University of Minnesota
St. Anthony Falls Hydraulic Laboratory

Project Report No. 129

MODEL STUDY OF PROPOSED EXPANSION - ENCINA POWER PLANT
SAN DIEGO GAS AND ELECTRIC COMPANY

by

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PIONEER SERVICE & ENGINEERING COMPANY
Chicago, Illinois

September 1971
Minneapolis, Minnesota

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San Diego Gas and Electric Company

I. INTRODUCTION

Because of present and anticipated future power demands, it is now necessary for the San Diego Gas and Electric Company to expand the generating capacity at its Encina Power Plant. In order to provide sufficient cooling water for the expanded plant, the existing inlet tunnels and associated structures must be modified. Because the complexity of the structure made it essentially impossible to analyze accurately the flow pattern and associated head losses mathematically, a model study was undertaken at the St. Anthony Falls Hydraulic Laboratory. The hydraulic model was built and tested to assist in the development of a satisfactory design and to verify the final design. In order to obtain answers to the questions raised, the following tests were conducted:

1. Scheme A-1: Units 1, 2, 3, and 4 operating, channel 4 supplying water to unit 4 with the vane and pier configuration shown in Fig. 1.
2. Scheme A-2: Units 1, 2, 3, and 4 operating, channels 3 and 4 supplying water to unit 4 with the pier configuration shown in Fig. 2.
3. Scheme B-1: All units operating with channels 3 and 4 supplying water to units 4 and 5 as shown in Figs. 2 and 3.
4. Scheme B-2: All units operating with channels 3 and 4 plus a gated opening 1-1/2 ft high at the end of channel 2 supplying water to units 4 and 5 as shown in Fig. 4.
5. Scheme B-3: All units operating with channels 3 and 4 plus a 10 ft diameter pipe as shown in Fig. 5 supplying water to units 4 and 5.

Throughout the entire series of tests, channel 1 was used to supply water to unit 1 and channel 2 was used to supply water to units 2 and 3. In addition to the basic operating tests, a series of tests measuring the effect of cleaning the east or west intake tunnel was also conducted. Included in this series of tests were:

1. Cleaning the east or west channel units 1, 2, 3, and 4, operating as shown in Fig. 2.

2. Cleaning the east or west channel units 1, 2, 3, and 4 at full power and with unit 5 at 0, 25, 50, 75, and 100 per cent of full power, using the geometry shown in Fig. 2.

As a result of the tests, a satisfactory design to supply the water to unit 4 was developed. A design that is adequate to supply the combined flow requirements of units 4 and 5 was also developed. It is strongly recommended that the modifications to the existing structure which are necessary to provide water for the future unit 5 be incorporated in the current construction phase.

II. DESCRIPTION OF MODEL

A 1:12 scale model of a portion of the cooling water intake channels for the Encina Power Plant was constructed based on drawings provided by Pioneer Service and Engineering Company. The general layouts of the various alternate models tested are shown in Figs. 1 to 5. Numerous changes in the geometry of the models were made as the various testing programs progressed. The geometries shown in Figs. 1 to 5 are the final designs and were judged to be the most satisfactory from the hydraulic point of view.

The overall model included several features to facilitate the test program. These included wall pressure taps at the locations shown in Fig. 6. There were flow meters and control valves in all discharge lines so that any desired flow combination could be obtained. A dye injection system was also provided to aid in visually observing the flow pattern and obtaining proper flow distribution through the unit 4 screen structure.

For a structure in which the free water surface is exposed to atmospheric pressure, dynamic similarity is obtained when the model-prototype relationships are determined by the Froude law of similarity.

The following relationships for velocity, discharge, and pressure in terms of the length scale ratio ($L_r = 12$) are then obtained:

$$\text{Velocity ratio} = L_r^{1/2}$$

$$\text{Flow ratio} = L_r^{5/2}$$

$$\text{Pressure ratio} = L_r$$

By utilizing the above equations, the model discharge can be determined and pressures and velocities as measured in the model can be readily translated to the prototype values. All values reported in the following sections are prototype values, not model values.

III. EXPERIMENTAL STUDIES

Scheme A - Units 1, 2, 3, and 4 operating

1. Channel 4 only supplying water to unit 4 (431 cfs)

Initially tests were conducted using the geometry shown schematically in Fig. 1, except no vanes were installed. For this condition the flow in channel 4 has sufficient momentum parallel to the centerline of the channel so that essentially no flow turned and passed through the openings cut through the center wall. As a result, all the flow passed inward to the pumps through the right screen area (where point 5 is located in Fig. 6) and a reverse flow (outward instead of inward) passed through the left screen area. Since one of the primary objectives of the study was to achieve uniform flow inward to the pumps through both the screens, the initial geometry tested was unsatisfactory.

After numerous tests, the wall opening and valve configuration shown in Fig. 1, Detail #1, were found to produce essentially equal flow at all depths of flow through both the left and right screens supplying water to unit 4. The determination of the equality of the flow was based on the visual observation of the movement of the dye cloud injected simultaneously into the flow entering the two screen areas. Other methods of determining the flow distribution (propeller meters and velocity profile measurement) were also tried, but the visual observation method proved to be superior to the other methods. After a vane configuration was developed which correctly distributed the flow for all probable depths of flow in the channel as well as minimizing the head loss, head loss data were obtained.

For this operational mode no problems in either total head loss or general flow pattern were observed. The flow distribution to the unit 4 screen structure is satisfactory if the valve configuration shown in Fig. 1 is used.

The pressure heads (in prototype dimensions) measured throughout the structure were obtained using wall pressure taps at the 16 points shown in Fig. 6. The results for this series of tests are given in Table I. The "sea level" at which a particular test was conducted is the water surface elevation at point 15; e.g. a value of +4 at point 15 means 4 ft above mean sea level. The surface elevations at points 1 through 7 were identical.

TABLE I

Measurements for Scheme A-1

Water Surface Elevations Relative to Normal Sea Level in Feet										H_L*
Points 1-7	8	9	10	11	12	13	14	15	16	Feet
3.8	4	4	3.7	3.9	3.9	3.9	3.7	3.9	3.9	0.1
1.5	1.8	2	1.5	1.8	1.8	1.8	1.6	1.8	1.8	0.3
-1.7	-1.1	-.9	-1.8	-1.1	-1.1	-1.1	-1.5	-1.1	-1	0.6
-3.2	-2.2	-3.2	-3.2	-2.4	-2.4	-2.4	-3	-2.3	-2.4	0.9
-5.9	-4.5	-4.2	-5.9	-4.3	-4.4	-4.3	-5.2	-4.3	-4.3	1.6
-7.2	-5.4	-5.1	-7.3	-5.3	-5.4	-5.3	-6.5	-5.3	-5.3	1.9

*Head loss is determined by subtracting the water surface level at point 4-5 from that at point 15; note that the water surface is lower at point 10 than at points 1 through 7. This does not represent a greater head loss, but rather a lower surface elevation caused by a high velocity at point 10. This characteristic existed at points 9 and/or 10 through all the tests.

The head loss between point 15 (Fig. 6) and the intake screens of unit 4 as a function of water surface elevation at point 15 is shown in Fig. 7. The mean annual minimum water level at point 15 shown on Fig. 7 can be established by subtracting the average yearly low tide and the decrease in head through the intake structure from mean sea level. Thus the mean annual lowest water surface elevation at 15 would equal zero sea level minus mean annual lowest tide of 4.5 ft minus 0.6 ft head decrease (taken from an estimate provided by Mr. A. Trott of Pioneer) or -5.1 ft. The maximum level would be mean annual highest tide minus the decrease in head or +4.0 ft elevation. The extreme low water level for the period of record would be 0.8 ft lower than the -5.1 value.

For the mean daily tidal range the head loss will be 1.20 ft and for the mean annual lowest tide the head loss will be 1.9 ft. It should be noted that if a low tide equal to the extreme low tide on record (Dec 1932 and 1933) would occur, the head loss would be 2.3 ft. The 2.3 ft of head loss would not cause a major problem; thus throughout the entire tidal range head loss should not cause any operational problems.

Because the screens were not modeled, all head loss values given in this report do not include screen losses. Therefore, these must be taken into consideration where appropriate.

The head loss between point 15 and units 1, 2, and 3 is very small, a maximum of 0.1 ft for unit 3. However, it should be noted that this head loss value was obtained only when both units 2 and 3 were being supplied by channel 2, an expedient to save time and costs in building and testing the model. If unit 2 is supplied by channel 2 and unit 3 by channel 3, as shown in Fig. 1 and proposed for this alternate, then the head loss for units 2 and 3 would be reduced, making it less than 0.1 ft.

2. Channels 3 and 4 supplying water to unit 4

If both channels 3 and 4 are used to supply unit 4, then the geometry shown in Fig. 2 should be used. This geometry accomplishes two important functions. First the head loss is significantly less than when channel 4 alone is used to supply unit 4. Second, unit 5 could be added at a later date with little or minimum power outages necessary for construction.

The head loss as a function of water level at point 15 for this condition is given in Table II; it is also shown in Fig. 7. By comparison one can see that for the mean daily low tide (-3.5 ft at point 15 including intake structure head loss) the head loss savings would be 1.00 ft of head (1.2 ft loss at -3.5 ft elevation using channel 4 only and 0.2 ft loss using channels 3 and 4). For lower tides the savings would be even larger.

TABLE II

Measurements for Scheme A-2

Water Surface Elevations Relative to Normal Sea Level in Feet										H_L Feet
Points 1-7	8	9	10	11	12	13	14	15	16	
2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	<0.1
-3.2	-3.2	-3.2	-3.2	-3.1	-3.1	-3.1	-3.2	-3.1	-3.1	0.1
-6.3	-6.3	-6.4	-6.4	-6.1	-6.2	-6.2	-6.3	-6.1	-6.1	0.2

Another important consideration may be the cleaning of channels 3 and 4. If two channels are used to supply unit 4, then one could be cleaned while the other supplies sufficient water on a temporary basis to maintain full power production. In contrast, if only one channel supplies unit 4, and the channel is cleaned, a 100 per cent temporary power outage on unit 4 would be necessary.

Based on the overall improvement in flow characteristics as well as other considerations, it is strongly recommended that the alternate shown in Fig. 2 for supplying water to unit 4 be selected.

3. Cleaning east and west intake tunnels

The head loss that occurs during the cleaning of either the east or west intake tunnel is shown in Fig. 8. These data were obtained using channels 3 and 4 to supply water to unit 4. If the alternate scheme (Fig. 1 geometry) of supplying water to unit 4 is selected, it is anticipated that the head loss would be essentially the same as that shown in Fig. 7 for flow with vanes.

No flow problems were encountered during the simulated east and west tunnel cleaning tests. Thus it is not anticipated that a power reduction would be required during the actual prototype cleaning operation, except as noted earlier.

Scheme B - Units 1, 2, 3, 4, and 5 operating

Once the testing of both units 4 and 5 operating simultaneously was begun, it became obvious that the total flow for both units could not be supplied by channel 4 alone. Tests were then undertaken to determine (1) if the geometry shown in Figs. 2 and 3 could be used satisfactorily and (2) if not, what would be the "best geometry" for satisfactory operation of units 4 and 5.

1. Channels 3 and 4 supplying water to units 4 and 5

a. Flow with guide vanes. Early tests indicated that this geometry might be satisfactory, but it was observed that while at times the flow through both the left and right side of the screen area would be essentially equal, at other periods of time the flow through the left side would decrease significantly and the flow through the right side would show a corresponding increase. The reverse tendency was not observed.

It should also be noted that because of the higher velocities associated with both units 4 and 5 operating, the turbulence level in the unit 4 screen area for Fig. 3 geometry is somewhat greater due to the presence of the vanes than it is for the geometry shown in Fig. 2.

The water surface elevations for various depths of flow using Fig. 3 geometry are given in Table III. The head loss at a function of water surface elevation at point 15 is shown in Fig. 9.

TABLE III

Measurements for Scheme B-1-a

Water Surface Elevations Relative to Normal Sea Level in Feet														H_L Feet
Points 1-2	3	4-5	6	7	8	9	10	11	12	13	14	15	16	
4.6	4.5	4.6	4.6	4.5	5.1	4	3.9	5.1	5.1	4.7	4.7	5.1	5.1	0.4
3.5	3.5	3.4	3.5	3.3	4.0	3.5	3.6	4	4.0	3.6	3.6	4.1	4.1	0.5
-1.1	-1.5	-1.1	-1.2	-1.7	-.2	-1.4	-1.4	-.2	-.2	-1	-1	-.3	-.4	0.8
-2.4	-3.1	-2.4	-2.6	-3.0	-1.3	-2.5	-2.5	-1.2	-1.2	-2.2	-2.2	-1.3	-1.4	1.1
- 3	-7.3	- 3	-6.7	-7.0	-3.7	-7.4	-7.2	-3.7	-3.7	-5.7	-5.7	-3.9	-4.2	2.4

To obtain the decrease in head from the ocean intake to point 15 for the increase in flow resulting from unit 5 being added a computation similar to that made by Mr. A. Trott for Scheme A-1 was made, except the total flow was increased by 720 cfs (flow for unit 5) to 1480 cfs. The resulting decrease in total head from entrance losses, expansion losses, velocity head at point 15, and friction along the intake tunnel is 2.25 ft. As a result of the head loss and tidal action, the mean daily low water surface elevation at point 15 would be 5.12 ft below mean sea level (2.87 ft mean daily tide +2.25 ft head loss). Any alternate that is to be seriously considered must be capable of functioning satisfactorily, at the very least, 5 ft below mean sea level. This alternate will not; therefore, it cannot be recommended.

b. Flow without guide vanes. The water surface elevations for various depths of flow using Fig. 2 geometry are given in Table IV. The head loss as a function of water surface elevation at point 15 for the data in Table IV plus some additional data is plotted on Fig. 9 also. Although the turbulence level and head loss is slightly reduced compared to the previous alternate, neither this alternate nor the previous alternate can supply sufficient water for units 4 and 5 operating at 100 per cent power if the water surface at point 15 is more than 4 feet below mean sea level. Since a workable alternate must be capable of functioning at least 5 ft below mean sea level, this alternate is also not satisfactory.

TABLE IV

Measurements for Scheme B-1-b

Water Surface Elevations Relative to Normal Sea Level in Feet														H_L Feet
Points 1-2	3	4-5	6	7	8	9	10	11	12	13	14	15	16	
3.6	3.4	3.6	3.5	3.3	3.9	3.5	3	4	4	3.7	3.4	4	3.9	.4
2.7	2.8	2.8	2.8	2.6	3.2	1.6	2.3	3.1	3.1	2.7	2.4	3.0	2.9	.2
1.5	1.3	1.5	1.4	1.2	1.9	1.4	.7	2	2	1.6	1.2	2	1.9	.5
-.6	-1	-.6	-.5	-.9	.1	-.5	-1.3	0	.1	-.4	-.9	0	-.1	.6
-.7	-1	-.7	-.7	-1.2	-.1	-.9	-1.7	0	0	-.5	-1	0	-.1	.7
-1.8	-2.1	-1.8	-2	-2.4	-1.1	-2	-3	-1	-1	-1.6	-2.2	-1	-1.2	.8
-3.1	-3.6	-2.9	-2.8	-3.6	-1.9	-2.8	-4.1	-1.8	-1.9	-2.7	-3.3	-2	-2.1	.9
-2.9	-3.5	-3	-3.3	-3.7	-2	-3.2	-4.3	-2	-2	-2.7	-3.8	-2	-2.2	1
-4.2	-4.9	-4.3	-5.1	-4.9	-3	-4.7	-6.7	-3	-3	-3.9	-4.7	-3	-3.3	1.3
-4.3	-5.1	-4.3	-4.4	-5.4	-2.9	-4.3	-5.9	-2.9	-2.9	-3.7	-4.6	-3.1	-3.3	1.2
-5.8	-6.3	-5.6	-6	-7	-3.3	-5.3	-7.3	-3.3	-3.3	-4.6	-5.3	-3.6	-3.7	2
-6.1	-6.7	-6.6	-6.7	-7.5	-3.9	-6.5	-8.5	-4.3	-4.3	-6.3	-6.2	-4	-4.3	2.6
-7.7	-9.7	-8.0	-8.7	-10.5	-4.1	-8.2	-8.9	-4.3	-4.3	-6.3	-6.2	-4.4	-4.7	3.6

2. Channels 3, 4 and gated opening at end of channel 2 supplying water to units 4 and 5

Since the geometry shown in Figs. 2 and 3 will supply sufficient water for units 4 and 5 only when the low tide is less than 1.75 ft in magnitude (a condition that occurs less than 10 per cent of the time; see Fig. 11), additional alternates were examined. One alternate method of supplying flow to units 4 and 5 is to install a slide gate at the end of channel 2, thus allowing a portion of the flow to pass through channel 2 and less flow in channels 3 and 4.

To insure that the screens upstream of channels 1 and 2 would not be overloaded, it was decided that the maximum combined flow in channels 1 and 2 should be equal to that of unit 4 alone (430 cfs). Unit 4 and 2 identical screens. Using an Ott meter to determine the proportion of the total flow for units 2, 3, 4, and 5 flowing in channels 2, 3, and 4 for openings of various heights at the end of channel 2, the data shown in Fig. 12 were obtained. Fig. 12 shows that a gate opening of 1.5 ft would allow a total flow through channel 2 of 290 cfs for high water levels (mean sea level) and 335 cfs for low water levels (5 ft below mean sea level). This flow added to the channel 1 flow would equal 400 and 445 cfs respectively, which is approximately equal to unit 4 flow.

Water surface level data for a gate opening of 1.5 ft are given in Table V. The head loss data for this condition along with data for no opening and complete opening are shown in Fig. 13.

To prevent excessive turbulence and surging at the entrance to unit 3 pumps, it would be necessary to construct the wing wall shown in Fig. 4, Detail #3. The length of the proposed wall was determined by successively increasing the length in the model until a satisfactory flow condition was obtained.

TABLE V

Measurements for Scheme B-2

Water Surface Elevations Relative to Normal Sea Level in Feet														H_L Feet
Points 1-2	3	4-5	6	7	8	9	10	11	12	13	14	15	16	
3.8	3.6	3.6	3.7	3.7	4.1	3.7	3.5	4	4	3.8	3.6	4	4	.4
1.7	1.6	1.6	1.7	1.6	2.1	1.6	1.3	2.1	2.1	1.7	1.5	2	2	.4
-.5	-.6	-.5	-.5	-.1	-.1	-.6	-1.2	0	-.1	-.5	-.8	-.1	-.2	.4
-.6	-.6	-.6	-.7	-.7	-.1	-.8	-1.2	0	-.1	-.5	-.8	-.1	-.2	.5
-1.6	-1.6	-1.6	-1.6	-1.6	-1.9	-1.6	-2.1	-.9	-1.1	-1.5	1.8	-1.1	1.2	.5
-2.7	-2.8	-2.9	-2.9	-2.9	-2.9	-3	-3.4	-1.9	-2	-2.6	-2.8	-2	-2.1	.9
-2.6	-2.7	-2.7	-2.7	-2.7	-1.9	-2.8	-3.3	-1.9	-2.1	-2.7	-3	-2.1	-2.3	.6
-3.8	-3.8	-3.8	-3.9	-3.9	-2.9	-3.8	-4.5	-2.8	-3.1	-3.8	-4.1	-3	-3.2	.8
-3.8	-4.1	-3.9	-4	-4.1	-3	-4.1	-4.6	-2.9	-3	-3.7	-4	-3.0	-3.1	.9
-5	-5.3	-5.2	-5.2	-5.2	-4	-5.4	-6	-3.9	-4	-4.9	-5.2	-4	-4.2	1.2
-5.2	-5.3	-5.2	-5.4	-5.3	-4.2	-5.6	-6.3	-4	-4.3	-5.2	-5.5	-4.2	-4.4	1
-6.5	-6.9	-6.6	-6.8	-6.9	-5.5	-7.3	-8	-4.8	-5.1	-6.2	-6.4	-5	-5.2	1.6
-6.6	-7.1	-6.7	-6.8	-7.1	-5.2	-7.5	-8.1	-4.9	-5.2	-6.5	-6.6	-5.1	-5.3	1.6
-7.7	-8.4	-7.9	-8.2	-8.4	-6.2	-9.1	-9.4	-5.5	-5.8	-7	-7	-5.5	-5.8	2.4
-8.9	-10.2	-9	-9.4	-10.4	-8.1	-9.3	-9.3	-5.4	-6.2	-7	-7	-5.6	-5.9	3.4
-8.8	-9.5	-8.8	-9.4	-10	-7.1	-9.6	-9.7	-5.7	-6.4	-7.3	-7.2	-5.8	-6	3

Even with complete removal of the wall at the end of channel 2, this alternate would not provide an adequate cooling water supply to units 4 and 5 approximately 15 per cent of the time. Thus, a power reduction would be necessary during the time period when low tide was more than approximately 3.75 ft below mean sea level. This, coupled with the fact that for complete wall removal channels 1 and 2 and thus the screens supplying these channels, would be carrying 520 cfs (90 cfs more than unit 4 screens) indicates that a gated opening is not a completely satisfactory alternate.

3. Channels 3, 4 and 10 ft diameter pipe to supply water to units 4 and 5

Using the geometry shown in Fig. 5, the data in Table VI was obtained. These data, along with data for the alternate Schemes B-1 and B-2, are shown in Fig. 14. Figure 14 shows that this alternate is the only alternate that will adequately supply water to units 4 and 5 during periods of low tide. Not until the water level reached -7.5 ft at point 15 did any flow problems develop. At this flow depth a hydraulic jump forms in the west intake tunnel.

For the mean annual low tide the water surface at point 15 would be approximately 6.75 ft below mean sea level. For this condition the model operated satisfactorily. For the lowest tide on record (-5.3 below sea level, Dec. 1932, and Dec. 1933) the water surface level at point 15 would be -7.55 ft. This alternate would be essentially adequate to supply water for all 5 units during the infrequent times this extreme low tide would occur. However, a weak hydraulic jump would exist in the intake tunnel.

For this alternate only the effect of a 10 ft diameter pipe was determined. No other pipe sizes were tested. Although it appears as though a slightly smaller pipe may be adequate, it should be remembered that screen losses were not included in the model tests.

TABLE VI

Measurements for Scheme B-3

Water Surface Elevations Relative to Normal Sea Level in Feet														H_L
Points 1-2	3	4-5	6	7	8	9	10	11	12	13	14	15	16	Feet
3.8	3.6	3.7	3.6	3.5	3.9	3.6	3.3	4	4	3.8	3.6	4	3.9	.3
1.7	1.6	1.7	1.6	1.5	1.9	1.6	1.2	2	2	1.7	1.5	2	1.9	.3
-.4	-.5	-.4	-.4	-.6	0	-.5	-.9	0	0	-.3	-.6	0	-.2	.4
-2.4	-2.5	-2.7	-2.5	-2.7	-1.9	-4.7	-3.1	-1.9	-1.9	-2.4	-2.7	-2	-2.2	.7
-4.6	-4.7	-4.7	-4.8	-4.9	-4	-4.7	-5.3	-3.9	-3.9	-4.4	-4.8	-4	-4.3	.7
-5.7	-5.9	-5.8	-5.8	-6	-4.9	-6	-6.6	-4.9	-4.9	-5.6	-5.9	-5	-5.4	.8
-7	-7.2	-7.1	-7.2	-7.4	-5.9	-7.1	-7.9	-4.9	-5.9	-6.7	-7.1	-6	-6.7	1.1
-8.1	-8.6	-8.3	-8.8	-8.7	-6.8	-8.4	-9.4	-6.9	-6.9	-7.9	-8.2	-7	-7.9	1.3
-9.1	-9.2	-9.1	-9.3	-9.4	-7.3	-9.2	-10.4	-7.3	-7.4	-8.5	-8.8	-7.5	-8.3	1.6

IV. SUMMARY AND RECOMMENDATIONS

Scheme A - Units 1, 2, 3, and 4 operating

While the geometry shown in Figs. 1 and 2 are both satisfactory, the geometry shown in Fig. 2 (Scheme A-2) has several important advantages. First, the head loss (see Fig. 7) is significantly less using Fig. 2 geometry. Over the lifetime of the structure this would result in a considerable saving in pumping power as well as more satisfactory operation at extreme low tides. Second, the channel cleaning operations would be better for Fig. 2 geometry since no power outage for unit 4 would be required as it would be for Fig. 1 geometry. Finally, looking to the future addition of unit 5, the construction of Fig. 2 geometry now would save considerable time and cost, and power outages would be virtually eliminated when unit 5 is constructed.

Scheme B - Units 1, 2, 3, 4, and 5 operating

First, it can be concluded that if unit 5, when constructed, is of the capacity currently anticipated then the alternate Scheme B-3 (Fig. 5) is the only alternate that will work. If, instead, unit 5 is reduced in size, then alternates presented in Schemes B-1 and B-2 (Figs. 2 and 4, respectively) may be suitable, depending on the actual final size of unit 5. In order to help clarify this last point the data shown in Figs. 10 and 15 were obtained for alternate Schemes B-1-b and B-2, respectively. From the figures it can be seen that Scheme B-1-b would be satisfactory if unit 5 required approximately 50 per cent of the currently anticipated cooling water supply and Scheme B-2 would be satisfactory if unit 5 were 80 per cent of present design flow.

V. POSSIBLE ADDITIONAL TESTS

It is recommended that additional tests be conducted to determine the effect of cleaning the east and west channels when unit 5 is operating at 50 per cent capacity using Scheme B-1-b, 80 per cent capacity using Scheme B-2 and 100 per cent capacity using Scheme B-3.

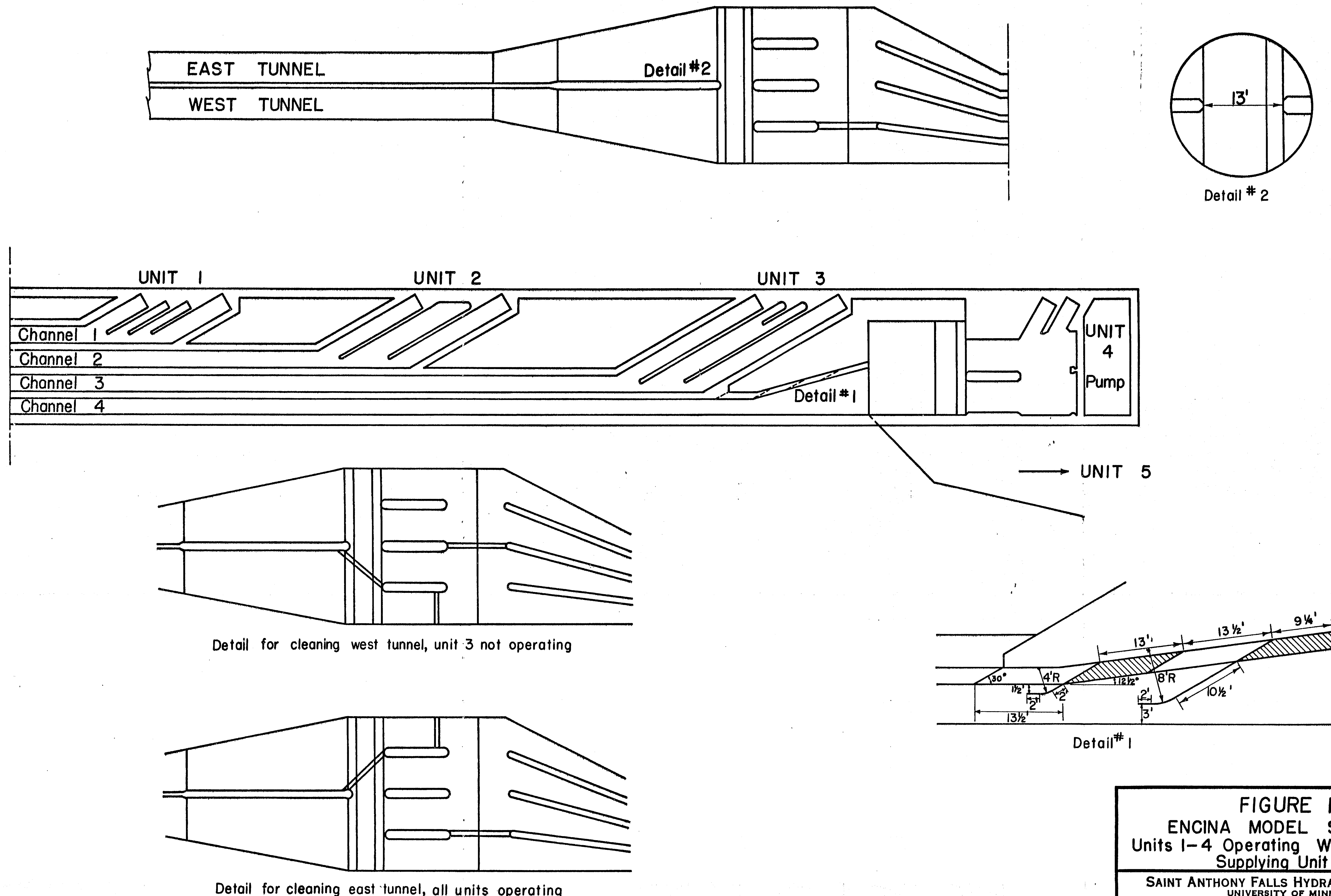

FIGURE I

ENCINA MODEL STUDY

Units I–4 Operating With Channel 4 Supplying Unit 4

SAINT ANTHONY FALLS HYDRAULIC LABORATORY
UNIVERSITY OF MINNESOTA

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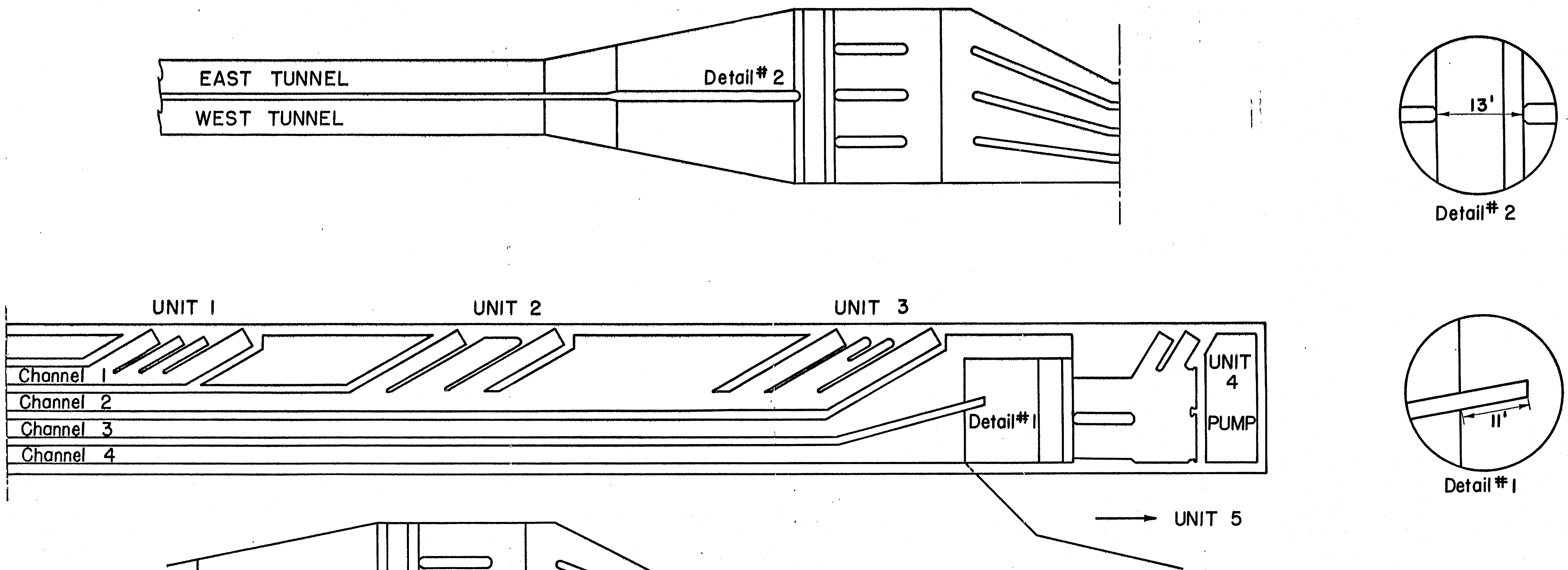
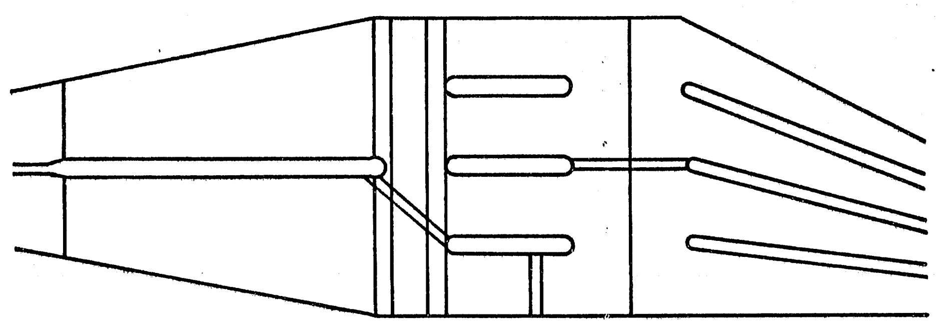
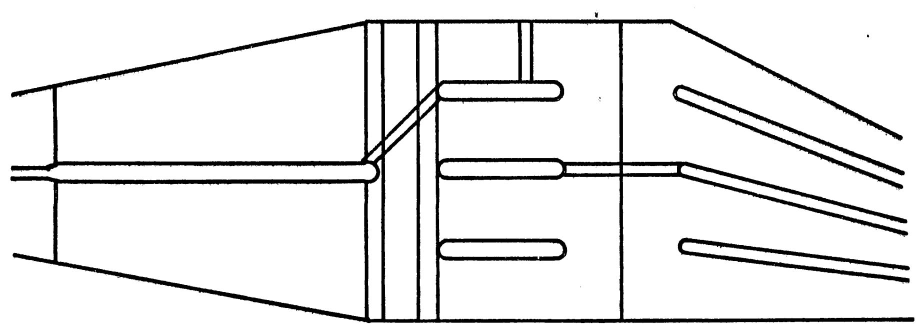
SCALE	DATE	DRAWING NO.





Configuration for cleaning west tunnel, reduced power for units 4 and 5 when both are operating



Configuration for cleaning east tunnel, unit 3 not operating

FIGURE 2
ENCINA MODEL STUDY
Channels 3 and 4 without vanes supplying water to unit 4 or to both units 4 and 5

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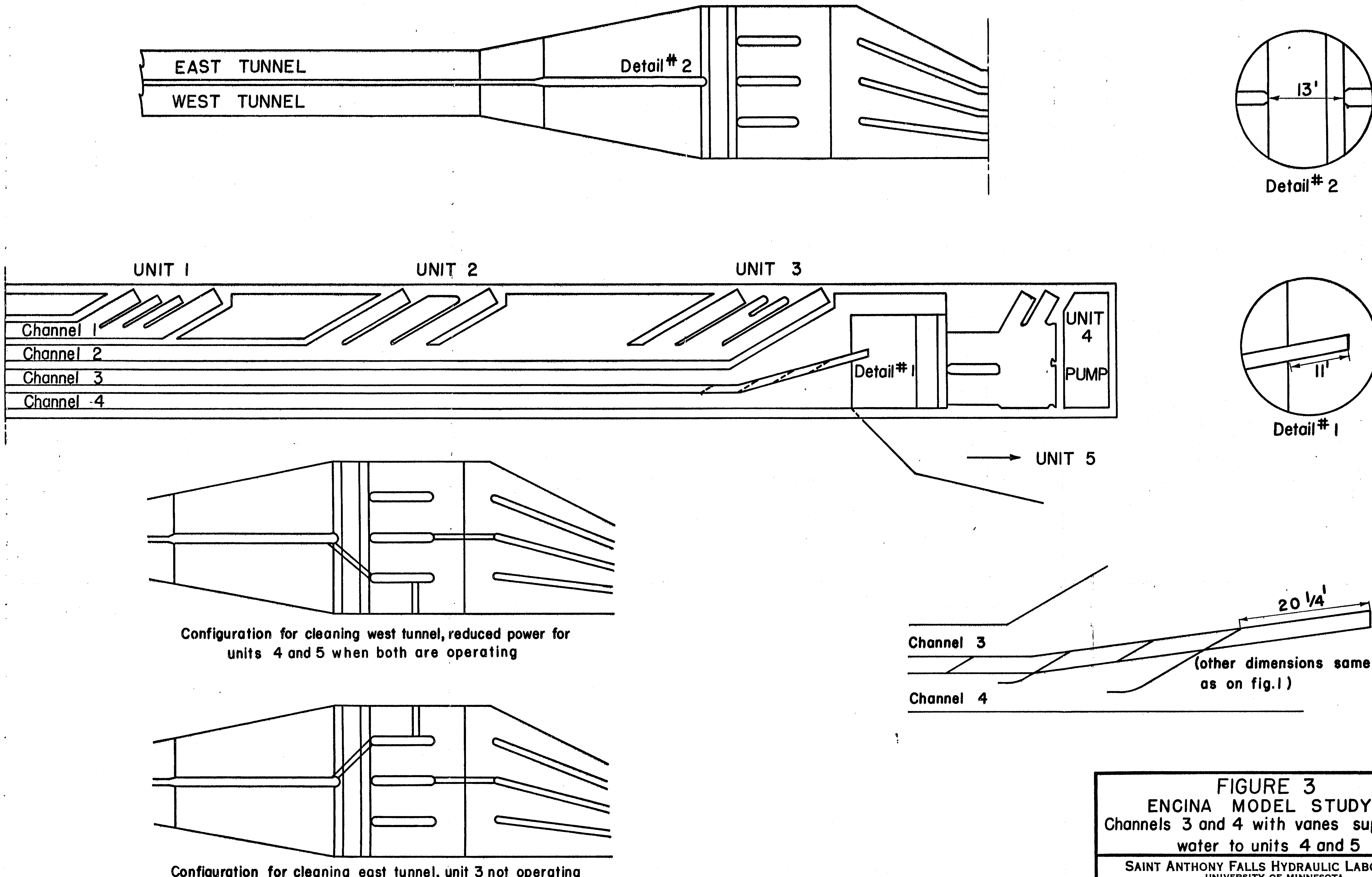

FIGURE 3
ENCINA MODEL STUDY
Channels 3 and 4 with vanes supplying water to units 4 and 5

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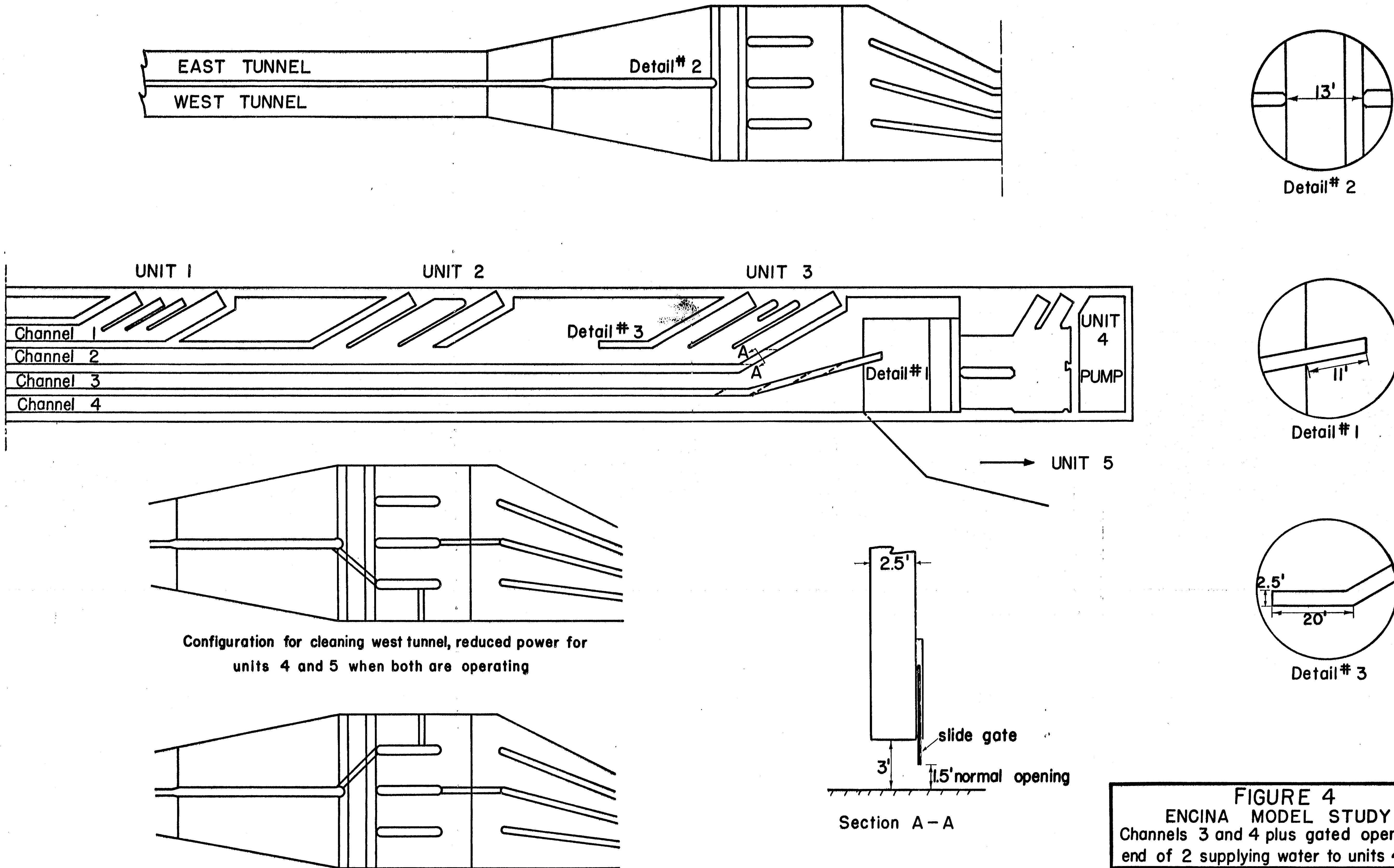

Configuration for cleaning west tunnel, reduced power for units 4 and 5 when both are operating

Configuration for cleaning east tunnel, unit 3 not operating

FIGURE 4
ENCINA MODEL STUDY
Channels 3 and 4 plus gated opening at end of 2 supplying water to units 4 and 5

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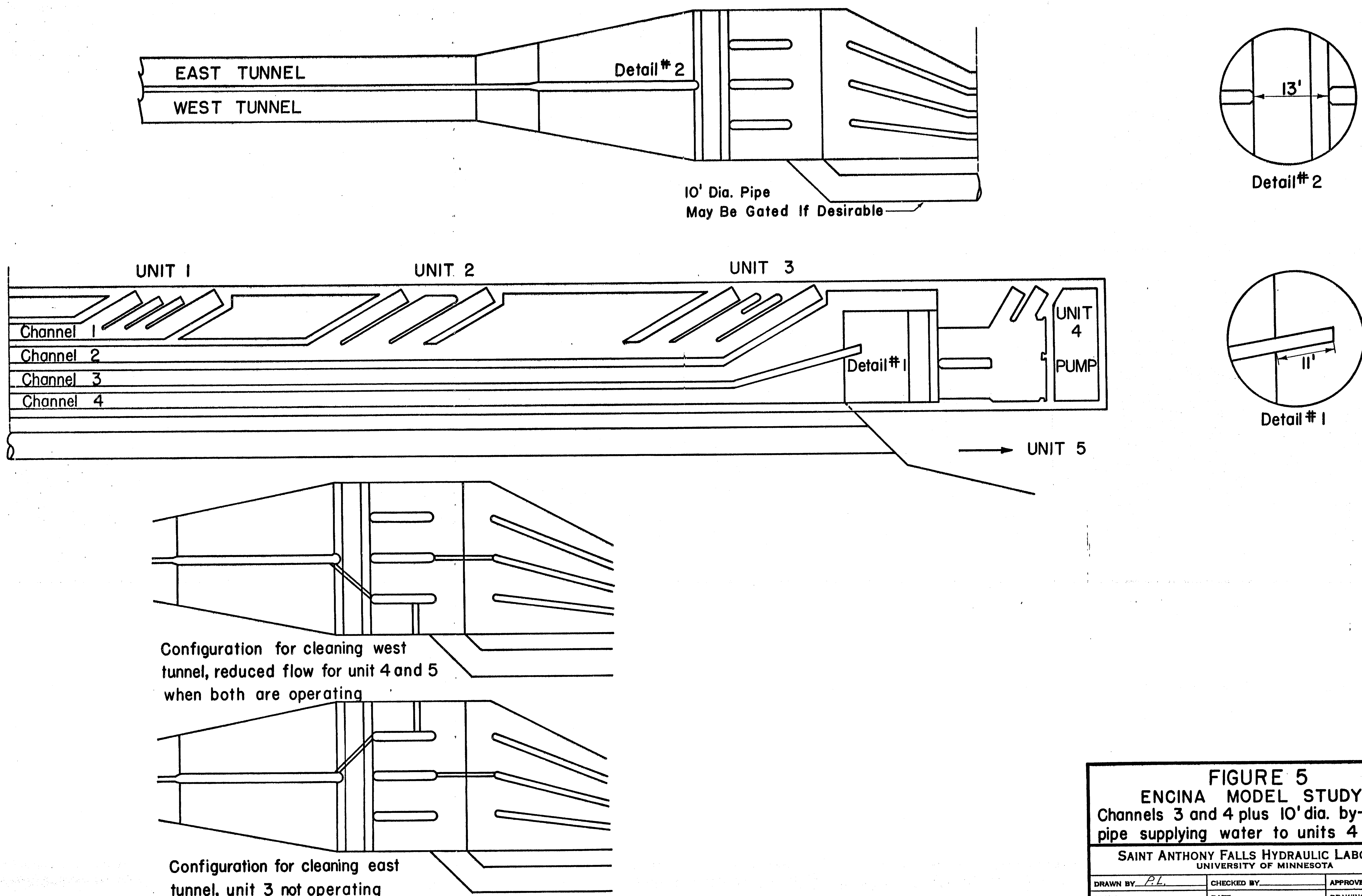

FIGURE 5
ENCINA MODEL STUDY
Channels 3 and 4 plus 10' dia. by-pass pipe supplying water to units 4 and 5

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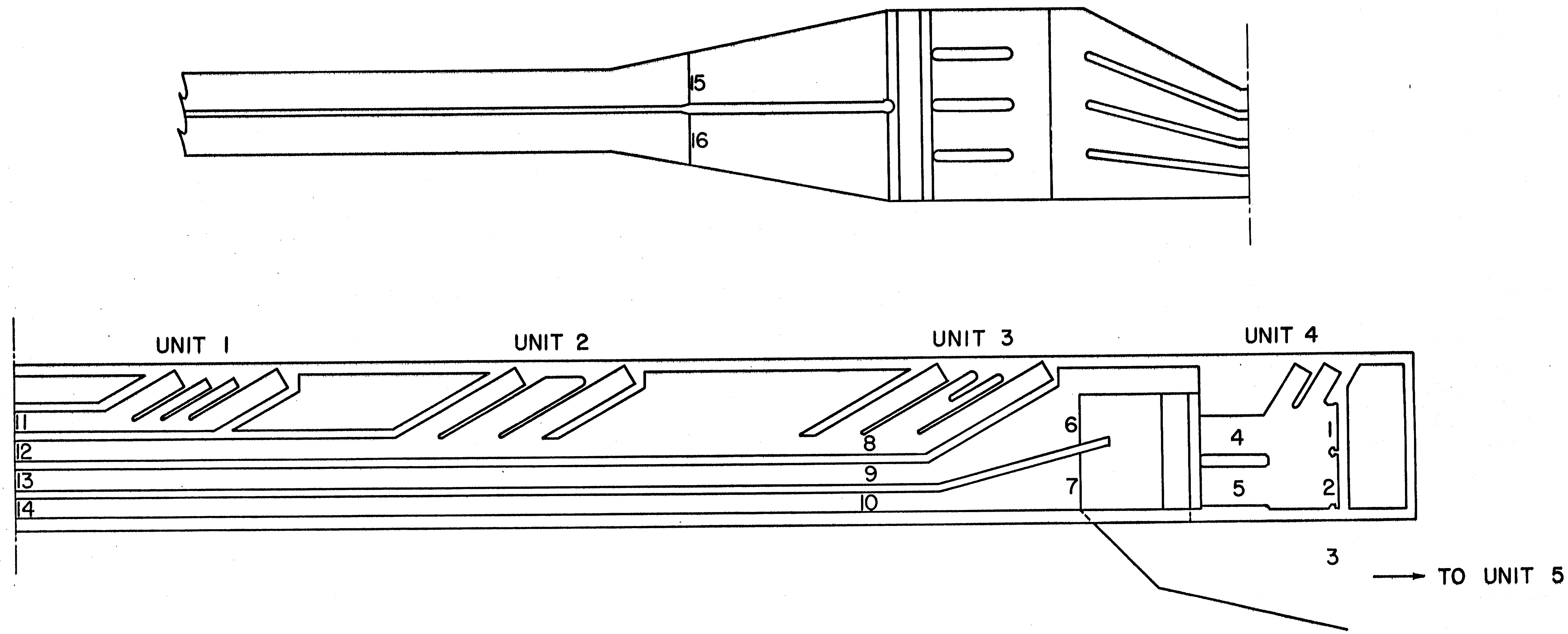

FIGURE 6
ENCINA MODEL STUDY
Pressure Tap Locations

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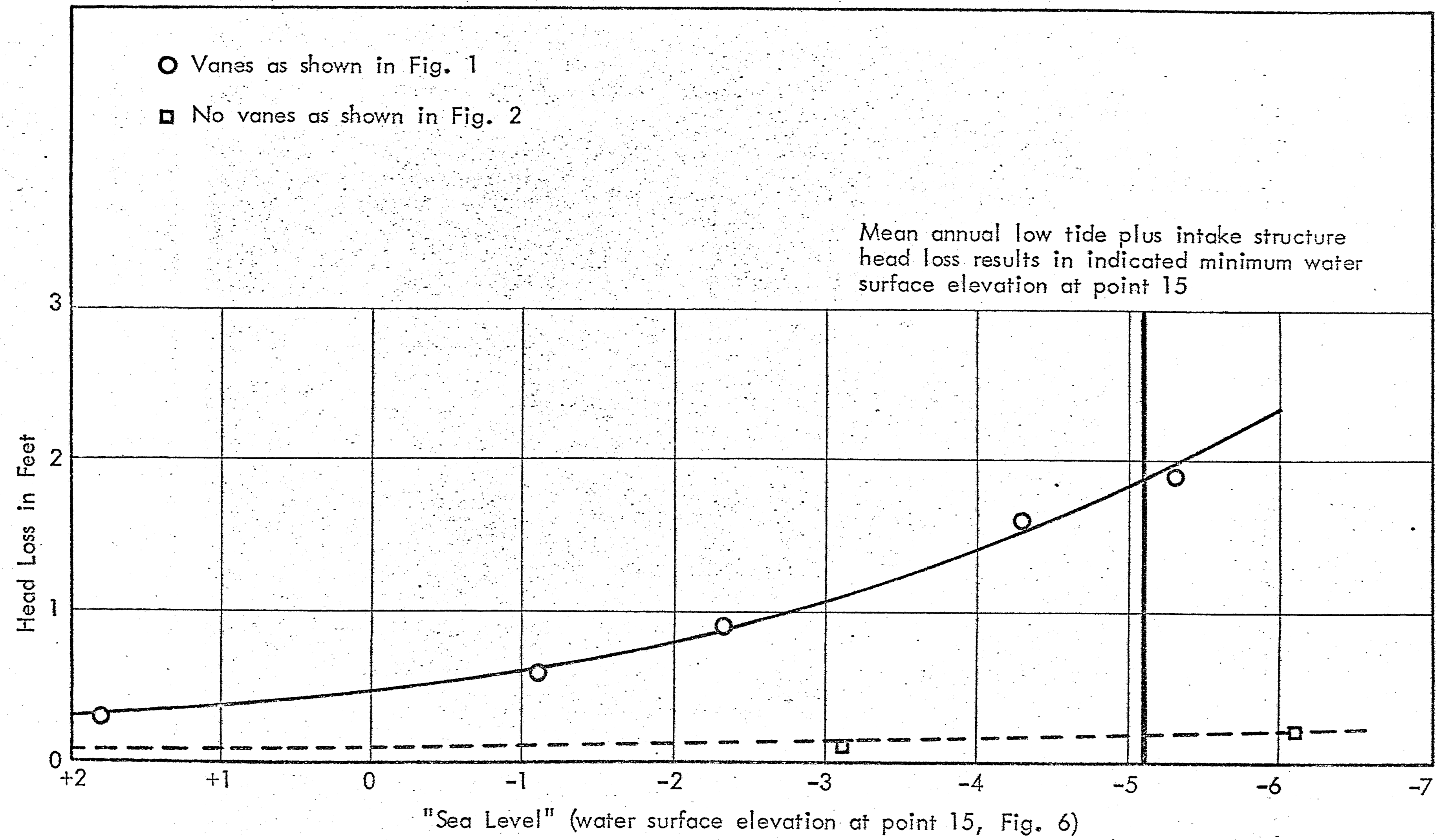

Fig. 7 - Head Loss as Function of Water Surface Elevation and Geometry for Supplying Water to Unit 4

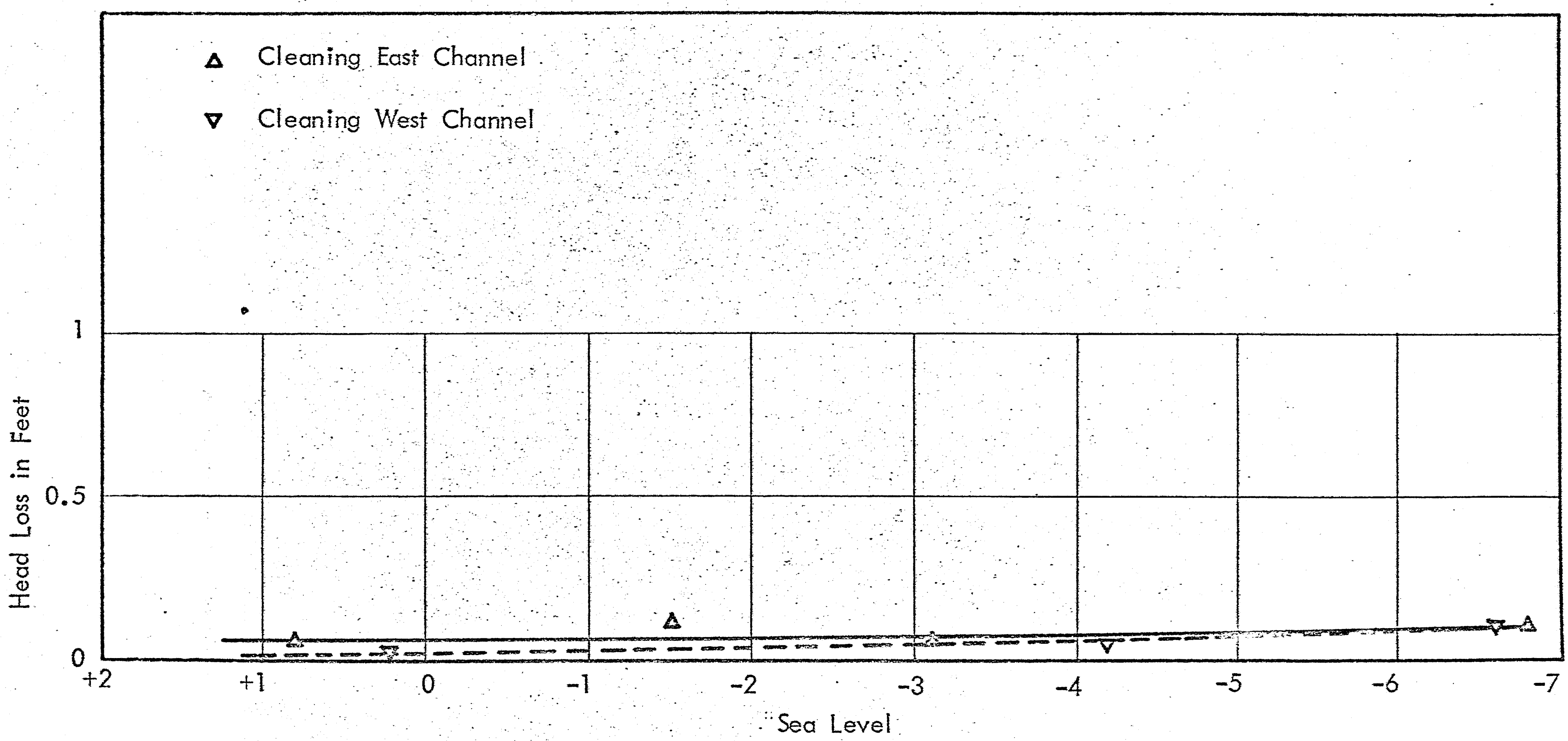

Fig. 8 - Cleaning East and West Tunnels, Units 1, 2, 3, and 4 Operating, using Fig. 2 geometry

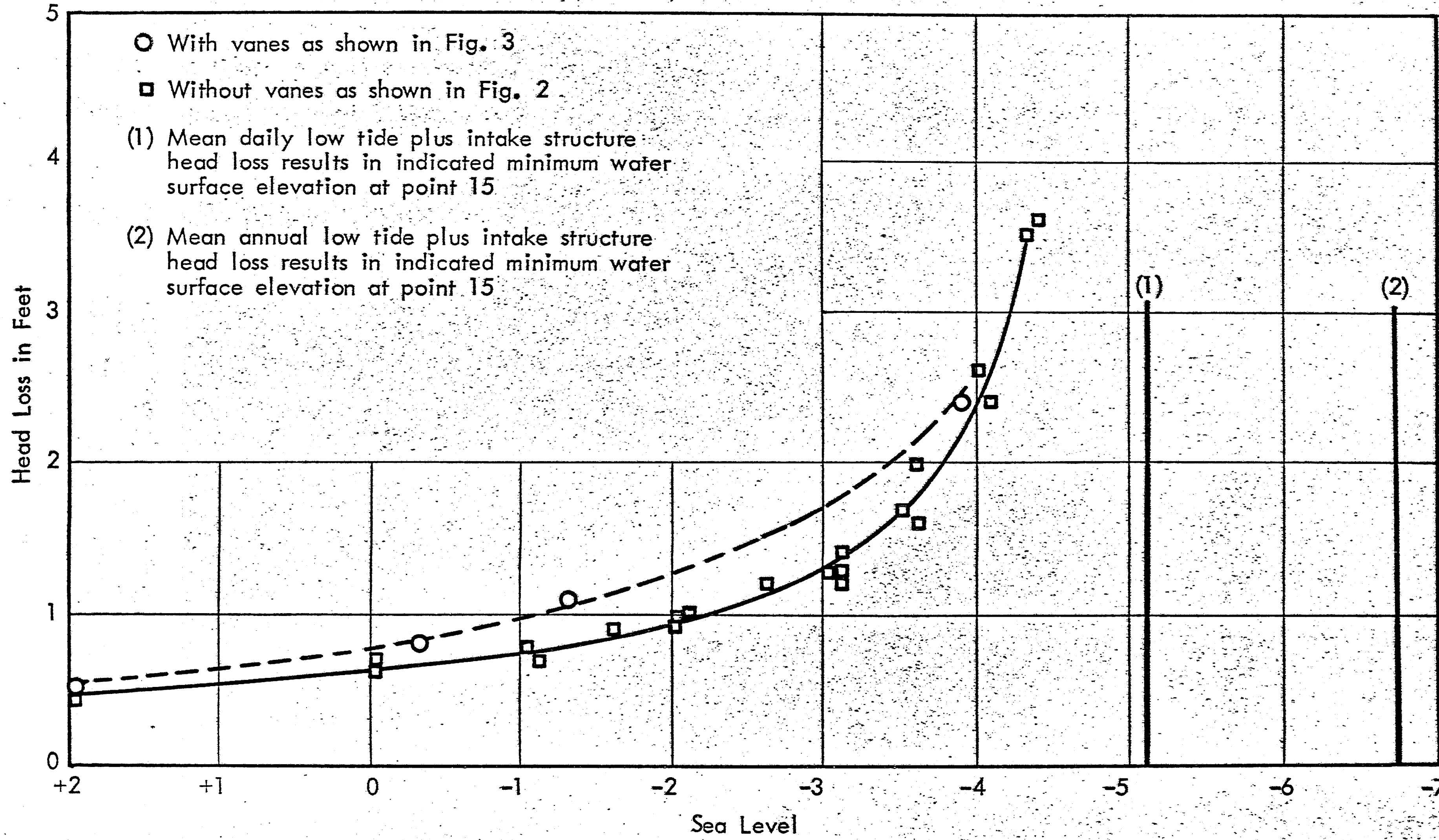

Fig. 9 - Head loss as function of Water Surface Elevation when Units 4 and 5 are supplied by Channels 3 and 4

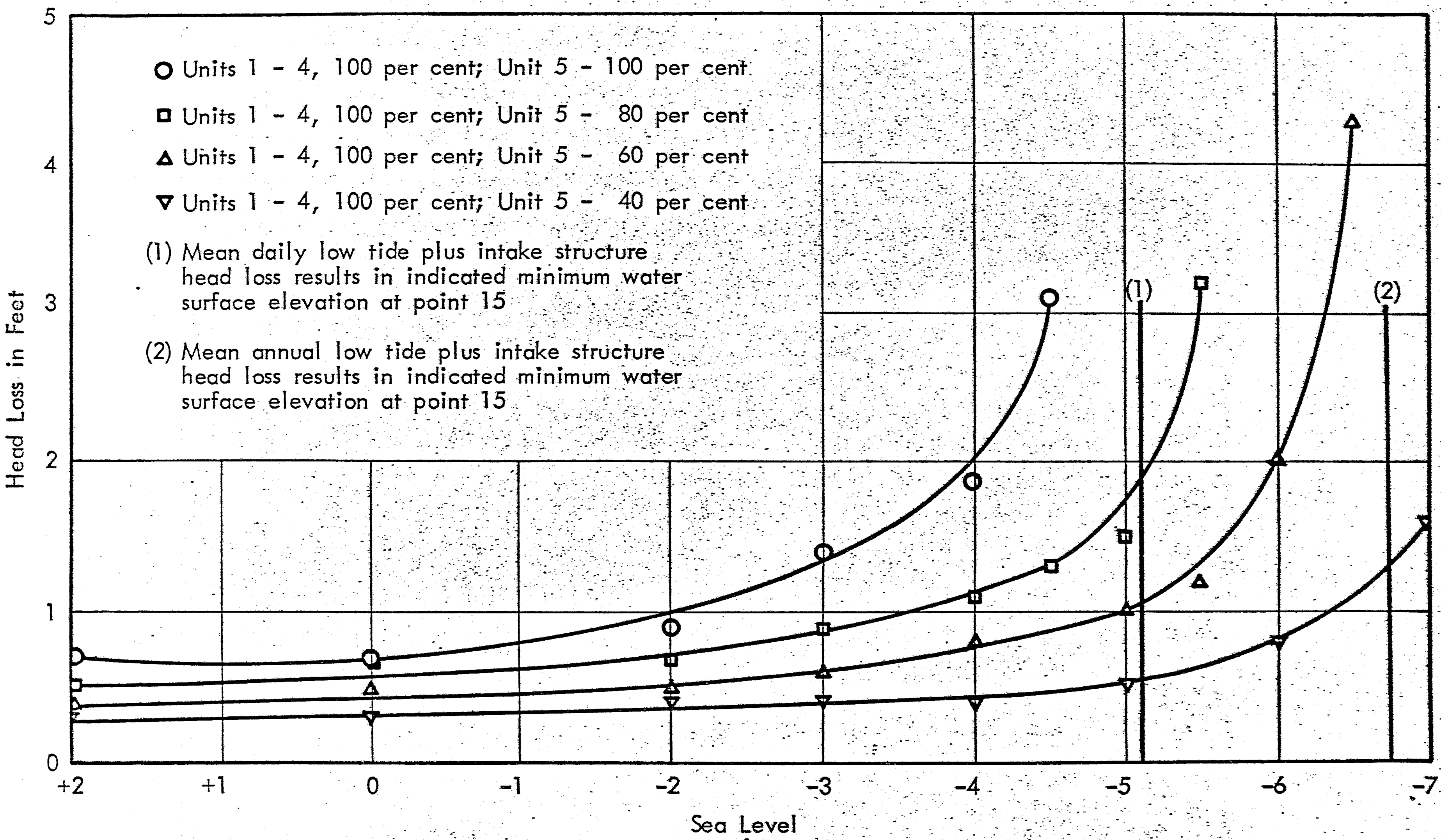

Fig. 10 - Head loss as a function of Water Surface Elevation and Unit 5 flow using Fig. 2 geometry

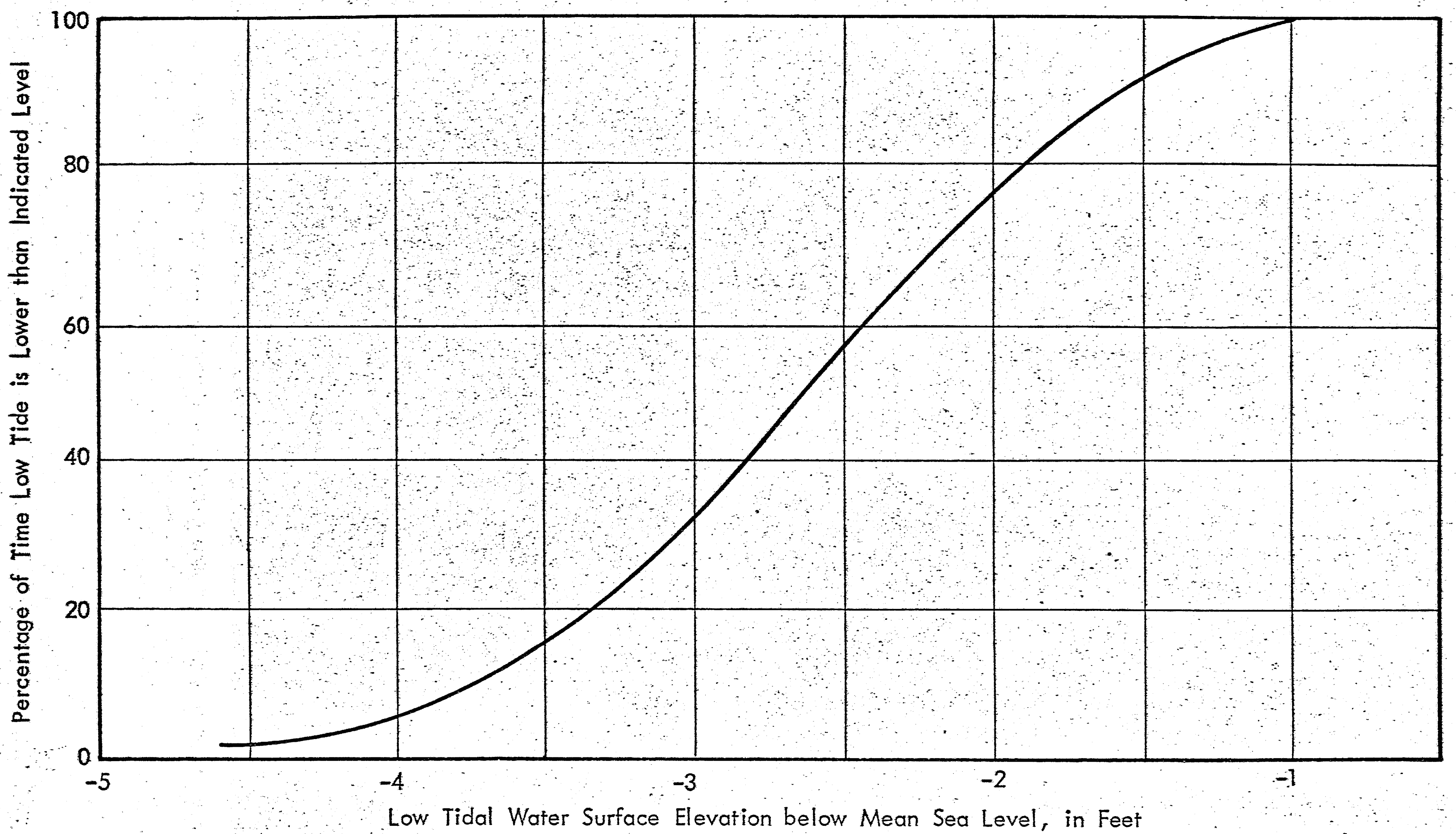

Fig. 11 - Tide-Time Relationship for Encina, California

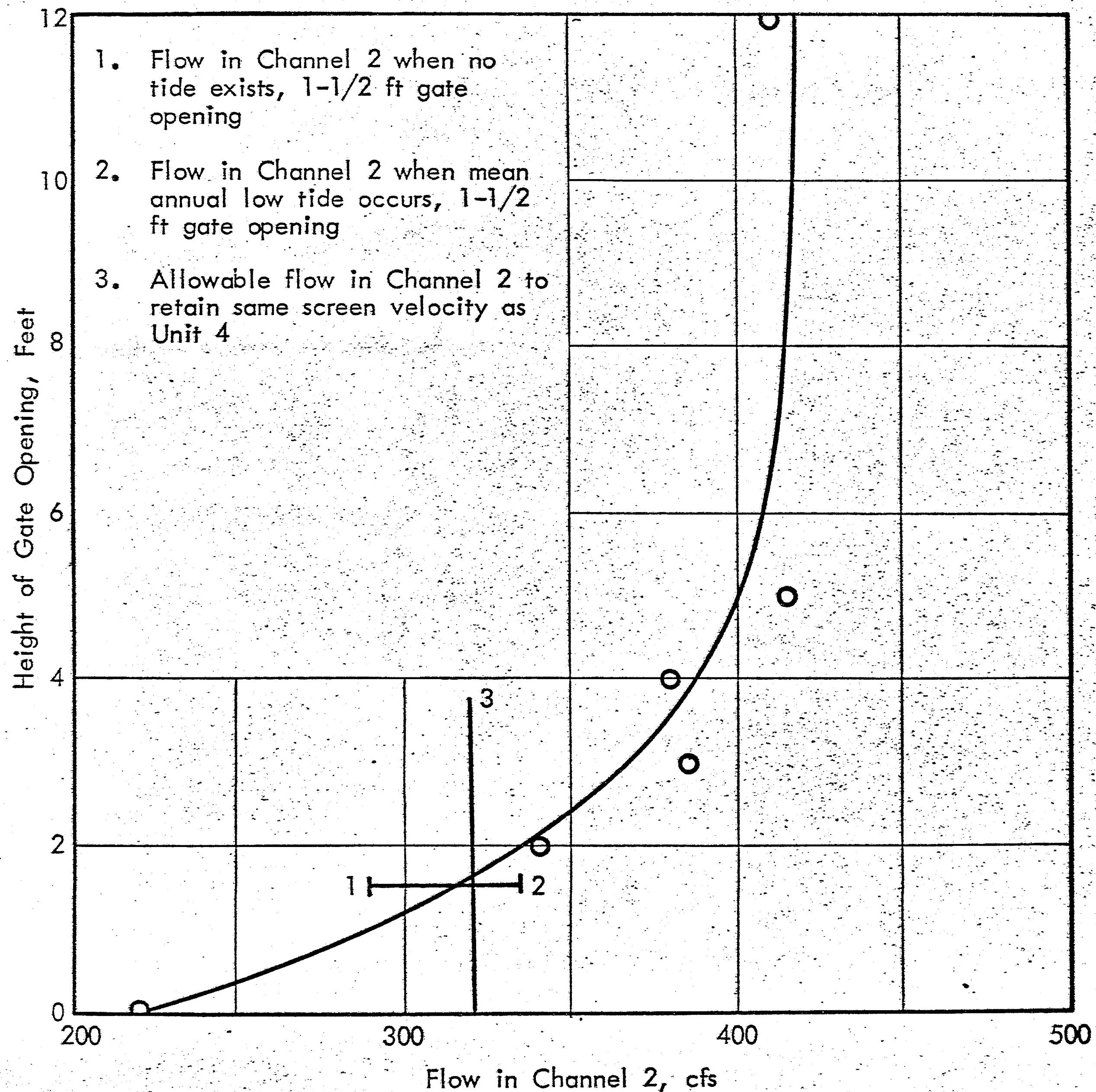

Fig. 12 – Flow in Channel 2 as a function of Gate Opening

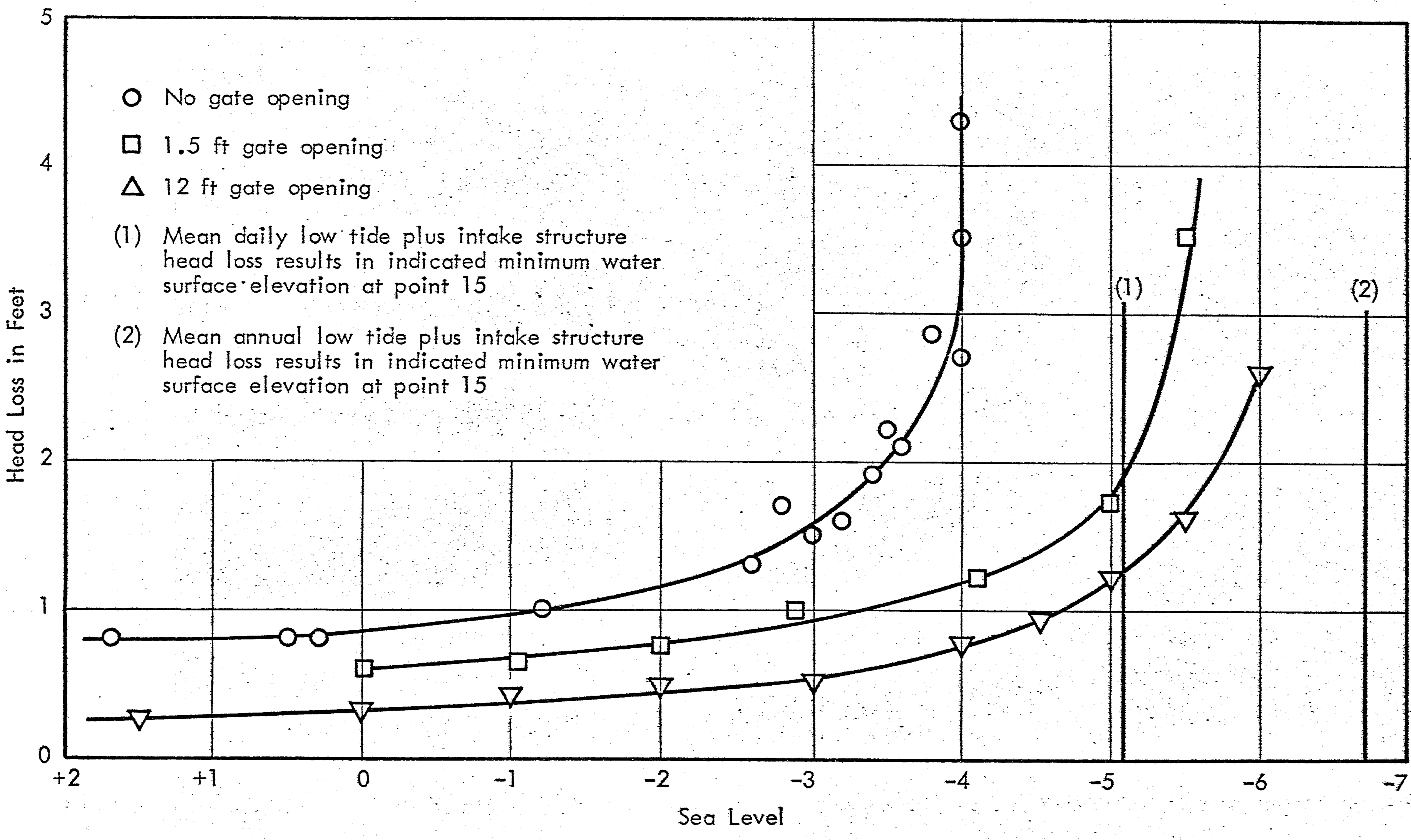

Fig. 13 - Head loss as function of Water Surface Elevation for 0, 1.5, and 12 ft Gate Opening at End of Channel 2, Units 1 - 5 Operating

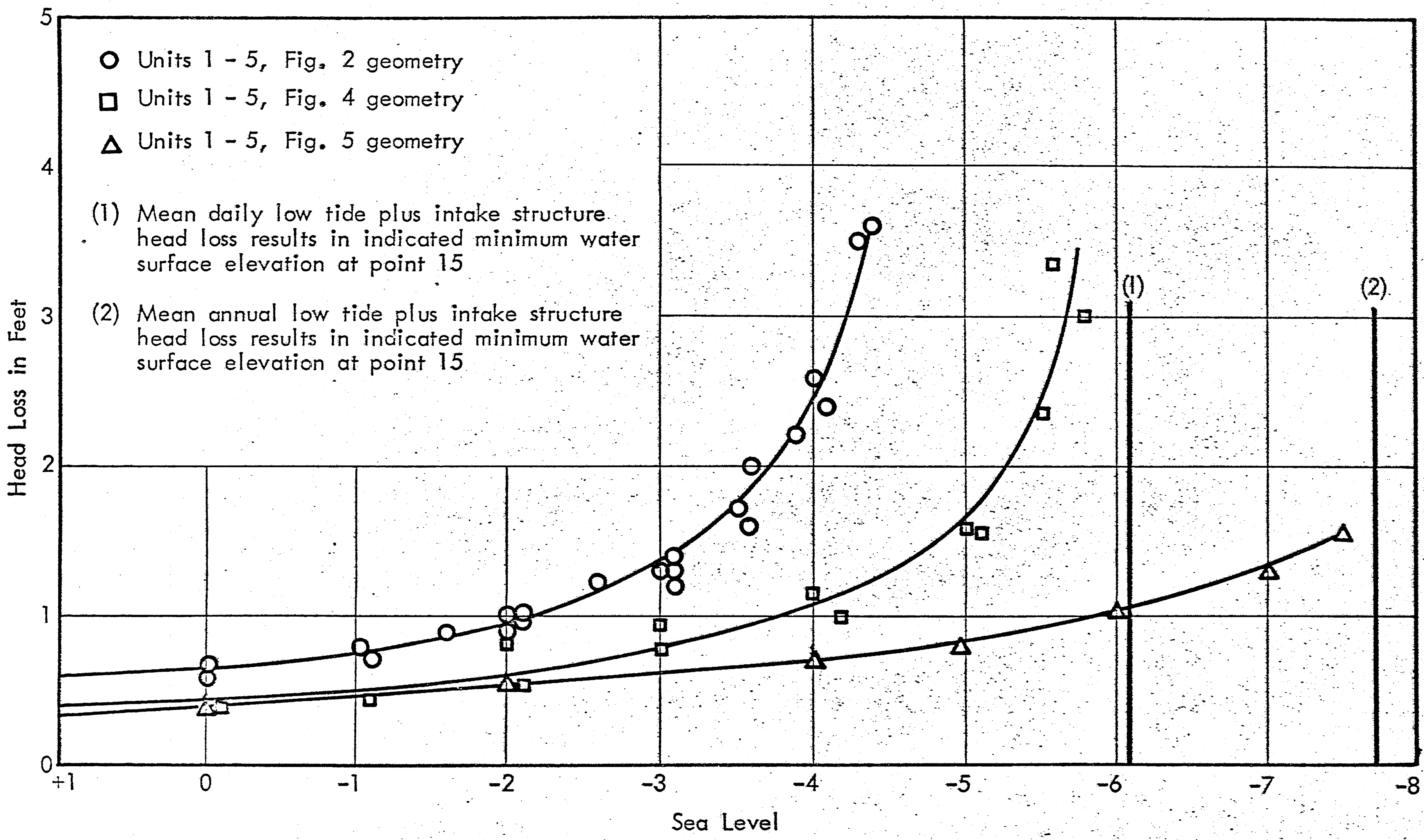

Fig. 14 - Head loss as function of Water Surface Elevation for Various Geometries

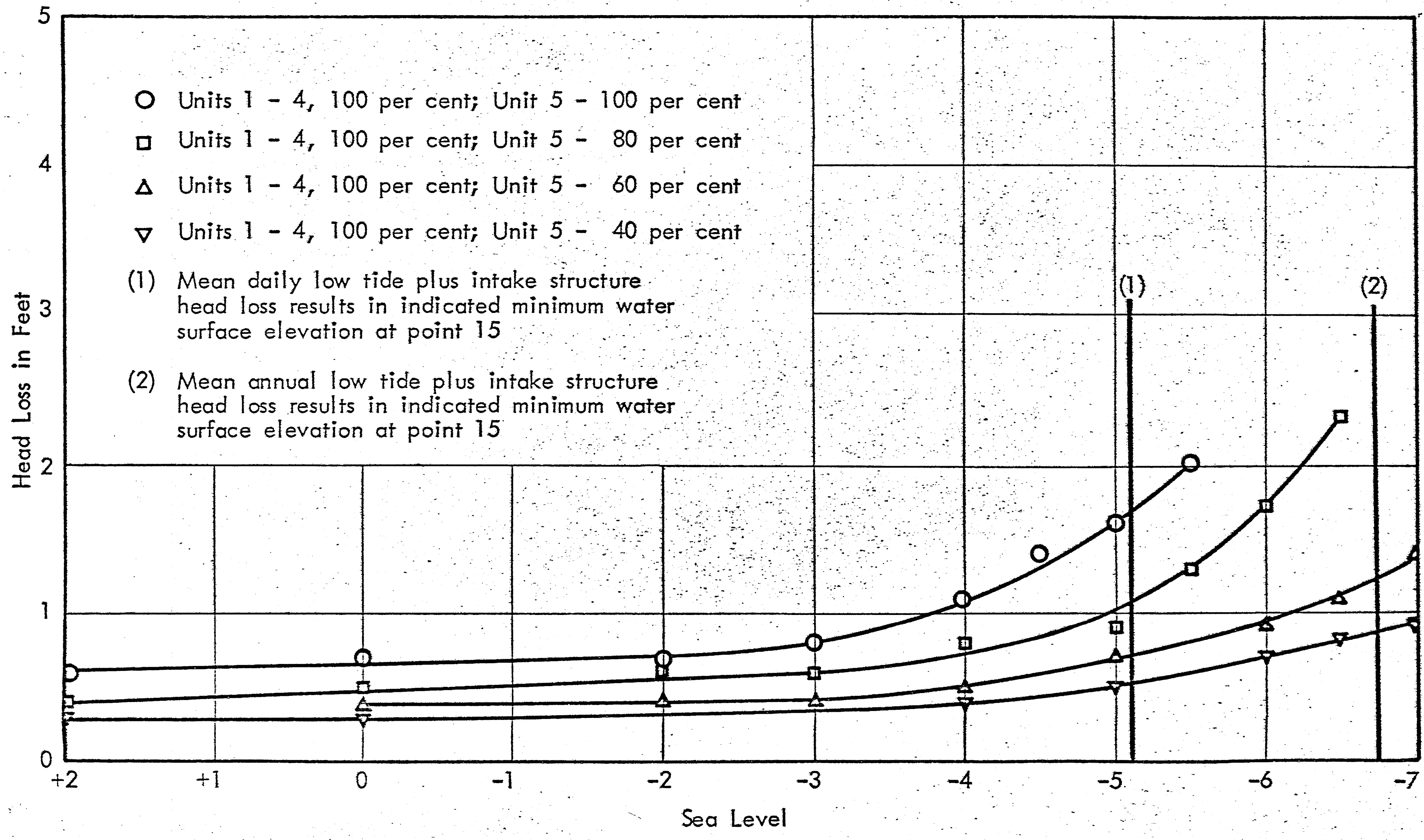

Fig. 15 - Head loss as a function of Water Surface Elevation and Unit 5 flow using Fig. 4 geometry