A successful case of water cut-off by ground freezing in the excavation of the shaft DST-1

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ABSTRACT: The construction of the shaft DST-1, 14 m diameter, 65 m deep of the Dugway Storage Tunnel Project (Cleveland), was completed following significant obstacles in the shaft excavation process. After applying different soil treatments (dewatering, jet grouting, permeation grouting) ground freezing was selected as a technically appropriate alternative. Since steel liner plates and ribs were installed in the initial excavation phase, consideration was given to the frost pressures generated by the freezing process. This paper describes the field and laboratory testing as well as the engineering analysis required for freezing to permit excavation below the existing liner plates and into the underlying bedrock. The exiting liner plates and ribs were instrumented to assess the effects of the freezing and compare additional forces with those evaluated during the ground freezing design.

1 INTRODUCTION

The Dugway Storage Tunnel (DST) is a combined sewer overflow deep tunnel project that will provide additional storage of combined sewer flows during wet weather events reducing the number of combined sewer discharges into the environment.

The tunnel alignment is approximately 4.5 km long, excavated with single shield hard rock TBM 8.22 m excavation diameter. The finished internal tunnel lining diameter is 7.30 m. Depths to tunnel invert generally range from 55 to 70 m below ground surface.

The project will have a total of 6 deep shafts with internal lined diameter between 5 m to 15 m and 4 adit connections between shafts and tunnel.

The 14 m diameter shaft DST-1 is the TBM launch shaft and is 65 m depth, was constructed through fill material, glacial till and lacustrine deposit.

Part of the shaft was constructed through soft ground and part also encounter chagrin shale bedrock.

Specifically, the excavation consisted of 29 m in soil and 36 m in shale rock.

Figure 1. Project overview

The shaft was excavated by conventional method using excavator and much box. Steel liner plates and steel ribs were installed for lateral ground support as the excavation proceeded downward.

After different soil treatments were attempted, soil freezing successfully impeded ground water infiltration in the excavation base during excavation.

2 GEOTECHNICAL SCENARIO

The subsurface is made up by 4 class of soil material as shown in *figure 2*

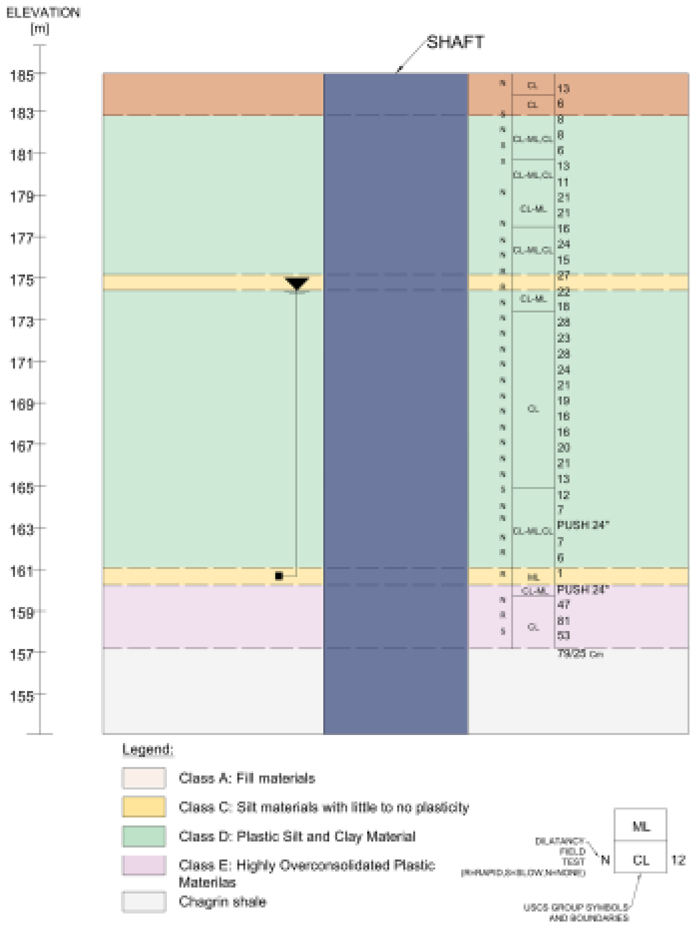


Figure 2. Geological section shaft DST-1 (From Project Geotechnical Baseline Report)

2.1 Class of material:

2.1.1 Fill material- Brown (Class A) found in the first meters. It includes variable material such as topsoil, various backfill and building materials.

2.1.2 Silt Materials – orange- with little to no plasticity (Class C) are comprised of silt, sandy silt, gravelly silt and clayey silt. These materials do not show a plastic behavior and exhibit rapidly dilatant behavior when tested using the field dilatancy test.

This class of material has been subject matter for discussion due to its properties. In fact, a portion of the Class C soil is *Rock Flour silt*, also known as *Bull's Liver*. (fig.3) Because of its smooth texture, this inorganic silt is often mistaken for clay, but may be readily distinguished from clay without laboratory testing. *Rock flour* silt when shaken in the palm of the hand expels water with a dilatant behavior associated. This particular type of inorganic silty soil is the most unstable soil of his category (Terzaghi et all, 1967).

2.1.3 Plastic Silt and Clay material - Green (Class D) are comprised of clay, silty clay and clayey silt. The plasticity index of this class is higher than 4, so they have a non-dilatant behavior. The main part of the soil excavation was executed in Class D soil, Silty Clay and Clay mainly. Fig.2 shows the different soil layer within a standard penetration test number associated.

From the top surface, El. 184 m until El. 165 m, a substantial SPT number gives an indication about the consistency and resistance of the soil. Below El. 165 m, it can be noted that the N Value decreased as the excavation approached the silt layer, where the blow count is 1. Furthermore, Class C is a pressurized layer contain water, so the silty clay just above manifest lower strength.

2.1.4 Highly over-consolidated plastic material, Purple (Class E) encountered during the subsurface exploration are comprised of clay, silty clay and clayey silt. Due to the over-consolidated nature, this class of soil given has a high N value and based on USCS it is classified as *hard* material.

Figure 3. Silty dilatant material behavior Class C soil
a) before shaking b) after shaking

The fine-grained deposit, Class D, Class E and shale formation were considered to be poor producers of groundwater. However, discontinuous silty sand lenses have contained large volumes of water. The Class C silty soil layer between El. 160 m and El. 161 m, contained a pressurized amount of water and so the main source of the bottom instability.

3 TEMPORARY EARTH RETENTION SYSTEM

The DST-1 temporary earth retention system in soft ground is generally designed as a 14.6 m diameter 7 gauge 2-flange steel liner plate with internal W12x72 ribs at 1.3 m spacing. The system is designed so the steel ribs carry the fully soil and hydrostatic loading in compression and the liner plates act as lagging members carrying the load to the steel ribs. Wood blocking is placed between the plates and ribs at approximately 0.5 m to transfer load between the plates and ribs.

Figure 4. DST-1 Tiers

4 CONSTRUCTION AND GROUNDWATER INFILTRATION

As excavation proceeded downward, an external eductor dewatering system was installed around the shaft. In addition, at the same time an internal well point system was installed directly at the bottom of the shaft, aiming to depressurize the silty layer located between El. 160 m and El. 161 m. The water pressure was monitored along the entire working activity by a vibrating wire piezometer installed into the pressurized silty layer. The design and construction plan was to depressurize and lower groundwater in the lower of the silt layers using external eductor wells which would allow excavation to a depth of approximately 25 m (Elevation 160 m) which would be close enough to then install internal vacuum wells and dewater the silt layer.

The dewatering would have allowed excavation of the silt layer and installation of the steel liner plates to occur. As the dewatering systems started, an initial reaction of the piezometer was observed. However, during excavation at a depth of approximately 20 m (Elevation 165 m) uncontrolled groundwater infiltration began at the excavation base. The infiltration also carried silt materials creating a loss of ground concern. The shaft was then flooded to stabilize the excavation. Several attempts were then made to install internal vacuum wells along with an extensive and fairly expensive external grouting program such as Jet-grouting and permeation grouting

However, these methods were not successful in preventing re-occurrences of the groundwater / silt infiltration. Ground freezing of the shaft perimeter to cut off the groundwater infiltration was then decided upon.

5 GROUND FREEZING

Ground freezing is the technique that turns the pore water to ice, resulting in a strong, water-tight barrier. The approach on this project was to drill and install a single row of 114mm freeze pipes at approximately 1.0m spacing. The design of the individual refrigeration pipe is shown in Fig. 5.

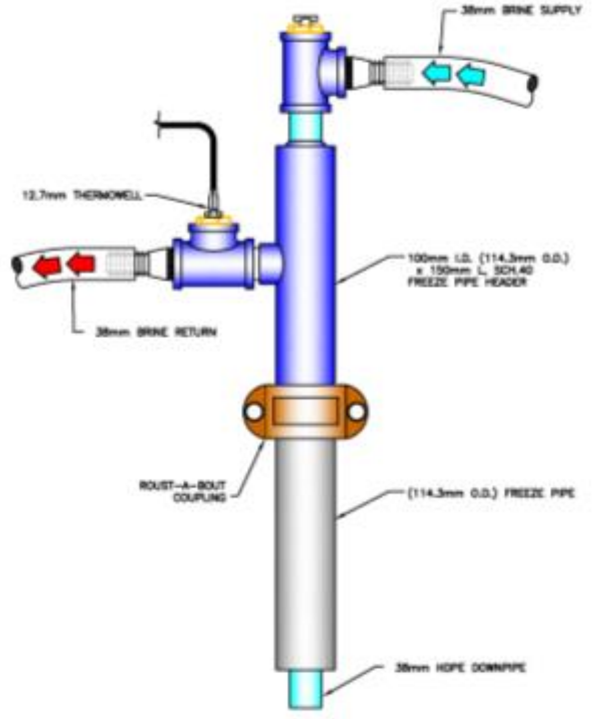


Figure 5. Freeze pipe detail

In order to complete the ground freezing design, that include refrigeration pipe spacing, refrigeration load, and required freezing time, it was necessary to evaluate the thermal properties of the dilatant silt. Prior to the design, additional exploratory borings were completed around the shaft. Relatively undisturbed samples were retrieved and tested on site. A KD-2 meter provided the required values of thermal conductivity and heat capacity.

The samples were then sent to a testing laboratory to evaluate volumetric expansion upon freezing and frozen unconfined compressive strength.

The resulting refrigeration design is shown in Fig. 6.

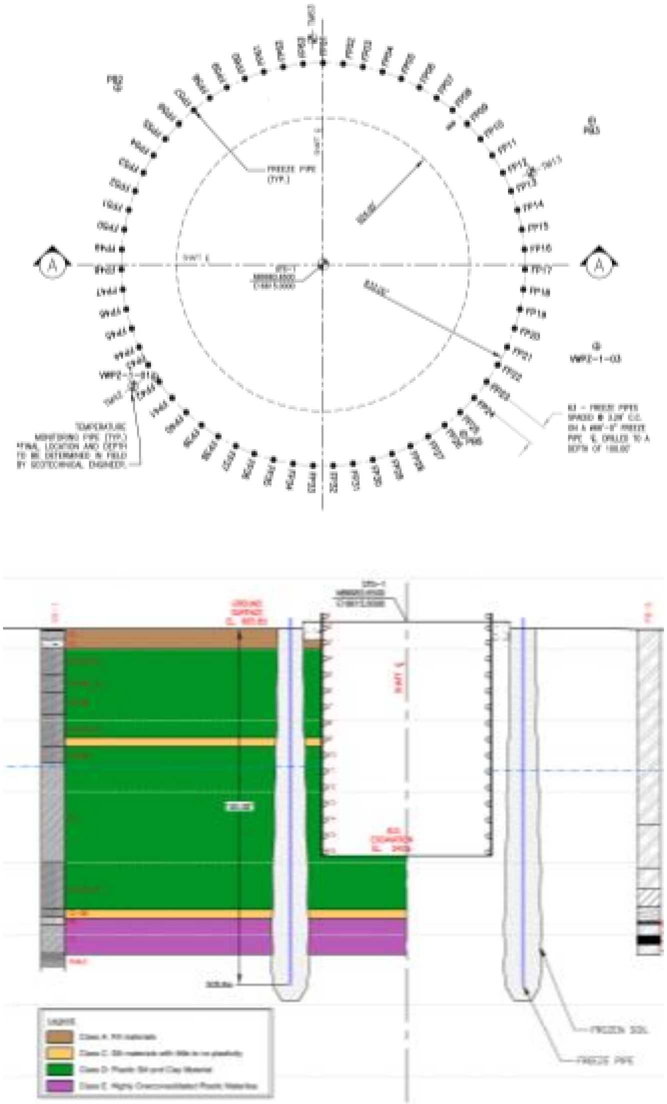


Figure 6. Freeze pipe plan and section

Since one of the performance requirements of the project to not only stabilize the Class C soil, but to also ensure bottom stability, the pipes were installed approximately 3m into the underlying shale.

A time dependent thermal finite element analysis was conducted to evaluate the freezing time and refrigeration requirements. This analysis concluded that freezing would take approximately four weeks and would required 250 kW of refrigeration (not electrical) power.
Freezing was accomplished using two mobile refrigeration plants. Each plant had a 260 kW electric motor. There was redundancy in freezing capacity in the event there was a need for back-up power. The plants utilized ammonia

as the primary refrigerant that chilled a calcium chloride solution to -30 ^{o}C.

Ground temperatures, groundwater levels and refrigeration system performance were measured using a SCADA system. All data was recorded and displayed as shown in Fig.7.

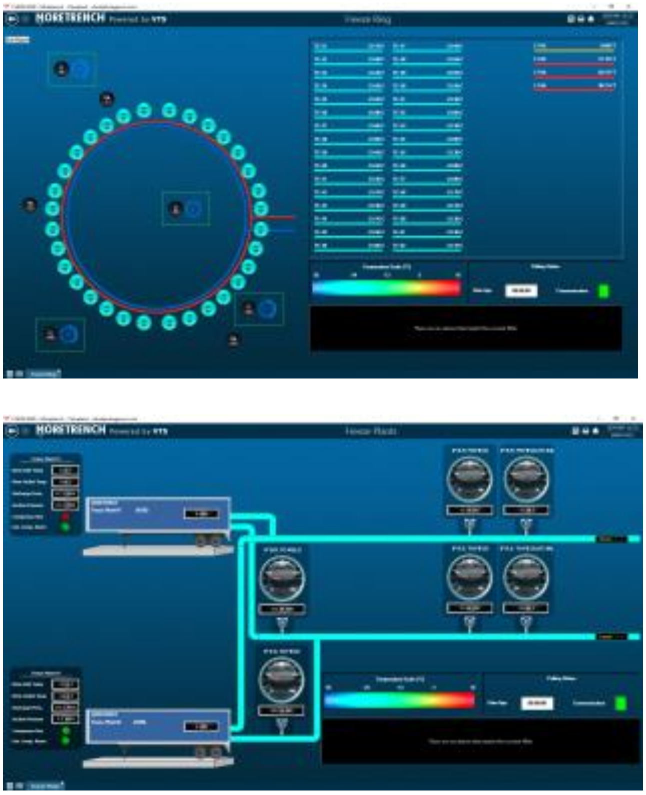


Figure 7. SCADA System

Ground temperature RTDs were installed in temperature monitoring pipes drilled on the exterior of the refrigeration pipes. An individual RTD was placed approximately every 3 meters. Additionally, pressure transducers were installed in the exteriors piezometers and on one internal piezometer. These were also connected to the SCADA system.

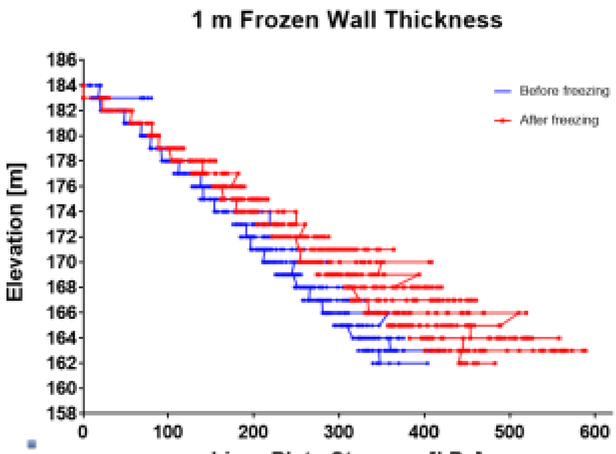
These piezometers were used during the pumping down of the shaft to ensure there was no communication between the internal and external water levels confirming full formation and function of the frozen earth barrier.

Figure 8. Refrigeration Pipe System

6 GROUND FREEZING EFFECTS ON TERS

Ground freezing results in an expansion of the subsurface soils which in turn creates compression on the already installed TERS. The TERS elements are generally designed as active pressure elements using a braced pressure distribution. This design assumes the elements can flex allowing some movement to relive soil pressure. However, since the elements were already installed to a depth of 20 m and soil movement had already occurred, any additional load created by the soil expansion would create additional compression in the TERS system elements. This compression required a re-examination of the expected loading on the shaft elements.

Expected loading was derived by using a Plaxis analysis for different freeze wall thicknesses.



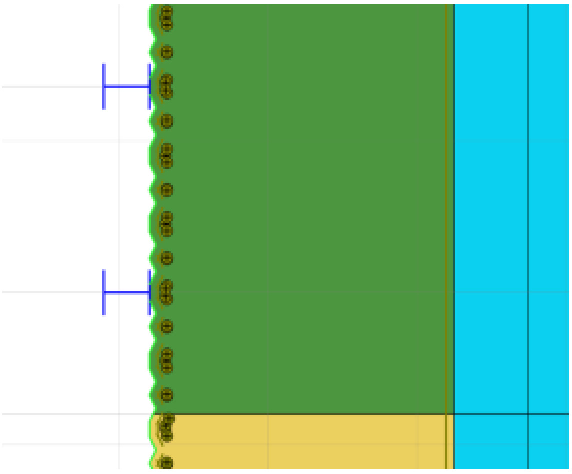


Figure 9. Plaxis freeze model

Based on the Plaxis analysis, we determined that the increase in soil/hydrostatic loading from the ground freezing would be approximately 0.14 MPa (21 psi). When combined with the ribs spacing and shaft diameter and assuming a compressive load of the soil pressure times the shaft radius, this load would result in a significant increase in compressive stress for the steel ribs which were already installed. Based on our estimation of the loading using a braced pressure distribution, we estimated the shaft ribs were already under a rib compressive stress of approximately 82.7 MPa (12 ksi) before the ground freezing operation began.

This combined stress of the induced stress and the in-situ stress would be above standard allowable stresses for the lower elevation steel ribs already installed.

7 FIELD MODIFICATIONS AND INSTRUMENTATION

Based on our analysis, the steel ribs below Elevation 169 m would be loaded beyond their design capacity after the ground freezing was in full effect.

Therefore, to increase their capacity, steel cover plates were installed on the inside (shaft side) flange of the three steel ribs below Elevation 169 m. Also, due to uncertainties involved with the excavation and analysis, installation of strain gauges was recommended to monitor stresses on the steel ribs.

8 INSTALLATION AND MONITORING OF STRAIN GAUGES AND PRESSURE CELLS

Spot welded strain gauges were installed on several of the steel ribs in the excavation. However, since the shaft had been flooded and the shaft could not be dewatered until the ground freezing had become effective, only the steel rib at Elevation 176.5 m (Rib 8) was accessible in the early stages of the ground freezing operation. The strain gauges were installed on May 27, 2016 which was approximately three days after the ground freezing was commenced. The gauges were installed on two opposite locations (stations) and consisted of a set of three gauges at each station. The gauges were installed in a horizontal direction with one gauge on the web face of the internal (shaft) side flange, one gauge across the top web, and one gauge on the web face of the external (soil) side flange.

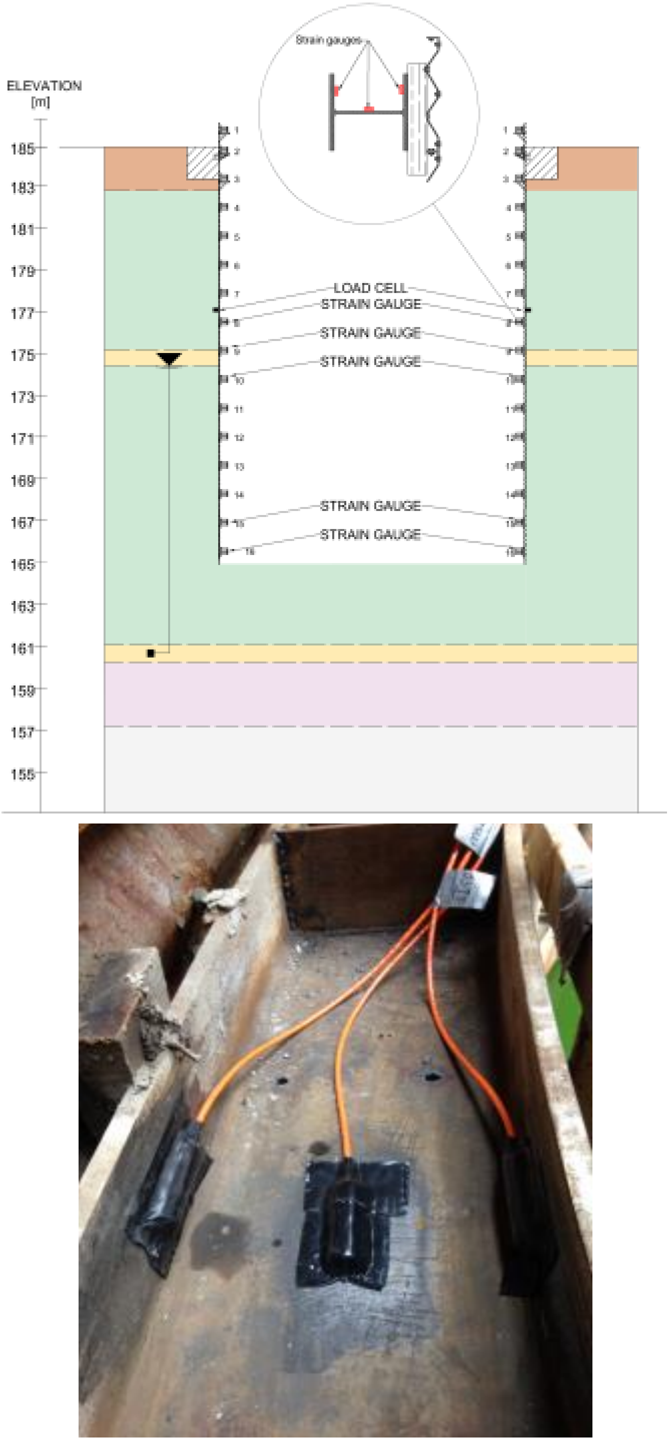


Figure 10. Strain Gage Placement

Two flat circular pressure cells were also installed behind the steel liner plate at the Rib 8 elevation on May 27th. The cells were installed by cutting out the liner plate, installing the cells, then replacing the soil and welding a cover plate. The gap between the plate and cells was packed tight with grout.

After the ground freezing became effective and the shaft could be dewatered, two stations of strain gauges were then installed in a similar manner on Rib 9 (Elevation 175.1 m), Rib 10 (Elevation 173.8 m), Rib 16 (Elevation 165.5 m), Rib 17 (Elevation 164.6 m), and Rib 18 (Elevation 163.7 m). However, since these gauges were installed after the ground freezing had begun, a true baseline to monitor the increase in stress from the ground freezing was not possible.

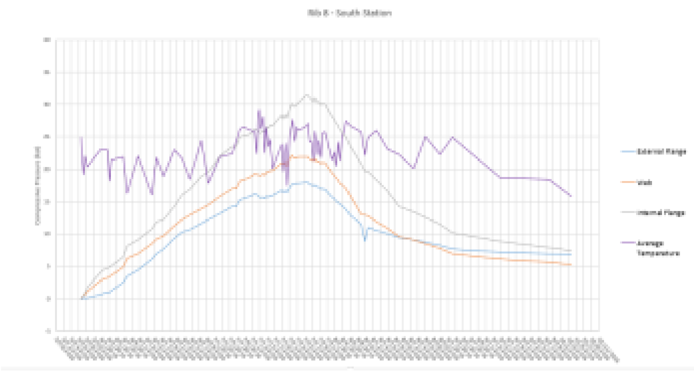
After several weeks of ground freezing, the interior of the shaft was incrementally dewatered and observed for signs of water infiltration. After the ground freezing was deemed effective in cutting off the ground water, the entire shaft was dewatered and excavation resumed to the top of the bedrock. After the excavation reached the bedrock, the ground freezing operation was discontinued.

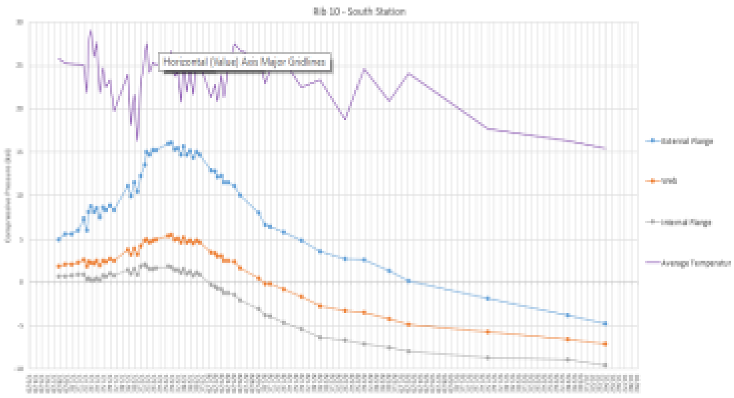
During the dewatering, excavation, and thawing of the ground, the strain gauges were read at regular intervals and presented on graphical forms to the project team.

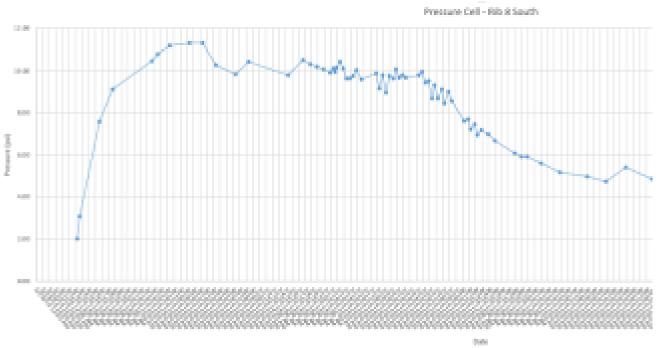
The strain gauges showed combined existing and imposed stresses that exceeded the allowable stress levels and approached the yield stresses of the steel ribs. However, no visible signs of distress were observed in the steel ribs themselves.

The monitoring system did show the increasing stress levels which prompted the construction team to work extended hours and weekends to complete excavation in a timely manner. Ground freezing began in late May 2016 and excavation began in late June 2016.
The excavation was completed and the freezing shut off in August 2016.

Typical readings of the pressure cells and strain gauges are as below:







9 EVALUATION OF STRAIN GAUGE AND PRESSUE GAUGE DATA

The strain gauges revealed an unsymmetrical increase in compressive stress which indicates some moment was developed in the steel ribs. This may be due to an axisymeterical loading or support around the perimeter of the shaft. Induced peak compression of up to 207 MPa (30 ksi) was developed in the first monitored rib (Rib 8). Combined with a pre-existing stress of 82.7 MPa, the total compression load was approaching the yield point of the steel ribs 344 MPa (50 ksi). However, no visible failure of the ribs or the joints was observed. The peak compressive stress loading was approximately 40 percent greater than what was originally expected mostly due to the apparent moment in the rib. The average induced rib compressive stress was on the order of 138 MPa (20 ksi) which was very close to the expected soil loading of 0.14 MPa (20.8 psi).

The pressure cells installed behind the steel liner plate at the Rib 8 elevation had readings as high as 0.08 MPa (12 psi). These measured values are somewhat below what was expected.

Overall, the average measured and evaluated induced soil loading appeared to be on the order of 0.08 to 0.14 MPa (12 to 21 psi). However, as these gauges were installed several days after the ground freeze had begun, we expect that actual loading would be slightly greater than that observed, but still within predicted ranges. Therefore, in conclusion, the predicted inducted stresses are in general agreement with those actually measured in the field. However, we would recommend, some additional allowance be included in any future such designs to allow for an inducted moment imbalance in the steel ribs.

10 CONCLUSION

Due to the soil conditions, this work represented a technical challenge. The excavation of the shaft was integrated with different activities to mitigate the risk of interruption. Ground freezing was finally selected, however there were potential risks of damaging the existing liner plates and reinforcing ribs. Frozen soils laboratory testing and complex analysis was warranted. Field instrumentation data revealed a good accuracy when compared with the analysis performed with the finite element method.
For the excavation of the shaft DST-1, ground freezing was the key to perform successfully the activity, therefore, with the cooperation and coordination between the Contractor Salini-Impregilo Healy JV, the Owner Northeast Ohio Sewer District, the Subcontractors Moretrench and FK Engineering was possible to achieve this important and successful result.

References:

K. Terzaghi et all, 1996. *Soil Mechanics in Engineering practice*

MWH and HMM, 2014. *Dugway Storage Tunnel Geotechnical Baseline Report*

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