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Geotechnical Engineers

E-1747-01

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September 27, 1996

Zeck-Butler Architects, P.S.
Paulsen Center, Suite 860
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Attn: Mr. Chip Missler
Project Manager

**REPORT OF SUBSURFACE EXPLORATIONS AND GEOTECHNICAL
ENGINEERING STUDIES FOR THE PROPOSED VETERANS ADMINISTRATION
HOSPITAL ADDITION - BUILDING 27, SPOKANE, WASHINGTON**

1.0 INTRODUCTION

This letter report summarizes the results of subsurface explorations and geotechnical engineering studies conducted for the proposed new Veterans Administration Hospital Building 27 Addition Project. The purpose of the work was to supplement previous work and develop site specific recommendations to assist in the design and construction of new foundations, below grade walls and related earthwork.

The study was accomplished in general accordance with our revised proposal, dated August 23, 1996. The work was authorized by Mr. William J. Zeck, AIA on August 28, 1996.

2.0 SITE AND PROJECT DESCRIPTION

The VA Hospital site is located in the northwestern part of Spokane. See Figure 1. The proposed construction, which has been identified as Building 27, will include a new entrance at the north end of Building 1, the main hospital building. The site is presently

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landscaped with lawn and walkways and includes an emergency entrance complete with a paved driveway and turn around, and a canopy covered entrance. A number of underground utilities cross the site including a chilled water service tunnel.

We understand that the proposed new building will be a multi-story, "L"-shaped structure with a total footprint area of about 8,600 SF. The new addition will abut existing Buildings 1, 25 and 26 as shown on Figure 2. Most of the new building area will have a full basement. Existing Building 1 is a three-story concrete structure. Buildings 25 and 26 are both single-story, steel-frame structures.

We understand that Building 1 and 25 have shallow foundations and crawl spaces beneath structurally supported first floors. The north end of Building 26 has a partial basement. Construction of the new Building 27 addition will require excavating adjacent to shallow foundations supporting Building 1 and 25, and 26.

3.0 SUBSURFACE EXPLORATIONS

Two exploratory borings were made at the site on September 4, 1996 at the approximate locations shown on Figure 2. Each boring extended 25 feet below existing ground surface. The drilling, totaling 50.0 linear feet, was accomplished using a CME-75, truck-mounted drill rig. The borings were advanced using a 7-1/4 inch O.D., 3-3/4 inch I.D., hollow-stem auger.

Representative soil samples were obtained at 2.5 foot depth intervals, using a 2 inch O.D. split-spoon drive sampler. Standard Penetration Tests (SPT) were performed in conjunction with the drive sampling. These tests (ASTM D 1586) involved driving the sampler a total of 18 inches with a 140 pound automatic drop hammer, freely falling a distance of 30 inches. In performing the test, the sampler is driven through three successive 6 inch increments of penetration. The sum of the number of blows for the last two increments, that is, the last foot of penetration, is defined as the Standard Penetration Resistance, or N-value. This value is a widely accepted empirical parameter that can be approximately correlated to certain engineering characteristics of the soils sampled.

Drilling and sampling operations were observed and recorded by Mr. David S. Phelps, our geotechnical engineer. Mr. Phelps collected and field classified samples and developed detailed field boring logs. Representative portions of the split-spoon samples

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were sealed in jars to preserve natural moisture contents and were returned to our laboratory for analysis. A total of 20 Standard Penetration Tests were conducted and 15 soil samples were recovered.

Summary boring logs are presented in Figures 3 and 4. Summary descriptions of the soil units shown on the logs are based on interpretation of the field and laboratory data. The logs show plots of Standard Penetration Test N-values and results of moisture content tests.

Upon completing the drilling work, the borings were backfilled with cuttings, and the upper 3 feet tamped to minimize settlement. Boring locations were determined by taping from existing site features. Relative ground surface elevations were estimated by hand level survey and are shown on the respective boring logs. A temporary benchmark (TBM) was established at an assumed elevation of 100.0 feet, at the south most bolt on the upper flange of a fire hydrant located directly east of the north end of Building 1. See Figure 2.

4.0 LABORATORY TESTING AND ENGINEERING PROPERTIES

Soil samples were visually classified as they were recovered. Representative portions were preserved in airtight containers for transport to our laboratory. Upon receipt in the laboratory, samples were reexamined to verify and refine field classifications, in general accordance with the procedures described in ASTM D 2488.

Natural moisture contents (ASTM D 2216) were determined on all recovered samples to aid in classifying the soil and evaluating engineering properties. Moisture contents are expressed as a percentage, based on the dry weight of the samples. Graphic plots of moisture content vs. depth are shown on the respective boring logs, Figures 3 and 4. Two grain size analysis were conducted in general accordance with ASTM D 422, to correlate field and visual classifications and for use in describing the soil units. Since the split-spoon drive sampler limits the maximum recoverable particle to about 1 inch, gradation analyses may not accurately reflect the full range of particle sizes present. The curves shown on Figure 6 should be considered representative only of the minus 3/4 inch fraction. The figure shows that sample 101-6 would classify as GP and sample 102-7 as SP based on the Unified Soil Classification (USC) System.

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5.0 SUBSURFACE INTERPRETATIONS

The subsurface soils at this site consist primarily of gravelly and sandy soils formed in glacial outwash. The boring logs show our interpretation of the subsurface stratigraphy based on samples recovered from 2.5 foot intervals. Actual soil conditions are probably more stratified or layered than shown on the logs. Generally, except for a thin surface layer of sod, three distinct soil units were encountered in the recent borings at the site:

- Silty GRAVEL
- Sandy GRAVEL
- Gravelly SAND

Each of these units can be described in general terms as follows:

Silty GRAVEL was encountered in both borings just below the sod. It extended about 2 to 3 feet below existing ground surface. This soil unit was described as consisting of loose to dense, brown, silty GRAVEL and appeared to have been disturbed or reworked, possibly during previous site grading. Standard Penetration Test (SPT) N-values of 7 and 22 blows per foot were obtained in this soil unit. A natural moisture content of 7.1 percent was determined for the sample obtained at Boring 101. No sample was recovered in this unit at Boring 102.

Sandy GRAVEL was encountered below the silty GRAVEL at both locations extending the full 25 foot depth of Boring 101 and to a depth of about 15 feet in Boring 102. This soil unit was described as consisting of dense to loose, brown to gray, sandy GRAVEL; moist, with occasional small cobbles and interbedded lenses of gravel and coarse sand. SPT N-values ranged from 8 to 52 blows per foot and averaged about 24 blows per foot. Natural moisture contents ranged from 1.3 to 4.7 percent and averaged about 2 percent. A gradation analysis on sample 101-6, from this unit, indicates it classifies as GP based on the Unified Soil Classification System

Gravelly SAND was encountered in Boring 102 from about 15 feet to the bottom of the boring at 25 feet. This soil unit was described as consisting of medium dense, brown to gray, gravelly SAND, SPT N-values ranged from 25 to 40 blows per foot and averaged about 31 blows per foot. Natural moisture contents ranged from 1.5 to 3.1 percent and averaged about 2 percent. A gradation analysis on Sample 102-7, from this unit, shows it classifies as SP based on using the Unified Soil Classification System.

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The average SPT N-value, obtained below proposed basement finish floor elevation at about 1910 feet was 31 blows per foot. No ground water was encountered in the borings.

Previous geotechnical explorations in this area include a log of boring PA-3 that was obtained from the War Department Corps of Engineers Drawing No. 18-01-01, Sheet of 2 of 2, dated 19 September 1947. Details on the method of exploration are not known; however, the boring log for PA-3 shows that subsurface soil conditions consisted of about 16 feet of silty gravel, overlying 18 feet of sandy gravel, which, in turn, was overlying coarse sand. No Standard Penetration Test blow count data is available for this boring. The soil descriptions shown on the PA-3 log are generally similar to the materials encountered in the recent borings. All materials are identified as sands and gravels, however, depths for various zones are somewhat different. Additional information shown on the 1947 drawing includes results of two plate load tests that were conducted at other locations on site. This information was used in conjunction with recent Standard Penetration Test data to estimate elastic soil modulus values for the sandy GRAVEL and gravelly SAND.

The following approximate soil engineering properties were established based on field observations, laboratory classifications and testing the standard penetration resistance values from the two recent boring.

MEASURED VALUES

Engineering Properties	Silty <u>GRAVEL</u>	Sandy <u>GRAVEL</u>	Gravelly <u>SAND</u>
SPT N Values*(Blows per ft.) Average and (Range)	15 (7-22)	24 (8-52)	31 (25-40)
Moisture Content (%) Average and (Range)	7.1 -	2.0 (1.3-4.7)	2.0 (1.5-3.1)
USC	-	GP	SP

* Below approximate basement finish floor elevation (1910 ft.)
average SPT N-values were 31 blows per foot.

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ESTIMATED ENGINEERING SOIL PROPERTIES

Unit In-place Weight (pcf)	120	125	125
Friction Angle ($^{\circ}$)	32	35	35
Cohesion (psf)	0	0	0
Poisson's Ratio	0.30	0.30	0.30
Elastic Modulus (ksf)	-	1290	815

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 Site Preparation

Demolition work prior to the start of excavation should include removing all structures, pavements, underground utilities, tree roots, sod and topsoil in the proposed new building area. Over-excavation necessary to remove existing foundations, underground utilities, etc. should be replaced with compacted Structural Fill.

The first floor of the new addition will match the existing floor in Building 1 at about 1922 feet. We understand that the Building 27 addition will include a basement with a finish floor at about elevation 1910 feet. Since the existing grade slopes down towards the north, this will require making a general excavation of about 8 to 12 feet deep.

When an excavation is made adjacent to an existing structure, there is always some concern about the stability of the existing foundations. We understand that the foundations for the north end of Building 1 and east sides of Building 25 and 26 are at variable depths and all are above the proposed Building 27 basement floor level. Since below the bearing surface, footings impose both vertical and horizontal stresses into the foundation soil, excavations that extend below the bearing level of adjacent foundations must be shored or otherwise designed to accommodate the lateral stress imposed by the footing load. This is frequently accomplished by underpinning or special shoring before making the new excavation. Underpinning transfers the existing footing loads to deeper depths and eliminates the need to consider their lateral load surcharge effect on the new below grade wall. Recommendations for underpinning and treating lateral surcharges are included in a subsequent section.

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After stripping and general excavation, we recommend proof-compacting subgrades scheduled to receive Structural Fill, slabs-on-grade or pavements. This should be done with a heavy, vibratory compactor, (6 ton minimum static weight) by making 6 to 8 passes over the surface. The purpose of proof-compacting will be to densify the surface and identify any loose areas that may require additional over-excavation and replacement with compacted Structural Fill. Where vibratory compaction may interfere with existing foundations, walls or temporary shoring, we recommend proof-compacting with a heavy, static compactor and increasing the number of passes. We recommend that proof-compaction be observed by an experienced geotechnical engineer, or engineering technician, representing the Owner.

6.2 Excavation, Backfilling and Structural Fill

In addition to general excavating required for stripping, subgrade and basement preparation, other excavations will be necessary for foundation construction and installing underground utilities. We believe that all of the site excavation work can probably be accomplished with conventional equipment, such as backhoes, loaders, dozers, etc.

Excavating close to existing structures should not proceed without considering the need for underpinning the existing building foundations. In general, if an existing foundation lies outside of a 1.0V on 1.0H slope extending up from the bottom of an adjacent excavation, foundation underpinning is normally not required and lateral and vertical pressures are not considered in the design of the excavation (or basement) wall; if the existing foundations lie between a 1.0V on 1.0H and a 2.0V on 1.0H slope, lateral and vertical pressures must be considered for the design of the excavation wall; if existing foundations lie inside a 2.0V on 1.0H slope, underpinning is typically required.

Since the maintenance of stable excavations is related to job safety, excavation stability should be the responsibility of the Contractor. All excavations should conform to Federal, State, and local standards. Based on information from the borings, we believe the silty GRAVEL, sandy GRAVEL and gravelly SAND, would classify as OSHA Type C, for excavation regulation purposes. For Type C soils, OSHA (and WAC 296-155-664, Part N) recommend that all unsupported, simple-slope excavations, 20 feet deep or less, have a maximum allowable slope angle of 1.0V on 1.5H.

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We recommend that permanent cut slopes be made no steeper than about 1.0V on 1.5H. This recommendation is based on maximizing long term stability and minimizing surface erosion potential. Final permanent slopes should be protected against surface erosion.

We believe that the on-site silty GRAVEL, sandy GRAVEL and gravelly SAND, could be reused for wall backfill and for other Structural Fill purposes.

In our opinion, imported soil for backfilling and Structural Fill purposes should consist of a clean, well graded, sandy GRAVEL, or gravelly SAND having a maximum size of 4 inches and containing less than 15 percent by weight of non-plastic fines; i.e., material passing the No. 200 sieve, based on the minus 3/4 inch fraction. Samples of imported fill material should be approved by the soils engineer.

Following proof-compaction of the subgrade, Structural Fill should be brought to optimum moisture content ($\pm 2\%$) and then placed in lifts not exceeding about 10 inches in loose measure, followed by compacting to produce a density of not less than 95% of the Modified Proctor Maximum Dry Density (AASHTO T-180).

Laboratory compaction testing should be performed on all potential Structural Fill materials to establish moisture and density criteria before fill placement begins. In our opinion, fill compaction will probably be best accomplished with vibratory compaction equipment, except possibly adjacent to existing structures or temporary shoring. We recommend that Structural Fill placement and compaction be monitored by an experienced soils engineer, or engineering technician, representing the Owner.

6.3 Foundation Design Considerations

Based on the results of the field explorations and our estimate of the engineering properties for the soil encountered at the site, we believe that the new structural loads could be supported on a system of conventional spread footing foundations bearing in the native, medium dense to dense, sandy GRAVEL, gravelly SAND, or compacted Structural Fill. For frost protection, we recommend providing a minimum of 3 feet of embedment for exterior foundation footings and 1.5 feet of embedment below adjacent grade for interior footings.

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We understand that several options are being considered for the new basement wall locations. If the new basement walls are located close to the existing buildings, floor and other structural loads from above will probably bear directly on top of them. If the basement walls are located farther away from the existing building wall lines, new floor and other super structure loads will have to be supported on footings constructed adjacent to the existing building footings. Foundation conditions are complicated for this project because of the relatively high foundation loads on the Building 1 footings.

We estimate that the existing foundation wall load at the north end of Building 1 is about 10 klf, and the new wall load from the proposed addition at about 4.5 klf. The existing footings are reported to be about 1.5 feet wide, suggesting that the soil bearing pressure is about 6700 psf. This is considerably higher than the bearing pressure we would normally recommend for the subgrade soil at this site. Based on soil properties estimated from the penetration resistance measured in the two recent borings, the ultimate soil bearing capacity would be about 12-14 ksf. Using a factor of safety of about 3, the recommended allowable soil bearing pressure should be about 4000 psf. Using this value, the existing 1.5 ft. wide north wall footing of Building 1 would have to be increased by about 2 ft. to accommodate the existing and expected new loadings at the reduced bearing pressure.

Based on the proposed basement finish floor elevation of 1910 feet, the foundations for the addition will probably bear in medium dense to dense, native sandy GRAVEL. In our opinion, all new foundations for the Building 27 addition, bearing in this soil unit or in compacted Structural Fill, could be sized for an allowable net soil bearing pressure of 4000 psf. In addition, we recommend that existing footings that will be required to support additional new loads, be increased in size such that the design bearing pressure is not greater than 4000 psf. This value includes a factor of safety of about 3 and could be increased by one third for transient loading conditions such as those from earthquake or wind forces. It also assumes that individual square spread footings have a minimum dimension of 3 feet and continuous footings have a minimum width of 2 feet and that all footings have a minimum embedment of 1.5 feet below adjacent grade.

6.4 Foundation Subgrade Preparation

Excavating for footings may loosen or disturb the soil at the subgrade elevation. Disturbance could be kept to a minimum by using a smooth-edged, rather than toothed, excavating equipment. We recommend that loose or disturbed zones beneath bearing

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areas be removed and replaced with compacted Structural Fill. Alternatively, if the depth of disturbance is not greater than about 6 to 8 inches, the bearing area could be tamped with a small compactor to provide a firm base for the footing.

Foundation subgrade and bearing conditions should be evaluated by an experienced geotechnical engineer or engineering technician just prior to placement of concrete to confirm that the subgrade is suitable for bearing and that all loose materials have been removed. Preparation of foundation bearing surfaces, in the sandy GRAVEL, gravelly SAND, or compacted Structural Fill should include proof-tamping with a small, vibratory compactor to densify soils loosened during excavation.

6.5 Foundation Settlement

Foundation settlement analyses were conducted using elastic methods typically considered appropriate for granular soil conditions. Elastic moduli were estimated from Standard Penetration Test blow count data. These values compared relatively closely to modulus values calculated from the 1947 plate bearing test results. For foundations constructed and loaded as described above, we estimate that individual spread footings could experience potential settlements of less than 1/8 inch. For continuous footings, we estimate maximum potential settlement would be less than about 1/4 inch. Maximum differential settlement between adjacent footings could be about half these amounts. If loose or soft subgrade soils are present beneath footings, settlements greater than these amounts could occur.

Because all of the site soils encountered at the approximate bearing elevations are granular, we expect they will perform in a generally elastic manner when subjected to the new loadings. Settlements will probably occur as the loads are applied and most will probably be built-out of the structure during construction. Long-term foundation settlement is expected to be negligible.

6.6 Foundation Lateral Resistance

Lateral forces from earth, wind, or seismic loadings will be resisted by base friction and by passive earth pressure acting against the buried portion of spread footings. In our opinion, the passive earth pressure from compacted backfill against the sides of footings can be estimated by using a soil pressure of $300H$ psf, where H is the depth below the

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adjacent grade in feet. This loading will have a triangular distribution against the outside of the footing and stem wall, with a maximum pressure at the bottom. It includes a factor of safety of about 1.5 and assumes that all backfill around the footing consists of medium dense to dense, sandy GRAVEL, or compacted Structural Fill.

For sliding resistance at the base of footings, we recommend using a coefficient of sliding friction of 0.30 between mass concrete and foundation subgrade consisting of sandy GRAVEL, gravelly SAND, or compacted Structural Fill. This value also includes a factor of safety of about 1.5.

6.7 Seismic Design Criteria

The project site is located in Seismic Zone 2B, as identified by the 1994 Uniform Building Code. For structural design purposes, the seismic zone factor would be 0.20, the site coefficient would be 1.2.

6.8 Lateral Earth Pressures

Below-grade basement walls must be designed to resist lateral pressures resulting from the backfill soil as well as any surcharges that may be present. The magnitude of the pressure that will act against the wall will depend on its ability to yield, or rotate, (its stiffness) as well as the nature of the backfill against it. Below-grade walls should also be designed to resist additional lateral earth pressure surcharge effects from nearby existing footings.

For normal conditions, the total lateral pressure against a below-grade wall is dependent upon the method of backfill placement, the degree of backfill compaction, the backfill slope, the type of backfill material, wall drainage conditions, and whether or not the wall can yield laterally upon placement of the backfill. When a wall is restrained against horizontal movement, or tilting, the pressure against the wall is considered to be the "at-rest" pressure. The designer should consider the "at-rest" pressure condition if a rigid structural network of floors is constructed prior to backfilling against the wall. Restraint can also occur at wall corners, or as the result of the inherent stiffness of the wall.

If the wall is free-standing, or otherwise allowed to tilt or move horizontally, so that the top can move an amount equal to about 0.001 of its height, then the lateral pressure

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from the wall would reduce to the "active" value. This is the minimum lateral pressure the backfill soil will exert against the wall. For this type of wall, and assuming native soil or Structural Fill backfill, we recommend designing on the basis of an "active" equivalent fluid pressure of $35H$ psf, where H is the height of the wall in feet. Non-yielding walls should be designed using an "at-rest" equivalent fluid pressure of $55H$ psf. These recommendations assume that the walls are drained to prevent the build-up of hydrostatic pressure and do not include the effects of surcharges from adjacent surface loadings or additional loading from nearby existing footings. For intermediate degrees of restraint, an appropriate lateral pressure could be obtained by interpolating between the above two values. The load against the wall from either of these two cases would be applied as a triangular force with the maximum load at the bottom of the wall. §3.51

To account for lateral pressure surcharge from floor loadings in adjacent structures, we recommend calculating design wall pressures using a wall height 2 feet higher than actual. Additional lateral pressure surcharge effects from existing adjacent footings will depend on the bearing pressure as well as the relative locations of the new wall and the existing footings.

We considered surcharge lateral loads that could develop for two possible new basement wall locations along the north side of the existing building. Since the north end of Building 1 has the highest wall loadings, this represents the extreme condition for all the new basement walls.

The first case, considered locating the new basement wall adjacent to the outside edge of the existing wall footing. Unless the footing is underpinned, it would be necessary to shore the edge of the excavation to support the soil under the existing footing and behind it under the floor slab. If the existing footing is loaded such that its bearing pressure is about 6700 psf, the effect of this vertical loading would be to apply an inverted triangular lateral load against the new basement wall, with a maximum pressure of about 3400 psf at the top and decreasing to zero at the bottom of the wall.

A new basement wall at this location would have to be designed to support this surcharge load plus the normal at-rest soil pressure. To cover additional surcharge from the floor loading behind the existing footing, we recommend calculating the normal at-rest soil pressure on the basis of a fictitious 2 ft. increase in the height of the wall.

The second case, considered locating the new basement wall about 8 feet away from the edge of the existing north wall footing. In this case, it would be necessary to provide

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some temporary shoring to install the new wall (because of the 1.0V on 1.0H and 1.0V on 2.0H slope criteria) and the new wall would have to be designed to resist some lateral surcharge loading. The loading will be less and the distribution will be different than for the first case. If the existing Building 1 north wall footing is enlarged to reduce the bearing pressure to 4000 psf the lateral surcharge load on the new basement wall could be simulated as a uniformly distributed rectangular loading having a maximum pressure of 350 psf. This load would be added to the normal soil pressure against the new wall. Figure 7 shows lateral earth pressure recommendations in more detail.

6.9 Slabs-On-Grade

In our opinion, slab-on-grade floor loads could be supported on the medium dense to dense, sandy GRAVEL, or compacted Structural Fill. Structural Fill should be placed and compacted as described previously.

We recommend that the slab-on-grade subgrade surface be compacted to 95 percent of its Modified Proctor Maximum Dry Density (AASHTO T-180). If any loose or soft subgrade areas are encountered during compaction, they should be over-excavated and replaced with compacted Structural Fill. We believe that the sandy GRAVEL, gravelly SAND or Structural Fill compacted to 95 percent density, would provide a base such that a subgrade modulus of about 200 pci could be used for floor slab-on-grade thickness design.

We recommend that the slab foundation consist of a 4 inch layer of 3/4 to 3/8 inch, well graded gravel, placed directly over the compacted sandy GRAVEL, gravelly SAND, or compacted Structural Fill subgrade, to act as a capillary break. The gravel should be proof-compacted and the subgrade below it should be compacted to a minimum of 95 percent of the Modified Proctor.

Typical current design for slabs-on-grade frequently includes a vapor retarder over the top of the capillary break. If this is used, we recommend that the vapor retarder consist of a puncture resistant material with a permeance (ASTM E 96) of less than 0.30 perms. To provide space for bleed water from the slab and to protect the vapor retarder, we recommend covering it with about a 4 inch thick layer of well graded, proof-compacted Top Course, generally meeting the WSDOT 9-0.39(3) specifications for crushed surfacing, with the added requirement of less than 2 percent passing the No. 200 sieve. In our opinion, these additional provisions for slab-on-grade construction are important

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for interior floors that will be surfaced with tile, carpeting, or epoxy coatings.

6.10 Site Drainage

Runoff from roof drains or pavements should be collected and discharged into storm sewers, drainage swales, or other storm facilities. The project site is located in an area mapped as Springdale gravelly sandy loam by the U.S.D.A. Spokane County Soil Survey. This soil unit is reported to have a permeability greater than 20 ft./day. In our opinion, the permeability of the native soils at the estimated elevation for drywells in this general area is probably on the order of 100 to 500 ft./day. Drywells for storm drainage disposal should be used with appropriate detention facilities, in accordance with local drainage regulations. Final design of grading for the site should promote drainage away from the new structures.

If drywells are used for storm drainage disposal, we recommend they be located at least 20 ft. away from the proposed new structures. Tightlines for control of roof drainage from downspouts should be provided with clean-outs at strategic locations to allow for periodic maintenance.

The sandy GRAVEL and gravelly SAND underlying the site appears to be relatively permeable and well drained, except possibly in some localized areas where it is more silty. For this reason, we believe that foundations constructed at this site will not require footing drains. This recommendation, however, should be reevaluated during construction.

7.0 LIMITATIONS

The analyses, conclusions and recommendations contained in this report are based on our interpretation of subsurface conditions and assume that the information obtained from the exploratory borings is representative of subsurface conditions throughout the site. If subsurface conditions different from those encountered in the explorations and described herein appear to be present beneath the site during construction, we should be advised at once, so that we can review conditions and reconsider our recommendations.

If there is a substantial lapse of time between submission of this report and start of work at the site, or if conditions have changed due to natural causes or construction

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operations at or adjacent to the site, or if the building location is changed, we should be advised at once, so we can review conditions and reconsider our recommendations.

This report does not include a review of historical data, sampling, reconnaissance, or testing to assess the presence of any hazardous substances that may be present on the site.

We recommend that we be retained to review those portions of the final plans and specifications that pertain to earthwork and foundations to determine that they are consistent with our recommendations. We also recommend we be retained to observe foundation construction, particularly site preparation, subgrade excavation, Structural Fill placement, footing excavations, and other work for foundation field operations that may be necessary.

In addition, we have the capabilities to provide special inspection and testing services (UBC Section 1701) which may be required during construction. Such tasks often include inspecting reinforced concrete, structural masonry, structural steel welding, high strength bolting, and spray-applied fire-proofing. Our technicians have current ICBO certifications and recent experience in all of these important construction quality control areas.

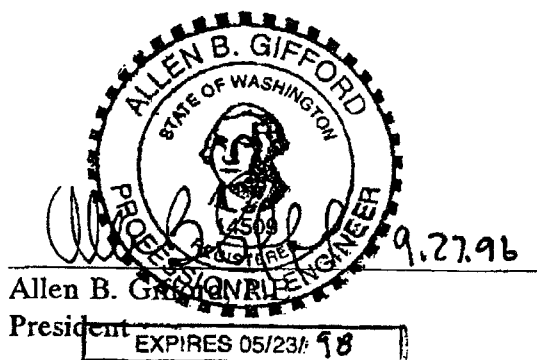
Unanticipated soil conditions are commonly encountered and cannot be fully determined by obtaining soil samples and making explorations. Unexpected conditions often require that additional expenditures be made to obtain a properly constructed project. For this project, we recommend establishing a contingency fund to accommodate potential unexpected conditions.

Sincerely,

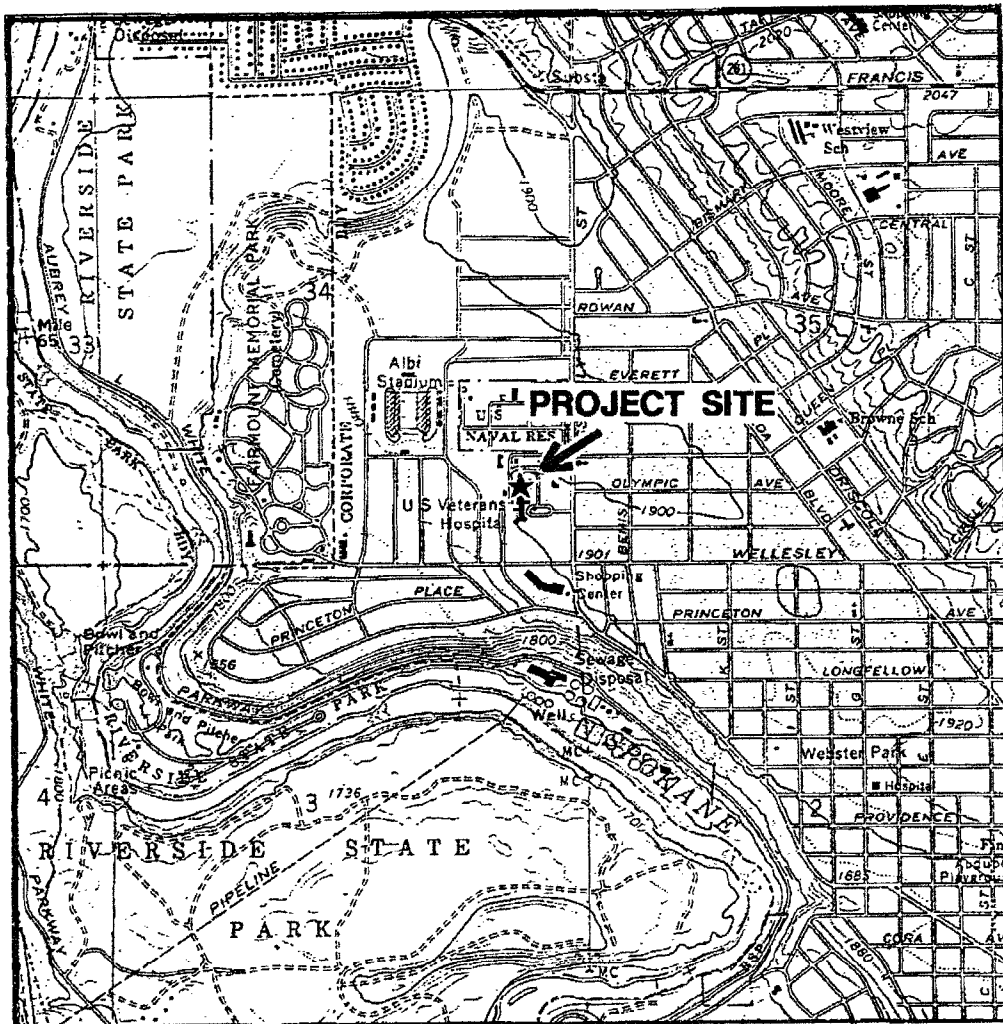
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David S. Phelps, E.I.T.
Geotechnical Engineer



Encls: Figures 1 through 7



SE 1/4, Section 34, T26N, R42E

BASE MAP FROM USGS 7.5 MIN. QUADRANGLE
 "SPOKANE NW, WASH." 1974 PHOTOREVISED 1986



SCALE 1"=2000'

V.A. HOSPITAL ADDITION - BUILDING 27
 Spokane, Washington

VICINITY MAP

SEPTEMBER 1996

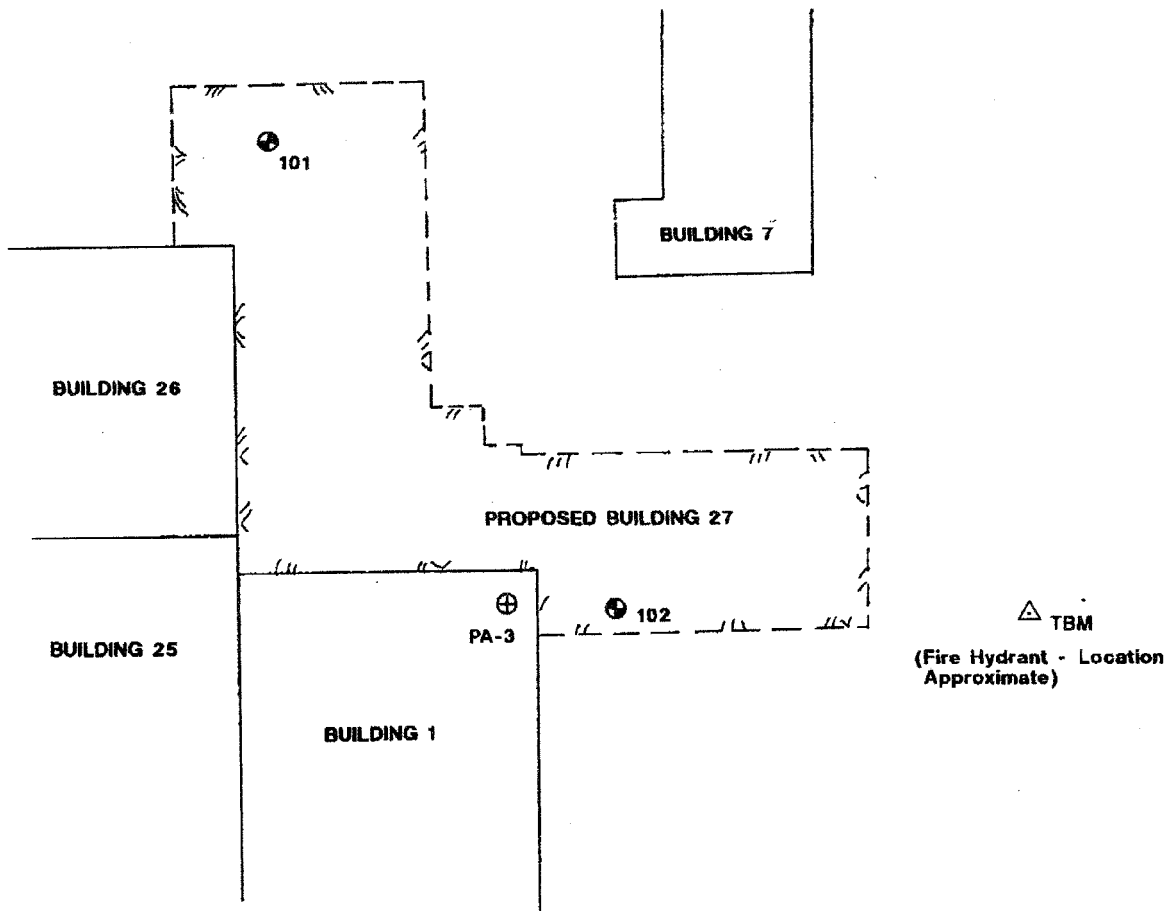
E-1747-01

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FIG. 1

Drawing Date 9-13-96 Drawn By JMG Checked By JSP Field Rep. JSP

LEGEND

- ⊕ 101 BORING LOCATION SEPTEMBER 1996
- ⊕ PA-3 BORING LOCATION SEPTEMBER 1947

DRAWING ADAPTED FROM SITE PLANS
PROVIDED BY ZECK-BUTLER ARCHITECTS



SCALE 1" = 40'

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Spokane, Washington

BORING LOCATION PLAN

SEPTEMBER 1996

E-1747-01

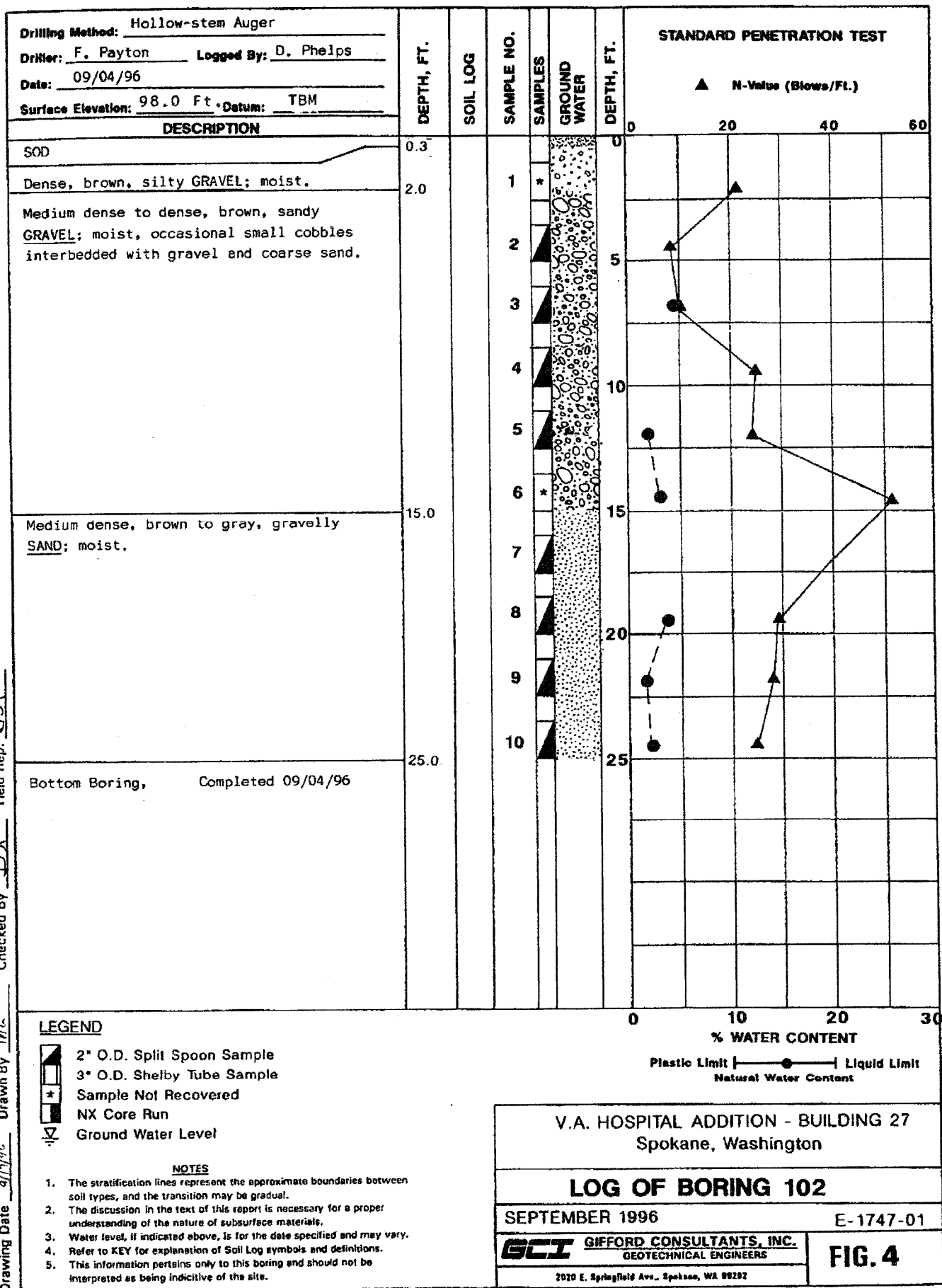
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FIG. 2

Drawing Date 9-13-96 Drawn By TILG Checked By JBP Field Rep. JBP

Drawing Date 9/17/96 Drawn By WAC Checked By JS? Field Rep. JS?


 Drawing Date 9/17/96 Drawn By TM Checked By DBP Field Rep. DBP

CLASSIFICATION OF SOIL CONSTITUENTS

- MAJOR constituents compose more than 50 percent, by weight, of the soil. Major constituents are capitalized (SAND).
- Minor constituents compose 12 to 50 percent of the soil and precede the major constituents (silty SAND). Minor constituents preceded by "slightly" compose 5 to 12 percent of the soil (slightly silty SAND).
- Trace constituents compose 0 to 5 percent of the soil (slightly silty SAND, trace of gravel).

MOISTURE CONTENT DEFINITIONS

Dry	Absence of moisture, dusty, dry to the touch.
Moist	Damp but no visible water
Wet	Visible free water, from below water table.

SOIL LOG SYMBOLS

	SOD
	SILTY <u>GRAVEL</u>
	GRAVELY <u>SAND</u>
	SANDY <u>GRAVEL</u>

GRAIN SIZE TERMINOLOGY

DESCRIPTION	SIEVE SIZE
FINES	< #200 (0.08 mm)
SAND* <ul style="list-style-type: none"> • Fine • Medium • Coarse 	<ul style="list-style-type: none"> • #200 - #40 (0.4 mm) • #40 - #10 (2 mm) • #10 - #4 (5 mm)
GRAVEL* <ul style="list-style-type: none"> • Fine • Coarse 	<ul style="list-style-type: none"> • #4 - 3/4" • 3/4" - 3"
COBBLES	3" - 12"
BOULDERS	> 12"

*Unless otherwise noted, sand and gravel, when present, range from fine to coarse in grain size.

RELATIVE DENSITY/CONSISTENCY

COARSE-GRAINED SOILS		FINE-GRAINED/COHESIVE SOILS	
N, SPT BLOWS/FT.	RELATIVE DENSITY	N, SPT BLOWS/FT.	RELATIVE CONSISTENCY
0 - 4	Very Loose	< 2	Very Soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium Dense	4 - 8	Medium Stiff
30 - 50	Dense	8 - 15	Stiff
Over 50	Very Dense	15 - 30	Very Stiff
		Over 30	Hard

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BORING LOG KEY

SEPTEMBER 1996

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FIG. 5

Drawing Date 9/17/96 Drawn By JMG Checked By JGP Field Rep. JGP

Drawn By TA/4 Drwg. Date 11/1/96 Checked By DSR Field Rep. DSR

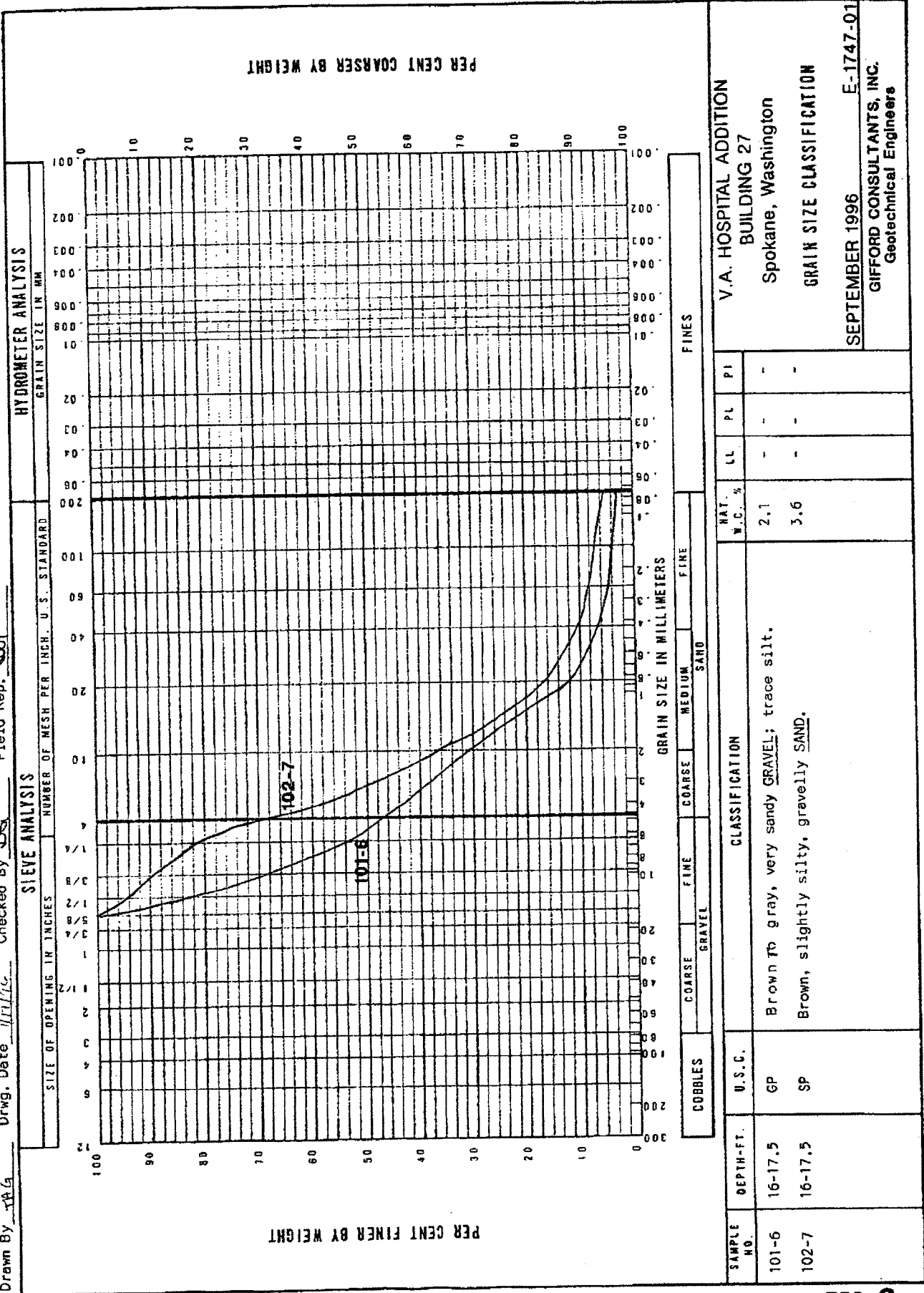
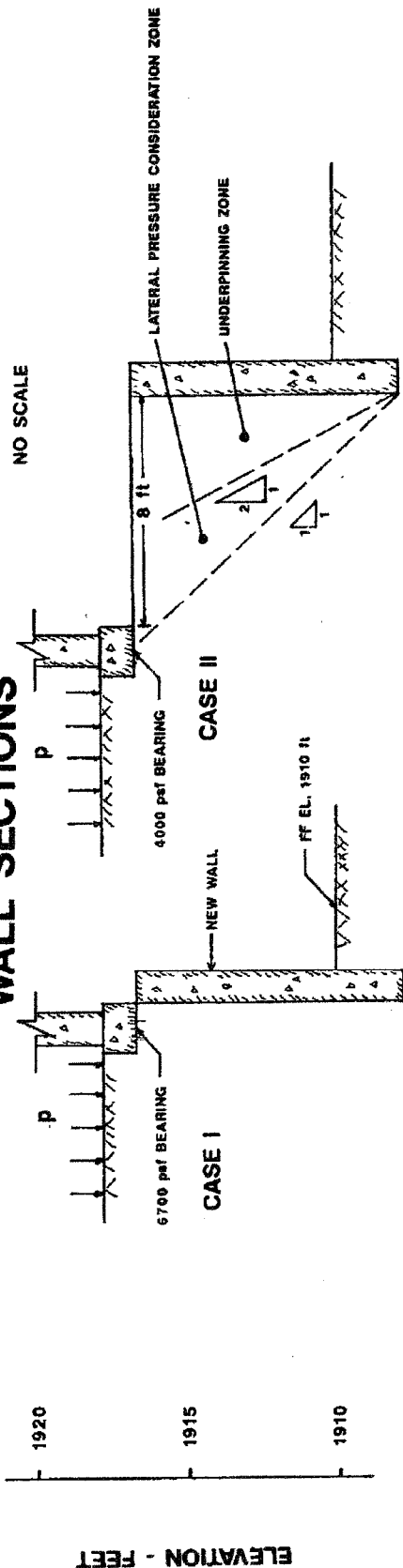


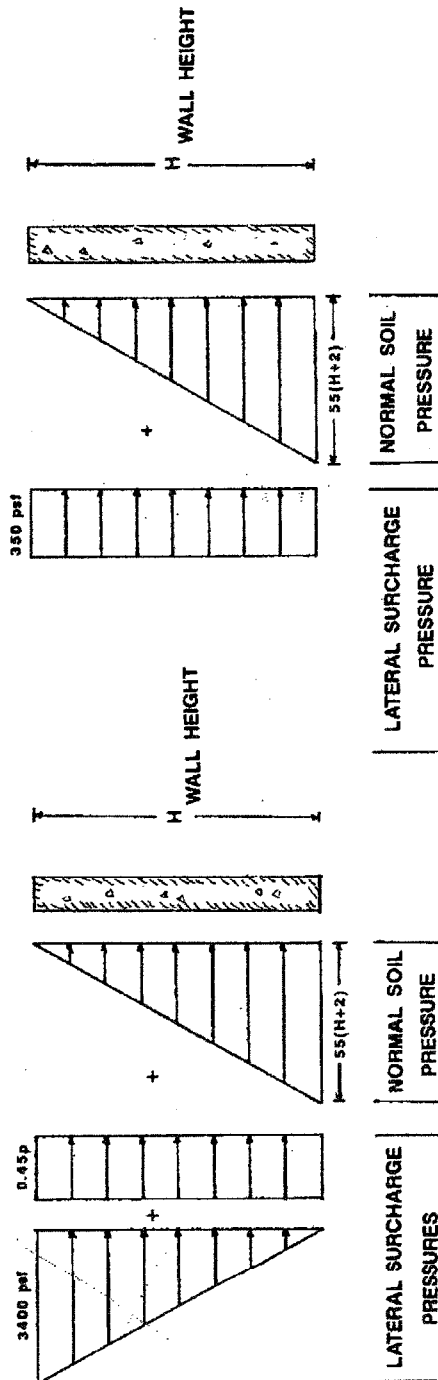
FIG. 6

Drawing Date 01/30/96 Drawn By DSP Checked By 431 Field Rep. DSP

WALL SECTIONS



LOAD DIAGRAMS



NOTE: PRESSURE DIAGRAM BASED ON THE FOLLOWING SOIL PROPERTIES:

$\phi = 35^\circ$
 $C = 0 \text{ psf}$
 $\gamma = 126 \text{ pcf}$

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Spokane, Washington

LATERAL EARTH PRESSURES

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FIG. 7

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FIG. 7



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November 13, 1996

Zeck-Butler Architects, PS
Paulsen Center, Suite 860
W. 421 Riverside Avenue
Spokane, Washington 99201

Attn: Mr. Kent Chadwell, AIA
Project Manager

VA HOSPITAL, PROPOSED BUILDING 27 ADDITION PROJECT, UNDERPINNING RECOMMENDATIONS

1.0 INTRODUCTION

This letter provides supplemental geotechnical recommendations for underpinning part of the north end of Building 1 at the Spokane VA Hospital prior to constructing the Building 27 Addition project.

Results of site explorations and initial geotechnical recommendations were provided in our report of September 27, 1996. At that time, design concepts assumed that new basement walls for Building 27 would be located sufficiently far away from the existing building so that underpinning Building 1 foundations would not be necessary. Review of costs and design issues related to this concept and the possibility of increasing the size of the new basement area, has resulted in a need to reconsider underpinning and develop design recommendations for accomplishing it.

2.0 UNDERPINNING CONCEPT

The present design concept envisions extending the new Building 27 basement area so that it abuts the northeast corner of Building 1. This will require underpinning about 15 feet along the east side and about 30 feet along the north side. In this area the existing foundation load is about 13 klf and the existing footing is about 1.5 feet wide.

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November 13, 1996

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Along most of this length, the footing bears on foundation soil at about elevation 1917 feet. At the west end of the north segment however, the footing steps down to about elevation 1910 feet to accommodate a utility duct. The finish floor elevation for the new basement is planned to be at about elevation 1910 feet.

Underpinning needs to be designed to support existing wall loads and expected future loads as well as the lateral earth pressure behind it. For this project, we recommend using continuous concrete wall underpinning by constructing incremental piers under the existing footing. The existing foundation loads would be transferred through the underpinning to bear on deeper strata. The usual procedure is to dig an approach pit in front of the footing to be underpinned. The pit is then extended under the footing so that the underpinning pier is centered beneath it. The sides of the pit are shored during excavation to minimize loss of ground. Once the pit is completed to required depth, the front of the slot is shored, and the pit is filled with concrete up to within about 3 inches of the under side of the footing. After the concrete has set, the slot between the bottom of the footing, and the top of the concrete underpinning pier is filled and tamped with dry pack concrete grout. The process is repeated in incremental fashion until the footing is continuously supported by the underpinning.

3.0 DESIGN LOADS

For this project, the underpinning must satisfy several design goals:

- Reduce contact pressure on the foundation soil.
- Support additional load from the Building 27 addition.
- Support the lateral earth pressure behind the underpinning wall.

The existing foundation supports an estimated wall load of about 13 klf. This means that the present foundation/soil contact pressure is about 9 ksf. We understand that the expected new load from the Building 27 addition could add an additional 5 klf to the existing foundation load. In addition, the net load increase from an 8 ft. high concrete underpinning element could add 0.5 to 1.0 klf, depending on its width. Analyses show that an underpinning wall, bearing at about elevation 1909 feet, should be sized for a foundation soil bearing pressure of 6.5 ksf in order to maintain a factor of safety of about 3 ($SF = 3$), the value typically used for soil bearing capacity. A 3 ft. wide underpinning wall would satisfy the requirement to reduce the current soil contact pressure and support the additional new loading.

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November 13, 1996

In our opinion, the underpinning wall should be designed to support an at-rest earth pressure, based on an equivalent fluid pressure of $55H$ psf. Where H is measured in feet. We recommend that the effective height of the wall be increased by 2 feet ($H+2$) to account for possible floor loads in the crawl space behind the wall where materials and supplies may be stored. Both the permanent wall elements (underpinning piers) and the shoring for the excavations necessary to construct the piers, should be designed for the $55(H+2)$ lateral loading. Based on Building 1 drawings, the nearest interior columns are located about 15 to 20 feet behind the existing foundation line. In our opinion, at that distance their influence on lateral pressure surcharge against the underpinning wall would be insignificant.

The lateral earth pressure against the underpinning piers and wall will be resisted by friction at the bottom, (between the concrete and the foundation soil), and at the top (by friction between the concrete underpinning pier and the bottom of the existing concrete footing). We recommend using a friction factor of 0.3 for the underpinning pier concrete/foundation soil contact. The friction factor for the pier concrete/footing concrete contact at the top of the underpinning piers would be considerably more. Preliminary calculations show that the pier concrete/foundation soil resistance would be more than sufficient to support the expected lateral earth pressure from the soil.

4.0 CONSTRUCTION

In our opinion, the maximum length of wall that could be temporarily left unsupported during underpinning pier construction should not be more than about 3 or 4 feet. The unsupported segments should be equally distributed along the length of the wall. The work should be accomplished so that the sum of the total of the unsupported lengths is never more than 20 percent of the total wall length. We recommend that piers be constructed in groups of six. The excavation sequence should begin with the first and fourth piers, then the second and fifth, and finally the third and sixth. To a very large extent the success of the work will depend on the care the Contractor takes installing shoring for the sides and backs of the slot excavations as well as his general care in excavating.

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November 13, 1996

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5.0 CONSTRUCTION MONITORING

Because the proposed new structure involves excavating adjacent to the foundations of an important existing building, there is concern for its safety. Any time an excavation is made below the footings of an existing adjacent structure, there is a risk that damage to the structure may occur even though a well designed system of shoring and underpinning is planned in advance. We recommend therefore, that a program of systematic observations be made prior to, and during construction, to measure performance of the existing building foundations during underpinning. This is important; if excessive movement is detected, remedial measures must be taken promptly to prevent damage to the existing building.

In our opinion, monitoring should include optical measurements to determine horizontal and vertical movements of points installed along the footing of the existing structure. The measuring system should have precision of at least 0.01 inch. We recommend installing the points a maximum of 10 feet apart and as low as possible on the existing footing. Survey measurements should be made from a baseline that can be easily reestablished from points located outside the influence of the construction work.

Several weeks prior to starting construction we recommend that the Owner and the Contractor make a structural inspection of the north end of the existing building. The inspection should be directed towards detecting any signs of damage, particularly those caused by settlement. Notes and photographs should be taken. Where necessary, crack monitoring gages should be installed. As construction starts, we recommend monitoring the survey points established along the top of the existing footing to determine horizontal and vertical displacements. Measurements should be made using conventional optical surveying equipment.

Several sets of measurements should be obtained prior to starting the general excavation in order to establish a data baseline from which to determine any changes that may occur. At the beginning of the underpinning work, we recommend monitoring measurements be made on a daily basis. For monitoring to be successful and useful, the data must be promptly analyzed. It is also important that the construction contract clearly spell out the responsibilities of the Owner, the Owner's Representative, and the Contractor regarding who is to make inspections, take readings, review data, and repair possible damage.

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6.0 LIMITATIONS

The supplemental recommendations described in this letter are based on our understanding of the current foundation concept for Building 27. It is subject to the same limitations described in our geotechnical report of September 27, 1996.

We appreciate the opportunity to be of service to you on this project. If you have any questions about this letter or we can be of any further assistance, please let us know.

Sincerely,

GIFFORD CONSULTANTS, INC.



Allen B. Gifford, P.E.
President



GIFFORD CONSULTANTS, INC.
Geotechnical Engineers

E-1747-01

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October 29, 1996

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Attn: Mr. Kent Chadwell, AIA
Project Architect

**COMMENTS REGARDING UNDERPINNING BUILDING 27 ADDITION PROJECT,
VETERANS ADMINISTRATION HOSPITAL, SPOKANE, WASHINGTON**

1.0 GENERAL

Below grade basement walls must be designed to support lateral earth pressure from soil backfill on the outside of the wall. They must also support surcharged lateral earth pressures from adjacent foundations or other loadings that lie within an influence zone outside of the wall. The magnitude of the lateral surcharge load is a function of the soil bearing pressure on the existing adjacent foundations. For the case of the Building 27 Addition Project, the surcharge lateral pressure from the existing foundation wall is a significant design load that depends on where the new basement wall is located. Lateral pressures from existing adjacent foundations also have significant implications for excavations (and shoring) that may be necessary to provide space for constructing the permanent below grade walls.

Structural engineering information shows that the Building 1 foundations apply soil bearing pressures on the order of about 9,000 psf. In our opinion, this value is considerably higher than what would normally be used in today's practice for the same soil, footing width and depth conditions. Although the building and the foundations have apparently performed well over the years, based on the information we have, it is unlikely that the same bearing pressure could be justified for new design.

2.0 UNDERPINNING

Underpinning is the installation of additional support to an existing foundation to deepen or improve its bearing capacity. Deepening or enlarging an existing foundation because of deeper, new adjacent construction is called precautionary underpinning.

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October 29, 1996

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Page 2

Underpinning to improve support beneath structures that have already settled, is called remedial underpinning. In our opinion, precautionary underpinning should be done at the north end and part of the northeast corner of Building 1 prior to making the basement excavations for the proposed new addition.

To reduce the design and construction problems related to the surcharge effects caused by the footing loads of the adjacent foundations, underpinning could be combined with the new basement wall construction for the addition. This would increase the basement area, provide shoring for the basement excavation work and simplify wall design. It could also be used to reduce the unit bearing pressure on the foundation soil.

In our opinion, underpinning of the existing basement wall should be continuous and should extend from the bottom of the existing footing to an appropriate depth below the bottom of the finish floor in the proposed new basement. We envision that this work would consist of making a series incremental narrow slot excavations (about 3-4 feet wide) under the footing and filling them with concrete. The usual procedure is to dig an approach pit in front of the footing to be underpinned. The approach pit is then extended under the footing so that the concrete underpinning pier will be centered beneath it. The sides of the pit are shored during excavation to minimize loss of ground. Once the pit is completed to the required depth, the front of the slot is shored and the pit is filled with concrete up to within about 3 inches of the underside of the footing. After the concrete has set, the slot between the bottom of the existing footing and the top of the concrete underpinning pier is filled and tamped with dry pack concrete grout. The process is repeated in incremental fashion until the entire wall is continuously supported at the planned deeper depth.

We hope the information in this letter is sufficient for your needs at this time. If you have any questions or we can be of any further assistance, please let us know.

Sincerely,

GIFFORD CONSULTANTS, INC.



Allen B. Gifford, P.E.
President