DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS

STABILITY ANALYSIS OF STRUCTURES

FRANKLIN FALLS DAM

FRANKLIN, NEW HAMPSHIRE

REPORT

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PART I

GENERAL

I - Section 1 - Project Criteria.

List of recent and updated stability criteria and instructions provided by the Corps of Engineers, New England Division:

Engineering Manuals:

EM 1110-2-2101 - Working Stresses for Structural Design (17 Jan. 1972).
EM 1110-2-2200 - Gravity Dam Design (25 Sept. 1958).
EM 1110-2-2400 - Structural Design of Spillways and Outlet Works (2 Nov. 1964).
EM 1110-2-2501 - Wall Design: Flood Walls (18 Jun. 1962).
EM 1110-2-2502 - Retaining Walls (25 Jan. 1965).

Engineer Technical Letters:

ETL 1110-2-184 - Gravity Dam Design (25 Feb. 1974).
ETL 1110-2-109 - Structural Design for Earthquakes (21 Oct. 1970).

Pertinent Hydraulic Data:

Hydrologic Data for Structural Stability - Analysis of Spillways.

List of design computations and drawings:

(1) Analysis of Design - 1938: Franklin Falls Dam.
(2) Analysis of Design - Appendices A, B-IV, and B-V.
(3) Plans for Construction of Franklin Falls Dam.

I - Section 2 - Description of the Dam and Operating Condition.

Franklin Falls Dam is located in New Hampshire on the Pemigewasset River, the main tributary of the Merrimack River which is formed about three miles downstream at the junction with the Winnipesaukee River. The dam was completed in 1943, and is made of rolled earthfill with a dumped rock shell 140 feet high and 1,740 feet long. The reservoir is operated for flood control purposes and is normally kept empty. Control gates in the outlet structure are operated to store floodwaters in the reservoir during times of flood. A concrete spillway, 546 feet long and founded on rock, is located on the west abutment. The spillway, with its crest elevation of 389.0, is 27 feet below that of the dam and would protect the dam from overtopping during passage of a maximum probable flood.

The spillway was designed for a peak flow of 18,800 cfs (the spillway design flood), which is 50 percent greater than the maximum six-day storm and snow-melt run-off. The outlet structure has a maximum discharge capacity of 18,500 cfs with full gate control to afford maximum use in flood control. Water level has never reached the spillway crest elevation of 389.0.

The hydrological data for structural stability, updated and furnished by the Contracting Officer, are as follows:

(a) Full Pool Condition (pool at spillway crest, minimum tail water):

Energy gradient at spillway (ft. msl)	389.0
Tail-water energy gradient	301.0

(b) Design Discharge Condition (reservoir at peak level of probable maximum flood and corresponding tail waters):

Energy gradient at spillway (ft. msl)	412.3
Tail-water energy gradient (ft. msl)	336.0
Tail-water water surface (ft. msl)	333.5

I - Section 3 - Criteria for Analysis.

The principal concrete structures and project features analyzed for stability consist of the following:

(a) Intake Tower
(b) Service Bridge Pier and Abutment
(c) Spillway Weir
(d) Spillway Retaining Walls
(e) Stilling Basin Head Wall and Retaining Walls
(f) Outlet Approach Channel Walls

Two members of our engineering staff visited the site on January 8, 1974 (copy of memorandum enclosed).

To check sliding resistance of structures under lateral loading, a method different from the original design calculations has been used. This is the shear-friction factor of safety formula, as outlined in the Engineer Technical Letter No. 1110-2-184 of 25 Feb. 1974. The sliding resistance is a function of the angle of internal friction and the unit shearing strength of the foundation material. Where the base of the concrete structure is embedded in rock, the passive resistance of the downstream layer of rock may be utilized in addition to the sliding resistance.

In the analysis of the Franklin Falls Dam structures, the shear-friction safety factor formula used includes all three contributing resistances: namely, the friction, the shearing strength, and the passive reaction where applicable.

For the spillway weir and the retaining wall, a minimum shear-friction factor of safety of 4 is required for all conditions of loading when earthquake is not considered. When earthquake is considered, this factor of safety should exceed 2-2/3. Retaining walls on earth require a sliding factor of safety of Tan ϕ/1.5.

The resistance to overturning is determined according to current criteria by the location of the resultant of vertical forces at the base. Without seismic forces, the resultant should be located within the middle third. When earthquake is considered, it is acceptable if the resultant stays within the width of the base. For retaining walls founded on rock, the resultant may be outside the middle third of the base if all other conditions are met, i.e., the foundation pressures are within allowable values and the factor of safety against sliding is sufficient.

The original design of 1938 did not consider earthquake loads. Because the Franklin Falls Dam is located in Zone 2 (moderate damage), as shown on the Seismic Risk Map of the U.S., included with ETL 1110-2-109, this analysis includes seismic forces as specified for that zone with acceleration of 0.10g.

The seismic forces applied to this stability analysis, in accordance with EM 1110-2-2200 of 25 Sept. 1958, are as follows:

(a) Inertia force P_{el} = 0.10W, acting horizontally through the center of gravity in any direction.

(b) Increase in water pressure by Westergaard's formula, first published in 1933, and expressed in terms of horizontal force P_{e2} and moment M_e at any depth y. Factor C = 51 lbs./ft.^{3} was used throughout, assuming t = 1 sec. This factor does not change appreciably for the height of structures up to 200 feet.

(c) Dynamic earth pressure in accordance with EM 1110-2-2502 of 25 Jan. 1965, was applied at about 2/3 of the fill height. This pressure is equal to about 20 percent of static lateral earth pressure. The backfill between a sloping wall and a vertical plane through the heel was added to the wall mass for calculation of inertia force P_{e1}.

(d) For walls with water on both sides, the seismic loads should include effects of increase on one side and decrease on the other side for free water. Horizontal water pressure in the soil is similar to uplift pressure and the effects of earthquake on it will be negligible; therefore, only increase in water pressure on one side is used in the design.

Ice pressure, used where applicable, is 5,000 psf x 2 feet = 10,000 pounds per linear foot of structure (refer to EM 1110-2-2200, Section 2-07).

The uplift pressure at any point under a structure is the tail-water pressure plus the pressure measured as an ordinate from tail water to the hydraulic gradient between the upstream and downstream sides. The uplift considered in the original design of 1938 was only 67 percent of these values. In this analysis, the uplift pressure is considered to act over 100 percent of the base area measured from the upstream edge to the downstream edge.

I - Section 4 - Evaluation of Foundations.

Reference is made to "Analysis of Design," Corps of Engineers, Boston, Massachusetts, 1938.

The subsurface exploration consisted of borings, trenches, test pits, and probes. Two-inch diameter samples were obtained in the overburden; and the bedrock was diamond drilled, generally to a 15-foot depth.

From field reconnaissance and surface information, the bedrock was found to be a solid, unweathered variation of micaceous gneiss and granular mica schist with numerous but minor veins of coarse granite. The bedrock is without large fissures, shatter zones, or faults and is only nominally fractured. The veins of coarse granite are too irregular for lateral correlation from the drill holes. The rock is structurally sound.

All of the concrete structures, except the service bridge pier and abutment, are founded on sound rock. Embedment of the foundation into sound rock is from 1 to 2 feet for the walls, about 5 to 7 feet for the spillway weir, and varies from 5 to 30 feet for the intake tower.

Allowable bearing pressure for the massive crystalline, igneous, and metamorphic rock with minor cracks may be as high as 80 tons per square foot. Foliated, metamorphic rock, such as schist, may be loaded up to 35 tons per square foot. Allowable shearing stress for the rock type described above and in the "Analysis of Design," 1938, based on shearing strength of the rock would be higher than the allowable stresses for bonded surface between rock and concrete, as given in Section I-5.

The service bridge pier and abutment are founded in the compacted, pervious fill of the dam. The report made in 1938 indicates that the compacted pervious fill would come from the structure excavation and borrow areas B and C. Information available indicates that the material used could vary from a silty fine sand to a well-graded gravelly sand. The allowable bearing pressure will vary from 1.5 tons per square foot to 3 tons per square foot. No visual overstressing of the soil or structure has been observed since the completion of the structures, even with the actual bearing pressures computed in this analysis of up to 3 tons per square foot, which indicates that the higher limit of allowable pressures (3 tons per square foot) can be used.

I - Section 5 - Allowable Unit Stresses at Interface of Concrete and Rock.

Allowable stresses at the bonded surface between concrete and rock are assumed to be the same as for 3,000 psi concrete or as allowable for the type of rock at the site. EM 1110-1-2101 refers to the ACI Building Code for allowable stresses in concrete with certain modifications. The following allowable stresses are used in this report:

(a) Concrete - Compressive strength $f_{c'}$ = 3,000 psi at 28 days.

(b) Rock (weathered or unweathered schistose gneiss, ETL 1110-2-184, 25 Feb. 1974) - Average compressive strength = 10,500 psi Average shear strength = 1,600 psi.

(c) Allowable bearing on rock - 35 tons/s.f. = 485 psi (less than allowable compression, direct or flexural, in concrete). Bearing on compacted pervious fill = 3 tons/s.f.

(d) Shear at interface between rock and concrete = 40 psi (based on ACI 318-63, Composite Concrete, allowable bond shear stress for rough and clean contact surfaces without mechanical anchorage).

(e) Coefficient of frictional resistance = 0.5 (based on tangent of the angle of internal friction for foundation material or angle of sliding friction).

These allowable unit stresses may be increased by 33-1/3 percent with Group II Loadings, such as wind, ice or earthquake (EM 1110-2-2101).

PART II

RESULTS OF THE ANALYSIS

II - Section 1 - Intake Tower.

The intake tower is located at the upstream end of the outlet conduits and is founded on solid rock. In the plan, the tower substructure from Elevation 334.0 to 416.0 and superstructure above Elevation 416.0 measures approximately 121 feet by 30 feet. The substructure below Elevation 334.0, including trash bar supports, piers, and transition to outlet conduits, with 5-foot thick base slab is approximately 70 feet wide by 107 to 121 feet long.

The tower was analyzed for stability at two levels: Elevation 334.0 and Elevation 295.0 (on rock). Loading cases applied are those listed in EM 1110-2-2400, Section 3-07c, entitled "Stability of Gate Structure at Upstream End." The structure was analyzed for Loading Cases I through V, and IA, IIA, IIIA, and IVA, with seismic acceleration of 0.10g for Zone 2. Obviously noncritical loadings were eliminated by comparison during the analysis. Thirteen loading cases were analyzed; eleven for stability at the base on weak axis (perpendicular to the flow), and two for concrete section at Elevation 334.0. Stability on strong axis (parallel to the flow) and on diagonal axis were also checked.

Maximum bending and shear stresses at Elevation 334.0, including seismic forces, are within allowable limits.

At the base of the intake structure, Elevation 295.0, for all the Loading Cases I through V, except IV, with emergency gates closed, the resultant is within the middle third of the base. For Loading Case IV with emergency gates closed, the resultant falls just outside of the middle third, but the factor of safety against overturning is adequate with 96 percent of the base in bearing. The factor of safety against sliding is more than 1.5 for all loadings, and the bearing pressures are within the allowable values.

For Loading Cases IA through IVA with seismic forces included, the resultant is always within the base with a minimum of 16.5 percent of the base in bearing. The maximum bearing pressure on rock is 20.9 tons per square foot which does not exceed the allowable of 1.33 x 35 = 46 tons per square foot. The analysis does not take into account extra stability provided by embedment of the tower base into rock.

The intake tower is stable under all of the specified loading cases and no modifications or strengthening is required.

II - Section 2 - Service Bridge Pier and Abutment.

The service bridge connecting the intake tower with the roadway on top of the dam is a two-span structure with one truss span of 140 feet and a girder span of 38 feet supported by one pier, one abutment, and a bridge seat at the intake tower. The design loading is AASHO H-15. The pier and the abutment are founded on the compacted pervious fill of the dam.

Loading cases considered are as given in EM 1110-2-2400, Section 3-07c, entitled "Stability of Gate Structure at Upstream End." In the analysis, soil pressure effect was neglected since the major portion of the structure is buried in the fill. Considering the large opening between columns above the footing, the difference in pressures on the two sides of the structure would be very small. Ice force of 10,000 pounds per linear foot was applied to the pier according to EM 1110-2-2200, entitled "Gravity Dam Design." Wind loading of 30 psf was applied at 30 degrees to the longitudinal axis of the bridge to produce maximum lateral load on the pier about its weak axis.

The following is a summary of the loading cases and the critical results for the pier and the abutment:

(a) Loading Cases:

I - Dead load plus wind, no water.

II - Dead load plus water level up to the top of the spillway, Elevation 389.0, plus ice and uplift.

V - Dead load plus floodwater level to Elevation 412.3, plus uplift.

IA, IIA - I or II plus earthquake.

Only Loading Cases I, V, and IA were applied to the abutment because the bottom of the footing at Elevation 390.5 is higher than the top of the spillway.

(b) Critical Results:

	Pier		Abutment	
	Factor of Safety	Load Case	Factor of Safety	Load Case
Minimum Factors of Safety:				
Against Overturning	3.13	IIA	3.19	IA
Resultant	In Middle	Third	In Middle	Third
Against Sliding	3.94	IIA	5.0	IA
Against Uplift	3.20	V	3.23	V
Maximum Foundation Pressure	3.65 tons/ square foot	IA	1.46 tons/ square foot	IA

No allowance was made for the service bridge connection at the top of the structure or the embedment of the foundation into the fill, both of which would make the structures more stable.

For the pier and the abutment, maximum foundation pressures are within the allowable values, the stability is adequate and no strengthening is required.

II - Section 3 - Spillway Weir.

The spillway weir is a concrete gravity wall section, approximately 550 feet long at the crest, low and wide in cross section. The structure is divided into thirteen monoliths, typically 42 feet long and separated by expansion joints. The expansion joints have two keys, 4 inches deep by 12 to 18 inches wide, and joint fillers at the ends. All monoliths have a minimum of 5 feet of embedment into sound rock.

The north end of the spillway weir has three monoliths of height varying from 15 to 31 feet at the upstream end, a base width of 42 or 60 feet and a rock embedment of 5 to 10 feet. The remainder of the typical spillway monoliths has a maximum height of 10 feet, a base width of 42 feet, a minimum concrete thickness of 5 feet and a rock embedment of 5 feet. These ten typical monoliths are anchored to the rock by 1-1/4 inch square steel rods set in 2-1/2 inch diameter by 6-foot long drilled holes approximately 5 feet on center. A gravel drain system with downstream outlet at Elevation 373.5 is provided under all of the typical monoliths and the first monolith adjoining the typical. Connection of the spillway weir with the spillway retaining walls at the north end is assumed to be with a keyed expansion joint but no dowels. At the south end, the weir is keyed into the rock. It also has an anchored concrete facing extending above the weir to the top of the slope.

Three sections, one typical and the other two at the north end, were analyzed for stability. Loading cases in accordance with EM 1110-2-2200, Section 3.01, were applied. The following loading cases were governing:

II - Normal Operating (full pool condition).

IV - Flood Discharge (reservoir at peak level of probable maximum flood).

VI - Normal Operating with Earthquake.

The critical values of the factors of safety and bearing pressures for each monolith analyzed are shown in Table 1. For the typical section, the resultant is within the middle third of the base when the 6-foot deep block of rock down to the bottom of existing anchors is included in the analysis for Loading Case II.

However, the typical Section A-A does not satisfy the stability criteria with Loading Case IV. Even with the anchored block of rock, the resultant falls outside of the base. This means that these monoliths may not be stable in case of a maximum design flood should the rock develop a cracked seam at the base of existing anchors. For other sections and loading cases, stability requirements are satisfied and the foundation pressures for all of the sections are within allowable values for the rock.

Remedial measures are recommended to strengthen all ten monoliths of Section A-A, for a total length of 420 feet. Vertical rock anchors of high strength steel would be installed into predrilled holes and extend a minimum of 20 feet into rock, making the total length of each anchor about 30 feet. Using design capacity of 73 kips per anchor, the spacing of rods would be approximately 3 feet 6 inches. The total estimated cost of this system to anchor ten monoliths of spillway weir is $234,000.

TABLE 1

SPILLWAY WEIR

Weir Section	Loading Case	Location of Resultant: In Middle Third	Location of Resultant: In Base	Percent Base In Bearing	Resistance to Sliding Factor of Safety(*)	Bearing Pressures on Rock, Tons/S.F.: Maximum	Bearing Pressures on Rock, Tons/S.F.: Minimum
A-A/46	II	Yes	-	100	15.7	1.2	0.06
	IV	No	No	-	9.7	-	-
	VI	No	Yes	84	20.6	1.0	-
C-C/46	II	Yes	-	100	11.0	0.9	0.34
	IV	No	Yes	59	6.0	1.3	-
	VI	Yes	-	100	11.0	0.8	0.48
D-D/46	II	Yes	-	100	8.6	1.0	0.67
	IV	Yes	-	100	4.6	1.0	0.10
	VI	Yes	-	100	7.0	1.1	0.62

*Factor of safety calculated for bond shear value of 40 psi and $\phi = 30^{\circ}$.

II - Section 4 - Spillway Wall.

The spillway wall is located at the southern end of the dam and separates the earth dam embankment from the spillway channel. The spillway wall is about 900 feet long and starts from one end of the intake tower, is connected to the spillway weir, and extends approximately 140 feet downstream along the spillway channel. The wall has two changes in direction and a slight curvature in plan. The wall section is a concrete gravity type, with maximum height of about 58 feet and a corresponding base width of 39 feet. The full length of the wall is founded on sound rock with 1 to 2 feet of embedment.

The retaining wall was analyzed in accordance with EM 1110-2-2502 for active earth pressures, disregarding fill in front of the wall. Walls founded on rock and designed for active earth pressures with the resultant in the middle third and satisfying all other critical values of sliding factor of safety and foundation pressures, will have sufficient stability for at-rest pressures. Loading cases listed below were based on design criteria given in EM 1110-2-2400 but not listed specifically for spillway walls.

Loading Cases:

Upstream Walls -

I - Normal operating condition, water level in channel to the top of the spillway, Elevation 389.0, backfill submerged to Elevation 389.0 or higher.

I-1 - Normal operating condition, channel empty, backfill submerged up to 50 percent of height. (Similar to Loading Case II with reduced water level in backfill.)

II - Sudden drawdown in channel water level to bottom of the channel, backfill water level to Elevation 412.3, maximum flood, or top of the wall.

III - Maximum flood condition, water level in channel to Elevation 412.3 or top of the wall, and backfill water level 50 percent of height.

IA - I with earthquake.

I-1A - I-1 with earthquake.

Downstream Walls -

I - Channel empty, backfill water level to midheight of wall.

II - Water level in channel to tail-water elevation, no water in backfill.

IA - I with earthquake.

Three sections of the wall on the upstream side and one section (A-A) on the downstream side were analyzed for stability. Section A-A was checked at the bottom of the gravity wall, at Elevation 362.0, and at the base of the channel, at Elevation 350.0. The latter analysis was combined with the anchored concrete facing and the engaged block of rock. These calculations for Loading Case I produced the resultant of all forces falling outside of the middle third.

The critical values of the factors of safety against sliding, location of resultant, and foundation pressures are shown in Table 2. The required factor of safety in sliding for the spillway walls is the same as for the weir since it retains the earth dam.

Using an allowable 40 psi bond shear with passive resistance of the fill in front of the channel and the wall embedment into the rock, the factor of safety for Section B-B/45 is 3.90. Considering the nature of Loading Case II, which assumes sudden drawdown from floodwater level of 412.3 and passive resistance of rock wedge in the front based on only 40 psi shear stress (actual strength is higher), this sliding factor of safety may be accepted. All other stability requirements for the section are satisfied.

For Sections D-D/45 and A-A/45, the resultant falls outside of the middle third for Loading Cases I or II but always remaining within the base. Other stability requirements for the sections are satisfied.

The spillway wall, except for Section B-B near the spillway weir, does not satisfy the stability requirements as outlined above. Therefore, remedial measures are recommended to strengthen the following sections:

A-A, from Sta. 27+85 to Sta. 28+67,	-	82 feet
C-C, from Sta. 23+25 to Sta. 25+05,	-	180 feet
D-D, from Sta. 19+72 to Sta. 20+92,	-	120 feet
Total	-	382 feet

The required strengthening could be achieved with horizontal or near horizontal tie rods anchored to concrete deadmen in the rear of walls. This system would require a limited amount of drilling and grouting through the walls and substantial excavating for placement of tie rods and deadmen. The total estimated cost is 382 feet at $150 per foot = $57,000, including contingencies. Additional mass concrete would probably cost more, as it would require larger volumes of excavation.

TABLE 2

SPILLWAY WALL

Wall Section	Loading Case	Location of Resultant: In Middle Third	Location of Resultant: In Base	Percent Base In Bearing	Resistance to Sliding Factor of Safety(1)	Bearing Pressures on Rock Maximum Tons/S.F.	Bearing Pressures on Rock Minimum Tons/S.F.
A-A/45 at Elev. 362.0	I	Yes	-	100	15.3	0.8	0.75
	II	Yes	-	100	21.6	1.0	0.6
	I-A	No	Yes	53	6.7	2.6	-
A-A/45 at Elev. 350.0	I	No	Yes	71	5.0	3.6	-
	I-A(2)	No	Yes	29	3.9	10.9	-
B-B/45	I	Yes	-	100	6.5	3.1	2.24
	I-1	Yes	-	100	4.3	4.0	2.20
	II	Yes	-	100	3.9	4.0	1.05
	III	Yes	-	100	4.6	5.7	0.06
	I-1A	Yes	-	100	2.95	4.3	1.90
C-C/45	I-1	Yes	-	100	10.6	3.0	0.55
	II	No	Yes	41	4.0	8.0	-
	III	Yes	-	100	9.4	3.4	0.32
	I-1A	No	Yes	74	6.7	4.8	-
D-D/45	I-1	No	Yes	67	5.1	4.6	-
	II	No	Yes	37	4.3	7.4	-
	III	Yes	-	100	17.2	2.4	0.45
	I-1A	No	Yes	27	4.1	11.9	-

(1) Factor of safety calculated for bond shear value of 40 psi and $\emptyset = 30^{\circ}$.

II - Section 5 - Stilling Basin Head Wall and Retaining Walls.

The stilling basin is located at the downstream end of the outlet conduits. The toe of the earth dam is protected by a low head wall for full width of the stilling basin. Flow from the two outlet conduits is separated by a center wall. The floor of the stilling basin slopes down in steps from Elevation 299.0 at outlet conduits to Elevation 273.0, and is anchored to the rock. The stilling basin is separated from the downstream terrace on the east side by a concrete gravity wall of varying height. The west wall is a small concrete gravity section situated on top of the anchored concrete facing and retains earth embankment between the stilling basin and spillway channel wall. A gravel drain system is provided between the rock and concrete facing at the west wall. All the stilling basin structures are founded on sound rock, with a minimum of 2 feet embedment. Part of the head wall, on top of the outlet conduit openings, is dowelled into the top slab of the conduit.

The stability analysis of the stilling basin was done in accordance with EM 1110-2-2400, Section 2-07f, and EM 1110-2-2502. The walls were designed for either active or at-rest pressure coefficients, modified where necessary for backfill slope. The wall sections were analyzed for the following loading cases:

I - Stilling basin empty, water level in the backfill at the height midway between base and bottom of rockfill.

II - Rapid closure of gates, water level inside at low flow elevation, backfill water level midway or higher between tail-water elevation before and after the reduction in flow.

IA - I plus earthquake.

One section at the head wall and west wall, two sections at the center wall, and three sections at the east wall were analyzed. Critical values of the factors of safety against sliding, location of resultant and maximum foundation pressures are given in Table 3.

Sections E/42 and D/42 at the east wall, Section G/42 at the west wall, and the center wall have resultants outside of the middle third under Loading Cases I or II, as shown in Table 3.

Analysis of the head wall, Section K/42, and east wall, Section C/42, shows that the resultant is in the middle third of the base under Loading Conditions I and II. With earthquake loading, Case IA, all sections produce the resultants outside of kern but safely within the width of the base.

Two sections of the center wall at the downstream end of the stilling basin were analyzed according to EM 1110-2-2400 for water level to the top of the wall; Elevation 312.0 on one side and low hydraulic jump profile elevation of 296.0 on the other side. A fifty percent increase in loads due to pulsation was included. The analysis was done assuming an independent section of the wall and floor slab between keyed joints on each side. Resisting moment provided by weight of concrete and water is slightly more than overturning moment produced by the horizontal water pressures, pulsating force and uplift. For earthquake loading, different water levels on two sides were used but pulsation force was not included.

Loading considered in the analysis assumes the hydraulic jump occurring at the sections analyzed, but from data available, exact limits could not be ascertained. The section of the center wall analyzed is at the downstream end of the stilling basin where the loading may not be applicable and the water levels on two sides would tend to equalize.

In order to satisfy the updated criteria for stability, the following retaining walls of the stilling basin should be provided with an anchorage system:

East Wall, Section E/42, from Sta. 23+25 to Sta. 23+91, 2 monoliths - 66 feet long.
East Wall, Section D/42, from Sta. 21+90 to Sta. 22+54, 2 monoliths - 64 feet long.
West Wall, Section G/42, from Sta. 21+90 to Sta. 22+54, 2 monoliths - 64 feet long.
Center Wall, Section C/42, from Sta. 22+93 to Sta. 23+25, 1 monolith - 32 feet long.

East and west wall strengthening can be done with tie rods and deadmen similar to the system recommended for the spillway walls. The estimated cost for 194 feet of walls is $29,000. The center wall would require rock anchors set in diagonally predrilled holes through the wall and into rock approximately 20 feet. Six rock anchors, three on each side, at design loading of 60 kips each, would cost approximately $2,700 each, for a total of $16,000. Total cost of remedial measures to the stilling basin would be $45,000, including contingencies.

TABLE 3

STILLING BASIN HEAD WALL AND RETAINING WALLS

Wall Section	Loading Case	Location of Resultant: In Middle Third	Location of Resultant: In Base	Percent Base In Bearing	Resistance to Sliding Factor of Safety(2)	Bearing Pressures on Rock Tons/S.F.: Maximum	Bearing Pressures on Rock Tons/S.F.: Minimum
East Wall E/42	I	No	Yes	97	4.3	5.8	-
	II	No	Yes	82	3.9	5.8	-
	IA	No	Yes	43	2.8	13.2	-
East Wall C/42	I	Yes	-	100	6.3	3.9	0.92
	II	Yes	-	100	5.7	3.8	0.62
	IA	No	Yes	84	4.0	5.8	-
East Wall D/42	I	Yes	-	100	4.1	3.8	0.90
	II	No	Yes	97	7.6	4.0	-
	IA	No	Yes	79	5.9	5.4	-
West Wall G/42	I	No	Yes	78	7.5	3.3	-
	II	No	Yes	30	5.8	7.2	-
	IA	No	Yes	39	5.4	6.5	-
Head Wall K/42	II	Yes	-	100	10.0	2.1	-
	IIA	No	Yes	78	6.7	2.7	-
Center Wall C/42	I (1)	No	Yes	33	8.6	5.2	-
	IA (1)	No	Yes	28	5.4	6.2	-
Center Wall at Sta. 22+75	I (1)	No	Yes	47	-	4.4	-
	IA (1)	No	Yes	34	-	6.4	-

(1) This calculation includes hold-down reactions at existing keys to the adjacent slab sections.
(2) Factor of safety calculated for bond shear value of 40 psi and $\phi = 30^{\circ}$.

II - Section 6 - Outlet Approach Channel Walls.

At the upstream end of the intake structure, the river flow is channelized between approach channel walls. The west wall is about 85 feet long and connected to the east wall by a floor slab at Elevation 300.0. The cross section of the west wall combines the side wall facing of the channel anchored into the rock with a gravity section of varying height founded on the rock. The backside of the west wall is stripped to the top of the rock. Design analysis assumes backfill to the top of the wall. The east wall is about 475 feet long, with the downstream end connected to the west wall by the channel floor slab. The cross section of the east wall varies over the length of the wall. At the two ends, it is a gravity section founded on rock with the channel floor part way up the front of the wall; whereas, the middle part of the wall is similar in cross section to the west wall. Backfill at the east wall is level with the top of the wall except at the upstream end where the backfill is banked to meet the existing grade for about 60 feet along the wall.

The approach channel walls were analyzed for Loading Cases I, II, and III, as indicated in EM 1110-2-2400, Section 2-07f, entitled "Approach Channel Walls," and in accordance with EM 1110-2-2502, "Retaining Walls," for active pressure conditions.

Loading cases considered were:

I - Channel empty with saturated backfill, partial height.

II - Sudden drawdown in channel with saturated backfill, full height.

III - Sudden floodwater increase in channel and low water level in backfill or floodwater on both sides of the wall.

IA - Loading I with seismic forces.

Three sections of the east wall and one section of the west wall were checked for stability. All sections, except one, were checked for overturning and sliding at two points; one at the base of the gravity section, and the other at the bottom of the channel floor. The critical values of factor of safety against sliding, location of resultant, and foundation pressures are shown in Table 4.

According to the design requirements of EM 1110-2-2502, the resultant for retaining walls should fall within the middle third of the base for all loadings, applying active lateral earth pressures, and remain within the base for loadings with the earthquake forces. All sections for Loading Case II and Section A for Loading Case III have the resultant outside of the middle third. Section D does not satisfy this criteria for Loading Cases I and II. Factors of safety against sliding based on shear-friction resistance are adequate. Maximum foundation pressures on rock in bearing are within allowable limits.

Therefore, the entire length of the outlet approach channel wall, from Sta. 11+07 to Sta. 15+83, total of 15 monoliths, 476 feet long, has to be strengthened. The recommended method of remedial measure is a system of tie rods anchored to concrete deadmen. The estimated cost of construction, including excavation and backfill, is $71,000.

TABLE 4

OUTLET APPROACH CHANNEL WALLS

Wall Section	Stability Elevation	Loading Case	Location of Resultant: In Middle Third	Location of Resultant: In Base	Percent Base In Bearing	Resistance to Sliding Factor of Safety(2)	Bearing Pressures On Rock, Tons/S.F.: Maximum	Bearing Pressures On Rock, Tons/S.F.: Minimum
East Wall A/26	290.0	I	Yes	-	100	4.3	3.6	0.40
	290.0	II	No	Yes	32	2.8	8.6	-
	290.0	III	No	Yes	88	3.6	3.8	-
	290.0	IA	No	Yes	70	3.0	5.8	-
	298.5	II	Yes	-	100	2.8	2.6	0.21
West Wall A/26	310.	I	Yes	-	100	15.1	0.9	0.57
	310.	II	No	Yes	85	8.2	1.4	-
	310.	IA	Yes	-	100	9.9	1.3	0.17
	297. (1)	I	No	-	100	7.7	2.3	-
	297. (1)	IA	No	Yes	51	12.3	3.0	-
East Wall C/26	313.	I	Yes	-	100	9.3	1.5	0.66
	313.	II	No	Yes	64	6.2	2.8	-
	313.	IA	No	Yes	94	5.1	2.3	-
East Wall D/26	300.	I	No	Yes	89	6.9	2.2	-
	300.	II	No	Yes	37	6.0	3.7	-
	300.	IA	No	Yes	46	5.0	4.3	-

(1) Including existing anchors and blocks of rock.
(2) Factor of safety calculated for bond shear value of 40 psi and $\phi = 30^{\circ}$.

CONCLUSIONS

Most of the Franklin Falls Dam concrete structures analyzed for stability do not satisfy the specified requirements.

Remedial measures are recommended with estimated costs as follows:

Spillway Weir: Vertical Rock Anchors	-	$234,000
Spillway Wall: Tie Rods with Concrete Deadmen	-	57,000
Stilling Basin Side Walls: Tie Rods with Concrete Deadmen	-	29,000
Stilling Basin Center Wall: Diagonal Rock Anchors	-	16,000
Outlet Approach Channel Wall: Tie Rods with Deadmen	-	71,000
TOTAL	-	$407,000

The intake tower, service bridge pier and abutment have adequate stability in accordance with the prescribed criteria and do not need any strengthening.

MEMORANDUM

Site Visit to Franklin Falls Dam on Pemigewasset River
Franklin, New Hampshire
January 8, 1974

The writer and Mr. Sanat Patwari arrived at the site at 10 a.m. and were shown around by Mr. Robert Mayo, dam manager. Our visual inspection included the following structures:

1. The Dam and Reservoir. Drove along full length and walked at the south end of it. Pavement is smooth. The riprap covered with snow, steps covered with ice. Water level 307.4 feet (the base level is Elevation 300), very low. On December 24, and 25, 1973, it reached Elevation 348.0; on July 4, 1973, Elevation 369.0, still 20 feet below spillway crest, which is at 389.0.

2. Intake Tower. Concrete appears to be sound. All eight gates were open. The manager told us that at water levels above Elevation 334.0, they had seen whirlpools near the intake tower; one clockwise and the other counterclockwise. Some vibration had been felt inside the tower.

3. Service Bridge. Steel arched truss, one pier, one abutment. The manager told us that the concrete has deteriorated at the bearing of the shorter span at the pier.

4. Intake Channel. This was half full with water. There are some seeping construction joints. They were not visible from the tower but the manager told us.

5. Spillway Weir. Only the crest of it is visible above existing ground and rock.

6. Stilling Basin Head Wall and Retaining Walls. All appear to be sound.

We did not notice any variances to conditions indicated on drawings and descriptions furnished to us that would affect the stability analysis of structures. Eight photographs were taken. The temperature was about 13 degrees, cold, windy, and sunny.

Jurgis Gimbutas

JG:vj
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