GROUNDFREEZING TECHNICAL PAPERS

Frozen Earth Cofferdam Design

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Abstract

Structures designed to transfer storm runoff water from surface collector systems to deep large diameter tunnels pass through a variety of subsurface soils in the Milwaukee, Wisconsin area. During construction, problems developed relative to the use of conventional earth support systems. Ground freezing was selected for temporary ground support and ground water control during excavation and construction. Irregular wail geometry required the use of reinforcement members to avoid excessive tensile stresses in the frozen soil at critical wall locations. Design and evaluation of the frozen earth structure required specialized computational methods to account for the time-, stress-, and temperature-dependent material behavior and adfreeze bond between frozen soil and reinforcement members. The analysis clearly showed where modifications in cofferdam design were required. Small displacements predicted for sides of an excavated wall opening were in agreement with no observed movement during the construction period.

Introduction

Structures designed to transfer storm runoff water from surface collector systems to deep large diameter tunnels pass through a variety of subsurface soils in the Milwaukee, Wisconsin area. During shaft construction, problems developed relative to the use of conventional earth support methods. Earlier failures with slurry diaphragm walls made ground freezing an attractive alternative method. Required excavation limits for a transition structure imposed an irregular wall geometry on the proposed frozen earth cofferdam. Preliminary design of this cofferdam, using simplified numerical methods, showed the potential for large tensile stresses in certain areas within the wall.

The frozen earth wall system selected included a deep shaft and three adjacent cells, two elliptical and one circular in cross-section. Required excavation involved cutting an opening through the frozen wall between cells. Excessive tensile stresses within the wall and adjacent to the wall openings were avoided by the use of steel-concrete reinforcement members placed so as to transfer loads to less critical areas. Stresses and deformation within the frozen earth cofferdam and reinforced frozen wall sections were evaluated for the excavation and construction phase using both an elastic and a time dependent nonlinear finite element analysis with soil stiffness and soil creep parameters. Field measurements during construction showed agreement with predicted wall movements.

The dropshaft and approach structures (Figure 1) were intended for transfer of storm runoff water from surface collector systems to a deep large diameter tunnel. The dropshaft, with an inside diameter of 5.64m, extended down through 50+m of water bearing soils and another 35m of rock. Each dropshaft included a vortex type entrance, an approach box structure, and a trash rack structure located near the ground surface (Figure 1).

Excavation for construction of the dropshaft and approach structures involved two phases, each with different earth support methods. A circular excavation with diameter of about 6.1 m and extending down to bedrock was required for the dropshaft. Construction schedules included the dropshaft as part of the first phase. The second phase included the trash rack and transition structure. The trash rack, with a 6.1 m square cross-section, required excavation to a depth of I 9.8m. The transition structure with width of 3.66m and varying depth extended from the trash rack to the dropshaft (Figure 1). These dimensions represented the minimum excavation limits required to build the dropshaft and approach structure.

Construction was located on a 4.8 ha site on the south bank of the Menomonee River in proximity to the Menomonee and Milwaukee River junction. Glacial till was the most abundant soil on all boring logs. There was no consistent soil stratification and the till was composed of a clay matrix with varying quantities of silt, sand, gravel and boulders. A generalized boring profile for the site is given in table 1.

Strata	Depth (m)	Description
I	0.0 - 4.0	Fill soils, sands and clayey silts
II	4.0 - 18.3	Post glacial estuarine silts
III	18.3 - 21.3	Glacial silty sand-clay mixtures
IV	21.3 - 33.5	Glacial lacustrine sands
V	33.5 - 38.1	Lacustrine sands and silts
VI	38.1 - 50.6	Top of rock
	50.6	

table 1. Generalized soil profile.

Frozen Wall Geometry

The trash rack, transition structure, and dropshaft (Figure 1) form a single structure in what would appear to be best suited for a rectangular excavation. Lateral earth support using straight frozen walls would include high tensile stresses and bending moments which are not acceptable for frozen soils. More conventional frozen walls are typically circular or elliptical in cross-section so as to create a structure with only compressive stress. Use of a single elliptical cofferdam would require a large number of freeze pipes making the single elliptical cofferdam economically unfeasible.

In order to minimize the frozen wall length, reduce the required refrigeration capacity, and limit the excavation volume, a series of four separate but interconnected frozen soil structures were selected (Figure 2). Each unit would be a curved compressive structure and could be frozen independently. The dropshaft was constructed to a depth of about 55m as part of Phase 1, hence the shaft wall was in place prior to freezing cells 1, 2 and 3. Cell 1 was of elliptical cross-section with a major axis of 12.2m, minor axis of 1 1.9m and extended to a depth of 42.7m. Cell 2 was also of elliptical cross-section with a major and minor axis of 12.8m and I 2.2m, respectively, and a depth of 30.5m. Cell 2 provided lateral earth support during excavation and construction of the entrance to the transition structure. Cell 3 was a circular frozen cofferdam with diameter of 10.7m and depth of 30.5m placed tangent to cell 2. The trash rack was constructed within cell 3.

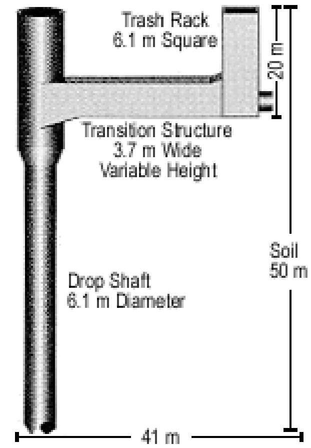

Figure 1. Proposed underground structures (from Contact Documents, 1983).

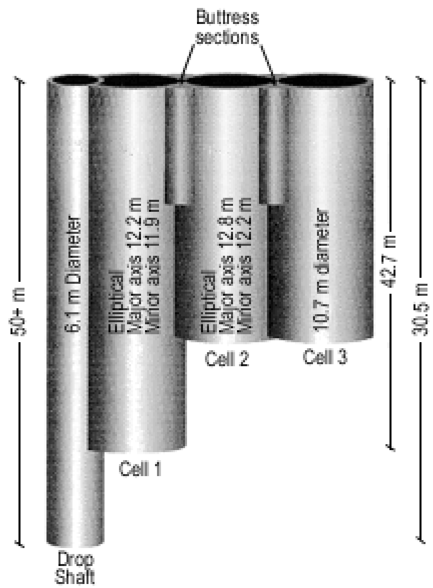

Figure 2. Three dimensional illustration of the proposed frozen earth structures. Frozen walls formed by installing 76 mm diameter steel freeze pipes on approximately 0.9m centers.

After freezing and excavation of these separate cofferdams (Figure 2), subsequent construction would involve cutting an opening through adjacent cell walls for construction of the transition structure. Of major concern was the development of tensile stresses within the frozen soil adjacent to the wall opening. To avoid this problem, vertical reinforcement members were placed, prior to ground freezing, to the buttress sections near the wall openings. A steel beam, embedded in concrete, served to transfer lateral loads to less critical areas above and below the wall opening. Stresses and displacements within the frozen wall cofferdam were evaluated using methods outlined in subsequent sections.

Material Properties

Soil properties required for evaluation of stresses and displacements within the frozen earth cofferdam are listed in table 2. Since weaker fine-grained soils control design limits, all samples were obtained from stratum II (estuarine silts in table 1) at elevations corresponding to proposed wall openings required for construction of the transition structure. Detailed information on sample preparation, uniaxial compression and tensile tests on frozen samples, and data analysis, is given by Sopko (1990). An upper limiting test temperature of - 109^{o}C was selected for this project. This temperature would be easily attainable for most refrigeration plants and selection of wall thickness recognized that temperatures would be colder near the freeze pipes and warmer near the outer wall surface. Soil parameters and bond link element values for the nonlinear analysis were obtained from work reported by Soo (1983).

*Frozen soil (Soo, 1983; Sopko, 1990)

Property	Compression	Tension
Parameter n	1.61	5.00
Parameter b	0.26	0.55
Proof stress (MPa)	95.45	9.38
Elastic modulus (MPa)	54.92	--
Poison's ratio	0.33	--
Uniaxial compressive strength (MPa)	3.11	--
Composite steel / concrete (Merrit, 1983)		
Elastic modulus (MPa)	54.92	54.92
Poison's ratio	0.45	0.45
Bond link elements (Soo, 1983)		
KY	9500	
KZ	3000	
PMX	100	
PMY	100	
Parameter n	1.6	
Parameter b	0.2	

*Temperature = -10 ^{o}C

Structural Analysis and Duscussion

Conventional methods of analysis (Sanger and Sayles, 1979) provide very conservative approximations for design of frozen earth cofferdams with complex wall geometry. More accurate methods of analysis were needed. Application of the finite elements method (FEM) to be frozen earth structures has been described by Klein and Jessberger (1979), Klein (1981), Jessberger (1982), Soo (1983), and Soo, et al (1985). Its use for analyzing frozen earth structures has been slow due to limited commercial availability of FEM programs that can handle the time-dependent behavior and material properties which vary with temperature and stress, typical of frozen soil materials. Use of the FEM with a three-dimensional model will be described.

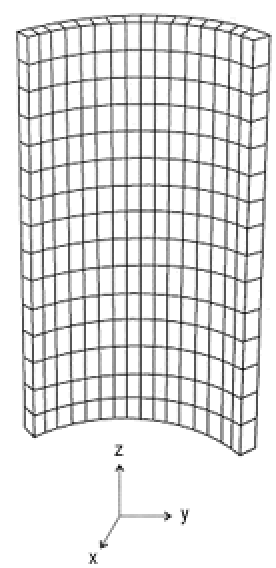

Figure 3. Three dimensional grid used for the elastic analysis.

5.1 ELASTIC ANALYSIS

Individual Cells. The linearly elastic SAP IV program using a three dimensional grid model (Figure 3) with steel/plate elements was used to determine both internal stresses for individual cofferdams and forces transferred to buttress sections (Figure 2). Dimensions were scaled to proposed field values for each cell. A four node shell element was selected for this phase of the analysis because it could model the three-dimensional geometry of the cofferdam while limiting the total number of nodes and required computer time. Nodal coordinates in the z-direction were selected to best correspond with major soil strata changes. Horizontal nodal coordinates were selected to best define the plan cross-section of each cell with spacing set so as not to exceed a 2-to- I aspect ratio for each three dimensional element. A thicker soil strata required more rows of elements. The element (wall) thickness initially selected for the dropshaft was 1.37m and 0.91 m for individual cells. These thicknesses could be readily attained in the field using a single row to freeze pipes. Note that these wall thicknesses are significantly less that those indicated by conventional analysis methods (Sanger and Sayles, 1979).

Boundary conditions were essentially the same for all four models. Each node at the bottom of the grid was restrained in the x y and z directions, and permitted to rotate about the x and y axis. Nodal points along the outer vertical edge were restrained as shown in Figure 4. To determine reactive forces imposed on buttress sections, it was necessary to replace the restrained nodal joints with boundary elements, one parallel to the x and y axis, respectively. The resultant forces and moments are computed later in the analysis. Nodal points representing portions of the frozen cofferdam embedded in unexcavated earth were restrained with springs directed radially toward the center of the shaft.

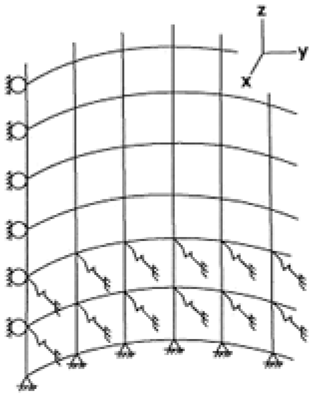

Figure 4. Boundary conditions imposed on the grid shown in Figure 3.

Since it was uncertain whether passive earth pressures would develop, some analyses were run without spring restraints for comparison. Normal pressures applied to the external surface of all elements were extrapolated from the pressure diagram available in project specifications (Contract Documents, 1 983). Internal stresses computed in this first analysis identified the location and magnitude of maximum stresses.

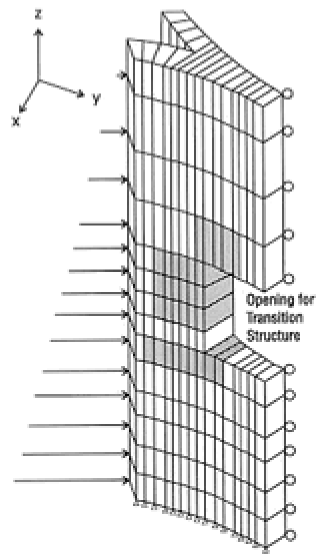

Figure 5. Three-dimensional grid for the buttress section showing location of excessive tensile stresses (shaded) for the elastic analysis.

Comparison with available compressive strengths showed that the design would be adequate except that the time-dependent behavior and a decrease in frozen soil strength with time had not been accounted for. Using results from compression creep tests conducted on the organic silt, Sopko (1 990) developed a relation between time and reciprocal of stress. Based on these relationships and the maximum computed compressive stresses, the service life or time to failure was determined for each cell. High compressive stresses at the cell 1 wall/dropshaft intersection indicated a potential problem area. To compensate for these high stresses an additional row of freeze pipes was installed so as to double the wall thickness at the critical areas.

Buttress Sections. The next concern was buttress sections and the potential for high tensile stresses when openings were excavated through the wall between two cells. Before proceeding with this phase of the analysis, reaction forces and moments, determined by boundary elements, were computed and summarized. Again a three-dimensional elastic model (Figure 5) was used to determine (1) nodal deflections in the y-direction, (2) magnitude and location of maximum compressive stresses, (3) location and magnitude of any tensile stresses, and (4) reactions in the y-direction for use in a subsequent analysis. The buttress section between cells 1 and 2, with the higher stresses, was selected for analysis. Elements illustrated in the FEM grid (Figure 5) are the eight-node brick (type 5) available in the SAP IV program. Loads, imposed at the various nodes, included forces and moments determined from the elastic analysis for cells I and 2, Nodes along the base of the model were restrained to represent hinges with three degrees of freedom. Boundary nodes along the internal side of the buttress section were restrained with boundary elements representing rollers with five degrees of freedom. Boundary elements were used to determine the y-component of loads imposed on the buttress section from the cells. These load components were used in a later two-dimensional analysis.

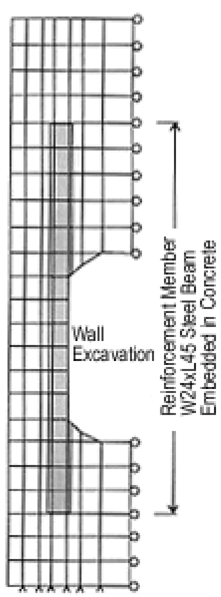

Figure 6. Two-dimensional FEM grid for a buttress section between cells 1 and 2.

Results of the analysis indicated that (1) excessive tensile stresses near the face of the opening through the buttress sections would likely result in structural failure (Figure 5), (2) compressive stresses within the buttress section were well within the acceptable range as compared with laboratory test results, and (3) deflections along the x-axis due to stress imbalance from adjacent cells were negligible. High tensile stresses showed that modifications in the cofferdam design were required. A review of several options indicated that use of reinforcement members designed and placed so as to transfer loads to less critical areas was the most practical. Preliminary considerations involved drilling a 0.9m hole prior to ground freezing, placement of a steel member, and backfilling with concrete. Questions remained concerning size of the composite steel/concrete reinforcement and whether adfreeze bonds would be sufficient for load transfer from frozen soil to the reinforcement members.

5.2 REINFORCED SECTIONS

Linear Analysis. The FEM grid shown in Figure 6 represents a vertical section through the buttress area between cells 1 and 2. A two-dimensional plane strain linear elastic model was used for initial evaluation of nodal displacements adjacent to the proposed wall opening and the magnitude of shear stresses developed between the frozen soil and the steel/concrete beam. Boundary conditions consisted of simulated hinges along the horizontal base of the grid with simulated rollers along the vertical symmetric slice. loads applied at nodal points were those obtained from boundary element forces for the FEM model shown in Figure 5. These loads would contribute to bending of the reinforcement member. Reinforcing elements were the same as those described by Merritt (1983). Material properties are listed in table 2.

Results of this analysis indicated that tensile stresses along the opening between cells 1 and 2 would be transferred to the steel/concrete reinforcing member. Computed displacements along the face of the opening where less that 1.6mm. Based on the elastic analysis, shear stresses at the interface between frozen soil and reinforcing members were less than 287 KPa, below the range which would cause adfreeze bond failure. A further analysis of the reinforced section considered the time-dependent properties of the frozen soil.

Time Dependent Nonlinear Analysis. A computer program described by Soo (1 983) appeared to contain the features needed for this analysis. A nonlinear plane stress element was capable of representing both linear and nonlinear material behavior in both tension and compression, ideal for the frozen soil. The reinforcing element (steel/concrete) was modeled with linear elastic elements using the same parameters in both tension and compression (Merritt, 1983). The adfreeze bond between frozen soil and the reinforcement member was modeled by a bond link element. These three element types permitted simulation of the time dependent creep deformation of the adfreeze bond up to some limiting deformation, at which time slip failure would occur. The two-dimensional nonlinear FEM grid for the buttress section is shown in Figure 6. The shaded area represents the reinforced section. Material properties required for the three element types are listed in table 2.

Time steps used to evaluate creep deformation were arbitrarily selected as one, seven and thirty-days. Compatible creep proof stresses and proof strain rates (table 2), selected by Soo's (1 983) method, ensured proper execution of the program. The analysis indicated that the reinforcement would transfer loads to less critical areas, thus avoiding high tensile stresses in the wall adjacent to the opening. The analysis showed that most of the displacement at the sides of the opening between cells occurred in the initial elastic phase. Very little deformation was predicted for times of one, seven and thirty-day intervals. Deformation indicated on Figure 7 are extremely small and were not observed during construction, possibly due to limitations of available measurement equipment.

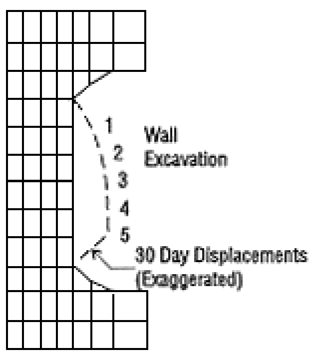

Figure 7. Computed nodal deformations at the side of the opening between cells 1 and 2.

Position	Time (days)	Nodal Displacement (mm) Vertical	Horizontal
	1	-0.30	1.34
1	7	-0.40	1.40
	30	-0.30	1.43
	1	-0.33	1.37
2	7	-0.37	1.43
	30	-0.33	1.49
	1	-0.30	1.37
3	7	-0.33	1.49
	30	-0.30	1.52
	1	-0.27	1.43
4	7	-0.30	1.52
	30	-0.30	1.58
	1	-0.24	1.46
5	7	-0.27	1.55
	30	-0.27	1.62

Conclusion

1. This project has demonstrated that the analysis techniques used are suitable for design of frozen earth cofferdams having complex wall geometry, and that they are capable of evaluating the effect which reinforcement members placed in the frozen wall have on stresses and wall deformation.
2. The linearly elastic SAP IV program with a three-dimensional model and shell/plate elements computed both internal wall stresses for each cell and the magnitude of forces transferred to the buttress sections. Stress levels indicated adequacy of cell wall thickness and loads were used for subsequent analysis of buttress sections.
3. The time-dependent nonlinear analysis of reinforced buttress sections provided both stresses and displacements at specified locations in the wall. The three element types available appeared to correctly represent the reinforced wall behavior. Small displacements predicted for sides of the excavated wall opening were in agreement with no

observed movement during the construction period.

4. Placement of reinforcement elements (before freezing) in the cofferdam buttress sections reduced tensile stresses in the frozen soil adjacent to the wall opening to acceptable levels and permitted construction to proceed using the most economical frozen wall geometry.
5. It is important that complete information, relative to frozen soil moduli and creep parameters, be available on future projects which include detailed structural analysis of complex wall geometry. Standardized laboratory test procedures are needed for evaluation of these moduli and frozen soil parameters.

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