Docket No. CP80-

(DESIGN MANUAL)

Exhibit Z-9.1
Pipeline and Civil Design

Volume VII

ALASKA SEGMENT

ALASKA NATURAL GAS TRANSPORTATION SYSTEM

Alaskan Northwest Natural Gas
Transportation Company

United States of America
Before the
Federal Energy Regulatory Commission

Docket No. CP80-

(DESIGN MANUAL)

Exhibit Z-9.1
Pipeline and Civil Design

Volume VII

Application of
ALASKAN NORTHWEST NATURAL GAS TRANSPORTATION COMPANY

For a Final Certificate of Public Convenience and Necessity Pursuant to Section 7 (C) of the Natural Gas Act, as amended, and Section 9 of the Alaska Natural Gas Transportation Act of 1976 to construct and operate the Alaska Segment of the Alaska Natural Gas Transportation System.

July 1, 1980

EXHIBIT Z-9.1

PIPELINE AND CIVIL DESIGN

ALASKAN NORTHWEST NATURAL GAS TRANSPORTATION COMPANY

ALASKA SEGMENT OF THE
ALASKA NATURAL GAS TRANSPORTATION SYSTEM

Docket No. CP80
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PIPELINE AND CIVIL DESIGN

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3.0	Frost Heave Design
4.0	Geotechnical Considerations
5.0	Hydrological Considerations

INTRODUCTION

This Exhibit provides the design basis and description of the pipeline and civil work associated with the pipeline on the Alaska Segment of the Alaska Natural Gas Transportation System.

The Exhibit has been divided into 5 Sections as follows:

Section 1.0 deals with the design criteria and design related to the pipeline route, mainline pipe and appurtenances attached to the mainline; pipeline ditch configuration; pipe stress analysis.

Section 2.0 deals with the design of civil works relating to the pipeline and pipeline construction zone.

Section 3.0 is a discussion of the frost heave phenomena as related to this pipeline and of the mitigation technique to be employed in the design of the pipeline to protect it from the deleterious effects of frost heave.

Section 4.0 discusses the geotechnical considerations and inputs that have to be taken into account in the design.

Section 5.0 is a consideration of the surface and the subsurface hydrological phenomena as related to the pipeline design.

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1.0 PIPELINE DESIGN

1.1 PIPELINE ROUTE SELECTION CRITERIA AND ROUTE DESCRIPTION

The following criteria were used in the selection of the pipeline route.

- Utilize existing transportation corridors.
- Maximize use of existing facilities such as workpads, highways, access roads, airports, material sites, disposal sites and communications.
- Minimize crossing Trans Alaska Pipeline and other pipelines.
- Minimize crossing roads and highways.
- Provide year-round, all-weather access to the proposed pipeline.
- Minimum separation between the proposed gas pipeline and Trans Alaska Pipeline to be not less than 80 feet.
- Locate the pipeline downslope of TAPS or the haul road.
- Minimize cross drainage blockage.
- Avoid thaw unstable slopes as much as possible.
- Minimize traversing areas with frost susceptible soils.
- Minimize the haul distance for construction materials.
- Avoid bracketing the Prudhoe Bay Haul Road between the gas pipeline right-of-way and existing rights-of-way.
- Avoid sensitive areas; minimize adverse impacts on the environment and the socioeconomic structure of the communities in the corridor and Alaska as a whole.
- Maximize route cost effectiveness.

Route Description

For the purpose of construction, the pipeline route has been divided into six sections, details of which appear in Exhibit Z-6. A brief description of the sections and some highlights of the route are as follows:

SECTION 1 - MP 0.00 TO MP 132.02

Section 1 (Figure Z-9.1-1-1) begins at the Prudhoe Bay Metering Station in Section 11, T11N, R14E, Meridian Umiat, approximately 5 miles northeast of TAPS Pump Station No. 1. The gas pipeline joins the TAPS workpad just south of TAPS Pump Station No. 1 and parallels TAPS to MP 23.69. At this point, the pipeline picks up the Prudhoe Bay Road and parallels it to MP 98.55, except for a short diversion at MP 97 where it descends an ice rich unstable slope. At MP 98.68, the pipeline rejoins TAPS for another 10 miles, then diverts westward to bypass TAPS Pump Station No. 3 and to pass through Future Compressor Station No. 3. The pipeline rejoins TAPS at MP 115.15 and follows it to MP 124.39, from which point it parallels the Prudhoe Bay Road to the end of the section. Pipe selection in this section is based on location Class 1.

Mainline Valves Nos. 1 through 9, Compressor Station No. 2, and Future Compressor Stations No. 1 and No. 3 are included in this section. The first 115 miles of Section 1 run nearly parallel to the Sagavanirktok (Sag) River and, consequently will cross several creeks and wet areas. Streams throughout the spread are characterized by abrupt break-ups with large discharges of short duration.

Section 1 traverses parts of two major physiographic divisions: The Arctic Coastal Plain and the Arctic Mountains. The Arctic Coastal Plain includes Teshekuk Lake Section, MP 0 to MP 17; and the Whitehills Section, MP 17 to MP 62. The Arctic Mountains Province includes: the Arctic Foothills to the north, MP 62 to MP 111; and the Arctic Foothills to the south, MP 111 to MP 132.

The Arctic Coastal Plain is a smooth plain rising imperceptibly from the Arctic Ocean to 600 feet at its southern margin. It is underlain by marine sediments, mainly sands and gravels, covered by a thin mantle of fluvial and eolian gravels, sands, and silts. Permafrost is essentially continuous. The plain is very poorly drained and swampy.

The Arctic Mountains section consists of rolling plateaus and low linear mountains. The bedrock is mostly tightly folded sedimentary rock of varying hardness. Major streams have swift, braided courses across wide gravel flats. Minor tributaries often drain alluvium-choked valleys. Moraines border larger stream valleys. Permafrost is generally present.

The workpad in this section averages 36 inches in thickness except for 28 miles of 24-inch insulated workpad where the gas line parallels the TAPS thermal workpad. A 50-foot pad will be used where the pipeline is on a separate location. This width is reduced to 40 feet where the pipeline is adjacent to the Prudhoe Bay Road, and to 25 feet where it is adjacent to the aboveground TAPS line. No pad is required where the belowground TAPS pad can be used. Material will be obtained from 25 sites along the Sag River to MP 126, and eight pipe storage yards will be located throughout the section. A bridge is required near MP3 to cross the Putuligayuk River. Franklin Bluffs and Happy Valley camps will be used during construction.

SECTION 2 - MP 132.02 to MP 229.58

From the beginning of Section 2 (Figure Z-9.1-1-2) to MP 148.01, the pipeline generally follows along the west side of TAPS; one detour in the vicinity of MP 141 allows the pipeline to cross the site of Compressor Station No. 4. From MP 148.01 to MP 150.83, the pipeline first follows a separate route and then joins the east side of the Prudhoe Bay Road. After crossing the road at MP 150.83, the pipeline either follows the west side of the road or the east side of the nearby TAPS to MP 165.82. At MP 165.82 the pipeline crosses TAPS and follows it along the Atigun River to MP 171.63. For the next few miles in Atigun Pass, the pipeline crosses TAPS and the Prudhoe Bay Road several times on either side of the Pass in a tight Right-of-Way. On the south side of Atigun Pass, the pipeline rejoins the east side of TAPS at MP 174.50 and remains on this route until MP 181.86. The pipeline crosses TAPS, the Prudhoe Bay Road, and the Dietrich River near MP 182, then to avoid a white spruce area follows a separate route along the Dietrich River until MP 184.22. From MP 184.22 to MP 197.83, it follows the west side of TAPS, then at MP 197.83 it crosses and leaves TAPS to follow the west side of the Prudhoe Bay Road until MP 205.59. After crossing the Dietrich River again at MP 206.70, the pipeline comes back to the west side of TAPS and generally follows it to MP 214.60. The pipeline then crosses the Middle Fork Koyukuk River and TAPS to relocate on the east side of TAPS at MP 215.42. The pipeline briefly follows the east side of TAPS before joining the west side of the Prudhoe Bay Road at MP 217.50. Except for minor deviations around MP 222.0 and 223.7, the pipeline remains on the west side of the road to MP 227.65. From that point to the end of Section 2, the pipeline follows a separate route for the crossings of the Middle Fork Koyukuk and Hammond Rivers.

For a total of 70 miles in this section, the pipeline runs along the floodplains of the Atigun, Chandalar, Dietrich, and Middle Fork Koyukuk Rivers, resulting in numerous river and creek crossings. Throughout this section the pipe will be Class 1.

Although the terrain in the mountain valleys is generally flat to gently rolling through most of Section 2, the pipeline crosses the Atigun Pass around MP 173.00. In this area, the terrain is mountainous with very steep grades and the Right-of-Way is restricted. The pipeline reaches its maximum elevation of about 4,750 feet in this section.

Permafrost is discontinuous in Section 2, floodplains and south-facing slopes are often unfrozen. The section lies entirely within the Arctic Mountains Province. The Arctic Mountains Province includes: 1) the southern Arctic Foothills from MP 132 to MP 145; and 2) the Central Brooks Range from MP 145 to MP 230. The area is composed of rugged, glaciated, east-trending ridges that rise to summits of 7,000 to 9,000 feet in the north and 4,000 to 6,000 feet in the south. The easterly grain is the result of belts of hard and soft sedimentary and volcanic rocks. The mountains have a bench-and-slope topography characteristic of glacially eroded bedded rocks. Soils along the pipeline route vary among floodplain gravels, alluvial fan deposits, and fine-grained slopewash deposits. Glacial tills are not common in the center of the Brooks Range but they occur along the north and south margins of the range.

The Atigun River (flowing north) and the Dietrich and the Middle Fork Koyukuk Rivers (flowing south) characterize the major rivers in the Brooks Range; these rivers flow in flat-floored, glaciated valleys approximately 1/2 to two miles wide. Smaller tributaries follow the grain of the rock, cutting V-shaped ravines and building alluvial fans where they debouch into valleys.

From the beginning of this section to MP 165, approximately 11 miles of structural pad, with a 36-inch average gravel depth, and 22 miles of thermal pad are required. The structural pad's thickness varies from 24 inches to 42 inches from MP 165 to the end of the section at MP 229.6. Material will be obtained from 20 sites spaced along the construction zone.

Compressor Station No. 4, Future Compressor Station No. 5, and Mainline Valves 10 through 14 are located in Section 2. Construction will be supported by camps at Toolik, Galbraith, Atigun, Chandalar, and Dietrich. Seven storage yards are planned.

SECTION 3 - MP 229.58 TO MP 375.65

From the start of Section 3 (Figure Z-9.1-1-3) at MP 229.58, the pipeline generally follows the west side of TAPS until MP 235.02 where it crosses TAPS and then meets the west side of the Prudhoe Bay Road. For the next 19 miles, the route follows the Prudhoe Bay Road, crossing the road from west to east at MP 247.07, from east to west at MP 250.63, an from west to east again at MP 251.67. At MP 254.02, the pipeline crosses and begins to follow the west side of TAPS. From here to the Yukon River at MP 361.50, the pipeline generally parallels TAPS, crossing it six times in between; the only main diversion is between MP 273.41 and MP 285.71, where the route is diverted to the east to reduce the number of river crossings and to bypass TAPS Pump Station No. 5. After the aerial crossing of the Yukon River, the pipeline follows the west side of the Prudhoe Bay Road for two miles to bypass TAPS Pump Station No. 6, rejoining TAPS near MP 363.68. From here to the end of the section, the route parallels TAPS, on the east side to MP 375.15, and on the west for the last 1/2 mile.

Mainline Valves Nos. 15 through 22, Compressor Station No. 7, and Future Compressor Stations Nos. 6 and 8 are located in this section. Pipe throughout the section will be Class 1.

The terrain in Section 3 is a series of ridges and valleys which includes parts of several major physiographic divisions. The route traverses the Arctic Mountains Province from the beginning of the section to MP 264; this province includes portions of both: 1) the Central Brooks Range, from the beginning of the section to Coldfoot Camp at MP 245, and 2) the Chandalar Ridge and Lowland Section, from Coldfoot to the crossing of the South Fork Koyukuk River at MP 264. From MP 264 to the end of the section, the route traverses the Kokrine-Hodzana Highlands of the Interior Province. In all, the pipeline crosses numerous creeks and four rivers: the Middle Fork Koyukuk, the South Fork Koyukuk, the Jim, and the Yukon. Soils are predominantly residual soils over bedrock and a complex of eolian, alluvial, and colluvial soils, mostly silts. Fluvial gravels are confined mostly to very narrow floodplains, and glacial deposits are found only north of Prospect Creek.

The workpad will typically be a 42-inch fill section on permafrost. For soils with better supporting value along some of the creeks and rivers (in areas comprising nearly one-fourth the section), pads to 24 inches will be used.

Camps at Colfort, Prospect, Old Man, and Five-Mile will be used during civil and pipeline construction. Eleven pipe storage yards, 32 material sites, and two bridges are planned.

SECTION 4 - MP 375.65 to MP 503.69

From the start of Section 4 (Figure Z-9.1-1-4) at MP 375.65, the pipeline generally parallels TAPS on the southwest side to MP 425.24 where it crosses TAPS from west to east. During this 50-mile stretch, the pipeline departs slightly from TAPS: to pass through Compressor Station No. 9 in the vicinity of MP 381; to cross Hess Creek, Lost Creek, and the Tolovana River at MP 386.70, MP 400.48, and MP 406.98 respectively; and, to bypass TAPS Pump Station No. 7 near MP 422. From MP 425.24 to MP 477.62 the pipeline remains on the northeast side of TAPS. The pipeline generally follows TAPS during this 52-mile run except at MP 426, to avoid TAPS snowpad construction, and MP 429, to avoid steep slopes at MP 439.98 crossing Washington Creek; at MP 446.34 crossing the Chatanika River; and between MPs 455.87 and 459.41 avoiding the Steese Expressway near Fairbanks. Beginning at MP 477.62, the pipeline proceeds along the Golden Valley Electrical Association (GVEA) Right-of-Way (ROW) for 2 miles. The pipeline rejoins TAPS at 479.57 and runs parallel to it until 490.11, crossing TAPS from east to west at MP 486.99. At MP 490.11, the pipeline departs westward passing through the site of Compressor Station No. 11 near MP 494. It then returns to the west side of TAPS at MP 494.54 proceeding alongside it to MP 496.34 where the pipeline veers to the south, circumventing TAPS Pump Station No. 8. The pipeline continues southerly until it meets and crosses the GVEA ROW at MP 498.88. It parallels the GVEA ROW to

MP 502.77 and then diverts to cross the Salcha River to the end of the Section on the east side of the river. The pipeline crosses numerous creeks and the Tolovana, Tatalina, Chatanika, Chena, Little Salcha, and Salcha Rivers. The pipe in this section is generally Class 1 except for two lengths of Class 2 pipe designated for populated areas near MP 463 and MP 468.

The terrain is primarily sharp and choppy; however, around the Fairbanks area it flattens (for about 32 miles) before turning to gently rolling at the end of the section.

This section includes parts of five physiographic subprovinces laying within the Interior Province: 1) the Kukrine-Hodzana Highlands (MP 376 to MP 383); 2) Rampart Trough (MP 383 to MP 388); 3) Livengood Upland (MP 388 to MP 464); 4) Chena-Tanana Lowland (MP 464 to MP 495); and, Salcha Upland (MP 495 to MP 504).

In general, the uplands are an area of northeast-trending ridges composed of schist with intermittent granitic intrusions. There are a few outcrops of bedrock; and profiles are smooth. Generally, ridge crests are unfrozen weathered bedrock or residual soil. Eolian and retransported silts form a mantle that thickens down slope; smaller valleys are choked with over a 100 feet of ice rich silts. Only larger streams in this section have unfrozen floodplain gravels and sands. Overall, the lowlands are a complex of sands, gravels, silty cover deposits, and organic channel fillings that are sporadically frozen.

The pad thickness will alternate between zero and 42 inches in this section with minimum material being required from MP 430 to MP 480.

Construction will be supported by camps at Livengood and Fort Wainwright. A double-jointing yard will be sited at Fairbanks and eight pipe storage yards will be located in this section. Material sites are planned for 35 locations. Seven bridges are required in this section.

Mainline Valves Nos. 23 through 29, as well as Compressor Stations No. 9 and No. 11 and Future Compressor Station No. 10 are located in this section.

<u>SECTION 5 - MP 503.69 TO MP 623.93</u>

From the start of Section 5 (Figure Z-9.1-1-5) to MP 522.29, the pipeline runs parallel and adjacent to the southwest side of the Golden Valley Electric Association (GVEA) Right-of-Way. The pipeline then follows TAPS on the southwest side for approximately five miles. Between MPs 528.28 and 547.84, the route runs adjacent to the GVEA ROW's northeast side. The pipeline traverses through nine miles of virgin territory (bypassing populated areas) and then meets the Haines Pipeline ROW at MP 556.95 and runs parallel to and alongside it for the remainder of the section switching from east to west several times. The pipeline departs from the above route: at Shaw Creek (MP 527.41), the Tanana (MP 538.85), Johnson (MP 588.73), and Robertson (MP 621.35) Rivers, and between MPs 592.21 and 594.69 in order to facilitate construction; the pipeline crosses several creeks at the beginning of the section, during the first 20 miles, and at the end of the section, during the last 30 miles. Except for a hilly area, between MPs 510 and 525, the terrain is generally flat or gently rolling. Pipe throughout this section is Class 1.

This section includes two major physiographic divisions: 1) The Interior Province comprised of the (a) Salcha Upland between MP 504 and MP 538, and (b) Delta-Gerstle Lowland, between MP 538 and MP 582; and 2) The Alaska Range Province, comprised of the Johnson-Robertson Foothills between MP 582 and MP 624.

The Salcha Upland consists of northeast trending rounded ridges with several hundred feet of relief. Weathered bedrock is exposed on ridges. The mantle of retransported silts thickens from the upper slopes (feather edge) to the valleys (over 100 feet). Only major streams contain floodplain gravels. Permafrost is discontinuous in this region.

The Delta-Gerstle Lowland is a long trough laying along the north flank of the Alaska Range. Streams and glaciers flowing from the Alaska Range have carried enormous quantities of gravel and sand that have forced the Tanana River to the north side of the valley; they have built an apron sloping northward across the valley. The Tanana's northern tributaries form meandering streams in flat swampy valleys. Permafrost in this area is discontinuous and sporadic.

The Alaska Range (Johnson-Robertson Foothills) barely impinges upon the pipeline in this area. The pipeline runs into a series of fluvial cones and rocky hillsides before descending once more into the valley.

The workpad thickness generally will be 42 inches to MP 538; 18 inches to MP 583; and will average 24 inches from that point to the end of the section. Mainline Valves Nos. 30 through 36, Future Compressor Station No. 12, and Compressor Station No. 13 are located in this section.

Construction will be supported by camps at Big Delta and Sears Creek. Material will be obtained from 28 sites and seven locations will be used for pipe and other storage. Seven bridges are planned.

SECTION 6 - MP 623.93 TO MP 743.19

Section 6 (Figure Z-9.1-1-6) begins at MP 623.93 to the west of a westerly meander of the Tanana River about 26 miles west and 9 miles north of Tok.

The pipeline proceeds south, then east along the Haines Pipeline Right-of-Way, passing below the Cathedral Bluffs to the east. The Alaska Highway is located 100 to 200 yards to the east of the pipeline until MP 632. At that point, the pipeline and the Haines ROW veer to the south, leaving the highway; Future Compressor Station No. 14 will be built in this run in the vicinity of MP 634, 1/2 mile before crossing Yerrick Creek. Between MPs 637.48 and 641.37 the pipeline follows a route through virgin territory to avoid marshlands. At MP 641.37 the route crosses the Alaska Highway and rejoins the Haines ROW. It then continues east to MP 646.04, passing to the south of Tanacross Airfield. At MP 646.04, the pipeline leaves the Haines ROW and the Alaska Highway and passes to the north of the Haines Pipeline Tok Pumping Station. It continues east to a point to the north of Tok where it turns to the southeast to join the Haines ROW at MP 660.65. For the next 5 miles, the pipeline parallels the Haines ROW until it reaches the Tanana River at MP 666.05.

An aerial crossing will be used at the Tanana River. It will be located about 200 yards to the north of the Alaska Highway Crossing. The pipeline then continues paralleling the Haines ROW to MP 680.29 where it leaves the Haines ROW and follows a route approximately 100 to 300 yards to the north of the Alaska Highway to MP 682.60. At this point, the pipeline crosses the highway and the Haines ROW and begins to follow the west side of the Haines ROW. At MP 684.69, the pipeline crosses back to the north side of the highway to pass through Compressor Station No. 15 near MP 685 and rejoins the Haines ROW at MP 685.50. From this point to the Canadian border the route generally follows the Haines ROW except for short diversions to improve construction conditions.

Throughout most of the section the route is never more than a few hundred yards from the Alaska Highway. Mainline Valves Nos. 37 through 44, Compressor Station No. 15, and Future Compressor Stations Nos. 14 and 16 are located in this section. Throughout this section the pipe will be Class 1.

Section 6 includes parts of two physiographic provinces: the Interior Province and the Alaska Range Province. The route begins in the Johnson-Robertson Foothills (MP 624 to MP 640) of the Alaska Range Province. It then traverses the Tok Fan (MP 640 to MP 666) and the Northway Upland (MP 666 to MP 743) of the Interior Province. Through the Tok Fan, the route passes through 26 miles of flat area south of the Tanana River. After the Tanana River crossing at MP 666, which is past Yarger Lake, the pipeline passes between the Tanana/Chisana River floodplain on the west and forested hills on the east. After MP 711, the route passes through approximately 20 miles of sand dunes, and passing through ridges and valleys near the end of the section at the Canadian Border.

The workpad in this section varies from 18 to 42 inches, averaging 30 inches in thickness. Material for construction will be obtained from 37 material sites. Seven bridges are required. Construction will be supported with camps at Tok Northway and by seven pipe storage yards.

1.2 Pipe Selection Criteria

Codes and Regulations

- 49 CFR 192.101 through 192.115
- API 5LX, Specification for High Test Longitudinal Seam Line Pipe
- API 5LS, Specification for Spiral-Weld Line Pipe
- Project Specification SP-4680-50-26 [1]

Design Criteria

- 48-inch outside diameter
- Maximum allowable operating pressure 1260 psig

[1] Specification SP-4680-50-26, Piping Material Purchase Specification, Pipe - Large Diameter, Low Temperature

- o Minimum design temperature for pipe material 0°F

- o Pipe material will be Grade 70 specified in accordance with the above codes and regulations plus additional specification requirements to provide improved mechanical properties, increased inspection requirements, restricted dimensional tolerances, and fracture toughness levels suitable for the operating conditions.

- o The steel will generally be a low carbon (0.10% maximum) controlled rolled type with the maximum levels of the other alloying elements established mainly to provide a weldable steel compatible with field requirements.

- o Fracture toughness levels have been specified based on Charpy V-Notch energy (CV100 criteria) to provide an inherent resistance to both fracture initiation and unstable fracture propagation at the design conditions. A drop weight tear test minimum shear area requirement at the design temperature has been specified for brittle fracture prevention.

- o For ductile fracture propagation resistance, toughness levels have been predicted using empirically derived equations which relate pipe properties and fracture velocity. When used in conjunction with the expected gas decompression chacteristics at the compressor suction and discharge, the toughness levels predicted by these equations are 70 and 105 ft-lb respectively. Accordingly, the pipe specification requires 105 ft-lb for 50% of the heats and 70 ft-lb for the remainder. These toughness levels are well above those that would be required for fracture initiation resistance and a large initial flaw size would have to exist before fracture propagation could occur.

- o A full scale pipe burst test program is currently underway to confirm the adequacy of these toughness levels. If necessary, appropriate adjustments of the fracture control methodology will be made based on the full scale test results.

- <u>Wall Thickness Calculation</u> - Pipe wall thickness was calculated in accordance with 49 CFR 192.105, using the following equation:

$$t = \frac{PD}{2S} \times \frac{1}{F \times E \times T}$$

Where:

t = Nominal wall thickness of the pipe in inches

P = Design pressure in pounds per square inch gauge - 1260 psig

S = Yield strength in pounds per square inch - 70,000 psi

D = Nominal outside diameter of the pipe in inches - 48"

F = Design factor - see table below

E = Longitudinal joint factor - 1.0

T = Temperature derating factor - 1.0

- Pipe wall thickness based on Class Location will be as follows:

Class Location (49 CFR)	Design Factor (F)	Maximum Allowable Operating Pressure (psi)	Wall Thickness (Inches)
1	0.72	1260	0.600
2	0.60	1260	0.720
3	0.50	1260	0.864
4	0.40	1260	1.080

1.3 Mainline Block Valve Assemblies

Codes and Regulations

- 49 CFR 192.111 (b) (4), 192.141, 192.143, 192.145, and 192.179
- API Standard 6D - Specification for Pipeline Valves
- Project Specification SP-4680-50-25 [2]

[2] NWA/PMC Specification SP-4680-50-25, Piping Material Purchase Specification, Ball Valves, - Large Diameter, Low Temperature

Design Criteria

- Mainline valves will be 48-inch ANSI 600 rated ball; trunnion mounted; ball full bore to match the inside diameter of 48-inch OD x 0.720-inch wall thickness pipe.
- Weld end connections with transition pieces for welding directly to 48-inch OD x 0.720-inch wall thickness API-5LX (5LS) Grade X70 pipe.
- Body materials will be suitable for gas temperatures down to 0°F.
- Top works will be suitable for ambient air temperatures to -50°F.
- Valves will be supplied complete with external corrosion coating for buried installation.
- Valve assemblies will be equipped with bypass and blowdown facilities.
- Above grade bypass and blowdown piping materials will be suitable for temperatures to -80°F.
- Nominal valve spacing will be 20 miles for Class 1 Locations. Reference Exhibit Z-9.0, Appendix E, Table E-1.
- Mainline block valves will be equipped with gas/hydraulic operators that will utilize the pipeline gas pressure as the primary source of power.
- A 12-inch blowdown bypass piping system will be installed around each mainline block valve to permit depressurization and purging of pipeline sections between block valves.

1.4 Pig Launchers and Receivers

Codes and Regulations

- 49 CFR 192.111, 192.147, 192.149, 192.153(b), and 192.157.

Design Criteria

- MAOP 1260 psig

- o Material suitable for gas temperatures down to 0°F.
- o Design factor 0.6.
- o Trap barrel outer diameter will be 6 inches to 8 inches greater than the mainline pipe diameter.
- o Launcher barrel will be of sufficient length to accommodate at least one pig downstream of the kicker line inlet.
- o Receiver barrel will be of sufficient length to at least hold two pigs downstream of the receivers outlet.
- o Access for loading and unloading of pigs will be provided through a quick opening power assisted end closure.
- o Internal trays will be provided to facilitate loading and unloading of pigs.

1.5 Pipeline Ditch Design

See typical drawings, Appendix E, Exhibit Z-9.0.

Codes and Regulations

- o 49 CFR 192.327

Design Criteria

- o Minimal environmental disturbance.
- o Minimum clearance, ditch sidewall to pipe, 12 inches.
- o Minimum bedding thickness where required, 6 inches.
- o Minimum padding thickness where required, 6 inches.
- o Minimum cover:

Location	Normal Soil	Consolidated Rock
Class 1	2'-6"	1'-6"
Class 2, 3, and 4	3'-0"	2'-0"

Ditch Types

- Type I Conventional Ditch - This type of ditch will be used in all areas excluding permafrost regions, frost-susceptible soils and bedrock. Bedding and padding will be used where required to provide protection for the pipe before backfilling with native or common backfill.

- Type II Permafrost Ditch - Four subtypes of this ditch configuration will be used in permafrost areas where the ditch will be opened by blasting

  - Subtype "A" will utilize insulation board ranging in thickness from 1-1/2 to 3-1/2 inches covered by a protective berm to maintain the ditch materials in a frozen state prior to startup. [3] This berm-over mode of construction is an extension of the workpad and will limit the amount of excavation needed in frozen ground.

  - Subtype "B" will use a 5-inch thick insulation board and will be employed in permafrost regions that have a deeper active layer (south of Brooks Range). Again, the purpose will be to prevent the disturbed materials in the ditch sidewalls from thawing prior to startup. The insulation board for Subtype B will be located as close to the active layer depth as practical.

  - Subtype "C" will be similar to Subtype B except that this subtype will apply to thaw-stable permafrost. Here, the insulation board will not be required and only bedding and padding will be used.

  - Subtype "D" will be similar to Subtype "A" except that it will apply to thaw stable permafrost; the insulation board will not be required.

- Type III Rock Ditch - This type of ditch will be used in bedrock areas where the ditch will be opened by blasting, and the depth of cover will be reduced from 2 feet 6 inches to 1 foot 6 inches (for Class 1 areas) as allowed by 49 CFR 192.327. Bedding and padding will be required for this type of ditch.

[3] Reference report, "Determination of Insulation Thickness Required for Ditch Configurations II A and II B".

- Type IV Deep Burial Ditch - This type of ditch will be used in relatively shallow frost-susceptible soils. The maximum depth of excavation, and hence the pipe burial depth, will be limited by the practical working depth that can be reached by standard construction equipment used on a pipeline spread. The limit presently considered is 15 feet. Bedding and padding will be used where required to provide protection for the pipe before backfilling with native or common backfill.

- Type V Mitigative Ditch - This type of ditch is designed to limit the adverse effects of frost heave on the pipeline by retarding the frost bulb growth by using insulation around the pipe and by replacing the frost-susceptible soil beneath the pipe with nonfrost-susceptible material. The use of insulation will reduce the rate of growth and the size of the frost bulb and will consequently limit frost heave. Backfill material used below the pipeline will be a nonfrost-susceptible granular material with silt content $\leq$ 6 percent.

1.5.1 Ditch Stability

Ditch stability will be a problem during warm weather when a ditch is opened in high moisture/ice content sandy and silty soils. The methodology and considerations for this subject are discussed in Section 4.0. Alignment sheets will identify these potential problem areas. Construction in these areas will be scheduled for the shoulder months.

1.5.2 Backfill materials to be used in Pipeline Ditch

- Bedding, where required to provide protection to the pipe from the ditch bottom, will be placed in the bottom of the ditch to a minimum depth of 6 inches utilizing an end-dump or equivalent method and will not require compaction. A minimum of 6 inches of padding material will be placed around the pipe to prevent damage resulting from backfill operations. This material will be end-dumped or placed by an equivalent method and hand dressed as required to provide adequate support and protection for the pipe and pipe coating. Bedding and padding material will be processed pit-run material with a size limitation of maximum 2-inch.

- o Native backfill material excavated from the ditch will be used for backfill purposes providing it is a stable material with rock content size limitation of maximum 6-inch. Bedding and padding will be placed under and over the pipe where required as discussed above.

- o Common backfill will be placed on top of padding material in areas where original excavated ditch material is not suitable for backfill purposes. Common backfill will normally be pit-run material with a rock content size limitation of maximum 6-inch.

- o Nonfrost-susceptible backfill will be used for backfilling overexcavated portions of the Type V mitigative ditch prior to lowering-in the pipe. Nonfrost-susceptible backfill will normally be a granular pit-run material with silt content $\leq$ 6 percent.

1.6 River and Stream Crossings

See typical drawings, Appendix E, Exhibit Z-9.0.

Codes and Regulations

- o 49 CFR Part 192.327 (e)

- o Department of Interior Stipulations

River Crossing Design Criteria

- o Pipeline Design Flood (PDF) will be used as design basis for stream crossings.

- o Minimum depth of cover will be 4 feet or maximum computed scour plus 20 percent, whichever is greater, measured from thalweg of stream to top of continuous concrete coating or top of set on weight. For unclassified stream crossings, minimum depth of cover will be 2 feet 6 inches. For a stream crossing whose trench is excavated in rock minimum depth of cover will be 2 feet.

- o Maximum scour at each stream crossing is dependent on the PDF magnitude, hydraulics of the crossing site and the size of streambed material. Methods employed for establishing this are discussed in section 5.0 on hydrology.

- Sag bends at stream crossings will be setback from the stream bank at a distance greater than the maximum predicted bank migration. The potential for bank migration is determined by comparison of old and new aerial photography, empirical formulas and by site specific investigations. The methods employed in this analysis are discussed in Section 5.0.

- For the purpose of pipeline stream crossing design, streams have been classified as major, minor, and unclassified according to the system developed in Section 5.0 on hydrology. Stream crossings requiring special construction considerations are listed in Table E-2, Appendix E to Exhibit Z-9.0.

- Major stream crossings, with the exception of the three aerial crossings, will have continuous concrete coating. Other stream crossings will be weighted using set-on weights as required.

- Based on a specific gravity of 1.0, a negative buoyancy requirement of 5 percent and a concrete density of 190 pcf for concrete coating and 140 pcf for set-on weights, the weighting requirements for 0.600-inch wall thickness and 0.720-inch wall thickness pipe will be as follows:

Concrete Coating

Wall Thickness (Inches)	Thickness of Concrete Coating: Bare Pipe	Thickness of Concrete Coating: Pipe with 6" of Insulation
0.600	3-3/4"	5-1/2"
0.720	3-1/2"	5-1/4"

Set-on Weights (1)

Wall Thickness (Inches)	Weight Spacing: 13,500# Weight	Weight Spacing: 25,000# Weight
0.600	13'-0"	14'-0"
0.720	15'-0"	14'-0"

(1) 13,500# weights used on uninsulated pipe
25,000# weights used on pipe with 6" of insulation

1.7 Road Crossings

See typical drawings, Appendix E, Exhibit Z-9.0.

Codes and Regulations

- o 49 CFR 192.111, Design Factor (F) for Steel Pipe
- o Alaska Department of Highways, Policy on Accommodation of Utilities on Highway Rights-of-Way (11/16/79).
- o API Recommended Practice 1102

Design Criteria

- o Casing dimensions - 56 inch O.D. x 0.250 inch wall thickness, API 5L Grade B minimum
  - 66 inch O.D. x 0.375 inch wall thickness, API 5L Grade B minimum
- o Carrier pipe wall thickness within the right-of-way of public roads based on class location will be as follows:

Class Location	Design Factor	Wall Thickness
1	0.60	0.720
2	0.50	0.864

- o The minimum depth of cover will be four feet from the lowest point within the road right-of-way to the top of either the pipe, for uncased road crossings, or the casing, for a cased crossing.
- o The angle of crossing between the pipeline and the roadway will be as nearly perpendicular as practical.
- o In nonfrost-susceptible areas the casing diameter will be 56 inches. Casing vents will not be used, the casing will be filled with a nonwater soluble inert casing filler and will be separated from the carrier pipe by full encirclement insulators.
- o In frost-susceptible soil areas, carrier pipe will have insulation and the casing diameter will be 66 inches. Casing vents will not be used. The casing will be filled with foam-type insulation.

- Where required, road crossings will be bored. Boring will be performed through the casing where casing is installed. Open trenching, if permitted by the state authorities, will be used whenever possible.

- Casing will be electrically insulated from the carrier pipe.

- Reference Exhibit Z-9.0, Appendix E, Table E-3.

1.8 Other Pipeline Crossings

See typical drawings, Exhibit Z-9.0, Appendix E.

Codes and Regulations

- 49 CFR 192.325

Design Criteria

- Minimum clearance, vertical support members (VSM) to ditch line - 15 feet

- Minimum clearance - 12 inches

- Crossing angle - 70° to 90°

- Heavy wall pipe wherever crossing TAPS

- The minimum clearance between the gas pipeline and other pipelines being crossed will be 12 inches.

- The angle of crossing between the gas pipeline and other pipelines will be as nearly perpendicular as practical.

- For TAPS crossings, the minimum crossing angle will be 70°. This angle is based on a minimum bending radius of 120 feet and a distance of 80 feet between pipelines in their parallel configuration. This crossing configuration is shown in Figure Z-9.1-1-7. Where required, the pipeline alignment may deviate from the 80-foot minimum parallel spacing in the area of the side bend to accommodate maintaining the minimum crossing angle. When TAPS is aboveground, the buried gas pipeline will cross as nearly perpendicular as practical to TAPS at the mid-point of the two adjacent vertical support members. When the TAPS oil pipeline is buried, the gas pipeline will cross above it as nearly perpendicular as practical and will be protected by an earthen berm.

The design of the crossings will evaluate any potential effects resulting from the growth of a frost bulb around the gas pipeline. An initial thermal analysis of the gas pipeline crossing over TAPS in a berm shows that the frost bulb will not penetrate sufficiently to influence or cause danger to TAPS. For further clarification of simulated computer analysis, see Section 3.0, Frost Heave.

- Locations where the pipeline crosses other pipelines are shown in Table E-4, Appendix E, Exhibit Z-9.0.

1.9 Pipeline Installation in Wetlands

See typical drawings, Appendix E, Exhibit Z-9.0.

Design Criteria

- Specific gravity of the immersing media 1.18
- 2 percent negative buoyancy
- Concrete density of 140 pcf

1.10 Pipeline Installation at Fault Crossings

An investigation program is continuing to locate or verify existing active faults along the pipeline route. At locations where the pipeline must cross an active fault identified by the survey, pipeline installation mode configuration will be designed to prevent a pipe rupture due to fault movement. The geometry of the pipeline and the ditch, the type of pipe coating, and the nature of backfill materials that are selected will affect the behavior of the pipeline when it is subjected to large fault movements. For each active fault crossing, a detailed analysis and specific design will be performed in order to select the most suitable construction method from the following crossing modes:

- Buried Configuration - Ditch designed to limit the resistance to pipe movement
- Buried Configuration - Pipeline inside a conduit
- Aboveground Configuration - Pipeline in a berm

1.11 <u>Pipeline Corrosion Control</u>

See typical drawings, Appendix E, Exhibit Z-9.0.

<u>Codes and Regulations</u>

- 49 CFR 192.451 through 192.491 and Appendix D
- NACE, RP-01-69

<u>Corrosion Control Design Criteria</u>

- High resistance, electrically insulating, external protective coating compatible with the cathodic protection system.
- Cathodic protection system capable of maintaining the pipeline at a minimum negative (cathodic) voltage of at least 0.85 VDC with reference to a saturated copper-copper sulfate half-cell with protective current applied.
- Electrically insulating joints to assure that the pipeline remains free from foreign contacts at each compressor site, at the beginning and end of the line and at any ties to other pipelines.
- Test stations for electrical survey of all points deemed necessary for proper system performance.
- Pipeline will be cathodically protected by impressed current rectifier and groundbed installations.
- Rectifiers will be constant potential units and will be connected to deep well groundbeds or, if suitable areas are available, to surface type groundbed systems. Rectifiers and groundbeds will be located at compressor station sites in order to utilize the existing ac power supply.
- Impressed current cathodic protection system will be supplemented by sacrificial anodes in areas such as river crossings, wetlands, unfrozen inclusions in permafrost, and areas of low resistivity in otherwise frozen soil.
- Telluric currents will be controlled by electrical isolation and segregation of the pipeline into short segments by installation of electrically insulating joints at compressor station sites; automatic poten-

tial control rectifier to sense and control potential variations; utilization of sacrificial strip anode materials at areas of probable current discharge as determined by resistivity and geophysical surveys.

- o Internal corrosion control will not be required because of the noncorrosive nature of the gas being transported.

- o A monitoring system will be installed for recording:

  - Rectifier and groundbed performance

  - Pipeline potential profiles

  - Telluric current activity

- o Test stations for measuring pipeline electrical potentials will be installed at:

  - Road crossings

  - River crossings

  - Other pipeline crossings

  - Insulating joints

  - At one-mile intervals where not covered by another type of test station

- o Test stations for measuring pipeline current flow patterns will be installed at:

  - Areas of sacrificial anode installation

  - At five-mile intervals

1.12 Pipelines Spatial Position and Physical Condition Monitoring System

A system will be provided for monitoring the spatial position of the pipeline and for checking the physical condition of the pipe to ensure compliance with the design criteria and to prognosticate any possible deviation.

Design Criteria

The monitoring system will:

- o determine changes in the pipeline plan and profile
- o determine changes in the pipeline radius of curvature
- o detect pipe ovalization, wrinkles, dents, and pipe wall loss
- o be capable of collecting and presenting data in a hardcopy form
- o be reliable
- o have minimum requirement for operating personnel
- o perform under normal pipeline operating conditions and under all weather conditions
- o have minimum time interval between data collection and output results
- o be cost effective
- o be accurate

1.13 Hydrostatic Testing

Codes and Standards

- o 49 CFR 191
- o 49 CFR 192.501, 192.503, and 192.505

Design Criteria

- o Detailed procedures will be developed for hydrostatic testing to prove the strength and integrity of the pipeline system using the following criteria and guidelines
- o In any test section, the pressure at the point of maximum elevation will be at least 1.25 times the maximum operating pressure MOP and the pressure at the point of minimum elevation will not cause a stress that would exceed 110 percent of the specified minimum yield strength (SMYS) of the pipe. This

corresponds to a maximum pressure of 1925 psig and a minimum pressure of 1575 psig for a maximum elevation differential in a test section of 807 feet.

- Minimum duration of test will be 8 hours.
- Testing will be performed during the period from April to October.
- Hydrostatic testing of certain portions of the pipeline may require the use of heated water. The final identification of these areas will depend on an analysis of backfill soil temperatures and fill water temperatures to be encountered at time of testing.
- In the event that pressure readings indicate that a leak exists in a section of pipe under test, steps will be taken to locate and repair the leak and the pipe section will be retested. A contigency plan will be prepared to handle various types or sizes of leaks.
- Pipeline will be thoroughly cleaned by repeated pigging operations prior to introduction of test water.
- Physical condition of the pipeline will be checked by running an instrumented pig (kaliper pig) to locate and record any damage.
- Any damage to the pipe will be rectified.
- Clean filtered water will be used for testing. Volume of water introduced into the test section will be metered.
- A pressure versus volume added plot will be made during the pressurizing of the test section. The plot will start at a pressure equal to 80 percent of the test pressure and will continue through to the full test pressure.
- Upon completion of the pressure-volume plot, the pressure in the pipe will be allowed to stabilize and the temperature of the test media, the pipe, and the backfill will be allowed to equalize prior to beginning the 8 hour hold test.

- Upon stabilization, the pressure in the test section will be brought back to the predetermined test pressure, locked in, and held for a minimum period of 8 hours. Temperature and pressure data will be recorded throughout the duration of the test to verify that there are no leaks.

- Instrumentation will be used to record water temperature, ground temperature, ambient temperature, and test pressure during the filling, stabilization, and hold test stages of the test.

- Records and documentation of the hydrostatic tests will be retained on file for the life of the pipeline.

- Upon completion of a successful test, the test section will again be inspected using the instrumented pig.

- The pipeline test section will be dewatered once the successful hydrostatic test has been completed.

- Hydrostatic test planning will emphasize the reutilization of the test water, from one section to another, in order to minimize the amount of test water being required and the amount of water being disposed.

- The water being disposed will be treated where required to the minimum State and Federal Environmental Standards prior to disposal in normal drainage patterns.

- Immediately after dewatering operations, pigs will be run through the test section in order to remove any water remaining in the pipeline. To eliminate the possibility any small quantities of water may remain in the pipeline, a final pigging run will incorporate a methanol slug run between two batching pigs.

- Water for hydrostatic testing will be obtained from naturally occurring sources after a thorough evaluation of the environmental impact of such an operation; the quality of the water; the quantity of water available, the rate at which the water may be obtained and its cost.

- o River crossings will be subjected to a hydrostatic test prior to installation. The crossings will be tested to a pressure corresponding to a stress level of 90 percent of SMYS for a minimum period of 4 hours. Pretested river crossings will subsequently be tested after installation as an integral part of the pipeline.

1.14 Pipeline Stress Analysis

This section outlines the basic stress analysis design criteria, design concepts, and design procedures established to ensure the fundamental safety and structural integrity of the buried gas transmission pipeline. Applicable regulations, codes, and standards are presented as they pertain to stress analysis. The types of loads, loading conditions, and combinations of loads are identified and discussed. The geotechnical conditions and soil responses are reviewed as they relate to geotechnical design criteria and methods which are used to model soil response. The basic design criteria for acceptable levels of stress and strain in the pipe are established for all identified loading conditions. In addition, the results of analyses are presented to demonstrate compliance with the criteria. The appropriate methods of analysis to be used in the final design are subsequently defined. The analytical procedures and methods will ensure that the levels of induced stress and strain do not violate the criteria.

Stipulations

The following Department of Interior stipulations pertain to the pipe stress analysis work:

- o Stipulation 3.1.1.1
- o Stipulation 3.2.1.1
- o Stipulation 3.2.1.2
- o Stipulation 3.2.2.2
- o Stipulation 3.3.1
- o Stipulation 3.6.1

Docket No. CP80-
Exhibit Z-9.1
Hearing Exhibit No.

DOT Regulations, Title 49 CFR 192

The following sections pertain to the basic structural requirements for the pipe stress analysis work:

- 192.103
- 192.105
- 192.111
- 192.113
- 192.159
- 192.161

This Federal Regulation places limitations on allowed internal pressure. It does not specify other loading conditions, combinations of loading conditions, methods of application of loads, or limitations on the combined state of stress and strain. Its basic structural requirements other than the above internal pressure limitation are in general terms only.

Codes and Regulations

Other codes and standards which are not mandatory, but are accepted by industry for use in gas pipeline design and construction are used as guidelines where appropriate in areas not specifically covered by the Stipulations and Federal Standards. These include:

- ASME Guide for Gas Transmission and Distribution Piping Systems.
- ANSI B31.8, "Gas Transmission and Distribution Piping Systems."
- API RP-5L1, "API Recommended Practice for Railroad Transportation of Line Pipe."
- API Standard 1102, "Liquid Petroleum Piping Crossing Railroads and Highways."
- AISC, "Manual for Steel Construction."
- ACI, "Building Code Requirements for Reinforced Concrete."
- AISC, "Manual for Structural Application of Steel Cables for Building."

1.14.1 Design Approach

The following general design approach has been established to ensure the structural integrity of the pipeline.

- Identify the loading conditions and establish the range of design loadings and geotechnical conditions to which the pipeline may be subjected during construction and operations. The effects of the following are directly considered in the stress analysis work:

  - Internal pressure
  - External pressure from overburden
  - Pipeline temperature changes
  - Dead load of pipe, contents, and insulation/coatings
  - Frost heave
  - Settlement
  - Buoyancy in saturated soils
  - Seismic ground motion
  - Potential movement at active fault crossings
  - Construction loads
  - Wind load
  - Snow and ice loads

  Certain occasional severe geotechnical loading conditions are not directly considered in structural stress analysis. It is recognized that such conditions may exist along the pipeline route. The risk from such conditions is minimized by careful selection of the pipeline route, by a geotechnical evaluation of hazardous areas in order to design mitigative measures to resist the occasional severe loading conditions, by identifying potential problem areas during construction, and by maintaining a regular and planned monitoring program during operation. These severe conditions include the following:

  - Slope instability caused by construction or natural processes
  - Seismic liquefaction and subsidence
  - Erosion
  - Thaw plug instability

- Establish critical pipe material behavior and acceptable levels of stress and strain which will not be exceeded to ensure that pipe deformations will not occur. Levels of maximum permissible stress are specified by 49 CFR 192. Additional limitations are placed on strain levels. The maximum axial compressive strain is limited to prevent local wrinkling or instability of the pipe wall. The maximum tensile strain is limited to maintain a ductile reserve and to mitigate the possibility of fracture initiation or propagation. In both compression and tension, the allowed strain is well below that which would produce rupture of the pipe.

- The minimum required pipe wall thickness is first determined in accordance with 49 CFR 192 based on design MAOP. Elastic analyses are then performed for primary loads and secondary loading conditions which produce membrane stresses in order to verify the acceptability of combined stress levels.

  Nonlinear analyses will be performed to consider the effects of secondary loadings produced by the movement of soils in combination with other applicable loadings. As the pipe material yields, the secondary loads are relieved since they are displacement limited. The criteria then becomes the limitation on the allowed strain level. Based on these limits, parametric analyses will be used to determine allowable pipe displacements and geometric configurations for the range of geotechnical loadings to which the pipeline may be subjected.

- A detailed mile-by-mile evaluation of the route will be conducted and used in conjunction with the parametric studies to establish the pipeline design along the route.

Loading Conditions

The conditions for which the pipeline is analyzed can generally be classified as follows:

- Transportation - Transportation loads are considered to be those imposed during handling, loading, shipping and stacking.

- Construction and Pre-Operations - Construction loads are those loads imposed during installation resulting from stringing, construction traffic, lifting, and placement of the pipe in the ditch. Lifting will cause flexural stress and strain. Weight and compaction of the backfill will cause circumferential bending stress and some ovalization in the unpressurized pipe. Construction traffic will contribute to overburden loads.

Pre-Operation loads include settlement that may occur after installation, prior to startup, and during the hydrostatic proof test.

- Design Operations - Design operating loads are the sustained loads imposed by normal operations of the pipeline and the maximum expected geotechnical loads resulting from movement at bends, frost heave, settlement, and design operating earthquake. For these loading conditions, the criteria are established to provide a sufficient factor of safety against failure.

- Design Maximum - Design maximum loads include design operating loads combined with occasional loads such as loads from extreme conditions with a low probability of occurrence during the lifetime of the pipeline. For such conditions, the combined strains are allowed to exceed the design operating criteria with a reduced factor of safety. Should these loadings occur, it may be necessary to shut-down the system, inspect the structure, and take action to relieve excess stress or strain. These loads include contingency earthquake effects and fault crossings.

Primary and Secondary Loading Effects

The loadings can be classified as to their effect on stress and strain:

- Primary Loads - A primary load is a load which is not self-limiting and cannot be relieved by yielding or distortion. Such a load induces stresses which are necessary to satisfy the laws of equilibrium of external and internal forces and moments. Primary loads on a buried pipeline include internal pressure, dead load of the pipe and its contents, soil overburden, and buoyancy in saturated soils.

- Secondary Loads - A secondary load is a load which is self-limiting and can be relieved by yielding or distortion. Such a load is caused by movement of supports, restraint of adjacent parts, or self-constraint of the structure. Failure is not stress limited and occurs only when strain levels or fatigue exceed the ductile capacity of the material or cause structural instability. Secondary loads on a buried pipeline include the effects of temperature changes, as well as the effects of movement of the surrounding soil media caused by displacement at bends, differential settlement, differential frost heave, seismic ground motion, and fault displacement.

Internal Pressure

Internal pressure is a primary load which induces primary circumferential tensile stress and strain in the pipe wall by expanding the pipe radially. If the pipe is unrestrained, pressure also induces axial tensile stress by expanding the pipe longitudinally. If the pipe is restrained the longitudinal pressure force is resisted by the longitudinal soil restraint, and axial tensile stress then develops as a result of Poisson's effect. Where changes occur in pipe alignment, internal pressure induces secondary longitudinal bending stresses. Internal pressure governs the required pipe wall thickness in accordance with the requirements of 49 CFR 192. This limits the hoop pressure stress to a designated percentage of the specified minimum yield strength. The pipeline system is then designed to accommodate or limit other loads so as not to exceed the limits of the design criteria.

Overburden

The weight of the backfilled soil over the pipe is a primary load, bearing on the upper surface of the pipe and induces a circumferential bending stress in the pipe wall. This load may be controlled by limiting the depth of burial or controlling the placement of backfill around the pipe. An uncontrolled overburden load might also cause excessive flattening (ovalling) of the pipe which could restrict the passage of internal cleaning or monitoring equipment. In areas where settlement may occur, the overburden load will induce secondary longitudinal bending stresses.

Dead and Live Loads

Dead loads include the weight of the pipe and any externally applied loads such as overburden, concrete coating, or weights. Live loads are construction traffic; the weight of the hydrostatic test medium used during the test; or the gas during operations.

Buoyancy

Buoyant uplift will occur in areas where the pipeline is partially or fully submerged in a water or saturated soil-water media. When the weight of the pipeline and its contents is less than the weight of the saturated soil it displaces, the uplift force will tend to push the pipe out of the ditch. Concrete coating or concrete weights will be used to offset buoyant effects as required.

Differential Temperature

A buried pipeline is partially or fully restrained from expansion or contraction by the backfill and surrounding soil medium. Any change in temperature of the pipe steel after installation will induce secondary longitudinal stress. The temperature of the steel at installation will be at or near ambient. An increase in temperature during operation or test will induce compressive membrane stress. The elastic analysis considers full restraint at maximum positive and negative temperature differential. The inelastic analysis considers the soil-pipe interaction to determine the degree of restraint and pipe movement in conjunction with bending stresses induced by soil movement.

Seismic Loads

Seismic activity induces secondary stress and strain in the pipe as a result of ground motion from seismic waves or deformation from faulting. Seismic shear and compression waves travel through the soil media causing displacement of the soil particles which in turn cause longitudinal strain on the pipe. Faulting may occur in the vertical or horizontal direction causing bending stress and longitudinal strain resulting from the soil restraint.

Geotechnical Loads

Geotechnical loads are considered to be those caused by movement of the supporting soil media and include soil deformation at bends, differential settlement, and differential frost heave. These are displacement limited secondary loads which cause bending movements and result in additional longitudinal strain. Bends are required at alignment changes or to accommodate elevation changes. The axial forces created at these bends are resisted by the bearing pressure of the soil. As a result, the soil will deform in the vicinity of the bend apex and allow pipe movement which induces longitudinal bending strains in the pipe. The amount of pipe movement is a function of the load-deformation characteristics of the soil and the longitudinal restraint provided by soil friction. Differential settlement may occur during pre-operation and testing as a result of thaw settlement of the soil, differential compaction, or loss of support due to erosion.

During operation, a frost bulb forms around the pipe preventing further differential settlement.

Chilled gas flowing through the pipe causes frost heave in certain soils as a result of the ice lense formation or ice transformation below the pipe. Frost heave exerts a vertical force on the pipe tending to cause upward movement. If this movement is uniform, no additional stress is induced. However, where transitions occur from frozen areas to unfrozen areas, differential movement may take place. Differential movement will then induce longitudinal bending strains. The amount of strain that occurs is a function of pressure, temperature, upward heave force, length of affected section, and uplift resistance. Uplift resistance results from the combined action of overburden weight, pipe weight, shear resistance of the frozen soil, and the stiffness of the pipe.

Stress-Strain Relationships

- Biaxial Stress-Strain - A high pressure gas transmission pipeline is essentially a straight or curved cylindrical shell. As such, it is subjected to a more complex state of stress and strain than that existing in a simple uniaxial test. The pipe is in a state of biaxial stress, consisting of hoop (circumferential) stresses and longitudinal stresses as shown in Figure Z-9.1-1-8. There are two well-established theories of elasticity and plasticity which relate the actual stresses and strains existing in the pipeline to the simpler uniaxial stress-strain relationship. These theories are the Tresca criteria and the von Mises criteria.

The biaxial stresses will be combined in accordance with the appropriate theory to produce a maximum "effective" stress or "stress intensity" which relates to the uniaxial tensile test. The hoop and longitudinal stresses are the major and

The Design Maximum condition includes sustained loadings for normal operating conditions combined with occasional loadings for extreme ambient influences which may require a shutdown of the system. Design Maximum conditions will occur rarely during the lifetime of the system. The loadings include all sustained loadings plus contingency earthquake, fault displacements, contingency differential settlement, or contingency frost heave.

Stress Criteria Basis

Elastic stress level limitations are imposed upon membrane stresses, the combination of membrane stresses, and those combinations of membrane stresses and bending stress due to primary loadings. For these cases, the pipe is considered fully restrained by the surrounding soil. The hoop stress is limited specifically by 49 CFR 192 on the basis of "Class Locations" and building and population density.

The combined effective membrane stress intensity is limited by the nature of loadings and the hazards involved. The stress intensity, derived from design pressure and temperature differential, is limited to 0.90 SMYS. This value is consistent with the philosophy of industry piping codes such as ANSI B31.3 and B31.4. The stress intensity (from design pressure and temperature differential) combined with design operating earthquake is 1.00 SMYS; and combined with design contingency earthquake is 1.10 SMYS. It is noted that the minimum specified ultimate strength requirement of the API-5LX (5LS), grade 70 pipe is 82,000 psi. Thus, the value of 1.00 SMYS is 85 percent of ultimate strength and the 1.10 SMYS is 94 percent of ultimate strength. In no case will the maximum effective stress intensity be allowed to exceed ultimate strength. These stress levels calculated elastically do not actually constitute stresses in the steel. For example, the temperature differentials and earthquake loading are secondary loads which are partially relieved by the strain when material behavior passes the proportional limit. This approach is merely a simplified calculational technique used in several codes to establish safe limits on plastic strains from secondary loadings when performing an elastic analysis.

The combined membrane stress and bending stress from primary loads is limited to 1.00 SMYS considering the nature of bending stress which varies linearly from a maximum compression to a maximum tension through the cross-section of the pipe. The maximum computed value will occur, therefore, only at the extreme fiber.

minor principle stresses, respectively. The intermediate principle stress, that is normal to the thickness, is assumed to be zero. Thus,

$$S_{eff} \text{ (Tresca Criterion)} = S_h - S_l \geq S_h \text{ or } S_l$$

$$S_{eff} \text{ (von Mises Criterion)} = (S_h^2 - S_h S_l + S_l^2)^{1/2}$$

Where: S_{eff} = Maximum stress intensity

S_h = Hoop stress

S_l = Longitudinal stress

In practice, both the Tresca and von Mises values will be used. The Tresca criterion will provide a slightly more conservative estimate of yield; its use is generally implied within piping codes. The von Mises criterion, on the other hand, more closely complies with test results for pipelines.

The above equations predict the onset of yielding under the combined hoop and longitudinal stresses and are used directly in elastic analyses. They do not indicate any of the inelastic stress and strain behavior beyond the proportional limit. In the elastic range, a unique relationship independent of load history exists between stress and strain. After the initiation of yielding, stresses and strains depend at any time on the previous load history and a unique relationship does not, in general, exist between them. It is, therefore, necessary to deal with stress and strain differentials and then proceed to obtain their total behavior by the appropriate theory.

Inelastic analysis of the pipeline is based upon application of the von Mises formulation. Nonlinear behavior is predicted through computer simulation by using accepted and consistent plasticity theory.

- Membrane Stresses, and Strains - Membrane stresses and strains are those stresses and strains which are uniformly distributed across the cross-section of the pipe wall thickness. Hoop membrane stress is induced by internal pressure; and longitudinal membrane stress is induced in a buried pipeline by internal pressure, differential temperature, and seismic ground motion.

- o Bending Stress and Strain - Bending of the pipeline will induce bending stresses and strains in the pipe wall. In beam analysis the strains are assumed to vary linearly from a maximum compression to a maximum tension through the depth of the pipe. In the elastic range, the corresponding stress also varies linearly and may be added directly to the membrane stress. In the inelastic range, the bending stress distribution is nonlinear.

Basis for Criteria

The Design Criteria is established to ensure safe and continuous operation of the pipeline during the expected lifetime of the system when subjected to design loadings. It incorporates the requirements of the Department of Transportation Regulations (DOT), 49 CFR 192, and is expanded to consider design loading conditions not specifically covered by these regulations. The criteria are in accordance with the industry's practice of arctic pipeline design and are consistent with accepted principles of structural analysis and theories of failure.

The establishment of criteria depends upon the nature of the load which induces stress, whether it is primary or secondary. A primary stress is imposed by a load or force which is not relieved by straining and is developed to satisfy only equilibrium. A secondary stress is due to the pipeline's constraints and is required to satisfy the conditions of compatibility. It is partially or fully relieved by plastic straining. The criteria also depend upon the effect of the stress induced i. e., whether membrane or bending stresses. Due to the nature of membrane stresses, a higher factor of safety is applied.

The establishment of criteria also is contingent upon another important factor i. e., the nature of the loading conditions or the Design Operating and Design Maximum Classification. Through this classification system the criteria can be related to the levels of hazard involved.

The Design Operating condition includes sustained loads such as those caused by internal pressure, temperature, live loads, dead loads, and environmental loads as operating earthquake, differential settlement, overburden, and movements at bends.

Strain Criteria Basis

The supporting soil will deform as a result of forces at bends, differential settlement, differential frost heave, seismic motion, and potential fault displacement. The amount of deformation and resulting stresses and strain is a function of the soil-pipe interaction and require an elastic-plastic analysis for proper assessment. For these cases, additional limitations are imposed on strain levels in order to prevent localized wrinkling of the pipe wall and to maintain a ductile reserve in the steel. This application of an elastic-plastic analysis, with an appropriate limitation on deformation, is an accepted industry practice. Both the Department of Transportation and the Department of Interior approved it for the design of TAPS.

In order to prevent localized bellows wrinkling in the pipe wall, the maximum limit for longitudinal compressive strain is assumed to be 0.6 percent (preliminary). This value is based on data derived from full scale pipe tests performed at the University of California at Berkeley for the Alyeska Pipeline Service Company. The tested pipe was 48 inches in diameter by approximately 1/2-inch wall thickness. It was subjected to bending while pressurized and axially loaded. Analytical studies are currently being performed to investigate the buckling behavior of the heavier wall pipe specified for the project.

The maximum limit for longitudinal tensile strains is assumed to be 0.5 percent (preliminary) i. e., the elongation value used to establish SMYS in accordance with API-5LX (5LS). A tensile strain limit helps mitigate the possibility of initiation and propragation of defects in the pipe or welds. NWA is continuing its fracture control program and studies. This work is expected to verify the tensile strain limit and may demonstrate that higher levels are acceptable.

A design factor is applied to the maximum strain limits based upon the nature of the loadings. The factor is 0.8 for the combination of design operating loads and for construction loadings. The factor is 0.9 for the combination of design maximum loads and for the test condition. The factor is 1.0 for the combination of design operating loads and loads from contingency frost heave, contingency settlement, or fault displacements.

1.14.2 Criteria Summary

- <u>Hoop Stress from Design Operating Pressure Calculated on an Elastic Basis (49 CFR 192)</u>

Class Location	Design Factor (F)
1	0.72 SMYS
2	0.60 SMYS
3	0.50 SMYS
4	0.40 SMYS

- <u>Combined Membrane Stress in Restrained Pipe Calculated on an Elastic Basis (Tresca Criteria)</u>

Effective stress intensity from design pressure and temperature differential	0.90 SMYS
Effective stress intensity from design pressure, temperature differential, and design operating earthquake	1.00 SMYS
Effective stress intensity from design pressure, temperature differential, and design contingency earthquake	1.10 SMYS

- <u>Combined Membrane and Bending Stress (From Primary Loads) Calculated on an Elastic Basis (Tresca Criteria)</u>

Effective stress intensity from design pressure, temperature differential, and hoop bending stress due to overburden or longitudinal bending stress due to buoyancy	1.00 SMYS

- <u>Combined Membrane and Bending Strain</u> - Combined membrane and bending strain will be calculated by elastic/ plastic analysis and will include the consideration of movement of the supporting soil. The tentative criteria for allowable longitudinal strain is as follows.

Longitudinal compressive strain (ε_c) at which localized bellows wrinkling is incipient.

Operation	0.60 Percent
Construction	0.35 Percent
Longitudinal tension strain (ε_t) selected to maintain ductile reserve	0.50 Percent

For design purposes, the allowed maximum total longitudinal strain that is computed inelastically will be reduced by the appropriate design factors as outlined below.

	A	B	C
Pressure, temperature differential, design operating earthquake, bend movement, and settlement or frost heave	0.8	0.48	0.40
Pressure, temperature differential, design contingency earthquake, bend movement, and settlement or frost heave	0.9	0.54	0.45
Test pressure, temperature, differential and bend movement	0.9	0.54	0.45
Pressure, temperature differential, movement at fault crossings, and design contingency earthquake	1.0	0.60	0.50
Pressure, temperature differential, design operating earthquake, and contingency frost heave or contingency settlement	1.0	0.60	0.50
Construction loads	0.8	0.28	0.28

Column A - Design Factor
Column B - Total Longitudinal Compressive (%)
Column C - Total Longitudinal Tensile (%)

- <u>Elastic Stability</u> - When the pipeline is considered as a straight column with fixed-end conditions, the maximum unsupported span caused by soil settlement or buoyant uplift will be limited to a 50 percent of the critical span.

- <u>Ovalling During Construction</u>

Maximum circumferential stress due to weight of overburden under zero internal pressure to protect against collapse of the pipe	0.80 SMYS

<u>Design Procedures</u>

The sound structural design of the pipeline has been evolved by systematically applying analytical procedures which will facilitate interfacing among various aspects of the design such as geotechnical, route refinement, compressor stations, stress analysis and structural design, fracture mechanics, river engineering, hydraulics, and environmental constraints.

The minimum required wall thickness is determined in accordance with 49 CFR 192 by class location based upon the design requirements for pressure. Elastic analysis has been performed for all primary loads and secondary loads which induce membrane stress. The analysis were conducted in order to verify compliance with the established criteria. Maximum stress levels are controlled, as required, by limitations on pipeline tie-in temperature, limitations on the depth of cover, or limitations on allowable free spans. Nonlinear analyses are then be performed in order to determine the effects of the secondary geotechnical loading. Parametric studies on pipe-soil interaction are conducted in order to ascertain conditions of bend configurations, differential settlement, differential frost heave, seismic effects, and fault crossings. The objective of these studies is to define structural and geotechnical guidelines for establishing the below ground configurations. Additionally, established below ground concepts will be used to assist in refining the route. Pertinent technical and economic factors encompassing the whole range of terrain conditions and topographies will also be considered.

Based upon the results of the parametric studies and interaction with other pipeline engineering disciplines, a series of design tables and curves representing allowable geotechnical loads and displacements will be produced for use in the final mile-by-mile design.

A number of special detailed analyses will be required during the final design phase. Some of the items included in these analyses are discussed below.

- <u>Below Ground Bend Restraint</u> - For given operating and soil conditions, allowable bend angles will be determined which do not require mechanical restraint. In order to optimize the alignment for special conditions, it may become necessary to use greater bend angles that require mechanical restraint. This situation will be evaluated and designed to minimize anchoring requirements.

- <u>Traps and Valves</u> - Traps and valves will involve the analysis and design of valve and piping supports, bend offset details, anchor requirements, site grading and drainage, access roads and fences required at trap and valve sites. Pipeline transitions from below ground to aboveground will be evaluated to ensure against the movement of valves and traps.

- <u>River Crossings</u> - All river crossings encountered along the route will be investigated and classified on the basis of the construction and design problems associated with each one. Standardized designs will be established for each classification group.

- <u>Road, Railway, Pipeline, and Other Crossings</u> - The structural integrity of the pipeline, as well as that of the structures crossed by the pipeline will be assured by configuration and location design. Loading conditions will be considered during construction and operation of the pipeline.

1.14.3 <u>Basic Data</u>

A summary of the pipe's structural properties are shown in Table 1-1. The basic data used in the preliminary analyses are outlined below.

<u>Elastic Stress Analysis</u>

When a pipeline is placed in service, it is subjected to nominal stresses and deformations. These stresses and deformations are limited in accordance to the design criteria in order to maintain the integrity and serviceability of the pipeline.

The stresses induced in the pipeline can be classified as membrane stresses and bending (or flexural) stresses. The membrane stresses are developed by gas pressure, temperature differential, and seismic movement during the operation of the pipeline. The gas pressure loading will be the operating gas pressure which will be essentially constant throughout the line. The temperature differential is the difference between the maximum or minimum operating temperature of the flowing gas and the tie-in temperature. The tie-in temperature is the temperature of steel when the pipeline is installed in the ditch and becomes essentially restrained. Seismic loadings have different magnitudes in different zones as required by the stipulations. For the elastic analysis, the seismic stress is related to the pipe strain caused by the ground strain.

The buried pipeline will be subjected to axial loadings due to internal pressure, restraint of the pipeline by the surrounding soil, and temperature changes. As long as the pipeline is continuously supported by the ditch bottom, there will be no substantial longitudinal bending stresses. However, sections of pipeline subjected to buoyant uplift force or to conditions of free span will undergo such primary bending stress. Circumferential bending stress will be induced by the pipe overburden.

Elastic analyses are performed by the procedures outlined here. Stress levels are verified to comply with the criteria. Maximum depth of cover and maximum unsupported pipe spans are determined.

Internal Pressure

Hoop stresses due to internal pressure in the pipe are limited according to the Department of Transportation Code, 49 CFR 192. For a given operating pressure, the wall thickness required to meet this limitation is calculated using Barlow's formula in the following form:

$$t = \frac{PD}{2S} \times \frac{1}{F} \times \frac{1}{E} \times \frac{1}{T} \qquad (1.1)$$

Where: t = pipe wall thickness, in

P = design operating pressure, psig

D = pipe outside diameter, in

S = specified minimum yield, psi

F = design factor

E = longitudinal joint factor

T = temperature derating factor

The factors F, E, and T are determined in accordance with Federal Standards 192.111, 192.113, and 192.115, respectively, and are equal to 1.0.

Under a given set of loading conditions, the component of hoop stress resulting from internal pressure is determined using Barlow's formula as presented below:

$$S_h = \frac{PD}{2t} \tag{1.2}$$

Where: S_h = hoop stress, psi

P = internal pressure, psig

D = outside diameter, in

t = pipe wall thickness, in

In the unrestrained condition, longitudinal stresses are induced by longitudinal expansion of the pipeline and are defined by the product of the internal pressure and the area of the pipe divided by the area of the steel.

$$S_1 = \frac{A_p\ P}{A_s} \tag{1.3}$$

Where: S_1 = longitudinal stress, psi

A_p = internal area of the pipe, in²

P = internal pressure, psig

A_s = area of pipe steel, in²

In the restrained condition, longitudinal stresses are induced by Poisson's effect due to radial expansion. These stresses are defined by the product of Poisson's ratio and the hoop stress:

$$S_1 = \mu \frac{PD}{2t} \qquad (1.4)$$

Where: S_1 = longitudinal stress, psi

P = internal pressure, psig

D = pipe outside diameter, in

t = pipe wall thickness, in

μ = Poisson's ratio, 0.3

Differential Temperature

Longitudinal membrane stresses induced into a fully restrained pipeline by thermal expansion or contraction are defined by:

$$S_t = E\ (-\alpha)\ (T-T_i) \qquad (1.5)$$

Where: S_t = longitudinal stress, psi

E = steel modulus of elasticity, psi

α = steel coefficient of thermal expansion in/in/°F

T = temperature in question, °F

T_i = installation temperature, °F

Earthquake Elastic Stress

The response of the buried pipeline to earthquake motions is discussed under analysis of earthquake effects. Current earthquake engineering studies are being conducted to better define the seismic-induced interaction between pipe and soil and the resulting pipe stresses and strains. For the preliminary stress analysis the following equation is considered conservative.

$$S_1 = \varepsilon_a E \qquad (1.6)$$

Where: S_1 = longitudinal membrane stress, psi

ε_a = maximum longitudinal membrane strain, in/in

E = pipe modulus of elasticity, psi

The maximum longitudinal strains from axial deformations are calculated using the following equations:

$$\varepsilon_a = \frac{V}{2C_s} \qquad (1.7)$$

Where: C_s = seismic shear wave velocity, ft/sec

V = maximum soil particle velocity, ft/sec

Overburden Load

The weight of the backfill above the pipe will deform and induce a hoop stress in the pipe. The overburden load may be taken as equal to the weight of soil prism above the pipe:

$$W = \Upsilon HD \qquad (1.8)$$

Where: W = overburden load, lb/ft

Υ = backfill unit weight, pcf

D = pipe outside diameter, ft

H = height of fill above the top of the pipe, ft

Unpressurized Pipe - The deformation of the unpressurized pipe can be expressed in terms of the backfill load, the properties of the pipe, and the properties of the backfill around the pipe. The deformation is calculated by the expression derived by Spangler (1973):

$$\Delta X = \frac{D_1\ K\ W\ R^3}{12(EI + 0.061\ E'R^3)} \qquad (1.9)$$

Where: ΔX = reduction in the vertical diameter, in

D_1 = deflection lag factor

K = bedding constant

R = pipe radius, in

E = steel modulus of elasticity, psi

E' = modulus of soil reaction of the sidefill material, psi

I = pipe moment of inertia, in^4

W = overburden load, lb/ft

Substituting the following values:

D_1 = 1.25 (as recommended by Spangler)

K = 0.108 (for a bedding angle of 30 degrees)

R = 24 in

E = 29×10^6 psi

I = $\frac{t^3}{12}$, where t = pipe wall thickness

Υ = 130 pcf

E' = 0 (assuming a poorly compacted sidefill)

$$\Delta X = 0.0335 \frac{H}{t^3} \qquad (1.10)$$

The hoop stress induced by the weight of the backfill on the unpressurized pipe is calculated on the assumption that the cross-section of the pipe will deform into an elliptical shape. For $\Delta X << D$, the hoop stress is given by:

$$S_b = 3.3\ E \frac{t}{D^2} \Delta X \qquad (1.11)$$

Where: S_b = hoop stress, psi

D = pipe outside diameter radius, in

E = steel modulus of elasticity, psi

t = pipe thickness, in

ΔX = reduction in vertical diameter, in

<u>Pressurized Pipe</u> - The deformation and hoop stress induced by the weight of the backfill on the pressurized pipe are calculated based on the conservative relationships developed by Spangler (1955).

The deformation is given by Spangler as:

$$\Delta X = \frac{0.108\ W\ R^3}{12(EI + 0.216\ PR^3)} \qquad (1.12)$$

Where: ΔX = reduction in the vertical diameter, in

W = overburden load, lb/ft

R = pipe radius, in

E = steel modulus of elasticity, psi

I = pipe moment of inertia, in^4

P = internal pressure, psi

Substituting the appropriate values:

$$\Delta X = \frac{64696\ H}{2.4 \times 10^6 t^3 + 2986\ P} \qquad (1.13)$$

The hoop bending stress is given by Spangler as:

$$S_b = \frac{0.117\ W\ E\ t\ R}{Et^3 + 2.592\ p\ R^3} \qquad (1.14)$$

Where: S_b = hoop bending stress, psi

P = internal pressure, psi

E = steel modulus of elasticity, psi

t = pipe wall thickness, in

W = overburden load, lb/ft

R = pipe radius, in

Using the values given above for the different terms in the equation, the hoop stress is expressed as:

$$S_b = \frac{42345\ H\ t}{29t^3 + 0.035831\ p} \qquad (1.15)$$

Structural Stability

When the pipeline is subjected to lateral loads such as settlement or buoyant uplift, it reacts like a beam-column and will fail due to structural instability if a compressive axial load exists and if the distance between anchored points exceeds the critical span length for stability. This critical span length can be found by using the Euler formula in the following form:

$$L = \frac{\Pi^2\ E\ I}{36\ F_t}\ ^{\frac{1}{2}} \qquad (1.16)$$

Where: L = critical span length for structural stability, ft

E = steel modulus of elasticity, psi

I = pipe moment of inertia, in^4

F_t = total axial force, lb

The total axial force (F_t) is the sum of the forces resulting from internal pressure and temperature differential. It is determined as follows:

$$F_t = F_1 + F_2 + F_3 \qquad (1.17)$$

Where: F_1 = axial force due to internal pressure, lb

F_2 = axial force due to temperature differential, lb

F_3 = axial force in contained gas, lb

and:

$$F_1 = A_s\ \mu\ \frac{PD}{2t}$$

$$F_2 = A_s E\ (-\delta)\ (T-T_i)$$

$$F_3 = PA_p$$

A_s = cross-sectional area of pipe steel, in^2

A_p = cross-sectional area of the pipe, in^2

P = internal pressure, psi

D = nominal pipe diameter, in

The standard sign convention (positive for tension and negative for compression) is used for all calculations.

Note that term PA_p does not develop a stress in a straight section of the buried line, but adds to the bending moment if the pipe is bent by some loading condition.

Results of Analysis - Results of the analysis and combinations of stress are shown in Tables 1-2 through 1-10. These results demonstrate compliance with applicable codes and the criteria.

Elastic-Plastic Analysis

The elastic-plastic analyses is performed on the pipe in order to consider the effects of secondary loadings produced by geotechnical conditions.

The objective of the elastic-plastic representation of structural behavior is to determine either load-deformation or load-strain relations for the structure and/or its individual elements. Elastic-plastic analyses are based upon an idealization of actual material properties and the simplified solution process. The analyses are complex mathematical models requiring the use of computer programs for their solution. The stress-strain relationship, as represented by a realistic stress-strain curve, is a basic input required for completing these computations.

Figure Z-9.1-1-9 shows a typical curve for a 48-inch diameter API-5LX/5LS, Grade 70 pipe and the idealization used for all preliminary analyses. An idealization of the soil's response as it interacts with the pipe, constitutes the second important input required for the analysis. The soil is generally modeled as bilinear springs as shown in Figure Z-9.1-1-10.

Studies have been conducted: (1) to investigate pipe stress and strain under anticipated design loading conditions and the resulting pipe-soil interaction for the significant design considerations; and (2) to demonstrate the analytical techniques which are being used by the design group for structural design of the pipeline. For final design, the soils will be classified in a manner that will correlate with actual field conditions.

The objective of the studies is to determine the limits of pipe displacement and deformation in terms of the strain levels defined by the criteria. Via the analysis, pipe curvature changes occurring under design loadings can be related to the strain levels. Once determined, these curvature limits may be used in the planned operation monitoring program.

The results of the studies will be used to develop the following tools for the final design:

- o Design tables representing the maximum allowable pipeline bend angles versus the depth of cover over the pipe.
- o Curves representing allowable differential soil settlement versus span of settlement based on depth of burial of the pipe.
- o Curves representing allowable differential displacement of the pipe caused by frost heave versus the span of the heaving section.
- o Verification of the adequacy of special design for identified fault crossings.
- o Analysis of special crossing designs.

In order to properly and economically consider these cases, the various loading conditions and complex interaction between the pipe and soil will require the use of several computer programs. The basic program to be used is PIPLIN. Other programs which will be used to supplement this work when it is necessary to consider dynamic and/or three-dimensional effects as described in detail in Appendix E, Exhibit Z-9.0 are:

PIPLIN
PIPANL
ANSR
DRAIN PIPE
ANSYS
SABOR/DRASTIC 7
EDS-SNAP

1.14.4 Bend Design

Soil Load/Deformation Functions - Computer programs, such as "PIPLIN," that are being used for pipeline stress analysis require soil load/deformation functions as input. These functions are approximations of the actual relationship between the soil resisting force, R, and the soil deformation, δ, caused by pipe displacement. These interaction functions have to be predicted for pipe displacement in the vertical (upward and downward) and the horizontal (axial and transverse) directions. These functions are needed for bend design and for evaluation and design of the buried line when subjected to differential movements. The latter includes situations such as differential frost heave, fault crossings and differential settlements.

Past practice for bend design in unfrozen soil has adequately approximated soil load/deformation by a simple elastic-plastic function as shown in Figure Z-9.1-1-11 The elastic-plastic approximation consists of a linear elastic portion and a horizontal plastic portion that represents the ultimate soil resistance. The intersection of these two portions represents the yield displacement at which ultimate soil resistance is reached. The criteria for obtaining load/deformation functions for bend design is discussed below.

Changes in pipe temperature or internal pressure will produce changes in axial stress along the pipeline. Pipe axial displacement will be restrained by soil friction along the pipeline and by the passive soil reaction force developed at bends. Excess pipe movement at bends or at transitions between belowground and aboveground sections (at compressor stations) is to be avoided. The design criteria will ensure that the pipe is adequately restrained.

Bend design criteria are based on conventional practice established for unfrozen soil. The only modification necessary is a special provision for sidebend and sagbend design when burial is in ice-rich permafrost.

When the pipe is buried in thaw stable permafrost or permafrost free soil, conventional practice based on unfrozen soil properties is both appropriate and conservative.

When the pipe is buried in ice-rich or thaw unstable permafrost special construction techniques will be used to prevent prestartup thawing of the surrounding soil. Overbend design and axial restraint (skin friction) criteria are conservatively based on the unfrozen properties of the backfill which may be unfrozen at startup. Sidebend and sagbend design criteria are based on the properties of the ice-rich natural soil.

Ice-rich frozen soils have very high immediate shear resistance, but may creep and allow excess pipe deformation if high stresses are sustained over a long time. For sidebend and sagbend design in ice-rich soil, the design shear strength (used to calculate soil resistance) will be selected to prevent excess creep deformation over the 25 year design life of the pipeline.

Bend analysis done by APSC established that the initial soil modulus has little influence on allowable bend angles. Design is almost exclusively controlled by the ultimate soil resistance. The detailed criteria for obtaining ultimate soil resistance are described below, with the soil resistance (R) expressed as a force per unit length of pipeline. These criteria are for static loading with hydrostatic ground water conditions. Conditions due to earthquake loading, including dynamic effects, loss of support or excess soil pore water pressure have not been included.

The hydrostatic pore water pressure influences the soil resistance through the effective density, γ, of the soil. When the water table is at the soil surface, the buoyant soil density will be used in design. The total soil density will be used only where the water table remains well below the bottom of the pipe throughout the year.

Axial Restraint

Friction between the select padding and bedding and the pipe provides the pipe with axial restraint. Axial restraint is being conservatively calculated for the unfrozen condition that may represent startup.

$$R = \pi D \gamma (C + \frac{D}{2}) \quad (\frac{1 + K_o}{2}) \quad \tan \beta$$

Where β is the angle of friction between the pipe and the selected padding and bedding, as shown in Figure Z-9.1-1-12(a), and D, C, γ, and K_o are as defined in Figure Z-9.1-1-12(b). For preliminary design, a conservative value of 20° will be used for β. A final design value may be established on the basis of the actual coatings selected. The pipe displacement required for maximum soil resistance is small, of the order of 0.1 to 0.3 inches, as revealed by extensive pile test data.

Sidebend Restraint

Outward horizontal forces are produced by the pipe at sidebends. These outward forces are restrained by the natural soil at the ditch wall. The compacted granular padding and bedding will assure good load transfer between the pipe and the ditch wall.

For cohesionless soil, the soil resistance is being calculated using the modification of anchor wall resistance suggested by Ovensen and Stromann.

$$R = \tfrac{1}{2}\ \gamma\ H^2\ \ A\ \ \tan^2\ \left(45 + \frac{\phi}{2}\right)$$

γ, H and ϕ are as defined in Figure Z-9.1-1-12(b). The reduction coefficient A is given in Figure Z-9.1-1-13(a). The soil yield displacement may be of the order of 1 percent to 4 percent of the embedment depth, H.

For frozen or unfrozen cohesive soils, the soil resistance is being calculated by a method proposed for anchor slabs by Tschebotarioff.

$$R = KsD$$

Where K is the lateral resistance coefficient given in Figure Z-9.1-1-13(b). For unfrozen, cohesive soils, s is the undrained shear strength. For frozen soils, s is ultimate, long-term shear strength that will prevent excess creep deformation in the frozen, native soil over the 25 year design life of the pipeline. The yield displacement may be of the order 2 percent to 6 percent of the embedment depth.

Overbend Restraint

Overbends are being designed to resist upward pipe thrust. Resistance against uplift is provided primarily by the soil backfill material. It will be conservatively calculated for the unfrozen condition that may represent startup. The pipe weight or buoyancy is covered in another section and is an additional load that is being included in overbend design. The passive resistance of the backfill consists of two parts, the fully mobilized weight of the overburden and the shear resistance of the overburden.

$$R = \gamma\ DC + \gamma(C + \frac{D}{2})^2\ K_o\ \tan\ \phi$$

Where γ, D, C, K_o and ϕ are as defined in Figure Z-9.1-1-12(b). The soil yield displacement may be of the order 2 percent to 5 percent of the depth to the springline, (C + ½ D).

Sagbend Restraint

Sagbends are being designed to resist downward pipe thrust. The pipeline will act essentially as does a strip footing with the ultimate bearing capacity established in the native soil beneath and adjacent to the trench. For the most general case

$$R = [cN_c + H\gamma(Nq - 1) + \tfrac{1}{2}\gamma BN_\gamma]B$$

N_c, N_q, and N_γ are bearing capacity factors commonly used in foundation design (Vesic, 1975) and c is the cohesion intercept of the soil. H, B and γ are as defined in Figure Z-9.1-1-12(b), except for the value of γ associated with N_γ. In this special case, the total soil density, γ_t, will be used only when the water table is at least one pipe diameter, 4 feet below the bottom of the pipe.

For unfrozen, cohesive (ϕ = 0) soils and for frozen soils, the equation reduces to

$$R = sN_c B$$

For unfrozen, cohesive soils, s is the undrained shear strength. For frozen soils, s is the ultimate, long-term shear stress that will not produce excess creep deformation over the 25 year design life of the pipeline.

Both equations account for the overburden load on the pipe, but not for the weight or buoyancy of the pipe, which must be included as an additional force. The soil yield displacement will be of the order 5 percent to 15 percent of the bearing width of the pipe, B.

Preliminary bend design was based on an unfrozen silt with ϕ = 26° and the densities shown below.

SOIL DENSITIES

	Dry Density (pcf)	Saturated Density (pcf)	Buoyant Density (pcf)
Undisturbed Soil	80	112	50
Nominally Compacted Backfill	90	120	58

These values were used to estimate suitable preliminary design elastic-plastic soil/deformation functions for the uninsulated 4 foot diameter pipe under 2-1/2 feet of cover, as shown in Figure Z-9.1-1-12(a). The calculated values shown below do not include the weight or buoyancy of the pipe.

UNFROZEN SOIL RESISTANCE AT BENDS

	Axial	Sidebend	Overbend	Sagbend
Yield Displacement (inches)	0.1-0.3	1-3	1-2	3-7
Ultimate Resistance (kips/ft)				
Dry Soil	1.4	4.2	1.3	8.2
Saturated, Low Water Table	1.8	5.9	1.8	11.5
Saturated, High Water Table	0.9	2.6	0.9	5.1

Time and Temperature Dependent Deformation of Frozen Soils

For design conditions that are load controlled, such as bend design, frozen soil properties are best represented by a series of creep tests. A frozen soil normally creeps under constant load (stress). Three stages of creep can be distinguished in terms of changing strain rate, as shown in Figure Z-9.1-1-14(a). After an initial elastic strain, the strain rate is high, but decays with time (primary creep), then remains nearly constant (secondary creep), and finally may begin to accelerate (tertiary creep). The beginning of tertiary creep is often regarded as the beginning of soil failure. However, for design purposes failure is defined at the maximum allowable soil strain that will not cause excess deformation in the pipeline.

The allowable frozen soil strength for design is being established from the results of creep tests under different loads and at different temperatures, as shown in Figure Z-9.1-1-14(b). The larger the load, the larger the strain and strain rate at any time, and the shorter the time to creep failure. An increase in temperature produces results similar to an increase in stress. A series of creep tests is used to define the frozen soil stress in terms of strain, temperature and load duration. The creep strength can be established at an allowable strain and expressed as a function of temperature and load duration as shown in Figure Z-9.1-1-14(c). Methods are available to extrapolate the results to long time periods. The allowable stress for bend design will be the 25 year creep strength at the appropriate soil temperature.

For frost heave related displacement and displacement rate controlled conditions, the procedure for calculating soil reaction described above may not be sufficient. The stress in the soil is a function of strain, temperature and applied strain rate. At a given soil strain, the stress decreases with decreasing strain rate, and decreases with increasing temperature. The relationship between the peak stress and strain rate corresponds to the relationship between creep stress and secondary creep rate. For frost heave design, soil resistance beyond the strain at peak strength may be important. A laboratory testing program designed to investigate and characterize these factors for project design is underway.

Frost Heave Interaction Effects

When a chilled pipeline is buried in unfrozen soil differential frost heaving may cause one section of the pipe to heave at a rate different than adjacent sections. The restraining action of frozen soil will result in pipe stress. Calculations of uplift resistance as used for preliminary design in "PIPLIN" assume simple elastic-plastic behavior of frozen soil and are based on the following equation.

$$R = 2s \left(C + \frac{D}{2}\right)$$

Where R is the ultimate uplift resistance (Kips/ft), s is the shear strength of the frozen backfill (Kips/ft^2), C is the thickness of the frozen cover (ft) and D is the diameter of the pipe plus insulation (ft). Additional terms can be included to account for the weight of backfill and the strength of any additional thawed cover, but these terms are small compared to the uncertainty associated with the assumption of elastic-plastic behavior of frozen soil.

For preliminary design purposes only, uplift resistance values were estimated for a 4 foot diameter pipeline with 6 inches of insulation buried under 2-1/2 feet of sand backfill cover. Shear strengths were chosen from a synthesis of published values to be representative rather than conservative. They account for the effect of soil temperature and include an estimation of the effect of load duration. A range of ultimate uplift resistance values were obtained for the summer and winter conditions as tabulated below.

FROZEN SOIL UPLIFT RESISTANCE

	Winter	Summer
Frozen Cover Thickness	2-1/2 ft.	1 ft.
Temperature	20°F	31°F
Loading Duration	8 months	4 months
Shear Strength		
Dense Sand	20 Kips/ft^2	5.5 Kips/ft^2
Loose Sand	11 Kips/ft^2	3.0 Kips/ft^2
Uplift Resistance		
Dense Sand	200 Kips/ft	39 Kips/ft
Loose Sand	110 Kips/ft	21 Kips/ft
Yield Displacements	1-3 inches	1-3 inches

As an important soil/pipe frost heave interaction effect, consider the worst-case frost-heaving situation shown in Figure Z-9.1-1-15. The pipeline crosses a perfectly abrupt transition between uniformly nonfrost-susceptible soil and uniformily highly frost-susceptible soil. The pipeline is caught in a "guillotine" between heaving and nonheaving frost bulbs. Relief is provided only by "crushing" the frozen soil below the heaving section and above the nonheaving section. The pipe is forced to bend into an S shape over a short span, creating high stress and strain in the pipe.

This analysis is unrealistic, and will overestimate pipe stress. The actual load will be redistributed and the effective bending length increased due to several interactive factors discussed separately below. The mitigative effects of these factors at potential frost heave transitions will be considered in final design.

- It is known that increased pressure at the freezing front will reduce the rate of frost heaving. The increased pressure in the heaving soil near the transition zone will reduce the differential frost heave in this critical region.

- Frost Bulb Interaction - Shear resistance will be mobilized in the frost bulb across the transition zone. The shear will be transmitted along the frost bulb by its bending resistance. The stiffness of the frost bulb will reduce the stress in the pipe.

- Summer Relief - The uplift resistance above the slower heaving section will be greatest in the winter, when the soil above the pipe is completely frozen and at its lowest temperature. If the pipe can withstand the winter heave, summer may bring relief. The uplift resistance will decrease as the soil above the pipe warms and partially thaws. Analysis is difficult, and must be made in a series of time steps. The uplift resistance will depend not only on the changing temperature, the thickness of the frozen soil and the heave rate, but also on the hereditary accumulation of soil strain.

- Initial Relief Due to Small Frost Bulb - For a period after startup, the frost bulb will be small. If it is not bonded to permafrost and the seasonal layer is thawed, then the uplift resistance will be established by the whole frost bulb being lifted, shearing the unfrozen soil above it. This resistance can be calculated by conventional means provided the size and shape of the frost bulb is known. Although this provides stress relief only for a short period after startup, this may be the period of highest heave rate.

- Fractural Relief - As high tensile strains accumulate in the frozen soil above the pipe, the soil may fracture, reducing the uplift resistance. More needs to be known about the roll of soil tension in uplift resistance before the amount of relief can be estimated. Further, there may be healing due to water infiltrating and freezing in the cracks.

- o Creep Compression Relief - A developing ice-rich frost bulb will be subject to heaving forces which will cause the frost bulb to creep, particularly in the warmer zones close to the frost front. This can significantly decrease heave rates and displacements by lateral spreading and densification of the frost bulb.

Fault Crossings

Pipeline fault crossings require site-specific crossing designs. Criteria for buried designs focus on providing sufficiently small soil resistance to pipe movement along the crossing such that the pipeline can safely accommodate the fault movement. Geotechnical input will consist of load/displacement functions appropriate for each design configuration as well as detailed examination and design integration of site-specific geotechnical conditions at each individual fault crossing.

Other Pipeline Crossings

At certain locations along the alignment the pipeline will cross the TAPS pipeline, fuel gas pipeline, and other pipelines. These crossings will require site-specific designs. Geotechnical input will consist of load/displacement functions appropriate for each design configuration as well as detailed examination and design integration of site-specific geotechnical conditions at each individual crossing.

Differential Settlement

A buried pipeline essentially will be uniformly and continuously supported by bedding material provided in the ditch bottom. As long as this support remains in contact with bottom of the pipe, no significant longitudinal bending stresses will develop in the straight pipe.

It is recognized, however, that certain geotechnical conditions could cause vertical movement of the supporting soil. Terrain disturbance occurring during construction may cause thawing of the permafrost and induce some thaw settlement prior to operation and creation of a frost bulb. Erosion or scour in stream beds may result in free-spanning pipe over some distance, thus causing the pipe to carry its own weight, the weight of the contained gas, and the load contributed by the soil above. For short sections, the pipe will be able to span the distance regardless of the amount of settlement without exceeding the elastic stress criteria. For longer spans, it is necessary to determine the amount of differential settlement which may be allowed. The vertical displacement of the pipe will be resisted by the stiffness of the pipe, by the strength of the supporting soil on each side of the span of settlement, and by the longitudinal restraint provided by the soil.

Preliminary parametric studies have been conducted to investigate pipe stress and strain under anticipated design loading conditions and the resulting pipe soil interaction. These studies have been run, (1) to identify critical configurations which will then be subjected to detailed investigation, and (2) to demonstrate the analytical techniques being used.

The computer program PIPLIN-II has been used for all typical analyses. The pipe characteristics and design conditions used for all analyses have been outlined in the section on design procedures. The stress-strain properties of the steel pipe are input as a number of points from a plot based on typical data from pipe manufacturers (see Figure Z-9.1-1-9). Two particular differential settlement situations have been modeled: a gradual settlement following a sinusoidal curve and an abrupt step function settlement. In both cases, the pipe is completely restrained by the soil at each end and with loads due to pressure, temperature change, and dead weight applied to the pipe, the ground is then allowed to settle away from below the pipe. Two different settlement depths were considered for both cases, and the additional effect of varying the span length was investigated for the gradual settlement case. The results of the analysis demonstrate that for depths of cover up to 10 feet, the pipeline can sustain the effects of soil settlement of 18 inches without exceeding the preliminary criteria limits.

1.14.5 Analysis of Earthquake Effects

The following seismic design procedures will be applied to the pipeline to protect it from the effects of earthquakes. These effects include:

- o Seismic shaking
- o Fault displacements (ground deformation)
- o Soils hazards (mass movements)

The risk resulting from these earthquake effects will be minimized by the use of one or more of the following methods:

- o Avoidance of the hazard, where practical.
- o Mitigation of the hazard, through site-specific design and analysis.
- o Qualification of representative pipeline segments, by analysis of earthquake effects.

The earthquake ground motions which will be used in the design of the pipeline have been established by Dr. Newmark. The analysis of ground shaking effects on buried pipeline segments will utilize the effective maximum ground motions for soils and buried structures defined by Dr. Newmark and is stated in Table 1-1. The minimum wave propagation velocity for various soils will be taken from the values quoted by Newmark. The analysis of above ground segments of pipeline and support structures will use the design response spectra defined by Newmark and will use equivalent ground motion time histories where appropriate.

The structural acceptance criteria are defined in the Criteria Summary. Their application to earthquake-related effects is as follows:

<u>Buried Pipeline</u>

Stress Criteria (Elastic Analysis)	Straight Pipe; membrane stresses due to ground shaking in combination with those due to pressure and temperature.
Strain Criteria (Elastic-Plastic Analysis)	Curved Pipe; membrane strains and secondary bending strains due to ground shaking in combination with strains due to pressure, temperature differential, settlement, and frost heave.
	Straight and Curved Pipe; strains due to fault displacements or soils hazards, combined with strains due to pressure, temperature differential, settlement, frost heave, and ground shaking.

Elevated Pipeline (Special Crossings)

Stress Criteria (Elastic Analysis) as defined in applicable code.

Support Structures (Bridges and cables)

Stress Criteria as stated in:

AISC Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings, latest edition.

American Concrete Institute - ACI Standard 318, latest edition.

AASTHO - Standard Specifications for Highway Bridges, latest edition (for the Yukon River Bridge).

AISI Manual for Structural Application of Steel Cables for Buildings (for cable-supported structures), latest edition.

Ground Shaking, Straight Buried Pipeline

Propagating seismic waves induce strain in a buried pipeline by causing relative displacements along its length. The small relative displacements induced across the pipeline diameter induce very small hoop bending strains which will be neglected. Displacements which are normal to the pipeline axis and which vary along the axis induce longitudinal bending and shear strains in the pipeline. The longitudinal bending strains in the straight pipeline are negligible and will not be included. The shear strains will be evaluated and included in the calculation of effective stress. The most significant strains induced in the straight pipeline are the longitudinal membrane strains induced by axial displacements which vary along the pipeline length. The maximum longitudinal strains caused by a propagating shear wave can be estimated as follows (Newmark):

$$\varepsilon_a = \frac{V}{2C_s}$$

Where: ε_a = maximum axial strain

V = maximum soil particle velocity

C_s = minimum wave propagation velocity

Provided the following conditions are satisfied:

- o The pipeline is buried in homogeneous soil.
- o There is no reduction in soil strain due to the presence of the pipeline (no pipe/soil relative stiffness effects).
- o There is no pipe/soil, pipe/frost bulb, or frost bulb/soil slippage.
- o Bending strains caused by the components of ground motion which are normal to the pipeline axis are negligible.

The application of this expression typically results in a conservative calculation of strain in the pipeline. However, in order to confirm and reduce this conservatism, a study will be performed as follows:

- o An assessment will be made of the potential for increased near-surface soil strain resulting from abrupt geological discontinuities or the condition of soft soils overlying firm soils or rock.
- o An evaluation will be made of the soil load/ displacement characteristics with respect to longitudinal pipeline motion.
- o Longitudinal strain and stress in the pipeline due to propagating seismic waves will be calculated.

Based on these analyses, design guides will be developed to estimate earthquake-induced longitudinal pipeline strain.

Ground Shaking, Buried Pipeline at Bends

Due to the geometric configuration at bends, traveling ground waves induce longitudinal bending strains as well as longitudinal membrane strains, shear strains, and negligible hoop bending strains in the pipeline. The amount of longitudinal bending caused by seismic motion depends upon the nature of the traveling waves, the operating conditions of the pipeline, the geometry of the bend, and the soil resistance at the bend location.

An evaluation of pipeline bends will be made considering the following parameters:

- Seismic wave propagation velocity and direction.
- Ditch backfill and surrounding soil stiffness effects, including frost bulb effects.
- Bend type - overbend, sagbend, sidebend.
- Bend radius and angle.

The above analyses will be used to develop design guides for the final design of the pipeline.

Additional site-specific analyses will be made as necessary for special cases such as tight multiple bend configuration at road crossings, Alyeska pipeline crossings, entries to compressor stations, sections of cased pipe where direct soil support to the pipe is prevented, and at intermediate traps, valves, and T-junctions. Effects of anchors on the seismic response of the pipeline will also be investigated.

To date, the preliminary elastic analyses of the seismic response of a typical bend configuration under average soil conditions yielded peak strains at the crown of the bend which were found to be within design limits.

Special Crossings

At certain locations along the pipeline, such as at major river crossings, tunnels, and compressor station interfaces, elevated pipeline configurations may be adopted. Once the pipeline is elevated, its response to earthquake-induced ground motions differs from the buried-mode behavior. Hence, the required analysis methodology also differs. Elevated pipeline segments will be designed and analyzed to meet dynamic considerations. In addition to the pipeline, the support structures (bridges and piers) will be designed and analyzed to consider seismic-induced loadings. Earthquake response spectra which will be used for the design and analysis are based upon Newmark's seismic criteria. These criteria will be established for specific application to above ground pipeline and structures for Design Operating Earthquake and Design Contingency Earthquake.

The effects of wave propagation over long elevated spans will be studied. In transition zones where the pipeline shifts from elevated to buried mode, analysis models will include below ground segments of sufficient length to develop virtual anchors.

Fault Displacements

The pipeline burial configurations will be modified, as necessary, at active fault locations to mitigate the effects of abrupt differential ground displacements on the pipeline. Representative analyses will be performed to demonstrate the adequacy of these special designs at the fault crossings.

The abrupt differential ground displacements experienced at a fault location have the effect of applying passive soil pressures normal to the pipe in the horizontal or vertical directions, depending upon the geometry of the fault motion. The pipe is also stretched or compressed, depending upon the angle at which the pipeline intersects the fault. The purpose of the special burial designs is to reduce the soil strength to permit the pipeline to move in both the axial and normal directions. In this manner, the imposed displacements are distributed over a sufficient length of pipeline to keep the material strains within acceptable limits.

The special burial designs will be qualified by two stages of anayses as described below:

- o Soil load/displacement characteristics for the displacement of the pipeline in relation to the soil will be determined.

- o Pipeline strains will be calculated for each active fault. The analysis will incorporate the following behavior phenomena:

  - Soil Behavior
  - Pipe Material Behavior
  - Internal Pipe Pressure
  - Initial temperature stresses.

Soils Hazards

Other earthquake hazards which may affect the pipeline include liquefaction and slope instability. These hazards will be avoided in the routing of the pipeline where practical. Where the hazards are not avoidable, analysis will be used to qualify the pipeline design as described below.

Liquefaction

The liquefaction potential of suspect soils in the immediate vicinity of the pipeline will be determined by appropriate methods as described in Section 4.0.

Slope Instability

The stability of suspect slopes in the immediate vicinity of the pipeline will be determined by appropriate static and dynamic analysis procedures as described in Section 4.0.

Seismic Monitoring System

A monitoring system will be designed and installed which includes procedures for the safe shutdown of the pipeline under seismic conditions that may affect pipeline integrity. The monitoring system as a minimum will include:

- Communication capability with all key operating control points on the pipeline, the gas processing plant, and other parties with seismic monitoring capabilities as appropriate
- A control center and alternate for the pipeline system
- Operating procedures establishing the actions to be taken if seismic conditions occur that may affect pipeline integrity
- Seismic sensors to supplement existing monitoring capabilities as necessary.

ROUTE DESCRIPTION – SECTION 1

F&T 1.0

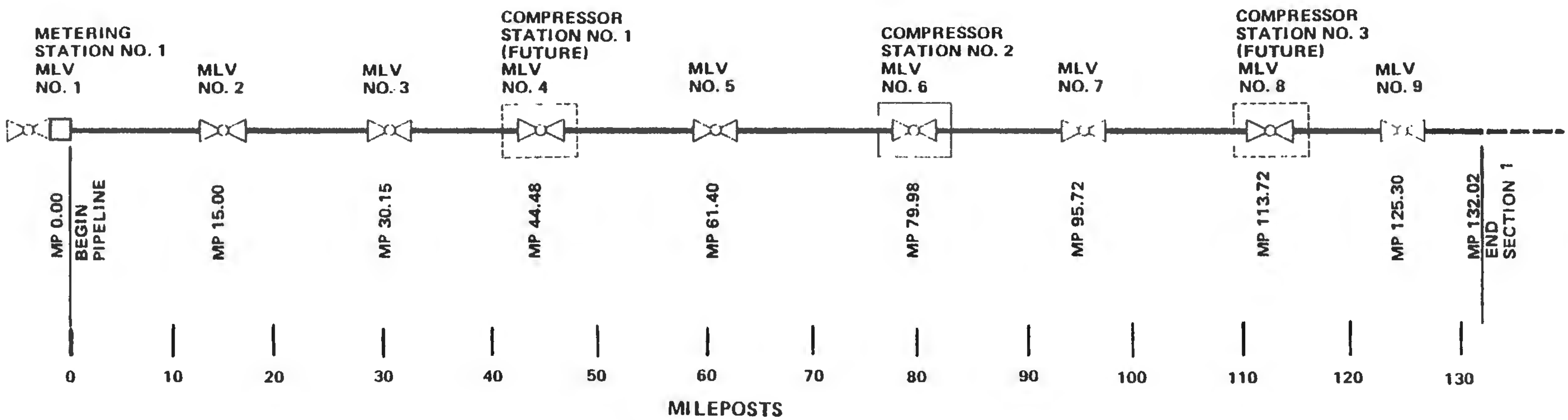


FIGURE Z-9.1-1-1

MAY, 1980

ROUTE DESCRIPTION – SECTION 2

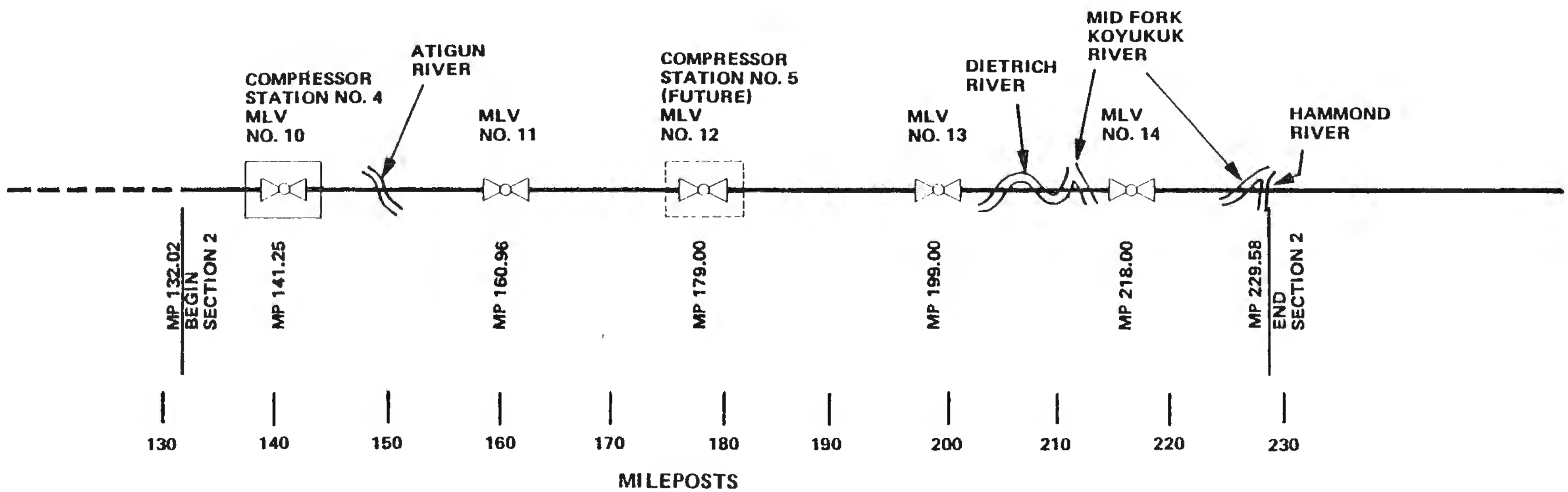


FIGURE Z-9.1-1-2

ROUTE DESCRIPTION – SECTION 3

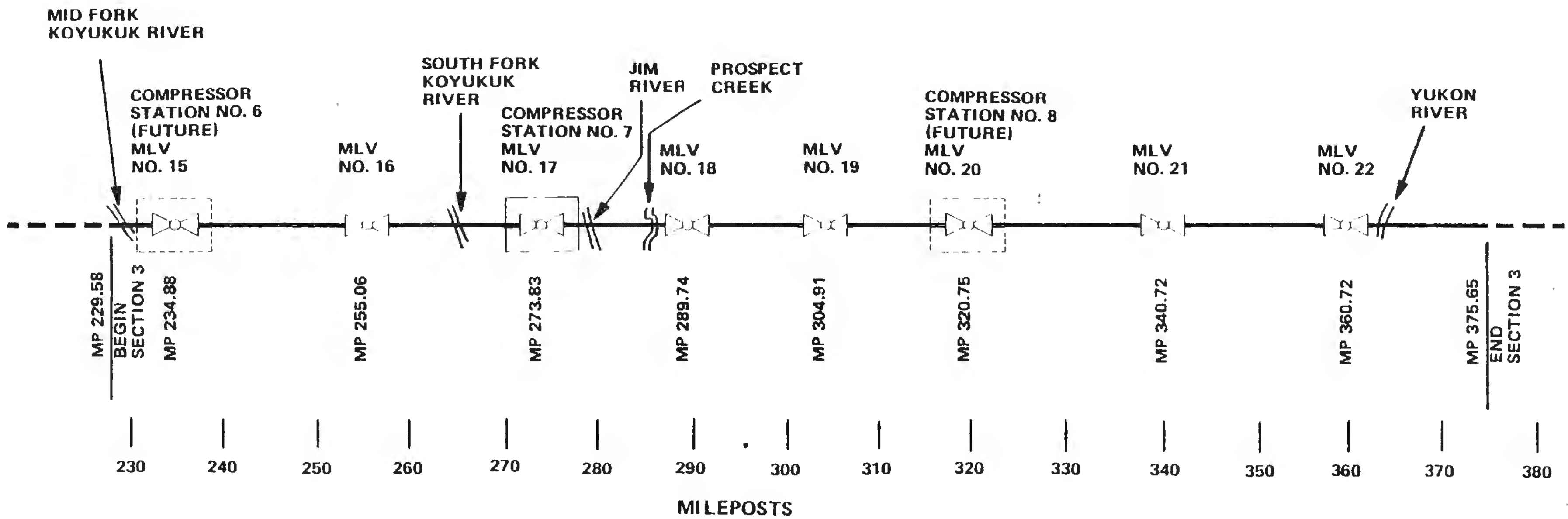


FIGURE Z-9.1-1-3

ROUTE DESCRIPTION – SECTION 4

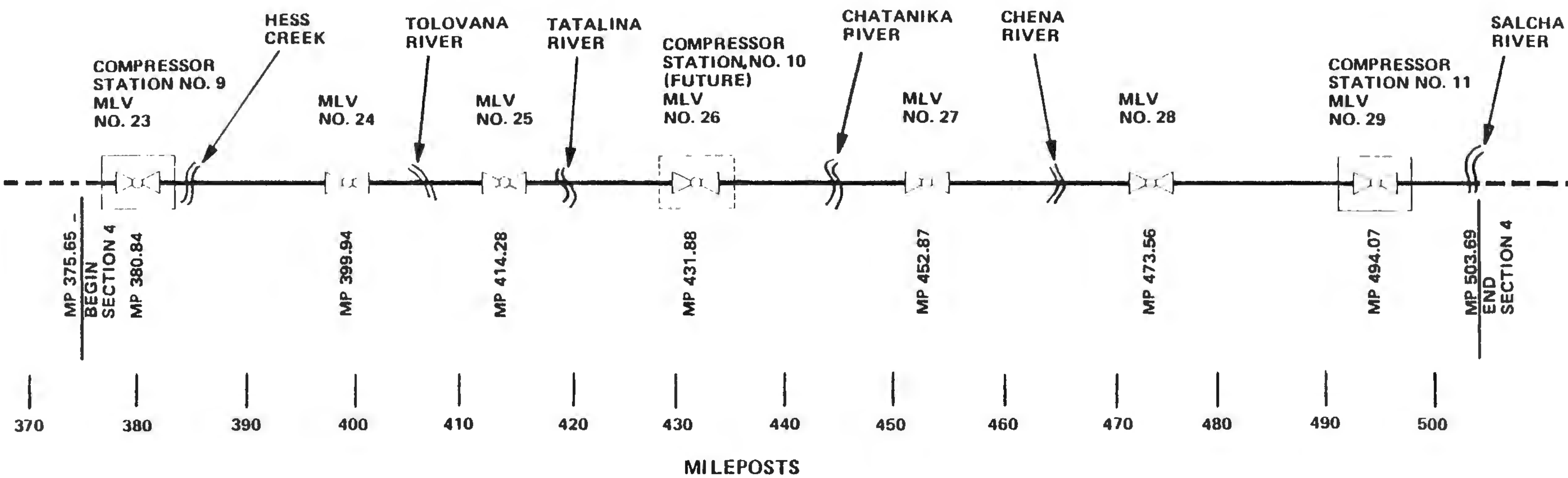


FIGURE Z-9.1-1-4

MAY, 1980

ROUTE DESCRIPTION – SECTION 5

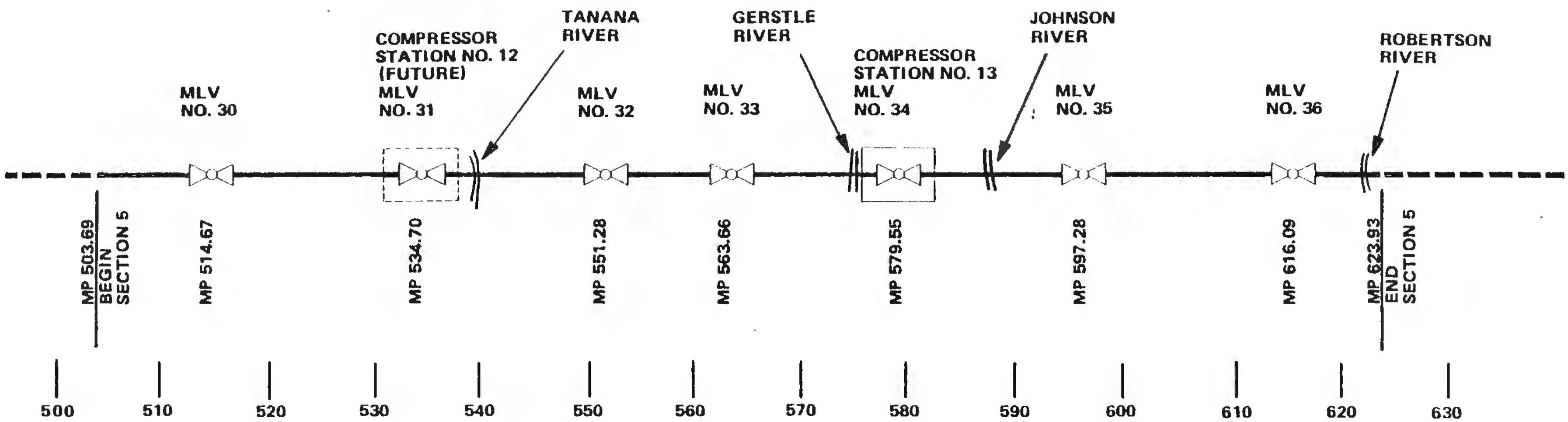


FIGURE Z-9.1-1-5

MAY, 1980

ROUTE DESCRIPTION – SECTION 6

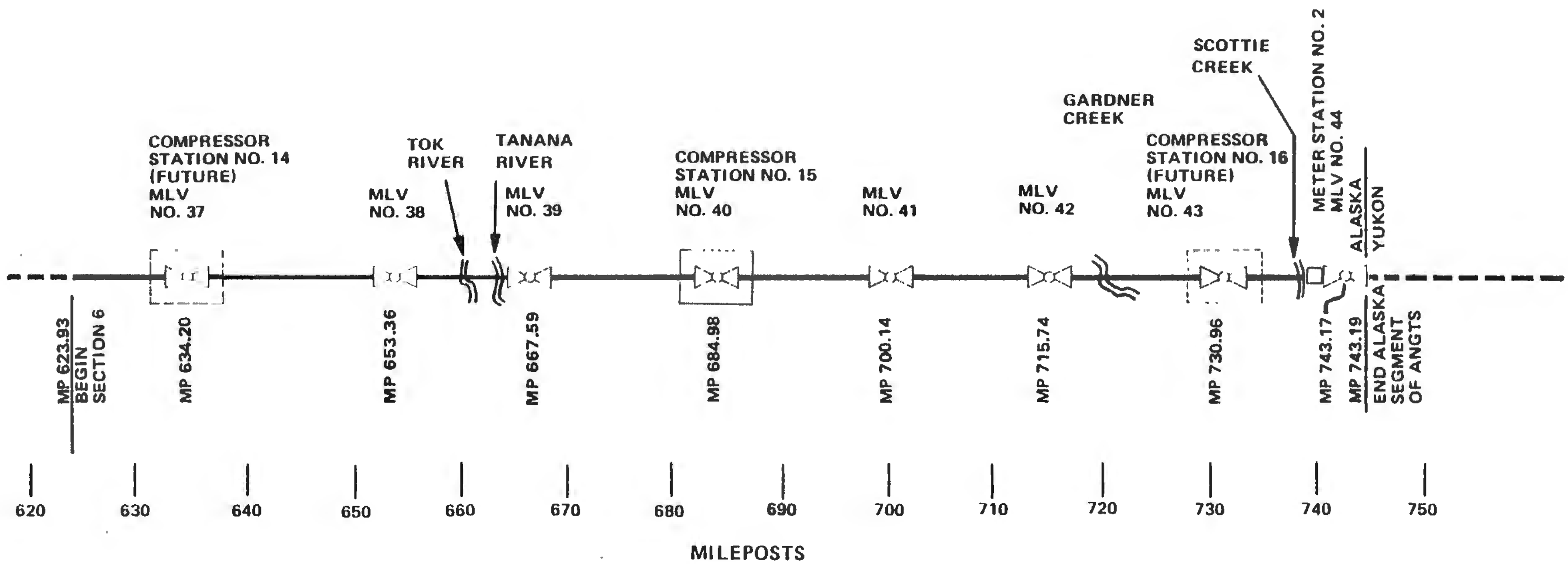
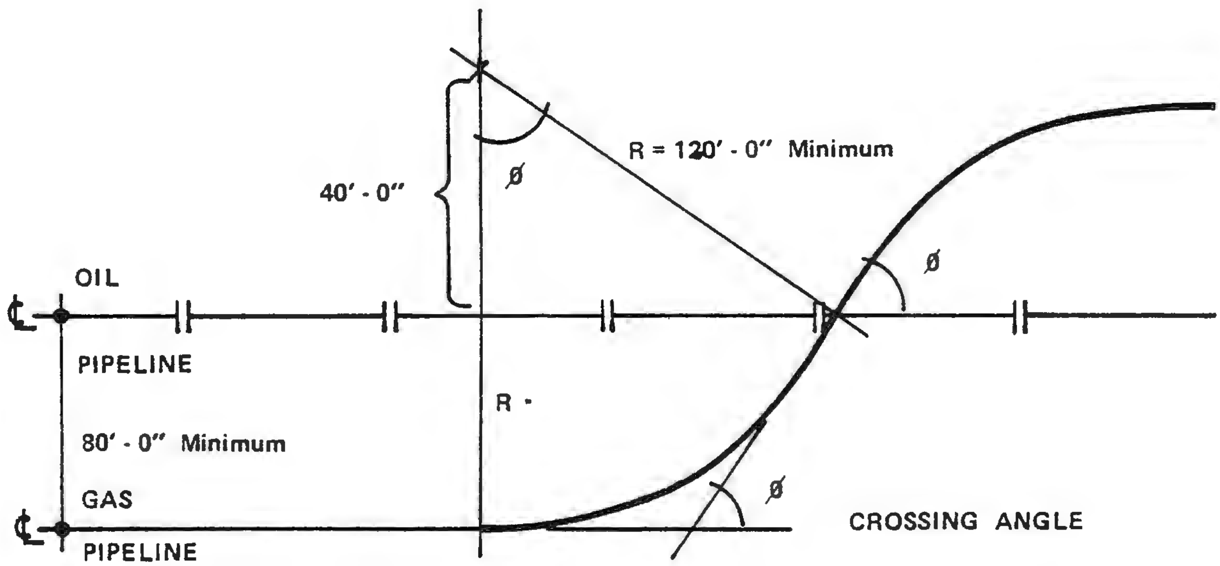


FIGURE Z-9.1-1-6

TAPS CROSSINGS



$\phi = \cos^{-1}(40/120) = 70^{\circ} \pm$

FIGURE Z-9.1-1-7

BIAXIAL STATE OF STRESS IN PIPE WALL

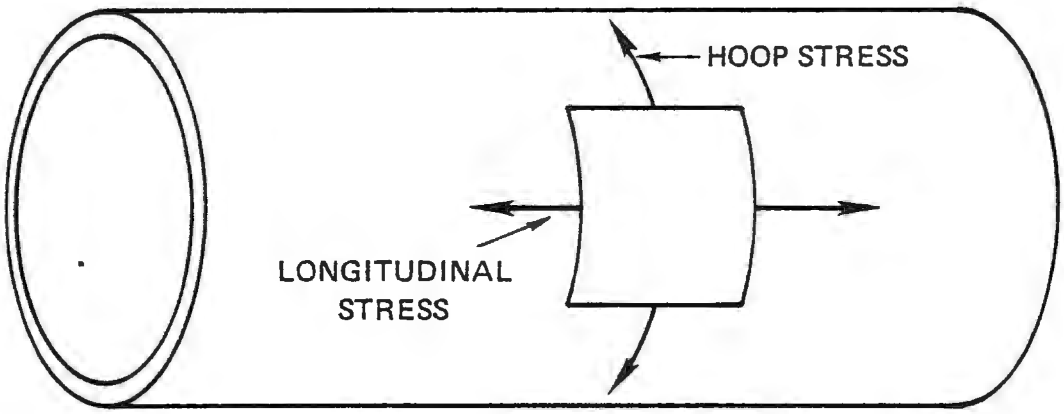


FIGURE Z-9.1-1-8

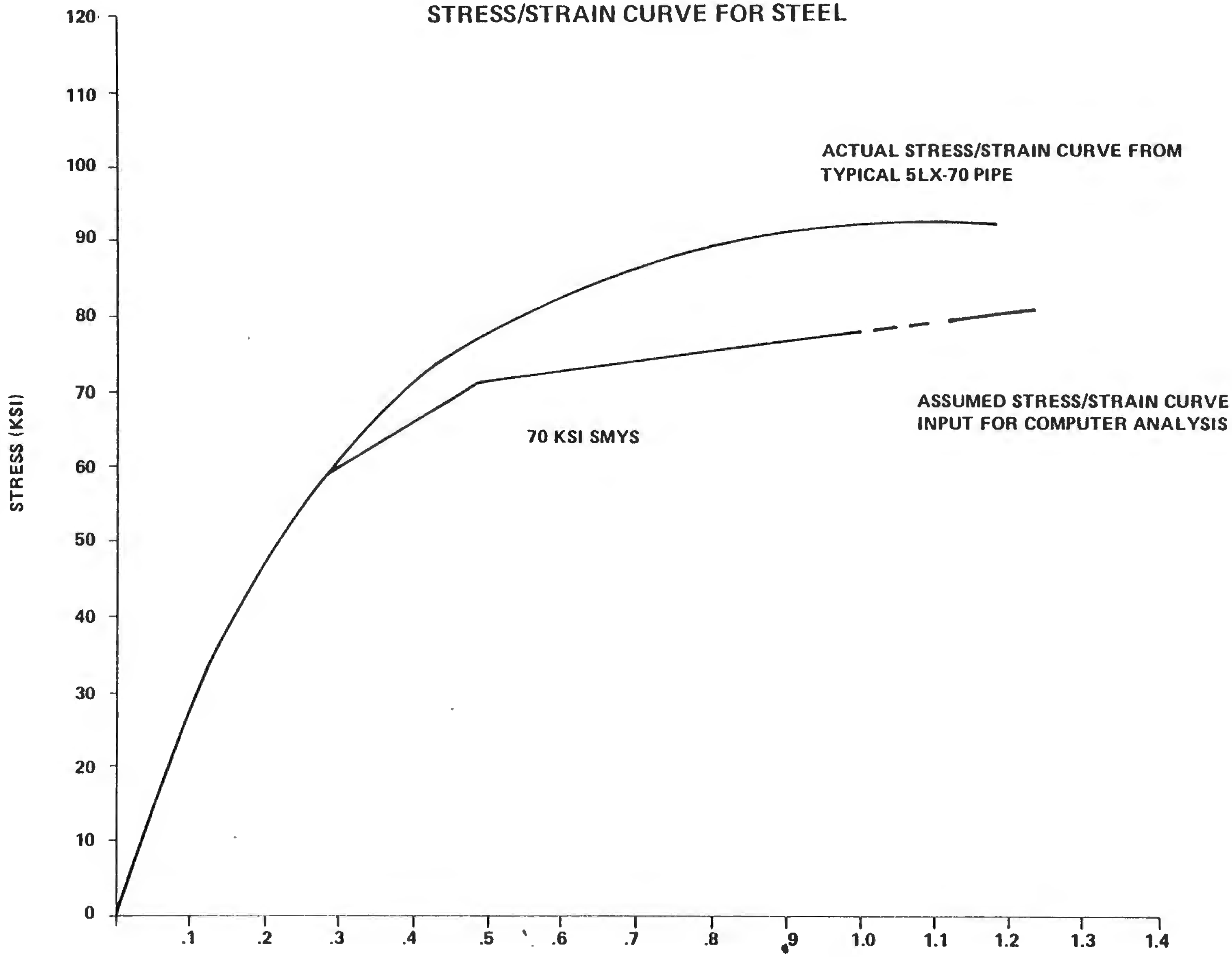


FIGURE Z-9.1-1-9

SOIL LOAD DISPLACEMENT CURVE – GENERAL FORM

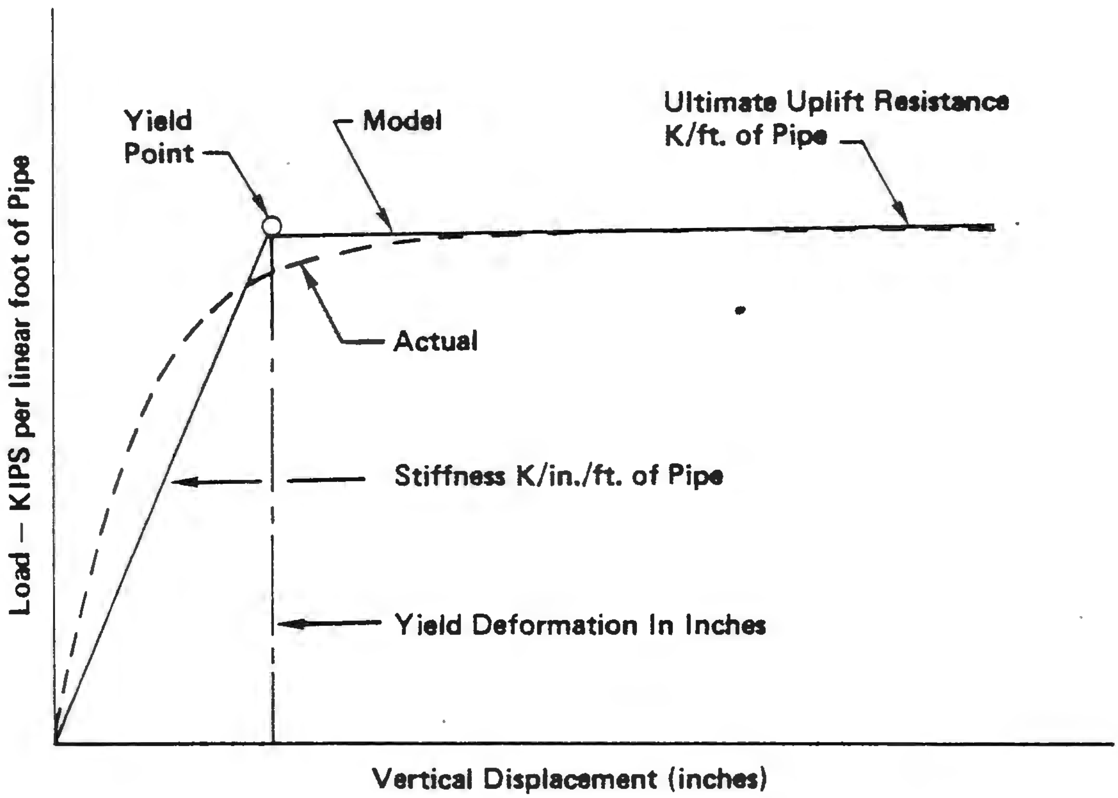


FIGURE Z-9.1-1-10

LOAD/DISPLACEMENT FUNCTION

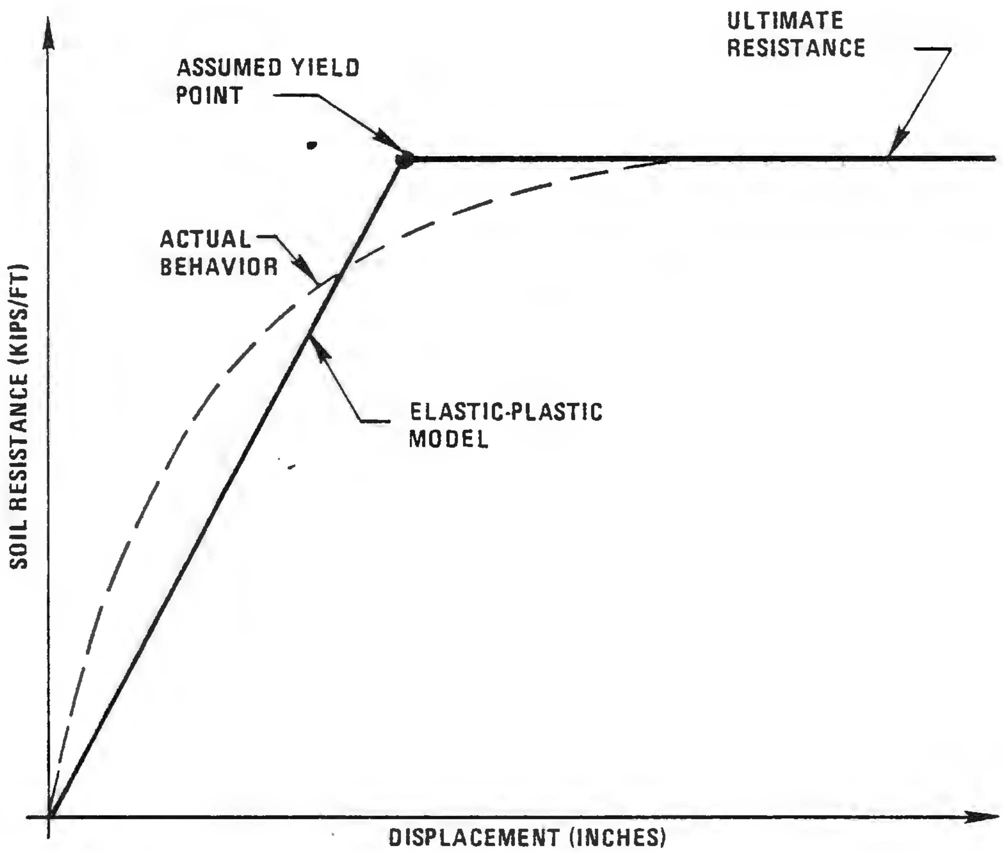
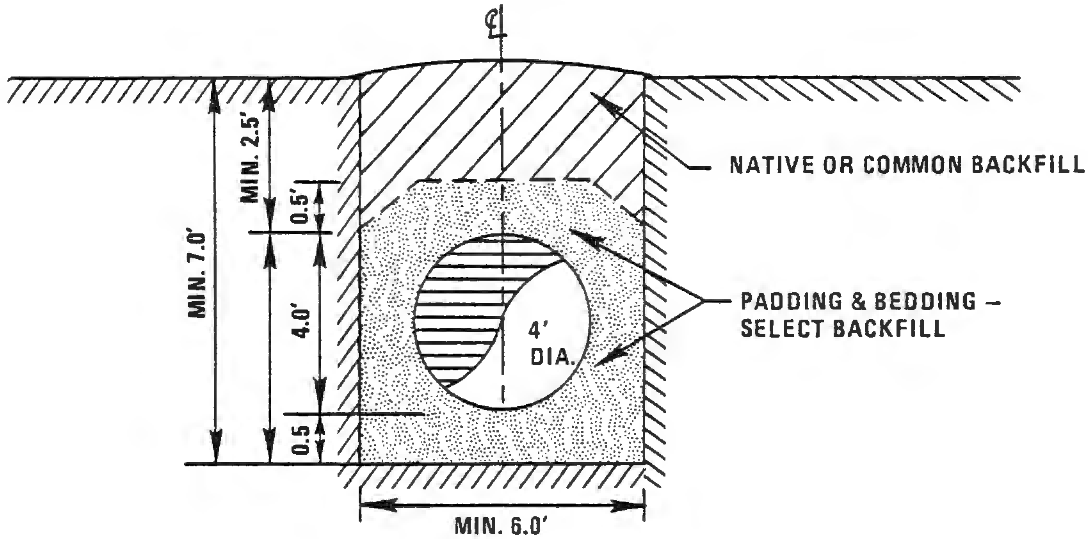
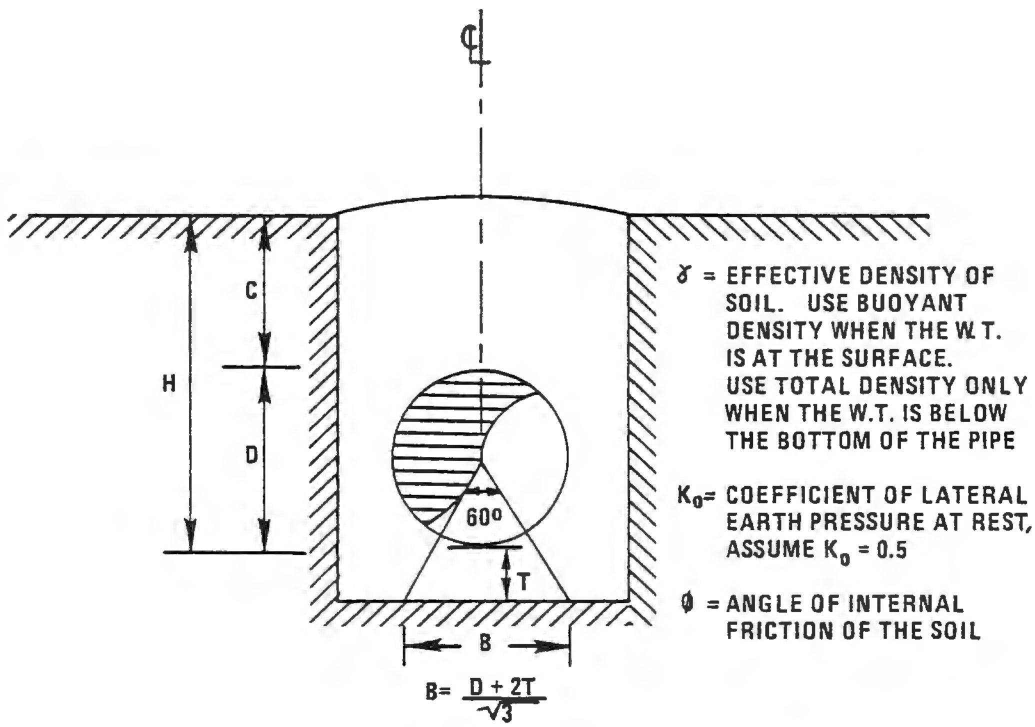


FIGURE Z-9.1-1-11

BURIED PIPE CROSS SECTION



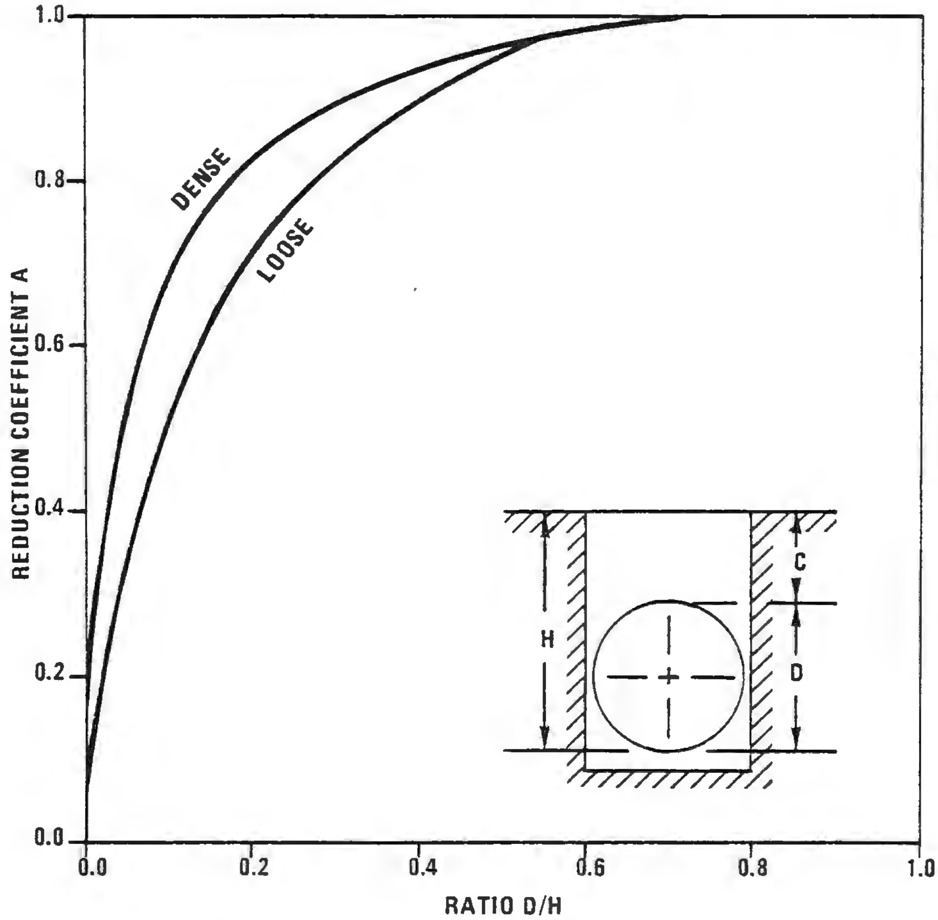
a) TYPICAL DITCH CROSS - SECTION



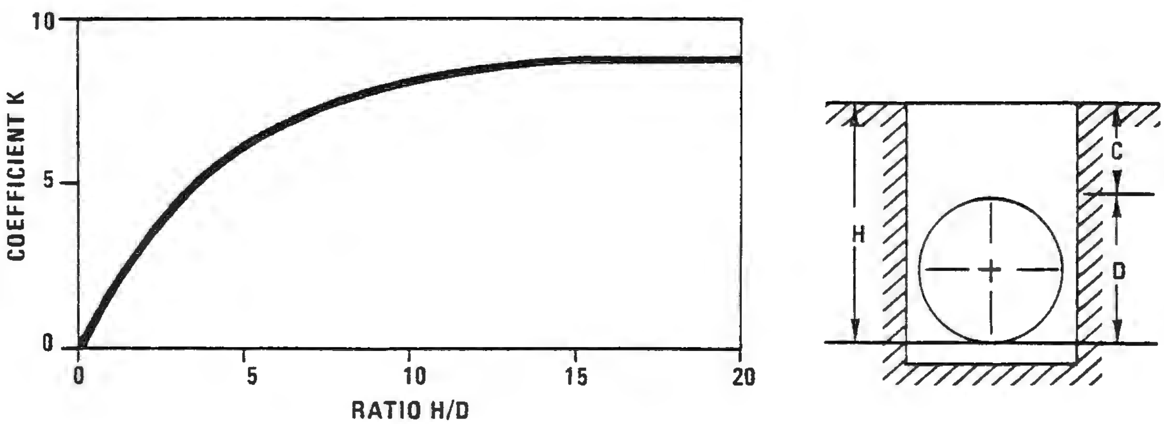
b) DEFINITION OF SYMBOLS

FIGURE Z-9.1-1-12

COEFFICIENTS FOR SIDEBEND RESISTANCE



a) REDUCTION COEFFICIENT FOR SIDEBEND RESISTANCE IN GRANULAR SOILS



b) LATERAL PRESSURE COEFFICIENT FOR SIDEBEND RESISTANCE IN FINE GRAINED SOILS

FIGURE Z-9.1-1-13

CREEP BEHAVIOR OF FROZEN SOIL

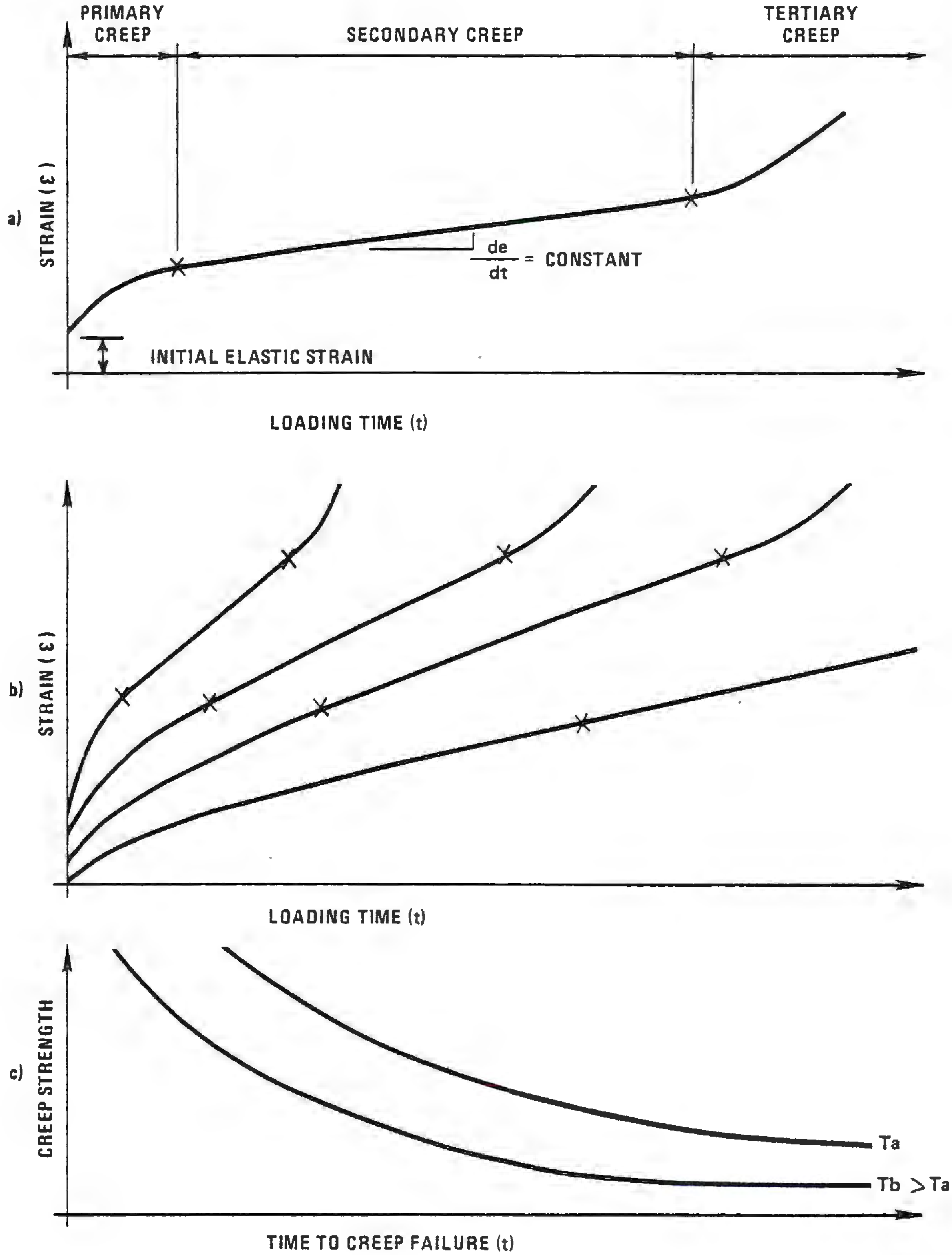


FIGURE Z-9.1-1-14

BENDING OF PIPELINE DUE TO DIFFERENTIAL FROST HEAVE
(EXAGGERATED WORST CASE)

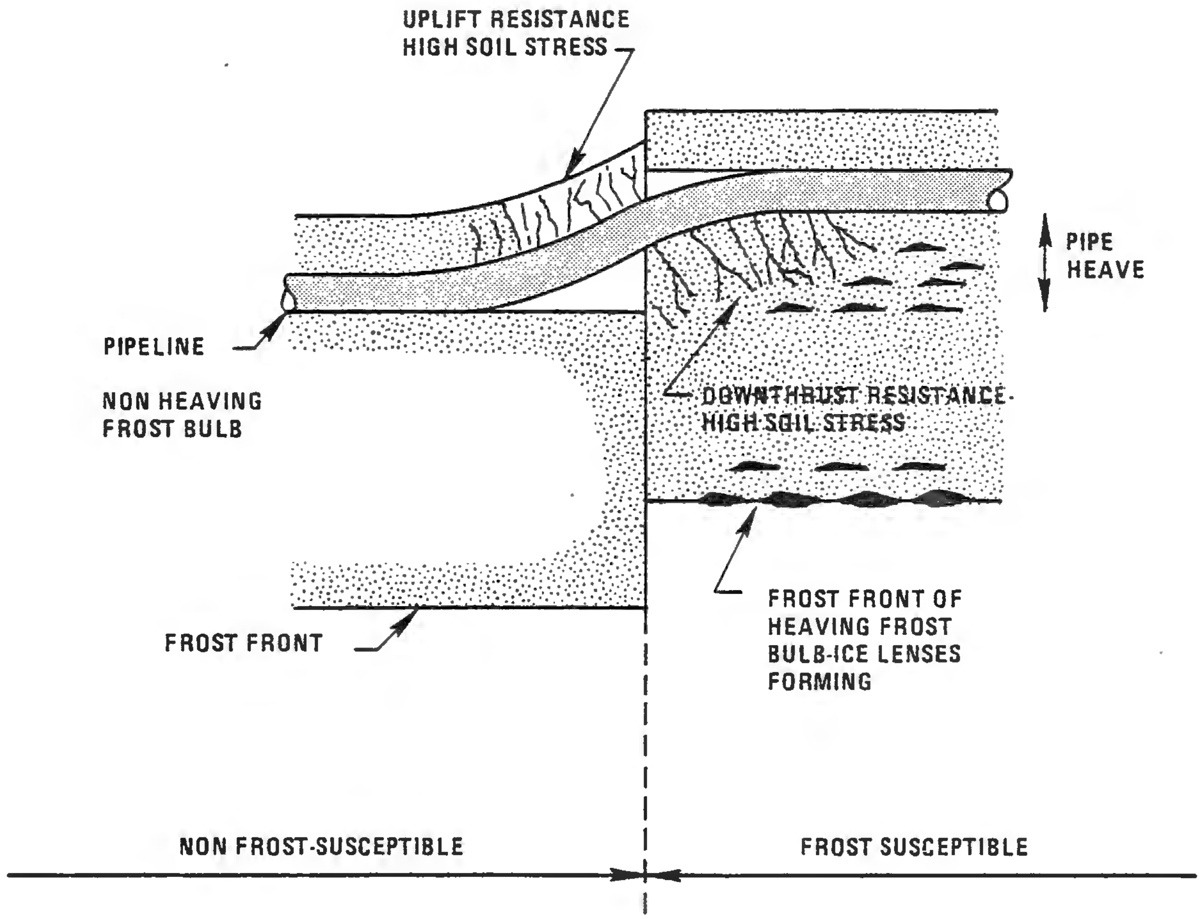


FIGURE Z-9.1-1-15

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TABLE 1-1
SUMMARY OF BASIC DATA

o Pipe Properties

Class Location	Pipe Size D X t (in x in)	Area of Steel A_s (in^2)	Internal Area (in^2)	Moment of Inertia I (in^4)	Pipe Weight (lb/ft)
1	48 x 0.600	89.3	1720	25,097	304
2	48 x 0.720	106.9	1703	29,890	364
3	48 x 0.864	127.9	1682	35,545	435
4	48 x 1.080	159.2	1650	48,830	542

$$A_S = \frac{\pi}{4} (D^2 - (D-2t)^2$$

$$A_p = \frac{\pi}{4} (D-2t)^2$$

$$I = \frac{\pi}{64} (D^4 - (D-2t)^4)$$

Where:

D = outside diameter (in)

t = wall thickness (in)

Steel Density = 490 lb/cu ft

Specified Minimum Yield Strength	70,000 psi
Grade	API-5LX (5LS)
Modulus of Elasticity	29×10^6 psi
Coefficient of Thermal Expansion (α)	6.5×10^{-6} in/in/°F
Poisson's Ratio (μ)	0.3
Minimum Field Bend Radius	120 ft

TABLE 1-1 (Cont.)

- Internal Pressure

Maximum Allowable Operating	1260 psig (.72 SMYS)
Test, Minimum	1575 psig (.90 SMYS)
Test, Maximum	1925 psig (1.10 SMYS)

- Minimum Depth of Cover

Normal soil	2.5 ft
Rock	1.5 ft

- Soil Density

Maximum unit weight for overburden load	130 lb/ft^3

- Temperature (Degrees Fahrenheit) for Thermal Stress Analysis

	Operating	Installation(*)
Maximum	+32	+80
Minimum	-10	0

- Differential Temperature (degrees)

	Operating	Hydrotest(*)
Maximum (Pos.)	32 (0°F to +32°F)	50 (0°F to 50°F)
Minimum (Neg.)	90 (+80°F to -10°F)	50 (+80°F to 30°F)

- Live Loads

Maximum Gas Weight	80 lb/linear ft
Test Medium (Max S.G. = 1.08)	805 lb/linear ft

(*)May be varied in final design.

TABLE 1-1 (Cont.)

- Seismic Ground Motions for Buried Pipeline Response Per Newmark's Preliminary Criteria

o Contingency Earthquake	Ground Acceleration	Ground Velocity In/Sec	Zone
Prudhoe to 67 degrees north	0.15g	7	1
67 degrees north to Big Delta	0.45g	22	3
Big Delta to the Canadian Border	0.35g	17	2

- Operating Earthquake

Prudhoe to 67° North	0.06g	3	1
67° North to Big Delta	0.18g	9	3
Big Delta to the Canadian Border	0.14g	7	2

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TABLE 1-2

48" DIAMETER PIPE

Hoop Stress Versus Allowable Stress
Based on Internal Operating Pressure

Class Location	Wall Thickness (inch)	Operating Pressure (psi)	Hoop Stress (ksi)	Allowable Hoop Stress (ksi)
1	0.600	1260	50.40	50.40
2	0.720	1260	42.00	42.00
3	0.864	1260	35.00	35.00
4	1.080	1260	28.00	28.00

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TABLE 1-3

48" DIAMETER PIPE

Effective Stress Intensity Due To
Design Pressure And Temperature Differential

Pipe Wall Thickness (inch)	Maximum Temperature Differential (°F)	Hoop Stress (ksi)	Long Stress Due to Pressure (ksi)	Long Stress Due to Temp. Differential (ksi)	Total Long Stress (ksi)	Effective Stress Intensity (ksi)	Allowable Stress 0.9 (SMYS) (ksi)
0.600	-90	50.40	15.12	+16.96	32.08	50.40	63.00
	+50	50.40	15.12	- 9.43	5.69	50.40	63.00
0.720	-90	42.00	12.60	+16.96	29.56	42.00	63.00
	+50	42.00	12.60	- 9.43	3.17	42.00	63.00
0.864	-90	35.00	10.50	+16.96	27.46	35.00	63.00
	+50	35.00	10.50	- 9.43	1.07	35.00	63.00
1.080	-90	28.00	8.40	+16.96	25.36	28.00	63.00
	+50	28.00	8.40	- 9.43	-1.03	29.03	63.00

Operating Pressure = 1260 psi
Poisson's Ratio = 0.3
Effective Stress Based On Tresca Criterion

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Hearing Exhibit No.

TABLE 1-4

48" DIAMETER PIPE

Effective Stress Intensity Due To
Design Pressure, Negative Temperature Differential,
And Design Operating Earthquake

Pipe Wall Thickness (inch)	Hoop Stress (ksi)	Long Stress Due to Pressure (ksi)	Long Stress Due to Temp. Differential (ksi)	Long Stress Due to design Oper. Seismic (ksi)	Max. & Min. Total Long Stress (ksi)	Effective Stress Intensity (ksi)	Allowable Stress 1.0 (SMYS) (ksi)
0.600	50.40	15.12	+16.96	±4.35	+36.43 +27.73	50.40	70.00
0.720	42.00	12.60	+16.96	±4.35	+33.91 +25.21	42.00	70.00
0.864	35.00	10.50	+16.96	±4.35	+31.81 +23.11	35.00	70.00
1.080	28.00	8.40	+16.96	±4.35	+29.71 +21.01	28.00	70.00

Operating Pressure = 1260 psi
Poisson's Ratio = 0.3
Ground Velocity = 9 inches/second
Velocity of Seismic Shear Waves = 2500 ft/sec.
$\Delta t = -90°F$

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Hearing Exhibit No.

TABLE 1-5

48" DIAMETER PIPE

Effective Stress Intensity Due To
Design Pressure, Positive Temperature Differential,
And Design Operating Earthquake

Pipe Wall Thickness (inch)	Hoop Stress (ksi)	Long Stress Due to Pressure (ksi)	Long Stress Due to Temp. Differential (ksi)	Long Stress Due to design Oper. Seismic (ksi)	Max. & Min. Total Long Stress (ksi)	Effective Stress Intensity (ksi)	Allowable Stress 1.0 (SMYS) (ksi)
0.600	50.40	15.12	-9.43	±4.35	+10.04 + 1.34	50.40	70.00
0.720	42.00	12.60	-9.43	±4.35	+ 7.52 - 1.18	43.18	70.00
0.864	35.00	10.50	-9.43	±4.35	+ 5.42 - 3.28	38.28	70.00
1.080	28.00	8.40	-9.43	±4.35	+ 3.32 - 5.38	33.38	70.00

Operating Pressure = 1260 psi
Temperature Differential = +50°F
Poisson's Ratio = 0.3
Ground Velocity = 9 inches/second
Velocity of Seismic Shear Waves = 2500 ft/sec.

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TABLE 1-6

48" DIAMETER PIPE

Effective Stress Intensity Due To
Design Pressure, Negative Temperature Differential,
And Design Contingency Earthquake

Pipe Wall Thickness (inch)	Hoop Stress (ksi)	Long Stress Due to Pressure (ksi)	Long Stress Due to Temp. Differential (ksi)	Long Stress Due to design Max. Seismic (ksi)	Max. & Min. Total Long Stress (ksi)	Effective Stress Intensity (ksi)	Allowable Stress 1.10 (SMYS) (ksi)
0.600	50.40	15.12	+16.96	±10.63	+42.71 +21.45	50.40	77.00
0.720	42.00	12.60	+16.96	±10.63	+40.19 +18.93	42.00	77.00
0.864	35.00	10.50	+16.96	±10.63	+38.09 +16.83	38.09	77.00
1.080	28.00	8.40	+16.96	±10.63	+35.99 +14.73	35.99	77.00

Operating Pressure = 1260 psi
Poisson's Ratio = 0.3
Ground Velocity = 22 inches/second
Velocity of Seismic Shear Waves = 2500 ft/sec.
Temperature Differential = -90°F

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Hearing Exhibit No.

TABLE 1-7

48" DIAMETER PIPE

Effective Stress Intensity Due To
Design Pressure, Positive Temperature Differential,
And Design Contingency Earthquake

Pipe Wall Thickness (inch)	Hoop Stress (ksi)	Long Stress Due to Pressure (ksi)	Long Stress Due to Temp. Differential (ksi)	Long Stress Due to design Max. Seismic (ksi)	Max. & Min. Total Long Stress (ksi)	Effective Stress Intensity (ksi)	Allowable Stress 1.10 (SMYS) (ksi)
0.600	50.40	15.12	-9.43	±10.63	+16.32 - 4.94	55.34	77.00
0.720	42.00	12.60	-9.43	±10.63	+13.80 - 7.46	49.46	77.00
0.864	35.00	10.50	-9.43	±10.63	+11.70 - 9.56	46.70	77.00
1.080	28.00	8.40	-9.43	±10.63	+ 9.60 -11.66	39.66	77.00

Operating Pressure = 1260 psi
Temperature differential = +50°F
Poisson's Ratio = 0.3
Ground Velocity = 22 inches/second
Velocity of Seismic Shear Waves = 2500 ft/sec.

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Exhibit Z-9.1
Hearing Exhibit No.

TABLE 1-8

48" DIAMETER PIPE

UNPRESSURIZED
Circumferential Bending Stress Due to
Overburden Load (Uncompacted Backfill)

Pipe Wall Thickness t (inch)	Depth of Cover H (feet)	Circumferential Deflection Δx (inch)	Circumferential Bending Stress S_b (ksi)	Allowable Circum. Bending Stress S_a (ksi)
0.600	2.50	0.39	9.72	56.00
	8.00	1.24	30.90	56.00
	12.00	1.86	46.35	56.00
0.720	2.50	0.22	6.58	56.00
	8.00	0.72	21.53	56.00
	12.00	1.08	32.30	56.00
0.864	2.50	0.13	4.67	56.00
	8.00	0.42	15.07	56.00
	12.00	0.62	22.25	56.00
1.080	2.50	0.07	3.14	56.00
	8.00	0.21	9.42	56.00
	12.00	0.32	14.36	56.00

Soil Density = 130 lbs/cu ft

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TABLE 1-9

48" DIAMETER PIPE
PRESSURIZED

Effective Stress Intensity Due To
Design Pressure, Positive Temperature Differential,
And Overburden Load (Uncompacted Backfill)

Pipe Wall Thickness t (inch)	Depth of Cover H (ft)	Bending Stress Due to Backfill (ksi)	Hoop Stress by Pressure (ksi)	Combined Stress Press. & Backfill (ksi)	Long. Stress by Temp. and Pressure *(ksi)	Total Comb. Effective Stress (ksi)	Maximum Allowable Stress (ksi)
0.600	2.50	1.24	50.40	51.64	5.69	51.64	70.00
	8.00	3.95	50.40	54.35	5.69	54.35	70.00
	12.00	5.93	50.40	56.33	5.69	56.33	70.00
0.720	2.50	1.36	42.00	43.36	3.17	43.36	70.00
	8.00	4.36	42.00	46.36	3.17	46.36	70.00
	12.00	6.54	42.00	48.54	3.17	48.54	70.00
0.864	2.50	1.43	35.00	36.43	1.07	36.43	70.00
	8.00	4.58	35.00	39.58	1.07	39.58	70.00
	12.00	6.88	35.00	41.88	1.07	41.88	70.00
1.080	2.50	1.40	28.00	29.40	-1.03	30.43	70.00
	8.00	4.48	28.00	32.48	-1.03	33.51	70.00
	12.00	6.72	28.00	34.72	-1.03	35.75	70.00

*-Obtained from Table 1.3
Pressure = 1260 psi
Δ Temperature = +50°F
Soil Density = 130 lbs/cu ft

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Hearing Exhibit No.

TABLE 1-10

48" DIAMETER PIPE

Effective Stress Intensity Due To
Design Pressure, Negative Temperature Differential,
And Overburden Load (Uncompacted Backfill)

Pipe Wall Thickness t (inch)	Depth of Cover H (ft)	Bending Stress Due to Backfill (ksi)	Hoop Stress by Pressure (ksi)	Combined Stress Press. & Backfill (ksi)	Long. Stress by Temp. and Pressure **(ksi)	Total Comb. Effective Stress (ksi)	Maximum Allowable Stress (ksi)
0.600	2.50	1.24	50.40	51.64	32.08	51.64	70.00
	8.00	3.95	50.40	54.35	32.08	54.35	70.00
	12.00	5.93	50.40	56.33	32.08	56.33	70.00
0.720	2.50	1.36	42.00	43.36	29.56	43.36	70.00
	8.00	4.36	42.00	46.36	29.56	46.36	70.00
	12.00	6.54	42.00	48.54	29.56	48.54	70.00
0.864	2.50	1.43	35.00	36.43	27.46	36.43	70.00
	8.00	4.58	35.00	39.58	27.46	39.58	70.00
	12.00	6.88	35.00	41.88	27.46	41.88	70.00
1.080	2.50	1.40	28.00	29.40	25.36	29.40	70.00
	8.00	4.48	28.00	32.48	25.36	32.48	70.00
	12.00	6.72	28.00	34.72	25.36	34.72	70.00

**-Obtained from Table 1.3
Pressure = 1260 psi
Δ Temperature = -90°F
Soil Density = 130 lbs/cu ft

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Hearing Exhibit No.

2.0 CIVIL DESIGN

The civil engineering design methods applied to the pipeline construction zone and related facilities are a means for accommodating construction, operation, and maintenance needs while appropriately considering environmental and economic constraints. Construction and restoration activities establish the controlling design requirements; and construction design requirements accommodate operation and maintenance needs. This section describes civil design criteria, and design features related to clearing, grading (earthwork), work pad, drainage, erosion control, restoration, and revegetation activities.

The design and construction of civil aspects of the pipeline will be in accordance with the requirements of the following agencies:

- United States Army, Corps of Engineers (COE)
- American Association of State Highway and Transportation Officials (AASHTO)
- American Concrete Institute (ACI)
- American Institute of Steel Construction (AISC)
- American Iron and Steel Institute (AISI)
- American National Standards Institute (ANSI)
- American Society of Testing Materials (ASTM)
- American Welding Society (AWS)
- Concrete Reinforcing Steel Institute (CRSI)
- Steel Structures Painting Council (SSPC)
- Code of Federal Regulations Title 18 (CFR)
- State of Alaska Department of Transportation and Public Facilities
- State of Alaska Department of Fish and Game (ADFG)
- State of Alaska Department of Environmental Conservation (ADEC)
- United States Coast Guard (USCG)
- United States Department of Defense (DOD)

2.1 Pipeline Construction Zone

The preliminary civil design of the pipeline construction zone is presented in the civil construction alignment sheets and the accompanying drawings (listed below). Cross section drawings depict clearing and earthwork design elements of the features which will be constructed at various reaches along the pipeline route. Cross section dimensions have been derived from the design criteria; they vary as conditions change along the route. Typical cross sections have been developed for reaches of the route where similar geometric conditions exist. The cross sections were subsequently adopted to specific right-of-way conditions and have been applied on a mile-by-mile basis. Special considerations will be given to the following:

- o Continuous, all-weather access along the right-of-way for the movement of construction and maintenance equipment, materials, supplies, and personnel.
- o Sufficient working space for efficient performance of ditching operations, including space for ditch spoil.
- o Sufficient pad surface width for safe and productive pipe stringing and laying operations.
- o Appropriate clearances for protection of existing facilities such as the TAPS pipeline, the haul road, the fuel gas pipeline, and public utilities.
- o Staging and erection space requirements adjacent to areas requiring special pipeline construction procedures.
- o Minimize overall impact to fish passage, fish habitat, terrestrial and wetland habitat, and impact to aesthetics.

2.1.1 Civil Construction Drawings and Sections

Civil construction drawings and sections are included in Exhibit Z-6.2.

2.1.2 Clearing

The areas to be cleared will be designated on the alignment sheets, construction zone cross sections and other design drawings. These areas will be the minimum quantity required for the proposed construction and will be based upon arctic and subarctic engineering methods, accepted engineering design, requirements of the activity to be performed, environmental concerns and seasonal constraints.

- o Clearing limits will be established with consideration for the minimum space requirements that are needed to safely and efficiently perform subsequent construction operations. Clearing limits will be located with special consideration of the following:

Restrictions requiring undisturbed buffer strips

Impact on visual resources

Requirements pertaining to the purchase of merchantable timber

Use of previously cleared or disturbed areas

- o Machine clearing methods will be employed within the clearing limits to remove trees, stumps, roots, and slash. Machine clearing will be scheduled for periods of optimum productivity and minimal environmental disturbance.

  Clearing by hand methods will be accomplished in areas where it is important to keep the upper vegetative layer undisturbed or undamaged, and unacceptable subgrade degradation could occur.

  Clearing operations will be scheduled so that areas of thaw unstable permafrost soils are cleared when the active zone is frozen.

2.1.3 Grading

Design for grading operations will be based on surface conditions, soil types and thermal regime. Detailed criteria is listed on the following drawings 4680-10-00-C-001 and 002 (Appendix E). General guidelines for design of the grading requirements are as follows:

- o Maximize the amount of summer month construction.
- o Use fill type construction if possible.
- o Schedule permafrost cuts during shoulder months.
- o Use cut sections when the gas pipeline is on the high side of TAPS or the Prudhoe Bay Road.
- o Minimize air and water pollution (dust, ice fog, erosion, and sediment).
- o Minimize impact on visual resources.

- Minimize disturbance of fish and wildlife resources.
- Minimize disturbance to terrain and wetlands through control of mobile equipment operations.
- Minimize disturbance of the thermal regime in ice-rich permafrost areas.
- Minimize cuts in ice deposits or ice-rich soils.
- Optimize balance between initial construction efforts and subsequent maintenance efforts.
- Optimize use of locations which provide competent subgrades.
- Optimize construction of embankments through use of materials readily available on the worksite.
- Remove detrimental surface deposits of snow and ice to prevent embankment settlement and contamination of excavated materials.
- Strip surface materials unsuitable for use in construction.

2.1.3.1 Excavation - To the extent possible, excavations for grading and for mining of embankment materials will be made in areas known to be thaw stable, thus avoiding ice deposits and ice-rich soils. At locations where it is necessary to excavate frozen materials, cuts will be designed to provide stabilizing measures which will promote self-healing of slopes where possible. Positive measures will be used to stabilize cuts where self-healing is not possible. These methods are included elsewhere. The design of excavations will give special consideration to the following:

- Minimizing disturbance of the thermal regime.
- Providing erosion control structures and measures where needed.
- Arrange slope stability.
- Designing for the physical properties and characteristics of the excavated material.

- o Considering requirements for the use of explosives.

2.1.3.2 Embankments - Embankments will be designed to provide for physical and thermal stability while simultaneously minimizing material and transportation requirements. The design of embankments will contain special consideration of the following:

- o Minimum surface width requirements.
- o Minimum thickness requirements for gradient, strength, insulation, and frost action.
- o Slope stability.
- o Availability of materials.
- o Erosion and siltation control.
- o Soil or rock materials to be used in embankments will be obtained from onsite excavation when deemed suitable or from designated material sites.
- o Materials selected will possess the physical properties and characteristics required for good compaction and the development of densities necessary to support imposed loads.
- o The design of embankments will also assess the use of insulation board and engineering fabrics for reducing embankment thickness at appropriate locations.

2.1.4 Disposal

The design will include the following guidelines:

- o Cleared timber will be disposed of by burial in the workpad or access roads whenever compatible with design.
- o Unburied timber will be burned, chipped, or sold subject to all applicable regulations.
- o Timber will be disposed of within a time frame consistent with insect infestation control.
- o Slash will be disposed of according to the methods utilized for timber disposal. Disposal of slash and timber will occur concurrently.

- Organic surface material, selected for later use in restoration, will be stockpiled.

- Unusable material will be disposed in selected disposal sites.

- Excess materials may be spread along the workpad and access roads, as appropriate.

2.1.5 Workpad Width

An analysis of the major construction activities indicates that safe and efficient accommodation of certain operations establish the minimum width requirements for the workpad. The following are the requirements found to establish the workpad width.

- The horizontal distance required between workpad shoulder point and the centerline of the welded 48-inch pipe on skids will be five feet.

- The clearance required between the centerline of the welded 48-inch pipe on skids and the near track of the pipe layer will be four feet.

- Workspace requirements for the operation of cradling pipe from cribbing will require one stationary pipe layer (Cat 583) adjacent to the cribbed pipe with counterweight extended, and another pipe layer (Cat 583) passing with boom and counterweight retracted. See Figure Z-9.1-2-1.

- Workspace requirements for lowering-in operations will require one pipe layer (Cat 594) with counterweight extended, and with the near track positioned four feet from shoulder point. A second pipe layer (Cat 594) with boom and counterweight retracted will be passing. See Figure Z-9.1-2-1.

- Maximum allowable cross slope is five percent.

- A 10-foot wide lane is required for continuous access for emergency, repair, service, and supervisory vehicles during the pipelaying operations.

- Organic surface material, selected for later use in restoration, will be stockpiled.

- Unusable material will be disposed in selected disposal sites.

- Excess materials may be spread along the workpad and access roads, as appropriate.

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- Maximum allowable cross slope is five percent.

- A 10-foot wide lane is required for continuous access for emergency, repair, service, and supervisory vehicles during the pipelaying operations.

2.1.5.1 <u>Ditch and Ditch Spoil Area</u> - Analysis of standard pipe ditch excavation and backfill operations indicate the following width requirements.

- o A 10-foot top-width pipe ditch will be required for most soil conditions.
- o A nominal two-foot catchment terrace of original ground will be required between the near edge of the ditch top and the toe of the workpad slope.
- o A nominal 30-foot width from the centerline of the pipe to the edge of the spoil area will normally provide adequate space for ditch. This width can be reduced to a minimum of 25 feet in certain situations.

2.1.5.2 <u>Workpad Relationship to Pipe Ditch</u> - The edge of the workpad should be maintained as near and parallel to the final pipe grade as practicable. Variations in the distance will be caused by workpad thickness, ditch depth, and cross-slope of natural ground. The following are requirements related maintaining the proper relationship between the workpad and pipe ditch.

- o The maximum allowable horizontal distance from workpad shoulder point to pipe ditch centerline will be 16 feet to keep such distance within reasonable boom overhang and to maintain lifting capacity limits of the pipe layers (Cat 594) during lowering-in operations.
- o The workpad embankment slope will normally be at a 1:1 ratio; however, as indicated in the typical sections, this slope will vary with the embankment material.
- o The standard pipe-laying direction is the pipe ditch on the left side of the workpad. This direction may be reversed for short sections with the pipe layers in a backing movement. Normally, the ditch is located to correspond with the pipe laying direction selected; however, laying direction may be controlled by terrain or other factors.

2.1.5.3 <u>Terrain Effects</u> - A five percent slope was selected as the maximum on which pipelaying equipment can operate. Cross slopes exceeding 5 percent must be graded to meet this requirement. The typical cross section types to meet the grading requirements are:

- o Fill section with uniform pad surface
- o Full cut section
- o Through-cut section
- o Cut-fill section

2.1.5.4 Location of Relative Facilities - The geometric requirements for construction zone cross sections when the gas pipeline is adjacent to either the TAPS or the Prudhoe Bay Road is depicted in the typical cross-sections. By maximizing use of these facilities, including the use of the TAPS workpad for half or all of the pad width, total space requirements and cost will be minimized. The Prudhoe Bay Road will be used for light vehicle operation. A protective barrier will be placed to protect the TAPS when it is in the aboveground mode.

2.1.5.5 Cross-Section Selection - Many combinations of cross-section types are possible due to variations in soil, to cross slopes along the pipeline route, and to the relative location to existing facilities. An identifying number system was developed to describe the selected cross-sections and to aid in identification and tabulations. This numbering system consists of three numeric-alpha groups separated by hyphens, for example IL - 30R - 6L. In the groups AX - BY - CZ, A, B, and C represent three Category Groups; X represents the location of an existing facility to the gas pipeline (either Right or Left); Y represents the transverse ground slope direction in Category B (either right or left); and Z represents the workpad location in Category C relative to the pipe centerline (either right or left). The three elements of the cross-section number are explained in the following sections.

- o Relative Facility Location - This is the first of the three category groups and the number assigned refers to the facility location referenced to the pipeline centerline. Table 2-1 lists the detail number assignments for each location traversed by the pipeline route with the corresponding location relative to the pipeline.

- o Ground Slope - The transverse slope of the terrain with respect to the centerline of the pipeline is important to the type of section that is selected because the surface of the workpad may not exceed a five percent slope. A 30 percent cross slope will have the downhill shoulder point of a 50-foot wide pad, 12-1/2 feet above the ground surface. If a fill with a 1:1 side slope ratio is placed on this slope, the toe will be 18 feet from the shoulder point. Work area requirements and sideboom reach limit the distance between the shoulder point and toe to nine feet. Consequently, the workpad must be a stepped fill, a cut, or cut/fill section to maintain the geometric requirements.

The ground cross slope and direction numbers for Category B are shown in Table 2-2. Slopes are divided into five percent increments to 55 percent. Left or right direction is added to the number and refer to the downhill side.

2.1.6 Workpad Thickness Requirements

The thickness of the workpad embankment on thawed subgrades will be designed on the following bases:

- o Pipe ditch location
- o Loading (earthmoving and pipelaying equipment, forces, frequencies, repetitions)
- o Soil bearing capacity
- o Depth of active layer
- o Condition of active layer during placement
- o Period and duration of use

The required thickness of workpad embankments for preventing thermal degradation of frozen subgrade will be dependent upon:

- o Pipe ditch location
- o Ambient temperatures
- o Soil characteristics
- o Insulation

- Construction season

- Exposure

Manufactured insulations will be evaluated as a means of maintaining a frozen subgrade and effecting an embankment thickness reduction.

Civil engineering fabrics will be considered for use where monetary savings might result from employing a reduced workpad or road thickness on weak subgrade soils.

2.1.6.1 Workpad Thickness Design - Structured and thermal design procedures were used to determine minimum workpad thickness. Thermal design methods were considered north of Atigun pass where the subgrade must be kept frozen when the pipeline parallels the TAPS thermal workpad. Structural design is presented for both nonpermafrost and permafrost soils. Structural design accounts for thawing of frozen soils to a depth of five feet. Structural and thermal design methods are presented in the following sections.

- Structural Design - Nonpermafrost - The design procedure for nonpermafrost soils is based on a method whereby the stresses induced in the embankment soil structure are controlled so that no layer of soil is stressed to a level causing the structure to fail. Stresses under the load decrease as the distance from the point of load application increases. Table 2.3 and Figure Z-9.1-2-2 were developed to limit stresses from the applied load to a level that will not result in excessive deformation or failure during the construction of the pipeline.

  The organic material found in nonpermafrost areas will consolidate when an embankment is constructed on it. The consolidation will be about one half the depth of the organic layer, so this thickness is added to the structural thickness as the underlying soil has little effect upon the embankment performance when three or more feet of organic material covers the soil. The embankment, in effect, becomes a raft and thickness is not increased above this value for increases in depth of organic material.

  Civil Engineering Fabrics (CEF) can be used to reinforce the underlying subgrade material; the thickness of the required embankment can then be reduced. The CEF have the effect of increasing the underlying soil strength by 75 percent. A soil that requires a 43 inch embankment to distribute stresses in supporting a 35-ton rock truck requires only 32 inches when CEF are used. The USC soil type and true slope of the area are required to enter Figure Z-9.1-2-2 to determine embankment structural thickness. Slope is expressed as units and slopes less than 0.6 percent are considered to be 0.6 percent. The CEF can also be used to reduce the embankment thickness over organic materials.

- <u>Structural Design - Permafrost</u> - The design procedure for embankments to be constructed on permafrost consists of two components -- structural strength and thaw settlement. As the subgrade material is thawing, the embankment will consolidate the underlying material. Clays and silts present the greatest problem as most contain some ice, and lenses two feet thick are not uncommon. An embankment causes the thaw bulb to penetrate deeper into the ground than normal. The weight of the embankment causes consolidation of this ice-rich material as it thaws. Very rapid thawing of high ice-content soils under an embankment might result in a slurry layer in which the water in the soil cannot dissipate. Vehicular traffic on an embankment over the above condition might result in the material being pumped up into the embankment, thus weakening and causing early failure of the structure. To compensate for this condition and the resulting consolidation, the embankment will be constructed to a thickness greater than that determined from Table 2-3. The additional thickness will be equal to the thaw settlement and will help prevent structural failure during the first thaw season. Only the structural thickness will project above the ground surface after the underlying soil has thawed and been consolidated. Civil engineering fabric can also be used to assist in preventing structural failure, and one half of the thaw settlement thickness may be replaced by a layer of CEF if the thaw settlement is greater than one foot. Thaw settlement is related to landform and USC soil type.

- <u>Thermal Design Procedure</u> - The thermal design procedure includes insulation, either manufactured insulation board or embankments five to seven feet-thick.

Use of thermal design workpad is indicated on the Civil Construction Alignment Sheets with the recommended insulation thickness.

2.1.7 Erosion Control and Drainage

Civil designs will provide for control of erosion, sediment production, transport and deposition.

The design will provide for the following requirements:

- o Minimum disturbance of natural waters
- o Minimum effect of soil thermal regime
- o Conformance with Project Stipulations, and State and Federal requirements
- o Erosion control structures will be designed to accommodate flows resulting from the maximum rainfall rate and snowmelt combination that is reasonably characteristic of the drainage basin
- o Thawing effects produced by ponded or flowing water on permafrost and ice effects will be minimized
- o A soil erosion classification system will be utilized to identify and classify the various soils. Application of erosion control design and construction procedures will be in agreement with this classification system
- o Siltation of natural waters will be minimized by the use of:
  - Location of facilities as previously outlined
  - Settlement basins
  - Water bars (transverse pad levees)
  - Ditch checks
  - Filters
  - Surface protection

- Revegetation

- o Slope angles will be designed to maximize stability and minimize erosion. Special attention will be paid to soil classification, characterization, and hydrologic conditions.

- o Concentrated flow will not be allowed over cut and fill slopes, except where such slopes are specifically protected by an appropriate erosion control structure.

- o Lined let-down structures will be employed if stream or culvert flow must pass over cut or fill slopes. Levees will be placed to intercept stream or sheet flow and will be designed to minimize head cutting.

- o Providing adequate embankment insulation or thickness.

- o Providing for drainage minimizing ponding adjacent to embankments.

- o Preserving organic mats above cuts.

- o Using diversion levees.

- o Providing ditch checks at the toe of slopes.

- o Insulating erodable areas.

- o Mulching erodable areas.

2.1.7.1 Soil Classification, Soil Erosion Code (SEC) - The Soil Erosion Code soil classification system (SEC), (Table 2-4) has been developed to aid in the design of erosion control measures for this project. Soils have been classified primarily according to particle size distribution of the mineral soil encountered within the soil profile, the permanent thermal state of the soil, and susceptibility or resistance of the soil to erosion.

Soils found throughout the construction area of the project will vary widely. Soils classified as having the same particle size distribution may perform quite differently depending upon their moisture contents, in-place densities, exposure to sunlight, and whether they are permanently or seasonally frozen. Specific erosion control procedures have been developed to accommodate these variations within classifications.

Particle size distribution (or percentages of different size materials constituting the soil matrix) affects the strength of the soil and other mechanical properties such as horizontal and vertical permeability. Large particles require more energy to be transported or displaced than do smaller particles. A description of the particle size distribution for soils is presented in Table 2-5.

Permafrost soils are those soils permanently frozen below the depth of seasonal thawing. Their texture varies from clays to gap-graded clean gravels. The moisture content, ice content, and patterns of these frozen soils may differ widely often within small areas. Ice content and patterns range from barely visible ice crystals to massive ice. Frequently occurring soil ice contents are:

- o Low ice - little ice is visible; the sample did not slurry in or contain excess water upon thawing.
- o High ice - visible ice constitutes greater than 20 to 25 percent of soil by volume; the sample slurried or contained excess water upon thawing.
- o Massive ice - visible ice lenses or layers thicker than one inch constitute over half of the soil volume.

Thawed soils are those soils permanently thawed below the depth of seasonal freezing. Thawed soils vary in texture and generally have a low moisture content.

2.1.7.2 <u>Erosion Control Structures and Measures</u> - This section deals with those measures which will be required to prevent erosion; i. e., channels, drainage ditches, water bars, diversion levees, drainage ditch checks, letdown structures, plunge basins, settling basins, and thermal erosion control.

Drainage ditches and channels will be specified where flows must be carried longitudinally through cut sections or through civil facilities such as material or disposal sites. The design of these channels and ditches is based upon the following assumptions:

- o Uniform flow through the structure and at transitions between the stream and the structure

- o Channels and ditches will have trapezoidal cross sections with variable bottom widths and side slopes

- o The minimum freeboard will be twelve inches

- o The maximum velocity will not exceed the limiting velocity of the material used for lining the channel

The side slopes for the channels and ditches will be selected according to the SEC Classification of the soils. (Table 2-6)

The size of the channel or ditch is dependent upon the slope, channel roughness coefficient for the lining material, the frequency design flow, and the allowable flow velocity. The Manning equation will be used to determine channel size.

- o Channel liners are used to protect a channel from erosion caused by scour or headward hydraulic action. Whenever flow passes a structure at velocities great enough to cause accelerated erosion, a channel liner will be used. A channel liner is considered to be suitable when it will resist erosion at design flow velocities. Suitable channel lining materials are:

  <u>Coarse Gravel</u> - Well graded coarse gravel (3/4-inch to 3-inch) will be used to provide protection where design velocities do not exceed 6 feet per second (fps).

  <u>Cobbles or Rock</u> - Well graded cobbles or rock (3 inch to 8 inch) will be used to provide erosion protection for design velocities less than 8 fps.

  <u>Class I Riprap</u> - Riprap meeting Class I (AASHTO) gradation requirements will be used to provide erosion protection for design velocities less than 10 fps.

  <u>Class II Riprap</u> - Riprap meeting Class II (AASHTO) gradation requirements will be used to provide erosion protection for design velocities 10 to 15 fps.

  <u>Gabions</u> - Gabions will be used in instances where Class II riprap is not available or its use is infeasible. Gabions are wire baskets

filled with cobbles or rock; they have a wire top that can be secured to prevent displacement of the smaller material.

- Diversion levees and letdown structures will be used to protect cut and fill slopes from accelerated erosion (rilling and head cutting). Diversion levees will be placed on natural ground above cut slopes to divert sheet and minor stream flow around cut slopes to locations where the flow will not cause erosion and resulting siltation.

- Water bars will be used to retard sheet flow and minimize erosion parallel to the centerline where the workpad is located on steep slopes. Water bars (look like speed bumps) are skewed to the centerline to direct water off of the road and into a prepared channel or off of the right-of-way.

- Ditch checks reduce the effective slope of a ditch, thereby decreasing the flow velocity to acceptable limits for the ditch bed material.

- Letdown structures are used where low quantities of flow must pass over a cut or fill slope. A letdown structure is an armored or metal channel used for slopes up to 100 percent. A plunge basin, or protected apron, is normally used with a letdown structure.

- Plunge basins are designed to dissipate the energy of water flowing from outlets of culverts, channels, or water crossings. They are required whenever outlet velocities exceed the allowable velocity of the streambed material at the outlets, where exit inverts are at a natural grade, or where culvert outlets are cantilevered from fill slopes. Plunge basins will not be used in fish streams.

- Rain and snowmelt in disturbed areas may result in rapid runoff with excessive quantities of sediment in the water. These suspended solids will be removed by sediment basins. Sediment basins are small dams that hold the flow for a period long enough to allow the suspended solids to collect in the pool.

Runoff is estimated by using a procedure in Reference.[1] Basins will be designed to store 70 cu yd of sediment per acre and 1/70 of the average annual runoff.

- Thermal erosion is caused by the thawing of fine-grained ice-rich soils. Thermal erosion may cause subsidence, slope failure, and siltation. Measures will be taken where ice-rich soil disturbance occurs to reduce the rate of thaw, to minimize siltation of waters, and to reestablish thermal equilibrium.

Initial control of thermal erosion on cut slopes will consist of cutting the slope at a ratio of 1:4 and hand clearing the area far enough above the slope to allow controlled ablation of the slope to a ratio of 1-1/2:1 or 2:1. Slash and timber will be removed from the cleared area in order to minimize the tearing of the organic mat caused by excessive weight of vegetation.

The organic mat will drape over the face of the slope as the cut degrades; this will shade and insulate the face. The mat will be reinforced if necessary. The sheet flow will be intercepted by diversion levees above the cut thus allowing water to be routed around the active area. Ditch checks will retain mineral soil at the toe of cut faces. Meltwater will be routed to sediment basins. The area will be revegetated as it stabilizes.

Thermally degrading areas will be covered with an effective mulch such as excelsior or a layer of straw. A secondary control, consisting of a gravel buttress or insulation board, may be required on some slopes.

2.1.7.3 Culverts and Bridges - Permanent culverts and bridges will be designed to accommodate Frequency Design Flood with a return frequency of fifty years in accordance with criteria established by the American Association of State Highway and Transportation Officials (AASHTO) and endorsed by the State of Alaska Department of Transportation and Public Facilities. (See Section 5.0). General guidelines are as follows:

[1] - "Hydrology for Small Drainage Basins" Northern Technical Services, Jan. 14, 1980.

- Traffic requirements
- Fish passage and protection requirements
- Small craft passage requirements
- Native soil conditions
- Proximity to existing facilities
- Embankment thickness
- Construction zone geometry
- Grades and cross slopes
- Discharge and velocity
- Permanent structures designed for 50 year frequency flood
- Temporary structures designed for 5 year frequency flood
- Culverts on fish streams to be sized for fish passage

Design Method - The structures on the Prudhoe Bay Road were designed using a modified BLM method, or a constant of 20 cfs/sq mi, north of the Brooks Range, and 30 cfs/sq mi, south of the Brooks Range.

The hydrologic method is also a modified BLM method and uses an approach similar to the Rational Method. It includes:

Drainage Area (sq mi)
Basin Elevation (EF factor)
Slope, Topography, Vegetation (LF factor)
Aspect (Orientation - lee or windward)
Storage
Rainfall Intensity (RF factor)
Unit Runoff (q, cfs/sq mi)

The discharge equation to calculate design discharge, i.e., Q in cfs is:

$$Q = A \times q \times RF \times LF \times EF$$

Area, slope, topography, vegetation, and aspect are basin characteristics that can be determined from USGS quadrangle maps, aerial photography, and field investigations.

The unit runoff is calculated using the expression q = C/A(0.2) where C is the coefficient dependent on the recurrence interval.

The various factors are determined in the following manner: the rainfall factor is based on rainfall intensity having a specified duration and recurrence interval, and is modified as required according to the size of drainage basin. The land factor is based on the average basin slope, general topography, and vegetal cover. Values range from 2, for straight deep basins, to 0.5 for flat areas with poorly defined drainage. The elevation factor is based on mean basin elevation and orographic effect; it may be as low as 0.8 on the leeward side and as high as 1.36 on the windward side.

Drainage Structures - Temporary and permanent drainage structures for cross drainages of small discharges will be of three general types: 1) culverts, 2) low water crossings, and 3) temporary bridges. Culverts generally will be selected in areas of embankment fill exceeding 30 inches in thickness. Low water crossings will be selected in areas with a cut-fill geometry, except for deeply incised channels which require culverts. Bridges will be built across large sensitive fish habitat streams or navigable streams where constant construction or maintenance traffic will be required.

Access will not be provided across channels where other means of crossing exist (haul road or a TAPS workpad).

Structure selection will be carried out using the following procedure:

- Culverts or bridges will be the structures of choice for fish streams
- Estimated frequency design discharge - If the estimated frequency design discharge cannot be accommodated by a standard structure, a special study of the crossing must be made.
- In areas where the construction zone will be adjacent to the TAPS or Prudhoe Bay Road and permanent access across the drainage is not required, a low water crossing not intended for traffic other than pipe spread equipment will be selected. Bridges will generally be selected where design frequency discharges exceed 100 cfs.

- o The construction zone geometry and embankment thickness will be evaluated to determine the choice between a low water crossing or a culvert.

- o When a low water crossing is selected, the SEC classification and thermal state of the natural soil encountered will be used to select the type of low water crossing.

- o When a culvert has been selected, the thermal state of the natural soil encountered will be used to select the appropriate culvert design.

The design of a culvert is based upon several assumptions. These assumptions are that:

- o The grade of the culvert will match streambed's grade (thalweg), but may be less in fish streams

- o The inlet of the structure will project from fill

- o The headwater at the specified frequency design flood is equal to the culvert's diameter or arch height (HW/D = 1.0)

- o The culvert flows with inlet control

- o Flow velocity in fish stream culverts will allow fish to pass through

- o The fish stream culvert inverts will be placed 6 inches below the thalweg of the stream

- o The culvert alignment will be determined in the field. The culvert's size will be determined according to the Federal Highway Administration (FHWA) Procedure except where necessary to adjust for fish passage. [2]

The selection and design of low water crossings is based upon the SEC classification of the streambed material and the following assumptions:

- o Low water crossings are not planned for use in fish streams during construction

- o Uniform flow through the structure and at transitions between stream and structure

[2] Hydraulic Engineering Circular No. 5, December 1965.

- o The side slopes (traffic surface) of the crossing will be 6:1 with a variable bottom-width
- o The maximum flow depth will be 2.0 feet at the specified frequency design flood
- o The maximum velocity and the SEC classification of the streambed material will be used to select the material for use in the construction of the structure

  Type I water crossings will be selected for rock, cobbles, or clean gravels (SEC G).

  Type II water crossings will be selected for sands and dirty gravels (SEC Q,D).

  Type III water crossings will be selected for silty and clayey soils (SEC S, C, L, U).

  Type IV water crossings will be selected for any soil in areas where the workpad is insulated.

  Type V water crossings will be selected where access is required for the pipe spread equipment and where streams are trafficable for this equipment.

The size of the structure is dependent upon the slope, channel roughness coefficient of the lining, frequency design discharge, and allowable velocity of discharge. The Manning equation will be used for calculating the above design.

Temporary Bridges - Temporary bridges will be specified where design frequency discharge exceeds 100 cfs. Bridges will be of the panel design type that can be erected for most spans and loadings. Bridges will be erected according to manuals' specifications provided by manufacturers of temporary panel type bridges. Mud sills or cribs will be used where possible; permanent abutments will be placed or driven as needed.

2.1.8 Restoration and Revegetation

Areas which have been graded, filled, or otherwise disturbed in the course of civil construction, pipeline construction, or related activities will be restored to satisfactory conditions subsequent to completion of use. Such restoration may include finish grading, permanent erosion control structures, revegetation, or other measures which will be required to leave the affected area physically stable and minimal change can occur to the topography or drainage patterns before native vegetation reclaims the disturbed area. Several field programs are planned prior to construction. (See Environmental Engineering Manual, Exhibit Z-1.1.)

2.1.8.1 Restoration - Restoration measures will be designed for all project facilities and will be implemented following construction. Restoration measures will include the following where necessary:

- Grading of areas to a stable geometry
- Installation of permanent erosion control structures
- Removal or retirement of temporary facilities
- Reestablishment of natural drainage patterns
- Redistribution of stripped material
- Revegetation of disturbed areas
- Treatment of areas with critical visual impact
- Restoration of survey monuments damaged or removed by construction operations

Finish grading of the workpad and access road fills will be indicated on the restoration plans. These fills will be placed at 1:1 for construction of the pipeline; they will require reshaping for restoration according to the slopes listed in Tables 2-7 and 2-8.

2.1.8.2 Revegetation

Temporary revegetation will consist of measures controlling erosion or siltation during construction.

Permanent revegetation measures will be specified for slope stabilization and restoration. Permanent revegetative measures will be implemented in order to minimize erosion and visual impact and to enhance the reestablishment of native vegetation. The following revegetation measures will be used:

- Surface preparation of areas to be revegetated will leave soil in a rough and friable condition.

- Fertilizer will be applied at suitable rates contingent on soil type and condition. Fertilizer mixes will be designed to add the required nutrients to the soil to promote hardy stand establishment.

- Temporary seeding will be used when the surface will be disturbed in the future. Fast establishing grasses will be used for temporary seeding.

- Permanent revegetation measures will be used to "put to bed" areas no longer needed for construction. These measures will be designed to enhance the reinvasion of native species. The measures will include seeding with selected perennial grasses, fertilization, and use of other techniques.

- Surface protection will be applied, where needed, to retain moisture, dissipate raindrop energy, and hold the seed in place.

Revegetation measures will be scheduled for implementation and hardening-off during the growing season, or during positive dormancy in the winter.

2.1.9 Site-Specific Areas

Standard site-specific items will be grouped into the areas listed below. Typical designs will be prepared for each group. Field measurements will be combined with typical designs to make each item site-specific.

- Highway Crossings - The designs will consider requirements for workpad, excavation, backfill, maintenance of traffic, and highway repair.

- Road and Trail Crossings - The designs will consider requirements for workpad, excavation, backfill, maintenance of traffic, and trail or road repair.

- Minor Stream Crossings - The designs will take into account requirements for crossing minor streams. Attention will be given to fish, wildlife, and siltation control measures requisite for minimizing environmental impact.

- Wetland Crossings - The designs will include the special requirements specified within the project stipulations for wetland construction.

- Special Site-Specific Areas - Special site-specific areas are those sites whose physical features create unusual construction difficulties, and therefore require major consideration. Atigun Pass is one area in this category due to its limited space and difficult terrain. Detailed engineering studies will be conducted of this area to evaluate alternative alignments and designs which will minimize detrimental effects on existing facilities and the site's environmental conditions.

2.2 CIVIL QUANTITIES

2.2.1 Quantity Estimate and Haul Analysis

Quantities are based on the grading section selected for the workpad, embankment by thickness, and terrain cross slope. Grading sections will be selected on the basis of minimum material requirements which will produce an economical earthwork design. The following items are considered in determining grading section limits:

- Material site quantities available
- Type of material available
- Quantity and location of material
- Haul conditions
- Haul distance
- Quantity and location of special materials required
- Cost-effectiveness of the selected site and adjacent material sites
- Material available from the right-of-way (suitable material from excavation)

The study and review of these considerations will usually clearly determine the break point or limit of the haul; however, in some instances a more detailed analysis of these factors will be required. For details see Table 2-9.

2.2.2 Workpad and Ditch Earthwork Quantities

Workpad construction quantities are primarily dependent upon the location of the construction zone, placement of the pipeline, and location of the workpad within the construction zone. The factors that determine workpad quantities are:

- o Soil type by Soil Erosion Code (SEC), Unified Soil Classification System (USCS), and Landforms
- o Soil condition (wet or dry)
- o Soil state (frozen or thawed)
- o Cross and longitudinal slopes
- o Stripping
- o Location relative to existing facilities (roads and pipelines)
- o Pipe-ditch cross sections and backfill requirements
- o Construction season (some workpads must be built during winter)

All of the above factors are considered in the design and earthwork quantity estimate for workpad and pipeline construction.

2.2.3 Worksheets

Three worksheets were developed for use in the quantity estimate and haul analysis. The worksheets are:

- o Common and Waste Material Haul Analysis Worksheet
- o Special Material Haul Analysis Worksheet
- o Clearing Quantities

2.2.4 Earthwork Factors

Shrinkage, spillage, maintenance, and additional quantity factors have been estimated for the Quantity Estimate and Haul Analysis. In order to determine these accurately, the following definition of cubic yard (cu yd) is used, that is, the in-bank volume at the source (material site for borrow or construction zone for suitable material, spoil, excess, unsuitable, and other materials).

2.2.5 Off Right-of-Way Quantities

Off right-of-way quantities are included in the analysis. The off right-of-way facilities include camps, storage yards, road crossings, airports, and other items requiring special construction.

2.2.6 Common and Waste Material Haul Analysis Worksheet

Materials required from off right-of-way sources are entered on this worksheet. One worksheet was used for each scheduled material site. The limits of haul and quantities involved are entered on the civil alignment sheets.

2.2.7 Special Material Haul Analysis Worksheet

Processing plant locations for special backfill materials will be situated 20 miles apart at the maximum, depending upon the terrain and quantity of materials required. A ten-mile average haul distance has been considered for processing plants in order to keep the number of setups to a minimum. Plants will be located closer together where hauling conditions are difficult.

Material entered on the Special Material Haul Analysis Worksheet will be processed to meet the specific requirements for a certain application. The primary applications are:

- o Bedding and padding material for the pipeline
- o Nonfrost-susceptible material for Type V ditches

2.2.8 Clearing Quantity Analysis

The selected clearing approach will allow machine clearing within the construction zone limits immediately preceding civil or pipeline construction. It will reduce environmental impact, maximize equipment efficiency, and ultimately, lower construction cost. This will be accomplished by clearing the workpad foundation and stripping soil by machine immediately ahead of construction, and then returning to machine clear the ditch and ditch spoil area at the time of excavation.

Hand clearing will be limited to areas where steep slopes are located adjacent to environmentally sensitive streams, and other areas where the use of heavy equipment would be detrimental to existing conditions.

2.2.9 Pay Items

Pay items for materials and cubic yard miles (cu yd-mi) are as listed below:

- o Common and Waste Material Haul Analysis Worksheet

Construction zone grading (excavation)	cu yd-mi
Workpad embankment (borrow)	cu yd & cu yd-mi
Ditch Backfill, Common	cu yd & cu yd-mi
Access Road Embankment	cu yd & cu yd-mi
Other Off ROW Embankment (borrow)	cu yd & cu yd-mi
Spoil Disposal	cu yd-mi

- o Special Material Haul Analysis Worksheet

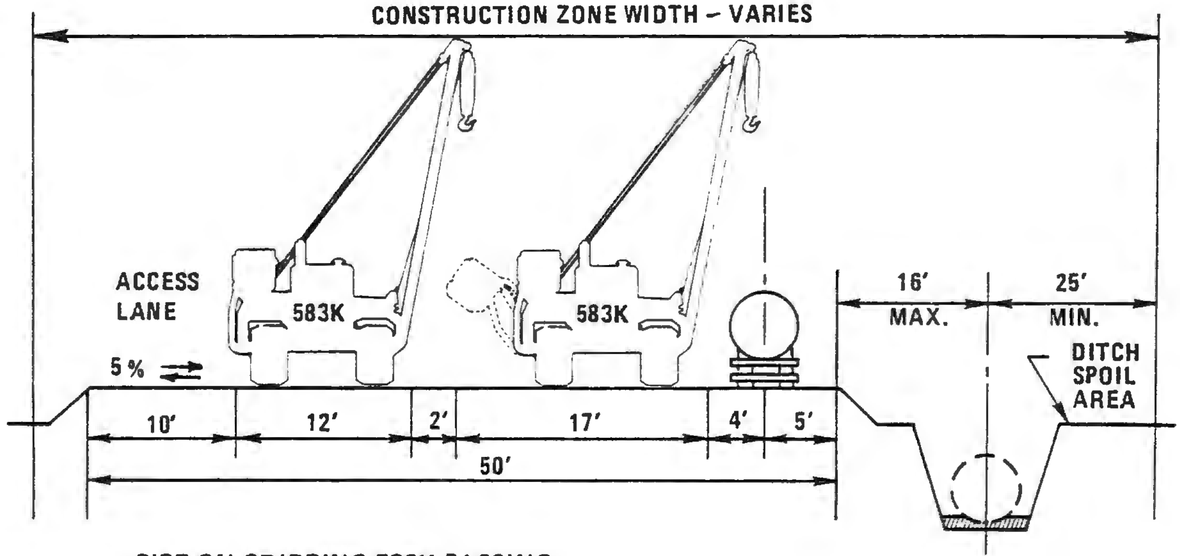
Ditch Backfill, Special	cu yd & cu yd-mi
Pipe Ditch (Excavation)	cu yd
Ditch Backfill, Special (Nonfrost)	cu yd & cu yd-mi

- o Clearing Quantities

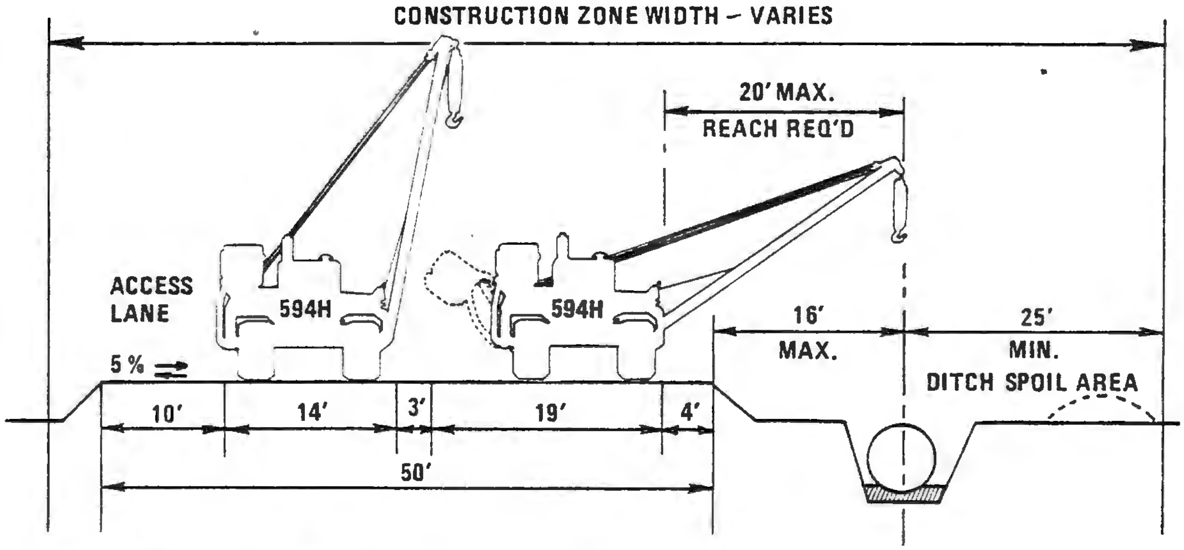
Hand Clearing	- Heavy	acre
Hand Clearing	- Medium	acre
Hand Clearing	- Light	acre
Machine Clearing	- Heavy	acre
Machine Clearing	- Medium	acre
Machine Clearing	- Light	acre

2.0 fast

TYPICAL CONSTRUCTION ZONE CROSS SECTIONS INCLUDING GEOMETRIC REQUIREMENTS

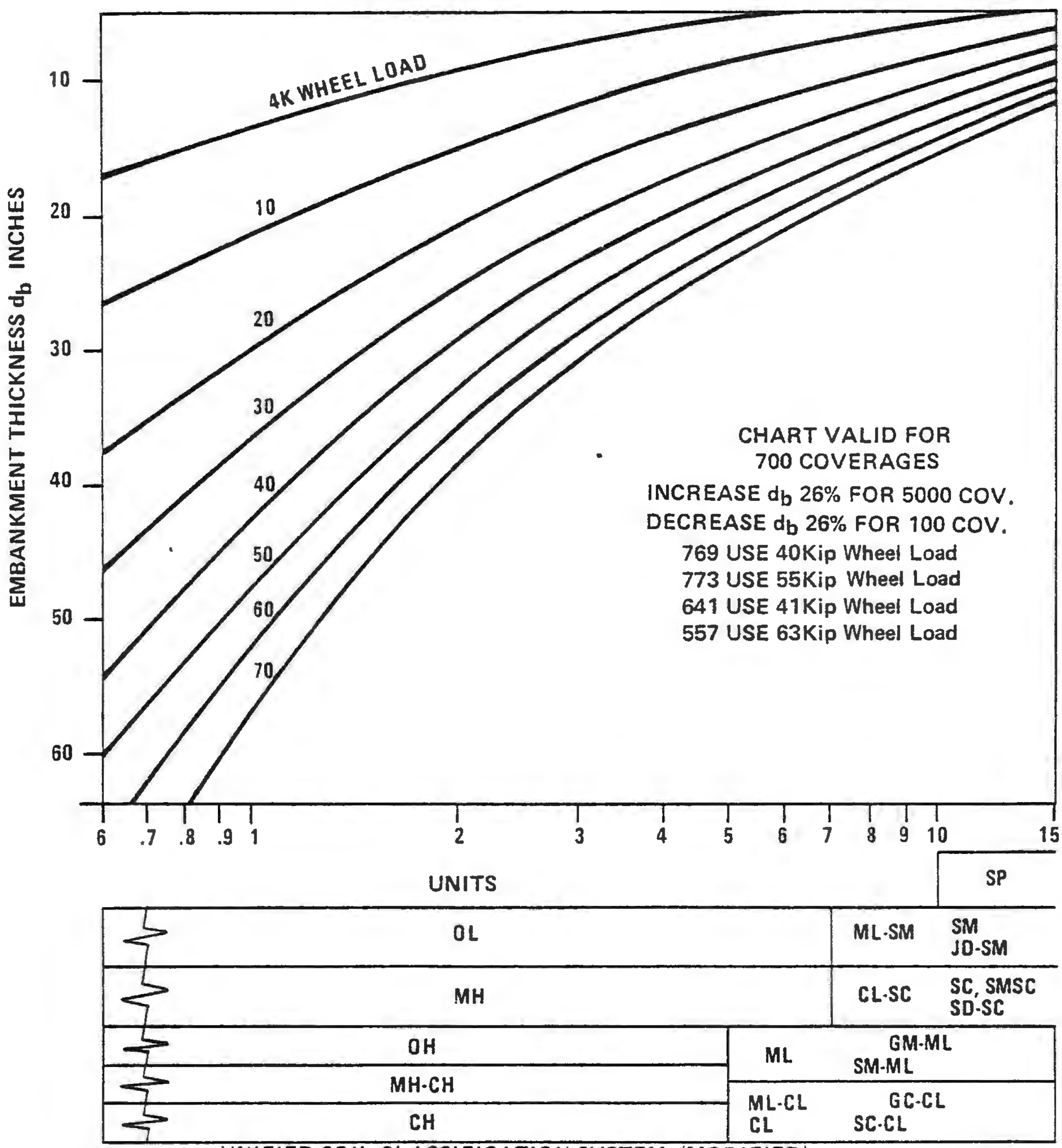


a. PIPE ON CRIBBING 583K PASSING



b. 594H PASSING PIPE LOWERING 594H

EMBANKMENT THICKNESS FOR LOW STRENGTH SOILS



UNIFIED SOIL CLASSIFICATION SYSTEM (MODIFIED)

FIGURE Z-9.1-2-2

TABLE 2-1

CATEGORY CHART A (EXISTING FACILITY LOCATION RELATIVE TO THE GAS PIPELINE CONSTRUCTION)

Adjacent to TAPS Line (0-9) and (10-19)

OL - TAPS A/G WP Lt of the Pipeline OR - TAPS A/G WP Rt of the Pipeline	Single Pad = 25' Ext. (Opposite side of TAPS Pipe)
IL - TAPS A/G Pipe Lt of the Pipeline IR - TAPS A/G Pipe Rt of the Pipeline	Double Pad = 50' Ext. (Adj. to TAPS Pipe)
10L - TAPS B/G WP Lt of the Pipeline 10R - TAPS B/G WP Rt of the Pipeline	Single Pad, use TAPS pad w/overlay
11L - TAPS B/G Pipe Lt of the Pipeline 11R - TAPS B/G Pipe Rt of the Pipeline	Double Pad, construct new pad

Adjacent to Prudhoe Bay Road (20-29)

20L Haul Road Lt of the Pipeline
20R Haul Road Rt of the Pipeline

Haines or Separate Location (30-39)

30 Haines ROW, GVEA ROW, or Separation Location (Left or Right not used)

TABLE 2-2

CATEGORY CHART B
GROUND CROSS-SLOPE PERCENT AND DIRECTION

Cross-Slope Nos. 0-50

No.	Slope Range	Avg. Slope for Quan. Calc.
OLR	0 - 5	0
5	5 - 10	7-1/2
10	10 - 15	12-1/2
15	15 - 20	17-1/2
20	20 - 25	22-1/2
25	25 - 30	27-1/2
30	30 - 35	32-1/2
35	35 - 40	37-1/2
40	40 - 45	42-1/2
45	45 - 50	47-1/2
50	50 - 55	52-1/2

L or R is used with ground slope numbers.

L - Ground slope with low side to the left.
R - Ground slope with low side to the right.

TABLE 2-3

WORKPAD STRUCTURAL THICKNESS COMPONENT
db FOR VARIOUS SOIL TYPES

Structural Thickness Component, db	Underlying Soil Type	Unified Soil Classification System (Modified)
0 inches	Gravels	GW, GP
6 inches	Sands & Gravels	SW, SP, GW-GM, SW-SM, SP-SM, GP-GM, GW-GC, GP-GC
16 inches	Silty or Clayey Gravels and Sands	GM, GM-GC, GC, SW-SC, SM, SM-SC, SC, SP-SC
18 inches	Gravely or Sandy Silts and Clays	SM-ML, SC-CL, ML-GM, ML-SM, GC-CL, CL-SC, GM-ML
20 inches	Low Plasticity Inorganic Silts and Clays	ML, ML-CL, CL
24 inches	High Plasticity Inorganic Silts and Clays	OL, MH, OH, MH-CH, CH

TABLE 2-4

SOIL EROSION CODE GRADATIONS

Soil Erosion Code Symbol	General Name	Modified Unified Soil Classification
G	Clean Gravel	GW, GP
D	Dirty Gravel	GM, GC, GP-GM, GM-ML, GW-GM, GP-GC, GC-CL, ML-GM, GM-GC, CL-GC, GW-GC
Q	Sands	SW, SP, SM, SC, SM-SC
L	Loess	ML
S	Silts	MH, ML, ML-SM, SM-ML OL, OH
FU	Frozen Upland Silt (Always Frozen)	MH, ML, ML-SM, SM-ML OH, OL
C	Clays	CL, ML-CL, CH, MH-CH, SC-CL, CL-SC, CL-ML
O	Organics	Pt, OL, OH
Rc	Competent Bedrock	
Rw	Weathered Bedrock	

TABLE 2-5

	SEC CLASSIFICATION			
	G	D	Q	L,G,S, FU,O*
Sieve Size	% of Particles Passing Sieve			
3 inch	40-100	10-100	60-100	50-100
#4	0-60	10-70	60-100	50-100
#200	0-10	10-50	0-40	50-100

*O soils usually have a large silt or clay content.

TABLE 2-6

SIDE SLOPES FOR STABLE CHANNELS

SEC Classification	Shallow Channels d 4 ft.	Deep Channels** d 4 ft.
Clean Gravels (G)	2:1	3:1
Dirty Gravels (D)	1-1/2:1	2:1
Sands (Q)	1-1/2:1	2:1
Silts (S)	1:1	1-1/2:1
Frozen Upland Silts (FU)	1/4:1	1/2:1
Clay (C)	1/2:1	1:1
Loess (L)	1/4:1	1/2:1
Organic Soils (O)*	Vertical	Vertical

*Most organic mats are thin (less than 2 feet); channel side slopes are designed for the underlying soil.

**Also used it for channels with 4 feet or less bottom width.

TABLE 2-7

SOIL EROSION CODE CUT SLOPE STANDARDS

SEC Symbol	Soil Cut-Slopes Permafrost[b]	Non-permafrost
G	2:1	1-1/2:1
D	2:1	2:1
Q	2:1	1-1/2:1
L	1/4:1	1/4:1
S	1/4:1	2:1
FU	1/4:1	- -
C	2:1	1-1/2:1
O	a	1/4:1
Rc	1/4:1	1/4:1
Rw	1-1/2:1	1-1/2:1

(a) Organic material (O) will have the same design cut slope as required for the underlying natural soil

(b) Maximum cut slope applies only to exposed cut faces, not to bench cuts.

Docket No. CP80-
Exhibit Z-9.1
Hearing Exhibit No.

TABLE 2-8

SOIL EROSION CODE FILL SLOPE STANDARDS

SEC Symbol	Soil Fill Slopes* Permafrost	Nonpermafrost
G	1-1/2:1	1-1/2:1
D	1-1/2:1	1-1/2:1
Q	1-1/2:1	1-1/2:1
L	2:1	1-1/2:1
S	2:1	1-1/2:1
FU	2:1	- - -
C	2:1	1-1/2:1
O	2:1	1:1
R_c	1:1	1:1
R_w	1:1	1:1

*After restoration - 1:1 fill slopes will be used during construction and will be graded to the slopes shown above during restoration.

Docket No. CP80-
Exhibit Z-9.1
Hearing Exhibit No.

TABLE 2-9
EARTHWORK FACTORS - CIVIL CONSTRUCTION

Construction Zone Grading (Workpad Construction)

Material Site Materials*

Shrinkage (during construction)	=	14.33%	In-Place at Material Site
Spillage (during construction)	=	2.00%	
Maintenance (after construction)	=	13.67%	
Total Loss and Maintenance		30.00%	

$\frac{1}{1.0-0.3}$ = 1.43 cu yd @ material site will provide 1.0 cu yd compacted material on workpad

Workpad construction (including shrinkage and spillage) =
0.8633 x total cu yd in-place @ material site

Maintenance (after construction) =
0.1367 x total cu yd in-place @ material site

Excavation From Cut Sections Suitable For Workpad Material*

Shrinkage (during construction)	=	14.33%	In-Place at Excavation Sites
Spillage (during construction)	=	2.00%	
Total Loss		16.33%	

Workpad Construction (including shrinkage and spillage):
0.8633 x 1.43 = 1.2345 cu yd of excavation (will provide 1.0 cu yd of compacted material on workpad)

Excavation From Cut Sections

Workpad Construction = 0.8633 x total cu yd in-place from excavation and material site

Material From Material Site

Workpad Maintenance = 0.1367 x total cu yd in-place from excavation and material site

TABLE 2-9 (Continued)
EARTHWORK FACTORS - CIVIL CONSTRUCTION

Construction Zone Grading (Workpad Construction)

Construction Zone Excavated For Disposal

1.0 cu yd Neatline Excavation = 1.0 cu yd of Disposal Material

Pipe Ditch - (Backfill and Excavation)

Material site or suitable excavation materials

COMMON BACKFILL

Shrinkage (during construction)	13.00%	In-Place at
Spillage (during construction)	2.00%	Material Site or
Total Loss	15.00%	Excavation Volume

$\frac{1}{1.0-0.15}$ = 1.1765 cu yd @ material site or the excavation volume will provide 1.0 cu yd of compacted common backfill

Stable Ditch - Additional Quantity = 25% (For mounding, standard over-excavation of pipe ditch and wall) (Ditch Slopes 1/4:1)

Unstable Ditch - Additional Quantity = 70% (For mounding, ditch wall sloughage and over-excavation) (Ditch Slopes 1/2:1)

Common Pipe Ditch Backfill (In-place @ material sites or excavation volume)

Stable Ditch Quantity = Neatline Ditch Area x 1.25 x 1.1765 x 5280/27 or Neatline Ditch Area in sq ft x 287.59 cu yd-mi

Unstable Ditch Quantity = Neatline Ditch Area x 1.70 x 1.1765 x 5280/27 = Neatline Ditch Area in sq ft x 391.12 cu yd-mi

TABLE 2-9 (Continued)
EARTHWORK FACTORS - CIVIL CONSTRUCTION

Pipe Ditch - (Backfill and Excavation)

Special Backfill Bedding, Padding, and Nonfrost-Susceptible Backfill

Shrinkage	11.00%
Spillage	2.00%
Total Loss	13.00%

$\frac{1}{1.0-0.13}$ = 1.1494 cu yd processed material @ material site will provide 1.0 cu yd bedding and padding or nonfrost-susceptible (select backfill) of compacted backfill in pipe ditch

Stable Ditch - Additional Quantity = 25% w/Ditch Slopes 1/4:1

Unstable Ditch - Additional Quantity = 70% w/Ditch Slopes 1/2:1

Special Pipe Ditch Backfill (In-Place @ Material Site)

Stable Ditch Quantity = Neatline Ditch Area in sq ft x 125 x 1.15 x 5280/27

or = Neatline Ditch Area in sq ft x 281.11 cu yd-mi

Unstable Ditch Quantity = Ditch Area x 1.70 x 1.15 x 5680/27

or = Neatline Ditch Area in sq ft x 382.31 cu yd-mi

Pipe Ditch Excavation For Disposal

1.0 cu yd Neatline Excavation = 1.0 cu yd of disposal material

* NOTE: The section of the workpad constructed on right-of-way excavation will be maintained with material from the material site. Material required for maintenance of the above workpad sections will be 0.1955 cu yd @ material site (0.1367 x 1.43). This material will provide the required quantity for each 1 cu yd of compacted material on the workpad.

3.0 FROST HEAVE DESIGN

Frost heave is the uplifting of the soil mass caused by the freezing of water. The total heave and the rate of heave is dependent on the thermal regime, the original moisture content of the soil mass, and on the amount of water migrating to the freezing front. Soil masses having different gradations, densities, overburden pressures, moisture contents, and groundwater availabilities may result in different heave rates and accordingly differential heave. Differential heave may induce intolerable strains on or in facilities within that soil mass.

This is important to a chilled pipeline because a differential heaving of the pipe could induce stresses that would endanger the integrity of the system.

The following sections present criteria, definitions and procedures for analyzing this phenomenon and for applying mitigative measures in the design.

3.1 FROST HEAVE DESIGN CRITERIA

The following are criteria used to assess the pipeline route frost heave potential on an area-by-area basis. The design approach is based on a buried pipe configuration.

3.1.1 Frost Heave Potentials

Frost Heave Potentials (FHP) have been assessed along the route based on prelimiary frost heave criteria. Three heave potentials (low, moderate, and high) were defined in terms of three field conditions: thermal, silt content, and groundwater conditions. The state and confidence level of each of the three conditions determined a segment Frost Heave Potential.

Table 3-1 shows how Frost Heave Potentials (FHP) were defined in terms of three conditions: thermal (frozen or unfrozen), silt content, and groundwater condition. Three heave potentials were used: Low Heave Potential (LHP), Moderate Heave Potential (MHP), and High Heave Potential (HHP). The state and confidence level of each of the three conditions determine Frost Heave Potential. Table 3-2 summarizes the geotechnical and thermal conditions, states, and confidence levels for each of the three FHPs. The frost heave criteria will be refined as new field and laboratory data becomes available.

Application of the frost heave criteria requires the geotechnical assessment of the states and confidence levels of thermal, silt content, and groundwater conditions encountered along the pipeline alignment. Geotechnical criteria for that assessment are shown in Table 3-3 specifically for use on an area-by-area basis with the stated frost heave criteria. The resultant assessment is termed the frost heave RG2C (route geotechnical characterization and classification). The assessment criteria are subject to modification and refinement as improvements are identified.

The geotechnical assessment process uses input data from: terrain unit maps, landform profiles, airphotos, borehole logs, laboratory data, and statistical tabulations of laboratory soil information. These inputs, the criteria, and geotechnical engineering/geological knowledge and judgments, form the basis of the frost heave RG2C. All geotechnical interpretations used in assessing frost heave potentials are documented, area-by-area.

3.1.2 Preliminary Heave Strain Values

Frost heave potentials are quantified by assigning heave strains to the different classifications for the purpose of preliminary frost heave design.

Preliminary Heave Strain Values

Low Heave Potential	(LHP)	= 0% Heave Strain
Moderate Heave Potential	(MHP)	= maximum 20% Total Heave Strain
High Heave Potential	(HHP)	= maximum 50% Total Heave Strain

Differential heave over limited pipe span lengths was assumed to effectively limit the above heave strain values. Preliminary pipe stress analysis results predicted allowable heaves for various soil strengths and heave lengths. To illustrate how these studies would impact design heave strain values, a heaving length of 100 feet was assumed. Stress analysis predicted an allowable heave of 18 inches to correspond to this heave length. For reasons listed below, it was assumed that over a span of 100 feet, the differential heave would be about 1/2 of the maximum total heave. That is, the difference in heave strains between any two points separated by 100 feet would not exceed 1/2 of the total heave strain at any point. This is equivalent to using 1/2 the assumed maximum heave strains for examining heave induced stress effects on the pipe. The resulting values are shown below:

Differential Heave Strain

LHP = 0 Percent Maximum Differential Heave Strain in 100 feet
MHP = 10 Percent Maximum Differential Heave Strain in 100 feet
HHP = 25 Percent Maximum Differential Heave Strain in 100 feet

The basis for these assumptions ultimately rests on engineering judgment; rigorous justification for any reasonable number is not possible with the current state of frost heave understanding. The values for differential heave strain have been strongly conditioned by several facts:

1. a) There is wide scatter in the (CRREL) data between frost heave magnitudes and soil types; b) potentially excessive heave behavior has been observed in the lab for nearly all natural soils.

2. The Calgary test site, located in silt and clay, shows a) only about 20 percent heave strain, and b) differential heave is a small fraction of total heave.

3. In situ freeze plate tests in White River and Beaver Creek (HAL) floodplains showed no heave in clean sands and gravels.

4. Pipe flexural rigidity and soil uplift resistance will a) both dampen total heave by imposing significant stress on heaving soils and reduce differential heave by stress redistributions over short span lengths. In addition, it will b) provide potential total and differential heave mitigation by soil creep in the frost bulb over the 25-year design life.

5. Natural limits on in situ water availability will restrict actual heave below potential heave as measured in lab tests.

6. Geotechnical variability is generally less over short distances relative to longer distances.

7. Pipe padding and bedding will tend to lessen differential heave over short distances.

8. Where used for frost heave mitigation, overexcavation and backfill with nonfrost-susceptible soil will lessen differential heave over short distances.

Fact "3" was used to set LHP to zero heave strain as previously shown. The unacceptability of excessive potential heave strains suggested by Fact "1" were judged unrealistic because of Facts "2a," "3," "4" and "5." Facts "2b," "4," "6," "7" and "8" suggested a reduction in total heave to account for reasonable differential heave over short span lengths.

3.2 COMPUTER MODEL DESCRIPTION

The EPR computer program simulates two-dimensional heat conduction with a change of state for a variety of boundary conditions. A variational technique is used to obtain temperature distributions and thaw or freeze-front locations at discrete times. The heat of fusion, and changes in heat capacity and thermal conductivity due to thawing or freezing are taken into account.

The work was divided into two phases:

The first phase examined six hypothetical insulation geometries. This phase employed a single set of soil properties (see table below) and an initially thawed soil profile in determining the effect of these proposed design modes on the growth of the frost bulb. Each analysis represented 25 years of chilled pipe operation. The insulating efficiency of each mode was related to frost growth around a bare pipe. Thermal properties of insulation were also considered.

SOIL PROPERTIES USED FOR PHASE ONE GEOTHERMAL ANALYSIS

	Heat capacity BTU/cu ft-°F		Thermal Conductivity BTU/ft-hour-°F		Latent* Heat BTU/cu ft
Material	Frozen	Unfrozen	Frozen	Unfrozen	Heave State
Silt	29.5	39.0	1.21	1.13	3217

*Note that latent heat varies as a function of temperature below 32°F. The listed value is the total extractable latent heat.

Based on the results of the first phase of study, the most efficient insulation configuration for a level soil surface was chosen for refined analysis. Embankments were not examined in this second phase. Phase two considered the effects of various ditch configurations and selected in situ soil and backfill properties on the frost bulb growth for various pipe temperatures.

3.2.1 Ditch Mode Analysis

The most efficient design mode to minimize the effect of frost heave, based on geothermal analysis of several buried chilled pipeline configurations, was a circular insulation placed around the pipe. The results were developed using the EPR computer program (Figure Z-9.1-3-1). Six inches of insulation was considered a practical (construction) upper limit for conceptual work. As additional site-specific data becomes available, insulation thickness refinements will be made as part of the mile-by-mile design.

Results and Applications

Based on the results of gas temperature and the ditch mode thermal analysis, a mitigative ditch mode was developed using:

- o An insulated pipe
- o An insulated pipe and overexcavation (replacement of frost susceptible soil with nonfrost susceptible soil)

The amount of over excavation (OX) required is found using the following equation:

$$OX = R - \frac{\text{Allowable Heave (AH)}}{\text{Ice Segregation Ratio (ISR)}}$$

R = Frost Depth below the pipe (from EPR printout)

AH = Allowable heave

A typical buried pipeline nomogram for determinating the total excavation for varying gas temperatures and insulation thicknessess is shown in Figure Z-9.1-3-2.

Crossings

Pipeline stream and river crossings will be buried below the scour depth. The effect of the chilled pipeline operating below major rivers will be minimal.

Other pipeline crossings were simulated using the EPR program. The chilled line was located in an aboveground berm and allowed to cross (approximately perpendicular) the buried TAPS pipeline. The preliminary results indicated that the frost bulb will not penetrate sufficiently to cause danger to the TAPS pipeline.

Mitigative Modes

The mitigative modes developed to minimize the effects of frost heave over a 25 year project life in problem soils include:

- Reroute to avoid soil problem areas
- Insulate the pipeline
- Insulate the pipeline and overexcavate

3.2.2 Insulation Analysis and Results

Computer simulations performed for two different insulation k-factors show that a 25 percent reduction in the k-factor, from 0.02 to 0.015 Btu/ft-hr-°F, results in a 15 percent reduction in the frost depth. A search of available literature, summarized below, indicated that it was reasonable to use a k-factor of 0.015 Btu/ft-hr-°F.

The foam insulations generally have thermal conductivity (k-factor) ranging from 0.010 to 0.014 Btu/ft-hr-°F. However, during the operational life of a foam insulated pipeline, temperature, water, air, exposure to ultra-violet rays and vapor migration can affect the thermal properties of the insulation. At the present time, several long-term aging tests have been performed. Results of these tests show that the k-factor of this type of insulation can decrease (17 percent to 30 percent) with regard to insulative value during its design life, depending on exposure. However, a buried section of insulation with aluminum-faced panels shows no change in k-factor.

3.2.3 Mode Geothermal Analysis (Phase Two)

Using six inches of circular pipe insulation with a k-factor equal to 0.015 Btu/ft-hr-°F, a study of the effect of variable ditch configuration on frost penetration was performed. This study incorporated the effects of backfill soil with properties different from the natural in situ soils.

Soil Properties

Two natural soils and one backfill soil were used in the simulations. The soil types, whose properties are shown on Table 3-4, consisted of:

- o A moderately dense moisture-saturated frost susceptible silt.
- o A dense moisture-saturated frost susceptible predominantly sandy soil.

The backfill was a sandy nonfrost susceptible 90 percent saturated soil. The soil properties listed in Table 3-4 require assumed values for the heave strain and the dry density of the thawed (unheaved) soil. Those parameters actually input to the EPR model are indicated by an asterisk (*) on the Table.

The values for the frozen and thawed heat capacities and the frozen and thawed thermal conductivities are representative values (not necessarily conservative with respect to the contribution to frost bulb growth).

ALP and GAM are parameters used in the EPR model to define the amount of soil moisture remaining unfrozen as a function of temperature below 32°F. The values chosen for silt are typical of a frost susceptible silt. The values chosen for the sand and sandy backfill closely approximate isothermal freezing of all soil moisture at 32°F.

Ground Temperatures

Preliminary results suggest that ground surface temperatures varying seasonally about a given mean temperature have almost exactly the same effect on soil temperatures below the pipe as does a constant surface temperature equal to that mean value.

These results apply strictly to level ground surfaces only. For nonlevel ground surface (e.g., embankments) or where seasonal frost behavior is important (e.g., frost jacking or pipe uplift resistance), it may be found that seasonally varying surface temperatures is a necessary refinement.

For phase two analyses, as in phase one, the temperature at the ground surface was constant with time at 32.1°F. Other warmer, less conservative, temperatures will be assessed in the future as their justification becomes warranted.

3.3 FROST HEAVE PIPE STRESS ANALYSIS

The objective of the preliminary stress analysis is to develop sufficient parametric study data to evaluate the tolerable displacements in a buried pipeline under loadings generated by frost heave action. Extensive studies and tests in structural design and pipe/soil interaction are scheduled for 1980 to considerably expand the capability for two-dimensonal modeling of the pipeline with greater accuracy than currently available.

The current analysis of stress and strain induced in a buried pipeline subject to the effects of frost heave action has considered the following parameters:

- Structural tolerance limits
- Pipe characteristics
- Pipeline operating conditions
- Geotechnical boundary conditions including imposed loadings or displacements

Preliminary studies have been performed to investigate the sensitivity of imposed pipeline displacements and generated loadings to variations in assumed geotechnical boundary conditions. These studies are used in conjunction with geotechnical and geothermal work to assist in the preliminary development of a frost heave design approach.

3.3.1 Modeling of Pipeline Heave

The computer program, PIPLIN - II, has been used for preliminary analysis. The pipe characteristics and design conditions used for analyses are outlined in Section 1.0. The stress-strain properties of the steel pipe have been input as a number of points from a plot based on typical data from a pipe manufacturer of 48-inch, 5LX-70 pipe. The restraining effect of the soil on pipeline movement idealized by the bilinear soil load-displacement curve shown in Figure Z-9.1-3-3. From this curve, it is seen that the soil resistance is assumed to increase linearly with increasing displacement until a yield deformation is reached. For displacement greater than this yield deformation, the resistance of the soil is constant and is equal to the ultimate uplift resistance "K" (in kips per linear foot of pipe). Preliminary soil yield displacement and uplift resistance values are discussed in Section 4.0. The effects of differential frost heave are modeled as a midspan displacement of the pipeline as it crosses a segment of frost susceptible soil subject to the strength of the surrounding soil. Curves representing allowable displacement of the pipe versus span of heave have been produced. Studies were performed for two basic modeling configurations for maximum allowed pipe displacement versus span. These configurations are as follows:

- o A uniform heave force model is shown in Figure Z-9.1-3-4. In this model, the effect of heave is considered to exert a net uniform distributed loading of equal magnitude over the full span of frost susceptible soil.

- o A uniform heave displacement model is shown in Figure Z-9.1-3-5. In this model, the frost bulb is considered to displace upward an equal distance over the full span. Uplift resistance "K" is mobilized in the frost span only when the pipe tends to deflect upward from the displaced soil profile.

A series of analyses were run for each configuration under operating pressure and maximum temperature differentials. Analyses were made for spans ranging from 20 feet to 100 feet, for uplift resistance values (K) ranging from 2 to 100 kips per linear foot. Based upon the results of these analyses, the following general observations were made:

- o The stress and strain induced in the pipe as the result of a given displacement decreased as the length of span of the heave increases. Thus, the allowable differential frost heave increases with span length.

- o The stress and strain induced in the pipe as the result of a given displacement increases as the uplift resistance of the soil increases.

- o The reactive force exerted by the pipe on the supporting soil and transmitted to the freezing front increases as the length of span decreases, and increases as the uplift resistance increases.

- o Analysis based on the assumption of an abrupt displacement at the interface between the nonheaving soil and heaving segment indicated that very high forces were generated on the freezing front and restraining soil. This assumption would, in effect, limit the maximum allowed pipe displacement to a value not exceeding that allowed for a span of approximately 50 feet, regardless of the actual length of heaving segment.

3.3.2 Basis for Preliminary Design

The preliminary design is based upon the results of the parametric studies, ongoing work, and typical conditions expected along the pipeline route. The frost heave action is assumed to exert a net uniform distributed heave load vertically upward throughout the heave span, with smooth transitions as shown in Figure Z-9.1-3-4. Soils uplift resistance values and yield displacements are based on the geotechnical input discussed in Section 4.0. These values are as follows:

	Uplift Resistance, kips/ft		Yields Displacement, in	
	Winter	Summer	Winter	Summer
Dense Sand	200	39	1 to 3	1 to 3
Loose Sand	110	21	1 to 3	1 to 3

Uplift resistance values for a medium dense backfill are assumed to be 150 kips per foot for the winter condition and 30 kips per foot for the summer condition. The effective continuous uplift resistance is then assumed to be 50 kips per foot due to the effect of seasonal relaxation. For preliminary design, the maxiumum allowed effective stress level is limited to 70,000 psi. Based on the limits used, the positive temperature differential controls the maximum allowed displacements. The allowed displacements are approximately 50 to 70 percent of that which would be allowed by using strain limits and applying a design factor of 1.0. Figure Z-9.1-3-6 shows the maximum allowed mid-point displacement due to frost heave plotted against heave span for a range of uplift resistance values, "K". Figure Z-9.1-3-7 shows the heave load generated at maximum displacement versus heave span.

As the frost front progresses away from the pipe, the generated upward force on the pipe tends to be spread over a longer span. To illustrate this effect, a simplified assumption may be made which considers the force to be transmitted from the frost front to the pipe uniformly over an effective span equal to the actual heave span plus twice the distance to the frost front as shown in Figure Z-9.1-3-8. Based on this assumption, Figure Z-9.1-3-9 shows the allowable heave plotted against the depth to the frost front for several spans of segregated ice. As an example of use for preliminary mile-by-mile design, assume a 100 foot span of segregated ice.

If the frost front moved five feet below the pipe with a 30 percent ice segregation ratio, the heave displacement would be 18 inches. Figure Z-9.1-3-9 depicts for a depth of frost front of five feet that the allowable heave would be approximately 18 inches. If the predicted heave based upon ice segregation ratio and depth of frost front exceeds that allowable for the span considered, pipe insulation, or over-excavation, and soil replacement or other alternate construction modes must be considered.

3.3.3 Further Study

The preliminary analysis has been based upon conservative assumptions and methodology in an attempt to define particular problem areas and assist in preliminary design. Further work is continuing to refine the analysis procedure, analysis tools, and soils parameters.

The results of the preliminary study showed that the allowable heave depends on the geometry of the transition zone as well as the mechanism used to model the loading produced by frost heave. A study is being planned to investigate the effects of longitudinal heave profiles and loading mechanisms on a straight section of pipe. The analysis will be performed by a finite element computer program which has the capability of performing linear-elastic analysis of periodically loaded prismatic solids. The length of heave segments and the spacing between heave segments will be varied.

The study is expected to include a two-dimensional model of a section perpendicular to the pipe axis with sufficient soil detail below and around the pipe to avoid boundary problems. This model will be used to determine the appropriate method for applying the heave mechanism, such as (1) applied distributed loads over elements constituting the bottom of the frost bulb, or (2) specifying swelling within the region of the frost bulb. The results of this work may provide the basis for expanded parametric analysis, as appropriate.

3.4 PROCEDURE FOR MILE-BY-MILE FROST HEAVE DESIGN

The frost heave design procedure is summarized in seven distinctive but interactive engineering tasks. These are:

- Task 1 Route Geotechnical Characterization and Classification
- Task 2 Frost Heave Effects and Prediction

Task 3 Develop Analytical Tools
Task 4 Separate Route into Segments
Task 5 Parametric Engineering Analysis
Task 6 Frost Heave Design Mode Development
Task 7 Area-by-Area Mode Selection

The frost heave design process will proceed as follows: Having identified the route and set operational criteria, Task 1 and 4, as conditioned by Task 2, establish route segments. Task 2, 3, 5 and 6 establish candidate design modes to mitigate frost heave effects. If a given segment has a frost heave potential, then the design enters Task 7, an area-by-area mode selection process to determine the appropriate mode for frost heave mitigation. Having selected a design mode for a given area, one proceeds to mile-by-mile design.

Task 1 Route Geotechnical Characterization and Classification

The effects exerted by frost heave will vary in a general way along the alignment because of variations in soil properties, groundwater and thermal conditions in addition to pipe operating temperature. The natural variation in geotechnical properties from point-to-point along the alignment results in the potential for differential frost heaving.

Task 1 coordinates the collection of field data and its subsequent geological and geotechnical synthesis into appropriate Field Design Soil Types (FDST). This task includes a procedure for characterizing the soil conditions between borings and accounting for geotechnical variability. This task is directed towards both the overland and river crossing portions of the route.

Task 2 Frost Heave Effects and Prediction

The potential for heave exists when a freezing front advances through soils having access to groundwater. If groundwater is attracted to this freezing front, ice lenses form, heave is generated, and the soil is then classified as frost susceptible. Alternatively, if groundwater is not attracted to the freezing zone, or if heave does not result, the soil is considered to be nonfrost susceptible. The requirement for pipeline design is a quantitative prediction of the amount and rate of frost heave for soils along the right-of-way.

The requirement for heave prediction begins with the proposition that some soils heave and some do not. The statement that certain soils do not heave must be confirmed, and is the first step in a systematic resolution of this problem. Having been able to set aside nonheaving soils, it is then necessary to quantify the magnitude of heave potential in the soils that can heave.

The physics of the frost heave phenomenon is not sufficiently well understood to accurately predict on any theoretical basis the response of a given right-of-way area in terms of practically obtainable soil properties. Therefore, efforts are being directed to obtain empirical predictions of this phenomenon based on laboratory experiments and in situ field testing.

Based on field and laboratory frost heave testing program results, soil, groundwater and thermal conditions along the alignment will be characterized in terms of frost heave susceptability. Similar soil, groundwater and thermal conditions along the route will be described in summary form by deriving Field Design Soil Types. Empirical predictive correlations will be utilized to relate frost heave and frost heave rate to Field Design Soil Types. In those soil types where a satisfactory predictive correlation cannot be obtained, appropriate conservative upper bound heave values will be adopted. The correlation between frost heave and Field Design Soil Types will be mode-dependent wherever the design mode configuration affects potential frost heave.

Task 3 Develop Analytical Tools

This task acquires or develops for project use the necessary analytical tools required in the project frost heave design process. Examples are geothermal conduction and convection models, and the model or models necessary for pipeline stress analysis.

Task 4 Separate Route Into Areas

The output of Tasks 1 and 2, when combined and conditioned by Tasks 5 and 6, result in the route being separated into areas.

Task 5 Parametric Analysis

Formation of ice in the ground beneath a chilled pipeline results in mechanical forces and displacements which will load the pipe structurally. These loadings vary in a general way along the alignment because of variations in geotechnical properties. These differential loadings will stress the pipeline; and if allowed to develop in an uncontrolled manner, could place unacceptable loading on the pipeline. The differential heave is the frost heave design problem.

For a given construction mode configuration, frost heave stress analysis requires an engineering analysis of the coupling of frost heave effects and other geotechnical factors with a structural analysis of the pipeline to quantify the influence of differential heaving. The major objective of these analyses is to quantitatively state for each construction mode the allowable operating conditions, such as differential heaving, that will keep the pipeline within operational tolerance limits.

Analysis of pipe stresses and strains due to prescribed frost heave induced pipe boundary conditions is being conducted on a parametric basis for each design mode configuration. Prescribed boundary conditions, soil load/deformation behavior, and time-dependent effects due to frost bulb growth are being considered, including creep effects and flexural rigidity of the composite pipe/frost bulb beam.

Within this task are other parametric studies including geothermal analysis and frost heave effects.

Task 6 <u>Frost Heave Design Mode Development</u>

Pipeline construction mode designs are being developed to mitigate potential frost heave to acceptable limits. Construction modes are integrated with pipeline frost heave monitoring systems and remedial action plans to provide for a variety of overall designs. These overall designs, called simply design modes, are being developed to provide reliable designs for all potential alignment frost heave conditions. Beginning with conventional burial, design modes will be ranked in order of increasing cost per typical mile.

Task 7 <u>Area-by-Area Design Mode Selection Process</u>

The seven tasks identified are precursors to the final selection of a frost heave design mode for the areas making up the frost heave susceptible portions of the route. The route will be moded area-by-area. Where the Field Design Soil Type for an area is nonfrost-susceptible, a conventional burial design is adopted directly.

If the Field Design Soil Type is frost-susceptible, the first trial design mode, conventional burial, is selected (based on least-cost) for evaluation of suitability. Using the Field Design Soil Type for the area, the conventional burial mode is checked against the results of Task 5 (parametric analysis) to determine if stresses and strains are maintained in the acceptable range. If so, conventional burial is adopted. If not, the next trial mode, in order of increasing cost, is selected and analyzed to determine its suitability. This process continues until a trial mode is found to be adequate. This final trial mode would constitute the least-cost acceptable mode for the area.

Before each trial mode is selected, a check will be made to determine if a reroute could be cost-effectively used to eliminate or mitigate the potential frost heave problem. The whole process is repeated area-by-area until all areas are properly moded.

Design refinements or any special alignment considerations requiring attention are addressed during the mode selection process. Further analysis or design changes are handled at that time on an area-specific basis.

Once an acceptable design mode has been selected for a particular route area, the design can then be implemented within the mile-by-mile design process.

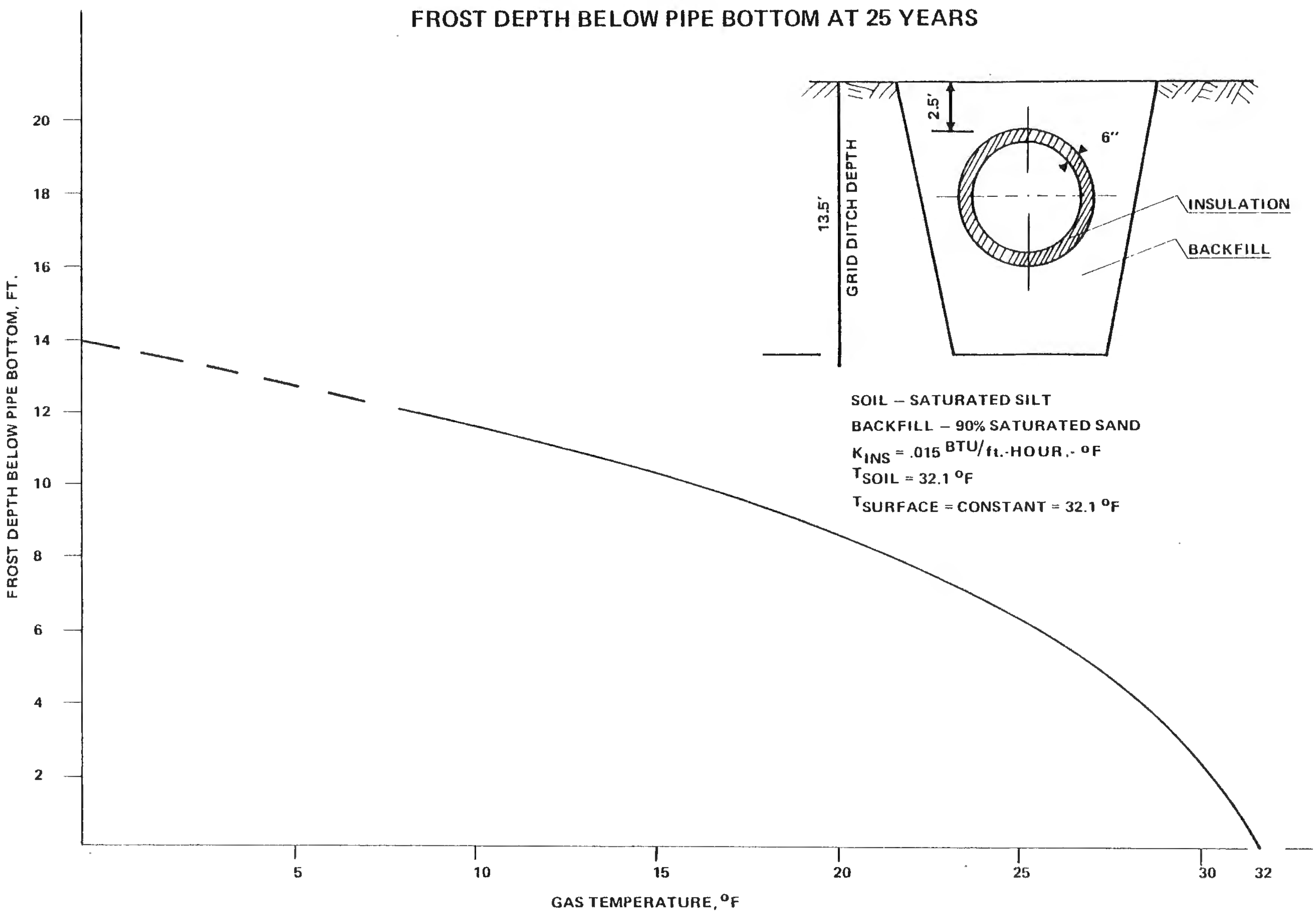
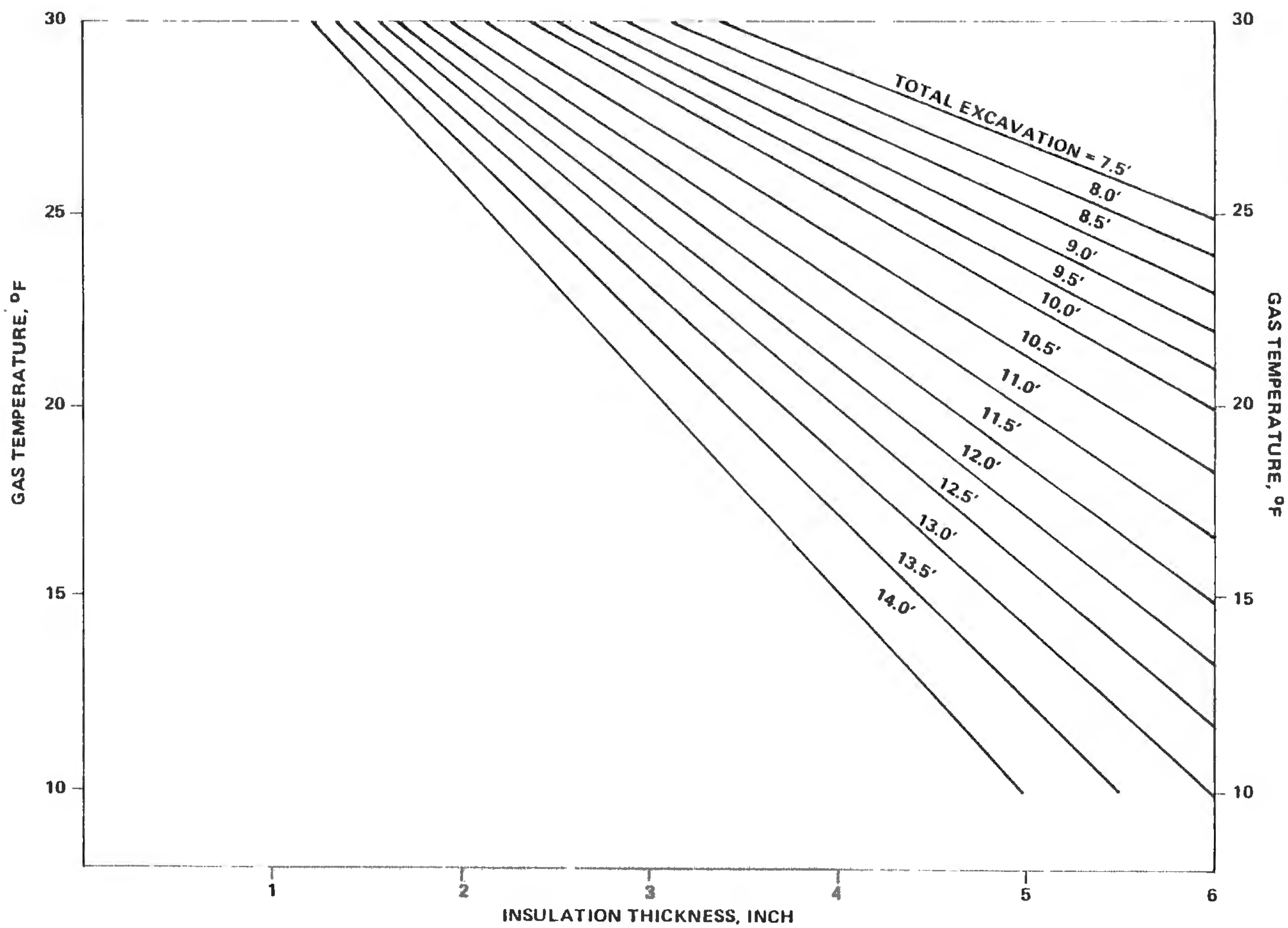


FIGURE Z-9.1-3-1

NOMOGRAM FOR DETERMINATION OF TOTAL EXCAVATION FOR VARYING GAS TEMPERATURES AND INSULATION THICKNESSES



ALL CALCULATIONS AND INTERPOLATIONS BASED ON K_{ins} = .015 BTU/ft. hour °F AND INITIAL BACKFILL DEPTH (GRID DITCH DEPTH) = 13.5′ SOIL – SATURATED SILT, BACKFILL – 90% SATURATED SAND

DESIGN LIFE – 25 YEARS

FIGURE Z - 9.1-3-2

SOIL LOAD DISPLACEMENT CURVE – GENERAL FORM

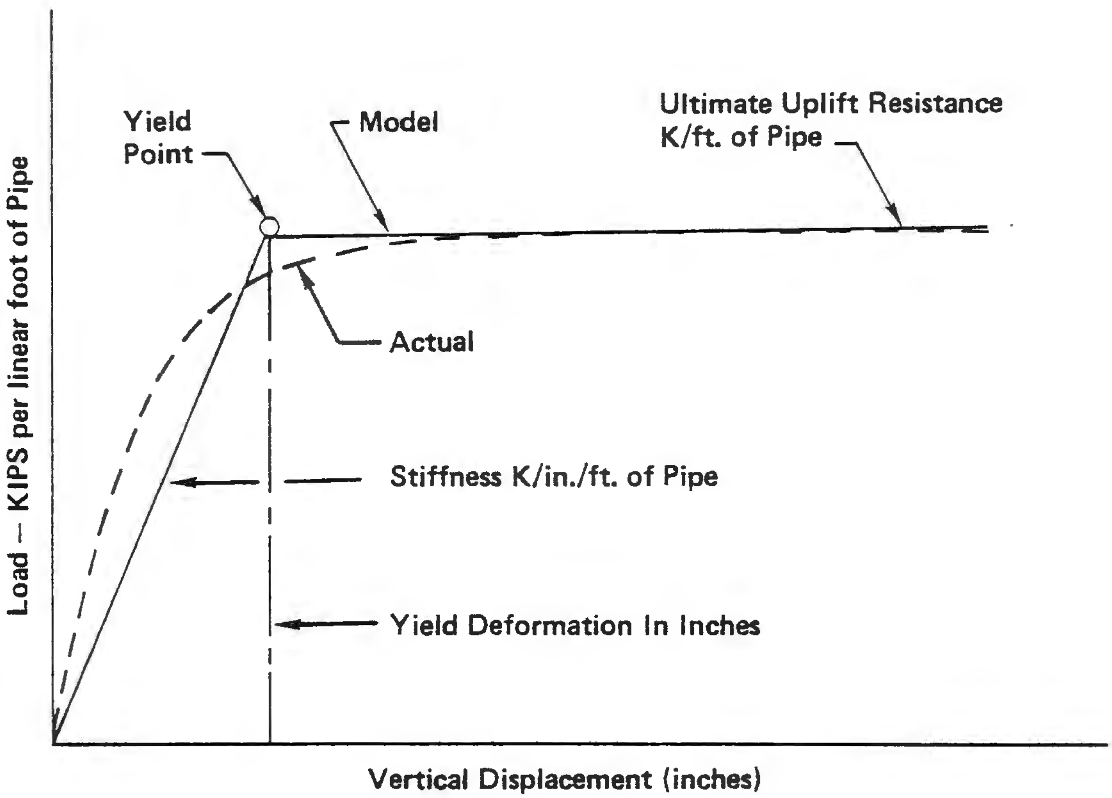


FIGURE Z-9.1-3-3

MAY, 1980

UNIFORM DISTRIBUTED HEAVE FORCE CONFIGURATION FOR COMPUTER ANALYSIS

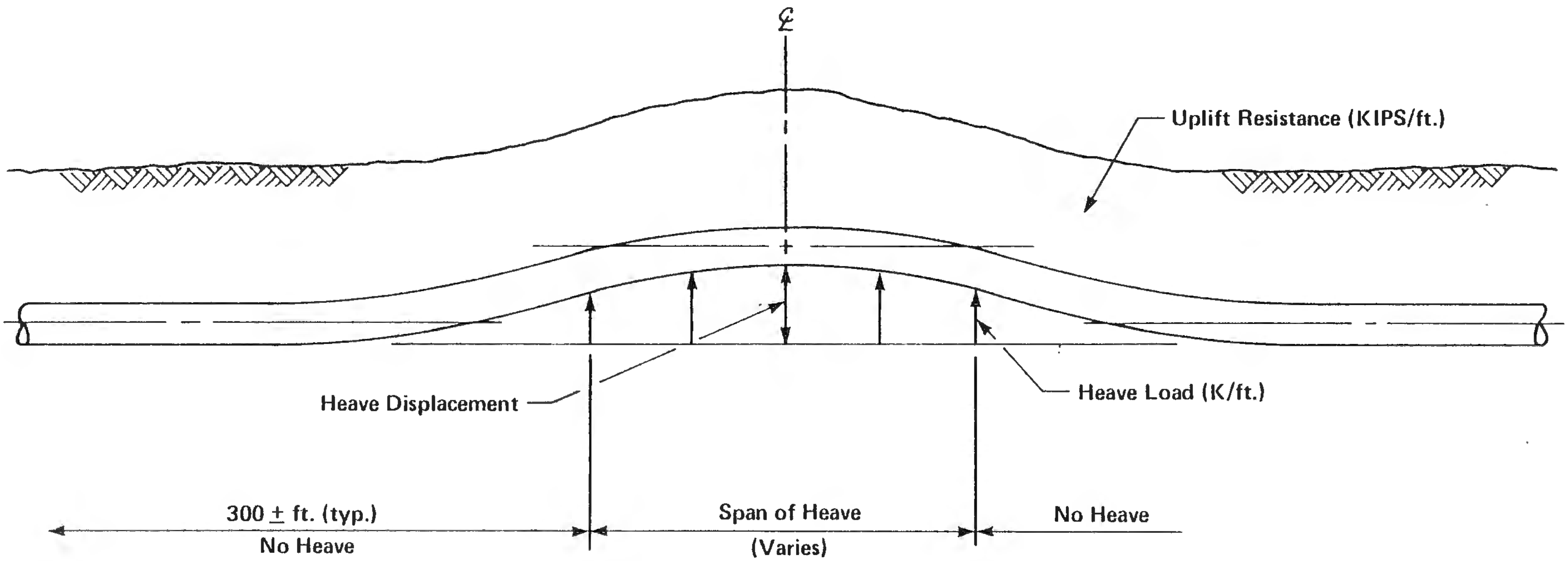


FIGURE Z-9.1-3-4

UNIFORM HEAVE DISPLACEMENT CONFIGURATION FOR COMPUTER ANALYSIS

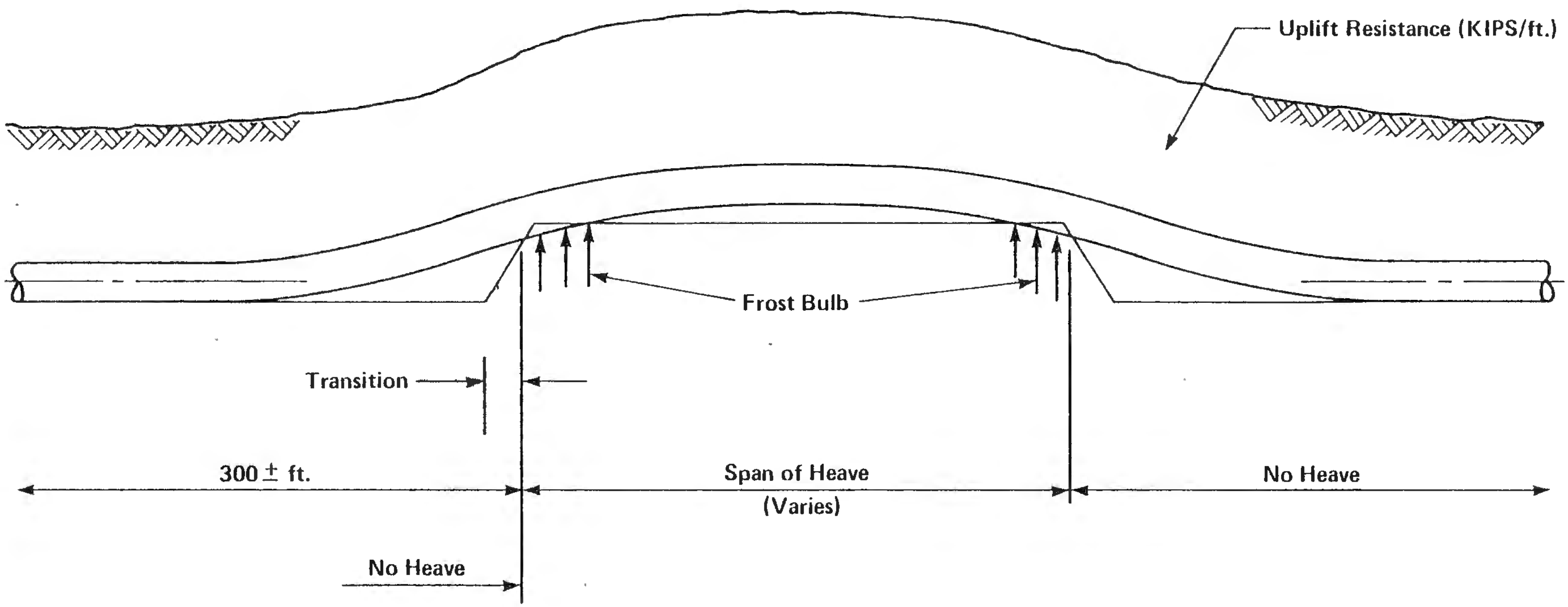
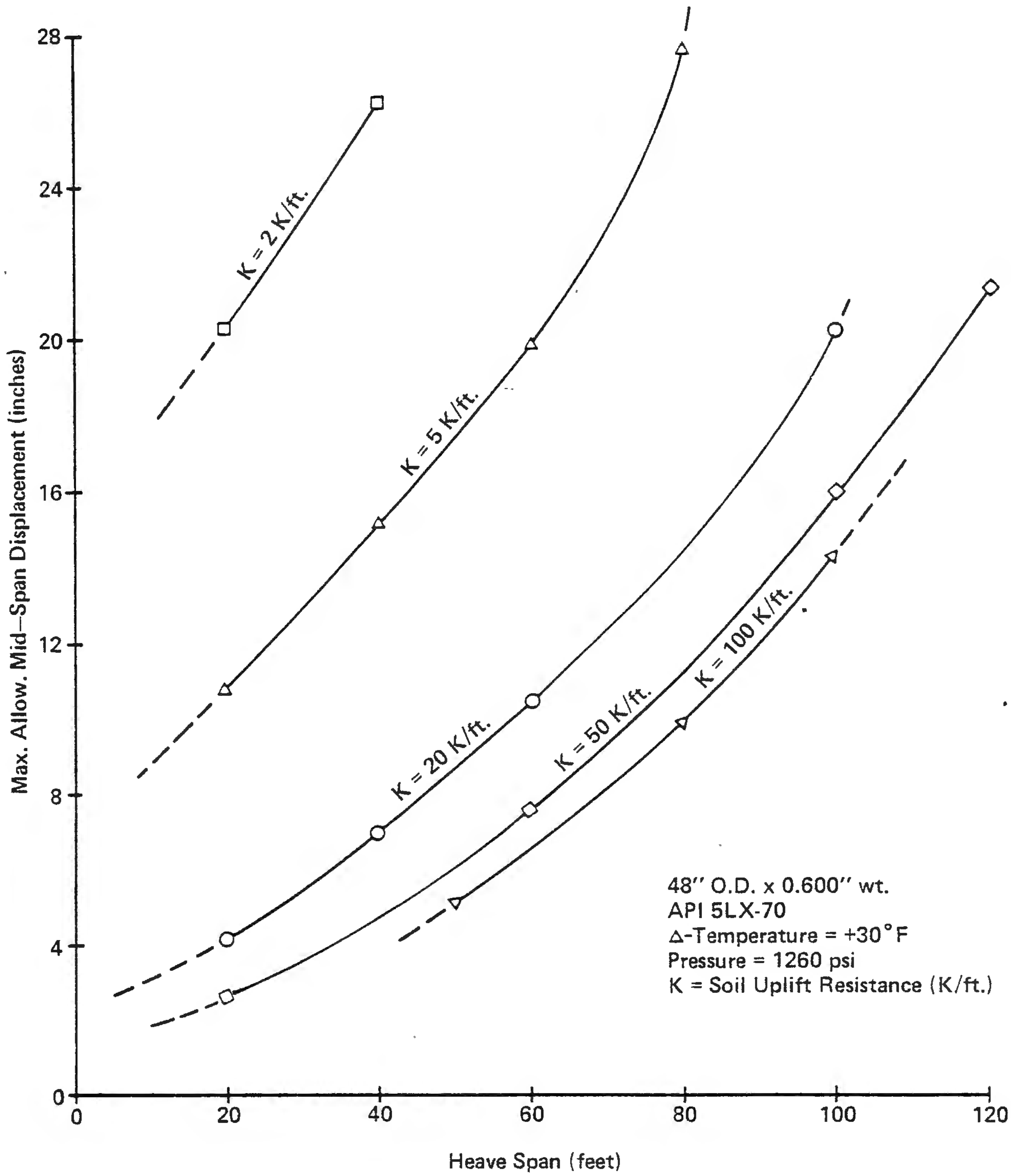


FIGURE Z-9.1-3-5

MAXIMUM MID– SPAN DISPLACEMENT Vs. SPAN
UNIFORM DISTRIBUTED HEAVE LOAD



UNIFORM DISTRIBUTED HEAVE LOAD Vs. SPAN AT MAX. MID–SPAN DISPLACEMENT

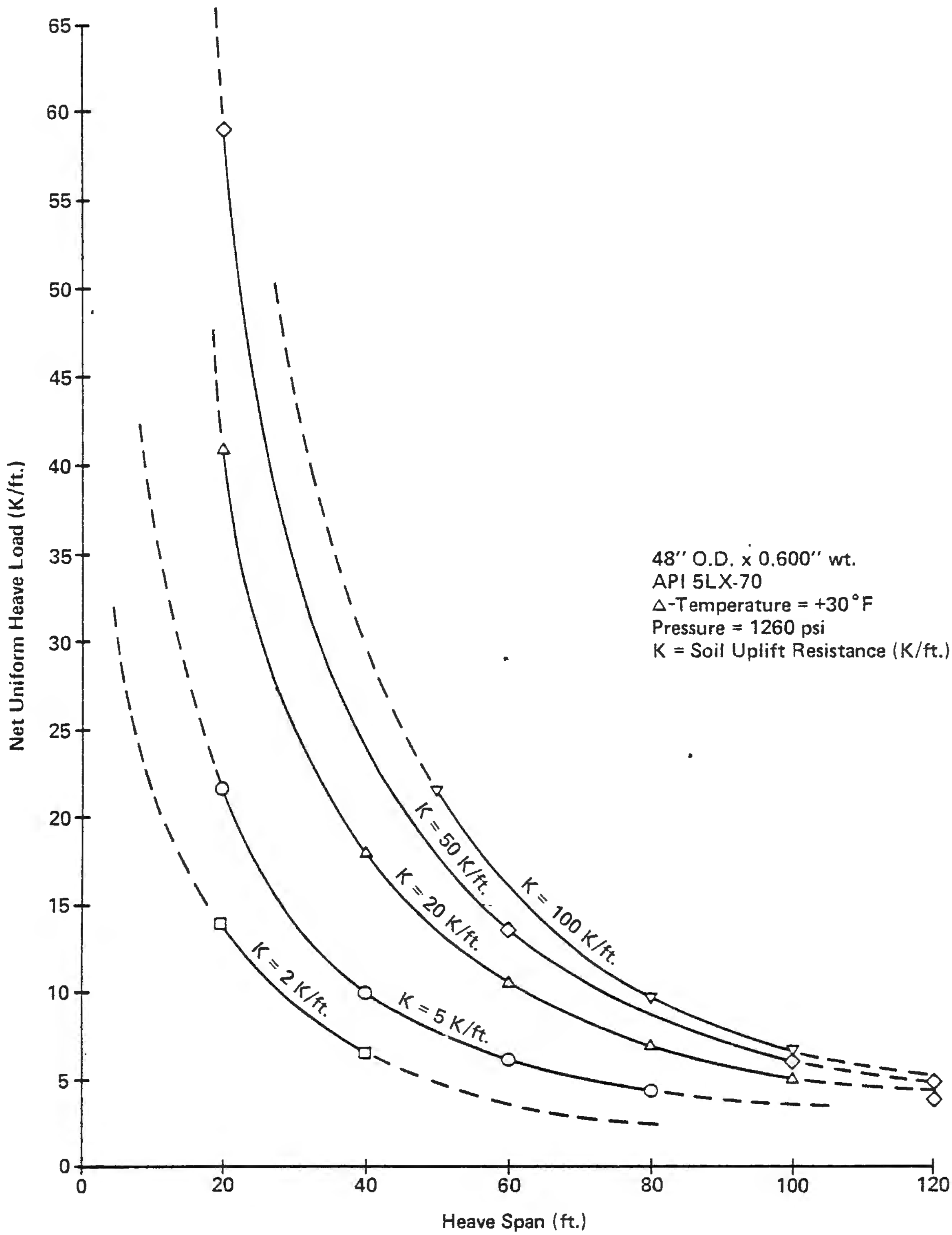


FIGURE Z-9.1-3-7

MAY, 1980

LOAD DISTRIBUTION ILLUSTRATION

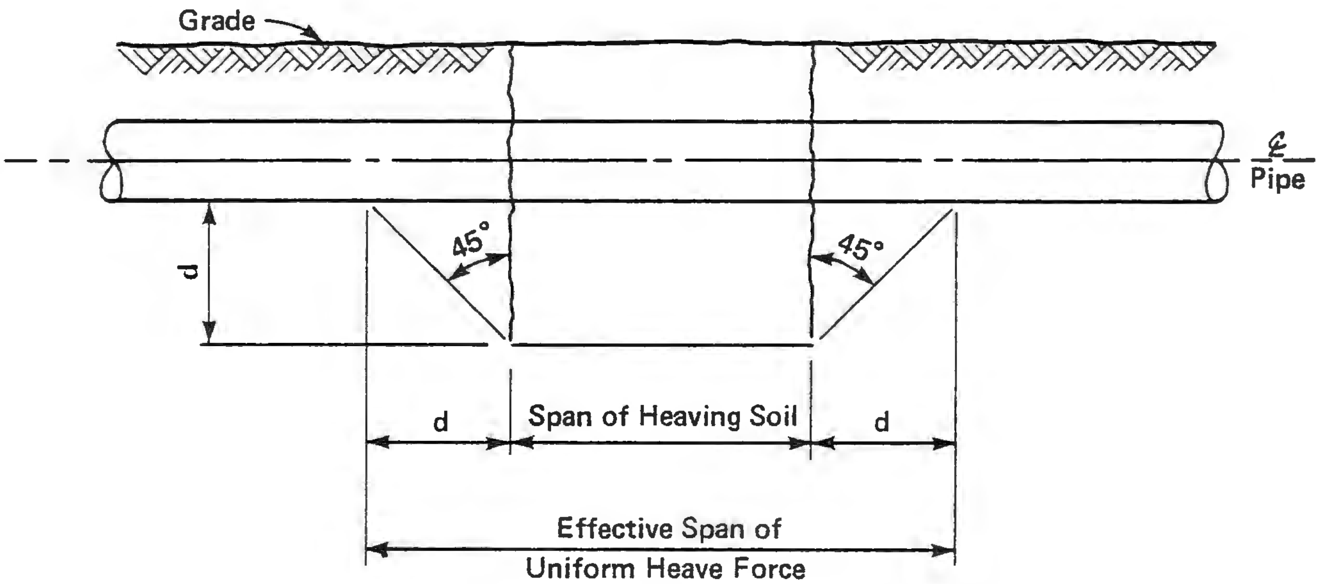


FIGURE Z-9.1-3-8

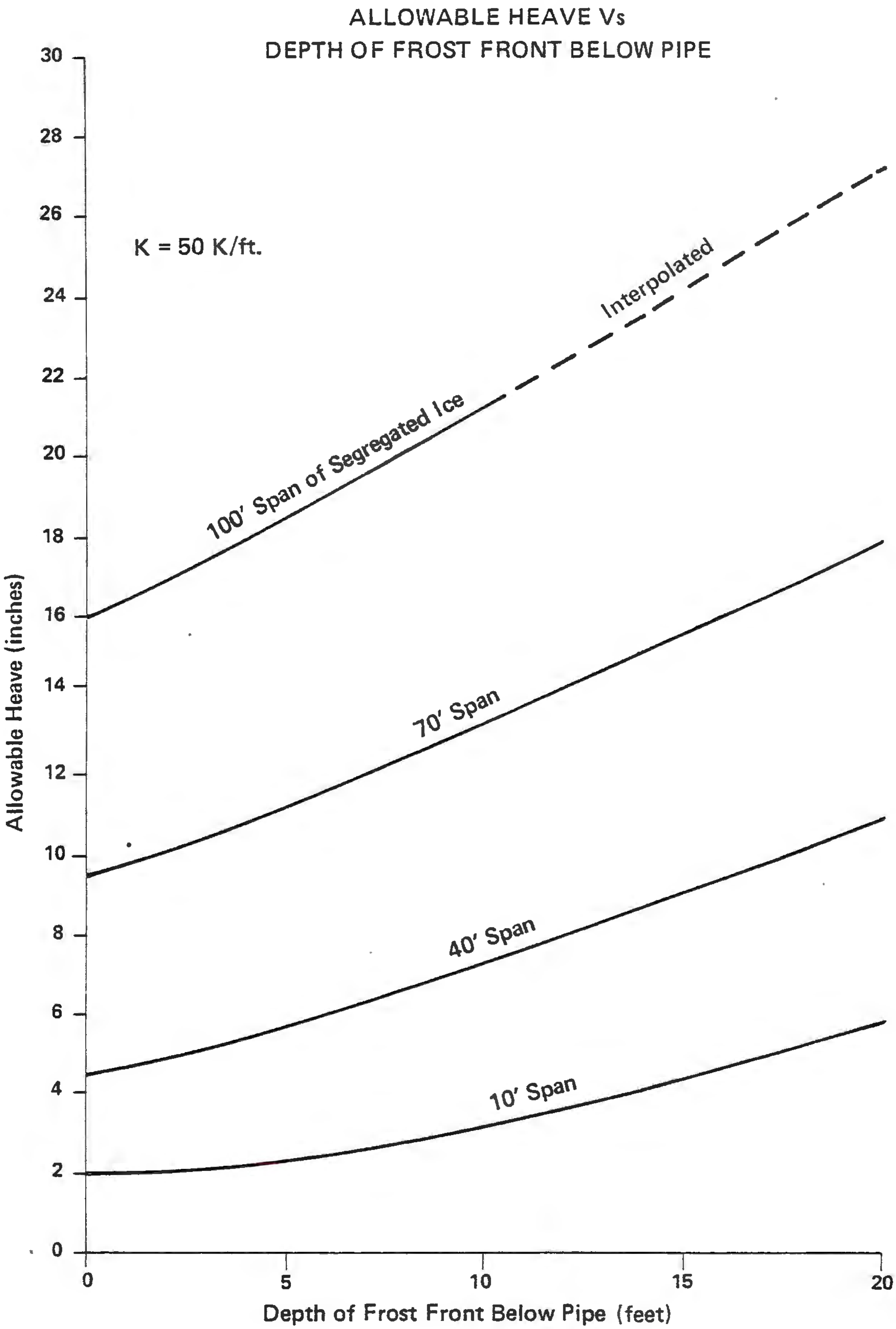


FIGURE Z-9.1-3-9

TABLE 3-1
CRITERIA USED TO DEFINE DESIGN
FROST HEAVE POTENTIALS

CONDITION	STATE	CONFIDENCE LEVEL[+]	FROST HEAVE POTENTIAL
(1) Thermal	Frozen* to 40' Unfrozen	High to Moderate	Check Condition 1A Check Condition 2
(1A) Preoperational Thaw	No thaw below pipe Thaw below pipe	High to Moderate	LHP Check Condition 2
(2) Silt Content	$\leq$6%[x] $\leq$(7 to 12%) >12%	High to Moderate High to Moderate N/A	LHP MHP Check Condition 3 HHP Check Condition 3
(3) Groundwater Table	Below 25 year Design Frost Bulb Depth (DFBD)	High to Moderate	LHP

* 40' refers to depth below nominal ground surface (40' criterion can be site-specifically modified).
\+ Confidence Level as defined in text.
x Silt refers to No. 200 sieve fraction.

LHP = Low Heave Potential
MHP = Moderate Heave Potential
HHP = High Heave Potential

TABLE 3-2
FROST HEAVE POTENTIALS RESULTING FROM GEOTECHNICAL CONDITIONS, STATES, AND CONFIDENCE LEVELS

LOW HEAVE POTENTIAL (LHP)

CONDITION	STATE	FROST HEAVE POTENTIAL
Thermal	Frozen	High to Moderate
Silt	Any	N/A
*Silt	≤6%	High to Moderate
Groundwater Table	Any	N/A
*Groundwater Table	>25 year DFBD**	High to Moderate
Silt	Any	N/A

MODERATE HEAVE POTENTIAL (MHP)

CONDITION	STATE	FROST HEAVE POTENTIAL
*Silt	≤(7 to 12%)	High to Moderate
Groundwater Table	Within 25 year DFBD	Reasonable

HIGH HEAVE POTENTIAL (HHP)

CONDITION	STATE	FROST HEAVE POTENTIAL
*Silt	>12%	Reasonable
Groundwater Table	Within 25 year DFBD	Reasonable

*Thermal Condition Unfrozen
**DFBD = Design Frost Bulb Depth

TABLE 3-3

Geotechnical Assessment Criteria For Frost Heave Potential Determination

Thermal State Determination

SOIL IN FROZEN STATE

Criteria

- o All boreholes in segment frozen continuously (V or N)* from 7 to 40 feet of depth.

  The 40 foot criterion is subject to site-specific revision supported by adequate documentation. Characters V, N, N_f and N_b refer to standard frozen soil classifications.

- o All thermistors read frozen (<32°F).

- o Any EM resistivity indicates frozen.

HIGH CONFIDENCE

High silt content soils (- No. 200 fraction >30 percent)

- o Any combination of V and N without "significant" N_f. ("Significant" will be defined by the area's documentation.)

Low silt content soils (- No. 200 fraction $\leq$30 percent)

- o Visible ice (V) in $\geq$50 percent of 7 to 40 foot interval. Note (1)

- o Not visible but well bonded (N_b) in <50 percent of 7 to 40 foot interval. Note (1)

MODERATE CONFIDENCE

High silt content soils (- No. 200 fraction >30 percent)

TABLE 3-3 (Continued)

- o Any combination of V and N

Low silt content soils (- No. 200 fraction $\leq$30 percent)

- o Any combination of V and N without significant N_f. ("Significant" will be defined by the area's documentation.)

SOIL IN UNFROZEN STATE

Any of the following conditions require the assumption of an unfrozen state:

- o Called out in log as frozen but without convincing evidence (such as visible ice, or bonding) of its frozen state.
- o Mixed frozen and unfrozen in the interval 7 to 40 feet in any borehole located in the segment. Note (1)
- o Insufficient information to make reasonable interpretation.
- o Boreholes show soil not frozen.

Groundwater Table (GWT) State Determination

GROUNDWATER TABLE BELOW BOTTOM OF 25 YEAR DESIGN FROST BULB DEPTH

Criteria

- o Test holes in area do not show GWT above bottom of 25 year design frost bulb.
- o Local and regional drainage features do not suggest the possibility of a high GWT.
- o Moisture contents below saturation.
- o Any piezometers show GWT below bottom of 25 year design frost bulb.

Note (1): 40 foot criterion subject to site-specific revision; adequate documentation required.

TABLE 3-3 (Continued)

HIGH CONFIDENCE

- o Average area silt content <12 percent.

MODERATE CONFIDENCE

- o Average area silt content >12 percent, but local drainage conditions limit the possibility of a GWT condition of design concern, adequate documentation required.

GROUNDWATER TABLE ABOVE BOTTOM OF 25 YEAR DESIGN FROST BULB DEPTH

- o No observed GWT in any test hole in area but average silt content too high to preclude possibility of GWT.

- o Local drainage features suggest the possibility of a high GWT during at least part of the year.

- o Insufficient information to make reasonable interpretation.

- o GWT observed above bottom of 25 year design frost bulb.

Silt Content State Determination

EFFECTIVE SILT CONTENT (ϕ) <6 PERCENT

Criteria

HIGH CONFIDENCE

- o All gravels and well-graded gravelly sands between 7 and 50 feet of depth:

  $F_\phi(\phi \leq 6 \text{ percent}) \geq \alpha$ = approximately 80 percent for Terrain Unit/Landform (TU/LF) statistical tabulations having more than about 100 samples.* $F_\phi(\phi)$ is the cumulative distribution function of minus No. 200 sieve size particles determined from soil laboratory tests of representative samples obtained from the TU/LF of the area being analyzed.

TABLE 3-3 (Continued)

- o Poorly graded sands between 7 and 50 feet:

  $F_{\phi}(\phi \leq 6$ percent$) \geq \alpha$ = approximately 80 percent for TU/LF statistical tabulation having more than about 100 samples*.

- o The weighted average, $\bar{x}$, of all samples between 7 and 50 feet depth $\leq$ 6 percent for segment, i.e., $\bar{x} \leq 6$ percent. Can be modified by adequate documentation.

MODERATE CONFIDENCE

- o Same as for high confidence but with α = approximately 70 percent.

* For TU/LF) tabulations not having 100 or more samples inferences using results from similar TU/LF's can be used if adequately documented.

EFFECTIVE SILT CONTENT ϕ <12 PERCENT

Criteria

HIGH CONFIDENCE

- o Same as for high confidence $\phi \leq 6$ percent but with $\phi \leq 12$ percent and $\bar{x} \leq 12$ percent.

MODERATE CONFIDENCE

- o Same as for high confidence but with α = approximately 70 percent.

EFFECTIVE SILT CONTENT ϕ >12 PERCENT

- o $F_{\phi}(\phi \leq 12$ percent$) < \alpha$ = approximately 70 percent
- o $\bar{x} > 12$ percent
- o Insufficient information to characterize silt content with confidence.

TABLE 3-4

SOIL PROPERTIES USED FOR PHASE II SIMULATIONS

	Heave Strain (Assumed) (%)	(Assumed) γ_d Thawed State (lbs/cu ft)	γ_d Heaved State (lbs/cu ft)	w Thawed (% dry weight)	w Heaved State (% dry weight)	C* Thawed (Btu/cu ft-°F)	C* Frozen (Btu/cu ft-°F)	K* Thawed (Btu/ft-hr-°F)	K* Frozen (Btu/ft-hr-°F)	L* Heaved State (Btu/cu ft)	ALP*	GAM
Silt	50	112	75	18.5	42.1	40	28	1.0	1.3	4260	7.0	0.24
Sand	20	130	108	11.0	18.7	40	28	1.25	2.0	2918	0.01	1.0
Backfill	0	130	130	10.7	10.7	40	25	1.5	2.5	2000	0.01	1.0

NOTE: It is assumed for this study that 2.6 percent (percent dry weight) of the water in the frozen silt remains in the liquid phase.

* EPR input parameter.

Docket No. CP80-
Exhibit Z-9.1
Hearing Exhibit No.

4.0 GEOTECHNICAL CONSIDERATIONS

Geotechnical considerations are an important part of the design of the pipeline system. These include subsurface conditions along the line in respect to the design, construction and operation of the pipeline.

The geotechnical program includes geotechnical data collection, identification of route soil conditions, assessment of potential ground movements, and selection of pipeline modes and support. This process is related in Figure Z-9.1-4-1. The pipeline route geotechnical assessment process involves an integrated procedure, as is shown in Figure Z-9.1-4-2.

Route soil condition data constitute the basis of all geotechnical evaluations. A generalized presentation of route soil conditions is summarized in Figure Z-9.1-4-3.

4.1 GEOLOGIC AND GEOTECHNICAL DATA COLLECTION

Geotechnical information on the proposed route of the gas pipeline has been obtained from literature, aerial photo-interpretation, field investigations, borehole drilling programs, engineering properties tests, and geotechnical analyses.

The enormous amount of available information requires the aid of a computer to ensure easy access to the geologic and geotechnical data needed for decision-making, during design, construction, and operational phases of the project. Most of the geologic and geotechnical data are included in the Geotechnical Information System (GIS) computer data bank. The GIS system will evolve with the changing needs of the project and will eventually include information gathered during the construction and operational phases of the project. This system is explained in Appendix D, Z-9.0.

4.2 FIELD EXPLORATION PROGRAMS

Drilling programs are utilized to supplement existing data. Initial field programs concentrated in the Delta South portion of the line because of the general absence of detailed geotechnical information for this area. The geotechnical information obtained in preliminary investigations provided the basis for the initial FPC hearings. Programs initiated to date have concentrated on estimating the amount and distribution of frozen ground, the nature and distribution of general soil conditions and terrain units, the determination of soil index and engineering properties, and groundwater location.

Over 800 boreholes have been drilled along the centerline and at compressor station sites. Approximately 302 miles of ground resistivity work has been completed. Results of these programs are used in this filing as part of the design basis.

4.3 LABORATORY TESTING

Laboratory tests conducted on soil samples recovered from the boreholes include index property tests and engineering property tests.

Index Property Tests - All tests in this category are standard tests that have been specified by ASTM, the U.S. Army Corps of Engineers, or the State of Alaska. Standard soil index tests include moisture content, dry density, grain size analysis, Atterberg limits (L.L. & P.L.), specific gravity, and organic content.

The results of the grain size analyses and the Atterberg limits are used for soil classification according to the Unified Soil Classification System (Modified) Figure Z-9.1-4-4. Classification followed ASTM designation D 2487-69 with the following exceptions: (1) additional classifications have been included for coarse-grained soils having 45 to 55 percent passing the No. 200 sieve, (2) the dual-symbol borderline area, which surrounds the "A-line" on a plasticity chart Figure Z-9.1-4-5 has been extended through the entire range of liquid limits. Index property test results are given in report form and are stored in the Geotechnical Information System (G1S).

Geotechnical Engineering Property Tests - Conventional and special engineering property tests were performed on representative soil samples to determine values of pertinent geotechnical properties of the soils along the proposed alignment, and to establish predictive correlations between geotechnical engineering properties and respective soil index property data. These relationships and the relationships existing between soil type and landform type are used to estimate engineering properties at any given location along the alignment, from available soil index property data.

Unfrozen engineering property tests included the general categories of shear strength, compressibility, and permeability. All tests were made using standard procedures.

Frozen engineering property tests included shear strength tests and uniaxial thaw consolidation tests. Shear strength tests are made to evaluate the shear strength of "undisturbed" frozen soils as functions of temperature, load duration, and allowable strain. Strain rate controlled tests are made in unconfined compression test equipment with apparatus to maintain an accurately controlled temperature below freezing to evaluate creep behavior. Uniaxial thaw consolidation tests are conducted on selected representative undisturbed samples to assess thaw strain potentials. Completed laboratory testing results are placed in the Geotechnical Information System (GIS) for use in engineering analysis and design, including Route Geotechnical Characterization and Classification (RG2C) development.

4.4 ALYESKA PIPELINE SERVICE COMPANY DATA

The incorporation of APSC data into the geotechnical design is currently in progress.

Selected APSC borehole and laboratory data are currently being input into the three distinct GIS data files.

- Thermistor Data File: Thermocouple data from the APSC "Route Thermocouple Report" have been input. These data are utilized by the TPLOT computer program for graphical presentation of soil temperature data.

- Soil Laboratory Data File: Data are presently input from APSC's Soil Index Properties report directly into this file. The SSP computer programs process these data statistically in order to help classify and characterize the proposed route.

- Soil Computer Log Data File: The Soil Index Properties and Borehole Logs for the Trans Alaska Pipeline Route reports provide the data from which Soil Computer Logs for APSC boreholes are generated. These data, along with laboratory data, are used by Soil Statistics Program (SSP) in the route geotechnical classification and characterization.

4.5 GEOTECHNICAL DATA PRESENTATION

Geotechnical data is presented in two basic formats: in map and graphic form on the Route Soil Conditions alignment sheets, and in computer printout form through GIS. Source documents that constitute the raw data used in the route soil sheets and the GIS include: soil computer logs, thermistor data, soil laboratory data, and other borehole data. Various types of source documents are available in the GIS hard copy library, the Central Receiving Group (CRG) files and Project Information Control (PIC) files.

4.6 ROUTE SOIL CONDITIONS ALIGNMENT SHEETS

The Route Soil Conditions Alignment Sheets present a summary of all available geologic and geotechnical information. This information is presented in a mile-by-mile format which presents the pipeline alignment, terrain units, and other pertinent information necessary for pipeline design. These data are presented in both plan and profile as well as in summary listings to facilitate use by the designers.

The sheets are coded: 4680-11-00-C or B-G-001 through 131.

Each Route Soil Conditions Alignment Sheet consists of five distinct parts:

- The terrain unit map band
- The landform type profile band
- The upper data band
- The lower data band
- The explanations and identification band

The terrain unit concept is the basis on which the line is subdivided for all design efforts. A terrain unit is defined as a three dimensional body having mappable boundaries and extending to an arbitrary depth of about 20 feet. Each terrain unit is composed of one or more land forms. Land forms are defined as "an element of the landscape that has a definite composition and range of physical and visual characteristics, such as topographic form, drainage pattern, and gully morphology, which occur wherever the landform is found."[1] The landform classification used here is genetically based as was the classification used by the Alyeska project.

[1] References 17 and 18 (Z-9.0, Appendix C)

Additional discussions of the Alyeska terrain evaluation system are available in papers by Kreig (1977)[2] and Kreig and Reger[3] (1976).

Use of the terrain unit concept facilitates the planning of field data gathering programs and allows the data to be extrapolated with a predictable degree of confidence within the limits of the individual terrain unit. A detailed explanation of landform type and terrain unit symbols is included with each set of Route Soil Condition Alignment Sheets.

4.6.1 The Terrain Unit Map Band

This band consists of a photo mosaic exactly like those used in the other alignment sheet series which will be overprinted with the following:

- The proposed gas pipeline centerline
- Terrain unit boundaries
- Ground surface contours
- Locations of all available boreholes
- Significant special areas such as major faults and fault crossings, liquefaction-prone soils, landslide debris, and potential soil instability.
- The location of resistivity traverses, other geophysical investigations or any other like item

4.6.2 The Landform Type Profile Band

This band is a graphic representation of the landforms (soils), permafrost, and groundwater directly beneath the centerline to a depth of 50 feet, and includes the following:

- The longitudinal ground surface profile along the centerline
- Selected boreholes are drawn on the landform type profile in schematic form
- Permafrost where present as indicated by a special symbol
- Groundwater where present as indicated by small symbols to the left of the borehole in which the water was observed

[2] Reference 20
[3] Reference 19

4.6.3 The Upper Data Band

This band consists of five individual lines.

The terrain unit line contains a lineal representation of the landforms present. It is a summary of the upper 20 feet of the landform type profile band.

The soil type line contains a lineal representation of the soil types present both by name and by Unified Soil Classification (USC). An individual entry may consist of a single soil type, a range of soil types, or several different soil types depending on the variation of soils within the unit both laterally and with depth.

The frost heave classification line contains symbols representing the potential of the soils to heaving upon freezing. The parameters utilized in assessing frost heave potential are soil type present, thermal regime and the amount and availability of groundwater; the presence of freezing temperatures is assumed.

The soil erosion code line contains symbols representing the susceptibility of the soil to erosion by running water.

The soil temperature range line shows the temperature in degrees Fahrenheit for those locations where data are available. The data from each thermistor have been reduced to the maximum and minimum temperatures ever measured at the 5 foot and 20 foot depths. When the period of record does not span one full year, the temperature shown is not necessarily representative of an annual extreme; thus, an asterisk will follow the listed value.

4.6.4 The Lower Data Band

This band consists of five individual lines.

The horizontal scale line simply indicates stationing relating directly to the profiles described in the preceding section.

The permafrost line contains the general, area wide permafrost description. Determination of the permafrost classification is based on borehole data supplemented with information from other similar nearby areas and an analysis of such modifying factors as slope aspect and clearings. The system is used to describe areas only; it does not apply to and is never used to describe vertical properties.

The ground ice line contains a short, generalized description of the character of the ground ice to be expected to a depth of up to fifty feet where appropriate.

The potential hazards line provides a space in which hazards such as avalanche, liquefaction, slope instability, flooding, faulting, and the like can be noted and the area of influence indicated. No entries were made for the FERC Filing in this line.

The reference documents line contains notes which indicate the document in which the reasons for the determination of the potential hazard are detailed. No entries were made for FERC Filing.

The explanations and identification line contains much of the explanatory and identification information necessary to interpret or understand the data presented in the four bands and to identify the alignment sheet.

4.7 ROUTE GEOTECHNICAL CHARACTERIZATION AND CLASSIFICATION

The Route Geotechnical Characterization and Classification (RG2C) is a system designed to determine mile-by-mile geotechnical mechanical and thermal input data for each of the geotechnical engineering considerations, including: frost heave; liquefaction; slope, thaw plug, and ditch stability; and soil/pipe interaction.

Criteria used to select RG2C data values include consideration of: (1) geotechnical variability, (2) constraints/limits imposed by the available data base, (3) details of the analysis methodology, and (4) sensitivity of final designs to error in the RG2C data set.

Each characterization consists of the data set required for the specific geotechnical engineering task. Parameter values are established by analysis and synthesis of data, including: Route Soil Conditions Alignment Sheets, borehole logs, soil laboratory data, and other pertinent data.

The RG2C data set is conditioned by the overall geotechnical engineering solution methodology developed for each specific task. This includes effects of geotechnical variability, constraints/limits imposed by the available data base, details of the analysis methodology and the sensitivity of final designs to error in the RG2C data set. Geotechnical variability is the natural variation in geotechnical, mechanical and thermal properties from point-to-point along the alignment. Constraints and limits in the available data base arise from limited field exploration and laboratory testing, data uncertainty and inaccuracies (due to assumptions implicit in data collection and development, and limitations in sample recovery and testing techniques). Higher sensitivity requires higher conservatism in the RG2C data set to maintain an acceptable failure risk.

For a given level of acceptable risk, the necessary conservatism of the RG2C data must increase in proportion to the geotechnical variability, data base limitations, and engineering sensitivity of the design solutions to nonconservative error in the RG2C input data. The process used to obtain a set of data for a task-specific RG2C is formulated to account for these considerations in a systematic way.

The two divisions of this RG2C system, "geomechanical" and "thermal" characterization and classification, are discussed in more detail in the following paragraphs.

4.7.1 Geomechanical Characterization and Classification

Each of the geotechnical analysis/design tasks involved in soil/pipe interaction and terrain stability will require as input parameters related to geotechnical-mechanical behaviors, i.e., soil strength and compressibility parameters. Each geotechnical analysis/design task has its Geomechanical Characterization and Classification (GCC) which is used for segment-by-segment analysis and design. Geometric "typicals" are used to relate GCC parameters for analysis and design. Task-specific GCC's are being developed concurrently with each of the geotechnical analysis/design tasks.

GCC's are developed from available route geotechnical data. Engineering properties are characterized using both actual testing and correlation. Correlations are based on limited testing of representative samples obtained from centerline drilling programs. Results are then generalized to relate required engineering properties to the soil index or other properties at those locations of the alignment not actually tested for engineering properties. Correlations are developed using statistical and geotechnical concepts, including pertinent geotechnical correlations available in the literature. Soil index properties are generalized exclusively from laboratory testing of representative samples.

4.7.2 Thermal Characterization and Classification

Thermal Characterization and Classification (TCC) relates to appropriate thermal parameters comprising the RG2C data set, such as thermal conductivity, heat capacity, and climatic factors.

Geometric "typicals" are used to relate thermal parameters in geometrical configurations for analysis and design. Task-specific thermal data sets are being developed as each of the geotechnical analysis/design tasks evolve.

Thermal analysis input parameter values are determined by (1) observations or measurements, (2) calculations based on other measurements, or (3) correlations based on semiempirical formulas, literature information, or engineering judgment. Work to date has progressed so that those parameters critical to the overall thermal balance (excluding groundwater movement) can be confidently assigned to meet design requirements.

The pipeline alignment crosses two major climatic zones: the Arctic Zone, north of the Continental Divide (ridgeline of the Brooks Range), and the Subarctic Zone, south of the Continental Divide. To provide a more refined and accurate picture of the climate along the pipeline, the alignment is divided into eight segments based on latitude, altitude and topography.

<u>Arctic Zone</u>

- Arctic Coast - North of 70°00'N latitude. The 70°00'N parallel at approximately 100 feet above mean sea level effectively defines the southern boundary of the strongest coastal winds.

- Arctic Foothills - 70°00'N latitude - 69°00'N latitude. The 69°00'N parallel at approximately 1000 feet above mean sea level defines the beginning of major elevational effect and the local influence of the Brooks Range. Shortage of data, however, has forced this segment to be considered with the Arctic Coast at least for the time being.

- Brooks Range North - 69°00'N latitude - 68°00'N latitude. The 68°00'N parallel closely coincides with the Continental Divide, the ridge of the Brooks Range, and marks the boundary between the Arctic and Subarctic Zones.

<u>Subarctic Zone</u>

- Brooks Range South - 68°00'N - 67°00'N. The 67°00'N parallel is at approximately 1000 feet above mean sea level and marks the end of the influence of the high elevation of the Brooks Range.

- o Interior North - 67°00'N - 66°00'N. The 66°00'N parallel approximates the Yukon River and defines the change from the southern foothills of the Brooks Range to the rolling uplands of Central Alaska.

- o Interior Central and Interior South - 66°00'N - 64°00'N. These zones are similar and thus far have been treated as one climatic segment. There are indications that Interior South may be influenced by the flow of marine air through the Alaska range. This will be investigated and refinements will be made as necessary.

- o Delta South - 64°00'N to Canadian border at approximately 62°30'N. This segment is climatically influenced by the massive Alaska range to the south and by increasing elevation toward the Canadian border.

Criteria used to select and analyze climatological data were based on the requirement to provide necessary input data to thermal analysis techniques used on the project, including the EPR thermal computer program. The climatological input data, used in the production of subsurface temperature profiles along the alignment is included on Table 4-1.

Soil thermal properties are characterized using field information, laboratory measurements of soil index properties, and Kersten's empirical correlations.[4] Soils with low dry density, very high moisture contents, or moderately high organic contents require additional interpretation.

The data base for Kersten's correlations represents four major soil groups: high quartz content sands/gravels, sands predominantly derived from basic igneous sources, fine grained soils of mixed mineralogical composition, and peat. In addition, thermal properties for bedrock and massive ice can be confidently assigned. Figure Z-9.1-4-6 is a preliminary representation of the potential variability in thermal properties for representative alignment conditions.

Soil thermal properties along the alignment are characterized on a segment-by-segment basis using the landform concept. Landform occurrence along the alignment has been mapped; statistical characterization of landform variability is being performed. Soil index properties for each landform will be used to estimate that landform's thermal properties.

[4] Reference #29

4.8 GEOTECHNICAL DESIGN PRODUCTS

Geotechnical design products deal with four general design concerns: (1) soil/pipe interactions, (2) terrain stability (liquefaction, thaw plug, and slope), (3) various geotechnical properties, used as input data by civil and pipeline designers, (e.g., thaw strain values and frost heave potentials), and (4) geotechnical thermal analyses for prediction of subsurface temperature regimes as input to further geotechnical analysis. For each of these design concerns, the general product presentation format is a mile-by-mile tabulation of design recommendations covering the entire alignment. These recommendations are then used as geotechnical design input to pipeline and civil design.

4.8.1 Soil/Pipe Interaction

Refer to Section 1.0 under stress analysis for a discussion of soil/pipe interaction analysis.

4.8.2 Terrain Stability

Terrain stability deals with geotechnical assessment of liquefaction, slope stability, and thaw plug stability along the alignment. Each stability form has its own specific assessment criteria, methodologies, and level of acceptable risk of potential instability. Unacceptable risks will require mitigative design application.

Geotechnical assessment of route terrain stability is done in two stages: (1) initial identification of potentially unstable segments along the route using generalized analysis techniques, followed by (2) site-specific detailed engineering analysis of potentially unstable areas. Mitigative design applications are specified if necessary to ensure required stability. The geotechnical assessment process considers the influence of present and expected future field conditions on stability, including effects of TAPS proximity and chilled pipe frost bulb growth.

4.8.2.1 Liquefaction of Level Ground - The tendency to densify and develop excess pore pressures during earthquake-generated ground motions can lead to liquefaction of saturated ground. When the ground is subjected to complete liquefaction, it behaves like a heavy fluid. But even if the ground does not lose its strength completely, the soil is subjected to shear deformations due to increased pore pressures and as these pore pressures dissipate the ground is subject to compaction settlements. In the arctic and subarctic regions, the possibility of liquefaction exists in the active layer in permafrost areas, saturated unfrozen layers below the frozen active layer (taliks) in the permafrost, as well as in unfrozen gravel, sand, or silty soils. Analysis of the liquefaction potential of susceptible ground and application of mitigative measures allow structures to be designed for any required standards of safety.

Two primary steps are involved in the analysis of liquefaction potential. These steps are the evaluation of the stresses generated at various points in the ground during the ground shaking caused by the design earthquake, and comparison of these with the stresses which will lead to the generation of liquefaction at those points. The first depends on the seismic status of the region and the wave propagation and energy absorption characteristics of the ground. The second is a function of the physical and engineering properties of the concerned soils including stress history.

Three conditions which require evaluation of liquefaction potential are where the pipe is buried in (1) a thawed saturated layer, (2) a frozen layer above a thawed layer (talik), and (3) close proximity to TAPS pipeline.

Mitigative measures of liquefaction are based on the understanding of various parameters affecting liquefaction as well as environmental, construction and cost constraints.

<u>Effect on Pipe and Environment</u>

In a thawed saturated layer, the gradual increase in pore pressures during an earthquake will correspondingly decrease the soil resistance. This could lead to deformation of the buried pipe relative to the ground due to inertial effects. The extreme case here could be the complete loss of strength of the surrounding soil which will lead to either settlement or floating up of the pipeline. However, the most probable mode of pipe behavior is that the pipeline will undergo the same deformation as the surrounding soils during the earthquake. For analyzing this, it will be necessary to compute the shear strain potential in the soil during the earthquake (with the implicit assumption that the presence of the pipeline does not affect the stresses within the earth).

Another related concern is the vertical deformation of the pipe due to the compaction settlements of underlying soils. In general, the settlement aspect will be considered when the ground is in an unsaturated state.

In a frozen active layer located above an unfrozen layer (talik), the generation of pore pressures in the unfrozen layer will tend to cause the rupture of the impermeable frozen layer above. The consequences of such a rupture on the buried pipeline is a highly indeterminate problem. Therefore, it is desirable to avoid the problem of initial liquefaction, i.e., the pore pressures equalling the over-burden pressure within the talik.

Where the NWA pipeline is located in close proximity to the TAPS pipeline, the thaw plugs under the TAPS constructions as well as the frost bulb which will be created around the chilled pipeline can affect groundwater conditions and hence the liquefaction potential of the terrain. Special attention is being directed to these factors.

Design is performed by ensuring that unacceptable liquefaction problems are eliminated by reroute or mitigative designs. This is to be accomplished by avoiding initial liquefaction in taliks, keeping the shear strains and the settlement potentials within permissible limits in the thawed layer and compaction settlements within permissible limits in unsaturated soils.

Design Applications

The alignment passes through areas of widely varying characteristics which influence both the earthquake induced stresses and the stresses required to cause liquefaction. The design approach being used takes this into consideration. The objectives being pursued are to:

- Characterize design earthquake parameters along the pipe route.
- Characterize the route soils with respect to need of detailed analysis for liquefaction.
- Determine the liquefaction potential of those soils in terms of initial liquefaction, cyclic shear strain potential and settlement potential.

- o Compare the above with the acceptable limits as per criteria for design.

- o Evaluate various types of mitigative measures for unacceptable segments to meet the design criteria.

- o Set simple criteria and procedures for field verification and field design change of the solutions.

The earthquake parameters used for analyzing the liquefaction potential are the maximum values of acceleration, velocity and displacement, and the magnitude (or the frequency content and the duration of significant shaking). The above have been arrived at on the basis of a detailed study. The route has been divided into three zones where the maximum probable magnitudes are identified. The other parameters are made to relate to these values. Professor N. M. Newmark is providing these parameters. [5]

The three principal aspects to be considered are the initial liquefaction, cyclic mobility, or shear strain potential, and compaction settlements. Evaluation of the liquefaction potential of deposits of various descriptions shall be done in terms of the magnitude of pore pressures developed, induced strains, and compaction settlements. This requires three separate but related sets of analysis for each location being analyzed. [6]

At each location, the following geotechnical factors which affect the liquefaction potential of the deposit are considered.

- o Soil profile - bedrock, permanently frozen soils, cohesive soils and very dense cohesionless soils do not pose any problem. The thawing active layer, taliks and unfrozen cohesionless soils are being analyzed.

  Special consideration is being given to the possible changes in the water regime due to TAPS as well as the proposed constructions.

- o Stress state prior to the occurrence of the design earthquake as well as stress history - Since the stress ratio during consolidation, over-consolidation ratio, and the strain history have been found to influence the process of development of pore pressures

[5] Reference 10
[6] Reference 11, 12, 13, 14

during cyclic loading, the initial stress state due to the ground profile, initial static excess pore pressure caused by thaw consolidation are being considered. [7]

- <u>Soil Properties</u> - The physical properties (grain sizes and their distribution, grain shapes and the packing characteristics shown by the densities or relative densities) are considered.

- <u>Soil-Structure Interaction</u> - The presence of the pipeline, the operation conditions and the disturbance that may be caused by construction may influence the stresses induced during the earthquake as well as the liquefaction potential of the ground.

<u>Mitigation Solutions</u>

The solutions fall into two categories, namely avoiding the hazardous area by rerouting or countering the hazard by suitable engineering construction. The latter includes:

- Burial below hazardous zones
- Replacement of hazardous zone with stable material
- Provision of drainage paths
- Compaction prior to construction
- Grouting of hazardous zone
- Prefreezing unfrozen soil

The measures listed above require analysis and integration with other project conditions and constraints, including frost heave effects, thaw plug stability, APSC proximity and other adjacent structures.

It is also necessary to confirm during actual construction the conditions assumed in analysis and design. As such, field verification and redesign for changed conditions are being developed concurrently with segment-by-segment analysis and design.

<u>Proposed Design Procedure</u>

Step 1: o Classify the deposits as per soil type

[7] Reference 15, 16

- o Classify the deposits as per land form and thermal conditions

- o Classify the deposits as per seismic zones

Step 2: From the soil exploration data, classify the route soils according to the density/SPT values for cohesionless soils and cohesion/PI values for cohesive soils.

Step 3: Establish critical values of density/SPT values considering a) soil type, b) deposit characteristics and c) seismic zone.

Establish critical values of cohesion/PI considering a) soil type, b) correlation between cohesion values and PI, c) densities and d) seismic zone based on the concept that the cohesion alone must be able to prevent failure of pipe buried at different depths. If observed values are larger than critical values, detailed analysis is not needed.

Step 4: Detailed Liquefaction Analysis - The procedure to be used will utilize direct comparison of the stress ratios developed at different depths during the design earthquake with those which have been seen to develop liquefaction in soils of given SPT values during an earthquake of given magnitude.

Suitable modifications and developments have been carried out using the available data in the literature to obtain design curves which can be used for obtaining the liquefaction potential in terms of a given amount of cyclic shear strain. The grain size characteristics are considered in the use of the curves.

Table 4-3 summarizes the basic philosophy and the outline of the procedure for assessing the Liquefaction Potential of the route soils. The details of the application procedure and the actual correlations used considering the different design parameters have been developed as backup data.

4.8.2.2 Slope Stability - Slope stability is concerned with disturbance of the existing equilibrium of slopes along the alignment due to natural events, including earthquakes, and construction impacts. The risk of slope instability must be below an acceptable level to provide pipeline integrity and environmental protection.

Criteria used to assess slope stability addresses both static factors of safety and earthquake-induced permanent displacements. Slopes will require mitigative designs or detailed site-specific field or analytical qualification where: (1) the computed static factor of safety is less than 1.5, (2) computed permanent displacements exceed five inches under design contingency earthquake loading, or (3) cyclic shear strains, if computed, exceed 10 percent.

An existing stable slope fails under the action of static loads alone if the toe is eroded, crest is overloaded, or if excess pore pressures develop within the slope. Potential increased pore pressures due to thaw consolidation following thermal disturbances caused by the construction processes are being evaluated. Ground motion effects caused by the design earthquake are being assessed with respect to equilibrium upsets due to inertia forces as well as dynamic pore pressures. The consequences of such events are being measured in terms of slips, settlements and cracking. Long-term stability of potential problem slopes along the alignment are being evaluated by detailed analysis.

Effect on Pipe and Environment

Types of slope instability concerns include soil/residuum failures, rock failures, and flow slides due to liquefaction. The buried pipeline cannot withstand excessive displacement in the longitudinal or transverse directions without impairing its integrity. Therefore, it is necessary to limit potential ground deformations to permissible values under design forces.

Ground deformations can be caused 1) by slipping of a wedge along the surface of least resistance when the driving forces exceed the strengths, 2) due to the velocity attained by a portion of the slope as a result of earthquake induced vibrations or 3) due to the loss of strength of a portion of the slope caused by dynamic pore pressure development during earthquakes. The amount of slope displacement which can be tolerated by the pipe depends on site-specific conditions.

Design Application

- Identification of potentially unstable slopes along the route on the basis of available literature, reconnaissance surveys, personal experiences, engineering judgment based on empirical rules and detailed geotechnical analysis. The modes being identified include soil/residuum failures, rock failures, and flow slides.

- Analysis of the effects of all factors which affect the equilibrium of the slopes. This includes the computation of increased pore pressures due to thaw consolidation from thermal disturbance, increased pore pressures due to earthquake loading (for design earthquake parameters), and pipeline construction effects. Assessment of gas pipeline effects include consideration of potential changes in groundwater flow/drainage characteristics due to the development of a frost bulb and other thermal disturbances. Mechanical effects due to soil removal or surcharge loads are likely to be limited to the construction period, and would be taken care of while analyzing the ditch stability.

- Determination of the factors of safety against failure and the magnitude of displacements. Comparison of these against stability criteria to identify hazardous slopes.

- Specification of mitigative measures for the hazardous slopes.

- Establishment of the field verification and field design change procedure for each slope.

Design considerations include the following: The mode of failure is dependent on the specific slope conditions and the potential disturbing forces. For rock failures, a block failure mechanism considering the geology of the site, jointing, cleavages, and bedding planes would be considered to compute the factor of safety against slipping of the block.

For soil/residuum failures the design consideration include the long-term stability of the slopes, the effects of thermal-induced thaw consolidation on stability, effects of the disturbance of the equilibrium brought about by construction, earthquake-induced ground motion, and weathering. Of the above, all the factors except the earthquake loading are evaluated in terms of suitable values of factors of safety against flows, slides and falls. The analysis accounts for soil properties, soil profile, thermal conditions and groundwater conditions. For earthquake loading both factors of safety from pseudostatic analyses and potential deformation limits are evaluated. Deformation analysis is especially important where deterioration of the strength takes place during the earthquake loading. Total stress analysis for computation of strains and subsequent integration to arrive at the deformations and effective stress analysis assuming slips along well defined failure surfaces is being considered. Selection of the best method is evaluated for individual sites.

The various types of mitigation solutions which are being used include:

- Rerouting
- Removal and replacement of hazardous reaches
- Deep burial of pipes
- Construction of buttresses, deflection barriers
- Rock bolting
- Control of groundwater
- Regrading of slopes
- Grouting, reinforced earth and other special methods

After identifying the hazardous slopes, the most appropriate form of mitigation solution is arrived at on the basis of its efficacy and sufficiency as indicated by analysis and integration with other factors including APSC and environmental concerns.

Most hazardous slopes will have been identified during design on the basis of reconnaissance, APSC reports, and detailed analysis of slopes found to be potentially unstable. However, due to the widely varying nature of the soils, it is further necessary to ensure against changed circumstances. Field verification and redesign procedures are being developed concurrently with segment-by-segment analysis and design. These allow field personnel to interpret changed conditions found during construction and apply redesigns where necessary.

<u>Proposed Design Procedure</u>

Step 1: Mile-by-mile survey to identify problem slopes utilizing the alignment sheets, published and other available reports, reconnaissance survey reports and field inspection of the route.

Step 2: Idealization of problem slopes as two-dimensional cases of average slopes and heights and soil conditions and water flow conditions with proper consideration of the construction and operation of both TAPS and the proposed pipelines.

Step 3: Static slope stability analysis using infinite slope models, simplified Bishop's method and Spencer's method. [8]

[8] References 27 and 28

Step 4: Computation of principal stresses along the failure surface using Seed's [9] procedure for the critical wedge.

Step 5: Computation of cyclic pore pressures considering the design earthquake. [10]

Step 6: Dynamic stability analysis for determining the yield acceleration considering the inertia forces and cyclic pore pressures.

Step 7: Computation of displacements using Newmark's Concept. [11]

Steps 5 to 7 may be replaced by a total stress analysis for computing strains, if appropriate.

Step 8: Perform a pseudostatic analysis of the slope with design acceleration coefficient.

Step 9: From the results of above, estimate the ratio of maximum dynamic shear stress to the minor principal stress during consolidation.

Step 10: From appropriate curves developed, between the strains (under anisotropic consolidation and for given number of cycles) and the ratio of shear stresses and the minor principal stresses during consolidation, estimate the strains.

Table 4-4 is a brief outline of the design procedure. Details are in backup material.

4.8.2.3 Thaw Plug Stability - A thaw plug can develop in or adjacent to the alignment right-of-way due to thermal disturbance from construction activities, workpad maintenance during system operation, or TAPS pipeline influence. It is expected that any developing thaw plugs will not be capable of adversely affecting pipeline integrity once a sufficiently large frost bulb builds up around the chilled line. Until that time, the risk of thaw plug instability must be below an acceptable level to provide pipeline integrity. Acceptable risks of potential instability must also be sufficiently low to assure workpad integrity both during construction and thereafter for use in system maintenance and monitoring operations.

[9] Reference 26
[10] References 15 and 16
[11] Reference 25

Criteria used to assess thaw plug stability addresses both static factors of safety and potential slope displacements during earthquakes. Thaw plugs affecting pipeline integrity will require mitigation or detailed site-specific field or analytical qualification where: (1) the computed static factor of safety is less than 1.5, or (2) computed permanent displacements exceed five inches under design contingency earthquake loading. Thaw plugs not affecting pipeline integrity will require mitigation only if the computed static factor of safety is less than 1.1.

The stability of an existing thaw plug could be adversely affected by adjacent construction or by earthquakes. The gas pipeline is concerned with existing stability conditions of TAPS thaw plugs which may be influenced by the proposed constuction activities. Failure of existing thaw plugs could lead to undesirable consequences to existing and future structures.

A limit equilibrium approach utilizing a two-dimensional infinite slope model is being used for all cases of loading. Where necessary, thaw consolidation pore pressures are taken into account. Excess pore pressures due to earthquake loading are also being considered as necessary. Where site conditions require, simple three-dimensional models are being developed for stability analysis.

Mitigation measures being used include reroute, deep bury, refreeze or buttress of slope toes. These measures are being integrated with all the project and environmental constraints.

Field verification and redesign procedures are being developed concurrently with segment-by-segment analysis and design. This allows field personnel to interpret changed conditions found during construction and apply redesigns where necessary.

The criteria for safety of the constructions against failure of thaw plugs is set in terms of factors of safety. The current design procedures require properly conservative values of factors of safety.

Step 1: Identification of locations of potentially unstable thaw plugs considering a) steep slopes, b) construction activities, c) ice-rich slopes.

Step 2: Identification of the geometry of the thaw plug which will be site dependent where appropriate and generalized (like an infinitely long prism) elsewhere.

Step 3: Computation of the factor of safety of the plug considering the driving forces and resisting forces in the direction of the slope; thaw consolidation pore pressures will be considered.

Step 4: Considering an inertia force to be acting in the direction of slip, compute the yield acceleration value and compute the displacements from the formula

$$d = \frac{V^2}{2gN} \cdot X$$

where d = Displacement

V = Peak velocity during the design earthquake.

N = Yield acceleration coefficient for the thaw plug

A = Acceleration coefficient such that $A \times g$ is the peak acceleration during the design earthquake

$$X = 6.0 \text{ if } \frac{N}{A} < 0.15$$

$$= (1 - \frac{N}{A}) \frac{A}{N} \text{ if } \frac{N}{A} > 0.15$$

4.8.2.4 Thaw Strain - Thaw settlement predictions will be utilized in the design of the workpad, access roads, permafrost cut slopes, the evaluation of thermal erosion and the development of methods for erosion control. Thaw strain potential is a primary factor used in the analysis or prediction of thaw settlement. It is a measure of the consolidation which occurs when frozen soils are thawed and the excess water drains out. The values of thaw strain can vary greatly with physiographic province, landform, soil type and frozen moisture content. The soil properties found in a landform in one physiographic province may not correspond to the soil properties found in a similar landform of a different physiographic province. The soil types found within a particular landform can vary and also frozen moisture content will vary within any soil type. Since an analysis of each combination of physiographic province, landform, soil type and frozen moisture content would be impractical, the thaw strain potential was analysed according to the average thaw strain expected in the particular landforms found in each physiographic province. Additionally only those landforms analysed where those which are found on the preliminary route soils alignment sheets along the proposed centerline.

The initial determination of average thaw strain potentials as shown in Table 4-2 was made using three different methods which are: 1) By using existing data on thaw strain, 2) By using laboratory data from existing boreholes and 3) By comparison.

The first method used was to analyze and evaluate the thaw strain data of the different landforms as found in APSC data.[12] This document presents the thaw strain potential in terms of landforms found in each physiographic province. It also states the amount of data used in calculating the thaw strain potential in each landform. This document does not cover all landforms found along the APSC pipeline route. It also has no information on soils along the gas line route from Delta Junction to the Canadian Border.

For the estimation of average thaw strains south of Delta Junction, the thaw strain potential was calculated from laboratory data for the bulk density and dry density of samples taken from boreholes drilled south of Delta. Calculations were made separately for dry density and bulk density of numerous soil samples classified according to landform.

For those landforms which had little or no data available from either the APSC or from borehole data, the thaw strain potential was estimated by comparing the engineering properties of that landform with those of a similar landform. From this a best estimate thaw strain potential was chosen. Where more than one landform was noted for a given area the more conservative thaw strain value was used.

Values of thaw strain potential are preliminary best estimates. Because averages were estimated, large deviations from these values are expected at specific locations. They are, however, representative of actual conditions on the average. These values will be refined in the design phase as additional soil exploration data and APSC thaw strain prediction information becomes available.

An estimate has been made of the thickness of the organic layer overlying mineral soil along the alignment of the

[12] Reference # 30

pipeline. The thickness of compressible organic material is one factor used in the estimation of the required thickness of workpad material.

4.8.2.5 Ditch Degradation Potentials - Geotechnical input to the pipeline design ditch stability assessment consists of Ditch Degradation Potentials (DDPs). These are determined on the basis of soil profile and groundwater conditions, frozen soil thaw degradation characteristics, and construction season timing.

Factors influencing ditch stability include thaw strain, ditch depth, thermal state, frozen zone location and thickness, thaw consolidation properties of ditch soils, soil strength properties, ground and surface water presence in the ditch, topography, length of time for construction, and other seasonal constraints.

Landform characterization will establish correlations with the soil strength parameters and, together with geotechnical thermal analysis, the potential for excess pore pressures. Landforms will be correlated with various subsurface soil properties and will provide a general basis for evaluating soil criteria with an influence on ditch wall stability. The primary parameters to be utilized are landform and percent thaw strain potential categories. The thaw strain potential indicates the potential susceptibility of a soil to thaw degradation. Soils with a high thaw strain potential have significant amounts of excess ice.

Different Ditch Degradation Potentials (DDP) will be assigned those areas with expected thaw strain potentials of 10 percent or less, greater than 10 percent but less than or equal to 20 percent, and greater than 20 percent.

Three DDP's will be designated -- low, moderate, and high, respectively.

Completely frozen ditch sections will have a higher DDP than completely unfrozen sections. Frozen over unfrozen ditch sections will have a lower DDP than unfrozen over frozen states because of a generally lower potential for excess pore pressures.

The presence of a groundwater table in unfrozen soils over frozen soils will have a significant impact on the DDP. This condition will indicate a high DDP regardless of the thaw strain value of the frozen material. Less common situations with significant groundwater in Taliks or in an unfrozen layer beneath a frozen layer will similarly have a high DDP.

Terrain with a cross-slope will have a higher potential for ditch wall instability than level terrain, particularly on the uphill side where ditch spoil will be placed. A cross-slope of greater than 20 percent may result in a one-step higher categorization of DDP. Topography also controls surface water flow, whose characteristics will also require a subjective evaluation. The proximity of ponds or cross-drainages may indicate a higher potential for thaw degradation.

Ditching near existing thaw bulbs (e.g., due to TAPS proximity) will increase DDP. This can be particularly significant if ditching occurs downslope of existing TAPS workpad or on cross-slopes.

Stability conditions will decrease with the duration of time the ditch remains open during periods of thawing ambient air temperature. Expected open ditch timing during periods of expected thawing temperatures will be considered for those areas exhibiting a high or moderate DDP.

Seasonal constraints that may be considered will be that ditching when the ambient temperature is below freezing will prevent thaw degradation. Ditching in the "shoulder months" will mitigate DDP. Ditching in the summer months will result in the maximum DDP for any given segment.

On the basis of above analyses, mile-by-mile design charts, can be prepared as shown conceptually below.

SEGMENT. (   )

DITCH DEGRADATION POTENTIAL
L Low; M Medium; H High

Days for which the ditch will be open													
	4N–3N	L	L	M	H	H	H	H	H	H	H	M	L
	3N–2N	L	L	M	M	H	H	H	H	H	M	M	L
	2N–N	L	L	L	L	M	M	H	M	M	L	L	L
		J	F	M	A	M	J	J	A	S	O	N	D

Month of Construction

1. Segment-by-segment evaluation of the Ditch Degradation Potential in terms of:

a) Landform

b) Thermal conditions (soil and meteorological)

c) Water Table

d) Thaw strain characteristics

e) Ditch depth

f) Cross slopes

2. Assessing the DDP for each segment based on average conditions of temperature and open ditch conditions.

3. Modifying the DDP's to account for the month of construction as well as days for which the ditch may remain open.

4. Providing field procedures for changed conditions such as changes in soil conditions or unforeseen weather conditions.

4.8.3 Geotechnical Thermal Analysis

Many aspects of geotechnical engineering analysis and design require knowledge of the ground thermal regime because of the extreme temperature dependence of soil behavior at and below freezing (32°F). Thus, assessment of geotechnical stability requires both thermal and structural/mechanical analysis. Because thermal analysis is analytically distinct from structural/mechanical analysis in geotechnical engineering, the two have been separated here for convenience. However, in all cases these two distinct analyses are coordinated. Temperature dependent geotechnical properties used in analysis tasks are kept consistent with the soil thermal regime.

The general procedure for segment-by-segment analysis is to develop thermal regime "typicals" for use in analysis and design. Site-specific concerns utilize appropriate site-specific geometry and thermal input for analysis.

The methods of thermal analysis vary in complexity. Simple analyses of freeze/thaw depth that use the various modifications of Stefan's solution often suffice where a one-dimensional approximation is appropriate. For example, in estimating gravel pad thickness required to keep thaw settlement within a given criterion, analysis employed an equation combining the modified Berggren equation with Aldrich's equation for heat flow in a multi-layered medium.

Another useful set of closed form equations relates to thermal environment near a buried chilled pipe. This set can be used to estimate the size of the frost bulb, temperature distributions, or heat fluxes at interfaces. These equations are useful in parametric analysis related to such design applications as sizing of test site cooling systems, estimation of insulation requirements, and the examination of mitigative techniques related to maximum frost bulb penetration.

SIMPLIFIED GEOTECHNICAL ENGINEERING FLOW DIAGRAM

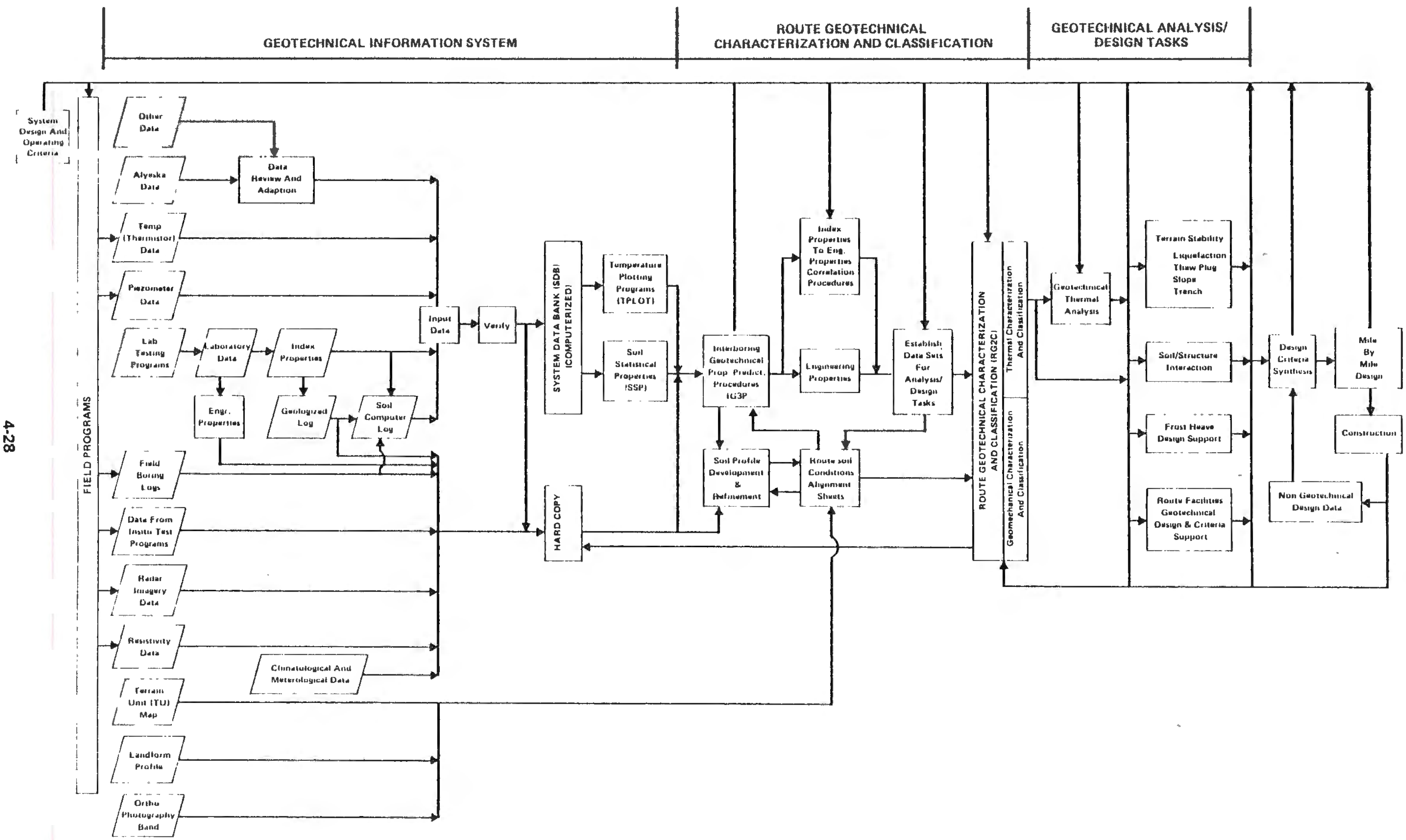


FIGURE Z - 9.1-4-1

MAY, 1980

PIPELINE ROUTE
GEOTECHNICAL ASSESSMENT PROCESS

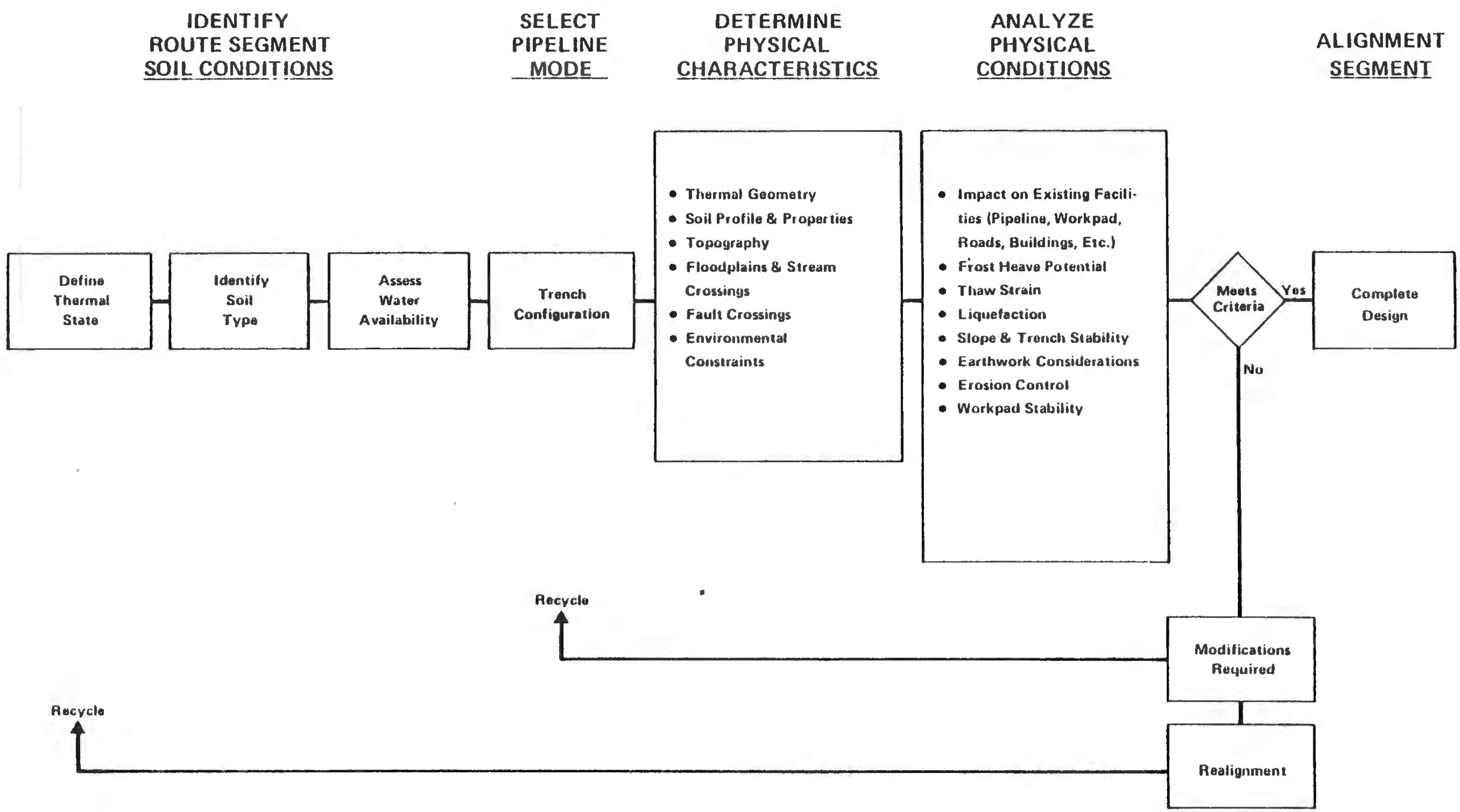


FIGURE Z - 9.1-4-2

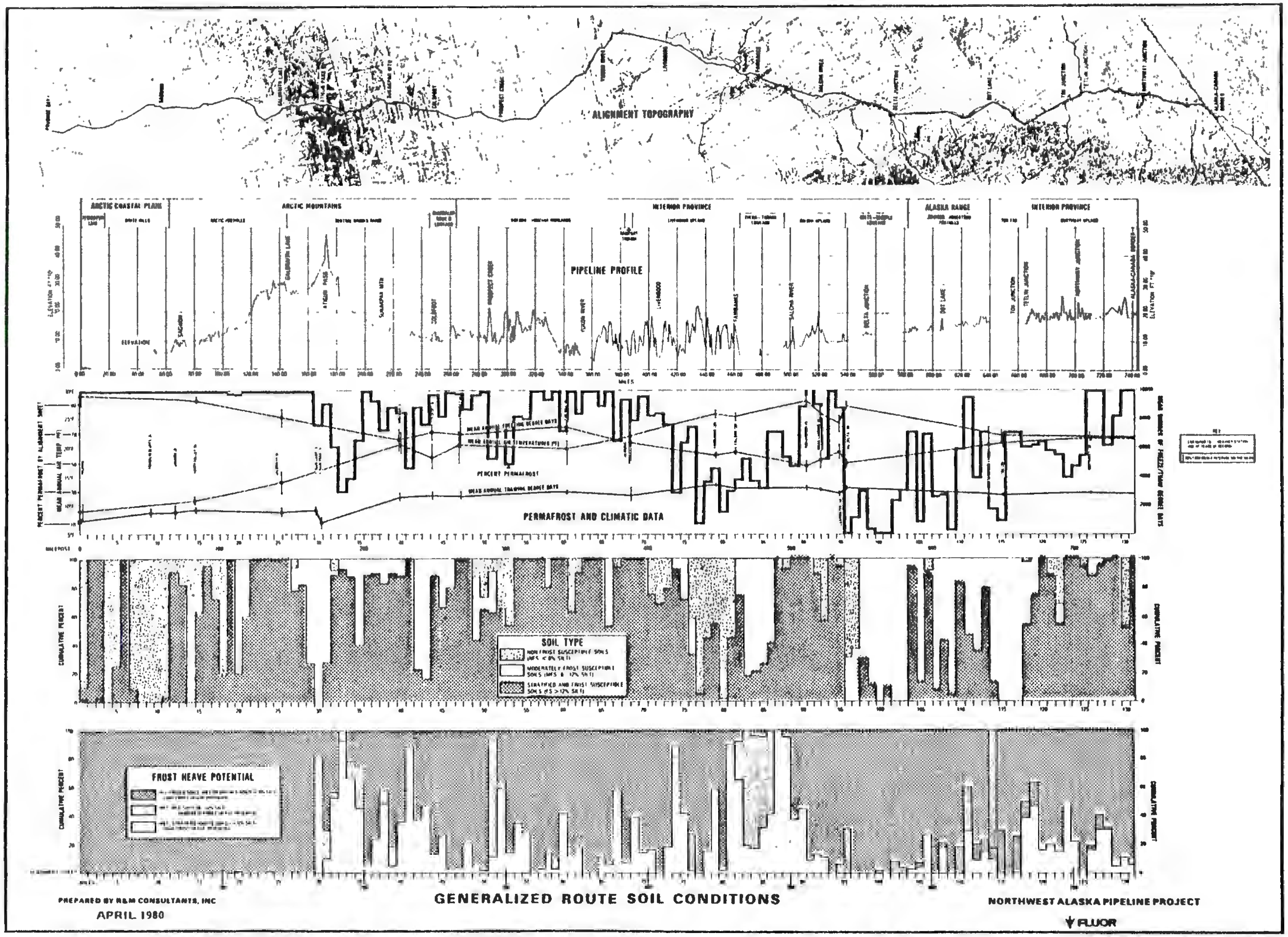


FIGURE Z-9.1-4-3

UNIFIED SOIL CLASSIFICATION SYSTEM (MODIFIED)

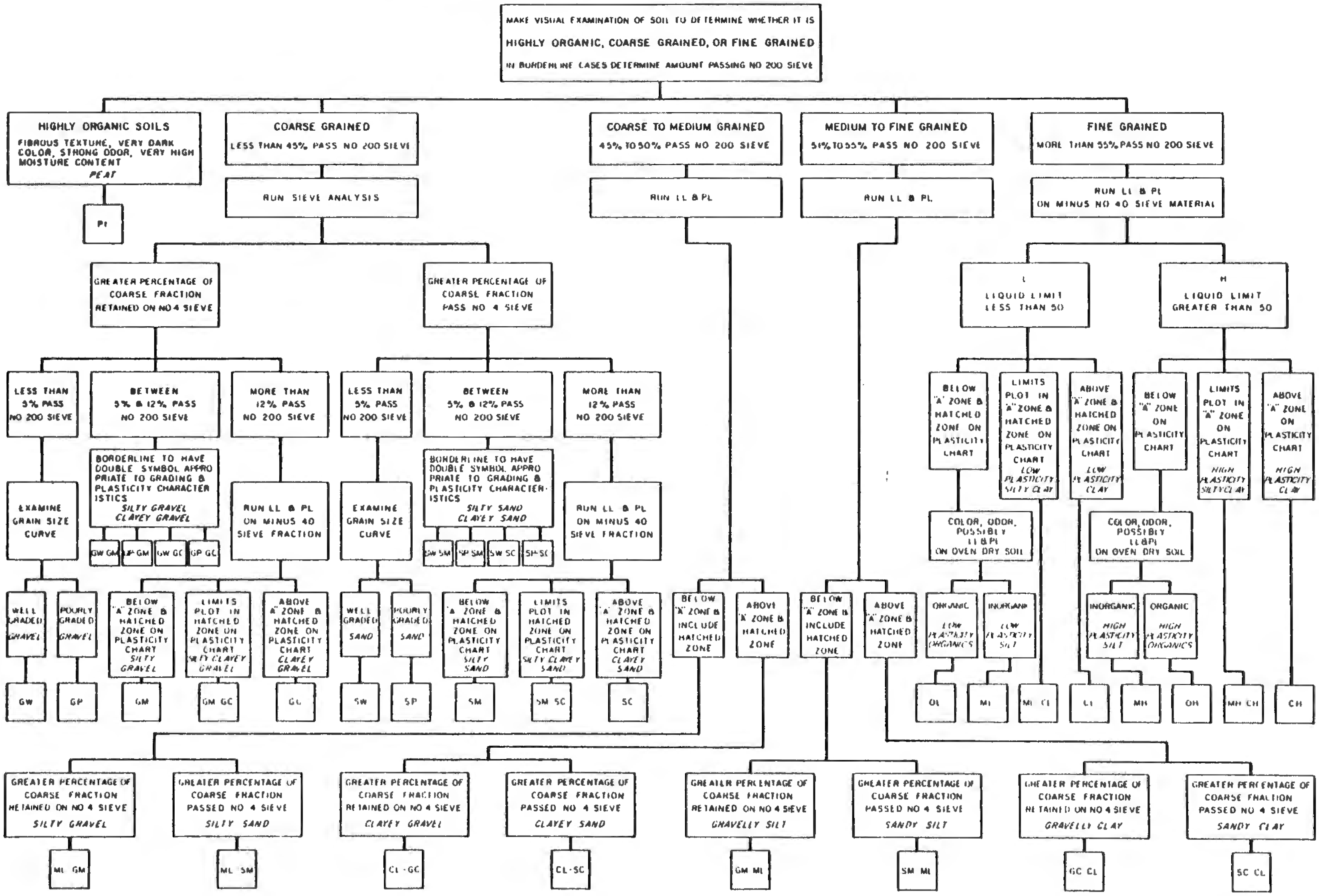


FIGURE Z - 9.1-4-4

PLASTICITY CHART (MODIFIED)

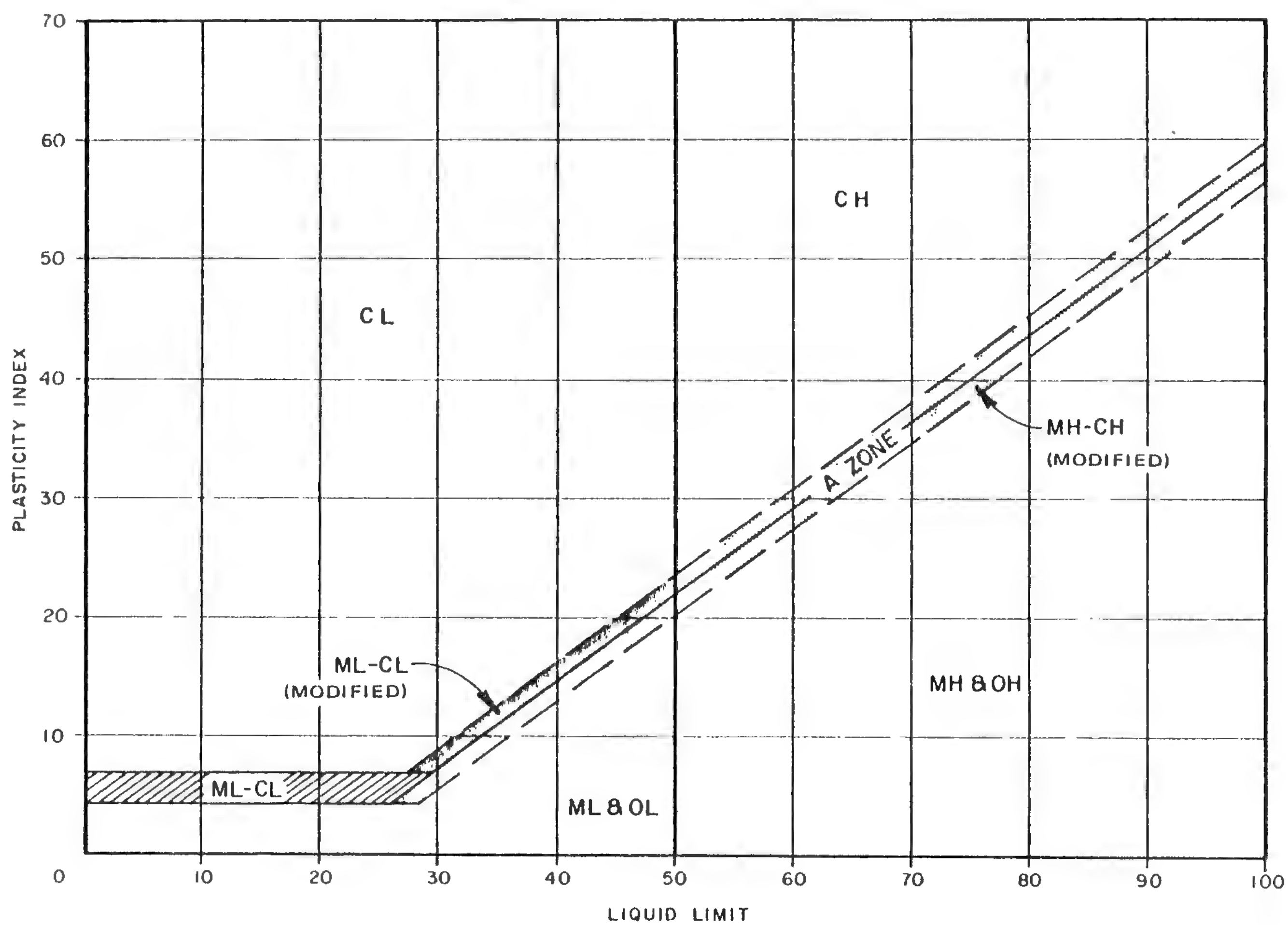


FIGURE Z - 9.1-4-5

MAY, 1980

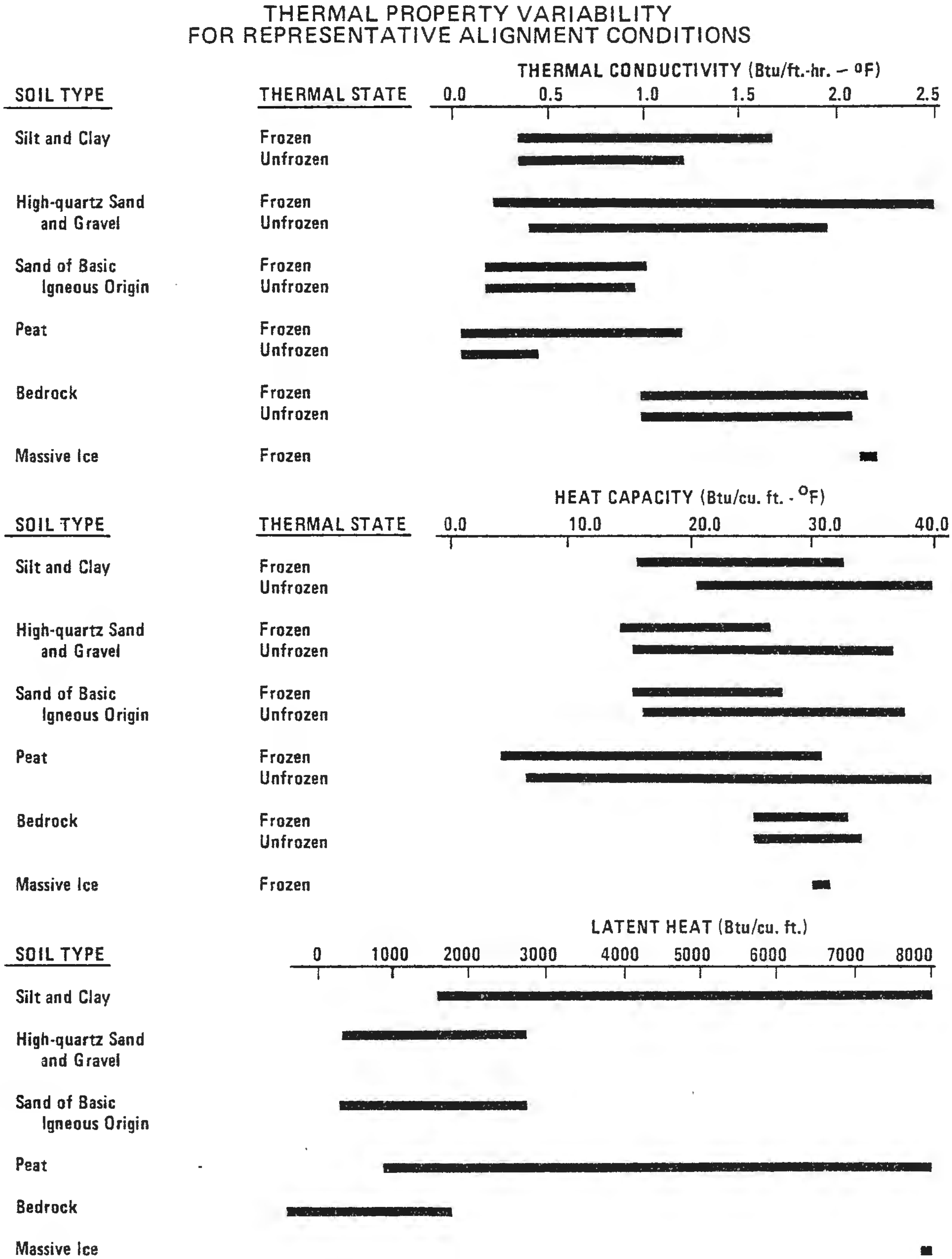


FIGURE Z - 9.1-4-6

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TABLE 4.1

CHARACTERIZATION OF THE CLIMATE ALONG THE ALIGNMENT AREA-BY-AREA

	Annual Mean Daily Temp (°F)	Monthly Mean Daily Temp (°F) Warmest Month	Monthly Mean Daily Temp (°F) Coldest Month	Total Annual Precipitation (ins of H_2O)	Total Annual Rainfall (ins)	Total Annual Snowfall (ft)	Mean Snow Density (gm/cm^3)
Arctic Slope Foothills	9.8	45.5	-27.4	4.4	1.9	7.1	0.32
Brooks Range N	8.2	63.0	-38.0	7.7	2.5	3.2	0.30
Brooks Range S	12.1	67.9	-33.2	13.4	5.7	7.8	0.29
Interior North	21.1	71.5	-24.6	13.4	7.5	6.4	0.25
Interior Central Interior South	26.2	72.4	-19.9	11.2	7.0	5.7	0.25
Delta South	25.8	72.1	-19.6	12.4	8.4	3.5	0.31

	Mean Annual Wind Speed (mph)	Mean Annual Cloud Cover (%)	Mean Daily Solar Radiation (BTU-ft/day)	Freezing Degree Days (°F)	Thawing Degree Days (°F)
Arctic Slope Foothills	11.8	72	1220 1260	8520	470
Brooks Range N	6.3	50	1280	9410	770
Brooks Range S	6.7	66	1310	8490	1980
Interior North	6.7	66	1340	6720	2780
Interior Central Interior South	5.4	70	1370 1390	5140	3170
Delta South	5.4	72	1420	5180	2940

TABLE 4.2
ESTIMATED AVERAGE THAW STRAIN FOR FROZEN SOILS IN LANDFORMS
OCCURRING FROM A/S 1 THROUGH 131

Average Thaw Strain (%)	5%	10%		20%		25%	30%	40%	50%
Landform	Bx(25-41)	Bx(42-88)	Ns-r	C	Gt	C+G	Cs	Fpa-c(99-131)	Fps-c*(99-131)
	Bx-w	Bx-r	S	C or F	Gt?	Ca+Ct	Cx(42-66)	Ns	Fps-c?
	F	Es	S-r?	C+F	Gt or Cl?	Ca+Fpb-c	E?		O
	F?	Es?	Sc-r?	Cx(1-24)	Gt or El?	Cl	El		
	Fpb-r	Es+Fsa		F+L(99-131)	Gt or F?	Cm	El?		
	Fp-r	F+L(25-41)		Ff	Gt+C	Cs-Cm	El+Fp-c		
	Fp-r?	Ffg		Ff?	Gto	Cs+Ft	El+Fs		
	G	Ffg+Fp-r		Ff or Gt?	Gtx	Elu	Ell		
	G+F	Ffg+G		Ff+Fp	Gty	Elx	Ell+Fp-c		
	G+GF	FG		Ffg+Ca	L(25-41)	Elx+Fs	Ell+Fp-c?		
	GF	Fp		Fp-c(89-131)		Ff+Cm	Ell+Lt		
	Hc	Fpa-r		Fp-c?		Ff+Fs(25-41)	Ff+Fs(99-131)		
	Hf	Fpm-r(67-88)		Fp-c+E		Fp+Ft	Ffs+Fs		
	Hf?	Fps-r		Fp-c+Fpa-c		Fp-c(1-88)	Fp+Fpa		
	Ht	Fpt		Fp-c+Fpa-c?		Fp-c+Fs	Fpa-c(1-24,42-88)		
	I-w	GF+L		Fpb-c		Fp-p	Fps-c(24-41)		
	Ib-u	GFo		Fpc-c		Fpm-c(67-88)	Fs(89-131)		
	Ib-w	I-r		Fpm-c(42-66)		Fpm-p	Fs?(99-131)		
	Ig-w	Ib		Fpm-r(42-66)		Fs(1-88)	Fs+Fp-c(89-98)		
	Ig-w?	Ib-r		Fps		Fs?(1-24)	L?		
	Mc	Ig		Fs+C		Fs+Cm	Lt		
	N(25-66)	Ig?		Fs+Ffg		Fs+Ell			
	N-w	Ig+n		Fsa		Fs+Elu			
	Ng-w	Ig-r		Fsa?		Fs+Elx			
	Nl-w	N(67-88,99-131)		Fsa+Es		Fs+F			
	Ns-w	N?		Fsa+Fp-c		Fs+Ff			
	Sc	N-r		Fsf		Fs+Fp-c(42-88)			
	Sc-w	N-r?		Fss		Fs+Fpm-c			
	Sh-w	Ng-r		Fss+Elx		Ft+Cm			
						L(42-66)			

Note: ( ) indicated thaw strain estimate applies only to alignment sheets indicated.

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TABLE 4-3
SEISMIC LIQUEFACTION* DESIGN PROCESS

I. CHARACTERIZE EARTHQUAKE GROUND MOTION DESIGN PARAMETERS ALONG THE ALIGNMENT FOR PROJECT LIQUEFACTION ANALYSIS - BY SEISMIC ZONE

   A. Determine Design Earthquake(s)

      1. Maximum acceleration, velocity, displacement
      2. Frequency content
      3. Duration of significant shaking

   B. Characterize Design Earthquake Motions in Terms of Parameters Appropriate for Project Liquefaction Analysis

II. CHARACTERIZE ALIGNMENT IN TERMS OF CYCLIC LIQUEFACTION (DENSIFICATION) BEHAVIOR OF ROUTE SOILS

   A. Characterization Parameters

      1. Cyclic stress ratio induced during Design Earthquake
      2. Cyclic stress ratio causing "initial liquefaction"
      3. "Shear strain potential"
      4. "Settlement potential"

   B. Geotechnical Factors Potentially Influencing Characterization Parameters

      1. Geologic history, by landform

         a. Depositional environments and genesis
         b. Stress state effects due to:

            o Overburden (ice loading and unloading, erosion, uplift, subsidence, etc.)
            o Topography
            o Soil/bedrock interfaces

      2. Seismic history, by seismic zone
      3. Soil properties

         a. Grain size distributions
         b. Grain shapes and textures

*Includes earthquake-induced liquefaction and compaction settlement

TABLE 4-3 (Continued)

c. Moisture contents
d. Density

- o SPT blowcounts
- o Dry densities
- o Relative densities

4. Groundwater conditions including probable changes
5. Initial static excess pore pressures from thaw-consolidation in thermally disturbed areas

III. ESTABLISH LIQUEFACTION DESIGN MITIGATION SOLUTIONS AND APPLICATION CRITERIA

A. Develop Design Solutions to Mitigate/Eliminate Potential Hazardous Liquefaction or Instability Conditions

1. Reroute around hazardous areas
2. Mitigate directly

a. Burial below hazardous zone
b. Replacement of problem soil with compacted backfill
c. Groundwater drainage
d. Increase density of problem soil (blast, dynamic compaction, vibroflotation, compaction piles)
e. Grout
f. Regrade slopes to flatter angles
g. Buttress slopes
h. Prefreeze unfrozen soil segment

3. No mitigation

B. Integrate Liquefaction Mitigation Solutions With Other Project Conditions and Constraints

1. Frost heave

a. Effects
b. Design modes

2. APSC pipeline proximity
3. Thaw plug stability

TABLE 4-3 (Continued)

a. Short term
b. Long term

4. River crossing designs
5. Environmental
6. Construction
7. Cost
8. Contingencies

C. Establish Application Criteria for Mitigative Design Solutions Suitable for Segment-by-Segment Design, based on:

1. Design liquefaction potential
2. Occurrence of other project conditions and constraints as per III.B. above.
3. Site importance (consequence of liquefaction)

a. Compressor stations
b. Pipeline

o Cross country
o River crossings

c. Adjacent facilities
d. Ancillary structures
e. Special cases

D. Establish Criteria for Field Verification and Redesign Based on Changed Conditions

1. Basis for determining significance of changed conditions
2. Basis for field redesign and construction implementation

IV. CONDUCT DESIGN LIQUEFACTION POTENTIAL ASSESSMENT SEGMENT-BY-SEGMENT

A. Identify Those Segments Where Liquefaction or Instability is Not a Potential Hazard

1. Where the system would not be sensitive to actual liquefaction or compaction settlement caused by earthquake-induced densification

2. Geotechnical conditions not susceptible to densification, liquefaction or settlement

TABLE 4-3 (Continued)

a. Bedrock
b. Frozen soils (which remain frozen)
c. Cohesive soils with sufficient cohesion
d. Very dense soils

B. Determine Design Liquefaction Potential of Segments Susceptible to Densification

1. Estimate design earthquake-induced seismic shear stresses. Account for:

a. Topography
b. Soil profile
c. Groundwater conditions
d. Thermal profile

- o Frozen over unfrozen soil
- o Unfrozen over frozen over unfrozen soils
- o Erratic frozen zones

e. Soil/bedrock interface
f. Initial stress state
g. Soil/structure interaction

- o Pipe
- o Frost bulb
- o Adjacent facilities

2. Utilize cyclic liquefaction characterization from II appropriate for the segment being analyzed with results of induced shear stresses from 1. above.

a. Calculate excess pore pressures in soil

- o Earthquake generated, plus
- o Thaw-consolidation generated

b. Estimate shear strain potential of soils experiencing initial liquefaction or instability

c. Estimate settlement potential of soils
* Note: Aging, grain size and any other site-specific parameter identified will be considered in these estimates

TABLE 4-3 (Continued)

d. Define "Design Liquefaction Potential" in terms of factor of safety, corresponding shear strain potential, and settlement potential.

V. SELECT LIQUEFACTION DESIGN SOLUTIONS LISTED UNDER III.A. SEGMENT-BY-SEGMENT BASED ON DESIGN LIQUEFACTION POTENTIAL

A. Segments Having No Liquefaction or Instability Potential -- No Further Concern (No Mitigation)

B. Segments With Liquefaction Potential -- Apply Criteria for Design Solutions (as Per III.C.)

1. Integrate design with other project conditions and constraints appropriate for the segment (as per III.B.)
2. Account for segment importance (consequence of liquefaction) (as per III.C.)

VI. ALLOW FOR FIELD VERIFICATION AND REDESIGN FOR CHANGED CONDITIONS ENCOUNTERED DURING CONSTRUCTION (AS PER III.D.)

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TABLE 4-4
OUTLINE OF SLOPE STABILITY DESIGN PROCEDURE

I. CATEGORIZE AND CHARACTERIZE ALIGNMENT IN TERMS OF EARTH MOVEMENT PHENOMENA

A. Soil/Residuum Failures

1. Characterization Parameters

a. Static factor of safety
b. Pseudostatic factor of safety
c. Flow slides
d. Seismic displacements

2. Geotechnical Factors Potentially Influencing Characterization Parameters

a. Geologic origin and history (land form)
b. Seismic zone (magnitude, recurrence interval)
c. Topographic slope
d. Soil type and properties
e. Groundwater and probable changes
f. Thermal state considerations

B. Rock Failures (slides, falls, topples)

1. Characterization Parameters

a. Static factor of safety
b. Pseudostatic factor of safety

2. Geotechnical Factors Potentially Influencing Characterization Parameters

a. Rock type and stratigraphy
b. Degree of weathering
c. Fracturing - orientation, spacing, continuity
d. Bedding - orientation
e. Clay seams
f. Cleft water pressures
g. Thermal state
h. Topographic slope
i. Seismic zone

TABLE 4-4 (Continued)

II. ESTABLISH DESIGN MITIGATION SOLUTIONS

A. Soil/Residuum Failures

1. Remove and Replace
2. Buttress
3. Groundwater Control
4. Reroute
5. Other (Grout, Reinforced Earth)

B. Rock Failures

1. Remove; Remove and Replace
2. Rockbolt
3. Deflection Barriers, Screens
4. Groundwater Control
5. Reroute
6. Other

C. Integrate Solutions with Other Project Conditions and Constraints

1. Pipe Mode
2. APSC Proximity
3. Liquefaction Constraints and Mitigation
4. Thaw Plug Constraints and Mitigation
5. Environmental Constraints
6. Construction Methods and Constraints
7. Cost
8. Contingencies

D. Establish General Application Criteria For Solutions

1. Probability of Occurrence; Degree of Activity and Size of Phenomena
2. Consequences to Pipe, Compressor Station, Ancillary Facilities

III. CONDUCT PRELIMINARY SEGMENT-BY-SEGMENT ANALYSIS

A. Literature Research

B. Air Photo Interpretation

C. Preliminary Field Reconnaissance

TABLE 4-4 (Continued)

D. Estimate Areas With Ground Movement Potential in Terms of Geotechnical Parameters

IV. CONDUCT SITE-SPECIFIC DETAILED INVESTIGATION

A. Geologic Mapping; Field Measurements

B. Obtain Sequential Air Photos, Fly Some Sites if Required

C. Conduct Subsurface Investigation (as appropriate)

D. Instrument and Monitor Sites (as appropriate)

E. Lab Testing (as appropriate)

F. Geotechnical Engineering Analysis

V. CONSTRUCTION INSPECTION, VERIFICATION, DOCUMENTATION; ACCOMMODATE CHANGED CONDITIONS

A. Establish General Criteria

B. Establish Documentation Guidelines, Format

C. Establish Field Redesign Procedure

5.0 HYDROLOGICAL CONSIDERATIONS

This section contains the surface water and groundwater hydrological considerations developed for the assurance of pipeline integrity, protection of adjacent property, and minimizing environmental impact.

5.1 SURFACE WATER HYDROLOGY CONSIDERATIONS

5.1.1 Design Floods

Two basic flood sizes will be used as well as two unusual flood considerations.

- Pipeline Design Flood (PDF) - The Pipeline Design Flood is derived by applying the most severe precipitation or snowmelt conditions which can reasonably be expected, excluding extremely rare combinations of events, to a mathematical model of the runoff characteristics of the particular watershed included. This flood is deterministically sized and does not have an associated return period.

  The pipeline design floods were estimated by using a regression equation which was derived from the results of analysis of nine representative streams from Delta south. The same approach was also applied to those stream crossings located north of the Brooks Range and to small drainage basins. During final detailed design a PDF will be computed for each stream, using that stream's basin and meteorologic parameters.

  For purposes of estimation, streams will be classified on the following basis:

  If PDF is greater than 10,000 cfs, it is classified as major.

  If PDF is between 1,000 cfs and 10,000 cfs, it is classified as minor.

  If PDF is smaller than 1,000 cfs, it is unclassified.

  Although this is a large flood, which might imply a higher initial cost, it is selected for the pipeline design flood for the following reasons:

  - The loss of system revenue from a protracted outage at a stream crossing is very large compared to the increased construction costs for a larger flood.

- The change in pipe burial depth at a particular stream crossing is primarily a function of change in water depth. For any large overbank flood of this type, depth normally increases only slightly with a large increase in discharge. Therefore, there is only a minimal reduction, if any, in costs for a smaller design flood.

- For many crossings the controlling costs will derive from avoiding impact on adjacent structures, property and the environment, as opposed to design flood magnitude.

- The selected pipeline design flood will be synonymous with Standard Project Flood, as developed by the Corps of Engineers and which is generally employed and accepted as the definitive standard throughout the hydrologic engineering profession. This flood was also used as the Pipeline Design Flood as a basis of design for the TAPS project.

o <u>Frequency Design Flood (FDF)</u>

For appurtenant and associated features of the gas line system, such as bridges, roads, culverts and other drainage structures, a smaller flood with an excedence frequency of 50 years will be used. A method of regional frequency analysis, being developed by the USGS as an update to a previous study, will be adopted. The method consists of a statistical approach in which data are analyzed by a multiple regression model. Most of the required input data are already available from existing records. The results are used to develop a relationship between existing regional frequency statistics and map-measurable variables. The major work effort in utilizing this statistical approach consists of updating flow records and preparing computer input. Distinct advantages associated with this method are:

- Additional records now available since the completion of the earlier studies.

- Use of hydrologically more homogeneous areas.

o <u>Glacier Outburst Floods (Jökulhlaups)</u>

The available literature does not indicate any glacierdammed lakes threatening the proposed gas pipeline route in Alaska. However, a field investigation to determine whether new lakes have formed or are likely to form along the alignment will be made, and recommendations will be offered on continuing investigations during the life of the project.

o <u>Seasonal Flood Variability</u>

Construction timing may be influenced by seasonal flooding. A review of expected seasonal flood sizes for certain streams is required. This study is elementary and will utilize many of the data developed in the Frequency Design Flood study.

5.1.2 Flood Levels and Velocities

The HEC-2 computer model, developed by the Hydrologic Engineering Center, will be used to determine flood levels and velocities for all stream crossings. The program uses a rigid-bed step-backwater process with the appropriate design floods and channel geometry as input to provide a design water surface and average velocities at each cross section location. The computed design levels will be field checked against the known flood levels (evidenced by high water marks). The assumption of a rigid bed may not be warranted in a few areas because of extensive stream alteration due to gravel mining or river-training. In these areas a movable-bed model such as HEC-6 would be used. HEC-6 accounts for raising and lowering of the bed elevation by eroding and depositing material in accordance with the stress imposed by a hydrograph. This method will not be routinely used because of the great amount of basic data input required. It will be used only where large changes in channel geometry are anticipated.

The water levels resulting from ice jams or aufeis will be determined on the basis of the available evidence and the application of hydrologic engineering judgment. No reliable analytical method of estimating aufeis levels exists at this time. However, it is known that ice jam levels tend to reach a limiting height slightly above the first floodplain terrace, and this limit is reached when sufficient conveyance around the ice jam is developed in the floodplain.

5.1.3 Scour

The determination of scour depth will be based on the following considerations:

o <u>General Scour</u>

A large number of interrelated variables affect the behavior of stream channels and therefore make definitive analysis difficult. Because of the complexity involved, no single method of estimating scour is entirely accurate and reliable. Four independent methods will be utilized and each method will not apply to each stream. The method most applicable for each stream will be weighted in making a reconciliation of the four methods discussed below:

- Armor Development Method - A limitation on the depth of a stream may be imposed by the development of an armor layer on the bed. This layer forms when flow drag force and uplift force are not sufficiently large to remove the coarser particles available. A number of procedures have been developed to estimate the flow depth for a given bed material composition and given flow conditions, from which a procedure will be developed for use on this project.

- Regime Formulation Method - Channels in alluvium tend to adjust their boundaries throughout their ranges of flow conditions until a generally equilibrium between depth, bed material and flow is reached. At this point, the channel is said to be "in regime." Although application of regime depth formulations is judgmental because of the high variability found in natural streams, utilization of these formulations will provide a good overall check.

- Sediment Transportation Relationship - Methods have recently been developed which allow efficient modeling of the interactions among bed material, suspended sediment, velocity and depth. These methods, which consider complete hydrographs and a long reach of stream, require large amounts of basic data and computer time. If used, they would probably be limited to reaches that are too complicated to be reliably analyzed by simpler means. At the present time, the most efficient means of applying these methods is through the use of the HEC-6 computer program model. This model is best suited to studying the long-term trends of scour or depositing in streams, considering changes which would result from encroachment within floodplains or gravel removal from streams.

- Evidence of Historic Scour Limits - Evidence of scour limits during large floods can often be found. This evidence may consist of the following:

  Buried organic material found in boreholes.

  Alterations of minerals in the alluvium resulting from scouring during large floods may be observed.

  Armor layers resulting from old floods may be observed in scour holes, test pits or sometimes in boreholes.

  In some coarse-bed streams evidence of scour depths and armor layers may be visible at selected locations.

This evidence must be carefully correlated with that developed by analytical methods. Instructions for those logging boreholes and performing field hydrologic investigations will be developed.

Present estimates are based on estimated stream bed material size (D50), drainage area, and pipeline design flood. The present criteria for estimating scour depth for unclassified streams are 2-1/2 and 4 foot minimum cover for drainage areas less than five square miles and ten square miles respectively. For some minor streams, the empirical relationship between scour depth and drainage area (Alyeska Data) was used. This will be confirmed during final design.

- <u>Local Scour</u>

  Some of the above methods will be used to predict scour in a limited constriction. None of them, however, can be used to predict scour caused by abrupt flow direction changes. In any case, if bridges or other structures are planned, then the local scour analysis will be required. This local scour is additive to the general scour and is more difficult to assess. Extensive literature exists and a scour assessment program best suited to the particular problems is available.

  For estimating purposes, streams are classified, in addition to flood magnitude, on the following basis.

  - If net scour depth for a stream is greater than four feet, it will be classified as major.

  - If net scour depth is greater than 2-1/2 feet and equal to or less than four feet, it will be classified as minor.

  - If net scour is smaller than or equal to 2-1/2 feet, it will be designated as unclassified.

Some of the procedures that will be utilized to assess local scour conditions are those advanced by Blench (modified for Alaska), Shen, Larvas and the Corps of Engineers.

5.1.4 Lateral Migration

An alluvial stream is constantly changing its position and shape due to its own hydraulic forces acting on its bed and banks. Changes may be slow or rapid and may evolve naturally or result from man's activities. Streams are the most actively changing of all geomorphic forms. In alluvial streams it is the rule rather than the exception that banks will erode, sediment will be deposited, and floodplains will be modified with time.

Pipeline sagbend and overbend setback distances are determined by site specific, historic, bank migration studies.

The assessment of bank migration will be based on judgment, available bank material data, channel patterns, and on empirical curves used by Alyeska. These curves are not reliable for site-specific bank migration design purposes. They are used at this time for estimation only. The most reliable and therefore the proposed approach for final design is to perform photo comparison. Ten feet of bank migration is adopted as the lower limit.

The design of pipeline crossings will take into account the lateral migration that might occur during the life of the project. This is accomplished by identifying the past migration for the reach of stream in question. Past lateral migration can be estimated by:

- o Comparison of historic air photos and maps.
- o Studies of the age of vegetation.
- o The accounts of residents and historic records.

Review of the above information can indicate future migration trends with regard to directions and rates of movement. Stream alteration, caused by riparian material sites and channel control structures, must also be considered.

5.1.5 Channel Control Structures

An important design consideration will be where and when to use channel control structures for the protection of stream banks and the pipeline from lateral migration. These structures may be used when all of the following conditions are met.

- o A specific economic analysis demonstrates cost-effectiveness.
- o The hydraulic effects on the stream do not result in unacceptable impacts on other structures or fisheries resources.

- The structures can be designed to protect the pipe and other property against the appropriate design flood. Extensive repairs after a large flood would be a possible alternative and must be considered in the analysis of cost-effectiveness.

- A satisfactory method for abandonment or removal of the structures after completion of use can be developed.

- There is continuing access for surveillance and maintenance during operation of the pipeline.

If channel control structures are used, three feet of free-board above the pipeline design flood level or the maximum recorded aufeis level will be used for design, whichever is greater. The top width of the structure will be adequate to allow for maintenance equipment and for increasing the structure's height if necessary in the future. Riprap and filter blankets will be sized in accordance with currently existing Corps of Engineers criteria. Spacing criteria will be based upon the intended purpose of the structure.

5.1.6 Stream Ice and Drift

At each proposed aboveground stream crossing, an analysis will be made to determine the characteristics of the stream in its natural state, or, if structures are in use, in its modified state. The analysis will include a general study to establish the nature and extent of ice conditions in the area. This evaluation will require information from various design data such as the design flood, water levels and cost. The primary concern is the maximum thickness of ice development in the stream.

5.1.7 Data Acquisition

The design of a pipeline stream or floodplain crossing requires the collection and analysis of hydrologic, hydraulic, sediment, topographic, geomorphic and environmental data. Many data are currently available but additional data are required for basic stream analyses.

Field data being collected for input to stream crossing design is outlined below.

- Prebreakup Survey

  Aufeis, stream ice and drift considerations will be essential in the design of crossings. At many of the major crossings where structures such as pipelines and highway bridges now exist, some ice and breakup data have been collected by Alyeska and others. There are many crossings, however, where no information is available regarding ice forming processes.

- Breakup Survey

  The purpose of this work will be to investigate high stream flows resulting from spring breakup. Observation of these flows will greatly increase understanding of how specific streams behave under potential flooding conditions.

  High water levels will be photographed and overflow channels noted at selected stream crossings, particularly where there are bridges, aboveground pipeline crossings, or river-training structures either existing or proposed.

- Panel Marker Installation

  The purpose of the panel marker program will be to install panel markers at each of the cross section locations at selected stream crossings. The installed panel markers will provide horizontal and vertical control essential for both the hydrologic and ground topographic field surveys. A temporary benchmark will be placed at each panel marker.

- Hydrologic Survey

  The purpose of the hydrologic survey will be to observe and measure hydrologic conditions at selected stream crossings.

  The scope of work involves:

  - Investigate and record changes which may have occurred since the existing maps, charts, and aerial photographs were issued.

  - Investigate and describe existing bridges, pipeline and other structures in the vicinity of the proposed pipeline crossing.

  - Examine debris, ice marks on the stream banks, and evidence of scour, high flood levels and past ice jamming.

  - Investigate overflow channels.

  - Obtain local information on past hydrologic events.

- Photograph stream channels, banks and overflow channels.

- Describe property which could be affected by backwater or scour.

- Assess roughness of bed and overbank areas.

- Determine stream velocities.

- Investigate surface evidence of possible armor layers which identify historic depths of scour.

- Estimate entrance and exist loss coefficients at existing bridges and culverts to be used in the hydraulic computations.

- Conduct pebble counts to assess surface bed material size.

o <u>Ground Topographic Survey</u>

The purpose of the ground survey will be to obtain cross sections and water surface profiles at selected locations along stream and floodplains. In addition, elevations will be determined of old high watermarks and of aufeis and stream ice levels previously flagged during the ice and prebreakup survey and the breakup survey.

The scope of work involves the following:

- Cross section the stream at pipeline crossings.

- Determine elevation of high watermarks and flagged stream ice and aufeis levels.

- Obtain bridge and culvert geometry required for backwater computations.

o <u>Unusual Floods</u>

In the event of significant floods, personnel will be deployed on short notice to obtain the needed measurements. These data will be some of the most useful but also the most transient of all field information available. The crews will coordinate fully with personnel from government agencies as well as those from Alyeska who will be gathering similar data.

o <u>Aerial Photography Surveys</u>

Aerial surveys will be conducted at each of the proposed stream crossings to obtain photogrammetric models which will be used to extend cross sections already surveyed during the ground topographic survey, permit backwater calculations when necessary, and develop sufficiently detailed contour maps. These surveys, combined with the cross sectioning and profile surveys conducted during the field hydrologic survey program, will provide:

Adequate topography for hydraulic computations outside of the active stream channel.

A basis for comparison with older photograph to ascertain historic rates of bank migration.

A source of information in determining and locating potential environmental constraints on the crossing design.

Base information for assessing possible changes in the stream regime (stream channel) imposed by construction of the pipeline.

Cross sections for backwater computations.

Detailed topography for training structures design, if required.

5.2 GROUNDWATER HYDROLOGY CONSIDERATIONS

Along the pipeline route there are five areas of groundwater related concerns. The five concerns are aufeis formation, slope stability, liquefaction, frost heave and thaw bulb.

Of the five concerns, four are discussed elsewhere. These four are slope stability, liquefaction, frost heave and thaw bulb.

5.2.1 Groundwater Program

The primary objective of the groundwater program will be to obtain baseline data, develop methods of analysis and establish predictive capabilities for the assessment of groundwater flow as it relates to the design of the chilled gas pipeline.

The design of a chilled gas pipeline must adequately address several major considerations: the integrity of the chilled gas pipe, the impact which the chilled gas pipe will have upon adjacent facilities, and the environmental consequences associated with a chilled gas pipeline in the arctic and subarctic regions.

The method of analysis depends upon the following procedures and data:

- <u>Estimation</u>

  This procedure involves an office effort in which the initial conditions and the effects of a cold pipe are estimated. The acceptability of this approach is judged on the basis of experience and the ability to predict and mitigate anticipated problems.

- <u>Full Scale Chilled Pipe Tests</u>

  This procedure employs full scale experiments in the field with a buried chilled pipe. Two types of test are being planned. These are the Frost Heave Field Tests and the Ice-Damming of Cross Drainage Tests.

The equipment to be used for the groundwater monitoring program includes:

- <u>Frost Probe</u>

  The frost probe collects data on the soil-water-ice temperature and resistivity. The resistivity of the soil has been shown to vary as the state of water changes from liquid to solid. Because water and ice can coexist near freezing, temperature data alone is not reliable for locating the freeze front.

  For this reason, measurement of the specific conductance or resistance allows for more accurate determination of the frozen boundary. The temperature sensors, in addition to measuring the active layer and permafrost temperature, can be used to monitor the air or snow temperature by extending the probe aboveground.

  The frost probe is metal-tipped fiberglass dowel of 3/4-inch diameter with variable sensor spacing, e.g., 16 to 24 sensors per 10 or 20-foot section of probe. The probe can be installed in a 1-inch diameter pilot hole or placed next to the MP system casing in the borehole annulus.

- <u>Tensiometer</u>

Tensiometers will be used in cases where soils experience seasonal unsaturated moisture conditions. The unit is designed to measure both negative (less than atmospheric) and positive (hydraulic head) pressures with automatic transfer between the pressure types. An onsite barometer is used to calibrate the data. The sensor contains a porous ceramic disk or stone behind which is a reservoir filled with a freeze depressant to prevent instrument damage during freezing. Laboratory tests have shown this instrument capable of discerning the large negative pressure gradient found adjacent to the freezing front which is caused by the attraction of water to the frost front.

The individual sensor is a cylinder 1-inch in diameter and 3 inches long. This small size makes it ideal for use in the active layer where alternating fine grained and coarser grained zones occur in a thinly bedded sequence. Installation of the tensiometer occurs in the same borehole used for the frost probe, with the vertical spacing of each unit depending on the in situ soil and water conditions. Clay backfill is used in the borehole to isolate each tensiometer at a specified depth.

Tensiometers will be used to measure variations in hydraulic head within the active layer. Observation wells, equipped with Multiple Piezometer (MP) systems will be used to determine aquifer permeability and pore water pressure distribution in thawed, fully saturated aquifers.

- <u>Multiple Piezometer (MP) System</u>

The Multiple Piezometer (MP) System of observation wells allows for multiple piezometer and soil permeability measurements to be obtained from a single borehole using a single casing string. The casing (2-inch diameter, O.D. PVC) comes in differing lengths called blanks. The blanks are joined with plain couplers or couplers that contain either piezometric or pumping ports. Packers are inflatable bladders that seal the annular space between the borehole and the casing. The packers isolate a zone around each port to ensure against communication with other zones. Each packer requires a 5-foot section of casing and two packers are needed to isolate each port.

The pumping ports are used to conduct falling head (or slug) permeability tests. With these ports open, water is discharged from the casing and into the surrounding aquifer matrix. The permeability is calculated from a plot of the change in water level with time. Permeability will be obtained for each significant change in formation encountered in the hole.

The pressure ports will be used to obtain pore water pressure (or potentiometric surface) measurements. A device that can be lowered through the inside of the MP System casing is used to open each pressure port. Sample containers may be attached to this probe for collection of isolated water samples, if required.

- Borehole Data

- Standpipe Observation

  Over 100 perforated standpipes have been placed in selected boreholes to help monitor groundwater levels. Periodic readings will be made throughout the year for a period of time prior to construction.

5.2.2 Aufeis

Groundwater may be redirected to the surface by the development of an underground obstruction which forms a barrier to the normal groundwater flow. Such an underground obstruction may be caused by construction of the chilled gas pipeline through thawed areas with active groundwater movement.

The extent of possible ice-damming is related to the velocity of the groundwater moving past the pipeline. The potential for frost bulb growth is decreased by the convective transport mechanism acting as a modifier to the freezing mechanism. Preliminary analysis indicates that when groundwater velocities are greater than 3 feet per day, frost bulb growth will be minimal. For velocities less than 1 foot per day, frost bulb growth can be considered to be unaffected by groundwater movement.

Groundwater velocity criteria for determining aufeis potential will be as follows:

- For $\upsilon < 1$ fpd, blockage will be assumed to equal depth of frost bulb penetration normal to flow direction under worst case (zero velocity) conditions.

- For $\upsilon > 3$ fpd, blockage will be assumed to equal depth of pipe (4 feet) normal to flow direction with no increase for frost bulb.
- For 1 fpd $< \upsilon < 3$ fpd, blockage will be assumed the same as in the case for $\upsilon < 1$ fpd because of the uncertainties present in this range.

The velocity will be calculated as a function of hydraulic conductivity, hydraulic gradient (assumed to be equal to the slope of the land surface), and effective porosity.

A mile-by-mile review will be made of the pipeline route to determine the potential for aufeis formation.