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Design Flood - An Overview of Indian Practice

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SUMMARY

A Committee of Engineers, appointed by Government of India in 1957, identified rational estimation of Design Flood as critically important for bridge foundation design. The report of the Committee initiated a landmark national Study. Regional synthetic unit hydrograph and design input storm parameters have been defined on the basis of this study, for the entire country divided into 26 hydrometeorologically homogeneous regions. The method applies to catchments of about 5000 km^2 area. For larger projects, flood frequency analysis is generally adopted when adequate gauging data is available. Flood with a return period of 100 years is now unambiguously defined as Design Flood. The evolution of current design practice is discussed with particular reference to road bridges.

1 INTRODUCTION

Indian River System is large and so also is the number of crossings by roads and railways. Rivers unlimited carry an annual flow of about 1700 billion m^3 [1]. They all swell in summer with high monsoon precipitation in their catchments. In Himalayan rivers, snowmelt may add to it. In the alluvial plains, many overflow the banks and flood the land around, often ravaging it.

Flood is the most familiar and frequent natural disaster in India. It afflicts one river basin or the other almost every year, with varying fury. Protection of vital communication links of roads and railways during floods aids disaster mitigation in a big way. This lends added socio-economic significance to designing bridge foundations safe against the severest floods.

Flood is an extreme natural event with many faces. The principal characteristics, usually represented in flood hydrograph, are i) Peak Discharge ii)Water Level iii)Volume iv) Flood Duration. Peak Discharge alone may be the prime concern in bridge hydrology. It will be deemed here as synonymous with flood. The current design practice of estimating design flood is reviewed here with particular reference to road bridges.

2 CODE SPECIFICATION AND PAST PRACTICE

The first Section of the national Bridge Code, IRC-5:1985 (referred as Code), lays down specifications relevant to design flood, currently valid for all road bridges[2]. However, codes tend to be static. Design practice, often, progresses much beyond the bounds of codes. For Indian bridges also, current code specification only defines past practice. Demerits of Code specifications, which eventually made them invalid for current use, are discussed below.

2.1 Historic Flood

The design flood is simply defined in Code as the maximum observed flood or historic flood, for a mandatory minimum period of record of 50 years[3]. This earliest method of selecting design flood has grown rather dated. Its demerits are well known. The probable frequency of the selected flood remained unknown. The design flood at the same site could increase as period of record increased. Insufficiency of flood records, more as a rule than exception, was, of course, its biggest flaw. Accepting it in its stride, Code offers a long list of alternative methods, obviously intended to find an equivalent. Two of them discussed below are of prime interest. These methods used in combination essentially defined past design practice, in conformity with Code.

2.2 Area Velocity Method[2]

This is really an extension of the method of Historic Flood. Instead of records for historic flood, the maximum water level reached in historic flood is sought to be estimated on the evidence of local witnesses. These may include flood marks on banks and structures close to project site or even fading memories of how high the highest flood rose on the ancient tree or building. The variability of bed profile and flood slope from those measured before/after flood is ignored. Computation of stream velocity relies on subjective selection of an empirical coefficient. The return period of the design flood is left to uncertainty.

2.3 Empirical Formula

A large family of empirical formulae for quick and ready estimation of design flood was developed in India. These have spilled from the past century into the present. The first one that made its debut in 1885 is Dickens formula[3]. It also happened to be the one most frequently used in bridge design, until recently. It read as-$Q = C * A^{0.75}$ where Q is design flood (ft^3/s), A catchment area ($mile^2$) and C a constant.

The formula was surely developed for small catchments, with limited data available and for a small region. These obvious limitations have been largely ignored. Its validity has been extended from regional into near national without many qualms. Although meant for small catchments, it has been used for catchments exceeding a few hundred thousand km^2

Invalid extrapolation used the simple expedient of varying constant C as wide as 200 to 2000. The critical choice of C was left to the subjective judgment of designer, who had little clues to go by except personal preference. The uncertainty around the frequency of the design flood resulting from the formula remained as the common malady.

2.4 Multiple Methods in Combination

Code preferred to rely on multiple methods to improve reliability, which could be elusive. It all boiled down to computing values twice over (or more), once by area velocity method and then again by Dickens formula (and/or equivalent). The values were compared and the largest only qualified for selection as the design value. The inherent fallacy should be obvious. If both methods were unreliable, comparison and combination may, in all probability, compound the errors. Overestimation by Dickens formula could easily negate the efforts of a more rational hydrologic analysis. The same fallacy recurred when design discharge adopted for bridges in vicinity was called in for comparison. If the reference values themselves were estimated by unreliable methods, any comparison could have little relevance to a rational estimation.

3 PRESENT PRACTICE AND THE CHANGE IN APPROACH

'Period of Empiricism' no longer rules the scene[4]. As disenchantment with it grew, search for rational methods for practical use began as far back as the fifties. Dickens formula is now invalid even for minor bridges. A simple rational substitute method of regional analysis was put to practice in 1973[5]. Further development through two decades has followed and brought in its wake a complete change in approach.

3.1 Report of Committee of Engineers and Follow-up

The report of a high powered Committee of Engineers on bridges appointed by Government of India (referred as Committee) was published in 1959[6]. It identified design flood and its rational evaluation as critically important. Following its recommendations, 'sustained and systematic collection of hydro-meterological data' was undertaken for the entire country on a short and long term plan[6].

The short term plan was completed in 1973. A regional Synthetic Unitgraph (SUG) method was evolved for estimation of design floods of bridges with catchments of 25 to 5000 km^2[5]. The long term plan has continued since with joint efforts of hydrologists, meteorologists and bridge engineers of roads and railways. The national cooperative study (referred as Study) was a landmark event in flood hydrology of Indian bridges. A brief description of the Study and the method of estimation of design flood evolved follows.

3.2 Basic Approach for the Study

The approach has to be tailored to availability of data and project size. Large investment intensive bridge projects should, of course, go in for detailed hydrologic analysis supported by project specific hydro-meterological investigation, if needed. For many large projects, gauging stations with adequate period of record may be available at

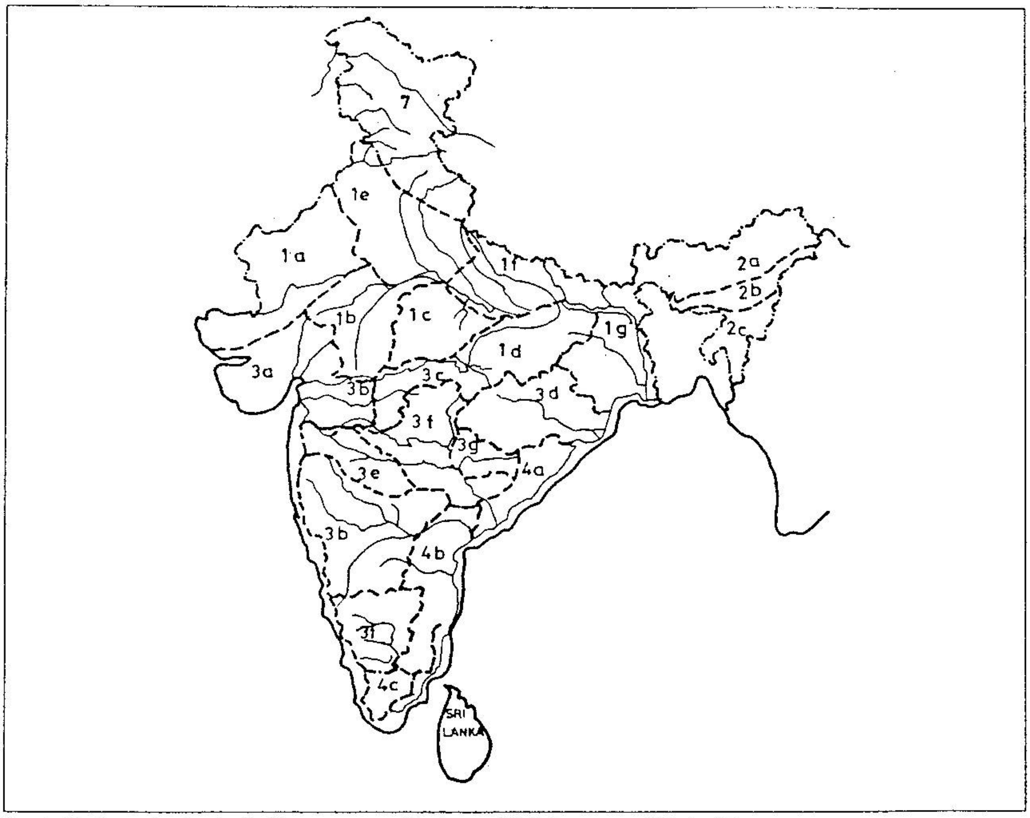


Fig. 1 Hydrometerologically Homogeneous Regions – 26 Subzones and Major Rivers

site or in vicinity. Flood frequency analysis could be feasible and preferred. An example has been discussed later.

The thrust area identified for the Study, therefore, related to bridges with catchments upto 5000 km ^{2}. These claimed the lion's share of total national investment in bridges. Most of their catchments were ungauged. Project specific investigation was not feasible and Regional Analysis was the obvious option open.

Two candidate approaches considered for regional flood estimation were i) flood frequency ii) hydro-meterological. The latter was adopted for better availability of data and in conformity with the recommendations of the Committee.

Regional flood estimation studies were taken up for hydro-meterologically homogeneous regions. For this purpose, the country was divided into 26 such regions (called subzones; principal zones number 7) as shown in Fig 1. The salient features of these subzones vary widely in drainage basin area, topography, rainfall, land use, etc. Results were reported separate for each subzone.

The Study has been jointly undertaken by four apex bodies of Government of India - Central Water Commission (CWC), Research and Standards Organisation (RDSO) of Ministry of Railways, India Meteorological Department (IMD) of Ministry of Science and Technology and Ministry of Surface Transport (MOST).

3.3 Study Methodology [7]

3.3.1 Flood Flow Data (RDSO/MOST)

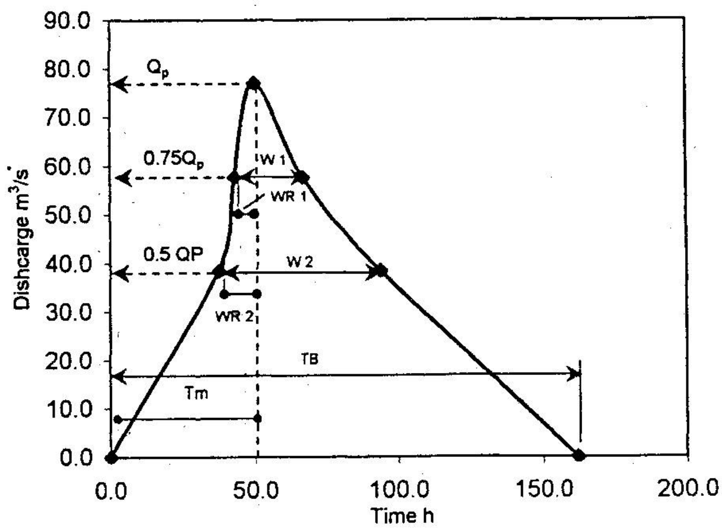
Rainfall and flood flow data were collected at selected representative railway bridge catchments (RDSO) numbering about 10 to 30 for each subzone. Period of observations in phases varied from 5 to 10 years beginning from 1965. These were supplemented by observations at total number of 45 road bridge sites (MOST), beginning 1979.

3.3.2 Storm Analysis (IMD)

Long term rainfall data for object subzone for a large number of raingauge stations, both ordinary and self recording, were collected by IMD from its National Data Centre. These were combined with rainfall data mentioned in 3.3.1. IMD made rainfall depth-duration-frequency analysis of data for each subzone and furnished the following components of design storm- i) Isopluvial maps of 24 h point rainfall of 25,50 and 100 year return period (T) ii) Ratios of Short Duration to 24 h Rainfall iii) Time Distribution Curves of Storms of various duration iv) Ratios of Areal to Point Rainfall. Hourly design storm rainfall increments could be redily estimated with the aid of maps, tables and charts given by IMD.

3.3.3 Hydrologic Analysis (CWC)

CWC collated concurrent rainfall and flood data furnished for gauged catchments in a subzone. After due scrutiny and finalisation of gauge and discharge rating, several storm/flood events were selected for study. One hour unit hydrographs (UG) were derived by usual methods. A few characteristics of UG curve were identified and measured for the several UG curves in view. These were correlated to physiographic characteristics of catchment by regression analysis. A simple relation of the form $y = k*x^n$ where k, n are constants, did suffice to define the SUG for ungauged catchments in a subzone. Fig 2 shows a typical SUG developed including the constants defining it.



River	Karmanasha	
Subzone	Sone 1(d)	
Catchment Area	$A = 2000$ km^2	
Length of Stream	$L = 180$ km	
Equivalent Slope	$S = 1.5$ m/km	
t_p	$0.314(L/S^{0.5})^{1.012}$	$= 50$
q_p	$1.664/(t_p)^{0.965}$	$= 0.0385$
W_2	$2.534/(q_p)^{0.976}$	$= 57$
W_1	$1.478/(q_p)^{0.86}$	$= 24$
W_{2R}	$1.091/(q_p)^{0.75}$	$= 13$
W_{1R}	$0.672/(q_p)^{0.719}$	$= 7$
TB	$5.526(tp)^{0.866}$	$= 162$
Tm	$t_p+t_r/2$	$= 50$
Q_p	$q_p \times A$	$= 77$
Design Storm Duration $1.1*t_p$		$=54$
Design LossRate		$=0.25$ cm/h
Base Flow		$=0.045*A$

Fig. 2 Synthetic Unit Hydrograph – An example

Average constant infiltration loss(ϕ-index) rate per hour, base flow per km^2 to be used in design estimation and design storm duration were also derived by CWC on a general basis, from analysis of a number of flood events.

3.4 Study Report and Method of Flood Estimation(CWC) [7]

Results of the study for each subzone have been reviewed by a co-ordination committee and published successively by CWC separate for each subzone. The reports lay down a method of estimation of design flood with return period of T = 25,50,100 years by SUG. It is based on the basic assumption that design storm of T years return period causes a flood of T years return period. No significant interception is presumed.

Deign flood with desired return period is computed in three simple steps- i) draw the SUG curve; tabulate its hourly ordinates ii)estimate the hourly rainfall increments (deduct losses) iii) Compute direct runoff; add base flow. Method of estimation of flood of T = 25, 50, 100 years is lucidly set out with tables, charts, maps and worked out examples to aid easy and unfettered use. Hydrographs can also be prepared.

The utility of these reports extends much beyond its prescription of a rational method of flood estimation for minor bridges. Each self complete subzone report contains detailed documentation of data collected, methods of analysis and results along with some general topographic, climatic, meteorological data. 21 separate reports, covering all but 2 subzones and 91% of the country, have been published by CWC to date[8]. A large national hydrometeorologic database has been compiled and deserves to be extended in future.

3.5 Design Flood Defined- Anomalies Abandoned

3.5.1 Anomalies in Definition

Estimation of Design Flood can only as good as Design Flood is defined. Anomalies in definition may undo all the rationale of evaluation. Some did creep into Committee recommendations quoted below [6].

"Committee felt that design discharge should be maximum flood on record for a period of not less than 50 years. Where adequate records are available extending over not much less than 50 years, design flood should be 50 year flood determined from probability curve on the basis of recorded floods during the period."

Committee thus defined Design Flood as Maximum Observed Flood (definition 1)and NOT as Flood with Return Period T = 50 Year or any other T fixed a priori (definition 2). Code definition is identical and the lack of logic has been discussed in 2.1[2].

Option of Committee for definition 1 is unambiguous. Definitions do not alter as a function of period of flood record. It would be highly anomalous to presume that definition 1 could be substituted by definition 2 if period of record just fell short of the threshold value of 50 years. Nor could flood frequency analysis be invalid for 50 years' record. When this context is ignored, anomalies arise. These are best illustrated by Table 1.

Table 1 Maximum Observed Floods and Return Periods
Case Study for Yamuna at Tajewala- Annual Flood Peak Series 1913-78.

Maxm Observed Historic Floods			**Floods of T year Return Period (Probability Curve)**	
Year	Flood m^3/s	T Year	T Year	Flood m^3/s
1924 Sept	25110	105	50	20320
1947 Sept	18390	35	100	25020
1955 Oct	13234	13	200	30240
1978 Sept	26410	130	1000	47550

Taking the annual peak series>50years (1913-78)into account, a flood with T as high as 130 years should be selected as design flood. Given a hypothetically truncated series of 50 years- (1925-75), a flood with T as low as 35 years would be selected as design flood as defined by Committee. The real dilemma, more commonplace, should arise when a hypothetically truncated series over 1938-78 (40 years) is considered. Maximum observed flood with T=130 years occurs in this series and selecting anything lower as design flood would amount to a gross violation of the basic definition. At best, the intent of Committee could be interpreted (for insufficient flood record) to find a probable flood in 50 years with probability of exceedance left anomalously undefined.

3.5.2 Foundation Design Flood [6], [2]

Committee also recommended two kinds of design floods for bridges-1) foundation design flood 2) deck design flood. The former claiming a higher safety level is obtained by incrementing design flood by a Factor of Safety varying (from 1.1 to 1.3) inversely as the catchment area. The latter with a lower safety demand is assigned factor of safety of 1. The length of deck or waterway should be determined by design flood. A higher value only applies to foundations with the higher safety levels.

Safety factors may not have much relevance in bridge hydrology. Desire for higher safety and higher safety factors can increase non-linearly following a flood event. There are instances of Factor of safety for foundation design flood rising as high as 1.5 for large projects.

Distinction between safety levels for waterway(deck) and foundation is quite impracticable. Scour around foundation is a function of Q/W, where Q is the design discharge, W length of waterway. So a lower waterway enhances the risk of foundation failure.

3.5.3 Rational Definition

Rational methods of estimation of design flood for road bridges in current practice could not but abandon the anomalies discussed above. No distinction is made between foundation design flood and other design flood. Nor is 50 year return period flood incremented by a factor of safety (1.3 or more) considered relevant.

Rational definition of design flood in terms of T year return period fixed a priori is only adopted in present practice. It applies uniformly to all bridges minor, medium and major alike. The probability of exceedance of a design flood with a given return period during design service life is shown in Fig. 3[10]. The design service life of road bridges in India can be notionally defined as 50 years. Fixing a return period of 50 years for design flood would yield a probability of exceedance of 65% which may be deemed too high. Higher return periods of 100 and 1000 year will reduce the risk to 39% and 5% respectively.

The optimal choice of a return period of 100 years defines present practice. The Study report includes estimation of floods with T = 100 years. The same return period applies to inflow design flood of small dams according to IS Guidelines[11]. It is interesting to recall that the very first version of Bridge Code draft dating back to 1946 opted for a 100 year design flood [12].

So the present definition of design flood as one with T=100 year abandons all anomalies of past practice. Choice of

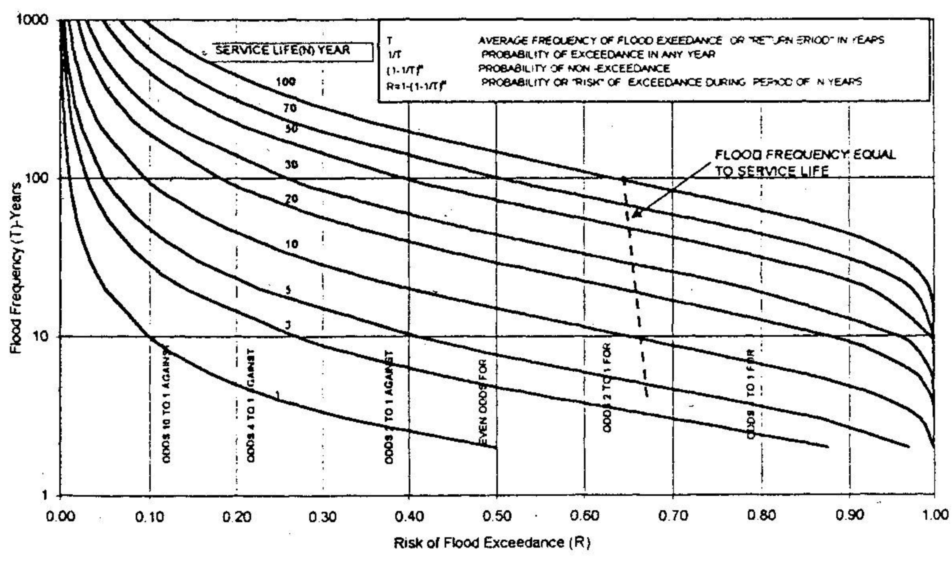


Fig. 3 Flood Frequency and Risk of Exceedance

Design Flood is a risk based decision aimed at socio-economic optimisation utility of a structure. India can hardly afford the luxury of designing bridges for 1000 year flood.

3.6 Regional Flood Frequency Analysis

Regional Flood Frequency Analysis has not been developed in the Study reports. Inadequacy of flood flow data is the obvious reason. However, the Regional Flood Frequency Model was developed with limited data for one Subzone(Sone)[13]. Data of 11 catchments spread over the subzone with areas varying from 30 to 500 km ^{2} were used. Annual flood peak series of 11 to 25 years was available. Gumbel EV-1 distribution was used. Values of floods (Q_T) for various return periods T = 2.33(mean annual flood Q_m),25(Q_{25}), 50(Q_{50}) ,100(Q_{100}) were obtained by fitting a straight line through plotted positions. The following ratios of Q_T/Q_m have been derived - Q_{25}/Q_m=2.83, Q_{50}/Q_m=3.38, Q_{100}/Q_m=3.82. The regional formula for mean annual flood is related to physiograpic characteristics of catchment derived by least squares method read as, $Q_m=2.33*A^{0.795}*S^{0.567}*F^{0.520}$, where Q_m is in m^3/s ,A catchment area in km^2, S equivalent storm slope in m/km, F form factorA/L^2 , L being basin length.

3.6.1 Mean Annual Flood

Results yielded by the method were acceptable and proved the potentials of future use. Incidentally, estimation of Q_m is key step in flood frequency analysis and it is equally so in foundation design. Accidental load combination like earthquake or barge impact often determines foundation design [14]. Code specifications lay down that Mean Annual Flood. and NOT Design Flood should be combined with earthquake. As such estimation of the former gets equal importance in current rational practice. Arbitrary coefficients promoted by cursory code prescriptions are worth ignoring[14]. Flood level corresponding to Q_m is also to be evaluated, as depth of scour is to be measured from it only.

3.7 Flood Frequency Analysis for Large Catchments

Large bridges need project specific investigation and analysis for rational flood estimation. Application of the regional analysis is limited to about 5000 km^2. Choice of methods is left to designer. This, of course, precludes any return to empiricism.

Single unit hydrograph cannot be applied to large catchments. The total drainage area has to be divided into a number of subbasins .Separate flood hydrographs may be derived for each sub-basin from analysis of different storms. These hydrographs are routed down river to site. Appropriate flood routing methods are used[1]. Calibration of flood hydrographs and flood routing parameters is essential.

However, a flood frequency analysis is the preferred method in practice. With the large network of gauge and discharge stations of CWC in major river basins, it is feasible to find one not far from the site. An example of a simple application of rational flood estimation procedure is given below [15].

The catchment area was as large as 368302 km^2. In large drainage basins($A>10000$ km^2) floods in tributary basins occur at different times at random. Combination of these make the flood event in the main river. As the number of tributaries increases, frequency distribution curve should tend to the normal distribution[16]. Flood gauge data were available at CWC station close to site for 1971-95. A rating curve was developed for determining discharges corresponding to the gauze records. The annual peak series was then analysed using normal distribution curve to yield the following flood values in m^3/s- Q_m=25000,Q_{50}=35800, Q_{100}=37.200. The corresponding flood levels(m) were given as 83.750, 87.540, 88.000 m. It was interesting to note that maximum observed flood (1978) was identical to Q_{100}. Dickens formula yielded a discharge 42155 m^3/s which exceeds 1000 year flood.

4. CONCLUSION

The method of estimation of design flood has been weaned away from the past practice of empiricism and irrational definition of design flood. Regional synthetic unitgraph (SUG) method is used instead. Parameters have been derived for the entire country divided into 26 regions. Design Flood is simply defined by a Return Period of 100 years. Maximum observed flood or Factors of safety for foundation design flood are no longer relevant. For large bridges and catchments regional analysis is precluded. Flood frequency analysis is usually adopted using nearest gauging station records.

There is little room for ambiguity in the present practice of flood estimation, given above. Yet, the boundaries between past practice and present may not be as clearly delimited as presumed in the paper. Traditions die hard and so do defunct Code prescriptions. These aberrations are better ignored without much ado.

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