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8 July 2011

Mr. Scott Unger
Executive Director
Allentown Economic Development Corporation
435 West Hamilton Street, No. 331
Allentown, PA 18101

Re: Preliminary Geotechnical Engineering Study
Allentown Arena
Allentown, Pennsylvania
Langan Project No.: 240022301

Dear Mr. Unger:

This letter report presents the results of Langan Engineering and Environmental Services, (Langan's) preliminary geotechnical engineering study for the proposed Allentown Arena in center-city Allentown, Pennsylvania. The study was performed for the Allentown Economic Development Corporation. The purpose of our geotechnical engineering study was to provide preliminary site-specific subsurface information with the potential of identifying subsurface liabilities (e.g., miscellaneous fills, soft silty/clayey layers, known karst hazards, and depth/condition of bedrock) at the due-diligence stage that may adversely affect the site's future development. A supplemental, structure-specific geotechnical engineering study will be needed once the proposed site layout and grading is finalized. This report summarizes our findings and presents preliminary geotechnical engineering concerns and impacts associated with site redevelopment.

Site Description and Existing Conditions

The project site of our preliminary geotechnical subsurface investigation occupies two city blocks of Allentown. These blocks are bounded by Linden Street to the north, Hamilton Street to the south, 6th Street to the east, and 8th Street to the west. A series of single-lane alleys crisscrosses the site in a north-south and east-west alignment. The location of the site is shown on the attached Figure 1. The site is fully developed and is occupied by a combination of at-grade parking lots, 2- to 3-story residences, and multiple-story residential and commercial buildings, many of which have full basements.

Of particular interest is the existing at-grade parking lot located just northwest of the intersection of Hamilton Street and 7th Street, which is the site of the former Corporate Plaza

office building. This former development is the site of a well-documented, very large sinkhole that occurred beneath the former office tower, resulting in substantial settlement of structural columns leading to irreparable structural distress, its condemnation and subsequent demolition. It is important to note that this sinkhole was filled with grout and also that the site was backfilled with demolition debris after the building was razed. Langan prepared a geologic and geotechnical evaluation for this sinkhole event in our report dated 21 December 1994. A synopsis of this report is presented in the 'Previous Geotechnical Investigations' section below.

Proposed Construction

The proposed development will involve the construction of an approximately 8,500-seat arena consisting of an event level, concourse level, lower suite level, upper suite level/press level, low roof, catwalk/rigging level and high roof level. An arena loading dock will be located on the northwest corner of the proposed arena, featuring a concrete cantilever retaining wall up to 16 feet high.

Anticipated column loads for the arena are 1,100 kips for the columns supporting roof trusses, and 500 kips at typical interior columns.

It is our understanding that the proposed arena will be located on the city block between 7th and 8th Streets between Linden and Hamilton Streets. As such, it overlaps the limits of the former Corporate Plaza development, now the large at-grade parking lot, that was the site of the massive sinkhole discussed above.

Local Geology

According to the Atlas of Preliminary Geologic Quadrangle Maps of Pennsylvania, 1981, Allentown East quadrangle, the site of the proposed arena development is underlain by the Allentown Formation. This bedrock formation is composed of laminated, medium grey dolomite and impure limestone, which are both carbonate rocks that are categorized as karst.

Karst is a type of geologic formation characterized by carbonate bedrock such as limestone or dolomite that is susceptible to dissolution when exposed to mildly acidic groundwater. Over time, the dissolving bedrock creates the features common to karst topography, including an irregular, pinnacled bedrock surface that is often highly fractured and ground subsidence in the form of sinkholes, or dolines. Karst topography presents a unique set of challenges to site development, particularly from a geotechnical engineering perspective.

Previous Geotechnical Investigations

A series of geotechnical subsurface investigations have been completed at the site throughout its development history. Prior to Langan's current subsurface investigation, the field investigations have focused on identifying the subsurface conditions beneath the former Corporate Plaza development, both before and after it existed. A brief summary of the previous geotechnical investigations performed by others is provided in the subsequent sections.

F&M Associates Investigation

Prior to the construction of the original Corporate Plaza development, the site's subsurface conditions were investigated by F&M Associates. F&M performed 32 borings in two separate geotechnical investigations. A series of 21 borings (TB-1 through TB-21) were drilled in August 1977, with an additional 11 borings (B-22 through B-32) drilled in August and September of 1984. The locations of the borings performed by F&M are included on the attached Figure 2 entitled "Boring Location Plan." These borings were drilled to depths ranging from 11 to 48 feet below existing ground surface (BGS). Rock coring was performed in 19 of the borings, with core lengths ranging from 3 to 21 feet.

Subsurface conditions revealed by the F&M borings consisted of up to 8 feet of uncontrolled fill material underlain by sandy silt or silty sand above a weathered limestone with variable rock surface elevations. The limestone bedrock surface was encountered from 8 to 40 feet BGS. Groundwater was encountered in two of the borings performed during the 1977 subsurface investigation, at 14 feet and 23 feet BGS in borings TB-2 and TB-3, respectively. No groundwater was encountered in the 1984 subsurface investigation. The boring logs from the F&M field investigations are provided in Appendix A.

Langan Engineering & Environmental Services Study

Langan conducted and investigation for the City of Allentown of the sinkhole formation that developed below the Corporate Plaza building and North 7th Street in Allentown, Pennsylvania on 23 February 1994. A copy of this report is included in Appendix B. The focus of the report was "to document the geologic conditions associated with the sinkhole, to document the sequence of events and effects of the sinkhole development, and to assess the possible causative mechanism(s) for the sinkhole". The following are excerpts from the Executive Summary" and "Conclusions" sections of Langan's 21 December 1994 report.

> "Direct observations of the sinkhole were limited to the period of 23 to 27 February 1994. The sinkhole was filled with concrete on 27 February 1994."

"The sinkhole below 7th Street was an elongate, east-west-trending open depression with depths as great as 17 feet. The overall size of the depression was approximately 150 ft. by 45 ft. The sinkhole extended below the Corporate Plaza building, and resulted in the partial collapse of that building and attached parking deck. An associated sinkhole also developed below a vacant building on the east side of 7th Street. The sinkhole resulted in the collapse of the pavement of North 7th Street and severance of underground utilities. Approximately three million gallons of water were discharged into the sinkhole from the city water supply system."

"The bedrock underlying the City of Allentown consists mostly of the Allentown Formation, which is composed of limestone. Sinkholes, which are common in the Lehigh Valley, occur when overburden soil filters downward into voids in the underlying soluble limestone bedrock. The 7th Street sinkhole occurred within a soil-filled, east-west-trending trough in the irregular bedrock surface, which is a typical sinkhole situation. A review of soil boring logs from a site investigation conducted as part of the Corporate Plaza development indicates that loose soil conditions within the bedrock trough were present at the time of site development. The greatest depth of subsidence within the sinkhole appeared to be below column A-5 on the east side of the Corporate Plaza building. It is in this area that the first loss of soil likely occurred."

"Based on the events which led to the development of the surface depression, the nature of the collapse of the street and buildings, and the geologic conditions, Langan concludes that there was a void in the bedrock below the site of the sinkhole into which overburden soil migrated from the overlying bedrock trough. The sink through which the soil migrated was located below the east facade of the Corporate Plaza building, and the 7th Street sidewalk. The loss of soil into the bedrock resulted in an unstable condition in the overlying soil which caused the building to settle and the water main to fail which led to the collapse of the Corporate Plaza building and adjacent street, and the subsequent subsidence of the buildings on the east-side of 7th Street."

"Based on the results of our investigation and technical evaluation, we conclude that there was an initial soil loss within the soil-filled bedrock trough below the east side of Corporate Plaza and 7th Street, into a void(s) in the underlying bedrock. The resulting soil loss propagated upward from the bedrock surface, creating a conduit of loose soil, with increased hydraulic conductivity which extended upward through the soil column. The timing for this initial loss of soil is not known, and can not be estimated accurately. When the shifting soil undermined the water main, the pipe sheared under the load of the overlying soil and road pavement, and water began to drain down the conduit and a rapid loss of soil ensued. This accelerated soil loss continued for approximately three hours, after which time the street and east side of the building collapsed. After the water supply lines were shut off, soil continued to sink slowly into bedrock over the course of the next six to eight hours, which resulted in the subsidence of Column D-6 on the northwest corner of the building, and development of the sinkhole below the vacant buildings east of 7th Street."

Refer to the full report and figures included in Appendix B for a complete description of the geologic conditions, sequence of events and Langan's model of the mechanism and development of the sinkhole.

Earth Engineering Incorporated Investigation

A previous geotechnical subsurface investigation was performed by Earth Engineering Incorporated (EEI), documented in their Preliminary Report of Subsurface Investigation dated 5 October 2009. EEI performed nine soil borings located within the limits of the existing asphalt parking lot formerly occupied by the Corporate Plaza development. The locations of the borings performed by EEI are included on Figure 2. The nine EEI borings were advanced to the top of bedrock, as interpreted by refusal of the soil auger, ranging in depth from 23 to 51 feet BGS. A single 5-foot run of rock coring was performed in two of the EEI borings.

Subsurface conditions revealed by the EEI borings consisted of a surficial layer of asphalt pavement underlain by 3.5 to 12 feet of demolition fill of variable density and composition. The demolition fill was underlain by a stratum of natural soil referred to as Stratum I, described as decomposed/weathered dolomite in the form of sandy silt to silty sand with rock fragments. The Stratum I soils extend from beneath the demolition fill to the top of the underlying limestone bedrock. Groundwater was not encountered during the EEI geotechnical subsurface investigation. The boring logs from EEI's field investigation are provided in Appendix C.

Langan Geotechnical Investigation

Langan performed a subsurface investigation consisting of eight borings distributed throughout the two city blocks included in our site. The borings were completed between 4 and 5 May 2011 by Earthcore Inc., using a truck-mounted Acker Soilmax drill rig and a track-mounted Acker AD-18 ATV drill rig, both equipped with hollow-stem augers. The boring locations are shown on the attached Figure 2. All field work was completed under the direct observation of a Langan field engineer.

The borings were advanced to depths ranging from 15.5 to 55 feet below existing ground surface (BGS). Soil samples were collected in conjunction with SPT testing continuously in the upper 12 feet and at 5-foot intervals thereafter in general accordance with ASTM D1586. Soil samples were classified in the field and recorded on our field logs along with other observations during drilling. The top of limestone bedrock was encountered at six of the eight boring locations at depths ranging from 15 to 40 feet BGS. Borings LB-2 and LB-3 were advanced to approximately 50 feet without encountering bedrock. Once encountered, bedrock

coring was performed to collect samples of the limestone and to evaluate the bedrock's condition. Between 5.5 to 25 feet of rock coring was performed in borings LB-1, LB-4B, LB-5, LB-6, LB-7, and LB-8. Cores were obtained using a NX/NQ-sized core barrel fitted with a diamond cutting bit producing 2.125-inch-diameter rock cores. Core run lengths were typically 5-feet-long.

All borings were backfilled with bentonite grout in the lower 10 to 15 feet of the borehole, followed by a combination of bentonite grout and soil cuttings to within 4 feet of ground surface. The final 4 feet of borehole was then backfilled with cement grout and the pavement surface was restored with cold patch asphalt in asphalt-paved areas or with cement grout in concrete-paved areas. The boring logs from Langan's field investigation are provided in Appendix D.

Laboratory Testing

Upon completion of the borings, the soil and bedrock samples were brought back to our office for further evaluation and laboratory testing. Soil classifications were verified by a senior geotechnical engineer and select samples were sent to our subcontracted geotechnical laboratory to determine index and engineering properties of the subsurface soils and bedrock. Laboratory testing was performed on six soil samples and three bedrock samples at a subcontracted laboratory and included the following:

(6) Water Content [ASTM D2216];
(5) Particle Size Analyses [ASTM D422];
(4) Atterberg Limits [ASTM D4318];
(1) Percent Finer than No. 200 Sieve [ASTM D1140]; and,
(3) Unconfined Compressive Strength of Rock Cores [ASTM D2938].

The laboratory test results are discussed in the following section under their respective soil strata. The complete laboratory reporting is provided in Appendix E.

SUBSURFACE CONDITIONS

In general, the subsurface conditions beneath surficial pavement materials consist of three soil strata overlying variable-depth bedrock. These soil strata in descending order from the ground surface include a 0 to 10 foot thick fill stratum underlain by residual soils, underlain by highly weathered limestone rock. The top of bedrock was encountered in six of the eight borings at depths ranging from 10 to 40 feet BGS. A brief description of each stratum is provided below.

Surficial Materials

Surficial layers encountered at the boring locations consisted of asphalt or concrete pavement. Asphalt pavement was encountered at four boring locations (LB-2, LB-3, LB-4 and LB-7). The thickness of the asphalt ranged from 1.5 to 4 inches and was placed on underlying stone aggregate subbase ranging in thickness from 4 to 6 inches. The existing asphalt pavement at the at-grade parking lots is in poor condition with abundant cracking and other damage. Asphalt pavement within the public right-of-ways is in significantly better condition, with only minor cracking and other damage.

Concrete pavement was encountered at three boring locations (LB-1, LB-5, and LB-6), ranging in thickness from 4 to 6 inches and was constructed on an underlying stone aggregate subbase layer ranging in thickness from 3 to 6 inches. No surficial materials were encountered at boring LB-8, where fill material was encountered beginning at ground surface.

Fill

Beneath the surficial layer exists a stratum of highly variable fill made up of sand, gravel, brick and concrete fragments, silt, and clay. Varying amounts of wood, asphalt fragments, glass, metallic debris, and cinders were also encountered within the fill. These fill materials were placed throughout the course of the site's development history. The fill consisted primarily of soil constituents with some man-made material; however, there were instances in which the fill consisted entirely of brick and concrete debris. The predominantly brick-concrete debris fill areas were only encountered within the limits of the former Corporate Plaza development that once occupied a portion of the site. In addition to these demolition debris materials, possible below-grade elements of the former construction, such as basement floor slabs and exterior foundation walls, may have been encountered at boring LB-4, where multiple offsets of the drilling equipment were necessary to avoid obstructions. Auger refusal was encountered at approximately 6 feet BGS at LB-4 and LB-4A. The obstruction encountered at borings LB-4 and LB-4A was not encountered at LB-4B. This obstruction may also be the grout used to fill the massive sinkhole since it was mapped at approximately this location. LB-8 also encountered a near-surface obstruction and is also roughly within the mapped limits of the sinkhole.

The fill stratum was encountered in all borings except for LB-5, ranging in thickness from 3 to 10 feet. The SPT N-values varied from 3 blows per foot (bpf) to refusal of the sampling equipment, at 50 blows over 0 inches of sampler penetration, indicating extreme variability in the in-place density of the fill materials. The average N-value of 21 bpf indicates a medium-dense state of in-situ density, but is not truly representative of the in-place density of this layer.

Soils laboratory testing was performed on two split-spoon samples of the soil component of this stratum collected during Langan's preliminary geotechnical investigation. The natural moisture content ranged from 7.2% to 12.0% and the fines content (silt and clay-sized particles) ranged from 2.6% to 62.3% in the tested specimens. Laboratory testing results and field observations show that the fill stratum consists of poorly-graded gravel with sand [GP] and sandy lean clay [CL]. A summary of the laboratory testing results for samples within this layer is provided in Table 1.

TABLE 1 - LABORATORY TEST RESULTS FOR FILL

Boring/ Sample	Depth (ft)	Water Content (%)	% Passing #200	Atterberg Limits			Laboratory Description
				LL	PL	PI	
LB-3/S-4	6-7.7	12.0	2.6	--	--	--	Poorly-graded GRAVEL with sand [GP]
LB-6/S-2	3-5	7.2	62.3	36	22	14	Sandy lean CLAY [CL]

Residual Soil

A stratum of fine-grained soil was encountered beneath the surficial material at boring LB-5 and beneath the fill material encountered at the remaining borings except for boring LB-8, where the fill is underlain by bedrock. This soil layer is residual material that mantles the weathered bedrock and is derived from the chemical weathering of the underlying limestone rock. The residual soil generally consisted of an upper zone of brown to brown-orange clayey silt with trace amounts of fine gravel and sand, and a lower zone of brown to yellow silt with varying amounts of friable weathered rock fragments. This stratum ranged in thickness from 2 to 36 feet where penetrated, and continued beyond the investigated depths of 48.3 and 51 feet at borings LB-2 and LB-3, respectively.

SPT N-values collected within this stratum ranged from 2 bpf to 78 bpf and averaged 21 bpf, indicating a very stiff state of consistency. Higher N-values were typically observed in samples containing higher percentages of gravel-sized particles. Soils laboratory testing was performed on three split spoon samples of the residual soil stratum. The results of this laboratory testing are included below in Table 2. The natural moisture content ranged from 19.8% to 32.8% and the fines content (silt and clay sized particles) ranged from 49.0% to 75.8% in the tested specimens. The index test results indicate that the residual soil ranges from lean clay with sand [CL], silty sand [SM], and sandy silt [ML].

TABLE 2 - LABORATORY TEST RESULTS FOR RESIDUAL SOIL							
Boring /Sample	**Depth (ft)**	**Water Content (%)**	**% Passing #200**	**Atterberg Limits**			**Laboratory Description**
				LL	**PL**	**PI**	
LB-1/S-3	4-6	21.5	75.8	31	22	9	Lean CLAY with sand [CL]
LB-3/S-11	34-36	19.8	49.0	NP	NP	NP	Silty SAND [SM]
LB-4B/S-3	10-12	32.8	66.3	NP	NP	NP	Sandy SILT [ML]

Weathered Rock

Weathered limestone bedrock was encountered below the residual soil in five boring locations (LB-1, LB-4B, LB-5, LB-6, and LB-7) at depths ranging from 9 to 36 feet BGS. The weathered rock stratum was from 2 to 6 feet thick and consisted of tan-grey gravel and sand with silt. The weathered rock was characterized by frequent refusal of the sampling equipment (i.e. 50 blows over less than 6 inches of sampler penetration). SPT N-values collected within the weathered rock ranged from 30 bpf to 50 blows over 0 inches of sampler, indicating a very dense state of relative density.

Laboratory testing was performed on one sample of the weathered rock stratum. The results of this laboratory testing are included below in Table 3. The natural moisture content was 5.8% and the fines content (silt and clay sized particles) was 18.6% in the tested specimen. The index test results indicate that the weathered rock is silty gravel with sand [GM].

TABLE 3 - LABORATORY TEST RESULTS FOR WEATHERED ROCK							
Boring /Sample	**Depth (ft)**	**Water Content (%)**	**% Passing #200**	**Atterberg Limits**			**Laboratory Description**
				LL	**PL**	**PI**	
LB-6/S-5	9-9.9	5.8	18.6	--	--	--	Silty GRAVEL with sand [GM]

Limestone Bedrock

Limestone bedrock was encountered in six of the eight borings performed during Langan's geotechnical investigation, at depths ranging from 10 to 40 feet BGS. Once encountered, rock cores were obtained of the bedrock. Coring lengths ranged from 5.5 feet to 25 feet. Rock core recovery ranged from 15% to 100%, and rock quality designation (RQD) ranged from 0% to 75.0%. A summary of the rock coring performed is provided below in Table 4.

The observations collected during the rock coring runs reflect a highly variable weathering profile of the limestone bedrock, as is expected in soluble carbonate rock formations that result in karst geology. At borings LB-4B and LB-8A, the observed weathering was minimal. No loss

of wash water was observed in these two borings, and recovery and RQD values were relatively high. The rock that was cored in the remaining borings exhibited more extensive weathering, as evidenced by the consistent loss of wash water, frequent soil seams or highly weathered seams within the cores, and the highly fractured nature of the recovered rock cores. Open voids were encountered in two boring locations: a 2-foot-high void was encountered at boring LB-6 from 23 to 25 feet BGS, and a 3-foot-high void was encountered at boring LB-5 from 41 to 44 feet BGS.

TABLE 4 – ROCK CORING SUMMARY

Boring	Run	Depth (ft)	Length (ft)	REC (%)	RQD (%)	Comments
LB-1	1	21-22.5	1.5	100	38.9	No return of wash water for LB-1
	2	22.5-26	3.5	47.0	36.9	Soil seam 22.5 to 23 feet, and 25 to 26 feet
	3	26-31	5	62.0	8.3	Soil seam 26 to 26.3 feet
	4	31-35.5	4.5	100	0.0	Highly fractured 31.5 to 34 feet, highly weathered 35 to 35.5 feet
	5	35.5-41	5.5	69.7	13.6	Highly weathered 37.5 to 38 feet
LB-4B	1	29-34	5	94.2	35.0	Water return throughout coring LB-4B
	2	34-39	5	95.0	75.0	Weathered zone 36.5 to 37 feet
LB-5	1	40-45	5	30.0	9.2	No return of wash water for LB-5. VOID 41 to 44 feet
	2	45-50	5	70.0	39.2	
	3	50-55	5	83.8	55.0	Highly fractured, weathered zone 53.5 to 55
LB-6	1	15-20	5	48.3	0.0	No return of wash water for LB-6. Soil seams at 16.9 to 17.1 feet, 18.3 to 18.4 feet, 18.8 to 18.9 feet, and 19.2 to 19.9 feet.
	2	20-25	5	30.0	8.3	Highly fractured from 21 to 23 feet. VOID 23 to 25 feet.
	3	25-30	5	15.0	0.0	Highly fractured zone 25-30 feet
	4	30-33.5	3.5	60.5	0.0	Soil seam 30 to 31 feet. Highly fractured, highly weathered from 31 to 31.5 feet.
	5	33.5-35	1.5	83.3	0.0	Highly fractured 33.5 to 35 feet
	6	35-40	5	98.3	45.8	
LB-7	1	23.5-26	2.5	53.3	0.0	No return of wash water for LB-7. Highly fractured 23.5 to 26 feet.
	2	26-29.5	3.5	47.6	0.0	Highly fractured 26 to 29.5 feet.
	3	29.5-31	1.5	100	0.0	
	4	31-34.5	3.5	100	11.9	Highly fractured 33.8 to 34.5 feet.
	5	34.5-36	1.5	100	30.6	
	6	36-41	5	81.7	10.0	
LB-8A	1	10-10.5	0.5	100	0.0	Water return throughout coring LB-8A
	2	10.5-15.5	5	85.0	56.7	

Three samples of intact rock core were tested in unconfined compression. The unconfined compression strength test results indicate the intact limestone rock has a moderate to high strength. The results of these tests are provided in Table 5.

TABLE 5 – LABORATORY TEST RESULTS FOR LIMESTONE BEDROCK

Boring	Run No.	Depth (ft)	Unit Weight, γ_{dry} (pcf)	Unconfined Compressive Strength, q_u (tsf)
LB-1	Run 2	23-24	175.4	1094.2
LB-4B	Run 2	35-36	173.3	1059.8
LB-7	Run 6	38-39	174.9	586.9

Groundwater

Groundwater was not encountered during Langan's subsurface investigation. Also, only 2 of the 41 borings drilled during prior geotechnical studies of the site encountered groundwater. These borings recorded groundwater at 14 and 23 feet below the ground surface corresponding to approximately elevation EL+337 to EL+346 based on an assumed ground surface elevation of EL+360. No long-term groundwater observations were made as part of this investigation. Seasonal fluctuations of groundwater levels should be expected with variations in precipitation, surface water, evaporation, pumping, and other natural or man-made factors.

EVALUATION AND RECOMMEDATIONS

Evaluation

The city block on which the arena is sited is the same block that experienced the formation of a massive sinkhole that caused the undermining of several column footings of Corporate Plaza office building in 1994 that lead to a partial collapse of that building. While that sinkhole was probably caused in part by the water main break washing tons of erodible residual soils away thereby undermining column footings, the breach in the main could have been caused by ground subsidence and sinkhole formation undermining and causing distress in the pipe eventually leading to its failure. Basically, the karstic nature of the underlying soil and bedrock formations was the root cause and these conditions must be heavily considered in the redevelopment of this site and in particular the selection of the foundation system.

The subsurface conditions encountered during the previous and current geotechnical investigations are typical of the local karst geology. The tendency for ground subsidence and potential collapse into sinkhole formation raises concerns over the support of the new foundations and floor slabs of the proposed development. The variable depth to bedrock and its pinnacled nature will make the rock excavation during utility installation and construction of below-grade levels difficult. In the consideration of deep foundations, the variable bedrock surface and erratic weathering profile will also impact the foundation length and load-bearing rock sockets.

Foundations

Considering the history of sinkhole formation and building collapse at this site, as well as the need to limit settlements to magnitudes that are tolerable for the successful operation and maintenance of an arena ice surface, we recommend constructing the proposed arena and office development on a deep foundation system consisting of drilled foundation elements socketed into sound limestone bedrock. The rationale for this recommendation is as follows.

The use of a shallow foundation system would require stabilizing the highly erodible residual overburden soils and the karstic limestone bedrock formation in order to try to eliminate the potential for a similar disastrous event such as the building collapse of 1994. A combined program of pressure grouting for the soil stabilization and proof-drilling and grouting of voids in the bedrock would be required to ensure a stable competent subgrade for spread and strip footings or a mat foundation. However, this does not appear to be a cost effective approach since the volume of grout required to perform this stabilization grouting can not be predetermined accurately even with a very comprehensive subsurface investigation consisting of borings, test pits and geophysics. As such, there is great exposure to excessive cost overruns due to need to significant expand the grouting program and extensive grout takes in proof-drilling and grouting program. Also, a maximum allowable design bearing pressure of only 4 ksf can be assigned to the residual soils resulting in footing sizes on the order of 17-feet-square for the truss load bearing columns of the arena. The bedrock surface is extremely variable in elevation and competency and can not be counted on for shallow foundation support. Therefore, consideration must be given to suitable deep foundation options.

Driven pile foundations are not suitable due to the difficulty in driving the piles in variably weathered pinnacled bedrock. The main concerns are the inability to advance the pile through weathered zones and solution cavities within the rock mass and the uncertainty as to whether or not voids may exist below pile tips once driving refusal has been achieved. These concerns also apply to certain other pile types such as auger cast-in-place piles, drilled displacement

piles, and similar augered piles that would be very difficult to advance through pinnacled bedrock. Adequate compressive capacities would also be difficult to achieve with these pile types.

In order for deep foundation elements to develop sufficient capacity, it is essential that they be advanced through the upper weathered zones of bedrock, that may contain solution cavities, and into the deeper sound, competent bedrock. Drilled deep foundation elements, such as rock-socketed drilled shafts (caissons) and micropiles, meet this requirement.

Mini-caissons (18 to 30 inches diameter drilled shafts) are a viable option but would require the use of a downhole hammer in to order to advance the casing to competent rock and to produce a rock socket of sufficient diameter and length. The larger diameter would result in a slow installation process. Also, the available compressive capacity of the mini-caissons, on the order of 1,500 to 2,500 kips per shaft, may be excessive considering the anticipated moderate column loads for this structure. The cost of a rock-socketed drilled shaft foundation system is likely to significantly exceed that of a comparable micropile system.

It is our opinion that micropiles are the most technically viable and cost effective deep foundation solution for this project. Micropiles would be cased through the burden fill, residual soil and weathered-pinnacles limestone bedrock with the casing advanced using a downhole hammer. Once the casing has been advanced to the top of competent bedrock, the downhole hammer is used to create a socket in the bedrock for load transfer. We anticipate that micropiles having rock socket diameters between 8.5 to 11 inches are appropriate for column support for this project. The competent bedrock at this site should be able to provide an allowable unit side shear of 120 pounds per square inch (psi). Therefore, micropile compressive capacities on the order of 380 to 970 kips should be attainable for sockets on the order of 10 to 20 feet in length. The following table provides a preliminary matrix of micropile compressive capacities for various socket diameters and lengths.

TABLE 6 – MICROPILE COMPRESSIVE CAPACITY MATRIX

Socket Diameter (in)	Micropile Capacity per Socket Length (kips)		
	10-Foot Socket	15-Foot Socket	20-Foot Socket
8.54	386	579	772
9.75	441	662	882
10.71	485	727	969

The preliminary borings show bedrock surface that is extremely variable in depth. However, this does not directly translate to the top of competent bedrock, which is where the top of the

micropile socket must begin. Our review of all of the available boring on the arena site indicates that the cased length of the micropiles should be expected to range from 40 to 80 feet below current site grades. The actual cased length must be calculated based upon the expected pile cap subgrade which can be based upon the proposed finished floor elevation. Then the rock socket lengths given above must be added to the cased length to get the total anticipated pile length. Anticipated total pile lengths will be in in the 60 to 100 feet range.

Floor Slab

The event level floor of the proposed arena can presumably be constructed on a slab-on-grade; however this would be inadvisable without first performing a ground improvement program to address the underlying highly weathered limestone bedrock that could produce ground subsidence and sinkholes if not stabilized prior to construction. As with the foundation system options discussed above, this stabilization can be achieved by performing a proof-drilling and grouting program, in which the soil-rock interface is sealed with high mobility grout in an effort to fill in any voids or solution cavities and prevent future collapse of overburden soils and subsequent ground subsidence and sinkhole formation. The main disadvantage of this ground improvement method is the inability to accurately estimate the volume of grout that will be required to seal the soil-rock interface. Depending on the degree of weathering, potentially large volumes of grout could be necessary.

Because of the uncertainty in the required grout volume and the potential for increased construction costs, it is our opinion that it will be most cost-effective to construct the heavily loaded, settlement sensitive event level floor slab as a micropile-supported structural slab system while supporting the other non-critical floors on a stabilized subgrade designed as a slab-on-grade. The spacing and capacity of the micropiles installed to support the structural floor slab will be dictated by the structural design of the floor slab including its thickness and reinforcement. A cost-effective combination of the slab thickness, pile spacing and pile capacity can be developed by the structural engineering with input from Langan.

Below-Grade Walls

Site retaining walls are proposed along the loading docks on the order of 14 feet high, and review of the transverse building section of the proposed arena shows an approximately 24-foot-high basement wall along the southern limit of the event level arena support area. Site retaining walls should be designed to resist earth pressure and surcharge loads. Unrestrained walls (walls that are free to move/rotate) and should be designed for active earth pressure and restrained walls (walls that are braced against movement/rotation) should be designed for at-

rest earth pressure. The soil parameters shown in Table 7 should be used for design of site retaining walls, assuming that the walls are backfilled with the native silty residual soil.

TABLE 7 – DESIGN PARAMETERS FOR RETAINING WALLS	
Parameter	**Recommended Value**
Wall Backfill:	Silty Residual Soils
Typical Backfill Unit Weight:	135 pcf
Friction Angle:	32 Degrees
Coefficient of Active Earth Pressure: (top of wall free to deflect)	0.31
Coefficient of At-Rest Earth Pressure: (top of wall restrained)	0.47
Allowable Soil Bearing Capacity	4,000 psf
Interface Friction Factor	0.35

Surcharge loads should also be considered in the design of retaining walls. The walls should be designed for an additional uniform pressure distribution equal to the corresponding coefficient of earth pressure (active or at-rest) times the anticipated surcharge load. The design surcharge load should include anticipated surcharge from construction equipment. Walls must also be designed for surcharge loads from adjacent structures if the walls extend below the area of influence of the adjacent foundations. The zone of influence of neighboring foundations can be estimated as the area below an imaginary 2 to 1 line (vertical to horizontal) extending downward from the base of the adjacent foundations.

The above parameters assume that the walls are fully backdrained to prevent the buildup of hydrostatic pressure. We recommend that retaining walls be fully drained and adequate drainage can be provided by a clean, crushed-stone drainage zone or a manufactured drain panel with weep holes or a connection drain.

Seismic Design Criteria

According to the International Building Code (IBC) 2009 the following seismic parameters should be used for design:

- Site Class = D – Stiff Soil Profile
- Maximum Considered Earthquake Ground Motions: S_s = 25.8%g and S_1 = 6.1%g
- Ground motions listed above should be adjusted for Site Class "D" effects using the following coefficients: F_a = 1.59 and F_v = 2.4

Development Recommendations for Karst Geology

Stormwater Management in Karst Areas

Stormwater management in karst hazard areas is typically best performed by surface detention ponds where access is relatively easy and subsequent sinkhole repair costs typically lower. However, we understand that as part of the current BMP, infiltration is often required.

Infiltration in areas with karst prone bedrock, while feasible, should be designed with caution and the understanding that infiltration increases the risk for future sinkhole formation at the site. All stormwater management should be designed in accordance with the karst hazard requirements. To help reduce the potential for infiltration related sinkholes to impact the proposed building, we also recommend the infiltration areas be kept a minimum of 30 feet away from the building foundations, and as far away from the building as practical for the proposed site development. The infiltration areas should be designed for the minimum feasible hydraulic head (maximize infiltration area of each individual bed) such that concentrated flow areas are minimized. Long term maintenance and monitoring of infiltration areas is critical, signs of sinkholes should be investigated and repaired immediately.

Sinkhole Mitigation

The following recommendations regarding sinkhole mitigation should be incorporated into the project design to help reduce the potential for long term sinkhole hazards at the site. Sinkholes typically form when surface water infiltration causes overlying soils to collapse into solution cavities within the bedrock. Controlling infiltration of surface water is critical to reducing the potential for sinkholes developing at the site. Proper construction procedures are also critical to ensure sinkholes are not created during construction. We recommend the following procedures be incorporated in the design plans and construction documents to help control infiltration and reduce the potential for sinkholes at the site:

- Provide positive surface gradients adjacent to the building to direct surface water away from the foundations and slabs towards suitable discharge facilities;
- Provide positive surface gradients away from any bedrock expose at the surface;
- Connect all roof downspouts to solid collector pipes that discharge to appropriate discharge facilities;
- Design site grades to prevent surface water from ponding, especially adjacent to buildings, on pavements and during construction;
- Construct drainage utilities with leak-proof pipe and watertight connections. Flexible connections should be considered at building locations;

- Backfill the upper portion of below grade walls with a low permeability clay material to reduce water infiltration;
- Protect excavations and trenches during construction. Cover all excavations during wet weather and backfill as soon as possible following excavation.
- Immediately repair all sinkholes exposed during construction.

Sinkhole Repair

If evidence of sinkholes is revealed during construction or over the life of the development, the geotechnical engineer should be notified immediately and the sinkhole repaired as soon as possible to limit migration of the sinkhole. The following repair sequence should be followed, under the direct observation of the geotechnical engineer:

- Excavate all loose soil to expose the throat of the sinkhole or bedrock. The use of high pressure water may aid in identifying the throat of the sinkhole;
- If a throat is encountered in soil, plug the throat with a concrete or grout cap that extends at least 1 foot into firm soil;
- If a throat is encountered in rock, plug all exposed holes and crevices with grout. Use of a low slump grout is permissible to reduce the amount of grout needed to cap the hole;
- Allow grout to cure for a period of at least 24-hours;
- Backfill area using excavated soil in accordance with the requirements for controlled fill presented below. All soil excavated during the repair should be temporarily stockpiled onsite in accordance with all applicable erosion and sediment pollution control requirements;
- Provide permanent positive drainage away from the sinkhole repair area;

Another option for sinkhole repair could include inverted filters, if approved by the geotechnical engineer. Where significant excavations below 20 feet are completed and a defined throat to the sinkhole cannot be found, the geotechnical engineer should be consulted for alternate repair options such as pressure grouting and flow fill.

CLOSING

Thank you for the opportunity to provide geotechnical engineering services for this project. Should you have any questions regarding the content of this report or if we can be of further service, please call us.

Sincerely,
Langan Engineering and Environmental Services, Inc.

Michael B. Fritzges, P.E.
Project Engineer

John J. McElroy Jr., P.E.
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Enclosure(s): Figure 1– Site Location Map
Figure 2 – Boring Location Plan
Appendix A – F&M Investigation Boring Logs
Appendix B – Langan 1994 Corporate Plaza Study Report
Appendix C – EEI Investigation Boring Logs
Appendix D – Langan Investigation Boring Logs
Appendix E – Soils Laboratory Test Results

cc: Mike Leone – Hammes
Chris Hager – Langan
Ron Boyer – Langan

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