INTRODUCTION

The Badami oil pipeline is being constructed by BP Exploration Alaska, and will run from Badami to a tie-in with the Endicott pipeline, a distance of about 40 km parallel to the Northern coastline of Alaska. The oil will then be transported to Prudhoe Bay in the existing Endicott line, for transfer to southern markets via the Trans Alaska pipeline. Early design concepts called for a fully buried line, and some of the design issues with such a pipeline are discussed in this paper. Later, much of the pipeline was changed to an elevated mode for reasons of process constraints and oil rheology. However, future projects that consider a buried pipeline option will be confronted with many of the same aspects discussed in the following.

The Badami project will transport about 5000 m³/day (33,000 bbl/day) of crude oil from the Badami field in a westerly direction to the Endicott pipeline. The route is underlain by thick, continuous cold permafrost, with mean ground temperatures of -10 to -7°C. The surface 2 to 4 m of soils are ice rich silts and silty sands, underlain by denser, thaw stable gravels. The surface silts can be extremely ice rich in many areas, and any thawing of in situ soils or re-worked backfill soils would be accompanied by large settlements. No significant slopes or slope stability issues are present along the route.

The route crosses 3 major and several minor rivers. The smaller of the these rivers have only minor deep seasonal thawed zones associated with their active flood planes. The exception to this is the Sag River, which has a significant talik (unfrozen zone) to a depth of 12 m or more in places.

As the oil flows down the pipeline, it gains or loses heat to the surrounding environment depending on the thermal properties of the oil and the surrounding soil. The temperature to which the pipeline loses or gains heat in the ground varies throughout the year due to large seasonal temperature fluctuations at the pipeline burial depth.

With typical project oil flow rates, the late August temperatures at the outlet point of the pipeline is predicted to be -0.6°C. The late February profile which is the coldest profile predicted for the analysis, predicts an outlet temperature of -12°C.

ABSTRACT

The Badami oil pipeline originally involved a cool buried oil line along its 40 km length. Axial stresses can result from ice wedge cracking. A solution for stresses induced by ice wedge cracking is developed, and results indicate strains are within allowable limits for the pipe section.

The backfill over a cool buried pipe will thaw each summer, resulting in potentially high thaw settlements in the surface icy soils. Thaw settlement tests on excavated frozen soil allowed the development of a “Ditch Settlement Index” (DSI). Amounts of granular backfill to balance anticipated trench backfill settlement could then be estimated.

Concerns for frost heave, and blockage of ground water flow due to chilled pipe operation in an unfrozen river bed were addressed. Frost heave tests indicated that such soils would not be susceptible to significant heave. Further, the convective action of flowing groundwater would cause the frost bulb to seasonally shrink, reducing the impediment to ground water movement.
No thawing was anticipated to take place beneath the pipeline under normal operating conditions. Occasional excursions of the oil inlet temperature above the soil melting point (0°C) could be tolerated during upset conditions, without undue melting or settlement in the underlying icy soils.

In summary, the construction and operation of a buried oil pipeline from Badami to Prudhoe appears feasible from a geotechnical and geothermal perspective, assuming the crude oil properties allow processing and pumping at colder temperatures. The following sections illustrate some of the design issues that were addressed during the development of the buried pipeline concept.

**Ice wedge cracking**

Ice wedge polygons form in permafrost with cold ground temperatures due to thermal contraction in the near surface soils, after a sudden cooling trend. The coefficient of thermal expansion (and contraction) for frozen soil is quite large, in the range of 50 to 200 x 10⁻⁶ °C⁻¹ for silts (Mackay, 1975), and as high as 400 x 10⁻⁶ for clays (Grechishchev, 1972). As a result, large tensile stresses can be induced in the ground near surface. If the tensile stresses exceed the tensile strength of the frozen soil, a narrow crack can open suddenly in the soil to depths of 2-4 m. Various reports (Mackay, 1975 at Garry Island, NWT) indicate that the cracks can be up to 8 mm wide, but more usually 2 to 5 mm wide (Knight, 1971 at Barrow, Alaska) at the permafrost table 0.6 m down. Fukuda (1981) reported average crack widths of 4 mm at Tuktoyaktuk, NWT. Cracks typically open about once per year, on a polygonal spacing in the range of 20 - 60 m. The crack later fills with water which freezes, and after many years this gives rise to the well known wedge shaped ice feature, that can be up to 1.0 m or more across at the top of the wedge.

Because the crack occurs quickly, stresses can be applied to a buried pipe through the tangential adfreeze bond formed between the pipe and the frozen soil at burial depth (Figure 2). An analysis can be developed to calculate the peak tensile stress induced in the buried pipe, and the resulting stress depends largely on the adfreeze bond strength developed between the pipe and the frozen ground. Pipe diameter, wall thickness, type of external coating, pipe/soil temperature and backfill type are other significant factors in the assessment of these stresses. Once these parameters are established by testing and/or literature review, calculations of the likely magnitude of tensile stresses due to ice wedge cracking can be carried out. The effects of ice wedge cracking on a buried pipe are not considered to be cumulative from year to year, due to annual warming in the surface soil layers, and creep relaxation of induced shear stresses with time.
PIPE - SOIL ADFREEZE STRENGTH

A series of laboratory tests were carried out by Shaw Pipe Protection in Calgary to measure the shear adfreeze bond strength between frozen sand/gravel and a coated pipe at -12°C. A cylindrical volume of sand about 11 cm long parallel to the pipe, and about 30 cm diameter was frozen for 1.5 days. A length of 10 cm diameter pipe was embedded in the sand prior to freezing. The pipe was then loaded extremely rapidly causing failure of the frozen bond between sand and coating. Peak stress was reached typically in about 2 seconds, appropriate for field rates of ice wedge cracking. Three different coatings were tested, for a total of 9 tests.

The YJII coating provided the lowest peak shear strength, with a range of 483-621 kPa. The FBE coating gave a somewhat higher range of 580-780 kPa. Finally, the HPCC coating gave a wider range of 435-704 kPa. The average value for each coating varied from 566 to 704 kPa. A larger pipeline diameter would tend to give slightly lower unit adfreeze stresses, according to pile design theory, so therefore the values for the smaller pipe tested would tend to be conservative (high). Yield during the testing took place at the exterior coating-frozen soil contact, and no damage to the pipe-coating bond was noted.

The above testing indicates that a peak shear stress of 690 kPa would be a reasonable value for design. The initial slope of the stress-strain curve was not measured, but is less important in the analysis for effects of ice wedge cracking. Initial estimates of the pre-yield slope of the load-displacement used published values for the dynamic Young’s modulus (13.8 x 106 kPa) to estimate the initial modulus.

ANALYSIS FOR ICE WEDGE CRACKING

Analysis for stresses induced in a buried cold ambient pipeline were carried out using a closed form (exact) solution developed here. The Nixon Geotech PIPSOL pipe structural program (Nixon, 1994) has been applied to the same problem, both to check the closed form solution, and to provide results for more complex situa-
tions involving a non-linear pipe, and the effects of internal pressure.

The pipe used in all cases was a 0.508 m diameter, 11.1 mm wall, Grade 448 MPa (X-65) steel. The soil is assumed to be frozen sand, which has a peak shear strength in the range of 690 kPa in contact with the pipe.

The ice wedge crack problem can be simulated by either (a) applying a displacement to the exposed end of the pipe equal to the crack half-width, as done in the closed form solution, or (b) applying a displacement to the end of the axial soil springs attached to the pipe, as done in the PIPSOL simulations. Either load mechanism applies the same relative pipe-soil displacement, and should result in the same stresses and displacements in the pipe. As illustrated on Figure 2, the problem is symmetrical about the wedge center-line, and only one half of the crack width need to be applied to the exposed end of the pipe.

The closed form solution is limited to a linear pipe, and a soil that has an elastic-perfectly plastic shear stress-displacement relationship. The advantage of the closed form approach is implementation on a spreadsheet, and dozens of cases can be studied in a short time.

A simpler version of the closed form solution can be developed as follows, based on other solutions for compressible elastic piles in elastic soil materials. The solution is given by

\[ u / d = \left\{ P_o / (2AE) \right\} \left\{ \exp \left( \frac{x}{L} \right) + \exp \left( \frac{L-x}{L} \right) \right\} / \left\{ \exp \left( \frac{L}{L} \right) - 1 \right\} \]

where

\[ \beta = \left\{ T_L / (Y_f AE) \right\} ^{0.5} \]

\[ T_L \text{ and } Y_f \text{ are the peak shear load and yield displacement, defined by} \]
\[ T = \left\{ T_L / Y_f \right\} Y_f < Y_f \text{ and } T = T_L, Y_f > Y_f \]
\[ x \text{ is the distance away from crack center-line;} \]
\[ A \text{ is the cross sectional area of pipe;} \]
\[ E \text{ is Young's modulus for pipe material;} \]
\[ P_o \text{ is the axial load applied to pipe at crack to cause displacement;} \]

![Figure 3. Axial pipe displacements and stresses with distance from ice wedge.](image)
L is the total length of pipe considered (sufficiently long that ice wedge shear stresses fall to zero along pipe exterior); and d is the pipe diameter.

This solution can be extended to the more general case of an elastic-plastic soil, and a more complex exact solution can be developed (Murff, 1975). This more complex solution has been solved using a spreadsheet solution.

In order to check the solution against a numerical PIPSOL simulation, a crack half-width of 2.5 mm was selected. Due to the symmetry of the problem, only one-half of the total crack width needs to be applied as the relative pipe-soil axial displacement. The comparison between the closed form and numerical PIPSOL results for axial pipe stresses and displacements are shown on Figure 3. The agreement is very good, indicating the accuracy of both approaches, at least for a linear pipe. The solution also indicates that the relative displacements between the pipe and soil dissipate over a relatively short distance from the ice wedge crack location. Within about 3 m of the crack, the pipe has adapted to the new free field axial displacements. The peak axial pipe stress for this case is about 170 MPa, or about one third of the pipe yield stress.

The PIPSOL program was then used for a series of cases to try to establish the important variables affecting pipe strains. Figure 4 shows the effect of internal pipe pressure on the peak axial strain developed at a crack. The crack half-width was varied from 0 to 5 mm. The case with internal pressure provides the greater pipe strains at the crack location, as internal pressure results in an initial tensile stress in the axial direction due to the Poisson effect. Clearly, using a zero temperature differential (difference between pipe operating temperature and pipe backfilling temperature) will provide the worst case. For a total crack width of around 10 mm, the peak axial strain in the pipe for this base case is about 0.2%. The corresponding axial stresses are given on Figure 5. For crack half-widths above about 2.5 mm, the pipe will be strained inelastically, and the axial tensile stresses will exceed the limit of proportionality.

Higher peak shear strength results in higher strains in the pipe. The peak tensile strain appears to be related approximately to the square root of the peak shear stress at the pipe-soil contact. Finally, the yield displacement in the soil load-displacement function appears to have little effect on the peak axial tensile strain in the pipe, provided that the applied crack width is greater than the yield displacement.
Ditch backfill and settlement

Replacement of native, icy soils in the pipeline trench would result in a significant settlement trough developing along the top of the buried pipe. Although this could be offset by periodic maintenance in the first few years after construction, these effects can be limited from the outset. This can be done by partial replacement of the native backfill in the trench by thaw stable backfill. Mounding of the remaining icy trench spoil over top of the trench will limit the amount of settlement immediately over the ditchline, yet preserves the natural insulating qualities of the fine-grained and organic soils on the ground surface. This would assist in maintaining cooler ground surface temperatures, limit water flow into the ditch, and help in re-vegetation.

In order to estimate the quantity of thaw stable, granular fill along the pipeline, a new test was devised to provide input on the anticipated amount of settlement in the ditch backfill. At the time, it was planned to excavate the pipeline trench with a wheeled ditcher during the winter, producing a relatively homogeneous mixture of frozen chips of soil and ice. Following replacement and subsequent thaw over the pipeline, the frozen, unsaturated backfill mixture would thaw, settle and saturate over time. The net settlement from the frozen, porous condition to the final thawed and consolidated saturated condition could not be estimated from any existing thaw strain correlations for natural soils.

A total of 47 samples were collected during auguring of the frozen soils in the top 1.5 m of the soil profile in the spring 1995 geotechnical field program. Each bulk, augured sample was sufficient to fill a 20 liter plastic pail, and this was shipped to Prudhoe Bay for testing. The samples were tamped gently in the frozen condition, and then allowed to thaw in an uncontrolled manner in a warm building. After completion of thaw, the overall thaw strain was denoted as "DSI", or ditch settlement index.

Figure 6 shows the distribution of the results with distance along the pipeline route. No particular trend with distance was noted. The average for all tests was 56%, with a standard deviation of 12.9%. No strong correlation with geological landform was observed, although fluvial deposits tended to strain less (50%) than surface deposits (56%), or icy thaw lake deposits (62%).

\[ 508 \text{ mm} \times 11.1 \text{ mm WALL, GRADE 443 MPa PIPE} \]
\[ P=9.9 \text{ MPa, DT=0}^\circ\text{C} \]

Figure 5. Effect of peak shear stress on maximum axial tensile strain.
values were then used to calculate the volume that would be required to make up the volume loss associated with thawing of the ditch backfill in the trench, and mounded up over the trench. As shown simplistically on Figure 7, some assumption must be made about the side slopes and shape of the backfill mound, so that the correct amount of granular fill can be placed in the ditch over the pipe. Typically, about 0.6 - 0.7 m of granular fill was required in the trench over the pipe. In the first year or so after construction, some re-working and restoration of the backfill mound would be required along much of the routing, as it is not possible to replace all the backfill mound directly over the excavated trench.

The use of select coarse-grained backfill in the lower part of the pipe trench will reduce the amount of thaw settlement in the trench backfill, and help control the formation of a surface trough along the top of the pipe. However, the presence of coarse, permeable sand backfill around the pipe might also result in an increased potential for subsurface ground water flow along the pipe trench. Low permeability ditch plugs were to be used to limit ground water flow in the trench parallel to the pipeline, in order to limit additional heat input to the soils around the pipe from flowing groundwater. Granular backfill over the pipe would also assist in limiting the possibility of upheaval buckling at low angle overbends (see Nixon and Burgess, 1998).

**Frost heave at Sag River crossing**

The potential for frost heave beneath the buried Badami chilled oil pipeline was assessed. Some of the gravels beneath the active channel at the Sag River are thawed, and a bulb of frozen soil will therefore form around the pipe beneath the river bed. These gravels are clean, and relatively deficient in fine sized soil particles, and are therefore not expected to exhibit significant frost susceptibility.

Samples were collected by D. Miller and Associates of Anchorage and tested in Calgary. In addition, water contents, grain size and laboratory permeability tests were carried out on the 2 bulk samples, in order to classify and further determine properties of the actual samples tested.

The lab frost heave test was carried in a 10 cm diameter cell maintained by the first author. Unique features of the cell include a constant temperature (−1°C) circulating jacket around the cell, that allows tests to be car-
ried out in a warm room. Based on the Konrad-Morgenstern (1983) frost heave approach, the rate of heave can be calculated from 1.09 times the SP parameter multiplied by the temperature gradient at the freezing front. Using this definition, the SP (Segregation Potential) parameter can be extracted (Figure 8). The SP values are very low, in the range of 0.1 to 5 mm²/day°C, when compared with typical values for silts and clays, which are in the range of 80-250 mm²/day°C. This indicates the low frost susceptibility of these materials.

The percentage fines (#200 sieve) is 1.5% and 1.9% for Samples 1 and 2. These values are entirely consistent with the low frost susceptibility confirmed for the frost heave tests samples. Saturated water contents of 12-13% by dry weight were observed for the samples at the end of the tests. Permeability tests gave an average permeability of 2.2 x 10⁻² cm/sec after correction to field temperatures.

Based on pipe structural analysis, the allowable frost heave for the Badami pipeline is about 0.3 m for an appropriate combination of loadings. Frost heave predictions based on the heave test results and geothermal predictions for frost bulb growth (Figure 10), would give rise to a pipe heave of 0.025 m or less. In view of the very small anticipated frost heave in relation to the allowable value, there was no indication that pipe displacements resulting from frost heave at the Sag River (or other crossings on this project) would pose any design concern for the pipeline.

**Interruption to groundwater flow**

A concern for subsurface blockage of ground water flow existed at the Sag River crossing, due to the presence of a significant thawed zone, (talik) within the zone of soil freezing beneath the pipe. If significant ground water flow occurred particularly through the winter, there is a potential for icings in the region of the buried pipe crossing. In order to more correctly predict these effects, and their possible impacts on ground
water flow and frost bulb growth, a more detailed and complex geothermal analysis of the crossing was made. If significant ground water velocities are present (i.e. greater than about 3x10^{-5} cm/sec), then the size of the frost bulb will be reduced, as will the potential for blockage of ground water flow.

The Nixon Geotech THERM2 model with a convective ground water flow option was used to predict the growth of the frost bulb around the buried pipe (Nixon, 1983). As outlined above, the average permeability of the fine gravel samples obtained from the Sag river bed was about 2.2 \times 10^{-2} \text{ cm/sec}. The river bed gradients in this area were estimated to be in the range of 0.005 or so. The product of these two numbers provides an estimated, typical initial ground water flow velocity of around 1x10^{-4} \text{ cm/sec}. This is in the range of flow velocities where convective heat transfer effects become significant.

Figure 9 shows the predicted shape of the frost bulb in late summer and winter, with no convective effects of flowing ground water. The figure also shows the frost bulbs for the same times, with the effects of convective heat flow incorporated. The frost bulb is changed less in late winter, but shrinks to almost nothing in late summer. This is partly due to the warmer pipe temperatures at this time of the year. Finally, Figure 10 shows the seasonal effects of frost bulb growth above and below the pipe. The frost bulb for the case with convection appears to have reached equilibrium after 2-3 years, extending about 1.5 m below pipe base.

Therefore, in areas where a large unfrozen zone exists, the size of the frost bulb should be limited by the flowing groundwater itself. In areas where ground water flows are low or zero, then the frost bulb will be larger. On the other hand, concerns for ground water related issues such as icing formation and blockage of subsurface flow will be lower for these cases of lower ground water flow.

**Figure 8. Segregation potential vs stress for sag river samples # 1 & 2.**

\[ \text{SEGREGATION POTENTIAL (mm2/day, C)} \]

\[ \text{EFFECTIVE STRESS (kPa)} \]

SP IS DEFINED AS HEAVE RATE DIVIDED BY 1.09 x TEMP GRAD

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Figure 9. Maximum and minimum size of frost bulb for groundwater velocity = 0. $1 \times 10^{-4} \text{ cm/sec}$.

Figure 10. Frost depth above and below pipe in artic river.
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References


