acknowledged for their valuable guidance and advice. Special thanks go to Barbara Turner for clerical assistance throughout the study and for typing this paper.

The opinions, findings, and conclusions expressed in this paper are mine and not necessarily those of the sponsoring agencies.

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Flood Frequency Analysis for Regulated Rivers

STEVEN G. BUCHBERGER

A case study of the Colorado River at Glenwood Springs, Colorado, is presented to demonstrate several statistical tests for identifying watersheds in which conditions are changing with time. Results of the tests indicate that annual peak flows of the Colorado River are influenced significantly by reservoir regulation. Consequently, conventional methods of frequency analysis are not suitable for obtaining flood estimates from the data series. Time-series analysis is a versatile approach to flood-frequency determinations when conventional statistical methods are not appropriate. The basic strategy of time-series analysis is to treat each value of the regulated annual peak-flow series as a combination of two elements—a deterministic component and a stochastic component. The deterministic component is quantified and removed from the flood series. The residual stochastic components, found to be stationary and independent, are then fitted to a probability distribution from which annual floods are estimated. Results of the time-series analysis show that the 2 percent and 1 percent chance floods, both required for Interstate highway design, are substantially less than corresponding log-Pearson type III estimates. Because the time-series analysis is able to detect and to treat the impact of reservoir regulation on the peak-flow series, the resulting flood frequency estimates are more representative of the watershed.

Analysis of the magnitude and frequency of floods is an important prerequisite of many engineering projects and consequently a routine practice in many engineering offices. During the past 60 years, a variety of techniques have been developed for peak-flow analysis (1). In an effort to promote a consistent approach to these peak-flow studies, the U.S. Water Resources Council (2-4) recommended the log-Pearson type III (LP III) distribution for determinations of flood frequency.

Because the LP III procedure is simple and well documented, it has become a popular method of flood flow determination. Application of this methodology, however, must not preclude engineering judgment. There are a growing number of situations--such as watersheds in which peak flows are altered by reservoir regulation--for which conventional statistical methods are inappropriate. The Colorado River in west central Colorado is a classic example. Experience has shown that myopic application of the LP III method results in flood estimates that are not representative of the Colorado River watershed.

The Colorado Department of Highways is now involved in final design of the uncompleted portions of I-70, much of which will parallel and at times cross the Colorado River. For public safety and project economy, it is imperative that the final design be based on peak-flow estimates that accurately reflect the flood characteristics of the Colorado River. The purpose of this paper, therefore, is to (a) present several objective methods for identifying watersheds in which reservoir regulation significantly influences annual peak flows and (b) demonstrate an alternate approach that combines time-series analysis and engineering judgment in order to obtain flood frequency estimates of regulated rivers.

BACKGROUND INFORMATION

I-70 is the major route for east-bound and west-bound traffic in Colorado. One of the few segments of I-70 that remains uncompleted is that through Glenwood Canyon, a narrow meandering gorge of sheer cliffs shaped over millions of years by the erosive action of the Colorado River. Although it is renowned for its scenic splendor, the canyon also serves as a vital transportation corridor for west central Colorado. Glenwood Canyon now accommodates US-6, the Denver and Rio Grande Western Railroad, and the Shoshone Dam and Power Plant of the Public Service Company of Colorado.

In 1969 the Colorado Department of Highways received and accepted a hydrologic report (5) of the Colorado River at Glenwood Canyon. The report included several LP III analyses for various periods of the annual flood record observed at Glenwood

Table 1. Flood frequency estimates of Colorado River at Glenwood Springs.

Annual Exceedance Probability	Estimated Flood Discharge[a] (ft^3/s) LP III (1900-1968)	LP III (1942-1968)	Recommended for Design
0.50	16 350	11 900	16 000
0.02	31 000	22 000	25 000
0.01	33 250	23 400	26 500

[a]From Huggins and Griek (5).

Table 2. Peak flows of Colorado River at Glenwood Springs.

Year	Discharge (ft^3/s)	Year	Discharge (ft^3/s)	Year	Discharge (ft^3/s)	Year	Discharge (ft^3/s)
1900	20 000	1920	24 300	1940	11 100	1960	9 730
1901	20 000	1921	29 000	1941	14 900	1961	7 680
1902	12 000	1922	16 100	1942	16 800	1962	14 600
1903	16 500	1923	20 400	1943	13 000	1963	5 470
1904	16 500	1924	24 500	1944	10 600	1964	7 580
1905	22 500	1925	11 200	1945	10 600	1965	11 900
1906	22 100	1926	23 000	1946	9 720	1966	4 840
1907	20 400	1927	18 400	1947	14 200	1967	9 200
1908	11 500	1928	27 400	1948	16 600	1968	8 100
1909	27 900	1929	21 400	1949	16 300	1969	7 120
1910	14 600	1930	15 500	1950	10 100	1970	13 220
1911	15 200	1931	9 710	1951	14 400	1971	9 970
1912	27 700	1932	17 300	1952	20 800	1972	7 300
1913	12 400	1933	20 600	1953	14 000	1973	12 220
1914	28 100	1934	8 140	1954	4 060	1974	9 620
1915	13 400	1935	21 300	1955	5 400	1975	8 270
1916	14 800	1936	16 900	1956	12 600	1976	4 240
1917	29 400	1937	11 400	1957	18 900	1977	2 340
1918	30 100	1938	20 900	1958	16 000	1978	11 180
1919	12 300	1939	13 100	1959	8 480	1979	11 860

Springs, a community near the western end of the canyon. Results of the study (Table 1) show that the flood estimates obtained from the short record (1942-1968) are 30 percent less than those from the entire record (1900-1968). The values recommended for design were a compromise between the flood estimates obtained from both periods.

A recent review (6) of the recommended values suggests that they are not representative of the watershed. For example, the annual peak flows of the Colorado River (Table 2) show that the 50 percent chance flood has not occurred for 22 consecutive years. Intuitively, it seems unlikely that this should happen; mathematically, it is simple to evaluate the probability that this would happen. Since annual peak flows are considered independent events, the probability P that a flood will not be exceeded for n consecutive years is

$$P = (1 - P_e)^n \tag{1}$$

in which P_e is the probability that the flood will be exceeded in any given year. For the case under consideration, P_e = 0.50 and n = 22. Hence,

$$P = (1 - 0.50)^{22} = 0.000\,000\,24 \tag{2}$$

or the probability of not exceeding the 50 percent chance flood for 22 consecutive years is about 1 in 4 million.

The remote possibility of this dry spell indicates that the recommended 50 percent chance flood is overestimated. Although this type of check can be extended to the 2 and 1 percent chance floods, the results would be inconclusive because the length of the annual peak-flow record is short in comparison with the expected frequency of occurrence of rare floods. Nevertheless, it is reasonable to suspect that a high bias also exists for the recommended 2 and 1 percent chance floods. Although inflated estimates may be condoned for providing an extra margin of safety, overconservativeness is not warranted, since economy of the I-70 project is linked inextricably to the magnitude of the design discharge.

A more compelling reason for an updated flood study, however, stems from consideration of the rationale used to obtain the recommended flood estimates. The 1969 report recognized that reservoir regulation affected the latter period of the peak-flow series. Nonetheless, the entire flood record was retained for its greater statistical base and subsequently used in the frequency analysis. As such, recommended flood estimates were derived from data collected during a time that no longer reflects prevailing conditions in the watershed. The shortcoming of this approach is obvious. Analysis of nonrepresentative flood data yields nonrepresentative flood estimates.

Many statistical tests are available to evaluate the suitability of an annual peak-flow series for conventional flood frequency determinations. Several of these tests are demonstrated following a brief discussion of the key data assumptions on which the frequency analysis is based.

DATA ASSUMPTIONS

In any statistical treatment of annual flood flows, the data must be stationary and reliable. Stationariness requires that the properties of the annual flood series remain time invariant; reliability implies that the flood record is free of substantial errors caused by measuring, transmitting, recording, and processing data. Further, the flood record must be independent and homogeneous. Independence means that peak flows from one year are not influenced by peak flows from previous years; homogeneity requires that all peak flows be from the same parent population or collection of all possible outcomes of annual floods.

A "well-behaved" flood series--one that is stationary, independent, reliable, and homogeneous--is suitable for flood frequency analysis by conventional statistical methods such as LP III. These prerequisites are reiterated in the Water Resource Council's guidelines (3,4), which caution, "Assessment of the adequacy and applicability of flood records is therefore a necessary first step in flood frequency analysis...."

TESTING ANNUAL FLOOD SERIES

Test for Stationariness

An effective test for stationariness involves detection of significant long-term trends in the data series (7). Although it may be possible to fit high-order polynomial functions to the data series, it is desirable to use simple relationships in order to keep the analysis tractable. Therefore, a linear trend is investigated here. For this case, least-squares regression is used to express the annual peak flows as a function of time:

$$q_i = a + bt_i \tag{3}$$

where

- a = regression constant,
- b = regression coefficient, and
- q_i = peak discharge observed during year t_i.

For a series that is stationary, the slope of the regression line is not significantly different than zero. So to test for stationariness, the following hypotheses are postulated: $H_0 : b = 0$ versus $H_1 : b \neq 0$. The appropriate test statistic is given by the following equation:

$$T = r[(n-2)/(1-r^2)]^{1/2} \sim t(n-2) \tag{4}$$

where r is the correlation coefficient of the linear regression and n is the amount of data in the series. The test statistic is assumed to be a random variable that has the t-distribution with n - 2 degrees of freedom. Let $t_c[1(\alpha/2)]$ be the critical value of the test statistic at the α-level of significance. Then H_0 is accepted if

$$|T| < t_c[1-(\alpha/2)] \tag{5}$$

Otherwise, H_0 is rejected. By using the 80-year flood record given in Table 2, least-squares regression gives the following:

$$q_i = 22\,037 - (180.4)t_i \tag{6}$$

where t_i is the year minus 1900 and r is -0.63. The minus sign in Equation 6 indicates that the annual peak floods of the Colorado River are decreasing with time. From any statistics book, at the 1 percent level of significance with n - 2 = 78, $t_c = 2.65$. Equation 4 gives T = -7.16. Since $|T| > t_c$, H_0 is rejected. Consequently, the annual peak-flow series of the Colorado River at Glenwood Springs is considered nonstationary.

Test for Independence

Serial correlation is a measure of the degree of linear dependence among successive observations of a series that are separated by k time units. For a series of annual floods, the units of k are given in years. If an annual flood series is independent in time, its serial correlation coefficients, denoted by r(k) in which k ranges from 1 to n - 1, are not significantly different than zero. To verify linear independence of the annual peak-flow series it is necessary to perform a test of significance for each serial correlation coefficient. From a practical standpoint, however, it is usually sufficient to check only r(1). For this case, the following hypotheses are postulated: $H_0 : \rho(1) = 0$ versus $H_1 : \rho(1) \neq 0$, in which $\rho(1)$ is the population value of the first serial correlation coefficient. If we assume a circular, normal, stationary series of annual floods, r(1) is given by the following:

$$r(1) = [\Sigma(q_i q_{i+1}) - n\bar{q}^2]/[(n-1)s_q^2] \tag{7}$$

where $\bar{q}$ is the mean of the annual flood series and $s_q{}^2$ is the variance. Because a circular series is one that closes on itself (q_n followed by q_1), the summation is taken over all n-values of the flood record. Under these conditions, confidence limits for r(1) are given by the following equation:

$$CL[r(1)] = \left\{-1 \pm z_c[1-(\alpha/2)](n-2)^{1/2}\right\}/(n-1) \tag{8}$$

in which $z_c[1 - (\alpha/2)]$ is the critical value of the standard normal deviate for a two-sided test at the α-level of significance. If r(1) falls inside the confidence limits, H_0 is accepted (8).

The data from Table 2 give r(1) = 0.349. At the 1 percent level of significance, $z_c = 2.576$ and Equation 8 yields CL[r(1)] = (-0.301, 0.275). Since r(1) falls outside these limits, it appears that the annual flood series of the Colorado River is not independent.

Recall that this test is based on a stationary series. The requirement for stationariness is necessary because long-term trends introduce significant positive correlation into a series (9). If nonstationariness is manifest as a linear trend (for example, in Equation 3), the positive correlation expected at r(1) is given by the following equation:

$$r(1)^* = (b^2/12s_q^2)(n^2 - 2n - 2) \tag{9}$$

By using b = -180.4 from Equation 6, the positive correlation expected from the linear trend is r(1)* = 0.388. Subtracting r(1)* from r(1) gives a value that is not significantly different than zero. Because the apparent significant serial correlation results from a negative linear trend and not from dependence among successive peak flows, the annual flood series of the Colorado River at Glenwood Springs should be considered independent.

Another test for linear independence of an annual peak-flow series is as follows. Define a "turning point" T whenever $q_{i-1} > q < q_{i+1}$ or $q_{i-1} < q > q_{i+1}$. For an independent series, confidence limits for t are given by the following:

$$CL[T] = \left\{2(n-2) \pm z_c[1-(\alpha/2)]\,[(16n-29)/10]^{1/2}\right\}/3 \tag{10}$$

The hypothesis that the annual peak-flow series is independent in time is accepted if T falls inside the confidence limits (10). For the annual flood series of the Colorado River, T = 45 and n = 80 - 3 = 77 since on three occasions observed peak flows are identical for two successive years. At the 1 percent level of significance, Equation 10 gives CL[T] = (40.6, 59.4). Since T falls inside the confidence limits, the annual floods of the Colorado River are considered independent. This conclusion agrees with the result of the serial correlation test.

Test for Homogeneity

Statistical tests designed to ascertain whether or not data are from different populations invariably require that the data be divided into two subsamples. For example, one common application is to test for significant differences in the characteristics of snowmelt floods and rainfall floods when both are present in the peak-flow series. For the Colorado River, however, the investigation concerns flood data that are changing with time. Therefore, in the test for homogeneity, annual floods observed during the early period of record will be compared with those observed more recently.

One method, which requires only that the data be independent, is the Mann and Whitney U-test (11). From a flood series that is ranked in order of decreasing magnitude, the following two statistics are calculated:

$$U_1 = uv + (u/2)(u+1) - R_u \tag{11}$$

$$U_2 = uv - U_1 \tag{12}$$

where u and v are the subsample sizes (u + v = n) and R_u is the sum of the ranks assigned to the sample of size u. Let U be the smaller of U_1 and U_2. Then the test statistic T is defined as follows:

$$T = [U - (uv/2)]/[(uv/12)(u+v+1)]^{1/2} \sim N(0,1) \tag{13}$$

If tied observations are present, the following correction is made:

$$C = (1/12)(t^3 - t) \quad (14)$$

in which t is the number of observations tied at a given rank and C is computed only for those tied observations that appear in both subsamples. The test statistic T now becomes the following:

$$T = [U - (uv/2)] / \{[uv/(n^2 + n)] [(n^3 - n)/12 - \Sigma C]\}^{1/2} \sim N(0,1) \quad (15)$$

For subsamples, both containing more than 20 observations, T is assumed to have the standard normal distribution. The hypothesis that both subsamples are from the same population is accepted at the α-level of significance if the following condition is met:

$$|T| < z_c [1 - (\alpha/2)] \quad (16)$$

By using the data given in Table 2, subsample 1 is taken as the data from 1900 to 1929 (u = 30) and subsample 2 covers 1930 to 1979 (v = 50). These subsamples yield $R_u = 682$, $U_1 = 1283$, $U_2 = 217$, and $\Sigma C = 1.0$. Equation 15 gives T = -5.36. At the 1 percent level of significance $z_c = 2.576$. Since $|T| > z_c$, the hypothesis that both subsamples are from the same population is rejected. Noting that the test for stationariness has previously identified a significant linear trend, the result of the Mann and Whitney U-test is expected.

Table 3. Tests to assess annual peak-flow series (1900-1979) of Colorado River at Glenwood Springs.

Condition	Test	Result[a]	Reference
Stationary	Linear trend	No	(7)
Independent	Serial correlation	Yes	(7, 8, 9)
Independent	Turning point	Yes	(7, 10)
Homogeneous	Mann and Whitney	No	(11)
Reliable	Examination	Yes	(4)

[a]Yes: does meet criteria for conventional flood frequency analysis; no: does not meet criteria for conventional flood frequency analysis.

Note on Reliability

The statistical methods reviewed thus far have roots in hypothesis testing. Similar techniques, however, are not effective for evaluating the reliability of flood information. Although flood records always contain inaccuracies, statistical tests are generally unable to discriminate between data variability due to chance fluctuation and data variability resulting from random error.

Nevertheless, there are several ways to screen the flood series for suspected errors. One is to compare the data against concurrent records from nearby watersheds. This check may signal discrepancies that warrant further investigation. In most cases, data reliability is not a controlling factor during flood frequency analysis (4).

Summary of Data Tests

Statistical methods have been presented to test an annual flood series for three properties: sta-

Figure 1. Watershed of Colorado River above Glenwood Springs.

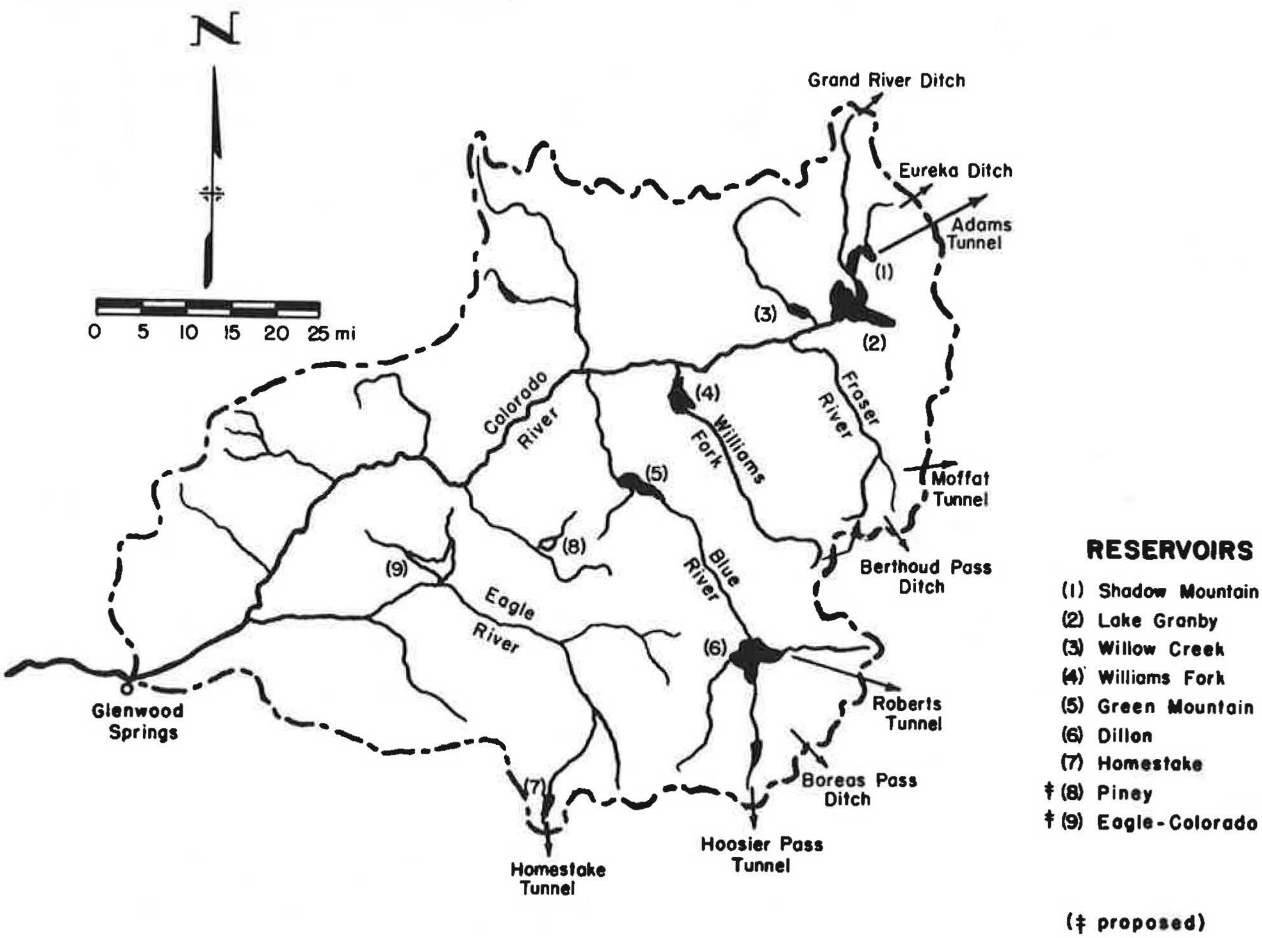


Table 4. Transmountain diversions from Colorado River and tributaries upstream of Glenwood Canyon.

Diversions	Year	Origin	Destination
Ewing Ditch	1880	Piney Creek	Arkansas River
Grand River Ditch	1892	Colorado River	Cache La Poudre River
Berthoud Pass Ditch	1909	Fraser River	West Fork Clear Creek
Boreas Pass Ditch	1914	Indiana Creek	Tarryall Creek
Fremont Pass Ditch[a]	1929	Tenmile Creek	Arkansas River
Columbine Ditch	1931	Eagle River	Arkansas River
Wurtz Ditch	1932	Piney River	Arkansas River
Moffat Tunnel	1936	Fraser River	South Boulder Creek
Eureka Ditch	1940	Colorado River	Big Thompson River
Gumlick Tunnel	1940	Williams Fork River	South Boulder Creek
Adams Tunnel	1947	Lake Granby	Big Thompson River
Hoosier Pass Tunnel	1952	Blue River	Middle fork of South Platte River
Roberts Tunnel	1963	Dillion Reservoir	North fork of South Platte River
Homestake Tunnel	1967	Eagle River	Arkansas and South Platte Rivers
Vidler Tunnel	1971	Montezuma Creek	Clear Creek

[a]Discontinued in 1943.

Table 5. Reservoirs from Colorado River or tributaries upstream of Glenwood Canyon.

Reservoir	Usable Storage (acre-ft)	Year Storage Began	Location	Operating Agency
Ralston	11 000	1937	East slope	DWB[a]
Marston	17 000	1939	East slope	DWB
Williams Fork	97 000	1939	West slope	DWB
Green Mountain	147 000	1942	West slope	WPRS[b]
Shadow Mountain	18 000	1947	West slope	WPRS
Lake Granby	466 000	1949	West slope	WPRS
Horsetooth	144 000	1951	East slope	WPRS
Willow Creek	10 000	1953	West slope	WPRS
Carter Lake	113 000	1954	East slope	WPRS
Gross	43 000[c]	1955	East slope	DWB
Montgomery	5 000	1957	East slope	CCS[d]
Dillion	254 000	1963	West slope	DWB
Homestake	43 000	1967	West slope	CCS
Strontia Springs	8 000	-[e]	East slope	DWB
Piney	40 000	-[f]	West slope	DWB
Eagle-Colorado	350 000	-[f]	West slope	DWB
Two Forks	860 000	-[f]	East slope	DWB, WPRS

[a]Denver Water Board. [b]Water and Power Resources Service. [c]To be increased to 113 000 acre-ft. [d]City of Colorado Springs. [e]Under construction. [f]Proposed.

Figure 2. Transmountain diversions from Colorado River watershed above Glenwood Springs.

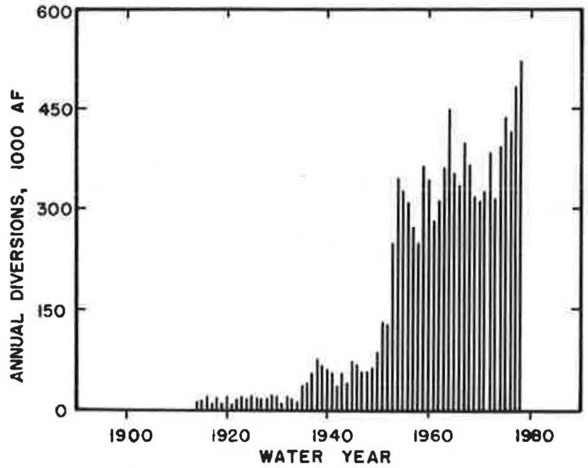
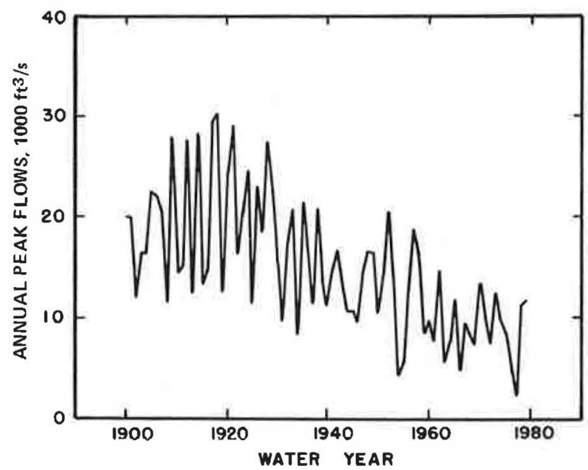


Figure 3. Annual peak flows of Colorado River at Glenwood Springs.



tionariness, independence, and homogeneity. Results of the tests applied to the annual peak-flow series of the Colorado River at Glenwood Springs are summarized in Table 3. The references cited in Table 3 demonstrate similar applications of other statistical tests available to aid practitioners involved in flood frequency analyses.

Results of the tests for stationariness and homogeneity reveal that conventional methods of analysis are not suitable for flood frequency determinations of the Colorado River at Glenwood Springs. Analysis of these data requires a technique that is sensitive to the sequential arrangement of the annual peak-flow series. This technique is known as time-series analysis. Before this methodology is presented, it is necessary to review the processes that affect the stream flow of the Colorado River above Glenwood Springs.

ANATOMY OF WATERSHED

Basin Description

From its headwaters high along the Continental Divide in Rocky Mountain National Park, the Colorado River flows southwest approximately 130 miles to Glenwood Canyon. Along this reach, as shown in Figure 1, five major tributaries--the Fraser, Williams Fork, Piney, Blue, and Eagle Rivers--join the Colorado River and contribute to a drainage area of 4560 miles^2.

Rugged snow-capped peaks, some rising more than 14 000 ft, frame the eastern boundary of the watershed. Below the headwaters, rolling alpine meadows yield to subalpine stands of aspen and conifer that make up extensive tracts of national forest. Drainage geomorphology evolves from intermittent streams of snowmelt cascading down glacial cirques to perennial meandering rivers flowing through broad U-shaped valleys. Although summer thunderstorms may cause appreciable flows on tributary reaches, annual peak flows on the Colorado River result from snowmelt runoff during the spring.

Diversions and Storage

The majority of Colorado's population live on the plains along the eastern foothills of the Rocky Mountains. In this arid region the water supply is not sufficient to support the demands of agricul-

Figure 4. Granby Reservoir, 1979 annual operating plan.

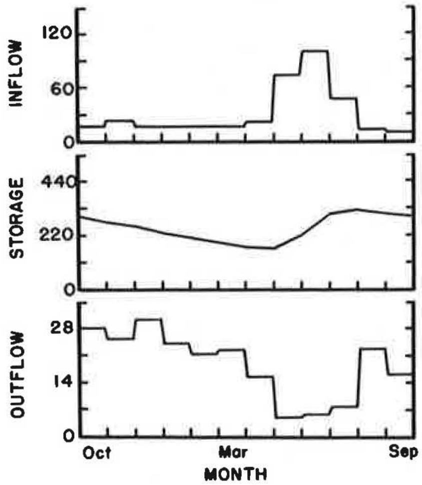
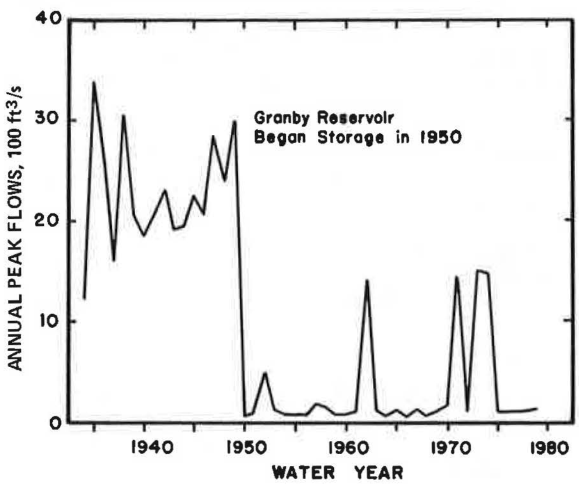


Figure 5. Annual peak flows of Colorado River at Granby.



tural, industrial, and municipal interests. To supplement the stream flow of the eastern-slope rivers, snowmelt runoff from the Colorado River watershed is diverted through the Continental Divide to the eastern plains. The transmountain diversions that operate above Glenwood Canyon are given in Table 4.

The annual hydrograph of the Colorado River exhibits pronounced seasonality. During months of low stream flow, it is impossible to meet demands for transmountain water. To help ensure a reliable supply, a network of storage reservoirs has been constructed (Table 5). Note that the reservoirs located east of the Continental Divide receive water from the Colorado River via one or more of the diversions listed in Table 4.

The projects shown in Tables 4 and 5 demonstrate that development of water resources in the Colorado River watershed has proceeded at a rapid pace, especially during the past 50 years. This point is substantiated further in Figure 2, which shows the annual volume of water exported from the watershed since 1914. Plans for additional storage reservoirs testify that inevitable future development along the eastern foothills will increase demands for transmountain water, and hence the trend in Figure 2 is expected to continue.

Streamflow Records

The Colorado River at Glenwood Springs has been monitored since 1900 by the U.S. Geological Survey. A time-series plot of the annual peak flows of the river is shown in Figure 3.

Reservoir Operation

A distinct feature of Figure 3 is the downward trend of the annual floods, particularly evident since the 1930s. Recall that the test for stationariness has indicated that this negative trend is statistically significant--that is, the trend cannot be attributed only to chance variation. This trend is likely the result of man-made river controls. It would be difficult to isolate and quantify the impact of each transmountain diversion and storage reservoir on the annual peak-flow series. Although it is not the intention of this paper to provide such an account, a brief description of the runoff and water-exchange system is included.

The primary objective of the reservoir network is to smooth out the seasonal fluctuations that characterize the hydrograph of the Colorado River and its headwater tributaries. To exploit the time-transient availability of water, reservoir operation is synchronized closely with the annual cycle of snowmelt runoff. In practice this means that the usable storage of each reservoir typically reaches its lowest level of the year just prior to the time of peak runoff. The depleted storage levels, which result from reservoir releases to downstream users and to transmountain diversions during periods of low stream flow, are replenished during the peak snowmelt season. Thus, annual operation of the reservoir network is a repeated pattern of water storage and water release during periods of high flow and low flow, respectively. A typical example appears in Figure 4 (12).

It should be emphasized that the timing of the reservoir filling operation guarantees maximum impact of the annual peak flows of the Colorado River at Glenwood Canyon. Of course, reservoir releases to the Colorado River are necessary to honor downstream senior water rights or to satisfy minimum required stream flows. However, these releases occur during periods of low snowmelt and consequently do not affect the peak flow of the river.

Significant changes in the characteristics of the annual peak-flow series are inevitable consequences of stream flow regulation. For example, consider the regulatory effect of Granby Reservoir, which has been in operation since 1950. As shown in Figure 5, this date coincides with an abrupt change in the sequence of annual peak flows at this site. The impact of Granby Reservoir is demonstrated further with the data given below:

Statistic	Before Regulation	During Regulation
Year	1934-1949	1950-1979
Mean (ft^3/s)	2200	300
SD (ft^3/s)	550	470

In summary, the construction of numerous storage reservoirs and transmountain diversions has significantly altered the virgin conditions of the Colorado River watershed above Glenwood Canyon. These water projects appreciably affect the annual peak discharge of the Colorado River. Therefore, a flood frequency analysis of the annual flood series must consider the impact of historical development within the watershed.

Figure 6. CMA of annual peak flows of Colorado River at Glenwood Springs.

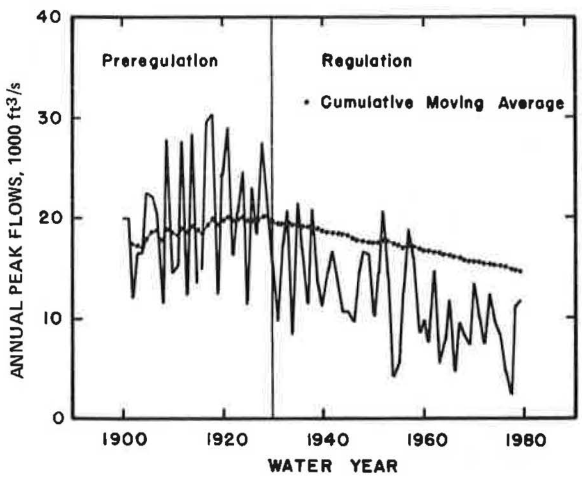
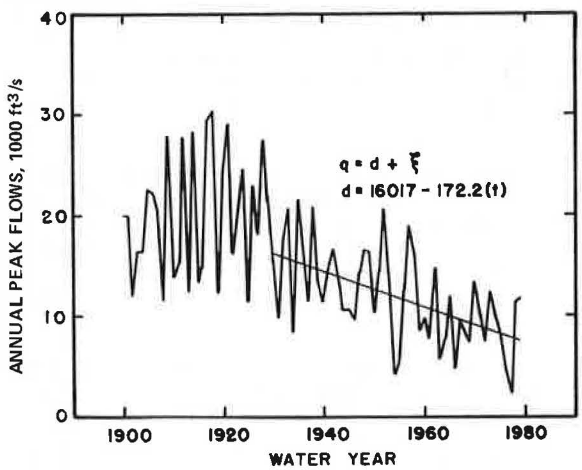


Figure 7. Linear-trend fit to regulation-period annual peak-flow series of Colorado River at Glenwood Springs.



OPTIONS FOR ANALYSIS OF REGULATED ANNUAL PEAK FLOWS

Reservoir-Simulation Approach

One method used to estimate frequencies of regulated peak flows is to route the entire historical runoff record through the reservoir system and then analyze the outflows by graphical techniques (13). This method requires a watershed model capable of simulating the annual operation of all existing and proposed reservoirs. At present, no watershed model has been developed for the Colorado River upstream of Glenwood Canyon. Consequently, this approach is not a practical option for estimating flood frequencies of the Colorado River.

Time-Series Approach

Another method that does not require reservoir modeling but instead deals directly with the observed sequence of annual peak flows is time-series analysis. In the time-series approach, each value of the flood series is considered to be a combination of a deterministic component and a random, or stochastic, component. The deterministic component represents the linear trend in the series. This trend has been shown to reflect two peculiar properties of the data--nonstationariness and serial correlation. The basic strategy of time-series analysis, then, is to remove the deterministic trend and investigate the properties of the residual stochastic components. If the stochastic residuals satisfy the data assumptions needed for flood frequency analysis, they can be used to obtain flood frequency estimates of the Colorado River.

TIME-SERIES ANALYSIS

Preregulation and Regulation Periods

In Figure 6, superimposed on the annual peak flows of the Colorado River at Glenwood Springs is a sequence of solid circles that represents the cumulative moving average (CMA) of the series. The CMA for any given year is equal to the average value of all annual floods that have occurred from 1900 through that particular year. The CMA sequence reveals that since 1930 there has been a progressive decrease in the mean value of the annual peak-flow series. For the purposes of this time-series analysis, the annual peak-flow record is separated into two periods--preregulation and regulation. Considering the scenario of development within the watershed and noting the trend of the CMA, the preregulation period is designated by the years 1900-1929 and the regulation period by the years 1930-1979.

A shortcoming of many flood frequency studies is a lack of data. In this case, however, the central issue concerns the representativeness of the data. As documented earlier (Figure 5), reservoir regulation can appreciably alter the characteristics of the annual flood series. Therefore, in order to better reflect conditions of the watershed that today influence the stream flow of the Colorado River at Glenwood Canyon, the time-series analysis treats only the regulation series.

Quantifying the Trend

Each peak-flow value of the regulation series is assumed to be composed of two parts--a deterministic component and a stochastic residual, which is stated mathematically as follows:

$$q = d + \xi \qquad (17)$$

where

q = annual peak flow,
d = deterministic component, and
ξ = stochastic residual.

The stochastic residuals constitute the portion of the regulation series attributed to chance variation. These residuals are assumed to be stationary, independent, and homogeneous random variables. The deterministic components represent the portion of the series described by any long-term trend. By definition, the outcome of a deterministic process is known for any appropriate input. Hence, the magnitude of the deterministic component is specified whenever its chronological position in the time series is given.

Following the same reasoning offered in the test for stationariness, the trend is assumed to be a linear decrease in the mean value of the regulation peak-flow series. By using least-squares regression, the deterministic linear trend of the regulation series is as follows:

$$d_i = 16\,017 - (172.2)t_i \qquad (18)$$

where t_i is the year minus 1930 and r is 0.54.

Figure 8. Adjusted regulation-period annual peak flows of Colorado River at Glenwood Springs.

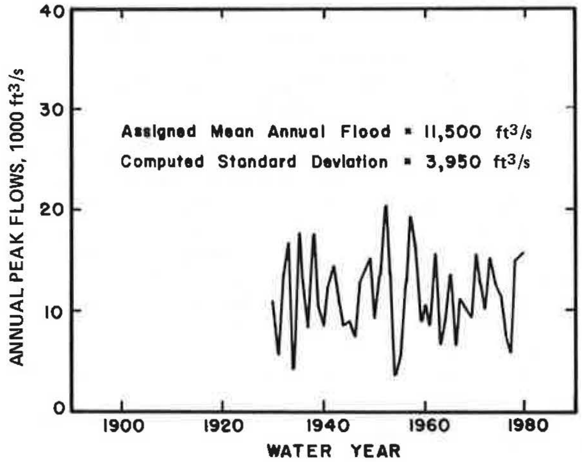


Table 6. Statistics for regulation-period annual peak flows (1930-1979) of Colorado River at Glenwood Springs.

Sample Statistic	Peak Flow[a]	Stochastic Residual[b]	Adjusted Peak Flow[b]
Mean (ft^3/s)	11 800	0.00	11 500[c]
SD (ft^3/s)	4 690	3950	3 950
Skewness	0.21	0.06	0.06

[a] Linear trend present. [b] Linear trend removed. [c] Assigned value.

This trend is shown as the straight line that extends from the year 1930 through 1979 in Figure 7. The test statistic given in Equation 4 indicates that the slope of this line is different than zero (α = 1 percent) and hence confirms the importance of the negative trend in the regulation-period annual flood series.

Removing the Trend

The stochastic residual is deduced as the difference between the observed peak flow and the deterministic component for each year of record or as follows:

$$\xi_i = q_i - d_i \tag{19}$$

Substituting Equation 18 into Equation 19 gives this equation:

$$\xi_i = q_i + (172.2)t_i - 16\,017 \tag{20}$$

The series of stochastic residuals has the following characteristics:

$$\bar{\xi} = 0 \tag{21}$$

$$s_\xi = (1 - r^2)^{1/2}(s_\phi) \tag{22}$$

where

$\bar{\xi}$ = mean of stochastic residuals,
s_ξ = SD of stochastic residuals, and
s_ϕ = SD of regulation-period annual floods.

In Equation 22, r^2 is the coefficient of determination of the linear regression and represents the percentage of s_ϕ^2 that is explained by the linear deterministic trend. The remaining, or unexplained, variance of the annual floods in the regulation series is attributed to the variance of the stochastic residuals.

SD of the regulation-period annual floods is as follows:

$$s_\phi = 4690\ ft^3/s \tag{23}$$

SD of the stochastic residuals is found by substituting Equation 23 and the correlation coefficient of Equation 18 into Equation 22:

$$s_\xi = 3950\ ft^3/s \tag{24}$$

Comparison of Equations 23 and 24 shows that removal of the deterministic component causes a substantial reduction in the SD-value. This result is expected since a measurable portion of the total variance of the regulation series is contributed by the deterministic trend.

The mean value of the regulation-period annual floods is as follows:

$$\bar{q}_\phi = 11\,800\ ft^3/s \tag{25}$$

However, Equation 21 shows that the residual series is centered about zero. Accordingly, some values of the residual series are negative. This is apparent by noting that about half the regulation-period peak flows are located below the linear trend in Figure 7.

Because some of the stochastic residuals are negative, they should not be interpreted as annual peak flows or be used in a flood frequency study. To remedy this condition, a representative mean annual peak flow must be selected and added to each value of the stochastic residual series. There are no equations to help redefine the representative mean annual peak flow. Instead, the decision is subjective. Specification of the mean annual flood must be tempered with an understanding of the conditions that prevail during times of peak runoff. At the high extreme, the value of 11 800 ft^3/s given in Equation 25 could be used and thereby preserve the mean peak flow of the regulation series. At the low extreme, extrapolation of the least-squares regression (Equation 18) shows that the mean peak discharge could approach zero cubic feet per second by 2020. The compromise is somewhere between these limits. Although future development within the watershed is planned (Table 5), it is unlikely that the future pace will continue at the past rate. Hence, extrapolation of the regression line probably does not offer a reliable indication of the future mean peak flow. Besides, other factors too complicated to be described by linear regression (e.g., downstream water rights and minimum required reservoir releases) will prevent the mean peak flow from approaching zero cubic feet per second. It seems more realistic, therefore, to select a value closer to the upper limit of 11 800 ft^3/s. Now, if we consider that the I-70 project through Glenwood Canyon is scheduled for completion in the mid-1980s and note that additional upstream reservoirs are not planned for completion until the 1990s, a value of 11 500 ft^3/s was selected as a conservative representative mean annual flood of the Colorado River at Glenwood Springs.

The adjusted regulation annual peak-flow series, obtained by adding the selected mean value of 11 500 ft^3/s to each stochastic residual, is shown in Figure 8. The statistics of the regulation peak-flow series, the stochastic residuals, and the adjusted regulation peak-flow series are summarized in Table 6.

Figure 9. Normal distribution fitted to histogram of adjusted regulation-period annual peak flow of Colorado River at Glenwood Springs.

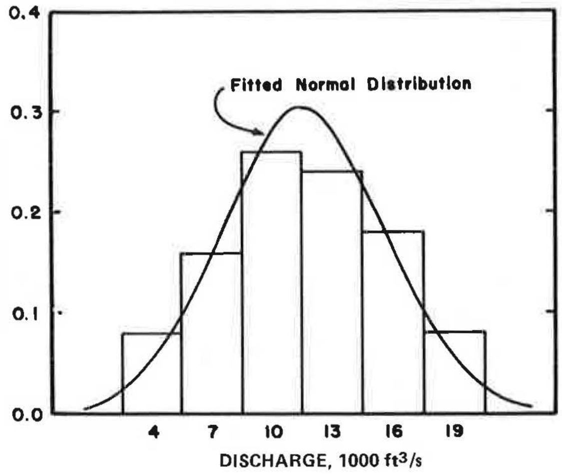


Table 7. Comparison of recommended flood-frequency estimates of Colorado River at Glenwood Springs.

Annual Exceedance Probability	Estimated Flow Discharge (ft^3/s) 1969 Study[a]	1979 Study[b]
0.50	16 000	11 500
0.02	25 000	20 000
0.01	26 500	21 000

[a]LP-III analysis of periods 1900-1968 and 1942-1968 (5).
[b]Time-series analysis of 1930-1979 (6).

ESTIMATING FLOOD FREQUENCIES

Testing Adjusted Flood Series

The data tests outlined earlier were used to investigate the properties of the adjusted regulation-period annual peak-flow series. Results show that the adjusted flood series is stationary, independent, and homogeneous. The adjusted series therefore qualifies for frequency analysis by conventional statistical methods.

Selecting the Distribution

The general relationship for estimating flood flows is as follows:

$$q_p = \bar{q} + k_p s_q \qquad (26)$$

where

q_p = flood that has p percent chance of being exceeded in any year,
k_p = frequency factor for p percent chance flood,
$\bar{q}$ = mean of annual floods, and
s_q = SD of annual floods.

The frequency factor is a function of the selected exceedance probability and the distribution of the annual floods. To obtain flood estimates of the Colorado River at Glenwood Springs, then, it is necessary to fit the adjusted regulation-period annual floods to a probability distribution.

The skewness coefficient, a statistic that measures the symmetry of a data sample, is sometimes helpful in selecting a distribution (14). Because the skewness coefficient of the adjusted regulation-period annual floods is close to zero (Table 6), any symmetrical distribution is a reasonable candidate for the underlying probability distribution. Two possibilities are the normal distribution and the two-parameter gamma distribution.

The adjusted regulation-period peak-flow series was fitted to each distribution by the maximum-likelihood method (15). By using the chi-square test to check for goodness of fit, both the normal and the two-parameter gamma distributions were accepted (α = 1 percent) as suitable approximations for the theoretical distribution of the adjusted regulation-period annual floods. The normal distribution was selected to represent the adjusted series because, according to the chi-square test, it provided a slightly better fit than the two-parameter gamma distribution did. In Figure 9 the histogram of the adjusted regulation-period floods is shown with the fitted normal curve.

If we substitute the statistics of the adjusted series, Equation 26 becomes the following:

$$q_p = 11\,500 + z_p (3950) \qquad (27)$$

in which z_p is the standard normal deviate that corresponds to the exceedance probability p. Equation 27 was used to estimate the flood flows of the Colorado River needed for design of I-70 through Glenwood Canyon. The results rounded to the nearest thousand in units of cubic feet per second are given in Table 7 under the heading for the 1979 study.

Discussion of Results

There may be some reluctance to accept flood flows estimated from a sample of adjusted data. Note, however, that individual values of the adjusted regulation-period annual flood series are byproducts of the methodology used to obtain the statistics of the adjusted series. Therefore, any concern should focus on the selected mean annual flood and the computed SD rather than on specific values of the adjusted flood series.

SD of the adjusted flood series is a direct analytical consequence of removing the linear deterministic trend from the peak-flow series. Of course, other types of trends could be investigated. However, because a linear relationship adequately describes the trend in the peak-flow series, there is no reason to pursue other more complicated trends that require estimation of additional regression parameters.

Although the mean value of the adjusted flood series was determined subjectively, the selection was based on careful consideration of existing and anticipated conditions in the watershed. This situation emphasizes that judgment remains an essential element in any flood-frequency analysis. For the purpose of estimating future flood flows, the computed values of SD and the assigned value of the mean annual flood are more representative of the watershed than are the statistics from the original flood series.

The results presented in Table 7 show that peak-flow estimates obtained with the time-series analysis are substantially less than those obtained previously with LP III analyses. The time-series approach was able to recognize and to treat the impact of reservoir regulation on the annual peak-flow series of the Colorado River. Therefore, the results of the time-series analysis more closely reflect the present-day character of the Colorado River at Glenwood Canyon.

CONCLUSION

Conventional methods of flood frequency analysis often are not suitable for watersheds in which conditions are changing with time. For such cases, a versatile alternative is time-series analysis, which considers the magnitude, the frequency, and the sequential order of the flood data. The time-series approach is formulated to reflect the factors and circumstances that significantly influence peak flows. Consequently, resulting flood estimates are representative of prevailing watershed conditions.

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Dynamics Approach for Monitoring Bridge Deterioration

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In conjunction with a fatigue test of a full-scale in situ three-span highway bridge, an investigation was undertaken to evaluate the use of changes in dynamic properties of the bridge as a possible means of detecting structural deterioration due to fatigue cracks in the girders. Cyclic-loading tests (transient and steady-state) were conducted to determine the changes in dynamic properties. The loading was imposed by a moving-mass, closed-loop electrohydraulic actuator system. Several different dynamic tests were employed in the investigation to determine the modal viscous damping ratios, stiffness, and mechanical impedance of the bridge at selected intervals during the fatigue loading. Acoustic emission sensors were also used to monitor the growth of fatigue cracks in the girders. The results show that changes in the bridge stiffness and vibration signatures in the form of mechanical-impedance plots are indicators of structural deterioration caused by fatigue. Stiffness coefficients were calculated from the experimental mode shapes on the basis of a multidegree-of-freedom system that uses modified coupling. The average reduction in stiffness was approximately 20 percent. This reduction was attributed to the combined deterioration of the bridge deck and steel girders. Mechanical-impedance plots were made from frequency-sweep tests, which included five resonant modes. Early changes in the mechanical-impedance plots were related to the deterioration of the bridge deck. Subsequent changes in these plots correlated with the fatigue cracking in the steel girders. An evaluation of the acoustic emission data showed that the sensors were able to detect the rapid critical crack growth in one girder.

At this time there is a substantial amount of research under way on the techniques used for monitoring structural deterioration. The types of techniques may be broadly classified under the following categories--nondestructive-testing methods and vibration-response methods. Much of the recent development in vibration-analysis techniques for monitoring structural integrity (1-4) stems from the needs of the offshore industry.

In general, nondestructive-testing procedures can be time-consuming and costly. This becomes evident when the structures are large, such as multispan bridges and offshore platforms. Nonetheless, the nondestructive tests that involve ultrasonic examinations and visual inspection are two of the most effective means of locating deterioration in a structure. As a consequence of the cost and time involved to accomplish nondestructive tests, alternative methods that will reduce the frequency of these tests are desirable.

Typically, a vibration-response method employs accelerometers to measure the response of the structure from either environmental forces or applied excitation forces. The data are analyzed to establish prescribed dynamic system parameters. Any significant changes in subsequent evaluations of these parameters are interpreted as fatigue damage in structural members or foundation settlement. In this approach to monitoring, vibration-response data provide a surveillance of the structure on a broad basis.

Many of today's highway bridges have a multitude of welded connections and details. These weldments