must be lined with filter fabric to prevent the migration of fine material from beneath the footing. These temporary sumps may be filled with stone and/or concrete during the pouring of the footing. It is imperative that footings be excavated and concrete placed in the same day in order to minimize disturbance to the footing subgrades.

During excavation for footings, some unsuitable, soft or loose natural soils or existing fill soils may be encountered, which must be removed by undercutting. If undercutting is required, sumps may also be required to be lowered. If the groundwater cannot be lowered below the footing subgrades, individual footing shoring may be required. The footing should be placed directly on the lowered subgrade. Alternatively, the undercut subgrade may be backfilled with lean concrete, allowing the footing to be placed at the designed elevation.

All footings should be at least 16 inches wide for shear considerations and a maximum slope of 2H:1V should be maintained between the bottom edges of adjacent footings where foundation grades are at different levels. Hand cleaning of the footing subgrades will be required to remove disturbed soils. This may occur from the backhoe excavation, ponding of water, trench collapse, etc. All footings should be placed at least 30 inches below final exterior grade for frost protection.

We expect that total foundation settlement of less than one inch and distortional settlement of 0.002 in/in can be expected when spread footings are placed in accordance with the recommendations provided in this report. The suitability of footing subgrades should be evaluated during construction by a geotechnical engineer from our office.

6.3 Spread Footings - Lightly Loaded Structures

We expect that some lightly loaded structures (wall loads of less than 3 klf and column loads of less than 50 kips) founded at nominal depths below existing grade may be required at the site. These structures may include small site walls, equipment foundations, etc. These lightly loaded structures may be founded on spread footings. The fill material found in Stratum A may not have been placed in a controlled manner; therefore, we believe that these existing fill soils are not suitable for direct support of footings. Where the fill soils of Stratum A or soft natural soils are encountered, the lightly loaded footings should be undercut to a minimum depth of 1B for square footings and 2B for strip footings, where B is equal to the footing width. If suitable natural soils are encountered prior to reaching the minimum depth indicated above, the undercut may be stopped. The resulting undercut excavation should be backfilled with new compacted structural fill. A design soil bearing pressure of 2.0 ksf is recommended for these lightly loaded footings founded on

natural soils or newly placed compacted structural fill when installed as described herein. Toe pressures for retaining walls should not exceed 3.0 ksf.

Undercutting of unsuitable existing fill soils should be expected where footings are founded near the existing ground surface. Unit rates and an allowance should be established for undercutting of unsuitable soil. The undercut and backfill should be performed in accordance with the earthwork recommendations in Section 5 of this report.

Strip footings should be at least 16 inches wide and column footings at least 30 inches for shear considerations. A maximum slope of 1.5H:1V should be maintained between the bottom edges of footings at different levels. Exterior footings and interior footings in unheated areas should be placed at least 30-inches below grade for frost protection.

Based on wall loads of less than 3 klf and column loads less than 50 kips, we estimate total settlements of less than one inch. Where spread footings are founded adjacent to below grade walls or within a slope of 1H:1V, the walls should be structurally evaluated for the ability to withstand the surcharge from the new footings.

7.0 FLOOR SLAB

7.1 Floor Slab Support

The floor slab subgrades are expected to consist of natural soils. Prior to placement of the floor slabs, the suitability of the slab subgrades should be determined by proofrolling. Proofrolling should be performed using a loaded 20 ton dump truck (or equivalent heavy-construction equipment) under the observation of a geotechnical engineer from our office. Any additional loose or unsuitable soils found during proofrolling should be removed and replaced with compacted fill. A stone working platform may be required to support heavy equipment during construction. The slab may be placed directly on this stone working platform if it is found to be stable during proofrolling.

As discussed in Section 5.5 of this report, an extensive dewatering system is required for construction of the lower levels. Due to the relatively low hydraulic conductivity of the soils, it is anticipated the bottom of excavation will weep and may become soft during construction. Temporary sumps will be required where this occurs in order to provide a suitable subgrade for slab support. The placement of a stone working platform underlain by a geotextile may be required to maintain a stable subgrade.

Floor slabs-on-grade may be designed using a modulus of subgrade reaction, k , equal to 90 pci. Groundwater is estimated to be up to 18 feet above the proposed finished floor grade of the lower level, and a special underfloor subdrainage system is considered necessary; see below. Included above the subdrainage system should be a vapor barrier or waterproofing membrane between the concrete floor slab and the gravel. The subdrainage system should be placed shortly before slab construction to minimize damage to the piping from construction operations.

7.2 Subdrainage System

The lower floor slab is expected to be at EL 363, which is approximately 18 feet below groundwater table. A subdrainage system designed to collect groundwater around the perimeter walls and below the floor slab of the structure is required to maintain groundwater below the floor level.

The use of both a waterproofing system and underfloor subdrainage system is recommended. The below grade walls and floor need not be designed for hydrostatic water pressure when subdrainage is installed as detailed herein; however, walls below grade and slabs-on-grade must be waterproofed.

General requirements of the drainage system are as follows:

The subdrainage system should consist of four-inch diameter, perforated, corrugated polyethylene tubing according to ASTM F405 with a maximum slot width of $\frac{1}{4}$ inch. Tubing should be placed with slots down using straight section and standard available connections.

Drainage lines may be placed without a slope, with inverts at least six inches below final floor grades. The subdrainage system may drain by gravity to daylight or to a storm drain line, provided that provisions are made to avoid back pressures from acting in the event storm sewers flow full. A maximum spacing of 30 feet between subdrainage lines below the floor slab should be maintained.

The use of a true waterproofing membrane such as Paraseal or Tegraseal, which are Bentonite/HPDE composites, is recommended directly below the floor slab. This membrane can be placed directly on the washed gravel drainage layer. All penetration and seals should be performed in accordance with the manufacturer's recommendations. This waterproofing membrane is to provide a seal that will minimize moisture vapor transmission through the floor slab.

A uniformly graded stone filter, (washed gravel), should be placed around the perforated drainage line. This stone filter should have a thickness of at least six inches at the bottom and sides. The drainage invert should be at least 12 inches below the slab subgrade. The stone filter should have uniform gradation and

AASHTO M43, Size No. 67 or 7 is recommended. The stone drainage filter should also be wrapped in geotextile. The underfloor washed gravel should be a minimum of 6 inches thick, and be underlain by a continuous layer of geotextile. The geotextile shall have an apparent opening size of greater than an equivalent opening size of the No. 70 sieve. (Mirafi 140N or equal).

The below grade walls should be constructed with a vertical drainage panel, which extends to within two feet of the ground surface, and with a gravel layer at the base of the exterior walls. The gravel layer should consist of open-graded washed gravel such as AASHTO M43, Size No. 67 or 7. Weep holes should consist of four inch PVC pipe packed with the same pea gravel and connected to the six inch layer of washed gravel or crushed stone below the floor slab. Weep holes if used in lieu of a perimeter drain should be spaced a maximum of six-feet on center. Typical subdrainage details are included in this report as Figure 7-1. Weepholes are unnecessary if the perimeter drain system is discharged independently of the underfloor system.

It should be noted that inspection of the subdrainage system should occur and the system may require flushing at periodic intervals if soil particles infiltrate the pipes. Clean-outs should be incorporated into the subdrainage system after each right angle bend to allow flushing of the system.

A design memorandum, dated October 17, 2014 discusses the expected discharge flows from the subdrainage system. Flow quantities ranging from 120 to 445 gpm were predicted. This design memorandum has been included as Appendix F of this report. Pump sizes may be selected based on the predicted flow quantities. A redundant power source, such as a backup generator, must be connected to the pumps so that the pumps do not stop running in the event of failure of the primary power source.

SECTION 8: LATERAL EARTH PRESSURES

8.1 Discussion of Excavation Support Systems

Based on a finished floor elevation of EL 363 for the lower level and a ground surface elevation of EL 384± to 394±, excavations of 20 to 30 feet are anticipated for the for this project. The retention of these excavations can be done by a variety of excavation support systems, but we recommend that a free-draining soldier beam and wood lagging wall be employed. This soldier pile wall may be cantilevered, braced with internal rakers, anchored with earth tiebacks, or a combination of the above. A soldier pile wall will require that piles be embedded into the dense residual soils, which may prove difficult to penetrate using vibratory or driven methods; therefore, pre-drilling will be required. Soldier pile shafts should be grouted during auger removal to prevent caving of the shaft. The soldier pile can then be inserted into the drilled, pre-grouted shaft.

Soldier pile spacing should be a maximum of eight feet, and the wood lagging should not be less than three inches thick, unfinished. The location of the wales, soldier beams, spacing, and embedment depth must be based on an analysis provided by the excavation support contractor. It is expected that braced excavation support is required to support the excavations near property lines. Tiebacks or internal rakers may be used for bracing. Typical arrangements of tiebacks or rakers are shown on Figure 8-1. Struts or tiebacks will require preloading in order to minimize wall movement. We expect that if tiebacks will be used, they will extend below the public rights of way, and would require easements. Research of the existing utilities and other below grade structures should be performed to verify that these structures will not interfere with the tiebacks. If easements cannot be obtained, internal rakers will be required.

A precondition survey of adjacent buildings, utilities, and structures should be performed prior to the start of work. The precondition survey should document the condition of the adjacent buildings/structures prior to construction. The adjacent buildings should be monitored for horizontal and vertical movement during construction, and this monitoring program should be required by the project specifications to be submitted and reviewed prior to construction. All foundations located within a 2H:1V slope of the lowest excavation level should be monitored for movement.

Any vertical surcharge load from temporary construction equipment should be added to the lateral earth pressure with a rectangular force diagram with an active earth pressure coefficient of 0.5 times the vertical surcharge load. The surcharge load from construction equipment should be a minimum of 250 psf.

Excavation is expected extend beneath the water table, and it is expected that a comprehensive dewatering system will be installed prior to excavation beneath the groundwater level, EL 381. Even with the dewatering system installed, it is expected that there will be some seepage through the walls of the excavation support system. Lagging beneath EL 381 should be stuffed with straw or filter fabric in order to prevent the migration of fine material into the excavation. If allowed to occur, this migration will cause settlements of the ground surface outside of the excavation.

The excavation support systems should also be designed in such a way that an acceptable lateral deformation of the excavation support systems and settlement of adjacent structures and utilities less than 1/4-inch and 1/2-inch, respectively, can be achieved.

All excavation support systems should be designed by a registered professional engineer. A submittal of the engineered system(s) including shop drawings and design computations should be reviewed by the structural engineer and D.W. Kozera, Inc. for approval prior to construction.

8.2 Temporary Sloped Excavations

Sloped excavation may be used on site where excavation depth is shallow, the extent of excavation is small, and ground movements as a result of excavation would not impact the performance of existing structures. It is expected that a sloped excavation may be utilized in the southern portion of the site, in combination with an excavation support system. The Occupational Safety and Health Administration

(OSHA) Technical Manual (OTM) Section V, Chapter 2, "Excavations: Hazard Recognition in Trenching and Shoring," provides guidelines that should be followed to ensure a safe excavation. The soils found on site can be considered Type C in accordance with the OTM; therefore, excavations of up to 20 feet deep with slopes of 1.5H:1V are permitted. All sloped faces should be covered with polyethylene sheeting to minimize the wetting and/or drying of the soils on the slope. Sloped excavations below the groundwater table are not recommended, and mechanical excavation support systems should be used if excavations extend below the groundwater table, EL 381. The construction dewatering system will lower the water table and temporary sloped excavations may be permitted below EL 381 in the dewatered zone.

8.3 Cantilevered/Braced Walls

The lower level is expected to be 20 to 30 feet below final exterior grade. As such, the perimeter walls are required to resist lateral earth pressure loads. Recommended parameters for cantilevered and braced retaining wall design are indicated on Figures 8-2 and 8-3, respectively. The ponding of precipitation behind the walls should be avoided during construction as the pressure diagrams included do not include hydrostatic pressure. A Factor of Safety of at least 1.5 should be used for evaluation of overturning and sliding of the walls using the parameters indicated on the lateral earth pressure diagrams.

All walls below grade, including those poured directly against sheeting and shoring, should be waterproofed and include a synthetic drainage panel to allow water against the wall to flow into the perimeter foundation drain. These walls will extend beneath the groundwater table and the lateral earth pressure diagrams do not include hydrostatic pressure. Conventional foundation subdrainage or weep holes must be used to prevent buildup of hydrostatic pressure behind walls. Typical subdrainage details are included in Figures 7-1 and 7-2.

Specific material and compaction requirements for fill against walls below grade are included in Section 5.4. Compacted fill behind and in front of the walls should be free of organics and rocks larger than three inches in diameter and should consist of soils classifying ML or coarser. Most of the on-site soils are expected to meet these criteria. Compaction equipment exceeding 3,000 pounds in dead weight should not be used within five feet of the walls in order to avoid overloading the walls. All building walls should be braced prior to backfilling, unless they are designed to be cantilevered walls.

SECTION 9: PAVEMENT DESIGN

9.1 Discussion

Proposed pavement construction includes new entrance drives from Maple Avenue and Nutley Street, as well as a delivery area to the south of the proposed building. It is suspected that all pavements will be subject to delivery truck loading, and a heavy-duty asphalt section has been provided. Pavement subgrade elevations are expected to be near existing grades and require cuts and fills of less than three feet. Therefore pavement subgrades are expected consist of existing fill soils, newly placed compacted fill, and/or natural soil. Depending on the support of excavation scheme utilized along the south wall, the southern pavement area may be supported on significant depths of newly placed fill.

9.2 Pavement Subgrade Preparation

Careful subgrade preparation, including the stripping of existing asphalt, topsoil and organic layers, and re-compaction and proofrolling, is strongly recommended. All subgrades should be proofrolled with a loaded 20 ton dump truck, and any unsuitable soft or loose areas detected should be removed and replaced with satisfactory compacted fill or stone base course. Some undercutting of unsuitable soils should be expected, in order to produce a pavement subgrade suitable for the pavement design provided below. The residual soils are expected to become unstable under construction activity and if they are exposed to wet weather. It is recommended that the subbase course be placed immediately after proofrolling in order to protect the subgrades. Compacted fill placed for pavement support should be placed in accordance with the recommendations contained in Section 5.4 of this report. The pavement subgrades should be observed carefully to determine that the soils meet the soil classification that is used in our analysis.

9.3 Design/Analysis

Pavements are expected to be subject to heavy loads of delivery trucks and a heavy-duty pavement section may be used for new pavements. Soil laboratory testing for the on-site soils indicated a CBR value of 1.0 for the SANDY SILT soils. The CBR value is generally considered as "poor" for pavement support and a granular subbase with a minimum thickness of 24 inches is recommended to stabilize the subgrades. We expect these soils to make up the majority of pavement subgrades. The laboratory CBR value was reduced by a factor of 0.67 to yield a design CBR value of 0.67. This design CBR value was used for our analysis.

Table 9-1: Recommended Heavy-Duty Pavement Section
500,000 ESALs
2.0 Inches Asphalt Surface Course
4.0 Inches Asphalt Base Course
8.0 Inches Stone Base Course
24.0 Inches Granular Subbase Course

The bituminous concrete surface and base course material should be selected by the civil engineer to provide a stable and relatively impervious pavement section. The stone base and granular subbase courses should meet the specifications of VDOT #21A or #21B, and be compacted to at least 100 percent of the maximum dry density per VTM-1.

9.4 Pavements During Construction

The recommended pavement sections are not designed to accommodate construction traffic. It should be expected that damage will occur due to overloading of the pavement sections if they are subjected to construction traffic. This will be prevalent especially if water is allowed to collect on or in the pavement subgrades, and if only the base course is placed prior to the completion of the construction. Provisions should be made to minimize damage to the pavements during construction including the use of subdrainage, temporary swales or berms, the limitation of construction traffic to certain areas, and/or an

increased thickness of stone or base asphalt. An allowance should be reserved for the cost of repairs to the base paving prior to completion of the final surface-course of asphalt.

The pavement design assumes that a continual maintenance program will be implemented during the service life of the project. This should include crack and surface sealing, and patching of deteriorated areas.

9.5 Rigid Pavement

A rigid concrete pavement should be used in areas of concentrated, repeated, heavy wheel loads such as at the trash dumpster area or loading dock area. The recommended rigid section is 7.0-inches of reinforced Portland cement concrete. The concrete should be 4,000 psi, air entrained, and be reinforced with a minimum 6x6, W2.9xW2.9, welded wire fabric. Construction and expansion joints should be doweled at 12 o.c., with 7/8-inch diameter, 18-inch long smooth dowels. Joint spacing should not exceed 14-feet in any horizontal direction. Subgrades for rigid pavement should be prepared in the same manner as those for heavy-duty flexible pavements, which includes both the 24 inch subbase, and 8 inch stone base courses.

10.0 CONSTRUCTION CONSIDERATIONS

Specific recommendations for foundation construction are given below:

10.1 Earthwork

The site surface asphalt should be stripped, as well as all existing topsoil from landscape areas and soft surface soils. The resulting subgrades should be proofrolled under the observation of our representative. Any soft or unsuitable soils encountered should be removed and replaced with compacted structural fill. Excavations are expected to extend up to approximately 25 feet beneath the water table and a comprehensive dewatering system is required.

10.2 Spread Footings

Care should be exercised during the excavation for all footings to minimize disturbance of the footing and fill subgrades. If subgrades are disturbed, the subgrades should be lowered to undisturbed soils. Due to the susceptibility of the on-site soils to damages from weather and/or construction activity, footings should be excavated and concreted the same day in order to avoid ponding of surface runoff water in footing excavations, and to avoid other disturbances such as freezing, extreme moisture variations (wetting or drying), etc. A mud mat consisting of a minimum of two inches of lean concrete may be placed to preserve the subgrades after the subgrade is approved by an engineer from our office. Hand cleaning of the disturbed soils left by the backhoe excavation will be required to produce a minimally disturbed subgrade. A flat-bladed excavation bucket will help to minimize the hand work.

10.3 Compacted Structural Fill

Compacted fill should meet the requirements outlined in this report. All compacted structural fill and backfill below slabs, and as backfill behind walls should be compacted to 95 percent of the maximum dry density per VTM-1. Compacted fill placed as subbase and base courses directly beneath pavements shall be compacted to 100% of the same standard. Moisture conditioning, such as wetting or drying, should be expected to be required depending on the time of year construction occurs. Soil additives such as lime, cement or kiln dust may be used to expedite compaction in soils above the optimum moisture for compaction.

10.4 Review of Construction Documents

Any deviation to the project design subsequent to the date of this report, such as changes in floor grades, building loads and building location, should be brought to our attention to determine if our recommendations contained herein remain valid.

The information provided in this report may be used to produce project drawings and specifications. We should be allowed to review the project drawings and specifications, as a follow-up to our design recommendations and as a precursor to our providing the geotechnical engineering services during construction.

10.5 Construction Observations and Testing

Regardless of the thoroughness of a geotechnical engineering exploration, there is always a possibility that conditions will vary from those encountered in the test borings, that conditions are not as anticipated by the designers, or that the construction process has altered the soil conditions. D.W. Kozera, Inc. considers construction observation and testing of the foundations and earthwork an integral part of the geotechnical design, and therefore these services should be provided by the geotechnical engineer of record. As actual subsurface conditions are exposed and observed by us during construction, modifications to our report recommendations can be made promptly and efficiently as needed.

Observations and testing should at minimum include full-time observations of the excavation of footing, fill, and floor subgrades, and field density testing of compacted structural fill. Other services, including

materials testing (concrete, reinforcing steel, bituminous concrete, masonry, etc) can be provided upon request.

Exhibit D

MAC ad hoc committee

Analysis of Current, Proposed and Potential MAC Developments - Dwelling Units per Acre (DU/A)

Development/Site	Total Acres	Actual MAC Projects DU/A	Range of Dwelling Units per Acre*													
			Applied to Approved, Proposed and Potential MAC Developments													
			15 DU/A	20 DU/A	25 DU/A	30 DU/A	35 DU/A	40 DU/A	45 DU/A	50 DU/A	55 DU/A	60 DU/A	65 DU/A	70 DU/A	75 DU/A	
Vienna Market (approved; 44 townhouses)	2.0	22	30	40	50	60	70	80	90	100	110	120	130	140	150	
444 Maple Ave (proposed; 160 apartments)	2.8	57	42	56	70	84	98	112	126	140	154	168	182	196	210	
380 Maple Ave W (potential; 45 condominiums)	.9	50	14	18	23	27	32	36	41	45	50	54	59	63	68	
Center and Maple	.7	--	11	14	18	21	25	28	32	35	39	42	46	49	53	
Rite Aid	1.0	--	15	20	25	30	35	40	45	50	55	60	65	70	75	
SunTrust	1.6	--	24	32	40	48	56	64	72	80	88	96	104	112	120	
Giant Shopping Center	10.4	--	156	208	260	312	364	416	468	520	572	624	676	728	780	

*Table includes a range of dwelling units per acre for medium density residential developments based on information found in the following reports: "Creating Great Neighborhoods: Density in Your Community" by the Local Government Commission in cooperation with U.S. EPA, "Higher-Density Development: Myth and Fact" and "Yes in My Backyard: How States and Local Communities Can Find Common Ground in Expanding Housing Choice and Opportunity" both by the Urban Land Institute, and "Visualizing Density" by the Lincoln Institute of Land Policy, as well as best practices of local jurisdictions.