US Army Corps
of Engineers
Hydrologic Engineering Center

Use of Non-Sequential Techniques in the Analysis of Power Potential at Storage Products

July 1981

TP-80

REPORT DOCUMENTATION PAGE

Form Approved OMB No. 0704-0188

1. REPORT DATE *(DD-MM-YYYY)* July 1981	**2. REPORT TYPE** Technical Paper	**3. DATES COVERED** *(From - To)*

4. TITLE AND SUBTITLE Use of Non-Sequential Techniques in the Analysis of Power Potential at Storage Projects	**5a. CONTRACT NUMBER** **5b. GRANT NUMBER** **5c. PROGRAM ELEMENT NUMBER**
6. AUTHOR(S) Gary M. Franc	**5d. PROJECT NUMBER** **5e. TASK NUMBER** **5F. WORK UNIT NUMBER**
7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES) US Army Corps of Engineers Institute for Water Resources Hydrologic Engineering Center (HEC) 609 Second Street Davis, CA 95616-4687	**8. PERFORMING ORGANIZATION REPORT NUMBER** TP-80
9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)	**10. SPONSOR/ MONITOR'S ACRONYM(S)** **11. SPONSOR/ MONITOR'S REPORT NUMBER(S)**

12. DISTRIBUTION / AVAILABILITY STATEMENT
Approved for public release; distribution is unlimited.

13. SUPPLEMENTARY NOTES
Presented at "Waterpower 81 Conference", Washington, D.C., June 1981.

14. ABSTRACT
An electronic computer procedure is described that uses an estimate of power storage available at a dam site to make estimate of average annual energy and dependable capacity using a non-sequential methodology. Comparisons of this method against the more traditional sequential routing analysis are performed.

15. SUBJECT TERMS
non-sequential analysis, critical drawdown period, storage adjustment, regression analysis, reservoir reallocation, dependable capacity, average annual energy

16. SECURITY CLASSIFICATION OF:			**17. LIMITATION OF ABSTRACT**	**18. NUMBER OF PAGES**	**19a. NAME OF RESPONSIBLE PERSON**
a. REPORT U	**b. ABSTRACT** U	**c. THIS PAGE** U	UU	24	**19b. TELEPHONE NUMBER**

Use of Non-Sequential Techniques in the Analysis of Power Potential at Storage Projects

July 1981

US Army Corps of Engineers
Institute for Water Resources
Hydrologic Engineering Center
609 Second Street
Davis, CA 95616

(530) 756-1104
(530) 756-8250 FAX
www.hec.usace.army.mil

TP-80

USE OF NON-SEQUENTIAL TECHNIQUES IN THE ANALYSIS OF POWER POTENTIAL AT STORAGE PROJECTS

Gary M. Franc[1]

INTRODUCTION

The analysis of hydropower storage projects has traditionally been performed by use of sequential reservoir routing techniques, whereas the use of non-sequential techniques has been traditionally limited to the study of run-of-river type projects.

While individual power storage projects should be analyzed by detailed sequential routings when sufficient funds and detail are available, the non-sequential or flow-duration technique (as modified herein) can be made to somewhat approximate the results of a sequential routing by modifying the flow duration curve to represent outflow conditions.

The intent of this paper is to briefly outline the technique developed to enable the analysis of power storage projects using a non-sequential approach and, more importantly, to illustrate the improvement in estimates of energy and capacity from employing the technique.

FLOW-DURATION CURVE ADJUSTMENT

A storage project, in general, accumulates excessive inflows for future use during low flow periods, thereby transforming the inflow-duration curve, based on inflows into the project, into a flatter outflow-duration curve, reflecting the operation and effect of the project's storage as depicted below:

[1]Hydraulic Engineer, Hydrologic Engineering Center, U. S. Army Corps of Engineers, Davis, CA 95616

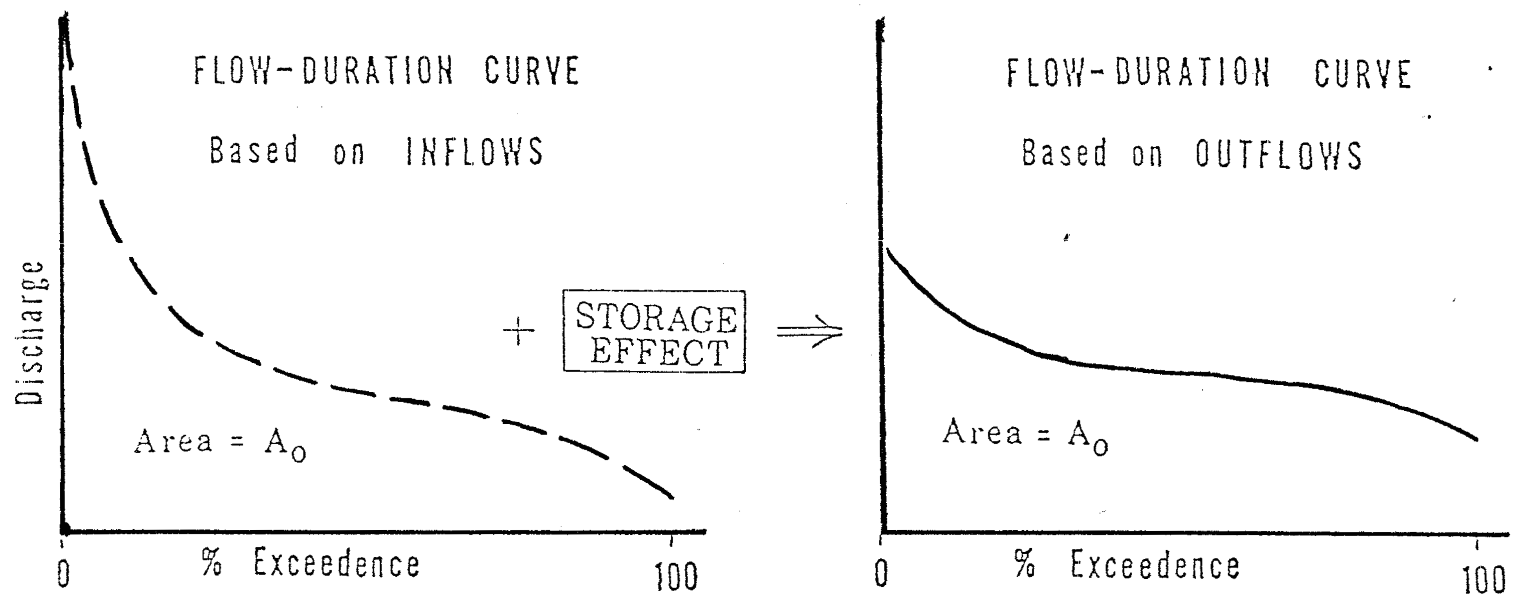

Figure 1

Superimposing the curves in Figure 1 results in the combined curve in Figure 2.

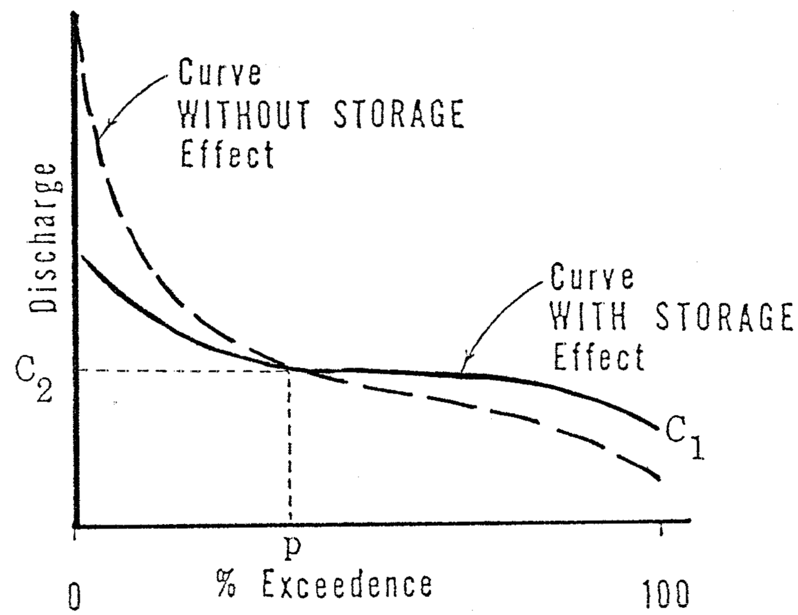

Figure 2

The area (A_o) under the original flow-duration curve is preserved and the modified flow-duration curve passes through points C_1 and C_2.

Where:

C_1 represents a discharge corresponding to 100% exceedence;

C_2 represents a point of intersection between the two curves.

An analytical technique was developed to transform the shape of the inflow-duration curve to the form of the outflow-duration curve. This mathematical algorithm will generate an outflow-duration curve and meet the following conditions:

1) the value of the function (flow-duration ordinate) at 100 percent exceedence must be C_1;

2) the value of the function (flow-duration ordinate) at some percent exceedence p must be C_2, where $0<p<1.0$;

3) the area under the modified outflow-duration curve must equal the area under the original inflow-duration curve (A_o).

PARAMETER DETERMINATION

Attention is now focused on making estimates for parameters C_1, C_2 and p.

The selected discharge value of C_2, corresponding to the percent exceedence point of intersection between the two duration curves, is critical in the mathematical algorithm for allowing feasible development of the outflow-duration curve. Comparison of inflow and outflow-duration curves for various storage projects tested revealed that the point of intersection between the two duration curves deviated unappreciably from the percent exceedence value corresponding to the average annual inflow A_o. Therefore, C_2 is assumed to be equal to A_o, which generally corresponds to percent exceedences ranging in value from 15 to 35 percent. The value of A_o is a constant and represents the area underneath the original inflow-duration curve, which is easily determined by integration. The selection for C_2 will automatically determine the value of p because C_2 and p are functionally related through use of the flow-duration relationship.

The value of C_1 is dependent on the storage capability of the site being analyzed. Accordingly, it seems reasonable to assume that C_1 can be estimated by considering the base flow component of the flow regime and the minimum flow contribution due to reservoir regulation during adverse flow conditions as follows:

$$C_1 = QMC + QMSC \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{Eq. 1}$$

where:

C_1 = the minimum flow value on the outflow-duration curve corresponding to 100 percent exceedence;

QMC = the minimum flow value on the original inflow-duration curve without regard to storage effects (100% exceedence value);

QMSC = the minimum flow contribution attributed to reservoir operation under critical low inflow conditions.

Critical low flow conditions occur whenever, over a sustained period of time, a reservoir is regulated to release additional flow in excess of upstream inflows as a means of satisfying designed project purposes. With regard to hydropower, this operational policy, if continued, can actually exaggerate the situation since depletion of power storage reduces the effective headwater and correspondingly the operating power head; requiring a continually increasing amount of flow to sustain energy requirements. The period of maximum drawdown can be defined as the period of time which begins with full power storage and ends when the power storage remaining is at a minimum. By definition, the period of maximum drawdown will then contain the most adverse streamflow conditions and will require the maximum withdrawal of water from the power storage. An estimate of QMSC can now be approximated by determining the depletion rate occurring throughout this period.

As an initial step, the power storage can be converted from units of volume, typically in acre-feet, to units of flow rate (cfs). To perform this conversion, a time period, say one year, must be selected. The resulting value expresses the power storage potential as the average amount of flow that can be extracted from an initially full power pool throughout a period of one year.

However, as defined above, the period of maximum drawdown is unconstrained with regard to the length of time required to complete the process, and actual reservoir operations have demonstrated this period of time varies from a few weeks to several years in length. Accordingly, the initial depletion rate (assuming a one-year length in the period of maximum drawdown) must be adjusted by a factor to reflect the project's actual length in time to minimum pool level as shown below:

$$QMSC = PS * ACF * ADJF \qquad \text{Eq. 2}$$

where

PS = power storage expressed as a volume (acre-feet);

ACF = a conversion factor (0.00138) which when multiplied by (PS) expresses the amount of power storages in terms of an average annual flow rate (cfs-yr);

ADJF = adjustment factor applied to the power storage to correct for variation in the length of the period of maximum drawdown.

From Equation 2, one can conclude that a length of the period of maximum drawdown exceeding one year requires the adjustment factor (ADJF) to be less than one. Conversely, a length in the period of maximum drawdown less than one year requires ADJF to exceed unity. In effect, ADJF can be alternatively defined as the reciprocal of the length in the period of maximum drawdown, when time is measured in years.

Several attempts at establishing a relationship to determine a value for ADJF were performed. The regression equation finally selected is based upon 113 existing and proposed hydro sites throughout the United States and is shown below:

$$ADJF = 0.65 + 1.113 * LOG(1/PSR) \qquad \text{Eq. 3}$$

where PSR represents a project's power storage to mean annual flow ratio.

Statistically, the above equation resulted in a R-squared value of 0.49 and a standard error of 0.45. A plot of this relationship can be seen in Figure 3.

The PSR is a dimensionless parameter which expresses the relative size of power storage to average annual inflow and is determined by converting power storage to an average one year flow rate, as previously suggested, and then dividing the result by the project's expected average annual inflow. In addition, this parameter is a relative measure of the ability to control the length of the period of maximum drawdown through regulation of project storage.

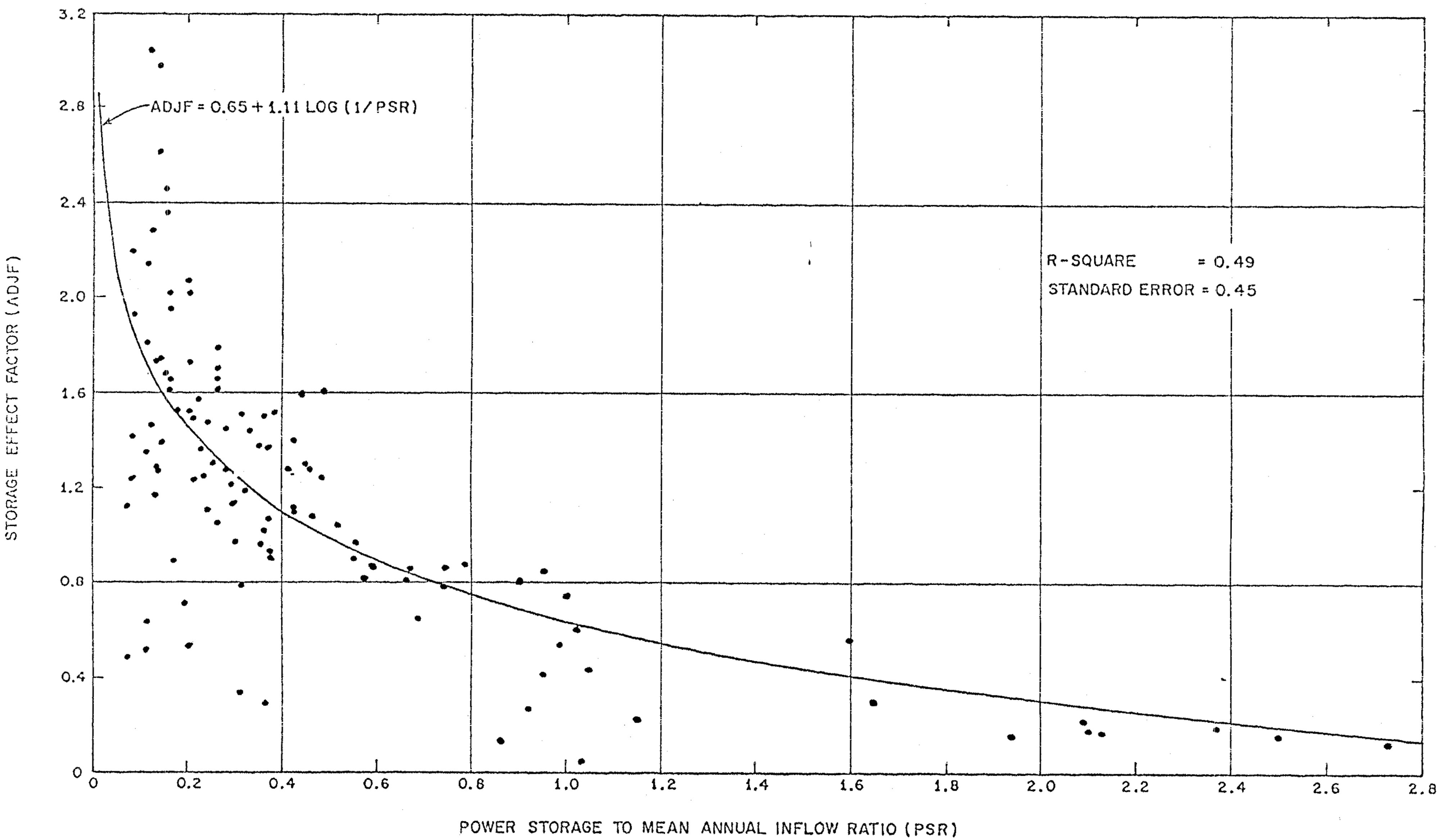

Figure 3 PLOT OF REGRESSION ANALYSIS RESULT

A relatively large PSR, say greater than 1.0, indicates sufficient storage capacity so that, on the average, there exists the capability of extending the period of maximum drawdown during sustained periods of low inflow. As the PSR falls below 1.0, this capability to attenuate diversity between inflow availability and project demands decreases, causing the average length of the period of maximum drawdown to decrease, accordingly. Another observation from Figure 3, substantiating this conclusion, is that as the PSR falls below 1.0, considerable increase in scatter of the data occurs. This implies that the storage effect is becoming relatively less important than the effect of diversity between inflow supply and energy demand. A direct determination of parameter C_1 is now possible by successive use of Equations 3, 2, and 1; allowing for a plausible solution to systematically produce a synthetic outflow-duration relationship for projects exhibiting power storage.

The substitution of the synthetic outflow-duration relationship for the original inflow-duration curve should substantially improve any estimate of average annual energy and should additionally enable an approximation of dependable capacity to be performed when used in a non-sequential power potential analysis.

AVERAGE ANNUAL ENERGY COMPARISON

Tables 1 and 2 illustrate the effect of using the flow-duration adjustment technique in the estimate of average annual energy.

Table 1 is comprised of twenty-six existing storage projects, each of which is currently installed and operating for energy production. Each project, defined by a site identification number and a project name, has been simulated on computer by using a computer program called HYDUR[1]. This program is design to perform power potential analyses using a non-sequential methodology. HYDUR can be operated in the traditional fashion or can be operated using the flow-duration adjustment option based on user input. Column 1 displays average annual energy estimates based on using standard non-sequential techniques. Column 2 contains energy estimates based on using the adjustment option. Column 3 represents the percent increase resulting from estimating average annual energy using the adjustment option as compared to standard non-sequential methods. In two cases where a decrease in the estimate of average annual energy occurred, the corresponding PSR's were 0.021 and 0.009; both relatively insignificant amounts of power storage. In general however, the use of the adjustment option resulted in an appreciable increase in the estimate of average annual energy. On the average, the increase was 13 percent. This trend is expected because the effect of the flow-duration adjustment is to produce a flatter and less "peaky" outflow-duration curve, thereby reducing the amount of average spill and increasing the amount of average annual energy.

TABLE 1 AVERAGE ANNUAL ENERGY ESTIMATE (GWH)

Site	Name of Project	PSR	Column 1	Column 2	Column 3
			No Storage effect	Storage effect	% Increase
SWT0418	Broken Bow	0.539	114.63	132.86	16
SWT0419	Eufaula	0.388	200.51	261.56	30
SWT0457	Keystone	0.069	203.66	232.55	14
SWF0015	Whitney	0.251	61.99	88.68	43
SWF0092	Sam Rayburn	0.817	99.09	106.09	7
SWL0005	Bull Shoals	0.482	633.14	636.77	1
SWL0013	Greers Ferry	0.445	198.76	249.91	26
SWL0010	Beaver	0.863	151.55	179.84	19
SWL0126	Table Rock	0.667	454.84	502.48	10
SWT0513	Fort Gibson	*0.009	190.40	178.46	-6
LMK0003	DeGrey Lake	0.740	83.20	89.56	8
LMK0008	Quachita Lake	1.075	147.57	169.73	15
LMK0026	Greeson	0.642	32.66	36.24	11
MRK0060	Harry S. Truman	*0.021	184.25	181.51	-1
MRK0067	Stockton Lake	0.714	47.58	50.61	6
MR00123	Big Horn Lake	0.140	922.45	1000.64	8
MR00158	Canyon Ferry	0.171	352.93	375.64	6
MR00215	Lake McConaughy	0.838	114.79	115.53	1
MR00274	Lake Francis Case	0.093	1976.93	2081.48	5
MR00326	Boysen Reservoir	0.406	80.03	86.16	8
MR00366	Glendo Reservoir	0.344	95.15	122.03	28
SAW0100	John H. Kerr	0.190	428.36	442.88	3
SAW0101	Philpott	0.557	29.81	30.37	2
SWL0004	Norfork	0.270	157.08	222.42	42
SWF0001	Toledo Bend	0.376	242.68	256.11	6
SWT0302	TanKiller	0.343	103.59	132.01	27
				Average	13

Tables 2A and 2B compare average annual energy estimates derived by sequential routing methods using computer program HEC-5[2], the non-sequential adjustment technique, and traditional non-sequential techniques; both latter estimates using computer program HYDUR. For each of the twenty-six existing power projects, three separate simulations were performed to evaluate the effect of reallocating additional storage volume to the production of energy. The first simulation estimates the average annual energy potential of the existing power project. Simulations two and three estimate the average annual energy potential after reallocating ten and twenty percent of the existing flood control storage to power, respectively.

The results indicate that the non-sequential flow-duration adjustment estimates of average annual energy were generally larger than the energy estimates resulting from sequential routing efforts. Conversely, the standard non-sequential estimates were generally smaller than the energy estimates resulting from sequential routings. On the average the fractional differences were 1.046 and 0.960 for the existing project comparison; 1.054 and 0.956 for the 10 percent reallocation comparison, and 1.059 and 0.952 for the 20 percent reallocation comparison, respectively.

TABLE 2A COMPARISON OF AVERAGE ANNUAL ENERGY
USING SEQUENTIAL AND STANDARD NON-SEQUENTIAL TECHNIQUES

SITE	NAME OF PROJECT	AVERAGE ANNUAL ENERGY (GWH)								
		EXISTING			10 PERCENT			20 PERCENT		
		HEC-5	HYDUR	FRACTIONAL DIFFERENCE	HEC-5	HYDUR	FRACTIONAL DIFFERENCE	HEC-5	HYDUR	FRACTIONAL DIFFERENCE
LMK0003	DeGrey Lake	87.63	83.20	0.949	88.37	83.86	0.949	89.08	84.51	0.949
LMK0008	Quachita Lake	167.99	147.56	0.878	168.95	148.52	0.879	170.22	149.56	0.879
LMK0026	Greeson	37.13	32.66	0.880	37.53	32.97	0.878	37.94	33.30	0.878
MRK0060	Harry S. Truman	178.20	184.25	1.034	203.30	205.53	1.011	220.90	221.69	1.004
MRK0067	Stockton Lake	47.96	47.58	0.992	49.39	48.51	0.982	50.66	49.49	0.977
MR00123	Big Horn Lake	933.40	922.46	0.988	942.60	931.36	0.988	951.61	940.06	0.988
MR00158	Canyon Ferry	359.40	352.93	0.982	367.50	359.83	0.979	374.80	365.82	0.976
MR00215	Lake McConaughy	109.20	114.79	1.051	109.10	116.12	1.064	113.60	117.54	1.035
MR00274	Lake Francis Case	2033.80	1976.93	0.972	2064.20	1995.46	0.967	2093.80	2008.26	0.959
MR00326	Boysen Reserv.	78.80	80.03	1.016	81.50	82.85	1.017	84.30	85.96	1.020
MR00366	Glendo Reserv.	100.37	95.15	0.948	102.98	96.81	0.940	105.47	98.68	0.936
SAW0100	John H. Kerr	435.96	428.36	0.983	448.11	442.71	0.988	459.44	451.31	0.982
SAW0101	Philpott	27.84	29.81	1.071	28.10	29.98	1.067	28.25	30.18	1.068
SWL0004	Norfork	201.70	157.08	0.779	204.80	159.00	0.776	208.10	160.80	0.773
SWF0001	Toledo Bend	255.80	242.68	0.949	256.10	243.62	0.951	256.20	244.20	0.953
SWT0302	TanKiller	116.80	103.59	0.887	119.30	105.66	0.886	121.40	107.57	0.886
SWT0418	Broken Bow	129.20	114.63	0.887	131.20	116.62	0.889	132.70	118.49	0.893
SWT0419	Eufaula	226.70	200.50	0.884	229.30	202.31	0.882	231.80	203.89	0.880
SWT0457	Keystone	234.40	203.65	0.869	244.70	211.60	0.865	252.80	218.20	0.863
SWF0015	Whitney	73.30	61.99	0.846	75.60	63.39	0.838	77.20	64.76	0.839
SWF0092	Sam Rayburn	100.50	99.09	0.986	101.90	100.92	0.990	103.10	101.86	0.988
SWL0005	Bull Shoals	598.78	633.14	1.057	616.92	646.48	1.048	637.41	662.39	1.039
SWL0013	Greers Ferry	236.29	198.76	0.841	239.72	200.86	0.838	242.98	203.05	0.836
SWL0010	Beaver	171.63	151.56	0.883	172.76	152.16	0.881	173.73	152.83	0.880
SWL0126	Table Rock	456.30	454.84	0.997	472.33	457.63	0.969	473.82	460.41	0.972
SWT0513	Fort Gibson	212.76	190.40	0.895	222.66	198.84	0.893	230.16	206.20	0.896
Total		7611.84	7307.62	0.960	7778.94	7433.60	0.956	7921.47	7541.11	0.952

TABLE 2B COMPARISON OF AVERAGE ANNUAL ENERGY
USING SEQUENTIAL AND NON-SEQUENTIAL ADJUSTMENT TECHNIQUES

SITE	NAME OF PROJECT	AVERAGE ANNUAL ENERGY (GWH) EXISTING HEC-5	EXISTING HYDUR	EXISTING FRACTIONAL DIFFERENCE	10 PERCENT HEC-5	10 PERCENT HYDUR	10 PERCENT FRACTIONAL DIFFERENCE	20 PERCENT HEC-5	20 PERCENT HYDUR	20 PERCENT FRACTIONAL DIFFERENCE
LMK0003	DeGrey Lake	87.63	89.56	1.022	88.37	90.38	1.023	89.08	91.15	1.023
LMK0008	Quachita Lake	167.99	169.73	1.010	168.95	170.99	1.012	170.22	172.36	1.013
LMK0026	Greeson	37.13	36.24	0.976	37.53	36.62	0.976	37.94	37.04	0.976
MRK0060	Harry S. Truman	178.20	181.51	1.019	203.30	211.27	1.039	220.90	233.29	1.056
MRK0067	Stockton Lake	47.96	50.61	1.055	49.39	51.92	1.051	50.66	53.38	1.054
MR00123	Big Horn Lake	933.40	1000.64	1.072	942.60	1013.27	1.075	951.61	1025.54	1.078
MR00158	Canyon Ferry	359.40	375.64	1.045	367.50	387.87	1.055	374.80	399.02	1.065
MR00215	Lake McConaughy	109.20	115.53	1.058	109.10	116.97	1.072	113.60	118.51	1.043
MR00274	Lake Francis Case	2033.80	2081.48	1.023	2064.20	2124.57	1.029	2093.80	2159.83	1.032
MR00326	Boysen Reserv.	78.80	86.16	1.093	81.50	90.23	1.107	84.30	94.56	1.122
MR00366	Glendo Reserv.	100.37	122.03	1.216	102.98	127.09	1.234	105.47	131.96	1.251
SAW0100	John H. Kerr	435.96	442.88	1.016	448.11	455.31	1.016	459.44	466.59	1.016
SAW0101	Philpott	27.84	30.37	1.091	28.10	30.56	1.088	28.25	30.77	1.089
SWL0004	Norfork	201.70	222.42	1.103	204.80	228.41	1.115	208.10	233.93	1.124
SWF0001	Toledo Bend	255.80	256.11	1.001	256.10	258.38	1.009	256.20	259.75	1.014
SWT0302	TanKiller	116.80	132.01	1.130	119.30	136.60	1.145	121.40	140.94	1.161
SWT0418	Broken Bow	129.20	132.86	1.028	131.20	135.23	1.031	132.70	137.56	1.037
SWT0419	Eufaula	226.70	261.56	1.154	229.30	266.00	1.160	231.80	270.20	1.166
SWT0457	Keystone	234.40	232.55	0.992	244.70	249.65	1.020	252.80	264.80	1.047
SWF0015	Whitney	73.30	88.68	1.210	75.60	95.95	1.269	77.20	101.17	1.310
SWF0092	Sam Rayburn	100.50	106.09	1.056	101.90	107.54	1.055	103.10	109.16	1.059
SWL0005	Bull Shoals	598.78	636.77	1.063	616.92	656.22	1.064	637.41	675.67	1.060
SWL0013	Greers Ferry	236.29	249.91	1.058	239.72	254.12	1.060	242.98	258.36	1.063
SWL0010	Beaver	171.63	179.84	1.048	172.76	180.81	1.047	173.73	181.87	1.047
SWL0126	Table Rock	456.30	502.48	1.101	472.33	506.97	1.073	473.82	511.46	1.079
SWT0513	Fort Gibson	212.76	178.46	0.839	222.66	217.76	0.978	230.16	231.05	1.004
Total		7611.84	7962.12	1.046	7778.94	8200.69	1.054	7921.47	8389.92	1.059

Additionally, comparison of average annual energy estimates calculated by both the sequential and non-sequential methods resulted in fairly constant deviations in the energy estimates for all three cases tested for each of the twenty-six projects. Although the absolute energy estimates differ on the average by five percent, the estimates of incremental energy gained from reallocating additional storage to power are averaging within about one percent.

These observations suggest that the flow-duration adjustment option is adequately redistributing the available streamflow for power potential analysis and therefore indicates that the primary cause of the difference in energy estimates is due to a difference in power head between the two techniques. This was expected because the average headwater elevation in all the non-sequential simulations was assumed to be at the top of power pool. A more practical selection would be to choose the normal pool headwater level as a fractional percent of the total power pool available. If the project is existing, past operation of the project may give a clue in establishing this fractional percent value. For proposed projects, results from Table 2B indicate that a fractional percent ranging between 0.85 to 0.95 is appropriate for projects exhibiting sufficient power storage (power storage to mean annual flow ratio greater than 0.10).

DEPENDABLE CAPACITY COMPARISON

Dependable capacity can be defined as the capacity which under the most adverse flow conditions on record, can be relied upon to carry system load, provide dependable reserve capacity, and meet firm power obligations, taking into account seasonal variations and other characteristics of the load to be supplied[3]. The association to the "most adverse flow conditions on record," in the definition of dependable capacity strongly supports the notion that parameter C_1 might be valuable as an indicator in approximating this capacity value. This assumption was tested and it was found that dependable capacity could be estimated by using the power equation as follows:

$$DCAP = C(C_1/PF)\, He \qquad \text{Eq.4}$$

Where:

DCAP = dependable capacity in kilowatts:

C = .084603 conversion factor which expresses power in kilowatts;

C_1 = the minimum flow parameter as previously defined;

PF = the average annual plant factor relating dependable capacity to its firm energy requirement;

H = net power head in feet;

e = overall efficiency (assumed equal to a constant 0.86)

The quantity (C_1/PF) represents the expected minimum flow adjusted for average time of hydropower plant operation.

Equation 4 was employed with data from all 113 projects used to develop the parameter ADJF and the resulting estimates of dependable capacity were compared to corresponding dependable capacity values estimated from sequential routing techniques (HEC-5). This comparison, depicted in Figure 4, resulted in an R-squared value of 0.985 and a standard error of 10,700 kilowatts. Comparison of empirical (non-sequential) to sequentially determined capacities varied over a range of 100 kilowatts to 650 megawatts. A departure in plotting position above an imaginary 45^{o} line represents an underestimate of dependable capacity determined empirical as compared to dependable capacity estimated using sequential routing techniques. Conversely, a departure below this line represents an exaggeration of dependable capacity. Maximum departure about this line, in terms of percent difference, occurs for small installations (i.e., projects having installed capacities less than 2 megawatts). Small installations are generally associated with projects possessing limited storage capacity and correspondingly small power storage to mean annual flow ratios (PSR). Since the regression equation used to estimate ADJF (Equation 3) was incorporated in the empirical determination of dependable capacity, the problem of increased scatter associated with small PSR's is most probably the underlying influence causing these maximum departures to occur in this capacity range.

An approach to alleviate this problem of increased scatter is to introduce a parameter into the regression analysis for determining ADJF which represents a measure of the diversity associated between energy demand requirements and inflow availability at a project. Although this approach was not performed for this paper, an initial attempt at defining this parameter can be suggested by plotting the firm energy demand requirements of a project against project inflow in the form of normalized distributions. As an illustration, assume the monthly distributions of inflow and firm energy demand can be determined from available data and are as follows:

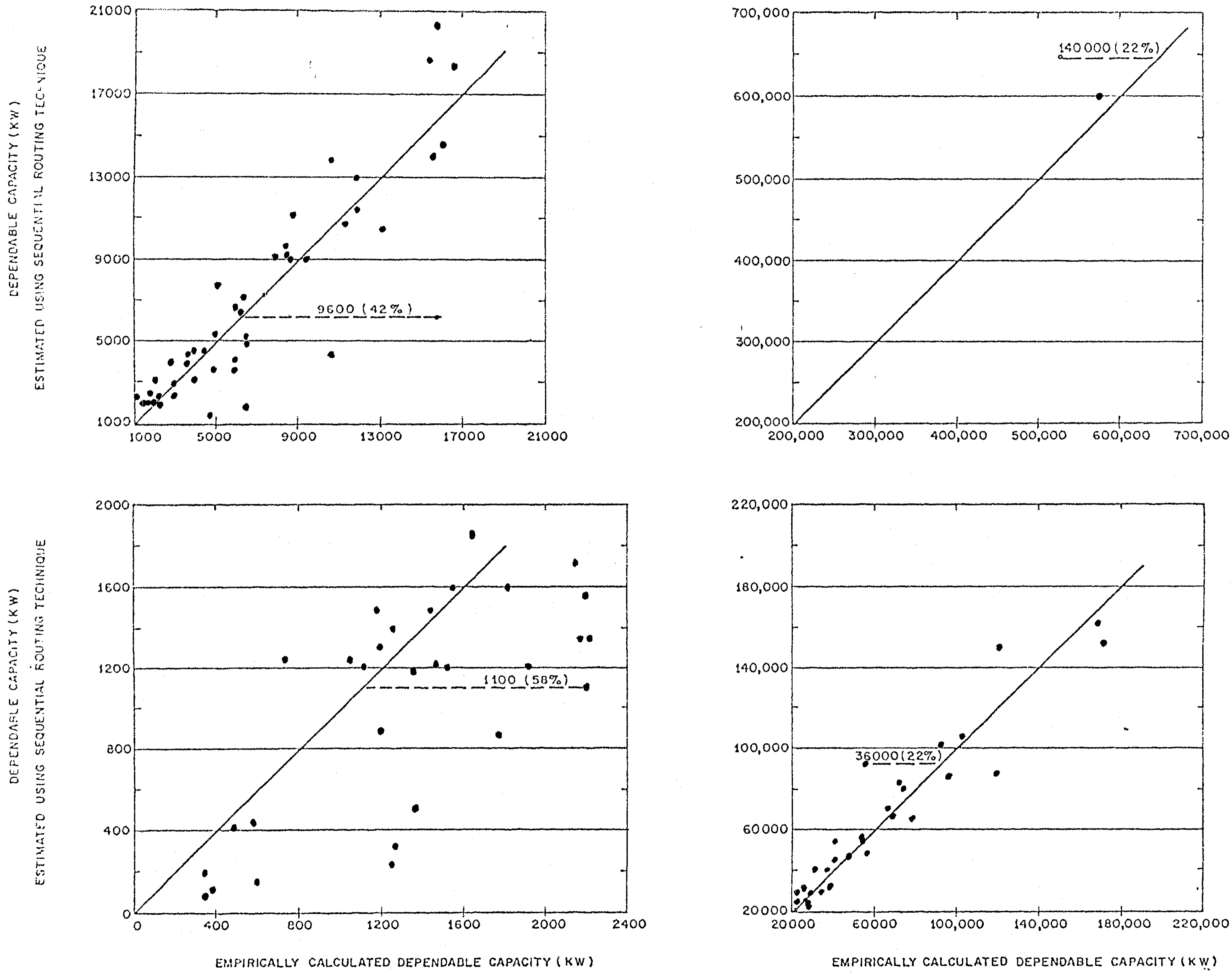

Figure 4 COMPARISON OF EMPIRICAL TO SEQUENTIALLY DETERMINED DEPENDABLE CAPACITIES

Monthly												
Inflow	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
(cfsm)	175	150	75	175	250	375	325	300	250	175	125	125
Monthly												
Firm Energy Required												
(MWH)	80	75	60	60	70	80	90	110	110	100	80	85

The distributions can be normalized by dividing the monthly values by annual inflow (2,500 cfsm) and annual firm energy (1,000MWH), respectively. Figure 5 is a superimposed plot of these normalized distributions, where the area under each distribution is unity. The plot graphically displays the diversity between energy demand and flow availability. In general, from April through August, this project will have sufficient storage available to meet all energy demands and might actually be accumulating storage and experiencing spill. From September to March, inflow recedes and storage depletion occurs to supplement flow needed for power generation. Therefore, an initial definition of the diversity parameter might be to accumulate the percent differences between the distributions throughout the year whenever percent of energy demand exceeds percent of annual inflow. This suggestion is only one of several alternatives which can be conceived to measure diversity. Future funding may allow for further investigation in this area.

In conclusion, the non-sequential adjustment technique has been shown to be a viable alternative to use in estimating energy and capacity values associated with power storage projects. It is recommended that this option be employed (through the use of computer program HYDUR) throughout the screening process of any basin wide power potential study. Once project alternatives become manageable, a more detailed sequential routing effort should be undertaken.

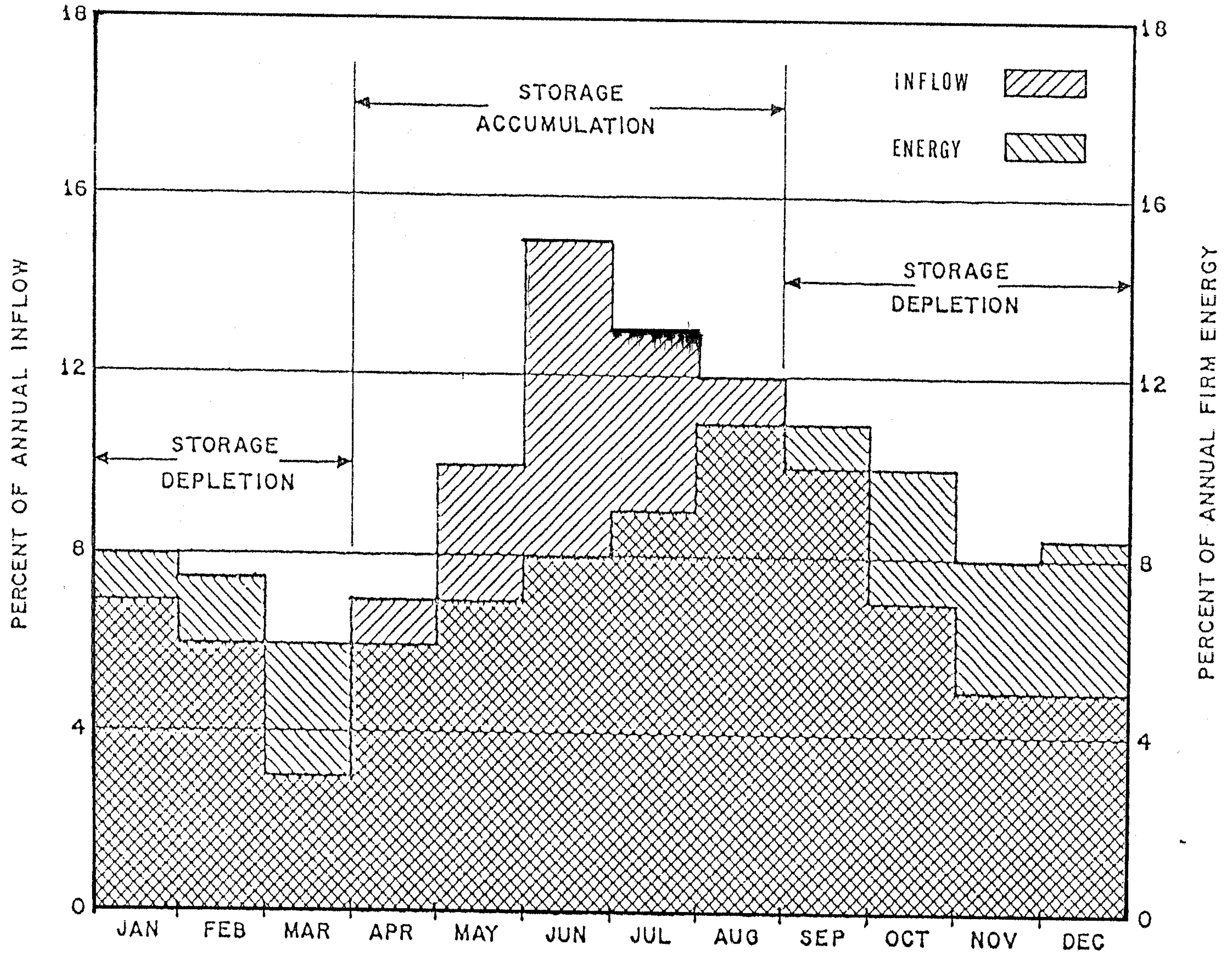

Figure 5 COMPARISON OF ANNUAL FLOW AND ANNUAL FIRM ENERGY DISTRIBUTIONS

References

1) Hydrologic Engineering Center, U.S. Army Corps of Engineers, "HYDUR-Hydropower Analysis Using Streamflow Duration Procedures", August 1980.

2) Hydrologic Engineering Center, U.S. Army Corps of Engineers, "HEC5-Simulation of Flood Control and Conservation Systems", June 1979.

3) Federal Energy Regulatory Commission, "Hydroelectric Power Evaluation", August 1979.

Technical Paper Series

TP-1 Use of Interrelated Records to Simulate Streamflow
TP-2 Optimization Techniques for Hydrologic Engineering
TP-3 Methods of Determination of Safe Yield and Compensation Water from Storage Reservoirs
TP-4 Functional Evaluation of a Water Resources System
TP-5 Streamflow Synthesis for Ungaged Rivers
TP-6 Simulation of Daily Streamflow
TP-7 Pilot Study for Storage Requirements for Low Flow Augmentation
TP-8 Worth of Streamflow Data for Project Design - A Pilot Study
TP-9 Economic Evaluation of Reservoir System Accomplishments
TP-10 Hydrologic Simulation in Water-Yield Analysis
TP-11 Survey of Programs for Water Surface Profiles
TP-12 Hypothetical Flood Computation for a Stream System
TP-13 Maximum Utilization of Scarce Data in Hydrologic Design
TP-14 Techniques for Evaluating Long-Tem Reservoir Yields
TP-15 Hydrostatistics - Principles of Application
TP-16 A Hydrologic Water Resource System Modeling Techniques
TP-17 Hydrologic Engineering Techniques for Regional Water Resources Planning
TP-18 Estimating Monthly Streamflows Within a Region
TP-19 Suspended Sediment Discharge in Streams
TP-20 Computer Determination of Flow Through Bridges
TP-21 An Approach to Reservoir Temperature Analysis
TP-22 A Finite Difference Methods of Analyzing Liquid Flow in Variably Saturated Porous Media
TP-23 Uses of Simulation in River Basin Planning
TP-24 Hydroelectric Power Analysis in Reservoir Systems
TP-25 Status of Water Resource System Analysis
TP-26 System Relationships for Panama Canal Water Supply
TP-27 System Analysis of the Panama Canal Water Supply
TP-28 Digital Simulation of an Existing Water Resources System
TP-29 Computer Application in Continuing Education
TP-30 Drought Severity and Water Supply Dependability
TP-31 Development of System Operation Rules for an Existing System by Simulation
TP-32 Alternative Approaches to Water Resources System Simulation
TP-33 System Simulation of Integrated Use of Hydroelectric and Thermal Power Generation
TP-34 Optimizing flood Control Allocation for a Multipurpose Reservoir
TP-35 Computer Models for Rainfall-Runoff and River Hydraulic Analysis
TP-36 Evaluation of Drought Effects at Lake Atitlan
TP-37 Downstream Effects of the Levee Overtopping at Wilkes-Barre, PA, During Tropical Storm Agnes
TP-38 Water Quality Evaluation of Aquatic Systems
TP-39 A Method for Analyzing Effects of Dam Failures in Design Studies
TP-40 Storm Drainage and Urban Region Flood Control Planning
TP-41 HEC-5C, A Simulation Model for System Formulation and Evaluation
TP-42 Optimal Sizing of Urban Flood Control Systems
TP-43 Hydrologic and Economic Simulation of Flood Control Aspects of Water Resources Systems
TP-44 Sizing Flood Control Reservoir Systems by System Analysis
TP-45 Techniques for Real-Time Operation of Flood Control Reservoirs in the Merrimack River Basin
TP-46 Spatial Data Analysis of Nonstructural Measures
TP-47 Comprehensive Flood Plain Studies Using Spatial Data Management Techniques
TP-48 Direct Runoff Hydrograph Parameters Versus Urbanization
TP-49 Experience of HEC in Disseminating Information on Hydrological Models
TP-50 Effects of Dam Removal: An Approach to Sedimentation
TP-51 Design of Flood Control Improvements by Systems Analysis: A Case Study
TP-52 Potential Use of Digital Computer Ground Water Models
TP-53 Development of Generalized Free Surface Flow Models Using Finite Element Techniques
TP-54 Adjustment of Peak Discharge Rates for Urbanization
TP-55 The Development and Servicing of Spatial Data Management Techniques in the Corps of Engineers
TP-56 Experiences of the Hydrologic Engineering Center in Maintaining Widely Used Hydrologic and Water Resource Computer Models
TP-57 Flood Damage Assessments Using Spatial Data Management Techniques
TP-58 A Model for Evaluating Runoff-Quality in Metropolitan Master Planning
TP-59 Testing of Several Runoff Models on an Urban Watershed
TP-60 Operational Simulation of a Reservoir System with Pumped Storage
TP-61 Technical Factors in Small Hydropower Planning
TP-62 Flood Hydrograph and Peak Flow Frequency Analysis
TP-63 HEC Contribution to Reservoir System Operation
TP-64 Determining Peak-Discharge Frequencies in an Urbanizing Watershed: A Case Study
TP-65 Feasibility Analysis in Small Hydropower Planning
TP-66 Reservoir Storage Determination by Computer Simulation of Flood Control and Conservation Systems
TP-67 Hydrologic Land Use Classification Using LANDSAT
TP-68 Interactive Nonstructural Flood-Control Planning
TP-69 Critical Water Surface by Minimum Specific Energy Using the Parabolic Method

TP-70 Corps of Engineers Experience with Automatic Calibration of a Precipitation-Runoff Model
TP-71 Determination of Land Use from Satellite Imagery for Input to Hydrologic Models
TP-72 Application of the Finite Element Method to Vertically Stratified Hydrodynamic Flow and Water Quality
TP-73 Flood Mitigation Planning Using HEC-SAM
TP-74 Hydrographs by Single Linear Reservoir Model
TP-75 HEC Activities in Reservoir Analysis
TP-76 Institutional Support of Water Resource Models
TP-77 Investigation of Soil Conservation Service Urban Hydrology Techniques
TP-78 Potential for Increasing the Output of Existing Hydroelectric Plants
TP-79 Potential Energy and Capacity Gains from Flood Control Storage Reallocation at Existing U.S. Hydropower Reservoirs
TP-80 Use of Non-Sequential Techniques in the Analysis of Power Potential at Storage Projects
TP-81 Data Management Systems of Water Resources Planning
TP-82 The New HEC-1 Flood Hydrograph Package
TP-83 River and Reservoir Systems Water Quality Modeling Capability
TP-84 Generalized Real-Time Flood Control System Model
TP-85 Operation Policy Analysis: Sam Rayburn Reservoir
TP-86 Training the Practitioner: The Hydrologic Engineering Center Program
TP-87 Documentation Needs for Water Resources Models
TP-88 Reservoir System Regulation for Water Quality Control
TP-89 A Software System to Aid in Making Real-Time Water Control Decisions
TP-90 Calibration, Verification and Application of a Two-Dimensional Flow Model
TP-91 HEC Software Development and Support
TP-92 Hydrologic Engineering Center Planning Models
TP-93 Flood Routing Through a Flat, Complex Flood Plain Using a One-Dimensional Unsteady Flow Computer Program
TP-94 Dredged-Material Disposal Management Model
TP-95 Infiltration and Soil Moisture Redistribution in HEC-1
TP-96 The Hydrologic Engineering Center Experience in Nonstructural Planning
TP-97 Prediction of the Effects of a Flood Control Project on a Meandering Stream
TP-98 Evolution in Computer Programs Causes Evolution in Training Needs: The Hydrologic Engineering Center Experience
TP-99 Reservoir System Analysis for Water Quality
TP-100 Probable Maximum Flood Estimation - Eastern United States
TP-101 Use of Computer Program HEC-5 for Water Supply Analysis
TP-102 Role of Calibration in the Application of HEC-6
TP-103 Engineering and Economic Considerations in Formulating
TP-104 Modeling Water Resources Systems for Water Quality
TP-105 Use of a Two-Dimensional Flow Model to Quantify Aquatic Habitat
TP-106 Flood-Runoff Forecasting with HEC-1F
TP-107 Dredged-Material Disposal System Capacity Expansion
TP-108 Role of Small Computers in Two-Dimensional Flow Modeling
TP-109 One-Dimensional Model for Mud Flows
TP-110 Subdivision Froude Number
TP-111 HEC-5Q: System Water Quality Modeling
TP-112 New Developments in HEC Programs for Flood Control
TP-113 Modeling and Managing Water Resource Systems for Water Quality
TP-114 Accuracy of Computer Water Surface Profiles - Executive Summary
TP-115 Application of Spatial-Data Management Techniques in Corps Planning
TP-116 The HEC's Activities in Watershed Modeling
TP-117 HEC-1 and HEC-2 Applications on the Microcomputer
TP-118 Real-Time Snow Simulation Model for the Monongahela River Basin
TP-119 Multi-Purpose, Multi-Reservoir Simulation on a PC
TP-120 Technology Transfer of Corps' Hydrologic Models
TP-121 Development, Calibration and Application of Runoff Forecasting Models for the Allegheny River Basin
TP-122 The Estimation of Rainfall for Flood Forecasting Using Radar and Rain Gage Data
TP-123 Developing and Managing a Comprehensive Reservoir Analysis Model
TP-124 Review of U.S. Army corps of Engineering Involvement With Alluvial Fan Flooding Problems
TP-125 An Integrated Software Package for Flood Damage Analysis
TP-126 The Value and Depreciation of Existing Facilities: The Case of Reservoirs
TP-127 Floodplain-Management Plan Enumeration
TP-128 Two-Dimensional Floodplain Modeling
TP-129 Status and New Capabilities of Computer Program HEC-6: "Scour and Deposition in Rivers and Reservoirs"
TP-130 Estimating Sediment Delivery and Yield on Alluvial Fans
TP-131 Hydrologic Aspects of Flood Warning - Preparedness Programs
TP-132 Twenty-five Years of Developing, Distributing, and Supporting Hydrologic Engineering Computer Programs
TP-133 Predicting Deposition Patterns in Small Basins
TP-134 Annual Extreme Lake Elevations by Total Probability Theorem
TP-135 A Muskingum-Cunge Channel Flow Routing Method for Drainage Networks
TP-136 Prescriptive Reservoir System Analysis Model - Missouri River System Application
TP-137 A Generalized Simulation Model for Reservoir System Analysis
TP-138 The HEC NexGen Software Development Project
TP-139 Issues for Applications Developers
TP-140 HEC-2 Water Surface Profiles Program
TP-141 HEC Models for Urban Hydrologic Analysis

TP-142 Systems Analysis Applications at the Hydrologic Engineering Center
TP-143 Runoff Prediction Uncertainty for Ungauged Agricultural Watersheds
TP-144 Review of GIS Applications in Hydrologic Modeling
TP-145 Application of Rainfall-Runoff Simulation for Flood Forecasting
TP-146 Application of the HEC Prescriptive Reservoir Model in the Columbia River Systems
TP-147 HEC River Analysis System (HEC-RAS)
TP-148 HEC-6: Reservoir Sediment Control Applications
TP-149 The Hydrologic Modeling System (HEC-HMS): Design and Development Issues
TP-150 The HEC Hydrologic Modeling System
TP-151 Bridge Hydraulic Analysis with HEC-RAS
TP-152 Use of Land Surface Erosion Techniques with Stream Channel Sediment Models
TP-153 Risk-Based Analysis for Corps Flood Project Studies - A Status Report
TP-154 Modeling Water-Resource Systems for Water Quality Management
TP-155 Runoff simulation Using Radar Rainfall Data
TP-156 Status of HEC Next Generation Software Development
TP-157 Unsteady Flow Model for Forecasting Missouri and Mississippi Rivers
TP-158 Corps Water Management System (CWMS)
TP-159 Some History and Hydrology of the Panama Canal
TP-160 Application of Risk-Based Analysis to Planning Reservoir and Levee Flood Damage Reduction Systems
TP-161 Corps Water Management System - Capabilities and Implementation Status