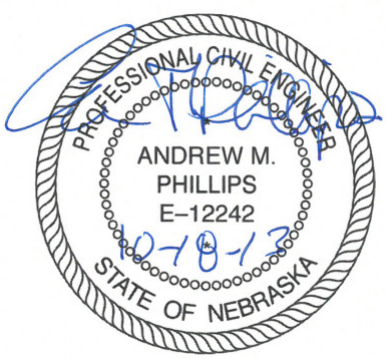
REPORT OF GEOTECHNICAL EXPLORATION

NDOR FACILITY ADDITION

NEAR THE INTERSECTION OF
COLFAX AVENUE AND NORTH DEPOT STREET
DORCHESTER, NEBRASKA

PREPARED FOR
THE ARCHITECTURAL OFFICES

PREPARED BY
OLSSON ASSOCIATES



October 18, 2013

OLSSON PROJECT NO: 013-2274

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A. PROJECT UNDERSTANDING

A.1. GEOTECHNICAL SCOPE

This report presents the results of the geotechnical subsurface exploration performed for the proposed NDOR building addition and pavement located northeast of the intersection of Colfax Avenue and North Depot Street in Dorchester, Nebraska.

The purpose of this exploration was to evaluate the subsurface conditions and provide recommendations regarding the design of foundations and floor slabs of the proposed building addition and pavement. We have completed the following scope of services for this project:

- Performed a site reconnaissance and reviewed geologic subsurface conditions.
- Drilled two soil test borings to depths ranging from 15 to 21.5 feet in the proposed building addition.
- Drilled one soil test boring to a depth of 10 feet in the proposed pavement areas.
- Performed laboratory tests on selected soil samples obtained during the drilling operations.
- Conducted a geotechnical engineering evaluation using information obtained from our observations, soil test borings and laboratory tests, and information available regarding the proposed construction.
- Preparation of this Report of Geotechnical Exploration presenting the soil test borings, laboratory test results, and a summary of our engineering evaluations and recommendations.

The scope of this exploration did not include any environmental assessment for the presence of wetlands and/or hazardous or toxic materials in the soil or groundwater on or near the site. Any statements in this report regarding odors, discoloration, or suspicious conditions are strictly for the information of our client.

A.2. SITE LOCATION AND DESCRIPTION

The proposed NDOR building addition is located northeast of the intersection of Colfax Avenue & North Depot Street in Dorchester, Nebraska. At the time of our field exploration, the site consisted of grass vegetation, trees, existing NDOR building structures, asphalt pavement, and concrete paving. The site was accessible with a truck-mounted drill rig. The approximate locations of the site are depicted on the Site Location Plan included in *Appendix A*.

A.3. PROJECT INFORMATION

The proposed NDOR facility addition project will include a new 2,000 square foot building addition. It was assumed that no portion of the building would include a basement. The new addition will be built at the location of the current NDOR office, south of the existing garage bays. The existing structure, not including the garage bays, will be demolished prior to construction of the new addition. The approximate location of the proposed building addition areas are depicted on the Boring Location Map included in Appendix A.

The ground in the area of the proposed building addition is at an approximate elevation of 99.0 and we have assumed a finished floor elevation (FFE) of 100.0, which matches the existing building's FFE. Based on the survey information completed by an ***Olsson*** drilling crew, the ground in the proposed buildings footprint is fairly level. Based on the anticipated finished floor elevation (FFE) of 100.0 for the new building addition and the boring elevations obtained by an ***Olsson*** drilling crew, a maximum of 12 inches of new fill and little to no excavation will be necessary in the building area.

Based upon conversations with Architectural Offices, the new addition will be supported on precast continuous footings. Structural design loads were not provided; however, based on our experience with similar structures, the maximum continuous wall load is not anticipated to exceed approximately 5 kips per lineal foot. If structural loads exceed the assumed value, ***Olsson*** should be contacted to verify that the recommendations contained in this report are valid.

Traffic loads on the proposed parking lot and driveway pavements were assumed to consist of automobiles, delivery vans, and garbage trucks. Since specific traffic volumes were not provided, the frequency of each vehicle driving on the proposed pavements was assumed to be similar to the traffic loads utilized in similar developments.

B. Exploratory and Test Procedures

B.1. Field Exploration

The field exploration program consisted of performing three soil test borings at the locations depicted on the Boring Location Plan presented in *Appendix A*. Ground surface elevations were surveyed by an ***Olsson*** drilling crew using the finished floor of the existing NDOR building as a benchmark. The finished floor of the existing building was assumed to be at elevation 100.0. Elevations were rounded to the nearest half foot.

The soil test borings were extended to depths ranging from 10 to 21.5 feet below the existing ground surface with a truck-mounted drill rig using continuous-flight augers. Soil samples were obtained at selected intervals in the soil test borings. Soils samples designated as “U” samples on the boring logs were obtained in using Thin-Walled Tube Sampling techniques. Soil samples designated as “SS” samples were obtained during Penetration Test using a Split-Spoon Barrel sampler. Recovered samples were sealed in containers, labeled, and protected for transportation to the laboratory for testing.

B.2. Laboratory Testing

Descriptions of the soils encountered in the soil test borings were prepared using Visual-Manual Procedures for Descriptions and Identification of Soils. Laboratory tests were also performed to evaluate the engineering properties for the recovered soil samples. Moisture content tests and density determinations were used to determine the existing moisture/density condition of the soils. Unconfined compression tests were used to help define the stress-strain characteristics and related shear strength of the cohesive soils. Two Atterberg limits tests were conducted to aid in the classification of the soils under the Unified Classification System and to indicate the shrink/swell potential of the soils. One consolidation test and one consolidation/swell test was conducted to determine the potential settlement and swell of the proposed structure. A summary of the laboratory test results is presented in *Appendix C*.

C. SUBSURFACE CONDITIONS

C.1. AREA GEOLOGY

The site is located in the Hill slopes region of Nebraska at the interface of the Crete silt loam soils. These soils are known to be moderately well drained and have a moderately low to moderately high permeability. The majority of the site was most recently utilized as grass vegetation and asphalt paving at the time of drilling operations.

C.2. TEST BORINGS AND LABORATORY SUMMARY

The generalized subsurface profile for this site, in descending order, generally consisted of grass vegetation or developed zone, asphalt, and concrete overlying fill, altered Peoria loess, Peoria loess, and Loveland formation soils. Specific soil descriptions are noted in more detail on the Soil Test Boring Logs in *Appendix* B.

Developed Zone

A developed zone with approximately 3 to 4 inches of topsoil was encountered in soil test boring B-1, which consisted of varying amounts of organics and roots. Organic material is typically considered unsuitable for structural support or for use as structural fill due to its high organic content. It should be noted that the developed zone encountered in the soil test borings is to be stripped and stockpiled outside of the construction area prior to the placement of structural fill.

Asphalt

Asphalt extended approximately 3 to 6 inches below the existing ground surface and was encountered in soil test boring B-2 and B-3.

Concrete

Concrete extended approximately 4 to 5 inches below the existing ground surface and was encountered in soil test boring B-2.

Fill

Existing fill was encountered in soil test boring B-1 and extended to a depth of 2.0 feet below the existing ground surface. Soils identified as fill were generally stiff, dark brownish grey, very moist, mostly fat clay with trace fine to coarse sand. Laboratory tests on recovered samples from this stratum depicted a moisture content of 25.5 percent and a dry density of 94.6 pounds per square foot (psf). An Atterberg limits test conducted on the fill material indicated a liquid limit of 53, a plastic limit of 23, and a plasticity index of 30.

Altered Peoria Loess

Altered Peoria loess was encountered in each of the soil test borings and extended to depths ranging from 1.5 to 3.0 feet below the existing ground surface. Soils identified as altered Peoria loess were generally stiff, yellowish brown to dark brown, moist, mostly lean clay with trace fine sand. Laboratory tests on recovered samples from this stratum depicted a moisture content of 24.8 and a dry density of 97.7 pcf, and unconfined compressive strength value of 2.5 tons per square foot (tsf). The unconfined compressive strength values indicated a very stiff consistency for the altered Peoria loess soils.

Peoria Loess

Peoria loess was encountered below the altered Peoria loess material in each of the soil test borings and extended to a depth ranging from 16.5 feet below the existing ground surface to the base of soil test borings. Soils identified as Peoria loess were generally firm to stiff, yellowish brown, moist, mostly lean clay with varying amounts of silt, fine sand, iron, and calcium. Laboratory tests on recovered samples from this stratum depicted moisture contents ranging from 24.1 to 31.8, dry densities ranging from 88.7 to 95.4 pcf, and unconfined compressive strength values ranging from 0.6 to 1.4 tsf. The unconfined compressive strength values indicated a firm to stiff consistency for the Peoria loess soils. An Atterberg limits test conducted on the Peoria loess indicated a liquid limit of 48, a plastic limit of 21, and a plasticity index of 27.

Loveland Formation

Loveland formation was encountered below the Peoria loess in soil test boring B-3 and extended to the base of the soil test borings. Soils identified as Loveland formation were generally firm to hard, reddish brown, moist, mostly lean to fat clay with trace fine sand, iron, and calcium.

C.3. GROUNDWATER SUMMARY

Groundwater was not encountered in the soil test boring at the time of drilling operations. Groundwater levels will fluctuate depending on seasonal variations of precipitation and other factors that may occur at higher elevations at some time in the future. ***Section D.4*** of this report will address any site drainage concerns with water elevations in this region.

D. Site Preparation

D.1. Demolition of the Existing Building

Initial site preparation for the construction site should commence with demolition/relocation of the existing utilities and the existing structure within the proposed construction area. Demolition should include removal of any at grade slabs, gravel and concrete associated with the building, driveways, sidewalks, and utilities. All broken concrete and other debris from the demolition operations should be removed from the site and disposed in accordance with applicable regulations. All disturbed soils should be undercut prior to placement of structural fill. All utility lines and driveways should be relocated to areas outside of the proposed construction area. Excavations created by removal of the existing lines should be cut wide enough to allow for use of construction equipment to recompact the structural fill. In addition, the base of the excavation should be thoroughly evaluated by an ***Olsson*** geotechnical engineer or engineering technician prior to the placement of structural fill. All fill should be placed in accordance with the recommendations presented in the section *D.4* of this report.

D.2. Site Preparation - Structure

In all new fill and excavation areas, vegetation, roots, topsoil (typically 0.5 feet thick), and other deleterious materials deemed unsuitable by the full-time field observer shall be removed from the proposed construction areas, and replaced with controlled fill. Site clearing, grubbing, and stripping will need to be performed only during dry weather conditions. Operation of heavy equipment on the site during wet conditions could result in excessive rutting and mixing of organic debris with the underlying soils.

Based on the laboratory results and site observations made during drilling operations, the existing fill soils encountered in the upper 2.0 feet below the existing ground surface were found to be variable in density and moisture content. Therefore, we recommend overexcavating the existing fill material in the proposed addition areas to a depth of 2.0 feet below the existing ground surface. The excavations of the existing fill should be extended horizontally 5 feet beyond the perimeter of the building. The sides of the excavation should be sloped at a 1(H):1(V) to permit controlled earth fill to be placed against the sides of the excavations to the specified degree of compaction as stated in the *Structural Fill* section of this report. Backfill placed on sloped areas within the addition shall be “benched” horizontally a minimum of two feet into the sides of the slope, so that the lifts of backfill are placed and compacted in as nearly a horizontal place as possible. Lifts of backfill material shall be placed and compacted in such a

fashion to allow overlying lifts to interlock with the underlying soils to reduce consolidation potential along the slope interface. In addition to the fill placement along the slopes, the base of the excavation should be at least wide enough to allow for compaction equipment to be utilized. A representative from ***Olsson*** should observe the area to document conformation with the above recommendation.

Care should be taken not to disturb the bearing soils beneath the existing building foundations. It is recommended that, where possible, excavations below existing footings not extend below an imaginary plane extending out and down from the outside edge of existing footings at a slope of approximately 2H:1V. Even with these criteria, excavations that extend below the level of existing foundations should be backfilled the same day they are excavated. Where this is impractical, shoring or underpinning of existing foundations may be required.

Once the base of the core-out operations is achieved or before placement of the new structural fill within the building area, we recommend scarifying and recompacting the upper 12 inches of the exposed soil subgrade. The soil subgrade should be compacted to at least 98 percent of the maximum standard proctor dry density at a moisture content ranging from optimum to +3 percent of optimum. We recommend contacting ***Olsson*** to determine if the base of the core-out areas should be proofrolled. To provide a working base in the core out area additional remedial measures may be necessary. Typical stabilization methods may include lifts of crushed stone material, either alone or in combination with geofabric or geogrid, overlying the soft soils. The identification of areas that may require undercutting and/or stabilization should be based on the actual conditions at the time of construction, and will depend on the location and extent of the soft area. It is recommended the appropriate proofrolling operation and stabilization method be determined after the core-out has been performed and depending on the visual observations noted during the core-out operation.

Based upon Atterberg limits results, the excavated fill soils appear to be unsuitable for re-use as structural fill. Based upon the results of the Atterberg limits and swell tests results, it appears that the Altered Peoria loess soils will be suitable for re-use as structural fill within the building area as long as it meets the requirements listed in section *D.4*. The suitability of the on-site soils for reuse as structural fill material should be verified with additional Atterberg limits test during grading operations by a representative of ***Olsson***.

D.3. Site Preparation – Parking Lot

In all new fill areas within the parking lot and drive lanes, vegetation, roots, topsoil (typically 6 to 8 inches thick), and other deleterious materials deemed unsuitable by the full-time field observer shall be removed and replaced with controlled fill. Site clearing, grubbing, and stripping will need to be performed during dry weather conditions. Operation of heavy equipment on the site during wet conditions could result in excessive rutting and mixing of organic debris with the underlying soils.

After the removal of topsoil and prior to the placement of new structural fill, we recommend scarifying and recompacting the upper 12 inches of the exposed soil subgrade. The soil subgrade should be compacted to at least 98 percent of the maximum standard proctor dry density at a moisture content ranging from optimum to +3 percent of optimum. After compaction of the subgrade, the parking and drive lane areas should be proofrolled with a loaded dump truck or similar rubber-tired equipment weighing at least 15-tons. Proofrolling operations should be observed by an ***Olsson*** representative. Unstable or otherwise unsuitable soils revealed by proofrolling should be scarified, allowed to dry, and then recompacted, or removed and replaced with new structural fill, or stabilized in place under the direction of ***Olsson***. Typical stabilization methods, if required, may include lifts of crushed stone material, either alone or in combination with geofabric or geogrid, overlying the soft soils. The identification of areas that may require undercutting and/or stabilization should be based on the actual conditions at the time of construction and will depend on the location and extent of the unstable area.

D.4. Structural Fill

During construction, we recommend that off-site borrow soils used as structural fill materials have a liquid limit less than 45 and a plasticity index less than 25 and that on-site soils that will be used as structural fill have a liquid limit less than 50 and a plasticity index less than 30.

In addition to the plasticity characteristics, the fill soils should be relatively free of organic materials (less than about two percent by weight) or other deleterious materials and should not contain particle sizes larger than 3 inches. Imported fill material should be tested prior to placement at the site to verify it complies with the criteria stated in this section of the report. Samples of the proposed imported structural fill should be submitted at least three days prior to placement so the necessary laboratory tests can be performed.

Suitable fill material should be placed in thin lifts (lift thickness depends on type of compaction equipment, but in general, lifts of 8 inches loose measurement is recommended). The soil should be compacted by heavy compaction equipment such as a Caterpillar 815 sheepsfoot roller. Within small excavations, such as in utility trenches (less than 24 inches in width), around manholes or behind retaining walls, we recommend the use of "wacker packers", "Rammax" compactors, or vibrating plate compactors to achieve the specified compaction. Loose lift thickness of 4 inches are recommended in small area fills.

We recommend that structural fill and backfill be compacted in accordance with the criteria stated in *Table 1*. A qualified field representative should periodically observe fill placement operations and perform field density tests concurrently to indicate if the specified compaction is being achieved.

TABLE 1

STRUCTURAL FILL PLACEMENT GUIDELINES

Areas of Fill Placement	Compaction Recommendation (ASTM D698-Standard Proctor)	Moisture Content (Percent of Optimum)
Granular Cushion Beneath Floor Slab (If used)	98%*	As necessary to obtain density
Floor Slab/Soil Subgrade – 1' below the base of the granular cushion (if used) or Floor Slab	98%	Optimum to +3 percent
Structural fill placed below the Floor Slab Subgrade and within 5 feet of the perimeter of the building pad	98%	Optimum to +3 percent
Pavement Soil Subgrade – 12" below the base of the granular subbase	98%	Optimum to +3 percent
Structural fill placed below the base of the Pavement Soil Subgrade	95%	Optimum to +3 percent
Utility Trenches	98%	Optimum to +3 percent
Beneath Landscaped/Grass Areas	92%	As necessary to obtain density

The moisture content of suitable borrow soils should generally be between the specified ranges in *Table 1*. More stringent moisture limits may be necessary with certain soils. Adjustment of moisture content may be necessary to allow compaction in accordance with project specifications. Dependent on the percentage of fines, the clean free-draining aggregates utilized in the granular cushion beneath the floor slab could alternatively be consolidated by means of a vibratory compactor to at least 70% "relative density", as determined in accordance with ASTM D 4253 (Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table) and D 4254 (Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculations of Relative Density).

D.5. Drainage and Groundwater Considerations

At the time of our drilling operations, groundwater was not encountered in the soil test borings and it is not anticipated that groundwater will likely impact construction operations. We note that groundwater levels will fluctuate depending on seasonal variations of precipitations and other factors that were not evident at the time of drilling and may occur at higher elevations in the future which could potentially impact construction operations.

In general, water should not be allowed to collect near the surface of the foundation or floor slab area of the structure during or after construction. Since soils generally tend to soften when exposed to free water, provisions should be made to remove seepage water from excavations, should it occur. Also, undercut or excavated areas should be sloped toward one corner to facilitate the collection and removal of rainwater or surface runoff.

Additionally, in order to minimize concerns related to improper drainage away from the building foundations and floor slab subgrades that tend to soften when exposed to water or can heave expansive soils, we provide the following general recommendations:

- Site grading should provide for efficient drainage of rainfall away from the building and pavement areas, with a minimum slope of 2% for pavement areas and 5% slope for grass or landscaped areas.
- Roof run-off should be collected and transferred directly to the storm sewer system, if possible, or to a location well away from the building. Conventional downspout drainage leading to splash blocks, though not as desirable, may be used.
- Install clay plugs in each utility trench for utility line penetrations entering the building pad. The compacted clay plug should extend a minimum distance of five feet out from the building exterior.
- External hose connections should incorporate splash blocks to prevent localized accidental flooding of foundation soils. External hose connections should have cutoff valves inside the building to prevent accidental or unauthorized use of external hose connections.
- Building maintenance personnel should be informed of the potential problems associated with watering in close proximity to the building. Excessive watering of shrubs or lawns near buildings should be avoided. Placement of deep-rooted or water-intensive shrubs near buildings also should be avoided.

D.6. Construction Equipment Mobility

Some of the soils encountered at this site may be highly susceptible to softening under the action of construction equipment traffic in combination with wet weather. Mitigation of equipment mobility problems and management of soft surficial soils will be greatly dependent on the severity of the problem, the season in which construction is performed and prevailing weather conditions.

Some general guidelines for reducing equipment mobility problems and dealing with soft, wet, surficial soils are as follows:

- Optimize surface water drainage at the site.
- Whenever possible, wait for dry weather conditions to prevail, and do not operate construction equipment on the site during wet conditions. Rutting the surface will only aggravate the problem.
- Use construction equipment that is well suited for the intended job under the site conditions. Heavy rubber-tired equipment typically requires better site conditions than light, track-mounted equipment, especially on granular and less cohesive soil conditions.
- Implement a construction schedule that realistically allows for rain days. Pressure to perform earthwork under a tight schedule is frequently counterproductive.

Ultimately, it may be necessary to take steps to aggressively improve construction mobility if construction must proceed under unfavorable conditions. Methods for addressing equipment mobility problems may range from removing several feet of soft wet soils, to utilizing crushed stone materials and/or appropriate stabilization fabrics. Other methods include cement modification of soils, lime stabilization, etc. The optimal approach should be determined by a representative of the geotechnical engineer at the time of construction.

Any additional disturbance below the proposed subgrade elevation should be the responsibility of the general contractor to stabilize by means which are in accordance with the project specifications and this report. Any site or soil conditions which are a result of adverse weather conditions and which may require additional measures to improve construction mobility and site conditions should be the responsibility of the general contractor. The contractor should not allow water to collect near the surface of foundation or floor slab areas, either during or after construction. Site grading should be designed to provide rapid and efficient drainage of water away from the building and pavement areas at all times during construction.

D.7. Temporary Slopes and Excavations

The owner and the contractor should make themselves aware of and become familiar with applicable local, state, and federal safety regulations, including current OSHA excavation and trench safety standards. Construction site safety generally is the sole responsibility of the contractor. The contractor shall also be solely responsible for the means, methods, techniques, sequences, and operations of construction operations. ***Olsson*** is providing the following information solely as a service to our client. Under no circumstances should ***Olsson's*** provision of the following information be construed to mean that we are assuming responsibility for construction site safety or the contractor's activities, as such responsibility is not implied and should not be inferred.

The contractor should be aware that slope height, slope inclination, and excavation depths (including utility trench excavations) should in no case exceed those specified in local, state, or federal safety regulations; e.g., *OSHA Health and Safety Standards for Excavations, 29 CFR Part 19266,* or successor regulations. Such regulations are strictly enforced and, if not followed, the owner, the contractor, or earthwork or utility subcontractors could be liable for substantial penalties.

For this site, the overburden soil encountered in our exploratory borings consisted of stiff lean clay. We anticipate that OSHA will classify the cohesive soils as type B. OSHA recommends a maximum slope inclination of 1(H):1(V) for type B soils.

Note: Soils encountered in the construction excavations may vary significantly across the site. Our preliminary soil classifications are based solely on the materials encountered in the widely spaced boring locations. The contractor should verify that similar conditions exist throughout the proposed area of excavation. If different subsurface conditions are encountered at the time of construction, ***Olsson*** recommends that they be contacted immediately to evaluate the conditions encountered. If any excavations, including utility trenches, are extended to depths of more than 20 feet, OSHA requires that the side slopes of such excavations be designed by a professional engineer registered in the state where construction is occurring.

As an alternative to temporary slopes, vertical excavations can be temporarily shored. The contractor or the specialty subcontractor should be responsible for the design of the temporary shoring in accordance with applicable regulatory requirements.

D.8. PERMANENT SLOPES

Permanent cut or fill slopes should be no steeper than 3.0(H):1.0(V) to maintain long-term stability and to provide ease of maintenance. Steeper slopes are susceptible to erosion, will be difficult to maintain, and could experience problems with instability. The crest or toe of cut or fill slopes should be no closer than 10 feet from any foundation. It is recommended that permanent slopes be vegetated, as soon as practical, in order to minimize the potential for erosion.

E. BUILDING AND STRUCTURES

E.1. SHALLOW FOUNDATION DESIGN

Based on the results of the soil test borings, laboratory testing and our engineering evaluation, it is our opinion that the subsurface conditions are suitable for supporting the proposed structure on a conventional shallow foundation system. Assuming the recommendations in ***Section D*** of this report are followed and a FFE of 100.0, the interior and exterior foundations will be supported on Altered Peoria loess. Based on the above soil properties and assuming the recommendations above are followed, we recommend that the foundations be designed for a maximum net allowable soil bearing pressure of 1,500 psf.

The net allowable bearing pressure refers to the bearing pressure at foundation level in excess of the minimum surrounding overburden pressure at the footing base elevation. The minimum surrounding overburden pressure should be based upon the lowest immediately adjacent finished ground surface or finished floor elevation. Footings should have minimum dimensions in accordance with local building codes. Exterior footings and footings in unheated areas should bear at a minimum depth of 3½ feet below the lowest adjacent final ground surface. It is recommended that interior footings in heated areas bear at a depth as shallow as possible below the lowest adjacent final ground surface. The analyses for interior and exterior footings utilized bearing depths of 2 and 3½ feet, respectively, below the finished floor elevation.

Provided the recommendations contained in this report are followed, total post-construction settlements are anticipated to be less than 1 inch with differential settlements anticipated to be less than 0.5 inches for the building addition. Differential settlements between the existing building and proposed addition are anticipated to be 1 inch or less. To reduce effects of differential settlement, a floating floor slab independent from the wall and column loads with expansion joints will be critical in minimizing the potential cracking that can occur along and around the proposed foundation system. Floor slab control joints should be used to reduce damage due to shrinkage cracks.

Some overlap in stress distribution from new and existing footings may occur, which may cause minor movement of the existing footings and supported structures. Maintaining a clear distance at least equal to the width of the new column spread footings between the edges of the new and existing footings could significantly reduce this risk. Connections between the new and existing structures should be designed to allow for the anticipated total post-construction settlement.

It is possible that some soils at the site will have an allowable soil bearing pressure less than the recommended design value. Therefore, foundation bearing surfaces should be performed by an ***Olsson*** representative during footing construction to aid in the identification of such soils. After foundation subgrades have been observed and documented and any required remedial measures are performed, concrete should be placed as quickly as possible to avoid exposure of the foundation subsoils to wetting, drying or freezing. If soils in the areas of foundation support are subjected to such conditions, the footings should be reevaluated.

E.2. FLOOR SLAB SUBGRADE PREPARATION

The soil subgrade in the areas of concrete slab-on-grade support is often disturbed during foundation and superstructure construction. Additionally, floor slab areas are often disturbed by construction equipment traffic between the time of initial grading and final construction. To prepare the floor slab subgrade, the top 12 inches of the subgrade in the building area should be scarified and re-compacted to a minimum of 98 percent of the maximum dry density as determined by the standard Proctor test (ASTM D698-91). The moisture content should also be controlled between optimum and +3 percent of the materials optimum. The final subgrade should be proofrolled and evaluated by a field representative immediately prior to placement of the concrete to detect any localized areas of instability. If required, the methods of stabilization will typically include the use of crushed stone and/or a geosynthetic fabric or grid installed over the soft soils. The identification of areas that may require undercutting and/or stabilization should be based on the actual conditions at the time of construction, and will depend on the location and extent of the soft area. In regards to the moisture levels of the subgrade, additional water application may be necessary to maintain moisture levels above optimum. It is imperative that the subgrade soil be at or above the optimum level prior to the placement of the aggregate base (if used) and concrete floor slab.

To reduce the potential for moisture leaching from the concrete after placement, it is also recommended that enough water be added to nearly saturate the granular cushion. The granular cushion should be compacted to a minimum of 70 percent of the materials relative density. Laboratory maximum and minimum index density for the relative density tests should be performed in accordance with ASTM D4253 and D4254. If these recommendations are implemented, a subgrade modulus of at least 125 psi/in for the floor slab design is acceptable.

E.3. LATERAL EARTH PRESSURES

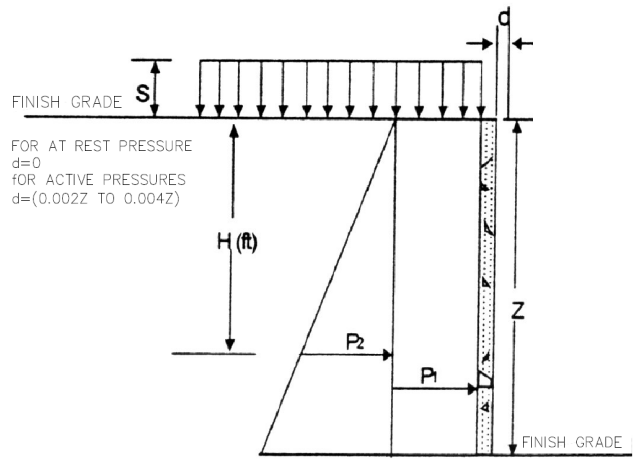
The following soil parameters are provided for use in designing grade retaining walls and/or foundation walls subject to lateral earth pressures. The parameters are based on the understanding that retained soils will be similar in composition to the on-site soils encountered during this investigation.

Walls which are rigidly restrained at the top and are essentially unable to deflect or rotate should be designed for "at rest" earth pressure conditions. Walls that are unrestrained at the top and are free to deflect or rotate slightly may be designed for "active" earth pressure conditions. The "passive" earth pressure condition should be used to evaluate the resistance of soil to lateral loads. *Table 2* presents recommended values of earth pressure coefficients based on our experience with soils in the area. Equivalent fluid densities are frequently used for the calculation of lateral earth pressures for the "at-rest" and "active" conditions and are therefore provided in *Table 2*. The equivalent fluid densities in *Table 2* do not include the effects of surcharge loading (P_1).

TABLE 2

EARTH PRESSURE PARAMETERS

LEGEND OF SYMBOLS	
Z	WALL HEIGHT (ft)
H	DEPTH BELOW SURFACE (ft)
d	WALL DISPLACEMENT (ft)
S	SURCHARGE LOAD (psf)
P_1	SURCHARGE PRESSURE (psf)
P_2	EARTH LOAD (psf)
K	COEFFICIENT OF EARTH PRESSURE
G	EQUIVALENT FLUID DENSITY (psf)



PRESSURE CALCULATIONS	
SURCHARGE PRESSURE	P_1 (psf) = K x S (psf)
EARTH PRESSURE	P_2 (psf) = G (pcf) x H (ft)

BACKFILL TYPE		FRICTION ANGLE	TOTAL SOIL DENSITY
COHESIVE -	Lean Clay (CL)	26°	120 pcf
GRANULAR* -	Less than 10% Fines (SP, GP)	32°	120 pcf

EARTH PRESSURE COEFFICIENT (K)			EQUIVALENT FLUID DENSITY (G)	
			DRAINED CONDITION	UNDRAINED CONDITION
AT REST (K_0)	Cohesive	- 0.56	68 pcf	95 pcf
	Granular*	- 0.47	56 pcf	
ACTIVE (K_a)	Cohesive	- 0.39	47 pcf	85 pcf
	Granular*	- 0.31	37 pcf	
PASSIVE (K_p)	Cohesive	- 2.00	240 pcf	170 pcf
	Granular*	- 3.00	360 pcf	

* If granular backfill is utilized, it is recommended the granular backfill be permanently drained

These design recommendations are based on the following assumptions:

- For active earth pressure, wall must rotate about base, with top lateral movements 0.002 Z to 0.004 Z, where Z is wall height.
- Drained conditions assume a permanent drainage system behind the retaining wall that will allow no development of hydrostatic pressure.
- Horizontal backfill.
- The upper 40 inches do not contribute resistance against horizontal movement if the soil is subject to frost action and seasonal volume change.
- Onsite backfill soils having a bulk unit weight of 120 pcf.
- Backfill soils placed within the height of the retaining wall consisting of selected lean clay should be tested to verify the lean clays exhibit low plasticity and can achieve a minimum friction angle of 26 degrees.
- Imported granular backfill soils having a minimum angle of internal friction of 32 degrees.
- Uniform surcharge, where S is surcharge pressure, in psf.
- Heavy equipment and other concentrated load components not included.
- No safety factor is included.

Backfill soils placed within a lateral distance from the face of the wall to seven tenths of the wall height could consist of selected lean clay exhibiting an Atterberg liquid limit of less than 40. For granular soils, the granular backfill must extend out from the base of the wall at an angle of 45 and 60 degrees from the vertical wall for the active and passive cases, respectively. To calculate the resistance to sliding on native soil and crushed angular limestone, an ultimate coefficient of friction value of 0.3 and 0.45 should be used, respectively, where the footing bears on suitable approved bearing soil. A factor of safety of at least 1.5 to 2 should be applied.

F. PAVEMENTS

F.1. PAVEMENT SUBGRADE PREPARATION

Based on the soil conditions encountered in the soil test borings, it is our opinion the pavement subgrade soils in the pavement areas can be adequately prepared by field conditioning and compaction. It is important that the subgrade support be relatively uniform, with no abrupt changes in the degree of support. Non-uniform pavement support can result from the transition at cut and fill areas, varying soil moisture contents, varying soil types, and where utility backfill have been placed in areas to be paved. Improper subgrade preparation such as inadequate vegetation removal, proofrolling, and compaction can also result in non-uniform subgrade support.

The subgrade soils should possess a moisture content between optimum to +3 percent of the standard proctor at the time of pavement construction. As a result of the timeline between initial subgrade preparation and installation of the concrete pavement, it may be necessary to appropriately adjust the moisture content of the subgrade soils immediately prior to the pavement construction.

To prepare the subgrade, the top 12 inches of the subgrade should be compacted to a minimum of 98 percent of the maximum dry density as determined by ASTM D698, Standard Proctor Moisture-Density Relationship at a moisture content between optimum and +3 percent of optimum. The final subgrade should be proofrolled immediately prior to the placement of the aggregate subbase to detect any localized areas of instability. For a subgrade prepared in this manner ***Olsson*** estimates a California Bearing Ratio (CBR) value of 2.5 and a modulus of subgrade reaction of 180 psi/in.

F.2. PAVEMENT DESIGN

Pavement design is influenced by the anticipated traffic loads and volumes, site subgrade conditions, pavement materials, traffic study and the desired design life. The recommended pavement thickness is based on our experiences with the alternate pavement types and equivalent pavement sections based on recognized structural coefficients. It is important to note that changes in traffic conditions that exceed the assumptions of this report can have a significant impact on the design life of the pavement.

The recommended design requires that the site be properly prepared in accordance with ***Section F.1*** of this report and that site drainage be provided to minimize the future wetting of the pavement subgrade. A pavement design life of 20 years is used. AASHTO pavement design procedures were used to estimate the required pavement thickness.

The following parameters were adopted for the thickness design:

- Subgrade: 12" subgrade compacted to at least 98% of ASTM D-698, at a moisture content between optimum and +3 percent of optimum
- CBR value: 2.5 (Assumed)
- Working stress (PCC) 668 psi

The Standard-Duty and Heavy-Duty pavement recommendations are based on a design life of 20 years, terminal serviceability = 2.5, reliability = 85%, initial serviceability = 4.4, drainage coefficient = .95, and standard deviation = 0.45 for flexible pavements and 0.35 for rigid pavements.

For each pavement section we have provided a total design ESAL's and how the total ESAL's correlate to the daily traffic count for passenger cars and truck delivery traffic. The pavement section chosen should be based off of expected traffic counts. For design purposes, the anticipated traffic on the proposed pavement areas was assumed to consist of automobiles, delivery trucks, and garbage trucks. If the traffic counts described with the pavement section are not sufficient for the proposed roadways please contact the geotechnical engineer.

Based on the above design parameters, we recommend the following **minimum** pavement design thickness:

General Pavement Cross Section:

Asphalt Pavement (Standard Duty: 41,647 ESAL)

Correlates to 500 - passenger cars, 2 – delivery trucks per day

Depth (in)	Material Designation	Material Specification
1.5		Surface Course: NDOR Section 1028, Asphaltic Concrete, Type SPR
4.5		Base Course: NDOR Section 1028, Asphaltic Concrete, Type SPR
12.0		Subgrade: NDOR Section 302, Subgrade Preparation

Asphalt Pavement (Heavy Duty: 96,320 ESAL)

Correlates to 500 - passenger cars and 5 – delivery trucks per day

Depth (in)	Material Designation	Material Specification
2.0		Surface Course: NDOR Section 1028, Asphaltic Concrete, Type SPR
6.0		Base Course: NDOR Section 1028, Asphaltic Concrete, Type SPR
12.0		Subgrade: NDOR Section 302, Subgrade Preparation

Concrete Pavement (Standard Duty: 47,507 ESAL)

Correlates to 500 - passenger cars, 2 – delivery trucks per day

Depth (in)	Material Designation	Material Specification
5.0		Concrete: NDOR Section 1002, Portland Cement Concrete
12.0		Subgrade: NDOR Section 302, Subgrade Preparation

Concrete Pavement (Heavy Duty: 115,148 ESAL)

Correlates to 500 - passenger cars and 5 – delivery trucks per day

Depth (in)	Material Designation	Material Specification
7.0		Concrete: NDOR Section 1002, Portland Cement Concrete
12.0		Subgrade: NDOR Section 302, Subgrade Preparation

Because parking lot areas are subjected to slow-moving and static load conditions, a PG64-28 asphalt cement is recommended for the asphalt. This grade of asphalt cement produces an asphaltic concrete that is less susceptible to rutting and creep caused by slow-moving or static loads during warm periods. Minimum surface course thickness of 1.5 inches in light duty areas and 2.0 inches in heavy duty areas are recommended for asphaltic cement concrete pavement sections. Surface drainage around the pavement and proper maintenance are also important for long-term performance. Curbs should be backfilled as soon as possible after construction of the pavement. Backfill should be compacted and should be sloped to prevent water from ponding and infiltration under the pavement. All pavement joints should be caulked and any cracks should be quickly patched or sealed to prevent moisture from leaching and softening the subgrade.

G. LIMITATIONS

The conclusions and recommendations presented in this report are based on the information available regarding the proposed construction, the results obtained from our soil test borings and sampling procedures, the results of the laboratory testing program, and our experience with similar projects. The soil test borings represent a very small statistical sampling of subsurface soils and it is possible that conditions may be encountered during construction that are substantially different from those indicated by the soil test borings. In these instances, adjustments to design and construction may be necessary.

This geotechnical report is based on the site plan and information provided to ***Olsson*** and our understanding of the project as noted in this report. Changes in the location or design of the proposed building could significantly affect the conclusions and recommendations presented in this geotechnical report. ***Olsson*** should be contacted in the event of such changes to determine if the recommendations of this report remain appropriate for the revised site design.

Soil stratification, as shown on the Boring Logs, represent soil conditions at the boring locations; however, variations may occur between or around the boring locations. The lines of demarcation represent the approximate boundary between soil types but the transition may be more gradual.

This report was prepared by an Engineer Intern and reviewed by a Professional Engineer registered in the State of Nebraska with the firm ***Olsson Associates (Olsson)***. The conclusions and recommendations contained herein are based on generally accepted, professional, geotechnical engineering practice at the time of this report within this geographic area. No other warranty is expressed or implied. This report has been prepared for the exclusive use of ***The Architectural Offices*** with specific application to the proposed project.

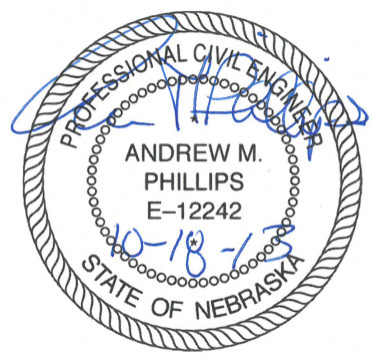
****

We trust that this report will assist you in the design and construction of the proposed project. ***Olsson*** appreciates the opportunity to provide our services on this project and looks forward to working with you during construction and on future projects. Should you have any questions, please do not hesitate to contact us.

Respectfully submitted,
Olsson Associates

Prepared by:

Reviewed by:



Sean Parks, M.S., E.I.
Assistant Engineer

Andrew M. Phillips, P.E.
Geotechnical Engineer

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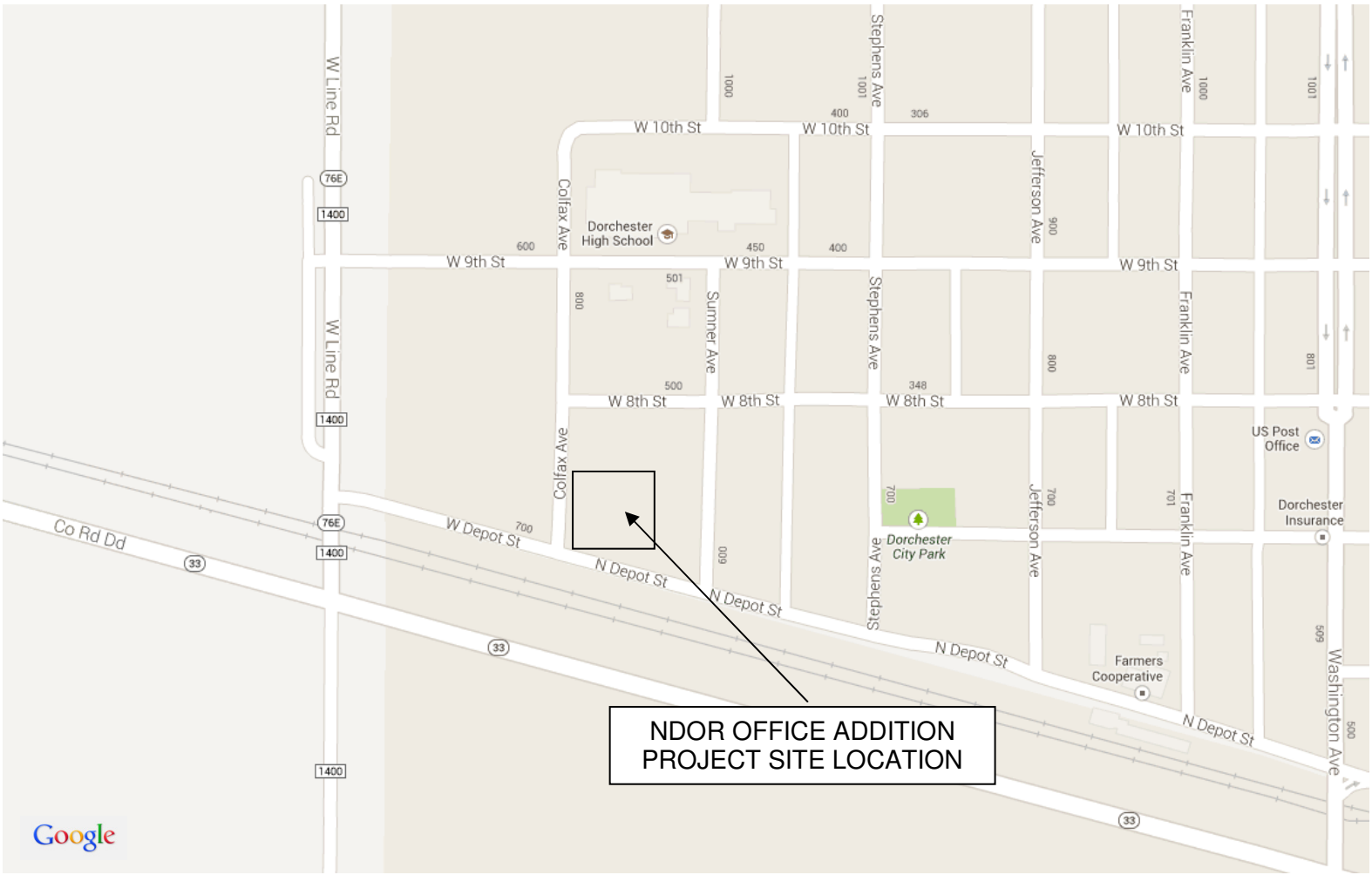
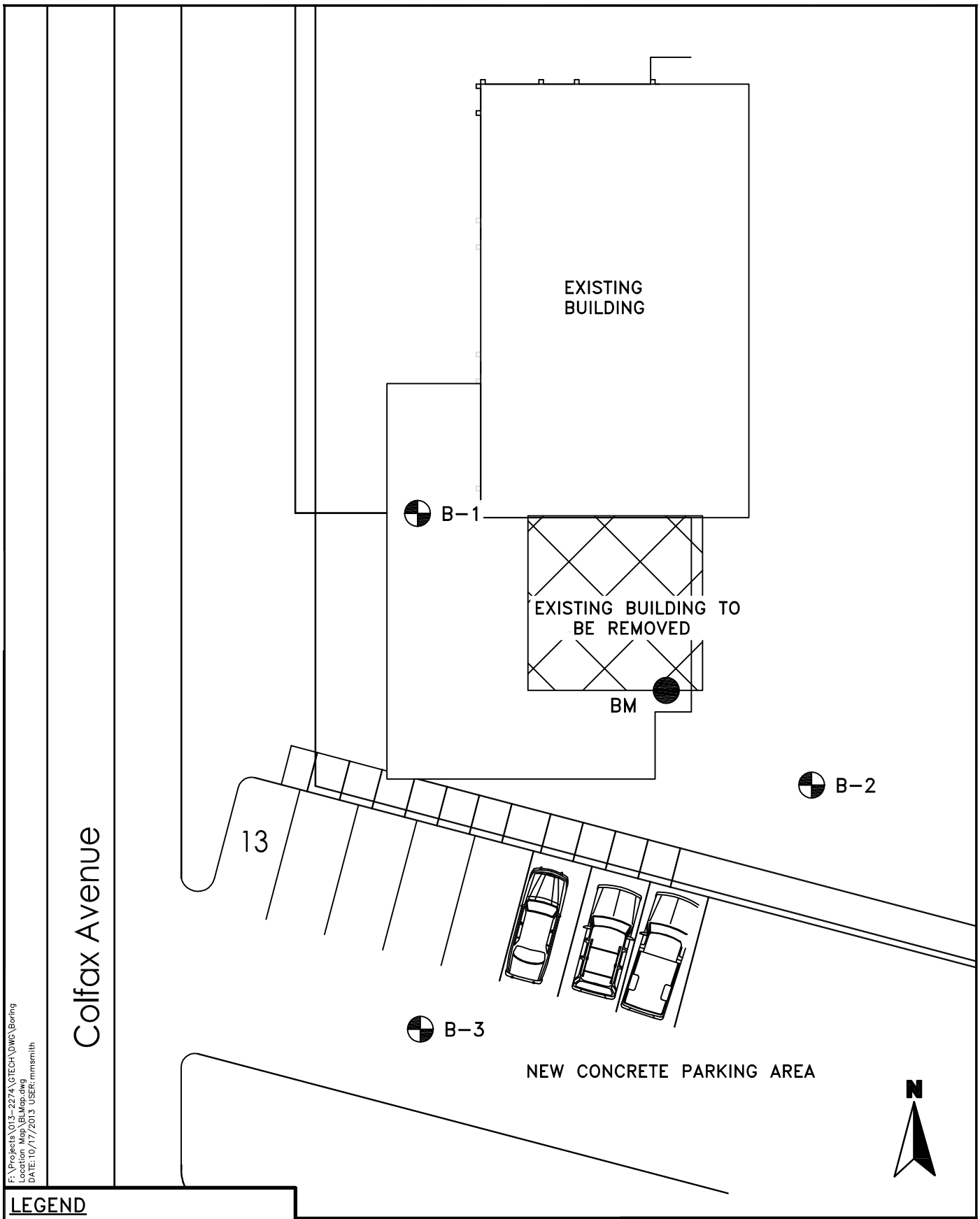
APPENDIX A

Site Location Plan
Boring Location Map



SITE LOCATION PLAN
NDOR OFFICE ADDITION
DORCHESTER, NEBRASKA
OA PROJECT NO. 013-1774



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DATE:10/17/2013 USER:mmsmith

LEGEND

- SOIL BORING LOCATION
- BENCH MARK
- APPROXIMATE EXISTING BUILDING LOCATION

0 10 20 40
SCALE IN FEET

PROJECT: 013-2274

DRAWN BY:MMS

DATE: 10/17/13

BORING LOCATION MAP
DORCHESTER, NEBRASKA

OLSSON ASSOCIATES
1111 Lincoln Mall, Suite 111
P.O. Box 84608
Lincoln, NE 68501-4608
TEL 402.474.6311
FAX 402.474.5160
www.olssonassociates.com

APPENDIX B

Symbols & Nomenclature
Boring Logs

SYMBOLS AND NOMENCLATURE

DRILLING NOTES

DRILLING AND SAMPLING SYMBOLS

SS:	Split-Spoon Sample (1.375" ID, 2.0" OD)	HSA:	Hollow Stem Auger	NE:	Not Encountered
U:	Thin-Walled Tube Sample (3.0" OD)	CFA:	Continuous Flight Auger	NP:	Not Performed
CS:	Continuous Sample	HA:	Hand Auger	NA:	Not Applicable
BS:	Bulk Sample	CPT:	Cone Penetration Test	% Rec:	Percent of Recovery
MC:	Modified California Sampler	WB:	Wash Bore	WD:	While Drilling
GB:	Grab Sample	FT:	Fish Tail Bit	IAD:	Immediately After Drilling
SPT:	Standard Penetration Test Blows per 6.0"	RB:	Rock Bit	AD:	After Drilling
				CI:	Cave-In

DRILLING PROCEDURES

Soil samples designated as "U" samples on the boring logs were obtained in using Thin-Walled Tube Sampling techniques. Soil samples designated as "SS" samples were obtained during Penetration Test using a Split-Spoon Barrel sampler. The standard penetration resistance 'N' value is the number of blows of a 140 pound hammer falling 30 inches to drive the Split-Spoon sampler one foot. Soil samples designated as "MC" were obtained in using Thick-Walled, Ring-Lined, Split-Barrel Drive sampling techniques. Recovered samples were sealed in containers, labeled, and protected for transportation to the laboratory for testing.

WATER LEVEL MEASUREMENTS

Water levels indicated on the boring logs are levels measured in the borings at the times indicated. In relatively high permeable materials, the indicated levels may reflect the location of groundwater. In low permeability soils, the accurate determination of groundwater levels is not possible with only short-term observations.

SOIL PROPERTIES & DESCRIPTIONS

Descriptions of the soils encountered in the soil test borings were prepared using Visual-Manual Procedures for Descriptions and Identification of Soils.

PARTICLE SIZE

Boulders	12 in. +	Coarse Sand	4.75mm-2.0mm	Silt	0.075mm-0.005mm
Cobbles	12 in.-3 in.	Medium Sand	2.0mm-0.425mm	Clay	<0.005mm
Gravel	3 in.-4.75mm	Fine Sand	0.425mm-0.075mm		

COHESIVE SOILS

Consistency	Unconfined Compressive Strength (Qu) (tsf)
Very Soft	<0.25
Soft	0.25 – 0.5
Firm	0.5 – 1.0
Stiff	1.0 – 2.0
Very Stiff	2.0 – 4.0
Hard	> 4.0

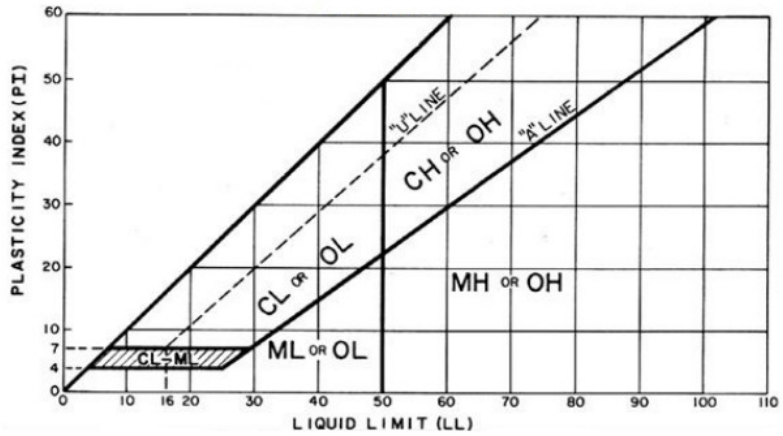
COHESIONLESS SOILS

Relative Density	'N' Value
Very Loose	0 – 3
Loose	4 – 9
Medium Dense	10 – 29
Dense	30 – 49
Very Dense	≥ 50

COMPONENT %

Description	Percent (%)
Trace	<5
Few	5 - 10
Little	15 - 25
Some	30 - 45
Mostly	50 - 100

PLASTICITY CHART



ROCK QUALITY DESIGNATION (RQD)

Description	RQD (%)
Very Poor	0 – 25
Poor	25 – 50
Fair	50 – 75
Good	75 – 90
Excellent	90 – 100

OLSSON ASSOCIATES ®

BOREHOLE REPORT NO. B-1

PROJECT NAME: **NDOR Facility Addition**	CLIENT: **The Architectural Offices**
PROJECT NUMBER: **013-2274**	LOCATION: **Dorchester, Nebraska**

Shelby Tube

APPROX. SURFACE ELEV. (ft): 99.1

ELEVATION (ft)	MATERIAL DESCRIPTION		DEPTH (ft)	SAMPLE TYPE NUMBER	CLASSIFICATION (USCS)	BLOWS/6" N-VALUE RQD	UNC. STR. (tsf)	MOISTURE (%)	DRY DENSITY (pcf)	LL/PI (%)	ADDITIONAL DATA/ REMARKS
	DEVELOPED ZONE	***0.3'***	0								
	FILL										
	Fat clay (CH) *Stiff, dark brownish grey, very moist, mostly fat clay, trace fine to coarse sand*	***2.0'***		U 1	CH			25.5	94.6	53/30	
	ALTERED PEORIA LOESS	***3.0'***									
95	*Lean clay (CL)* *Firm, brown, very moist, mostly lean clay, trace fine sand*			U 2				30.7	91.6		
	PEORIA LOESS		5								
	Lean clay (CL) *Firm, light brown to brown, very moist, mostly lean clay, iron*										
	Lean clay (CL) *Firm, light brown to brown, very moist, mostly lean clay, iron*			U 3			0.6	31.8	95.4		
90	*Lean clay (CL)* *Firm, light brown to brown, very moist, mostly lean clay, iron*		10	U 4				29.0	90.0		
85	*Lean clay (CL)* *Firm, light brown to yellowish brown, very moist, mostly lean clay, few silt, iron*	***15.0'***	15	U 5							

BASE OF BORING AT 15.0 FEET

WATER LEVEL OBSERVATIONS	
WD	Not Encountered
IAD	Not Encoutered
AD	Not Performed

OLSSON ASSOCIATES
1111 LINCOLN MALL, SUITE 111
LINCOLN, NEBRASKA 68508

STARTED	8/21/13	FINISHED	8/21/13
DRILL CO.	OLSSON	DRILL RIG	CME 55
DRILLER	AMA	LOGGED BY	IJS
METHOD	CONTINUOUS FLIGHT AUGER		

BOREHOLE REPORT NO. B-2

OLSSON ASSOCIATES

PROJECT NAME	CLIENT
NDOR Facility Addition	The Architectural Offices
PROJECT NUMBER	**LOCATION**
013-2274	Dorchester, Nebraska

Shelby Tube | No Recovery | Split Spoon

APPROX. SURFACE ELEV. (ft): 99.1

ELEVATION (ft)	MATERIAL DESCRIPTION	DEPTH (ft)	SAMPLE TYPE NUMBER	CLASSIFICATION (USCS)	BLOWS/6" N-VALUE RQD	UNC. STR. (tsf)	MOISTURE (%)	DRY DENSITY (pcf)	LL/PI (%)	ADDITIONAL DATA/ REMARKS
	ASPHALT 0.3'	0								
	CONCRETE 0.4'									
	ALTERED PEORIA LOESS 1.5'									
	Lean clay (CL) Firm, dark brown, very moist, mostly lean clay, trace fine sand		U 1	CL		0.8	30.2	89.7	48/27	
	PEORIA LOESS 3.5'									
95	*Lean clay (CL) Firm, yellowish brown, moist, mostly lean clay, trace fine sand*	5	U 2				25.0	92.7		
	Lean clay (CL) Stiff, yellowish brown, very moist, mostly lean clay, few silt, manganese 7.0'		U 3			1.4	28.3	91.6		
90	*Lean clay (CL) Firm, yellowish brown, moist, mostly lean clay, trace fine sand* 12.0'	10	U 4							
85	*Lean clay (CL) Firm, yellowish brown to dark brown, moist, mostly lean clay, little silt, trace fine sand, iron* 16.5'	15	U 5							
	LOVELAND FORMATION									
80		20	NR NA							

CONTINUED NEXT PAGE

WATER LEVEL OBSERVATIONS	
WD	Not Encountered
IAD	Not Encoutered
AD	Not Performed

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LINCOLN, NEBRASKA 68508

STARTED	8/21/13	FINISHED	8/21/13
DRILL CO.	OLSSON	DRILL RIG	CME 55
DRILLER	AMA	LOGGED BY	IJS
METHOD	CONTINUOUS FLIGHT AUGER		

BOREHOLE REPORT NO. B-2

PROJECT NAME	CLIENT
NDOR Facility Addition	The Architectural Offices
PROJECT NUMBER	**LOCATION**
013-2274	Dorchester, Nebraska

Shelby Tube | No Recovery | Split Spoon

ELEVATION (ft)	MATERIAL DESCRIPTION	GRAPHIC LOG	DEPTH (ft)	SAMPLE TYPE NUMBER	CLASSIFICATION (USCS)	BLOWS/6" N-VALUE RQD	UNC. STR. (tsf)	MOISTURE (%)	DRY DENSITY (pcf)	LL/PI (%)	ADDITIONAL DATA/ REMARKS
			20								
	LOVELAND FORMATION *Lean to fat clay (CL/CH) Hard, reddish brown, moist, mostly lean to fat clay, trace fine sand, iron and calcium* ***21.5'***			SS 6		12-16-15 N=31					

BASE OF BORING AT 21.5 FEET

WATER LEVEL OBSERVATIONS	
WD	Not Encountered
IAD	Not Encoutered
AD	Not Performed

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LINCOLN, NEBRASKA 68508

STARTED	8/21/13	FINISHED	8/21/13
DRILL CO.	OLSSON	DRILL RIG	CME 55
DRILLER	AMA	LOGGED BY	IJS
METHOD	CONTINUOUS FLIGHT AUGER		

BOREHOLE REPORT NO. B-3

PROJECT NAME	CLIENT
NDOR Facility Addition	The Architectural Offices
PROJECT NUMBER	**LOCATION**
013-2274	Dorchester, Nebraska

Shelby Tube

APPROX. SURFACE ELEV. (ft): 98.3

ELEVATION (ft)	MATERIAL DESCRIPTION	GRAPHIC LOG	DEPTH (ft)	SAMPLE TYPE NUMBER	CLASSIFICATION (USCS)	BLOWS/6" N-VALUE RQD	UNC. STR. (tsf)	MOISTURE (%)	DRY DENSITY (pcf)	LL/PI (%)	ADDITIONAL DATA/ REMARKS
	ASPHALT *0.6'*		0								
	ALTERED PEORIA LOESS										
	Lean clay (CL) *Firm, dark brown, moist, mostly lean clay, few fine sand* *2.0'*			U 1			2.5	24.8	97.7		
95	***PEORIA LOESS***										
	Fat clay (CH) *Very stiff, yellowish brown, moist, mostly fat clay, trace fine sand, iron* *4.0'*		5	U 2				24.1	95.3		
	Lean clay (CL) *Firm, yellowish brown, moist, mostly lean clay, trace fine sand*										
				U 3							
90											
	Lean clay (CL) *Firm, yellowish brown, very moist, mostly lean clay, trace fine sand*		10	U 4			0.8	31.2	88.7		

BASE OF BORING AT 10.0 FEET

WATER LEVEL OBSERVATIONS	
WD	Not Encountered
IAD	Not Encoutered
AD	Not Performed

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LINCOLN, NEBRASKA 68508

STARTED	8/21/13	FINISHED	8/21/13
DRILL CO.	OLSSON	DRILL RIG	CME 55
DRILLER	AMA	LOGGED BY	IJS
METHOD	CONTINUOUS FLIGHT AUGER		

APPENDIX C

Summary of Laboratory Test Results

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LINCOLN, NEBRASKA 68508

SUMMARY OF LABORATORY RESULTS

PROJECT NAME The Architectural Offices NDOR Dorchester Facility Addition

CLIENT The Architectural Offices

PROJECT NUMBER 013-2274

PROJECT LOCATION Dorchester, Nebraska

BORING NUMBER	SAMPLE I.D.	SAMPLE DEPTH (ft)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	VOID RATIO	SATURATION (%)	UNCONFINED STRENGTH (tsf)	STRAIN (%)	ATTERBERG LIMITS			P-200	USCS CLASS.
									LIQUID LIMIT	PLASTIC LIMIT	PLASTIC INDEX		
B-1	**U-1**	1.0 - 2.5'	25.5	94.6	0.782	88.1			53	23	30		CH
B-1	**U-2**	3.5 - 5.0'	30.7	91.6	0.839	98.8							
B-1	**U-3**	6.0 - 7.5'	31.8	95.4	0.766	100.0	0.6	14.9					
B-1	**U-4**	8.5 - 10.0'	29.0	90.0	0.874	89.5							
B-2	**U-1**	1.0 - 2.5'	30.2	89.7	11.531	47.1	0.8	4.3	48	21	27		CL
B-2	**U-2**	3.5 - 5.0'	25.0	92.7	11.126	40.4							
B-2	**U-3**	6.0 - 7.5'	28.3	91.6	11.268	45.3	1.4	4.9					
B-3	**U-1**	1.0 - 2.5'	24.8	97.7	0.725	92.4	2.5	3.9					
B-3	**U-2**	3.5 - 5.0'	24.1	95.3	0.769	84.8							
B-3	**U-4**	8.5 - 10.0'	31.2	88.7	0.900	93.5	0.8	4.8					

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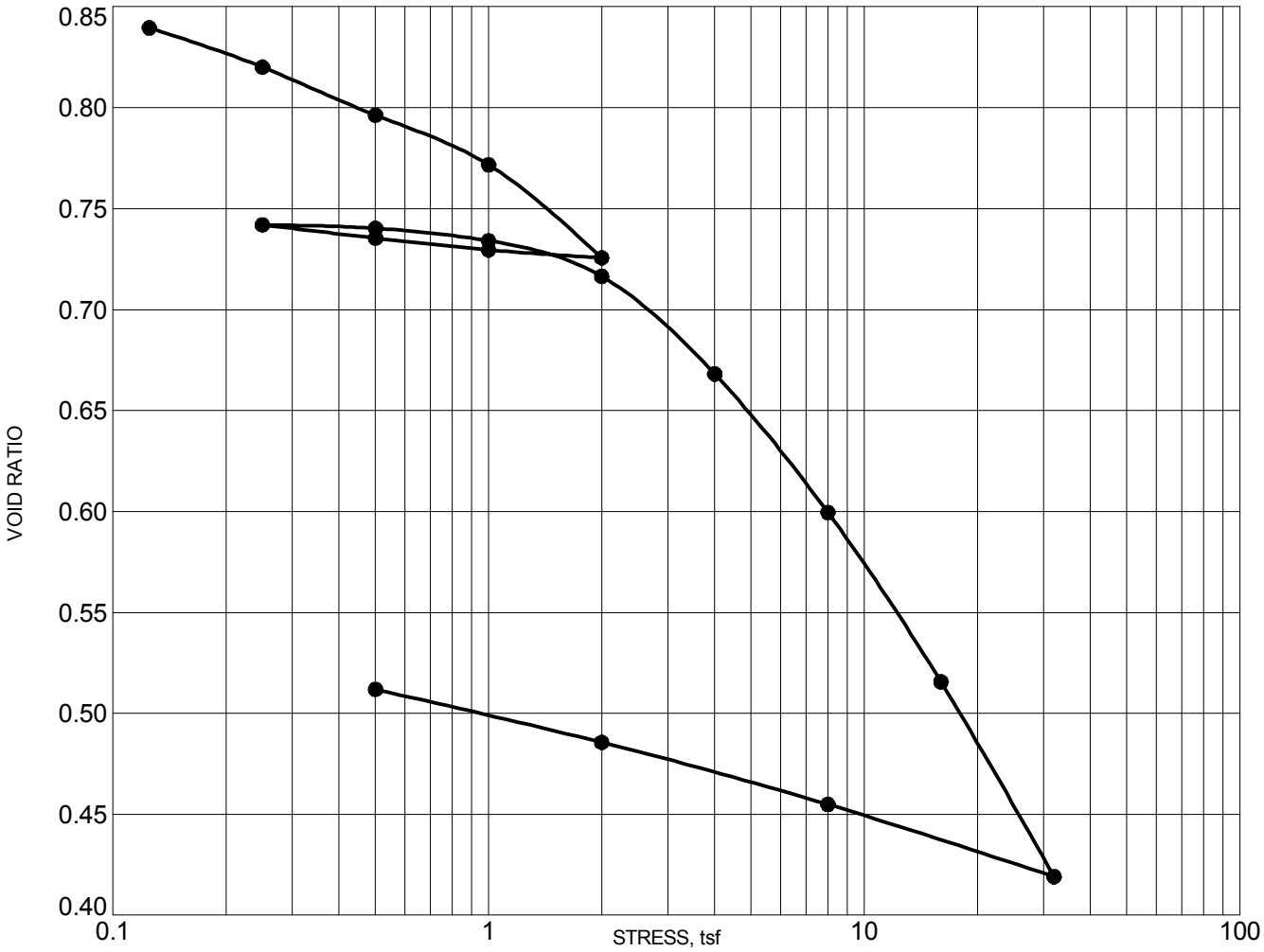
LINCOLN, NEBRASKA 68508

CONSOLIDATION TEST

OLSSON ASSOCIATES

PROJECT NAME NDOR Facility Addition **CLIENT** The Architectural Offices

PROJECT NUMBER 013-2274 **PROJECT LOCATION** Dorchester, Nebraska



Boring No:	B-1	Initial Water Content (ft):	29.0	Est. Preconsolidated Stress (tsf):	2.6
Sample ID:	U-4	Final Water Content (ft):	24.0	Laboratory Water Type:	Distilled Water
Sample Depth:	8.5 - 10.0'	Initial Dry Density (pcf):	90.0	Test Procedure Method:	NA
Start Date:	8/27/13	Initial Void Ratio:	0.870	Interpretation Procedure:	NA
Technician:	DDK	Final Void Ratio:	0.510	Stress at Inundation (psf):	50.0
Apparatus:	LT II - 66	Initial Degree of Saturation (%):	89.5	Specimen Trimming Method:	Turntable
Specific Gravity:	2.7	Final Degree of Saturation (%):	100.0		

ATTERBERG LIMITS

LL	PL	PI	Classification

Sample Description: Peoria Loess: Light brown, Lean clay (CL)

Notes:

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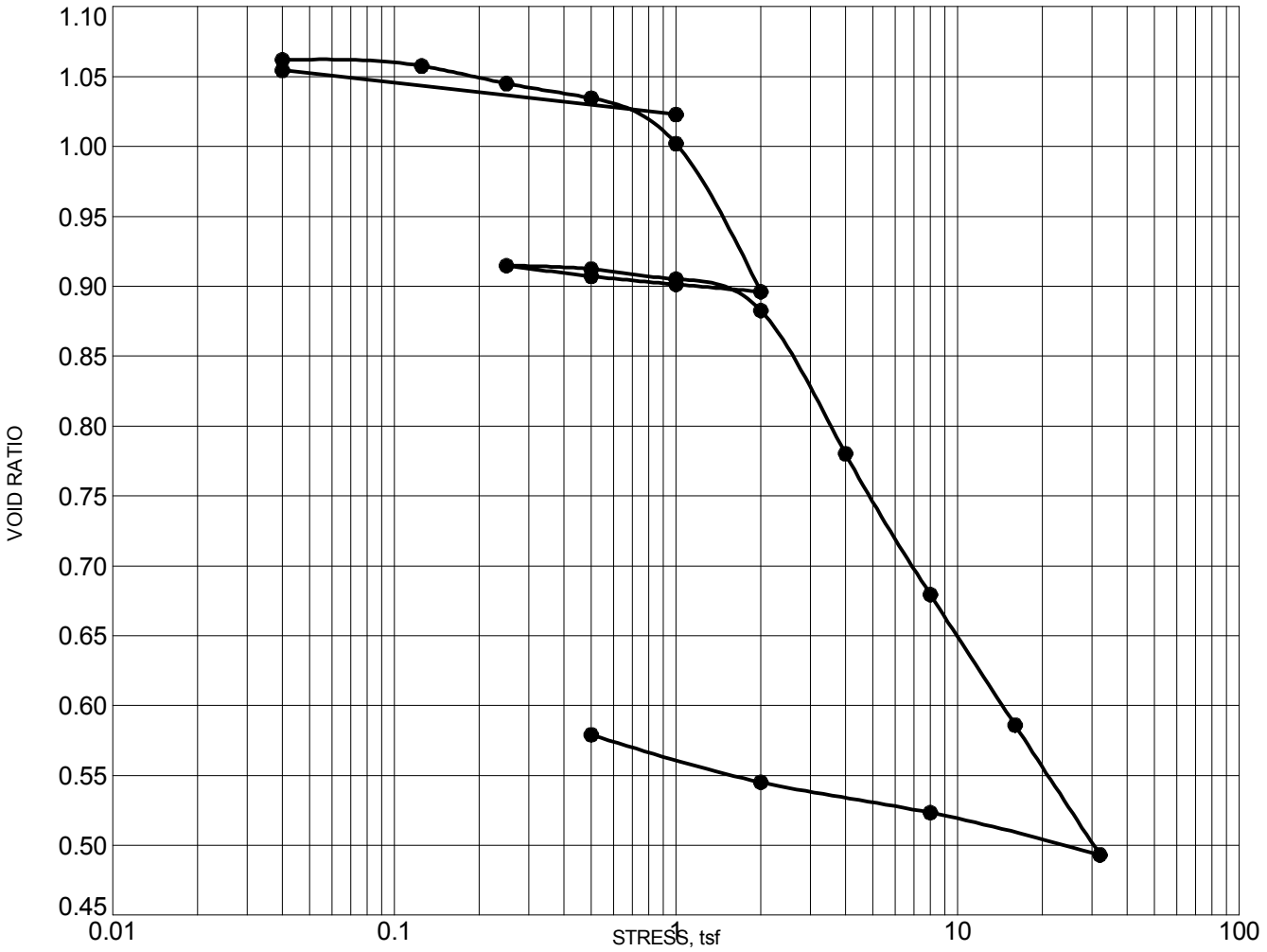
LINCOLN, NEBRASKA 68508

SWELL / CONSOLIDATION TEST

PROJECT NAME NDOR Facility Addition **CLIENT** The Architectural Offices

PROJECT NUMBER 013-2274 **PROJECT LOCATION** Dorchester, Nebraska



Boring No:	B-2	Initial Water Content (ft):	27.5	Est. Preconsolidated Stress (tsf):	1.0
Sample ID:	U-1	Final Water Content (ft):	21.5	Laboratory Water Type:	Distilled Water
Sample Depth:	1.0 - 2.5'	Initial Dry Density (pcf):	80.5	Test Procedure Method:	NA
Start Date:	9/19/13	Initial Void Ratio:	1.100	Interpretation Procedure:	NA
Technician:	DDK	Final Void Ratio:	0.580	Stress at Inundation (psf):	80.0
Apparatus:	LT II - 66	Initial Degree of Saturation (%):	67.7	Specimen Trimming Method:	Turntable
Specific Gravity:	2.7	Final Degree of Saturation (%):	100.0		

ATTERBERG LIMITS

LL	PL	PI	Classification
48	**21**	**27**	**CL**

Sample Description:

Notes:

APPENDIX D
PAVEMENT DESIGN

WinPAS

Pavement Thickness Design According to

1993 AASHTO Guide for Design of Pavements Structures

American Concrete Pavement Association

Rigid Design Inputs

Agency:
Company: Olsson Associates
Contractor:
Project Description: NDOR Facility Addition
Location: Dorchester, Nebraska

Rigid Pavement Design/Evaluation

PCC Thickness	4.00	**inches**	**Load Transfer, J**	3.60	
Design ESALs	47,507		**Mod. Subgrade Reaction, k**	187	**psi/in**
Reliability	85.00	**percent**	**Drainage Coefficient, Cd**	0.95	
Overall Deviation	0.35		**Initial Serviceability**	4.40	
Modulus of Rupture	668	**psi**	**Terminal Serviceability**	2.50	
Modulus of Elasticity	3,860,000	**psi**			

Modulus of Subgrade Reaction (k-value) Determination

Resilient Modulus of the Subgrade	3,635.1	**psi**
Resilient Modulus of the Subbase	0.0	**psi**
Subbase Thickness	0.00	**inches**
Depth to Rigid Foundation	0.00	**feet**
Loss of Support Value (0,1,2,3)	0.0	

Modulus of Subgrade Reaction	187.40	**psi/in**

WinPAS

Pavement Thickness Design According to

1993 AASHTO Guide for Design of Pavements Structures

American Concrete Pavement Association

Flexible Design Inputs

Agency:
Company: Olsson Associates
Contractor:
roject Description: NDOR Facility Addition
Location: Dorchester, Nebraska

Flexible Pavement Design/Evaluation

Structural Number	2.40		**Soil Resilient Modulus**	3,635.10	**psi**
Design ESALs	41,647		**Initial Serviceability**	4.40	
Reliability	85.00	**percent**	**Terminal Serviceability**	2.50	
Overall Deviation	0.45				

Layer Pavement Design/Evaluation

Layer Material	Layer Coefficient	Drainage Coefficient	Layer Thickness	Layer SN
	0.00	0.00	0.00	0.00
Asphalt Cement Concrete	0.44	0.95	6.00	2.51
	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00
			Σ SN	2.51

WinPAS

Pavement Thickness Design According to

1993 AASHTO Guide for Design of Pavements Structures

American Concrete Pavement Association

Rigid Design Inputs

Agency:
Company: Olsson Associates
Contractor:
Project Description: NDOR Facility Addition
Location: Dorchester, Nebraska

Rigid Pavement Design/Evaluation

PCC Thickness	4.54	**inches**	**Load Transfer, J**	3.60	
Design ESALs	115,148		**Mod. Subgrade Reaction, k**	187	**psi/in**
Reliability	85.00	**percent**	**Drainage Coefficient, Cd**	0.95	
Overall Deviation	0.35		**Initial Serviceability**	4.40	
Modulus of Rupture	668	**psi**	**Terminal Serviceability**	2.50	
Modulus of Elasticity	3,860,000	**psi**			

Modulus of Subgrade Reaction (k-value) Determination

Resilient Modulus of the Subgrade	3,635.1	**psi**
Resilient Modulus of the Subbase	0.0	**psi**
Subbase Thickness	0.00	**inches**
Depth to Rigid Foundation	0.00	**feet**
Loss of Support Value (0,1,2,3)	0.0	

Modulus of Subgrade Reaction	187.40	**psi/in**

WinPAS

Pavement Thickness Design According to

1993 AASHTO Guide for Design of Pavements Structures

American Concrete Pavement Association

Flexible Design Inputs

Agency:
Company: Olsson Associates
Contractor:
roject Description: NDOR Facility Addition
Location: Dorchester, Nebraska

Flexible Pavement Design/Evaluation

Structural Number	2.95		**Soil Resilient Modulus**	3,635.10	**psi**
Design ESALs	96,320		**Initial Serviceability**	4.40	
Reliability	85.00	**percent**	**Terminal Serviceability**	2.50	
Overall Deviation	0.45				

Layer Pavement Design/Evaluation

Layer Material	Layer Coefficient	Drainage Coefficient	Layer Thickness	Layer SN
	0.00	0.00	0.00	0.00
Asphalt Cement Concrete	0.44	0.95	7.50	3.14
	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00
			Σ SN	3.14