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Evaluation of Airport Pavement Designs for Seasonal Frost and Permafrost Conditions

March 2023

Final Report

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16. Abstract

In recent years, the Federal Aviation Administration (FAA) Alaskan Region reported performance issues with some flexible airport pavements. The issues were attributed to changes in the thermal regime of the permafrost layers due to a warming trend, but the poor pavement performance could have been exacerbated or caused by factors such as improper design and construction. The FAA Airport Technology Research and Development Branch reviewed the pavement failures, design, and construction histories to determine the cause(s) and contributing factors of flexible pavement performance issues. The goal was to determine if and where modifications to FAA guidance concerning design and construction in frost and permafrost zones were needed.

FAA selected three runways with an asphalt surface and one runway with a gravel surface at four airports in Alaska for detailed evaluation. The airports selected represent a variety of local conditions and permafrost zones. FAA collected design, construction, and traffic documentation from the selected airports. Researchers also conducted interviews with Alaska Department of Transportation and Public Facilities (Alaska DOT&PF) and FAA Alaskan Region engineers involved in the most recent pavement rehabilitation to gain additional insight into the pavement design, particularly with respect to the permafrost condition. The study focused on identifying frost and permafrost issues at the airports, identifying the design criteria used at the airports, and determining how closely the design criteria and construction practices followed FAA guidance. The study also re-created the pavement design for each runway following both the FAA guidance in effect at the time of the design and Advisory Circular 150/5320-6G. Researchers identified areas of the FAA frost and permafrost design criteria that are silent or subject to conflicting interpretation.

The study used findings from the design reviews and provided recommendations for possible improvements to the current FAA guidance with respect to frost and permafrost design. The recommendations targeted Advisory Circulars 150/5320-6G and 150/5370-10H and are intended to address the design- and construction-related issues.

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LIST OF SYMBOLS AND ACRONYMS

AC	Advisory Circular
Alaska DOT&PF	Alaska Department of Transportation and Public Facilities
AIP	Airport Improvement Program
AKOD98	Alaska Overlay Design Program for Flexible Highway Pavements
APEB	Aviation Project Evaluation Board
ARA	Applied Research Associates, Inc.
ARC	Airport reference code
CBR	California Bearing Ratio
CFP	Complete frost protection
DOS	Disk Operating System
ELSYM5	Layered elastic system analysis program
FAA	Federal Aviation Administration
FAARFIELD	FAA Rigid and Flexible Iterative Elastic Layered Design
FASBC	Foamed asphalt stabilized base course
FDD	Freezing-degree days
FG	Frost group
HMA	Hot-mix asphalt
IFP	Instrument Flight Procedures
IOAA	IFP, Operations, and Airspace Analytics
ksi	Kilopounds per square inch
LFP	Limited frost protection
MALSR	Medium-Intensity Approach Lighting System with Runway Alignment Indicator
MoS	Modification of Standards
M&O	Maintenance and operation
M_r	Resilient modulus
NOAA	National Oceanic and Atmospheric Administration
NREGS	Northern Region Engineering Geology Section
NRMS	Northern Region Materials Section
NSS	NextGen Shared Services
PCASE	Pavement-Transportation Computer Assisted Structural Engineering
PCI	Pavement Condition Index
RAP	Recycled asphalt pavement
RSA	Runway safety area
RSS	Reduced subgrade strength
SNAP	Scenarios Network for Alaska and Arctic Planning
STA	Station
TDD	Thawing-degree days

EXECUTIVE SUMMARY

In recent years, Alaskan airports have been challenged by premature failure of flexible pavements. While the warming trend and altering thermal regime of the permafrost layers have contributed to pavement failures, they are not necessarily the sole source of poor pavement performance at a specific airport. The premature failure in a permafrost area could be due to other factors such as improper design and construction.

The Federal Aviation Administration (FAA) initiated research to identify the cause(s) of flexible pavement performance issues at airports in Alaska to determine if and where FAA existing standards concerning design and construction in frost zones should be revised. FAA Advisory Circular (AC) 150/5320-6G includes requirements for frost protection and design of airport pavements in seasonal frost and permafrost areas. However, the guidelines should be reevaluated to address the challenges of changing permafrost.

The purpose of this study was to evaluate and improve design criteria for flexible airport pavements subject to seasonal frost and permafrost. To support this goal, the FAA selected three runways with asphalt pavement (Nome Airport; Ralph Wien Memorial Airport, Kotzebue; and Wiley Post-Will Rogers Memorial Airport, Utqiagvik) and one runway with gravel surface (Robert Curtis Memorial Airport, Noorvik) within the State of Alaska for detailed evaluation. Nome is located in a discontinuous permafrost zone, and the other three airports are located in a continuous permafrost zone. The FAA collected relevant documentation from the selected airports and conducted interviews with the Alaska Department of Transportation and Public Facilities (Alaska DOT&PF) and the FAA Alaskan Region engineers involved in the most recent rehabilitation or reconstruction to gain additional insight into the pavement design, particularly with respect to the permafrost condition.

Researchers used information from these documents and the interviews to identify frost and permafrost issues at the airports. The studies indicated a historical warming trend and loss of permafrost in the State of Alaska. In Nome, geotechnical investigations found that the permafrost has been degraded in some sections of the runway for more than 30 years. Thawing of the permafrost layer has significant influence on the airport pavement performance. The most common pavement distress type in the airports with asphalt pavement was localized differential settlements. It is likely that the loss of permafrost caused the settlements. This was aggravated by the inherently poor compaction of permafrost subgrade materials and incorrect timing and sequence of construction. The localized repair of the settled areas did not seem to be effective, as the settlements started to re-occur within a few years after the repair efforts. The full-depth repairs tend to significantly alter the thermal stability of the permafrost layer, making the subgrade thawing more detrimental.

Researchers identified the design methods and criteria for each of the airports. The intent was to determine whether the methods were based on the FAA guidance at the time of design, or if the airports used alternative methods based on best practices as accepted by engineers in the FAA Alaskan Region. Researchers re-created the pavement design for each runway following the guidance in effect at the time of the design. Areas where the FAA design guidance is silent or subject to conflicting interpretation were recorded, specifically criteria related to frost and permafrost protection. In addition, researchers reproduced the designs using the standards in

AC 150/5320-6G and FAARFIELD 2.0 for comparison, noting any areas where the criteria are silent. The design review determined which methods were used by the design engineer for frost protection in each case, and the amount of subsidence or heave considered tolerable under the design assumptions. In the event the actual input parameters (e.g., thermal parameters) deviated from the values used by the design engineer, researchers re-created the design using both the actual and the designer input parameters. The intent was to compare the actual design with the built design to determine if the poor performance of these pavements was due to faulty design assumptions, inadequacies in the FAA design method, or factors unrelated to the design method.

The design review indicated that loss of permafrost contributed to the pavement performance issues in the airports. Most of the designs would have been adequate if the permafrost remained. Loss of permafrost was partly caused by a warming trend that was not anticipated at the time of the design. The current FAA recommendation for calculation of thaw depth, based on the three largest thawing indices over 30 years, may have underestimated the depth of thaw from the warming trend. Construction-related issues also contributed to the degradation of permafrost and increase in thaw depth. Construction timing and sequence are critical factors to ensure the permafrost will remain protected for the entire design period. Pavement excavations during construction efforts altered the thermal balance of the subgrade which led to additional thawing of the permafrost. FAA guidance lacks standard procedures to address construction timing and sequence in frost/permafrost zones.

In addition, some design- and construction-related factors directly contributed to the performance issues, independent of loss of permafrost. In some instances, the designers did not use the FAA guidance in effect at the time of the design and instead used other methods for pavement design. In one case, the frost protection method implemented was not consistent with the FAA recommendations. Reviews also indicated that the rehabilitation efforts in most of the cases were not able to provide long-term solutions and pavement distresses reappeared a few years after the rehabilitation efforts.

The study used findings from the design reviews and provided recommendations for possible improvements to the current FAA guidance with respect to frost and permafrost design. The recommendations targeted ACs 150/5320-6G and 150/5370-10H and are intended to address the design- and construction-related issues. The recommendations include investigating a more conservative depth-of-thaw calculation to guard against warming trends and accounting for the change in thermal balance and permafrost due to construction and excavation. Revisions to the application of Complete Frost Protection and Reduced Subgrade Strength methods are also recommended. Partial frost protection seems of limited value, and it is recommended to remove it from the guidance. Reduced Subgrade Strength seems of limited value based on the small sample in this study. An additional recommendation includes adding proper procedures for construction and design of insulating panels to FAA guidance. The panels must be able to protect the permafrost layer for a prolonged time while withstanding overburden weight and aircraft loads.

1. INTRODUCTION

1.1 BACKGROUND

In recent years, premature failures of flexible pavements have been a challenge at Alaskan airports. While the cause of these failures has been attributed to the overall warming trend that has altered the thermal regime of the permafrost layers, poor pavement performance could also be due to other factors such as improper design and construction. Federal Aviation Administration (FAA) Advisory Circular (AC) 150/5320-6G (FAA, 2021) includes requirements for frost protection and design of airport pavements in seasonal frost and permafrost areas. There is widespread agreement that existing guidance should be reevaluated to address the challenges of changing permafrost. To that end, the FAA initiated research to identify the cause(s) of flexible pavement performance issues at airports in the Alaskan Region and to determine if and where the FAA should revise existing standards concerning design and construction in frost zones.

The FAA selected four runways at four airports in Alaska for detailed evaluation. Three have asphalt surfaces, and one has a gravel surface. The study focused on identifying frost and permafrost issues at the airports, identifying the design criteria used at airports exhibiting frost issues, and determining how closely the criteria followed FAA guidance. The FAA collected relevant documentation from the selected airports and conducted interviews with the Alaska Department of Transportation and Public Facilities (Alaska DOT&PF) and FAA Alaskan Region engineers involved in the most recent rehabilitation or reconstruction. The purpose of the interviews was to gain additional insight into the pavement design, particularly with respect to the permafrost condition. These personnel coordinated with the pavement consultants that provided geotechnical investigations, pavement designs, and other engineering services, as necessary. The interviews identified the conditions that led to the rehabilitation or reconstruction of the runways and whether the rehabilitation effort was intended to provide a long-term solution or a short-life repair. Interview questions were designed to clarify aspects of the design and construction history that could not be determined from the design documents.

1.2 SCOPE

The purpose of this study was to evaluate and improve design criteria for flexible airport pavements subject to seasonal frost and permafrost. To support this goal, this effort used information from the four airports and conducted a detailed design review. The goals of this design review were to:

- Determine the methods used for pavement design for each airport and whether the FAA standards were applied.
- Determine the causes of identified performance issues.
- Determine whether the identified performance issues were due primarily to incorrect design assumptions (specifically warming trends) or to inadequacies in the FAA design guidance.

Based on the findings from the design review and analysis, possible improvements of existing design guidance are recommended.

Sections 2 through 6 of this report document the pavement condition and design assumptions for the four runways in the Alaskan Region. These sections also include design review and analysis of each runway. Section 7 recommends potential changes to the FAA design and construction guidance.

On the recommendation of FAA Alaskan Region and Alaska DOT&PF personnel, the FAA selected three asphalt-surfaced runways at Nome Airport; Ralph Wien Memorial Airport, Kotzebue; and Wiley Post/Will Rogers Memorial Airport, Utqiagvik (formerly Barrow), for this investigation. These runways have had major rehabilitation/reconstruction projects in the past 15 years and have experienced localized failures or more generalized premature pavement and embankment failures. One runway with gravel pavements at Robert Curtis Memorial Airport, Noorvik, was studied. All selected runways are in continuous or discontinuous permafrost zones. The FAA collected the following documentation about the runways from each airport:

- Pavement design engineering reports
- Reports of geotechnical investigations and environmental assessments
- Airport Improvement Program (AIP) documentation, including FAA Forms 5100-1 (Airport Pavement Design) and any requests for Modification of Standards (MoS)
- Construction documents (drawings and specifications)
- Relevant standards and specifications
- Pavement condition inspection reports, work, and maintenance histories
- Runway usage data

Researchers used information from these documents and the interviews to identify design decisions for each of the airports. The intent was to determine whether the decisions were based on the FAA guidance at the time of design, or if the airports used alternative methods based on best practices accepted by engineers in the FAA Alaskan Region. Researchers re-created the pavement design for each runway following the guidance in effect at the time of the design. Areas where the FAA design criteria were silent or subject to conflicting interpretation were recorded, specifically those criteria related to the frost and permafrost protection. In addition, researchers reproduced the design using the standards in AC 150/5320-6G (FAA, 2021) and FAARFIELD 2.0 for comparison, noting any areas where the criteria are silent. The design review determined which methods were used by the design engineer for frost protection in each case, and the amount of subsidence or heave considered tolerable under the design assumptions. In the event the actual input parameters (e.g., thermal parameters) deviated from the values used by the design engineer, researchers re-created the design using both the actual and the designer input parameters. The intent was to compare the actual design with the built design to determine if the poor performance of these pavements was due to faulty design assumptions, inadequacies in the FAA design method, or factors unrelated to the design method.

1.3 FROST PROTECTION METHODS IN FAA STANDARDS

This section provides a summary of frost and permafrost design methods in FAA AC 150/5320-6G (FAA, 2021) and its predecessors ACs 150/5320-6C (FAA, 1978) and 150/5320-6D (FAA, 1995). These current and historical FAA standards all provide similar methods for frost protection. The FAA guidance requires special consideration to protect the pavement against detrimental effects of seasonal frost actions and thawing of permafrost. Seasonal frost refers to the condition in which the top layer of soil is frozen for part of the year. Seasonal freeze-thaw can cause non-uniform heave during frost action and/or loss of soil strength during the thawing period. The adverse effect of seasonal frost actions occurs when freezing temperatures penetrate frost-susceptible and moist soil. Permafrost is soil that remains frozen to considerable depths for many years and is common in the Alaskan region. The depth of permafrost may vary over time. The warmer summer temperatures can thaw the upper layer of permafrost. Thawing of frozen subgrade and embankment can lead to severe loss of load-bearing capacity, which, in turn, results in differential settlement. Pavement construction is another factor that can change the thermal balance of the underlying ground leading to degradation of the permafrost.

Calculation of the depth of frost penetration and depth of seasonal thaw are required for designing pavements seasonal frost and permafrost areas. AC 150/5320-6G (FAA, 2021) recommends using the freeze index and thaw index based on the average of the three coldest and warmest years, respectively, in the past 30 years of record. AC 150/5320-6D (FAA, 1995) provides charts (Figures 2-7 and 2-8) for calculation of frost and thaw penetration depth based on freezing and thawing index. AC 150/5320-6G refers to the Pavement-Transportation Computer Assisted Structural Engineering (PCASE) software program (U.S. Army ERDC, 2022) for calculating the frost depth. The FAA guidance categorizes soil into four frost groups (FG)-1, FG-2, FG-3, and FG-4, with the higher frost group number representing more frost susceptibility.

AC 150/5320-6G provides three methods for the design of pavements in seasonal frost areas:

- Complete Frost Protection (CFP). The total thickness of pavement structure must be equal to or greater than the depth of frost penetration to contain frost penetration within non-frost susceptible materials and prevent subgrade from seasonal freezing. This method applies only to soils in FG-3 and FG-4 and is recommended for large hub airports or in areas with minimal frost penetration.

- Limited Frost Protection (LFP). Frost is allowed to penetrate to a limited degree into the underlying frost susceptible layers. Non-frost susceptible materials are required for 65% of the depth of frost penetration. This method allows limited frost heave (typically less than 1 inch). This method applies to soils in all frost groups and is recommended for airports serving aircraft heavier than 60,000 lb gross weight.

- Reduced Subgrade Strength (RSS). The pavement structure must provide adequate load-carrying capacity to support design traffic during the weakest condition of the subgrade during frost-thawing period. The pavement thickness should be designed using reduced subgrade strength during thawing period or 50% of the subgrade design strength. This method applies to soils in FG-1, FG-2, and FG-3, which are uniform in horizontal extent or where the functional requirements of the pavement permit some degree of frost heave. This method is commonly used at non-primary airports serving aircraft less than 60,000 lb gross weight.

AC 150/5320-6G also provides three methods for the design of pavements in permafrost:

- CFP. The total thickness of pavement structure with non-frost susceptible materials must be equal to or greater than the depth of thaw penetration to contain the permafrost to remain frozen year-round. The application of this method is analogous to the CFP method for seasonal frost design.

- RSS. The pavement structure must provide adequate load-carrying capacity to support design traffic during the weakest condition of the subgrade during permafrost thaw. The application of this method is analogous to the RSS method for seasonal frost design.

- Insulated Panels. Insulating panels are placed above the frozen subgrade, which coincides with the top of permafrost, to protect against degradation of the permafrost.

AC 150/5320-6D (FAA, 1995) recommended frost protection methods similar to those in AC 150/5320-6G. The main difference between them is that for the RSS method, AC 150/5320-6D recommended assigning reduced subgrade California Bearing Ratio (CBR) values based on the frost group. AC 150/5320-6C (FAA, 1978) provided a Reduced Subgrade Frost Protection method in addition to the other methods in versions D through G. The source of the frost protection methods in FAA guidance is FAA Technical Report FAA-RD-74-30, *Design of Civil Airfield Pavement for Seasonal and Permafrost Conditions* (Berg, 1974). AC 150/5320-6C and later refer to the Berg (1974) report for the frost and permafrost design procedure details.

2. NOME AIRPORT

Nome Airport (OME) is situated on the south-central coast of the Seward Peninsula in northwestern Alaska. The main runway (10/28) is 6,000 feet long and 150 feet wide with an asphalt concrete pavement and unpaved shoulders. The crosswind runway (3/21) is 5,600 feet long and 180 feet wide. Runways 10/28 and 3/21 were previously designated 9/27 and 2/20, respectively. This effort focuses on Runway 10/28. Figure 1 shows the airport with runway ends labeled.

Figure 1. Nome Airport

Researchers reviewed the following documents for Nome, which are summarized in the following sections:

- Alaska Geological Consultants. (1970). *Soil and Foundation Investigation of the Nome Airport East-West Runway*
- Alaska Department of Transportation & Public Facilities. (1982). *Engineering Geology and Soil Report*
- R&M Consultants. (1986). *Investigation and Analysis of the Nome Runway Settlement Problems*
- R&M Consultants. (1987). *Nome Airport Runway Repair Study Volume I: Data Review, Acquisition, and Conceptual Design Report*
- R&M Consultants. (1988). *Nome Airport Runway Repair Study Volume III: Subsurface Investigations and Site Condition Report*
- R&M Consultants. (1991). *Subsurface Investigations and Site Conditions Report for Nome Airport East-West Runway Report*
- Alaska Department of Transportation & Public Facilities. (1997). *Nome East-West Runway Settlement Evaluation*
- R&M Consultants. (2001a). *Nome Airport Runway 27 Rehabilitation: Geotechnical Report*
- Alaska DOT&PF Northern Region. (2001b). *Runway 27 Rehabilitation and Obstruction Removal* (AIP 3-02-0199-1201)
- R&M Consultants. (2007). *Geotechnical Investigation and Recommendations Report*
- PDC Inc. Engineers. (2012). *Nome Airport Master Plan Update*
- Mappa Geotechnical. (2013). *Nome Airport Runway 10-28 Rehabilitation*

- Alaska DOT&PF Northern Region. (2013). *Nome Airport Safety Area Improvements Project: Geotechnical Investigation*
- R&M Consultants. (2020a). *Geotechnical Data Report: Nome Airport Operating Settlement Repair—Stage II*
- R&M Consultants. (2020b). *Nome Airport Rehabilitation—Engineer's Design Report*
- Alaska DOT&PF Northern Region (2020b). *Nome Airport Rehabilitation*, (AIP 3-02-0199)
- Alaska Airport Pavement Inspection Reports (2004, 2008, 2009, 2012, and 2018)
- Nome Airport As-Built Plans (1971, 1985, 1990, 1995, 1998, 2000, 2007, 2008, 2009, 2012, and 2016)

2.1 CONSTRUCTION AND REHABILITATION HISTORY

The original airport was constructed in the early 1940s. In 1958 and 1959, each runway was reconstructed with an extensive subdrain system beneath. Since the 1970s, large portions of each runway have been reconstructed numerous times to repair pavement distress mainly associated with subgrade settlement and thawing of frost-susceptible soil materials. In 1974 and 1975, the entire Runway 10/28 and 2,000 feet of Runway 3/21 were reconstructed, and the grade was raised 2 to 3 feet. The construction work involved removal of the existing pavement, compaction of the subgrade, and placement of 3 inches of asphalt over 9 inches of a crushed aggregate base course over an 11-inch subbase course (R&M Consultants, 1987). By 1979, new settlements appeared at seven areas along the centerline and along the entire pavement edge of Runway 10/28 (R&M Consultants, 1986). The 1986 geotechnical study (R&M Consultants, 1986) attributed the settlement to thawing of the permafrost caused by the warm summer and winter in 1978. In 1985, local pavement repairs were carried out to fix the settled areas. The western end of Runway 10/28 was then reconstructed in 1994. The objective of that effort was to completely remove soil materials causing differential settlement and heave by means of deep sub-excavation (approximately 30 feet). The sub-excavation was intended to remove nonuniform, soft, ice-rich and thaw-unstable soils. However, settlement areas appeared almost immediately after construction. A field study in 1998 concluded that settlements occurred because the sub-excavation failed to remove all the areas targeted for removal. The excavation took place in winter and the contractor was not able to fully compact the area due to flooding of the excavated area. The consequent problems occurred mainly due to lack of dewatering the excavation. Frozen material was also placed in the excavation. Settlements were severe enough to be considered detrimental to aircraft operations and were patched in fall 1997 (Alaska DOT&PF, 1997). Between 2002 and 2003, Runway 10/28 was reconstructed from the Runway 3/21 intersection eastward (R&M Consultants, 2001). The construction effort involved removal of the asphalt, base and subbase, and preparation and compaction of the subgrade. In some areas, deep excavation (approximately 20 feet) of subgrade material was carried out and backfilled with borrow materials. The new pavement section consisted of 4 inches of asphalt over 14 inches of crushed aggregate base over 12 inches of subbase material. Depressions on Runway 10/28 west of the intersection were also patched. A geotechnical study in 2007 identified seven areas with settlement and heaving of the pavement. Depressions along the west portion of Runway 10/28 were up to one-half foot deep and were distributed across the runway (R&M Consultants, 2007). The depressions were re-leveled and resurfaced in 2007. In 2013, the settlement areas were again repaired. In this effort, dynamic compaction of the subgrade was carried out in the affected areas in an attempt to densify the loose soils and mitigate further pavement settlement in the treated area (Alaska DOT&PF Northern Region, 2020b). A field survey by R&M Construction in 2017 identified six areas with pavement distresses, consisting mostly of depressions and bumps, and one area with high-severity transverse cracking. The distressed areas were leveled and resurfaced in 2019. The geotechnical report by R&M Consultants (2020a) summarized the airport improvement project as shown in Table 1. R&M Consultants (2020b) developed a pavement rehabilitation design, which included rehabilitation of the entire length of Runways 10/28 and 3/21, and deeper repair of select areas of Runway 10/28 experiencing severe depressions.

Table 1. Summary of Improvement Projects at Nome Airport (R&M Consultants, 2020a)

Year		Project No.		
Plans	Construction Year	AIP	DOT&PF	Scope of Construction
1971	1975	–	–	Reconstruct
1984	1986	3-02-0199-01	60438	Local pavement repairs (settlement areas)
1991	1993	3-02-0199-05	64188	Reconstruct (STA 6+00 to 49+00); with deep dig-\out from about STA 6+00 to 19+00
1997	1998	3-02-0199-09	66958	Local pavement repairs (settlement areas)
1999	2000	3-02-0199-10	60529	Reconstruct (STA 5+85 to 33+20)
2001	2002/2003	3-02-0199-12	60905	Local pavement repairs (STA 37+23 to 44+11)
2006	2007	3-02-0199-15	62774	Level and repave (STA 5+90 to 49+05)
2012	2012	3-02-0199-21	61731	Repair settlement areas; including deep dynamic compaction from STA 14+00 to 16+50, 85'R to 85'L
2013	2014/2015	3-02-0199-22	61413	Relocate threshold to STA 6+00 (from STA 5+90); extend embank to STA 3+00 as blast pad/RSA; extend embank to STA 3+00 as blast pad/RSA
2018	2019	3-02-0199-24	–	Reconstruct STA 8+00 to 10+50, STA 13+00 to 15+00, STA 33+00 to 34+75, STA 39+75 to 41+00

STA = Station
RSA = Runway safety area

2.2 GEOTECHNICAL AND PERMAFROST CONDITIONS

Nome is located in a discontinuous permafrost zone. The original runways were partially constructed on land that was filled by a mining company in the 1930s using dredged materials. Several geotechnical investigations have been conducted over the past five decades to evaluate the subsurface condition of the airport. The investigations included soil boring, embankment material testing, drainage evaluation, environmental and thermal evaluation, and pavement evaluation.

The 2007 geotechnical report (R&M Consultants, 2007) classifies soils underlying the Nome Airport into three general groups: (1) undredged areas, (2) dredged areas (mostly on the west part of the runway), and (3) fills including areas of deep sub-excavation. Each of these soils overlies relatively shallow bedrock. Undredged soils included glacial tills (mostly silt), alluvial sands, and organic material including peat. Since dredging typically indicates removal of underwater material, it is assumed that the term “dredged area” is used here to indicate a fill area or embankment where the fill material was obtained by dredging. This has reversed the order of soil deposition, i.e., fine-grained materials overlaid by coarser material. Substantial amounts of gravel were observed on the surface of the dredged areas. The fill material was either pushed into place during the initial airport construction or hauled from offsite sources. The 2007 geotechnical report by R&M Consultants concluded that “the quality and gradation of the fill is highly variable, and much of the material is moderately to highly frost susceptible, and highly degradable.” The geotechnical investigations revealed subgrade materials in frost groups FG-1 to FG-4, with the majority of the materials in FG-3.

Over the years, the dredging and numerous excavations during periods of pavement construction have altered the properties of subgrade materials, the thermal balance, and the freeze-thaw regime. The soil material has thawed as a result of initial dredging. Since the initial construction, the thermal condition and heat balance throughout the runway foundation have changed multiple times. Several reports indicate that, at some point, the thermal balance shifted to positive, i.e., the hot summer prevented the winter freeze-back leading to loss of permafrost. These actions and the warming temperature trend have changed the thermal balance to influence the depth of active layer.

The R&M Consultants (2007) investigation concluded that “warmer air temperatures may have caused deepening of the active layer and thawing of some of the finer grained deposits” in dredged area. The report also states that “the ground temperatures in this area are very close to the thawing point, then any small positive convective heat influx may be responsible for significant deviation in the depth of freezing and thawing predicted by purely conductive analyses.” The 2001 geotechnical investigation indicated that seasonal frost action has not been a serious issue in the runways. The seasonal frost heaves measured were generally less than 0.1 foot (R&M Consultants, 2001).

R&M Consultants (2020a) published a report in 2020 that contains results of a geotechnical investigation conducted in 2019 on the subsurface layers of the areas repaired in previous years. Findings showed that in one area that underwent several repairs and reconstructions (including the 2019 repair effort), almost all of the previous boreholes drilled prior to 2019 encountered weak- to well-bonded permafrost soils, generally below depths of about 15 feet. However, no permafrost soil was encountered during the 2019 drilling, indicating that the permafrost had thawed considerably in the previous 12 years. In another area, previous boreholes (prior to 2019) encountered weak- to well-bonded permafrost soils with variable ice content, generally below depths of about 9 to 16 feet. In contrast, the 2019 test borings in this investigation area showed that variable ice-rich permafrost was actively degrading. The 2020 report concluded that “permafrost is often continuous with the seasonal active layer in the undredged and unworked areas, but discontinuous or absent in areas that were dredged. However, the depth to and extent of permafrost in all areas has been degrading over the past decades as a result of the airfield pavement improvements” (R&M Consultants, 2020a).

The average freezing-degree days (FDD) and thawing-degree days (TDD) for the years 1986-2016 were 3,900 °F-days and 2,250 °F-days, respectively (R&M Consultants, 2020a). While this indicates a net freezing thermal balance, the permafrost in the dredged areas is almost gone. That the permafrost has largely disappeared in a relatively short time despite the net freezing thermal balance may indicate a different measure or better model of thermal conditions is required. The historic weather data indicate a gradual warming trend in both winter and summer air temperatures, which over time has a detrimental effect on the permafrost conditions.

2.3 PAVEMENT DISTRESSES

The geotechnical investigations prior to the 1994 construction reported that most of the major differential settlement areas occurred over the transitions between "dredged" and "undredged" ground. The gradual but continuing settlement in the "undredged" area was attributed to thawing of ice-rich organic soil because of the increased thaw depth below the base layer during warm years, and to the consolidation of soft and loose soil with high organic content (R&M Consultants, 1987; Alaska DOT&PF, 2004).

A 2000 survey of Runway 10/28 identified transverse cracking, localized depressions, and some block cracking (R&M Consultants, 2001). The transverse cracking was attributed to excessive stresses induced by diurnal and annual temperature cycles and the depressions were attributed to settlement in the subgrade.

The 2007 study identified seven areas within the runway with differential settlement, bumps, and cracks (R&M Consultants, 2007). Of these seven areas, six were west of the Runway 3/21 intersection. Three of the settlement areas lay in the sub-excavation area that was excavated to approximately 30 feet and backfilled during the 1994 runway rehabilitation. The report attributed the new settlements to incomplete dewatering, insufficient compaction on the lower part of the backfill, and soft foundation that was left beneath the backfill during 1994 construction. The report also concluded that the poor pavement performance was due to frozen material dumped into the backfill and pond ice buried beneath the backfill during the excavation. One of the bump/crack areas was located east of Runway 3/21 at the limit of the dredged area. This area was reconstructed in 2002–2003, and the cracks and bumps have reoccurred since that time.

Investigations and observations made by R&M in January 2001 suggested that the consequences of heave in winter and reduced bearing capacity during thaw periods have not been highly significant in the project area (R&M Consultants, 2001). The 2007 geotechnical studies (R&M Consultants, 2007) attributed the occurrence of surface settlements to four general subgrade conditions: "(1) Considerable variation in the soil profiles (e.g., different vertical sequencing and variable consistency or relative density and thermal state within each soil unit); (2) Settlements associated with thawing ice-rich permafrost in response to regional climate change; (3) Consolidation settlements of unfrozen, softer fine-grained soil units under the embankment load; and (4) Gradual densification of shallow, loose, coarse-grained materials possibly in response to repeated aircraft loadings and groundwater fluctuations."

The 2017 survey reported differential settlement resulting in pavement cracking on Runway 10/28. A report of the Airfield Aviation Project Evaluation Board (APEB) prepared for a Nome airport paving project, dated January 2018, identified considerable wear, frequent thermal cracking, wide cracks, and joints with settlement and isolated alligator cracking on Runway 10/28 (APEB, 2018). A 2020 survey indicated that the west end of Runway 10/28 exhibited a reoccurring settlement area. The source of the current (in 2020) surface settlement is presumed to be long-term settlement of the loose foundation soils (R&M Consultants, 2020a),

Alaska Airport Pavement Inspection Reports that report Pavement Condition Index (PCI) are available for the following years: 2004, 2008, 2009, 2012, and 2018 (Alaska DOT&PF). These reports summarized the PCI values for different pavement sections within the airport. Details of the stress type and severity are not included in the reports. Figure 2 shows the overall runway PCI at different years. The PCI deteriorated rapidly, from 100 to 56 within 3 years after the 2003 rehabilitation/reconstruction project. The PCI also dropped by 30 points over 5 years, after the 2007 rehabilitation project.

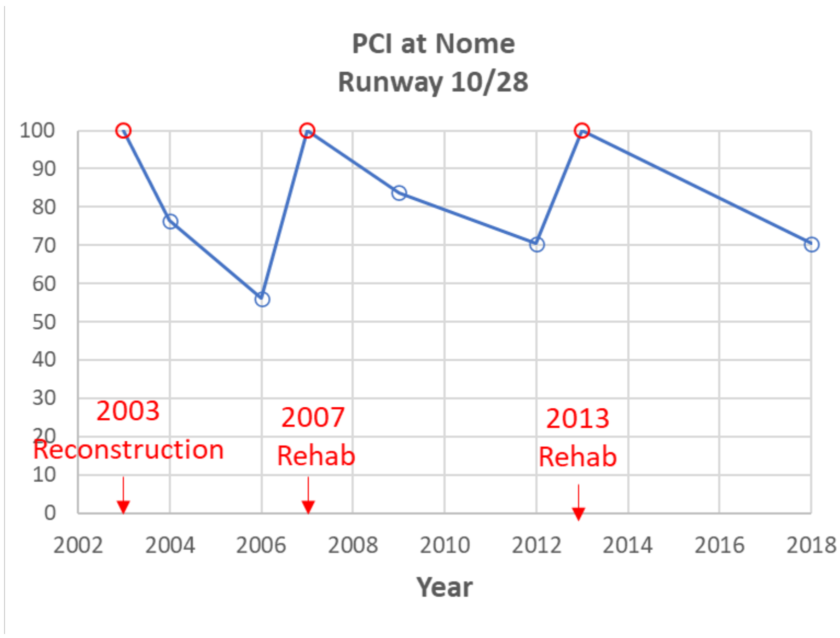

Figure 2. Nome Airport PCI Variation

2.4 PAVEMENT DESIGN

The pavement in the 1970s construction at Nome Airport was designed to accommodate Approach Category C aircraft in Design Group III. In the 1980s construction, both Runways 10/28 and 3/21 were designed to support a 150,000-pound, dual-gear aircraft. The design of the runway geometry was in accordance with FAA AC 150/5300-12, Change 1, dated March 14, 1985 (R&M Consultants, 1987). There is no record of a pavement design for the 1970s and 1980s construction using the then-current AC 150/5320-6C or other design methods.

R&M Consultants performed the pavement rehabilitation design for the 2002–2003 reconstruction of Runway 10/28 east of Runway 3/21 (R&M Consultants, 2001). The rehabilitation recommendation for Runway 10/28 also included full-depth repair of the settled area west of Runway 3/21. The pavement design used a mechanistic approach to evaluate stresses, strains, and deflection given the total aircraft loads and pavement layer strength (characterized by the resilient modulus, Mr). Seasonal variation in Mr was considered to account for the reduced bearing capacity in the base and subbase during periods of thaw. The report did not provide further details of the mechanistic approach. The report states that the results were compared with the minimum pavement thickness determined using the CBR design approach in AC 150/5320-6D (1995), but details of the analysis were not provided. Table 2 provides a summary of the aircraft mix used for design.

Table 3 presents the material properties assumed for the design. R&M did not perform a detailed thermal analysis or geotechnical investigation and relied on past investigations. The design freeze and thaw indices were 5,600 °F-days and 2,574 °F-days, respectively. The depth of the active layer was determined based on results of geotechnical borings, and not empirical methods such as the modified Berggren equation (see Berg, 1974). The designers recommended deep excavation of the subgrade under the newly constructed area and the repair area to reduce or mitigate the potential settlement due to seasonal thaw and material consolidation.

Table 2. Aircraft Mix Design for 2003 Rehabilitation Project (R&M Consultants, 2001)

Aircraft Mix	Maximum Takeoff Weight (lb)	Total Annual Operations	
		Airport	Runway 10/28
B737-400	140,000	4,455	2,228
B727-200	200,000	131	105
DC-6	100,500	786	629
C-130	175,000	100	80

Table 3. Pavement Material Properties for 2003 Rehabilitation Project (R&M Consultants, 2001)

Material Layer	Resilient Modulus, MPa (ksi)			Poisson's Ratio		
	Summer	Winter	Spring Thaw	Summer	Winter	Spring Thaw
Asphalt Concrete	1400 (203)	8300 (1200)	2800 (406)	0.45	0.30	0.40
Crushed Agg. Base	350 (50)	850 (123)	90 (13)	0.35	0.25	0.45
Classified Subbase	200 (29)	350 (50)	50 (7.2)	0.35	0.35	0.25
Subgrade	140 (20.31) (say 20)	350 (50.76) (say 50)	35 (5.07) (say 5)	0.35	0.35	0.25

ksi = kips (kilo pounds) per square inch

In 2007, R&M Consultants performed the pavement thickness design using the mechanistic design method contained in the *Alaska Flexible Pavement Design Software v1.0.64*. This software employs the computer program ELSYM5 (Kopperman, Tiller, & Mingstan, 1985) to calculate the stresses and strains at critical locations by assuming the pavement structure as a system of elastic layers. The designer used seasonal variations in layer properties. The pavement design also followed AC 150/5320-6D to determine minimum layer thickness. The average departures by month were determined for the Boeing 737-200, 737-400, 737-700, and 727-100; McDonald Douglas DC-6; and Lockheed L-100-30. These departures were converted to equivalent departures of a 737-200, which accounts for more than 50% of the annual departures. A pavement design life of 15 years was considered with a growth factor of 5% to estimate the total loads. Note that the standard FAA design life is 20 years, and no reason was provided why this was not used. The loads were then divided by season and used for seasonal design. It was assumed that Runway 10/28 would serve 70%, and the 3/21 Runway would serve 30% of the traffic. The subbase was modeled as 24 inches of granular fill. Due to poor quality of the subgrade material, a low modulus value was considered. A relatively high modulus value was considered for the base layer because it was assumed that the base would be improved by adding a layer of new base course, or a layer of old base course mixed with about 50% recycled asphalt pavement (RAP). The minimum calculated pavement section was 4 inches of asphalt, 4 inches of base (new or 50/50 RAP blend), over the existing base course. Note that this pavement design was never implemented.

In 2020, R&M Consultants performed a rehabilitation design for the entire length of Runways 10/28 and 3/21 using FAARFIELD pavement design software. A 20-year design life was considered. The proposed rehabilitation option involved removing 8 inches of existing pavement and replacing it with 4 inches of P-401 hot mix asphalt (HMA) and 4 inches of P-318 Foamed Asphalt Stabilized Base Course (FASBC). R&M also proposed alternatives for full-depth repair of three areas on Runway 10/28.

2.5 TRAFFIC

The 2012 Nome airport master plan reported 59,978 base passenger enplanements for the year 2008. The plan assumed a 0.60% annual increase, with enplanements reaching 67,601 by 2028. According to the master plan, "the 737-400 in the Combi configuration is the primary aircraft flown into Nome. 737-400 will remain as the design aircraft in the near term (0 to 5 years), with the 737-800 becoming the design aircraft for the intermediate (5 to 10 years) and long-term (10 to 20 years) planning periods." The airport reference code (ARC) for Nome runway was established based on the FAA AC 150/5325-4B recommendations (FAA, 2005). The most demanding ARC that met FAA's substantial use threshold of at least 500 annual operations was C-III through 2015. The 2012 airport master plan recommended that "beyond 2015, the ARC should be D-III. This change in ARC reflects an expectation that Alaska Airlines will switch from the 737-400 to the 737-800 aircraft."

Researchers obtained traffic data from 2014 to 2019 on both runways from the Instrument Flight Procedures (IFP), Operations, and Airspace Analytics (IOAA) Tool managed by MITRE Corp. for the FAA. Runway usage data in IOAA are highly granular for individual aircraft arrivals and departures. Information includes arrival/departure date and time, aircraft identification and type, and the probable arrival or departure runway end (with assigned probabilities).

2.6 INFORMATION FROM INTERVIEWS

Representatives from Alaska DOT&PF, FAA Alaskan Region, FAA William J. Hughes Technical Center, and Applied Research Associates, Inc. (ARA) held a teleconference on November 5, 2020, to discuss the pavement issues identified in the reports on Nome Airport. The discussion focused on the most recent rehabilitation project on Runway 10/28. While most of the recent pavement settlements were attributed to the thawing of the subgrade, complete frost protection could not be applied during the rehabilitation project, and future pavement settlement is expected. Appendix A provides the complete interview.

2.7 DESIGN REVIEW

This section reviews the 2002-2003 pavement reconstruction project for the east portion of Runway 10/28. This was the only project for which the design documents were available. The designed section consisted of 4 inches of asphalt concrete, over 14 inches of crushed aggregate base and 12 inches of subbase on prepared and compacted subgrade. The following sections identify design assumptions and input parameters as used by the design engineers and as required by the FAA standards and re-create the pavement design accordingly.

2.7.1 Design Based on the Original Design Inputs

The original design was similar to, but not exactly the same as, the RSS method in AC 150/5320-6D. Design engineers used a mechanistic approach to evaluate pavement stresses, strains, and deflections under the design aircraft loads. They considered seasonal variation of M_r to account for the reduced bearing capacity in the base, subbase, and subgrade during periods of thaw (see Table 3). Table 4 shows the aircraft mix and subgrade CBR used for the design in the 2001 design report (R&M Consultants, 2001). The CBR values in this table were obtained by dividing the design M_r (expressed in ksi) by 1.5. The design report does not identify criteria for the acceptable level of pavement distress. The design FDD and TDD in the design report, defined as the average three coldest winters and warmest summers over a 30-year period, were approximately 5,600 °F-days and 2,600 °F-days, respectively. However, the design documents do not include calculation of freeze and thaw depths, likely because the designers did not make use of the exact frost or thaw depth in the modified RSS method. The designers used observations from geotechnical boreholes only to establish that the seasonal frost depth exceeded the depth of the proposed pavement structure, thus requiring use of the RSS method.

Table 4. Summary of Input Parameters by Design Engineers at Nome Airport

Season	Traffic Mix				Subgrade CBR
	B727	B737	DC-6	C-130	
Summer	31	635	180	23	13.5
Winter	20	412	117	15	33.8
Spring	3	67	19	2	3.4

Researchers used input parameters from the design engineers to re-create the design. The total required thickness was determined using the FAA design spreadsheet F806FAA.xls[1], which is based on the design charts in Figures 3-2 and 3-3 of AC 150/5320-6D (FAA, 1995). The AC

[1] See https://www.faa.gov/airports/engineering/design_software/previous. Note: the 2005 version available at this website is a more recent version than the 2002/2003 that was used in this research.

150/5320-6D method uses the concept of “design aircraft,” i.e., the aircraft type in the traffic mix which by itself requires the greatest pavement thickness when using the design curves. Each aircraft in a mix containing a variety of aircraft with different landing gear types and different weights must be converted to the same landing gear type as the design aircraft. The equivalent annual departures of the design aircraft must be calculated based on the ratio of the wheel load of each aircraft in the mix to the wheel load of the design aircraft. The B737 with dual gear (150,000 lb) was considered the design aircraft. The number of equivalent annual departures of the design aircraft was calculated for the traffic mix of each season. Table 5 presents the design summary. Note the as-built thickness of crushed aggregate base (14 inches) exceeds the required amount (8 or 9 inches, depending on whether seasonal or year-round traffic is used). The total as-built thickness is adequate to support traffic during any season if the excess crushed aggregate base is treated as a high-quality subbase with an equivalency factor of 1.4 as recommended in Table 3-6 of AC 150/5320-6D (FAA, 1995). However, this approach does not consider the fact that damage is cumulative across the seasons and the guidance at the time did not provide a way to calculate accumulated damage. The RSS method in AC 150/5320-6D requires that the pavement must have adequate load-carrying capacity to support the total traffic for the weakest subgrade condition during the thawing period. Table 5 includes the required pavement section using combined traffic from all seasons. Results indicate that the existing pavement thickness is inadequate to support year-round traffic. The design spreadsheet for the spring condition design is shown in Figure 3. Design spreadsheets for other cases are included in Appendix B.

Table 5. Design Summary Based on Design Engineer Inputs at Nome Airport

Design Method	Season	Equivalent Annual Departure	Subgrade CBR	Required Total Thickness (in.)	Required Thicknesses (in.)		
					Asphalt	Base	Subbase
Design Engineer	Summer	776	13.5	16.0	4.0[(1)]	8.0[(1)]	4.0
	Winter	506	34.0	7.5	4.0[(1)]	8.0[(1)]	0
	Spring	86	3.4	32.5	4.0[(1)]	8.0[(1)]	20.5
Combined Traffic	Year-Round	1,354	3.4	41.5	4.0[(1)]	9.0	28.5

[(1)] Governed by the minimum thickness required for asphalt surface and granular base

FLEXIBLE PAVEMENT DESIGN FOR			program date 5/15/03
Nome RWY 10-28			*AC Method*
Nome, AK			
Engineer -	Applied Research Associates	AIP No.	
Spring Design			

32.5"	**Total Thickness Required (inches)**
	No thickness adjustments required

Stabilized Base/Subbase Are Required

Initial Pavement Cross Section		**Stabilized or Modified Cross Section**		Factors
4"	Pavement Surface Layer (P-401)	**4"**	P-401 Plant Mix Bituminous Pavements	
8" (6.04)	Base Layer (P-209)	**8"**	Not stabilized	1
20.5"	Subbase #1 (P-154) CBR= 20	**20.5"**	Not stabilized -- P-154	1
0"	Subbase #2 CBR= 0	**0"**	Material as defined by user	
0"	Subbase #3 CBR= 0	**0"**	Material as defined by user	

( ) = Subminimal base thickness calculation

Frost Considerations		
110 lb/cf	Dry Unit Weight of Soil	
4500	Degree Days ºF	
138"	Frost Penetration Depth	
3.4	Original CBR value of subgrade Soil	
3.4	CBR Value used for the Subgrade Soil	Non-Frost Code for Subgrade Soil
20	CBR Value used for subbase #1	Non-Frost code for Subbase #1
0	CBR Value used for subbase #2	Non-Frost code for Subbase #2
0	CBR Value used for subbase #3	No frost selection made for Subbase #3

Design Aircraft Information			
The Design Aircraft is a DUAL150 - 150,000 lbs -- ( )			
150000 lbs	Gross Weight	**20**	Design Life (years)
86	Equivalent Annual Departures		

Subgrade Compaction Requirements for Design Aircraft			
Non-Cohesive Soils		**Cohesive Soils**	
Compacti	Depth Required	Compaction	Depth Required
100%	0 - 20	95%	0 - 8"
95%	20 - 34.5"	90%	8 - 15"
90%	34.5 - 49.5"	85%	15 - 22.5"
85%	49.5 - 64.5"	80%	22.5 - 30"

Figure 3. Design Detail for Spring Condition at Nome Airport

The original design assumptions did not follow the FAA standards in effect at the time of the design for these reasons:

- AC 150/5320-6D (FAA, 1995) recommends the RSS method for uniform subgrade material and FG-1 to FG-3. This method is not appropriate for this runway since the subgrade material is highly variable and contains FG-4 material.

- Table 3-1 of this AC (FAA, 1995) recommends assigning reduced subgrade strength based on the frost group. However, the design engineers used seasonal values.

- The mechanistic approach is not an FAA standard.

- The pavement must support total design traffic during the thawing period. However, the designer used the traffic during the thawing period to check the adequacy of pavement structure.

2.7.2 Design Based on AC 150/5320-6D

This section re-creates the pavement design following AC 150/5320-6D. Because the permafrost was already degraded at the time of design in some areas, the pavement must be designed for the effect of seasonal frost. Given the nonuniform subgrade condition and presence of frost-susceptible material in the FG-4 category, the CFP method is most appropriate for thickness design.

AC 150/5320-6D (FAA, 1995) states that if the implementation of the CFP method is not practical, the design may be based on the RSS method, but designers should expect large seasonal frost heave. AC 150/5320-6D also allows the LFP method for seasonal frost protection. In this method, 65% of the depth of frost penetration is made up of non-frost-susceptible material. Researchers re-created the design using all three methods. Given the prevalence of FG-3 material, a reduced subgrade CBR value of 4 was assumed for RSS, following Table 3-1 of AC 150/5320-6D. AC 150/5320-6D is silent about what subgrade strength should be used for CFP and LFP seasonal frost protection design. The subgrade was unfrozen at the time of construction, so researchers assumed this was its normal condition. Thus, the subgrade CBR during the thawing period and during summertime (Table 4) were used for the design. The thawing CBR was used in the LFP method, assuming the weakened subgrade condition.

Figure 4 and Figure 5 show the FDD and TDD obtained from the National Oceanic and Atmospheric Administration (NOAA) Alaska-Pacific River Forecast Center website. The design FDD and TDD indexes, defined as the average of the three coldest winters and the three warmest summers, respectively, over a 30-year period before the year of design (2001), were 5,560 °F-days (FDD) and 2,680 °F-days (TDD). According to Figures 2-7 and 2-8 of AC 150/5320-6D (FAA, 1995), the frost penetration depth is approximately 11.5 feet and thaw penetration is greater than 11 feet. Table 6 presents design traffic used for all the methods described above.

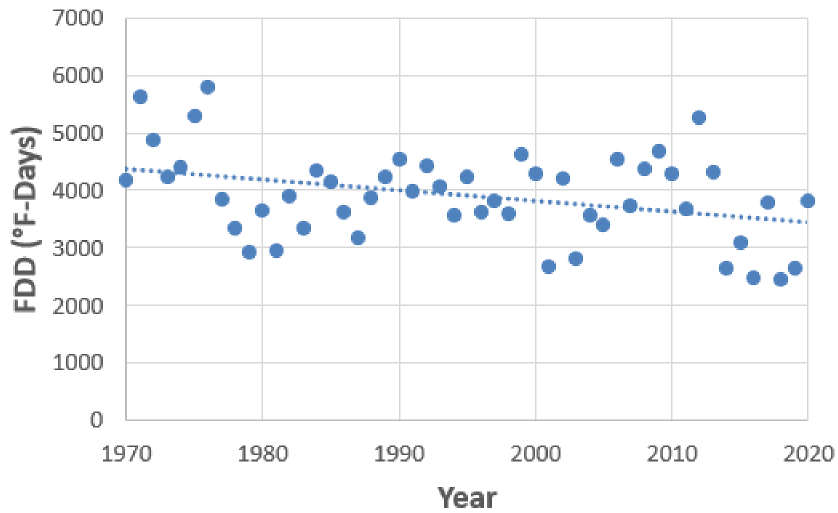

Figure 4. Change in Freezing-Degree Days in Nome 1970–2020
(Data from http://weather.gov/aprfc [Alaska-Pacific River Forecast Center])

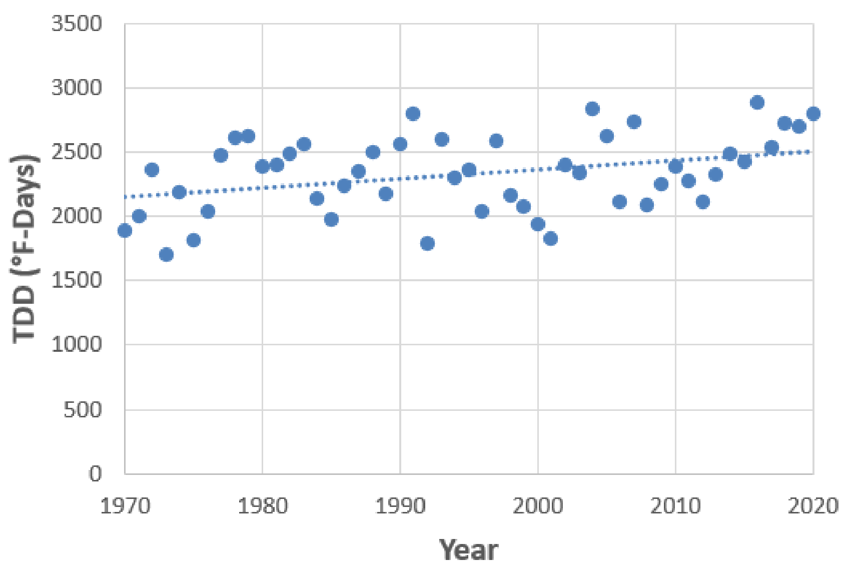

Figure 5. Change in Thawing-Degree Days in Nome 1970–2020
(Data from http://weather.gov/aprfc [Alaska-Pacific River Forecast Center])

Table 6. Design Traffic at Nome Runway 10/28

Aircraft Type	Annual Departure
B-727	55
B-737	1,114
DC-6	315
C-130	40

The total required thickness was determined using the FAA design spreadsheet F806FAA.xls[2]. The required asphalt thickness was determined to be 4 inches. The thickness of the base layer was determined using the same charts but using CBR = 20 for the subbase layer and subtracting the 4-inch asphalt layer from the result. The minimum base thickness is 6 inches per Table 3-4 of AC 150/5320-6D (FAA, 1995). The B-737 with dual gear (150,000 lb) was the design aircraft. Using the traffic mix in Table 6, researchers computed 1,354 equivalent annual departures of the design aircraft. Table 7 summarizes the thickness designs following AC 150/5320-6D. The existing pavement structure is not adequate to support the traffic in any of the frost protection methods. The additional thickness of subbase required to protect against frost action (beyond the thickness required to support the design traffic) is given in the table for each of the methods. Note that both RSS and LFP have lesser thickness requirements than CFP. For both RSS and LFP, differential settlements and frost heave should be expected due to the nonuniform subgrade conditions. A sample of a design spreadsheet is shown in Figure 6 for the RSS method. Design spreadsheets for the other methods are included in Appendix B.

Table 7. Design Summary Based on AC 150/5320-6D Inputs at Nome Airport

Design Method	Frost Protection Method	Subgrade CBR	Total Required Structural Thickness (in.)	Required Thicknesses (in.)			
				Asphalt	Base	Subbase (Structural)	Additional Subbase for Frost
150/5320-6D	RSS	4.0	37.5	4.0	9.0	24.5	0.0
	LFP	3.4	41.5	4.0	9.0	28.5	48.5
		13.5	17.0	4.0	9.0	4.0	73.0
	CFP	3.4	37.5	4.0	9.0	28.5	96.5
		13.5	17.0	4.0	9.0	4.0	121.0

2.7.3 Design Based on AC 150/5320-6G

The CFP method is also the most appropriate method for this runway according to AC 150/5320-6G. AC 150/5320-6G (FAA, 2021) indicates that the LFP method could result in frost heave up to 1 inch. The RSS method was not applied to this runway because AC 150/5320-6G recommends this method only for uniform subgrades in FG-1, FG-2, and FG-3, and for non-primary airports serving aircraft less than 60,000 lb gross weight. AC 150/5320-6G is also silent about what subgrade strength should be used for CFP and LFP seasonal frost design. To re-create the CFP design, researchers used the subgrade CBR during the thawing period and during summertime. The thawing CBR was used in the LFP method. The pavement design was checked for both the design traffic in Table 6 and for the average actual traffic from 2014 to 2019 (Table 8). Researchers used FAARFIELD 2.0 for the design.

[2] See https://www.faa.gov/airports/engineering/design_software/previous. Note: the 2005 version available at this website is a more recent version than the 2002/2003 that was used in this research.

FLEXIBLE PAVEMENT DESIGN FOR		program date 5/15/03
Nome RWY 10-28		*AC Method*
Nome, AK		
Engineer - Applied Research Associates	AIP No.	
AC 150/5320-6D Table 3-1		

37.5"	**Total Thickness Required (inches)**	
	No thickness adjustments required	
		Stabilized Base/Subbase Are Required

Initial Pavement Cross Sectio		**Stabilized or Modified Cross Section**		Factors
4"	Pavement Surface La	**4"**	P-401 Plant Mix Bituminous Pavements	
9"	Base Layer (P-209)	**9"**	Not stabilized	1
24.5"	Subbase #1 (P-154) C	**24.5"**	Not stabilized -- P-154	1
0"	Subbase #2 CBR= 0	**0"**	Material as defined by user	
0"	Subbase #3 CBR= 0	**0"**	Material as defined by user	

Frost Considerations		
110 lb/cf	Dry Unit Weight of Soil	
4500	Degree Days ºF	
138"	Frost Penetration Depth	
4	Original CBR value of subgrade Soil	
4	CBR Value used for the Subgra	F-3 Frost Code for Subgrade Soil
20	CBR Value used for subbase #1	Non-Frost code for Subbase #1
0	CBR Value used for subbase #2	Non-Frost code for Subbase #2
0	CBR Value used for subbase #3	No frost selection made for Subbase #3

Design Aircraft Information			
The Design Aircraft is a DUAL150 - 150,000 lbs -- ( )			
150000 lbs	Gross Weight	**20**	Design Life (years)
1,354	Equivalent Annual Departures		

Subgrade Compaction Requirements for Design Aircraft			
Non-Cohesive Soils		**Cohesive Soils**	
Compacti	Depth Required	Compaction	Depth Required
100%	0 - 20	95%	0 - 8"
95%	20 - 34.5"	90%	8 - 15"
90%	34.5 - 49.5"	85%	15 - 22.5"
85%	49.5 - 64.5"	80%	22.5 - 30"

Figure 6. Design Detail for the AC 150/5320-6D Reduced Subgrade Strength Method at Nome Airport

Table 8. Average Annual Traffic (2014–2019) at Nome Runway 10/28

Aircraft	Annual Operation	Annual Departure
737-200	438	219
737-300	164	82
737-400	86	43
737-700	1,506	753
737-800	381	190.5
L-100	342	171
MD-8	160	80
MD-9	105	53

Figure 7 shows the required pavement thickness computed by FAARFIELD using the design traffic and spring CBR. Complete FAARFIELD pavement design reports are provided in Appendix B. Table 9 presents a summary of required pavement structure to support the design and actual traffic for both the spring and summer CBRs. The required subbase thickness for frost protection is also included in Table 9. Results indicate that the existing pavement structure is not adequate to protect the pavement for the effect of seasonal frost. Table 10 summarizes the pavement design from different methods.

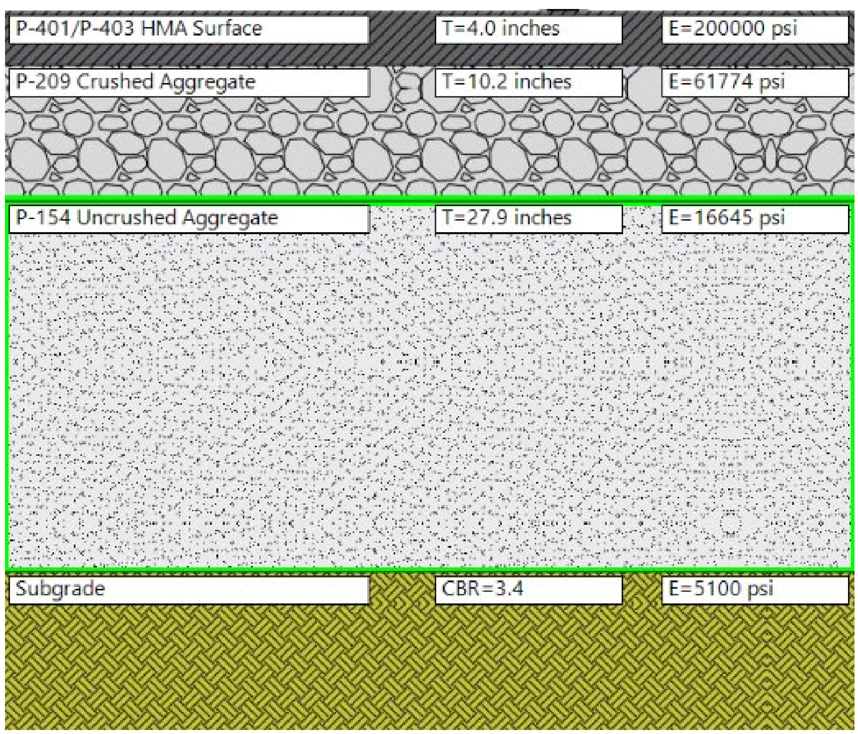

Figure 7. FAARFIELD Design Based on AC 150/5320-6G Using Design Traffic at Nome Airport

Table 9. Design Summary Based on AC 150/5320-6G Inputs at Nome Airport

Design Method	Frost Protection Method	Traffic	Season[a]	Required Structural Thickness (in.)			Required Frost Protection Thickness (in.)
				Asphalt	Base	Subbase	Subbase
AC 150/5320-6G	LFP	Design Traffic	Spring	4.0	10.2	27.9	76
			Summer	4.0	10.2	4.2	76
		Actual Traffic	Spring	4.0	10.8	27.8	75
			Summer	4.0	10.8	4.1	75
	CFP	Design Traffic	Spring	4.0	10.2	27.9	124
			Summer	4.0	10.2	4.2	124
		Actual Traffic	Spring	4.0	10.8	27.8	123
			Summer	4.0	10.8	4.1	123

[a]Spring CBR = 3.4; Summer CBR = 13.5

Table 10. Comparison of Design Methods at Nome Airport

Layer	Existing	AC 150/5320-6D			AC 150/5320-6G	
		RSS	LFP	CFP	LFP	CFP
Surface	4 in. Asphalt	4 in. P-401	4 in. P-401	4 in. P-401	4 in. P-401	4 in. P-401
Base	14 in. Crushed Agg	9 in. P-209	9 in. P-209	9 in. P-209	10.2 in. P-209	10.2 in. P-209
Subbase	12 in. Fill	24.5 in. P-154	77 in. P-154(1)	125 in. P-154(1)	77 in. P-154(1)	125 in. P-154(1)
Subgrade	CBR=3.4(2)	CBR=3.4(2)	CBR=3.4(2)	CBR=3.4(2)	CBR=3.4(2)	CBR=3.4(2)

(1) Governed by frost depth
(2) Spring value

3. RALPH WIEN MEMORIAL AIRPORT, KOTZEBUE, ALASKA

Ralph Wien Memorial Airport (OTZ), Kotzebue, Alaska, is bordered on three sides by water and has two runways. The airport has two runways: the main runway, 9/27, and the crosswind runway, 17/35. This effort focuses on Runway 9/27, which is 5,900 feet long and 150 feet wide with an asphalt concrete pavement and gravel shoulders. This runway was previously designated 8/26. Figure 8 is a diagram of the airport with the runways labelled.

Researchers reviewed the following documents for Kotzebue, which are summarized in the following sections:

- Ellerbe Associates Inc. (Fairbanks, Alaska Office). (1983). *Kotzebue Airport Extension Feasibility Study*

- TRA/Farr. (1986). *Ralph Wein Memorial Airport, Kotzebue, Alaska, Airport Master Plan*
- *Kotzebue Airport Master Plan* (1988)
- Northern Region Materials Section, State of Alaska. (1990). *Engineering Geology and Soils Report Kotzebue East-West Resurfacing*
- R&M Consultants. (1992). *Kotzebue Runway Resurfacing Preliminary Design Study Report*
- Northern Region Materials Section, State of Alaska. (1993). *Kotzebue Airport E-W Runway Resurfacing Supplemental Report No. 1*
- R&M Consultants. (1993). *Ralph Wien Memorial Airport Runway Resurfacing Final Design Study Report*
- *Kotzebue Airport Master Plan* (1998)
- Shannon & Wilson, Inc. (2001). *Kotzebue Airport Potential Borrow Site Investigation—Geotechnical Study*
- Northern Region Materials Section, State of Alaska. (2010). *Kotzebue Airport and Safety Area Improvements Runway 9-27 Rehabilitation*
- USKH, Inc. (2010b). *Ralph Wien Memorial Airport Pavement Condition and Repair Memo*
- USKH, Inc. (2010a). *Kotzebue Airport and Safety Area Improvements*—Stage I
- Alaska DOT&PF (2010). *Kotzebue Airport and Safety Area Improvements As-Built Plan*
- *Alaska Airport Pavement Inspection Reports* (2004, 2007, 2010, 2013 and 2016)
- *Kotzebue Airport Layout Plans* (1999 and 2011)

Figure 8. Kotzebue Airport

3.1 CONSTRUCTION AND REHABILITATION HISTORY

Runway 9/27 was constructed in the early 1950s and originally was 3,750 feet long with a gravel surface. Approximately 1,600 feet of the east end of the runway was constructed on fill material placed in a lagoon. The runway was extended by 1,250 feet to the east in 1964. In 1968, the runway was further extended an additional 1,000 feet to the east shore of the lagoon. The entire runway was paved in 1970. During paving, polystyrene insulation board was placed at a depth of 30 inches from station 3+00 to station 22+00 (measured from east of the west end) to protect the permafrost and limit ground settlement associated with permafrost degradation. In 1980, stations 22+00 to 26+00 were reconstructed with a pavement section that included the polystyrene insulation board (R&M Consultants, 1992). The 1980 pavement section consisted of 3 inches of asphalt concrete, 8 inches of base, and nearly 20 inches of subbase. The runway was again overlaid in 1994 and the asphalt thickness was increased to 4 inches. The localized settlement areas were repaired in 2005. In the 2010 runway rehabilitation project, the top 4 inches of existing asphalt was reclaimed and used as a base layer, and a 4-inch thickness of new asphalt was placed on the surface.

3.2 GEOTECHNICAL AND PERMAFROST CONDITIONS

R&M Consultants performed an extensive geotechnical investigation in 1992 (R&M Consultants, 1992, 1993). The report divides the subsurface conditions beneath the airport pavements into three areas: beach, spit, and lagoon (Figure 9). The embankment ranged from about 3 to 5 feet thick in the beach and spit areas, to 12 to 15 feet thick where the runway crosses the lagoon.

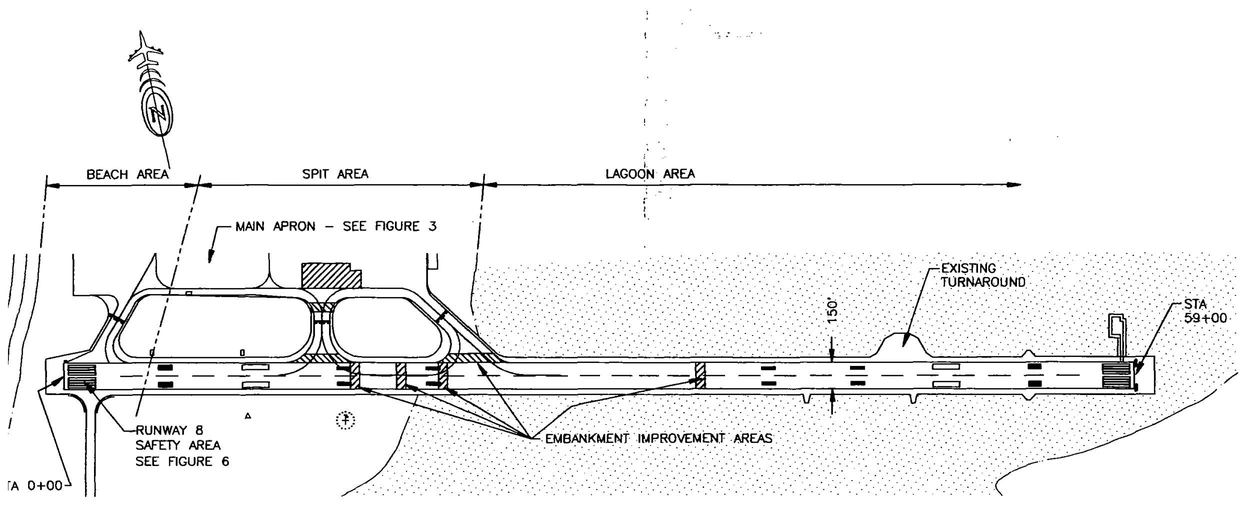

Figure 9. Three Geotechnical Areas within Runway 9/27 at Kotzebue Airport (R&M Consultants, 1992)

The R&M report (1992) describes the foundation condition in each area:

> The "beach" area extends from the west edge of the airport to the west diagonal taxiway. The foundation (native) soils generally consisted of silty sands and gravels randomly interlayered with sandy silts and organic silts and sands. There also appeared to be an

intermittent thin layer of peat to organic silt between the embankment fill and native soil. The top of permafrost is variable, but typically about 10 feet below the surface. Relative to the rest of the airport, the foundation soils in the west beach area appeared to be fairly well-draining and thaw-stable, at least within the apparent seasonal active layer which was estimated to range from about 8 ft. to 10 ft. under pavements not insulated, to about 5 ft. under insulated pavements.

The foundation in the "spit" generally consists of two to seven feet of interlayered peat and organic silt (or occasionally sand), over interlayered sand and silt occasionally intermixed with some organics and gravels. Under the insulated pavements, the top of permafrost appeared to generally coincide with the top of native ground, typically about one-half to 1 foot below the insulation. The embankment in this area appeared very thaw-unstable and loose soils. Board insulation was originally (in 1970) installed in this area to improve the pavement stability. From 1970 to the 1992 inspection, no significant areas of thaw settlements or consolidation were observed.

The "lagoon" area extends from east taxiway to the runway east end. The foundation soils generally consist of silt with variable organic materials, over silt with some sand. Based on test holes from 1968, no permafrost was identified within at least 40 feet of the current pavement surface.

Permafrost is discontinuous in the airport vicinity and the extent of it is decreasing. The subsurface thermal regime at the airport is complex owing to the variety of soil types and water table and temperature conditions. The active layer is significantly less where insulation board are buried, or fine-grained soils are present.

A geotechnical report by the Alaska DOT&PF Northern Region Materials Section (NRMS) (2010) presented the results of a geotechnical investigation conducted in 2009 on Runway 9/27. This investigation evaluated subsurface conditions in problem areas and assessed asphalt and base materials. Differential settlement was observed in different areas within the western half of Runway 9/27, mostly in the spit area. The investigation found organic silt with ice beneath the runway embankment in the spit area. The investigation also found that foundation soil was generally frozen, but some zones were within active layers and identified the potential for loss of soil strength due to thawing. It appeared that earlier earthwork to partially remove organic silt in some sections and to add insulation reduced, but did not eliminate, differential settlement of the runway.

Thirteen pavement bores drilled on Runway 9/27 indicated varying asphalt thickness along the runway. The base course thickness varied from 6 to 12 inches, which was thicker than shown on the record drawings. The runway embankment depth ranged from 3 to 7.5 feet. According to the geotechnical report, seven test holes were frozen to the depth drilled, thus neither top of permafrost nor depth of seasonal frost could be determined. The 2010 report also indicated that "Of the six test holes with a thawed zone, seasonal frost reached depths of 3 to 7 feet. In one of the holes, the thawed zone extended a foot into silt foundation soils, beneath the fill section with 6 inches of insulation" (NRMS, 2010). The 2010 study further indicated that seasonal thaw reached thermally unstable and frost-susceptible foundation soils to a depth of up to 3 to 4 feet below the insulation between Stations 14+00 to 26+00 in the spit section of the runway.

The NRMS report suggested that the thickness and configuration of insulation boards appeared to be inadequate to keep foundation soils frozen in all areas. Despite their initial purpose of keeping the subgrade frozen, it is likely that the insulation boards also reduced ground heat loss in winter. Due to past construction-related ground disturbances and proximity to water bodies, the freezing regime of the subsurface has been significantly changed. The report concluded that the combination of insulation and the 4.5-foot-thick embankment has not been sufficient to maintain a frozen foundation. Therefore, a partial removal of the organic silt layer approach that has been used in the past will likely not fix the settlement problems. The report stated that "long term stability would require subexcavating the organic silts down to sandy soils and replacing with compacted structural fill. This may require excavations of 8 to 12 feet in places" (NRMS, 2010).

The *Kotzebue Airport and Safety Area Improvements—Stage I* technical memorandum (USKH, 2010a) indicates that in the spit section where the subgrade consists of peat and silt layers with high ice content, thawing caused subsequent settlement. The study suggests that "the insulation in this area is more than likely severely degraded because of the vertical movement resulting from the underlying thaw consolidation. This degradation of the insulation allows the thaw penetration to extend even deeper and perpetuates the thaw penetration." In sections of the pavement where the subgrade is relatively thaw stable, the thaw penetration had little adverse effect.

3.3 PAVEMENT CONDITION

The 1993 study by R&M Consultants describes the following distresses across Runway 9/27:

> From west taxiway to approximately the center taxiway, the predominant failure was alligator and map cracking with major thermal transverse cracking extending across the runway width. From center taxiway to the diagonal taxiway, the runway is rough and there exists differential settlements, extensive cracking in addition to repaired areas from 15 to 35 feet long extending the runway width.

Frost was visible during the geotechnical sampling. The 1993 report concluded that high ground water levels, subgrade material with high natural moisture content within the thaw zone, and visible frost have contributed to the poor performance of the embankment. Repeated freeze-thaw cycles might have compromised the strength of the embankment material. The report concluded that the high silt content of the subgrade materials makes them unsuitable for use in the upper portion of the embankment.

USKH, Inc. performed a pavement condition and rehabilitation evaluation in 2010. They observed four predominant distresses including moderate block cracking near each end of the runway, surface raveling and weathering, severe transverse cracking at a spacing of approximately 200 feet, and pavement depressions and patching between in the spit area. Some of the transverse cracks are wide and extend through the underlying subbase and embankment layers. These types of transverse cracks are likely due to the permafrost-related volume changes in the embankment that reflected up to the surface.

The USKH technical memorandum (2010a) indicates that there are no indications of structural failure for most of the runway. However, depressions due to thawing subgrade have repeatedly developed in the spit section, due to the subgrade underlain by ice rich peat and silt layers of permafrost. Alligator cracking and rutting were also reported in the spit section.

PCI survey reports are available for the following years: 2004, 2007, 2010, 2013, and 2016. These reports summarized the PCI values for different pavement sections within the airport. Details of the stress type and severity are not included in the reports. Figure 10 shows the PCI values at those dates. The PCI dropped by 45 points nearly 10 years after the 1994 construction. The 2005 rehabilitation must have restored the PCI, but the PCI dropped to 55 in 2009. The PCI increased to 95 in 2013, approximately one year after the 2012 rehabilitation.

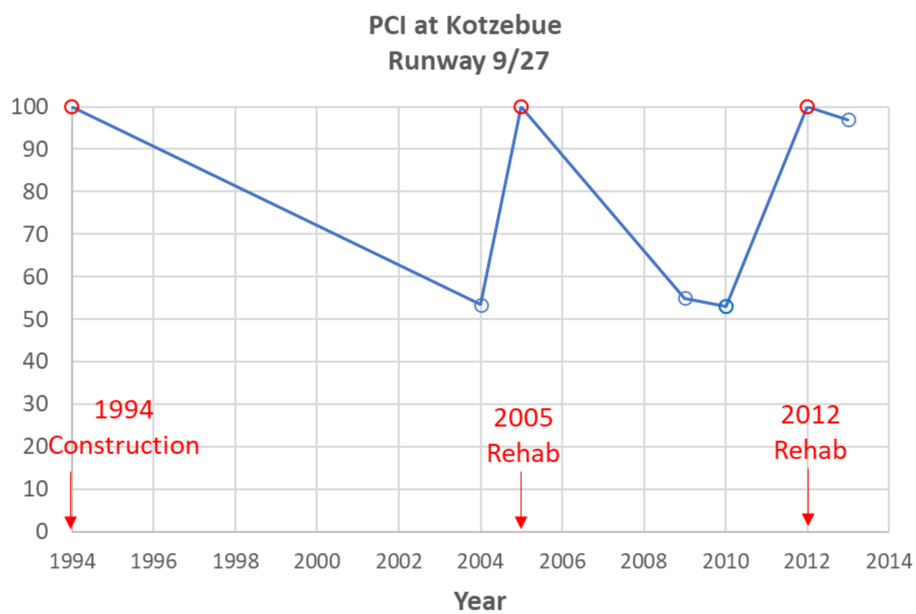

Figure 10. The PCI Values for Runway 9/27 at Kotzebue Airport

3.4 PAVEMENT DESIGN

The 1970 construction of Runway 9/27 was based on accommodating these loads: S (100,000 lb), D (128,000 lb), 2S (162,000 lb) and 2D (240,000 lb). (Kotzebue Airport Master Plan, 1988).

R&M Consultants provided a recommendation for pavement rehabilitation design in 1992. In the R&M report, the runway was classified as Design Group III (wingspan less than 118 feet) facility and designed to accommodate approach Category C aircraft (approach speed less than 141 knots). The pavement design was based on the Asphalt Institute's Design Manual *Full- Depth Asphalt Pavements of Air Carrier Airports, Third Edition* (Asphalt Institute, 1987). For comparison purposes, the FAA AC 150/5320-6C pavement design method was also evaluated. The design life was 20 years. Only the number of large aircraft operations of major air carriers was considered for the pavement design. The Asphalt Institute method determined that a 4-inch thickness of asphalt concrete underlain by 8 inches of base course and 8 inches of subbase was adequate. The FAA method indicated 4 inches of asphalt in critical areas, and 3 inches of asphalt in noncritical areas, underlain by 7.5 inches of base course. The design recommendation by R&M Consultants (1992) was to pave the runway keel from centerline to 25 feet left and right with 4 inches of asphalt, and to use a varying thickness of 3-4 inches asphalt for areas outside the keel (25 to 50 feet offset from centerline). A CBR of 20 was assigned to the subbase and subgrade. The design report indicates this was considered a conservative value.

USKH (2010b) provided recommendations for stabilizing the pavement structure in the area of reoccurring settlements. The rehabilitation recommendations involved excavating the existing pavement structure to frozen soil or to a minimum depth of 6.5 feet. A geotextile separation fabric was recommended to be placed on the frozen soil, followed by the placement of thaw-stable subbase. The subbase would be brought up to the elevation where 6 inches of high-density polystyrene insulation would be placed, with the top elevation set at 4.5 feet below finished grade. The recommended pavement section was subbase, 8 inches of base course, and 4 inches of asphalt pavement. The report concluded that "the additional 2 in. of insulation and thaw-stable subbase to a total depth of 6.5 feet will keep the thaw penetration within the pavement section, and the portion of the section above the insulation will match the remaining existing runway pavement section, which structurally, has performed very well."

Researchers obtained historical traffic data for OTZ from 2014 to 2019 from the IOAA Tool managed by MITRE Corp. for the FAA. Types of data obtained are similar to those obtained for OME (see section 2.5).

3.5 INFORMATION FROM INTERVIEWS

Representatives from Alaska DOT&PF, FAA Alaskan Region, FAA William J. Hughes Technical Center, and ARA held a teleconference on November 5, 2020, to discuss the pavement issues identified in the reports on Kotzebue Airport. The discussions corroborated the findings from the reports that the insulation boards were not effective. They found that timing and sequence of construction were key factors for an effective insulation. Appendix A provides the complete interview.

3.6 KOTZEBUE RUNWAY 9/27 DESIGN REVIEW

This section reviews the 1994 and 2010 pavement rehabilitation projects. The 1994 designed section consisted of 4 inches of asphalt, 8 inches of crushed aggregate base, and approximately 20 inches subbase. Insulation boards were at 30 to 40 inches below the surface on top of the subgrade. The 2010 section consisted of 4 inches of asphalt, 4 inches of reclaimed base, and 4 inches of crushed aggregate base over the exiting subbase. The following sections (a) identify the design assumptions and input parameters as used by the design engineers and as required by the FAA standards, and (b) reproduce the design.

3.6.1 Re-creation of 1994 Design at Kotzebue

The design of this runway was based on AC 150/5320-6C on the premise that the subgrade was frozen and would remain frozen. Insulating boards were used in an effort to protect the permafrost. The design engineer assumed CBR 20 for both subbase and subgrade. The flexible pavement design in AC 150/5320-6C is based on the CBR method of design. Figure 11 shows the TDD from 1964 to 2020. The design TDD index, defined as the average of the three warmest summers from

1964 to 1994 (year of design), was 2,596 °F-days, which equates to 12 feet of thaw penetration. The insulation boards were installed to prevent the frost-susceptible permafrost from thawing. If the insulation performs as intended, the subgrade can be assumed frozen and considered conservatively as CBR 20 in the pavement design. However, in fact the insulation boards proved to be ineffective for containing the permafrost, and thawing has penetrated below the boards into the subgrade. Researchers reproduced the pavement design based on two opposing scenarios:

1. Insulation board functioning: Design is based on the premise that the subgrade is frozen with CBR 20.

2. Insulation board not functioning: Design is based on the RSS method to account for lower load-carrying capacity due to subgrade thawing. Although the RSS is not the most appropriate method for this runway due to the presence of non-uniform and frost-susceptible subgrade materials, this design scenario attempts to check whether the existing pavement can support the traffic if the subgrade is thawed given the failed insulation. Data on CBR during the thawing period is not available. AC 150/5320-6C is silent about how much reduction should be considered. The Berg report provides design charts for calculating the required pavement thickness in the RSS method (Berg, 1974). However, charts are only provided for a few specific aircraft types, and no chart is available for the B737. Researchers used recommendations from AC 150/5320-6D to determine reduced subgrade strength. Given the high percentage of fine material in the subgrade, the most prevalent material is FG-3. A reduced subgrade CBR value of 4 was adopted from AC 150/5320-6D, Table 3-1 (FAA, 1995).

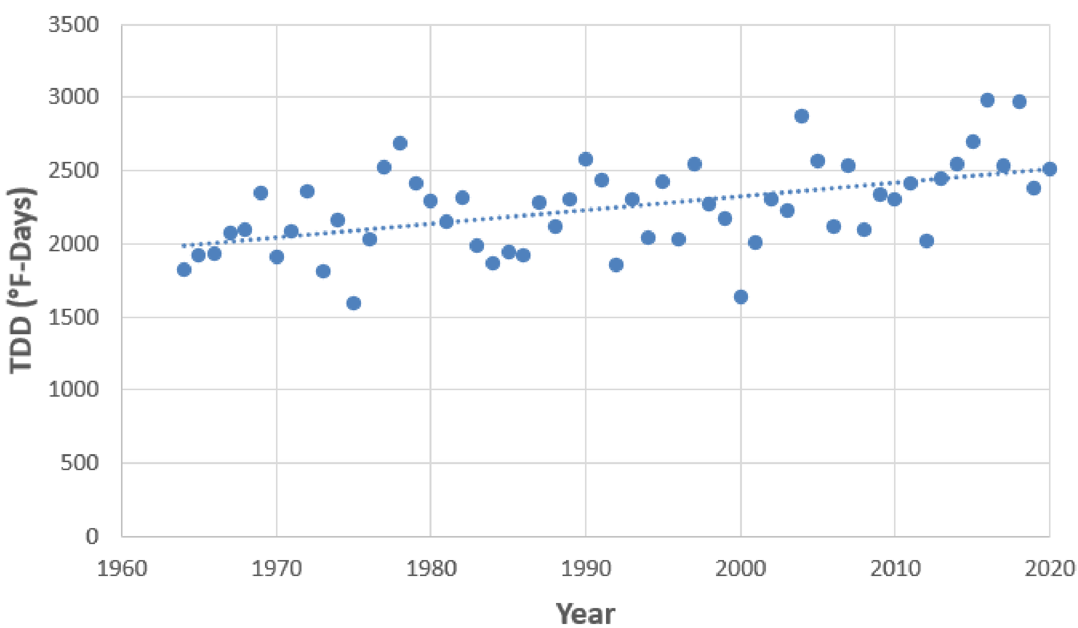

Figure 11. Change in Thawing-Degree Days in Kotzebue 1970–2020
(Data from http://weather.gov/aprfc [Alaska-Pacific River Forecast Center])

AC 150/5320-6C also uses the concept of design aircraft. Table 11 presents the design aircraft mix and the calculated equivalent annual departures of the design aircraft (B737-200). Table 12 presents a design summary based on the two scenarios above. The total required thickness was determined from design charts in Figure 3-4 of AC 150/5320-6C (FAA, 1978). Calculations show that the existing pavement structure is sufficient to support the design traffic if the insulation boards function properly. Note that the pavement structure does not have a stabilized base, although guidance at the time of design required one for the design traffic. Considering Design Scenario 2, where the insulation is not functioning, the pavement structure is adequate in most areas to support the design traffic assuming reduced subgrade strength. However, due to variable soil conditions along the length of the runway and varying insulation performance, differential settlement can be expected during the life of the pavement.

Table 11. Design Traffic Calculation for Kotzebue Airport

Aircraft	Gear Type	Annual Departures	Dual Gear Departures	Take-off Weight (lb)	Wheel Load (lb)	Design Wheel Load (lb)	Equivalent Annual Departures
B727-100	D	225	225	160,000	38,000	23,750	945
B737-200	D	1,475	1,475	100,000	23,750	23,750	1,475
B737-400	D	340	340	138,500	32,894	23,750	953
L-100-20	2S	185	148	155,000	36,813	23,750	665
DC-6	D	525	525	104,000	24,700	23,750	594
Total							4,632

Table 12. Design Summary for Kotzebue Airport Based on AC 150/5320-6C

Design Scenario	Frost Protection Method	Subgrade CBR	Total Thickness Required (in.)	Design Thickness Required (in.)		
				AC	Base	Subbase
Insulation functioning	Insulation boards	20 (Frozen)	11.5	4.0	7.5	0
Insulation not functioning	Reduced subgrade strength	4 (FG-3)	34	4.0	8.0	22.0

3.6.2 Design Based on AC 150/5320-6G

Researchers re-created the pavement design based on AC 150/5320-6G for both the fully functioning insulation boards and failed insulation scenarios. The pavement structure was checked against the RSS method in the failed insulation scenario. For the RSS method, AC 150/5320-6G states that the pavement design should be determined using 50% of the design subgrade strength, or the subgrade strength during frost-thawing periods. It is not clear from the AC whether "design subgrade strength" indicates the frozen subgrade condition or other conditions. Although 50% of frozen subgrade strength seems high, researchers nevertheless used this value (CBR 10) to re-create the design in FAARFIELD. The pavement thickness was determined using both the design traffic and the actual average traffic from 2014 through 2019 (Table 13). The actual traffic is approximately two-thirds of the design traffic. The traffic mix includes B737 aircraft with a gross weight greater than 100,000 lb, and AC 150/5320-6G requires a stabilized base for design of pavements serving aircraft heavier than 100,000 lb. For this reason, researchers considered both stabilized and granular base design scenarios.

Figure 12 shows the required pavement thickness computed by FAARFIELD using design traffic for the failed insulation condition. The complete FAARFIELD pavement design reports are provided in Appendix B.

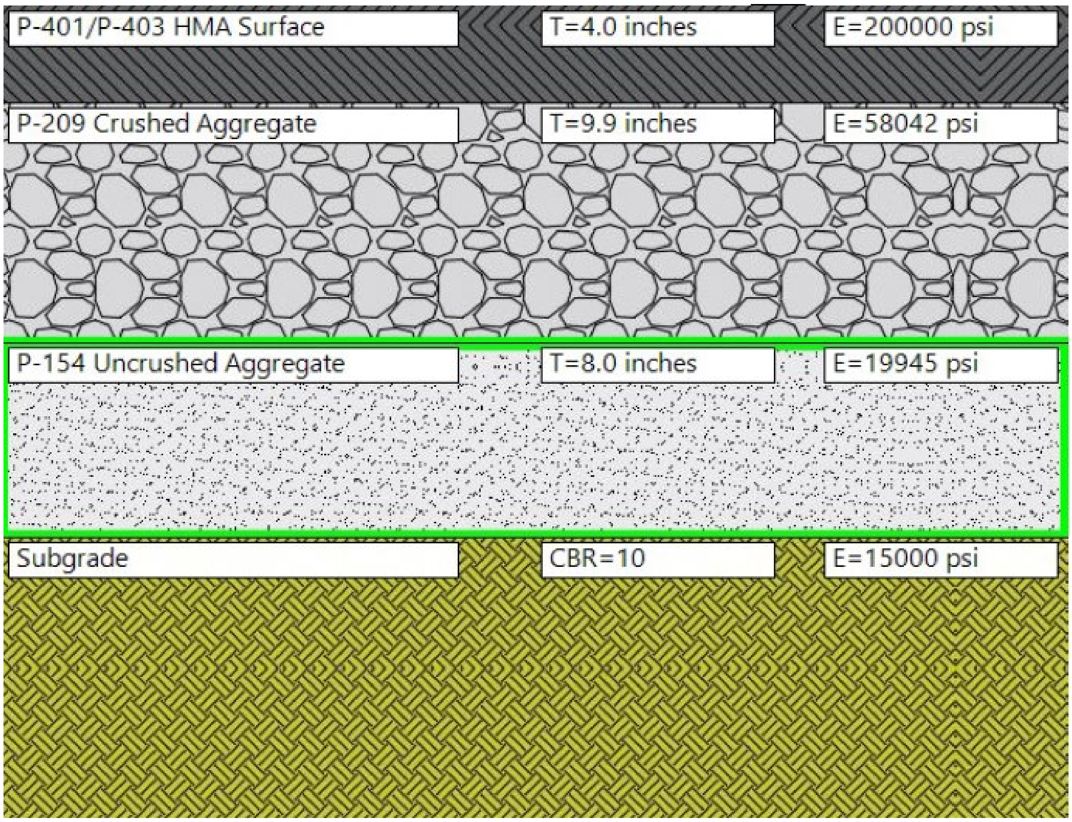

Figure 12. FAARFIELD Design Based on AC 150/5320-6G Using Design Traffic and Frozen Subgrade CBR at Kotzebue Airport

Table 14 summarizes required thicknesses for both the functioning and failed insulation conditions, assuming both the original design traffic (Table 11) and the actual traffic (Table 13). FAARFIELD results indicate that while the total thickness of the 1993 pavement is adequate to support the design traffic, the granular base layer thickness is not sufficient. The 2010 section with a 12-inch base layer is adequate to support the actual traffic under both failed and functioning insulation conditions. compares the existing 1993 pavement section to the required pavement thickness based on ACs 150/5320-6C (FAA, 1978) and 150/5320-6G (FAA, 1995). Table 16 shows the existing 2010 pavement section and the required pavement thickness to support the actual traffic.

Table 13. Average Annual Traffic (2014–2019) at Kotzebue Runway 9/27

Aircraft	Annual Departure
B737-200	256
B737-300	100
B737-400	38
B737-700	784
B737-800	130
C-130	117
LJ45	46
PA31	112
DC93	68

Table 14. Design Summary Based on AC 150/5320-6G Inputs at Kotzebue Airport

Design Method	Design Scenario	Traffic	Subgrade CBR	Required Thickness (in.)			Base Type
				Asphalt	Base	Subbase	
AC 150/5320-6G	Insulation functioning	Design Traffic	20.0	4.0	9.9	4.0	Granular
				4.0	5.2	4.0	HMA Stabilized
		Actual Traffic		4.0	10.5	4.0	Granular
				4.0	5.7	4.0	HMA Stabilized
	Insulation not functioning	Design Traffic	10.0	4.0	9.9	8.0	Granular
				4.0	5.2	8.3	HMA Stabilized
		Actual Traffic		4.0	10.5	7.7	Granular
				4.0	5.7	7.3	HMA Stabilized

Table 15. Comparison of Required and Existing 1993 Pavement Thickness at Kotzebue Airport

Property	Existing (1993)	AC 150/5320-6C (Design Traffic)		AC 150/5320-6G (Design Traffic)	
		Failed Insulation	Functioning Insulation	Failed Insulation	Functioning Insulation
Surface	4 in. Asphalt	4 in. P-401	4 in. P-401	4 in. P-401	4 in. P-401
Base	8 in. Crushed Aggregate	8 in. P-209	7.5 in. P-209	9.9 in. P-209	9.9 in. P-209
Subbase	18-28 in. Fill	22 in. P-154	0 in. P-154	8 in. P-154	4 in. P-154
Subgrade	CBR=20 (Frozen)	CBR=4 (FG-3)	CBR=20 (Frozen)	CBR=10*	CBR=20 (Frozen)

* 50% of design CBR

Table 16. Comparison of Required and Existing 2010 Pavement Thickness at Kotzebue Airport

Property	Existing (2010)	AC 150/5320-6G (Actual Traffic)	
		Failed Insulation	Functioning Insulation
Surface	4 in. Asphalt	4 in. P-401	4 in. P-401
Base	12 in. Crushed Aggregate	10.5 in. P-209	10.5 in. P-209
Subbase	18-28 in. Fill	7.7 in. P-154	4 in. P-154
Subgrade	CBR=3.4	CBR=10*	CBR=20

* 50% of design CBR

4. WILEY POST/WILL ROGERS MEMORIAL AIRPORT, UTQIAGVIK (FORMERLY BARROW)

Wiley Post/Will Rogers Memorial Airport (BRW) in Utqiagvik (formerly Barrow) is the nation's northernmost airport, located on the north coast of Alaska. Utqiagvik is situated in the Arctic Climatic Zone of Alaska with moderate summer temperatures and arctic winter temperatures. Figure 13 shows the airport layout. It has one runway, designated 8/26, in the east-west direction.

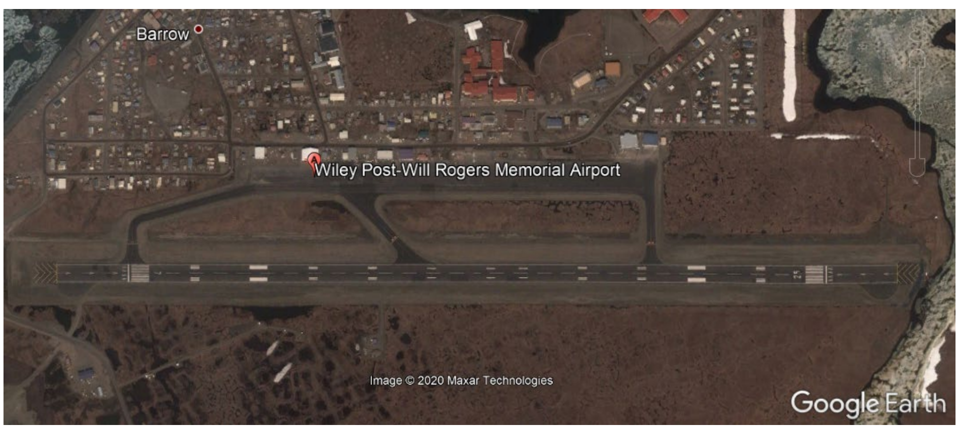

Figure 13. Utqiagvik (BRW) Airport Diagram

Researchers reviewed the following documents for BRW, which are summarized in the following sections:

- R & M Consultants, Inc. (1980). *Design Documents Paving and Grading Improvements*
- Northern Region Design and Construction, Alaska DOT&PF. (1984). *Engineering Geology and Soils Report*
- Alaska DOT&PF, Northern Region Design and Construction. (1990). *Safety Area and Apron Expansion at Barrow*
- Alaska DOT&PF. (2000). *Wiley Post-Will Rogers Memorial Airport Master Plan 2000*.
- Alaska DOT&PF, Northern Region Design and Construction. (2001). *Supplemental Geotechnical Report: Barrow Airport Runway and Apron Paving/Apron Expansion*

- Alaska DOT&PF, Design & Engineering Services. (2002). *Barrow Runway Pavement Design*
- AIP. (2009). *Runway and Apron Paving As-Built Plan*
- Alaska DOT&PF, Northern Region. (2014). *Geotechnical Investigation Report Barrow Airport Building Sites and Apron Expansion Project*
- Golder Associates Inc. (2016). *Maintenance Facility Geotechnical Recommendations, Barrow, Alaska*
- Shannon & Wilson, Inc. (2017). *Geotechnical Studies Barrow Airport Pavement Overlay*
- Alaska DOT&PF Airport Pavement Inspection Reports (2002, 2008, 2009, 2012, 2015, and 2018)

4.1 CONSTRUCTION AND REHABILITATION HISTORY

The BRW airfield was originally constructed in 1964. The original runway was designated 6/24, and its embankment consisted of an average of 7.25 feet of fill material and asphaltic surface. This runway was extended in 1968, and the extension had an average of 6.1 feet of total embankment thickness. The runway was overlaid with 2 inches of asphalt concrete layer in 1983. In 2001, the Alaska DOT&PF proposed improvements including construction of a new runway 125 feet south of the original runway. The new runway, initially designated Runway 7/25, was completed in 2012 and is 150 feet wide and 7,100 feet long with an asphalt surface and unpaved runway safety areas. This runway was later redesignated Runway 8/26. The new runway pavement consists of 3 inches of asphalt concrete surface over 6 inches of RAP base course on borrow embankment. The record drawings show that a 1.5- to 3-inch asphalt mill and overlay took place in 2014. A geotechnical investigation by Shannon & Wilson (2017) observed 6 to 11 inches of asphalt and fill to depths of 8 feet to 11 feet below ground surface at nine boring locations within the runway.

4.2 GEOTECHNICAL AND PERMAFROST CONDITIONS

Utqiagvik is in the Arctic Climate Zone. The region's climate has a marine influence in the summer months when the ice pack diminishes. The region's climate is more continental in the winter months, resulting in relatively cold and dry winters (Alaska DOT&PF Northern Region, 2014).

The region lies within a zone of continuous permafrost. A report prepared by R&M Consultants in 1980 included findings of an investigation to evaluate available local materials for use as embankment, base, and surface course pavement layers. Sample materials from three sources around the airport were identified as "gravelly sand containing very little coarse granular material or silt-sized binder." The report concluded that the materials were not suitable for crushing (to meet aggregate fracture face specifications) due to the lack of large aggregate. Therefore, it was recommended that standard surface aggregate specifications be modified for this project to allow the use of local material without crushing. The recommendation was based on the idea that crushing of the aggregate does not significantly increase the stability of the surfacing material.

The 1980 report used the Modified Berggren Method (see Berg, 1974) to calculate depths of freeze and thaw. The calculations suggested that for a 10-year return period that was selected for design, a fill of non-frost-susceptible material to a depth of 7.4 feet is required. Calculations also anticipated that 8.2 feet of thaw will occur on a 30-year return interval.

In 1984, the Materials Section of the Alaska DOT&PF performed a geotechnical investigation on a potential material site at BRW (Northern Region Design and Construction, 1984). The purpose of the investigation was to obtain information for a project that required material for a base course and for paving aggregate. The 1984 report found that the entire site was underlain by permafrost. The surface organic material thawed only in summer and refroze to permafrost during the winter. Thawed, saturated material was observed within the permafrost in some boreholes. Free ice was noted in all materials extracted from the boreholes.

In 2001, the Alaska DOT&PF Northern Region Engineering Geology Section (NREGS) conducted an additional investigation of the material source located on airport property to determine availability and quantity of materials for construction of the new runway (Northern Region Design and Construction, 2001). They performed drilling on the material site located approximately one-half mile southwest of the west end of the runway. The investigations were carried out in the winter (November) when the ground was frozen throughout. Underlying permafrost was found at varying depths. The report stated that "One test hole showed no separation between the seasonal frost and the permafrost. Two holes, drilled to depths of 20 and 24 feet, showed no permafrost below the seasonal frost. In the test trenches dug in July 2000, depth to frozen ground ranged from 3 to 3.6 feet. Frozen soils extended to the bottoms of all trenches. Additionally, in shallow pits dug into the face of the current excavation areas, frozen soils were found under approximately 1.5 feet of unfrozen soils."

A geotechnical investigation in 2014 (Alaska DOT&PF Northern Region, 2014) hypothesized that embankment consolidation and frost heave/thaw settlement can occur due to poorly compacted embankment soil. It also calculated design FDD and TDD for the Utqiagvik area by using the information from the three coldest winters and three warmest summers between 1981 and 2010:

- Design FDD: 9,160 °F-days
- Design TDD: 1,080 °F-days

In 2016, Golder Associates Inc. performed a geotechnical investigation for the Maintenance and Operation (M&O) building at BRW (Golder Associates, 2016). Data from the University of Alaska, Scenarios Network for Alaska and Arctic Planning (SNAP) (https://uaf-snap.org/) were used to forecast thermal data for the 25- to 30-year service life. Engineers calculated the depth of thaw based on forecast 2040-2049 design TDD for a pad fill section with 0, 4, and 6 inches of rigid insulation near the pre-fill tundra surface. To investigate the soil properties and ground thermal states within the building footprint, six geotechnical boreholes were advanced to depths ranging from 20 to 40 feet below ground surface over a frozen tundra surface in the footprint of the M&O building. The report indicated that "the active layer in the undisturbed tundra area within the M&O building pad area is approximately 2 feet thick. Below the active layer bonded continuous permafrost was encountered in all six boreholes. Immediately below the inferred active layer, a very ice rich organic and mineral soil and massive ice was encountered. Ice content of the recovered samples was in excess of 50-percent by volume by visual estimate in this zone."

A geotechnical investigation in 2017 indicated that permafrost was encountered between 7.7 feet and 10 feet below the ground surface. The 2017 study concluded that "seasonal thaw depths were generally contained within the fill embankment and extended to within 1 foot of the subgrade, except for one boring (17-02) which was drilled in a relatively thin area of the embankment." Shannon & Wilson also drilled nine borings through the embankment in unpaved areas. They observed fill depths between 5.5 to 12 feet below ground surface. Permafrost was encountered between 4.5 and 8 feet below ground surface. They encountered ice 4 to 12 inches thick in three of the borings, at depths of 8 feet to 15 feet below ground surface. They did not observe a consistent groundwater level in the borings. They observed wet thawed samples above the permafrost and concluded that "local perched groundwater tables are likely throughout the embankment when soils are thawed." The investigation indicated that seasonal thawing is generally controlled in the runway embankment fill and does not extend into the ice-rich subgrade. The study concluded that subgrade thawing into ice-rich soils is not likely contributing to the observed settlements. The report suggests that subgrade thaw might have occurred during construction and could have contributed to the initial surface settlement, and that future thaw could occur during unusually warm periods or due to warming trends. (Shannon & Wilson, 2017)

4.3 PAVEMENT DISTRESSES

The 1980 investigation by R&M indicated that the original runway (Runway 6/24) section exhibited little differential settlement (R&M Consultants, 1980). However, the extended runway area was in poor condition, exhibiting asphalt cracking and significant differential settlement as a result of thawing of ice-rich soils beneath the embankment.

As indicated in section 4.1, the original runway was replaced in 2012. Shannon & Wilson (2017) performed a geotechnical survey in 2014 on the new runway 7/25 (later redesignated 8/26), which had just opened two years previously. The 2014 survey identified "a prominent dip at the west end of the runway with settlement near 8 inches, relative to constructed elevations, that included the centerline of the runway out to the runway edges." The study related the observed runway embankment differential movement to these factors:

- "consolidation of poorly compacted wet embankment fills,
- construction and placement of embankment fill over more than one construction season resulting in frost heaving and ice growth of intervening winters,
- continued frost heaving of wet frost susceptible embankment soils and thaw-weakening of high-moisture-content soils subjected to freeze-thaw cycles, and
- subgrade thawing during and post construction followed by refreezing and possible ice growth."

(Shannon & Wilson, 2017)

The geotechnical study by Shannon & Wilson reviewed the PCI data and identified three factors that caused pavement degradation: "1) the performance of embankment and subgrade as it relates to the surface depressions and need for patching; 2) climate conditions (extreme temperature variations) which impact the frequency and severity of longitudinal and transverse cracking; and 3) the quality of the AC surface (observed raveling)" (Shannon & Wilson, 2017).

4.4 PAVEMENT DESIGN

The 2000 Airport Master Plan for BRW (Alaska DOT&PF, 2000) indicates the design aircraft is the C-130 (Airport Reference Code C-IV). The master plan implies that other aircraft use the airport, but no other aircraft types are listed. Table 17 provides the demand forecast.

Table 17. Aviation Demand Forecasts (Alaska DOT & PF, 2000)

Operations	1998	2005	2010	2020
Enplaned Passengers	39,467	45,335	50,054	61,015
Cargo	12,000	14,362	16,329	21,107
Commercial	12,050	12,376	12,634	13,226
General Aviation	3,900	4,238	4,514	5,173
Military	50	50	50	50
Total Operations	16,000	16,663	17,198	18,450

The Design and Engineering Services of the Alaska DOT&PF performed pavement analysis and design in 2002 as part of the runway reconstruction project (Alaska DOT Design & Engineering Services, 2002). The designers analyzed the pavement using a computer software program called "Alaska Overlay Design Program for Flexible Highway Pavements" (AKOD98). The program is based on a mechanistic approach and employs the ELSYM5 (Kopperman, Tiller, & Mingstan, 1985) layered elastic analysis program to calculate stresses and strains. The AKOD98 program used the critical strains to calculate the accumulated rutting or fatigue cracking. The proposed pavement consisted of 3 inches of asphalt concrete, 6 inches of recycled base, and a minimum of 84 inches of borrow material to be used as subbase/subgrade. Designers used the program's default modulus and Poisson's ratio for each layer and adjusted the values for winter conditions. Subbase modulus values of 30 ksi and 90 ksi were used for summer and winter, respectively. The B737-400 was the design aircraft with base 1,900 annual operations in 2002 and anticipated 2,937 annual operations by 2020. The design was based on the average of two values. Designers assumed 62.5% of the design aircraft operations would occur in the summer and 37.5% in the winter. The minor effects of the B727-200 (180 annual operations) and C-130 (50 annual operations) were also considered in the design.

There were a number of MoS involved in this project including the use of RAP as base course and deletion of crushing requirements in item P-401. The complete list of modifications is shown in Table 18.

Researchers obtained traffic data for BRW from 2014 to 2019 from the IOAA Tool managed by MITRE Corp. for the FAA. Types of data obtained are similar to those obtained for OME (see section 2.5).

4.5 INFORMATION FROM INTERVIEWS

Researchers submitted a set of questions to the Alaska DOT&PF and the FAA Alaskan Region regarding the runway reconstruction project at BRW, which began in 2001. The responses to these questions helped to clarify the project timeline and design process. Interviewees indicated that the Alaska DOT&PF decided to perform a major rehabilitation due to the recurrence of pavement settlement in the original runway (6/24). However, because there was no second runway to shift the traffic to during construction, the new runway (17/25) was built south of the existing runway. Installation of a MALSR (Medium-Intensity Approach Lighting System with Runway Alignment Indicator) could have caused permafrost degradation on one end of the project after completion. Appendix A provides the complete interview.

4.6 DESIGN REVIEW OF BRW RUNWAY 7/25

This section reviews the 2002 project design for pavement reconstruction at BRW. The designed section consisted of 3 inches of asphalt, 6 inches of RAP base, and approximately 84 inches of fill subbase. The original design was not based on AC 150/5320-6D, the FAA standard in effect at the time. Instead, design engineers used the AKOD98 computer program for pavement design. The AKOD98 program uses a mechanistic approach to calculate critical pavement strains under the design aircraft loads.

Table 19 lists the design traffic. The design considered seasonal variation of the layer modulus to account for increased modulus values in the winter. Modulus values of 30 ksi and 90 ksi were used for the subgrade in the summer and winter, respectively. While the design report does not specify which frost protection method was used, it appears that the CFP method was likely used to protect the permafrost. The base and subbase courses consisted of non-frost susceptible material according to the design documents. However, due to the MoS implemented on the embankment fill materials (see Table 18), modified specifications may have deviated from FAA requirements for non-frost-susceptible material. The design report does not state criteria for acceptable levels of pavement distress or thaw depth penetration.

Table 18. List of MoS in 2002 BRW Pavement Reconstruction

Item P-152 Excavation and Embankment		
1.1, Description		
1.2, g.	Paragraph Changed	Reduces the amount of silt allowed in borrow material.
2.1 Construction Methods		
2.5	Paragraph Deleted	Deletes requirement to remove organic mat from areas to be embanked to protect the permafrost.
2.6	Paragraph Deleted	Spec changed to require that 3 feet of embankment be placed in the winter.
3.1 Method of Measurement		
3.1	Sentences Added	States that no payment will be made for excavation. Payment will be made for any item for which excavated material is used.
Item P-209 Crushed Aggregate Base Course		
1.1, Description		
1.1	Sentences Added	Adds RAP as the material that may be used as base course.
2.1 Aggregate		
2.1	Paragraph Changed	Deletes aggregate testing requirements and requires that crushed RAP be used for aggregate base course.
Table 1	Changes Table	Changes table to require that RAP is crushed to 1" minus material.
4.1 Method of Measurement		
4.1	Sentences Changed	Adds RAP base course as the item to be measured.
5.1 Basis of Payment		
5.1	Sentence Changed	Adds RAP base course as the item to be measured.
Item P-401 Plant Mix Bituminous Pavement		
2.1 Aggregate		
2.1 a	Sentences Deleted	Deletes crushing requirements.
2.1 b	Sentences Changed	Allows 50% natural sand to be used as fine aggregate
2.3 Bituminous Material		
2.3	Table Added	Changes Bituminous material from AC-5 to PG graded, polymer modified.
3.1 Composition		
Table 1	Table Modified	Changes mix design requirements to reflect what is achievable at Barrow.
Table 2	Table Modified	Changed VMA minimum from 14% to 11%.
Table 3	Table Modified	Modified table to show only 1/2 inch minus material and modified the gradation based on the material at Barrow.
4.1 Construction Methods		
Table4	Table Modified	Changed minimum temperatures to reflect Barrow's summer.
4.2.	Sentence Added	Added capacity requirement for the bituminous mixing plant.

Table 19. Design Traffic at BRW Runway 7/25

Aircraft	Annual Departures
B737-400	1,209
B727-200	90
C-130	25

4.6.1 Design Based on AC 150/5320-6D

Researchers obtained a copy of the AKOD98 design program from Alaska DOT&PF. However, the program is Disk Operating System (DOS)-based and unable to run on modern computers. Therefore, it was not feasible to re-create the design with that program. Instead, researchers re-created the pavement design following AC 150/5320-6D. Given the relatively thick subbase and presence of deep permafrost in the area, it was assumed that the CFP method was used to protect the permafrost. Following the design inputs used by the design engineers, a subgrade modulus of 30 ksi (or CBR 20) was used for the design representing a frozen subgrade year-round. The equivalent number of departures of the design aircraft (B737-400) is 1,373 using the traffic mix shown in Table 19. The design TDD, defined as the average of the three warmest summers over a 30-year period before the year of design (2002), is 1,100 °F-days (Figure 14). According to Figure 2-8 of AC 150/5320-6D, the thaw penetration is approximately 7.5 feet.

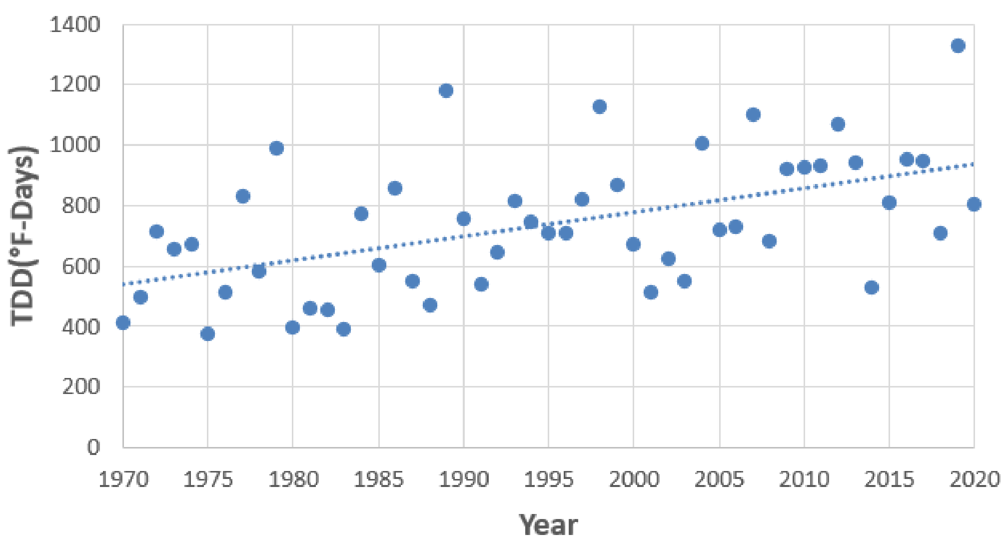

Figure 14. Change in Thawing-Degree Days in BRW 1970–2020
(Data from http://weather.gov/aprfc [Alaska-Pacific River Forecast Center])

AC 150/5320-6D requires a stabilized base and subbase for new pavements designed to accommodate aircraft heavier than 100,000 pounds. Stabilized courses may be substituted for granular courses using standard equivalency factors. Figure 15 presents a summary of the pavement design following AC 150/5320-6D for both granular and stabilized courses. The total required thickness determined from design charts in Figure 3-3 of AC 150/5320-6D is 12.5 inches. The pavement requires 9 inches of granular base with no subbase in the non-stabilized base course option (“Initial Pavement Cross Section”). The stabilized base course option (“Stabilized or Modified Cross Section”) requires 7.5 inches of stabilized base and 1.5 inches of granular subbase. However, a minimum of 13 inches of granular subbase is required to cover the CBR 20 subgrade. These results indicate that although the existing total pavement thickness is greater than is required to support the traffic, the base layer is not adequate according to the FAA standards in effect at the time. The 3-inch asphalt is also thinner than the minimum asphalt surface thickness required by AC 150/5320-6D for critical areas. Given the thaw penetration depth, the existing pavement structure is adequate to contain the seasonal thawing within the non-frost susceptible material.

4.6.2 Design Based on AC 150/5320-6G

This section covers the pavement design following AC 150/5320-6G and using the CFP method. A subgrade modulus of 30 ksi (equivalent to CBR 20 using the conversion E (psi) = 1,500 × CBR embedded in FAARFIELD) was used for the design, assuming that the top of the permafrost coincides with the top of the subgrade. The original 3-inch asphalt is less than the minimum required surface thickness. The 2014 mill and overlay effort increased the asphalt thickness to 4.5 inches. AC 150/5320-6G requires a stabilized base for new pavements designed to accommodate aircraft heavier than 100,000 pounds. (The RAP base course is not considered a stabilized material.) FAARFIELD designs were created using both the design traffic from 2002 and the actual average traffic counts for the period 2015 through 2018 (Table 20), as shown in Figure 16 and Figure 17. The actual traffic is approximately two-thirds of the design traffic. Similar to the AC 150/5320-6D design, FAARFIELD determines that the total thickness is adequate to contain the seasonal thawing within the non-frost-susceptible layer, but the base thickness is thinner than is required.

Table 20. Average Annual Traffic (2015–2018) at BRW Runway 7/25

Aircraft	Annual Departures
B737-200	169
B737-300	36
B737-400	258
B737-700	183
B737-800	231
C-130	7
MD80	11
MD82	10
DC6	16
DC93	31

FLEXIBLE PAVEMENT DESIGN FOR program date 5/15/03

Barrow *AC Method*

Barrow, AK

Engineer - Applied Research Associates AIP No.

12.5" **Total Thickness Required (inches)**

No thickness adjustments required

Stabilized Base/Subbase Are Required

Initial Pavement Cross Section	
4"	Pavement Surface Layer (P-401)
9"	Base Layer (P-209)
-0.5"	Subbase #1 (P-154) CBR= 20
0"	Subbase #2 CBR= 0
0"	Subbase #3 CBR= 0

Stabilized or Modified Cross Section		Factors
4"	P-401 Plant Mix Bituminous Pavements	
7.5"	P-401, Plant Mix Bituminous Pavements	1.2
1.5*"	Not stabilized -- P-154	1
0"	Material as defined by user	
0"	Material as defined by user	

*13" thickness required to cover CBR=20

Frost Considerations		
110 lb/cf	Dry Unit Weight of Soil	
4500	Degree Days °F	
138"	Frost Penetration Depth	
20	Original CBR value of subgrade Soil	
20	CBR Value used for the Subgrade Soil	Non-Frost Code for Subgrade Soil
20	CBR Value used for subbase #1	Non-Frost code for Subbase #1
0	CBR Value used for subbase #2	Non-Frost code for Subbase #2
0	CBR Value used for subbase #3	No frost selection made for Subbase #3

Design Aircraft Information

The Design Aircraft is a DUAL150 - 150,000 lbs -- ( )

150000 lbs	Gross Weight	20 Design Life (years)
1,373	Equivalent Annual Departures	

Subgrade Compaction Requirements for Design Aircraft

Non-Cohesive Soils Compaction	Depth Required	Cohesive Soils Compaction	Depth Required
100%	0 - 20	95%	0 - 8"
95%	20 - 34.5"	90%	8 - 15"
90%	34.5 - 49.5"	85%	15 - 22.5"
85%	49.5 - 64.5"	80%	22.5 - 30"

This software is currently under development and is not officially adopted as a FAA standard. Designs developed using this program should be checked against AC 150/5320-6D to insure accuracy and conformance to existing

Figure 15. Pavement Design Detail at BRW Based on AC 150/5320-6D

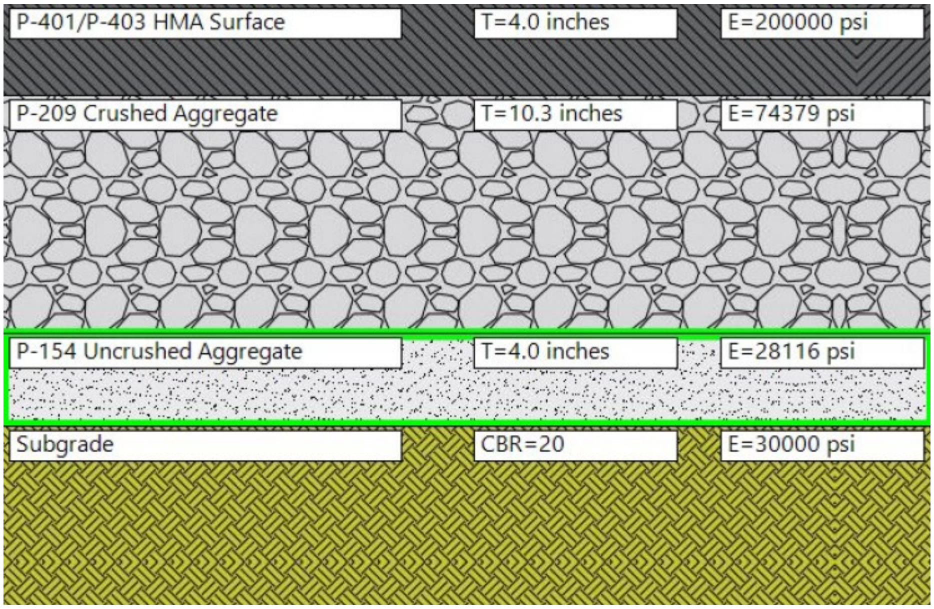

Figure 16. FAARFIELD Design Based on AC 150/5320-6G Using Design Traffic and Frozen Subgrade CBR at BRW

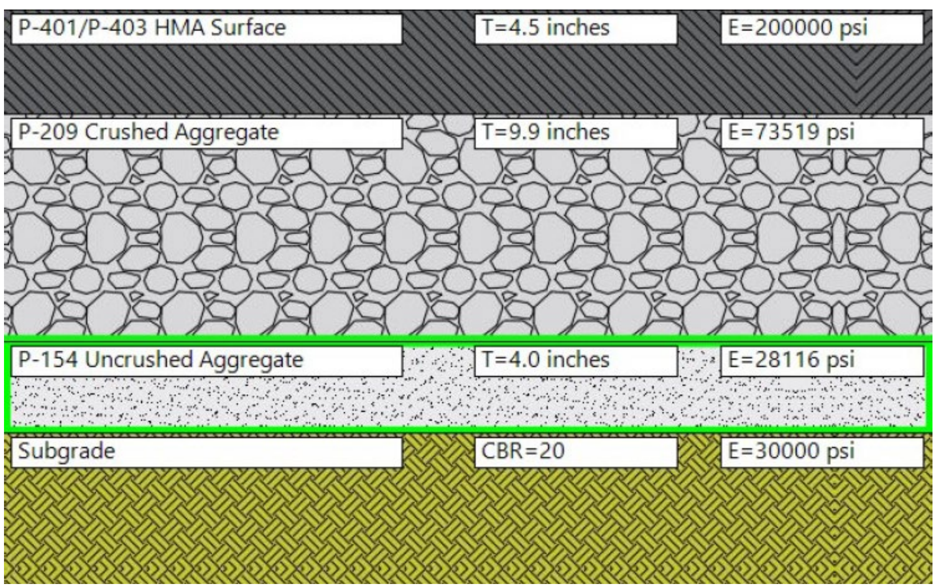

Figure 17. FAARFIELD Design Based on AC 150/5320-6G Using Actual Traffic and Frozen Subgrade CBR at BRW

Table 21 compares the pre-overlay 2002 pavement section with the required pavement thickness to support the traffic and meet the CFP method requirements, based on ACs 150/5320-6D and 150/5320-6G. This table also compares the post-overlay 2014 section with the FAARFIELD designed section using actual traffic.

Table 21. Comparison of Required and Existing Pavement Thickness at BRW

Layer	As-Built		Design Requirement		
	Pre-overlay	Post-overlay	AC 150/5320-6D	AC 150/5320-6G	
			Design Traffic	Design Traffic	Actual Traffic[c]
Surface	3 in. HMA	4.5 in. P-401	4 in. P-401	4 in. P-401	4.5 in P-401[b]
Base	6 in. RAP		9 in. P-209	10.3 in. P-209	9.9 in. P-209
Subbase	84 in. Fill		77 in. P-154[a]	76 in. P-154[a]	76 in. P-154[a]
Subgrade	CBR 20 (permafrost)		CBR 20 (permafrost)		

[a] Governed by thaw depth in CFP method
[b] P-401 thickness to match as-built post-2014 overlay
[c] Average annual departures 2015-2018

5. NOORVIK AIRPORT

Noorvik is an Inupiat village located on the Lower Kobuk River about 40 miles east of Kotzebue in northwest Alaska. The airport serving Noorvik (Robert Curtis Memorial Airport) has a single gravel-surfaced runway. Noorvik is in the transitional climate zone of Alaska, characterized by high daily temperature variations. The airport has one runway, designated 6/24. Figure 18 shows the airport.

Figure 18. Noorvik Airport Diagram

Researchers reviewed the following documents for Noorvik, which are summarized in the following sections:

- HDR Engineering, Inc. (1997). *Geotechnical Investigation Airport Material Sites, Noorvik, AK*
- APEB_2010-10-27_D76_Airport_Rehabilitation_Nomination
- Alaska DOT&PF Northern Region. (1997). *Noorvik Airport Runway Relocation* [Record drawings] (A.I.P. No. 3-02-0201-2/64022, 1996)

- Alaska DOT&PF Northern Region. (2020). *Geotechnical Investigation Report: Noorvik Airport Rehabilitation Project*
- Alaska Airport 5010 Inspections (2005, 2008, 2011, and 2015)

5.1 CONSTRUCTION HISTORY

Runway 06/24 has a gravel surface and was constructed in 2001. The runway is 4,000 feet long and 100 feet wide. The runway pavement section is illustrated in Figure 19 (Alaska DOT&PF Northern Region, 1997). A 2-inch insulation foam panel (indicated by the heavy dashed line in Figure 19) was installed approximately 3 feet below the embankment surface.

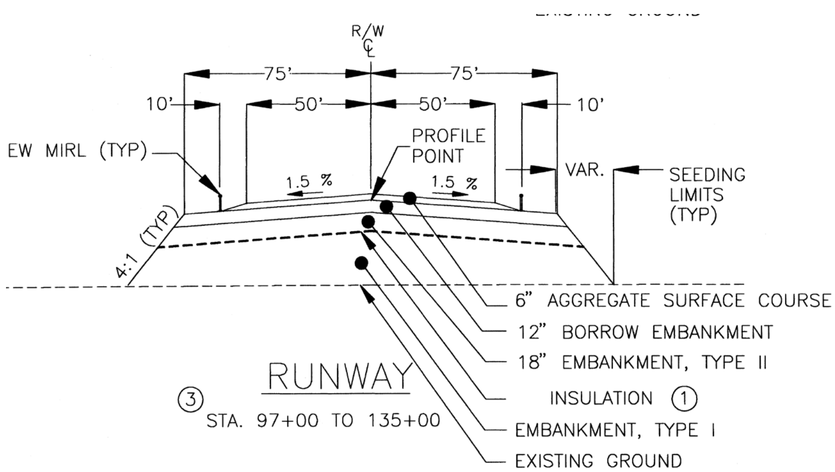

Figure 19. Runway 6/24 Pavement Cross Section

5.2 GEOTECHNICAL AND PERMAFROST CONDITIONS

A geotechnical investigation by HDR Engineering, Inc. (1997) states that "most of the lowland areas are underlain by moraine deposits which are often mantled by thick stream and lake deposits. Permafrost is present throughout the area but is discontinuous near large bodies of water. Drainage is often poor due to the low topographic gradient."

Due to lack of thermal data for the project site, the 2020 report by the Alaska Northern Region used data from Kotzebue to calculate the thermal indices (Alaska DOT&PF Northern Region, 2020a). The design FDD (defined as the average of the three coldest winters over the previous 30-year period) and TDD (defined as the average of the three warmest summers over the previous 30-year period), are shown in Table 22. The HDR Engineering (1997) report suggested that Noorvik would be expected to have a higher thawing index than Kotzebue.

Table 22. Thawing- and Freezing-Degree Days, Kotzebue Airport, 1976 to 2005 (Alaska DOT&PF Northern Region, 2020a)

Design Property	Value
Design TDD	2,673 °F -days
Design FDD	6,762 °F-days

5.3 PAVEMENT CONDITION

The 2005 pavement survey (Alaska Airport 5010 Inspections, 2005) characterized the surface condition as "fair" even though the gravel exhibited cracks 1 to-2 inches wide extending the width of the runway. No photos of the distress were in the survey report. The Alaska Airport 5010 Inspection Pavement Surveys for 2008, 2011, and 2015 noted the pavement condition was good. A rehabilitation project was defined in 2010 to repair the runway shoulders (APEB, 2010), which were sloughing just outside the runway lights resulting in significant longitudinal cracking and settling.

In 2020, the Alaska Northern Region carried out a geotechnical investigation of the Noorvik Airport rehabilitation. Researchers drilled six test holes on the runway to explore thaw settlement and shoulder rotation. Their investigation showed that "shoulder cracking and rotation (i.e., settlement) are most severe in the segments where the deepest thaw penetration and greatest surface polygon development were observed, due to thaw consolidation of ice rich foundation soil at the toe of the embankment." In the 2020 report of the investigation, researchers suggested that "cracking may be exacerbated by the upper 3 to 4 feet of embankment slipping along the inclined surface of the foam insulation, where shoulder settlement resulted in the foam panels dipping toward the embankment toe." The investigation encountered 2 inches of foam insulation in four test holes where the embankment was less than 8 feet thick. The investigation did not encounter insulation in two test holes with the approximately 11-foot-thick embankment near the runway east end. Thaw-unstable foundation soil was encountered in all five test holes. The report states that the deepest and widest thaw penetration was observed where surface polygons are most developed. According to the same report, "the top of permafrost appears to lie between the base of the embankment (original ground surface) and the top of visible ice encountered about 3 feet below it." (Alaska DOT&PF Northern Region, 2020a)

The 2020 report also identified shoulder rotation and embankment cracking as the main issues for the runway embankment. Cracking was observed between the light line and the upper embankment slope, resulting in openings up to 18 inches wide and up to 3 feet deep. The 2020 report states that the following factors contribute to shoulder rotation and cracking:

- "Thaw consolidation begins near the embankment toe, where the organic mat is compressed, and its insulation value is lost.

- Thaw consolidation results in formation of ponds trapped against the embankment, which contributes to further thaw consolidation. Ponds are wider and deeper in ice-rich permafrost areas.

- Deposition of snow on the embankment slope from snow removal and drifting insulate this area from winter cold, contributing to soil thawing.

- Thaw consolidation is most severe in areas of ice-rich permafrost as indicated by pronounced pattern polygon development.

- Buried insulation board at the embankment edge settles differentially, dipping away from the embankment. This results in a sloping water barrier and potential sliding surface which can contribute to the severity of cracking. Shoulder rotation and cracking were not observed in local roads where insulation board is absent."

(Alaska DOT&PF Northern Region, 2020a)

The report also concludes that "structural core of the embankment appears to be thermally stable. Permafrost is likely accreting into the embankment where it is 15 feet thick and not insulated with foam" (Alaska DOT&PF Northern Region, 2020a).

5.4 PAVEMENT DESIGN

The pavement design report for this runway was unavailable. Subgrade strength values and design traffic were also not available.

5.5 NOORVIK RUNWAY 6/24 DESIGN REVIEW

This section reviews the 2001 design for the gravel-surfaced runway pavement at Noorvik Airport. The designed section consisted of 6 inches of aggregate surface over 30 inches of borrow fill base over 2 inches of insulation boards over a prepared subgrade. The pavement design report is unavailable and design subgrade moduli and design traffic are unknown. The only information available is the actual traffic from 2015 through 2018 obtained from the IOAA Tool (see section 2.5) as presented in Table 23. Although the B737-400 (approximately 150,000 lb gross weight) operates at Noorvik, the gravel-surfaced runway does not meet design requirements for runways accommodating Airplane Design Group III aircraft.

Table 23. Average Annual Traffic (2015–2018) at Noorvik Runway 6/24

Aircraft	Annual Departure
B737-400	74
Cessna 208	76
PA31	43
Beech 200	25

In light of unavailability of key design inputs, researchers reverse-engineered the subgrade CBR from the existing thickness. The analysis used actual traffic data presented in Table 23. Researchers used both ACs 150/5320-6D and 150/5320-6G to back-calculate the subgrade CBR.

5.5.1 Design Based on AC 150/5320-6D

AC 150-5320-6D includes a design procedure for aggregate-turf pavements, but that procedure is only appropriate for airports serving light aircraft (<12,500 lb gross weight). While there is no FAA standard procedure for design of aggregate surface pavements in a higher aircraft weight range, researchers assumed that for higher aircraft gross weights, the design charts for flexible pavements can reasonably be used for aggregate-surfaced pavements. Figure 3-3 of AC 150/5320-6D (FAA, 1995) indicates that a CBR 4.5 subgrade along with 36 inches of aggregate course, can provide adequate support for 74 annual departures of a 150,000-pound D gear aircraft. The resulting subgrade CBR is reasonable, considering that the subgrade is protected by the insulation boards and expected to be frozen year-round, with the actual CBR greater than 4.5.

5.5.2 Design Based on AC 150/5320-6G

Researchers also reverse-engineered the design using FAARFIELD. The surface layer was modeled as moderate-stiffness, user-defined material (E = 80,000 psi). Results indicate that the existing structure over a subgrade with a CBR as low as 4.5 is adequate to support the actual traffic. Given the presence of insulation board, the actual subgrade is likely to remain in frozen condition with an expected CBR much greater than 4.5. Based on this analysis, the existing pavement is structurally adequate to support the traffic. Figure 20 shows the designed section in FAARFIELD.

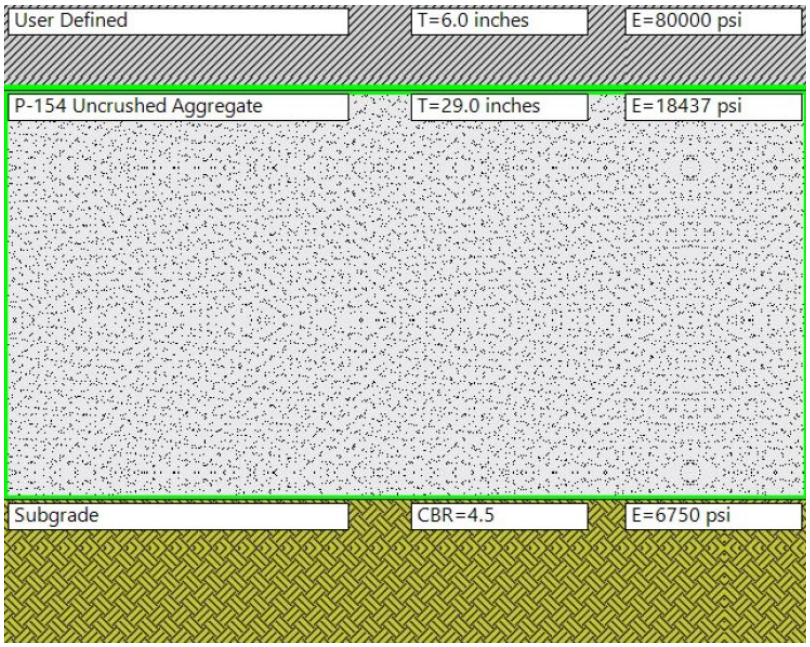

Figure 20. AC 150/5320-6G Design Using Actual Traffic at Noorvik Airport

6. SUMMARY OF DESIGN REVIEWS

The previous sections reviewed the pavement design, construction, and performance of four runway pavements at four airports in Alaska, all of which have reported performance issues. Researchers re-created the design for each of the runway pavements, identified methods for seasonal frost and permafrost design, and determined whether those methods were consistent with the FAA standards. Researchers found that the pavement performance issues can be attributed to loss of permafrost and to the designers' incorrect assumptions that the permafrost existed or will remain intact. Although most of the designs would have been adequate had the permafrost remained, some design- and construction-related factors also contributed to the performance issues. These are summarized as follows:

1) Design-related issues

   a) Designers failed to follow the FAA design criteria consistently. In some instances, the designers did not use the FAA guidance in effect at the time of the design and instead used other methods for pavement design. In one case, the frost protection method implemented was not consistent with the FAA recommendations.

   b) Degradation of permafrost, due to an increase in thaw depth over the long term and/or induced by construction activities in the short term, was not accounted for. In Nome, the depth of thawing exceeded the depth of refreezing over the years, resulting in a gradual lowering of the top of the permafrost. It is possible that the designers were aware of the unstable permafrost condition at this airport, but they were not aware of the extent of it and did not assume the permafrost would thaw. It is also possible that other cost- or construction-related limitations prohibited the appropriate frost design.

   c) The FAA guidance does not address a warming trend. The current FAA recommendation for calculation of thaw depth is based on the three warmest summers of record over the previous 30 years and might not capture the warming trend expected in the near future.

   d) The FAA guidance is silent or unclear about what subgrade strength should be used for the LFP method.

   e) There is a lack of standard procedures for pavement rehabilitation in frost and permafrost zones. The rehabilitation efforts in some of the airports were not able to produce long-term solutions. In most cases, pavement distresses reappeared a few years after the rehabilitation efforts.

   f) There were insufficient standards at the time of original pavement construction in 1960s and 1970s.

   g) Overloading was not reported (except for the gravel runway at Noorvik), but whether overloading occurred specifically during the thawing period is not known. It is possible that the original designs were based on the premise that very limited operations would take place during thawing periods, but this actually did not happen. If this was in fact the design assumption, then overloading could be the reason for some of the premature pavement distresses.

2) Construction-related issues

   a) There is a lack of standard procedures to address construction timing and sequence in frost/permafrost zones. The successful implementation of frost protection methods requires special consideration during construction to ensure that the subgrade layer being protected is in the desired (frozen) condition.

   b) There is a lack of standard methods for design and construction of insulation panels. The ineffective insulation at Kotzebue airport was attributed to incorrect construction timing (specifically, installation when the ground was not frozen).

c) Unexpected events during construction can negatively affect compaction and/or the thermal balance of the subgrade. This happened in one of the Nome rehabilitation projects where flooding during excavation prevented complete dewatering of the subgrade layer and resulted in poor compaction and subsequent reoccurring pavement settlements.

d) Scarcity of non-frost susceptible fill material in some areas led to reliance on MoS. The nonstandard materials that were used above the permafrost layer might not meet the FAA criteria for non-frost susceptible material.

3) Although cost was not directly reported as an issue in any of the investigated airports, the researchers speculate that some of the decisions that led to deviations from the FAA standards were made because strictly implementing those standards would have been prohibitively expensive.

7. RECOMMENDATIONS FOR IMPROVING CURRENT DESIGN GUIDANCE

This section provides recommendations for possible improvements to the current FAA guidance with respect to frost and permafrost design. The recommendations are based on findings from the design review of the four runways investigated in this study. The design-related issues are intended to be addressed as updates to AC 150/5320-6, *Airport Pavement Design and Evaluation* and construction-related issues are intended to be addressed as updates to AC 150/5370-10, .

Key recommendations from the study are as follows:

- Increase in Thaw Depth

    - Account for the warming trends in calculation of thaw depth. This may be more critical in the continuous permafrost zone. Designers may already account for warming trends to some extent by assuming permafrost may thaw in discontinuous permafrost zones.

    - Incorporate projected future conditions in the design thawing index calculation. The current method of using the average of three warmest years over the past 30 years cannot capture the increased warming trend.

    - Investigate the use of cool pavements, which use a reflective coating to prevent heat from building up during summer and reduce thawing. Skid resistance and cost are the main issues to be addressed.

- Frost Condition of Subgrade

    - Strengthen guidance on which design approaches are appropriate for specific permafrost zones (continuous permafrost, discontinuous permafrost).

    - Account for changes in thermal balance and permafrost condition induced by construction and excavation. This may be more critical in discontinuous permafrost zones.

- If the subgrade is not frozen at the time of construction, assume permafrost does not exist.
    - Even if CFP or insulation panel methods are used, the required pavement thickness must not be based on the assumption of frozen subgrade.
    - Pavement must be designed for the effect of seasonal frost and thaw.
    - Consider disallowing insulating panels if permafrost is not present at the time of construction.
- Consider weather conditions during construction. Construction during warmer weather exposes permafrost to non-freezing temperatures and allows it to thaw. After exposing permafrost during warm weather, delay paving over the winter to expose the thawed material to freezing temperatures and to allow permafrost to refreeze. However, this method may only be feasible for construction of a new pavement and not rehabilitation of existing pavements with daily jet operations.

- Application of CFP Method
    - Partial frost protection seems to be of limited value based on the small sample in this study. Designers and operators may not understand that differential settlement is expected with this approach. Consider either removing it from the guidance, or if retained, evaluating where its use is appropriate and strengthening the warning about expected issues.
    - Allow complete frost protection for the FG-1 and FG-2 frost groups.
- Application of RSS Method
    - Provide recommendations for how much reduction in subgrade strength is needed when the strength during the thawing period is not available.
    - Interpret design guidance to assume the entirety of design traffic occurs while the subgrade is in its weakest state. Since traffic occurs throughout the year, including non-thaw periods with a relatively strong subgrade, designers should be allowed to apply traffic seasonally to different subgrade strengths.
    - Perform research to investigate the reduction in strength during a thawing period. The recommended 50% reduction in AC 150/5320-6G is arbitrary and might not be realistic.
    - Clarify what is the “design strength” being reduced. Is this frozen subgrade or dry subgrade or else?
    - Consider that reduced strength can be a percentage of design subgrade strength, or recommended values in AC 150/5320-6D based on frost group, whichever is lowest.

- Avoid the RSS method, which seems of limited value based on the small sample in this study. All four airports serve aircraft heavier than 60,000 pounds (Boeing 737), for which the RSS method is not allowed.

- Clarify how much differential settlement can be expected when using RSS design methods.

- Insulating Panels

  - Panel Construction:

    - Guidance must emphasize that construction timing and sequence is critical.
    - Soil must be frozen when installing the insulating panels.
    - Panels should not be placed on uneven soil.
    - Uniform insulation is recommended.
    - Insulating panels must extend to the shoulders and embankment edge.

  - Panel Design:

    - Frost condition of subgrade

      - Account for changes in the thermal balance and permafrost due to construction and excavation.

      - Avoid using frozen subgrade strength for design to account for partial thawing of the subgrade below the insulation to optimize safety.

    - Strength and durability of panels

      - Confirm the thickness of insulation panels is sufficient to protect the panels against overburden weight and aircraft loads.

      - Investigate more durable and effective insulating materials such as lightweight cellular concrete, and polystyrene concrete.

    - Clarify whether the insulating panel method is applicable to all frost groups.

- Other Considerations

  - Avoid localized repair/rehabilitation, which is unlikely to fix the settlements of thaw-unstable subgrade.

  - Investigate the effect of high ground water levels in determining freeze and thaw susceptibility.

  - Dewater subgrade or frost-susceptible layers to help reduce frost susceptibility.

  - Avoid piling snow on the embankment slope.

8. SUMMARY AND CONCLUSIONS

Information was gathered and reviewed on the pavement performance issues of three runways with asphalt pavement (Nome, Kotzebue, and Utqiagvik/Barrow) and one runway with gravel surface (Noorvik) within the State of Alaska. Nome is in a discontinuous permafrost zone, and the other three airports are in a continuous permafrost zone. The study reviewed various types of documents such as airport pavement design and construction, geotechnical investigations, pavement condition surveys, and environmental investigations. Additional information was obtained through two teleconference meetings with personnel from Alaska DOT&PF, the FAA Alaskan Region, and the FAA William J. Hughes Technical Center. Additional questions were submitted to Alaska DOT&PF, who in turn obtained additional details from the pavement evaluation and design consultants the state used for projects at the airports.

Some of the affected airports have had to do multiple iterations of repair and have spent significant amounts of effort and money to keep their pavements serviceable. In many cases, airports sacrificed long-term pavement performance for repair expediency. Short-life repairs have led to fix-the-fix syndrome and have resulted in a higher life cycle cost for the repairs and poor value to the National Airspace System (NAS).

Localized differential settlements were the most common pavement distress type in the airports with asphalt pavement. It is likely that the loss of permafrost caused the settlements. This was aggravated by the inherently poor compaction of permafrost subgrade materials and incorrect timing and sequence of construction. The localized repair of the settled areas did not seem to be effective, as the settlements started to re-occur within a few years after the repair efforts. The full-depth repairs tend to significantly alter the thermal stability of the permafrost layer, making the subgrade thawing more detrimental.

The cases studied indicated a historical warming trend and loss of permafrost in the State of Alaska. In Nome, geotechnical investigations found that the permafrost has degraded in some sections of the runway for more than 30 years. Thawing of the permafrost layer has significant influence on the airport pavement performance.

Transverse cracks were also observed in all airports with flexible pavement. These cracks were primarily thermal cracks that occurred due to the prolonged freezing days. Some of the transverse cracks were also caused by the differential settlements of the thawed subgrade that were reflected to the surface.

Non-frost susceptible construction materials were not readily available in the vicinity of the Kotzebue and Utqiagvik/Barrow airports. Local material in Nome can meet non-frost susceptible requirements.

The design methods and input parameters used by the design engineers were also reviewed, identified, and determined whether they were consistent with the FAA guidance. This report documents the researchers' efforts to re-create the pavement design for each runway following both the FAA guidance in effect at the time of the design and current (as of the date of this report) AC 150/5320-6G. Areas of the FAA frost and permafrost design criteria that are silent or subject to conflicting interpretation were identified. The study determined that the loss of permafrost and thawing of the frost-susceptible pavement layers was the root cause of the majority of performance issues in the airports. The proximate cause of the failures included issues with guidance, design assumptions, and construction techniques incompatible with a warming trend. Researchers speculated that either the unsteady condition of the permafrost was not understood at the time of the original design, or the pavement designers assumed permafrost would remain. Comparison of the actual design with the existing pavement determined that faulty design assumptions and failure to follow the FAA guidance by the design engineers contributed to some of the performance issues. Inadequacies in the FAA design methods were also responsible for some of the performance issues. The performance issues were also linked to the construction practices. Lack of standard construction procedures to address construction timing and sequence in the areas of permafrost has led to ineffective frost-protection methods. Lack of standard procedures for pavement rehabilitation in frost and permafrost zones was also responsible for ineffective rehabilitation projects that led to reoccurrence of pavement settlements. Some of the efforts that resulted in ineffective rehabilitation and frost protection were also driven by unexpected events during construction, construction difficulties, and scarcity of standard non-frost susceptible materials.

This report provides recommendations for possible improvements to the current FAA guidance with respect to frost and permafrost design. Findings from the design reviews of the four airports formed the basis of recommendations. The recommendations targeted ACs 150/5320-6G and 150/5370-10H and are intended to address the design- and construction-related issues.

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APPENDIX A—INTERVIEW QUESTIONS AND ANSWERS FROM 05 NOVEMBER 2020

A.1 NOME INTERVIEW DISCUSSIONS

Q1: What were the pavement design assumptions with respect to the thawing index, frost protection method selected and in-situ soil characterization? What were the sources of data (e.g., freeze and thaw index calculation) and how current?

A1: Due to active layer depth, complete frost protection is not feasible. Except for select areas, this was a rehabilitation project which did not evaluate subgrade improvements for frost action. Select areas had subgrade repairs, however only extended to a portion of the active layer and design focused on matching adjacent typical section to ensure uniform frost action. Deep ground improvements on Runway 10 had not permafrost detected in most recent borings and the water table was a more limiting factor such that only limited frost protection is provided, similar to adjacent sections.

Q2: Were paper charts or a computer program such as ModBerg used for establishing the depth to freezing and thawing?

A2: Depth of freeze is based on geotechnical investigations (drill logs and thermistor logs).

Q3: What were the amount of distress considered tolerable for the design?

A3: Some amount of settlement is still expected, even if no permafrost was encountered during the recent geotechnical exploration. (This is why the micropiles were considered.) Uniform settlement is typically acceptable. Differential settlement is the trigger. User complaints (e.g., air carriers) are the typical driver, but are on the order of abrupt changes of an inch.

Q4: What amount of distress triggered the rehabilitation actions in 2020 project?

A4: Settlement in the runway. Water collection (enough to impede directional control from hydroplaning) even though entire runway is grooved. Order of magnitude 25 ft long by width of runway. Depth of water 3″–4″ or more. DOT patched these areas in 2019. Current rehabilitation is driven by surface condition (cracking). Cracks could be 1″–4″+ wide and be very deep. Near the intersection of the runways.

Design of runways was for a complete resurfacing, so no distinction was made during design for level of existing distress. Apron resurfacing extents were based on surface deformation and constructability such that the final surface meets FAA grade standards and has positive drainage. Records review did not reveal a specific trigger that was used for this rehabilitation project.

Q5: Local availability of suitable construction materials and equipment. Were non-frost-susceptible material available?

A5: Local material in Nome can meet non-frost susceptible requirements.

Q6: What were the unique or challenging aspects of the design?

A6: Complicated geotechnical situation. Consists of undisturbed, dredge, fill that are all mixed.

Continuation of historic settlement areas on Rwy 10 with no apparent cause. Continuation of a historic crack on Rwy 10/28 near Twy F. How to repair these areas in a lasting but cost and time effective manner.

Q7: Were there design details used that could have affected performance (for example, drainage structures, or treatment of transitions between sections with different properties)?

A7: All details affect performance in some way. Proven design details are used when applicable and cost effective for the project. Excavation limits and transition and select reconstruction areas are limited by cost and project scope to the minimum necessary at the time of design, thereby accepting some level of risk.

Q8: Was there embankment slope failure observed? Were there any design tradeoffs (For example, steep embankment slopes may avoid disturbing adjacent wetlands, but could also contribute to premature failure.)?

A8: Slope failures are not noted at Nome and are not part of the current project scope.

Q9: Was there consideration given to using innovative design features (for example, insulating boards) that are not part of the current FAA standard?

A9: Micropiles with caps along the centerline.

The 2020 project was a surface rehabilitation, not reconstruction so subsurface repairs, other than the select deep repairs, were not evaluated. Deeper repairs were designed using a consistent subgrade typical section as the adjacent sections to ensure a uniform frost action.

Q10: What were the cost considerations during pavement design? Was there any cost factor that prevented a certain design?

A10: 2019 rehab was a stopgap (that continued into 2020) project kept finding more and more areas to rehab. The 2020 rehab covers the entire runway and was partially intended to mitigate the issues discovered during the 2019 project rather than keep applying “band-aids”.

Deep repair design recommendations were highly influenced by cost and schedule (runway closure) requirements.

A.2 KOTZEBUE AIRPORT INTERVIEW DISCUSSIONS

Q1: What were the pavement design assumptions with respect to the thawing index, frost protection method selected and in-situ soil characterization? What were the sources of data (e.g., freeze and thaw index calculation) and how current?

A1: The 2010 project was a pavement rehabilitation project with the exception of a few areas experiencing settlement, and no thermal analysis was performed. Historic borings showed very sporadic permafrost and ice lenses and the existing embankment was over 20 years old and believed to be thermally stable. The design for the dig-out sections focused on improving the thermal stability of the sections experiencing settlement, while closely matching adjacent sections to minimize any potential for differential settlement.

Q2: Were paper charts or a computer program such as ModBerg used for establishing the depth to freezing and thawing?

A2: Depth to freeze and thawing for the existing section was based on drill logs and thermistor logs. Depth to freeze and thawing for the design section was not analyzed.

Q3: What were the amount of distress considered tolerable for the design? What amount of distress triggered the rehabilitation actions?

A3: Settlement in two spots. Approximately 4 inches. Airport manager reports that they start receiving complaints when settlement reaches 5 inches to 6 inches. Seeing 3-4 inches of settlement in the same area that was rehabilitated in 2011. Expect to need another rehab in near future.

Q4: What were the unique or challenging aspects of the design?

A4: The challenge during construction. This was excavated during summertime, so there is uncertainty when you open up the ground. There were ice lenses found during construction. We believe the excavation itself caused some thawing of the permafrost. The contractor was not able to remove all of the material that was intended during the 2011 rehabilitation due to water and thawing during construction. A mix of frozen and un-frozen areas can be observed in the embankment/subgrade.

Q5: Were there design details used that could have affected performance (for example, drainage structures, or treatment of transitions between sections with different properties)?

A5: Geotechnical investigations concluded that insulation was not effective; it kept the cold out and prevented re-freezing. Seems to be location based; works better in areas with very short summers. Timing of insulation is important. If you put insulation in during spring the ground is as cold as it gets, and the insulation is more effective. If you put it in during fall, the ground is warmer, and the insulation keeps the cold out. The ground may come back to equilibrium, but it may take years, and there is a lot of damage in the meantime.

Existing insulation board was broken into small pieces no larger than 12″ and blended into the bottom 18″ layer of gravel fill. This was thought to be a cost-effective solution to utilize existing, old insulation. Likely the most important factor affecting performance was not the design, but instead the sequencing of construction. The work was performed in summer. Therefore, excavation and re-construction of the digout section(s) would have introduced and trapped heat in the embankment, likely contributing to thermal degradation immediately during and after construction.

Q6: Was there embankment slope failure observed? Were there any design tradeoffs (For example, steep embankment slopes may avoid disturbing adjacent wetlands, but could also contribute to premature failure.)?

A6: No slope failure observed or documented.

Q7: What were the cost considerations during pavement design? Was there any cost factor that prevented a certain design? What was the role of budget limitation on selection of rehabilitation type?

A7: Due to the depth of the active layer and sporadic existence of ice lenses in the embankment, it was cost prohibitive to reconstruct the digout sections down to the bottom of the active layer. The design was highly influenced by construction sequencing, balancing construction with the need to minimize impacts to operations.

Expectation is that another dig out will not work, but no decisions on alternative solutions.

Comment from the geotechnical consultant: The primary factor affecting performance is likely attributed to the construction sequencing more than the design itself. We have learned, through practice, that placement of insulation board can be effective so long as the work does not introduce and trap additional heat in to thaw-unstable soils. If excavation and placement of insulation is performed in above freezing temperatures, the insulation board actually traps heat in the ground and further degrades the permafrost. Therefore, it’s a best practice to perform this type of work in the winter.

A.3 BARROW AIRPORT INTERVIEW DISCUSSIONS

Q1: What situations/factors led to the decision of shifting the runway and constructing new runway in 2001?

A1: At the time the East end of the runway had been painted white for a decade or more due to thaw settlement that had occurred shortly after it was paved. Barrow Airport does not have a parallel taxiway so the existing main runway would need to be used during construction of any improvement. The runway was shifted to the south and grade was raised to keep underlying soils frozen and to facilitate construction while using the existing runway.

Q2: What were the pavement design assumptions with respect to the thawing index, frost protection method selected and in-situ soil characterization? What were the sources of data (e.g. freeze and thaw index calculation) and how current?

A2: Barrow had many years of temperature data. The data was analyzed with respect to recurrence. At that time recent warm years were noticed in the data. The computer Program Mut1D was used to simulate different soil and temperature conditions. The sensitivity analysis showed that the variation in average yearly temperature was the key factor. Final selection of design average temperature was based on a 10- or 20-year warm year.

Q3: Were paper charts or a computer program such as ModBerg used for establishing the depth to freezing and thawing?

A3: The Computer Program Mut1D was used to establish estimated thaw depth. This thaw depth was used to establish the new runway profile grade.

Q4: What were the unique or challenging aspects of the design?

A4: Local materials were fine beach sands and gravel. The pavement and embankment design utilized this material for all construction. Hot Mix Asphalt was produced from 3/8- sand and gravel.

Q5: Were there design details used that could have affected performance (for example, drainage structures, or treatment of transitions between sections with different properties)?

A5: In pavement threshold and MALSR light may have caused permafrost degradation on one end of the project after completion. The choice to place the first layer of material on original ground may have led to some settlement in the embankment.

Q6: Was there embankment slope failure observed? Were there any design tradeoffs (For example, steep embankment slopes may avoid disturbing adjacent wetlands, but could also contribute to premature failure.)?

A6: The primary cause of slope failures is due to erosion. The 500′ wide safety area and relatively flat cross and longitudinal slope causes water to collect and concentrate in wheel tracks in the safety area. The water concentrates until it overflows the wheel track and then runs off the embankment. The concentrated flows easily produce rill and gully erosion in the fine beach sands and gravels. Vegetation in Barrow is especially difficult to establish.

Q7: What were the cost considerations during pavement design? Was there any cost factor that prevented a certain design?

A7: Barged material was considered and rejected due to cost.

APPENDIX B—PAVEMENT DESIGN CALCULATIONS

B.1 NOME DESIGN SUMMARY BASED ON ADVISORY CIRCULAR 150/5320-6D

FLEXIBLE PAVEMENT DESIGN FOR program date 5/15/03

Nome RWY 10-28 *AC Method*

Nome, AK

Engineer - Applied Research Associates AIP No.

Reduced Subgrade Strength

37.5" **Total Thickness Required (inches)**

No thickness adjustments required

Stabilized Base/Subbase Are Required

Initial Pavement Cross Section	
4"	Pavement Surface Layer (P-401)
9"	Base Layer (P-209)
24.5"	Subbase #1 (P-154) CBR= 20
0"	Subbase #2 CBR= 0
0"	Subbase #3 CBR= 0

Stabilized or Modified Cross Section		Factors
4"	P-401 Plant Mix Bituminous Pavements	
9"	Not stabilized	1
24.5"	Not stabilized -- P-154	1
0"	Material as defined by user	
0"	Material as defined by user	

Frost Considerations

110 lb/cf	Dry Unit Weight of Soil	
4500	Degree Days °F	
138"	Frost Penetration Depth	
4	Original CBR value of subgrade Soil	
4	CBR Value used for the Subgrade Soil	F-3 Frost Code for Subgrade Soil
20	CBR Value used for subbase #1	Non-Frost code for Subbase #1
0	CBR Value used for subbase #2	Non-Frost code for Subbase #2
0	CBR Value used for subbase #3	No frost selection made for Subbase #3

Design Aircraft Information

The Design Aircraft is a DUAL150 - 150,000 lbs -- ( )

150000 lbs Gross Weight **20** Design Life (years)

1,354 Equivalent Annual Departures

Subgrade Compaction Requirements for Design Aircraft

Non-Cohesive Soils		Cohesive Soils	
Compaction	Depth Required	Compaction	Depth Required
100%	0 - 20	95%	0 - 8"
95%	20 - 34.5"	90%	8 - 15"
90%	34.5 - 49.5"	85%	15 - 22.5"
85%	49.5 - 64.5"	80%	22.5 - 30"

This software is currently under development and is not officially adopted as a FAA standard. Designs developed using this program should be checked against AC 150/5320-6D to insure accuracy and conformance to existing

FLEXIBLE PAVEMENT DESIGN FOR program date 5/15/03

Nome RWY 10-28 *AC Method*

Nome, AK

Engineer - Applied Research Associates AIP No.

Limited Frost Protection

17" **Total Thickness Required (inches)**

No thickness adjustments required

Stabilized Base/Subbase Are Required

Initial Pavement Cross Section	
4"	Pavement Surface Layer (P-401)
9"	Base Layer (P-209)
4"	Subbase #1 (P-154) CBR= 20
0"	Subbase #2 CBR= 0
0"	Subbase #3 CBR= 0

Stabilized or Modified Cross Section		Factors
4"	P-401 Plant Mix Bituminous Pavements	
9"	Not stabilized	1
4"	Not stabilized -- P-154	1
0"	Material as defined by user	
0"	Material as defined by user	

Frost Considerations		
110 lb/cf	Dry Unit Weight of Soil	
4500	Degree Days °F	
138"	Frost Penetration Depth	
13.5	Original CBR value of subgrade Soil	
13.5	CBR Value used for the Subgrade Soil	Non-Frost Code for Subgrade Soil
20	CBR Value used for subbase #1	Non-Frost code for Subbase #1
0	CBR Value used for subbase #2	Non-Frost code for Subbase #2
0	CBR Value used for subbase #3	No frost selection made for Subbase #3

Design Aircraft Information

The Design Aircraft is a DUAL150 - 150,000 lbs -- ( )

150000 lbs	Gross Weight	**20**	Design Life (years)
1,354	Equivalent Annual Departures		

Subgrade Compaction Requirements for Design Aircraft

Non-Cohesive Soils		**Cohesive Soils**	
Compaction	Depth Required	Compaction	Depth Required
100%	0 - 20	95%	0 - 8"
95%	20 - 34.5"	90%	8 - 15"
90%	34.5 - 49.5"	85%	15 - 22.5"
85%	49.5 - 64.5"	80%	22.5 - 30"

This software is currently under development and is not officially adopted as a FAA standard. Designs developed using this program should be checked against AC 150/5320-6D to insure accuracy and conformance to existing

FLEXIBLE PAVEMENT DESIGN FOR program date 5/15/03

Nome RWY 10-28 *AC Method*

Nome, AK

Engineer - Applied Research Associates AIP No.

Limited Frost Protection

41.5" **Total Thickness Required (inches)**

No thickness adjustments required

Stabilized Base/Subbase Are Required

Initial Pavement Cross Section	
4"	Pavement Surface Layer (P-401)
9"	Base Layer (P-209)
28.5"	Subbase #1 (P-154) CBR= 20
0"	Subbase #2 CBR= 0
0"	Subbase #3 CBR= 0

Stabilized or Modified Cross Section		Factors
4"	P-401 Plant Mix Bituminous Pavements	
9"	Not stabilized	1
28.5"	Not stabilized -- P-154	1
0"	Material as defined by user	
0"	Material as defined by user	

Frost Considerations		
110 lb/cf	Dry Unit Weight of Soil	
4500	Degree Days °F	
138"	Frost Penetration Depth	
3.4	Original CBR value of subgrade Soil	
3.4	CBR Value used for the Subgrade Soil	Non-Frost Code for Subgrade Soil
20	CBR Value used for subbase #1	Non-Frost code for Subbase #1
0	CBR Value used for subbase #2	Non-Frost code for Subbase #2
0	CBR Value used for subbase #3	No frost selection made for Subbase #3

Design Aircraft Information

The Design Aircraft is a DUAL150 - 150,000 lbs -- ( )

150000 lbs	Gross Weight	**20**	Design Life (years)
1,354	Equivalent Annual Departures		

Subgrade Compaction Requirements for Design Aircraft

Non-Cohesive Soils		**Cohesive Soils**	
Compaction	Depth Required	Compaction	Depth Required
100%	0 - 20	95%	0 - 8"
95%	20 - 34.5"	90%	8 - 15"
90%	34.5 - 49.5"	85%	15 - 22.5"
85%	49.5 - 64.5"	80%	22.5 - 30"

This software is currently under development and is not officially adopted as a FAA standard. Designs developed using this program should be checked against AC 150/5320-6D to insure accuracy and conformance to existing standards

B.2 NOME DESIGN SUMMARY BASED ON ADVISORY CIRCULAR 150/5320-6G

Federal Aviation Administration FAARFIELD 2.0 Section Report

FAARFIELD 2.0.7 (Build 09/14/2021)

Job Name: Spring CBR - Design Traffic

Section: Nome Runway 10-28

Analysis Type: HMA on Aggregate

Last Run: Thickness Design 2022-01-12 15:23:56

Design Life = 20 Years

Total thickness to the top of the subgrade = 42.2in.

Pavement Structure Information by Layer

No.	Type	Thickness in.	Modulus psi	Poisson's Ratio	Strength R psi
1	P-401/P-403 HMA Surface	4.0	200000	0.35	0
2	P-209 Crushed Aggregate	10.2	61774	0.35	0
3	P-154 Uncrushed Aggregate	27.9	16645	0.35	0
4	Subgrade	0	5100	0.35	0

Airplane Information

No.	Name	Gross Wt. lbs	Annual Departures	% Annual Growth
1	B737-400	150500	1114	0
2	C-130	155000	40	0
3	DC9-32	109000	315	0
4	B727-200 Advanced Basic	185200	55	0

Additional Airplane Information

Subgrade CDF

No.	Name	CDF Contribution	CDF Max for Airplane	P/C Ratio
1	B737-400	0.69	0.70	1.2
2	C-130	0.00	0.00	1.51
3	DC9-32	0.00	0.00	1.25
4	B727-200 Advanced Basic	0.31	0.31	1.15

Federal Aviation Administration FAARFIELD 2.0 Section Report

FAARFIELD 2.0.7 (Build 09/14/2021)

Job Name: Spring CBR

Section: Nome Runway 10-28

Analysis Type: HMA on Aggregate

Last Run: Thickness Design 2022-01-12 15:00:46

Design Life = 20 Years

Total thickness to the top of the subgrade = 42.6in.

Pavement Structure Information by Layer

No.	Type	Thickness in.	Modulus psi	Poisson's Ratio	Strength R psi
1	P-401/P-403 HMA Surface	4.0	200000	0.35	0
2	P-209 Crushed Aggregate	10.8	62641	0.35	0
3	P-154 Uncrushed Aggregate	27.8	16620	0.35	0
4	Subgrade	0	5100	0.35	0

Airplane Information

No.	Name	Gross Wt. lbs	Annual Departures	% Annual Growth
1	B737-200	116000	219	0
2	B737-300	140000	82	0
3	B737-400	150500	43	0
4	B737-700	155000	753	0
5	B737-800	174700	191	0
6	L-100-20	155801	171	0
7	MD-83	161000	80	0
8	MD-90-30 ER	168500	53	0

Federal Aviation Administration FAARFIELD 2.0 Section Report

FAARFIELD 2.0.7 (Build 09/14/2021)

Job Name: Summer CBR

Section: Nome Runway 10-28

Analysis Type: HMA on Aggregate

Last Run: Thickness Design 2022-01-12 14:59:03

Design Life = 20 Years

Total thickness to the top of the subgrade = 18.9in.

Pavement Structure Information by Layer

No.	Type	Thickness in.	Modulus psi	Poisson's Ratio	Strength R psi
1	P-401/P-403 HMA Surface	4.0	200000	0.35	0
2	P-209 Crushed Aggregate	10.8	64237	0.35	0
3	P-154 Uncrushed Aggregate	4.1	22267	0.35	0
4	Subgrade	0	20250	0.35	0

Airplane Information

No.	Name	Gross Wt. lbs	Annual Departures	% Annual Growth
1	B737-200	116000	219	0
2	B737-300	140000	82	0
3	B737-400	150500	43	0
4	B737-700	155000	753	0
5	B737-800	174700	191	0
6	L-100-20	155801	171	0
7	MD-83	161000	80	0
8	MD-90-30 ER	168500	53	0

B.3 KOTZEBUE DESIGN SUMMARY BASED ON ADVISORY CIRCULAR 150/5320-6G

Federal Aviation Administration FAARFIELD 2.0 Section Report

FAARFIELD 2.0.7 (Build 09/14/2021)

Job Name: Kotzebue CBR=20_Design Traffic

Section: Design Traffic

Analysis Type: HMA on Aggregate

Last Run: Thickness Design 2022-01-19 11:48:30

Design Life = 20 Years

Total thickness to the top of the subgrade = 17.9in.

Pavement Structure Information by Layer

No.	Type	Thickness in.	Modulus psi	Poisson's Ratio	Strength R psi
1	P-401/P-403 HMA Surface	4.0	200000	0.35	0
2	P-209 Crushed Aggregate	9.9	73480	0.35	0
3	P-154 Uncrushed Aggregate	4.0	28116	0.35	0
4	Subgrade	0	30000	0.35	0

Airplane Information

No.	Name	Gross Wt. lbs	Annual Departures	% Annual Growth
1	B737-200	116000	1475	0
2	B737-400	150500	340	0
3	DC9-32	109000	525	0
4	B727-100C Alternate	170000	225	0
5	L-100-20	155801	185	0

Additional Airplane Information

Subgrade CDF

No.	Name	CDF Contribution	CDF Max for Airplane	P/C Ratio
1	B737-200	0.00	0.00	1.51
2	B737-400	0.00	0.00	1.48
3	DC9-32	0.00	0.00	1.59
4	B727-100C Alternate	0.00	0.00	1.45
5	L-100-20	0.00	0.00	2.34

Federal Aviation Administration FAARFIELD 2.0 Section Report

FAARFIELD 2.0.7 (Build 09/14/2021)

Job Name: Kotzebue CBR=20

Section: New Section 1

Analysis Type: HMA on Aggregate

Last Run: Thickness Design 2021-10-15 16:14:53

Design Life = 20 Years

Total thickness to the top of the subgrade = 18.5in.

Pavement Structure Information by Layer

No.	Type	Thickness in.	Modulus psi	Poisson's Ratio	Strength R psi
1	P-401/P-403 HMA Surface	4.0	200000	0.35	0
2	P-209 Crushed Aggregate	10.5	74742	0.35	0
3	P-154 Uncrushed Aggregate	4.0	28116	0.35	0
4	Subgrade	0	30000	0.35	0

Airplane Information

No.	Name	Gross Wt. lbs	Annual Departures	% Annual Growth
1	B737-200	116000	256	0
2	B737-300	140000	100	0
3	B737-400	150500	38	0
4	B737-700	155000	784	0
5	B737-800	174700	130	0
6	C-130	155000	117	0
7	Learjet 45/55B	21500	46	0
8	DC9-32	109000	68	0

Federal Aviation Administration FAARFIELD 2.0 Section Report

FAARFIELD 2.0.7 (Build 09/14/2021)

Job Name: Kotzebue CBR=20_Design Traffic

Section: Design Traffic

Analysis Type: New Flexible

Last Run: Thickness Design 2022-03-14 18:50:59

Design Life = 20 Years

Total thickness to the top of the subgrade = 13.2in.

Pavement Structure Information by Layer

No.	Type	Thickness in.	Modulus psi	Poisson's Ratio	Strength R psi
1	P-401/P-403 HMA Surface	4.0	200000	0.35	0
2	P-401/P-403 HMA Stabilized	5.2	400000	0.35	0
3	P-154 Uncrushed Aggregate	4.0	28116	0.35	0
4	Subgrade	0	30000	0.35	0

Airplane Information

No.	Name	Gross Wt. lbs	Annual Departures	% Annual Growth
1	B737-200	116000	1475	0
2	B737-400	150500	340	0
3	DC9-32	109000	525	0
4	B727-100C Alternate	170000	225	0
5	L-100-20	155801	185	0

Additional Airplane Information

Subgrade CDF

No.	Name	CDF Contribution	CDF Max for Airplane	P/C Ratio
1	B737-200	0.00	0.00	1.77
2	B737-400	0.00	0.00	1.73
3	DC9-32	0.00	0.00	1.72
4	B727-100C Alternate	0.00	0.00	1.68
5	L-100-20	0.00	0.00	2.67

Federal Aviation Administration FAARFIELD 2.0 Section Report

FAARFIELD 2.0.7 (Build 09/14/2021)

Job Name: Kotzebue CBR=10_Design Traffic

Section: Design Traffic

Analysis Type: HMA on Aggregate

Last Run: Thickness Design 2022-03-15 23:14:22

Design Life = 20 Years

Total thickness to the top of the subgrade = 21.9in.

Pavement Structure Information by Layer

No.	Type	Thickness in.	Modulus psi	Poisson's Ratio	Strength R psi
1	P-401/P-403 HMA Surface	4.0	200000	0.35	0
2	P-209 Crushed Aggregate	9.9	58042	0.35	0
3	P-154 Uncrushed Aggregate	8.0	19945	0.35	0
4	Subgrade	0	15000	0.35	0

Airplane Information

No.	Name	Gross Wt. lbs	Annual Departures	% Annual Growth
1	B737-200	116000	1475	0
2	B737-400	150500	340	0
3	DC9-32	109000	525	0
4	B727-100C Alternate	170000	225	0
5	L-100-20	155801	185	0

Additional Airplane Information

Subgrade CDF

No.	Name	CDF Contribution	CDF Max for Airplane	P/C Ratio
1	B737-200	0.00	0.00	1.42
2	B737-400	0.40	0.41	1.41
3	DC9-32	0.00	0.00	1.51
4	B727-100C Alternate	0.59	0.59	1.34
5	L-100-20	0.00	0.00	2.13

Federal Aviation Administration FAARFIELD 2.0 Section Report

FAARFIELD 2.0.7 (Build 09/14/2021)

Job Name: Kotzebue CBR=10_Design Traffic

Section: Design Traffic

Analysis Type: New Flexible

Last Run: Thickness Design 2022-03-14 18:54:56

Design Life = 20 Years

Total thickness to the top of the subgrade = 17.5in.

Pavement Structure Information by Layer

No.	Type	Thickness in.	Modulus psi	Poisson's Ratio	Strength R psi
1	P-401/P-403 HMA Surface	4.0	200000	0.35	0
2	P-401/P-403 HMA Stabilized	5.2	400000	0.35	0
3	P-154 Uncrushed Aggregate	8.3	20049	0.35	0
4	Subgrade	0	15000	0.35	0

Airplane Information

No.	Name	Gross Wt. lbs	Annual Departures	% Annual Growth
1	B737-200	116000	1475	0
2	B737-400	150500	340	0
3	DC9-32	109000	525	0
4	B727-100C Alternate	170000	225	0
5	L-100-20	155801	185	0

Additional Airplane Information

No.	Name	CDF Contribution	CDF Max for Airplane	P/C Ratio
1	B737-200	0.00	0.00	1.52
2	B737-400	0.43	0.44	1.49
3	DC9-32	0.00	0.00	1.6
4	B727-100C Alternate	0.56	0.56	1.46
5	L-100-20	0.00	0.00	2.36

Federal Aviation Administration FAARFIELD 2.0 Section Report

FAARFIELD 2.0.7 (Build 09/14/2021)

Job Name: Kotzebue CBR=10 Actual Traffic

Section: Actual Traffic

Analysis Type: New Flexible

Last Run: Thickness Design None

Design Life = 20 Years

Total thickness to the top of the subgrade = 22.2in.

Pavement Structure Information by Layer

No.	Type	Thickness in.	Modulus psi	Poisson's Ratio	Strength R psi
1	P-401/P-403 HMA Surface	4.0	200000	0.35	0
2	P-209 Crushed Aggregate	10.5	75000	0.35	0
3	P-154 Uncrushed Aggregate	7.7	40000	0.35	0
4	Subgrade	0	15000	0.35	0

Airplane Information

No.	Name	Gross Wt. lbs	Annual Departures	% Annual Growth
1	B737-200	116000	256	0
2	B737-300	140000	100	0
3	B737-400	150500	38	0
4	B737-700	155000	784	0
5	B737-800	174700	130	0
6	C-130	155000	117	0
7	Learjet 45/55B	21500	46	0
8	DC9-32	109000	68	0

Federal Aviation Administration FAARFIELD 2.0 Section Report

FAARFIELD 2.0.7 (Build 09/14/2021)

Job Name: Kotzebue CBR=10 Actual Traffic

Section: Actual Traffic

Analysis Type: New Flexible

Last Run: Thickness Design 2022-03-15 23:08:58

Design Life = 20 Years

Total thickness to the top of the subgrade = 17.1in.

Pavement Structure Information by Layer

No.	Type	Thickness in.	Modulus psi	Poisson's Ratio	Strength R psi
1	P-401/P-403 HMA Surface	4.0	200000	0.35	0
2	P-401/P-403 HMA Stabilized	5.7	400000	0.35	0
3	P-154 Uncrushed Aggregate	7.3	19746	0.35	0
4	Subgrade	0	15000	0.35	0

Airplane Information

No.	Name	Gross Wt. lbs	Annual Departures	% Annual Growth
1	B737-200	116000	256	0
2	B737-300	140000	100	0
3	B737-400	150500	38	0
4	B737-700	155000	784	0
5	B737-800	174700	130	0
6	C-130	155000	117	0
7	Learjet 45/55B	21500	46	0
8	DC9-32	109000	68	0

B.4 BARROW DESIGN SUMMARY BASED ON ADVISORY CIRCULAR 150/5320-6D

FLEXIBLE PAVEMENT DESIGN FOR program date 5/15/03

Barrow *AC Method*

Barrow, AK

Engineer - Applied Research Associates AIP No.

12.5" **Total Thickness Required (inches)**

No thickness adjustments required

Stabilized Base/Subbase Are Required

Initial Pavement Cross Section	
4"	Pavement Surface Layer (P-401)
9"	Base Layer (P-209)
-0.5"	Subbase #1 (P-154) CBR= 20
0"	Subbase #2 CBR= 0
0"	Subbase #3 CBR= 0

Stabilized or Modified Cross Section		Factors
4"	P-401 Plant Mix Bituminous Pavements	
7.5"	P-401, Plant Mix Bituminous Pavements	1.2
1.5*"	Not stabilized -- P-154	1
0"	Material as defined by user	
0"	Material as defined by user	

*13" thickness required to cover CBR=20

Frost Considerations

110 lb/cf	Dry Unit Weight of Soil	
4500	Degree Days °F	
138"	Frost Penetration Depth	
20	Original CBR value of subgrade Soil	
20	CBR Value used for the Subgrade Soil	Non-Frost Code for Subgrade Soil
20	CBR Value used for subbase #1	Non-Frost code for Subbase #1
0	CBR Value used for subbase #2	Non-Frost code for Subbase #2
0	CBR Value used for subbase #3	No frost selection made for Subbase #3

Design Aircraft Information

The Design Aircraft is a DUAL150 - 150,000 lbs -- ( )

150000 lbs Gross Weight **20** Design Life (years)

1,373 Equivalent Annual Departures

Subgrade Compaction Requirements for Design Aircraft

Non-Cohesive Soils Compaction	Depth Required	Cohesive Soils Compaction	Depth Required
100%	0 - 20	95%	0 - 8"
95%	20 - 34.5"	90%	8 - 15"
90%	34.5 - 49.5"	85%	15 - 22.5"
85%	49.5 - 64.5"	80%	22.5 - 30"

This software is currently under development and is not officially adopted as a FAA standard. Designs developed using this program should be checked against AC 150/5320-6D to insure accuracy and conformance to existing

B.5 BARROW DESIGN SUMMARY BASED ON ADVISORY CIRCULAR 150/5320-6G

Federal Aviation Administration FAARFIELD 2.0 Section Report

FAARFIELD 2.0.7 (Build 09/14/2021)

Job Name: Barrow CBR=20 -Actual Traffic

Section: New Section 1

Analysis Type: HMA on Aggregate

Last Run: Thickness Design 2022-01-21 11:00:24

Design Life = 20 Years

Total thickness to the top of the subgrade = 18.4in.

Pavement Structure Information by Layer

No.	Type	Thickness in.	Modulus psi	Poisson's Ratio	Strength R psi
1	P-401/P-403 HMA Surface	4.5	200000	0.35	0
2	P-209 Crushed Aggregate	9.9	73519	0.35	0
3	P-154 Uncrushed Aggregate	4.0	28116	0.35	0
4	Subgrade	0	30000	0.35	0

Airplane Information

No.	Name	Gross Wt. lbs	Annual Departures	% Annual Growth
1	B737-200	116000	169	0
2	B737-300	140000	36	0
3	B737-400	150500	258	0
4	B737-700	155000	183	0
5	B737-800	174700	231	0
6	C-130	155000	7	0
7	DC9-32	109000	47	0
8	MD-83	161000	21	0

Federal Aviation Administration FAARFIELD 2.0 Section Report

FAARFIELD 2.0.7 (Build 09/14/2021)

Job Name: Barrow CBR=20 -Design Traffic

Section: New Section 1

Analysis Type: HMA on Aggregate

Last Run: Thickness Design 2022-01-21 10:41:10

Design Life = 20 Years

Total thickness to the top of the subgrade = 18.3in.

Pavement Structure Information by Layer

No.	Type	Thickness in.	Modulus psi	Poisson's Ratio	Strength R psi
1	P-401/P-403 HMA Surface	4.0	200000	0.35	0
2	P-209 Crushed Aggregate	10.3	74379	0.35	0
3	P-154 Uncrushed Aggregate	4.0	28116	0.35	0
4	Subgrade	0	30000	0.35	0

Airplane Information

No.	Name	Gross Wt. lbs	Annual Departures	% Annual Growth
1	B737-400	150500	1209	0
2	C-130	155000	25	0
3	B727-200 Advanced Basic	185200	90	0

Additional Airplane Information

Subgrade CDF

No.	Name	CDF Contribution	CDF Max for Airplane	P/C Ratio
1	B737-400	0.00	0.00	1.46
2	C-130	0.00	0.00	2.32
3	B727-200 Advanced Basic	0.00	0.00	1.37