REPORT
ANALYSIS OF PIPELINE EXPOSURE DATA AND SCOPING REVIEW OF EXPOSURE SCENARIOS

PETROLEUM TECHNOLOGY ALLIANCE CANADA
PIPELINE ABANDONMENT RESEARCH STEERING COMMITTEE 013

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Report prepared for Petroleum Technology Alliance Canada (PTAC), August 2018.

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EXECUTIVE SUMMARY

This report provides the Petroleum Technology Alliance Canada (PTAC) with initial scoping of several topics within a broader planning level discussion regarding pipeline abandonment and management of long-term risk associated with pipe abandonment in place. The request by PTAC was the 013 work package entitled "Analysis of Pipeline Exposure Data and Scoping Review of Exposure Scenarios." The work involved four topics:

- Interpretation of pipe exposure data to estimate the likelihood of future pipe exposure of abandoned pipelines.

- Description of expected buoyancy control longevity for various practices, and recommendations of future studies to improve predictions of buoyancy control longevity.

- Preparation of a recommended work plan to evaluate low impact pipe removal methods for sensitive ecological areas.

- Preparation of a recommended work plan to measure the potential for pipe exposure due to frost heave.

The following is a brief summary of the study results.

An estimate of exposed pipeline frequency after abandonment was derived from published incident data for operating pipelines in several world regions. About one pipe exposure per 100 years per 1,000 km of pipe should be used for planning purposes after abandonment, assuming intervention or mitigation of long-term hazards at the time of abandonment. In Canada, this would equate to about 8 pipe exposures per year along the 840,000 km of operating transmission, gathering and distribution pipelines. Most of these "incidents" are expected to occur at water crossings. Mountainous areas and erodible soils are expected to have a relatively higher frequency of exposures. Removal of pipe at selected hazard locations may reduce the frequency of pipe exposures. The estimate is qualitative because regulatory agencies do not yet publish the exposed pipe frequency of abandoned pipelines.

Most buoyancy control measures will likely outlive the integrity of the steel wall of abandoned pipelines, unless the pipeline continues to be cathodically protected (in which case, the pipe steel may last longer). The corroded pipe will therefore likely fill with water and eliminate the buoyancy force prior to the degradation of the buoyancy control measure. The buoyancy control measure that may be most at risk from degradation is steel anchors. Anchor degradation over the long term will depend significantly on the initial thickness of the various steel components, on whether they were coated, on local soil corrosivity, on the groundwater level, and on groundwater mineral content. Other buoyancy control measures such as concrete weighting and geotextile bags are expected to last many decades, even after pipe abandonment.

At sensitive ecological locations such as river crossings and wetlands, we describe several removal methods. We assume that pipelines will be removed in locations where rivers are progressively eroding or the alignment is unstable, and where a future degraded pipe may result in a drainage diversion. The proposed next steps include a workshop to more fully explore the environmental considerations and potential consequences of removing abandoned pipes for various site conditions.

This report also describes the potential for frost action to impact abandoned buried pipelines, and concludes that this is highly unlikely. A work plan is described to confirm this conclusion, and a detailed assessment of a previous PTAC report explains the conclusions.

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DEFINITIONS AND ABBREVIATIONS

Abbreviation	Definition
AER	Alberta Energy Regulator
ASME	American Society of Mechanical Engineers
CAPP	Canadian Association of Petroleum Producers
CCC	Continuous Concrete Coating, a concrete layer of specified thickness that encircles the pipe and provides buoyancy control in watercourses, canals, or water-filled pipeline trenches
CCC	Continuous concrete coating
CEPA	Canada Energy Pipeline Association
CGA	Canadian Gas Association
Cold permafrost	Permafrost (rock or soil material that has remained below 0 ºC continuously for two or more years) which has an average yearly temperature below approximately -2 ºC
DNV	Det Norske Veritas
EUB	Energy and Utilities Board
Freeze-back	The refreezing of a thawed active layer in continuous permafrost. Refreezing (freeze-back) occurs from the ground surface downward, and to a lesser degree from the permafrost table upward.
Frost front	The interface between frozen and unfrozen soil which exists when frost penetrates into unfrozen soil. The frost front is typically identified by the 0 °C isotherm within the soil.
Frost heave	Volume change in soil caused by the development of discrete ice lenses in a soil.
Frost jacking	Permanent vertical displacement of a structure buried in soil caused by frost heave adjacent to the structure over one or more freeze-thaw cycles.
Geohazard	Natural hazards, sometimes referred to as “ground movement” hazards, associated with geotechnical, hydrotechnical, tectonic, snow and ice, and geochemical processes that can … threaten the integrity of … infrastructure or impact the environment (Porter et al. 2016).
GSW	Geotextile “Wrap” Swamp Weights, a geotextile fabric draped over the pipeline and up the trench walls and backfilled with mineral soil to a minimum specified thickness to provide buoyancy control
HDD	Horizontal directionally drilled
Incident	A pipeline leak, rupture, or if an object strikes (hits) the pipeline, even if the hit does not cause a loss of product (AER 2013).
Lagging	Wood boards placed between river weights and usually banded with metal strips
NEB	National Energy Board
NPS	Nominal Pipe Size
PARSC	Pipeline Abandonment Research Steering Committee
PCA	Portland Cement Association
Phi Angle	Assumed internal friction angle; and for the purposes of this chapter - mainly related to organics or organic/mineral soil mixes

Abbreviation	Definition
PipeSak™ Weight*	Prefabricated granular filled bags with lower “baffles” that are set on the pipeline to provide buoyancy control. PipeSak is a registered trademark of the PipeSak Inc. company of Ontario
Post-glacial rebound	The rise of land masses after the lifting of the huge weight of ice sheets during the last glacial period, which had caused isostatic depression (i.e., the sinking of the earth’s crust due to the heavy weight of ice).
PTAC	Petroleum Technology Alliance Canada
PW	Plate Weight, a solid high-density thin plastic “hat” placed over the pipeline, which extends laterally out from the pipeline, and is backfilled with mineral soil to a minimum thickness to provide buoyancy control
RoW	Right-of-way
RW	River Weight or “bolt-on” encirclement type concrete weight that provides buoyancy control in trenches with “deep” standing water
SA	Screw Anchor, steel rods typically with a single helix (but can have two or more helices) embedded in competent mineral soil and attached to a saddle draped over the pipe that provides buoyancy control
Soil Density	The unit weight of soil in kg/m^3 (total or wet unit weight unless otherwise specified)
SP	Segregation potential
Springline	The line extending across the diameter of a pipeline from the 9 o’clock to 3 o’clock positions on the pipe when viewed in cross-section.
SW	Swamp Weight or saddle weight or “set on” type concrete weight that provides buoyancy control in drier trenches.
TAPS	TransAlaskan Pipeline System
Terrain Type	A representation of the surface and near surface soil types on the basis of geological origin, landform, and texture.
Upfreezing surface	The upwardly freezing of the top of the permafrost table (that is, at the bottom of the active layer) during refreezing of the active layer as winter approaches in in continuous permafrost environments.
UV	Ultraviolet
WT	Pipeline Wall Thickness

1 INTRODUCTION

The following report highlights key pipe abandonment topics as requested by the Petroleum Technology Alliance Canada (PTAC) Steering Committee for Pipeline Abandonment Research (PARSC). This report is intended to comply with the PARSC 013 scope of work for the "*Analysis of Pipeline Exposure Data and Scoping Review of Exposure Scenarios*."

Pipeline abandonment refers to the "permanent removal from service of a pipeline." This may involve pipeline sections being abandoned in-place or removed, with the choice dependent on multiple factors. One factor to consider with in-place pipeline abandonment is the possible future exposure of pipeline segments and the consequences of exposure, such as interference with land use.

Overall, PTAC is developing a risk management framework for abandonment of pipelines. There are several key data gaps to be addressed before the framework can be completed. This study was intended to address some of those gaps.

The work was focused on the following topics:

- Chapter 2 - Interpretation of pipe exposure data to estimate the likelihood of future pipe exposure of abandoned pipelines.
- Chapter 3 - Description of expected buoyancy control longevity for various practices, and recommendations of future studies to improve predictions of buoyancy control longevity.
- Chapter 4 - Preparation of a recommended work plan to evaluate low impact pipe removal methods for sensitive ecological areas.
- Chapter 5 - Preparation of a recommended work plan to measure the potential for pipe exposure due to frost heave.

We further understand that this information will be utilized by others as part of a recommended risk management framework for abandoned pipes. The following report chapters are organized by the selected four topics.

The report was developed based on published literature, plus interpretation of the available information by experts in the topics to be addressed.

2 ANALYSIS OF PIPELINE EXPOSURE OCCURANCES

2.1 Background

Pipeline incidents are reported by regulatory agencies around the world for operating pipelines, and are often summarized by industry literature such as conference papers. The statistics for incidents are normally presented by age of the pipe, the size of the pipe, the media flowing through the pipe, or by the mechanism of failure, including damage caused by geohazards. The incidents may involve a pipeline exposure, and pipeline exposures can be counted as incidents, but many incidents may not involve a pipeline exposure.

> Incident = a pipeline leak, rupture, or if an object strikes (hits) the pipeline, even if the hit does not cause a loss of product (AER 2013) that results in a significant adverse effect on the environment, a death or serious injury, an unintended fire or explosion or unintended release of low pressure hydrocarbon more than 1500 litres or unintended release of high pressure hydrocarbon gas (NEB)

Regulatory agencies generally do not report statistics for exposed pipelines, and do not report statistics for exposed pipelines that are abandoned in place. However, pipelines regulated by the Canada National Energy Board (NEB) (i.e., pipelines that cross provincial and international boundaries) are now required to report exposed pipelines (as of April 1, 2018). Previously, the reporting requirement was limited to conducting any excavation work along the pipeline (NEB 21 day notification). A notification would be reported if an exposed pipe is being remediated, although not all exposed pipes require remediation. Some pipes are allowed to operate exposed if it is determined that they can continue to operate safely.

In the absence of direct historical records for abandoned pipelines or for operating pipelines that are exposed, the following description is an interpretation of available literature to estimate the relative potential for abandoned pipelines to be exposed. It is based on the published literature plus interpretation according to the professional experience of the authors. Literature is plentiful on the topic of pipeline asset integrity, and on the management of pipeline geohazards. There are simply too many knowledgeable and experienced authors to reference them all. A few key reports stand out in terms of the industry data and descriptions that they provide.

The key reports used in this study are:

- Report 2013-B: Pipeline Performance in Alberta, 1990-2012 (AER 2013).
- Gas Pipeline Incidents: 9th Report of the European Gas Pipeline Incident Data Group; period 1970-2013 (EGIG 2015).

- Oil Pipeline Characteristics and Risk Factors: Illustrations from the Decade of Construction (Kiefner 2001).
- Integrating Terrain and Geohazard Knowledge into the Pipeline Lifecycle (Porter 2014).
- Updated Estimates of Frequencies of Pipeline Failures Caused by Geohazards (Porter 2016).
- Pipeline Geo-Environmental Design and Geohazard Management (Rizkalla 2008).
- The Assessment and Management of Pipeline Geohazards (Rizkalla 2007).
- Terrain, Ground Conditions and Geohazards: Evaluation and Implications for Pipelines (Hengesh et al. 2005).
- Pipeline Abandonment – a Discussion Paper on Technical and Environmental Issues (NEB 1996).
- NEB website safety performance dashboard http://www.neb-one.gc.ca/sftnvrnmnt/sft/dshbrd/dshbrd-eng.html.
- US Department of Transportation, Pipeline and Hazardous Materials Safety Administration website Http://phmsa.dot.gov/pipeline/library/data-stats/pipelineincidenttrends.

2.2 Pipe Incident Data

Incidents along operating pipelines may not involve an exposure or a leak, based on incident reporting requirements that vary amongst regulatory jurisdictions. However, about 90% of incidents involve leaks (AER 2013). The incident rate varies from year to year, and it varies by cause. Most incidents are due to pipe material issues such as corrosion, welding, over-pressures, valve/fittings, operator error, or damage by others. The statistics for these causes are important for the management of operating pipelines, but not as relevant for the management of abandoned pipelines with no flowing media. For example, long-term corrosion and deterioration of an abandoned pipeline could be expected and acceptable in many cases.

> Geohazard, sometimes referred to as "ground movement" hazards = natural hazards associated with geotechnical, hydrotechnical, tectonic, snow and ice, and geochemical processes that can … threaten the integrity of … infrastructure or impact the environment (Porter et al 2016).

Incidents that might cause the exposure of an abandoned pipeline are more likely due to geohazards, including earth movement. Most agencies report incident statistics resulting from geohazards. For example, 1.7% of all incidents in Alberta involve earth movement (AER 2013). Similarly, 8% of incidents in Europe are due to geohazards (EGIG 2015). The following discussion provides a basis for interpreting the potential exposure risk of abandoned pipelines using geohazard statistics.

The reported incident rates for pipelines due to geohazards are:

- 2.3 average geohazard incident rate based on recent data, in terms of incidents per 1,000 km of pipeline per 100 year timespan, equivalent to 2.3×10^{-5} per km per year
    - 3.1 geohazard incidents in Alberta (1990-2012) (AER 2013)
    - 1.2 for US onshore gas transmission (2005-2014) (Porter 2016)
    - 2.7 for US onshore hazardous liquids (2005-2014) (Porter 2016)
    - 2.6 for gas transmission in Europe (1970-2013) (EGIG 2015)
    - 2.6 for NEB pipelines in Canada (2009-2018) (www.neb-one.gc.ca safety dashboard, assuming natural force damage incidents excluding operations beyond design limits)
- 0.4 average geohazard incident rate based on data available 10 years ago (Rizkalla 2008)
    - 0.45 for the US (1984-2001)
    - 0.16 for gas pipelines in Alberta (1980-1997)
    - 0.32 for oil pipelines in Alberta (1980-1997)
    - 0.42 for gas pipelines in Canada (1984-2003)
    - 0.54 for oil pipelines in Canada (1984-2003)

The geohazard incident rates are reported above in terms of incidents per 1,000 km of pipeline over a lifespan of 100 years, for numerical convenience and to help describe the context for long-term abandonment.

The average geohazard incident rate for the most recent data in North America and Europe is 2.3, based on a total of 1.3 million km of pipelines and incident statistics for NEB regulated pipelines in Canada (2009 to 2018), all pipelines in Alberta (1990 to 2012), US (2005 to 2014), and Europe (1970 to 2013). The average is about 6x the previously reported rates derived from data 15 to 20 years older (Rizkalla 2008). The disparity between newer and older data on geohazard incidents may be due to several factors, including: higher incidents for older pipelines, and improved reporting of incidents.

The higher incident rates for older pipelines have been documented to exhibit 4x the incident rate compared to newer pipelines (Kiefner 2001). However, the higher failure rates of older pipelines are primarily due to engineering practices that no longer exist. Early pipelines in the 1920s developed "defective pipe seam" because seams between pipe segments were not as strong as the pipe steel. Another innovation by the 1950s involved coatings and cathodic protection to reduce the failure rate due to external corrosion. Newer pipelines with improved connections and cathodic protection have lower incident rates in comparison to older pipelines. Therefore, many of the common factors for incidents along operating pipelines are not expected to affect future abandoned pipelines, such as: age of pipe, application of cathodic protection, coatings, etc.

In addition to improved engineering practices, abandoned pipes do not require the same pipe integrity needed to avoid leaks along an operating pipeline. This would effectively reduce the incident rate for pipelines that are abandoned in place. Other influences on the incident rate of abandoned pipelines are the geohazards that could result in a pipe exposure. Some geohazards have the potential to increase the incident rate over time – especially if the geohazards have a longer return period frequency compared to the operating life of the pipe.

We could conservatively estimate that 100% of the 2.3 geohazard incidents (over 100 years along 1,000 km of pipeline) will result in an exposed pipe along an abandoned pipeline. This conservative estimate includes almost 20 years of additional data compared to a previous average of 0.4 (Rizkalla 2008). Some of this difference may be due to changes in reporting, such as some incidents that are now reported as a geohazard incident but may have been previously reported as a corrosion issue – whereby ground movement resulted in damage to the corrosion protection. The difference may also be due in part to the frequency of geohazards, some of which require a longer time horizon to be observed. For example, southern Alberta experienced a large flood in 2013 that exposed a number of pipelines in rivers. The flood magnitude was about 100 year return period. Prior to that, the largest flood magnitude in Calgary over the 80 years prior to 2013 was equivalent to a 10 year return period event.

Regardless of the differences in available data or reporting measures, the conservative assumption above will likely over-estimate pipe exposures for abandoned pipe as many geohazard incidents do not involve a pipe exposure. For example, NEB data from 2009 to 2018 reports that "Natural Force Damage" (i.e., geohazard) incidents resulted in adverse environmental effects just 10% of the time from 2009 to 2018.

2.3 Geohazards

There are a variety of geohazards that might affect a pipeline. Geohazards in general are often grouped into tectonic, geotechnical, hydrotechnical, and geochemical hazards. A representative list of pipeline geohazards is presented in Table 1, based on several sources (Hengesh et al. 2005, Rizkalla 2007, and Porter 2016). Many of the geohazards do not require pipe exposure to stress and damage an operating pipeline. For example, the shear stress from slope creep could rupture a pipeline without ever exposing the pipe. These kinds of hazards are often mitigated as part of the pipe design.

The majority of geohazards along pipelines tend to be hydrotechnical. BGC, a consultancy in Canada, reports that 85% of all pipeline geohazards that they have documented are hydrotechnical, with an average of 1 hazard every 4 km (Porter 2014). Of these, hydrotechnical hazards are more numerous in mountainous areas – 2x more numerous than prairies (Porter 2014). In Europe, the majority of geohazard incidents occur in the Alps Mountains – at 40x the incident rate compared to the rest of Europe (EGIG 2015). However, reporting agencies do not differentiate geohazards by categories such as geotechnical or hydrotechnical – likely because the root cause may not always be clear. Many landslides (geotechnical hazard) along valley walls are initiated by undercutting of the toe by river bank erosion (a hydrotechnical hazard).

Table 1 Inventory of Potential Geohazards

Geohazards	Description
Tectonic	
Earthquake fault displacement	Movement along existing faults → Shear displacement/loading of pipe
Earthquake dynamic liquefaction	Sudden loss of strength and/or movement of soil subjected to dynamic loading → Lateral spreading of soil on RoW, pipe uplift (buoyancy), or pipe settlement leading to flexural strain and/or exposure
Seismic ground motion	Ground shaking due to seismic loading → Dynamic loading of pipe
Ash falls	Accumulation of ash → Change in burial depth resulting in increased load on pipe, obstructed access to pipeline
Lahars	Mudflow comprising pyroclastic material and water → Change in burial depth resulting in increased or decreased load on pipe, obstructed access to pipeline
Pyroclastic flow	Accumulation and flow of pyroclastic material → Change in burial depth resulting in increased or decreased load on pipe, possible pipe exposure, obstructed access to pipeline
Geotechnical	
Landslide, deep-seated (longitudinal or cross-slope)	Deep rotational of complex failure of steep longitudinal or cross slopes → Rapid loading and deformation of pipe, exposure of pipe, or deeper burial of pipe; disturbance of ditch and RoW; pipe vulnerability depends on loading direction
Slope creep	Gradual downslope mass movement of soil on frozen or unfrozen slopes → Gradual loading and deformation of pipe (longitudinally or laterally)
Slope creep rupture	Sudden rupture of warm permafrost on slope following gradual downslope mass movement → Gradual loading and deformation of pipe leading to rapid loading and deformation (longitudinally or laterally)
Rock fall or avalanche	Downslope movement of either individual blocks or a disaggregated mass of rock → Sudden vertical loading of pipe and increased burial depth; possible obstruction of RoW and ditch; possible exposure to pipe
Metastable soil (loess)	Open textured silty soils which collapse on wetting, structural loading or seismic loading
Geochemical	
Karst sinkhole collapse	Attach on pipe by acid groundwater → Localized metal loss and possible reduction in structural capacity
Acid rock drainage attack on pipe	Ground subsidence due to sinkhole development (collapse of active karst or paleokarst solution cavities) below pipe → Potential for pipe exposure and unsupported pipe span depending on size and depth of collapse feature
Saline soil/bedrock attack on pipe	Attack on pipe by saline groundwater → Localized metal loss and possible reduction in structural capacity

Geohazards	Description
Hydrotechnical	
Coastal inundation, flooding, and shoreline erosion	Change in sea level in low-lying coastal areas or water level of inland waterbodies → Inundation of ROW, buoyancy effects on pipeline, possible erosion due to wave action (tsunami)
Gullying or other soil erosion by water	Removal of soil by water action across and adjacent to the pipeline. Existing gullies prone to enlargement by erosion and scour of banks and headwall.
River progressive degradation (vertical scour)	Vertical hydraulic erosion of material over pipe in existing channel → Reduction of cover or pipe exposure possible unsupported pipe span with vortex shedding
River bank erosion	Migration of channel (lateral scour) in flood plain causing erosion of soil over pipe beyond the deep burial portion of the pipeline → Reduction of cover or pipe exposure; possible unsupported span leading to pipe strain aggravated by debris caught on pipe; possible toe erosion of valley slopes
River local scour	Local scour is the erosion occurring over a region of limited extent due to local flow conditions, such as may be caused by the presence of hydraulic structures such as bridge piers or abutments or culvert outlets.
River avulsion across floodplain	Relocation of a river within a floodplain, resulting in pipe exposure at a location with relatively shallow burial.
River avulsion across alluvial fan	Relocation of a river within an alluvial fan, resulting in pipe exposure at a location with relatively shallow burial.
Debris flow	Downslope movement of water-saturated debris → Sudden vertical loading of pipe and increased burial depth; possible obstruction of ROW and ditch; possible exposure of pipe
Drainage diversion along pipe due to backfill erosion	Hydraulic transport of ditch backfill and associated loss of restraint on pipe → Pipe flexure/exposure due to upheaval displacement of pipe; exacerbated by soil warming/strength loss
Drainage diversion along ROW due to backfill erosion or settlement	Hydraulic transport of soil particles along ROW → Loss of soil cover on ROW; possible stream siltation
Drainage diversion along pipe due to subsurface piping erosion	Hydraulic transport of subsurface material through groundwater flow → Loss of support beneath pipe, possible void development, possible stream siltation
Groundwater flow disruption by buried pipe crossing	Subsurface impoundment or slowing of groundwater flow through a wetland or alluvial aquifer due to a buried pipe.
Drainage through degraded pipe	Diversion of water through a degraded pipe (abandoned pipe)
Buoyant uplift in water bodies	Low soil mass with elevated groundwater table or failure of mechanical buoyancy control leading to pipe uplift → Potential exposure of pipe; pipe flexure due to upheaval displacement in unrestrained pipe section

Geohazards	Description
Intermittent wetlands	Some wetlands may be intermittent during extended dry climate cycles. There is a potential for pipe exposure if the pipe was never buried (i.e., the pipe is laying on the bed of the wetland)
Rapid lake drainage (e.g. beaver dam impoundment)	Breach in lake impoundment causing rapid drainage of water and associated erosion along drainage path across ROW → Potential exposure of pipe and/or an unsupported span; vortex shedding aggravated by debris caught on pipe in extreme case
Additional less common hazards	
Thawed layer detachment	Shallow slope instability involving downslope movement of thawed soil layer above permafrost → Reduced cover and exposure of pipe; possible longitudinal or lateral loading on slopes with deep thawed layer
Snow avalanche	Downslope movement of either individual snow, ice and debris → Sudden vertical loading of pipe and increased burial depth; possible obstruction of ROW and ditch; possible exposure of pipe
Frost heave (pipe)	Pipe uplift from frost bulb development in unfrozen spans → Pipe strain caused by flexure/uplift resistance of adjacent frozen soil
Frost heave (ditch)	Surface heave over ditch due to ground freezing → Disruption of surface drainage across ROW due to elevated ditch line
Frost bulb (on land)	Reduction of soil hydraulic conductivity due to ground freezing → Disruption of subsurface drainage beneath and above pipeline; possible increased pore pressure and aufeis on side-slopes with active groundwater systems
Frost bulb (at water crossing)	Reduction of soil hydraulic conductivity and ground heave due to ground freezing; constriction of water flow path → Disruption of watercourse flow and fish habitat or fish migration; intervention during operation potentially disruptive to watercourse
Frost blister	Ground heave due to increased hydraulic pressure beneath frozen layer from subsurface drainage disruption on cross slopes → Elevated pore pressure and possible erosion associated with release of trapped water or melting of ice upslope of pipeline
Ice wedge cracking	Sudden extensional cracking of ice wedges in frozen ground → Generation of localized extensional displacement/tensile stress in pipe, and associated shear stress near location of ice wedge cracking
Thaw settlement (pipe)	Differential settlement of the pipe due to thawing of permafrost → Pipe strain caused by pipe flexure in areas of differential settlement in frozen ground, or at frozen/unfrozen interfaces
Thaw settlement (ROW)	Subsidence along ROW due to thawing of permafrost → Disruption of surface drainage across ROW due to ground subsidence
Thaw bulb	Rapid thawing of ice-rich permafrost on slopes → Pore pressure generation and loss of soil strength leading to thermal erosion; possible void formation beneath pipe; contributing factor to slope movement
Thermokarsting	Thawing of massive ice or ice wedges leading to surface subsidence and possible ponding/sustained thawing → Disruption of surface drainage across ROW due to subsidence, and possible buoyancy issues

Geohazards	Description
Boulder / cobble / rock indentation	Interaction of pipe with cobble/boulders or shallow rock at bottom of ditch → Point loading/denting of pipe
Static liquefaction	Collapse of sensitive (flocculated) soil structure due to surface loading or groundwater effects → Disruption to right-of-way; potential for pipe exposure
Sensitive and residual soil	Disturbance to sensitive or residual soils during pipeline or access construction → Rapid loss of soil strength leading to flow or ground instability on ROW possibly inducing strain in pipe due to lateral loading, deeper burial, or loss of support
Dune migration	Wind transported desert soil → Change in burial depth resulting in increased or decreased load on pipe, possible pipe exposure, possible deeper burial
Flash flooding/scour at wadis	Water transported desert soil → Change in burial depth resulting in decreased load on pipe, possible pipe exposure

2.4 Geohazards along Abandoned Pipelines in Canada

Of the many geohazards in Canada, hydrotechnical hazards are likely candidates for exposing abandoned pipelines and causing negative consequences. For example, a landslide (geotechnical hazard) may result in negative consequences but finding an abandoned pipe among the debris probably does not result in a greater severity of the disaster. An exposure of a pipe in a river, however, may result in a navigation issue along a larger river or a fish passage issue along a smaller river. Several of the hydrotechnical hazards may be more relevant to abandoned pipes compared to operating pipes, and at least one hydrotechnical hazard is only related to abandoned (empty) pipes: “Drainage diversion into a degraded pipe.” Table 2 describes the various hydrotechnical hazards and highlights the hazards that may be more prominent in the future exposure of pipelines that are abandoned in place.

Table 2 Hydrotechnical Hazards in Canada

Geohazard (hydrotechnical)	Locations	Key factors	Future frequency of events	Event	Potential consequences of abandoned pipe exposure
Coastal inundation, flooding, and shoreline erosion	Coastal areas susceptible to hurricanes or volcanic eruption or tectonic activity	Susceptibility to coastal disasters (e.g. hurricanes)	**Subject to climate change, progressively higher frequency with sea level rise**	Exposure on land or in rivers	Reclamation cost
Gullying	Hill slopes, agricultural areas with gradual slopes	Erodible soils, land use changes	Depends on future development	Exposure on land	Reclamation cost
River progressive degradation (vertical bed scour)	Post-glacial valleys	Erodible, immature landscapes with land use changes	**Progressively higher**	Exposure in river	Fish passage, navigation
River bank erosion	Rivers prone to migration	Geomorphology of river	Similar frequency compared to current conditions along an operating pipeline	Exposure in river	Fish passage, navigation
River avulsion across floodplain	High sinuosity, flat floodplain, or high sediment load	Geomorphology of river	Similar	Exposure in river	Fish passage, navigation
River avulsion across alluvial fan	Mountainous channel confluence in larger valley	Location of pipe within the fan	Similar	Exposure in river	Temporary exposure of pipe
Drainage diversion along pipe or ROW	Gradually sloping ROW across or along drainage	Backfill competency, alignment of flow vs pipe, ROW gradient	Similar	Erosion, dewatering of creek	Loss of habitat, local flooding
Drainage diversion into degraded pipe	Gradually sloping pipe across drainage	Depth of valley, pipe alignment	**Progressively higher**	Dewatering of creek	Loss of habitat, local flooding
Buoyant uplift in water	Wetlands, large rivers, lakes	Lifespan of buoyancy control measures, pipe	Depends on material lifespans	Exposure in river, wetland	Fish passage or navigation along rivers
Groundwater flow disruption by buried pipe crossing	Wetland channels with poorly defined valley	Size of pipe, alluvial aquifer	Event occurred in the past	Partial dewatering of downstream alluvial aquifer	Temporary impact on wetland integrity

Geohazard (hydrotechnical)	Locations	Key factors	Future frequency of events	Event	Potential consequences of abandoned pipe exposure
River local scour	Near infrastructure such as road, rail	New infrastructure (e.g. road crossings)	Depends on future development	Exposure in river	Temporary exposure of pipe
Debris flow	Mountainous channel confluence in larger valley	Location of pipe within valley or alluvial fan susceptible to debris flow	Similar	Exposure in river	Reclamation cost
Intermittent wetlands	Wetlands with poorly defined drainage outlet	Wetland water level may drop during dry climate cycle	Subject to climate	Exposure in water body	Exposed pipe
Rapid lake drainage (e.g. beaver dam impoundments)	Flat topography, beaver-prone areas with steeper downhill gradients	Large upstream beaver ponds with erodible steep downstream gradient	Similar	Exposure in river	Exposed pipe

Some of the hydrotechnical hazards may result in exposure of an abandoned pipe. In most cases, the frequency and consequences are similar to an operating pipeline because the hazards are external to the pipeline and do not change over time. Two (2) of these hazards have a higher probability potential in the future:

- River progressive degradation (vertical bed scour) occurs along rivers in erodible soils, especially in regions with relatively immature landscapes. The profile of most rivers is relatively stable because the sediment regime is relatively balanced (i.e., incoming sediment = outgoing sediment), but some rivers are highly erodible and the incoming sediment volume is not sufficient to replace the eroded material. For example, the badlands of southern Alberta have highly erodible soils and weak bedrock layers that progressively erode channels. Many of the rivers south of Grande Prairie Alberta are also located along erodible soils in an immature landscape that was dominated recently by glaciers. Progressive degradation may result in several metres of erosion over a span of decades for small streams, or over centuries for large rivers. The consequences may be fish passage issues along creeks where the exposed pipe acts as a small dam until flow is established under the pipe. In a large river, an exposed pipe may create navigation issues for boaters.

- Drainage diversions into degraded pipes may occur over the long term as pipes begin to corrode and decay. Sloping pipes may accumulate local groundwater or drainage such that flow is diverted and re-emerges at the topographic low. In some cases, this may result in diversion of a creek with no surface flow (or habitat) for the sloping distance. This hazard does not exist along large rivers where the topographic low for the pipe is below the river bed. The hazard will tend to occur along escarpments where a local drainage or a creek flows across or along the sloping pipe. The potential consequences are local loss of habitat or connectivity of aquatic habitat, and potential downstream flooding where the water re-emerges from the pipe. Depending on the location, the loss of habitat may go undetected.

Overall, hydrotechnical hazards that do not otherwise result in a temporary consequence may need to be mitigated or monitored in the event of long-term risk of unacceptable consequences, including:

- River progressive degradation (vertical bed scour) throughout landscapes that are relatively immature, and with erodible soils, in locations where fish passage or river navigation have the potential to be impacted.

- River bank erosion at pipeline crossings where the river is prone to migration and the pipe setback distance is small (pipe crossings using horizontal direction drilling techniques tend to have large setback distances), at locations where fish passage or river navigation have the potential to be impacted.

- River avulsion across floodplains, equivalent to the entire river moving to a new alignment within the floodplain, and resulting in similar consequences to river bank erosion in locations where the pipe becomes exposed.

- Drainage diversions by eroding the ROW or backfill around the pipe, in locations where the pipe is sloping down from the water crossing or where the ROW is nearly parallel to the creek; such that the diversion may result in a loss of habitat or local flooding.

- Drainage diversions into a degraded pipe (as described previously).

- Buoyant uplift in water along wetlands, large rivers, or lakes; depending on the expected lifespan of the pipe's buoyancy control measures and the environmental consequences of removing the pipeline prior to abandonment.

- Coastal inundation, flooding, and shoreline erosion in a location that is susceptible to hurricanes or future sea level rise; depending on the potential consequences of blocking navigation channels and the consequences of disturbing the pipeline near levees or dykes.

In the future, these hydrotechnical hazards have the potential to increase or decrease the risk of exposure depending on climate changes. However, most of the hazards will tend to exist with or without climate change. The one exception is the coastal flooding and shoreline erosion hazard, which may be exacerbated by sea level rise, increasing frequency of hurricanes, or additional post-glacial rebound around northern lakes.

Some of the geohazard issues may be mitigated as part of the abandonment process. For example, drainage diversions into a degraded pipe could be mitigated with a plug insert into the pipe to prevent flow inside the pipe down a slope away from the established water body.

Other hazards that have the potential for temporary effects may need to be assessed on a case-by-case basis.

2.5 Conclusions

This chapter describes the relative frequency and mode of geohazard incidents along pipelines in Canada. Altogether, the future exposure rate of abandoned pipes is likely over-estimated by the use of average geohazard incidents for operating pipelines. The more likely exposure rate is less than this average, for the following reasons:

- Incident rates are influenced by old engineering practices that will not be relevant for abandoned pipes.

- An incident along an operating pipeline may not qualify as an incident along an abandoned pipeline.

- Not all geohazard incidents result in a pipe exposure.

- Newer technologies such as horizontal directional drilling across river valleys will likely further reduce the incident rate for hydrotechnical hazards by using deep burial to eliminate the hazard.

For purposes of long-term planning, the estimated exposure rate for abandoned pipelines is about one exposure per 100 years per 1,000 km of pipeline. This is a qualitative interpretation of the available literature, assuming that a priority list of hazards are mitigated prior to abandonment or monitored over the long-term. The estimated exposure rate for abandoned pipelines might therefore result in 8 exposures per year in Canada, based on 840,000 km of operating transmission, gathering and distribution pipelines (Natural Resources Canada, www.nrcan.gc.ca/energy/infrastructure/18856). The actual rate of exposure for pipelines that are abandoned in place will vary across the country depending on regional differences, and will depend on various choices by pipeline operators at the time of abandonment. For example, a pipeline operator may choose to remove the pipe in high hazard locations – thereby reducing the future frequency of abandoned pipe exposures.

Some of the remaining abandonment hazards are not prominent relative to operating pipelines because there would no longer be consequences such as leakage from an operating pipeline (i.e., incidents involving pipeline leakage would no longer exist). Therefore, the geohazard assessment for pipeline abandonment may need to focus differently and consider different rules for triggering mitigation actions during abandonment.

The remaining chapters of this report discuss some of the key topics for establishing abandonment rules:

- Buoyancy control longevity – to help answer the question of whether buoyancy control has an operating lifetime that is longer or shorter than integrity of the pipe, and therefore result in additional pipe exposures.
- Pipe removal methods for sensitive ecological locations – to help answer the question of whether additional disturbance to remove some pipelines is practical or desirable.
- Frost heave – to address questions regarding frost action and its potential to result in pipe exposure.

3 LONGEVITY OF BUOYANCY CONTROL MEASURES

3.1 Introduction

This chapter presents information on pipeline exposure risks from the potential long-term degradation of various buoyancy control measures that have been installed along onshore operating pipelines. No consideration has been given to offshore pipelines.

The following methodology was used to address issues related to long-term integrity of buoyancy control:

- conduct a literature review of various historical and currently used buoyancy control measures
- identify potential issues that may affect the long-term integrity of the identified buoyancy control measures
- identify data gaps associated with the issue of the long-term integrity of various buoyancy control measures, and
- summarize the results

This section is organized as follows:

- background
- literature review:
    - buoyancy control methods
    - buoyancy control design considerations
    - possible factors affecting buoyancy control longevity (failure modes)
- buoyancy failure risk assessment, and
- recommended future studies

3.2 Use of Buoyancy Control along Pipelines

Pipelines can be found under several types of water bodies including streams, lakes, muskeg, irrigation canals, or inland lakes (possibly placed using marine lay methods). Many of these pipelines were reasonably-assumed to be abandoned in-place – after due consideration by way of Certificate approval (DNV 2010).

Pipelines are subject to natural positive buoyancy when placed below the water table. The magnitude of the buoyant effect is proportional to the pipeline diameter. The need to mitigate pipe buoyancy depends on many factors. These include the pipeline material, the pipe diameter and wall thickness, the pipe product, the installation timing, the pipeline burial depth, and the density and type of the trench backfill material. Even during operation, pipelines carrying a heavy product may still require the use of buoyancy control measures as they may be subject to excessive buoyancy force during the period between pipe installation and full operation or during maintenance events.

Abandonment conditions may differ from an operating pipeline. For example, thermal expansion forces that may lead to overbend instability and pipe buckling are not a concern for an abandoned pipeline, but a reduction in cover of the installed pipeline may exist at overbends due to thermal expansion forces that occurred during pipeline operation. This may reduce the forces acting against pipeline buoyancy.

Buoyancy control philosophy has not been consistent over the past several decades. The philosophy may have ranged from field personnel placing buoyancy control only in areas suspected to be at risk, typically low-lying wetlands (swamps) and watercourse crossings, to conservatively placing buoyancy control along a high percentage of the overall pipeline length, to a more rigorous engineering design desktop study with a subsequent detailed field investigation. However, there are only a few standard options available, although there are now a few more options available, even if these options are not in widespread use.

3.3 Buoyancy Control Methods

The following descriptions of buoyancy control methods focus on those that are more typical, but a few other less well-known methods are also presented. The less well known and rarely used methods are acceptable buoyancy control measures and may have been used by some companies under certain circumstances. The buoyancy design values are presented to provide a basis for understanding both the low risk of exposure for small diameter pipelines, and the typical safety factors used for various buoyancy control measures.

Buoyancy control measures include:

- Concrete weights:
  - saddle weights, also known as set-on or swamp weights
  - river weights
  - continuous coating
  - rock shield wrap
- Soil weights:
  - deeper trench
  - bag weights
  - slot weight
  - geotextile wrap weights
  - plate weights

- Anchors:
    - screw anchors
    - flip anchors
    - grouted anchors

A majority of these buoyancy control measures are well known to the pipeline industry. However, a few may not be so well known. A summary of where the various buoyancy control methods would typically be used is provided in Table 3.

The most common and best-known weights are various concrete weights. A description of each method follows. Soil weights use either native soil or imported soil to provide additional mass over the pipe to counteract buoyancy. These weight types are used almost exclusively for overland buoyancy control as opposed to watercourse crossings.

Anchors differ in that they are designed to not rely solely on mass above the pipeline to counteract buoyancy force. Anchors utilize the strength and mass of the underlying soil or bedrock to resist pipe uplift. Anchors typically consist of two metal rods screwed, drilled, or otherwise placed into the subsurface soil, one on either side of the pipeline. The metal rods extend up to approximately the mid-point of the pipeline. A saddle or strap is then placed over the pipeline and attached to both rods. The pull-out resistance of the anchors provides resistance to the buoyant force of the pipeline. Mechanical flip anchors are a little different in that a metal cable is used instead of a metal rod.

Anchors can be installed in most soil conditions, but the anchor type will vary with the subsurface conditions. Anchor types differ in one way by the method of installing the steel rod or plate, and by how the rod or plate is held in-place. Although there are several anchor types, the screw anchor is the main anchor type used on Canadian pipelines.

Table 3 Buoyancy Control Method use for Typical Various Field Situations

	Terrain Condition							
	Shallow Organics		Deep Organics				Wet Organic Areas (Fens) or Minor Water Crossings	Major Water Crossings
Buoyancy control method	Unfrozen ground or warm permafrost	Cold frozen ground	Unfrozen ground or warm permafrost	Over cold frozen ground	Over shallow bedrock	Over granular soils		
Saddle Weights	XX	X	XX		X	X		
River Weights	X	X	X	X	X	X	XX	
Continuous Concrete Coating			X	X	X	X	X	XX
Imported Fill	X	X						
Deeper Ditch	XX	X						
Geotextile Bag Weights	X	XX	XX	X	X	X	X^2	
Geotextile Wrap Weights	X							
Plate Weights[1]	X							
Screw or Mechanical[2] Anchors	X		XX					
Grouted Anchors[1]			X		XX	X		
Freeze-Back Anchors[2]		X		XX	X			

[1] Very limited use as a research project.
[2] Not known to have been used to date on a Canadian pipeline.
XX – Typical primary usage, but dependent on local economics, local terrain, and lengths of buoyancy control required.
X^1 - if cold permafrost
X^2 - if pre-attached

3.3.1 Saddle Weights (Swamp Weights or Set-on Weights)

Concrete saddle weights, also known as swamp weights or set-on weights (see Figure 1), have been used for many decades. They were a common weighting option for drier organic terrain but have been replaced to some extent in the last few years by screw anchors or by bag weights due to the greater stability of bag weights. Swamp weights typically are used in areas where the pipeline can be welded beside the trench and lowered-in using conventional land lay methods, or in terrain where the trench either does not fill up with water or can be sufficiently dewatered. If the trench cannot be pumped dry or nearly dry, river weights, or continuous concrete coating, or pre-attached bag weights are used. Saddle weights are lowered onto the pipeline after it has been lowered into the trench.

Figure 1 Concrete Saddle or Set-on or Swamp Weights (photo courtesy of T. Bossenberry)

3.3.2 River Weights

Cast iron bolt-on weights are briefly referred to in a PipeSak® brochure titled “Natural Hazards.” The timing and extent of their use is uncertain as it is generally understood that they have not been used since at least the late 1980s.

Concrete river weights (RW), sometimes called bolt-on weights (see Figure 2), are typically used in smaller watercourses, or in organic terrain that cannot be sufficiently dewatered. A wood lagging is often placed between the weights to maintain proper weight spacing during installation. RW typically are preferred over continuous concrete coating for smaller pipe diameters or shorter weighted lengths due to the reduced cost. River weight halves are placed on each side of the pipeline and then bolted together prior to the pipeline being lowered into the trench.

Figure 2 Concrete River (bolt-on) Weight (photo courtesy of J. Harris)

3.3.3 Continuous Coating

Continuous concrete coating (CCC) as shown on Figure 3, is commonly used on larger watercourse crossings or small lakes where it is easier to drag the pipeline into place versus carrying it into place. CCC is also used in long organic areas or low weight backfill areas where rapid water inflow into the pipeline trench cannot be effectively removed. CCC also may be used in areas where the backfill soils are subject to liquefaction (assuming, from a strain relief viewpoint, that it is not preferable to allow the pipe to come to the surface.)

For either RW or CCC, they are typically used to sink the pipeline to the bottom of the excavated trench. Once the trench has been properly backfilled, the native mineral soil usually provides sufficient mass to counteract the pipe's buoyancy. The weighting is essentially redundant unless there is extreme scour over a longer distance.

Figure 3 Continuous Concrete Coating (photo courtesy of T. Bossenberry)

3.3.4 Rock Shield Wrap

Rock shield wrap is similar to CCC in that it is a continuous "coating" placed around the pipeline. The difference is that the wrap is pre-manufactured and brought to site. There are various types of rock shield wrap but the type referred to here is a gunnite wrap as opposed to a plastic type wrap. The gunnite provides both mass and protection from external impact. For smaller diameter pipelines, the thin wrap can provide sufficient mass to counteract low buoyancy forces.

Additional resources are available at: http://www.rockguard.biz/rockguard-hd.html

3.3.5 Deeper Trench

Using a deeper trench does not involve an imported manufactured item or fill material being placed on the pipe; it is simply using a greater thickness of native mineral soil backfill over the pipe. Therefore, its long-term effectiveness relies on the backfill remaining in place.

3.3.6 Bag Weights

Bag weights are now common on many pipelines. A photograph of a bag weight shown on Figure 4. The first PipeSak® bag weight was installed in 1994 (Connors 2013. Pers. Comm.).

Additional resources are available at:

- Soil filled "bag" weights: https://www.keymay.com/products/geotextile-pipeline-weights/

- Soil filled “slot weight”®: http://www.bkwinc.com/images/16_april_900.jpg (see also July 2015 newsletter)

Figure 4 Soil (granular) Filled Bag Weights (photo courtesy of G. Connors, PipeSak®)

3.3.7 Slot Weight®

The slot weight® is made with polyurethane and uses nylon straps to create a “slot” on either side of a pipe to hold mineral soil. It is not known if this weight type has been used in Canada. The manufacturer of this weight type is located in the United States (http://www.bkwinc.com/images/16_april_900.jpg)

3.3.8 Geotextile Wrap Weights

Geotextile wrap weights were first used in the early 1990s but are not commonly used. They consist of a geotextile fabric being draped over the pipeline and a mineral soil backfill being placed on the geotextile and then the geotextile is wrapped around the backfill as shown on Figure 5).

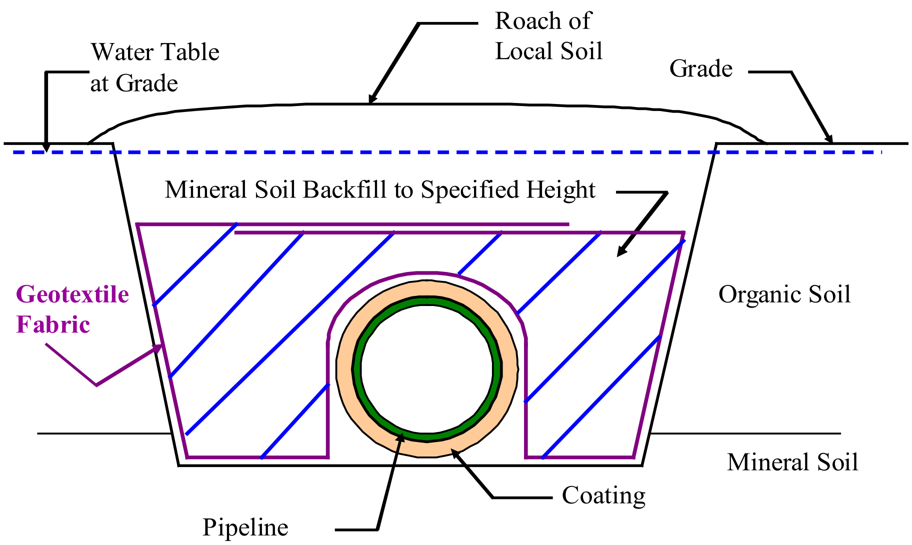


Figure 5 Soil Filled Geotextile "Wrap Weight" Schematic

3.3.9 Plate Weights

Plate weights, like geotextile wrap weights, are not a common weight type. This weight type was a research and development weight tested by a Canadian company on two different pipe diameters in the early 1990s. Plate weights consist of a thin, high-density plastic plate backfilled with mineral soil as shown on Figure 6. The plastic plates are made using high-pressure injection and a pre-made stainless steel mould. They are placed over the lowered-in pipeline and then backfilled with mineral soil. Similar to deeper trench or a geotextile wrap weight, they use the mass of the native local mineral soil to counteract buoyancy forces. As with the deeper trench method, its long-term effectiveness relies on the backfill remaining in place.

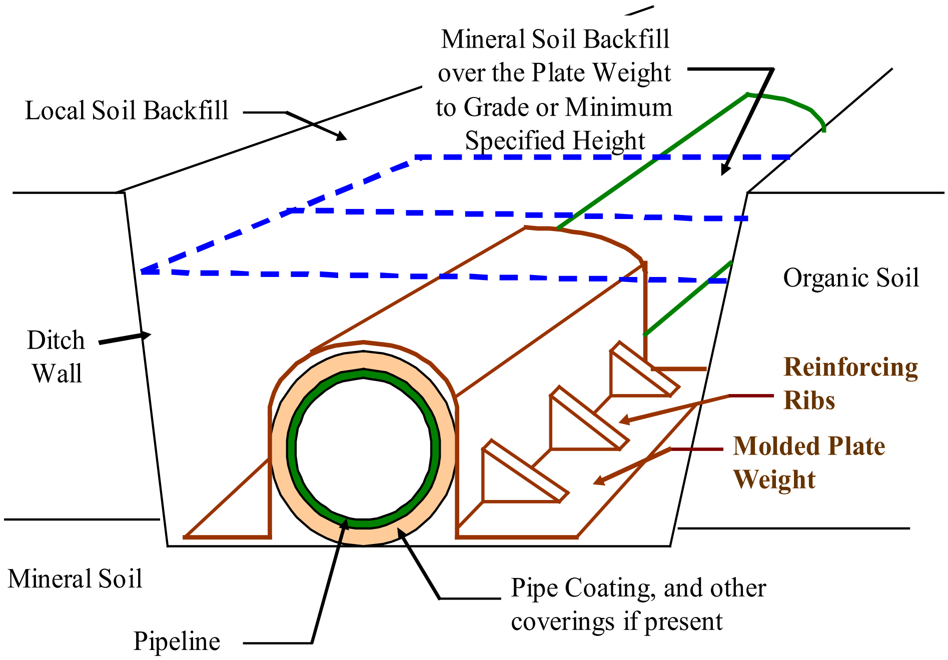


Figure 6 **Molded Plastic Plate Weight Schematic**

3.3.10 Screw Anchors

A screw anchor consists of a leading steel rod with single or multiple helices welded onto the rod (see Figure 4). Helices are available in a range of diameters. There may be one or more extensions attached to the leading steel rod; the number required depends on the depth to competent soil and the buoyant force to be restrained. Figure 7 through Figure 10 show screw anchors and various saddles and connections.

Figure 7 Power Driven Steel Screw Anchor Lead Section (typically only a single helix, see also the connecting bracket at top left; photo courtesy of A. Brown, AHB Consulting)

Figure 8 Screw Anchor Extensions (1.8 m long) (photo courtesy of A. Brown, AHB Consulting)

Figure 9 Example (1) Steel Screw Anchor and Polyester Strap Saddle and Connection (photo courtesy of A. Brown, AHB Consulting)

Figure 10 Example (2) Steel Screw Anchor and Polyester Strap Saddle and Connection (photo courtesy of A. Brown, AHB Consulting)

Figure 11 Mechanical "Flip" Anchors & Cable (Platipus anchor: http://www.platipus-anchors.ca; photo courtesy of J. Keaton, Wood PLC)

3.3.11 Mechanical Flip Anchors

Mechanical flip anchors (see Figure 11) are driven into the soil using one of various equipment types, depending on the anchor size, and depth and load requirements. Once driven to the required depth, the anchors are flipped into position using a cable attached to the anchor. The bearing surface area and the soil strength provide the uplift resistance. It is uncertain if this anchor type has been used on a Canadian pipeline.

3.3.12 Grouted Anchors

Grouted anchors differ from screw anchors in that they do not use a lower helix, only straight rods. The steel rods are placed into undisturbed competent soil below the upper organic or soft soils (see Figure 12). The rods can be placed into a pre-drilled hole, either cased or uncased, or they can be driven directly into the ground. The rods are then grouted into place using a prescribed minimum volume of grout combined with observations on grout return. When placed into granular soils or bedrock, the anchors can provide excellent load carrying capacity. This anchor type was a research and development item tested by a Canadian company and it is unlikely that they have been used outside of the single test site.

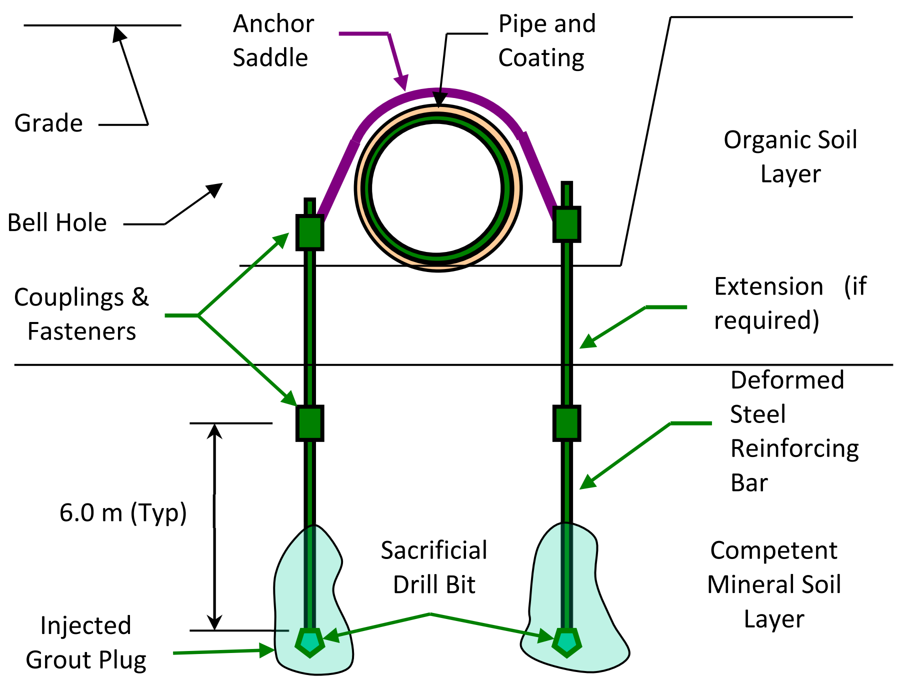


Figure 12 Grouted Steel Anchors Schematic

3.3.13 Freeze-Back Anchor

Freeze-back anchors, or sometimes referred to as slurry anchors, are only suitable in cold permafrost terrain. They are analogous to the freeze-back piles used in northern building construction. Similar to the grouted anchor, steel rods, with or without an end plate, are placed into a pre-drilled hole (see Figure 13). The hole is then backfilled with a sand and water slurry that subsequently freezes. The adfreeze bonds between the ground and slurry and the slurry and steel rod provide the uplift resistance. This anchor type was a research and development item tested by a Canadian company and it is unlikely that they have been used outside of two northern Canada test sites.

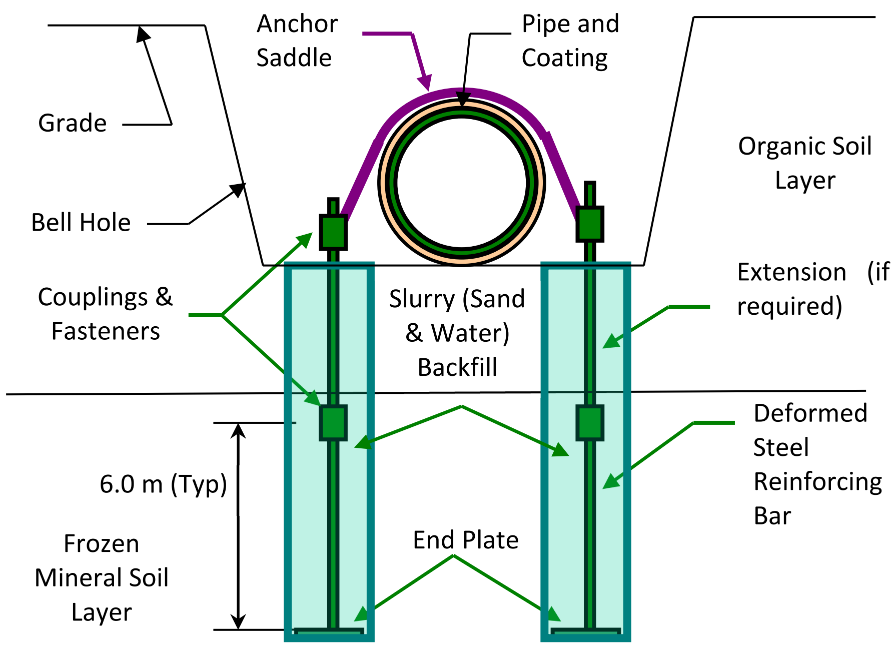


Figure 13 "Freeze-back" Steel Anchor Schematic (for use in continuous permafrost)

3.4 Buoyancy Control Design Considerations

The various inputs into buoyancy control calculations can affect the probability of pipe exposure due to failure of a particular buoyancy control measure. Natural mineral soil and organic soil density varies widely as do their friction angles. There are even wider variations to consider when noting that the values used must be representative of the highly disturbed condition of the backfill placed into an excavated pipeline trench. The selected soil density for design is subject to local conditions, judgement, degree of conservatism in all of the buoyancy factors, safety factor, plus the designer's or owner's level of acceptable risk. The benefit of using higher soil densities or friction angles in buoyancy calculations must be assessed against the realistic potential for measured verification during construction. The added cost of some additional buoyancy control is usually much less than post-construction costs to remediate shallow cover or a pipe exposure.

3.4.1 Mineral Soil Density

Soil wet density can range from nominally 1,600 to 2,200 kg/m^3. Ideally, the actual unit weight for the site-specific replaced trench material would be used in buoyancy calculations. However, the site-specific in-situ density is difficult to determine accurately. It is necessary to adopt an appropriate soil density assumption for design purposes. In the absence of multiple direct field measurements of soil density, a reasonable but conservatively low density usually is adopted.

A mineral soil density of 1,650 kg/m^3 is assumed by some in industry. Simmonds and Thomas, 1998, used a density of 1,650 kg/m^3, while Couperthwaite and Marshall, 1987, quote 1,600 kg/m^3.

These values differ by only 3%. Soil densities of 1,600 to 1,650 kg/m^3 are low compared to typical undisturbed bulk soil densities, and as such could be considered to be conservative. However, as stated previously the backfill soil is less dense than undisturbed soil, for at least two to three years after construction. Furthermore, in cases where the organic layer is 300 mm thickness or more, a lower soil density provides an allowance for a reduced amount of mineral soil A low soil density assumption also provides an allowance for some loss of soil to the roach over the pipeline assumption or an improperly centered roach.

The selected mineral soil density has two main effects:

- it will affect the minimum thickness of mineral soil required above the pipe before buoyancy control is needed
- the required spacing between soil weights, where the in-situ soil provides the resistance to buoyancy

The presence of mineral soil above the pipeline where other weighting methods are used, such as concrete or anchors, provides an additional safety factor against pipeline uplift. This safety factor may in some cases counteract the failure of some buoyancy control measures.

3.4.2 Backfill Shear Resistance

The internal friction angle (shear strength) of mineral or organic soil also provides additional resistance to pipe buoyancy (Simmonds and Thomas 1998). Undisturbed shear strength values for various soils are available in many soil textbooks. However, trench backfill is highly disturbed soil and may contain numerous voids, therefore lower shear strength values should be used in initial design. Another alternative, employed by many pipeline companies and consultants, is to ignore soil shear strength entirely for buoyancy calculations in overland areas, essentially providing zero buoyancy resistance. Assuming zero shear strength of backfill is another inherent safety factor (conservatism), particularly for small diameter pipelines. For large diameter pipelines, the soil shear strength provides little additional safety factor for buoyancy resistance.

For pipelines up to about NPS 24, only a relatively thin layer of mineral soil is needed for a pipeline cover of 0.7 m to counteract pipeline buoyancy of an empty pipeline. For small pipes, a high percentage of the buoyancy control measures could fail and the pipe would still remain buried. This is illustrated on Figure 14, which shows the trigger for requiring buoyancy control based on the portion of organic soil (i.e., muskeg) for a pipe with 0.7 m cover. All small pipes up to NPS 16 would not require buoyancy control even if all of the cover was muskeg/organic soil. At NPS 20 (and a wall thickness of 8.6 mm), only about 40 mm of mineral soil is needed to counteract the negative buoyancy. The largest pipes would require buoyancy control regardless of the portions of mineral and organic soil. Figure 14 therefore describes the maximum amount of muskeg/organic soil that can be allowed before buoyancy control is required on a pipe with 0.7 m cover.

The data shown on Figure 14 is based on a backfill soil friction angle of 15 degrees. Simonds and Thomas (1998) noted in their paper that soil friction angles measured in the laboratory ranged from 28 degrees to 62 degrees. Thus, the Figure 14 data is considered to be conservative for a pipeline that has been in service for several decades and the backfill has had many years to consolidate.

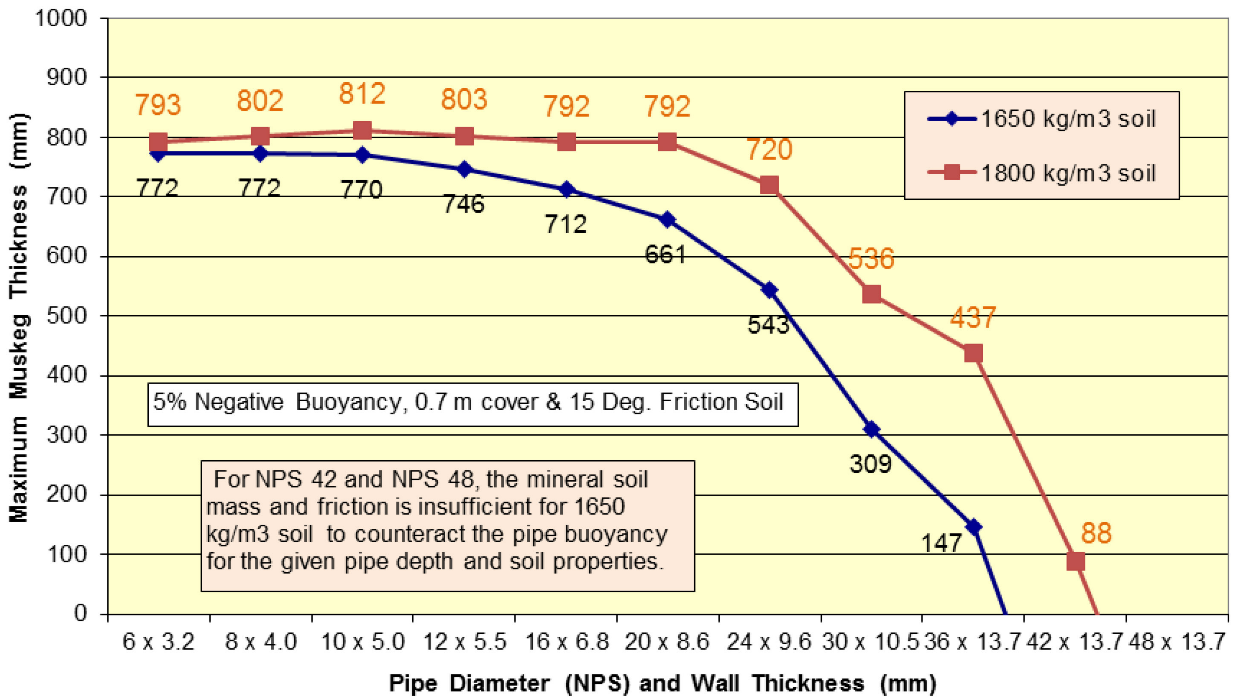


Figure 14 Maximum Muskeg Thickness for Various Pipeline Diameters and Two Mineral Soil Densities

3.4.3 Density of Water

The densities of ditch water and other components are also required to calculate pipe buoyancy. Common design values for these items are:

- Use a density of fresh water of 1,000 kg/m^3 at watercourses. If the water is brackish, a density proportionate to the salt content should be used.
- Use a density of water in muskeg areas of 1,040 kg/m^3 with the water table assumed to be at the ground surface. The higher density of muskeg water relative to fresh water accounts for suspended sediment. Again, if the water is brackish, an increased density proportionate to the salt content above the 1,040 kg/m^3 should be used.

Some designers may use higher water densities than these when calculating the buoyancy force. This practice increases the conservatism of the analysis and the safety factor of the design.

3.4.4 Safety Factor

The buoyancy safety factor is the ratio of the downward (negative) buoyancy force on the pipe to the upward (positive) buoyancy force on the pipe. For pipelines crossing watercourses, typical safety factors are small, nominally 1.1 to 1.2 (i.e., the resultant downward acting forces on the pipe are 10% to 20 % greater than the upward forces). At watercourses where significant water velocity exists, higher safety factors may be used to facilitate lowering-in of the pipeline.

For overland pipeline buoyancy control, a typical safety factor is 1.05. These relatively low safety factors take into account the conservatism in the input parameters. However, some owners may use a higher safety factor.

3.4.5 Soil Liquefaction

During seismic events such as earthquakes, some soils can become liquefied, essentially losing all shear strength and becoming a liquid for a short period of time until high pore water pressure dissipates and the soil regains its strength. When liquefied, the soil no longer provides a downward negative buoyancy force on the pipeline, but instead provides an upward positive buoyant force. In these cases, a density that represents the density of the liquefied soil must be used in the buoyancy calculations to ensure the pipeline does not float. Liquefied soil greatly increases the buoyant force relative to that from water. For abandoned pipelines that have not been appropriately weighted, soil liquefaction may cause the pipeline to float to the surface. Where liquefaction is possible, the pipeline may need to be removed or filled with a material denser than the liquefied soil to prevent a pipeline exposure.

3.4.6 Anchor Spacing

Weight spacing calculations for buoyancy control can be found in a variety of publications including American Society of Mechanical Engineers (ASME; Rizkalla 2008).

Anchor spacing calculations are not, unlike mass type buoyancy control, dependent on backfill soil density and backfill shear strength. The three main criteria for anchor spacing are limiting pipeline wall stress, limiting anchor load to that suitable for the anchor type and soil conditions, and limiting pipe uplift between anchors. The typical factors considered in the spacing calculations include:

- longitudinal bending stress
- longitudinal membrane stress
- combined hoop and longitudinal stress
- combined stresses for restrained spans
- longitudinal stress due to sustained loading
- contact stresses at the anchor hold down strap on the pipeline
- maximum allowable uplift between anchors (often owner company specified)
- anchor type, and

- potential underlying soil types (to a degree)

Anchor spacing calculations often consider various pipe stresses and combination of those stresses and this may, and likely does, reduce the anchor spacing from that strictly required to counteract the pipeline buoyancy. For example, a NPS 36 pipe with a wall thickness of 12.0 mm may have had anchors installed at approximately 30 m spacing because of stress considerations at maximum operating pressure. However, based strictly on buoyancy forces, the anchors could have been spaced at 37 m before anchor loads exceeded allowable design levels. This results in an additional safety factor of 23% for an abandoned pipeline without operating pressures. In other words, roughly every 4th anchor could fail post-abandonment and the pipeline would not be exposed. In this scenario, there may be additional uplift of a few centimeters of the pipeline between the remaining anchors depending on the backfill soils, but unless the pipeline was already close to the ground surface, this is unlikely to be a significant concern.

Power screw anchors typically have a design factor of safety on the order of three. This provides an allowance for the failure of some anchors without resulting in excess pipe uplift and pipe exposure. This assumes that the remaining anchors maintain sufficient structural integrity to hold loads above the original design load, which should be the case given the relatively high original design safety factor used for anchors.

Similarly, most anchor calculations assume no soil mass or soil shear strength in the backfill, providing another inherent safety factor. As was noted previously, this additional increment of safety factor may be of relatively low importance for large diameter pipelines, (NPS 30 and larger), unless there is a high percentage of mineral soil in the backfill.

Calculations for the load capacity of anchors are available from a number of sources including the Encyclopedia of Anchoring produced by Hubble Power Systems, and Mitsch and Clemence (1985).

3.5 Factors Affecting Buoyancy Control Longevity (Failure Modes)

During pipeline design, a service life (in years) is typically specified. The service life is unlikely to account for abandonment over many decades, and certainly not for hundreds of years. Buoyancy control measures were likely designed only for the service life, with some safety margin. As noted previously, abandoned pipelines may become exposed, particularly in wetlands, due to the failure of pipe buoyancy control measures.

The literature review for buoyancy control longevity focused on articles that discussed degradation of the primary materials used in the manufacture of common buoyancy control measures, including buried concrete, buried steel and buried polymers, such as those used in geotextile bag weights or anchor saddles.

3.5.1 Concrete Longevity

Concrete weights have been used on pipelines for many decades. Buried concrete weighting's only function is to provide mass. Whether the structural strength of the concrete, as quantified by compressive strength, is 5 MPa or 25 MPa, is of secondary importance. Thus, although, references were not found on the required design strengths of concrete weights produced in the mid-20th century, this is not an important uncertainty. Currently, and for the past approximately 30 years, concrete weight design includes a minimum compressive strength before weights are installed on the order of 25 MPa. This requirement ensures the weights can be handled safely during transport and installation. Older weights, even if the specified compressive strength was lower, still needed to withstand the rigours of installation and as such, there was at least a nominal compressive strength. Typically, concrete compressive strength increases with age, as even after some time there can be "an appreciable amount of unreacted cement" (Volpi 2015). This is shown in Neville (2011, Figure 6.32, where 50 year compressive strength averaged 2.4 times the 28 day strength. Even weights with lower compressive strength would very likely have gained strength with age. Although the compressive strength of weights is not a key issue, higher strength weights likely would better resist longer term deterioration from a number of factors (discussed in the next sections) than would lower strength weights. This would aid in weights maintaining their mass and integrity of shape over the longer term.

Neville (2011) is a good source of information regarding a multitude of concrete properties and concrete durability. A few similar points are presented in the short Concrete Information brochure IS536 prepared by the Portland Cement Association (PCA) entitled "Types and Causes of Concrete Deterioration" (PCA 2002). Concrete deterioration is "rarely due to one isolated cause" (Neville 2011), and it is often the result of additional adverse factors that leads to concrete damage. Neville (2011) discusses various potential impacts on concrete durability, noting that deterioration can be due to "external factors or internal causes," and the "actions can be physical, mechanical or chemical." Each of these is discussed further below.

The possible detrimental mechanical actions noted by Neville (2011) are abrasion, erosion and cavitation. The first two would not be issues for abandoned weights, except possibly for the rare third party contact with a weight. Cavitation may an issue in very rare cases. Cavitation occurs when a weight is exposed in turbulent water flow. The air bubbles in the turbulent water can cause pitting on the weight surface. In this situation though, it would be expected that the weights and exposed pipe section would be removed within a short time.

For concrete weights on an abandoned pipeline, the most significant deterioration concern is expected from chemical causes. A single physical cause, ongoing freezing and thawing, may also be a source of some weight degradation over the long term.

According to Neville (2011), chemical causes of concrete degradation include internal alkali-silica or alkali-carbonate reactions or actions by external aggressive ions such as chlorides, sulfates or carbon dioxide. Some natural or industrial liquids or gases may also be a concern, but industrial liquids or gases would rarely be of concern for a pipeline. The internal chemical issues, alkali-silica or alkali-carbonate reactions, can cause cracking, which may lead to the exacerbation of the chemical causes of degradation. However Neville (2011) notes that the alkali-silica reaction, “in most cases,” does not affect the integrity of the concrete. The alkali-carbonate reaction causes a small amount of swelling in certain dolomite based aggregates (Neville 2011), however, it seems that for a pipeline concrete weight, this issue is not a concern. Another useful reference regarding external chemical causes is Roy and Jiang (1995). It provides additional information regarding concrete chemical degradation.

There are three fluids that can enter concrete: water, (pure or carrying aggressive ions), carbon dioxide and oxygen (Neville 2011). The transport of these fluids depends significantly on the hydrated paste structure as this affects the concrete “penetrability.” Factors that reduce the penetrability and thus aid in protecting the concrete are lower water/cement ratios, proper curing, proper compaction and age (Neville 2011 and Volpi 2015). Air permeability will be lower for wet, saturated concrete (Neville 2011) and thus many buried weights would not be subject to the ingress of airborne contaminants.

Carbonation, which occurs when carbon dioxide (CO_2) in air enters concrete, lowers the pH (increases the acidity) of the pore water in concrete which can negatively affect the passivity layer on reinforcing steel. “Passivity is the formation of a thin, non-conductive, oxide surface film that hinders the flow of electrical current and reduces the rate of corrosion,” (Perko 2001). Carbonation can thus promote steel corrosion. The carbonation rate is slow in wet concrete but the highest rate occurs at water saturation of 50 to 70% (Neville 2011). This saturation level may exist in many concrete weights where the groundwater level fluctuates from above the weight to some depth below the top of the weight seasonally or annually because of wetter or drier years in a particular region. The ingress of CO_2 can also occur where there is “moorland” or peaty water as it can have relatively high CO_2 concentrations (Neville 2011). Peaty water with CO_2 and a low pH can be particularly aggressive.

Dissolved salts cause concrete degradation (Neville 2011). Salts such as sodium, potassium, magnesium, and calcium can occur in groundwater or in soil and fertilizers can also add to the overall salt content (Neville 2011). The salts attack the hydrated cement paste and produce expansive products (Volpi 2015), causing strength loss to occur. This may lead to cracking and surface spalling (Volpi 2015). As mentioned earlier, loss of concrete strength is not of concern for abandoned weights. Sulfate resistant cement can reduce concrete strength degradation from dissolved salts, but it is not known how often this type of cement was or is used in the manufacture of concrete weights. Some current specifications do not specify the use of sulfate resistant cement.

In addition to concrete degradation, chemical attacks can also lead to surface cracking or to corrosion of the reinforcing steel. Corrosion does not occur where a weight is fully immersed in water because of lack of oxygen. In addition, corrosion rates are reduced at lower temperatures. Submersion in water and relatively cool temperatures would preclude corrosion risks in possibly a high number of buried weights. Although minor steel corrosion is not a concern, the corroded steel occupies a larger volume than the initial non-corroded steel (Neville 2011 and PCA 2002) and this can increase the penetrability of concrete, leading to higher corrosion rates. Steel corrosion can also cause the surface concrete layer (the concrete cover layer over the steel) to crack away from the rest of the concrete, otherwise known as delamination (Neville 2011). This situation can result in a significant loss of concrete mass. As stated earlier, concrete buoyancy control may only have a 5% to 20% safety factor in terms of the mass needed to counteract pipe buoyancy. Delamination, if it occurs over a sufficiently large area, could lead to a decrease of the safety factor to below unity.

Rock shield wrap in the context of pipe buoyancy is considered to be a cement-based wrap and thus, its degradation would be similar to that of concrete. As this wrap would be used for buoyancy control only on small diameter pipelines and predominately at watercourses, only minimal wrap deterioration would be expected below the water table. Also, as previously mentioned, the native soils at watercourses will likely provide sufficient resistance to buoyancy.

Volpi (2015) presents a number of methods that can be used to check, or infer, if there has been corrosion of the reinforcing steel. Each of these methods appears to require access to the concrete surface and also in some cases to the reinforcing steel as well. One or more these methods could possibly be used if specific problem areas were identified. However, its cost effectiveness would need to be balanced against pipe and weight removal without going through the testing and analysis process.

The last concrete degradation topic is freeze-thaw. Freeze-thaw cycling may be of concern for some concrete weights, depending on local climatic conditions, snow cover, pipe burial depth and water table. It is expected that the upper part of the weighting would be most susceptible. Neville (2011) provides a discussion on the process of freeze-thaw and he notes that freeze-thaw cycling can cause significant damage to concrete over the long term. Saturated concrete (Neville 2011 Figure 11.1) is particularly at risk, and this would be expected for a vast majority of buried weights. Freezing and thawing of the pore water in concrete over multiple cycles can lead to surface scaling or to complete disintegration of the exposed concrete. This allows the degradation to systematically progress inwards (Neville 2011).

The amount of damage to the concrete will depend in part on the number of freeze-thaw cycles. Air entrainment and increased hydration (which results in the removal of water from the paste) before being exposed to freeze/thaw improves concrete's resistance to freeze/thaw. Increased hydration would be the case for abandoned concrete weights as they very likely had limited exposure to freeze-thaw prior to abandonment due to the pipeline's above freezing operating temperature. It is not known if air entrainment was typically used for concrete weights or coating as the concrete generally would not be expected to freeze during operation. Thus, freezing and thawing could lead, in some more extreme situations, to a sufficient loss of concrete mass that the original safety factor may be reduced below unity. It is not known how prevalent this situation may be for actual weights, and thus some data on weight integrity observed during a number of pipeline digs would be beneficial.

There is very limited information on the lifespan of concrete swamp weights. Figure 15 shows an excavated swamp weight. The weight is understood to be over 50 years old and was located on a pipeline in Northwestern Canada. The photograph shows some organic soil at the surface, which means that some of the concrete degradation conditions described above may have been present, but the presence of specific chemicals was not verified by testing. It can be seen that the weight corners are still well defined and there is not any noticeable spalling on the surface or visible large cracks. Also, the thinner legs of the weight appear to be in very good condition. As such, it does not appear that the weight has lost any mass after more than 50 years of service.

Figure 15 Concrete Set-on Weight, Over 50 Years Old, with no Noticeable Deterioration (photo courtesy of D. Madden, TransCanada Pipelines)

Similarly, saddle weights were to be used on the TransAlaska Pipeline System (TAPS) but were surplus at the conclusion of construction. Rather than disposing of these concrete weights, they were used for bank protection along a major river in northern Alaska. Figure 16 shows a number of these swamp weights. Having been placed in the mid-1970s, the weights are approximately 40 years old in the photograph. Note that the river is subject to significant floods almost annually and to severe winter icings at times. Even with this significant exposure to the elements, the weights do not appear to have suffered any visible damage.

Figure 16 Saddle Weights Used as Bank Protection in Northern Alaska (photo courtesy of W. Veldman of WMV Consulting Inc.)

The lifespan of river weights depends in part on the bolts that connect the two concrete halves. Figure 17 (courtesy of Mr. D. Madden, TransCanada Pipelines) shows an excavated river weight from northern Alberta that is understood to be approximately 60 years old. The weight halves are still joined together even though the lower half of the weight appears to be unsupported as a result of the excavation. This indicates that the bolts connecting the two halves remain intact and have not deteriorated to the extent that the halves could separate. Also, similar to the saddle weight on Figure 15, there is no noticeable concrete deterioration along the edges or the surface of the weight. Although the view in the photograph is limited, there are no visible large cracks and it does not appear that the weight has lost any mass.

Mr. Madden noted in his email communication of March 13, 2018 that “The majority of the weights we have taken off were in good condition and did not have to be replaced.” (However, there has not been any clarification to date on what “majority” means. Also, weight replacement in many cases could be due to deterioration of the top lifting hooks for swamp weights rather than deterioration of the concrete.)

Photographs of aged CCC were not available for inclusion in this report; however Figure 18 shows a section of exposed concrete coating. The approximately 8 year old coating was exposed due to a major flood on a river in the United States and the coating looks to be in good condition.

Figure 17 Concrete Bolt-on Weight, Approximately 60 Years Old, with no Noticeable Deterioration and the Weight Halves Still Locked Together (photo courtesy of D. Madden, TransCanada Pipelines)

Figure 18 Exposed Continuous Concrete Coating, no Noticeable Deterioration after Approximately 8 Years (photo courtesy of G. Connors, PipeSak®)

Senior chief pipeline construction manager, Mr. J. Ness, stated via email on February 9, 2018 that, "All the weights (I have) dug up over the years, did not seem to show any deterioration. There must be some but of course (it was) not measured." Mr. Ness did not have any photographs of excavated weights.

Based on the information provided above regarding potential causes that may negatively affect concrete longevity and the photographs provided by various pipeline engineering practitioners, the risk of a loss of buoyancy control appears to be minor for abandoned pipelines weighted by concrete. This statement is based on the apparent minor risk of a significant loss of mass due to concrete deterioration supported in part by the photographs and anecdotal information about existing long-term pipelines.

3.5.2 Screw Anchor Longevity

Screw anchors were used on pipelines in the 1950s (Robertson and Curle 1995). Their use apparently continued until the late 1960s or early 1970s, but were then used only sparingly or abandoned. Screw anchor use resurged in the early 1990s when Nova Gas Transmission started using them again on major projects. Initially, it is understood the saddles used were steel but were later changed to either straps (see Figure 9) or polyester saddles (see Figure 10).

Screw anchor longevity will depend on the longevity of its various components, which consist of:

- anchor lead section with helix
- anchor extensions
- couplings between lead section and extensions
- bolts and nuts at the couplings
- top coupling, and
- saddle (steel, polyester or other)

The rate of deterioration of the steel components will depend on several factors. These include:

- steel coating (if any)
- soil type and its associated resistivity
- groundwater level, and
- soil pH (acidity)

The steel components (anchor lead sections, the helix, the extensions, and the couplings) are typically not coated and their thickness is designed for a specific service life. The service life is specified by the pipeline owner and then the anchor supplier typically provides a steel thickness suited to the expected corrosion conditions. The ultimate load capacity over and above the design load capacity also provides a built-in allowance or safety factor for a loss of some steel by corrosion. Steel saddles typically were not coated or did not use galvanized steel, but this depended in part on pipeline owner specifications. If steel saddles were not coated, a suitable thickness would have been chosen to account for the required design life and expected local corrosion conditions.

Bolts and nuts used in steel anchors are usually coated. It is understood that the coating may be xylan, a galvanized layer, or other suitable material. Couplings may also be coated but as with steel saddles, this may not have always been the case. The longevity of the nuts is not considered critical for long-term anchor integrity, because the upward force on the anchor will act to secure a bolt in place, even in the absence of a corresponding nut.

Perko (2001) provides a good review regarding corrosion of steel anchor components. Perko describes corrosion as an "exothermic chemical transformation of a metal or metal alloy to a non-reactive covalent compound" and notes that the "electrochemical composition of the aqueous solution almost always governs the rate of corrosion." Thus, the soil type and the presence of dissolved salts are significant factors in the corrosion rate. Perko also notes that the corrosion rate depends on the quantity of dissolved and free oxygen. The paper presents a rating of soil corrosivity based on soil resistivity. Low soil resistivity is indicative of soils with a high pH, high moisture content, high salt content, and an ample supply of dissolved oxygen. High to very high corrosivity occurs when soil resistivity is less than 2000 ohm-cm. Peats and some clays fall into this category (Perko 2001 Figure 1). However, natural peats typically have lower dissolved oxygen with depth (Priest 2012) with correspondingly lower corrosion rates at depth. Very low corrosivity corresponds to soil resistivity values greater than 10,000 ohm-cm.

Perko (2001) discusses the galvanizing process. He notes that coated steel generally has a corrosion rate 50% to 98% lower than uncoated steel. This indicates that the coated bolts, nuts and connectors, will have significantly reduced corrosion rates. Corrosion rates for galvanized steel shown in Perko (2001 Figure 2) are on the order of less than 0.1 oz/sf/yr to 1.0 oz/sf/yr in one test area.

Perko (2001 Table 2) presents the estimated life span for bare steel, coated steel and galvanized steel in a variety of soil electrical resistivity ranges. For low resistivity (severe corrosivity) soils, the 95% probability minimum anchor lifespan is listed as 30 years for bare steel and 75 years for galvanized steel. In a "high" corrosivity soil, the life span more than doubles to 70 years and 170 years for bare and coated steel respectively. In a low corrosivity environment, the life span exceeded 300 years for the bare steel. Life span was defined as the length of time required for corrosion to extend 1/3 into the helical blade (a 1/3 reduction of blade thickness) and 1/2 reduction of the central shaft's thickness. The portion of the anchor at highest risk is likely the portion of the shaft nearest the ground surface as this is where water table variations may occur and where the access to oxygen will be highest (Perko 2001).

Steel saddle plates would also be subject to corrosion, the rate of which may be similar to that of upper sections of screw anchors.

Consideration must be given to the relative corrosion rates of the abandoned steel pipeline and that of steel screw anchors. If anchors are corroding relatively quickly due to a corrosive environment, then the steel pipeline is likely also being subject to higher corrosion rates. If this is the case, then eventually pin holes would develop in the wall of the steel pipeline allowing for water ingress to the pipe. This would result in a concomitant reduction in pipeline buoyancy, particularly at the lowest elevations of the anchored pipeline section where water initially would collect.

Polyester saddles are unlikely to be subject to significant degradation as this material has a buried service life exceeding 100 years. This is further discussed later in the section regarding geotextile swamp weights.

Odom and Thorsten (1994), presents a review of two anchors, each with four helices, which were buried for 8 years. One of the anchors was galvanized while the second was not. The anchors were buried in a high-plastic clay that may have been saturated. The soil pH was listed at 6.7 to 7.8 while the soil's resistivity was listed as 1,200 to 2,900 ohm-cm, which, based on the soil resistivity values, is high to very high corrosivity. The authors observed that the "anchors appeared in excellent condition with no rust visible on the galvanized anchor." Neither anchor had any pitting or scaling. They concluded that "no appreciable metal loss occurred at this site from corrosion." The time frame of 8 years is of course relatively short. Also, if the anchors were placed below the water table, this would significantly restrict the access of oxygen to the anchors and thus significantly limit corrosion. However, the soil corrosivity was in the high to moderate range per its resistance and because no visible pitting or scaling was observed, this does provide a favourable indication of anchor resistance to degradation.

It is noteworthy that a senior chief pipeline construction manager, Mr. J. Ness, stated in an email dated February 9, 2018, that over the last 25 to 30 years, he had "never heard of a screw anchor failure due to corrosion."

Pipeline technologist, Mr. A. Brown, as indicated in an email dated February 13, 2018, was involved with an excavation of an existing pipeline where a screw anchor, likely installed in the late 1960s or early 1970's, was exposed and removed, as shown on Figure 10. The anchor would have been nominally 50 years old and was located in northeast British Columbia. The soils were silt and sand with organic layers (see Figure 20). The anchor consisted only of a 2.4 m long lead section with a single helix, estimated to be 250 mm diameter with a thickness of approximately 9.5 mm. The rod for the lead section was approximately 25 mm diameter. The anchor rod and helix appeared to be bare steel. The saddle was a steel band. Mr. Brown recalled that the shaft was pitted, but only minimally. The helix looked to be in good condition. Mr. Brown stated that the "only noticeable area of corrosion (of the shaft) was at the top of the water table." It was estimated that the shaft diameter was reduced to approximately 19 mm in this area. If one magnifies the photograph and zooms into the steel saddle as shown on Figure 21, the saddle and holes where the anchor shaft connected to the saddle look to be in reasonable shape. There are not any obvious signs of pitting or metal loss. Unfortunately, only the bottom side of the saddle is visible.

It is possible to add sacrificial anodes to anchors to extend their service life. This may be an area of further study as to where it may be effective (i.e., high corrosive areas).

The longevity of the other anchor types, such as flip and grouted anchors, would be expected to be similar to screw anchors. As with steel screw anchors, corrosion of the steel components likely controls the anchor's serviceable lifespan.

Figure 19 Excavated 40+ Year Old Screw Anchor in Northeast BC (photo courtesy of A. Brown of AHB Consulting)

Figure 20 Soil and Water Conditions in Area of Excavated Screw Anchor in Northeast BC (photo courtesy of A. Brown, AHB Consulting)

Figure 21 Excavated 40+ Year Old Screw Anchor Saddle in Northeast BC (photo courtesy of A. Brown, AHB Consulting)

3.5.3 Geotextile Bag Longevity

It was previously noted that the first PipeSak® weight was installed in 1994. It is believed that this was the first use of this type of weight with other varieties having followed since. As such, the oldest bag weights are about 24 years old. PipeSak® weights were initially made with polypropylene for the bag section and polyester for the strapping (per Mr. G. Connors of PipeSak®). Today, the PipeSak® bags are made entirely of polypropylene because of the longer life of polypropylene relative to polyester. Other bags weights are made with polyurethane or nylon according to bag weight product information available online.

Ultraviolet (UV) radiation in sunlight poses the greatest risk to the various polymer-based weights. UV light can significantly degrade the properties of polymer chains by breaking bonds in the polymer structure (Amoco Fabrics 1992). UV exposure occurs only for bags or weights left in sunlight. It is understood that exposure for a few months is not problematic for most if not all bag weights, but longer exposure durations can negatively impact polymer bags unless they are coated with a specific treatment. Once buried, UV exposure is no longer a concern. Currently most, if not all, fabrics used in the manufacture of geotextile bags are understood to be treated to be resistant to UV light.

Amoco Fabrics (1996) notes that polypropylene geotextiles are highly resistant to chemical degradation. These geotextiles are recommended for use even in high chemical leachate environments such as landfills and as such should perform well in the relatively benign conditions of most overland pipelines. It is expected that all polymers used in bag manufacturing would be resistant to chemical degradation.

Dr. K. Rowe, Ph.D., P.Eng., a professor at the University of western Ontario, was engaged by PipeSak® to assess the longevity of the polypropylene used in PipeSak® bags. Dr. Rowe concluded on the basis of his testing (Rowe, 2003) that the bags would last under normal service for "well over 380 years." Mr. Connors of PipeSak® has indicated via personal communication that the life for polyester was judged to be only marginally less (Connors G. 2013. Pers. Comm.)

Figure 22, for example, shows a series of geotextile bag weights that were exposed following a major flood in a river. The bags are only 8 years old, but an inspection of the bags revealed that there was not any damage or deterioration.

Bag weights are therefore expected to last many decades if not centuries buried underground. However, it is not known what long-term testing has been done on polymer type saddles under constant load for anchor applications.

Plate weights are another type of soil weight that has been used, but relatively rarely. These plates are made of a molded plastic and thus are likely to be practically non-biodegradable. As far as is known during this review, only a very few plate weights were installed on about two small diameter pipelines, and as such are not considered a significant concern in the broader context of abandoned pipeline buoyancy.

Figure 22 Exposed Geotextile Bag Weights, no Noticeable Deterioration of Tears. Weights Were Approximately 8 Years Old (photo courtesy of G. Connors, PipeSak®)

3.6 Overall Long-Term Buoyancy Integrity Risk

The integrity of buoyancy control measures was assessed in terms of the integrity of the component materials such as concrete or steel, and our reality check was supplied by the field observations of several long-standing industry professionals.

Anchors may be most at risk of long-term failure. This is based on corrosion concerns for two particular anchor components: the saddle-to-anchor connections, and the upper part of the anchor stem, both of which are located where there is the highest likelihood of steel corrosion. Corrosion may be higher for these components due to fluctuating groundwater levels and oxidizing conditions. Metal loss at both of these locations over the longer term may be sufficient to result in anchor component failure. Anchor failure may be more likely to occur where anchors are restraining higher loads relative to ultimate load capability for the whole anchor system.

The other situation where abandoned pipeline buoyancy control may be at risk is where a deeper trench was used to provide backfill mass over the pipeline. Long-term backfill erosion (local erosion patterns or general local lowering of the land due to land use) may result in a sufficient loss of soil mass to allow for pipe uplift. This situation may manifest itself during a period of extensive rainfall and an increase in the level of the local groundwater table.

The long-term integrity of geotextile bag weights is considered to be relatively high. This is due to the long lifespan of polymers in relatively benign environments, as would be found along the vast majority of onshore pipelines. A possible issue with geotextile bags may be loss of strength over time of the connecting webbing or straps. This would be at least partially compensated for due to the safety factor used in bag design.

Similarly, the relatively benign environments for concrete limit long-term integrity risks for concrete buoyancy control. Thus, concrete weighting is also expected to endure for many decades barring a unique set of soil conditions and assuming the concrete weighting was initially manufactured to the specified requirements.

3.7 Recommended Future Studies

Additional studies are recommended to help clarify abandoned pipeline buoyancy issues:

- Develop a new information standard for pipeline companies to provide consistent pipeline integrity dig information, and inspection information for exposed pipelines, when buoyancy control devices are excavated (but excluding any sensitive information). Provide a list of data parameters to those companies on what information to collect when buoyancy control devices are exposed so that consistent data is received to be entered into a database.

- Develop a new reference resource for industry by mapping those areas where conditions of high soil corrosivity and high microbial activity may be present. The mapping would be made available to owner companies as aids in assessing potential longevity of some existing buoyancy control measures and could also be used by owner companies to make informed decisions when planning for pipe abandonment.

- Conduct a comparative assessment of the relative long-term integrity risk of buoyancy control failure versus the long-term risk of through-wall corrosion of the steel pipeline. If through-wall corrosion occurs and allows a pipe to fill with water, this negates the concern of pipe exposure due to buoyancy control failure.

- If data gathered from the three studies mentioned above identified a significant risk (i.e., high frequency of occurrence and significant consequence to safety and environment), then assess the technical, environmental and cost implications of having owner companies, or as a PTAC research program, add sacrificial anodes to steel anchors in areas of high risk of anchor failure. These areas may be where high corrosive soils are present or where there are fluctuating water tables. This option could then be weighed against other options, such as the previously noted allowing the pipeline to fill with water or the removal of the pipeline and buoyancy control over the length of pipeline at risk of exposure.

3.8 Conclusions

This section presents information on abandoned pipeline exposure risks from the long-term degradation of various buoyancy control measures that are known to have been used for onshore pipelines. The need to mitigate pipeline buoyancy at the time of construction depends on many factors such as the pipeline material, the pipe diameter and its wall thickness, the installation timing (summer vs. winter), the pipeline burial depth and the density and type of the trench backfill material. Even for pipelines that will carry or may have carried a heavy product, buoyancy control measures may still have been installed due to the pipeline being subject to excessive buoyancy forces in the period between pipeline installation and full operation or during maintenance events. During pipeline design and construction, ideally all relevant factors are or were taken into account when determining the optimum type and frequency of buoyancy control measures to mitigate pipeline buoyancy in susceptible areas.

One possible cause of pipeline exposure may be the failure of buoyancy control measures due to the long-term degradation of those measures. Long-term degradation may occur for several reasons depending on the type of buoyancy control measures that were installed during construction of the pipeline. Causes may include: spalling of concrete weights, corrosion of reinforcing steel used in concrete weighting, corrosion of steel in anchors, corrosion of saddle-to-anchor connections, breakdown of the polymer materials used in anchor saddles or in geotextile weights, or the loss of mineral soil backfill over the pipeline. If pipe exposure does happen or has a high probability of occurring due to a failure of buoyancy control measures and it results or could result in land use interference, then other abandonment practices may need to be considered. These practices may include pipe removal or filling the pipe with an inert substance (and natural groundwater may be an option) to reduce buoyancy.

After reviewing the various buoyancy control measures that are known to have been used in Canada, it is considered that most of these measures likely will outlive the integrity of the steel wall of the abandoned pipeline, unless the pipeline continues to be cathodically protected. The expected lifespan of steel pipe was reported as part of the PARSC 001 report (DNV 2015) in terms of the likelihood of collapse under the weight of a truck. The PARSC 001 report suggests that an uncoated medium diameter pipeline with shallow burial in extremely corrosive and poorly draining soil conditions may collapse within 100 years. Conversely, a coated pipe in good soil conditions may be structurally sound for several thousand years.

The buoyancy control measure that may be most at risk from long degradation is steel anchors. Anchor degradation over the long term will depend significantly on the initial thickness of the various steel components, whether they were coated, local soil corrosivity, groundwater levels and their variation, and groundwater mineral content. The other major buoyancy control measures such as concrete weighting and geotextile bags are expected to last many decades, even after pipe abandonment.

For small diameter pipelines, on the order of NPS 16 or smaller, even if installed buoyancy control measures fail, these pipelines will have a low likelihood of exposure due to buoyancy. This is because the consolidated backfill, even in organic soils, will provide sufficient resistance to the relatively low buoyancy force.

The long-term integrity of buoyancy control measures must also be considered in conjunction with the long-term integrity of the pipeline wall itself. If through-wall corrosion occurs at only a few locations in a wetland, these points will allow water into the pipe and the pipe's natural buoyancy will be reduced or negated.

Finally, the effect of a buoyancy control failure along an abandoned pipeline may also be mitigated by neutralizing the pipeline buoyancy, such as cutting the pipe or drilling holes to perforate the pipe wall. This style of mitigation will need to be considered in conjunction with related hydrotechnical hazards to avoid diverting a watercourse along the abandoned pipeline.

The buoyancy considerations discussed in this chapter are applicable to onshore pipelines. No consideration has been given in this report to offshore pipelines.

4 LOW IMPACT REMOVAL METHODS OF SENSITIVE ECOLOGICAL AREAS

4.1 Introduction

The decision by a pipeline operator whether to abandon a pipeline in place should consider added environmental factors in sensitive ecological areas where additional ground disturbances may result in more harm than good. Sensitive ecological areas include water bodies and especially wetlands where ecological functions may be sensitive to disturbances, where the removal of the pipe may result in excessive disturbances, and where the ecological impact of the disturbances may extend beyond the pipeline right-of-way. In addition to water bodies, other terrestrial areas may also have sensitive ecological areas. However, this study focuses on water bodies because of their potential to be impacted outside of the right-of-way. Sensitive terrestrial areas will still be subject to other abandonment compliance factors.

The following chapter describes the previous industry guidelines, keys for considering pipeline removal methods, an assessment of various common pipeline removal methods, examples of pipeline removal projects, and recommended next steps to further evaluate pipe removal methods at sensitive ecological areas.

4.2 Previous Industry Guidelines and Evaluations

In November 1996, a Steering Committee comprised of representatives from the Canadian Association of Petroleum Producers (CAPP), the Canada Energy Pipeline Association (CEPA), the Alberta Energy and

Utilities Board (EUB) and the National Energy Board (NEB) prepared a *Discussion Paper on Technical and Environmental Issues* with respect to pipeline abandonment (NEB 1996).

Some highlights in the Discussion Paper are:

- whether "*to abandon the pipeline in place or through removal should be made on the basis of a comprehensive site-specific assessment*"
- the site-specific factors to consider for water crossings are "*the size and dynamics of the water body, the design of the pipeline crossing, soil characteristics, slope stability and environmental sensitivities*" and,
- the Steering Committee concluded that "*in most cases it can be expected that abandonment-in-place will be the preferred option*."

In June 2016 the NEB published *Regulating Pipeline Abandonment*. The document:

- lists the required measures for abandoning a pipeline and monitoring it post-abandonment
- specifies that "*the proposed abandonment will be carried out in a safe manner and that potential environmental, socio-economic, economic and financial impacts are identified and addressed*"
- that the abandonment method should be "*appropriate for the ecological setting where it is located*"
- stresses that financial resources are to be set aside for "*abandonment work, including the activities to deal with unforeseen events*" such as "*pipeline settlement or exposure of the pipe that might occur after abandonment*"

In the NEB's *FAQs on Pipeline Abandonment* it is noted that "*issues such as pipeline exposure through erosion or depressions caused by settling may require ongoing monitoring for as long as the pipeline remains in the ground*." It is further indicated that owners are responsible "*…for any unforeseen events that might occur after abandonment.*"

Abandonment of pipelines, to date, is rare. In 2013 of the 73,000 km of pipelines regulated by the NEB, only 317 km or less than 0.5% of the lines going back to 1959, were abandoned. In March 2018, TransCanada Corp. received the go ahead for the largest abandonment approved by the NEB to date – 266 km. The Abandonment Plan involves the removal of only 9 km of the 266 km total length, the portion of the line running through a First Nations Reserve Land.

4.3 Keys to Assessing the Advisability of Pipeline Removal

From the above-referenced guidelines, the keys to deciding whether to abandon in place or remove a pipeline crossing of a water body depends on:

- whether the pipeline is exposed now or could likely become exposed in the future due to changing environmental conditions or loss of buoyancy control methods
- a site-specific assessment of the trade-offs of pipe removal versus abandoning the pipe in place

The assessment of trade-offs may be very site specific. For example, excavating into the shoreline of a wetland to remove a pipe could, through its disturbance, increase seepage from and drainage of the wetland (for an overall assessment of the water conduit issue; Parbery 2017.

The following section expands on the trade-off assessment considerations. In all instances, it is assumed that pipeline abandonment (in place or removed) will satisfy the following:

- the pipeline will have been fully cleaned internally, and
- measures are implemented to prevent the pipeline from becoming a water conduit such that the pipeline will not divert water from the water body.

4.4 High Level Assessment of Low Impact Removal Methods

The decision to remove a pipe during abandonment should be considered on a case-by-case basis to account for local factors. Although pipeline removal experience is limited, knowledge and techniques developed from the installation of new pipelines is applicable in many respects (CAPP 2005 and ASME 2008).

The experience of the authors plus the experience of the surveyed construction personnel indicated the following pipe removal feasibility for abandoned pipes:

- The feasibility of removal of a horizontal directionally drilled (HDD) crossing is highly unlikely due to the presence of the drilling fluid or bentonite.
- The significant typical HDD pipe burial depth and setback into the bed and banks respectively results in a minimal potential for pipe exposure and thus minimizes or eliminates the need to remove the line.
- Surficial settlement, due to deterioration and subsequent collapse of the pipeline at an HDD crossing, is expected to be insignificant due to the pipe profile and depth.
- Thus it is expected in all cases that HDD watercourse crossings will be abandoned in place.

- With recent trenchless construction technologies such as micro-tunneling or direct drilling which do not use bentonite, their removal may potentially be feasible depending on soil characteristics, length and profile of the crossing and pipe size. Removal would involve excavating bell holes adjacent to the crossing, cutting the line and removing it via pulling using either an HDD rig or more likely, via large dozer(s).

- For trenched crossings, set-on or bolt-on concrete weights preclude pulling out the crossing. Assuming equipment operations are permitted and feasible instream, and depending on the depth of the line, the line could be lifted and the weights and pipeline removed individually or as one unit.

- A small non-weighted 50-100 m long crossing, up to 500 mm in diameter, can likely be "pulled out". The feasibility of this scenario was surmised from discussions with several construction specialists, depending on:

    - depth of cover – removal could either be via a 'pull', for which depth of cover is not a significant factor, or by lifting up through the remaining cover via side booms for which depth of cover is a factor,

    - material conditions – removal is more likely feasible for granular or clean sand bed rivers, and less likely for silty or clay conditions, and

    - flow depth and aquatic values of the stream – 'walking' equipment across the river while lifting may or may not be feasible depending on flow depth or local aquatic resources.

- A longer crossing could be removed in segments, with minimal instream works, if mid-channel gravel bars or islands permit multiple bell holes to be excavated in non-flowing conditions thus enabling pipeline removal via a number of sections.

- If a watercourse crossing cannot be pulled through the bottom material and depending on its burial depth, material type, water depth and environmental considerations, the pipeline could potentially be lifted up through the remaining cover with the staging and sequencing of side booms – the reverse of pipe lowering. Backhoes could be used instead of or to aid side booms. This could enable the removal non-exposed concrete weighted lines.

In summary:

- It is highly unlikely that an HDD crossing can be removed. And likely little or no reason to remove it.

- A non-concrete weighted crossing might, depending on its pipe size and crossing length, be removed via pulling in one piece or in segments. It is unlikely that a concrete weighted pipe, especially one having concrete set-on or bolt-on weights, can be pulled out.

- A concrete weighted pipe could, for a crossing with shallow water depths, be removed via side booms or backhoes in a manner being the reverse of pipe lowering.

- As noted in the industry and regulatory guidelines, pipeline exposure in a sensitive area may trigger or increase the need for its removal. Site-specific conditions would no doubt still govern – exposure in a remote area would be addressed differently than in a high public use area.

4.5 Examples of Low Impact Removal Methods

Feasible, practical and environmentally sound low impact pipeline removal methods, in part developed from years of new pipeline construction, are possible for certain pipeline and stream crossing conditions.

Some examples of exposed or shallow watercourse crossings and the associated removal methodology and steps are outlined and illustrated on Figure 23a through Figure 23e. As detailed in the industry guidelines, the optimum method will depend on site-specific watercourse and environmental considerations.

4.6 Recommended Work Plan to Evaluate Pipe Removal Methods

The investigation and evaluation of low impact pipe removal methods for sensitive ecological areas is an initiative that is multidisciplinary and will require the input of various discipline specialists. We expect that the majority of sensitive areas will involve water crossings, although other settings may also be sensitive. We further expect that pipes will be removed at locations where a future exposure is inevitable, such as a shallow buried pipe in a river that is progressively (and rapidly) degrading – that is, where the river bed is becoming progressively lower over time. Other sensitive areas may require more careful consideration of the unintended consequences of additional disturbances as part of the pipe removal.

We recommend the following work plan:

- solicit from the individual PTAC members and organizations, examples, photos and details of low impact pipeline removal methods that have been employed,
- organize a workshop of experienced pipeline construction and environmental professionals to review and assess the collective knowledge:
- prepare an industry guide, similar in format and stakeholder input, to CAPP's 2005 guideline for the construction of watercourse crossings.

The primary focus of this work plan is a workshop organized similar to a small conference that will present industry pipe removal examples and promote panel-discussion style working conversations to refine the following:

- types of site conditions that should qualify as sensitive ecological areas,
- types of low impact pipe removal methods available,
- group decision analysis exercise to evaluate potential fatal flaws and rate the potential pipe removal methods, and
- deliver a summary of results to describe the conditions or situations where pipe removal should be considered and the acceptable low impact methods for removing the pipe.

Conditions

- Buried in the banks
- Horizontal pipe profile
- No concrete weighting
- Narrow crossing with low vegetated banks
- High aquatic value stream

Removal Methodology and Steps
Overall Approach

- Instream equipment operation to be avoided – due to aquatic value – and not deemed to be necessary as equipment is able to work from both banks. Maintain vegetation on the banks.

Steps

- Dig bell holes in right-of-way, back from the vegetated banks, both sides, to expose the line.
- Cut the line in both bell holes.
- Remove the line, via a horizontal pull.
- As deemed necessary – depending on the natural sediment/bed material movement in the stream – backfill the pipe ditch with imported clean granular fill and in a manner to minimize disturbance of the vegetated banks.

Figure 23a Low Impact Pipe Removal Methods - River Crossings Fully Exposed, Minor Crossing

Conditions

- Steep, boulder stream
- Banks of variable height
- Width ranges from 5 m to 25 m
- Low to minimal aquatic value
- Pipeline may be weighted (coating or bolt-on) or may be non-weighted

Removal Methodology and Steps

Overall Approach

- Instream equipment operations permitted – due to low aquatic value and feasible due to shallow depth.
- Schedule work during low flow periods.

Steps

- Dig bell holes in right-of-way, back from the banks, both sides.
- Cut the line in both bell holes.
- Cut the line in middle of stream (depending on the width of the crossing).
- If pipe is not weighted:
  - lift up or pull out the crossing in one or more sections
- If pipe is weighted, especially with set –on weights or bolt-on weights, pulling not feasible through the bank area.
- Then:
  - expose the line from the bell holes to the stream crossing,
  - lift and remove the two line sections

Figure 23b Low Impact Pipe Removal Methods - River Crossings Fully Exposed, Minor to Intermediate Crossing

Conditions

- Concrete weighted (continuous coating or weights)
- Width up to 25 m
- Flow less than 1.0 to 1.5 m3/s during allowable instream work period.
- Moderate to high aquatic value

Removal Methodology and Steps

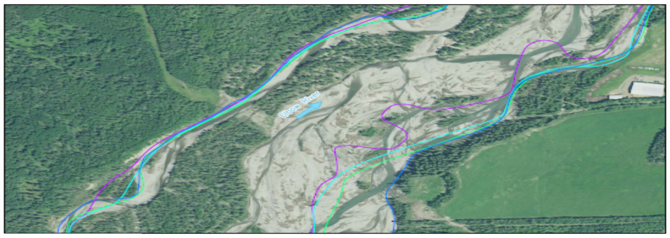
Overall Approach

- Flow isolation required to enable instream work in high aquatic value stream - technique same as used for new pipeline installation.

Steps

- Install sump and flow diversion pump(s).
- Install upstream and downstream diversion dams using 1 m^3 or smaller sand bags or "Aquadam".
- Excavate clean water interceptor ditch, inside the perimeter of the isolation structure, if seepage under/through the dams is high, and pump seepage around the site.
- Install ditch pump and discharge to an approved natural vegetated depression and/or a constructed sediment settling area.
- Excavate for and remove the deactivated line.
- Backfill the pipe ditch.
- Remove the ditch pump.

Figure 23c Low Impact Pipe Removal Methods - River Crossings Shallow Pipe, Minor to Intermediate Crossing



Airphoto.

Simple temporary crossing.

Bellhole excavation in bank or on a gravel bar.

In shallow water, booms can be used to lift the pipe out through the remaining shallow cover.

Conditions

- Shallow pipeline in active channels concrete weighted.
- Cover depth may be several meters deep across gravel bars and mid-channel islands.
- Wide braided river with mid channel gravel bars or vegetated islands.
- High aquatic value.

Removal Methodology and Steps

Overall Approach

- Minimize instream work and equipment operations. Work done during low flow. Excavate bellholes and cut the total crossing length into manageable lengths re: pulling or lifting the line out.

Steps

- From surveys of pipe profile and river crossing conditions, establish:
  - locations of bellhole excavations in non-flowing conditions
  - need for and type of temporary crossing(s) of subchannels
- Dig/expose/cut the line at the selected locations. Lift up (via booms) or pull depending on channel and bank conditions.
- As necessary across the bars and islands, first lower the grade to the water level to facilitate lifting out of the pipe.
- Restore right-of-way across channels and vegetated areas as necessary. Natural infilling of ditch area typically occurs in a braided river during the first increase in flow.

Figure 23d Low Impact Pipe Removal Methods - River Crossings Shallow Pipe or Locally Exposed, Major Crossing

Conditions

- Pipeline adequately set back into both banks.
- High aquatic value.
- Flow isolation not feasible due to high flow.
- Crossing width greater than 25 m.
- Steep, high banks.
- Concrete coated.

> **NOTE**
> Due to the significant extent of instream equipment operations required, this is not a "low impact" example. Provided as a comparison to the other examples.

Removal Methodology and Steps

Overall Approach

- Instream equipment operations required. Flow, even during the low flow period, too high to permit isolation

Steps

- Excavate down to the line to expose it
- Excavate into the banks
- Cut line at both banks/edge of River
- Lift and remove, restore the streambed/with the excavated material. Natural infilling of the pipe ditch may be adequate
- Restore streambanks using native large boulders

Figure 23e Low Impact Pipe Removal Methods - River Crossings Shallow Pipe, Major Crossing

5 POTENTIAL PIPELINE EXPOSURE FROM FROST JACKING

5.1 Introduction

In the NEB-commissioned Pipeline Abandonment Scoping Study (DNV, 2010), frost heave was enumerated as a potential mechanism for pipeline exposure. It was noted in that literature review that there is no indication this mechanism has ever caused exposure of an abandoned pipeline. The subsequent report sponsored by PTAC PARSC 003 (Stantec, 2014) provided an initial assessment of frost action as a potential mechanism for abandoned pipeline exposure.

This chapter was written to clarify the potential for frost action effects on abandoned buried pipelines, (as requested by PTAC), and addresses the following objectives:

- Provide a sound technical background regarding frost action in soils, including the physics of frost heave in soils, frost heave on pipelines, and the distinction between frost heave and frost jacking.
- Demonstrate soil temperatures and vertical frost front migration in winter around an abandoned pipeline using a two-dimensional (2D) finite element model for discussion purposes.
- Provide the scope and technically sound methodologies for laboratory and field investigations of pipeline frost jacking, including the ideal attributes of candidate field sites to investigate the potential for pipeline frost jacking.
- Provide a review of the conclusions and recommendations made in the previous PTAC PARSC 003 report (Stantec, 2014).

5.2 Technical Background

5.2.1 Seasonal Ground Temperature Variation and Maximum Frost Depth

The temperature of the ground surface varies seasonally throughout the year and is governed by heat energy exchange at the ground surface. This energy exchange is modulated by convective heat transfer between the ground surface and the atmosphere, incoming solar radiation, outgoing longwave radiation at the ground or snow surface, and summer evapotranspiration.

The seasonal variation (amplitude) of ground temperature variation is greatest at the ground surface and decreases with depth. Below the ground, there is a delayed temperature response, and the delay in that response increases with depth. For example, the coldest ground surface temperatures may occur in January each winter, but the coldest temperatures at typical pipeline burial depth may not occur until sometime in February or March.

The maximum depth that frost penetrates downward from the ground surface each winter (the maximum frost depth) depends on the severity and duration of subfreezing temperatures, the latter of which can be measured by the freezing index. The freezing index is calculated as a summation of the daily air temperature values which are below 0°C during the fall, winter and spring months. The greater the freezing index, the more severe the winter, with corresponding deeper frost penetration.

The maximum frost depth is also particularly sensitive to the soil moisture content at a particular location. Frost depth in moist fine-grained (clay and silt) soils can be on the order of 1.5 m in Alberta and increase to more than 2.0 m in relatively dry coarse-grained (sand and gravel) soils.

5.2.2 Frost Heave in Soils

To understand the potential for abandoned pipeline exposure from frost action in soils, it is first necessary to have some understanding of the physical mechanisms which cause frost heave in soils.

Upon cooling to subfreezing temperatures, water freezes from liquid to solid (ice) phase at 0°C. However, in soil pore spaces, ice and liquid water coexist at subfreezing temperatures. As shown schematically on Figure 24, pore water consists of water that is bound to solid soil particles and remains in liquid phase at subfreezing temperatures, and free water which will solidify to ice at cold temperatures. Ice first forms at the center of soil pores, and with cooler freezing temperatures the ice will penetrate deeper into the soil pore throats.

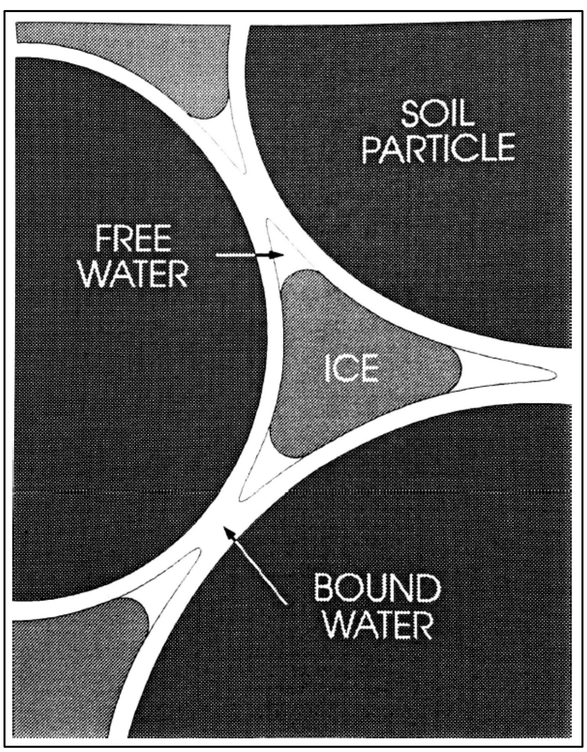


Figure 24 Unfrozen Water in Frozen Soil (from Coutts, 1991)

In soil, the water to ice transition occurs over a subfreezing temperature range that depends on soil pore size. As shown on Figure 25, the unfrozen water content in a soil decreases with colder subfreezing temperatures, and at any particular temperature (for example at -2°C) fine-grained (clay and silt) soils have significantly higher unfrozen water content than coarse-grained (sand and gravel) soils. This is a direct result of the extremely small pore sizes in fine-grained soils compared to coarse-grained soils.

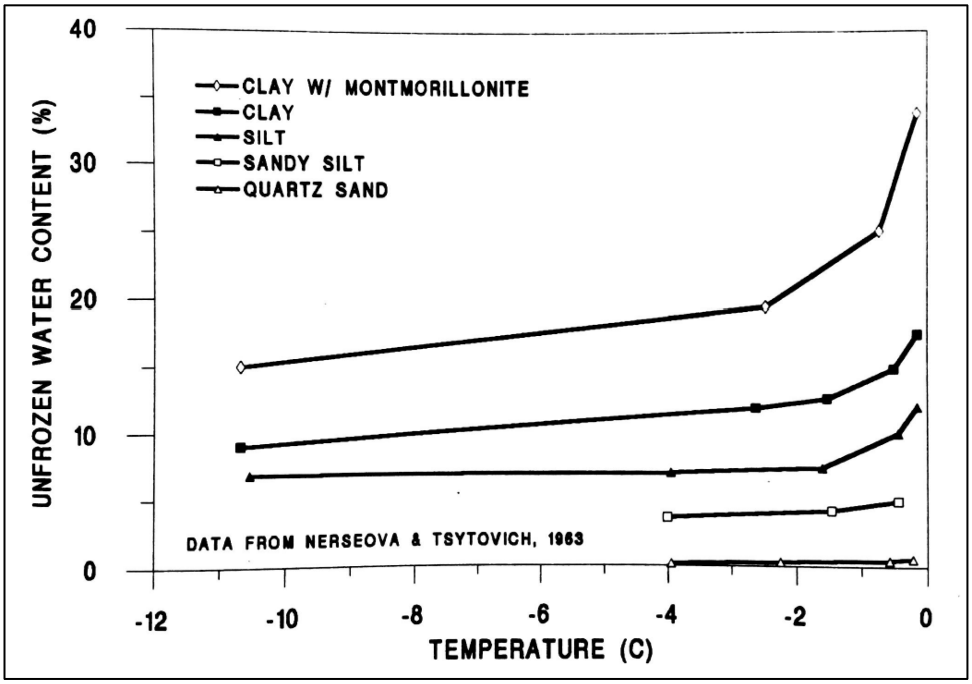


Figure 25 Unfrozen Water Content in Frozen Soils (from Coutts, 1991)

Inside soil pores, water and ice coexist in thermodynamic equilibrium with ice occupying the central portion of pore spaces, and with liquid water surrounding individual soil grains and within the pore throats (Figure 24). The curved water/ice interfaces in the soil pores are particularly important as they cause a difference in pressure to occur between the ice and water phases. As a result of this pressure differential, the pressure in the free water is lower than that of the ice. This same capillary phenomenon occurs in a capillary tube where water pressure is less than atmospheric pressure causing water to be drawn upward against gravity.

In a soil undergoing freezing, (for example when a frost front is migrating downward during winter freezing), the suction from negative pore water pressure at the frost front will draw water toward the frost front, and in fine-grained soils this can cause discrete ice lenses to form, (also known as segregated ice). Frost heave is the result of volumetric expansion of the soil from the formation of segregated ice.

Frost heave (specifically, the formation of segregated ice lenses) does not occur in coarse-grained soils because the pore ice fills the majority of the pore volume, resulting in very low hydraulic conductivity and very high resistance to water flow at the frost front, thereby thwarting the development of segregated ice lenses.

From the very brief description of the mechanism for frost heave in soils above, it can be summarized that three necessary conditions are required for frost heave to occur:

- subfreezing soil temperatures
- fine-grained soil
- groundwater available to migrate to the frost front

If any of these three conditions are not met, frost heave cannot occur. In light of the above, zero to minimal frost heave occurs in sand and gravel soils, and frost heave does not occur in even in fine-grained soils where the water table is located deep below the ground surface.

Depth to the water table along a pipeline route can be uncertain, and therefore it is conservative to assume that frost heave can occur wherever fine-grained soils occur along the route. Terrain analysis can be used to identify or to infer terrain landforms that may be fine-grained, and thus where frost heave is susceptible, along a particular route.

5.2.3 Segregation Potential Model of Frost Heave in Soils

Currently, the best method for quantifying soil frost heave susceptibility for use on engineering projects is the segregation potential (SP) model for frost heave in soils (Konrad and Morgenstern, 1981; and Konrad and Morgenstern, 1984). Through laboratory testing of frost heave susceptible soils, Konrad and Morgenstern (1981) observed that the rate of frost heave for a particular soil under a specific load (i.e., the soil effective stress condition) depends linearly on the thermal gradient in the frozen fringe.

As shown on Figure 26, the frozen fringe is a small zone at the frost front where soil temperatures range from the ice initiation temperature in the soil (T_i, typically 0°C) to an ice lens located on the cold side of the frost front (at the segregated ice lens temperature, T_s, typically -0.1°C to -0.2°C, depending on the soil effective stress). By measuring the frost heave rate of a soil undergoing freezing and the temperature gradient through the frozen fringe during a laboratory frost heave test, the soil SP is determined as a proportionality constant relating the heave rate to the temperature gradient in the frozen fringe during the test.

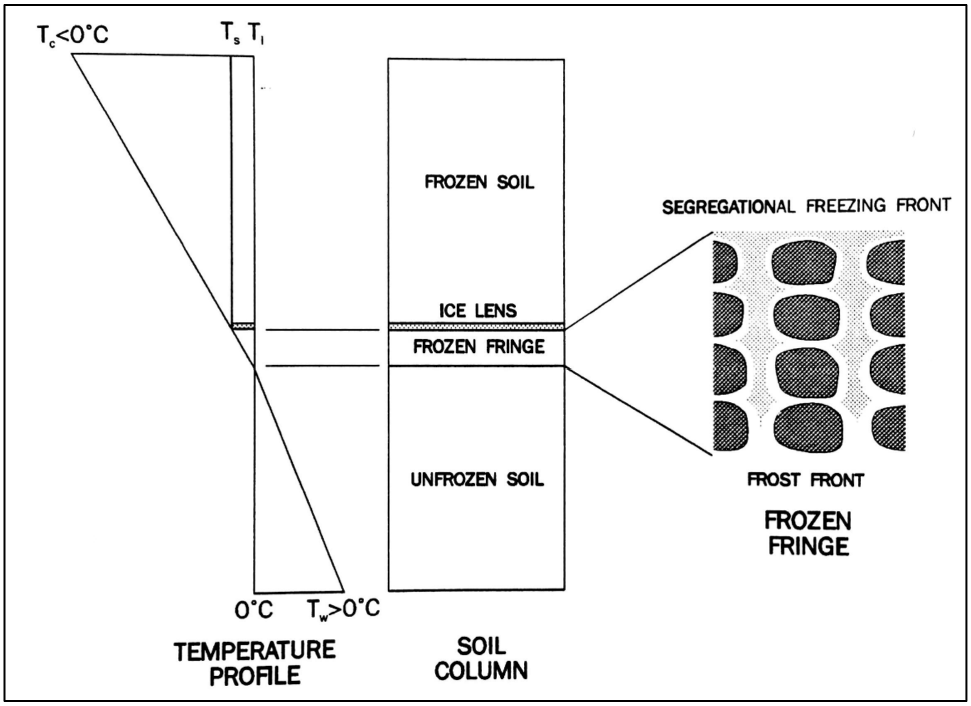


Figure 26 Frozen Fringe in Freezing Soil (from Coutts, 1991)

Once SP is known for a particular soil, it can be used to determine the frost heave rate in the soil at any point in time as follows:

$$\dot{h} = 1.09 \cdot SP \cdot \nabla T_{ff} \tag{1}$$

where: $\dot{h}$ = rate of frost heave
SP = segregation potential of the soil
∇T_{ff} = temperature gradient at the frost front
1.09 = volumetric expansion of water upon freezing

The rate of frost heave in soils is observed to decrease as soil stress (loading) increases. As such, SP decreases exponentially with increasing pressure (effective stress), and is expressed as:

$$SP = SP_0 \cdot \exp(-a \cdot \sigma') \tag{2}$$

Where:

σ' = effective stress (overburden load minus pore pressure)
SP_0 and *a* = intercept and slope of SP versus pressure on a semi-log plot

Figure 27 schematically shows the SP as a function of applied pressure from Equation 2 plotted arithmetically.

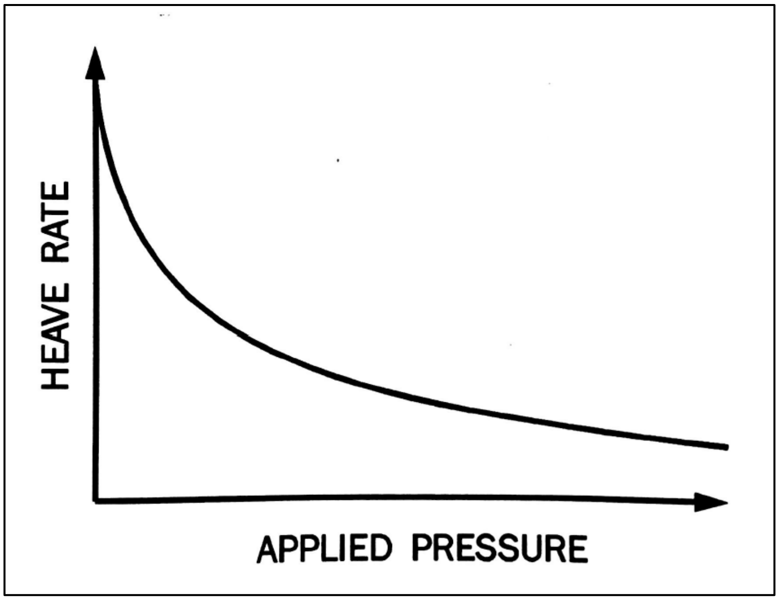


Figure 27 Effect of Applied Pressure on Frost Heave Rate

5.2.4 Frost Heave of Chilled Pipelines

Arctic gas pipelines are designed to minimize the potential for permafrost thaw caused by the pipe. Accordingly, these pipelines are designed to operate at subfreezing temperatures in continuous and discontinuous permafrost regions.

In discontinuous permafrost regions where a chilled gas pipeline crosses spans of unfrozen soil, a frost bulb will develop around the pipe, and frost heave can occur. The SP model for frost heave in soils described earlier is used to predict the frost heave of the pipeline under these conditions.

It is important to understand that a chilled gas pipeline is a thermally active system, meaning that the chilled pipe extracts heat from the soil and directly causes frost heave at the base of the frost bulb, vertically lifting both the frost bulb and the pipeline it contains. In contrast, an abandoned pipeline is a thermally passive system that does not develop a frost bulb by heat extraction from the soil by the pipe, and as such, an abandoned pipeline cannot cause frost heave.

5.2.5 Frost Jacking

It is important to distinguish between frost heave and frost jacking. Frost jacking refers to the non-recoverable vertical displacement of a structure relative to the soil at the base of the structure, caused by frost heave of the soil layer adjacent to the structure which freezes each winter. Non-recoverable vertical displacement refers to the upward displacement that occurs during freezing which is not fully offset by the downward displacement during thawing.

Cumulative non-recoverable vertical displacement from seasonal freeze-thaw cycles over a period of many years can cause significant upward displacement of structures. In fact, over several winters, frost jacking can lift a structure out of the ground. Examples of frost jacking on a concrete pile and on concrete post footing are shown on Figure 28 and Figure 29.

Figure 28 Frost Jacking of a Concrete Footing Foundation (Woodgears, 2018)

Figure 29 Frost Jacking of Posts (Woodgears, 2018)

Frost jacking of a structure embedded in soil susceptible to frost heave would occur during winter when the frost front migrates downward through the soil. The frozen soil adheres to the structure and as it heaves, it imparts an upward force on the structure. As the structure lifts, a void can develop between the bottom of the structure and the soil below. If there is inadequate downward resistive force to counteract the upward force exerted by the heaving soil, or if some soil sloughs into the void below the structure, the structure will experience a permanent non-recoverable vertical displacement.

Figure 30 summarizes the mechanism for frost jacking of a post, which can be considered generically as a vertically oriented cylindrical structure. The first panel on the figure shows a newly installed post where the surrounding soil has not undergone frost heave. During the first winter, soil frost heave lifts the structure upward by an amount H, and creates a void space below the structure, as shown on the second panel. By the following summer, after the soil has completely thawed, the structure does not settle vertically by the same amount that it was lifted, because the structure is light enough that it will not settle to its original position, or because some soil has filled the void space below the structure, or a combination of both of these mechanisms. In any event, the structure experiences a non-recoverable vertical displacement of Δh, as shown on the third panel. After a number of years, the cumulative non-recoverable vertical displacements lift the structure out of the ground as shown on the last panel.

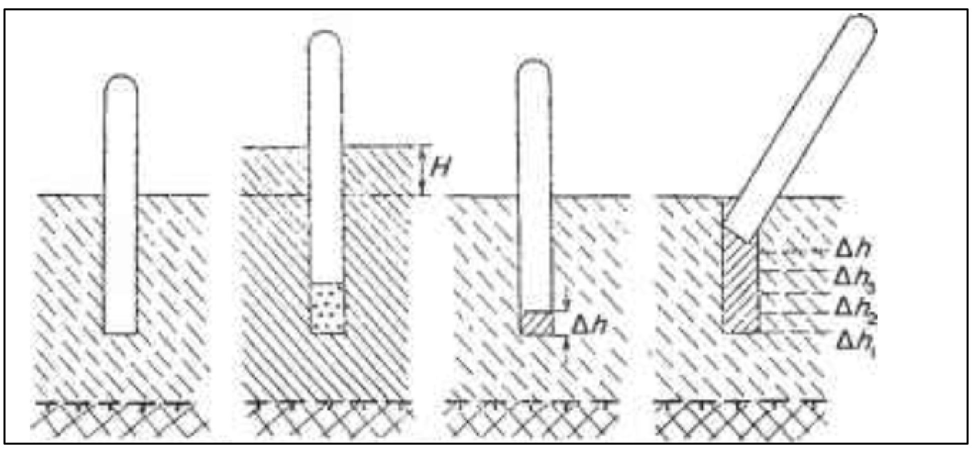


Figure 30 Frost Jacking Mechanism

For frost jacking to vertically displace a structure embedded in frost heave susceptible soil, it is necessary for a vertical force to develop between the heaving soil and the structure itself. For a structure such as a vertical pile or post, the shear force develops around the circumference of the vertically oriented cylindrical structure as the frozen soil adheres to the structure and lifts it vertically.

For frost jacking to vertically displace a structure embedded in frost heave susceptible soil, it is necessary for a vertical force to develop between the heaving soil and the structure itself. For a structure such as a vertical pile or post, the shear force develops around the circumference of the vertically oriented cylindrical structure as the frozen soil adheres to the structure and lifts it vertically.

In contrast, an abandoned pipeline is a horizontally oriented cylindrical structure located at some depth below the ground surface. For an abandoned pipeline, frost jacking cannot occur before the frost front penetrates to the top of the pipe. At the point in time when the frost is at the top of the pipe, to jack the pipe, it would be necessary for the frozen soil to adhere to the top of the pipe and lift it upward and out of the unfrozen soil below. Intuitively, this seems highly unlikely to happen before the frost front reaches the springline of the pipe. After the frost front gets to a depth beneath the pipe springline, it seems plausible that the pipe may be lifted vertically by frost heave in the surrounding soil; however clear evidence of such frost jacking of pipelines has not been reported.

One reference to pipeline frost jacking was found regarding a 240 km fuel gas pipeline which supplies natural gas to turbines in the northernmost four pump stations of the TransAlaska Pipeline System (TAPS), (Johnson and Hegdal, 2008). The gas line was 10" (0.25 m) diameter for the first 55 km and then reduced to 8" (0.20 m) diameter for the remaining 183 km with a burial depth of 1 m (36"). In the paper it is mentioned that the fuel gas line was buried adjacent to the Dalton Highway and had experienced increased surface heat exchange caused by construction disturbance and dust from the highway. Although the inlet temperature of the pipe was typically -2.0°C, temperature excursions of up to +20°C had occurred. The paper states that deep permafrost thaw had occurred in some places along the pipeline and that "*this has resulted in heaving of the line due to thermal expansion, frost jacking during freeze-back and loss of cover over the pipe*." No evidence for the statement regarding frost jacking of the pipe was provided. The effect of pipe uplift from pipe buckling has been observed in other arctic pipelines when warm pipe temperatures and relatively weak moisture-saturated soils occur as a result of permafrost thaw. It is therefore unknown from the paper of Johnson and Hegdal (2008) if frost jacking actually occurred on the gas fuel line, or if the possibility of it was strictly speculation.

Frost jacking of an abandoned pipeline seems intuitively unlikely because it is reasonable to expect that the maximum uplift force on the pipe would be smaller than the force resisting the pipe from upward displacement. This is based on the geometry of a horizontal cylinder (the pipe) relative to the frost direction (vertical upward lifting).

The DNV report (DNV 2010) when referring to "recent findings" regarding in-place pipeline abandonment in sensitive ecological areas stated "*removal may be the best option in northern areas where soil, groundwater and temperature conditions encourage extensive frost heaving, potentially resulting in surface exposure of the pipeline (Mackay et al, 1979)*". Review of Mackay and Burrous 1979 paper reveals that the subject matter of the paper concerns vertical displacement of solid objects by an upfreezing surface in continuous permafrost. Upfreezing occurs at the base of the active layer (the maximum seasonal thaw depth) in permafrost when the ground begins to cool each fall, when "*upward freezing of the permafrost table can accompany downward freezing from the ground surface during the freeze-back period*," (Mackay and Burrous, 1979). It is not valid to apply the observations of upward displacement of solids as a result of freeze-back of the active layer in continuous permafrost to the mechanism of pipeline frost jacking in non-permafrost soils as they not the same physical mechanism.

When considering the mechanism of pipeline frost jacking (in non-permafrost soils), it can be instructive to consider four time periods defined by the location of the frost front relative to the pipe each winter season:

- **Frost Front above Top of Pipe:** Before the frost front reaches the top of the pipe, there will be no frost heave in the soils adjacent to the pipe and there will be no uplift force on the pipe from frost heave. Therefore the pipe cannot be displaced vertically before the frost front reaches the top of the pipe.

- **Frost Front between Top and Springline of Pipe:** During the time from when the frost front reaches the top of the pipe to when the frost front reaches the pipe springline (i.e., vertically half the pipe diameter from the top of the pipe), the only upward force that can develop on the pipe would be from freeze bonding of the soil to the pipe. During this time period, the uplift force would be a tensile force on the pipe, not a shear force, and is expected to be a relatively small force compared to the force resisting upward displacement of the pipe. The minimum force resisting upward pipe movement would include the weight of the pipe itself, the weight of the soil associated with an area adjacent to the pipe from the springline of the pipe upward to the frost front, and the unfrozen soil shear resistance along the two shear planes upward from the pipe springline (at the 9 and 3 o'clock positions on the pipe). In fact, vertical displacement of the pipe may not be kinematically possible before the frost front penetrates below the pipe springline.

- **Frost Front between Springline and Bottom of Pipe:** During the time period when the frost front penetrates from the pipe springline to the bottom of the pipe, the upward frost heave force would develop on the perimeter of the pipe between the frost front depth and the pipe springline depth. The upward force would consist of compressive and shear force components. The force resisting upward movement of the pipe would consist of the weight of the soil above the pipe (area of soil calculated by multiplying pipe diameter soil cover depth), the weight of the pipe itself, and shear force within relatively high strength frozen soil above the pipe. During this time period, the vertical pipe displacement could create a void space below the pipe. The void space would be largest at the base of the pipe and would decrease to zero at some point along the bottom perimeter of the pipe toward the springline. It is important to note that for large diameter pipes, the maximum frost depth may not penetrate below the springline of the pipe. The likelihood of the frost front penetrating below the pipe springline is greater for smaller diameter pipes.

- **Frost Front below Bottom of Pipe:** If the frost front penetrates below the bottom of the pipe, frost heave would lift the soil and the pipe together, but there would be no differential vertical displacement of the pipe relative to the soil from frost heave during this time period.

5.3 Soil Temperatures near Abandoned Pipelines

The location of the frost front relative to a buried pipeline governs if and when frost jacking of the pipe can occur. Numerical modelling was used to demonstrate the soil temperatures near an abandoned pipeline using TEMP/W (Geo-Slope, 2018). The modelled case considered a 12” (0.30 m) diameter pipe buried in a fine-grained soil with a cover depth of 36” (0.91 m). The ground surface boundary condition accounted for energy flux components from solar radiation, longwave radiation, convective heat transfer with the atmosphere, and evapotranspiration (Hwang, 1976). The ground surface boundary condition considered a climate representative of northern Alberta (Fort McMurray 30-year climate normals, Environment Canada 2018) mean monthly values for air temperature, wind speed, solar radiation and snow depth. For any particular day of the year, these climate values were obtained by interpolation between the mean monthly values. The model simulation was run to periodic steady-state, which means that at any spatial location in the model, the temperature varies on an annual basis but is identical at the same point in time each year. The objective of the model was to observe the penetration of the frost front relative to the pipe, which is informative when considering frost jacking of abandoned pipelines.

Figure 31 shows the model-computed temperatures and frost front location near the example abandoned pipeline case representative of the fourth week of January. It is noted that the frost front away from the pipe reaches a depth of 1.11 m and adjacent to the pipe the frost front depth is 1.18 m, the difference of 0.07 m being caused by the heat transfer in the vicinity of the pipe and through the steel wall of the pipe which has significantly higher thermal conductivity than the surrounding soil.

Figure 31 also illustrates the frost front at a time when it is located between the springline and the bottom of the pipe, which as was discussed earlier, is the only time period when the pipe could possibly be vertically lifted by frost heave from the unfrozen soil below the pipe. Note that the resistance to pipe uplift during this time period is substantial, as it would require deformation of frozen soil above the pipe, and as such, it would seem unlikely that this could occur.

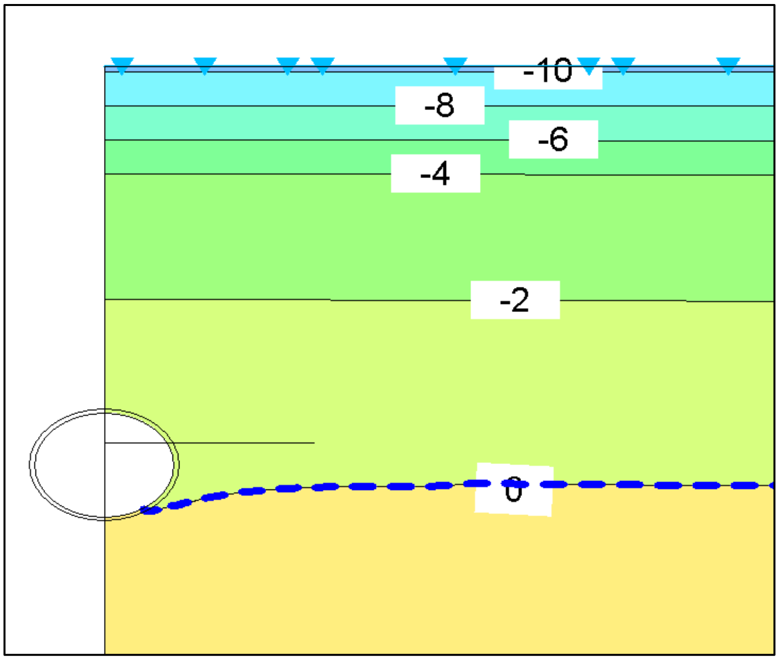


Figure 31 Temperatures and Frost Front from Abandoned Pipeline Example (4th Week of January)

5.4 Methodologies for Assessing Abandoned Pipeline Exposure from Frost Jacking

Although it seems unlikely that frost jacking could expose an abandoned pipeline, methods are presented here to measure and observe the potential for frost jacking in a laboratory or field setting, as was requested by PTAC (PTAC, 2017). These methods are not intended as recommendations for future work, but instead are intended to suggest the scope of such investigations if they were ever deemed necessary to obtain further evidence regarding frost action on abandoned pipelines. Note that frost jacking is not readily amenable to desktop assessment because of the uncertainties associated with the multiple interacting physical mechanisms which cause frost jacking of buried structures (i.e., heat transfer, soil frost heave, soil-structure friction, and soil sloughing into void space).

Culverts have been discussed as a potential proxy for pipeline frost jacking (DNV, 2010; Stantec, 2014), but culverts are a poor proxy for the investigation of pipeline frost jacking. Unlike buried abandoned pipelines, culverts are exposed to atmospheric temperatures, and in winter, cold temperatures inside a culvert will cause freezing in the adjacent soils around the culvert, and could possibly induce frost heave at the base of the resulting frost annulus around the pipe, analogous to frost heave around chilled gas pipelines in discontinuous permafrost as discussed earlier. For this reason, culverts should not be considered as a proxy to study frost jacking along abandoned pipelines.

5.4.1 Laboratory Methods

Background and Objectives

The objective of a frost jacking laboratory test would be to determine the degree to which pipeline frost jacking could occur, if at all, as a result of multiple freeze-thaw cycles. The main advantage of laboratory testing over field observations is the testing control which can be applied in the laboratory environment. This includes but is not limited to control of soil type, pipe diameter, burial depth, temperature conditions, and the number of freeze-thaw cycles.

Laboratory testing of pipe uplift resistance loads in frozen soils have been completed on multiple arctic gas pipeline projects. For these tests, a pipe is buried into a homogenized natural soil contained in a soil box and then frozen. The entire testing apparatus is housed in a cold room at sub-freezing temperature for the duration of the test which can last 30 days or more. During the test, the pipe is then displaced vertically at a slow rate while the uplift loading on the pipe is measured. The test produces a pipe load versus displacement function which is subsequently used for structural analysis of pipe strains caused by frost heave.

A similar test apparatus to that used for pipe uplift resistance testing could be used for pipe frost jacking. Pipe diameters for pipe uplift resistance testing varied from nominally 4” to 12” (0.10 m to 0.30 m). Soil boxes for these tests ranged in size from about 1.5 x 1.0 x 1.0 m, to 2.0 x 1.0 x 1.3 m (horizontal length x horizontal width x vertical height). For pipe diameters larger than 12” (0.30 m), larger soil boxes may be necessary, depending on the ability for the laboratory results of relatively small diameter pipes to be scaled to larger pipe diameters.

Natural fine-grained soils (clayey silt or silty-clay) which are frost heave susceptible and water saturated would be used for laboratory pipeline frost jacking tests.

Test Apparatus Fabrication

One or more soil boxes would need to be fabricated for laboratory frost jacking testing. The soil box would need to be insulated on all sides except the top. The box would need to be impermeable to water on all vertical sides, but water access along the bottom of the box would be necessary to supply water to the soil undergoing frost heave. The soil surface would need to be protected from moisture loss via desiccation during the test.

Temperature Cycling

The entire test apparatus would need to be located in a cold room which would be temperature-cycled. Each freezing temperature cycle would simulate a winter season, and therefore subfreezing temperatures would need to be maintained long enough for the frost front penetration from the soil surface to reach a (scaled) depth similar to that which would be experienced by a real pipe in a relatively cold climate such as in northern Alberta. Several freezing cycles would likely be required to observe any non-recoverable upward pipe displacement (frost jacking displacement), which may require tests of 30 days or more.

Measurements and Instrumentation

During a laboratory pipeline frost jacking test, several data measurements would be required, the first and foremost of these being the vertical displacement of the pipe which can be measured with linear variable displacement transducers (LVDTs). Displacement of the soil surface at various locations would be used to track the soil frost heave.

Temperature measurements vertically adjacent to the pipe during the test would be required to monitor the frost penetration during each temperature cycle. Other temperature measurements would be required for quality control purposes, such as at other locations within the soil and in the cold room, would be necessary as well. Temperatures in laboratory tests such as these are typically measured using thermistors.

A water reservoir feeding into the base of the soil box would be used to provide water for soil frost heave. Monitoring the water volume intake and expulsion from soil during frost heave would be useful to cross-check the soil surface frost heave monitored by LVDTs. Water volume changes can be monitored electronically using a digital mass scale.

Measurements of temperature, displacement and water volumes would be recorded on a set frequency (likely every 2 to 15 minutes) using a data acquisition system.

Test Design and Development

It is important to recognize that laboratory testing of pipeline frost jacking is essentially a research and development activity requiring some initial desktop design of the testing apparatus followed by iterative test-to-test trial-and-error development of the test apparatus in the laboratory. However, the design and development of the testing apparatus should be of relatively low to moderate technical risk as the principles regarding what is required for a successful test are well understood by experienced practitioners. The uncertainty in the testing derives more from what will work practically in a laboratory testing environment.

The first step for undertaking laboratory testing would be a preliminary design of a frost jacking soil test box. This would require first considering heat transfer to ensure that a test box can be suitably designed to simulate vertical freezing from the ground surface that would occur in field conditions. Heat losses from the box sides and bottom, and heat transfer horizontally along the pipe itself are of particular importance. Other design issues of importance include the friction between the soil and the box sides during frost heave (teflon-like interior side materials may be of use here) and visibility of the pipe ends so that pipe heave and any void below the pipe can be observed directly. The system to supply water to the soil at the bottom of the box would require design effort as well.

Bulk Soil Sample

To undertake laboratory pipeline frost jacking tests, a frost heave susceptible soil of sufficient quantity to fill one or more test boxes would be required. The entire quantity of soil would be sieved to remove any stones and retain the fine-grained silt and clay.

A well-known frost heave susceptible soil used in previous frost heave studies is Calgary silty-clay. Previous in-situ testing of pipeline frost heave at the Calgary Frost Heave Test near the north side of the University of Calgary was performed in Calgary silty-clay Facility (Carlson and Nixon, 1988). Since then, this soil has become a "reference" soil in laboratory frost heave testing programs.

Obtaining a sufficient quantity of Calgary silty-clay will require some effort to find a suitable source location in the Calgary area and may or may not require permitting for removal.

Once a suitable quantity of soil has been acquired, one or two frost heave tests should be performed on the soil to determine the SP of the sieved and homogenized soil sample to fully quantify its frost heave susceptibility.

5.4.2 Field Methods

The objective of a pipeline frost jacking field study would be to monitor vertical pipe displacement over at least three winter seasons, (ideally several more), to observe for any frost jacking of the pipe. Very small changes in seasonal pipe displacement would need to be observed, on the order of a few millimeters of vertical movement annually.

One or more study sites would be required to investigate pipeline frost jacking in the field. The ideal site would have the following characteristics:

- An operational but soon to be inactive pipeline or a pipeline that had recently been abandoned.
- Easy access by road, within 50 km from a populated area.
- Fine-grained soils with a relatively high water table at or just below the base of the pipe (such that buoyancy controls measures were not installed).
- An open, winter wind-swept, right-of-way (providing minimal winter snow cover).
- A relatively cold winter climate such as northern Alberta.

Locating such candidate sites would require working with pipeline owner companies together with available spatial data.

Excavation (likely with hydrovac) to top of pipe and backfilling would be required in several locations to install vertical survey points accessible at the ground surface. Care would need to be taken to ensure that the excavation disturbance was limited to minimize disturbance of the soil properties and overall annual heat transfer at the ground surface. The distance between the vertical survey posts would need to be based on site-specific considerations, but the spacing could be on the order of 7 m to 15 m.

At each survey location, an 8-10 m deep borehole should be drilled at a distance of 2-3 m perpendicular from the pipe. The borehole would be used to log the soil profile and to permanently install a thermistor string (or potentially a continuous fiber optic temperature measurement device) to measure the ground temperature variations with depth throughout the year. A thermistor string could also be placed on the vertical survey post attached to the pipe to obtain temperature in the soil directly above the pipe and at the top of the pipe itself.

A series of 3 to 5 slotted standpipe piezometers should also be installed across the site to characterize and monitor the groundwater flow at the site. Data loggers could be used to provide continuous readings of the thermistors and potentially the groundwater levels on an appropriate reading frequency.

Ideally, a continuous displacement monitoring technology (likely optically-based) would be employed to provide pipe displacement data. Alternatively, manual surveys could be used to observe the vertical pipe displacement. More frequent measurements increase the likelihood of isolating the timing and conditions for the pipe displacements.

The survey locations would need to be secured from public access for protection of both the public and the installed monitoring equipment. Any fencing around the test area would need to be suitably far from the pipeline (such as at the edge of the ROW) to avoid snow drifting caused by the fencing from extending over the pipeline, as this would increase the snow depth over the pipe, and reduce the natural heat extraction from the ground during winter.

Combined monitoring of pipe displacement and ground temperatures over a few to several seasons, at one or multiple sites, would provide direct evidence regarding pipe frost jacking under site-specific conditions, if it occurs at all.

5.5 Review of Previous PTAC Frost Heave Report

The previous report prepared for PTAC PARSC 003 (Stantec, 2014), regarding the exposure risk to abandoned pipelines caused by frost action in soils, was reviewed as part of this study. The following review comments focus on the conclusions and recommendations presented in that report.

The PARSC 003 report states that "*the literature searches confirmed that the conclusion drawn by DNV in 2010 remains true (i.e. there is no published information on the risk of abandoned pipeline exposure attributable to frost heave)*". The report also stated that "*a review of the broader literature on the frost heave process does, however, suggest that frost heave could pose an exposure risk to abandoned pipelines under certain geoclimatic conditions*". It was not specified under which specific geoclimatic conditions that an exposure risk from frost action would occur, but given the information provided in this report (i.e., physical mechanisms for frost action in soils and heat transfer around a buried pipe), together with the absence of observations of abandoned pipeline exposure from frost action, it seems unlikely that certain geoclimatic conditions could lead to frost jacking of an abandoned pipeline.

The PARSC 003 report also concludes that "*there are at least four factors that may cause preferential formation of ice lenses around an abandoned pipeline, and particularly immediately below that pipeline*". The four factors are examined below:

- First, it was stated that "*the steel in the pipeline wall is a much stronger thermal conductor than the surrounding moist soil. It is known that the two most essential driving forces leading to heave and heaving pressures in soils are the water intake rate (or the upward Darcy flux), and the heat extraction rate at the soil surface*". Both of these statements are true, but neither of these statements addresses how this relates to upward displacement of a buried pipe from frost action. Within the report, it is surmised that high thermal conductivity of a steel pipe will cause the frost front to rapidly extend to the base of the pipe. Thermal modelling presented in this report has shown that the unfrozen soil surrounding a pipe will limit the vertical extent of sub-freezing temperatures in the steel pipe.

- Second, it was stated that "*if the pipeline is positioned above the local groundwater table, there will be more 'free water' available at the bottom of the pipeline for ice lens formation than at positions closer to the soil surface, and the unsaturated hydraulic conductivity will be higher. The Darcy (water) flux, driven by the temperature gradient, is the major contributor to ice lens formation and frost heave in soils*". Again, both of these statements are true, but it is unclear how this relates to pipe displacement through the soil from frost action. Even if the water table was above the pipe, the previous report does not discuss how the pipe could displace relative to the surrounding soils, except by incorrectly assuming frost penetrates through the steel to the base of the pipe much sooner than occurs in the soil adjacent to the pipe.

- The third statement was "*Ice segregation in a frozen soil matrix (including ice lens formation) preferentially occurs in soil zones where overburden pressures are lower. The overburden pressure directly beneath an abandoned pipeline (i.e., air- or N2-filled [ambient]) is significantly lower than in adjacent soil areas beyond the pipeline trench (i.e., at the same depth). The mass of an empty pipeline segment is significantly lower than the wet bulk density of the soil that it displaces, particularly for larger diameter pipelines*". The overburden (vertical) soil pressure at any depth is estimated based on the weight of soil above. At the very base of a buried pipe, the vertical pressure will be somewhat less than in the adjacent soil because of the soil volume occupied by the presence of the pipe. For example, for a pipe buried with a cover depth equal to the pipe diameter, the vertical soil pressure directly under the pipe (ignoring the weight of the steel pipe) will be nominally half of the vertical pressure of the soil adjacent to the pipe. Moving horizontally from the bottom center of the pipe toward the adjacent soil, the vertical soil pressure increases and becomes equal to that in the adjacent soil within a distance equal to the pipe radius. In this case, the average vertical soil pressure beneath the pipe will be nominally 75% of the vertical soil pressure in the adjacent soil. This percentage will increase with increasing pipe burial depth. However, the importance of such relatively small vertical soil pressure differences could only be important if the frost front were to advance to the bottom of the pipe significantly faster than through the soil, which does not occur. Therefore differences in the vertical soil pressure below the pipe as stated in the previous PTAC report are unimportant with regards to pipe exposure from frost action.

- The fourth statement was "*if ice lenses form incidentally a short distance away from the pipeline, they may relocate to the area beneath the pipeline (where overburden pressures are lower) through the process of 'regelation'* ". Regelation is the process whereby ice melts under pressure and the resulting meltwater flows to another area in the system and refreezes. A fine wire hung by weights over the top of an ice block is often cited as an example of regelation. Melting occurs on the bottom side of the wire, the meltwater flows around the wire and refreezes on the top side of the wire, and eventually the wire passes completely through the block of ice. However, the pressure required to melt ice is extremely high, (on the order of tens of MPa), and even in the case with the wire and ice block, the force applied on the wire may not exceed the pressure required to induce regelation. It seems therefore extremely unlikely, perhaps even physically impossible, for the relatively small difference in vertical pressure between soil adjacent to a buried pipe and that directly below the pipe, to cause any regelation and redistribution of ice in frozen soil near a buried pipe. In instances where regelation does occur, it is an extremely slow process, and would therefore be negligible over the brief time period of one winter season.

The PARSC 003 report also concludes: "*Abundant ice lens growth directly beneath the pipeline could potentially jack the abandoned pipeline toward the surface over time*". This statement by itself is true, however no substantial evidence was provided in the previous PTAC report regarding how abundant ice lens growth beneath an abandoned pipeline could occur.

Four recommendations were made in PTAC PARSC 003 report. The recommendations and review comments for each are as follows:

- "*Recommendation #1: It is recommended that PARSC/PTAC should bypass the proposed 'Stage 2' (Laboratory soil column freezing tests) of this multi-stage investigation of frost heave risk to abandoned pipelines, and proceed directly to 'Stage 3' (Field measurements and observations)*." We agree with the recommendation that laboratory soil column freezing tests are not necessary as there is adequate published information regarding the frost heave susceptibility of various soil types, and certainly enough information in this regard to estimate quantitative soil properties such as segregation potential for any site-specific analysis. Proceeding directly to field measurements and observations may be premature, unless there is a requirement to demonstrate definitively that there is little to no frost jacking of existing or newly abandoned pipelines. As an alternative, pipeline operators could observe a select set of abandoned pipeline sections in locations where the pipe is buried in wet fine-grained soils, perhaps on an annual basis or longer frequency, to note the absence or occurrence of pipe exposure. Some such long-term visual observations could serve as evidence regarding the occurrence or absence of pipeline exposure caused by frost jacking.

- "*Recommendation #2: It is recommended that PARSC/PTAC put resources into fully developing the Groenevelt_HI1.0 model (largely conceptual at present), which is based on sound thermodynamic principles and has little dependency on PTFs (i.e., would likely only require a PTF to estimate saturated and unsaturated hydraulic conductivity), unlike the Konrad_SP1.0 model*." Implementing this recommendation is unlikely to provide value given that the segregation potential (SP) model for frost heave in soils already exists and is used in geotechnical engineering practice, including in pipeline design. This recommendation also implies that the SP model (Konrad and Morgenstern, 1981; and Konrad and Morgenstern, 1984) is not based on sound thermodynamic principles, which is false. In fact, the SP model has proven to be an excellent tool for engineering design as it strikes a balance between theoretical rigour and the availability of soils data during typical geotechnical and pipeline projects. It is doubtful that developing a new model for frost heave would decrease any business risk to engineering projects beyond what the SP model already delivers.

- "*Recommendation #3: It is recommended that efforts be continued to either i) locate a non-Fortran version of the SHAW 1D model (by contacting researchers who have published on SHAW 1D in the last decade, including its creator Dr. Flerchinger), or ii) initiate a new attempt to re-write the Fortran program code into the Python programming language*." The review comments above regarding Recommendation #2 apply to this recommendation as well.

- "*Recommendation #4: It is recommended that a more in-depth study be carried out on the pipeline segment (diameter and length) matter from a 'pipeline design engineer' perspective*." The "matter" referred to in this recommendation regards consideration of the length and diameter of a pipeline segment to vertical displacement and surficial exposure via frost action. The report discusses some pipe/soil stress interaction concepts, in particular the concepts of soil springs and virtual anchor length. The report is unclear exactly how the virtual anchor approach would apply in a case where differential vertical pipe displacement were to occur over a pipeline segment of limited length. Without a well-considered definition of the problem, it is unclear how the application of a virtual anchor length would be used in an analysis.

In summary, PTAC PARSC 003 report implied that frost action could expose an abandoned pipeline. However, the information provided in this report chapter clarifies that although it may be theoretically possible for frost action to expose an abandoned pipeline, it would be highly unusual and not reasonably likely.

5.6 CONCLUSIONS

This chapter provided technical background regarding frost action in soils, which included a description of seasonal ground temperature variation and maximum winter frost depth, the physical mechanisms which cause frost heave in soils, the distinction between frost heave and frost jacking, the segregation potential model for frost heave in soils, and example numerical modelling results showing the frost front relative to an abandoned pipeline.

Frost jacking is a well understood process caused by frost heave which results in a permanent non-recoverable vertical displacement of buried structures, in particular vertically oriented structures such as pile and post foundations. Based on consideration of a frost front penetrating downward from the ground surface during winter, frost jacking forces can only affect a buried pipe during the time period when the frost front intersects the pipe between the pipe springline and the bottom of the pipe. During this limited time period however, upward displacement of the pipe relative to the adjacent soils can only occur if the upward force on the pipe from frost heave exceeds the pipe uplift resistance provided by the frozen soil above the pipe. For the example case of a 12” (0.30 m) diameter pipe buried with a cover depth of 36” (0.91 m), the frost front at the pipe extended only 0.07 m below the frost depth in the soil adjacent to the pipe. Therefore it seems unlikely that winter frost penetration from the ground surface could induce frost jacking (characterized as upward vertical displacement of the pipe relative to adjacent soil) on an abandoned pipe.

Although frost jacking of an abandoned pipe is unlikely, a scope for laboratory and field investigations of the potential for abandoned pipeline frost jacking was provided in this report as per the requirements of the project request for proposal (PTAC, 2017). These investigations are not intended as recommendations for future work, but instead were provided to suggest a project scope if such investigations were ever deemed necessary to obtain further evidence regarding frost action on abandoned pipelines.

Finally, review comments were provided for the previous PTAC PARSC 003 report (Stantec, 2014) regarding the exposure risk of abandoned pipelines caused by frost action in soils.

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