Upward Integration of Geotechnical Curricular Content Using a Project in Seattle, Washington, USA

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ABSTRACT: *The King Street Center project occupies a city block in the Pioneer Square area of Seattle, Washington, USA, and was completed in 2000. The project was comprised of a mid-rise building that includes two below-grade levels of underground parking, requiring excavation ranging from 3.0 to 9.1 m below grade. The site was bounded by historic city utilities and structures that required careful consideration of construction alternatives for shoring, dewatering, and foundation support. Key geotechnical design elements of the project included soldier pile and tieback walls, sheet pile walls, secant walls, gravity walls, shallow foundations, deep driven and augered-in-place piles, construction and permanent dewatering, and construction vibration mitigation. Design elements were not only influenced by on site soil and groundwater conditions, but also by off-site sensitive historic structures and utilities, as well as pavement sections immediately adjacent to the site. This project forms a case study that was used to develop several student design assignments and final projects.*

KEYWORDS: explorations, foundations, retaining walls, seepage, curriculum.

SITE LOCATION: IJGCH-database.kmz (requires Google Earth)

INTRODUCTION

The King Street Center project in Seattle, Washington, USA, is a development that occupies a city block in the lower downtown region of Seattle. The project design began in 1987 and construction was completed in 2000. The project is a mid-rise building that consisted of two below-grade levels of underground parking and seven levels above grade. Project construction required excavation ranging from 3.0 to 9.1 m below grade. The bottom of the excavation to facilitate construction was at approximately elevation -3.0 m (MSL NAD 83). Of particular importance at the site was excavation approximately 4.6 to 6.1 m below the preconstruction groundwater levels. Subsurface conditions at the site consisted of soft and loose fill over loose to medium dense marine deposited sand, over a very dense silty sand glacial till. The site was bounded by historic city utilities and structures that required an alternatives analysis of construction methods for shoring, dewatering, and foundation support. The project required geotechnical design of soldier pile and tieback walls, sheet pile walls, secant walls, gravity walls, shallow foundations, deep driven and augered-in-place piles, construction and permanent dewatering, and construction vibration mitigation. Design was primarily influenced by on-site soil and groundwater conditions, however off-site sensitive historic structures and utilities, as well as pavement sections immediately adjacent to the site were also considered in design.

The author was the geotechnical engineer of record for the project prior to entering academics. Therefore, substantial site data was available to use in forming the project as a case study. Site data consisted of over three years of groundwater level monitoring (both pre and during construction), 10 subsurface borings conducted for geotechnical design, 5 borings and 5 monitoring wells installed for construction monitoring and 9 geophones installed as part of construction vibration monitoring. Pile installation records and observation diaries from the on-site engineer were also available.

After joining the faculty at South Dakota State University (SDSU), the author used the project to form the basis for several student design assignments and final projects. The assignments and projects are used in multiple geotechnical courses, both

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as undergraduate technical electives and in Masters-level graduate courses. This approach allowed for the student to become familiar with a specific site and apply a myriad of learned design methods. It also allowed for an increasing level of difficulty as the student progresses in the geotechnical curriculum. This is commonly termed upward integration of curricular content. For the geotechnical program at SDSU, this consists of specific assignments in subsurface exploration interpretation, soldier pile wall design, sheet pile wall design, seepage analyses, shallow and deep foundation design, and gravity wall design. For upper level graduate courses, multiple elements of design are combined for final projects. Learning outcomes appropriate to these types of designs are associated with each assignments and appropriately assessed.

This paper summarizes the project and geotechnical site conditions that formed the basis for the overall case study. The paper also presents how the various geotechnical design elements were incorporated into the geotechnical curriculum. It also presents how upward integration of geotechnical curriculum can be used to provide enhanced student learning within a geotechnical curriculum. Assessment for each element of the geotechnical curriculum is also presented.

SITE DESCRIPTION AND SUBSURFACE CONDITIONS

Site Description

The King Street Center site is located in Seattle, Washington, and is bounded by South Jackson Street to the north, South King Street to the south, Second Avenue South to the west, and the extension of Third Avenue South to the east as shown in Figure 1. The site was a paved parking lot adjacent to the King Street Station used by national Amtrak train service, and was at street level on three sides, with Jackson Street rising about 5.2 m above the lot surface at the northeast corner of the site. Remnants of an old brick retaining wall were present along the north, and subsequent exploration at the base of the wall showed large granite block footings supporting the wall. Construction consisted of two levels below grade for parking and seven levels above grade for office space.

Historical records of the area placed the site at the edge of the old Seattle tideflats, and some evidence suggested that a Native American fishing village may have been present at the site. Subsequent studies identified no artifacts, including careful observation of conditions during shoring installation and excavation. The historical review also indicated that the dense supporting soil underlying the site would likely be variable in depth.

Subsurface Conditions

Through several phases of geotechnical and environmental analyses beginning in 1987 and extending through 1990, historical reviews and explorations were completed to define the soil and groundwater conditions at the King Street Center site. Seven hollow-stem auger borings were completed within and around the site, with five of the explorations completed as ground-water monitoring wells. A cross section of the soil and groundwater conditions is shown in Figure 2. This profile is not provided the students and is provided here for the academic reader to gain understanding of site subsurface conditions. In general, the soil conditions consisted of fill thicknesses ranging from about 3.0 to 6.1 m. Below the fill was loose to medium dense sand ranging from about 3.0 to 4.6 m thick, with dense glacial deposits below that consisting of sand and gravel. Deeper supporting soils included hard glacial silt and clay. Two large test pits excavated on the site disclosed caving ground, buried timbers and piles, and other subsurface debris. Note on Figure 2 that “Good Fnd. Support” signifies the bearing layer for retaining structures and deep foundations.

Due to project development delays, no significant new geotechnical work was conducted between 1990 and 1996 for the project. Beginning in 1996, the studies were revived, targeting a late 1997 construction start. By 1997, only three of the monitoring wells remained readable. For the purposes of design, static groundwater level was established at elevation 1.5 m (1.5 m depth). This elevation was the highest noted in any of the wells over the several years of periodic measurements. The groundwater levels were not affected by the tides in Elliott Bay (at sea level approximately 700 m to the west), as verified by a tidal fluctuation study completed in 1997. Measurements in the wells indicated a gradual northeast to southwest gradient, generally matching the regional topography and expected slope of the underlying silt/clay units across the site. In situ permeability measurements were 10^{-5} to 10^{-6} m/s within the zone of excavation and required dewatering. Prior to construction, three additional groundwater monitoring wells were installed to monitor construction dewatering levels.

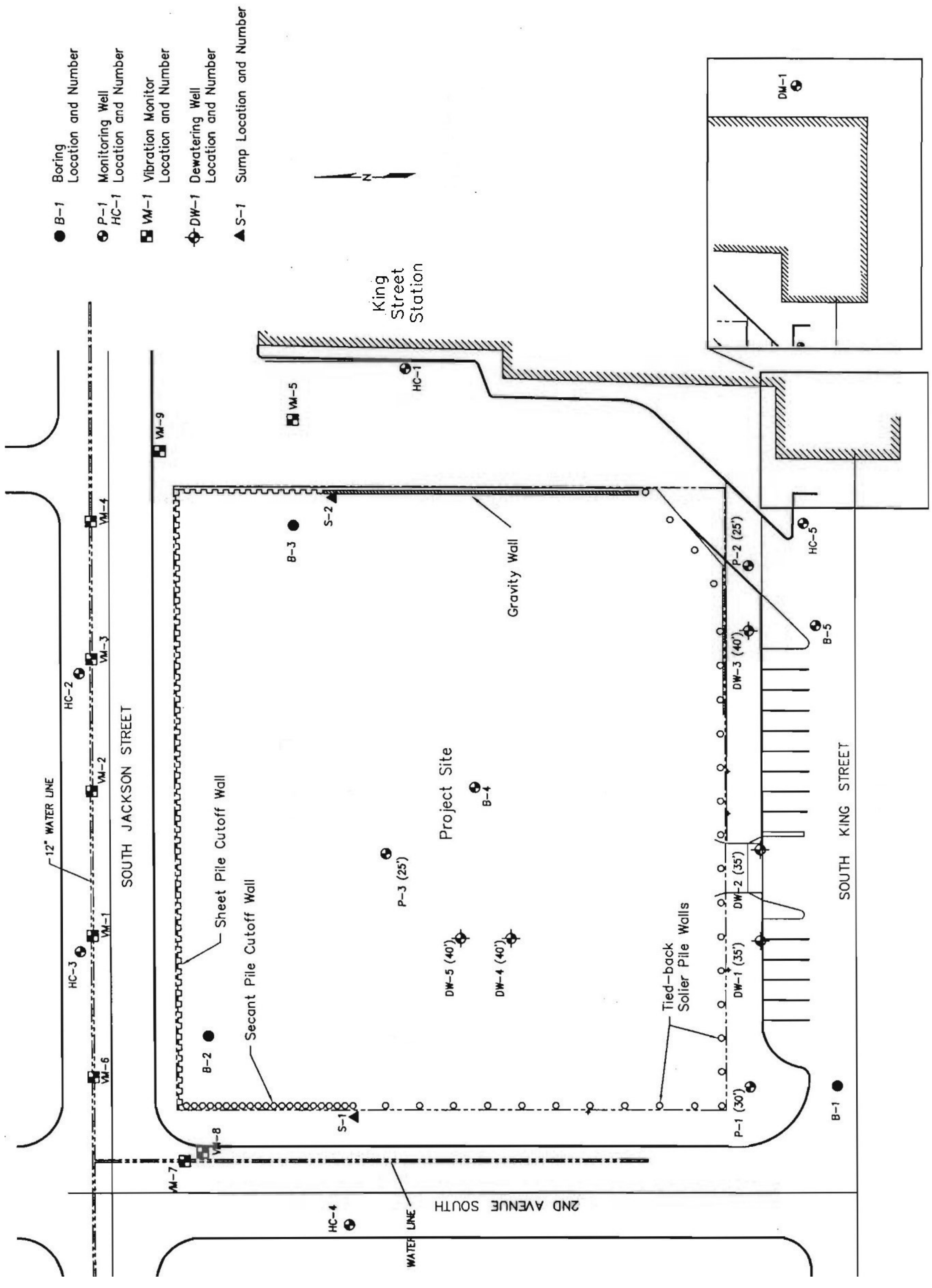

Figure 1. Site Plan (approximate distance between boring B-2 to B-3 is 60 m).

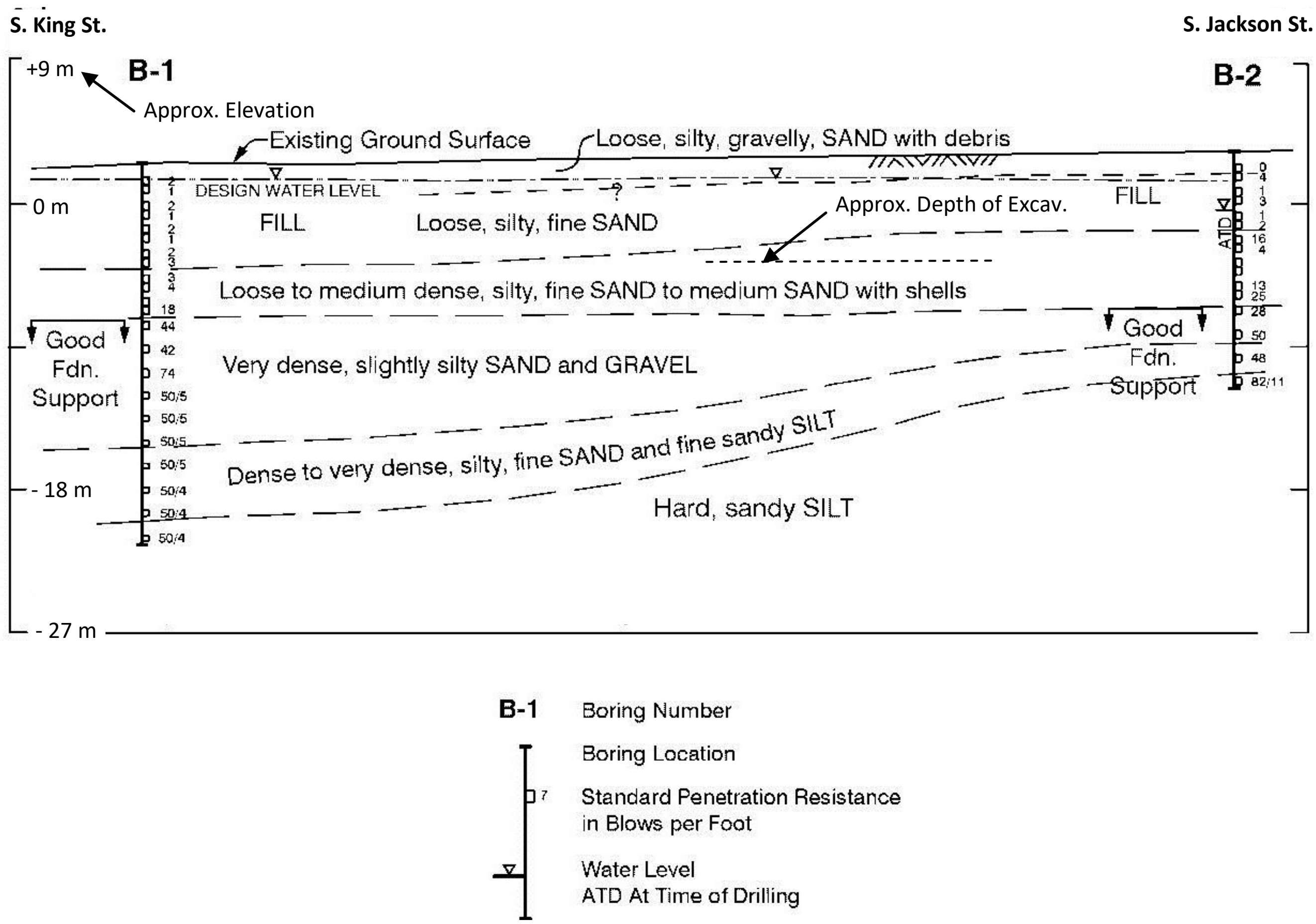

Figure 2. Generalized Subsurface Profile (distance between B-1 and B-2 is approximately 60 m, vertical elevations noted).

Development of Design Criteria

Shoring

Several shoring alternatives were considered for shoring around the site. The depth of cut ranged from less than 4.6 m to about 9.1 m. Key factors affecting the shoring were the upper soft and loose soils, groundwater, constructability, and cost. The site developer was active in selecting shoring alternatives in conjunction with the shoring contractor. Conventional soldier pile and lagging was considered around the entire site but the groundwater on the upgradient (Jackson Street to the north) side would make construction difficult and performance questionable. Modified soldier pile walls such as secant or tangent pile walls were considered for use around the entire site, but cost was a major disadvantage. Other alternatives such as slurry walls or soil mixing walls were also considered but the relatively high costs offset project goals.

Estimated active soil pressures were not atypically high for the conditions at 0.052 kPa equivalent fluid pressure for cantilever and single support conditions, and a 26H envelope (trapezoidal distribution) for multiple support conditions. In this case, 26H is the maximum ordinate of the lateral earth pressure distribution. The difficulty in selecting shoring alternatives was in the passive resistance, which was much smaller than usual because of the submerged soil conditions with estimated passive resistances of about 18.9 kN/m^3 (equivalent fluid pressure). Vertical supports for the selected shoring system would need to be extended down well below the bottom of the excavation to resist kickout. Support conditions were further affected by the poor soils expected in the zone of tieback resistance. Anchors were in fill and loose sand or silt in much of the area, resulting in very low unit resistance values for conventional anchors. As a result, the design recommended use of pressure-grouted tiebacks.

Groundwater Control

Based on the groundwater measurements at the site, the soil permeabilities, and the expected depth of cut, the groundwater flows into the site were estimated to be about 190 to 380 lpm. Although this flow is not considered major, it is enough to slow construction. Control of the groundwater using ditches, sumps, and well points or deep wells was considered. It was particularly noted that dewatering required to keep the excavation in a workable condition for the contractor would likely result in significant drawdowns of the water table around the site. This could have potentially led to ground settlement and possible damage to surrounding building foundations and slabs, and area streets, sidewalks, and utilities. A groundwater cutoff wall along the north would reduce the flows into the site, reduce the need for external site dewatering, and reduce the requirements for internal dewatering beyond simple sumps and pumping.

Sheet pile walls had not seriously been considered because of the depth of the cut along the north side of the site (requiring relatively long sheets with multiple supports), and because of the potential vibrations and obstructions likely to cause problems during installation. However, cost estimates prepared by the contractor for the sheet pile alternative compared to tangent or secant piles cutoff walls showed substantial cost savings, therefore the design team began to analyze the use of a sheet pile cutoff/shoring wall along the north.

Final Design

Final design called for a continuous sheet pile wall along the north side and extending a third of the way along the east side of the site, an open cut along the remainder of the east side, conventional tied-back soldier pile wall along the south and western two-thirds of the site, and a secant pile wall at the north end of the west side (because of a water line, subsequently discussed) as shown in Figure 1.

By significantly reducing groundwater flow into the site from the north/northeast, ditches and sumps could control water in the base of the excavation, with a few well points or deep wells inside the main excavation. Little or no drawdown of the groundwater levels along the north and west were expected, thus protecting the nearby utilities and century-old buildings from settlement. South of the site was a street and a parking lot; some drawdown of the water levels in this direction was tolerable, but no more than a foot or two was expected depending on how many deep wells were required inside the excavation. The greatest risk of groundwater level drawdown was on the east side, adjacent to the King Street Station. The Station was timber pile-supported and drawdowns of a foot or two would not result in significant increases in effective stress and settlement since the soils were granular within this zone and the groundwater level had likely fluctuated through the years. Estimates of potential settlement due to groundwater-induced settlement were less than 13 mm for even the worst combination of conditions.

CONSTRUCTION OBSERVATIONS

Installation of the Sheet Pile Wall

Vibration Monitoring Results

As part of identifying the potential for vibratory damage, a literature search was performed of similar projects with construction vibration measurements as well as construction vibration data generated during the construction of the Seattle Access portion of Interstate 90 in Seattle (Hart Crowser, Inc. 1987). Clough and Chameau (1980) demonstrated that in similar soil conditions to the King Street Center site, ground surface settlements resulting from sheet pile driving were near zero at a distance of 12.2 m from the pile. They also showed that peak particle velocity (PPV) values were well below the damage threshold of 0.051 m/s for distances as close as 6.1 m from the piles. Several other studies (Konon and Schuring 1985; Massarsch 1992; and Holloway et al. 1980) were reviewed for the effects of driven pile vibrations, ground surface settlements, and sensitivity of historic structures. The data further suggested that field measurements of peak particle velocity alone would be an acceptable indicator of damage potential, and could be used as a cutoff value, even though vibration frequency content is recognized as important.

Prior to the King Street Center project, sheet piles had not been used for major excavation support in the downtown core of Seattle. Therefore, the design team, contractor, and City agreed that preconstruction testing of sheet pile installation would be needed to better define the effects of vibrations. Vibratory pile driving equipment was mobilized to the site and four test sheet piles were installed. During installation testing, several points across the site were instrumented to measure PPV using geophones and a vibration monitoring device. Peak particle velocities, noted in Table 1, were generally low (less than 0.005 m/s) during the test installation, except where obstructions were encountered. Velocities tended to be slightly higher when sheet piles were installed adjacent to previously installed piles.

The finalized monitoring plan consisted of monitoring PPVs in the center of adjacent streets spaced 18.3 m apart. The contractor installed six monitoring points in the street, finishing the points using monitoring well monuments. The contractor also installed one monitoring point along King Street Station for monitoring the brick structure and two monitoring points at locations near the fire hydrants. Vibration frequencies were not measured. During sheet pile installation, if the threshold PPV was exceeded, pile installation would stop, the sheet pile removed, and the sheet area would be predrilled to clear the obstruction.

The 15.2 m to 18.3 m long sheet piles that spanned the entire north side and the northern portion of the east side of the site were installed using an ICE 812 vibratory hammer. Peak particle velocities were measured in the longitudinal, transverse, and vertical directions and summarized as vector sums (subsequent values are reported as vector sums).
Construction vibrations were continuously monitored during installation (and removal when necessary) at the two monitoring locations nearest the work with threshold values applied in the monitoring plan. The velocity threshold for utility protection was also applied in monitoring vibrations near King Street Station, even though this threshold limit is lower than that commonly used to protect masonry structures (Holloway et al. 1980; and Konon and Schuring 1985).
Measured peak particle velocities during sheet pile installation did not exceed the threshold limits at the fire hydrants and King Street Station. Measured velocities over the water line did periodically exceed the set threshold value; however, those exceedances occurred over very short time steps. To mitigate the exceedances, the sheet pile contractor installed half sheets at a time rather than pairs and predrilled for the sheets to remove obstructions. In general, the highest PPVs were recorded during hammer startup and shutdown.

Obstructions/Ease of Installation

Obstructions were repeatedly encountered along the length of the north wall and significantly hindered the installation of the sheet piles. After unsuccessful attempts to completely install multiple sheet piles, the contractor elected to probe using an H-pile every 1.5 m to 3.1 m along the wall. As a result, the wall location was shifted several feet to the north to avoid a row of preexisting timber piles. In addition, at the east end of the north wall, the contractor predrilled for obstructions that were encountered. At another location, the contractor excavated 3.1 m to 4.6 m below grade in an attempt to remove timber piles and decking obstructions. During this excavation, velocities ranging from 0.003 m/s to 0.024 m/s were recorded above the water line. The higher readings appeared to be associated with the trackhoe bucket hitting the timber piles. Several sheet piles ultimately were left slightly high of design tip elevation because of obstructions and high velocity measurement during attempts to complete installation.

Installation of Driven Foundation Piles

During the initial phase of foundation pile driving, the vibrations were monitored at various locations including inside the excavation, above the excavation near the source, and off-site. The maximum measured velocity inside the excavation measured 3.56 cm/sec at 7.6 m from the source. The velocity decreased to 0.46 cm/sec when the distance was doubled (15.2 m). Doubling the distance again to 30.4 m from the source resulted in a measured velocity of 0.002 m/s. Outside (above) the excavation at a distance of 12.2 m from the source, the maximum measured velocity was 1.07 cm/sec. At a distance of 18.3 m, the maximum velocity was 0.003 cm/sec. Velocities measured along King Street Station, at distances of 45.7 m and greater, were nominal at 0.002 m/s and less. The nearest pile driving to the station occurred at a distance of about 24.3 m, where vibrations induced from driving were below the established threshold. These data were consistent with the literature sources that formed the basis for the design criteria.

General Excavation Dewatering

Shallow groundwater conditions in loose fill provided a significant dewatering challenge. The strategy utilized for controlling groundwater included balancing two conflicting objectives:

1. To optimize dewatering efforts to provide stable and workable ground conditions at the base of the excavation, and

2. To minimize the potential impact of dewatering on the surrounding buildings and their foundations.

Table 1. Summary of Test Sheet Pile Installation Velocities.

Sheet Pile No.	Method	Time (seconds)	Peak Particle Velocities (cm/sec)		Comments
			6.1 m from Source	12.2 m from Source	
1	Drive	0	0.13	0.13	Vibratory Hammer
		30	0.10	0.10	RPM = 1750
		60	0.15	0.13	
		90	0.18	0.13	
		120	0.13	0.10	
2	Drive	0	0.13	0.08	Vibratory Hammer
		30	0.30	0.18	RPM = 1500
		60	0.48	0.48	
		80	0.53	0.46	
1	Pullout	0	0.61	0.46	Vibratory Hammer
		5	0.08	0.10	RPM = 1500
		15	0.08	0.10	
		35	0.08	0.20	
		50	0.10	0.10	
2	Pullout	0	0.20	0.15	Vibratory Hammer
		10	0.13	0.08	RPM = 1500
		40	0.10	0.08	
		55	0.13	0.08	
3	Drive	0	0.28	0.10	Vibratory Hammer
		30	0.20	0.08	RPM = 2000
		60	0.23	0.10	
		90	1.09	1.04	obstruction
		105	1.37	1.32	obstruction
3	Pullout	0	0.28	0.28	
		15	0.23	0.10	
		30	0.20	0.08	
		45	0.20	0.08	
		55	0.10	0.08	
3	Redrive	0	0.18	0.10	Second Installation
		30	1.57	1.17	obstruction
		45	0.97	0.28	obstruction
		55	0.25	0.13	
4	Drive	0	0.25	0.20	Vibratory Hammer
		30	0.58	0.28	RPM = 2200
		60	0.38	0.28	
		75	0.48	0.56	
		105	0.36	0.30	
3 & 4	Pullout	0	0.36	0.46	3 & 4 Interlocked
		30	0.18	0.13	
		60	0.18	0.18	Stop
		230	0.15	0.18	Restart
		260	0.46	0.23	
		290	0.13	0.10	Stop
		475	0.10	0.13	Restart
		505	0.13	0.13	

No pre-construction dewatering was performed; therefore the contractor expected wet conditions at the base of the excavation. Nine monitoring wells were used to monitor groundwater levels around the site. The cutoff walls were installed first. Soldier-pile walls with lagging and tiebacks were installed next on the west and south sides. During initial installation of the soldier pile walls, groundwater inflow created boiling and heaving sand in the southern areas of the site when excavated to a depth of about 3.1 m. Water pressure below the base of the excavation combined with the looseness of the fill caused running ground conditions.

Five deep dewatering wells (Figure 2) were installed to reduce the underlying groundwater pressure; three on the south side of the site, outside the excavation, and two in the southwest-central portion of the site. The wells were installed as deep as 13.7 m, and were immediately effective in lowering water levels within the excavation area. In addition, the contractor also utilized several surface and shallow sumps at various locations across the site to mitigate localized ponding of surface water.

The combination of dewatering wells used and pumping times were continually adjusted based on monitoring and quantity of water being pumped. On average, total pumping quantities early in the project using all deep wells and several shallow sumps were around 0.00725 m^3/s (435 lpm) and ranged from approximately 0.00568 m^3/s (341 lpm) to 0.01073 m^3/s (644 lpm) with full-time pumping. Several weeks after the start of dewatering, the highest producing wells were limited to pumping only in the morning and later with deep wells limited to five hours per day. From mid-May through June of 1998, the deep wells were limited to progressively shorter periods of pumping or shut off altogether. Pumping quantities reduced substantially and ranged from 0.00378 m^3/s (227 lpm) to 0.00788 m^3/s (473 lpm), or 0.00095 m^3/s (57 lpm) to 0.00190 m^3/s (114 lpm) normalized to absolute time pumping. Approximately 10 weeks after the start of dewatering, all deep wells were abandoned. Utilizing only surface and shallow sumps, dewatering volumes ranged from 0.00441 m3/s (265 lpm) up to 0.0142 m3/s (852 lpm) for about 6 hours per day, or 0.00095 m^3/s (57 lpm) to 0.00251 m^3/s (151 lpm) normalized to absolute time pumping. These pumping rates were slightly below the predicted levels of 0.00315 m^3/s (189 lpm) to 0.00631 m^3/s (379 lpm).

To protect surrounding structures, the maximum allowable drawdown that could be tolerated without causing structural distress to adjacent foundations and floor slabs was estimated. This was used to establish concern levels for monitoring wells. It was estimated that drawdowns of 1.2 to 1.8 meters could theoretically result in about 0.013 m to 0.019 m of settlement of the underlying granular soils. It was also expected that the water levels had varied several feet historically, so the baseline level was closer to elevations of 0 to 0.6 m rather than to the design level of elevation 1.5 m. Target maximum drawdown elevations were established as elevation of -0.3 to -0.6 m.

Figure 3 shows the progression of the water levels during and after excavation. Of particular note is that the initial water levels were somewhat lower than expected, ranging from elevation 0 m in HC-4 (Second Avenue to the west) to about elevation 1.2 m in HC-1 (King Street Station). The major drops in the water levels (in April and June) were due to water-bearing zones causing temporary significant increases in pumping to keep the excavation conditions under control. Major upturns in the water levels can be correlated with significant rainfall events. The two monitoring points behind the cutoff wall along the north, HC-2 and HC-3, depressed only slightly during excavation, and appear to have rebounded fully. The water level at the monitoring point along Second Avenue (west) remains depressed, although only by about 0.3 to 0.7 m. Near King Street Station the monitoring points HC-l, P-2, and HC-5 show water levels that were about 1.2 m lower than the initial values. The levels eventually recovered, although at a slow rate.

Long-term water levels around the new building were expected to be lowered permanently by about 0.3 to 0.7 m due to the influence of the underslab drainage system. That system consisted of behind wall drains and an underslab system with cycling pumps. Equilibrium pumping rates after construction averaged 189 lpm.

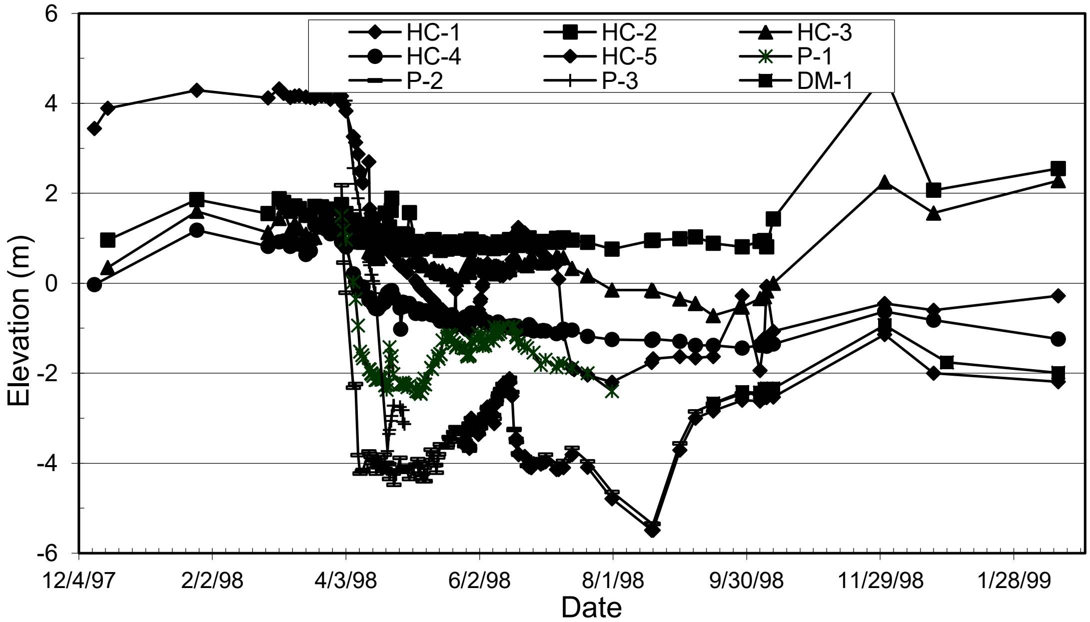

Figure 3. Monitoring Well Elevations (MSL NAD83).

THIS PROJECT AS A TEACHING CASE STUDY

Designing Curriculum

The discussed site forms the basis for a progression of course content that follows the student through a well-defined geotechnical curriculum. The premise behind using a common site for various geotechnical topics is to provide *upward integration of curricular content*. This curriculum model is well established in the medical field (Malik and Malik, 2011) and has been well documented in the literature as *vertical integration*, *integrated curriculum* (Orkwis et al, 1997), etc. Cawelti (1993) of The Association for Supervision and Curriculum Development (ASCD) offered these principles to guide course planning:

1. Offer a balanced core of learning in each course;

2. Adopt in-depth study of a limited number of important topics – this will have a more lasting effect than a course that tries to cover too many pieces of information;

3. Design course outcomes to focus on results, with multiple indicators (assessments) of performance;

4. Design authentic assessments that will encourage originality, insightfulness, and problem-solving, along with mastery of important information; and

5. Get students "doing" early in the course rather than studying all the principles and basics prior to performing.

The author also uses Outcomes-Based Education (OBE). Outcomes-based education is generally accepted as defining, organizing, focusing, and directing aspects of a curriculum on the subjects we want all learners to demonstrate successfully when they complete the course or program. Outcomes-based education is a student-centered, results-oriented curricular design strategy that implies the following (Boschee and Baron, 1993):

1. Subject matter that students are expected to learn are clearly identified;

2. Student's progress is based on demonstrated achievement;

3. Student's learning needs are addressed through multiple instructional strategies and assessment tools; and

4. Each student is provided time and assistance to realize his/her potential.

Curricular Map

This case study was used for both course curricular materials (examples in class, discussions, etc.) as well as assessment materials (homework, final projects, etc.). These courses included:

1. A senior-level technical elective course cross-listed with an introductory graduate course that introduces advanced soil mechanics topics (e.g., advanced seepage and consolidation, slope stability, etc.) designated *Course 1*;

2. A senior-level technical elective course cross-listed with an introductory graduate course that introduces the theoretical and design aspects of foundation engineering and earth retention structures designated *Course 2*); and

3. An advanced graduate-level course that continues with the theoretical and design aspects of the same subject materials designated *Course 3*.

Table 2 summarizes the courses and Table 3 summarizes how curricular content is mapped to the courses.

Table 2. Course Descriptions.

Course Designation	Course Name and Level	Prerequisites
Course 1	CEE 446/546 – Advanced Geotechnical Engineering, senior level technical elective/ introductory graduate course.	Soil Mechanics (undergraduate junior level required course)
Course 2	CEE 447/547 – Foundation Engineering, senior level technical elective/ introductory graduate course	Soil Mechanics
Course 3	CEE 632 – Advanced Foundation Engineering, graduate level course	Soil Mechanics

Table 3. Curricular Map of Subject Material to Courses.

Main Subject Area	Applicable Curricular Subject Matter	Course
Subsurface investigations	SPT Monitoring wells Subsurface interpretation Cross sections	*Course 2* *Course 1* *Course 2* *Course 2*
Retaining wall design	Gravity Anchored soldier pile Anchored secant/sheet pile	*Course 2* *Course 2* *Course 2*
Seepage analyses	Applying flow nets Basal heave Construction dewatering	*Course 1* *Course 1* *Course 1*
Foundation design	Shallow foundations Deep foundations Uplift pressures Permanent dewatering	*Course 2* *Courses 2 and 3* *Course 1* *Course 1*

ASSESSMENT OUTCOMES

The remainder of this paper focuses on how the summarized case study is used for assessing student learning. It is not the intent of this paper to summarize curricular content that is presented in a lecture-type setting. It is the author's opinion that large case studies are appropriate for assessment in the form of homework and final projects. Therefore, the following sections present how the case study is formed into homework and final projects and the associated assessment.

General Assignment Layout

The following is offered the student as an introduction to the first homework that uses the case study:

"A downtown office structure is being constructed at the site shown on the attached Site and Exploration Plan (Figure 1). Key considerations of the retaining walls are soil support for below grade construction and appropriate groundwater cutoff to maintain a stable and workable excavation. Key considerations for foundations is that they are stable and do not settle excessively. The top of the north wall (TOW) is elevation 3.81 m (above MSL) and excavation level (EL) is at elevation -1.92 m (below MSL). The attached packet of information contains subsurface explorations and groundwater level measurements (Appendix A and Figure 3). Appendix A also contains a key to the explorations logs. Performed laboratory testing is included in Appendix B. Assume a uniform surcharge pressure of 12 kPa for the street loading along South Jackson Street."

Note that the exploration logs and laboratory data in Appendix A and B, respectively, are presented in the same "raw" form that is presented to the student. This is by design so that the student gains skills in reading various forms of presented geotechnical data that is standard of practice.

Subsurface Investigations

The student is asked to perform an analysis of the provided subsurface and laboratory data. This analysis helps students develop skills to interpret subsurface soil and groundwater conditions. The student's analysis and interpretation includes the entire site and then focuses towards a part of the site where knowledge is applied and skills are formed. The interpretation includes the dimensions and soil properties for the various soil sublayers the student deems necessary to include in the design in relation to retaining walls and foundations. The student also summarizes groundwater conditions they deem necessary to use in their design. The students are cautioned to consider how conditions encountered in the subsurface explorations will affect design and construction, and how the timeline of groundwater conditions effect their design.

The student is required to provide:

1. An engineering cross-section of site soil and groundwater conditions similar to that shown in Figure 2. This forms the basis of their analysis.

2. A summary of overall site subsurface soil and groundwater conditions in a tabular form. Soil layers are grouped into units with similar or like engineering properties. Groundwater conditions, as summarized as well, with commentary on how subsurface soil and groundwater conditions might affect design and construction.

3. A summary of general site preparation techniques that would be used for the various types of civil engineering structures. Site preparation techniques are actions a contractor would need to do prior to building a geotechnical design element (footing, pavement, etc.). The students are encouraged to summarize conditions on a construction method basis.

4. A list of the predominant exploration types used at the site with commentary on why each type is used and the type(s) of information that can be obtained from each for these specific explorations.

5. Commentary on the lack of subsurface information at the site for the specified designs and an analysis of how they would have conducted the subsurface exploration program differently (with specific proposed additions/changes, types, depths, etc.), if at all.

6. General comments on site soil and groundwater variability. Given the complexities of the site, a formal spatial variability analysis is not required.

General assessment consists of format, organization, and technical content. The following is used to assess technical content that corresponds to defined learning outcomes:

1. Cross section
   a. Location of cross section shown on plan
   b. Provides detailed justification for the location of the section
   c. Identifies explorations used
   d. Identifies projected or interpolated explorations on the section
   e. Properly connects divisions between layers
   f. Shows groundwater conditions (perched, static, confined, etc.)
   g. Establishes correct elevations
   h. Provides a legend
   i. Provides horizontal and vertical scale
   j. Indicates vertical limit of explorations (depth limit)

2. Subsurface summary
   a. Properly groups major soil units
   b. Includes summary soil description
   c. Includes depths and/or thicknesses
   d. Includes comments on groundwater (and be technically correct)
   e. Include effects on design and/or construction

3. Site Preparation (summary by load type)
   a. Foundation types
   b. Pavements
   c. Utilities
   d. Comments on site preparation

4. Exploration types
   a. Type
   b. Advantages and disadvantages of each
   c. List of information for each

5. Evaluating exploration plan
   a. Cite source for quantifying explorations (justification from the literature)
   b. Types of explorations

c. Number of explorations

d. Depths of explorations

Retaining Wall Design

The case study site is used in several homeworks. Retaining wall types are prescribed with specific locations provided.

Gravity

The student is asked to provide detailed geotechnical design-level recommendations for a gravity retaining wall along the east property boundary. The anticipated top of wall elevation is 3.20 m. At a minimum, the recommendations are expected to include lateral earth pressures and location; resultant force and location; wall dimensions to adequately provide for bearing capacity, lateral translation, overturning, etc.; drainage; minimum embedment; open cut excavations; and anticipated deformations.

The students are assessed (again corresponding to learning outcomes) on the following:

1. Appropriate design criteria are established

   a. Property line conditions

   b. Groundwater conditions are appropriately established

   c. Surcharges behind the wall are established

   d. Adjacent footing surcharge loads are recognized

2. Correct design input is established

   a. Soil pressure

   b. Passive pressure

   c. Active earth pressure

   d. Coefficient of friction

   e. Soil density

3. Design recommendations

   a. Wall stem, heal, and toe design

   b. Overturning, sliding, and eccentricity calculations

   c. Drainage design

   d. Foundations

Anchored Soldier Beam

The students are asked to design a permanent anchored soldier beam and timber lagged wall for the west wall. When construction of the wall is completed, a 7.3 m wide entrance ramp will be constructed 3 m behind (parallel to) the wall. The wall is to be constructed in the soil conditions shown in the profile in Figure 1. No existing structures, or underground utilities, are located within 20 m of the top of the proposed wall location. Given this type of wall, the student is instructed that no groundwater pressures are present on the west portion of the site (simulated conditions for the purposes of the assignment). The assignment consists of:

1. A subsurface analysis focused towards the west wall. The previously described assignment is the basis.

2. Considered alternatives.

3. Design (sizes, lengths, spacing, capacities, pressures, etc.) of:

    a. H-beam sections,
    b. Lagging,
    c. Anchors, and
    d. Wales.

4. Vertical and lateral deformation estimates.

Assessment of technical content that corresponds to defined learning outcomes:

1. Load types acting on the wall.

2. Location of the critical failure surface.

3. Apparent earth pressures.

4. Other lateral loads acting on the wall.

5. Anchor loads, maximum bending moments, and reaction forces.

6. Anchor design (unbonded length, bonded length, and capacity).

7. External stability.

8. Selection of structural elements including:

    a. Anchor tendons, and
    b. Soldier beam section.

9. Timber lagging.

10. Axial and lateral capacity of the soldier beam.

11. Wall deformations (vertical and lateral).

Anchored Sheet Pile

The goal of this assignment is to design (part of) an anchored sheet pile retaining wall. The wall is intended to be temporary along the north property line. This progression of subject matter is divided into four distinct parts:

1. A subsurface investigation focused towards the north wall. The previously described assignment is the basis.

2. A preliminary design of the sheet pile wall. The design includes design lateral stresses/forces and vertical forces. In a preliminary design, the student provides at least the following figures:

    a. A figure that shows the design lateral/vertical pressures/forces they intent to base their design on.
    b. A figure that shows the expected failure zone and how it might affect the design.

c. A figure that shows the final geometries of the wall elements and the selected structural properties including anchors. In this part of the design, the depth of embedment the student determines is based on limit equilibrium only, not seepage (see Seepage Analysis).

3. A seepage analysis later defined in the next section.

4. Final design.

The following is used to assess technical content learning outcomes that correspond to what the student is required to provide:

1. Proper selection and computation of lateral earth pressure coefficients.

2. Proper computation of active and passive soil stresses.

3. Proper computation of surcharge and water stresses.

4. Proper computation of net passive.

5. Selection of proper method of analysis.

6. Proper execution of analysis.

7. Correct results including anchor force, bending moment and depth of embedment.

8. Final cross-section showing wall design including selected section, problem geometry, anchor force, etc.

Seepage Analysis

The limit equilibrium analysis and seepage analysis are separate assignments given the complexity of the material involved. It also allows the student a more in-depth study of the topics. The seepage portion of the case study includes:

1. The student is asked to perform a seepage analysis of the north wall conditions to determine the required depth of embedment of the sheet pile wall to achieve a factor of safety of at least 2.0 against uplift. The student is reminded that the sheet pile wall is being used to ensure that there is necessary cutoff of groundwater flow to prevent heave at the excavation level, and to achieve a workable excavation for the contractor.

2. The analyses are performed using commercially available computer programs (two are provided to the students and they are free to choose) presented in the classroom setting and required to check the results using a hand drawn flow net.

3. The student is required to provide a figure of their flow net, calculated inflow and seepage pressures. The hydraulic conductivities are estimated from the subsurface investigation analysis previously discussed using published correlations (USEPA, 1984; Baum et al, 2008).

4. The student is then asked to compare the results of embedment depth from the limit equilibrium and seepage analysis. The student is then asked to comment on what controls design, stability or seepage.

The student is then asked to complete a "final" design that is formatted and organized with well-documented calculations. The following is used to assess technical content learning outcomes that correspond to what the student is required to provide:

1. A subsurface analysis of the north wall using the previously performed site subsurface analysis.

2. Defining appropriate boundary conditions.

3. Selecting an appropriate seepage flow model including:

    a. Analysis type,
    b. Soil layers, and
    c. Soil properties giving specific attention to hydraulic conductivity functions.

4. Presentation of the seepage analysis output for evaluation (pressures, heads, flow lines, etc.).

5. Tip elevation of the sheet pile wall for the defined factor of safety against heave.

Foundation Design

The student is asked to provide a detailed foundation assessment of the structure and provide design-level geotechnical recommendations. The students are instructed to consider alternatives and consider the feasibility of a shallow foundation system (footings and floor slab) to support the structure. The assignment requires the student to provide geotechnical design-level recommendations that includes estimated structural loads, computed ultimate and allowable bearing capacity, soil properties developed as part of the assessment/design, resistance to lateral loads, temporary and permanent drainage (and how it affects the design), settlement, and construction considerations relevant to the design.

<u>Shallow Foundations</u>

The students are asked to provide:

1. Estimated structural loads on continuous and discrete foundation elements, as well as slabs-on-grade.

2. The factor of safety on bearing capacity, if the footing is dimensioned so as to limit total settlement to 2.5 cm.

3. Estimate of total settlement of proposed footings immediately after construction. In their solution, they are asked to show a plot of the distribution of the strain influence, standard penetration resistance and equivalent modulus of elasticity directly on a scanned copy of the boring log.

4. Estimate of the size of the square footing necessary to limit the immediate settlement to 2.5 cm for the anticipated column loads (same timeframe as 3).

<u>Deep Foundations</u>

If the shallow foundation analysis is performed correctly, the students determine that the use of a shallow foundation system will result in excessive settlement (and inadequate factor of safety). Therefore, they design a deep foundation system that determines the type, size, depth, and capacity to support the anticipated loads. Assessment is performed similarly as previously described.

A follow-on assignment is used that explores verification of predicted pile capacity. The student is provided the driving log presented in Appendix C and asked to assess the vertical load carrying capacity of the pile. It also allows the student to provide overall comments, method of assessment and any qualifiers for that type of verification.

Photographs of site construction are included in Appendix D for completeness.

CONCLUSIONS

The King Street Center case study site supports several conclusions:

1. This site provides a wide range of geotechnical design elements that can be used for student learning in the classroom.

2. Various retaining wall types at a site can achieve the desired design goals. The same applies to foundations.

3. Specific assessment corresponds to learning outcomes to define how elements of the case study are used in student learning.

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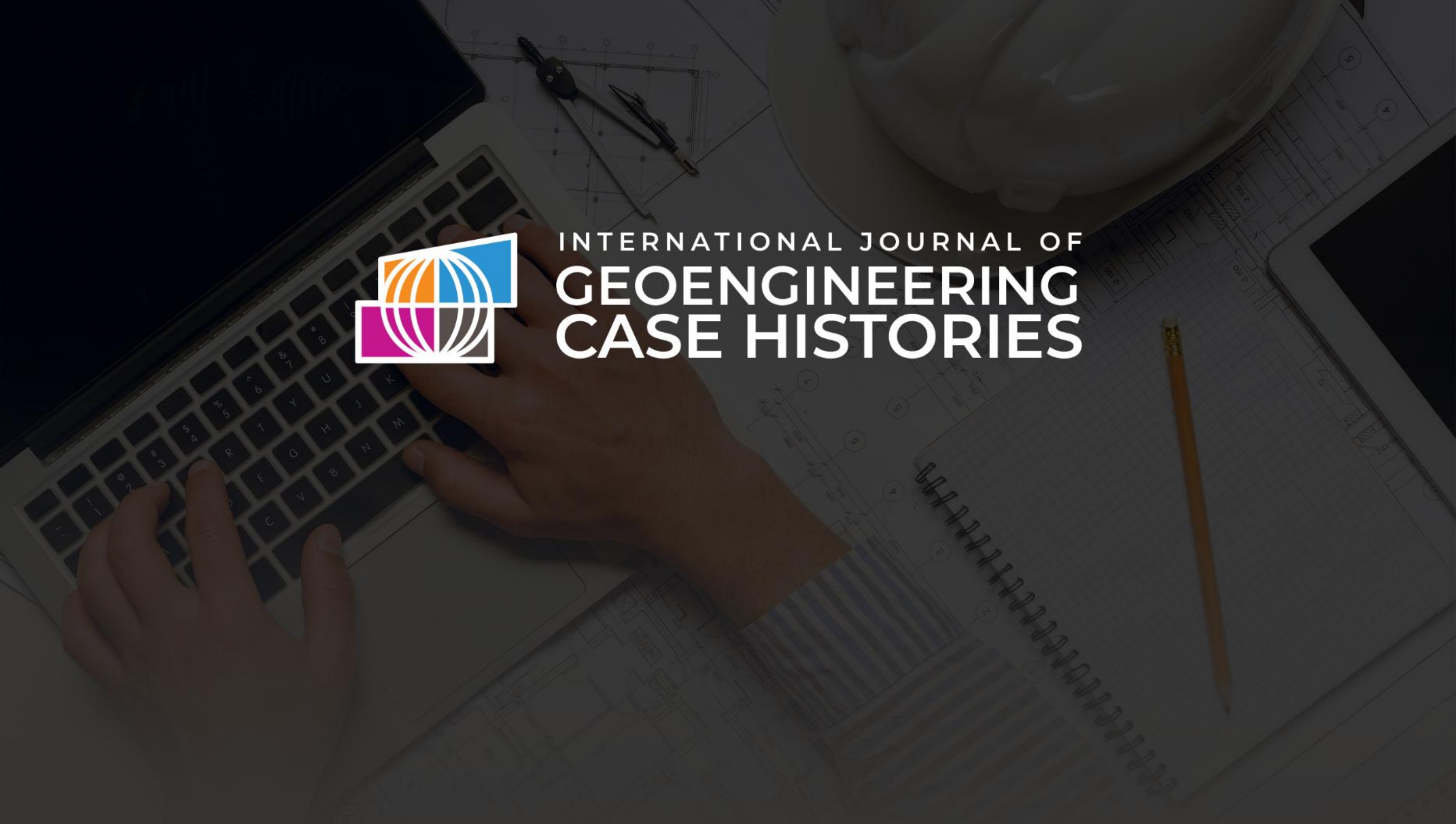
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