Canal. Geologic studies for the Red Bluff-Black Butte conveyance facilities were limited to an appraisal of ground water conditions along the canal route. Depth to ground water along most of the route would be 15 to 18 m (50 to 60 ft), except in the vicinity of Thomes Creek where it would be about 3 m (10 ft). Some perched water might also be encountered but dewatering should not present any great problems.

Near Black Butte Reservoir, the conveyance facilities would encounter volcanic rocks of the Lovejoy Formation, which overlie mudstones and sandstones of the Black Butte Formation. In 1978, a brief geologic reconnaissance was conducted of foundation conditions for the pumping-generating facilities that would be located near Black Butte Dam. The plant would be founded on semi-consolidated mudstone or sandstone and tunnelling conditions between the plant and Black Butte Reservoir would be similar to those encountered in the existing Black Butte outlet tunnel. Hard rock excavating methods would be required in the basalt and breccia that cap the plant and intake channel areas to a depth of about 21 m (70 ft).

Early planning studies considered building a small on-line regulating reservoir on Sour Grass Creek at about the 104-m (340-ft) elevation. In 1978, the prospective damsite was explored with five bucket auger holes and primary soil tests were performed on samples of the foundation material. The entire reservoir is underlain by the Red Bluff Formation, poorly-sorted cobbles and gravels with a reddish clayey or sandy matrix. A memorandum geologic report prepared on the Sour Grass Dam site studies in January 1979 concluded that the site would be suitable for the 18-m (60-ft) dam then under construction. Similar geologic studies were conducted in 1978 for a Kirkwood Dam, located about 8 km (5 mi) farther downstream on Sour Grass Creek. (Kirkwood Reservoir would have been used in connection with the low-level conveyance alternative.) These studies included nine bucket auger holes and additional laboratory testing. Kirkwood Reservoir would also be founded on Red Bluff Formation soils. Although neither Sour Grass nor Kirkwood Reservoirs is included in the current plan, the geologic information developed for them is applicable to the appraisal of canals and generating plants that would be constructed in the same general areas.

Sediment

Suspended sediment concentrations in the Sacramento River are comparatively low, but a large volume of water would be diverted to Glenn Reservoir and the cumulative sediment volume would be significant. The USGS monitored daily suspended sediment at or near Red Bluff from 1957-58 through 1969-70 and from 1977-78 to the present. The average annual load during 14 years of available record was 1 690 000 t (1,860,000 tons). Average sediment composition was 23 percent sand, 36 percent silt, and 41 percent clay. The estimated dry unit weight of this sediment after deposition is 930 kg/m^3 (58 lb/ft^3); at this unit weight, the total annual suspended sediment load of the Sacramento River at Red Bluff would occupy a volume of about 1 800 dam^3 (1,500 ac-ft).

The Department's Staff Sedimentation Engineer prepared a memorandum report appraising sediment conditions relating to diversion of water to Glenn

Reservoir. That report estimated that diversion of an average of 540 000 dam^3/yr (440,000 ac-ft/yr) would capture about 120 dam^3/yr (100 ac-ft/yr) of sediment. (With the current plan formulations outlined in Chapter 6, more water would be diverted and some 20 to 30 percent more sediment would be admitted into the conveyance system.) The report recommended that a sill and stilling basin be included in the inlet works, but estimated that about 70 percent of the suspended sediment entering the stilling basin would pass on through to the conveyance system. This remaining sediment would be the finer material; it would tend to remain in suspension until reaching a quiescent area.

Sacramento River--Black Butte Reservoir Facilities

Figures 9-1 and 9-2 show a plan view and profile of the Glenn Reservoir Plan conveyance facilities between the Sacramento River and Black Butte Reservoir. The following sections cover the portion of those facilities that would deliver 283 m^3/s (10,000 ft^3/s) of Sacramento River water to Black Butte Reservoir.

Intake Facilities

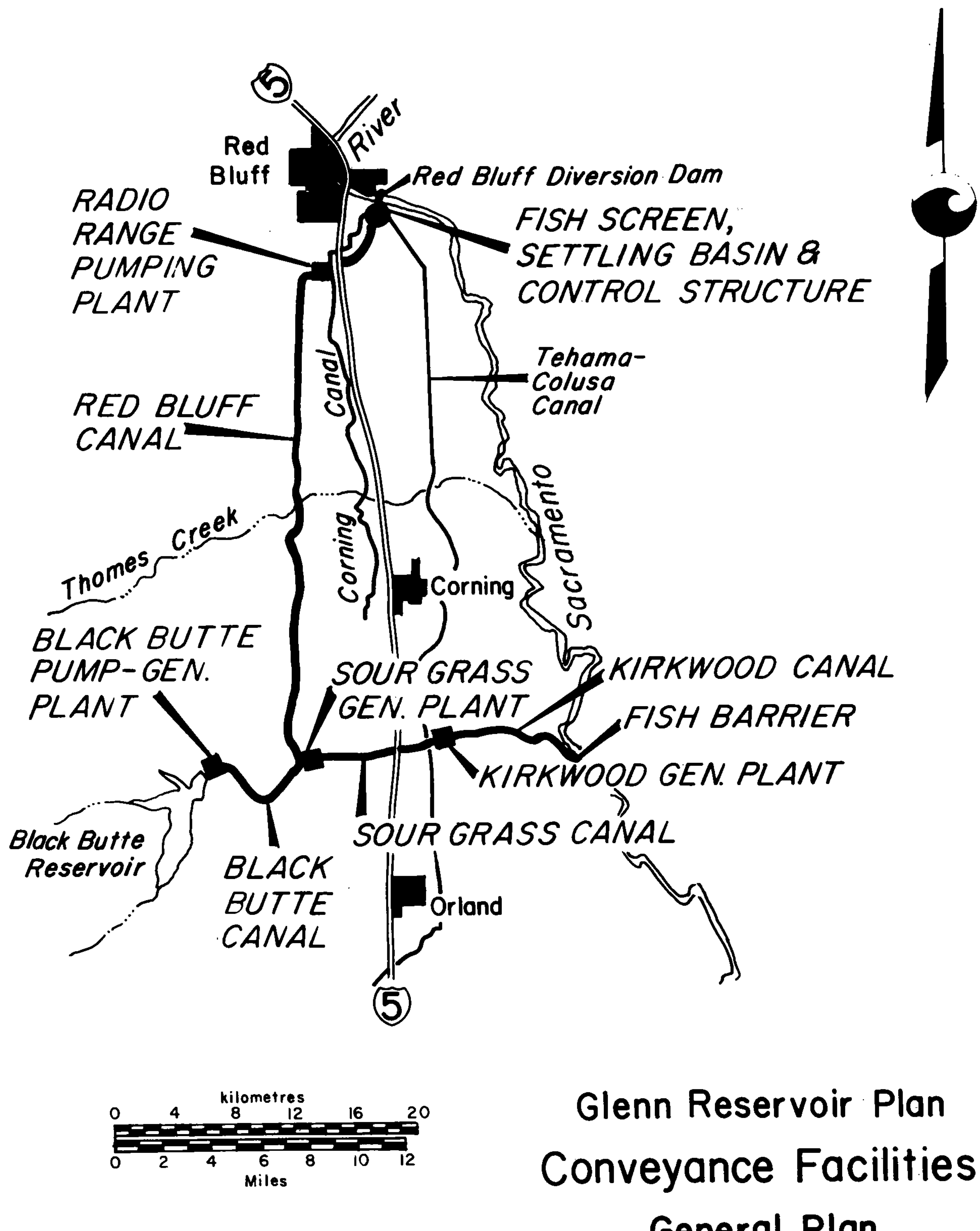
The intake facilities at Red Bluff Diversion Dam would have to exclude debris, sediment, and fish from the canal. With the large flows that would be diverted, these facilities would be fairly extensive, yet they would have to fit in with the existing Tehama-Colusa and Corning Canal intakes.

Some rudimentary designs and cursory cost estimates have been made for the new intake facilities. These studies give a general indication of the cost, but much more work will be required to define the facilities. The preliminary studies focus on the fish screening problem; the cost estimate provides for twenty horizontal drum screens, each 6.1 m (20 ft) in diameter and 7.6 m (25 ft) long. These would be similar to the drum screens presently in use at Red Bluff Diversion Dam, but their performance has not been entirely satisfactory--small fish are too often able to get past them. Any future studies should include analysis of other types of screens.

The assessment of Sacramento River sediment problems was completed long after the preliminary cost estimates for the intake facilities. That assessment recommended a very large stilling basin at the head of the Red Bluff Canal. The earlier cost estimate had provided only for a much smaller stilling basin, so additional work is also needed on this aspect of the intake facilities.

Finally, the preliminary cost estimates do not appear to have taken full account of the debris problems that would be associated with large diversions during flood periods. At flood stage, the Sacramento River carries a heavy load of floating debris, including plentiful numbers of logs and entire trees. Some rather elaborate facilities would be needed to remove all of the large debris and much of the smaller debris before it reached the fish screens.

Figure 9-1

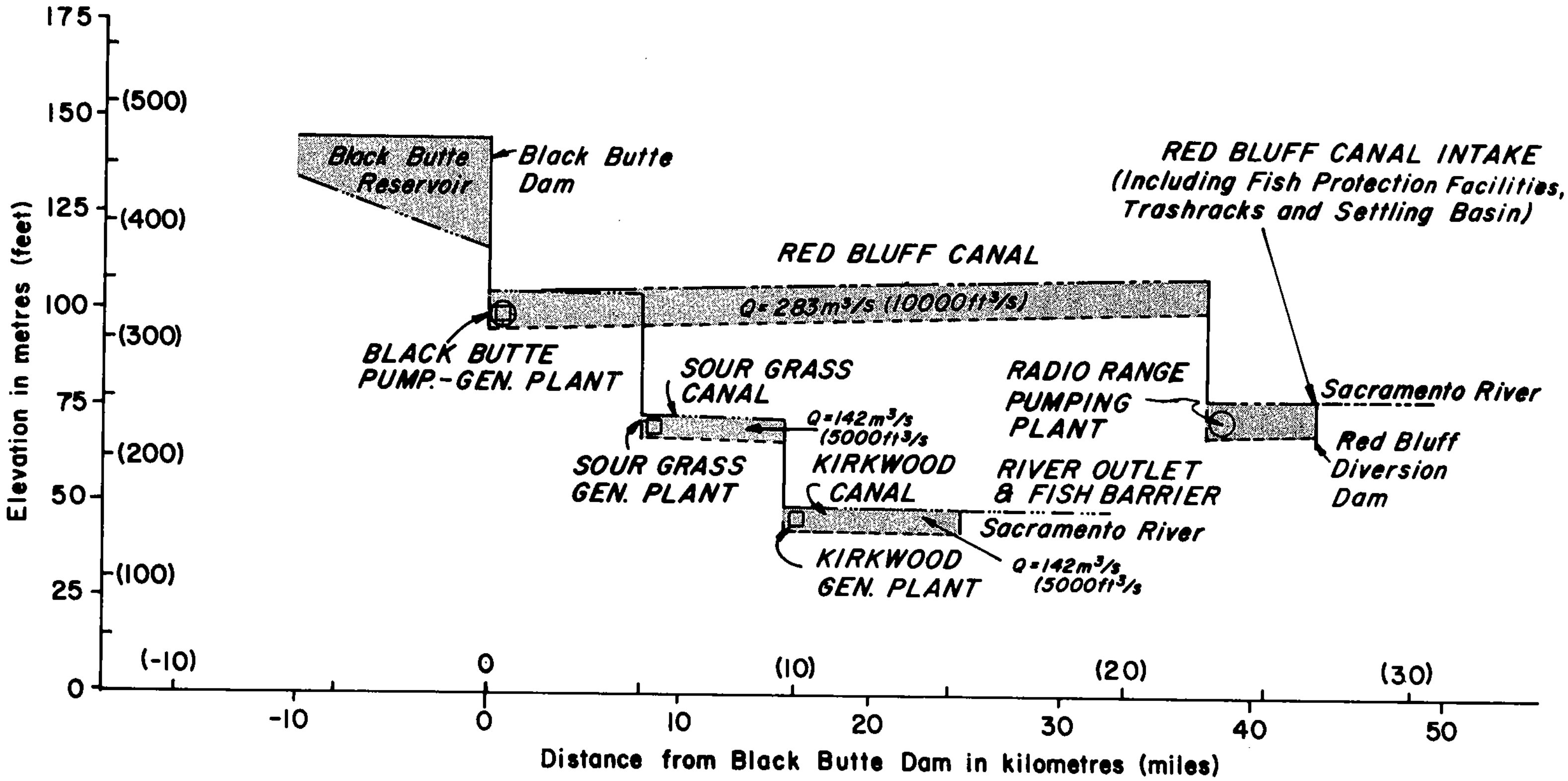


Glenn Reservoir Plan

Conveyance Facilities

General Plan

Figure 9-2



Glenn Reservoir Plan

Conveyance Facilities

Profile

Radio Range Pumping Facilities

The Radio Range Pumping Plant would be located 5 km (3 mi) southwest of the intake facilities at Red Bluff Diversion Dam. (The intervening intake channel is treated as part of the Red Bluff Canal in the following section of this chapter.) The plant would lift water about 30 m (100 ft) to the main portion of the Red Bluff Canal. It would be a conventional five-unit indoor pumping plant, typical of others throughout the State Water Project system. Three of the units would be designed to pump about 47 m^3/s (1,667 ft^3/s) each and two would deliver 71 m^3/s (2,500 ft^3/s) each; by running various combinations of units, the plant output could be tailored to the amount of surplus water available. Total installed plant capacity would be about 100 MW.

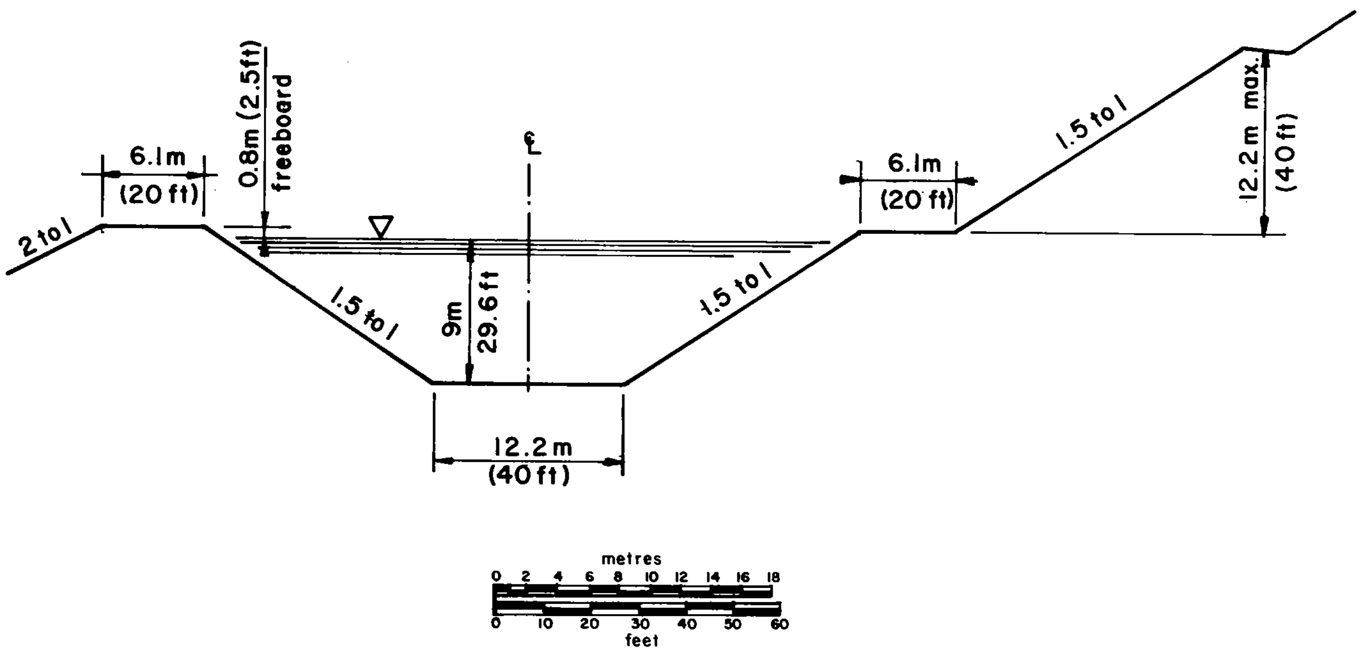
Ground surface elevation at the site of Radio Range Pumping Plant is about 98 m (320 ft) and the water surface elevation in the intake channel would be about 77 m (250 ft). The total depth of cut to the bottom of the plant foundation would be about 30 m (100 ft). All of the excavation would be in the gravelly soils of the Red Bluff Formation. Some ground water would be encountered in the excavation, but it should not present any major obstacle to construction.

Discharge lines for the Radio Range Pumping Plant would be buried steel pipe. The discharge lines for the two larger units would be about 5 m (16 ft) in diameter; for each of the smaller units, a 4-m- (13-ft-) diameter line would suffice. The five discharge lines would terminate at a concrete outlet structure that would be equipped with radial gates to prevent backflow under emergency conditions.

Red Bluff Canal

The Red Bluff Canal would include a 5-km (3-mi) lower reach between the intake facilities at Red Bluff Diversion Dam and Radio Range Pumping Plant and a 34-km (21-mi) upper reach between the pumping plant outlet structure and the junction with Black Butte Canal near Sour Grass Creek. The major portion of the canal alignment would lie just east of the two sets of power transmission lines that parallel Interstate 5. Both reaches of the canal would be conventional concrete-lined trapezoidal channels, as shown on Figure 9-3. Maximum flow velocity in the canal would be about 1.2 m/s (4.0 ft/s).

Design and construction of the canal would be routine. Most of the canal would be founded on soils of the Red Bluff Formation; some alluvium would be encountered at creek crossings. Ground water levels are generally below the level of the canal invert, except near Thomes Creek and possibly at a few other isolated locations. The preliminary layout showed that the Red Bluff Canal would require about 11 500 000 m^3 (15,000,000 yd^3) of excavation, but only about 3 500 000 m^3 (4,600,000 yd^3) of embankment. Future design studies should consider adjustments in alignment to more nearly balance the earthwork; otherwise, a substantial area would be needed for spoil disposal.



Glenn Reservoir Plan

Conveyance Facilities

Typical Canal Section

(for 283 cubic metres per second)
(10,000 cubic feet per second)

Although the Red Bluff Canal would pass through a relatively undeveloped area for much of its length, a number of major crossings would be required. These include:

Roads and Railroads	Creeks
Southern Pacific Railroad	Coyote Creek
State Highway 99E	Oat Creek
Interstate Highway 5	Elder Creek
Rawson Road	McClure Creek
Rawson Avenue	Thomes Creek
Ottman Avenue	Burch Creek
Clark Road	
Simpson Road	
Corning Road	
Chittenden Road	

The initial layout of the canal would have involved two crossings of the Corning Canal and one additional county road crossing. However, an alternative alignment could avoid these crossings. The listed major creek crossings would be made with multi-barrel concrete box siphons. Check structures with radial gates would be incorporated with the siphon crossings of Oat, McClure, and Burch Creeks; these would divide the canal into reaches no more than 11 km (6.8 mi) long. Cross drainage from smaller creeks would pass either above the canal in overchutes or under it in concrete pipe culverts.

Black Butte Canal

In the original layouts, the Red Bluff Canal would have terminated at a small regulating reservoir on Sour Grass Creek. From there, the Black Butte Canal would have run west to the Black Butte Pumping-Generating Plant. As more was learned about expected sediment conditions, it was feared that the small reservoir would simply act as a settling basin. Since the reservoir would not be essential for operational purposes, it was decided to omit it from the plan.

In the revised plan, the Red Bluff Canal would simply turn east and become the Black Butte Canal. (The distinguishing feature is that the Black Butte Canal would carry flow in either direction.) Intake facilities at the junction of the two canals would lead to the conveyance system that would carry releases back to the river, as described later in this chapter.

The Black Butte Canal would be 7.2 km (4.5 mi) long. It would be identical to the Red Bluff Canal, except that it would involve deeper cuts. The canal water surface elevation would be about 104 m (340 ft) and its invert would be near elevation 94 m (310 ft). The maximum ground elevation along the selected preliminary alignment is 152 m (500 ft), so the maximum depth of cut would be about 58 m (190 ft). Much of the canal excavation would be in Red Bluff Formation soils, but some hard rock would be encountered in the deeper portions near Black Butte Dam. The Black Butte Canal would pass through a rather isolated area; only one crossing would be required--a county road near Black Butte Dam.

An alternative alignment could avoid some of the deeper cuts by staying closer to Stony Creek. However, the selected canal water surface elevation of 104 m (340 ft) is about 9 m (30 ft) lower than the existing streambed level at the toe of Black Butte Dam. It appears that future studies might be able to improve the plan considerably by increasing the pump lift at Radio Range Pumping Plant by about 9 m (30 ft) and tying the Black Butte Canal directly into a small afterbay below Black Butte Dam. One possible problem with this adjustment is that the alignment of the Red Bluff Canal would be shifted westward into the existing power transmission line corridors. Nevertheless, this change should be considered in any future study of the Glenn Reservoir Plan.

Black Butte Pumping-Generating Facilities

The Black Butte Pumping-Generating Plant would be located just downstream from the existing Black Butte Dam. When running in the pumping mode, it would lift as much as 283 m^3/s (10,000 ft^3/s) of surplus Sacramento River water from the Black Butte Canal to Black Butte Reservoir. Some of the units would be reversible, so that they could recover energy whenever releases were being made back to the Black Butte Canal. For this study, a capacity of 142 m^3/s (5,000 ft^3/s) was selected for all of the generating facilities in the plan; detailed studies may show that the optimum generating capacity would be either greater or smaller.

Under the Glenn Reservoir Plan, Black Butte Reservoir would be stabilized near its spillway elevation of 144 m (474 ft). The water surface elevation in the Black Butte Canal would always be near 104 m (340 ft), so the static head on the Black Butte Pumping-Generating Plant would be essentially constant at 41 m (134 ft). The plant would be a conventional indoor facility, with three pump-turbine units of 47-m^3/s (1,667-ft^3/s) capacity each and two pump units, each rated at 71 m^3/s (2,500 ft^3/s). Total installed plant capacity would be about 130 MW (pumping) and 45 MW (generating).

The Black Butte Pumping-Generating Plant would require an excavation about 27 m (90 ft) deep. The upper 21 m (70 ft) of that excavation would be in basalt and breccia; the plant would be founded on mudstone and sandstone of the Black Butte Formation.

Two tunnels, each 7 m (23 ft) in diameter, would connect the pumping-generating plant to intake/outlet structures in Black Butte Reservoir. The preliminary cost estimate for these facilities was based on high-level tunnels that would not require Black Butte Reservoir to be drawn down excessively during construction. Each tunnel would be about 550 m (1,800 ft) long. A gate shaft near the upstream end of each tunnel would provide access to a 3.7- by 5.5-m (12- by 18-ft) tunnel gate. A high-level intake/outlet structure would be constructed at the end of each tunnel. (Future studies should consider the possibility that a multi-level intake might be desirable for the tunnel through which generation releases would be made.) Tunneling conditions should be similar to those experienced in the existing outlet tunnel in the right abutment of Black Butte Dam (where some difficulty was encountered with sheared rock).

Black Butte--Glenn Reservoir Facilities

From Black Butte Reservoir, the conveyance system to Glenn Reservoir could follow either the North Fork or the main stem of Stony Creek. The 1978 design studies examined both routes and found that the North Fork Stony Creek alternative (with Tehenn Reservoir) would be significantly lower in cost. This comparison, however, was based on the concept of constructing Glenn Reservoir in a single stage. Next, attention was directed to the possibility of constructing a Thomes-Newville Plan first and incorporating it into a Glenn Reservoir Plan later. In this case, the North Fork Stony Creek route would be far less attractive. Either Newville and Tehenn Reservoirs would have to be drained during construction of the second stage, or major expenditures would have to be included in the Thomes-Newville Plan to provide for the eventual expansion. (These provisions would include discharge tunnels, intake and outlet works, pumping plant foundations, etc.) Either of these options would involve costs that would more than offset the savings associated with the North Fork Stony Creek route. Therefore, it was concluded that the conveyance facilities for a Glenn Reservoir Plan should follow the main stem of Stony Creek if the Thomes-Newville Plan were built first. If Glenn Reservoir were to be built in a single stage, the North Fork Stony Creek route should be selected.

The facilities along the North Fork route would be essentially the same as those already covered by the discussion of the Tehenn diversion route in Chapter 5. They would include a Tehenn Canal and a Tehenn Reservoir, with pumping-generating facilities at Tehenn and Newville Dams. The main differences with the Glenn Reservoir Plan would be the larger flow capacities and the stabilized level of Black Butte Reservoir, which would simplify the design and construction of both the Tehenn Canal and the Tehenn Pumping-Generating Plant. Because these differences would not have any significant effect on the basic physical feasibility of the plan, the discussion in Chapter 5 is applicable here. The remainder of this section is devoted to the features of the conveyance facilities that would follow the main stem of Stony Creek. The layout of these facilities is shown on Figure 1-4.

Millsite Canal

The Millsite Canal would be excavated up the main stem of Stony Creek from Black Butte Reservoir to a pumping-generating plant at Millsite Dam. Like the Tehenn Canal, it would simply be an unlined excavation, generally following the existing creek channel.

For the 283-m^3/s (10,000 ft^3/s) size, the Millsite Canal would have a bottom width of 18 m (60 ft). The total length of the canal would be 13 km (8 mi), of which about 1.3 km (0.8 mi) would be within the normal Black Butte Reservoir area. Maximum depth of cut would be about 44 m (145 ft). The total volume of excavation would be 11 500 000 m^3 (15,000,000 yd^3). Based on limited seismic refraction surveys conducted in 1978, the upper 3 to 9 m (10 to 30 ft) of excavation would be in alluvium or Tehama Formation soils. The underlying weathered mudstone and shale should be rippable, but some light drilling and blasting might be required. Construction of the Millsite Canal would be routine. Most of the work should be concentrated in the summer months, when Stony Creek flows could be controlled at relatively low levels. The entire canal could be completed without drawing Black Butte Reservoir down to an unusually low elevation.

Millsite Reservoir

This diversion route would be selected only if the Glenn Reservoir Plan were to be developed as an expansion of a previously constructed Thomes-Newville Plan. In this case, the Millsite diversion alternative should be selected for the initial development and Millsite Dam would already be constructed as described in Chapter 5.

Millsite Pumping-Generating Facilities

This plant would be located at the head of the Millsite Canal near Millsite Dam. It would lift water from a stabilized Black Butte Reservoir at elevation 144 m (474 ft) to Millsite Reservoir at elevation 183 m (600 ft). The plant would be similar to the others in the Glenn Reservoir Plan. Two pump units would pump 71 m^3/s (2,500 ft^3) each and three reversible pump-turbine units would each handle 47 m^3/s (1,667 ft^3/s). In the pumping mode, the total installed plant capacity would be about 130 MW. Generating capacity would be about 45 MW.

The Millsite Pumping-Generating Plant would be founded on mudstone and shale. The maximum depth of excavation for the plant foundation would be about 49 m (160 ft). The upper portion of the foundation could be excavated by ripping, but drilling and blasting would probably be required in the fresher rock at depth.

Intake-discharge conduits for the Millsite Plant would pass through the upper left abutment of Millsite Dam. The reservoir would be drained while the two 7.0-m- (23-ft-) diameter cut-and-cover conduits were cast in place. Each conduit would terminate at a high-level intake/outlet structure within Millsite Reservoir. Slide gates would be included in both conduits for emergency use. Downstream from the slide gates, the conduits would be steel-lined.

Rancheria Pumping-Generating Plant

The final pump lift to Glenn Reservoir would be made via a plant located near the toe of Rancheria Dam. From the upper end of Millsite Reservoir, the static lift to Rancheria Reservoir would range from 32 to 122 m (105 to 400 ft). This is a very wide range of pumping heads, but operation studies indicate that very little pumping would be done at the lower heads after initial filling of the reservoir.

The Rancheria Pumping-Generating Plant would be a conventional indoor facility with four pump units and four pump-turbine units. All units would be designed for a flow of 35 m^3/s (1,250 ft^3/s). The total installed plant capacity would be 440 MW pumping and about 130 MW generating.

Foundation excavation conditions would be similar to those at the Millsite Plant, but the maximum depth of cut would be only about 30 m (100 ft). The intake-discharge conduits for the Rancheria Pumping-Generating Plant are treated in Chapter 8 as part of the Rancheria Dam facilities.

Release Facilities to Sacramento River

Glenn Reservoir releases would pass back through two pumping-generating plants to Black Butte Reservoir. From there, about 22 percent of the water would be released to Stony Creek, mostly to meet prior downstream commitments, but occasionally as spills. The remaining release, averaging some 925,000 to 965,000 dam^3 (700,000 to 750,000 ac-ft) annually, would be discharged through the Black Butte Pumping-Generating Plant to the Black Butte Canal and thence to the Sacramento River in a separate conveyance system. This system, illustrated on Figures 9-1 and 9-2, would avoid erosion or detrimental ground water impacts along lower Stony Creek, as well as permit maximum energy recovery. The separate release facilities would begin at the downstream end of the Black Butte Canal and include the features described in the following sections.

Sour Grass Generating Facilities

The Sour Grass Generating Plant would develop the 32 m (105 ft) of head between the Black Butte and Sour Grass Canals. As with the upstream generating facilities, the Sour Grass Plant would be designed for a flow of 142 m^3/s (5,000 ft^3/s). Maximum generating capacity would be about 35 MW. The plant would include three equal-sized units. Design and construction would be routine. The maximum depth of cut would be about 34 m (110 ft), all in Red Bluff Formation soils.

The intake to the Sour Grass Generating Plant would take the form of a canal turnout constructed at the junction of the Red Bluff and Black Butte Canals. (This approach would be used in preference to constructing a small reservoir that could create a problem with sediment deposition during winter diversion from the Sacramento River.) Buried steel penstocks would connect the intake to the generating plant.

Sour Grass Canal

This canal would carry releases eastward from the Sour Grass Generating Plant to the intake of the Kirkwood Generating Plant. It would be 7.5 km (4.5 mi) long and would generally follow Sour Grass Creek. The water surface elevation in the unlined canal would be about 72 m (235 ft) and water depth would be about 9 m (30 ft). The total depth of cut at the east end of the canal would be about 30 m (100 ft); at the west end, the water surface would be about 5 m (15 ft) above original ground level. Three major bridge crossings would be required for the divided lanes of Interstate 5 and the adjacent Highway 99E.

Kirkwood Generating Facilities

This plant would develop the final 23 to 27 m (75 to 90 ft) of head between the water surface levels of the Sour Grass Canal and the Sacramento River. Except for the lower head, the Kirkwood Plant would be virtually identical to the Sour Grass Plant. When discharging the design flow of 142 m^3/s (5,000 ft^3/s), its three generating units would have a combined capacity of about 24 MW. Water would enter the plant through buried steel penstocks from an intake at the east end of the Sour Grass Canal. Like the Sour Grass Plant, the Kirkwood Generating Plant would be founded on soils of the Red Bluff Formation. The deepest excavation for the plant foundation would be about 29 m (95 ft).

Kirkwood Canal

The Kirkwood Canal would serve as a tailrace channel for the Kirkwood Generating Plant. It would be an unlined canal, 9.2 km (5.7 mi) in length, discharging to the Sacramento River near the mouth of Burch Creek (River Mile 207.2). The water surface level in the Kirkwood Canal would fluctuate with the water level of the river; at low flows, the river water surface elevation at the mouth of Burch Creek is about 44 m (145 ft) and at flood stage it rises by as much as 5 m (15 ft).

Near the Kirkwood Generating Plant, about 24 m (80 ft) of excavation would be required for the Kirkwood Canal. At the river, the depth of excavation would be only about 8 m (25 ft). The canal invert would be lower than the adjacent river water surface and ground water would be expected to occur at relatively shallow depths. Since it would be unlined, the canal could be excavated by dragline or other such techniques that would not require dewatering.

The Kirkwood Canal would involve a number of major crossings, including: (a) the Tehama-Colusa Canal; (b) the Southern Pacific Railroad; and (c) three county roads. The Tehama-Colusa Canal would be siphoned under the Kirkwood Canal in a concrete box culvert; the road and railroad crossings would be conventional bridges.

A river outlet and fish barrier would be installed at the terminus of the Kirkwood Canal. The fish barrier would consist of stainless steel fish racks to prevent entry of migrating adult salmon and steelhead into the canal. Siting and design of the river outlet structure would be especially challenging because of the continual changes in the river course that occur in the area. The preliminary design and cost estimate prepared in 1978 do not fully reflect potential problems with river meandering; future studies might well demonstrate a need to stabilize the river channel in the vicinity of the Kirkwood Canal river outlet.

Cost Estimates

Table 9-1 summarizes cost estimates for the various conveyance features described in this chapter. The costs were derived by multiplying spring 1978 cost estimates by a factor of 1.20 to approximate spring 1980 price levels. The costs do not provide for escalation during the several years that would be required for construction. Costs of land acquisition are not shown in the table; a detailed estimate prepared in 1978 indicated a cost of about $8 million for all of the right-of-way needed for conveyance facilities from Red Bluff to Glenn Reservoir and from Black Butte Reservoir to the Sacramento River.

Conclusions and Recommendations

Examination of a wide range of alternatives has led to the following conclusions relating to Glenn Reservoir conveyance facilities:

1. Between Red Bluff Diversion Dam and Black Butte Reservoir, the mid-level diversion route is the most promising. It would involve two pumping plants, one near Red Bluff and another near Black Butte Dam.

2. A direct westward diversion from the Sacramento River to Black Butte Reservoir would be less costly to build than any diversion from Red Bluff, but would increase the total pumping lift by about 30 m (100 ft); this alternative deserves additional study when better projections of future energy values become available.

3. If the Glenn Reservoir Plan were to be constructed in a single stage, the diversion between Black Butte and Glenn Reservoirs should follow the North Fork of Stony Creek (via Tehenn Reservoir).

4. If Glenn Reservoir were to be constructed as an expansion of a Thomes-Newville Plan, the diversion between Black Butte and Glenn Reservoirs should follow the main stem of Stony Creek (via Millsite Reservoir, which would have already been constructed as a feature of the Thomes-Newville Plan).

5. Although they would be extensive and costly, none of the Glenn Reservoir Plan conveyance facilities would be of unusual size or complexity. The most challenging technical aspect would be the design of the intake and fish screening facilities for diversion from the Sacramento River.

Planning studies conducted to date have necessarily been somewhat general, because of the large number of alternatives considered. If planning were to be resumed on the Glenn Reservoir Plan, the following recommendations should be taken into account:

1. Large-scale topographic maps should be prepared for corridors along the various conveyance routes. Geology and ground water surveys should be conducted, with particular emphasis on the sites of major structures.

TABLE 9-1

GLENN RESERVOIR PLAN CONVEYANCE FACILITIES
PRELIMINARY COST ESTIMATES
(Price Basis -- Spring 1980)

Pumping Capacity: 283 m^3/s (10,000 ft^3/s)
Generating Capacity: 142 m^3/s (5,000 ft^3/s)

	Estimated Costs			
	Contract	Contingencies	Engineering	Total
Sacramento River to Black Butte Reservoir				
Intake Facilities	$ 15,020,000	$ 1,500,000	$ 3,800,000	$ 20,320,000
Radio Range Pumping Plant a/	59,520,000	7,940,000	16,450,000	83,910,000
Red Bluff Canal	107,720,000	10,780,000	27,250,000	145,750,000
Black Butte Canal	20,040,000	2,000,000	5,080,000	27,120,000
Black Butte Pumping-Generating Facilities a/	84,240,000	10,550,000	22,790,000	117,580,000
Totals	$286,540,000	$32,770,000	$75,370,000	$394,680,000
Black Butte to Newville Res. b/				
Tehenn Canal	$ 12,220,000	$ 1,220,000	$ 3,100,000	$ 16,540,000
Tehenn Reservoir	25,080,000	2,510,000	6,350,000	33,940,000
Tehenn Pumping-Generating Facilities a/	82,430,000	12,460,000	22,900,000	117,790,000
Newville Pumping-Generating Plant c/	136,930,000	19,840,000	38,980,000	195,750,000
Totals	$256,660,000	$36,030,000	$71,330,000	$364,020,000
Black Butte to Rancheria Reservoir d/				
Millsite Canal	$ 54,360,000	$ 5,440,000	$13,750,000	$ 73,550,000
Millsite Reservoir e/	-	-	-	-
Millsite Pumping-Generating Facilities a/	89,300,000	11,220,000	24,190,000	124,710,000
Rancheria Pumping-Generating Plant c/	111,230,000	15,610,000	53,830,000	180,670,000
Totals	$254,890,000	$32,270,000	$91,770,000	$378,930,000
Release Facilities to Sacramento River				
Sour Grass Generating Facilities a/	$ 31,810,000	$ 4,180,000	$ 8,750,000	$ 44,740,000
Sour Grass Canal	13,220,000	1,320,000	3,350,000	17,890,000
Kirkwood Generating Facilities a/	33,880,000	4,340,000	9,240,000	47,460,000
Kirkwood Canal and River Outlet	24,650,000	2,460,000	6,240,000	33,350,000
Totals	$103,560,000	$12,300,000	$27,580,000	$143,440,000

a/ Includes associated discharge lines and inlet/outlet structures.
b/ Alternative that would be used if Glenn Reservoir Plan were to be built in a single stage.
c/ Costs shown are for a Glenn Reservoir elevation of 305 m (1,000 ft); discharge lines and inlet/outlet structure costs are included with costs of adjacent dam.
d/ Alternative that would be used if Thomes-Newville Plan were built first.
e/ Assumed to be built already as part of Thomes-Newville Plan.

2. Much more detailed work needs to be done on the layout and design of intake facilities on the Sacramento River. Special attention should be devoted to sediment, debris, and fish screening aspects.

3. The Red Bluff and Black Butte Canals should be rerouted to reach Black Butte Reservoir at about elevation 113 m (370 ft). This would allow the Black Butte Canal to be realigned to avoid deep cuts and would permit Stony Creek releases to pass through the Black Butte Pumping-Generating Plant.

4. The need for a multi-level intake for releases from Black Butte Reservoir should be considered.

5. Economic studies should be conducted to determine the optimum capacity of the five generating facilities that would be included in a Glenn Reservoir Plan. The $142\text{-m}^3/\text{s}$ ($5{,}000\text{-ft}^3/\text{s}$) capacity used in the most recent studies was selected rather arbitrarily. The optimum capacity of the Sour Grass and Kirkwood Generating Plants would probably be lower than that of the other three plants because 22 percent of the total Black Butte Reservoir release would flow down Stony Creek rather than through these two plants.

6. The siting and design of the Kirkwood Canal river outlet and fish barrier should be reexamined with full recognition of the instability and meandering tendency of the Sacramento River in the area.

APPENDIX A

REGIONAL GEOLOGY, FAULT AND SEISMIC CONSIDERATIONS

APPENDIX A. REGIONAL GEOLOGY, FAULT AND SEISMIC CONSIDERATIONS

The Thomes-Newville and Glenn Reservoir Plans would be located along the boundary between the Sacramento Valley and the Coast Ranges. Rock units in the area may be divided into three distinct groups: (1) Cenozoic and Recent fluvial sedimentary deposits and volcanic rocks; (2) Jurassic and Cretaceous sedimentary rocks of the Great Valley Sequence (GVS) deposited on a basal ophiolite; and (3) the deformed and lithologically diverse rocks of the Franciscan complex. (See Plate A-1.)

Most of the reservoirs and all of the dams and appurtenant structures would be located on rocks of the GVS. This sequence is one of the thickest and most complete upper Mesozoic sections in North America. The section consists principally of clastic sedimentary rocks that occur in simple stratigraphic order, are folded and faulted locally but are not disrupted in detail, and are not affected by other than mild metamorphism. At the base of the GVS is a dismembered ophiolite complex consisting of serpentinite and mafic volcanic and intrusive rocks. An ophiolite is considered to be a fragment of ocean crust and upper mantle.

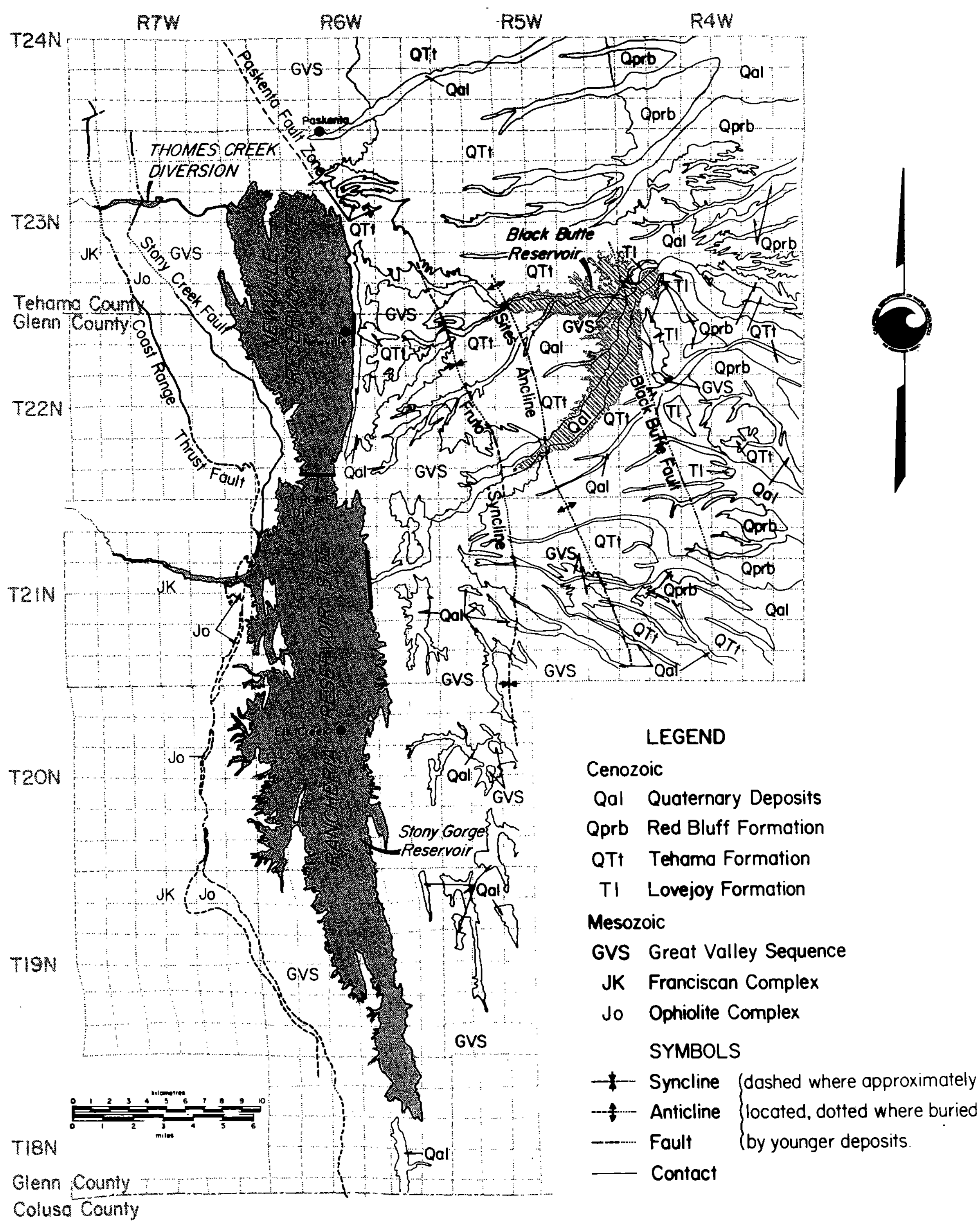
The extreme western part of the Rancheria Reservoir area is on rocks of the Franciscan complex. This unit is lithologically heterogeneous and structurally complex. Various types of graywacke, metagraywacke, shale, argillite, chert, limestone, mafic and ultramafic rocks have been intermingled by pervasive shearing and the formation of melanges. Low- and high-grade metamorphic rocks may occur with unmetamorphosed sedimentary rocks in a single outcrop.

Of particular interest to current planning is the fault contact between the basal ophiolite of the GVS and the Franciscan complex to the west, designated the Coast Range thrust. The contact between the ophiolite and the overlying GVS to the east is also believed to be a fault, called the Stony Creek fault. These faults will be discussed in detail in following sections.

Cenozoic and Recent deposits occur in the Sacramento Valley and locally cap the Franciscan and Great Valley rocks. These deposits include alluvium in stream channels, landslides, terrace deposits, the Red Bluff Formation, Tehama Formation, and the Lovejoy basalts.

Tectonic Setting

The tectonic setting of the Coast Ranges has long been a puzzle. The GVS rocks were believed to be younger and overlie in depositional contact the rocks of the Franciscan complex (Taliaffero, 1943). Irwin (1957, 1964), however, recognized that the two units were the same age and that the structural position of GVS above the Franciscan complex was the result of regional overthrusting. With the introduction of plate tectonics, Hamilton (1969) set forth essentially all the implications for the Mesozoic evolution of California. He argued that the Franciscan complex represents a variety of



Regional Geologic Map Glenn Reservoir Area

ocean floor and trench deposits dragged downward against, and added to, the continental margin by plate consumption. The thrust fault contact between the Franciscan complex and the GVS then marks the former position of a subduction zone, and the Sierra Nevada batholith represents the eroded plutonic base of a volcanic chain that stood landward to the Franciscan trench, just as the volcanic Andes in South America are related to the Peru-Chile trench today. The sedimentary material of the GVS was eroded from the volcanic chain and deposited between the chain and the trench (arc-trench gap deposits).

The oldest part of the GVS apparently was originally deposited concordantly on oceanic crust. At several localities in the Coast Ranges, Upper Jurassic beds pass gradationally downward into pillow lavas and pillow breccias that form the upper horizons of an ophiolite sequence (Bailey and others, 1970). In most places, however, the ophiolite is severely dismembered, structurally dislocated in part by underthrusting of the Franciscan complex beneath it during the Mesozoic, and in part by reverse-faulting in the steep limbs of much younger Tertiary folds (Raymond, 1973).

In mid-Tertiary time, the Farallon Plate and its spreading center, or mid-ocean ridge, was subducted under the continent near the present Gulf of California. Subduction in the Franciscan trench then ceased at that point. The San Andreas fault was born, formed as the triple junction of the Pacific, North American and Gorda plates migrated northward along the coast of California. Below the triple junction, located today at Cape Mendocino, the San Andreas fault separates the Pacific and North American plates and above the junction, the Gorda plate is being subducted under the continent at a very slow rate.

As underthrusting along the Franciscan trench ceased, geologically rapid uplift occurred (Silver, 1971; Suppe, 1978). Marine deposition continued in the Great Valley sedimentary basin until mid-Tertiary time. Since then, nonmarine deposition occurred in the Sacramento Valley (Hackel, 1966), indicating that the basin had been uplifted above sea level. During this latter part of its history, the Great Valley was not a simple forearc basin, because the San Andreas transform fault system was developing in place of the trench along the continental margin.

Stratigraphy

Franciscan Complex

The heterogeneous assemblage of sedimentary, volcanic and metamorphic rocks that stretches nearly the length of coastal California is referred to as the Franciscan complex. The rocks range in age from at least as old as late Jurassic to as young as early Tertiary, generally becoming younger toward the coast. Just west of the proposed reservoir, the most common Franciscan rock type is schist, with some melange occurring in the Stonyford area.

Blueschist (high pressure, low temperature) grade phyllitic sedimentary rocks of the South Fork Mountain schist member of the Franciscan complex are exposed just west of the Coast Range thrust. Included within the schist are blocks of greenstone and serpentinite, and interbeds of metatuff.

The metatuff layers are very little deformed and do not exhibit the secondary chevron-folding characteristic of the associated micaschist. Potassium-argon (K-Ar) age determinations on the tuff vary from 159 to 61 million years, with unique metamorphic events at 155 $\pm$ 5 million years, 120 million years, and possibly younger ages (Fritz, 1975).

The Stonyford melange unit of the Franciscan complex contains exotic blocks of ultramafic rock, glaucophane schist, metavolcanic, and volcanic rocks in a shaley matrix. It structurally underlies the South Fork Mountain schist along a low angle fault.

The rocks of the Franciscan complex form the rugged mountainous terrain of the Coast Ranges. They support a thick covering of brush and trees. The steep-walled canyons are often mantled with landslide debris. The Stonyford melange is less resistant and forms a knobby low terrain covered with grass and scattered trees.

Ophiolite Complex

The various rock types in the ophiolite complex do not always occur in a typical ophiolite sequence in the reservoir vicinity, but are chaotically mixed and in places best described as a serpentinite melange. Included in the ophiolite are (1) volcanic breccia of angular volcanic fragments in a basaltic matrix, (2) bedded radiolarian chert, (3) gabbro, (4) basalts (some with pillow structure) that are altered in places to greenstone, (5) unmetamorphosed to slightly metamorphosed blocks of mudstone and graywacke, and (6) highly weathered and sheared serpentinite as the predominant rock type.

Fritz (1975) has determined the age of the ophiolite in the Paskenta area to be late Jurassic to possibly earliest Cretaceous, based on radiolarian ages of Valanginian to Tithonian. Her K-Ar dates on gabbro from the ophiolite range from 163 to 140 million years.

Radiometric dates on gabbroic rocks indicate that the igneous rocks of the ophiolite are late Jurassic in age (Lanphere, 1971), the same age as the GVS. Deposition of the GVS thus began on newly formed ocean crust, rather than on crust inherited from some older episode of seafloor spreading. As Schweickert and Cowan (1975) have outlined, this crust was probably formed within an interarc basin by backarc spreading behind an east-facing island arc. The arc collided head-on with the continental margin in the late Jurassic. During this event (the classic Nevadan orogeny), the migratory island arc lodged in the subduction zone along the foothills of the antecedent Sierra Nevada, and subduction stepped outward into the Coast Ranges.

The ophiolite complex now crops out on the west side of the reservoir area at the base of the Coast Range between the Franciscan complex and the GVS. It also lies at depth beneath the reservoir area. Surface exposures are easily identified in the field or in color aerial photos by a distinctive greenish-gray soil and sparse vegetation of brush and digger pines. The moderate relief of the outcrop area and the incompetence of serpentinite have produced many large landslides, but which pose no hazard to the proposed reservoir.

Great Valley Sequence

The section of interbedded and intertonguing mudstone, sandstone, and conglomerate, 3 700 to 7 600 m (12,000 to 25,000 ft) in thickness, that underlies most of the Glenn Reservoir area is part of the GVS. The sequence was named by Bailey, et al (1964) to distinguish the Jurassic and Cretaceous rocks from the similar age rocks of the Franciscan and ophiolite complexes. The GVS consists principally of clastic sedimentary rocks in simple stratigraphic order. It is folded and faulted locally, but not disrupted into a melange, and it is affected only by mild metamorphism (Dickinson, et al, 1969; Ingersoll, et al, 1977).

Once regarded as mainly shallow continental shelf and slope deposits, the GVS is now thought to have been deposited mostly as deep-marine turbidities. The sources of these sediments are thought to have been the Sierra Nevada batholith to the east and the Klamath Mountains to the north. They were deposited onto oceanic crust, and locally include materials derived from that source.

Turbidity currents deposited coarse-grained materials by gravity-flow mechanisms at different times and places along the margins of the trough. These turbidities are therefore lensoid, fan or tongue-shaped units set in a continuum of fine-grained deposits. The individual sandstone and pebble conglomerate beds wedge out or grade into finer-grained material over distances of less than 1 km (0.6 mi), or as much as 65 km (105 mi) (Ingersoll, 1977), and probably represent distributary channels in the mid-fan and outer fan environment of submarine valleys.

The GVS includes numerous series, formation, and members. A more complete discussion of the nomenclature and stratigraphy is in Dickinson and Rich (1972) and Ingersoll, et al (1977). Within the study area, about 70 percent of the Great Valley rocks are mudstones, 15 percent are sandstones, and 15 percent conglomerates. The mudstones are dark gray to black, massive to thin-bedded, and in places brittle, and closely fractured. They contain scattered pebbles and limestone concretions. The mudstones form the low areas between more resistant rock units. Dark gray or brown graywacke sandstones form the hills in the reservoir area. The sandstones are interbedded with conglomerates and mudstones. Massive lenticular conglomerates form prominent ridges such as Rocky Ridge and Williams Butte. The conglomerates are composed of small, subrounded clasts in a sandstone matrix.

According to Ingersoll, et al (1977), the stratigraphic nomenclature of the GVS has undergone a varied and controversial evolution. Such stratigraphic names as Knoxville Formation, Shasta Series, and Chico Group are based on faunal rather than lithologic criteria and are of doubtful value as rock stratigraphic units.

The rock stratigraphic units of Dickinson and Rich (1972) and Ingersoll, et al (1977), are based on petrographic analyses of mineral constituents. The differences in mineral content can be observed in outcrop once the nature of the differences has been established by microscopic petrography. In general, rock units thus defined are regionally continuous because the sediments reflect the progress of igneous activity and erosion of the antecedent Sierra Nevada. Each rock unit contains distinct mineral and rock fragments. These units are, from oldest to youngest:

- The Stony Creek Formation, derived mostly from volcanic rocks. The lower part of the section contains sandstones rich in basaltic and andesitic debris.

- The Lodoga Formation, representing erosion from granitic and sedimentary rocks exposed during a period of volcanic quiescence.

- The Boxer Formation, which is rich in volcanic, hypabyssal and plutonic fragments. Boxer time saw the first appearance of extensive quartz-rich volcanic activity and potassium feldspar.

- The Cortina Formation, somewhat similar to Boxer, but with a higher mica content, suggesting that more granitic terrane was exposed to erosion.

- The Rumsey Formation, derived primarily from the vast quartz monzonite plutons of the Cathedral Range intrusive epoch (Evernden and Kistler, 1970), whose volcanic cover had been stripped away by Rumsey time.

Lovejoy Formation

The only occurrence of basalt of the Lovejoy Formation in the study area is in the vicinity of Orland Buttes. Basalt underlies Orland Buttes and Black Butte, with scattered occurrences between the two, and to the south along Walker Creek. Four additional outcrops are present along Salt Gulch, south of Stone Valley, in Sections 8, 16, and 17, T20N, R4W (Chuber, 1961). The basalt forms conspicuous flat-topped buttes with steep to near-vertical flanks. These buttes differ sharply from the more common rounded hilltops and gentle slopes in adjacent areas.

The Lovejoy Formation consists of a series of black olivine basalt flows that has their source to the east (Durrell, 1966). It has a thickness of up to 24 m (80 ft) in the vicinity of Orland Buttes, and consists of at least two distinct flows with a discontinuous basal volcanic breccia (USCE, 1963). The upper flow has a crude columnar structure with fracture spacings of 80 to 600 mm (3 to 24 in). The lower flow lacks any columnar structure, but has blocky fracturing, the fractures being spaced generally 0.3 to 0.6 m (1 to 2 ft) apart. The rocks are dense, brittle to extremely brittle, black to medium gray on hard, fresh surfaces, weathering to brown or gray.

The age of the Lovejoy basalt is thought to be Oligocene or early Miocene (37 to 21 million years old) (Redwine, 1972). At Orland Buttes, the basalt rests unconformably on Cretaceous sediments, and is overlain in places by Tehama deposits. However, the basalt does not continuously underlie the Tehama, but forms isolated patches. The discontinuous nature of the basalt is the result of paleotopography and subsequent erosion (Redwine, 1972).

Near Black Butte Reservoir, the Lovejoy is locally underlain by the Black Butte Series (USCE, 1959). The basal member is 8 m (25 ft) of basaltic conglomerate in a clay matrix. This is overlain by a greenish gray mudstone that has been thermally altered by molten lava which flowed over its surface.

Tehana Formation

Deposits of the late Pliocene to early Pleistocene Tehama Formation underlie much of the area east of the Glenn Reservoir site, cropping out in a belt 3 to 19 km (2 to 12 mi) in width extending nearly continuously from north of Elder Creek to south of Funks Creek (Anderson and Russell, 1939; Jennings and Strand, 1960). Where exposed, the Tehama Formation forms smooth to gently undulating low hills, decreasing slightly in elevation to the east.

The Tehama Formation consists of silty clays, sands, and gravels. Near the base of the formation is a prominent and distinctive tuff member known as the Nomlaki tuff (Anderson and Russell, 1939). Many of the finer-grained sediments within the lower portion of the formation are also tuffaceous. The part of the Tehama Formation above the Nomlaki tuff member consists of as much as 650 m (2,150 ft) of poorly sorted fluvial sand, silt, and clay with lenses of crossbedded sand and gravel. These sediments represent flood plain deposits similar to those being deposited today along the western margin of the Sacramento Valley. A northwestern source for the Tehama sediments was inferred by Anderson and Russell (1939) because of an abundance of minerals and rock types derived from the Coast Range and Klamath Mountains, data from pebble imbrications and foreset beds, and a general eastward decrease in grain size. Within the study area the Tehama Formation commonly dips between 2 and 4 degrees toward the east (Olmsted and Davis, 1961), which might represent an initial depositional dip. A Pleistocene orogeny uplifted and gently folded the western part of the study area, and appears to have terminated deposition of the Tehama Formation (Anderson and Russell, 1939).

Red Bluff Formation

The Red Bluff Formation (originally defined by Diller, 1894, redefined by Russell, 1931) was deposited on the eroded surface of the Tehama Formation. The formation is locally as much as 30 m (100 ft) thick, but generally is less than 15 m (50 ft) thick, and consists of poorly sorted gravels with a reddish silty to sandy matrix (Olmsted and Davis, 1961).

The Red Bluff Formation is an alluvial deposit consisting largely of gravel with a minor amount of interbedded sand and silt. The gravel consists of sub-angular to sub-rounded boulders, cobbles, and pebbles in a matrix of sandy clay that is commonly deep brick-red in color. Although both the Tehama and Red Bluff Formations vary considerably in grain size at any locality where both are exposed, the Red Bluff is usually much coarser. This feature, together with the red color of the Red Bluff, usually distinguishes it from the underlying Tehama.

The Red Bluff Formation is Pleistocene in age. It was deposited post-Tehama (less than 1.3 million years) and pre-late Pleistocene (Steele, 1979). It lies nearly flat except for an anticlinal fold near Corning (Steele, 1979) and the Red Bluff arch, an east-west trending anticline between the City of Red Bluff and Cottonwood Creek (Olmsted and Davis, 1961).

Quaternary Deposits

These sedimentary deposits include terrace deposits, landslides, and Recent Alluvium. Terraces commonly occur next to major streams in the reservoir area, particularly where they flow through mudstone terrain. Steele's (1979) terrace study is a comprehensive reference covering the reservoir area. He delineated six terrace levels by morphological and pedological analysis, developed methods of identification, and determined age relationships. From youngest to oldest, these terraces are designated as Qt_1 (4,000 years old), Qt_2 (10,000 years), Qt_3 (30,000 years), Qt_4 (130,000 years), Qt_5 (between 0.25 and 1.25 million years), and Qt_6 (1.25 million years).

In the study area, thicknesses of the alluvial terrace material range from 0 to about 12 m (40 ft). The deposits are not bedded, for the most part, although some vestiges of bedding and crude sorting of the debris may exist locally in the finer-grained sections. Terrace deposits consist of coarse-grained sand and gravel, containing cobbles and boulders as large as 1 m (3 ft) in diameter. The average grain size appears to decrease downstream and the rounding and sorting of some of the larger fragments may become more pronounced. Often there is a thin irregular layer of silt and clay near the base of the material. This does not appear to be an old soil horizon, but is more likely the result of weathering of alluvial constituents and the redistribution of the finer fraction by percolating water. Ophiolite and Franciscan metasedimentary and metavolcanic rock derived from the Coast Ranges make up most of the terrace material, although fragments of mudstone and shale of the GVS are also intermixed.

Levels of stepped terrace surfaces develop through alternating periods of landscape stability when gently sloping erosion surfaces are formed, and during periods of degradation when streams become entrenched and begin to cut new terrace benches at lower elevations. Renewed downcutting by the streams may result from tectonic uplift or from changes in streamflow due to climatic fluctuations. Terraces are also caused by episodic landslide and flood activity. Steele (1979) concluded that stepped terrace surfaces in the Glenn Reservoir area resulted from Pleistocene climatic fluctuations, thus the various terrace levels and the soils formed on them can be correlated with glacio-eustatic fluctuations in order to establish their relative ages.

Because the terrace surfaces are the only widespread Quaternary marker horizons in the reservoir area, they provide the primary means for assessing recent fault activity. Much of the effort of the Earth Sciences Associates (ESA, 1980) investigation was focused on examining these surfaces where they crossed fault traces to determine whether or not they exhibit evidence of faulting. Details of these examinations are described in the ESA report in Section III, B, D, and E, and in Appendices A, E, and G. With the exception of Qt_4, which may be offset 6 to 18 m (20 to 60 ft) along the Stony Creek fault, none of the other terrace surfaces near the reservoir area exhibit any evidence of tectonic disturbance.

Landslides in the vicinity of Glenn Reservoir are generally in the Franciscan and Ophiolite complex, where steep topography, incompetent material, and actively eroding streams combine to produce an environment favorable to landslide development. Slumps and earthflows are the landslide types that predominate in these units. A few small landslides also occur in the Great Valley rocks, terrace material, and Tehama Formation.

Alluvial deposits are present in nearly all the Glenn Reservoir area. These unconsolidated deposits range in grain size from gravel to clay, with most being in the sand-to-silt size fraction. Plant material is commonly scattered throughout the deposits. The alluvium consists of active stream channel deposits and older, partially stabilized, deposits that form flood plains in the lower reaches of most streams.

Within the Coast Range, alluvium is composed of locally derived fragments of schist, serpentinite and Franciscan type rocks. East of the range front, the alluvium consists of rock fragments derived from the Coast Range and sandstone, shale and conglomerate clasts of the GVS. In general, there is a gradual reduction of grain size eastward, with a greater percentage of large boulders in the alluvium within the Coast Ranges than in alluvium east of the range front.

The thickness of alluvium varies throughout the study area, ranging up to about 15 m (50 ft). Locally, within narrows and constrictions, alluvium is absent in channel bottoms, and bedrock is exposed. Alluvium often interfingers with landslide deposits at the base of hillslopes.

Regional Geologic Structure

The Glenn Reservoir area is characterized by three regional north-trending structural features: (1) the folded, dominantly east-dipping beds of the GVS; (2) the Stony Creek fault forming the boundary between the Coast Range upland and the Sacramento Valley foothills; and (3) the Coast Range thrust to the west of the reservoir area. Subsidiary structures pertinent to the current study include the Paskenta fault zone, Willows fault, Black Butte fault, and various unnamed cross faults within the reservoir area. The San Andreas fault zone, 130 km (80 mi) to the west, must be considered for seismic design of reservoir structures.

Folds

The Great Valley Sequence forms a broad homoclinal structure with dips generally from 40 to 80 degrees east throughout the Glenn Reservoir area. Near the Stony Creek fault, at the base of the Coast Ranges west of the reservoir, the rocks are steepened and locally overturned. East of the reservoir, the Fruto syncline and Sites anticline (the two largest folds on the west side of the Sacramento Valley) are superimposed on the homocline.

The flexures termed Sites anticline by Kirby (1943) and Fruto syncline by Chuber (1961) extend in a northerly direction parallel to the Coast Ranges for at least 64 km (40 mi) from the town of Sites to at least the Newville Road, and possibly to northeast of Paskenta. The folds developed in middle Cretaceous sediments from east-west compression prior to Pliocene (Tehama) deposition (Chuber, 1961). In addition to these major folds, minor folds occur locally in the Great Valley rocks.

The Franciscan complex has a predominant northwest structural trend and is highly deformed as a result of pervasive shearing and folding.

Stony Creek Fault

The contact between the base of the Great Valley Sequence and the Great Valley ophiolite complex is referred to as the Stony Creek fault. The contact zone has been variously interpreted as a major dislocation and as a depositional contact. Both interpretations are probably correct, depending on the area examined. Original offsets probably occurred during the late Jurassic, although movement in more recent tectonic regimes could have occurred.

Detailed mapping in the Elk Creek area by Chuber (1961) led him to conclude that the Stony Creek fault dips west at a high angle and probably has more than 900 m (3,000 ft) of reverse displacement. He also describes as much as 450 m (1,500 ft) of crumpled beds of the Sanhedrin Formation (now the Stony Creek Formation) adjacent to the Stony Creek fault. The contact itself is marked by a 12-m (40-ft) thick gouge zone at Grindstone Creek with many subparallel faults having small vertical displacements exposed in the area.

Brown (1964) put the Stony Creek fault at the serpentine-siltstone contact in the Stonyford quadrangle and considers it to represent a zone of major thrust faulting.

The depositional nature of the Great Valley ophiolite contact was first described in 1933 by Anderson in a regional context. He acknowledges that small displacement faults do exist at the contact, but this condition "does not appear to be general" and that they may be "disregarded as being unimportant" (p. 1242).

More recently, Bailey et al (1970) have described the contact as depositional. They find several localities where upper Jurassic beds pass gradationally downward into pillow lavas and pillow breccias that form the upper horizons of an ophiolite sequence. In this interpretation, the ophiolite is common with the overlying strata and is underthrust by the Franciscan complex. In most places, the ophiolite is severely dismembered, structurally dislocated in part by underthrusting during the Mesozoic, and in part by reverse faulting in steep limbs of much younger Tertiary folds (Raymond, 1973).

However, in the Glenn Reservoir study area, Bailey et al describe "an uncontestable depositional succession, extending up from ultramafic rock through mafic volcanic rock to mudstone, exposed in the channel of the South Fork of Elder Creek" (p. C72). At Stonyford, they reinterpret Brown's (1964) work to show a standard depositional sequence that has been offset along the Stony Creek fault to only a minor degree. These exposures and studies at eight other sites led investigators to conclude that the ophiolite complex was Mesozoic oceanic crust with Great Valley rocks resting depositionally above.

Mapping of trenches, road cuts, and creeks by ESA (1980) suggests the contact is a fault in a number of places. On the south side of Thomes Creek near Hatch Flat, the fault strikes north-northwest and dips 55 degrees east. The fault zone is about 30 m (100 ft) wide with 15 m (50 ft) of mixed and recrystallized shale on the east and about 15 m (50 ft) of sheared and strongly foliated serpentinite on the west. Near the Grey Eagle chromite mine west of Chrome, the fault strikes nearly north and dips 45 to 65 degrees west.

The peridotite adjacent to the fault is sheared and brecciated in a zone 30 to 150 m (100 to 500 ft) in width. A cross fault appears to offset the Stony Creek fault near the mine, but it could not be traced through this shear zone.

At Hull Road, the fault is sharp, 0.15 to 0.3 m (1/2 to 1 ft) in width, with carbon deposits in the center of an extremely punky, powdery zone. Both the serpentinite and shales are moderately sheared beyond this fault zone. At Forest Highway 7, northwest of Elk Creek, the contact trends north-south and dips 25 to 45 degrees to the west. The fault is very clean, with little mixing of surrounding rocks.

The evidence is that there are faulted and depositional contacts between the GVS and the ophiolite. The contact probably represented a zone of weakness that failed in some but not all places during Cretaceous compression.

ESA found no Tertiary and Quaternary faulting along the trace of the Stony Creek fault. However, two correlative terraces (dated as 130,000 years old) on opposite sides of the fault (Steele, 1979), with an apparent vertical offset of 6 to 18 m (20 to 60 ft), were interpreted as a fault offset by ESA (1980). Terrace surfaces 30,000 years old were not offset. Last movement on the fault would then have between 30,000 and 130,000 years ago (ESA, 1980). Landslides, debris dams, waterfalls, or other sharp changes in stream gradient are other possible explanations for the apparent offset. This area needs further study.

Coast Range Thrust

The Coast Range thrust (CRT) is the contact zone between the Franciscan complex on the west and the Great Valley ophiolite complex on the east. The Coast Range thrust generally lies 0.4 to 4.8 km (1/4 to 3 mi) west of the Coast Range front in the reservoir area. It passes through low foothills southwest of Elk Creek. The fault is easily recognized on false color infrared photography, owing to vegetation and topographic differences of the juxtaposed rocks. The thrust was active during late Jurassic time and may not have been active as a thrust fault since that time. Geologic evidence suggests that the most recent movement was before at least 130,000 years ago.

Irwin (1964) noted that the contact zone is a regional thrust fault of considerable magnitude. There is evidence, however, to suggest that much of the actual fault displacement occurred in the Franciscan complex (Maxwell, 1974; Raney, 1976; Suppe, 1979) and not along the thrust contact.

The CRT, however, was caused by east-west compressive stresses from subduction of oceanic crust beneath the continent. This changed to transform faulting 5 to 10 million years ago (Atwater, 1970) at the latitude of the

Glenn Reservoir area. Since that time, the tectonic regime has changed from compression to north-south transform shear and isostatic rebound. It is possible that since the CRT is a zone of weakness, renewed movements coincident with uplift of the Coast Ranges and right lateral strike slip along the San Andreas could have occurred or may occur in the future.

Paskenta Fault Zone

This fault was mapped in detail by Jones et al (1968, 1969) by using detailed biostratigraphic correlations of Buchia zones, high altitude aerial photography, and side-looking airborne radar imagery. The fault zone is one of the most striking structural features near the Glenn Reservoir complex area. It can be followed for more than 11 km (7 mi) near Paskenta, merging with the Stony Creek fault at the north and becoming either a bedding plane fault or dying out at the south. Total displacement along the fault zone is more than 8 km (5 mi). The zone was probably active during Cretaceous and early Tertiary time and could be the result of tear faulting associated with "Coast Range" type thrusting. Movement along the Paskenta fault zone had ceased prior to the beginning of Tehama deposition, 3.3 million years ago, and the fault need not be considered in formulating seismic design criteria.

Willows Fault

The Willows fault is a north-to-northwest trending, steeply dipping reverse-to-normal fault. It has a length of 61 km (38 mi), extending from just west of Artois, south to a point 22 km (14 mi) east of Williams. It is recognized only in the subsurface, on the basis of gas well data. In the northwest, it offsets late Miocene deposits as well as the basal portion of the late Pliocene Tehama Formation. The most recent movement in the southeastern portion could possibly have been post-Tehama Formation. A low level of historic seismicity has occurred near the Willows fault, but correlation of relocated epicenters with the fault is not convincing because of the apparent scatter in the data. No surface rupture has been reported.

Black Butte Fault

The Black Butte fault has been reported to be a north 10 degrees to 20 degrees west trending high-angle normal fault, with the west side downdropped relative to the east. This fault has a surface length of about 17 km (11 mi) and a suspected vertical throw of about 760 m (2,500 ft). Numerous splay faults are associated with it. The fault trace has been mapped as offsetting late Pliocene-early Pleistocene Tehama deposits (USCE, 1963).

Investigations by ESA (1980) showed no faulting or deformation of Tehama deposits along the trace of the fault. They concluded that the Black Butte fault and associated splay faults should be considered pre-Tehama (pre 3.3 $\pm$ to .4 million years old) and need not be considered in formulating seismic design criteria.

Cross Faults

Numerous minor faults have been mapped that are oriented oblique to the north-south regional structural grain in the proposed reservoir area. Since none of these has been found to displace geologically young materials, they have been grouped under the term "cross faults". The faults are probably Cretaceous in age and need not be considered except where engineered structures would be located on them.

Seismicity

The Glenn Reservoir would be in an area of low-to-moderate seismicity. Epicenters plotted in the region total 180 since 1928. Several historic events between 1903 and 1921 are also included. The data were compiled by ESA (1980) and include events from the files of the University of California, Berkeley, the U. S. Geological Survey, California Division of Mines and Geology, and California Department of Water Resources.

Epicenters are scattered throughout the area. ESA concludes that there is a vague association between mapped faults and seismicity, but that the association lacks any special significance for predicting future earthquake activity. Primary emphasis for assessing future hazards must therefore be on the geologic record. The largest quake in the vicinity had a magnitude of 4.7 and was about 50 km (30 mi) from the Glenn Reservoir site.

Focal mechanisms could not be determined for most of the earthquakes, with the exception of a small earthquake swarm near Alder Springs. These were determined to be indicative of horizontal tensile stresses. ESA (1980) believes that this sequence is not indicative of the regional stress regime, but rather, represents a local stress concentration. However, it is also possible that these tensile stresses conform to regional uplift of the Coast Ranges. Analyses of future seismicity may clarify this problem.

Epicenters of 15 earthquakes appear to be associated with the subsurface trace of the Willows fault, located about 30 km (20 mi) east of the reservoir area. The seismicity, ranging in magnitude to 4.7, could be associated with the Willows fault, and thus it should be considered a fault capable of a magnitude 5 shock at any time (ESA, 1980).

ESA believes the Stony Creek fault should be considered potentially active and capable of a magnitude 4 shock at any time. This is based on seismicity nearby ranging from 2.8 to 3.6 and the possibility of movement along this fault between 30,000 and 130,000 years ago. ESA assigns a maximum credible earthquake of 6.5 to this fault.

Reservoir-Induced Seismicity

Both Newville Reservoir and Glenn Reservoir would be deep (greater than 92 m--300 ft) and very large in volume. Although available data and understanding of reservoir-induced seismicity are limited, evaluation of the data does indicate that it is much more likely to occur at large, deep reservoirs than at smaller ones. One survey of 234 large reservoirs found

29 accepted cases of reservoir-induced seismicity. A total of 45 cases have been accepted worldwide through 1978 (Packer et al, 1979).

A total of ten accepted or questionable cases of reservoir-induced seismicity with maximum magnitude of five or more have been documented. Eight have occurred at deep and/or very large reservoirs. Among the ten reservoirs, eight have evidence indicating active faulting nearby and the other two probably have active faults, although the evidence is not conclusive (ESA, 1980).

Most of the faulting in the Glenn Reservoir area took place during Mesozoic to early Cenozoic time. At that time, east-west compressive forces tilted, folded, and displaced the Jurassic and Cretaceous sediments to their present configuration. Table A-1 summarizes the important faults in the area and the age of most recent movement on each fault. This is the age that can be established on the basis of field evidence; the actual age of latest movement is likely to be considerably older on all faults except possibly the Stony Creek fault.

TABLE A-1

SUMMARY OF FAULTS IN THE VICINITY OF GLENN RESERVOIR*

Name and Predominant Fault Type	Closest Distance to Reservoir km	Closest Distance to Reservoir (mi)	Maximum Length km	Maximum Length (mi)	Most Recent Displacement
Stony Creek Fault (reverse-normal, oblique)	0	0	480	300	over 130,000 years
Coast Range Thrust (thrust)	0	0	105	65	30,000-130,000 years
Paskenta Fault Zone (left-lateral)	0.3	0.2	16	10	pre-late Pliocene (over 3.3 million years)
Willows Fault (reverse-normal)	24	15	61	38	late Pliocene
Black Butte Fault (normal)	14	9	18	11	pre-late Pliocene (over 3.3 million years)
Cross Faults (lateral, oblique)	0	0	10	6	pre-late Pliocene (over 3.3 million years)

*From Earth Sciences Associates, 1980

The Stony Creek fault is the only fault in the vicinity of the reservoir that exhibits evidence of movement as young as late Pleistocene. Terrace surfaces (Qt_3) mapped on both sides of the fault trace at Thomes Creek indicate that no displacement has occurred on the fault in the last 30,000 years. However, an older set of terrace surfaces (Qt_4), approximately

130,000 years old, may have been vertically displaced between 6 to 18 m (20 to 60 ft). Geomorphic relationships (ESA, 1980) also suggest that significant uplift of the Coast Range front has occurred on the Stony Creek fault since deposition of the Tehama Formation (Plio-Pleistocene). Latest movement on the Stony Creek fault is, therefore, pre-Holocene, but may be as young as late Quaternary. Although, on this basis,the fault is not considered active for the purpose of evaluating reservoir-induced seismicity, ESA feels it would be prudent to consider the fault to be "potentially active".

The selection of a maximum credible event for possible reservoir-induced seismicity at Glenn Reservoir is mainly a professional judgment guided by the interpretation of the evidence and the present understanding of faulting and fault activity in the vicinity of the site. It is also contingent upon the judgment that reservoirs may act as triggering mechanisms for earthquakes, but are not capable of inducing significant earthquakes without the presence of active faults. ESA (1980) assigned an earthquake magnitude in the range of 6.0 to 6.5 as a conservative engineering estimate of the maximum credible reservoir-induced event for Glenn Reservoir.

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APPENDIX B

NEWVILLE DAM SITE GEOLOGY

APPENDIX B. NEWVILLE DAM SITE GEOLOGY

Newville Dam site would be located on the North Fork of Stony Creek, where it passes through Rocky Ridge about 29 km (18 mi) west of Orland. Access to the site is by paved road from Orland or partially paved road from Paskenta, 16 km (10 mi) to the north.

Newville Dam would be an embankment-type structure some 90 to 120 m (300 to 400 ft) high. This appendix discusses foundation conditions for the dam and appurtenant structures. Appendix C covers the geology of Rocky Ridge, which would form the east rim of the reservoir.

Previous Studies

The Department first appraised Newville Dam site in the mid-1950's as a part of the studies for Bulletin 3, "The California Water Plan". Later, a reconnaissance geology report was prepared under the North Coastal Area Investigation (DWR, 1959).

Follow-up studies concentrated on Rocky Ridge geology; these studies included core drilling and water pressure testing at various saddles along the ridge (DWR, 1961). No damsite foundation drilling was done during the Rocky Ridge study, but one diamond core hole was drilled and water pressure-tested in the saddle south of the right abutment. Concurrent with the Rocky Ridge study, the Department conducted investigation of earth material sources that included drilling four flight-auger holes east of the damsite (DWR, 1965).

The Department's final report on the North Coastal Area Investigation included an evaluation of the geologic conditions at Newville Dam site (DWR, 1965). The appraisal consisted primarily of reconnaissance geologic mapping of the foundation and did not include drilling at the site.

After the Department's report, the Bureau of Reclamation mapped Newville Dam site geology at a scale of 1:2400 with a contour interval of 1.5 m (5 ft). Ten vertical and angle diamond drill core holes were drilled and water pressure-tested in the foundation. Twelve bucket auger holes were drilled near the damsite to investigate construction materials. The Bureau prepared a comprehensive report on these studies as a part of its feasibility analysis of the Paskenta-Newville Unit (USBR, 1967).

Earth Sciences Associates (ESA) recently investigated the reservoir area to determine fault and seismic hazards (ESA, 1980). Their study at the damsite included four backhoe trenches, geologic mapping and terrace age-dating. Their findings are included in this report.

Dr. Douglas D. Campbell, a consultant for the Department, made a brief reconnaissance of the reservoir area in May 1979 as part of a study of Rocky Ridge. Although his comments (Dolmage, Campbell and Associates, 1979 and 1980) are not specifically directed to the damsite, his conclusions on stability, permeability, and grouting are applicable to the site.

Site Geology

Outcrops of the Stony Creek Formation of Jurassic to Cretaceous age are moderately well exposed in the area. The formation consists of conglomerate, sandstone, and mudstone. These sedimentary rock units strike nearly north-south and dip steeply (50-80°) east. Detailed descriptions of the lithology and physical properties of rocks are found in Appendix C. Along the crest and upper flanks of Rocky Ridge, resistant conglomerate forms bold, massive outcrops. Good exposures of sandstone occur on the steep western slope of the ridge and along secondary strike ridges. Mudstone is the least resistant rock in the area. It is covered by colluvium and soil in most places, but crops out in North Fork Stony Creek and along road cuts.

The rocks are grouped into five units on the geologic map of the dam site (Plate B-1). Mudstone, sandstone, and conglomerate are described briefly below. The two other units, which are combinations of these three lithologies, are pebbly mudstone and interbedded mudstone and sandstone. These units are transitional, grading into the other rocks. In places, however, they occur as distinct types and will be so discussed.

As used here, "mudstone" designates clay, silt, claystone, siltstone, shale and argillite. The term is used when the clay, silt and sand fractions vary from place to place so that more precise terms are not possible. It is dark gray, soft to moderately hard, and usually thin-bedded. It is generally interbedded with fine-grained sandstone. The mudstone slakes to small angular fragments when exposed to air. Where the mudstone contains considerable sand-sized particles, this slaking is greatly reduced.

The sandstone is a gray, well indurated and moderately hard graywacke. It has an argillaceous cement, but may be locally cemented with calcite. It varies from fine- to coarse-grained with the fine-grained variety dominating. The sandstone in the central portion of the ridge is massively bedded to 6 m (20 ft) in thickness. It contains pebble lenses, and grades to conglomerate. Near the flanks of the ridge, the beds average less than 0.3 m (1 ft) in thickness and are interbedded with mudstone.

Conglomerate is the most resistant rock in the area. It is gray when fresh, moderately hard, and varies from poorly to well indurated. It has an argillaceous cement, but may locally be cemented with calcite. The conglomerate consists of rounded clasts of chert and metavolcanic rocks in a sand and clay matrix. The clasts are generally 3 to 20 mm (2.5 to 10 in) in diameter. The conglomerate usually occurs interbedded with sandstone in massive, lenticular beds.

Colluvium, Recent alluvium, and terrace deposits cover about 20 percent of the foundation at Newville Dam site. Colluvium, consisting of gravelly clay, is generally less than 1.5 m (5 ft) thick in the foundation area. Recent alluvial sands and gravels cover parts of the channel section up to 1.5 m (5 ft) thick.

Extensive terrace deposits designated Qt_3 1/ occur upstream and downstream from the damsite and cover part of the foundation in the channel. These deposits consist of 1.5 to 6 m (5 to 20 ft) of sandy clay overlying 1 to 5 m (3 to 15 ft) of silty-to-clayey sand and gravel. Scattered remnants of older terraces--$Qt_{3,5}$, Qt_4, and Qt_5--occur at various elevations in the foundation area and consist of compact cobble and boulder-bearing clayey gravel. These terraces were used by ESA to evaluate the age of faulting at the damsite.

A zone of right-lateral faulting trends northeasterly through Newville Dam site. Geologic mapping and drilling show that the faults dip steeply and offset lithologic units. Complex fault movement makes the total amount of displacement across this zone difficult to determine, but it is apparently 370 to 1 200 m (1,200 to 4,000 ft). This faulting is confined to the Jurassic-Cretaceous bedrock and is considered by ESA (1980) to be at least pre-Tehama Formation in age (3.3 million years ago).

The principal effect of faulting on the physical character of sandstone and conglomerate beds along Rocky Ridge appears to be the development of a fractured and broken zone adjacent to the fault plane. At places it is more pervious. Drill hole N-106, a north-bearing angle hole in the left abutment, showed no water loss during a water pressure test of the faulted and fractured 54.4-57.5 m (178.6-188.6 ft) interval. In contrast, drill hole N-107, a north-bearing angle hole located in the channel, shows that open fractures, as determined by water tests, persist to 67 m (220 ft) below ground surface. Except for these peripheral effects, the faults have not significantly altered the physical properties of the rocks at Newville Dam site. For example, the fault (encountered in drill hole N-106) appears to be about 1 m (3 ft) wide. Most of the faulted interval consisted of sandstone fragments iwth only 100 mm (3 in) of gouge recovered. In outcrop, faults are in places healed with calcite and are nearly as hard as the surrounding rocks.

Faults appear to widen and branch irregularly in the mudstone beds. The inferred westward continuation of a major fault at Newville Dam site was crossed in drill hole N-105 between 12.8 and 27.7 m (41.9 and 90.8 ft). The entire interval consisted of closely fractured and slickensided rock; drillers reported numerous "mud seams" that washed away during drilling. Caving and sloughing were severe, and it was necessary to follow the hole advancement closely with casing.

One 0.6-m (2-ft) wide fault is exposed in the bed of Heifer Camp Creek about 180 m (600 ft) southwest of drill hole N-102. The fault is filled with sheared but compact mudstone and calcareous gouge that appears to be nearly as competent as the surrounding rock. In places, calcite-filled faults occur in creek bottom exposures standing in relief above the surrounding mudstone beds.

Two sets of joints occur at Newville Dam site: one set strikes northeast and dips near vertical; the second set strikes parallel to the ridge and dips east or west at zero to 45 degrees. Joint spacing is widest in the

1/ The various terraces in the area are discussed in Appendix A.

conglomerate beds, 0.6 to 2.1 m (2 to 7 ft) typically, but up to 1 m (1 to 3 ft). Outcrops of mudstone are close and irregularly checked due to air slaking combined with the jointing.

Foundation Conditions

Subsurface foundation conditions have been explored by the Bureau of Reclamation, who drilled 10 core holes to depths of 93 m (305 ft). ESA excavated four backhoe trenches to 4.6 m (15 ft) deep. The saddle south of the damsite was drilled to 28.5 m (93.6 ft) by the Department during the investigation of Rocky Ridge (DWR, 1961).

In 1961, the Department tested 31 fresh sandstone and conglomerate samples from drill holes along Rocky Ridge. These were from the same geologic units mapped at the damsite. Unconfined compressive strengths averaged 60 000 kPa (8,700 lb/in^2) for the conglomerate, and 81 400 kPa (11,800 lb/in^2) for the sandstone (DWR, 1965). These data indicate the foundation has sufficient strength for an embankment-type dam of the height being considered.

Right Abutment

The right abutment rises evenly at about 40 percent to an elevation of 315 m (1,035 ft). There are a few scattered rock outcrops, but most of the abutment is covered with up to 1 m (3 ft) of colluvium and organic debris. There are also occasional boulders of conglomerate to 4 m (12 ft) in diameter.

The two NX core holes drilled on the right abutment (N-103) and N-108) show that the abutment is intensely weathered to a depth of 3 to 7 m (10 to 20 ft) with pockets as deep as 20 m (60 ft). Fresh rock is found at about 15 m (50 ft). The drilling pads constructed during the exploration showed that in places the upper 3 m (10 ft) of intensely weathered rock is rippable.

Foundation preparations should include the removal to an average depth of 5 m (15 ft) of all colluvium and the intensely weathered bedrock. The cutoff trench should be excavated to fresh rock to an average depth of 9 m (30 ft). Stripping of the upper 3 m (10 ft) of colluvium and intensely weathered rock should be accomplished by common methods.

Channel

The active stream channel is 18 to 24 m (60 to 80 ft) wide and is covered locally by stream gravel deposits that average 1.5 m (5 ft) in thickness. In places, where the stream has eroded the alluvium, the bedrock appears to be fresh to lightly weathered to a depth of 0.3 to 0.9 m (1 to 3 ft). Terrace deposits and colluvium cover the bedrock near the flanks of the abutments to a maximum depth of 6 m (20 ft) and in the center of the channel, upstream from the axis, to a depth of 9 m (30 ft). There are also a few boulders of conglomerate to 8 m (25 ft) in diameter.

The six NX core holes drilled in the channel section (N-101, N-101A, N-102, N-105, N-107, N-109) show that fresh rock occurs up to 9 m (30 ft) below the terrace deposits. Water pressure tests show that the rock is impermeable except locally along small fractures. All water losses were less than 75 litres per minute (l/min) (20 gal/min), and most were less than 20 l/min (5 gal/min).

Foundation preparations should include the removal of all alluvium, colluvium, terrace deposits, and intensely weathered bedrock. The average depth of stripping would be about 8 m (25 ft). The cutoff trench should be excavated to fresh rock, which will be at a depth of 12 m (40 ft) locally. Approximately the top 8 m (25 ft) of stripping should be accomplished by common methods.

Left Abutment

The left abutment has a gentle 30 percent slope and rises evenly to an elevation of 305 m (1,000 ft), where it flattens to a saddle-and-knob topography to elevation 365 m (1,200 ft). Bedrock crops out almost continuously on this abutment, with soil and colluvium pockets locally up to 1 m (3 ft) in thickness. There are a few boulders of conglomerate up to 2 m (5 ft) in diameter.

The two NX core holes drilled in the left abutment (N-104 and N-106) show that the abutment is intensely weathered to a depth of 3 m (10 ft). The depth of fresh rock varies from 9 m (30 ft) to 20 m (65 ft). Dozer cuts during drill pad construction showed that in places the upper 3 m (10 ft) of intensely weathered rock is rippable.

Foundation preparations should include the removal of all colluvium and intensely weathered bedrock. The average depth of stripping would be 3 m (10 ft). The cutoff trench should be excavated to fresh rock to an average depth of 10 m (30 ft). The upper 1 m (3 ft) of stripping should be accomplished by common method.

Grouting

Water pressure testing on all drill holes has shown that the foundation rocks are essentially impervious. The data available on faults indicate that these structures are not pervious, but may contain fractures that could contribute to local leakage. A grout curtain should be constructed to control possible leakage. The grout take should be low, although a few intervals of high grout take would occur in open-jointed rock. Care should be taken to avoid excess grout pressure, since some water tests showed increasing permeabilities throughout the test cycle, suggesting that fractures were being progressively opened. As far as can be determined, depths of intensely weathered rock not judged groutable are fairly uniform over the abutments and beneath the channel section of the dam. Some deeper zones of intense weathering that would require additional excavation might be encountered next to faults and in poorly cemented conglomerate lenses. However, except for one such interval in drill hole N-108, the location of these zones of rock weakness cannot be predicted. It is not considered necessary to completely remove all material from deep and small zones of intense weathering.

Spillway

Geologic reconnaissance of Rocky Ridge shows many suitable spillway sites. Investigations to date have revealed no significant geologic problems (DWR, 1980). All spillway alignments would be excavated in conglomerate and sandstone, with the lower chute in mudstone. Any spillway alignment would parallel the east-northeast-trending faults for a considerable distance. The general area must be carefully explored to establish the spillway alignment. Excavation should remove all intensely weathered rock, and the spillway should be lined. Cut slopes should be stable to 1:1 with berms every 9 to 12 m (30 to 40 ft). Neither the jointing nor air slaking would contribute to rapid slope degradation.

Intake Structure

An inclined intake structure located on either abutment would have its base on mudstone, but most of its foundation would be on conglomerate and sandstone. Most of the weathered rock, which extends to a depth of about 9 m (30 ft), would be removed by the required excavation. The upper 3 m (10 ft) would probably be common excavation. Locally, the foundation might have to be overexcavated to remove pockets of intensely weathered rock, and then backfilled. Differential settlement of the foundation rock should not be a problem and the cut slopes should be stable to 1:1.

Diversion and Outlet Tunnels

Any tunnel driven through the ridge should encounter good rock conditions in fresh rock. Whenever the tunnel had less than 3 m (30 ft) of cover, weathered rock would be expected. Typical tunneling conditions for a tunnel through the ridge are shown in Table B-1.

Surface exposures and drill-hole data indicate that faults in sandstone and conglomerate beds are generally less than 1 m (3 ft) wide, and for 2 to 3 m (7 to 10 ft) on either side, the rock is more closely fractured than the average. The east-northeasterly trending faults would parallel the tunnel alignment for a considerable distance, so it would be essential to explore the general alignment carefully to establish an alignment that was as free of faults as possible, or that crossed perpendicular to the faults.

Water inflows in portions of the tunnel should be slight. Estimated maximum short-duration inflows would be about 40 l/min (10 gpm) from closely jointed zones in the conglomerate and sandstone.

TABLE B-1

TYPICAL TUNNELING CONDITIONS IN ROCKY RIDGE

% of Tunnel	Rock Type	Rock Conditions (Terzaghi Factors)	Rock Load Factors	Support
40	Conglomerate	Moderately blocky and seamy	0.35 $(B+H_t)$	Light
40	Alternating beds of sandstone and mudstone	Very blocky and seamy	0.70 $(B+H_t)$	Moderate
12	Sandstone	Moderately blocky and seamy	0.30 $(B+H_t)$	Light
7	Mudstone	Very blocky and seamy	0.70 $(B+H_t)$	Moderate with wire mesh to control the effect of spalling upon exposure to air
1	Fault zone	Crushed	1.10 $(B+H_t)$	Heavy

Conclusions

1. The Newville Dam site foundation is suitable for an embankment-type dam at least 128 m (420 ft) high or to elevation 311 m (1,020 ft).

2. There are no active faults at the site. The inactive faults would pose no unusual construction difficulties.

3. The rocks in the foundation are essentially impervious. Some grouting would be required due to higher permeabilities along faults and joints.

4. No significant geologic problems should be encountered for the spillway, intake structure, or outlet works.

Recommendations

1. Detailed geological formation mapping should be carried out for the dam and appurtenant structures.

2. A seismic refraction survey should be conducted to clearly define the depth to unweathered bedrock in the foundation of all structures.

3. Faults near all foundations should be trenched to determine their widths and character.

4. Diamond core drilling investigation should be made to study the (a) lithology; (b) width, character, and continuity of joints; and (c) depth of weathering at the proposed sites of the planned structures.

5. Water pressure tests should be made in conjunction with the drilling to investigate foundation permeability.

6. Laboratory and in situ rock strength tests should be conducted on foundation rocks.

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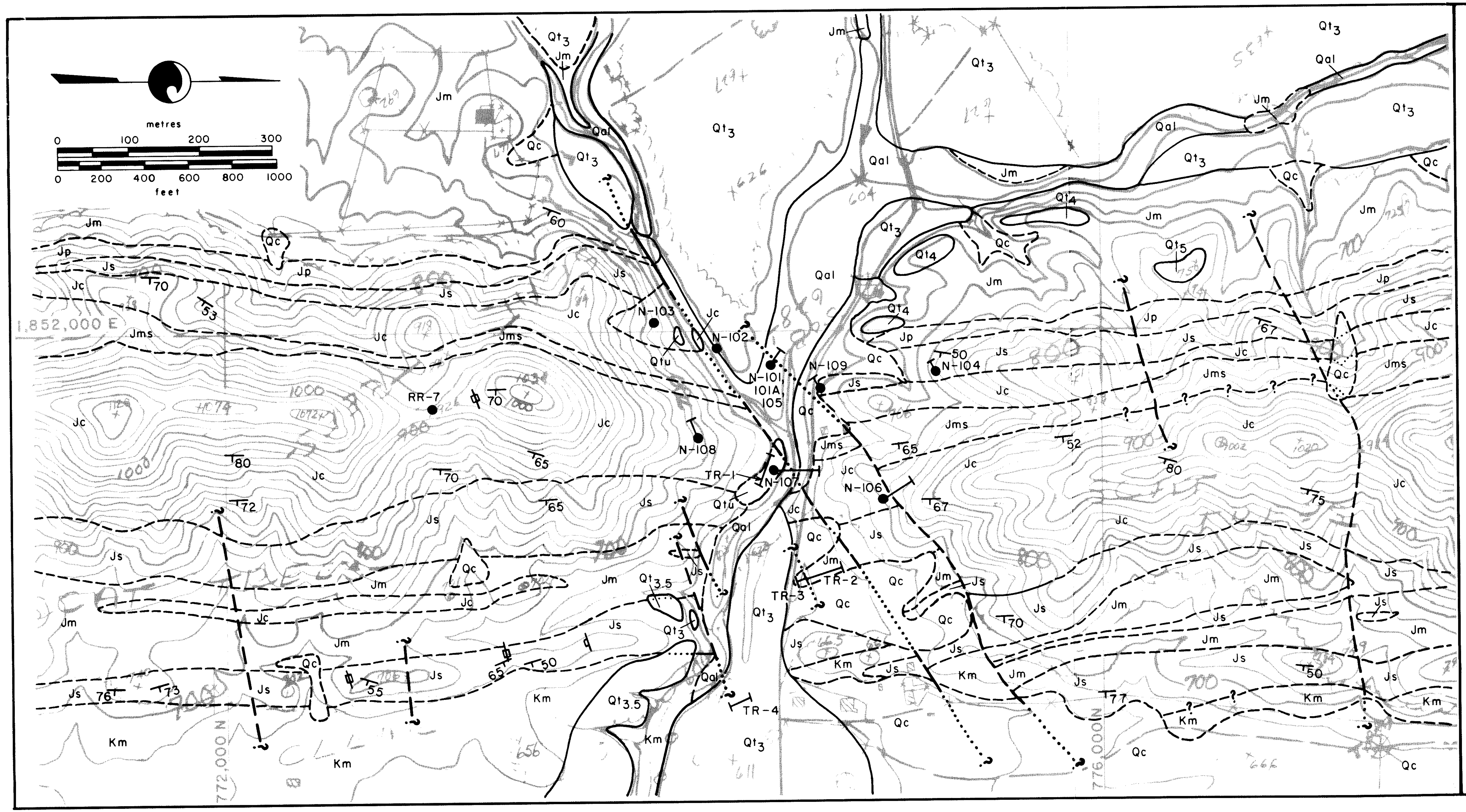

LEGEND

QUATERNARY

Qal	Recent Alluvium – sand and gravel
Qc	Colluvium
Qt_3	Terrace deposits – clayey gravels, number indicates relative age (Qt_u where undifferentiated)

CRETACEOUS – LODOGA FORMATION

Km	Mudstone

JURASSIC/CRETACEOUS – STONY CREEK FORMATION

Jms	Interbedded mudstone and sandstone
Js	Sandstone with a few interbeds of conglomerate and mudstone
Jc	Conglomerate with a few beds and lenses of sandstone
Jm	Mudstone
Jp	Pebbly mudstone grading to conglomerate

SYMBOLS

Geologic contact dashed where inferred or projected

50 Strike and dip of bedding

N-101 Diamond drill hole showing projection, if any, on surface.

TR-2 Location of exploratory backhoe trench

40 Strike and dip of joint

---? Fault; dashed where approximately located, dotted where covered by younger material, queried where uncertain.

STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES
NORTHERN DISTRICT

AREAL GEOLOGY
NEWVILLE DAM SITE
1980

APPENDIX C

ROCKY RIDGE GEOLOGY

APPENDIX C. ROCKY RIDGE GEOLOGY

Rocky Ridge is a thin north-south trending ridge that would form the eastern rim of a Newville Reservoir (or the Newville compartment of a Glenn Reservoir). This appendix assesses the stability and leakage potential of Rocky Ridge as an element of a Newville Reservoir development.

Previous Studies

There were references to dam proposals for reservoirs west of Rocky Ridge in various reports before 1959, but lower reservoirs were under consideration up to that time and no attention was given to possible difficulties concerning Rocky Ridge. Serious studies of the ridge began in 1960, when W. A. Brown, Supervisor of the Department's office of Dam Safety, toured the proposed Glenn Reservoir area and questioned the ridge's competence in a memorandum of May 10, 1960. The Department then retained a consulting board consisting of John S. Cotton, consulting engineer, and Roger Rhoades, consulting geologist, to examine the feasibility of a high Newville Reservoir. This examination included stability and leakage potential of the ridge. On May 27, 1960, the consultants reported that by surface examination, Rocky Ridge was stable, or could be made stable without extraordinary measures for a reservoir not exceeding 305 m (1,000 ft) (Cotton and Rhoades, 1960).

After the 1960 report, the Department conducted a geologic mapping and drilling program on Rocky Ridge. Results of this study were reported in a memorandum report (DWR, 1961). The information was restated in the final report of the North Coastal Area Investigation (DWR, 1965).

In the summer and fall of 1965, the Bureau of Reclamation mapped the geology and drilled core holes at the Newville Dam site. Detailed calculations on seepage potential (considering rock permeabilities, fractures, and fault leakage) were made for Rocky Ridge (USBR, 1967).

In studies conducted in 1978, the Department's Division of Design and Construction made limited stability calculations for the thinnest and steepest portion of the ridge. Recognizing the many uncertainties involved in the analysis of natural slopes, the Division concluded that it would be prudent to limit Newville Reservoir to elevation 297 m (975 ft) (DWR, 1979).

In early 1979, the Department again addressed questions of the stability and leakage potential of Rocky Ridge. Realizing that the exploration since 1960 had never been reviewed in its entirety by a consultant, the Department retained Dr. Douglas D. Campbell. In May 1979, Dr. Campbell reviewed and evaluated the exploration and submitted a formal report (Dolmage Campbell and Associates, 1979). He envisioned no significant problems in stability or leakage by increasing the maximum water surface elevation of the reservoir to 305 m (1,000 ft), from Design and Construction's limit of 297 m (975 ft).

Dr. Campbell recommended further detailed geological mapping of specific sites on Rocky Ridge, additional core drilling and water pressure tests. All of these were completed in the summer of 1979. Dr. Campbell reviewed the results and submitted a final report (Dolmage Campbell and Associates, 1980).

In the fall of 1979, Earth Sciences Associates (ESA) was contracted to conduct a seismic and fault activity study of the reservoir. Their work revised the location and orientation of faults on the geologic map of Rocky Ridge.

Location and Topography

Rocky Ridge trends approximately north-south for about 16 km (10 mi) between Sec. 16, T23N, R6W and Sec. 4, T21N, R6W. It is beyond the western edge of the Sacramento Valley in Glenn and Tehama Counties, about 50 km (30 mi) southwest of Red Bluff. Access is provided by paved roads west from Corning, Orland, and Willows.

The Department's Glenn Reservoir topographic mapping with a scale of 1:4,800 and contour interval of 6.1 m (20 ft) covers the entire ridge area. The USGS Newville and Chrome 7.5-minute quadrangles also cover the area at a scale of 1:24,000 and a contour interval of 12.2 m (40 ft).

The profile of the ridge is uneven, with peaks separated by saddles, and an elevation range from 183 m (600 ft) in the stream channel at Newville Dam site to 402 m (1,320 ft) at the highest peak. The crest of the ridge is sharp and some of the saddles are narrow. The lowest saddle is Burrows Gap, at elevation 259 m (850 ft); it is located 4.7 km (2.9 mi) south of Newville Dam site. Five other Rocky Ridge saddles range between elevations 282 and 300 m (926 and 984 ft) and five more are in the 305- to 309-m (1,002- to 1,014-ft) range. The Rocky Ridge saddles have been assigned letter designations in north-to-south order (A through L), as shown on Plate C-1.

Both sides of the ridge have approximately the same slope. On the upper elevations the slope averages 2½:1, it flattens to 4:1 part way down, and flattens more at the toe.

Geology

Rocky Ridge is a narrow hogback ridge composed of steeply dipping beds of conglomerate and conglomeratic sandstone. These beds form the spine of the ridge. The lower ridge slopes are underlain by sandstone and are flanked by mudstone in the valleys to the east and west. (As used here, mudstone designates clay, silt, claystone, siltstone, shale and argillite. The term is used when the clay, silt, and sand fractions vary from place to place so that more precise terms are not possible.)

The rocks forming Rocky Ridge are part of the Stony Creek Formation. The age is late Jurassic to early Cretaceous (ESA, 1980). A regional geological description of the area that includes Rocky Ridge can be found in Appendix A of this report.

Lithology

Conglomerate, sandstone and mudstone are the three principal rock types that form Rocky Ridge (see Plate C-1). Conglomerate beds are nearly continuously exposed along the crest of the ridge and on the upper eastern slope. Some good exposures of sandstone occur on the steep western slope of the ridge in deep gullies. Exposures are sparse on north and lower east slopes, where heavy brush, deep soil cover or slopewash, and conglomerate float cover underlying rock.

Conglomerate. The core of the ridge is composed of conglomerate that ranges in thickness from 90 to 150 m (300 to 500 ft). It includes lenses of sandstone and conglomeratic sandstone as well as intercalated, discontinuous beds of mudstone. Individual conglomerate beds differ considerably in physical characteristics. The most common variety is a poorly sorted, well indurated pebble conglomerate with well rounded, vari-colored pebbles of chert, volcanics, greenstone and other minor rock types. A few beds are poorly indurated and deeply weathered. The average pebble size is in the range of 25 mm (1 in), grading down to grit and locally up to cobbles 300 mm (12 in) in size. The cement consists of argillaceous material and, in places, small amounts of calcium carbonate. The rock is generally massive, dense, hard, and relatively impermeable where unweathered.

Weathering proceeds initially along bedding and fracture planes, then diffuses through the rock, decomposing the cement, and reducing the rock to a mass of hard gravel in a loose matrix (Dolmage Campbell and Associates, 1979). The conglomerate is predominantly gray where fresh and brown where weathered.

Wide exposures of conglomerate hundreds of metres long are common along the ridge. The exposures are fractured and jointed, both normal and parallel to the bedding. Water pressure tests and core examination indicate that the conglomerate is only slightly fractured beneath the weathered zone. Many old fractures have been cemented with calcite, but some open fractures in fresh rock are noted, as shown by iron-oxide staining on the fractures, due to water percolation.

The conglomerate is the most resistant rock type in the ridge, but it is usually not as strong as the sandstone (Table C-1). Tensile splitting tests were plotted against corresponding unconfined compressive strength, and average shear strength was found to be 10 000 kPa (1,450 lb/in^2) (DWR, 1961).

Sandstone. Sandstone occurs throughout the ridge, generally as lenses within the conglomeratic sequence. The beds range up to 6 to 9 m (20 to 30 ft) thick, and the bedding is usually very faint or indistinguishable. Bedding planes between the sandstone and other rock types in the core are commonly tight and impervious. Fracturing in the sandstone is similar to that in the conglomerate, along with general lithology and physical characteristics.

The sandstone varies from predominately fine-grained to predominately coarse-grained. Unconfined compression test results cannot be correlated to grain size variability (Table C-1). Tensile splitting tests were plotted against corresponding unconfined compressive strength, and the average shear strength was found to be 10 600 kPa (1,540 lb/in²), with a lowest shear strength of 5 900 kPa (850 lb/in²) (DWR, 1961).

TABLE C-1

SUMMARY OF ROCK STRENGTHS FOR ROCKY RIDGE

Rock Type	1960 1/ Unconfined Compressive Strength kPa (lb/in²) Range	Average	1979 2/ Average Unconfined Compressive Strength kPa (lb/in²) Dry	Saturated
Fine-grained sandstone	65 600 - 125 000 (9,500 - 18,200)	90 300 (13,100)	136 000 (19,700)	48 500 (7,030)
Medium-grained sandstone	-	-	207 000 (30,000)	57 600 (8,350)
Coarse-grained sandstone	39 300 - 88 900 (5,700 - 12,900)	68 600 (9,950)	184 000 (26,700)	89 600 (13,000)
Fine-grained conglomerate	47 600 - 119 000 (6,900 - 17,300)	68 900 (10,000)	115 000 (16,700)	47 800 (6,940)
Medium-grained conglomerate	31 000 - 97 200 (4,500 - 14,100	56 200 (8,150)	-	-
Coarse-grained conglomerate	35 200 - 98 600 (5,100 - 14,300)	66 900 (9,700)	117 000 (17,000)	35 600 (5,160)

1/ Department of Water Resources, 1961.
2/ Department of Water Resources, 1980.

Mudstone. The mudstone usually occurs on the outer flanks of the ridge on both sides of the conglomerate and sandstone units. In some places it occurs near the ridge crest because of fault displacement.

The mudstone is generally a soft, slightly fissile, moderately lithified silty shale. It slakes like normal shale upon exposure (wetting and drying). The depth of slaking is about 0.3 m (1 ft), beneath which the rock is reasonably compact. The tendency to slake is less as the smount of sand in the mudstone increases. In some places the mudstone alternates with thin beds of very fine-grained sandstone.

The strength of the mudstone was not determined because of its slaking tendency (DWR, 1961), but the average unconfined compressive strength for similar rock type is estimated to be between 10 300 and 13 800 kPa (1,500 and 2,000 lb in) (DWR, 1965).

Structure

Strike and Dip. Rocky Ridge is part of an eastward dipping homocline in which the beds dip uniformly at about 60° to 80° (DWR, 1965). The ridge trends nearly north-south, but ranges from N 10° to 20° W. Local abrupt topographic and lithologic discontinuities are commonly the result of numerous cross faults which cut the ridge. Some of the irregularities are also due to minor folds or flexures associated with or resulting from the faults (ESA, 1980). In addition, lenticular conglomerate and sandstone beds result in anomalous strike and dip orientations.

Faults and Joints. Numerous faults cross the ridge, but not all offset the ridge. The strike of the faults varies from N 50° E to N 80° E. The dips are hard to determine, but are probably steep as the traces of the faults tend to cut straight lines across the topography (DWR, 1961). The faults occur at intervals along the ridge of at least every 3 km (2 mi), with a possible closer spacing of as little as 300 m (1,000 ft). It must be kept in mind that many of the lenticular beds die out near topographic saddles (where continuous rock exposures are often covered with slopewash and colluvium) and apparent truncation by faulting is falsely suggested (ESA, 1980). Apparent right lateral displacements along the faults vary from a few centimetres to at least 340 m (1,100 ft). Normal fault movement may also have occurred.

At Newville Dam site, the rocks are offset about 370 m (1,200 ft) along a series of faults through the gap, although the largest apparent offset on any one fault is about 150 m (500 ft) (USBR, 1967). It appears that the major saddles along the ridge are underlain by faults or groups of faults having the larger displacements and the minor saddles are underlain by faults having lesser displacements. The faults appear to widen and branch irregularly in the mudstone beds. The lateral extent of most of the faults that cut Rocky Ridge cannot be accurately determined because of alluvial and colluvial cover on side slopes; however, most are estimated to be less than 300 m (1,000 ft) in length (ESA, 1980).

The observed fault zones are narrow - less than 3 m (10 ft). Even though broken rock appears adjacent to the fault plane, permeability problems are not likely, due to development of gouge and clay within the fault zones. Some of the fractures are calcite-healed and nearly as hard as the surrounding rock (DWR, 1961).

The faults grade into joints by loss of visible displacement. East-northeast striking joints with near vertical dips cross all rock types along Rocky Ridge. The joints are widely spaced in massive conglomerate beds with spacings up to 6 m (20 ft). Joint spacings in drill core from conglomerate beds usually range between 0.3 and 1.5 m (1 and 5 ft). Joint spacings in sandstone beds average 0.3 to 1 m (1 to 3 ft) and outcrops of mudstone beds show very close checking, as a result of air slaking combined with the jointing. The conglomerate beds also exhibit a series of shallow-dipping joints that strike parallel to the ridge. These dip from zero to 15 degrees east and west, and are not uniformly spaced in outcrop. Joints dipping 15 to 35 degrees are spaced 0.6 to 2 m (2 to 7 ft) in conglomerate and 0.15 to 0.6 m (0.5 to 2 ft) in sandstone. In fresh rock, the joints are commonly healed with calcite.

Potential Problems

The suitability of Rocky Ridge as a reservoir rim is dependent on: (1) stability of the ridge; (2) closure of topographic saddles; (3) prevention of excessive leakage; and (4) protection of weaker portions of the ridge against wave action.

Stability

Rocky Ridge appears to be "thin" because it stands conspicuously above a flat base. Consequently, concern has been expressed as to its stability as a natural water-impounding structure.

There are three basic possibilities to consider in analyzing conditions that could cause failure: (1) assuming that the entire ridge is impervious, do all portions of it have sufficient stability to resist failure by sliding; (2) if the ridge is not impervious, does a permeable layer or lens occur in the ridge along which hydrostatic pressure could develop and cause a slide or blowout on the downstream side of the ridge; and (3) are fractures present along which hydrostatic pressure could develop and cause failure?

Failure due to piping is unlikely since most of the beds that form the ridge are well indurated. The uncommon poorly indurated beds that do occur in the ridge are sandwiched between well indurated beds.

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Assuming that there are no pervious beds in the ridge, then a section crossing the entire ridge would have to slide in order for failure to occur. Topographic cross-sections show that even the thinnest saddles in the ridge have sufficient cross-section to resist failure by sliding.

If the entire ridge is not impervious and hydrostatic pressure could develop in some pervious, confined beds in the ridge, failure could occur if the rock at that point had insufficient strength and mass to resist the hydrostatic pressure. The core from the Department's drill holes (RR-1 through RR-16) and the geologic mapping indicate that all of the unweathered rock intersected, including the mudstone, is compact, massive, and sparsely fractured. In addition, the sandstone and conglomerate are highly competent. The weathered rock that was intersected comprised conglomerate and sandstone and generally did not core well, if at all; however, water testing indicated that the weathered material had low permeability and was compact.

For unsafe hydrostatic pressure to develop, the fractures would have to: (1) strike parallel to the ridge so the fracture would not surface to allow a release of pressure; and (2) be watertight in order for the maximum hydrostatic head to develop. Fractures that are parallel to the ridge are intersected by transverse fractures, allowing the drainage necessary for the prevention of a high hydrostatic pressure.

The slopes on the west side of the ridge in the vicinity of Newville Dam site show only minor evidence of surficial instability. The slopes on the east side are stable. Cut slopes parallel to the strike of the beds would be stable at 2:1 in weathered rock and at 1:1 with berms in lightly weathered to fresh rock. Near vertical cut slopes would be stable in fresh and weathered rock if cut normal to strike of bedding and parallel to strike of east-northeast trending joints.

Original designs for the proposed reservoir permitted a maximum water surface elevation of 297 m (975 ft). Questions were raised concerning any additional problems that would occur if that water level was increased to 305 m (1,000 ft). No bedrock stability problems of any consequence are foreseen (Dolmage Campbell and Associates, 1979).

Topographic Saddles

Five saddle dams would be required in Rocky Ridge for a reservoir surface elevation of 297 m (975 ft). (A sixth saddle would be used as a spillway site). The saddle dams would range in height from 3 to 44 m (11 to 145 ft).* The most significant effect of raising the reservoir level to 305 m (1,000 ft) would be the addition of five more saddle dams.

The topography at the ridge crest varies from saddle to saddle. Some saddles are best suited for embankment-type dams, while others are better suited for concrete gravity structures. The bulldozing of drill platforms and access roads, together with the results of 1979 drilling, indicate that the weathered conglomerate and sandstone are compact and possess sufficient bearing strength to act as suitable foundations for either embankment-type or concrete dams. Necessary excavation to a suitable foundation could be done by bulldozing and ripping to depths of 3 m (10 ft) or less (Dolmage Campbell and Associates, 1980). Because of fracturing and weathering, considerable foundation grouting could be required.

Construction material sources for the saddle dams would be the same as those proposed for Newville Dam. Availability of construction materials within a short distance of construction sites appears likely (see Appendix D).

Leakage

Examination of the bedrock and water test data reveal a consistent low permeability of all pertinent rock types. However, the presence of joints and brecciated intervals adjacent to faults raises the prospect

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*The saddle dam crests would be 6 m (20 ft) higher than the reservoir elevation.

of appreciable fracture permeability across the ridge. Average permeabilities are highest in intensely weathered conglomerate within 6 m (20 ft) of the ground surface and least in fresh, interbedded mudstone and sandstone below about 24 m (80 ft) of depth. Permeabilities appear to be related chiefly to the degree of weathering to about 18 m (60 ft), and to intensity of fracturing below 18 m (60 ft).

The core from the drill holes shows many old fractures tightly cemented with calcite and a few open fractures. The open fractures are easily recognized as they are usually stained with iron-oxide. Minor seeps and springs occur along the ridge, suggesting that some fractures transmit water. Many of these seeps occur near faults. All evidence indicates that the fractures, caused by fault movement, are not continuous through the fault zone (they are not found in the mudstone beds), so seepage is a local phenomenon.

The most likely place for leakage to occur appears to be in low saddles where the water surface would closely approach the crest of the ridge. In these areas the path of percolation would be short and might not encounter fresh rock. The Bureau of Reclamation estimated that about 4 300 dam^3 (3,500 ac-ft) of water would leak through the ridge annually with a reservoir elevation of 297 m (975 ft) and no foundation grouting. The Bureau further concluded that almost 95 percent of this leakage would occur at Newville Dam and the five saddle dams along the ridge. As the foundations of these structures would be grouted or blanketed to impede leakage as a normal part of construction, only about 310 dam^3 (250 ac-ft) per year of leakage would be expected to occur after normal foundation treatment. If the ridge segment between Saddle H and Burrows Gap (Saddle L) were grouted or blanketed, indicated leakage would be reduced to 60 dam^3 (48 ac-ft) annually.

Raising the water level to 305 m (1,000 ft) would flood a greater amount of more deeply weathered rock; however, this increment is insignificant relative to the remaining width of unweathered rock in the ridge. The increase in water level would probably increase the leakage through weathered rock in one or two restricted sections, such as Saddle Dam A where the rock profile at 305 m (1,000 ft) elevation is narrow and entirely weathered. No difficulties would be anticipated in alleviating this problem, either by grouting or by a modest blanket (Dolmage Campbell and Associates, 1979).

Ridge Protection

The steeper and thinner portions of the ridge would have to be protected from damaging erosion by wave action. The eastward dip of the rocks of the ridge is favorable to resist erosion, but future studies will likely indicate the necessity for protection, probably by riprap, at least in some places. It has been suggested that the riprap could be installed economically by barge as the reservoir filled and could be later maintained in the same way. Only the upper part of the ridge would need protection, so the riprap would need support from below to prevent its loss downhill on steep slopes. This could be accomplished by placing riprap from the base of the ridge to the crest, or starting from a bench cut part way down the slope.

Exploration and Testing

Information regarding Rocky Ridge has been gathered through various stages of study from 1960 to 1980 by the Department, the Bureau of Reclamation, and several private consultants. These studies are discussed in chronological order.

John S. Cotton and Roger Rhoades made a brief surface investigation of the ridge for the Department in 1960. They concluded:

> "... that it is entirely feasible, from the standpoint of stability and leakage, to construct the Glenn Reservoir to an elevation not to exceed 1,000 feet, and that this can be done by construction measures of a standard and precedented kind."

They pointed out that this conclusion was based on very brief study and recommended a program of exploration and testing (Cotton and Rhoades, 1960).

Later in 1960, the Department performed an investigation of the ridge that included geologic mapping, diamond drilling in conjunction with water pressure tests, and rock testing of the core obtained by diamond drilling (DWR, 1961). Eight holes, with a total footage of 332 m (1,080 ft) were drilled in seven different saddles, one a proposed spillway site. This was done because the lithology of the ridge varies in short distances, and portions of the ridge have been shifted by faults, so it was decided that as many potential problem areas as possible should be drilled. Water pressure tests were conducted at irregular intervals in all holes.

The Department used a base map with a scale of 1:4,800 in the geologic mapping of the ridge. Detailed mapping was not considered necessary at this stage of the investigation (individual beds in the ridge were not mapped), and the ridge was divided into units based on the predominant rock type.

The water tests were conducted to estimate rock permeabilities and leakage potentials. Occasionally during drilling, all return water was lost and an iron-oxide stained fracture could be observed in the core at that depth. During water tests, the greatest water loss occurred in the top of the holes where weathering had opened fractures. Some of the water losses in the weathered zone were as high as 130 l/min (35 gal/min) with a pressure of 550 kPa (80 psi) over a zone of 4 m (13 ft).

Some of the fractures took considerable water at the start of a test, but the flow diminished steadily during the test. In some holes, an artificial artesian condition was developed by the water test and most of the water came back into the hole when the packer was released. This indicates that some of the fractures are not connected and would not contribute to leakage through the ridge. In Hole RR-8, artesian water was encountered at about 20 m (65 ft) and a small amount (less than 4 l/min) (1 gal/min) seeped out of the hole after completion.

The Department also performed unconfined compression tests and tensile splitting tests of the rock from Rocky Ridge. The tensile splitting tests were done by laying rock core on its side and loading it to failure in the direction of its diameter. Mohr's circles of tensile and unconfined compressive strengths were then plotted and the shear strength determined.

The factor of safety against sliding of the ridge was computed for several of the thinner portions. The lowest safety values were found in the low saddles near Newville Dam site, where the ridge is thinner down to a lower elevation than at any other place on the ridge. To the north and south of the damsite, the lower portions of the ridge are much wider.

During the summer and fall of 1965, the Bureau of Reclamation mapped the geology at Newville Dam site on a 1:2,400 scale topographic map with a 1.5 m (5 ft) contour interval. The Bureau also drilled ten core holes and twelve bucket auger holes. Ten earth materials samples were tested at their Willows Laboratory.

To estimate potential leakage through Rocky Ridge, analyses were made of 114 water tests made in Bureau drill holes at Newville Dam site and 30 tests made in Department drill holes located along Rocky Ridge. Most Bureau tests were of 3 m (10 ft) sections and were made at pressures ranging from gravity to 690 kPa (100 lb/in^2), while Department tests were usually made at pressures of 275 to 550 kPa (40 to 80 lb/in^2) with considerable variation in the length of the section tested.

Tests were generally made in a cyclic pattern, i.e., 172 kPa (25 lb/in^2) - 345 kPa (50 lb/in^2) - 172 kPa (25 lb/in^2), and were generally maintained at constant pressure for 5 minutes during Bureau tests. Time intervals in Department tests varied from 1 to 10 minutes.

In order to compare test results, percolation test data were converted to permeability by the method outlined in Designation E-18, Earth Manual (revised reprint, 1963). Average permeabilities were calculated for each test section and the data plotted against rock type, degree of weathering and depth.

Potential leakage through Rocky Ridge was then calculated by the slope-area method (USBR, 1967). In order to establish an impervious basement layer, it was assumed that an average permeability of one Meinzer unit (K = 14.9 m/yr - 48.8 ft/yr) represented an essentially impervious condition. Impervious conditions were found to prevail below 37 m (120 ft) in sandstone and conglomerate and below 18 m (60 ft) in mudstone.

A profile was constructed along the crest of the ridge to determine areas with the greatest leakage potential. The profile extended from the vicinity of Saddle A to about 1 800 m (6,000 ft) south of Saddle L (Burrows Gap). This is the portion of the ridge for which reconnaissance mapping is available and along which leakage potentials appear greatest.

At water surface elevation 299 m (981 ft) approximately 60 percent of the ridge extends less than 35 m (120 ft) above the water surface. Of the 60 percent, about 72 percent is conglomerate, 25 percent sandstone, and 3 percent mudstone. Permeabilities weighted to reflect rock type and depth were calculated for each potentially pervious portion of the ridge.

Slopes were determined for cross sections of the ridge at each location and areas were obtained by counting squares between the maximum water surface and the assumed "impervious basement".

In 1978, the Department's Division of Design and Construction appraised the maximum safe level of the Newville Reservoir. A limited study was made, including Swedish slip circle and wedge method analyses. The section selected for study was Saddle F, the thinnest and steepest portion of the ridge. This study concluded that it would be prudent to limit the water surface elevation to 297 m (975 ft) until uncertainties concerning the strength and dynamic response of the ridge materials were resolved (DWR, 1979).

The most recent Department studies were suggested by Dr. Douglas Campbell, who was retained to advise on evaluating Rocky Ridge. Dr. Campbell completed a reconnaissance investigation and reviewed all previously accumulated data. Following recommendations of his May 1979 report, the Department drilled eight vertical diamond drill core holes (RR-9 through RR-16, see Plates C-2 and C-3), each approximately 61 m (200 ft) in depth. Five holes were in Saddle A and three in Saddle F. These two saddles were chosen because Saddle A is strongly faulted and Saddle F is unfaulted. All of these holes were water pressure tested and petrographic analyses were made of some core samples.

The water testing was performed using a packer to seal off the lower 3 m (11 ft) of the hole while water was pumped in at a maximum pressure equal to 23 kPa/m (1 lb/in^2/ft) of depth from the ground surface to the packer. The tests were made in a cyclic pattern, starting with one-half the maximum pressure, then going to maximum pressure and then back to one-half maximum. Each pressure test lasted for 5 minutes. The holding test was then performed by sealing the hole and measuring the shut-in pressure for 1 minute at 15-second intervals. However, the rock was generally of such poor quality in the first 6 m (20 ft) that water tests were done using gravity or by pressuring against the seal that the casing made with the formation.

Conclusions and Recommendations

The conclusions and recommendations summarized below are a result of all data that has been collected and reviewed, up to the time of this report. Most of these have been adopted from Dr. Campbell's reports (Dolmage Campbell and Associates, 1979, 1980).

1. The unweathered interbedded massive conglomerate and sandstone that comprise the central spine of Rocky Ridge are compact, massive, sparsely fractured and highly competent. The weathered rock is considered compact and of low permeability. The sandstone beds are more massive, competent and unfractured than previously anticipated. Because the mudstone beds lie well out on the lower slopes of the ridge, and because of their steep dip, they do not comprise a significant factor in the stability of the ridge.

2. All the study results confirm that the ridge itself is highly competent and stable, with no probability of failure by rupture, slides, or other deterioration.

3. The 1979 water pressure tests indicate that the mass of the unweathered ridge is essentially impermeable, with permeability generally less than 40 $l/day/m^2$ (1 $gal/day/ft^2$), principally because it is so sparsely fractured. The tests also indicate that the weathered layer of bedrock, which is consistently about 8 to 15 m (25 to 50 ft) in depth in Saddle A and 15 m (50 ft) in Saddle F, is not significantly more permeable than the underlying unweathered rock. Weathered rock permeabilities range from 120 to 160 $l/day/m^2$ (3 to 4 $gal/day/ft^2$). Local fractured zones in the unweathered rock show permeabilities up to about 240 $l/d/m^2$ (6 $gal/day/ft^2$) over lengths of 3 m (10 ft), but these zones are uncommon and, in any case, such permeabilities are very low and readily groutable.

   It is of interest that the lowest 3 to 6 m (10 to 20 ft) of the weathered layer is everywhere somewhat more permeable than either the underlying unweathered rock or the overlying weathered rock. Permeabilities in this layer range from 400 to 1 200 $l/day/m^2$ (10 to 30 $gal/day/ft^2$). This zone or layer would have to be grouted or blanketed to ensure a positive cutoff beneath saddle dams.

   Water testing of drill hole RR-12, which intersected the Saddle A fault, or a major branch of it, indicated that the permeability in the fault zone is essentially no greater than the surrounding rock. The permeability of the fault zone was less than 40 $l/day/m^2$ ($1/gal/day/ft^2$), at a depth of 33 to 38 m (109 to 125 ft) below surface. This suggests that the fault zones would probably act as vertical impermeable barriers rather than as water courses.

   The exploratory water testing and coring done in 1979 shows that Rocky Ridge is for all practical purposes an impermeable barrier whose natural leakage would be of little consequence. Minor local seeps and springs on the downstream side of the ridge should be expected, but probably only at higher elevation where the rock is weathered.

4. Prior to the 1979 drilling program, it was assumed that the entire layer of weathered rock, to depths of 9 m (30 ft) or more, would comprise a permeable zone that would need to be rendered watertight. The most practical solution then appeared to be upstream blanketing. It is now evident that both the weathered rock and the faults are generally of very low permeability and that any relatively permeable zones within them can be readily made impermeable by conventional cement grouting. For this reason, it is now recommended that the saddle dams be designed with single line grout curtains to depths of at least 15 m (50 ft), which is the depth of the deepest permeable zone at the bottom of the weathered layer. This single line curtain should be flanked by

two rows of shallow holes 5 to 6 m (15 to 20 ft) in depth. The drilling of the grout curtain holes should be exploratory, leaving provision for deepening or angling holes where advisable, particularly in the area of faults.

There appears no need now to consider upstream blankets. The compactness and general low permeability of the weathered rock does not warrant blanketing or consideration of slurry trenches to effect a cutoff.

5. It is recommended that consideration be give to designing some of the smaller saddle dams as concrete gravity structures instead of earthfill embankments. This would eliminate the problem of the long thin upstream and downstream "skirts" that would result from fill dams on the narrow saddles.

6. There is no indication that raising the proposed maximum reservoir elevation to 305 m (1,000 ft) in contrast to the earlier proposed 297 m (975 ft), would significantly decrease the stability of the ridge. It would, however, require marginal increases in the sizes of the saddle dams.

7. Where the freeboard with the ridge top is less than about 9 m (30 ft), the crest and upper part of the ridge might need protection from wave action. Riprap appears to be the best method of protection.

In Dr. Campbell's opinion, there is enough known about the physical and geological characteristics of Rocky Ridge now to proceed to the design stage of the project. He believes there is no need for further feasibility exploration or testing; however, considerable site-specific investigation should be done before or during construction, particularly in grouting and foundation treatment. It would, therefore, be worthwhile to locate and trace the faults through the saddle sites by means of backhoe trenches.

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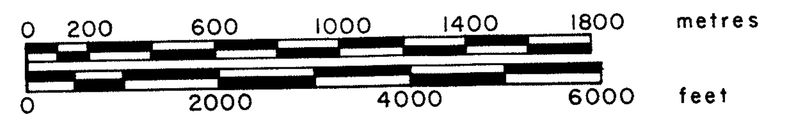

LEGEND

– – –	Geologic Contact — approximate, inferred, or projected
	Diamond Core Holes
•RR-6	individual core hole
● RR-13→15	group of core holes e.g. RR-13, 14 and 15
╱	Fault — solid where there is general agreement among investigators, dashed where D.W.R., W.P.R.S and E.S.A. do not agree, dotted where concealed.
"A"	Saddle Designation

Q	Quaternary — recent, undifferentiated, terrigenous sediments
	Jurassic — Cretaceous Stony Creek Formation
M	mudstone, chiefly silty claystone
MS	interbedded mudstone and sandstone
P	pebbly mudstone grading to conglomerate
C	conglomerate with occasional beds and lenses of sandstone
S	sandstone with occasional interbeds of conglomerate and mudstone

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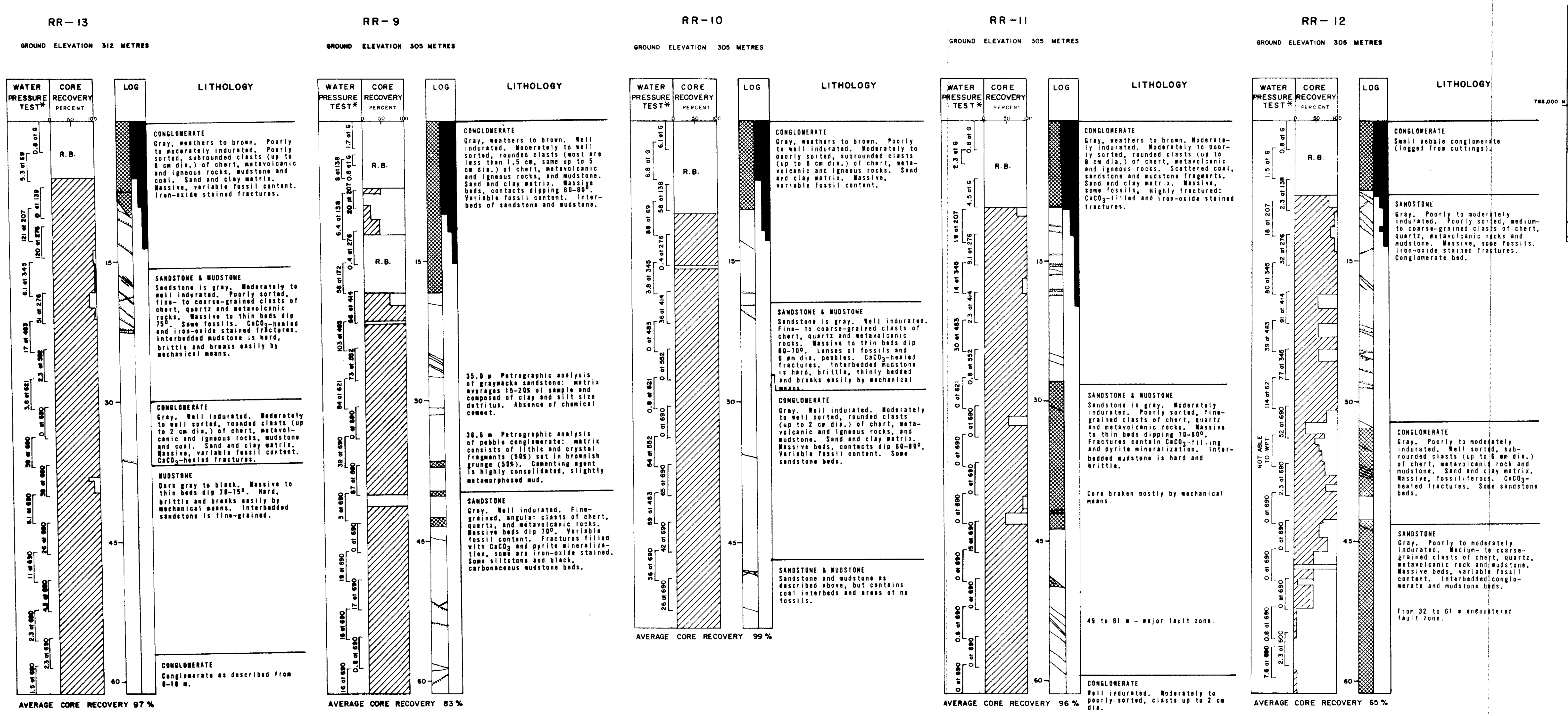
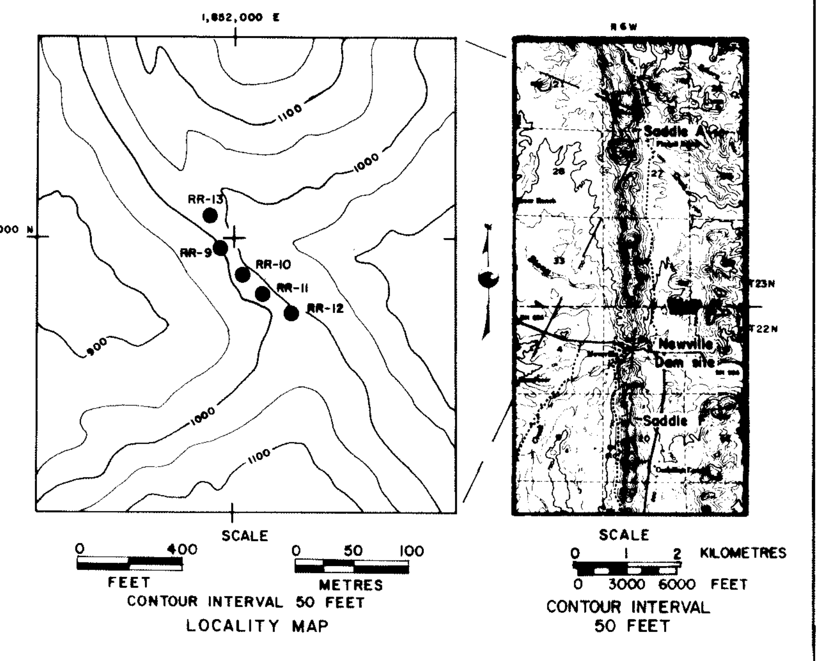
NORTHERN DISTRICT

LITHOLOGY AND STRUCTURE
ROCKY RIDGE
GLENN RESERVOIR PLAN
1980



* NOTE: Water pressure tests were measured in gpm at specified psi and converted to litres/minute at kilopascals. Example: 30 at 483 means 30 l/min at 483 kPa. G equals gravity.



EXPLANATION OF SYMBOLS

- NX Core Recovery
- RB — Rock Bitted Interval
- **Weathering, intense.**
- **Weathering, moderate.**
- **Weathering, slight.**
- Intensely Fractured Zone (Includes zones of broken rock) Recovered core could not be oriented.
- Fractures — symbols spacially oriented, — iron-oxide staining, — pyrite mineralization.
- ● Vertical NX Core Hole

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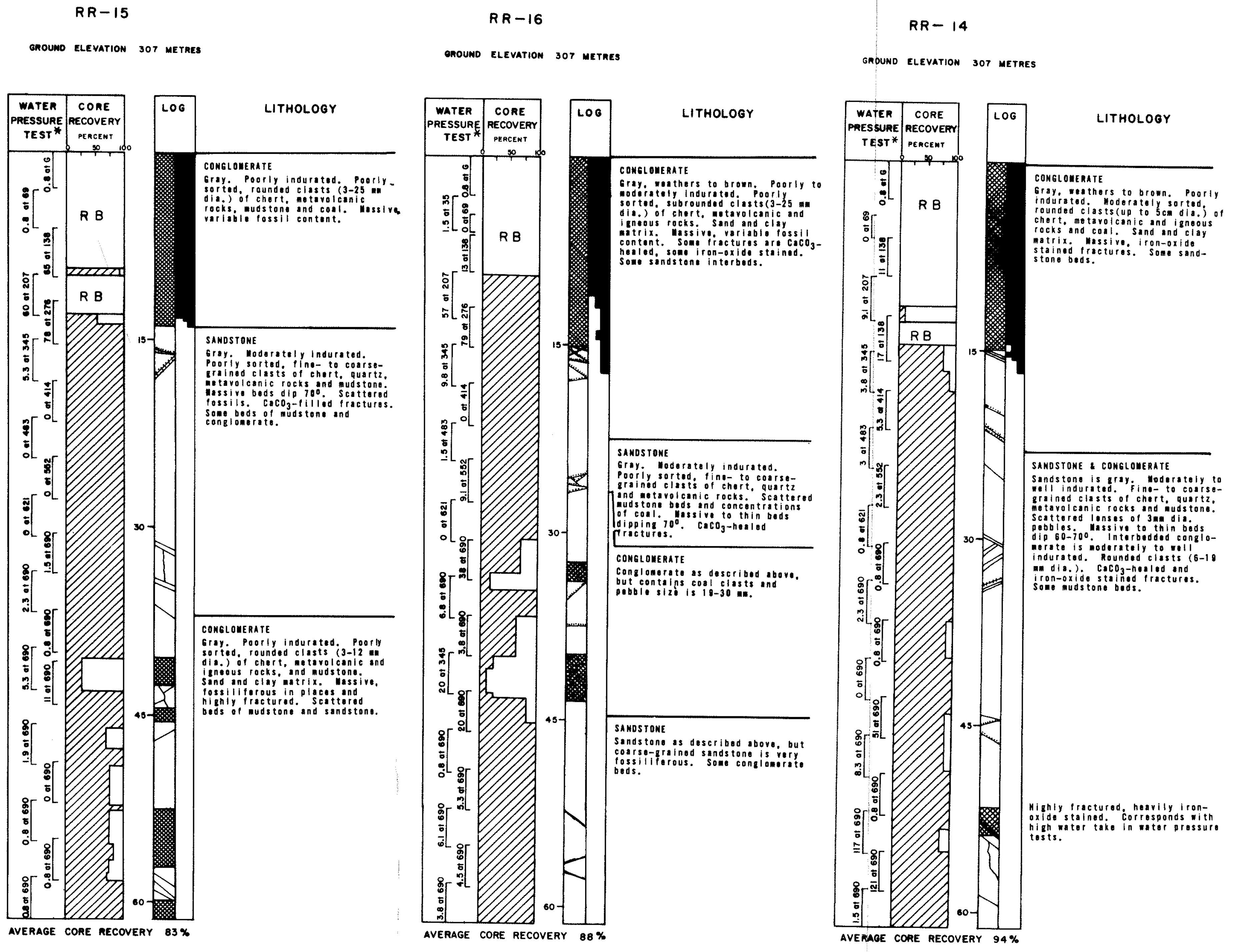
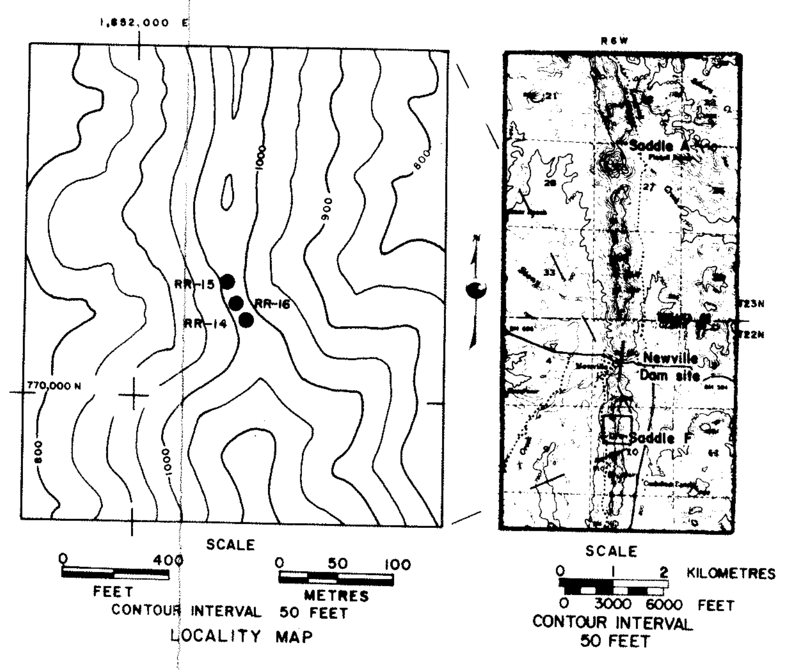
LOGS OF CORE HOLES ROCKY RIDGE SADDLE A GLEN RESERVOIR PLAN 1980





EXPLANATION OF SYMBOLS

- NX Core Recovery
- RB Rock Bitted Interval
- Weathering, intense.
- Weathering, moderate.
- Weathering, slight.
- Intensely Fractured Zone (Includes zones of broken rock) Recovered core could not be oriented.
- Fractures — symbols spacially oriented, — iron-oxide staining, — pyrite mineralization.
- Vertical NX Core Hole

* NOTE: Water pressure tests were measured in gpm at specified psi and converted to litres/minute at kilopascals. Example: 30 at 483 means 30 l/min at 483 kPa. G equals gravity.

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LOGS OF CORE HOLES ROCKY RIDGE SADDLE F GLEN RESERVOIR PLAN 1980

APPENDIX D

CONSTRUCTION MATERIALS

APPENDIX D. CONSTRUCTION MATERIALS

This appendix summarizes the investigations of construction materials for the Thomes-Newville and Glenn Reservoir Plans that have been conducted over the years by the Department, the Corps of Engineers, and the Bureau of Reclamation (now Water and Power Resources Service). It specifically discusses the availability of impervious embankment materials, pervious material and concrete aggregate, and quarry rock sites.

For a more detailed discussion of the borrow areas, refer to the reports listed under References at the end of this appendix. Location maps of the auger holes and dragline pits, geologic logs of the auger holes, laboratory test results, and electric resistivity and refraction seismic exploration results are on file at the Department's Northern District office. Borrow areas identified in this report are shown in Plate D-1.

Impervious Embankment Material

Two sources of impervious materials are available. Stream terrace deposits, which contain an estimated volume of 57.6 million m^3 (75.3 million yd^3), are within the Newville, Tehenn, Rancheria, and Millsite reservoir areas. The Tehama Formation, an almost unlimited source of impervious material, is located outside the prospective reservoir areas (see Appendix A).

All exploration of impervious deposits, except deposits N-3 and N-4, was by flight auger. Much of the exploratory drilling did not reach the bottom of the deposit when the maximum depth of the flight auger was reached at 7.6 m (25 ft). Difficulties encountered by the drilling included hard clay lenses, large rocks, or caving conditions. The Bureau explored the N-3 and N-4 deposits by bucket auger, drilling 12 holes to a maximum depth of 9.8 m (32 ft) (USBR, 1967).

The terrace and Tehama Formation borrow areas are grass-covered with a few scattered oak trees. Development of borrow areas at these sites would require stripping an average 0.6 m (2 ft) of surface material. Tree removal would require locally deeper excavation.

Terrace Deposits

Bulletin 136, Appendix E, discussed 34 terrace and slope-wash deposits. The Bureau modified the boundaries of these areas and explored an additional terrace, N-4 (USBR, 1967). Table D-1 summarizes the exploration, testing, and volume estimates of the deposits. Preliminary test results indicate that the terrace materials are suitable for use as impervious fill.

Based on morphological and pedological analyses, Steele (1979) developed methods for identifying different terrace levels and determining

TABLE D-1

SUMMARY OF TERRACE AND SLOPEWASH DEPOSITS

Borrow Area	1/ Terrace	Volume (million m^3)	2/ Exploration	Lab Tests 3/ L.L.	P.I.	Specific Gravity	% < 200 Sieve	Secondary Tests	Depth (m) to Bedrock 3/	Depth (m) to Ground Water 3/
TA-1	T-1	1.76	9 (fa)	-	-	-	-	No	1.8/+7.6	2.1/3.0
	T-2	0.11	-	-	-	-	-	No	-	-
	T-3	0.44	3 (fa)	-	-	-		No	1.8/ 4.6	Not Encountered
	T-4	0.14	1 (fa)	-	-	-	-	No	2.3	Not Encountered
	T-5	0.23	2 (fa)	-	-	-	-	No	1.8/ 2.4	Not Encountered
	T-6	0.18	3 (fa)	-	-	-	-	No	1.5/ 2.1	Not Encountered
	T-7	0.10	1 (fa)	-	-	-	-	No	1.8	Not Encountered
	T-8	0.37	4 (fa)	-	-	-	-	No	2.1/ 2.3	Not Encountered
	T-9	0.70	4 (fa)	-	-	-	-	No	2.3/ 4.0	Not Encountered
	T-10	0.67	--	-	-	-	-	No	-	-
		4.70	27 (fa)							
TA-2	T-11	0.99	6 (fa)	43	22	2.83	50	No	1.7/ 2.7	2.3
	T-12	1.83	2 (fa)	35	17	2.78	20/80	Yes	2.6/+7.0	5.2
	T-13	4.59	3 (fa)	31/43	17/28	2.72/2.74	51/78	Yes	+7.6	Not Encountered
	T-14	2.29	3 (fa)	31/39	16/25	2.73/2.77	39/75	Yes	+7.6	Not Encountered
	T-15	4.74	10 (fa)	41	23	2.79	23	No	0.9/+7.6	Not Encountered
	T-16	0.37	3 (fa)	-	-	2.68	34/84	No	2.7/ 4.3	1.2/4.0
		14.81	27 (fa)							
TA-3	T-17	0.08	1 (fa)	21	6	2.75	33	No	2.1	Not Encountered
	T-18	0.20	4 (fa)	25	9	2.74	22/57	No	2.1/ 5.3	Not Encountered
	T-19	0.11	4 (fa)	37	21	2.76	58	No	+5.2	Not Encountered
		0.39	9 (fa)							
TA-4	T-20a	0.69	--	-	-	-	-	No	-	-
	T-20b	0.84	2 (fa)	29	12	2.78	46	No	5.8/ 6.7	Not Encountered
	T-21a	0.92	--	-	-	-	-	No	-	-
	T-21b	0.27	--	-	-	-	-	No	-	-
	T-22	0.31	--	-	-	-	-	No	-	-
	T-23	6.42	19 (fa) 7 (s)	31/42	12/23	2.72/2.78	27/37	Yes	1.8/+7.6	5.2/70
		9.45	21 (fa) 7 (s)							

1/ See Plate D-1 for locations.
2/ Flight auger (fa), bucket auger (ba), seismic lines (s).
3/ More than one value expressed as: minimum value/maximum value.

TABLE D-1 (Continued)

SUMMARY OF TERRACE AND SLOPEWASH DEPOSITS

Borrow Area	Terrace 1/	Volume (million m^3)	Exploration 2/	Lab Tests 3/ L.L.	Lab Tests 3/ P.I.	Lab Tests 3/ Specific Gravity	Lab Tests 3/ % < 200 Sieve	Secondary Tests	Depth (m) to Bedrock 3/	Depth (m) to Ground Water 3/
TA-5	T-24	3.21	--	-	-	-	-	No	-	-
	T-25	0.84	--	-	-	-	-	No	-	-
	T-26	1.30	1 (fa)	-	-	-	-	No	+4.6	Not Encountered
	T-27	1.53	1 (fa)	-	-	-	-	No	6.6	Not Encountered
	T-28	1.22	3 (fa)	-	-	-	-	No	0.9/ 4.4	Not Encountered
		8.10	5 (fa)							
TA-6	T-29	0.99	--	-	-	-	-	No	-	-
	T-30	1.07	--	-	-	-	-	No	-	-
	T-31	1.07	--	-	-	-	-	No	-	-
	T-32	1.53	--	-	-	-	-	No	-	-
	N-4	8.60	4 (ba)	30/42	9/21	2.70/2.75	14/88	Yes	1.2/ 8.2	5.5/6.1
		13.26	4 (ba)							
TA-7	T-33	1.38	3(fa)	43/49	13/30	2.74/2.78	42/83	No	6.1/+7.6	Not Encountered
	T-34 (N-3)	5.47	3(fa) + 8(ba)	30/44	10/28	2.70/2.78	14/88	Yes	2.1/ 9.4	4.0/7.3
		6.85	6(fa) + 8(ba)							

1/ See Plate D-1 for locations.
2/ Flight auger (fa), bucket auger (ba), seismic lines (s).
3/ More than one value expressed as: minimum value/maximum value.

their ages. Earth Sciences Associates (1980) summarized the studies of terraces in the reservoir areas, and classified the terraces into six distinctive levels (Qt1 through Qt6). The most common terraces in the project area are Arbuckle (Qt3), Perkins (Qt4), and Corning (Qt5). Terraces of a common level may be composed of similar materials, but the few soil tests conducted by the Department and the Bureau are inconclusive in this respect.

The terrace deposits, grouped into seven general borrow areas, are discussed below. The individual deposits are identified and plotted on Plate D-1.

Area TA-1. Terrace deposits T-1 through T-10 are along Stony Creek between the mouth of Grindstone Creek and Millsite Dam site. The average haul distance to Rancheria Dam site is 1.6 km (1 mi). The deposits of sandy, clayey gravel have a total surface area of 174 ha (430 ac), and a volume of 4.7 million m^3 (6.1 million yd^3). Ground water should not pose a problem in the borrow area.

Twenty-seven flight auger holes were bored into the TA-1 terraces; however, laboratory tests were not performed on the materials. Geologic logs of the auger holes describe graded deposits of clay, sand, and gravel with lenses of silty clay to sandy gravel. None of the auger holes encountered the water table, but wet soils were reported in two holes at 2.1 and 3.0 m (7 and 10 ft).

Area TA-2. Terrace deposits T-11 through T-16 are west of Stony Creek, above the mouth of Grindstone Creek and below Stony Gorge Dam. Haul distances average 5.6 km (3.5 mi) to Rancheria Dam site. The surface area of the deposits is 380 ha (950 ac). The deposits contain about 16.3 million m^3 (21.3 million yd^3) of sandy clay and sandy clayey gravel, based on average thickness of about 4 m (14 ft).

Twenty-seven flight auger holes penetrated the terrace deposits of Area TA-2. The thickness of terraces T-12, T-13, T-14, and T-15 exceeded the 7.6-m (25-ft) depth limit of the auger. Thus, these deposits may contain substantially more impervious material than the estimated volume.

In situ moisture contents in terraces T-12, T-13, and T-14 ranged from 7.5 to 14.0 percent. Ground water was encountered at 2.3 m (7.5 ft) in one auger hole in terrace T-11, at 5.2 m (17 ft) in a hole in T-12, and at 1.2 and 4.0 m (4 and 13 ft) in two holes in T-16. Twenty-three auger holes did not encounter ground water, and only scattered local occurrences of ground water are expected in these terraces.

Compaction tests on composite samples from terraces T-12, T-13, and T-14 yielded maximum dry densities of 2 030 kg/m^3 (127 lb/ft^3) and 1 840 kg/m^3 (115 lb/ft^3) at optimum moisture contents of 11 and 16 percent.

Area TA-3. Terrace deposits T-17 through T-19 lie along Grindstone Creek about 4.8 km (3 mi) west of Rancheria Dam site. They cover 18 ha (45 ac) with 2.1 to 5.2 m (7 to 17 ft) of clayey sand and gravelly, clayey sand. The deposits contain 0.4 million m^3 (0.5 million yd^3) of impervious material that overlie bedrock of the Stony Creek Formation. Saturated conditions were not encountered in any of the nine auger holes.

Area TA-4. Six terraces, designated T-20a, T-20b, T21a, T-21b, T-22, and T-23 are near Chrome, midway between Rancheria and Newville Dam sites. These terrace deposits have a total surface area of about 160 ha (400 ac) and contain 9.4 million m^3 (12.3 million yd^3) of material. An average of 5.8 m (19 ft) of gravelly sandy clay overlie mudstone bedrock of the Stony Creek Formation.

Two auger holes penetrated terrace T-20b and 19 penetrated terrace T-23. None of the other terraces was explored. The auger holes ranged from 1.8 m (6 ft) to 7.6 m (25 ft) in depth. Cobbles up to 250 mm (10 in) in diameter were encountered. Six of the holes reached ground water or very wet materials at 5.2 to 7.0 m (17 to 23 ft) below the surface.

Terrace T-20b is composed of sandy gravelly clay 6 to 7 m (19 to 22 ft) thick. The gravel contains serpentinite and schist cobbles and boulders. Ground water was not encountered and the material is described as moist.

Geologic logs of auger holes in terrace T-23 indicate that a clayey, sandy gravel layer about 1 to 3 m (4 to 10 ft) thick overlies a silty clay layer with up to 20 percent pebbles. Schist and serpentinite are the common rock types that make up the gravel. Atterberg limit tests indicate the silty clays have low-to-moderate plasticity.

Consolidation and triaxial shear tests were conducted on undisturbed samples of terrace T-23. The samples contained less than 4 percent gravel and 24 percent sand. Six consolidation tests yielded maximum dry densities ranging from 1 590 to 1 700 kg/m^3 (99 to 106 lb/ft^3) and optimum moisture contents from 21 to 27 percent. A triaxial shear test showed an 18.5-degree angle of shear resistance and 7 600 kg/m^2 (0.78 tons/ft^2) cohesion intercept.

In 1967, the Department conducted a refraction seismic survey along the proposed axis of Chrome Dike. The interpretation of the seismic survey closely agrees with results from the auger drilling. The seismic survey indicates that bedrock is 2.4 to 12 m (8 to 40 ft) below the land surface in part of terrace T-23.

Area TA-5. Terrace deposits T-24 through T-28 are near Heifer Camp Creek, about 1.6 to 6.4 km (1 to 4 mi) south of Newville Dam site in the reservoir area. These deposits contain approximately 8.1 million m^3 (10.6 million yd^3) of material.

The Department bored five auger holes into terraces T-26, T-27, and T-28. Terraces T-24 and T-25 were not explored. Geologic logs of the auger holes describe a variety of materials ranging from clayey silt to mixtures of clay, sand, and gravel. Laboratory tests were not performed. The depth to bedrock varied from 1 to 7 m (3 to 22 ft). Ground water was not encountered and most of the terrace material was dry to slightly moist.

Area TA-6. Terrace deposits T-29 through T-32 and N-4 are 0.2 to 4.8 km (0.1 to 3 mi) north and west of Newville Dam site. T-29 through T-32 have a total volume of 4.7 million m^3 (6.1 million yd^3), and N-4 has 8.6 million m^3 (11.2 million yd^3) of material. They are underlain by the Stony Creek Formation. Terraces T-29 through T-32 have not been explored or tested.

The Bureau drilled four bucket auger holes into terrace N-4 near Newville and found mudstone bedrock at depths from 1 to 9 m (4 to 31 ft). Lean sandy clay, 5 to 6 m (16 to 20 ft) thick, overlies 0.3 to 3 m (1 to 11 ft) of clayey gravel or clayey sand. In situ moisture content varies from 9.6 to 25.9 percent. Three holes found ground water at depths from 5.5 to 6.1 m (18 to 20 ft).

The Bureau tested samples from two auger holes. Maximum dry densities range from 1 710 to 1 730 kg/m^3 (107 to 108 lb/ft^3) and optimum moisture contents from 19.0 to 19.3 percent.

Area TA-7. Terraces T-33 and T-34 (N-3) are along North Fork Stony Creek, an average 1.6 km (1 mi) haul distance below Newville Dam site. The deposits cover 100 ha (246 ac). Fourteen auger holes were drilled in and near the terraces. Ground water was found in four holes at depths from 4.0 to 7.3 m (13 to 24 ft).

Three auger holes penetrated terrace T-33. About 2 to 3 m (7 to 10 ft) of gravelly clay overlie silty clay or gravelly sand layers up to 4.6 m (15 ft) thick. Atterberg limits of the fines were above the A-lines within the moderate plasticity range. Lenses within the terrace contain up to 45 percent sand and 40 percent gravel. The maximum gravel size was 130 mm (5 in) in diameter. Ground water was not encountered.

Eight auger holes penetrated terrace T-34. Most holes found clayey sand or sandy clay layers 2 to 6 m (7 to 20 ft) thick overlying either a gravelly sand layer 0 to 5 m (0 to 17 ft) thick or bedrock. Atterberg limits were above the A-lines in the low-to-moderate plasticity range. A compaction test of sandy clay had 1 790 kg/m^3 (112 lb/ft^3) maximum dry density and 17.7 percent optimum moisture content. In situ moisture content was 12 to 26 percent. Ground water was below 6.6 m (21.5 ft) in two holes and below 4 m (14 ft) in a hole near the borrow area periphery.

Tehama Formation

Four large Tehama Formation borrow areas, TF-1 through TF-4 have been outlined. They are 5 to 13 km (3 to 8 mi) from Newville and Rancheria Dam sites (see Plate D-1). The volume of impervious material in these areas exceeds 490 million m^3 (640 million yd^3).

The Department performed compaction, triaxial shear, and consolidation-permeability tests on composite samples from area TF-1, TF-2, and TF-3. The results of these tests are summarized in Table D-2. The Tehama Formation contains a large volume of high-strength impervious material suitable for dam core fill.

TABLE D-2

SUMMARY OF TEST INFORMATION 1/ ON TEHAMA FORMATION

Test	Lab. Number	No. of Tests	Max. Size of Sample	Moisture Content % (optimum)	Dry Density kg/m³ 2/	Shear Strength φ Angle Effect.	Shear Strength φ Angle Total	Shear Strength C = kg/m² Effect.	Shear Strength C = kg/m² Total	Permeability K (mm/day)	Consolidation (%)	% Fines (-200 Mesh)	LL	PI	Remarks
Triaxial Compression 4/ (CU) 5/	1-2467	1	19 mm	10.9	1 893 3/	26.4°	11.5°	0.0	3 900			27	46	29	)Samples are representative of
	1-2468	1	19 mm	9.8	1 997	31.0°	14.2°	2 150	2 150			29	45	28	)various gradations encountered
	1-2466	1	19 mm	8.9	1 996	35.2°	16.2°	3 710	2 930			22	56	34	)during exploration.
Compaction	1-2463	1	19 mm	10.4	2 018							30	49	28	)Samples are representative of
	1-2464	1	38 mm	8.3	2 058							22	47	27	)various gradations encountered
	1-2465	1	38 mm	8.8	2 026							30	48	30	)during exploration.
Permeability	1-2470	1								6/ 0.076 @ 2 060 kg/m³		20	49	28	)Samples tested represent high
	1-2469	1								0.002 @ 1 990 kg/m³		34	44	28	)and low -200 mesh for range)of values.
Consolidation (Maximum load applied = 39 060 kg/m²)	1-2469										7/ 10.3% @ 39 060 kg/m²	34	44	28	)Samples tested represent high
	1-2470										9.6% @ 39 060 kg/m²	20	49	28	)and low -200 mesh for range)of values.

1/ Secondary testing on terrace and gravel deposits was halted in 1961 when funds were depleted. Additional funds were requested to complete the Tehama Formation testing only. The properties of these materials were thought to be the most important due to the homogeneous character of the proposed Glenn Reservoir dams.

2/ All density values listed here will be affected slightly (increased) by the increase in gravel content in the field samples. The maximum field size observed (see MA diagrams) was 51 mm to 76 mm. Specific gravity of the +No. 4 material = 2.73 - 2.83; of -No. 4 material = 2.73 - 2.76.

3/ This sample indicates lower value expected from any of the finer grained Tehama Formation. Selective borrowing and field mixing should enable keeping this type of material to a minimum in any proposed dam.

4/ Triaxial compression samples chosen as representative as to grading of the various borrow areas except that maximum size would be closer to 51 mm than 19 mm. Not enough +19 mm material was contained in the samples to warrant testing them with maximum size greater than 19 mm. The three samples tested had an average of 10 percent gravel of size 19 mm to 76 mm. This material was scalped prior to sample Triaxial testing.

5/ Consolidated undrained, pore pressures measured.

6/ Permeability tests run concurrent with consolidation tests. Permeability above is at estimated maximum dry density.

7/ Restraining load of 12 200 kg/m² needed to prevent swelling of this sample prior to beginning consolidation testing. Auger log indicates this sample may contain tuffaceous material. No problems in using this material in the fill are anticipated; however, care should be taken to minimize use of tuffaceous zones as compacted fill beneath concrete structures.

Area TF-1. Area TF-1 has a surface area of about 13 km^2 (5 mi^2) east of Millsite Dam site and south of Stony Creek. The deposit contains 151 million m^3 (198 million yd^3) of well-graded clayey, sandy gravel.

The Department drilled 15 flight auger holes in Area TF-1 and ground water was not encountered. The Department Soils Laboratory tested 12 samples of material taken from the auger holes. An average of 19 percent was fines, which pass through the No. 200 sieve. Liquid limits ranged from 53 to 59, and plasticity indexes from 33 to 37. The apparent specific gravity was 2.70 to 2.75 for material passing through the No. 4 sieve.

Area TF-2. TF-2 includes two sites between Burris and Stony Creeks. The deposits contain 108 million m^3 (141 million yd^3) of sandy, clayey gravel. The average sample had 43 percent gravel larger than the No. 4 sieve and 21 percent fines that pass through the No. 200 sieve. Ground water was not encountered.

The deposit was explored with 14 auger holes. Much of the drilling was unsuccessful due to the loosely bound cobble beds. Liquid limits ranged from 41 to 58 and averaged 51. Plasticity indexes ranged from 23 to 39 and averaged 33. The apparent specific gravity of minus No. 4 sieve material ranged from 2.73 to 2.76.

Area TF-3. South of North Fork Stony Creek, Area TF-3 has a surface area of nearly 13 km^2 (5 mi^2) that contains more than 196 million m^3 (257 million yd^3) of graded clayey, sandy gravel. The average sample contained 24 percent fines. Exploration auger holes did not encounter ground water.

The Department Soils Laboratory tested 21 samples from 10 auger holes. Liquid limits ranged from 49 to 53, and plasticity indexes from 28 to 33. Material passing through the No. 4 sieve had apparent specific gravities from 2.72 to 2.76.

Area TF-4. Tehama Formation material covers an area of about 5.2 km^2 (2 mi^2) between North Fork Stony and Kendrick Creeks. The Department estimates the deposit contains 35 million m^3 (46 million yd^3) of sandy, clayey gravel. The material in Area TF-4 has not been explored or tested.

Pervious Material and Concrete Aggregate

Potential pervious and aggregate sources include Recent stream alluvial deposits within the Stony, North Fork Stony, Grindstone, Burris, Salt, and Heifer Camp Creek channels. Eighteen channel deposits, containing an estimated volume of 28.7 million m^3 (37.6 million yd^3), are grouped into five general areas, GA-1 through GA-5.

A flight auger bored holes into most of the channel deposits. Department field personnel sieved samples from 21 auger holes to determine the fractions of the larger-sized gravel. Table D-3 summarizes the results of the mechanical analyses. Sand and fine gravel passing through the 19 mm sieve comprised 64 percent of all the samples.

TABLE D-3

SUMMARY OF MECHANICAL ANALYSES OF STONY AND GRINDSTONE CREEK CHANNEL DEPOSITS

Stream Area 1/	Channel Deposit	Auger Hole	Size Fractions Percent by Weight +76 mm (+3")	76 to 38 mm (3 to 1.5")	38 to 19 mm (1.5 to .75")	-19 mm (-.75")	+19 mm (+.75")
GA-1	G-1	MS-77	6	14	15	65	35
	G-1	MS-73	9	16	20	56	44
	G-1	MS-70	3	9	21	67	33
	G-3	MS-86	9	8	12	72	28
	G-3	MS-87	5	11	22	62	38
	G-5	MS-98	5	17	19	58	42
	G-6	MS-66	5	14	15	65	35
Average Percent			6	13	18	64	36
GA-2	G-7	MS-20	22	18	16	44	56
	G-7	MS-22	7	7	6	80	20
	G-7	MS-25	8	17	18	58	42
	G-8	MS-9	-	-	-	75	25
	G-8	MS-11	-	-	-	38	62
Average Percent			12	14	13	59	41
GA-3	G-9	MS-14	-	19	23	58	42
	G-10	MS-12a	-	22	17	61	39
	G-10	MS-13	-	11	12	76	24
	G-10	MS-15	-	18	22	60	40
	G-11	MS-18	3	5	14	78	22
	G-12	MS-3	-	-	-	58	42
	G-12	MS-4	-	-	-	75	25
	G-12	MS-5	-	-	-	62	38
	G-12	MS-6	-	-	-	74	26
Average Percent			-	15	18	67	33

1/ See Plate D-1 for locations.

Gravel from Stony Creek and its tributaries would probably make suitable concrete aggregate. The Corps tested Stony Creek gravel deposits in the Black Butte Reservoir area, and found the material adequate for concrete aggregate (USCE, 1959).

Area GA-1. The stream channel alluvium in Stony Creek below the mouth of Grindstone Creek includes deposits G-1 through G-6. Plate D-1 shows the location of the deposits. The combined volume estimate for Areas GA-1 and GA-3 was revised (DWR, 1968) from 14.7 to 23.7 million m^3 (19.2 to 31.0 million yd^3) after further exploration of the deposits. Exploration of Area GA-1 included 26 flight auger holes, 15 refraction seismic survey lines, 7 electrical resistivity survey stations, and 3 dragline pits. Laboratory tests included mechanical analyses, specific gravity, and moisture adsorption tests. Samples extracted from below the ground water level in dragline pits in Areas GA-1 and GA-3 contained less than 4 percent fines.

Area GA-2. GA-2 comprises two channel deposits, G-7 and G-8, which are in Grindstone Creek. Exploration included five refraction seismic lines and ten flight auger holes. Average thicknesses ranged from 2.7 to 3.4 m (9 to 11 ft). The ground water level averaged 1.5 m (5 ft) below the surface. The volume of material in Area GA-2 is 2.7 million m^3 (3.5 million yd^3).

Laboratory tests showed the average sample had 7 percent fines passing the No. 200 sieve. The material passing through the No. 4 sieve had an apparent specific gravity of 2.72.

Area GA-3. Deposits G-9 through G-12 are in the channel of Stony Creek between Stony Gorge Dam and the mouth of Grindstone Creek. The revised volume estimate, for Areas GA-1 and GA-3 combined, is 23.7 million m^3 (31 million yd^3). Exploration included 14 flight auger holes, 4 dragline pits, 5 refraction seismic lines, and 5 electric resistivity surveys. Laboratory tests of samples from the auger holes included mechanical analyses and specific gravity (which ranged from 2.70 to 2.76). Moisture adsorption of two samples was 1.2 percent.

Area GA-4. GA-4 is made up of stream channel deposit G-13 on Burris Creek and G-14 on North Fork Stony Creek near the Burris Creek confluence. The deposits cover an area of 70 ha (180 ac). The Department explored the deposits with three shallow shovel holes. An average 56 percent of the material was gravel larger than 5.0 mm (0.2 in) in diameter and 11 percent was finer than the No. 200 sieve.

GA-4 has a volume of 1.4 million m^3 (1.8 million yd^3) based on an estimated average depth of about 2 m (6 ft). However, a Department refraction seismic survey conducted in 1978 included two lines in deposit G-14 that suggest a 3.2 to 4.6 m (11 to 15 ft) thickness for the gravel.

Area GA-5. Stream channel deposits G-15, G-16, G-17, and N-9, which comprise Area GA-5, are in North Fork Stony, Salt, and Heifer Camp Creeks. These deposits have been mapped at the reconnaissance level but have not been explored or tested. The Department estimates the volume of material in deposit G-15 at 38 000 m^3 (50,000 yd^3), deposit G-16 at 1 500 m^3

(2,000 yd^3), and deposit G-17 at 34 000 m^3 (45,000 yd^3). Deposit N-9 is a large area that includes all of deposit G-16 and part of G-17. The Bureau (1967) estimates the volume of N-9 at 1.0 million m^3 (1.3 million yd^3).

Quarry Rock

Several potential rock quarry sites are in competent deposits of the Great Valley Sequence. The Department identified 10 deposits (QA-1 through QA-10), and the Bureau identified three deposits (N-17 through N-19). The locations of the prospective quarry sites and exploratory diamond core holes are shown on Plate D-1.

Exploration of the quarry sites included six diamond core holes drilled at four of the sites, a small test blast, and surface rock sampling and testing at one site. Exploration of the other eight sites consisted of reconnaissance-level geologic mapping.

Area QA-1. The quarry site is on Williams Butte 10 km (6 mi) northwest of Newville Dam site. Area QA-1 contains about 3 million m^3 (4 million yd^3) of rock. Diamond core hole TH-5 was drilled into the saddle west of Williams Butte, penetrating unweathered conglomerate at 5.8 m (19 ft). Core from TH-5 combined with core from hole TH-6 in Area QA-3 was tested in the Department Laboratory. The results of these tests (Table D-4) indicate marginal suitability of the rock for use in a high earth-rock embankment.

A small test blast near TH-5 found weathered rock from the surface to 9 m (30 ft). A D-8 tractor with hydraulic ripper could excavate the upper 3 m (10 ft) of weathered material.

Area QA-2. A strike-ridge 10 km (6 mi) northwest of Newville Dam site contains conglomerate and sandstone deposits similar to Area QA-1. The Department (1965) estimated there is 4.4 million m^3 (5.8 million yd^3) of fresh rock, 1.5 million m^3 (2.0 million yd^3) of random fill, and 1.1 million m^3 (1.4 million yd^3) of waste material. Later design studies placed the ratio of fresh rock to waste rock at 1:1 (DWR, 1966).

Area QA-3. Another strike ridge, 8 km (5 mi) northwest of Newville, contains sandstone and conglomerate with interbedded shale similar to Areas QA-1 and QA-2. The volumes of rock, random fill, and waste are about equal to those at Area QA-2.

Core hole TH-6 was drilled 44.7 m (147 ft) into Area QA-3. Highly weathered conglomerate and sandstone extends from the surface to 5.5 m (18 ft), lightly weathered conglomerate from 5.5 to 8.1 m (18 to 27 ft), highly weathered conglomerate from 8.1 to 11.2 m (27 to 37 ft), unweathered shale from 11.2 to 14.4 m (37 to 47 ft), and unweathered sandstone with interbedded shale below 14.4 m (47 ft).

Areas QA-4, QA-5, and QA-6. These areas, located 6 to 10 km (4 to 6 mi) west of the damsites, contain metavolcanic and basic igneous rock of the Great Valley Sequence ophiolite. A volume estimate is unavailable due to the lack of adequate exploration.

TABLE D-4

SUMMARY OF PHYSICAL TEST RESULTS
ON CONGLOMERATE NX DRILL CORES 1/

Lab. No.	Field Sample	Specific Gravity Bulk S.S.D.	Specific Gravity Apparent	Adsorption %	Los Angeles Abrasion % Loss "F" Grade 100 Rev.	Los Angeles Abrasion % Loss "F" Grade 1000 Rev.	Na_2SO_4 Soundness % Loss in 5 Cycles		Compressive Strength, kPa
60C-128	1	2.61	2.66	1.0	5.5	38.7	13.4 2/	8.2 3/	(A) 47,000 (B) 51,500
60C-129	2	2.59	2.66	1.7	7.7	42.6	43.1 2/	31.3 3/	57,600
60C-130	3	2.59	2.66	1.5	6.0	38.5	35.2 2/	27.3 3/	(A) 98,400 (B) 91,700 (C) 24,000

1/ These samples were selected as representative of the conglomerate cores from holes TH-5 and TH-6 (Williams Butte and Hufford Quarry areas).

2/ Percent loss on full-size sieve.

3/ Percent loss on ½ finer sieve.

NOTE: Compressive strength specimen 60C-130(C) was capped on the ends with a sulfur compound because of pitting of the ends when cut with diamond saw.

Area QA-6 is a deposit of serpentinized peridotite. The Department tested this rock in 1979 with the following results:

Los Angeles Abrasion % Loss "A" Grade		Sodium Sulfate Soundness % Loss	Adsorption	Specific Gravity 19 mm to + No. 4		Los Angeles Rattler - No. 4 Sieve
100 rev.	500 rev.	5 cycles	%	Bulk S.S.D.	Apparent	% Loss
8.1	32.3	48.2	3.4	2.51	2.65	46.7

The results show that this rock is unsuitable for use in free-draining zones of a high earth-rock embankment.

Areas QA-7 and QA-8. About 1.6 km (1 mi) south of Millsite Dam site are two north-south elongated deposits of sandstone and conglomerate rock. The deposits have not been explored or tested. Geologic mapping is insufficient for volume computations.

Area QA-9. Area QA-9 is 5 km (3 mi) north of Newville Dam site on Rocky Ridge. Erosion-resistant beds of conglomerate and sandstone with minor amounts of interbedded mudstone make up the ridge. The Department drilled three diamond core holes into the ridge in 1979 and tested core samples. Exploration to date suggests that there are 16 million m^3 (21 million yd^3) of fresh rock and 5 million m^3 (6.5 million yd^3) of weathered rock and mudstone above elevation 311 m (1,020 ft).

Rocky Ridge is part of a homoclinal geologic structure with conglomerate and sandstone beds striking northward and dipping 60 to 80 degrees eastward (see Appendix C). The conglomerate varies from fine- to coarse-grained with well rounded cobbles to 30 mm (12 in) in diameter. The sandstone is a graywacke ranging from massive- to thin-bedded and coarse- to fine-grained. Both the conglomerate and sandstone deposits contain beds with widely varying physical characteristics, which affect the strength and durability of the rock. Mudstone makes up about 5 percent of the rock mass.

The rock is weathered to a depth of about 6 m (20 ft). Between 6 and 30 m (20 and 100 ft), weathering is along fractures and below 30 m (100 ft), weathering is rare. The rock fractures are usually well developed, numerous, and generally have good continuity. Fracture spacing suggests that rock sizes larger than 100 to 150 mm (4 to 6 in) in diameter are obtainable.

The Department Soils Laboratory tested representative samples from core holes in Area QA-9. Table D-5 compares average physical properties of the rock core with the acceptable criteria for each test. Saturated conglomerate is weak in triaxial shear strength and may not be suitable as upstream rockfill. The minimum friction angle of the sandstone and conglomerate is about 35 degrees at 1 030 kPa (150 lb/in^2) confining pressure and 15 percent strain.

Table D-6 compares mechanical properties of QA-9 rock to test results of Pyramid Dam rock. The high percentage of losses in the sodium sulfate tests may indicate an internal structural weakness for the QA-9 rock.

TABLE D-5

AVERAGE PHYSICAL PROPERTIES OF ROCK CORES FROM ROCKY RIDGE MATERIAL

Lab No.	Description	Unconfined Compressive Strength Dry psi	Unconfined Compressive Strength Sat psi	Young's Modulus Compression Dry psi X 10^6	Young's Modulus Compression Sat psi X 10^6	Poisson's Ratio Dry	Poisson's Ratio Sat	Brazilian Tensile Strength Dry psi	Brazilian Tensile Strength Sat psi	Bulk Specific Gravity SSD Basis	Percent Absorption SSD	Slake Test Percent Loss
79-961	Conglomerate		5160(4)*		2.60(3)		.57(3)			2.59	1.2	
79-962	"	16,700(4)*		4.39(3)		.15(3)				2.58	1.6	
79-963	"		4790(1)					1,288(2)	605(2)	2.61	1.2	
79-964	"		9090(1)		3.32(1)		.17(1)					
79-959	"	22,100(1)		4.22(1)		.12(1)		1,990	1320	2.59	1.2	
79-960	"	11,900(1)		3.92(1)						2.59	1.2	
Average	"	16,800	5750	4.26	2.78	.15	.47	1,522	844			
79-964	" **A**		8350		2.46		.33			2.61	1.0	0(7 cycle)
	B		6670		2.07		.44			2.60	1.1	0(14 cycle)
	C									2.60	1.3	0(28 cycle)
79-952	Sandstone	19,700(5)		2.77(3)		.16(3)				2.51	2.0	
79-961	"		12,800(5)		2.70(3)		.23(3)			2.54	2.1	
79-953	"		8350(1)		1.68(1)		.52(1)	1,725	564(2)**	2.53	2.1	
79-959	"	30,000(1)		3.71(1)		.09(1)		1,980(1)	1480(1)	2.54	2.1	
79--65	"	26,700(1)	13000(1)	4.03(1)	3.29(1)	.10(1)	.25(1)	2,160(1)		2.62	1.2	
79-966	"								1870(1)	2.62	1.2	
Average	"	22,160	12160	3.21	2.61	.13	.29					
79-954	" **C**		7210		0.83					2.55	2.3	0(7 cycle)***
	D		6850		0.96							0(14 cycle)***
	E											0.2(28 cycle)****
Acceptable Criteria		7,500 min.								2.5 min.	3.0 max.	

* () No. of specimens
** Specimens are badly cracked before testing
*** Specimens have vertical cracks before testing
**** Specimen cracked at 21 cycles

TABLE D-6

ROCKFILL SUITABILITY TESTS ON CRUSHED ROCK CORES FROM ROCKY RIDGE QUARRY FOR NEWVILLE DAM

Lab No.	Material Type	Drill Hole	Bulk Specific Gravity SSD*	Percent Absorption SSD*	Sodium Sulfate Soundness 1½"x3/4" % Loss	Wet-Dry Test 10 cycles % Loss 1½"x3/8" Average	Los Angeles Abrasion Test Grading A % Loss: Before Wet-Dry Test 100 Rev.	Before Wet-Dry Test 500 Rev.	After Wet-Dry Test 100 Rev.	After Wet-Dry Test 500 Rev.	Freeze-Thaw Test 25 cycles 1" x ½" % Loss Average	Water Soak Test, 14 Days 1" x ½" % Loss
79-965-966	Coarse-grained Sandstone	RRQ3	2.62 2.62	1.2 1.2	5.4	0.2	9	23			1.5	0.1
79-949-959	Medium-grained Sandstone	RRQ3	2.54 2.54	2.9 2.1	45.8 56.5	3.0	7	33			5.4	0.2
79-952-954	Fine-grained Sandstone		2.51 2.55	2.0 2.3	28.3	0.4	12	29			15.7	0.6
79-960-961	Coarse-grained Conglomerate		2.59 2.59	1.2 1.2	68.4 70.6	0.9	8	28			4.4	3.3
79-962-967	Fine-grained Conglomerate		2.58 2.62	1.6 1.0	19.8	0.9	7	27			4.0	0.5
Acceptable Limits for Concrete Aggregate				2-3	5-10		10	45				
Pyramid Dam Shale			2.52	0.8	6	2.9*** 0.3**	4	18	5	20	0.9**	0.2

*Values Computed from data on uncrushed cores
**Minus #4
***Minus 3/8

A comparison was made with the Deer and Miller engineering classification chart (Figure D-1). The average results of unconfined compressive strength and Young's Modulus in compression for the Pyramid Dam, Oroville Dam, Venado Sandstone, and Rocky Ridge materials have been plotted. According to the classification chart, the saturated conglomerate would be a D rock (low strength, average modulus), and the saturated sandstone would be C rock (medium strength, average modulus). Both dry sandstone and conglomerate would be B rocks (high strength, average to low modulus).

Consolidation-permeability tests indicate that the permeability rate ranges from 3 to 6 m/day (10 to 20 ft/day) for the sandstone and 15 to 30 m/day (50 to 100 ft/day) for the conglomerate under loads up to 195 000 kg/m^2 (20 t/ft^2).

Area QA-10. Quarry site QA-10 is 3.2 km (2 mi) north of Newville Dam site on Rocky Ridge. The deposit is a round knoll approximately 430 m (1,400 ft) in diameter with moderate-to-steep slopes that support a moderate growth of oak. The geology is similar to Area QA-9. Subsurface exploration and material testing have not been done. Based on reconnaissance-level geologic mapping, Area QA-10 contains approximately 2.5 million m^3 (3.1 million yd^3) of fresh rock and 1.5 million m^3 (2 million yd^3) of weathered rock and mudstone.

Area N-17. On Rocky Ridge 1.6 km (1 mi) north of Newville Dam site, the Bureau identified deposits of lightly weathered to fresh sandstone and conglomerate. An average 7 m (24 ft) of slopewash and intensely to moderately weathered rock overlie the quarry rock.

Preliminary estimates indicate the deposit contains about 4.7 million m^3 (6.1 million yd^3) of rock and 2.1 million m^3 (2.7 million yd^3) of random fill. Subsurface exploration of the deposit and laboratory testing of the rock have not been done.

Area N-18. About 2.3 km (1.5 mi) south of Newville Dam site, the Bureau located a deposit of sandstone and conglomerate similar to Area N-17. The deposit contains an estimated 1.7 million m^3 (2.2 million yd^3) of rock and 1.8 million m^3 (2.3 million yd^3) of random fill. Stripping would require the removal of 7 m (24 ft) of slopewash and weathered rock for quarry development. Subsurface exploration and laboratory testing of the rock have not been done.

Area N-19. A deposit of sandstone and conglomerate is 1.6 km (1.0 mi) southeast of Chrome, on the southern end of Rocky Ridge. In 1959, the Department drilled a 48-m (158-ft) core hole into the deposit. About 5.5 m (18 ft) of intensely weathered and fractured sandstone and shale overlie unweathered deposits of very fine- to fine-grained sandstone with some interbedded shale and conglomerate. The Bureau estimated 23.2 million m^3 (30.4 million yd^3) of rock and 5.5 million m^3 (7.2 million yd^3) of random fill are in this deposit.

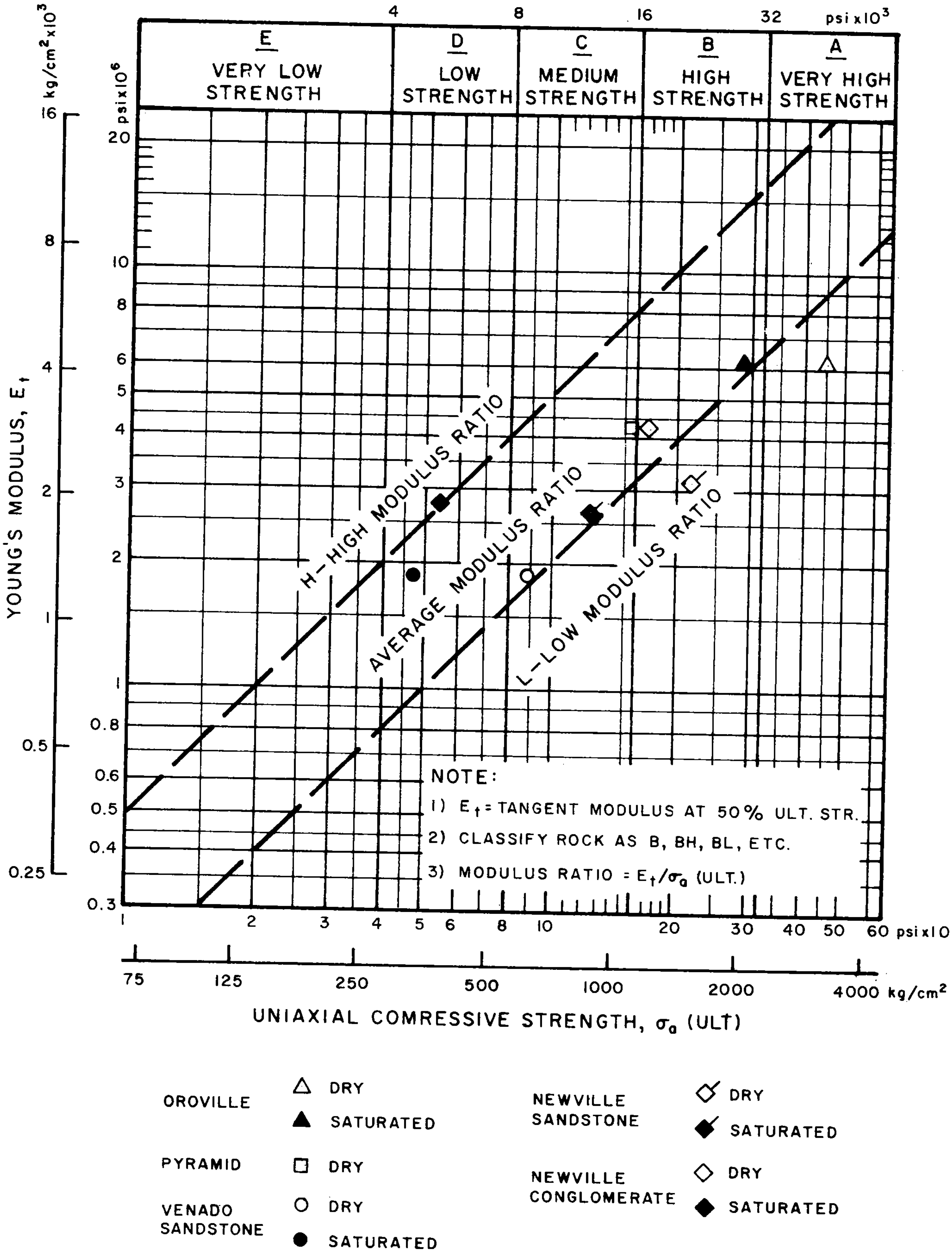


FIGURE D-1 ENGINEERING CLASSIFICATION FOR INTACT ROCK

Conclusions and Recommendations

Impervious Material

Previous exploration and testing of impervious deposits focused primarily on Tehama Formation deposits, which are outside of the prospective reservoir areas. Potential borrow areas including about 460 million m^3 (600 million yd^3) of impervious Tehama Formation material have been explored and tested. Terrace deposits within the reservoir areas should be further explored and tested. For example, the unexplored terraces T-24, T-25, T-29, T-30, T-31, and T-32 are within 6.5 km (4 mi) of Newville Dam site and contain about 8.7 million m^3 (11.4 million yd^3) of impervious material.

Exploration is needed to accurately determine the quantity and quality of the terrace materials available from Areas TA-2, TA-4, TA-5, and TA-6. The deposits should be mapped on 1:4800 scale topographic maps. A bucket auger capable of drilling to 12-m (40-ft) depths should be used to explore and sample the terrace materials. Exploration should include auger holes, trenches, in-place density tests, and ground water level determination. Selected samples should be tested in the laboratory for gradation, Atterberg Limits, specific gravity, moisture content, consolidation-permeability, compaction, and triaxial shear strength.

Pervious Material

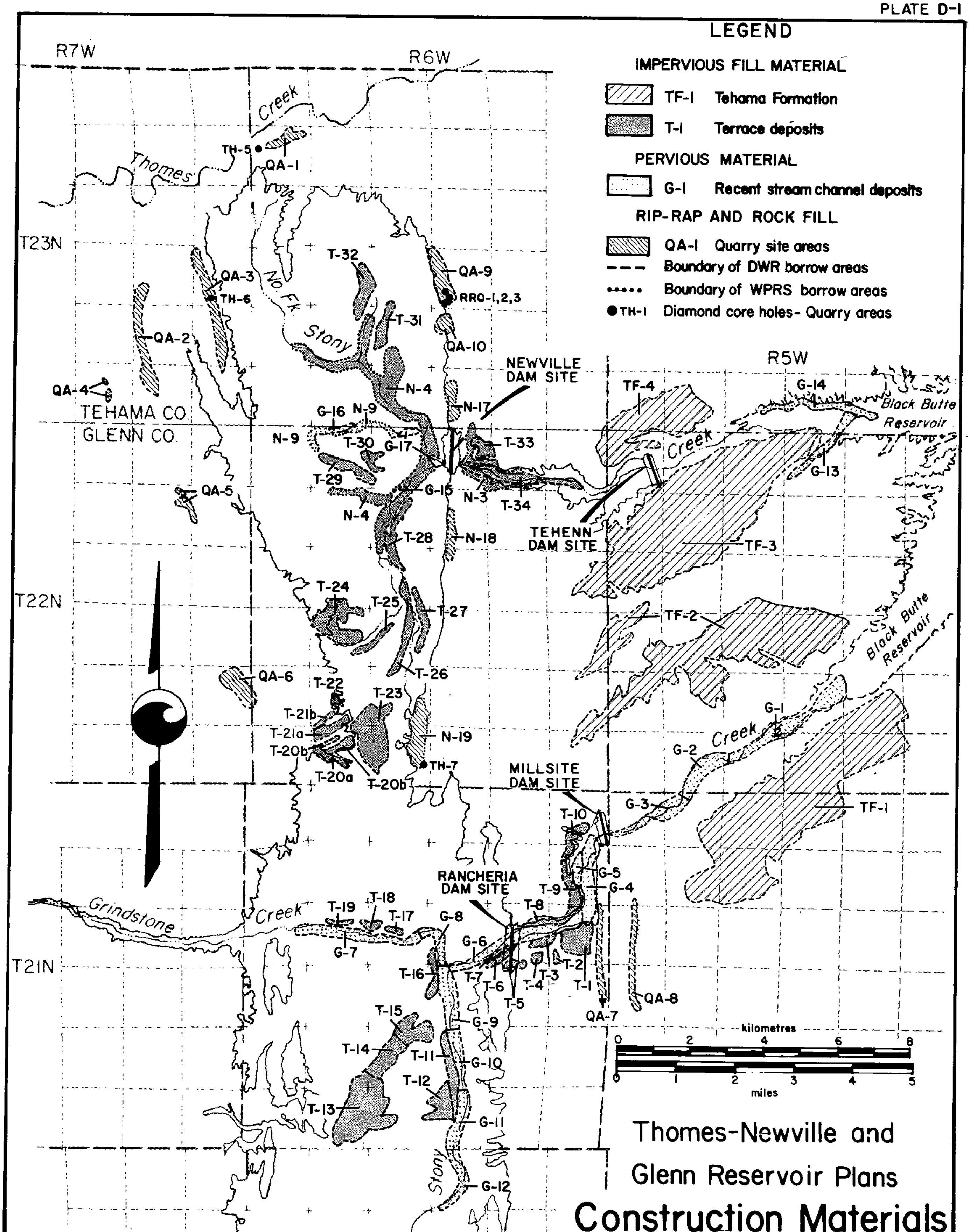
More than 26 million m^3 (34.5 million yd^3) of quality pervious material have been explored and tested (DWR, 1968). Exploration is needed of the remaining pervious stream channel deposits closer to Newville Dam site (Salt, Heifer Camp, North Fork Stony, and Burris Creeks and possibly, Stony Creek below Millsite Dam site). The areas should be mapped on 1:4800 scale topographic maps and dragline pits and refraction seismic studies should be used to determine gravel thickness. Ground water levels and material testing are needed from the dragline pit exploration. The suitability of the gravel for rockfill and concrete aggregate should be determined.

Quarry Rock

A detailed investigation of potential rock quarry sites is needed. Many source areas have been identified, but limited exploration and testing have yielded inconclusive results. At present, it appears that stream channel deposits would be less costly than quarried rock for use in the shells of Newville Dam.

Further exploration of QA-9 is needed if quarried rock is to be further considered. A test quarry and a test fill would be required to properly evaluate breakage and to provide large samples for laboratory testing. Detailed geologic mapping, additional diamond core holes and geophysical surveys would be needed to evaluate the potential quarry quality and boundary conditions.

Near Rancheria and Millsite Dam sites, quarry areas QA-7, QA-8, and N-19 need detailed geologic mapping and additional subsurface exploration. Weathered sandstone and conglomerate from these areas and on Rocky Ridge should be explored and tested for possible use as pervious random fill material.


LEGEND
IMPERVIOUS FILL MATERIAL
TF-1 Tehama Formation
T-1 Terrace deposits
PERVIOUS MATERIAL
G-1 Recent stream channel deposits
RIP-RAP AND ROCK FILL
QA-1 Quarry site areas
Boundary of DWR borrow areas
Boundary of WPRS borrow areas
TH-1 Diamond core holes- Quarry areas
R7W
R6W
R5W
T23N
T22N
T21N
Thomes Creek
No. Fk. Stony
TEHAMA CO.
GLENN CO.
NEWVILLE DAM SITE
TEHENN DAM SITE
MILLSITE DAM SITE
RANCHERIA DAM SITE
Black Butte Reservoir
Grindstone Creek
Stony
kilometres
0 2 4 6 8
miles
0 1 2 3 4 5
Thomes-Newville and
Glenn Reservoir Plans
Construction Materials

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APPENDIX E

RANCHERIA DAM SITE GEOLOGY

APPENDIX E. RANCHERIA DAM SITE GEOLOGY

Rancheria Dam site is on Stony Creek, 32 km (20 mi) west of Willows, in T21N, R6W, Sections 14 and 23, MDB&M. Access to the site is by paved road from Willows to within about 0.8 km (0.5 mi) of the site, and by private graveled road that crosses the axis. A Department topographic map, with a scale of 1:4,800 and contour interval of 6.1 m (20 ft), covers the site.

Previous Studies

The Department previously reported on Rancheria Dam site in Bulletin 136 (DWR, 1965). Subsurface investigations included auger holes in the channel and in the terraces on the right and left abutments. A geologic map at a scale 1:4,800 and contour interval of 6.1 m (20 ft) was prepared as part of the report.

The Department conducted further subsurface investigations in 1967. These included drilling and water pressure testing nine diamond drill holes in the dam foundation and two holes at a potential spillway site. Results of this work are presented in a separate office report (DWR, 1968). Concurrently with the drilling program, the Department conducted shallow seismic refraction and resistivity surveys of the gravels along Stony and Grindstone Creeks (DWR, 1967).

The Corps of Engineers made reconnaissance designs and cost estimates for Rancheria Reservoir in 1969. The Corps' report briefly describes the site geology and provides a basis for cost estimates (USCE, 1969). This geologic investigation consisted only of reevaluation of existing data.

Under contract with the Department, Earth Sciences Associates (ESA) conducted a seismic and fault activity study of the proposed Glenn Reservoir complex from August through December 1979 (ESA, 1980). Their study at Rancheria Dam site included geologic mapping, terrace age dating, and photogeologic interpretation. The results of their work are incorporated in the site geology of this appendix.

Site Geology

Outcrops of bedrock are relatively scarce at Rancheria Dam site. The upper portions of the abutments are mantled by a thin soil cover, the lower portions are covered with terrace deposits, and a wide deposit of Recent alluvium blankets the channel. Bedrock is well exposed only in steep banks next to the channel and locally on the hill slopes. Since the rock units at the site have a consistent attitude, a geologic map of the foundation was prepared mainly by projecting the geologic contacts from the outcrops near the channel to the abutments.

The foundation rock at the site belongs to the Lodoga Formation of the Great Valley Sequence of Lower Cretaceous age. It consists of mudstone with interbedded sandstone. (As used here, mudstone designates clay, silt, claystone, siltstone, shale, and argillite. The term is used when the clay, silt, and sand fractions vary from place to place so that more precise terms are not possible.) The beds strike nearly north-south, paralleling the dam axis and dip about 65 degrees east, or downstream.

For geologic mapping purposes, the rock types were grouped into three units: (1) mudstone with interbedded sandstone (approximately 90 percent mudstone); (2) mudstone with interbedded sandstone (approximately 65 percent mudstone); and (3) sandstone with minor interbedded mudstone (approximately 85 percent sandstone). The contacts between the three units are usually gradational and based on interpretation. Overlying the bedrock are terrace deposits at two elevations and gravels in the stream channel. These units are shown on the Rancheria Dam site geologic map (Plate E-1).

The mudstone is dark gray, soft to moderately hard, and usually thinly bedded, although the bedding is difficult to see. It is composed of a mixture of clay, silt, and minor amounts of fine sand. After prolonged weathering, the mudstone slakes to a depth of about 0.3 m (1 ft).

The sandstone is gray, moderately hard, mostly fine-grained, and usually thin-bedded. The sandstone is lightly jointed normal to the bedding. The individual beds average about 75 mm (3 in) in thickness, but the thickness varies from a negligible amount to 0.6 m (2 ft). The sandstone beds are usually separated by several feet of mudstone, but in places the sandstone beds are sandwiched together and form units up to 30 m (100 ft) thick. The greater resistance of the sandstone to erosion has created the ridges that form the abutments at the site. The valleys that extend perpendicular to the stream channel upstream and downstream from the axis have been eroded in the softer mudstone.

Terrace deposits at the site consist mostly of clayey gravels and average about 2 m (7 ft) in thickness, based on the results of the auger drilling. The maximum known thickness is about 4.6 m (15 ft). These deposits are found in two ages of terraces, designated Qt_3 and Qt_4 (Plate E-1). Terrace Qt_3 is approximately 30,000 years old and found at elevation 192 m (630 ft). Terrace Qt_4 is about 130,000 years old and found at elevation 210 m (690 ft).

The stream channel is underlain by a wide deposit of sand and gravel. Auger holes indicate the alluvium averages about 3.7 m (12 ft) in thickness at the site. The deposits consist of moderately clean sands and well-rounded gravels and cobbles up to 200 mm (8 in) in diameter.

Continuous faults or prominent shear zones resulting in disrupted bedrock or terrace deposits were not observed at the site. Two minor shear planes cutting the mudstone are exposed on the south side of the streambank below the youngest terrace level. The more westerly shear strikes N 55° W, dipping 70° to the southwest, and displays 0.6 to 0.9 m (2 to 3 ft) of apparent dip slip and/or left lateral displacement along the shear plane.

The easterly shear is vertical, striking N 35° W with unknown displacement; minor dragging of adjacent mudstone beds suggests the southwesterly beds are displaced upward. Except for this minor dragging, bedding attitudes are consistent on both sides of the two small shears. Both shear planes are thin, slightly arcuate locally, and are commonly lined and/or filled with calcite, resembling many other small old faults in Great Valley rocks (ESA, 1980).

Other bedrock discontinuities in the damsite area occur approximately 600 to 900 m (2,000 to 3,000 ft) west of these faults and along a north-flowing tributary drainage ravine south of the Stony Creek channel. No faulting or shearing is exposed at any of these areas. It is believed that any faults in the foundation are minor.

Foundation Conditions

In 1962, several auger holes were drilled in the channel and terrace deposits in connection with the construction materials investigation for Glenn Reservoir. In general, these holes were shallow and refusal was at the bedrock contact. The average depth was 2.1 m (7 ft) except on the left abutment, where bedrock was encountered at 5.9 m (19.5 ft). Most of the material was a clayey gravel possibly suitable for use as impervious fill in the dam embankment.

Foundation conditions in the dam embankment area were explored during the spring of 1967 with nine diamond holes ranging in depth from 9.8 to 61 m (32 to 201 ft). A potential spillway location was explored with two holes, 23.3 and 40.5 m (76.6 and 133 ft) in depth. The drill logs of the holes are included in the memorandum report, "Engineering Geology of Rancheria Damsite and Construction Materials Data" (DWR, 1968). Drilling was augmented by seismic surveys along the axis to provide stripping estimates. Plate E-1 shows the location of drill holes and seismic survey lines.

The foundation rock comprises a sequence of stratified layers of sandstone and mudstone. Mudstone generally predominates over sandstone in the embankment area, but in the area along the damsite axis sandstone is predominant. Sandstone, the stronger rock type, is therefore favorably situated in the area where the greatest loads would be concentrated. In the downstream and upstream portions of the embankment, the foundation rock would be largely mudstone.

Because mudstone is the weakest of the rock types present, unconfined compressive strength tests were conducted on eight specimens from hole LC-2. Strengths ranged from 5 400 to 39 300 kPa (780 to 5,700 lb/in^2). Average of the eight values was 19 100 kPa (2,770 psi) (DWR, 1968). These strengths are considered to be adequate for the dam heights under study.

Right Abutment

The right abutment is irregular in shape with two broad terraces at elevations 192 and 210 m (630 and 690 ft). A long ridge runs from elevation 268 to 274 m (880 to 900 ft) and ends in a fairly even 30° slope at the top of the abutment. Due to the lack of bedrock outcrops, most of the subsurface interpretations are based on drilling and seismic data.

Both terraces average 3 m (10 ft) in thickness, with some evidence of channeling in the upper terrace. About 1.5 m (5 ft) of slaked and weathered bedrock occurs beneath the terrace materials.

The abutment between elevations 210 and 308 m (690 and 1,010 ft) has an average soil cover of 0.6 m (2 ft). The soil is clayey silt. Beneath the soil is about 3.7 m (12 ft) of intensely weathered and slaked bedrock. Fresh rock is found at about 9 m (30 ft).

Foundation preparations should include the removal of all soil, terrace deposits and intensely weathered bedrock. The cut-off trench should be excavated to fresh rock to an average depth of 9 m (30 ft).

Channel

The channel is about 300 m (750 ft) wide at the damsite. The width of flowing water varies from 9 to 90 m (30 to 300 ft) at the axis. Alluvium fills the channel section to an average depth of 4.6 m (15 ft). Several shallow buried channels were located as a result of the refraction seismic survey. Foundation preparation of the channel should include the removal of all alluvium and 0.3 m (1 ft) of slaked and weathered bedrock. The cut-off trench should be excavated to fresh rock at an average depth of 5.5 m (18 ft).

Left Abutment

The left abutment has an irregular shape both normal and parallel to the stream channel. The terrace deposits average 4.5 m (15 ft) in thickness with some local channeling indicated by the seismic data. Beneath the terrace deposits is about 1.5 m (5 ft) of intensely weathered and slaked bedrock. The abutment between elevations 204 to 308 m (670 to 1,010 ft) has a few outcrops of weathered bedrock, but is mostly covered by soil consisting of clayey silt averaging 0.6 m (2 ft) in thickness. Beneath the soil is 3.7 m (12 ft) of intensely weathered and slaked bedrock. Fresh rock is found at the depth of about 12 m (40 ft).

Foundation preparations should include removal of all soil, terrace deposits, and intensely weathered bedrock. The average depth of stripping would be 4.3 m (14 ft). The cut-off trench should be excavated to fresh rock to an average depth of 12 m (40 ft).

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Grouting

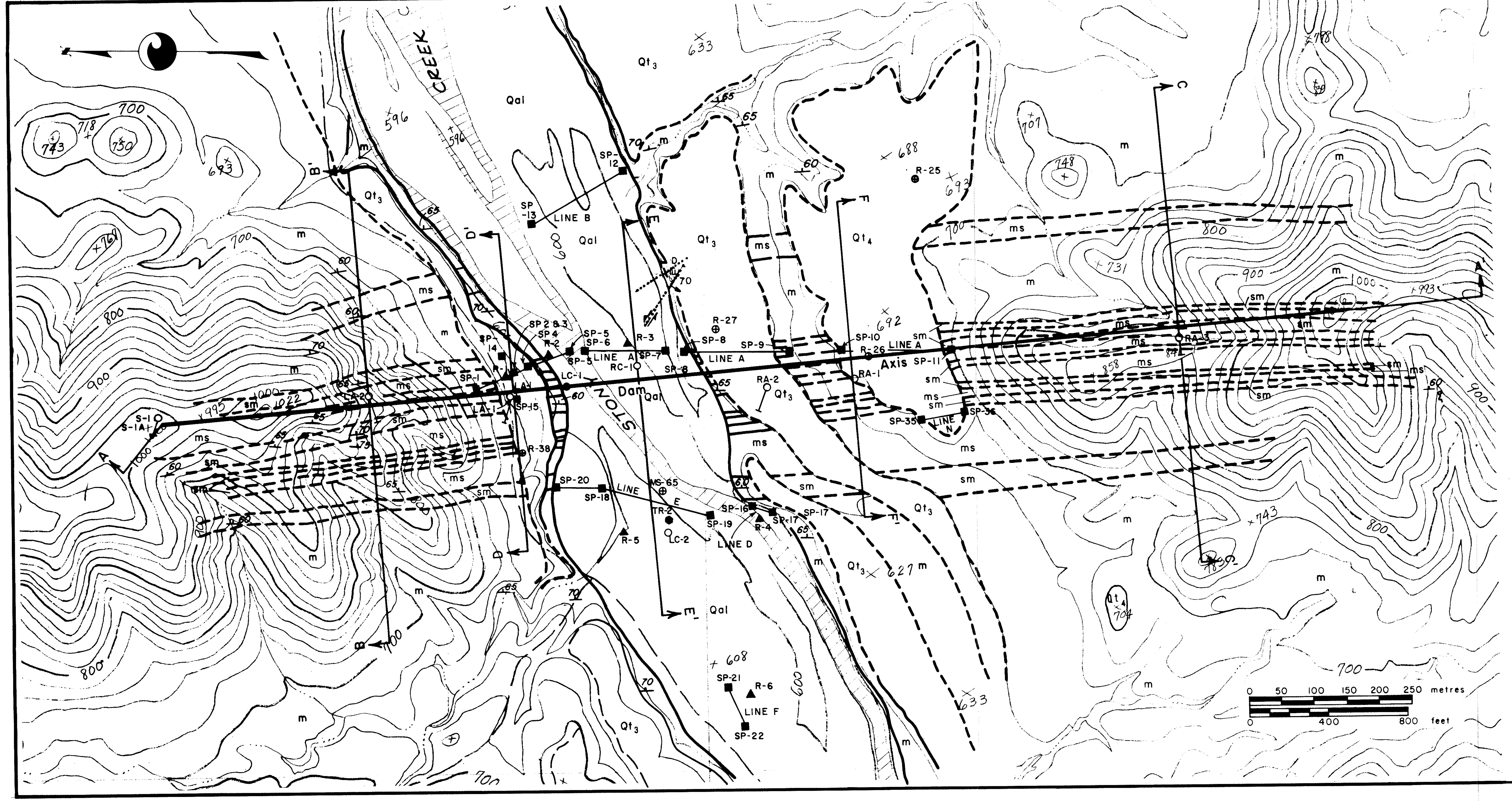
Water pressure tests on all drill holes show that the foundation bedrock is essentially tight. Generally, water losses at a pressure of 340 kPa (50 psi) were only a few litres per minute. Higher losses were confined to short intervals of fractured sandstone, shear zones, and weathered zones. Leakage at the site would not be expected to be a problem, since the rock is lightly jointed and the fault zones observed are neither extensive nor highly permeable. It appears the grout take would be low.

Conclusions

1. The proposed damsite appears suitable for an embankment-type dam; the maximum height would be limited by topography and cost.
2. The foundation is mainly composed of mudstone, a relatively weak rock, and sandstone, a relatively strong rock.
3. There are no active faults at the site. The small inactive faults would pose no unusual construction difficulties.
4. Foundation preparation should include the removal of all soil, alluvium, terrace deposits, and intensely weathered bedrock.
5. A light grouting program would be required; grout takes would be expected to be low.

Recommendations

Before final design, further drilling, water testing, and seismic exploration should be done to better define stripping estimates, locations of fault zones, and other factors affecting the strength and water-tightness of the foundation.



LEGEND

QUATERNARY

Qal	Recent Alluvium - sand and gravel
Qt_3	Terrace deposits - clayey gravels, number indicates relative age.

CRETACEOUS - LODOGA FORMATION

m	Mudstone with interbedded sandstone, approximately 90% mudstone.
ms	Mudstone with interbedded sandstone, approximately 65% mudstone.
sm	Sandstone with interbedded mudstone, approximately 85% sandstone.

SYMBOLS

Geologic contact - dashed where inferred or projected.

50 Strike and dip of bedding.

RA-2 Diamond Drill hole showing projection, if any, on surface.

R-38 Auger Hole.

SP-8 LINE A SP-9 Seismic Line and Shot Point.

R-6 Resistivity Station

TR-2 Test Pits

A A Geologic Section Line (see DWR 1968 for cross sections, plates 2 and 3)

U D 70 Fault, dotted where covered by younger material, direction of dip indicated by arrow, U=up thrown side, D=down dropped side, direction of movement.

Vertical fault.

STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES
NORTHERN DISTRICT

AREAL GEOLOGY
RANCHERIA DAM SITE

APPENDIX F

SUMMARY OF PRIOR PLANNING

APPENDIX F. SUMMARY OF PRIOR PLANNING

The Thomes-Newville and Glenn Reservoir Plans are the latest in a long series of water development activities affecting the Stony and Thomes Creek basins. This appendix summarizes the general water history of the area and traces the various specific planning studies that led to the current plans.

Early Water Development

Settlers began moving into the Stony and Thomes Creek areas in the latter 1840s. Their first crops were dry-farmed wheat and barley, but production was undependable and individuals and small groups soon began diverting from the creeks to irrigate nearby lands. (The earliest water right priorities on Stony Creek date to 1864.) In 1866, Will S. Green, known as the "Father of Irrigation in the Sacramento Valley", made a survey for a canal that would divert from Stony Creek near the present Black Butte Dam and extend down the west side of the valley as far south as Willows. The Stony Creek Irrigation Company was formed in 1888 and built several miles of canal near Green's route. Two of California's earliest irrigation districts were formed in 1887-88 to serve Stony Creek water to some 16 000 ha (40,000 ac), but their plans were abandoned due to an inability to sell bonds. At about the same time, there was an abortive start on what later became the Glenn-Colusa Irrigation District's main canal from the Sacramento River. Sometime before 1892, the Corning Irrigation Company began diverting water from Thomes Creek across a divide to the Corning area via Squaw Hollow and the Thomes Creek Irrigation Company was organized to irrigate about 1 600 ha (4,000 ac) along lower Thomes Creek. About 1897, the Lemon Home Water, Power, and Light Company built a short canal to irrigate lands to the north of Stony Creek.

All of these early irrigation developments depended on the natural flow of the creeks and were able to furnish sufficient water only through early summer. Diminishing crop yields in the 1890s and the extremely dry season of 1898 caused a number of farm failures and emphasized the need for reservoir storage. In about 1900, the California Water and Forest Association asked the U. S. Geological Survey (USGS) to study potential reservoir sites on Stony Creek, which was considered one of the most promising streams for irrigation development. The USGS work focused attention on the unfinished Glenn-Colusa Canal (then called the Central Canal) and led to completion of the upper reach and the beginning of operation in 1905. The Glenn-Colusa Canal crosses Stony Creek about 6 km (4 mi) above its mouth via a temporary gravel dam that is reinstalled each spring; any flow remaining in Stony Creek at this point during the irrigation system is thus diverted to the canal.

The 1903 report on the USGS study described a survey of the entire Stony Creek Basin. Three damsites were studied in detail: (1) Briscoe

Creek, just west of the present Stony Gorge Reservoir, (2) East Park on Little Stony Creek, and (3) Millsite on Stony Creek about 5 km (3 mi) downstream from the present Rancheria Dam site. The USGS also examined Newville Dam site on the North Fork of Stony Creek and, finding its natural runoff inadequate to fill any substantial reservoir, made a cursory study of diverting water from Thomes Creek to Newville Reservoir. The USGS report mentioned that a reservoir site on Thomes Creek, upstream from Paskenta, "was surveyed by others several years ago". No information on that earlier study could be obtained during the present investigation.

The Orland Project

Following the lead of the USGS study, a group of landowners along lower Stony Creek formed a water users' association in 1906* and petitioned the Secretary of the Interior to proceed with an irrigation project under the Reclamation Act of 1902. The newly created U. S. Reclamation Service responded energetically and the Orland Project was born. After reexamining the various reservoir sites, the Reclamation Service recommended construction of East Park Reservoir on Little Stony Creek about 64 km (40 mi) above the Stony Creek Irrigation Company diversion. The Interior Secretary authorized this proposal in October 1907 and the construction contract for East Park Dam was awarded in October 1908. Construction began on June 11, 1909, and the dam was completed on June 15, 1910. East Park Dam is a concrete gravity-arch structure that rises about 27 m (90 ft) above streambed; it was initially designed to impound a 57 000-dam^3 (46,000-ac-ft) reservoir.

As soon as East Park Reservoir was completed, the two dry seasons of 1911-12 and 1912-13 showed that the natural inflow would not assure its filling each spring. To help overcome this problem, the 11 km (7 mi) East Park Feed Canal was constructed to divert up to 6 m^3/s (200 ft^3/s) from the main stem of Stony Creek to East Park Reservoir. The canal and the associated 9-m- (28-ft-) high Rainbow Diversion Dam were completed in 1914. After these diversion facilities were added, the spillway of East Park Reservoir was modified to increase reservoir storage capacity to the present 63 000 dam^3 (51,000 ac-ft).

To serve Orland Project lands lying south of Stony Creek, the South Diversion Dam was completed in 1915 at the site of the Stony Creek Irrigation Company diversion. The South Diversion Dam was a concrete sill about 1 m (3 ft) high; it was removed during construction of Black Butte Dam (1960-63) and replaced with a direct diversion from the dam's outlet works. The North Diversion Dam was constructed in 1913 8 km (5 mi) downstream from the South Diversion Dam; it replaced the original diversion dam of the Lemon Home Water, Light, and Power Company. The North Diversion Dam was partially replaced in 1954 and remains in service today. It is a concrete overpour dam about 2 m (7 ft) above tailwater; wooden flashboards are installed during the irrigation season to raise the water surface an additional 0.6 m (2 ft). The initial Orland Project distribution systems were designed to serve 5 800 ha (14,300 ac), but with the addition of the East Park Feed Canal and the raising of the East Park spillway, expansions were begun to serve an

*The association was reorganized in March 1907 under its present title, Orland Unit Water Users Association.

additional 2 400 ha (6,000 ac). The distribution system eventually came to include about 63 km (39 mi) of canals and 160 km (100 mi) of laterals, most all of which are concrete-lined.

When the Orland Project first began operating, the annual diversion ranged up to about 93 000 dam^3 (75,000 ac-ft) in wet years. But, substantial deficiencies were encountered in the dry years of 1918 and 1920, and attention was soon drawn to the need for more storage. The Reclamation Service again surveyed potential damsites, including Briscoe Creek, Millsite, Stony Gorge, and Stonyford. Millsite Dam site was explored in detail in 1923 with three test pits and 19 diamond drill holes; these studies were directed at a 41-m- (135-ft-) high dam and a 142 000-dam^3 (115,000-ac-ft) reservoir that would have extended up Stony Creek to within about 2 km (1 mi) of the town of Elk Creek. However, in 1923 the water users voted down a proposal to build Millsite Dam (on the grounds of excessive cost). The following year, 1924, brought a severe drought and the project's water supply was reduced to about 20 percent of normal. Late that year, water users voted in favor of building Stony Gorge Reservoir and the association became very active in support of the plan (which included no expansion of service areas).

The U. S. Bureau of Reclamation (successor to the Reclamation Service) awarded the contract for Stony Gorge Dam in 1926 and it was completed in October 1928. Stony Gorge Dam is a concrete slab and buttress structure that rises 37 m (120 ft) above original streambed; it impounds a 61 900-dam^3 (50,200-ac-ft) reservoir. The added storage increased the project water supply as planned. In the 50 years since Stony Gorge Reservoir went into operation, the total annual diversion to the North and South Canals has ranged up to about 160 000 dam^3 (130,000 ac-ft) and averaged about 123 000 dam^3 (100,000 ac-ft). About two-thirds of the water is served via the South Canal and one-third via the North Canal.

Planning in the 1930s and 1940s

In 1930, the California Division of Water Resources published its Bulletin 25, "Report to the Legislature of 1931 on State Water Plan". This plan included a 142 000-dam^3 (115,000-ac-ft) Millsite Reservoir on Stony Creek, in recognition of the fact that East Park and Stony Gorge Reservoirs did not fully develop the runoff of the basin. More detail on the Millsite Reservoir plan was presented in 1931 in the Division's Bulletin 26, "Sacramento River Basin".

Throughout the 1930s and early 1940s, water development planning for the northern Sacramento Valley focused on Kennett (Shasta), Iron Canyon and Table Mountain Reservoirs on the Sacramento River. In the mid-1940s, the Corps of Engineers made a comprehensive flood control survey of the Central Valley. The Corps recommended construction of 25 reservoirs, including one at the Black Butte site on Stony Creek. The Corps' studies also included examination of a Paskenta Dam on Thomes Creek, but a program of channel and levee improvements on lower Thomes Creek was found more economical. Congress authorized both Black Butte Dam and a Thomes Creek levee project in the Flood Control Act of 1944, but the levee project was never built. The State agreed with the Black Butte Dam proposal and concluded that it should be substituted for the Millsite Dam included in the 1931 State Water Plan.

The Flood Control Act of 1944 also authorized Table Mountain Dam, but opponents of an additional dam on the Sacramento River suggested that equivalent flood protection could be achieved by a system of smaller dams on tributary streams and levee improvements. In response, the Bureau of Reclamation undertook a study of the tributary plan in 1944. That study included a 168 000 dam^3 (136,000 ac-ft) Paskenta Reservoir on Thomes Creek, but concluded that the tributary plan was not an economically feasible alternative to main stem storage.

Bulletin 3 - The State Water Plan (1957)

In 1947, the Legislature directed the State Water Resources Board to investigate California's water resources and to formulate plans for their orderly development. This investigation culminated 10 years later in the Department of Water Resources' Bulletin 3, "The California Water Plan". Bulletin 3 outlined a framework of projects to meet local and statewide water needs under "ultimate" conditions of development. This framework included the authorized 197 000 dam^3 (160,000 ac-ft) Black Butte Reservoir on Stony Creek and suggested a pumped diversion from East Park Reservoir to serve the Bear Valley area within the Cache Creek Basin. Bulletin 3 also showed an 83 000-dam^3 (67,000-ac-ft) Paskenta Reservoir on Thomes Creek, with provisions to spill excess Thomes Creek water to a 1 170 000-dam^3 (950,000-ac-ft) Newville Reservoir on North Fork Stony Creek. The water supply of Newville Reservoir would be further increased by a 61-km (38-mi) gravity diversion canal from upper Stony and Grindstone Creeks. Bulletin 3 estimated the annual yield of Paskenta Reservoir as 68 000 dam^3 (55,000 ac-ft) and of Newville Reservoir as 252 000 dam^3 (204,000 ac-ft).

Follow-up Studies to Bulletin 3: Birth of the Glenn Reservoir Concept

After completing Bulletin 3, the Department launched a major reconnaissance investigation of the north coastal area. During the early stages of that study, attention was focused on potential sites within the Sacramento Valley for storage of large quantities of water diverted from the Eel, Trinity, and Klamath River systems. Huge reservoirs would have been needed in such developments and one of the first possibilities considered was a Millsite-Newville Reservoir. This would require Newville and Millsite Dams much higher than previously considered, with the two reservoirs merging at around elevation 283 m (930 ft). The planners soon discovered that the topography at Millsite Dam site was not as favorable as at an alternative site 5 km (3 mi) upstream. The new site was named Rancheria Dam site, after the nearby Grindstone Indian Rancheria. Geologic studies and materials exploration work for the Rancheria site began in 1958-59 and the name "Glenn Reservoir Complex" was first applied at about this time. A topographic map of the entire Glenn Reservoir area was completed in 1960. The Glenn Reservoir Complex concept was formally introduced in the Department's "Progress Report on North Coastal Area Investigation", May 1961. That progress report noted that Glenn Reservoir could provide very large storage capacity (at favorable cost) for regulation of water imported from the north coastal area. It also mentioned the unique potential of Glenn Reservoir for staged construction in a variety of possible physical combinations.

Black Butte Reservoir

The Corps of Engineers completed preconstruction planning for Black Butte Reservoir in the late 1950s and awarded the main construction contract in May 1960. The 43-m- (1,140-ft-) high earthfill dam was completed in August 1963 and storage began in October of that year. Black Butte Reservoir has a gross storage capacity of 197 000 dam^3 (160,000 ac-ft) and is operated primarily for flood control. The reservoir provides an average of 70 100 dam^3 (56,800 ac-ft) of new irrigation yield annually. /More recent figures refer to a 72 800 dam^3 (59,000 ac-ft) yield./ The State contracted for the Black Butte Reservoir yield in 1960, but no definite service areas could be identified and the water supply function of Black Butte Reservoir was integrated into the CVP in October 1970. Under the present arrangement, conservation releases are diverted into the Glenn-Colusa Irrigation District system where the District's main canal crosses Stony Creek. The District receives the Black Butte Reservoir releases as part of its CVP allotment, in lieu of pumping from the Sacramento River. Virtually no Stony Creek water reaches the river during the irrigation season.

The Department of the Interior Survey of 1960

In 1960, the U. S. Department of the Interior completed a comprehensive reconnaissance survey, "Natural Resources of Northwestern California". The accompanying water development appendix (by the Bureau of Reclamation) summarized reconnaissance plans for major diversions from the north coastal rivers to the Central Valley. Among these plans was a tunnel diversion from the Middle Fork Eel River to a "Stony Creek" Reservoir (which was the same as the State's Rancheria Reservoir) at elevation 259 m (850 ft). Stony Creek Reservoir was shown linked to a Newville Reservoir via an interconnecting cut. No diversion from Thomes Creek was included in this plan, but the possibility was mentioned in conjunction with imports from the Klamath River and enlargement of Stony and Newville Reservoirs.

Tehama County's Paskenta Project Proposal (1961)

In 1961, the Tehama Flood Control and Water Conservation District commissioned Clair A. Hill and Associates to prepare a feasibility report on a Paskenta Project. The report analyzed an 80 000-dam^3 (65,000-ac-ft) Paskenta Reservoir to provide recreation, salmon enhancement on Thomes Creek, and an annual yield of 62 000 dam^3 (50,000 ac-ft) for local irrigation use. Flood control was not included in the formulation because it was anticipated that the project would eventually be integrated with a larger State or Federal Newville Reservoir. The District applied for Davis-Grunsky funds (grant of $3,090,000 and loan of $127,000) and proposed that the remainder of the project cost ($4,238,000) be obtained via a PL 984 loan. The Department of Water Resources recommended that the State participate with the local agency to build a larger reservoir that could produce some SWP yield. However, the local proposal stalled and expired due to problems with overlap of the proposed service area with the Corning Canal service area and pessimism of the Department of Fish and Game over the projected fishery enhancement benefits.

Bulletin 136 (1964)

The Department completed its 7-year reconnaissance investigatiion and published Bulletin 136, "North Coastal Area Investigation", in 1964. Bulletin 136 concluded that a project drawing surplus water from the upper Eel River would be the most favorable initial north coastal development for augmenting SWP supplies in the Delta; it noted that upper Eel River water could be routed to the Delta either via Clear Lake or via elements of Glenn Reservoir. (Additional studies were begun in 1964 to select the optimum project formulation.) Bulletin 136 also concluded that the Paskenta-Newville Unit was one of the more favorable remaining potential developments in the Sacramento Valley and that it could be integrated later with facilities to import water from the north coastal area. The bulletin outlined a plan with an 86 000-dam^3 (70,000-ac-ft) Paskenta Reservoir diverting surplus Thomes Creek flows to a 1 390 000-dam^3 (1,130,000-ac-ft) Newville Reservoir; annual yield was estimated as 247 000 dam^3 (200,000 ac-ft). This plan would function entirely with local runoff from Thomes and North Fork Stony Creeks, and Bulletin 136 suggested it as a possible early-stage addition to the State Water Resources Development System.

Bulletin 150 (1965)

While the North Coastal Area Investigation was underway, the Department was conducting a similar comprehensive study of the northern Sacramento Valley area. That 6-year study concluded in March 1965 with publication of Bulletin 150, "Upper Sacramento River Basin Investigation". Bulletin 150 concluded that the Iron Canyon Project was not economically justified, but that four tributary reservoirs were (Hulen, Dippingvat, Millville, and Paskenta). The Paskenta Plan called for a 130 000-dam^3 (105,000-ac-ft) reservoir for flood control, recreation, salmon enhancement, and water supply; local irrigation yield was calculated as 64 000 dam^3 (52,000 ac-ft) per year and an additional 42 000 dam^3 (34,000 ac-ft) of yield would be available to augment flows in the Delta. Bulletin 150 recommended feasibility studies of the four selected tributary reservoirs.

A Push Begins for the Paskenta-Newville Unit

The December 1964 flood caused substantial damage along Thomes Creek and heightened interest in the potential developments described in Bulletins 136 and 150. In June 1965, the Bureau began a feasibility study of the Paskenta-Newville Unit and early in 1966, the State Legislature adopted resolutions requesting the Department to consider early construction of the unit as a joint State-Federal venture. In September 1966, the California State-Federal Interagency Group formally assigned responsibility for feasibility planning of the Paskenta-Newville Unit to the Bureau; the Department was assigned responsibility for Rancheria Reservoir and the Corps was assigned the Cottonwood Creek Project. In December 1966, the Department responded to the Legislature, recommending support for Federal authorization of the Paskenta-Newville Unit, with State participation under the Water Supply Act of 1958 or some similar arrangement.

The State Reclamation Board contracted with the Ralph M. Parsons Company in 1967 to reassess the potential of tributary reservoirs to control Sacramento River floods. Parsons' November 1968 report recommended 13 tributary projects as an alternative to a Butte Basin Bypass. Early construction was urged for five, including the Paskenta-Newville Unit and the Cottonwood Creek Project.

Eel River Studies of the Late 1960s

In August 1967, the Department completed the first phase of its advance planning studies on Eel River development with publication of Bulletin 171, "Upper Eel River Development--Investigation of Alternative Conveyance Routes". The report concluded that an eastward diversion from the Middle Fork Eel River was superior by engineering and economic standards to the alternative Clear Lake route. The "most likely plan of development" was identified as a Dos Rios Reservoir on the Middle Fork Eel River with a tunnel diversion to features of the Glenn Reservoir Complex. The Corps of Engineers was in the midst of an Eel River planning study, which was concluding that a large Dos Rios Reservoir should be the initial basin development. On October 5, 1967, the Department and Corps executed a memorandum of understanding providing for the Corps to construct Dos Rios Dam, with the Department contracting for conservation storage under the Water Supply Act of 1958. The Department was to construct the conveyance facilities from Dos Rios Reservoir to the Sacramento Valley.

Then, just as all the pieces appeared to be coming together, a rapid unravelling process began. Bulletin 171 met with swift, determined opposition, particularly from Lake County, where Eel River imports had long been viewed as a panacea for Clear Lake's water quality, water supply, and flooding problems. This opposition led the California Water Commission to recommend (in August 1968) that the Department perform additional studies relating to the routing issue. Meanwhile, the Corps completed its report on the Dos Rios Project and transmitted it for State review. Dos Rios Dam became the subject of widespread controversy, primarily over the prospective inundation of Round Valley and the displacement of some 1,400 people (including many of the 350 residents of the Round Valley Indian Reservation). In May 1969, Governor Reagan asked the Department to make further analyses of alternatives that would avoid flooding Round Valley. In response, the

Department prepared Bulletin 172, "Eel River Development Alternatives", in December 1969. Bulletin 172 presented six alternatives, three of which involved eastward diversions from the Middle Fork Eel River to a Rancheria Reservoir. (Earlier Eel River studies had considered a diversion to Newville Reservoir, but a longer, more costly tunnel would be involved and the Paskenta-Newville Unit being planned by the Bureau would not have had additional storage potential for use with an Eel River development.) Following Bulletin 172, the Department resumed its studies of the Eel River diversion routing issues, as recommended by the California Water Commission in 1968. However, the scope of study was expanded to cover the broader question of how development of the Eel River might best be accomplished.

The Bureau of Reclamation Reports on Paskenta-Newville (1971)

In August 1971, the Bureau released a status report on its studies of the Paskenta-Newville Unit of the Central Valley Project. The Bureau report outlined a plan of development as follows:

	Paskenta	Newville
Dam height	71 m (233 ft)	119 m (390 ft)
Dam crest elevation	312 m (1,023 ft)	301 m (980 ft)
Normal reservoir elevation	307 m (1,006 ft)	297 m (975 ft)
Reservoir storage	160 000 dam^3	3 684 000 dam^3
" "	(130,000 ac-ft)	(2,987,000 ac-ft)

* * * * * * * * * * * *

Annual Project water supply		
Local irrigation	53 000 dam^3	(43,000 ac-ft)
Mandatory releases	33 000 dam^3	(27,000 ac-ft)
Export to CVP	493 000 dam^3	(400,000 ac-ft)
	579 000 dam^3	(470,000 ac-ft)

The Bureau calculated a benefit-cost ratio of 1.46 to 1 (based on direct benefits), with 83 percent of the benefits attributed to CVP export yield. The large export yield was derived by assuming that the Paskenta-Newville Unit would be operated integrally with the remainder of the CVP system, but it would be reserved for dry period peaking supply only. (Under current Department criteria, the water supply of the Paskenta-Newville Unit would not justify nearly as much storage as the 1971 Bureau formulation indicated--even if the unit were operated at the extremely low load factor that the Bureau used.) The Bureau report concluded that the Paskenta-Newville Unit would be justifiable, but noted that additional CVP water supply would not be needed until the late 1990s; accordingly, it recommended that further study of the project be deferred until a need could be demonstrated for its water supply.

Wild Rivers Legislation: Termination of Eel River Studies

In the wake of the Dos Rios Dam controversy, several bills were introduced into the 1971 and 1972 Legislatures relating to possible wild river status for the major north coastal streams. Late in 1972, the Legislature passed a bill establishing the California Wild and Scenic Rivers System, including most of the Klamath, Trinity and Eel Rivers and their major tributaries. The act forbids construction of major dams but authorized the Department to continue studies of potential Eel River developments. (The Department is also directed to report to the Legislature on the need for water supply and flood control development on the Eel River; this report is due in March 1985.) In response to the Wild and Scenic Rivers Act, the Department wrapped up its major Eel River studies in the December 1972 report, "Alternative Eel River Projects and Conveyance Routes". That report described five alternative Eel River diversion plans that would involve Rancheria Reservoirs of from 1 550 000 to 5 900 000 dam^3 (1,260,000 to 4,780,000 ac-ft) capacity. Some environmental studies were continued in the Eel River area on a gradually declining basis, but these were completely terminated on June 30, 1977.

Reanalysis of Sacramento Valley Alternatives (1970-75)

As the prospects for Eel River development began to dim in the early 1970s, the Department began a reexamination of potential alternative sources of water within the Sacramento River Basin. The potential for additional water development in the upper basin had long been recognized through studies of Iron Canyon Reservoir and various packages of tributary reservoirs. By the 1970s, an Iron Canyon or other main stem development was clearly out of the question for economic and environmental reasons, so attention shifted to offstream storage and tributary developments. In February 1975, the Department completed a progress report entitled, "Major Surface Water Development Opportunities in the Sacramento Valley". This report describes four plans in detail:

1. The Tributary Storage Plan, a composite of the two reservoirs of the Corps' Cottonwood Creek Project, Millville Reservoir on South Cow Creek, Wing Reservoir on Inks Creek (with diversions from Battle and Paynes Creeks), Schoenfield Reservoir on Red Bank Creek, Gallatin Reservoir on Elder Creek, Newville Reservoir (with diversion from Thomes Creek), and Rancheria Reservoir.

2. The Tuscan Buttes Reservoir--River Diversion Plan, a large offstream storage development on Inks and Paynes Creeks (a plan devised by the Bureau in the late 1960s).

3. The Glenn Reservoir--River Diversion Plan, a new plan to couple the large storage potential of Glenn Reservoir to the surplus water of the Sacramento River.

4. The Colusa Reservoir--River Diversion Plan, an expanded version of the Bureau's plan to use the Tehama-Colusa Canal to convey surplus Sacramento River water for pumping to Sites Reservoir.

The report also briefly describes the prospects for an Enlarged Shasta Reservoir and an Enlarged Lake Berryessa offstream storage plan.

The Glenn Reservoir Plan outlined in the 1975 report was the first formal consideration of using Glenn Reservoir for offstream storage of Sacramento River water. The plan featured a 10 700 000-dam^3 (8,700,000-ac-ft) reservoir, supplied via a 142 m^3/s (5,000-cfs) diversion from the existing Lake Red Bluff. Total static pumping lift would be about 233 m (765 ft). Based on energy values at the time, off-peak pumping was selected and the possibility of pumped-storage power operation was considered. To allow off-peak pumping, a 40-km (25-mi) low-level gravity canal was selected for the diversion from Lake Red Bluff; it would terminate at a Kirkwood Forebay east of Black Butte Reservoir. From Kirkwood Forebay, the 1975 plan envisioned a single off-peak pump lift to Black Butte Reservoir. The flood control reservation in Black Butte Reservoir would be transferred to Glenn Reservoir and Black Butte was to have been held at a constant level near its design maximum pool elevation by installing gates in its presently ungated spillway. (This alteration of Black Butte Reservoir was first suggested by the Bureau of Reclamation in the 1960 appendix to "Natural Resources of Northwestern California"; this was before construction of Black Butte Reservoir had begun.) From the stabilized Black Butte Reservoir, a deep cut would lead upstream to another off-peak pumping plant at the toe of Newville Dam. Both the Black Butte and Newville Plants would have been equipped with reversible pumping-generating units to produce hydroelectric energy during times of water release.

The 1975 report made a major departure from the previous Glenn Reservoir and Paskenta-Newville Unit plans, which had used Paskenta Reservoir to divert surplus flows from Thomes Creek to the North Fork Stony Creek drainage area. In 1970, Department of Fish and Game studies showed that Paskenta Reservoir would inundate a critical wintering area for a large population of migratory deer. To avoid this, the Department of Water Resources devised a Thomes Creek diversion plan using a small reservoir about 8 km (5 mi) upstream from Paskenta Dam site and a 3.9-km (2.4-mi) canal. This diversion alternative, probably quite similar to that considered by the USGS in about 1903, was incorporated into the 1975 report.

The Delta Reappraisal: Bulletin 76 (1975-78)

Following completion of the 1975 report, the Department shifted its planning emphasis away from specific surface reservoir developments; no follow-up studies were programmed for Glenn Reservoir in 1975-76 or 1976-77. Early in 1975, a reappraisal was begun of the Peripheral Canal and surrounding issues. As this reappraisal progressed, it was recognized that resolution of the Delta problem would require a comprehensive plan of physical and institutional measures to reduce water demands, augment water supplies, protect the Delta, and provide for water transfer through the

Delta. Glenn Reservoir was identified as one of the more promising possibilities for development of additional water supplies in Northern California and reconnaissance-level planning studies of the Glenn Plan were resumed at a modest level in July 1977. Later that year, the Glenn Reservoir planning studies were expanded and incorporated into the newly formed comprehensive State Water Project Future Supply Program.

The results of the Department's reappraisal of Delta and related issues were reported in Bulletin 76, "Delta Water Facilities", July 1978. Bulletin 76 sets forth a total program of Delta protection and water transfer measures, water conservation and recycling, and surface and ground water storage. The program's key components north of the Delta are the Cottonwood Creek Project and a Glenn Reservoir--River Diversion Unit adopted directly from the 1975 report on Sacramento Valley water development opportunities. The Colusa Reservoir--River Diversion Plan from the 1975 report is included as a "partial alternative" to the Glenn Reservoir Plan. Major efforts were mounted for authorization of the Bulletin 76 plan by the State Legislature in both 1978 and 1979, but neither was successful.

Recent Glenn Reservoir Studies (1976-79)

While Bulletin 76 was being prepared, additional planning studies were being conducted of offstream storage plans involving Glenn, Colusa, and enlarged Berryessa Reservoirs. These studies led to several significant changes in the Glenn Reservoir Project configuration:

1. Because of the diminished prospects for low-priced off-peak energy, the plan was reformulated on the basis of continuous pumping rather than off-peak pumping.

2. A more westerly route (using two pumping plants) was selected for the diversion canal between Red Bluff and Black Butte Reservoir; this route would have less impact on agricultural land and existing development. Such higher routes were not previously feasible due to the need for regulatory storage to accommodate off-peak pumping.

3. Because of the realignment of the Red Bluff-Black Butte Canal, an entirely new and separate conveyance/power recovery system had to be added between Black Butte Dam and the Sacramento River.

4. In response to concerns about dam safety aspects, the plan was adjusted so that Black Butte Reservoir would be maintained at its spillway crest elevation of 144 m (474 ft) rather than maximum pool elevation of 155 m (510 ft).

5. With the lower Black Butte Reservoir level, a very deep excavation would be needed to reach the Newville Pumping-Generating Plant. To avoid this, an additional dam and pumping-generating plant were proposed on North Fork Stony Creek. The new site was named "Tehenn" because it is nearly on the Tehama-Glenn County line.

All of these changes were incorporated into an October 1978 memorandum. In contrast to previous studies, the 1978 memorandum presented a range of "example" plans rather than a single formulation. This approach reflected the uncertainty over the operational criteria a Glenn Reservoir or similar plan would have to meet. For example, if the development were to be dedicated to meeting only extreme dry period needs, it would have a very large storage capacity (usually near full), large critical period yield, and very small average yield. At the other extreme, if the development were to be operated to maximize average yield, the storage capacity would be relatively small (usually near empty), and the critical period yield would be minimal. An entire spectrum of other possibilities exist between these extremes; three such intermediate possibilities were examined in the 1978 memorandum. These resulted in the following three example Glenn Reservoir formulations:

Reservoir elevation, m (ft)	290 (952)	300 (984)	305 (1,002)
Gross reservoir storage, dam^3	7 576 000	9 498 000	10 628 000
" " " (ac-ft)	(6,142,000)	(7,700,000)	(8,616,000
Dry period yield, dam^3/yr	972 000	1 246 000	1 410 000
" " " (ac-ft/yr)	(788,000)	(1,010,000)	(1,140,000)
Average yield (1922-71), dam^3/yr	778 000	588 000	422 000
" " " (ac-ft/yr)	(631,000)	(477,000)	(342,000)
Average yield ÷ dry period yield (K)	0.80	0.47	0.30

The Thomes-Newville Plan Concept (1978)

As serious attention shifted to the Glenn Reservoir--River Diversion Project during Bulletin 76 studies, local opposition arose over the prospective inundation of the town of Elk Creek, the Grindstone Indian Rancheria, and the surrounding ranches. Late in 1978, the Department examined the possibility of omitting the Rancheria Reservoir compartment, which would overcome most of the objections. Three such projects were considered:

Plan 1: A Newville Reservoir, with diversion from Thomes Creek (essentially the project described by the USGS in 1903). Although this project would be hydrologically similar to the Bureau's Paskenta-Newville Unit, current formulation criteria would lead to a relatively small Newville Reservoir with a dry-period yield of only about 110 000 dam^3 (90,000 ac-ft) per year (for a "K factor" of 0.47).

Plan 2: An expanded version of Plan 1, with the water supply augmented by pumping surplus flows from Stony Creek. This is the plan featured in this report. As shown in Chapter 2, it would have a Newville Reservoir capacity of 2 050 000 dam^3 (1,662,000 ac-ft) and a dry period yield of about 273 000 dam^3 (221,000 ac-ft) per year (for K = 0.47).

Plan 3: An expanded version of Plan 2, with the water supply augmented further by winter diversions of Sacramento River water via the Bureau's Tehama-Colusa Canal. This would justify a larger Newville Reservoir, approaching the potential of the site. However, Plan 3 would preempt the Bureau's planned West Sacramento Canal Unit, which would use the Tehama-Colusa Canal to supply a Sites Reservoir in Colusa County.

When the studies of the reduced versions of the Glenn Project were completed in November 1978, several advantages of the preceding Plan 2 became apparent. First, it would avoid the major controversies associated with the full-scale Glenn Reservoir proposal. Second, it would not preempt eventual development of a Glenn Reservoir. Third, it would avoid many institutional obstacles because it would not interfere with existing or planned development of others. Fourth, it would have a relatively small environmental impact compared to the full-scale Glenn Reservoir Plan. Finally, because of its size and relative simplicity, it should be easier to get the necessary authorization, approvals, and funding so it could be in operation sooner. Accordingly, the Department decided to increase its planning emphasis on Plan 2, considering it as a first stage of the entire Glenn Reservoir Plan.

This marked the beginning of the studies covered by this report. Subsequently, revised projections of water demands and supplies showed that a development of the scale of the Glenn Reservoir Plan would not be needed until beyond the turn of the century. It is now considered that a full-scale Glenn Reservoir Plan and the enlargement of Shasta Reservoir will be considered as alternatives in the planning studies of the 1980s. Meanwhile, the Department will move ahead on the smaller Thomes-Newville Plan, considering it as an independently viable development in its own right and not necessarily as a first stage of a larger plan.

APPENDIX G

THOMES-NEWVILLE AND GLENN RESERVOIR PLANS--
CHRONOLOGICAL BIBLIOGRAPHY

APPENDIX G. THOMES-NEWVILLE AND GLENN RESERVOIR PLANS--CHRONOLOGICAL BIBLIOGRAPHY

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Preliminary designs and cost estimates for a range of sizes of Glenn Reservoir features and various alternative features.

California Department of Fish and Game. October 1979. Fish and Wildlife Considerations in State Water Project Planning.

Includes study recommendations for Glenn Reservoir.

Foundation of the California State University, Sacramento. October 1979. A Phase I Cultural Resources Planning Summary and Preliminary Field Work Proposal for Three Reservoir Locations in Central California: Los Vaqueros (Contra Costa County), Los Banos Grandes (Merced County), and the Glenn Complex (Newville and Rancheria Reservoirs) (Glenn and Tehama Counties).

Stout, Heather A., and Paul F. Collins. October 1979. Preliminary Survey of the Glenn-Tehama Reservoir Site Floristics.

Results of a 1979 survey of native plants within the Glenn Reservoir area; no rare or endangered plants were found but this limited study does not guarantee that none are present.

California Department of Water Resources, Division of Design and Construction. December 1979. SWP Future Supply Program, Glenn Reservoir Complex, Reconnaissance Study and Cost Estimates for Thomes Creek Diversion Facilities. Memorandum Report.

Preliminary designs and cost estimates for facilities to divert water from Thomes Creek to Newville Reservoir. Covers three sizes to divert 85, 184, and 283 m^3/s (3,000, 6,500, and 10,000 ft^3/s).

Earth Sciences Associates. January 1980. Seismic and Fault Activity Study, Proposed Glenn Reservoir Complex.

Results of a study conducted under contract to the Department of Water Resources. Concludes that there has been no recent faulting near the sites of structures. Concludes Stony Creek fault would be most critical for establishing design criteria. Recommends further studies.

Dolmage Campbell and Associates, Ltd. February 1980. Ridge Project, Review of 1979 Fieldwork. Letter Report.

Consultant's report on Rocky Ridge; follow-up to June 1979 report; gives final conclusions.

California Department of Water Resources, Division of Planning. March 1980. Status of Water Conservation and Water Supply Augmentation Plans for the State Water Project. Bulletin 76-80. Manuscript.

Appraisal and comparison of all plans under consideration to match State Water Project supplies and demands, including the Thomes-Newville and Glenn Reservoir Plans.

California Department of Water Resources, Division of Planning. April 1980. California Central Valley Natural Flow Data.

Includes calculated natural (unimpaired) flows of Stony Creek at Black Butte Dam site for 1920-21 through 1977-78.

California Department of Water Resources, Northern District. September 1980. Transmittal of Newville Project's Byrne Act Assessment. Memorandum Report.

Data from which to assess impacts of construction of the Thomes-Newville Plan on local populations, police and fire protection, schools, sanitation and health services.

California Department of Water Resources, Division of Design and Construction. October 1980. Reconnaissance Study and Cost Estimate for Thomes-Newville Project. Memorandum Report.

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CONVERSION FACTORS

Quantity	To Convert from Metric Unit	To Customary Unit	Multiply Metric Unit By	To Convert to Metric Unit Multiply Customary Unit By
Length	millimetres (mm)	inches (in)	0.03937	25.4
	centimetres (cm) for snow depth	inches (in)	0.3937	2.54
	metres (m)	feet (ft)	3.2808	0.3048
	kilometres (km)	miles (mi)	0.62139	1.6093
Area	square millimetres (mm^2)	square inches (in^2)	0.00155	645.16
	square metres (m^2)	square feet (ft^2)	10.764	0.092903
	hectares (ha)	acres (ac)	2.4710	0.40469
	square kilometres (km^2)	square miles (mi^2)	0.3861	2.590
Volume	litres (L)	gallons (gal)	0.26417	3.7854
	megalitres	million gallons (10^6 gal)	0.26417	3.7854
	cubic metres (m^3)	cubic feet (ft^3)	35.315	0.028317
	cubic metres (m^3)	cubic yards (yd^3)	1.308	0.76455
	cubic dekametres (dam^3)	acre-feet (ac-ft)	0.8107	1.2335
Flow	cubic metres per second (m^3/s)	cubic feet per second (ft^3/s)	35.315	0.028317
	litres per minute (L/min)	gallons per minute (gal/min)	0.26417	3.7854
	litres per day (L/day)	gallons per day (gal/day)	0.26417	3.7854
	megalitres per day (ML/day)	million gallons per day (mgd)	0.26417	3.7854
	cubic dekametres per day (dam^3/day)	acre-feet per day (ac-ft/day)	0.8107	1.2335
Mass	kilograms (kg)	pounds (lb)	2.2046	0.45359
	megagrams (Mg)	tons (short, 2,000 lb)	1.1023	0.90718
Velocity	metres per second (m/s)	feet per second (ft/s)	3.2808	0.3048
Power	kilowatts (kW)	horsepower (hp)	1.3405	0.746
Pressure	kilopascals (kPa)	pounds per square inch (psi)	0.14505	6.8948
	kilopascals (kPa)	feet head of water	0.33456	2.989
Specific Capacity	litres per minute per metre drawdown	gallons per minute per foot drawdown	0.08052	12.419
Concentration	milligrams per litre (mg/L)	parts per million (ppm)	1.0	1.0
Electrical Conductivity	microsiemens per centimetre (uS/cm)	micromhos per centimetre	1.0	1.0
Temperature	degrees Celsius (°C)	degrees Fahrenheit (°F)	$(1.8 \times °C)+32$	$(°F-32)/1.8$