Environ Earth Sci (2017) 76:743
DOI 10.1007/s12665-017-7086-7

THEMATIC ISSUE

Adaptive reservoir flood limited water level for a changing environment

Xiaoqi Zhang[1,2] · Pan Liu[1,2] · Hao Wang[3] · Xiaohui Lei[3] · Jiabo Yin[1,2]

Received: 8 November 2015 / Accepted: 21 October 2017 / Published online: 31 October 2017

Abstract As streamflow is non-stationary due to climate change and human activities, adapting reservoir operation in the changing environment is of significant importance. Specifically, the flood limited water level (FLWL) needs to be re-established to ensure flood safety when the reservoir inflow is altered. The aims of this study are: (1) to clarify the relationship between the FLWL and streamflow when statistical parameters of the flood peak and volume vary through time and (2) to re-establish the FLWL when the reservoir inflow changes under the non-stationary condition. The adaptive FLWL is derived based on flood routing of non-stationary design floods, and the flood risk probability is then estimated. With China's Three Gorges Reservoir (TGR) as a case study, the changing pattern of FLWL is quantified when statistical parameters (i.e., mean, C_V and C_S) of design floods have a linear temporal trend. The results indicate that the FLWL is sensitive with design floods, i.e., (1) means of design flood peak, 3-day volume, 7-day volume, 15-day volume and 30-day volume yearly decrease by 33 m^3/s, 0.008, 0.021, 0.482 and 0.905 billion m^3, respectively, (2) when the non-stationary design flood is used, the cumulative flood risk probability of the reservoir water level exceeding 175.0 m during 2011–2030 decreases from 1.98 to 1.82% with the conventional FLWL scheme and (3) the FLWL of the TGR could be re-set without increasing the flood risk probability, and the FLWL would increase about 4.7 m by 2030 in this non-stationary streamflow scenario. These findings are helpful to derive the FLWL in a changing environment.

Keywords Non-stationary · Flood limited water level · Adaptive operation · Flood risk

This article is a part of a Topical Collection in Environmental Earth Sciences on Climate Effects on Water Resources, edited by Drs. Zongzhi Wang and Yanqing Lian.

✉ Pan Liu
liupan@whu.edu.cn

1 State Key Laboratory of Water Resources and Hydropower Engineering Science, Wuhan University, Wuhan 430072, China

2 Hubei Provincial Collaborative Innovation Center for Water Resources Security, Wuhan 430072, China

3 China Institute of Water Resources and Hydropower Research, Beijing 100038, China

Introduction

Streamflow regime has been changed due to climate change and human activities (Greenwood et al. 1979), and a series of hydrologic extreme events have increased over the period of record (Obeysekera et al. 2011). As a result, some hydrologists have stated that "stationarity is dead" (Milly et al. 2008), and the hydrologic probability distribution estimation theories and methods which are based on stationary conditions are no longer helpful for studying the long-term variation pattern about water resources and flood evolution. Specifically, studies have demonstrated that hydrologic records in some rivers show non-stationarity in the form of increasing or decreasing tends (e.g., Olsen et al. 1999; Lins and Slack 1999; Douglas et al. 2000; Strupczewski et al. 2001b; Lin et al. 2014a), upward and downward shifts (e.g., Potter 1976; Salas and Boes 1980; McCabe and Wolock 2002; Fortin et al. 2004) or their combination (Villarini et al. 2009). Hence, adapting reservoir operation in a changing environment is of significant importance. Specifically, the flood limited water level (FLWL), which is the most key parameter for controlling the trade-off between activities of flood control and water conservation (Liu et al. 2008; Yun and Singh 2008; Li et al. 2010), needs to be re-established to ensure flood safety when the reservoir inflow alters.

The existing method to determine the FLWL is flood routing of design flood hydrographs by using the predetermined reservoir operating rules as follows (MWR 2006): (1) estimate the design flood hydrograph based on the maximum flood samples and (2) determine the FLWL based on the design flood hydrograph, i.e., routing the flood by setting it as the initial water level under the condition that the flood prevention risk does not increase. This approach can be conducted through a trial and error method.

The non-stationary issue of design flood has been addressed, where the most popular methods for non-stationary frequency analysis are the reductive method (Xie et al. 2009) and the time-varying moment method (Strupczewski and Kaczmarek 2001). In the reductive method, the non-stationary hydrologic series are decomposed by deterministic (a non-stationary term) and stochastic components. The stochastic component should be removed before the frequency analysis (Xie et al. 2009). Strupczewski and Kaczmarek (2001) and Strupczewski et al. (2001a, b) directly embed the linear and quadratic trinomial trend into the first and second moment of the extreme flood sequence's distribution to describe non-stationarity, and the frequency analysis is carried out directly on the original hydrologic sequence. In order to describe the variation tendency of the hydrologic sequence, the most popular method is that the distribution parameter is expressed as a function of time, such as linear, exponential, polynomial, logarithmic, spline interpolation function (Kharin and Zwiers 2005; Rigby and Stasinopoulos 2005). It can intuitively describe the changing trend of hydrologic sequences during the observed period. Recently, some hydrologists (Villarini and Serinaldi 2012; Xiong et al. 2014) have focused on the frequency analysis of non-stationary hydrologic series which is based on physical factors, and the relation between statistical parameters and the variable is often described by using the regression equation.

The reservoir FLWL can be controlled dynamically based on hydrologic forecasting. Aiming at the problems about dynamic control of reservoir operation, the multiple near-optimal solutions (Liu et al. 2011) and a variety of calculation models (Liu et al. 2006) were proposed. Zhou et al. (2014) proposed a model for a mixed reservoir system, which consists of a dynamic control operation module for a single reservoir, a dynamic control operation module for cascade reservoirs, and a joint operation module for mixed cascade reservoir systems. Zhao and Zhao (2014) proposed an improved multiple-objective DP algorithm for reservoir operation optimization. Zhang et al. (2015) used the Bayesian model averaging model to optimize the uncertainty of reservoir operating rules. Ouyang et al. (2015) proposed an optimal design model for the FLWL to optimize the flood control risk. Diao and Wang (2011) and Wang et al. (2011) putted forward a risk control model to determine the limit scope of the FLWL. Condon et al. (2014) focused on the flood risk under non-stationary condition. However, there is no systematic research on the design of reservoir FLWL when the influence of a changing environment is considered.

The aims of this paper are: (1) to clarify the relationship between the FLWL and the reservoir inflow when statistical parameters (mean, C_V and C_S) of the design flood have a linear temporal trend and (2) to re-establish the FLWL without increasing the flood risk probability under the non-stationary condition. The paper is organized as follows: second section describes the methods on estimation of statistical parameters module, the flood risk evaluation module and the FLWL derivation module. Third section addresses a case study of China's Three Gorges Reservoir (TGR), and then, results and discussions are given. Finally, conclusions are given in fourth section.

Methods

The steps to derive the adaptive reservoir FLWL are as follows (Fig. 1):

1. Based on the observed data, moving average and *L*-moment estimation are used to fit the linear function of statistical parameters. Namely, mean, C_V and C_S of streamflow vary through time (Section estimation of statistical parameters).
2. The flood risk is evaluated with two scenarios, i.e., stationarity and non-stationarity, in which the FLWL keeps at the original design value (Section flood risk evaluation).
3. The re-establishment of FLWL is derived based on flood routing of the non-stationary design flood (Section FLWL derivation).

Estimation of statistical parameters

In order to identify the relationship of statistical parameters over time, the moving average method is used to produce sample series, and then, the *L*-moment method (Hosking and Wallis 1997) is used to estimate the mean, C_V and C_S. Finally, functions of mean, C_V and C_S over time are quantified by using linear curve fitting method, respectively.

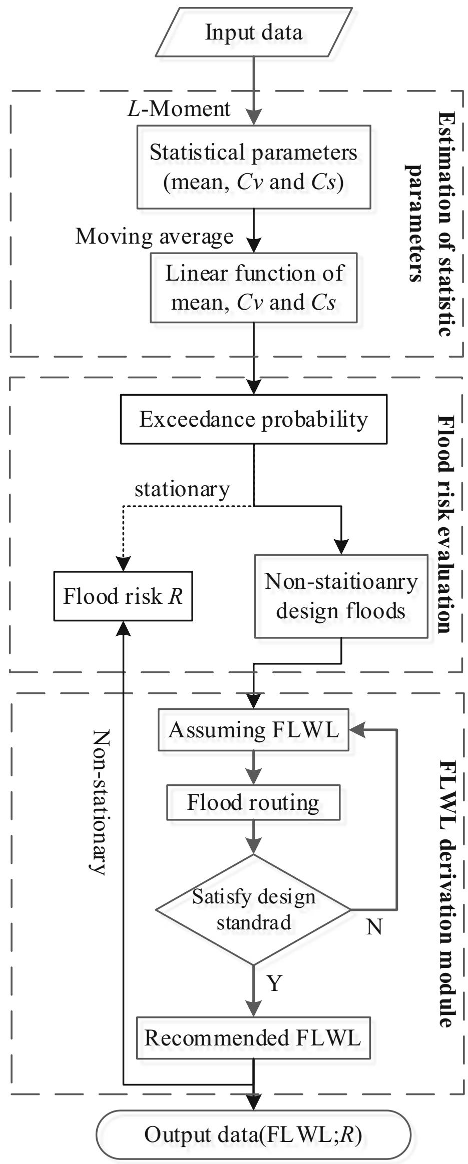

Fig. 1 Flowchart of the method for adaptive reservoir FLWL

The *L*-moment method is based on the probability weighted moment (PWM) (Greenwood et al. 1979), which is defined as follows:

$$\alpha_r = \int_0^1 x(1 - F(x))^r \mathrm{d}F(x) \tag{1a}$$

$$\beta_r = \int_0^1 xF(x)^r \mathrm{d}F(x) \tag{1b}$$

where x is a random variable, $F(x)$ and $f(x)$ are distribution and density functions, respectively. Hosking and Wallis (1997) define the *L*-moment (λ_r) based on PWM,

$$\lambda_r = \int_0^1 xP^*_{r-1}(F(x))\mathrm{d}F(x) \tag{2}$$

in which $P^*_r(u) = \sum_{k=0}^{r} \frac{(-1)^{r-k}(r+k)!}{(k!)^2(r-k)!}u^k$.

In order to facilitate the definition of the dimensionless *L*-moment, ratios of *L*-moment are as follow:

$$\tau_r = \lambda_r/\lambda_2 \quad r = 3, 4 \tag{3a}$$

where τ_3 (which means *L*-skewness) reflects skewness characteristics, while τ_4 (which means *L*-kurtosis) reflects kurtosis characteristics, and $L\text{-}C_V$ in Eq. (3b), which is used to reflect scale features.

$$\tau_2 = \lambda_2/\lambda_1 \tag{3b}$$

Combined with the distribution density function of P-III (Eq. 4), the *L*-moment method can be used to estimate statistical parameters of streamflow.

$$f(x) = \frac{\beta^\alpha}{\Gamma(\alpha)}(x - a_0)^{\alpha-1}e^{-\beta(x-a_0)} \quad (\alpha, \beta > 0; x > a_0) \tag{4}$$

Specifically, α, β and a_0 are undetermined parameters while $\Gamma(\alpha) = \int_0^\infty x^{\alpha-1}e^{-x}\mathrm{d}x$. Moreover, an approximate algorithm (Hosking and Wallis 1997) has been given in Eqs. (5), (6), (7a) and (7b),

$$\lambda_0 = a_0 + \alpha/\beta \tag{5}$$

$$\lambda_2 = \sqrt{\pi}\Gamma(\alpha + 0.5)/\Gamma(\alpha)/\beta \tag{6}$$

$$\tau_3 = \alpha^{-\frac{1}{2}}\frac{A_0 + A_1\alpha^{-1} + A_2\alpha^{-2} + A_3\alpha^{-3}}{1 + B_1\alpha^{-1} + B_2\alpha^{-2}} \quad (\alpha \geq 1) \tag{7a}$$

$$\tau_3 = \frac{1 + E_1\alpha + E_2\alpha^2 + E_3\alpha^3}{1 + F_1\alpha + F_2\alpha^2 + F_3\alpha^3} \quad (\alpha < 1) \tag{7b}$$

where A_0, A_1, A_2, A_3, B_1, B_2, E_1, E_2, E_3, F_1, F_2, F_3 are constant coefficients.

It is assumed that the sample is $x_{1:n} \leq x_{2:n} \leq \cdots \leq x_{n:n}$, and the calculation formulas of sample moment l_1, l_2 and l_3, which are corresponding to λ_1, λ_2 and λ_3, respectively, are as follows:

$$l_1 = b_0 \quad l_2 = 2b_1 - b_0 \quad l_3 = 6b_2 - 6b_1 + b_0 \quad \tau_3 = l_3/l_2 \tag{8a}$$

$$b_0 = \frac{1}{n}\sum_{j=1}^{n} x_{j:n} \quad b_1 = \frac{1}{n}\sum_{j=2}^{n}\frac{(j-1)}{(n-1)}x_{j:n}$$
$$b_2 = \frac{1}{n}\sum_{j=3}^{n}\frac{(j-1)(j-2)}{(n-1)(n-2)}x_{j:n} \tag{8b}$$

Flood risk evaluation

The traditional methods to derive return period and flood risk should satisfy two essential conditions, namely (1) extreme events follow a stationary distribution and (2) the occurrence of extreme events is independent or weakly dependent (Gumbel 1961; Leadbetter 1983; Salas and Obeysekera 2014).

It is assumed that Q_P is a design flood peak value of streamflow series, and the probability of streamflow Q_i, which is the random variable, exceeding Q_P, is expressed as p_i. It is assumed that Q_i exceeding Q_P would occur in the x^{th} year for the first time.

On the stationary condition, p_i is equal to a constant p whatever i is, and hydrologic series are independent. Hence, the probability of exceedance event occurs in year x is as follow:

$$f(x) = P(X = x) = (1 - p)^{x-1} p, \quad x = 1, 2, \ldots \tag{9}$$

which follows geometric probability distribution. Then, the mathematical expectation $E(X)$ is equal to $1/p$, which is known as the return period T.

$$T = E(X) = \sum_{x=1}^{\infty} xP(X = x) = \frac{1}{1 - (1 - p)} = \frac{1}{p} \tag{10}$$

It is assumed that the project life of a hydraulic structure is designed for n years, and the failure of the structure to facing a flood exceeding the design flood occurs before or at year n (Salas and Obeysekera 2014), and hence, the flood risk probability R is shown as follows:

$$R = P(X \le n) = p\sum_{x=1}^{n} f(x) = p\sum_{x=1}^{n} (1 - p)^{x-1} = 1 - (1 - p)^{n} \tag{11}$$

Due to the fact that streamflow is non-stationary in a changing environment, the exceedance probability would vary through time, namely $p_1, p_2, p_3, \ldots, p_t$. Therefore, the probability of exceedance event occurs in year x is as follows:

$$f(x) = P(X = x) = (1 - p_1)(1 - p_2)(1 - p_3)\cdots(1 - p_{x-1})p_x \tag{12a}$$

$$f(x) = P(X = x) = (1 - p_1)(1 - p_2)\cdots(1 - p_{x-1})p_x, x = 1, 2, \ldots \tag{12b}$$

Then, the flood risk probability R is as follows:

$$R = P(X \le n) = p_1 + p_2(1 - p_1) + \cdots + p_n(1 - p_1)(1 - p_2)\cdots(1 - p_{n-1}) \tag{13a}$$

$$R = P(X \le x) = \sum_{x=1}^{n} p_x \prod_{t=1}^{x-1} (1 - p_t) \tag{13b}$$

FLWL derivation

The FLWL derivation module is established based on flood routing of the design flood hydrographs. The steps to derive the design flood are as follows (Liu et al. 2015): (1) selection on a series of typical flood peak and flood volume values, (2) design floods based on hydrologic frequency analysis, (3) same frequency magnify method to obtain the design flood hydrograph, where the amplification coefficient of peak, the maximum flood volume for 3 days, 7 days, 15 days and 30 days are as follows:

$$K_Q = \frac{Q_P}{Q_D} \tag{14a}$$

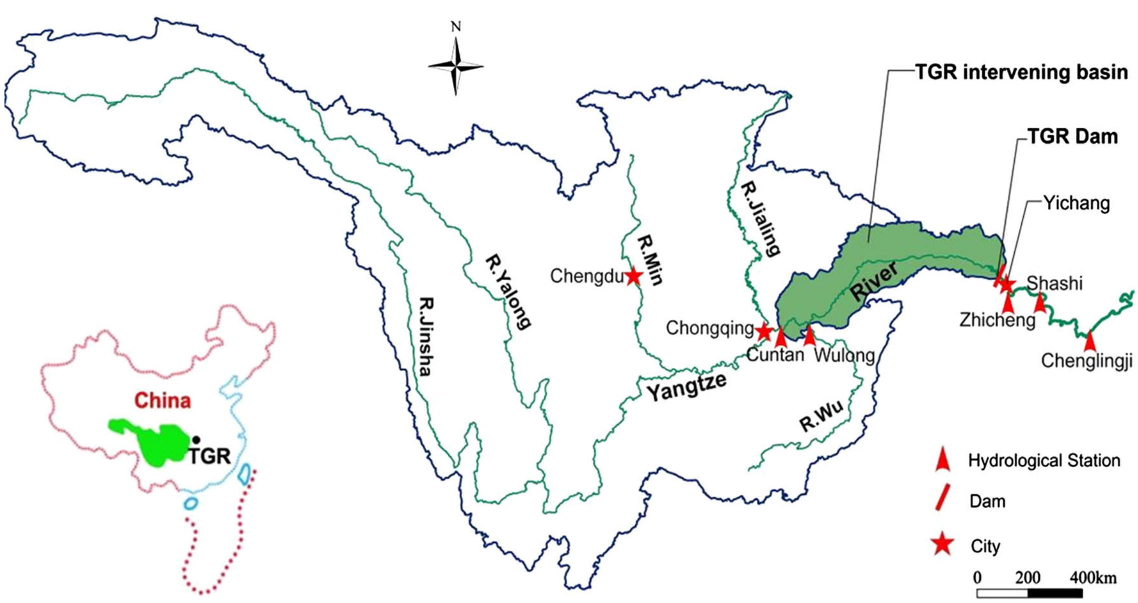

Fig. 2 Location of the China's Three Gorges Reservoir Basin

Table 1 Statistical parameters of TGR for stationary

Statistical Interval	Statistical parameters			Frequency (%)			
	E_X	C_V	C_S/C_V	1	0.1	0.01	0.01 (10% increment)
Q_P (m^3/s)	51,200	0.21	4.0	82,500	97,400	111,300	122,400
W_3 (billion m^3)	12.79	0.21	4.0	20.63	24.35	27.82	30.60
W_7 (billion m^3)	27.15	0.19	3.5	41.60	48.05	53.95	59.35
W_{15} (billion m^3)	51.51	0.19	3.0	78.27	89.79	100.23	110.25
W_{30} (billion m^3)	92.43	0.18	3.0	137.58	156.79	174.14	191.55

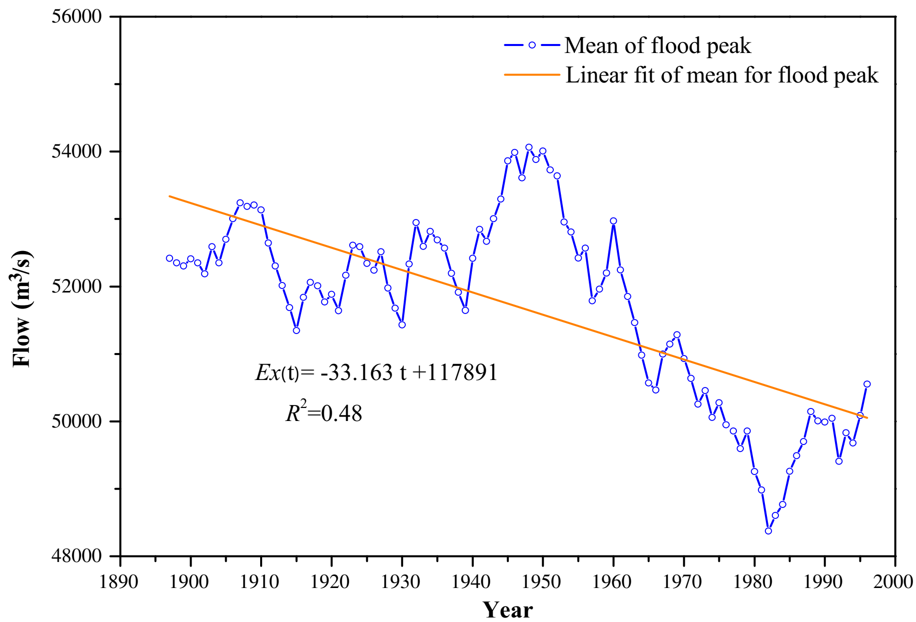

Fig. 3 Mean of flood peak $Ex(t)$ as a function of time t under non-stationary condition: The solid line is the linear fitting result, and the mean has a decline with time

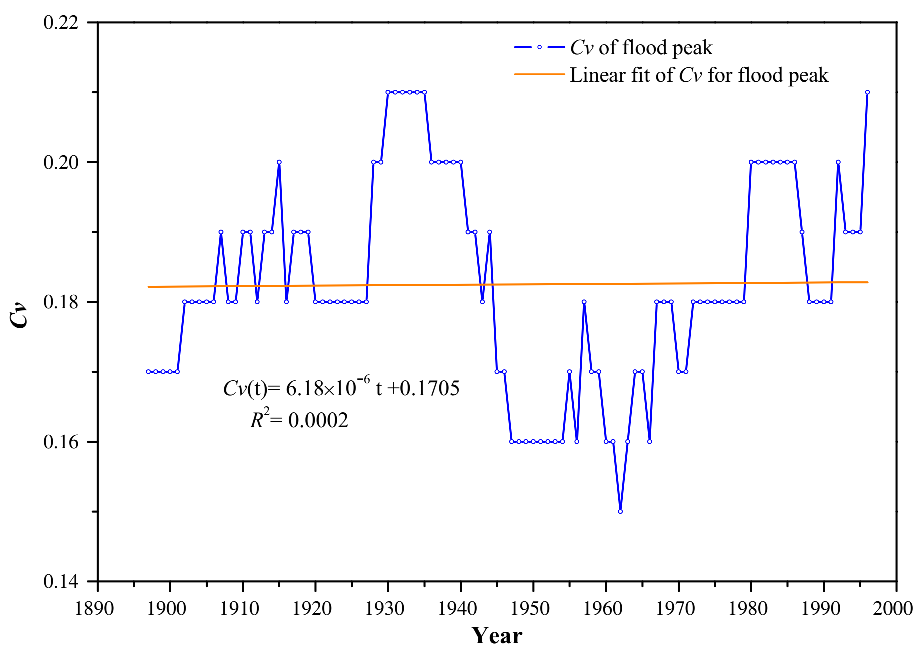

Fig. 4 The C_V of flood peak as a function of time t under non-stationary condition: The solid line is the linear fitting result, but the amplitude of variation of C_V changes with time is insignificant

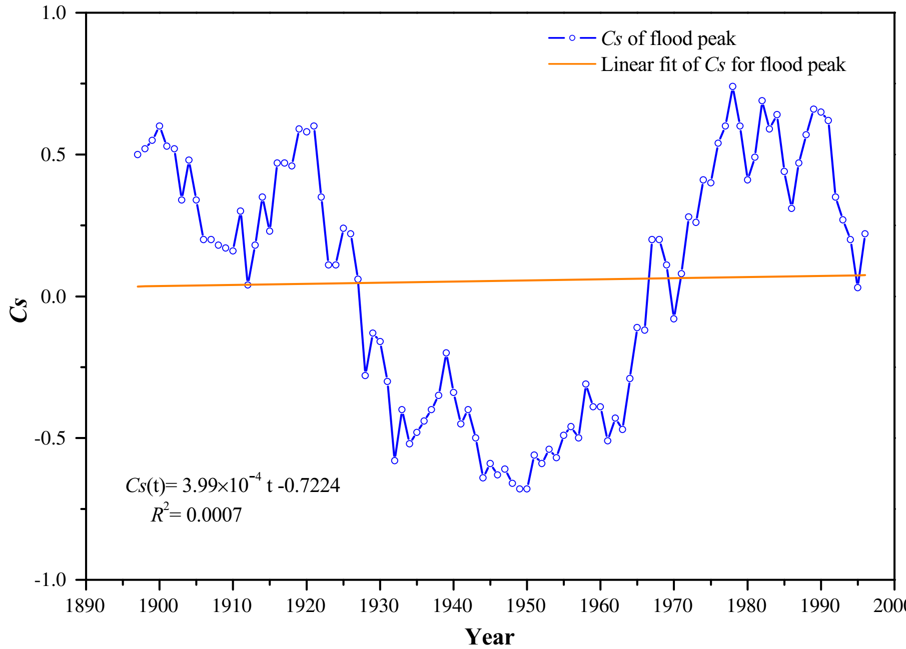

Fig. 5 The C_S of flood peak as a function of time t under non-stationary condition: The solid line is the linear fitting result, but the amplitude of variation of C_S changing with time is insignificant

Table 2 Estimated parameter values of flood peak for the Pearson-III distribution

Suppose	Statistical parameters for flood peak
Stationary	$E_X = 51{,}200$ m^3/s, $C_V = 0.21$, $C_S/C_V = 4.0$
Non-stationary	$E_X(t) = -33.163\ t + 117{,}891$, $C_V = 0.21$, $C_S/C_V = 4.0$

$$K_{w3-1} = \frac{W_{3P} - W_{1P}}{W_{3D} - W_{1D}} \quad (14b)$$

$$K_{w7-3} = \frac{W_{7P} - W_{3P}}{W_{7D} - W_{3D}} \quad (14c)$$

$$K_{w15-7} = \frac{W_{15P} - W_{7P}}{W_{15D} - W_{7D}} \quad (14d)$$

$$K_{w30-15} = \frac{W_{30P} - W_{15P}}{W_{30D} - W_{15D}} \quad (14e)$$

where Q_P is the design flood peak value, Q_D is the typical flood peak value, W_{iP} and W_{iD} are the maximum design flood volume value for i days ($i = 3$, 7, 15, 30) and the maximum typical flood volume value for i days, respectively.

Flood routing is formulated by the reservoir water balance equation (Deng et al. 2015) and the discharge equation, i.e., Eqs. (15a) and (15b),

$$\frac{Q_1 + Q_2}{2}\Delta t - \frac{q_1 + q_2}{2}\Delta t = V_2 - V_1 \quad (15a)$$

where Δt is the time interval, $Q_{\rm i}$ and q_i are reservoir inflow and release, respectively, V_i is reservoir storage, and $i = 1$, 2 denote the beginning and end of the time period Δt, respectively.

$$q(t) = f[V(t)] \quad (15b)$$

The discharge equation shows the relationship between the reservoir release and the reservoir storage, which is determined by reservoir operating rules. The trial and error method is used to resolve above equations.

Case study

Located in Yichang, Hubei Province (Fig. 2), China's Three Gorges Dam is 185.0 m high, the reservoir water is up to 175.0 m high. The hydropower station is equipped with 32 hydropower units, the single capacity of which is 700 MW. Overall, China's Three Gorges Project is a large-scale hydraulic project which combines multi-purpose uses simultaneously including flood control, power generation, shipping and so on.

The average annual discharge at the dam site is 14,300 m^3/s, and the average annual runoff is 451 billion m^3. The total area of the TGR is 1084 km^2, and the TGR belongs to the channel reservoir with the length and the width of about 600 km and 1.1 km, respectively. Furthermore, the total capacity of the reservoir is about 39.3 billion m^3.

Annual maximum data from 1882 to 2010 and eight historical floods (i.e., occurred in years of 1153, 1227, 1560, 1613, 1788, 1796, 1860 and 1870) are used for the case study. Furthermore, statistical parameters of the flood

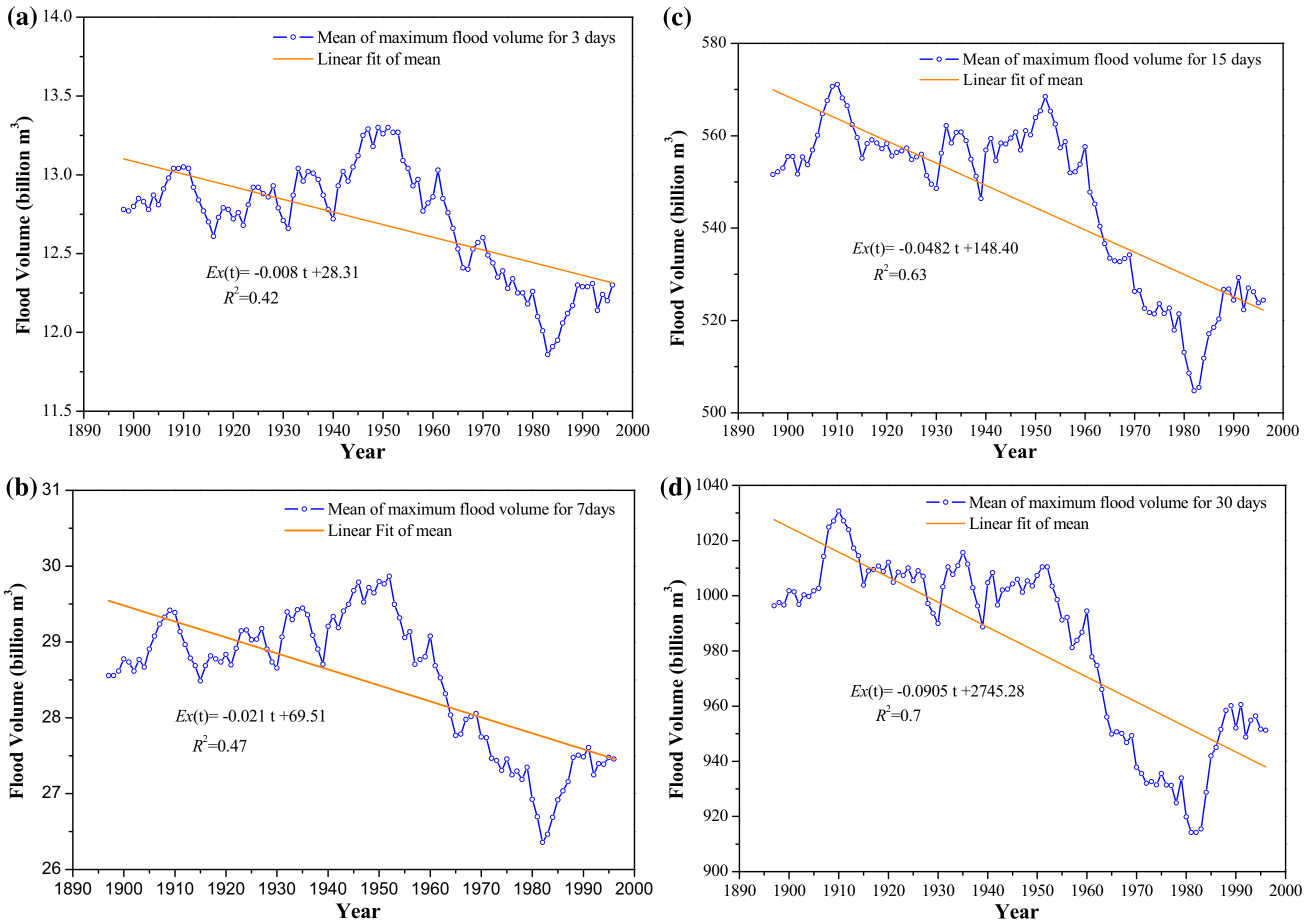

Fig. 6 Linear fitting results for the mean of maximum flood volume for *i* days ($i = 3, 7, 15, 30$): **a** the mean of maximum flood volume for 3 days, **b** the mean of maximum flood volume for 7 days, **c** the mean of maximum flood volume for 15 days, **d** the mean of maximum flood volume for 30 days

Table 3 Functions for estimated parameter values of flood volume under non-stationary condition for the Pearson-III distribution

Statistical Interval	Statistical parameters
MFV for 3 days	$E_X(t) = -0.0080\ t + 28.31$; $C_V = 0.21$; $C_S\ /\ C_V = 4.0$
MFV for 7 days	$E_X(t) = -0.0211\ t + 69.51$; $C_V = 0.19$; $C_S\ /\ C_V = 3.5$
MFV for 15 days	$E_X(t) = -0.0482\ t + 148.40$; $C_V = 0.19$; $C_S\ /\ C_V = 3.0$
MFV for 30 days	$E_X(t) = -0.0905\ t + 274.53$; $C_V = 0.18$; $C_S\ /\ C_V = 3.0$

MFV means maximum flood volume

peak and volumes for the TGR under stationary conditions are shown in Table 1.

Results and discussion

Results of statistical parameters

Linear functions of statistical parameters

With the 129-year observed data, the moving average is used to sample time series for each continuous thirty years. Statistical parameters (i.e., mean, C_V and C_S) of the TGR are then estimated by using the *L*-moment method. Finally, the changing tendency of statistical parameters over time is derived by using linear model. In this study, statistical parameters of the flood peak and the maximum flood volume for *i* ($i = 3, 7, 15, 30$) days are estimated, respectively.

Figures 3, 4 and 5 show the result of linear fitting for statistical parameters of flood peak over time. Specifically, Fig. 3 indicates that the mean is non-stationary with the changing pattern of the mean over time declining. However, C_V or C_S does not change over time with the

Table 4 Means of flood peak and flood volume for the Pearson-III distribution

Mean Year	Q_P (m^3/s)	W_{3d} (billion m^3)	W_{7d} (billion m^3)	W_{15d} (billion m^3)	W_{30d} (billion m^3)
2011	51,200	12.79	27.15	51.51	92.43
2012	51,170	12.78	27.13	51.46	92.34
2013	51,130	12.77	27.11	51.41	92.25
2014	51,100	12.77	27.09	51.37	92.16
2015	51,070	12.76	27.07	51.32	92.07
2016	51,030	12.75	27.05	51.27	91.98
2017	51,000	12.74	27.02	51.22	91.89
2018	50,970	12.73	27.00	51.17	91.80
2019	50,940	12.73	26.98	51.12	91.71
2020	50,900	12.72	26.96	51.08	91.62
2021	50,870	12.71	26.94	51.03	91.52
2022	50,840	12.7	26.92	50.98	91.43
2023	50,800	12.69	26.90	50.93	91.34
2024	50,770	12.69	26.88	50.88	91.25
2025	50,740	12.68	26.86	50.84	91.16
2026	50,700	12.67	26.84	50.79	91.07
2027	50,670	12.66	26.81	50.74	90.98
2028	50,640	12.65	26.79	50.69	90.89
2029	50,600	12.65	26.77	50.64	90.80
2030	50,570	12.64	26.75	50.59	90.71

correlation coefficient between C_V and time is 0.0002, while that of C_S is 0.0007. Their t test statistics are 0.1217 and 0.2620, respectively. In this case, the uncorrelated hypothesis is accepted since the threshold is 1.6606 with a significance level $\alpha = 0.95$.

The linear fitting may not the best form, but it can simply derive the changing tendency of statistical parameters over time, which provides a foundation for the research on the adaptive FLWL under a changing environment. In order to keep consistency with design floods, the mean for 2011 year is adjusted to the designed value which is determined under stationary conditions. Table 2 lists the estimated parameter values of flood peak for the Pearson-III distribution.

Similarly, statistical parameters of the maximum flood volume for i days ($i = 3, 7, 15, 30$) are estimated (in Fig. 6), and their results are shown in Table 3. Annual maximum data of the TGR are from 1882 to 2010, and functions of statistical parameters (in Tables 2, 3) are used to predict parameter values from 2011 to 2030. Finally, Table 4 shows the results of mean values of flood peak and maximum flood volume for i days ($i = 3, 7, 15, 30$).

Significant uncertainties in estimate of design floods arise for future projects, due to the limited data records, sampling variability and model errors (Obeysekera and Salas 2014; Lin et al. 2014b, c). This indicates that the uncertainties involved in data and methods cannot be avoided. For example, this study uses the P-III frequency curve and L-moment estimation for the frequency analysis, which leads to errors for the flood risk evaluation. However, this issue is out of our scope since the presented study aims at how to use the non-stationarity flood frequency analysis to design the reservoir FLWL.

Design flood hydrograph

Design frequencies of the TGR are 1, 0.1 and 0.01% (10% increment), and peak discharge values of four typical years (1954, 1981, 1982 and 1998.) are 66,100, 69,500, 59,000 and 61,700 m^3/s, respectively.

Figure 7 shows the design flood hydrograph of the TGR for 2011, where design parameter values are shown in Table 1.

Flood routing

The flood operating rules of the TGR are as follows: (1) When the reservoir inflow is less than a 100-year design flood, the reservoir release is no more than 53,900 m^3/s to ensure that the highest reservoir water level does not exceed 175.0 m and (2) when the reservoir water level exceeds 175.0 m, the reservoir release is equal to the minimum value between the full discharge capacity and the maximum inflow (Li et al. 2010). Combined with the reservoir water balance equation (Eq. 15a), flood routing of the TGR is acquired by the trial and error method.

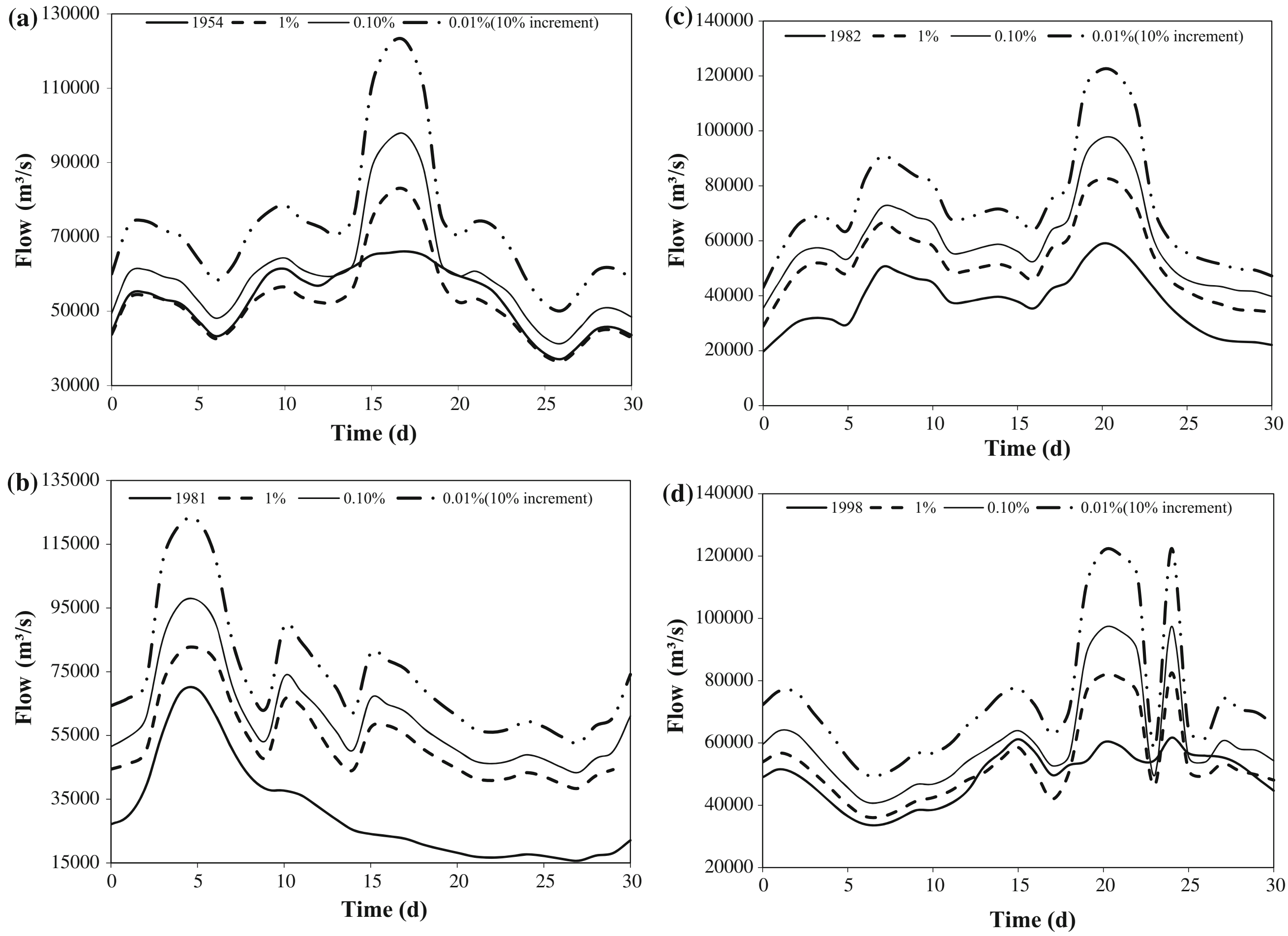

Fig. 7 Design flood hydrograph for 2011 [there are four lines, i.e., typical year (solid line), 1% (dashed line), 0.1% (thin solid line), 0.01% (10% increment) (double dot dash line) in all the cases]: (**a**) the 1954 year, (**b**) the 1981 year, (**c**) the 1982 year, (**d**) the 1998 year

Estimation of flood risk

On the non-stationary condition, it is assumed that mean values of inflow decrease linearly in the forms of functions in Tables 2 and 3, and the FLWL keeps at 145.0 m. When the mean of inflow varies through time, the exceedance probability would also vary over time from 2011 to 2030. The p_t/p_x in Table 5 shows the values of $p_1, p_2, \ldots, p_t$ and $p_i \leq p = 0.01$ $(i = 1, 2, \ldots)$, which is determined by flood routing. The probability distribution of the waiting time is derived from Eqs. (12a) or (12b), i.e., $P(X = x)$ in Table 5. In order to distinguish the t and x in Table 5, that $x = 3$ is an example, and $p_x = p_3 = 0.0990\%$. The exceedance probabilities for $t = 1, 2, 3$ are $p_1 = 0.1\%$, $p_2 = 0.0999\%$ and $p_3 = 0.0990\%$, respectively. Then, Eq. (12a) gives $f(3) = P(X = x) = (1 - p_1)(1 - p_2)p_3 = (1 - 0.1\%) \times (1 - 0.0999\%) \times (1 - 0.0990\%) = 0.0988\%$.

Theoretically, the flood risk probability R for non-stationary conditions could be derived by Eq. (13b). It can be expected that the flood risk probability would be less than that for stationary conditions due to the decrease in p_x values. For example, Table 5 shows that the flood risk probability under non-stationary conditions is 0.5884% when n is six, whereas the flood risk probability for the stationary condition is equal to $R = 1 - (1 - 0.1\%)^6 = 0.5985\%$.

On stationary conditions, the exceedance probability is equal to a constant $p_0 = 0.01$. The probability distribution of the waiting time is derived from Eq. (9), i.e., $P(X = x)$ in Table 6. The flood risk probability R_0 and the return period T_0 under stationary conditions could be derived by Eqs. (11) and (10), respectively, and the return period T_0 for stationary is 1000 years.

Figure 8 reflects the difference between the non-stationary flood risk probability R and the stationary flood risk probability R_0 with different design life. Clearly, the flood risk for stationary is greater than that of non-stationarity when the design life n of the hydraulic structure is fixed.

Table 5 Derivation of the return period T and flood risk R on non-stationary condition, in which the mean of inflow varies with time and FLWL keeps at 145.0 m

Year	Time t/x	p_t/p_x (%)	$P(X = x)$ (%)	$xP(X = x)$ (%)	Design life n years	Risk (%)
2011	1	0.1000	0.1000	0.1000	1	0.1000
2012	2	0.0999	0.0998	0.1995	2	0.1998
2013	3	0.0990	0.0988	0.2964	3	0.2986
2014	4	0.0979	0.0976	0.3904	4	0.3962
2015	5	0.0970	0.0966	0.4830	5	0.4928
2016	6	0.0960	0.0955	0.5731	6	0.5884
2017	7	0.0950	0.0944	0.6608	7	0.6828
2018	8	0.0940	0.0934	0.7472	8	0.7762
2019	9	0.0930	0.0923	0.8308	9	0.8684
2020	10	0.0921	0.0913	0.9128	10	0.9597
2021	11	0.0910	0.0901	0.9916	11	1.0499
2022	12	0.0901	0.0891	1.0696	12	1.1390
2023	13	0.0892	0.0882	1.1466	13	1.2272
2024	14	0.0884	0.0873	1.2220	14	1.3145
2025	15	0.0874	0.0863	1.2941	15	1.4008
2026	16	0.0865	0.0853	1.3652	16	1.4860
2027	17	0.0857	0.0844	1.4345	17	1.5705
2028	18	0.0848	0.0834	1.5020	18	1.6539
2029	19	0.0839	0.0825	1.5678	19	1.7365
2030	20	0.0830	0.0815	1.6309	20	1.8180

Table 6 Derivation of the return period T and flood risk R on stationary condition, in which the mean of inflow is equal to 51,200 m^3/s

Year	Time t/x	p_t/p_x	$P(X = x)$	Design life n years	Risk
2011	1	0.1	0.1000	1	0.1000
2012	2	0.1	0.0999	2	0.1999
2013	3	0.1	0.0998	3	0.2997
2014	4	0.1	0.0997	4	0.3994
2015	5	0.1	0.0996	5	0.4990
2016	6	0.1	0.0995	6	0.5985
2017	7	0.1	0.0994	7	0.6979
2018	8	0.1	0.0993	8	0.7972
2019	9	0.1	0.0992	9	0.8964
2020	10	0.1	0.0991	10	0.9955
2021	11	0.1	0.0990	11	1.0945
2022	12	0.1	0.0989	12	1.1934
2023	13	0.1	0.0988	13	1.2922
2024	14	0.1	0.0987	14	1.3909
2025	15	0.1	0.0986	15	1.4895
2026	16	0.1	0.0985	16	1.5881
2027	17	0.1	0.0984	17	1.6865
2028	18	0.1	0.0983	18	1.7848
2029	19	0.1	0.0982	19	1.8830
2030	20	0.1	0.0981	20	1.9811

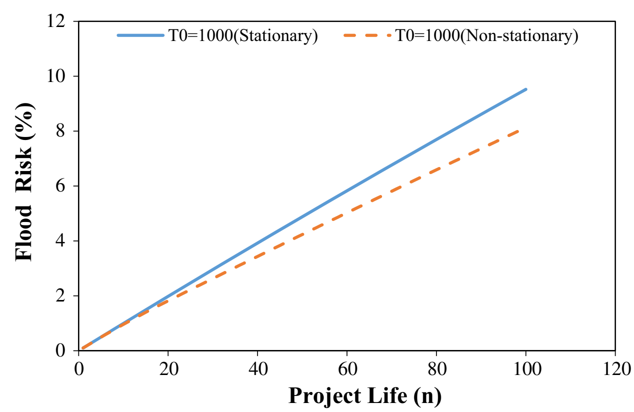

Fig. 8 Flood risk probability as a function of project life n [the solid line shows the flood risk for the stationary condition, while the dashed line shows the flood risk for the non-stationary condition in which the FLWL keeps at 145.0 m]

Table 7 Engineering design parameters of TGR

Parameter		Value
The normal storage level		175.0 m
Flood control limit level		145.0 m
Total capacity		39.3 billion m^3
The flood control capacity		22.15 billion m^3
The maximum water level	$P = 1\%$	166.9 m
	$P = 0.1\%$	175.0 m
	$P = 0.01\%$ (10% increment)	180.4 m

Table 8 Results of flood routing: (a) the typical year is 1954; (b) the typical year is 1981; (c) the typical year is 1982; (d) the typical year is 1998

P	Flood control limit level (m)	Maximum water level (m)	Maximum inflow (m^3/s)	Maximum release (m^3/s)	Reservoir storage (billion m^3)
(a)					
1%	144.9	160.02	82,500	53,900	9.12
	145.0	160.09	82,500	53,900	9.12
	145.1	160.16	82,500	53,900	9.12
0.10%	144.9	174.05	97,400	53,900	21.24
	145.0	174.10	97,400	53,900	21.24
	145.1	174.15	97,400	53,900	21.24
0.01%	144.9	178.13	122,400	109,840	25.45
(10%	145.0	178.07	122,400	109,692	25.34
increment)	145.1	178.10	122,400	109,751	25.32
(b)					
1%	144.9	162.36	82,500	53,900	10.84
	145.0	162.43	82,500	53,900	10.84
	145.1	162.49	82,500	53,900	10.84
0.10%	144.9	175.00	97,400	97,400	22.20
	145.0	175.00	97,400	97,400	22.15
	145.1	175.00	97,400	97,400	22.10
0.01%	144.9	175.07	122,400	101,881	22.27
(10%	145.0	175.03	122,400	101,773	22.18
increment)	145.1	175.08	122,400	101,900	22.18
(c)					
1%	144.9	161.45	82,500	53,900	10.15
	145.0	161.51	82,500	53,900	10.15
	145.1	161.58	82,500	53,900	10.15
0.10%	144.9	175.00	97,400	97,400	22.20
	145.0	175.00	97,400	97,400	22.16
	145.1	175.01	97,400	97,400	22.11
0.01%	144.9	175.01	122,400	101,717	22.21
(10%	145.0	175.06	122,400	101,844	22.21
increment)	145.1	175.04	122,400	101,810	22.15
(d)					
1%	144.9	161.45	82,500	53,900	10.15
	145.0	161.52	82,500	53,900	10.15
	145.1	161.58	82,500	53,900	10.15
0.10%	144.9	173.39	97,400	53,900	20.58
	145.0	173.44	97,400	53,900	20.58
	145.1	173.49	97,400	53,900	20.58
0.01%	144.9	175.02	122,400	101,754	22.22
(10%	145.0	175.01	122,400	101,736	22.17
increment)	145.1	175.01	122,400	101,716	22.11

Results of FLWL

The FLWL of the TGR is 145.0 m based on the observed annual maximum data from 1882 to 2010 under stationary conditions; however, the FLWL needs to be re-established to ensure flood safety when the reservoir inflow is altered. In this study, the FLWL is re-set with the assumption that the flood risk probability is equal to the original design value. Means of inflow from 2011 to 2030 are derived from the functions in Tables 2 and 3.

In order to illustrate the determination method about the FLWL of the TGR, 2011 is chosen for an example. Firstly,

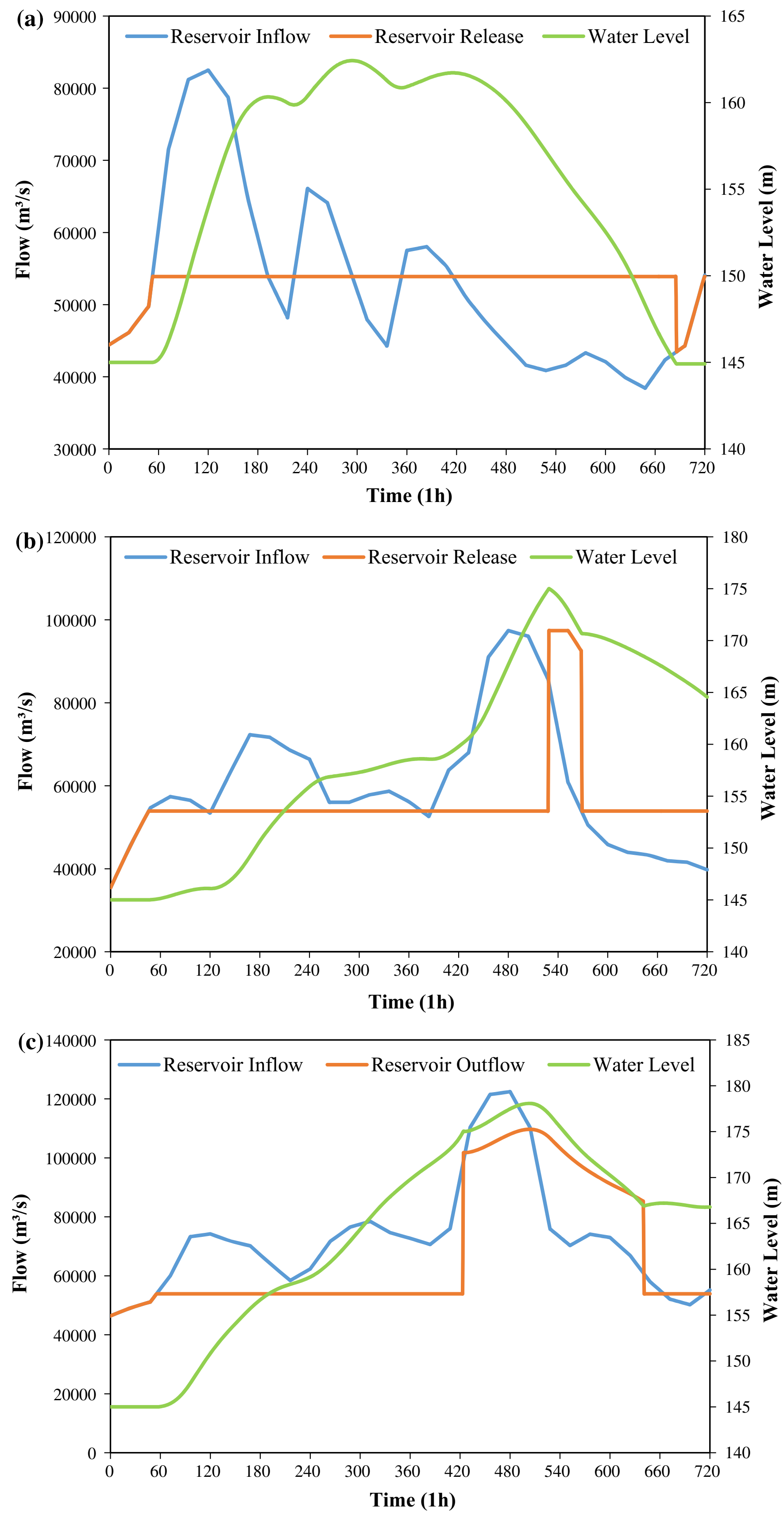

Fig. 9 Flood process line when the FLWL of TGR is 145.0 m [there are three lines, i.e., inflow hydrograph (blue solid line), release (orange solid line) and water level change (green solid line)]: **a** design frequency is 1%, and the typical year is 1981. **b** Design frequency is 0.1%, and the typical year is 1982. **c** Design frequency is 0.01% (10% increment), and the typical year is 1954

Table 9 Results of flood routing on non-stationary condition that the mean changes with time

Year	Flood control limit level (m)	Year	Flood control limit level (m)
2011	145.0	2021	147.5
2012	145.2	2022	147.8
2013	145.4	2023	148.0
2014	145.8	2024	148.3
2015	146.3	2025	148.4
2016	146.5	2026	148.7
2017	146.6	2027	149.0
2018	147.0	2028	149.2
2019	147.1	2029	149.5
2020	147.4	2030	149.7

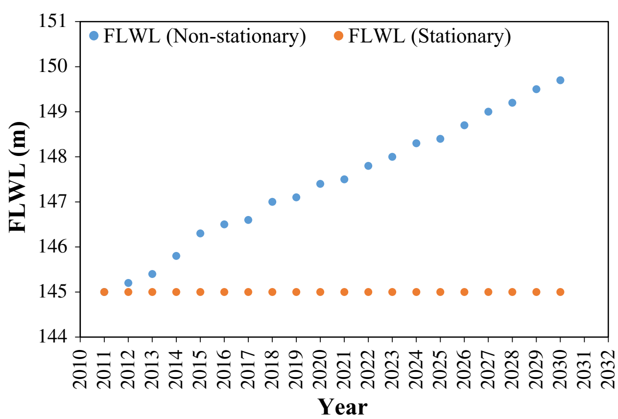

Fig. 10 Results of the flood routing [two lines, i.e., FLWL on the non-stationary condition (blue), FLWL on the stationary condition (orange)]

it is assumed that the design standard (such as flood risk, design flood peak value) of the TGR does not change, and Table 7 shows engineering design parameters of the TGR. Then, statistical parameters (such as mean) of inflow are estimated, and the reservoir FLWL is assumed to 144.9, 145.0 and 145.1 m, respectively, referring to the original FLWL scheme of the TGR. Finally, the result of flood routing is shown in Table 8, and only that the FLWL is 145.0 m can satisfy all requirements for the water level. Figure 9 shows the concrete process of the flood routing.

Table 9 shows the result of flood routing with the reservoir streamflow alters. When the mean value decreases from 2011 to 2030, the FLWL can be raised, which are shown in Fig. 10.

Conclusions

The FLWL is the most significant parameter for trade-off between activities of flood control and water conservation, and hence, it is necessary to re-establish FLWL to ensure flood safety when the reservoir inflow is altered. Based on results of the case study for the TGR, main conclusions are summarized as follows:

1. Moving average and *L*-moment estimation are used to fit the linear function of statistical parameters based on the observed data. Specifically, means of design flood peak, 3-day volume, 7-day volume, 15-day volume and 30-day volume yearly decrease by 33 m^3/s, 0.0080 billion m^3, 0.0211 billion m^3, 0.0482 billion m^3 and 0.0905 billion m^3, respectively. However, the amplitude of variation of C_V or C_S changing with time is insignificant.
2. When the streamflow decreases, the flood risk for non-stationary conditions is less than that for stationary conditions with the exceedance probability varies from each year. Thus, the traditional method for designing the flood risk may not be applicable for the projects under non-stationary condition. In this case study, the cumulative flood risk probability of the reservoir water level exceeding 175.0 m during 2011 to 2030 decreases from 1.98 to 1.82% when the non-stationary design flood is used.
3. The FLWL is sensitive to the design flood, and when streamflow changes, the FLWL needs to be re-established without increasing the flood risk probability. The FLWL of the TGR can increase 4.7 m by 2030.

Since only linear trend has been focused in the non-stationary environment, the design flood can be improved for nonlinear and jump formulation. The FLWL, which is re-established in each year, may not conform to the practical operation and needs to be further researched.

Acknowledgements This study was supported by the National Key Research and Development Program (2016YFC0400907), the Excellent Young Scientist Foundation of NSFC (51422907) and the National Natural Science Foundation of China (51579180). The authors would like to thank the editor and the anonymous reviewers for their comments that helped improve the quality of the paper.

References

Condon LE, Gangopadhyay S, Pruitt T (2014) Climate change and non-stationary flood risk for the upper Truckee River basin. Hydrol Earth Syst Sci 19:159–175

Deng C, Liu P, Guo SL et al (2015) Estimation of nonfluctuating reservoir inflow from water level observations using methods based on flow continuity. J Hydrol 529:1198–1210

Diao YF, Wang BD (2011) Scheme optimum selection for dynamic control of reservoir limited water level. Sci China Technol Sci 54(10):2605–2610

Douglas EM, Vogel RM, Kroll CN (2000) Trends in floods and low flows in the United States: impact of spatial correlation. J Hydrol 240(1–2):90–105

Fortin V, Perreault L, Salas JD (2004) Retrospective analysis and forecasting of streamflows using a shifting level models. J Hydrol 296(1–4):135–163

Greenwood JA, Landwehr JM, Matalas NC et al (1979) Probability weighted moments: definition and relation to parameters of several distributions expressible in inverse form. Water Resour Res 15(5):1049–1054

Gumbel EJ (1961) The return period of order statistics. Ann Inst Stat Math 12(3):249–256

Hosking JRM, Wallis JR (1997) Regional frequency analysis: an approach on L-moment. Cambridge University Press, London

Kharin VV, Zwiers FW (2005) Estimating extremes in transient climate change simulations. J Clim 18(8):1156–1173

Leadbetter MR (1983) Extremes and local dependence in stationary sequences. Theory Relat 65(2):291–306

Li X, Guo SL, Liu P et al (2010) Dynamic control of flood limited water level for reservoir operation by considering inflow uncertainty. J Hydrol 391:124–132

Lin KR, Lian YQ, Chen XH et al (2014a) Changes in runoff and eco-flow in the Dongjiang River of the Pearl River Basin, China. Front Earth Sci 8(4):547–557

Lin KR, Lian YQ, He YH (2014b) Effect of baseflow separation on uncertainty of hydrological modeling in the Xinanjiang Model. Math Probl Eng 8:1–9

Lin KR, Liu P, He YH, Guo SL (2014c) Multi-site evaluation to reduce parameter uncertainty in a conceptual hydrological modeling within the GLUE framework. J Hydroinform 16(1):60–73

Lins HF, Slack JR (1999) Streamflow trends in the United States. Geophys Res Lett 26(2):227–230

Liu P, Guo SL, Xiong LH et al (2006) Deriving reservoir refill operating rules by using the proposed DPNS model. Water Resour Manag 20(3):337–357

Liu P, Guo SL, Li W et al (2008) Optimal design of seasonal flood control water levels for the Three Gorges Reservoir. IAHS-AISH publication, Wallingford

Liu P, Cai XM, Guo SL (2011) Deriving multiple near-optimal solutions to deterministic reservoir operation problems. Water Resour Res 47(8):W08506

Liu P, Li LP, Guo SL et al (2015) Optimal design of seasonal flood limited water levels and its application for the Three Gorges Reservoir. J Hydrol 527:1045–1053

McCabe GJ, Wolock DM (2002) A step increase in streamflow in the conterminous United States. Geophys Res Lett 29(24):38.1–38.4

Milly PCD, Betancourt J, Falkenmark M et al (2008) Stationarity is dead: whither water management. Science 319:573–574

MWR (Ministry of Water Resources) (2006) Regulations for calculating design flood of water resources and hydropower projects. Water Resources and Hydropower Press, Beijing **(in Chinese)**

Obeysekera J, Salas JD (2014) Quantifying the uncertainty of design floods under nonstationary conditions. J Hydrol Eng 19(7):1438–1446

Obeysekera J, Irizarry M, Park J et al (2011) Climate change and its implications for water resources management in south Florida. Stock Environ Res Risk Assess 25(4):495–516

Olsen JR, Stedinger JR, Matalas NC et al (1999) Climate variability and flood frequency estimation for the Upper Mississippi and Lower Missouri rivers. J Am Water Resour Assoc 35(6):1509–1523

Ouyang S, Zhou JZ, Li CL et al (2015) Optimal design for flood limit water level of cascade reservoirs. Water Resour Manag 29(2):445–457

Potter KW (1976) Evidence for nonstationarity as a physical explanation of the Hurst phenomenon. Water Resour Res 12(5):1047–1052

Rigby RA, Stasinopoulos DM (2005) Generalized additive models for location, scale and shape. J R Stat Soc Ser C (Appl Stat) 54(3):507–554

Salas JD, Boes DC (1980) Shifting level modeling of hydrologic series. Adv Water Resour 3(2):59–63

Salas JD, Obeysekera J (2014) Revisiting the concepts of return period and risk for nonstationary hydrologic extreme events. J Hydrol Eng 19(3):554–568

Strupczewski WG, Kaczmarek Z (2001) Non-stationary approach to at-site flood frequency modeling II. Weighted least squares estimation. J Hydrol 248(1):143–151

Strupczewski WG, Singh VP, Feluch W (2001a) Non-stationary approach to at-site flood frequency modeling I. Maximum likelihood estimation. J Hydrol 248(1):123–142

Strupczewski WG, Singh VP, Mitosek HT (2001b) Non-stationary approach to at-site flood frequency modeling III. Flood frequency analysis of Polish rivers. J Hydrol 248(1):152–167

Villarini G, Serinaldi F (2012) Development of statistical models for at-site probabilistic seasonal rainfall forecast. Int J Climatol 32(14):2197–2212

Villarini G, Serinaldi F, Smith JA et al (2009) On the stationarity of annual flood peaks in the continental United States during the 20th century. Water Resour Res 45(8):W08417

Wang XJ, Zhao RH, Hao YW (2011) Flood control operations based on the theory of variable fuzzy sets. Water Resour Manag 25(3):777–792

Xie P, Chen GC, Lei HF et al (2009) Surface water resources evaluation methods on changing environment. Sci Press, Beijing **(in Chinese)**

Xiong LH, Jiang C, Du T (2014) statistical attribution analysis of the nonstationarity of the annual runoff series of the Weihe River. Water Sci Technol 70(5):939–946

Yun R, Singh VP (2008) Multiple duration limited water level and dynamic limited water level for flood control with implication on water supply. J Hydrol 354(1–4):160–170

Zhang JW, Liu P, Wang H et al (2015) A Bayesian model averaging method for the derivation of reservoir operating rules. J Hydrol 528:267–285

Zhao TTG, Zhao JS (2014) Improved multiple-objective dynamic programming model for reservoir operation optimization. J Hydroinform 16(5):1142–1157

Zhou YL, Guo SL, Liu P (2014) Joint operation and dynamic control of flood limiting water levels for mixed cascade reservoir systems. J Hydrol 519:248–257