2. Geotechnical/Foundation Analysis

Goal: **The intent of this section of the study is to address any issues related to the structural soils and geotechnical information surrounding the dam itself, and within the reservoir area. The study will determine if there are any issues that will prevent this project from continuing to the next phase or any major issues that will need to be taken into account during design that could add significant costs to the project.**

Summary: **At this stage of the investigations there appears to be no major issues that should prevent this project from continuing on to the next stage. However, some areas of concern do exist with regard to the soils at the dam site itself. These issues can be resolved with structural measures and the upgrading of the type of spillway that would be constructed. These resolutions will add additional cost to the project.**

Next Steps: **The next steps include expanding on the geotechnical information by completing borings in the field. Test borings and laboratory testing of soil samples will be needed to develop more detailed information on the subsurface strata and the soil engineering parameters. Verification of the condition/location of the limestone bedrock is also important, especially in the right abutment where the bedrock is relatively shallow.**

Geotechnical and Foundation Considerations

General Geologic Conditions

Geologic and hydrogeologic information was collected from various sources in order to investigate the subsurface conditions in the project area. In addition, well log information in the vicinity of the proposed dam was reviewed and evaluated. Based on this information, it appears that the bedrock consists of limestone and varies in depth from 50′ to over 200′ in the area. The overburden generally consists of glacial outwash overlain by alluvial soils. The overburden is quite variable and consists of both highly permeable sand and gravel as well as low permeability cohesive soils. Pump tests performed in the granular overburden indicate that the granular soils have high coefficients of hydraulic conductivity. The continuity of the granular and cohesive layers is important in evaluating the foundation conditions for the proposed dam and reservoir.

Subsurface Information from Well Logs

Individual well logs (see Appendix B for well logs and location map) near the proposed dam alignment were carefully evaluated. Unfortunately, very few wells are located at the proposed dam site. Based on well log #146650 located in the right abutment area, bedrock was encountered at a depth of 85′ and was described as gray limestone; this depth corresponds to approximately elevation 807. The surface of the bedrock drops rapidly closer to the river. Well log #352317, located closer to the river and approximately 400′ south of well #146650, reported the limestone bedrock surface at a depth of 122′, which would be about elevation 712.

The wells located near the river along the proposed dam alignment were relatively shallow and did not encounter bedrock. Several wells were located near the river a couple thousand feet downstream of the proposed dam centerline. These wells encountered limestone bedrock between approximate elevations 750 and 770.

The well located closest to the left abutment was #312998; unfortunately, no subsurface information was shown on this log other than stating that gravel was present at 40′. The next well closest to the left abutment was #146635, which is located about 700′ upstream of the proposed dam centerline. In this well limestone bedrock was encountered at a depth of 109′, which corresponds to about elevation 760.

Analysis/Subject Matter (continued)

Although detailed information about the limestone bedrock is not reported on any of the well logs, general geologic literature reports that the limestone in the area is likely to have karst features. However, other than in the right abutment, the bedrock is probably deep enough so that it will not significantly impact the foundation of the dam or the ability of the reservoir to retain water. The condition of the bedrock will need to be verified with site-specific borings.

As previously indicated, the overburden conditions vary widely. In well #146650 in the right abutment, the upper 9′ of soil was described as clay, and was underlain by 76′ of sand and gravel. In well #352317, located about 400′ farther to the south, the overburden was 122′ thick, with most of the soil described as clay; however, four layers of sand and/or gravel were reported in this well log, each about 10′ thick.

Two relatively shallow wells were located about 700′ downstream of the main section of the proposed dam; these two wells are #146625 and #146630. Well #146625 was drilled to a depth of 23′ and encountered mostly cohesive material, except for a gravel layer between 13′ and 18′. Well #146630 was drilled to a depth of 35′ and encountered alternating layers of clay and sand.

In well #146635, near the left abutment and approximately 700′ upstream of the proposed dam centerline, mostly sand and gravel were encountered to a depth of 75′, which was underlain by clay and hardpan.

Recommendations

Based on the existing well log information that has been collected, it appears that a variety of granular and cohesive soils will be present in the foundation for the dam. It is believed that these soils will provide adequate strength to support the proposed earth dam with typical side slopes of 3(H) on 1(V). A key trench should be constructed beneath the longitudinal centerline of the dam. Additional subsurface information will be needed to determine the continuity of the granular layers and whether a deeper cutoff will be needed to reduce under seepage.

Although the soils are probably strong enough to support the proposed earth embankment with 3:1 side slopes, foundation loads from a narrower concrete ogee section would be much higher and would probably be greater than the upper soils can support. Based on the limited existing information, strong soils described as "hardpan" or bedrock are located at significant depths below the streambed. Consequently, if a typical concrete spillway section were to be used, extended foundations (driven piles) would probably be needed to support the concrete section; differential settlement between the concrete and earthen sections of the dam would also have to be evaluated. Because of the significant additional cost of an extended foundation and the potential differential settlement, it is suggested that a chute spillway be used instead of an ogee section. The chute spillway could be constructed at a flatter slope, similar to the side slope of the earthen dam. It is also recommended that the chute spillway be supported on roller compacted concrete (RCC) in order to provide a more stable foundation for this critical part of the dam. This is important because of the constant flow of water over the spillway if it is ungated, and because of the vibrations that will occur during gate operations if a gated spillway is constructed.

Additional Investigations

Test borings and laboratory testing of soil samples will be needed to develop more detailed information on the subsurface stratigraphy and the soil engineering parameters. Verification of the condition of the limestone bedrock is also important, especially in the right abutment where the bedrock is relatively shallow. The additional borings will be used to determine the following:

- need for underseepage cutoff, type of cutoff wall and depth.
- need for any soil treatment or overexcavation needed for foundation support of the dam and spillway.
- location of a borrow source for construction of the dam.
- strength, compressibility and permeability of the foundation soils and borrow material to be used in the earthen dam.
- depth and condition of potentially karst limestone bedrock.

Analysis/Subject Matter

3. Hydrology and Hydraulic

DAM LOCATION AND SIZING

Goal: **The intent of this section of the study is to analyze items associated with the sizing of the dam and reservoir capacity, determine potential potable water yield, and analyze the potential for flood control.**

Summary: **At this stage of the investigations there appear to be no issues that should prevent this project from continuing to the next stage. The size of the dam spillway will range from 700 to 1,100 feet long, for pool elevations of 870 and 875 respectively. The potential water yield from the reservoir is about 50 MGD during severe drought conditions. Flood control is a possibility with controlled gates on the dam, however a detailed study of gate operations would be required to optimize the multiple objectives of the dam.**

Next Steps: **Provide the yield numbers to potential customers. Determine if a gated structure is cost effective and can meet multiple objectives including flood control, water supply and recreational needs. Address the pros and cons of the final pool elevation in a range between 870 and 875 feet. Complete additional needed calculations from borings to be completed as a portion of the next steps in the Geotechnical Section.**

Dam Layout

The valley associated with the West Fork White River through Anderson, Indiana is well defined with significant relief on both sides which present optimal locations for a dam across the river. The current proposed location is along an alignment just west of SR 9 where the length of the dam would be at a minimum. At this location, the width of the dam would be approximately 2800 ft. The primary intent of the dam is to provide water supply, economic benefits and recreational benefits.

The basis for design of the pertinent features of the proposed dam at Anderson is the requirement to safely pass the Probable Maximum Flood (PMF) as mandated by the Indiana Department of Natural Resources (IDNR). Some of the features of the dam are set by the local topography and by previous evaluations. The maximum top of dam based on the elevations of the adjacent valley walls. Previous evaluations indicated that pool levels at 870 ft and 875 ft would provide sufficient storage for water supply purposes. Consequently, normal pools at 870 and 875 ft are considered in this study for detailed evaluation. PMF studies were then conducted to determine the lengths of the uncontrolled spillway for these two crest levels to safely pass the PMF without overtopping the top of dam set at 890 ft. Additional studies were also conducted to determine the required spillway configurations with gates; for these studies the spillway crest elevations were set 10 feet lower. The disadvantage of using gated structures is that these would need to be continuously operated to pass normal flows and control the pool to the desired elevation; the advantage is that you could use the structure for flood control purposes as well and better manage the water supply during periods of drought.

Analysis/Subject Matter (continued)

As discussed in the geotechnical section of this report the geologic conditions are not well suited the use of an ogee spillway because of its narrow base width and consequently large foundation pressures. For this reason, a chute type spillway is utilized for this study. The discharge coefficient of the associated broad crested spillway is 20 -25% lower than with an ogee spillway, and thus less efficient. Also, since the spillway section is relatively long, it will be cost effective to use a roller compacted section for the spillway.

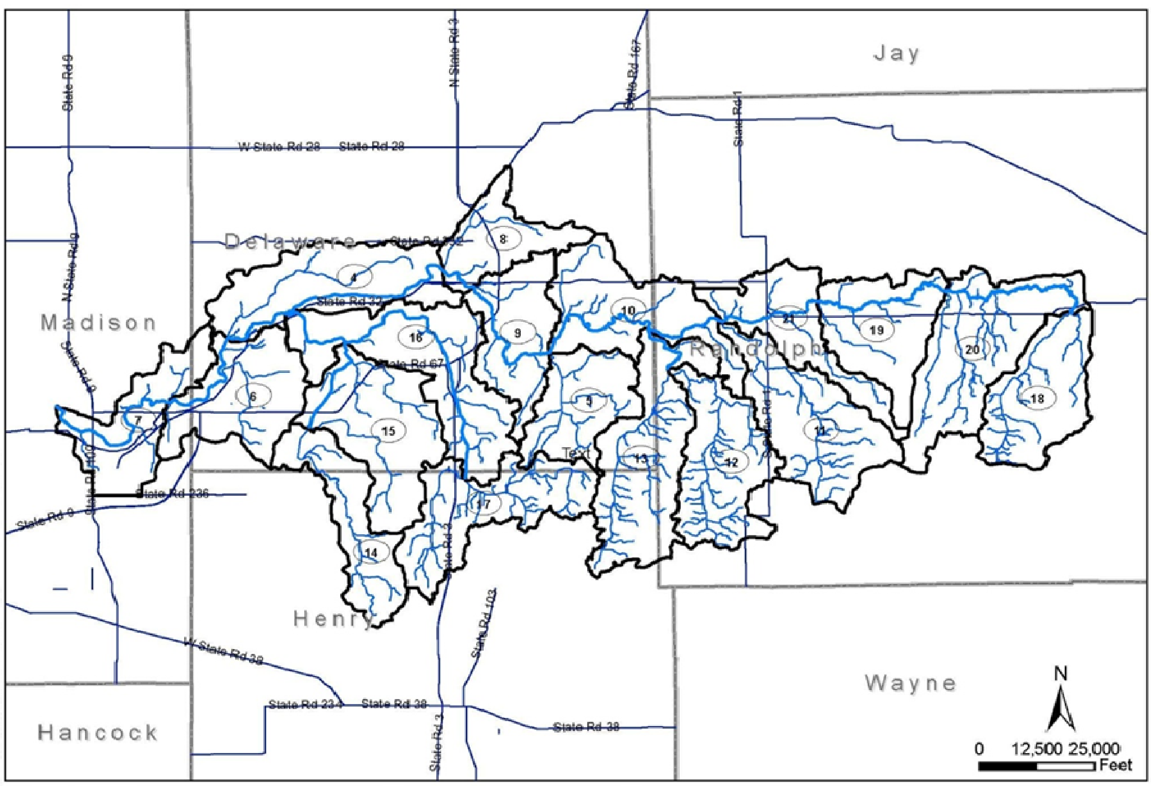
The principal data required for the analysis is the estimate of the PMF inflow from the contributing drainage area which is about 400 sq. miles. There is a USGS gage just downstream of the dam site which can be used to determine peak discharges associated with the more frequent events. The principal assumptions and features associated with the estimation of the PMF and higher frequency storm events are presented in this section.

Probable Maximum Flood Study

The PMF is determined based on the "General Guidelines for New Dams and Improvements to Existing Dams in Indiana", 2001 Edition, IDNR

Watershed Delineation

The Anderson Dam watershed is approximately 400 square miles. In order to develop a reliable estimate of the PMF, the drainage area to the lake was divided in to 18 sub basins (see Figure 3.1). The sub basin delineations were accomplished in the ArcGIS platform using topographic data from the USGS national elevation dataset.



Sub basin No./Name	Area Sq. Miles
4	26.03
5	16.96
6	21.75
7	18.43
8	13.44
9	18.41
10	24.6
11	25.9
12	22.83
13	29.33
14	17.73
15	26.89
16	31.6
17	23.89
18	21.03
19	20.5
20	29.69
21	17.79
Total	406.8

Figure 3.1 Anderson Lake Drainage Area Delineation

Probable Maximum Precipitation (PMP)

The depth of rain for the Probable Maximum Precipitation (PMP) is determined using the PMP isohyetals in Appendix D of IDNR's guidelines (see Table 3.1).

Area	Duration (hours)		
(mi2)	6	12	24
10	27.0	31.0	33.0
200	19.0	22.5	24.0
1000	13.5	17.0	19.0
5000	8.5	11.5	13.5

Table 3.1: PMP Estimates for the Study Area

Since the watershed is greater than 10 square miles, the PMP depth is reduced using procedure outline in HMR 51 (see Figure 3.2).

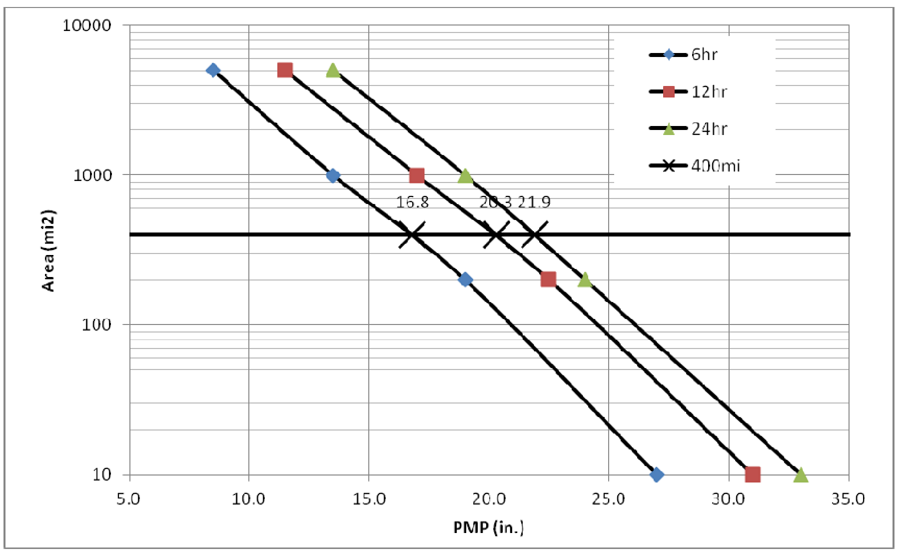


Figure 3.2: Depth-Area Relation for the PMP

The 24-hr PMP storm is used for the PMF generation since the time of concentration is expected to be larger than 12 hours. The SCS Type II distribution is used for the temporal distribution.

PMF Analysis

The PMP depth in conjunction with the SCS method is used as the meteorological model in HEC-HMS. Loss computations are carried out using the runoff curve number (RCN) method developed by the SCS. For this purpose, hydrologic soil type and land use delineations were performed for each sub basin. Data for the hydrologic soil type was obtained from the Soil Survey Geographic (SSURGO) Database using the NRCS online interface (Figure 3.3). The land cover data is extracted from the National Land Cover Data (NLCD) from the USGS website (Figure 3.4).

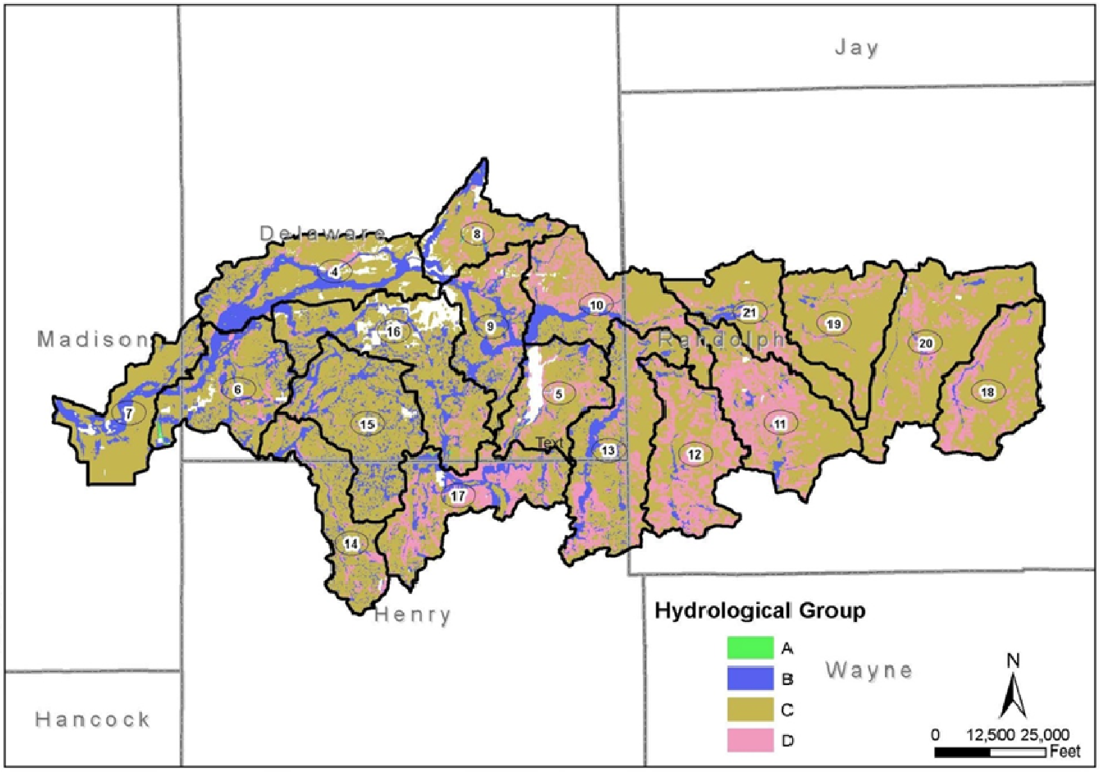


Figure 3.3: Soil Use Distribution

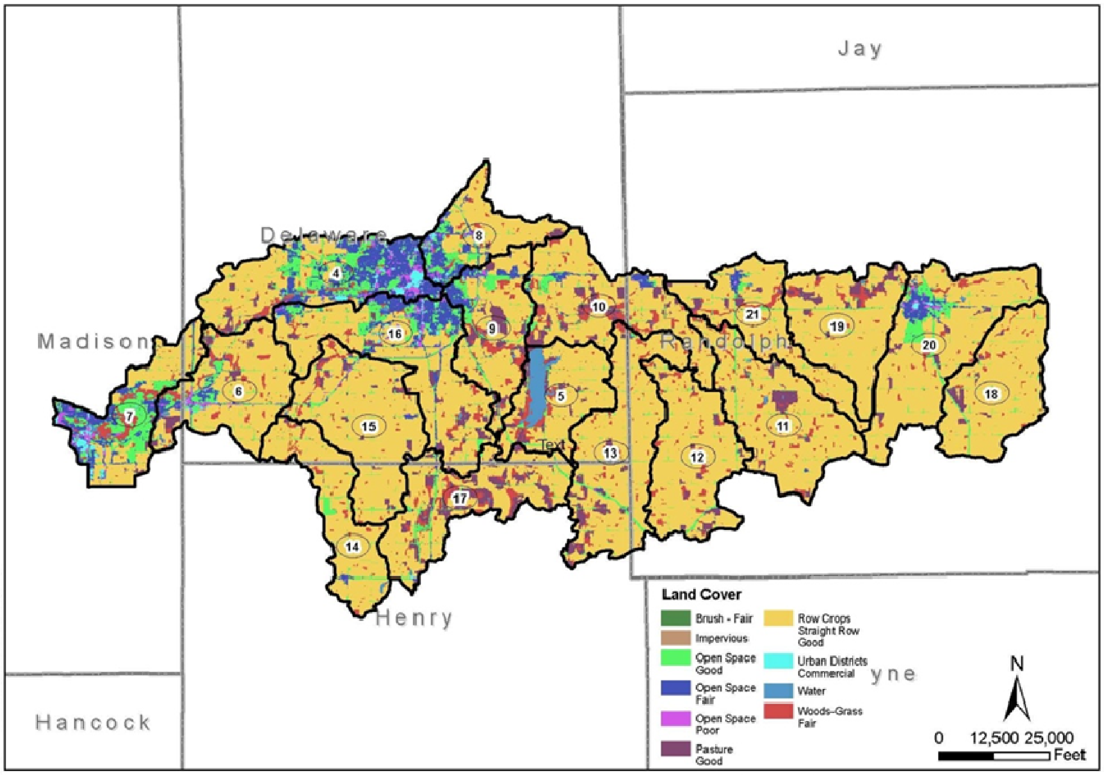


Figure 3.4: Land Use Distribution

The SCS method is based on a matrix of curve numbers (CN) that have been developed for various land uses and soil groups. The curve numbers utilized in this study correspond to average antecedent moisture conditions (AMC II). The land use and soil group maps were overlaid on the watershed maps. The areas of matching land use and soil types are accumulated and a curve number assigned to the specific area. A weighted curve number is then determined for each sub basin (see Table 3.2).

The unit hydrograph theory was used to convert the runoff volume into a flood hydrograph that would flow into the reservoir created by Anderson Dam. Though IDNR guidelines suggest the use of the SCS unit hydrograph, it is felt that this would be inappropriate for watershed areas as large as 400 sq. miles. The SCS unit graphs were originally developed to be used for drainage areas that are in the order of 10 sq. miles. For this reason, the unit hydrograph method developed by the Pittsburgh District of the Corps of Engineers for PMF studies of large drainage areas in the State of Ohio is utilized. This is reasonable since the east central areas of Indiana (where this project is located) are hydro-meteorologically similar to Ohio. The Corps method is based on developing 6-hour unit hydrographs for each sub basin that are dependent on specific geometric features of the basin. For more detail, 1-hr unit hydrographs were developed from the 6-hour graph using the S-Curve method. Details of the unit hydrograph generation are provided in Appendix A.

Once the rainfall patterns, curve numbers and unit hydrographs were developed, the HEC-HMS model was used to generate the probable maximum flood (PMF) from each sub basin. The schematic of the HEC-HMS model is shown in Figure 3.5. These flows are then combined and routed through stream reaches as required to generate the PMF inflow. The stream reach routing is accomplished using the SCS Lag Method.

Sub basin No./Name	Composite RCN
4	77.7
5	72.5
6	79.2
7	74.9
8	79.4
9	78.6
10	80.9
11	83.4
12	85.0
13	82.0
14	83.0
15	81.8
16	80.5
17	79.9
18	81.9
19	80.3
20	80.8
21	82.1
Basin CN	80.4

Table 3.2: RCN Values

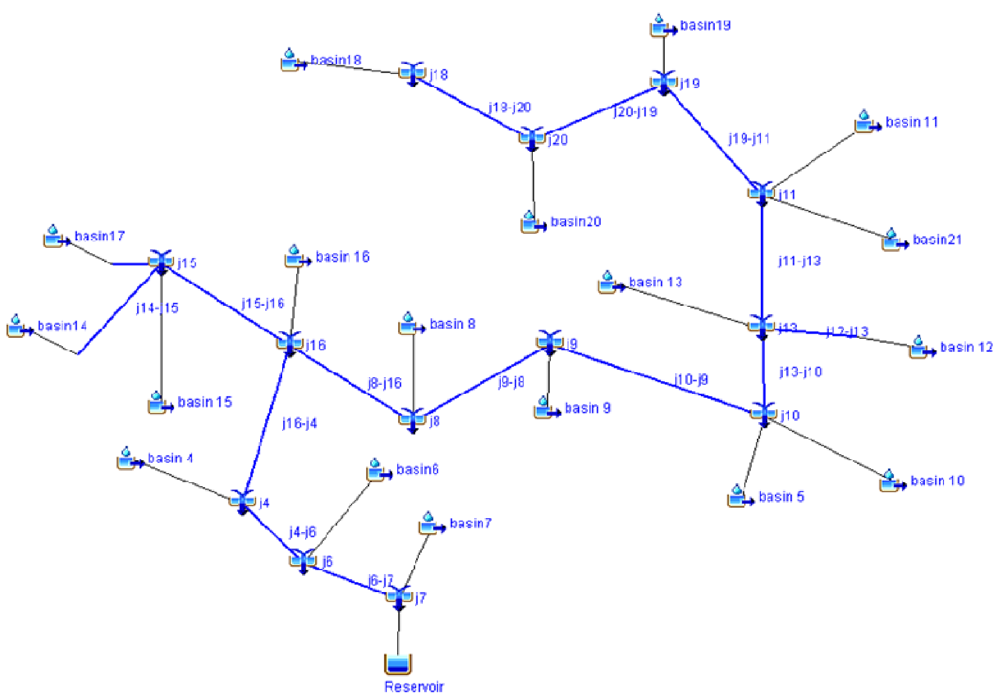


Figure 3.5 HEC-HMS Model Schematic

The PMF inflow hydrograph to Anderson Reservoir as generated by the HEC-HMS program is shown in Figure 3.6. The peak inflow is 191,695 cfs. The total inflow volume is 416932 ac-ft which corresponds to a depth of 19.2″ over the watershed area.

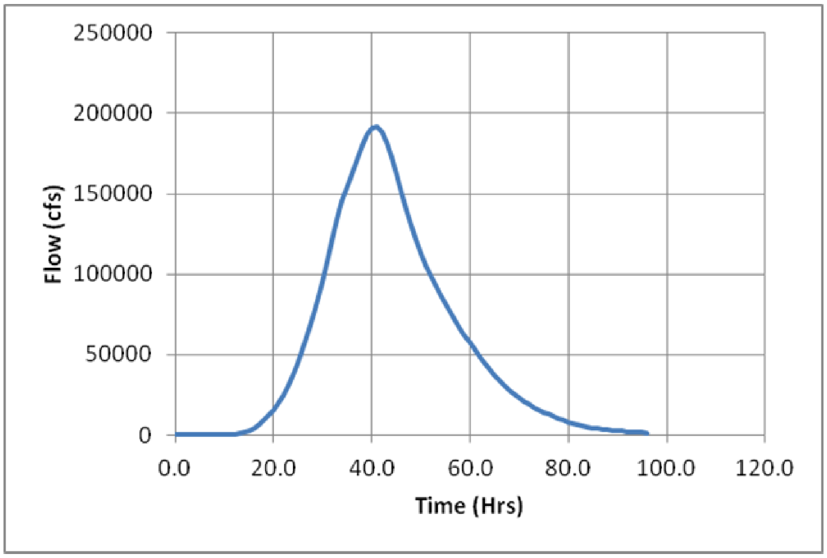


Figure 3.6 PMF Inflow Hydrograph

Reservoir Flood Routing

The HEC-HMS computer package was used to route the PMF through the reservoir. The flood routing is based on the level pool method for which the principal parameters are storage capacity of the reservoir and the hydraulic capacity of the broad-crested spillway system. The storage curve for Anderson Dam is shown in Figure 3.7.

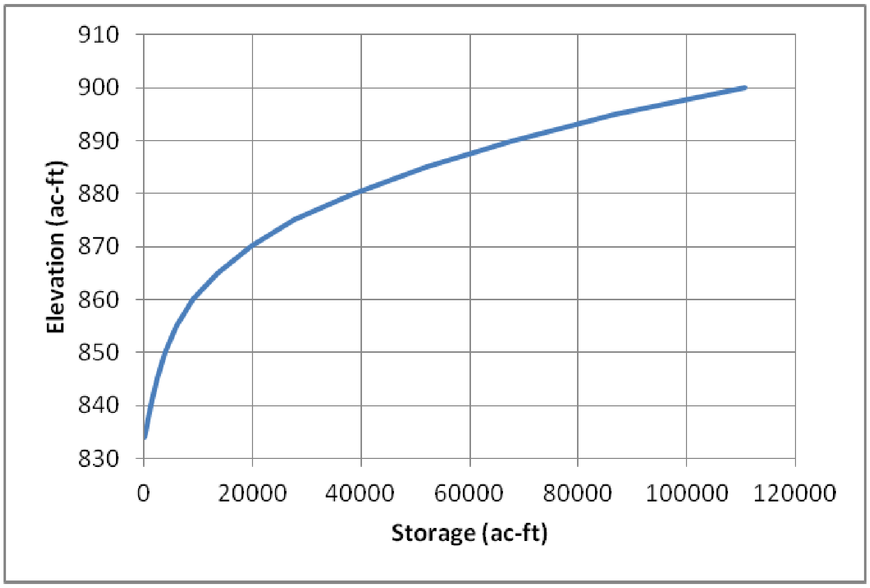


Figure 3.7 Anderson Dam Storage Curve

Results of PMF Analysis/Reservoir Routing

The results of routing the PMF inflow through the reservoir with the pool at 870 ft and a spillway length of 700 ft indicate a peak elevation of 889.6 ft and a peak outflow of 182,253 cfs. Since the top of the embankment is at 890.0 ft, the dam will not be overtopped by the PMF. Similarly, the results of routing the PMF inflow through the reservoir with the pool at 875 ft and a spillway length of 1100 ft indicate a peak elevation of 889.7 ft and a peak outflow of 185,741 cfs. Since the top of the embankment is at 890.0 ft, the dam will not be overtopped by the PMF.

This analysis indicates that for the PMF to be contained by the dam, an uncontrolled spillway length of 700 ft and 1100 ft is required with the pools at 870 ft and 875 ft, respectively.

Alternatively, if gates are provided on the spillway, the crest of the spillway could be lowered and the gates suitable operated to provide the required pool elevation (870 ft or 875 ft). If the crest of the controlled spillway is lowered to 860 ft, a clear spillway width of 360 ft is required to keep the PMF peak elevation below the top of dam. For this condition, the peak pool level is 889.6 ft and the peak outflow is 173,850 cfs. It is assumed that the gates would be wide open (with the bottom of the gate above 890 ft) for the PMF condition. The gated spillway configuration could be achieved by having 12 bays, each 30 ft wide, equipped with suitably sized radial or vertical lift gates that would be operated by an electrical/mechanical system mounted on a bridge deck above the spillway. The overall spillway width would be 415 ft assuming 5 ft piers between bays. The advantage of using gates is that such a system could be used for flood control as well, by lowering the pool by an additional 10 ft in advance of a flood and retaining the stored water as required. The disadvantage with this system is that the gates will constantly hold back and pass water under them that would result in recurring operations and maintenance costs. A comprehensive spillway operations plan will need to be developed to accomplish all of the required objectives. It is possible that with an appropriate spillway operations plan, the peak 100-yr discharge could be substantially reduced downstream of the dam. This could result in areas and properties downstream of the dam to be removed from the 100-yr floodplain. This would potentially eliminating the need for flood insurance on affected properties.

Pool Levels for the 100-yr Event and Possible Floodplain Benefits

For determining the pool levels for the 100-yr, 50-yr, 25-yr, and 10-yr events, a simplified HEC-HMS model using a smaller number of sub basins was utilized along with the SCS curve number method for determining the basin losses, and the built-in SCS unit hydrograph for the rainfall-runoff computation which is developed using the lag method. The meteorological model uses the NOAA Atlas 14, 24-hr duration storm depths for the associated frequency in conjunction with the SCS Type II rainfall distribution. These inflow hydrographs are routed through the lake using the level pool method previously described with the pool at 870 ft and an uncontrolled spillway with a length of 700 ft. The peak inflow/outflow discharges and associated pool levels are shown in Table 3. The comparison of the peak inflow values with the Indiana StreamStats estimates (based on flow data at the USGS Gage 03348000 just downstream of the proposed dam) is quite good and is shown in Table 3.3.

	NOAA Atlas 14	HEC-HMS Model			STREAMSTATS
Return Period (Yr)	24-Hr PPN (in)	Peak Inflow (cfs)	Peak Outflow (cfs)	Pool Elev (ft)	Peak Inflow (cfs)
500	6.97	31630	29800	875.9	30200
100	5.70	23615	22627	874.9	23600
50	5.17	20362	19570	874.4	20800
25	4.64	17184	16563	874.0	17900
10	3.97	13447	12895	873.4	14100

Table 3.3: Parameters/Results for Higher Frequency Events

There is about a 5% reduction in the 100-yr peak flow downstream with the uncontrolled spillway as a result of the storage effects associated with the pool. This reduction is not very significant in terms of removing areas from the 100-yr floodplain downstream. However, as stated previously, there would be significant reductions if a gated spillway structure were used for whose operations flood control is made a specific objective. It is difficult to quantify these benefits at this stage because a detailed study would be required determine the optimum gate operations policy.

Potable Water Yield

The principal data utilized to analyze the firm yield from the proposed reservoir at Anderson is the stream flow data at USGS Gage 03348000 on the West Fork of White River just downstream from the proposed location of the dam. Daily flow data is available from 10/1/1925 though there are data gaps between 10/1/1926 – 3/31/1932 and 12/21993 – 9/30/2006.

Flow Data Analysis

The flow exceedance curve for the average annual flow for all available water years and the flow exceedance curve for all available daily flows are shown in Figure 3.8.

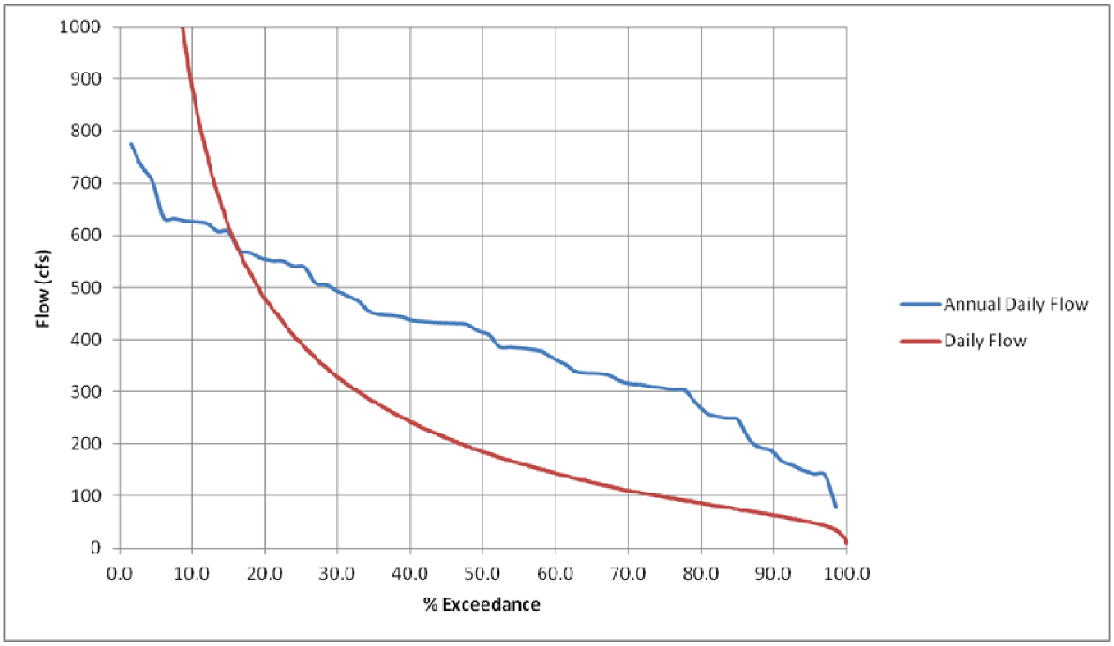


Figure 3.8 Flow Exceedance Curves

The above figure indicates that there are significant variations in the daily flows that are smoothed over when using average annual daily flows. Similar differences can be expected even with monthly averaged daily flows so all analysis for yield calculations are conducted with the daily flow data. Since continuous daily data is preferred, the period of 10/1/1932 to 9/31/1993 which covers 62 contiguous water years is utilized.

The analysis of the data indicated the following:

a. On a water year basis, considering all available water years:
    i. The lowest one-year inflow is in 1940, followed by 1953, and 1933.
    ii. The lowest continuous two-year inflow is 1939-1940, followed by 1933-1934, followed by 1940 -1941.

b. On a daily flow basis, considering the data for the contiguous period of 10/1/1932 to 9/31/1993:
   i. the lowest 180-day running averages begin in June 1940
   ii. the lowest 270-day running averages begin in June 1944
   iii. the lowest 360-day running averages begin in June 1940
   iv. the lowest 550-day running averages begin in June 1940
   v. the lowest 720-day running averages begin in Oct/Sept 1939

Based on the above, the critical drought period for water supply purposes is likely to be between September1939 to February 1945.

Yield Analysis

Reservoir yield analysis is based on the water balance principles which use daily inflow as the primary data. Proper accounting is made for additional losses such as net evaporation and seepage. The reservoir capacity curve is adjusted for potential loss in storage due to future sedimentation. A spreadsheet program is developed to automate the computations. The input variable is then the daily yield which is adjusted through trial and error such that for the specific pool level under consideration, the reservoir does not run dry for the entire period of record under consideration. Specific data utilized in these computations is presented below.

Average monthly net evaporation is estimated based on monthly estimates of precipitation in Anderson and projected evaporation rates in Indianapolis (based on data from Geist Reservoir) as shown in Table 3.4. These monthly rates are converted to daily rates for use in the spreadsheet program. Reservoir surface area used is that associated with the pool level under consideration.

	Evaporation Estimates (in inches)		
	Av Precip - Anderson	Average Evap	Net Evap
Month	Sewage Plant	Indianapolis	
Jan	2.09	0.39	0
Feb	2.28	0.61	0
Mar	3.24	1.41	0
Apr	3.84	2.57	0
May	4.08	4.07	0
Jun	4.21	4.96	0.75
Jul	4.28	5.46	1.18
Aug	3.43	4.45	1.02
Sep	2.95	3.11	0.16
Oct	2.77	1.91	0
Nov	3.68	0.79	0
Dec	2.97	0.38	0
		Total	3.11
Precipitation - Data Period = 1971 - 2010			
Evap - Data Period = 1911 - 1962, Indianapolis			
from "Lake Evaporation in Illinois", Illinois Water Survey, 1967			

Table 3.4 Net Evaporation Calculations

For seepage, a constant seepage loss of 2 cfs is assumed. This is reasonable since geologic investigations did not reveal any connections to karst features that would result in excessive seepage losses. Annual sedimentation rate at the proposed reservoir is estimated based on available data at Eagle Creek (27 ac-ft/yr) and Geist (42 ac-ft/yr).

Assuming a straight line relationship to the drainage area, the annual sedimentation at the proposed reservoir is estimated to be 74 ac-ft/yr. Assuming a 75-year life for the proposed reservoir, the total volume of sediment inflow is estimated to be 5550-ac-ft which is linearly spread through the elevation range up to the normal pool (for these calculations, a normal pool of 870 ft was used). The storage-capacity curve for the reservoir was adjusted to reflect this loss of storage.

Since the release for water supply from the dam will be made into the river itself, there is no low-flow limitation that will need to be met. However, there are losses associated with flow in the West Fork White River between the release point and the location at which the water is withdrawn for consumptive use.

The analysis using the spreadsheet program indicates that for the 870 ft pool elevation, the firm yield from the reservoir is about 75 cfs (48 MGD). For the 875 ft pool elevation, the firm yield is 88 cfs (57 MGD). For both these cases, the permissible minimum reservoir elevation is set to 845 ft. The critical period with the lowest reservoir elevation for the 870 ft pool elevation occurred on 2/11/1945, and on 12/23/1941 for the 875 ft pool elevation.

Assuming a 5 MGD loss in the river between the release point and the location for withdrawal of water, the available water at the location of withdrawal would be 43 MGD with the pool at 870 ft and 52 MGD with the pool at 875 ft.

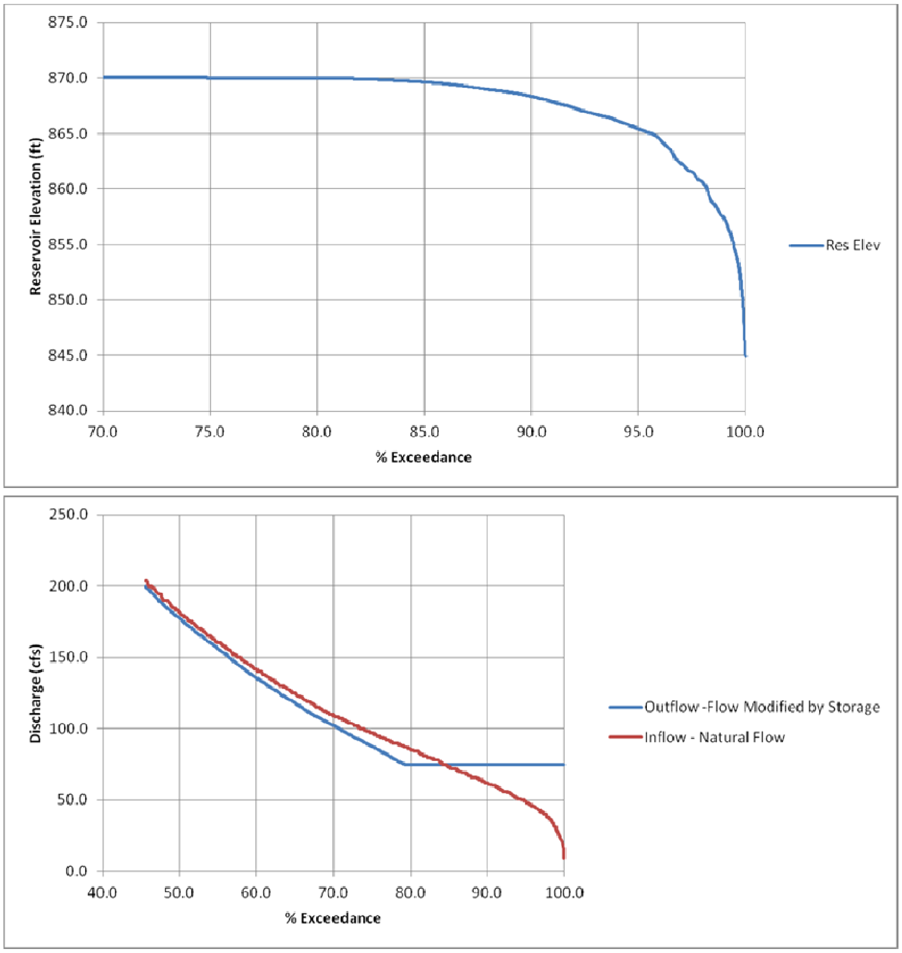


Figure 3.9 Exceedance Curves with Pool at 870 ft

Analysis/Subject Matter (continued)

It should be noted that the pool is filled to the 870 foot pool elevation about 85% of the time. The inflow/outflow exceedance curves show that reservoir storage is used about 20% of the time.

Similar curves for the pool at 875 ft are shown in Figure 3.10.

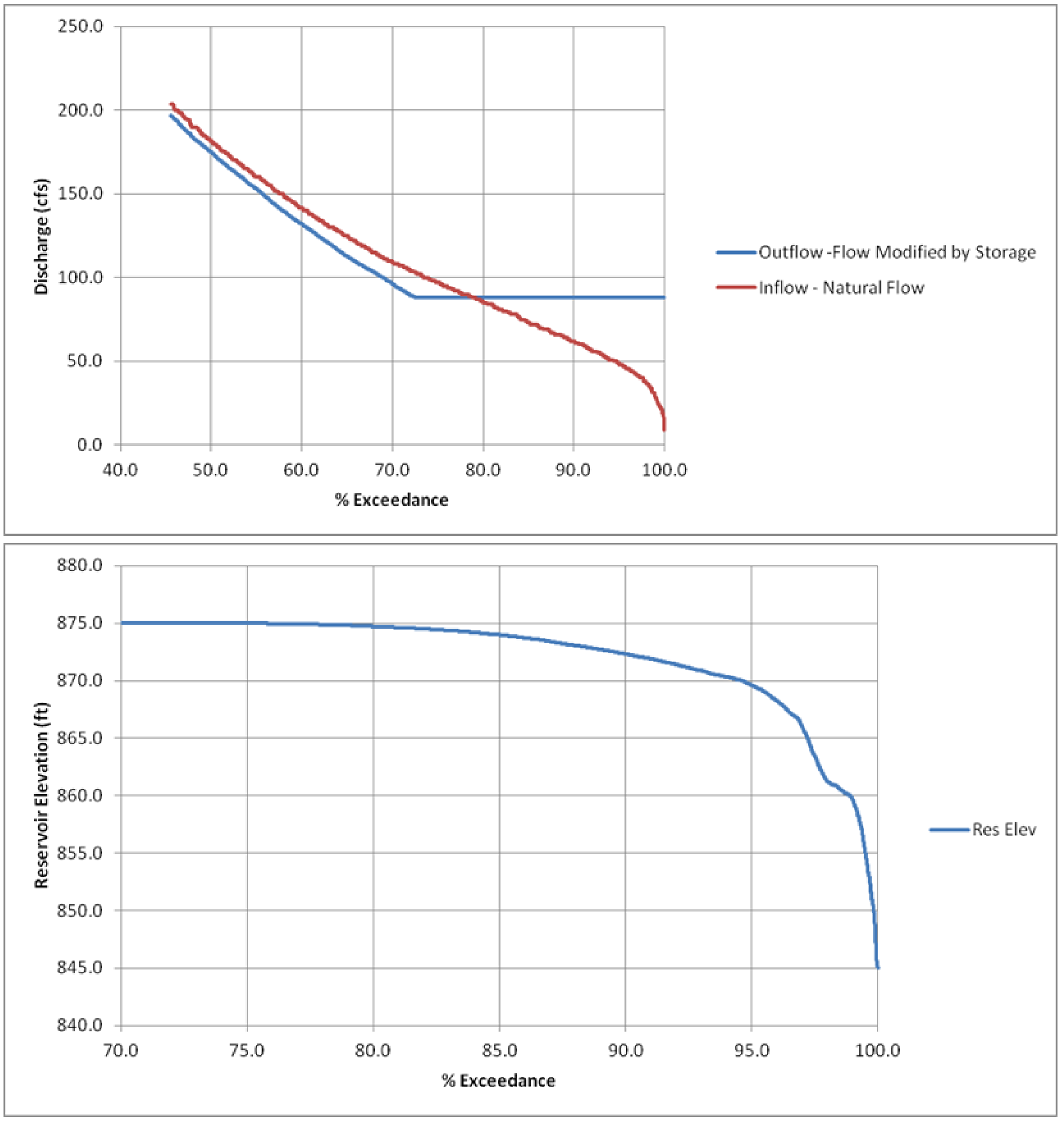


Figure 3.10 Exceedance Curves with Pool at 875 ft

The above curves show that the pool is filled to the 875 foot pool elevation about 80% of the time. The inflow/outflow exceedance curves show that reservoir storage is used about 30% of the time.

In conclusion, the firm yield of the reservoir is 43 MGD with the 870 pool and 52 MGD at the 875 pool assuming 5 MGD losses in the river system between the reservoir and point of withdrawal. These lower yields only occur during 15% to 20% of the time. During the remaining 80% to 85% of the time, the excess water flow could be used for pumping operations to create water retention at other locations in the area such as Geist and Morse Reservoirs.