Annex H

Railroad and Roadway Damage Repair Plan

Annex H: Railroad and Roadway Damage Repair Plan

H.1 Introduction

This portion of the study addresses of the potential effects of drawdown on railroad and roadway embankments from the Snake River's confluence with the Columbia River to the Idaho state line. Those effects are settlement and slope stability directly impacted by the drawdown of the reservoir. Problems and anticipated modifications required to resist the erosive forces of the river on the embankments are described in Annex F.

There is no doubt that many of the railroad and highway embankments will be damaged as a result of rapid reservoir drawdown. As drawdown occurs, areas of the embankments along the river are anticipated to fail due to steep slopes, saturated soils, and pore pressure increase. This annex describes the critical elements that contribute to embankment failures from rapid drawdown. It summarizes the observations from the 1992 test drawdown and, from those observations, projects damages resulting from a full reservoir drawdown. It discusses the necessity and impacts of the selected drawdown rate.

H.2 Review of 1992 Drawdown

A test of the reservoir drawdown concept was performed in March 1992, using Lower Granite and Little Goose Dams. The purpose of the test was to gather information regarding the effects of substantially lowering existing reservoirs. The drawdown test was scheduled to be completed within the month of March in order to minimize potential negative impacts to Snake River migrating fish. On March 1 the Lower Granite reservoir was drafted from its starting point of normal minimum operating pool (elevation 223.4 meters [733 feet]) at a rate of 0.6 meters per day for 14 days. Elevation 214.9 meters (705 feet) was achieved on March 15. During subsequent phases. Little Goose reservoir was lowered a total of 3.8 meters and Lower Granite Reservoir was further lowered to elevation 212.4 meters (697 feet) for a total drawdown of 11.0 meters.

During the drawdown the Corps monitored road and railroad embankments along the two reservoirs for potential problems. The following damage on the Lower Granite reservoir was reported:

- Camas Prairie Railroad (CPRR) embankment experienced cracking, movement, and track misalignment;
- Whitman County Road 9000 embankment experienced extensive movement and cracking in 33 areas (cracks varied in width from a few millimeters to 0.4 meters, and some over 60 meters in length) and damage to roadway and guardrail;
- State Highway 193 between Steptoe Canyon and Red Wolf bridge experienced cracking and movement;
- U.S. Highway 12 had two small slides (generally minor) near Red Wolf Marina and soil piping was noted; and
- Cracking and movement of the road and railroad embankments disturbed many survey monuments.

It was noted that most of the sliding activity associated with the drawdown occurred within slopes consisting of natural deposits of silts, sands, and gravels. For the purposes of this study, stability of natural slopes was not addressed, and efforts focused on man-made embankments. Drawdown of each reservoir of up to 30 meters cannot be assumed to occur without embankment failures.

H.3 Embankment Geometry and Material Considerations

The key to understanding how embankments will behave under drawdown conditions is to understand the embankment materials. Embankments constructed from materials that are so “free-draining” that the soil saturation level falls quickly will have increased stability under drawdown conditions. Stability is decreased if the soil saturation level lags behind the reservoir drawdown level. Therefore, the rate of drawdown associated with a minimal lag is related to the “free draining” ability of embankment materials. Greater permeability and porosity of soils results in a greater ability of the material to be “free draining.” Although a material may be free draining, the rate of reservoir drawdown may be too fast, resulting in a greater saturation level lag and reduced embankment stability.

The man-made embankments along the lower Snake River are, in general, constructed from locally borrowed materials, and were not subject to the same quality control efforts (grain size and compaction control) which were used in construction of major embankment dams. Also, internal drainage features such as pipes or clean stone drains were not incorporated into the designs. According to railroad and roadway relocation reports and drawings, many embankments were constructed from “random fill” or “granular fill” materials. Compaction was probably used in placing these materials, but it is not clear how much compactive effort was used and what methods were employed. The nature of “random fill” available for borrow in the vicinity of the lower Snake River varies, although the material is predominately sand and gravel with varying amounts of fines (silts and clays passing the No. 200 sieve) and cobbles. The CPRR relocation report (Lower Granite DM 9.2) states that embankment foundations along the relocated alignments consists of bedrock or materials described as relatively clean talus rock, silty talus rock, alluvial material, and wind-deposited sand and silts. Similar materials were used for construction of the relocated road and railroad embankments.

The amount of fines controls the ability of an embankment material to be “free draining,” and the amounts of fines in silty talus rock and wind-deposited sands and silts could be significant enough to preclude free draining conditions. Alluvial materials obtained from local terrace gravel deposits and clean talus rock materials likely consist of a predominantly granular mixture of sand, gravel, and cobbles, with a lower percentage of fines than the silty materials. Although aeolian silt often exists on the ground surface of the terrace gravel deposits, it is not likely that significant amounts of fines are present in the alluvial random fill mixtures. The ability of the embankments to be free draining, and therefore more stable during drawdown, depends on the borrow source used to construct the embankments.

Man-made embankments were generally constructed with slopes of 2h:1v, with riprap or rockfill slope protection within the normal reservoir surface operating range. Some embankments, particularly on the Ice Harbor reservoir, have buttress fills against the toe of embankments with slopes of 2.5h:1v to 3h:1v. The embankments along the reservoirs have various top and toe elevations, and the drawdown range will vary from approximately 30 meters just upstream of each dam to nearly no drawdown, or possibly a slight increase in water level, just downstream of each dam. There are many embankment and drawdown rate configurations, and when the variations in embankment geometry, material types and compaction criteria are considered, there are an infinite number of material parameter and geometric combinations.

H.4 Rate of Reservoir Drawdown

The man-made embankments along the four lower Snake River reservoirs were constructed by various entities (including the federal government, state transportation department, and railroad companies) over an extended period of time. Embankment characteristics which vary include the method of embankment construction, embankment geometry, materials used in the embankments, surrounding land topography, embankment foundation materials, and vertical distance of drawdown from the normal reservoir surface elevation. All of these characteristics result in embankments which will behave differently under a drawdown scenario. Behavior may vary from no visible movement or damage to few tension cracks and minor movement or sloughing, to the extreme case of slope failure with extensive movement.

The rate of reservoir drawdown is an important parameter in establishing the schedule for overall embankment dam removal and reservoir drawdown. There are several biological and weather factors which influence the beginning, end, and duration of drawdown. The primary constraint in determining the rate of drawdown is the time period during which the reservoir must be lowered and the embankment removed. Reservoir evacuation cannot begin in any year prior to 1 August. This is because the spring runoff flows extend into June and July and downstream fish migration continues until this time. By January of any year the probability of high flows in the river increases dramatically. These beginning and end point constraints require that the drawdown to be done during this 5-month period. This time is further reduced to allow sufficient time to excavate the embankment and remove cofferdams.

The drawdown rate will be controlled at each dam by the spillway and powerhouse gates. Consequently, a nominal drawdown rate of 0.6 meter (2 feet) per day has been assumed for feasibility level construction planning. While some latitude may be possible as designs and schedules are further developed, the drawdown rate of 0.6 meter per day may only be slightly reduced.

H.5 Methods

The location and extent of embankment failures is extremely difficult to predict based on the uncertainty and variability of materials and methods used in constructing the embankments. However, embankment damage data from the 1992 drawdown of Lower Granite was useful in making such predictions. Table H1 summarizes the specific areas where damage was observed after the 1992 test drawdown. A rational methodology was desired to determine potential damages and subsequent repairs. To estimate the potential for road and railroad embankment failures from observed embankment distress, the study team made the following assumptions:

1. Drawdown would remove hydrostatic support from saturated materials.
2. The sections anticipated to undergo settlement are those that are in similar physical positions (height and distance) as the sections that exhibited settlement along the Lower Granite Reservoir during the 1992 drawdown.
3. The anticipated failure type and characteristics are theoretical and are based on an infinite-slope analysis. Some parameters are based on field observation, and some are based on information resources such as topographic maps and aerial photographs.

Table H1. Measurements of Distress from Observations of 1992 Drawdown page 1 of 2

Station	Station Location	Feature	Description	Natural Slope (%)	Distance From River (ft)	Height Above River (ft)	Embankment Slope (%)	Materials
2431+14	Rd. 9000	Pavement Crack	149 ft long, 1 in wide	30	50	30	60	0 ft to 14 ft: silt with scattered rock fragments
2452+26	Rd. 9000	Pavement Crack	58 ft long, 1/2 in wide	17	50	20	40	0 ft to 15 ft: rock fragments in sandy silt matrix
2457+54	Rd. 9000	Pavement Crack	19 ft long, 1/4 in wide	18	50	20	40	0 ft to 15 ft: fine sandy silt with rock fragments
2552+58	Rd. 9000	Pavement Crack	422 ft long, 10 in wide	4	50	20	40	0 ft to 15 ft: fine sandy silt with rock fragments
2605+38	Rd. 9000	Pavement Crack	248 ft long, 1 ft wide	60	30	20	60	0 ft to 15 ft: interbedded silt and sand
2605+38	Rd. 9000	Pavement Crack	63 ft long, 1/4 in wide	60	30	20	60	0 ft to 15 ft: interbedded silt and sand
2626+50	Rd. 9000	Pavement Crack	341 ft long, 9 in wide	6	50	20	40	0 ft to 14 ft: silt with scattered rock fragments
2637+06	Rd. 9000	Pavement Crack	154 ft long, 3 in wide	13	20	10	50	0 ft to 15 ft: silt with scattered rock fragments
2684+58	Rd. 9000	Pavement Crack	80 ft long, 1/4 in wide	8	50	20	40	0 ft to 14 ft: sandy silt
2710+98	Rd. 9000	Pavement Crack	24 ft long, 6 in wide	27	50	20	40	0 ft to 15 ft: silt with scattered rock fragments
2742+66	Rd. 9000	Pavement Crack	221 ft long, 3/4 in wide	50	30	20	65	0 ft to 14 ft: silt with scattered rock fragments
2753+22	Rd. 9000	Pavement Crack	45 ft long, 2 in wide	17	30	20	65	0 ft to 14 ft: rock fragments in silty and ash matrix
2753+22	CPRR	Pavement Crack	197 ft long, 15 in wide	17	30	20	65	0 ft to 14 ft: rock fragments in silty and ash matrix
2758+50	CPRR	Pavement Crack	33 ft long, 6 in wide	30	30	20	65	0 ft to 14 ft: rock fragments in silty and ash matrix
2758+50	CPRR	Pavement Crack	51 ft long, 7 in wide	30	30	20	65	0 ft to 14 ft: rock fragments in silty and ash matrix
2763+78	Rd. 9000/ CPRR	Pavement Crack	191 ft long, 6 in wide	25	40	20	50	0 ft to 40 ft: interbedded silt and sand
2763+78	Rd. 9000	Pavement Crack	48 ft long, 2 in wide	25	40	20	50	0 ft to 40 ft: interbedded silt and sand
2779+62	Rd. 9000	Pavement Crack	81 ft long, 6 in wide	18	50	20	40	0 ft to 3 ft: sand and gravel, 3 ft +: bedrock
2784+90	Rd. 9000	Pavement Crack	118 ft long, 13 in wide	12	40	20	50	0 ft to 14 ft: rock fragments in silty matrix

Table H-1, continued. Measurements of Distress from Observations of 1992 Drawdown

page 2 of 2

Station	Station Location	Feature	Description	Natural Slope (%)	Distance From River (ft)	Height Above River (ft)	Embankment Slope (%)	Materials
2784+90	Rd. 9000	Pavement Crack	102 ft long, 4 in wide	12	40	20	50	0 ft to 14 ft: rock fragments in silty matrix
2784+90	Rd. 9000	Pavement Crack	228 ft long, 13 in wide	12	40	20	50	0 ft to 14 ft: rock fragments in silty matrix
2790+18	Rd. 9000	Pavement Crack	289 ft long, 7 in wide	40	50	20	40	0 ft to 14 ft: rock fragments in silty matrix
2800+74	Rd. 9000	Pavement Crack	313 ft long, 11 in wide	17	50	20	40	0 ft to 14 ft: rock fragments in silty matrix
2806+02	Rd. 9000	Pavement Crack	116 ft long, 9 in wide	40	30	20	65	0 ft to 14 ft: rock fragments in silty matrix
2806+02	Rd. 9000	Pavement Crack	254 ft long, 10 in wide	40	30	20	65	0 ft to 14 ft: rock fragments in silty matrix
2811+30	Rd. 9000	Pavement Crack	241 ft long, 1 in wide	10	50	20	40	0 ft to 14 ft: rock fragments in silty matrix
2816+58	Rd. 9000	Pavement Crack	56 in long, 1/8 in wide	20	60	30	50	0 ft to 14 ft: rock fragments in silty matrix
2849+94	Rd. 9000	Pavement Crack	50 ft long, 1/4 in wide	30	50	20	40	0 ft to 14 ft: rock fragments in silty matrix
2890+50	Rd. 9000	Pavement Crack	204 ft long, 1/4 in wide	26	30	10	30	0 ft to 14 ft: rock fragments
2901+06	Rd. 9000	Pavement Crack	253 ft long, 5 in wide	19	40	15	40	0 ft to 14 ft: rock fragments in silty matrix
2948+58	Rd. 9000	Pavement Crack	15 ft long, 1/4 in wide	15	40	15	40	3 ft to 6 ft: gravel 6 ft to 12 ft: silt
2953+86	CPRR	Pavement Crack	123 ft long, 6 in wide	4	150	20	13	volcanic ash, silt, and sand
2959+14	CPRR	Pavement Crack	30 ft long, 4 in wide	7	50	20	40	0 ft to 4 ft: talus and colluvium 4 ft+: bedrock
2959+14	Rd. 9000	Pavement Crack	162 ft long, 14 in wide	7	50	20	40	0 ft to 4 ft: talus and colluvium 4 ft+: bedrock
2959+14	Rd. 9000	Pavement Crack	758 ft long, 14 in wide	5	50	20	40	0 ft to 35 ft: interbedded silt and sand
2964+42	Rd. 9000	Pavement Crack	278 ft long, 2 in wide	18	60	20	30	0 ft to 35 ft: interbedded silt and sand

The team developed materials estimates for making repairs to the road and railroad embankments using the following assumptions:

1. The dimensions for road and railroad cross sections were assumed to be the same as the typical sections used for the road and railroad relocations prior to reservoir establishment. The team also assumed that road and railroad embankments would be constructed with materials meeting current standards.
2. Material sources were selected from existing sources identified on maps and aerial photographs. All sources were assumed to be available for use and no ownership issues were considered. Haul distances were based on sources shown on maps and aerial photographs.
3. The embankment repair quantities were assumed to be cumulative for each project.
4. Since the water level would be far below the structures, the team assumed that riprap would only be needed for shoreline protection in the active water surface zone.
5. Quantities were based on the following thicknesses:
    - Asphalt surfacing - 75 millimeters
    - Surface course - 150 millimeters
    - Base course - 300 millimeters
    - Ballast - 900 millimeters
    - Sub-ballast - 300 millimeters.

Combinations of theoretical and practical methods were used to evaluate potential railroad and roadway damage during drawdown. Practical methods were based on observations made during the 1992 Lower Granite Reservoir drawdown. The drawdown test section consisted of Whitman Co. Road No. 9000 and the Camas Prairie Railroad along the Lower Granite Reservoir (Steptoe Canyon to Wawawai Canyon). It appeared that many failures occurred along the contact between the structure fill and the natural foundation material. At other locations, it was evident that the failure extended into the foundation material. Therefore, both modes of failure had to be taken into account. The measurements taken at the time of the observations are summarized in Table H1.

Also, from the observations along the test section, it was evident that nearly all failures occurred at locations that were within 15 meters horizontal distance and 6-meter vertical distance of the reservoir perimeter, and on slopes less than 50 percent (greater than 50 percent would indicate shallow bedrock and greater stability). Therefore, the study team concluded that sections along the river in similar positions with similar physical characteristics would display a similar response. The team also assumed that sections at a horizontal distance of 15 meters to 30 meters and vertical distance greater than 6 meters from the reservoir would display only about 10 percent of the failures of the more closely adjacent sections. The areas of settlement within the test section along the Lower Granite Reservoir are marked on 1 inch = 1,000 feet maps, contract drawing maps, and copies of aerial photographs in the *1992 Reservoir Drawdown Test, Lower Granite and Little Goose Dam* (Corps, 1993). Using U.S. Geological Survey, 7.5 minute, 1:24,000 scale quadrangle maps, the study team delineated the sections in both modes of failure types and measured the approximate distance in feet of each.

The study team estimated that a total of 68 potential failure areas could result. These anticipated failure areas are shown in Table H2.

The study team also used a theoretical approach to determine the possibility of failure of natural slopes. Using the infinite slope equations for slope stability, the team calculated the factors of safety according to the following parameters:

- Slopes: 10 to 50 percent
- Soil: silt (classified as ML) with scattered cobbles and boulders
- Angle of internal friction: 30 degrees
- Height of phreatic surface above bedrock: 0.0 meter to 4.5 meters
- Saturated density: 1,954 kg/m^3
- Moist density (10 percent moisture content): 1,666 kg/m^3
- Depth to bedrock: 4.5 meters

While holding other parameters constant, the slope and height of the phreatic surface was varied according to the limits expressed above. Slopes range from 10 percent to 50 percent and are shown in radians. The phreatic surface ranges from 0.0 meter to 4.5 meters (anticipated ground surface) above the bedrock surface. The resulting factors of safety are shown in Table H3. The data shown indicate that, at slopes greater than about 30 percent, the factor of safety drops below one when the phreatic surface remains at the ground surface. Typical rates of permeability for silts and sandy silt mixtures (3.5 by 10^{-5} m^3/s or less) show that the phreatic surface would remain at the ground surface for a reservoir lowering rate of 2 feet per day, creating conditions of slope instability for slopes greater than 30 percent. For slopes of 40 percent and 50 percent, the instability would be much greater.

The study team devised a typical anticipated small failure from the observed data of the 1992 drawdown and a theoretical model based on natural slope instability. The following parameters were used:

- Length: 25.9 meters
- Width: 3.7 meters
- Depth: 1.5 meters

A cross section of the anticipated typical failure is shown in Figure H1. The quantities of construction materials for repair were calculated for the model using typical cross sections developed for the relocation of the County Road 9000 and the Camas Prairie Railroad. The quantities of the repair materials were then calculated for all projected small failures along the Snake River by multiplying the unit quantities (cubic meters per meter) by the number of feet of projected failure (also shown in Figure H1).

Figure H2 shows the cross section of a hypothetical large failure. The failure criteria, dimensions, and associated construction material quantities are also shown in Figure H2. It is anticipated that there would be at least two large failures on both the Little Goose and Lower Granite reservoirs, and one large failure on both the Ice Harbor and Lower Monumental reservoirs.

Table H2. Potential Failure Areas Resulting from a Permanent Drawdown page 1 of 4

Feature	Location	Legal Description	Potential Failure Segment (m)	Class	Estimated Failure Length (m)	Mat. So. No.	Cubic Meters Required	Haul (kilometers)
Ice Harbor Reservoir								
BNRR	North Bank	S18, T9N, R32E	121.9	Low	1.4	1.0	107.8	4.6
BNRR	North Bank	S18, T9N, R32E	182.9	High	20.6	1.0	1,617.1	3.8
BNRR	North Bank	S18, T9N, R32E	91.4	Low	1.0	1.0	81.0	3.0
BNRR	North Bank	S18, T9N, R32E	152.4	High	17.2	1.0	1,349.5	2.4
BNRR	North Bank	S7, T9N, R32E	304.8	Low	3.4	1.0	269.9	2.3
BNRR	North Bank	S8, T9N, R32E	487.7	High	55.0	1.0	4,320.0	1.2
BNRR	North Bank	S4,5, T9N, R32E	1,066.8	Low	12.0	1.0	945.0	1.2
BNRR	North Bank	S4, T9N, R32E	182.9	High	20.6	1.0	1,620.2	2.3
BNRR	North Bank	S3,T9N, R32E	335.3	Low	3.4	1.0	269.9	2.4
BNRR	North Bank	S34,T10N, R32E	152.4	Low	1.7	1.0	134.6	3.3
BNRR	North Bank	S26,T10N, R32E	152.4	High	17.2	2.0	1,349.5	1.7
BNRR	North Bank	S26,T10N, R32E	1,066.8	Low	12.0	2.0	945.0	2.4
BNRR	North Bank	S23,S26,T10N, R32E	1,371.6	Low	15.5	2.0	1,215.7	0.9
BNRR	North Bank	S24,T10N, R32E	274.3	Low	3.1	2.0	243.9	0.6
BNRR	North Bank	S13,T10N, R32E	274.3	High	30.9	3.0	2,429.1	0.3
BNRR	North Bank	S12,T10N, R32E	792.5	Low	8.9	3.0	701.1	2.1
BNRR	North Bank	S4,T10N, R33E	701.0	Low	6.2	3.0	488.6	6.7
BNRR	North Bank	S27,34, T11N, R33E	1,371.6	Low	15.5	3.0	1,215.7	14.6
BNRR	North Bank	S14,23, T11N, R33E	670.6	Low	7.6	4.0	593.3	11.0
Burr Cyn. Rd.	North Bank	S19, T12N, R34E	121.9	Low	1.4	4.0	107.8	4.6
Burr Cyn. Rd.	North Bank	S18, T12N, R34E	426.7	High	24.1	4.0	1,890.1	3.7
Burr Cyn. Rd.	North Bank	S 8,17, T12N, R34E	548.6	High	61.9	4.0	4,858.3	2.4
Wilson Cyn. Rd.	North Bank	S4,9, T12N, R34E	2,438.4	High	275.1	4.0	21,596.1	1.2
Gravel Road	South Bank	S19, T9N, R32E	609.6	High	68.8	13.0	5,398.8	0.6

Table H-2, continued. Potential Failure Areas Resulting from a Permanent Drawdown page 2 of 4

Feature	Location	Legal Description	Potential Failure Segment (m)	Class	Estimated Failure Length (m)	Mat. So. No.	Cubic Meters Required	Haul (kilometers)
UPRR	South Bank	S9, T9N, R32E	1,828.8	Low	20.6	14.0	1,620.2	0.9
UPRR	South Bank	S3,4, T9N., R32E	1,828.8	High	206.3	14.0	16,196.5	1.2
UPRR	South Bank	S2, T9N., R32E	1,676.4	High	189.1	14.0	14,847.0	3.7
UPRR	South Bank	S36, T10N, R32E	396.2	High	44.7	14.0	3,508.0	4.9
UPRR	South Bank	S8, T10N, R33E	1,524.0	Low	1.7	15.0	133.8	1.2
UPRR	South Bank	S34, T11N, R33E	701.0	Low	7.9	15.0	619.3	3.7
UPRR	South Bank	S26, T11N, R33E	762.0	Low	8.6	16.0	675.1	2.4
UPRR	South Bank	S24, T11N, R33E	609.6	High	68.8	16.0	5,398.8	0.2
UPRR	South Bank	S12, T11N, R33E	3,048.0	High	343.8	16.0	26,995.0	2.3
UPRR	South Bank	S30,31, T12N, R33E	3,048.0	Low	3.4	19.0	270.7	0.9
UPRR	South Bank	S17,19, T12N, R34E	5,181.6	High	402.3	17.0	31,590.2	2.1
UPRR	South Bank	S8,9, T12N, R.34E	1,219.2	Low	13.7	17.0	1,079.6	0.5
Lower Monumental Reservoir								
UPRR	South Bank	S35,36, T13N, R34E	1,828.8	High	206.3	18.0	16,201.9	0.8
UPRR	South Bank	S30,36, T13N, R34, 35E	1,219.2	High	137.5	20.0	10,793.1	0.9
UPRR	South Bank	S26,27,.28,29, T13N, R35E	7,315.2	High	825.1	21, 22	64,783.8	1.8
UPRR	South Bank	S21, T13N, R36E	304.8	High	34.4	23.0	2,699.0	1.6
UPRR	North Bank	S2,3, T12N, R37E	1,524.0	Low	17.2	5.0	1,349.5	4.0
UPRR	North Bank	S36, T13N, R37E; S31, T13N, R38E	1,524.0	Low	17.2	5.0	1,349.5	1.2
HWY. 261	South Bank	S3,4, T12N, R37E	609.6	High	68.8	25.0	5,398.8	4.7
Deadman Creek Rd.	South Bank	S32,33, T13N, R38E	609.6	High	68.8	26.0	5,398.8	1.8
Little Goose Reservoir								
CPRR	North Bank	S22,23, T13N, R38E		Low	5.2	5a	406.8	2.7
CPRR	North Bank	S22,23, T13N, R38E	1,066.8	High	120.4	5a	9,452.7	2.7

Table H2, continued. Potential Failure Areas Resulting from a Permanent Drawdown page 3 of 4

Feature	Location	Legal Description	Potential Failure Segment (m)	Class	Estimated Failure Length (m)	Mat. So. No.	Cubic Meters Required	Haul (kilometers)
CPRR	North Bank	S24, T13N, R38E	243.8	Low	2.7	5b	215.6	2.4
CPRR	North Bank	S19,24, T13N, R38E	1,066.8	High	120.4	5b	9,453.5	0.9
CPRR	North Bank	S20,21, T13N, R38E	1,524.0	High	171.9	5b	13,497.5	2.4
CPRR	North Bank	S22, T13N, R38E	457.2	High	51.8	5b	59,398.7	4.6
CPRR	North Bank	S7,11,14,23, T13N, R39,40E	2,590.8	High	756.5	5b	59,399.5	8.5
CPRR	North Bank	S7,12,14,23, T13N, R39,40E	1,219.2	Low	1.0	5b	81.0	10.4
CPRR	North Bank	S13,14,22,23,27, T14N, R40E	4,876.8	High	550.2	6.0	43,198.4	3.1
CPRR	North Bank	S13,17,18, T14N, R40,41E	1,524.0	High	171.9	7.0	13,497.5	3.1
CPRR	North Bank	S15,16,17, T14N, R41E	3,962.4	High	446.8	9.0	35,083.7	1.8
CPRR	North Bank	S20, T14N, R42E	1,219.2	High	137.5	9.0	10,798.4	8.5
CPRR	North Bank	S20,21, T14N, R42E	914.4	Low	10.3	9.0	808.9	8.5
CPRR	North Bank	S13,14,23, T14N, R42E	3,048.0	Low	343.8	9.0	26,995.0	15.3
CPRR	North Bank	S13,18,19, T14N, R42E,43E	1,828.8	High	206.3	10.0	16,197.3	14.6
Hwy 127	South Bank	S9, T13N, R40E	1,219.2	High	137.5	27.0	10,797.7	1.2
Deadman Creek Rd.	South Bank	S18,19,30, T14N, R43E	1,219.2	Low	13.7	28.0	1,079.6	1.5
Lower Granite Reservoir								
CPRR	North Bank	S33,34, T14N, R43E S2, T13N, R43E	4,267.2	High	481.3	10.0	37,788.1	9.1
Test Section	North Bank	Wawawai Creek to Steptoe Creek	16,254.4	High	1833.4	10 and 11	143,951.2	5.7
BNRR	North Bank	Steptoe Creek to RM 138.4	16,459.2	Low	185.9	11.0	14,598.5	4.0

Table H2, continued. Potential Failure Areas Resulting from a Permanent Drawdown page 4 of 4

Feature	Location	Legal Description	Potential Failure Segment (m)	Class	Estimated Failure Length (m)	Mat. So. No.	Cubic Meters Required	Haul (kilometers)
Whitman Co. Rd. 9000	North Bank	Steptoe Creek to RM 138.4	11,582.4	High	1306.4	11.0	102,571.9	6.4
Hwy 12	South Bank	Alpowa Creek to Red Wolf Bridge	10,972.8	Low	123.7	29 and 30	9,716.5	5.2
Hwy 129	West Bank	RM 140.5 to 143	5,486.4	High	618.7	32.0	48,581.9	5.2
Nez Perce Co. Rd.	East Bank	Hwy 12 to RM 143	5,486.4	Low	62.2	31.0	4,882.0	3.3

Table H3. Factors of Safety for Slope Stability

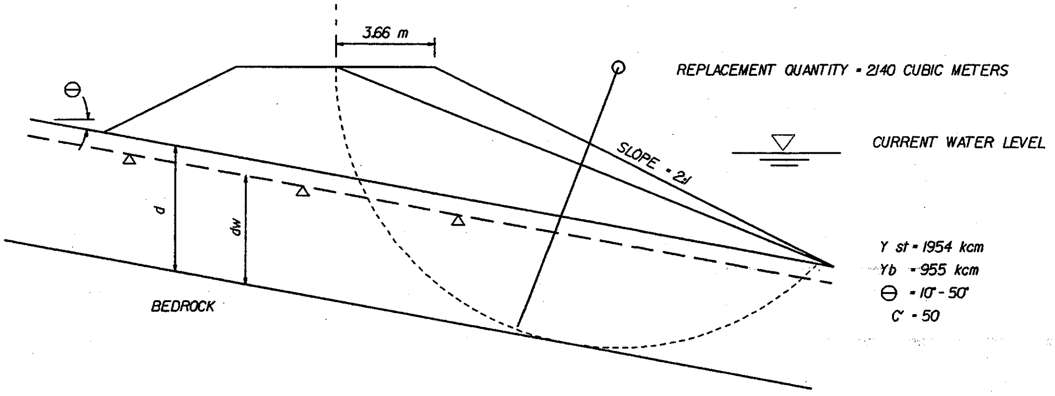
Degree Slope	Saturated Material Thickness (m)	Factor of Safety	Degree Slope	Saturated Material Thickness (m)	Factor of Safety	Degree Slope	Saturated Material Thickness (m)	Factor of Safety	Degree Slope	Saturated Material Thickness (m)	Factor of Safety	Degree Slope	Saturated Material Thickness (m)	Factor of Safety
5.7	0.0	6.11	11.3	0.0	3.06	16.7	0.0	2.04	21.8	0.0	1.54	26.6	0.0	1.23
5.7	0.3	5.88	11.3	0.3	2.94	16.7	0.3	1.96	21.8	0.3	1.48	26.6	0.3	1.19
5.7	0.6	5.65	11.3	0.6	2.83	16.7	0.6	1.89	21.8	0.6	1.42	26.6	0.6	1.14
5.7	0.9	5.43	11.3	0.9	2.72	16.7	0.9	1.81	21.8	0.9	1.37	26.6	0.9	1.10
5.7	1.2	5.21	11.3	1.2	2.61	16.7	1.2	1.74	21.8	1.2	1.31	26.6	1.2	1.05
5.7	1.5	5.00	11.3	1.5	2.50	16.7	1.5	1.67	21.8	1.5	1.26	26.6	1.5	1.01
5.7	1.8	4.79	11.3	1.8	2.40	16.7	1.8	1.60	21.8	1.8	1.21	26.6	1.8	0.97
5.7	2.1	4.59	11.3	2.1	2.30	16.7	2.1	1.53	21.8	2.1	1.16	26.6	2.1	0.93
5.7	2.4	4.39	11.3	2.4	2.20	16.7	2.4	1.47	21.8	2.4	1.11	26.6	2.4	0.89
5.7	2.7	4.19	11.3	2.7	2.10	16.7	2.7	1.40	21.8	2.7	1.06	26.6	2.7	0.85
5.7	3.0	4.00	11.3	3.0	2.00	16.7	3.0	1.34	21.8	3.0	1.01	26.6	3.0	0.81
5.7	3.4	3.81	11.3	3.4	1.91	16.7	3.4	1.28	21.8	3.4	0.96	26.6	3.4	0.77
5.7	3.7	3.63	11.3	3.7	1.82	16.7	3.7	1.22	21.8	3.7	0.92	26.6	3.7	0.74
5.7	4.0	3.45	11.3	4.0	1.73	16.7	4.0	1.16	21.8	4.0	0.87	26.6	4.0	0.70
5.7	4.3	3.28	11.3	4.3	1.64	16.7	4.3	1.10	21.8	4.3	0.83	26.6	4.3	0.67
5.7	4.6	3.10	11.3	4.6	1.55	16.7	4.6	1.04	21.8	4.6	0.78	26.6	4.6	0.63

H.6 Conclusions

Drawdown would cause significant damage to road and railroad embankments. Most embankment failures are expected to occur after the reservoirs are significantly drawn down, when the excess weight of the water in the embankment materials would cause a failure. Temporary road detours may be required during and after drawdown to allow vehicle traffic to use roadways. However, railroad embankment failures may result in a shut down of rail traffic until repairs can be made. Rapid response approach to railroad repairs will be critical to minimizing the impacts of interruption of rail service.

H.7 Construction Schedule

Embankment repairs cannot be performed until after drawdown is accomplished. Also, in some areas, it may be necessary to wait several weeks after drawdown to allow the materials to drain and stabilize before repairs can be initiated. The exact number and extent of failures cannot be predicted prior to drawdown. Therefore, multiple equipment rental contracts would be awarded prior to drawdown, allowing repairs to be performed as failures occur. It is anticipated that most damage and consequent repairs would be completed within a few months and up to 1 year after drawdown is complete.



ASSUMED TYPICAL FAILURE SECTION

NOT TO SCALE

DESIGN CRITERIA AND TYPICAL SMALL FAILURE PARAMETERS

I. SLIDE CHARACTERISTICS
  - A. LENGTH = 25.9 METERS
  - B. WIDTH = 3.7 METERS
  - C. DEPTH (VERTICAL DISPLACEMENT) = 1.5 METERS

II. EXCAVATION
  - A. QUANTITY OF MATERIAL ABOVE FAILURE ARC = 2140 CU. METERS

III. AVERAGE HAUL DISTANCES
  - A. ICE HARBOR POOL = 11.3 KILOMETERS
  - B. LOWER MONUMENTAL POOL = 11.7 KILOMETERS
  - C. LITTLE GOOSE POOL = 12.9 KILOMETERS
  - D. LOWER GRANITE POOL = 12.9 KILOMETERS

IV. MATERIAL TYPES AND QUANTITIES FOR SINGLE FAILURE
  - A. ROAD (8.5 METERS WIDE)
    - 1. SURFACING, COLD MIX ASPHALT = 7.3 CUBIC METERS
    - 2. SURFACE COURSE = 14.4 CUBIC METERS
    - 3. BASE COURSE = 11.5 CUBIC METERS
    - 4. FILL = 2090 CUBIC METERS
      - a. USE IN-PLACE MATERIAL. EXCAVATE AND RECOMPACT IN .3048 METERS LIFTS TO 95%
  - B. RAIL ROAD (5.5 METERS WIDE)
    - 1. BALLAST = 100.9 CUBIC METERS
    - 2. SUBBALLAST = 43.4 CUBIC METERS
    - 3. FILL = 1996 CUBIC METERS
      - b. USE IN-PLACE MATERIAL. EXCAVATE AND RECOMPACT IN .3048 METERS LIFTS TO 95%

V. MATERIAL TYPES AND QUANTITIES FOR ALL ANTICIPATED SMALL FAILURES (CUBIC METERS)

MATERIAL	ICE HARBOR	MONUMENTAL	LITTLE GOOSE	LOWER GRANITE
ASPHALT SURFACING	26	26	51	51
SURFACE COURSE	51	51	102	102
BASE COURSE	102	102	203	203
ROAD FILL	91,752	91,752	91,752	91,752
BALLAST	356	356	713	713
SUBBALLAST	153	153	306	306
RAIL ROAD FILL	91,752	91,752	91,752	91,752

SHEET MAIN SCALE:

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

RAILROAD AND ROADWAY REPAIR
SMALL SLOPE FAILURES

Figure:
H1

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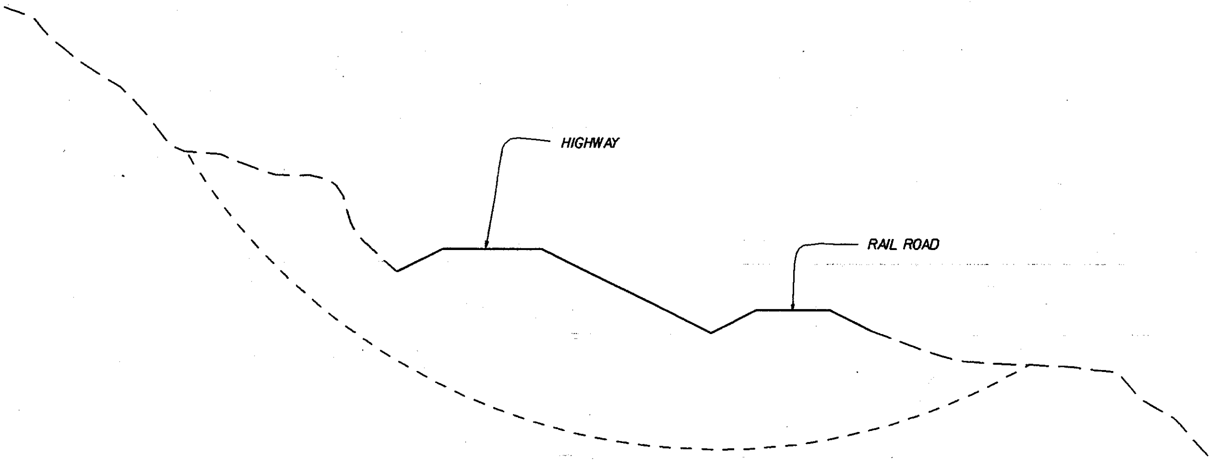

HYPOTHETICAL LARGE SCALE FAILURETYPICAL SECTION

NOT TO SCALE

DESIGN CRITERIA AND TYPICAL LARGE FAILURE PARAMETERS

I. SLIDE CHARACTERISTICS
  A. LENGTH = 91 METERS
  B. WIDTH = 91 METERS
  C. DEPTH = 46 METERS

II. ESTIMATED QUANTITY OF MATERIAL ABOVE FAILURE ARC
  A. 91,752 CU. METERS

III. AVERAGE HAUL DISTANCES
  A. ICE HARBOR POOL = 11.3 KILOMETERS
  B. LOWER MONUMENTAL POOL = 11.3 KILOMETERS
  C. LITTLE GOOSE POOL = 12.9 KILOMETERS
  D. LOWER GRANITE POOL = 12.9 KILOMETERS

IV. MATERIAL TYPES AND QUANTITIES FOR ALL ANTICIPATED LARGE FAILURES (CUBIC METERS)

MATERIAL	ICE HARBOR	MONUMENTAL	LITTLE GOOSE	LOWER GRANITE
ASPHALT SURFACING	560	385	911	1,293
SURFACE COURSE	1,107	763	1,802	2,558
BASE COURSE	2,220	2,243	3,604	5,131
ROAD FILL	160,962	162,634	261,302	372,094
BALLAST	7,771	7,852	12,616	17,965
SUBBALAST	3,338	3,373	5,420	7,717
RAIL ROAD FILL	159,721	155,318	2,459,240	355,353

SHEET MAIN SCALE:

US Army Corps
of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

LARGE SLOPE FAILURES

Figure:
H2

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