KGE THE ELECTRIC COMPANY

KANSAS GAS AND ELECTRIC COMPANY

GLENN L. KOESTER
VICE PRESIDENT - NUCLEAR

November 6, 1981

Mr. Harold R. Denton, Director
Office of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

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U.S. NUCLEAR REGULATORY COMMISSION

KMLNRC 81-132
Re: Docket No. STN 50-482
Ref: Letter of 9/22/81 from BJYoungblood, NRC, to GLKoester, KG&E
Subj: Hydrologic Engineering - Environmental

Dear Mr. Denton:

The Reference requested additional information concerning the hydrologic characteristics presented in the Wolf Creek Generating Station, Unit No. 1 Environmental Report - Operating License Stage (WCGS-ER(OLS)). Transmitted herewith are responses to the questions in the Reference. The responses will be formally incorporated in the WCGS-ER(OLS) in the next revision. The attached information is hereby incorporated into the Wolf Creek Generating Station, Unit No. 1 Operating License Application.

Yours very truly,

Glenn L Koester

GLK:bb
Attach

cc: Dr. Gordon Edison (2)
Division of Project Management
Office of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

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S1/1

Mr. Thomas Vandel
Resident NRC Inspector
P.O. Box 311
Burlington, Kansas 66839

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C PDR

OATH OF AFFIRMATION

STATE OF KANSAS)
) SS:
COUNTY OF SEDGWICK)

I, Glenn L. Koester, of lawful age, being duly sworn upon oath, do depose, state and affirm that I am Vice President - Nuclear of Kansas Gas and Electric Company, Wichita, Kansas, that I have signed the foregoing letter of transmittal, know the contents thereof, and that all statements contained therein are true.

KANSAS GAS AND ELECTRIC COMPANY

ATTEST:

By Glenn L Koester
Glenn L. Koester
Vice President - Nuclear

W.B. Walker
W.B. Walker, Secretary

STATE OF KANSAS)
) SS:
COUNTY OF SEDGWICK)

BE IT REMEMBERED that on this 6th day of November, 1981, before me, Evelyn L. Fry, a Notary, personally appeared Glenn L. Koester, Vice President - Nuclear of Kansas Gas and Electric Company, Wichita, Kansas, who is personally known to me and who executed the foregoing instrument, and he duly acknowledged the execution of the same for and on behalf of and as the act and deed of said corporation.

IN WITNESS WHEREOF, I have hereunto set my hand and affixed my seal the date and year above written.

EVELYN L. FRY
NOTARY PUBLIC
STATE OF KANSAS

Evelyn L. Fry
Evelyn L. Fry, Notary

My Commission expires on August 15, 1984.

Question 240.16

a) Section 2.4.1.2.2, p. 2.4-8 states that there are 34 water right permits granted for irrigation use along the Neosho River from the mouth of Wolf Creek to Oklahoma. However, Table 2.1-19 lists only 30 of these permits. Please update Table 2.1-19 to include the additional 4 irrigation permits.

b) The maximum rate of appropriated surface water from the John Redmond spillway location to the Oklahoma state line is stated in Section 2.4.1.2.2, p. 2.4-8 to be 239, 404 gpm. Table 2.1-19 indicates that the authorized maximum diversion rate from the Neosho River downstream of the confluence of Wolf Creek is 115,469 gpm. Please explain the discrepancy in these values. If the discrepancy is the result of diversions between the John Redmond Reservoir and Wolf Creek please furnish the appropriate information as given in Table 2.1-19.

c) The maximum annual quantity of water authorized to be diverted from the Neosho River as stated in Section 2.4.1.2.2, p. 2.4-8 (117,065 acre-feet) is four times larger than the total quantity indicated in Table 2.1-19 (29,989 acre feet). Please explain the discrepancy as in b) above.

Response

a) The 34 water right permits granted for irrigation represent the number of permits along the Neosho River from the Oklahoma state line to John Redmond Dam, not to the mouth of Wolf Creek. The discussion in Section 2.4.1.2.2 has been corrected to reflect this fact and the irrigation permits not listed in Table 2.1-19 are shown on new Table 2.1-19(a). The information presented in Table 2.1-19(a) was originally provided in Wolf Creek Final Safety Analysis Report (FSAR) Table 2.4-4.

b) and c) The value for the authorized maximum diversion rate (115,469 gpm), and that for the authorized maximum annual quantity (29,989 acre feet) indicated in Table 2.1-19 represents authorized diversions from the Neosho River from the mouth of Wolf Creek to the Oklahoma state line. Authorized diversions between Wolf Creek and John Redmond Dam are shown in new Table 2.1-19(a). The totals indicated in Section 2.4.1.2.2 have been corrected to reflect the data presented in Tables 2.1-19 and 2.1-19(a).

TABLE 2.1-19(a)

Additional Water RIGHTS ON THE NEOSHO RIVER
BETWEEN JOHN REDMOND DAM AND WOLF CREEK

Application Number	Location of Diversion(a)	Map Key(b)	Approximate River Mile(c)	Owner	Source	Authorized Maximum Diversion Rate (gpm)	Authorized Maximum Annual Quantity (acre-feet)	Principal Use
14626	E/NW/NW 10-21-15	9	343	KG&E	Neosho River	24,685(d)	25,000	Industrial
19882	E/NW/NW 10-21-15	35	343	KG&E	Neosho River	76,300(d)	57,300	Industrial
3782	NW/NE/SE 9-21-15	47	343	Kansas Fish & Game Commission	Neosho River	12,000	150	Recreation
4259	SW/NE/NE 15-21-15	49	342	J. Decker	Neosho River	1,050	156	Irrigation
21453	NW 26-21-15	44	339	City of Burlington Coffey Co. RWD 2 and 3	Neosho River	1,000	767	Municipal
271	NE/SW/NW 26-21-15	50	339	W. Strawn	Neosho River	400	40	Irrigation
VSC2	NW 26-21-15	41	338	City of Burlington	Neosho River	800	245	Municipal
2636	W/SW/SE 26-21-15	45	338	F. Robrahn	Neosho River	650	61	Irrigation
11078	NW/NW/SE 35-21-15	8	337	Nelson Motors, Inc.	Neosho River	1,500	350 125	Recreation Irrigation

[a]Locations are specified by section division, section, township, and range.

[b]See FSAR Figure 2.4-8 for locations.

[c]Mouth of Wolf Creek is approximately at Neosho River Mile 334.5; John Redmond Dam at approximately River Mile 343.7.

[d]Withdrawal of natural flows in Neosho River only at such times as minimum of at least 250 cfs remains immediately downstream from the intake.

Source: Kansas State Board of Agriculture, 1979. Open file material: Division of Water Resources, Topeka, Kansas (March).

Incorporated municipal water supply systems from Coffey County to Oklahoma, which utilize the Neosho River as the source of supply, are listed in Table 2.4-5. These include domestic, commercial, industrial, and public-use water requirements. Rural water districts in Kansas utilizing the Neosho River as the source of supply, either directly or indirectly, are also listed in Table 2.4-5. They have been formed in those areas where groundwater resources are limited.

There are 34 water right permits granted for irrigation use along the Neosho River from John Redmond Dam to Oklahoma. The maximum rate of appropriated surface water from the John Redmond spillway location to the Oklahoma state line is 233,854 gallons per minute, with a maximum annual quantity of 114,183 acre-feet (Kansas State Board of Agriculture 1979).

Further description of water use is provided in Section 2.1.3.4.

2.4.1.3 <u>Wolf Creek Cooling Lake</u>

The cooling lake is formed by a "main" earth dam constructed across Wolf Creek and saddle dams built along the periphery of the lake. The tops of the dams are at an elevation of 1,100 feet above mean sea level (MSL) to provide sufficient freeboard and prevent overtopping of the dams by the probable maximum flood and wind and wave action. Service and auxiliary spillways are provided on the east abutment of the main dam to pass floods of elevations up to and including that of the probable maximum flood. The maximum cooling lake elevation of 1,095 feet MSL occurs when the probable maximum flood is preceded by the standard project flood and both are routed through the spillways.

The normal operating elevation of the cooling lake is 1,087 feet MSL. At this elevation the lake has a capacity of 111,280 acre-feet and a surface area of 5,090 acres. Estimated sedimentation during the life of the plant in the cooling lake from the Wolf Creek stream flow and from the makeup pumped from the Neosho River below the John Redmond Reservoir is 1 percent of the lake's storage volume at its normal operating level and thus does not affect the functioning of the lake.

2.4.1.3.1 Makeup Water Supply to Wolf Creek Cooling Lake

A major source of makeup water to the cooling lake is the conservation storage of the John Redmond Reservoir, providing that the low flow downstream water requirements are satisfied. Additional makeup water is supplied by natural runoff

Question 240.17 (ER)

Table 240.14/240.15-1 gives the 100-year peak flood flow for Wolf Creek below the cooling lake dam under natural conditions as 8363 cfs. How does this value compare with the peak flood flow used to arrive at the flood prone area due to the 100-year flood found in Flood Hazard Boundary Maps for Coffey County?

Response

According to References 1 and 2, the 100-year peak flood discharge used to delineate the flood prone area in the Flood Hazard Boundary Maps of Coffey County is 13,900 cfs at the confluence of Wolf Creek with the Neosho River. This flow was based on a regression equation developed by the Kansas Water Resources Board for estimating 100-year flood peaks for ungaged streams in the State of Kansas. (Ref. 3).

The peak 100-year flow under natural conditions at the location of the cooling lake dam as given in Table 240.14/240.15-1 is 8,363 cfs. This flow was estimated by developing a unit hydrograph for the Wolf Creek drainage area and applying the 100-year rainfall distribution as discussed in Section 2.4 of the FSAR addendum. The corresponding discharge at the confluence of Wolf Creek with the Neosho River, estimated in proportion to the drainage areas is 10,680 cfs as compared to 13,900 cfs used in the Flood Hazard Boundary Maps of Coffey County.

Though there is a difference in the peak flood flow under natural conditions (without the existence of the cooling lake dam estimated by the two different methods, the conclusion demonstrated in the response to Q. 240.14/240.15, that the flooding of areas below Wolf Creek dam due to Wolf Creek flood flows is much reduced after the construction of the cooling lake dam, remains unchanged.

References:

1. Mr. R. G. Chappel, Chief, Engineering Branch, Federal Emergency Management Agency, Washington, D.C. undated letter to G. V. Komanduri, Sargent & Lundy, (Received October 26, 1981).
2. Mr. Werrer Miller, Michael Baker Inc., Harrisburg, Pennsylvania, telephone conversation with Mr. G. V. Komanduri, Sragent & Lundy, October 27, 1981
3. Kansas Water Resources Board, "Magnitude and Frequency of Floods in Kansas, Unregulated Streams," Technical Report No. 11, 1975.

Question 240.18

In Section 2.4.2.1.1 influent conditions on the Neosho River are purported to result in horizontal migration into the alluvium of 100 to 200 feet. Please provide the data to support this estimate, and what method(s) and parameter values were used.

Response

Section 2.4.1.2.2 provides data regarding the flow characteristics of the Neosho River. Low flow for the river (base flow) represents flow which is sustained primarily by ground water. As stated in Section 2.4.1.2.2, the mean daily discharge during a representative low flow period (November 1950) was 200 cfs at New Strawn and 224 cfs at Iola. The increase of 24 cfs (2.07×10^6 ft^3/day) is assumed to be the ground water contribution between the two stations. The distance between the two stations is approximately 48 miles or 2.53×10^5 feet. Assuming that half the river flow contribution comes from each side of the river, the inflow for one bank would be 1.04×10^6 ft^3/day.

From Figure 2.4-6 a reasonable estimate of the average hydraulic gradient is about 0.004 toward the Neosho River. For some areas it is less and for others it is greater. Section 2.4.2.1.1 indicates that the Neosho River alluvium has a maximum thickness of 20 feet. Assume that the average saturated thickness is about 18 feet.

From Darcy's law $Q = Kia$, or $K = Q/ia$, the average permeability can be estimated for the alluvium along the river.

$$K = 1.04 \times 10^6/(0.004)\ (18)\ 2.53 \times 10^5 = 57 \text{ ft/day or } 2.0 \times 10^{-2} \text{ cm/sec.}$$

This value of permeability is within the expected range of permeabilities for clean sand as indicated by Freeze and Cherry (1979 p. 29).

During the rising part of the flood hydrograph, water from the stream enters bank storage (Freeze and Cherry, 1979, p. 225). Likewise, during the recession part of the flood hydrograph ground water leaves bank storage and enters the stream. As the stream level rises and water starts to enter bank storage, the hydraulic gradient driving the water into the bank storage is initially relatively high but very quickly decreases as the water enters bank storage.

Assume that for a given flood the average hydraulic gradient is 0.5 for the duration of the rising part of the flood hydrograph. From Darcy's law, and using an effective porosity of 0.25, the average linear velocity of water entering bank storage would be:

$$v = \frac{Ki}{n} = \frac{(57)\ (.5)}{.25} = 114 \text{ ft/day}$$

Figure 2.4-11 of the Wolf Creek Final Safety Analysis Report (FSAR) illustrates the hydrograph of the July 1951 flood of the Neosho River. This is one of the largest floods on record which occurred prior to construction of the John Redmond dam. Because of the regulatory effect of the dam, most subsequent floods are expected to be much smaller than this one. Reference to this extreme flood of July 1951 is made only for the purpose of estimating an approximate maximum value for the duration of the rising part of the flood hydrographs which may be expected on the Neosho River. It should be emphasized that extreme floods on the Neosho River, especially if unregulated, overflow the river banks and cause flooding of the entire floodplain of the Neosho River which is in excess of a mile wide in many areas. During periods of overbank flooding when the entire floodplain is inundated, vertical downward infiltration of water will occur over the entire floodplain for distances up to a mile or more from the normal river channel. Under such circumstances estimations of horizontal migration of water into bank storage from the main stream channel are meaningless for all practical purposes.

The July 1951 flood has three peaks. The rise time for the first peak was half a day. The total rise time for the three peaks can be estimated to be about one day. Thus, for about one day stream water would be entering bank storage. From the average estimated seepage velocity of 114 feet per day, the average distance penetrated into the river banks would be about 114 feet during the storm runoff period. For most floods the average penetration distance can be expected to be less. Thus, the statement that the horizontal migration distance of river water back into the alluvium is "on the order of up to 100 to 200 feet" is a reasonable estimate for high-water conditions of the Neosho River.

Reference: Freeze and Cherry, 1979, Ground water, Prentice-Hall, Inc.

Question 240.19

In the first sentence of the last paragraph on Page 2.4-12 is written, "where it is saturated, the weathered bedrock (except limestone) has a greater permeability than the overlying soil zone". Please provide data to support this statement because comparable values for soil and bedrock are not presented in Table 2.4-7 nor anywhere else in relevant position of the text. Also, it is inferred (in the same sentence) that weathered limestone members probably do not exhibit permeability greater than or equal to the soil or bedrock shale members. Yet the latter are often confining units of the limestone aquifers. Furthermore, data presented in Table 2.4-7 show that the Plattsmouth Limestone has permeabilities approximately one to two orders of magnitude greater than some weathered shale members. Please explain these contradictions.

Response

Laboratory permeability tests were conducted on samples of the soils overlying the Heumader Shale and the Plattsmouth Limestone members. The results of these tests are presented in Wolf Creek Final Safety Analysis Report (FSAR) Table 2.5-35. In this table the soil samples obtained from Boring HS-6 overlie the Heumader Shale. All other soil samples were obtained from locations where the soil overlies the Plattsmouth Limestone. As described below, the test data confirm that the weathered bedrock has a greater permeability than these overlying soils.

The laboratory permeability test data indicate a permeability of 5.6×10^{-8} cm/sec for soil over Heumader shale and lower in the case of a test which had no flow in two days. Table 2.4-7 of the ER(OLS) indicates a value of permeability of 6×10^{-6} cm/sec for the weathered zone (0-20 feet depth) of the Heumader Shale. This indicates that the permeability is significantly greater for the weathered shale zone than for the overlying soil.

Laboratory test data indicate values of permeability for soils overlying the Plattsmouth Limestone ranging from 5.6×10^{-9} to 1.1×10^{-7} cm/sec. Table 2.4-7 of the ER(OLS) indicates the average permeability of the Plattsmouth Limestone as 2×10^{-5} cm/sec for the depth interval 0-20 feet. Here again the data indicate that overlying soil is significantly less permeable than the underlying bedrock.

Regarding the relationship between the permeability of saturated weathered bedrock and overlying unsaturated soil, the moisture content θ and the permeability K of unsaturated soils are functions of the soil moisture pressure head Ψ which is negative for unsaturated soils. Because $K = K(\Psi)$ and $\theta=\theta(\Psi)$, it follows that $K=K(\theta)$. Thus, the permeability of the unsaturated soil increases with increasing moisture content and reaches a maximum value when the soil is completely saturated. Dry soils at the land surface characteristically have a relatively low permeability because of their low moisture content. In the case of soils which contain swelling and shrinking clays, however, the average infiltration rate for the soil may be greater (because of the presence of desiccation cracks at the land surface) than the permeability of individual soil samples which are not cracked. Rainfall can thus penetrate the soil surface rapidly at first by way of desiccation cracks. However, the penetration rate is reduced when the bottoms of the cracks are reached and when the soil swells and closes the cracks.

The statement "where it is saturated, the weathered bedrock (except limestone) has a greater permeability than the overlying soil zone", is not intended to infer that weathered limestone members probably do not exhibit permeability greater than or equal to soil or bedrock shale members. Limestone was being excluded from the general discussion of weathered bedrock because the Plattsmouth Limestone was not weathered in the plant site.

Question 240.20

A water level recorder chart is shown on Figure 2.4-13 for a monitor well. Please provide a map showing the wells exact location. What depth and stratigraphic interval does the data represent?

Response

The location of the well is shown on Figure 2.1-27. The depth of the well is 35 feet. Based on a map showing the geology of the area (Figure 2.5-6), the dug well extends into the sandstone unit of the Jackson Park Shale Member.

Question 240.21

Please provide data to support the effective porosity values used to determine average linear velocities in the Plattsmouth Limestone and Shale members. Based on attached references the reported values are unreasonably high.

Response

The porosity values of 0.05 for the Plattsmouth Limestone and 0.20 for the Jackson Park and Heumader Shale members were estimated on the basis of examination of drill cores. It should be emphasized her that porosity refers to fracture porosity and not to unconnected interstitial pores. As reported in Section 2.4.13.3.3 of the Wolf Creek Final Safety Analysis Report (FSAR), the total porosity of nine Heumader shale samples was also measured on the basis of bulk density and found to be 0.15. The effective porosity was estimated to be 80 percent of the total porosity or 0.12 on the basis of Routson and Serne (1972).

If one uses an effective porosity of 0.12 for the shale members rather than 0.20, the average linear velocity would be 13.5 feet per year and the travel time for ground water to travel from the cooling water lake to the outcrop would be about 60 years rather than 100 years. Likewise, if one uses an effective porosity of 0.01 rather than 0.05 for the Plattsmouth Limestone, the travel time for ground water to travel from the cooling water lake to the outcrop would be 480 years rather than 2400 years. A significant point to emphasize here is that even if lower estimated effective porosities are assumed, the average permeabilities of the members are still very low and ground water travel times are extremely long.

Reference:

Routson, R. C., and Serne, R. J., 1972, Experimental support studies for the percol and transport models: Battelle Pacific Northwest Laboratories, Richland, Washington, BNWL-1719.

Question 240.22

Is the Heumader Shale Member considered to be an aquifer or aquitard or both within and proximal to the cooling lake area? Please support your position with data from tables and/or references.

Response

The terms aquifer and aquitard are generally relative terms. For two saturated geologic formations which are in direct contact with each other, the one with the higher permeability could be considered to be an aquifer and the one with the lower permeability could be considered to be an aquitard. Also, a formation with a certain permeability in one region might be considered to be an aquifer, whereas a formation having the same permeability in a different region might not be considered an aquifer. The term aquifer also involves economic factors. An aquifer can be defined as a water bearing geologic formation which is capable of yielding water to wells in economic quantities. In Table 2.4-6 the Heumader Shale Member is indicated as having a permeability of about 3.0×10^{-6} cm/sec and is indicated as yielding less than three gallons per minute to wells. If this limited quantity would be sufficient to supply the water needs of a particular user, then that user might consider the Heumader Shale to be an aquifer. However, if one compares the permeability of the Heumader Shale (3.0×10^{-6} cm/sec in Table 2.4-6) with the permeability the overlying Jackson Park Shale (4.4×10^{-5} cm/sec and 1.9×10^{-5} cm/sec in Table 2.4-6), the Heumader Shale could be considered to be an aquitard.

However, from Table 2.4-7 which gives ranges of permeability for the members of

Jackson Park Shale: 5×10^{-7} to 5×10^{-5} cm/sec

Heumader Shale: 3×10^{-7} to 3×10^{-5} cm/sec

it appears that on the average there is not any significant difference between these two members.

Question 240.23

You state that in-situ permeability tests were performed using falling head methods. These methods however, are subject to numerous problems ranging from construction of the infiltration sump to chemical incompatability of the water used in the test. To assess the validility of the tests run, please provide a detailed description of the methods, techniques, and an analysis of these tests, including construction, completion and development of test wells.

Response

The in-situ permeability tests were conducted in the piezometers installed in boreholes at the Wolf Creek site. The piezometer installations and the methods used for conducting the falling head permeability tests are described in Wolf Creek Final Safety Analysis Report (FSAR) Sections 2.5.4.3.2.2 and 2.5.4.3.2.3.

Boreholes were drilled in rock utilizing NX-wireline core barrels. After completion of drilling operations the water was blown out of the boreholes prior to the installation of piezometers. The piezometers consisted of 0.75-inch I.D. PVC pipe, perforated throughtout the length of the zone being monitored.

Gravel was placed around the piezometers in the monitored zones, and the zones were sealed above and below with bentonite pellets or cement grout. The remainder of the borehole was filled with cement grout or gravel. When more than one piezometer was installed in a boring, this procedure was repeated for each piezometer. The top of the borehole was sealed with cement to prevent entry of surface runoff and to provide protection for the piezometer pipes.

The in-situ falling head permeability tests were conducted in the piezometers using the following procedure:

1. Initial water level readings were recorded to determine the static water level before testing;

2. The piezometer was rapidly filled to the top with water obtained from the New Strawn, Kansas water system. The volumes of water used and time for filling were recorded;

3. Over a period of 20 to 50 minutes, the rate that the water level dropped in the piezometer was recorded by determining the water level readings at even-minute intervals; and

4. Water levels in other piezometers within the boring were rechecked to determine if the piezometers were properly sealed.

The field observations permitted calculation of the permeabilities of the zones monitored by each piezometer. The field data were reduced and analyzed to obtain values of transmissivity (T) and permeability (K) using the methods of Ferris et al (1962), and Cedergren (1967).

The Ferris (1962) method used a plot of (s) versus ($1/t_m$) on arithmetic graph paper to determine the transmissivity (T) by the equation

$$T = \frac{114.6\, q \frac{1}{t_m}}{s}$$ where s = residual head and t_m = measurement time

The permeability K was determined by the equation $K = \frac{T}{m}$ where m is the saturated thickness tested.

The Cedergren (1967) method employed the basic time lag Tlag from a semi-log plot of h/ho. The shape factor F was determined by:

$$F = \frac{2\pi L}{\ln\left(\frac{L}{R}\right)}$$ where L = slotted interval and R = radius

The permeability K was determined by

$$K = \frac{A}{F\ \mathrm{Tlag}}$$ where A = 0.00326

<u>References</u>:

Ferris, J. G., D. B. Knowles, R. H. Browne, and R. W. Stallman. 1962. Theory of aquifer tests: U.S. Geol. Surv. Water-Supply Paper 1536-E.

Cedergren, H. R. 1967. Seepage, Drainage and Flow Nets. John Wiley & Sons, New York.

Question 240.24

In Section 2.4.2.4.2 you state that seepage rates from the cooling lake will not increase due to quarrying of Plattsmouth and Toronto Limestones prior to filling. As most of the restriction to flow is reportedly caused by the overburden materials which will be removed during quarrying, your conclusion about the seepage rates appears to be unsupported. Please provide the rationale for this statement.

Response

The statement in Section 2.4.2.4.2 is that quarrying of portions of the Plattsmouth and Toronto Limestones during construction will not "significantly" increase the rate of seepage after the filling of the cooling lake.

In Section 2.4.2.4.2 the seepage rates Q were estimated by use of the Darcy equation Q = kia where (k) is the permeability of the medium transmitting the water flow, (i) is the hydraulic gradient and (a) is the area of the aquifer in a plane normal to the direction of ground water flow. The permeability values listed in Table 2.4-10 are conservative in that the values used in the calculations are those of the bedrock units and are not permeability values of the overburden.

Section 2.4.2.4.2 does not state that most of the restrictions to flow is caused by the overburden materials, and the effect of overburden materials was not included in the seepage calculations. If the flow restrictions caused by overburden materials were to be taken into account, the calculated seepage rates would be less than those reported in Section 2.4.2.4.2 and Table 2.4-10.

The quarries are located primarily in Sector 4 of Figure 2.4-17. From Table 2.4-10 the estimated seepage through Sector 4 for the Plattsmouth Limestone is .00269 ft^3/min. For the Toronto Limestone the estimated seepage is .0011 ft^3/min. The total estimated seepage for these two members in Sector 4 is, thus, .00379 ft^3/min. The total estimated seepage from the cooling lake is 0.82 ft^3/min. Thus, the estimated seepage from these two formations in the area of the quarries is approximately 0.46 percent of the total estimated seepage from the cooling lake.

If it is assumed that quarrying of the two limestone members has reduced the pathway length by one-half for water to travel from the cooling lake to the formation discharge points in the west slope of the hill, then the hydraulic gradient would be doubled for these two members in Sector 4. By doubling the hydraulic gradient, the seepage rate would be doubled, according to the Darcy equation, thereby increasing the estimated seepage flow rate from 0.00379 to .00758 ft^3/min. This would increase the total estimated seepage rate from the cooling lake from 0.82 to 0.82379 ft^3/min. This represents an estimated increase in flow rate of about 0.46 percent which is not a significant increase in the estimated seepage flow rate from the cooling lake.