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"DAMAGES CAUSED ON KANGRA VALLEY ROAD BY FLOODS OF AUGUST, 1944 AND THEIR RESTORATION"

BY

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CHAPTER I

Kangra Valley :—Its severe rainfall and damages caused by floods of August 1944

The Kangra valley road starts from Chakki near Pathankot, and runs at the foot of Dhaula Dhar range of the Outer Himalayas. The road has a mean elevation of 3,000 ft. above sea level. The reduced levels of important places on this road, and their distances from Pathankot are given belows :—

Name of place.	Distance from Pathankot.	Height above sea level.
Pathankot ...	0 ...	1,093 feet
Chakki	7 ...	1,318 ,,
Nurpur	16 ...	1,833 ,,
Kotla ...	29 ...	1,715 ,,
Shahpur ...	40 ...	2,472 ,,
Gaggal ...	48 ...	2,453 ,,
Kangra ...	52 ...	2,494 ,,
Nagrota ...	58 ...	2,897 ,,
Palampur ...	72 ...	4,070 ,,
Baijnath ...	82 ...	3,267 ,,
Jogindernagar ...	96 ...	3,998 ,,
Urla ...	108 ...	4,745 ,,
Mandi ...	131 ...	2,419 ,,

The Dhaula Dhar range has an elevation of more than 15,000 ft. It runs more or less parallel to the road, especially from Shahpur to Palampur, and at several places is not more than ten miles (in a horizontal direction) from the road. This snow-clad range with its steep slopes, dominates the whole valley, and makes it look picturesque and beautiful. The steep slopes however cause severe damages to the lands in the valley, by hill slides, flow of debris, and meandering of torrential streams after heavy rainfall.

On account of high altitude of the range, the monsoon clouds get intercepted, causing very heavy rainfall in the valley. The rainfall in this area is heaviest in Northern India. At Dharmsala and Palampur the average rainfall is about 120 inches per annum.

The average rainfall during months of July and August at Dharmsala and Palampur is approximately 40 inches. The rainfall is more severe in the month of August, and a maximum of 70 inches has been recorded at McLeod Gunj (Upper Dharmsala) in August 1943. The rainfall at McLeod Gunj during the year 1943 reached a figure of 172 inches.

* "The rainfall varies remarkably as one moves away from the range. Some of the European planters around Palampur used to have lines of rain gauges every quarter of a mile away from Dhaula Dhar range, and these could record an average annual diminution of about one inch between successive gauges." Kangra which is at a distance of about eight miles from Dharmsala (as crow flies) has an average of 80 inches rainfall during the year.

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* Gazetteer Kangra District.

Not only the total rainfall, but the intensity of rainfall is very severe. A precipitation of 2 inches per hour is not an abnormal occurrence. On 2nd September, 1945, a rainfall of 7·4 inches was recorded at McLeod Gunj in a period of $2\frac{3}{4}$ hours from 1-15 p.m. to 4 p.m. in the presence of the Executive Engineer, Kangra Provincial Division. The rainfall was heavy but not exceptional. This gave an intensity of $2\frac{1}{2}$ inches per hour. The adjoining nalas did not flow their maximum, and practically no damage occurred anywhere. Generally the cloud bursts are local, and therefore do not cause much damage. They cause moderate to severe floods in hill torrents which die very soon. When however the rain is widespread and its intensity is very severe, practically all the streams get in spate. The force and momentum of flood water results in severe damage.

During the night between 4th and 5th of August 1944, there occurred in the Kangra Valley a cloud burst of unprecedented ferocity. It is estimated that about 24 inches of rain fell in the vicinity of Palampur. The record of rainfall as obtained from the Civil Authorities is given below :—

Nurpur	...	0·68 inches.
Lower Dharmsala	...	10·82 inches.*
Upper Dharmsala	...	7·25 inches.*
Palampur (Civil)	...	12·4 inches.
Palampur Agricultural Farm	...	20.4 inches.**
Alhilal Camp	...	19·4 inches.

The rainfall in the upper reaches of Dhaula Dhar range must have been heavier still, as floods which caused colossal damages cannot be accounted for otherwise. The people who witnessed this deluge state, that rainfall in the form of a blinding wall of water, fell from the stricken sky, and gushed down like a tidal wave in the hill torrents. The force of water was so great that massive bridges of concrete, stone masonry, steel girders, went down with a crash, creating a large number of breaches along the road.

A statement showing list of breaches along with their location, and details of spans of bridges and culverts over these is given in Appendix I.

Some of the important bridges which were damaged or washed away, along with nature and cause of damage are described below :—

Manjhi Bridge in mile 115 of A. P. K. Road. This bridge is built immediately below the junction of two branches, *i.e.* (*a*) and (*b*) of Manjhi stream as shown in plate I, while the third branch, *i.e.* (*c*) of this stream is crossed by a bridge having 3 spans of 22 feet. Normally a proportionate quantity of water flowed through each bridge. During this flood, a considerably large quantity of water was diverted to the

* The record of rainfall at Lower Dharmsala and Palampur seems to be wrong. At both places the rain gauges overflowed, and no reliable person actually measured the rainfall.

** The Officer-in-Charge states that the total quantity of rainfall could not be recorded as the rain-gauge was overflowing at the time of recording.

big Manjhi bridge on account of heavy flow in stream (*b*) while a comparatively smaller quantity flowed through the smaller Manjhi. The original plans of Manjhi bridge which was built in 1895 show that the stream had a tendency to flow towards the right bank. The bridge had been built a little towards the left, while the opening on the right had been blocked by a long approach road protected on both sides by dry stone retaining walls. A protection bund was also built on the right bank which does not seem to have been properly maintained.

The discharge in the nala was so great that the bridge formed a bottle neck. The flood reached the top of the bridge, which acted like a weir for sometime. Ultimately the wing walls on the right bank, and approach road with dry stone retaining walls were washed away creating a gap of about 80 ft. The bridge stood intact. After the flood it was noticed that heavy silting had occurred on the left bank with heavy scour near the right bank.

Manuni bridge in mile 117/1 of A. P. K. Road. The bridge consisted of a single arch span of 33 ft. Water topped the road surface and bridge collapsed. The foundation of right abutment which were only about 4 feet below bed level had been scoured and the abutment masonry had toppled over. The foundation of left abutment however was not scoured. It seems that a larger quantity of water from the big Manuni Nala flowing close by was diverted into this Nala and had caused damage.

The bridge over big Manuni Nala consisting of 2 spans of 22 ft. and one span of 28 ft. was not damaged. This bridge was only at a distance of about 200 ft. from the small Manuni bridge which was washed away.

Bridges between Nagrota and Palampur. These bridges were of small spans, the more important being Ran Khud 19 ft., Soon Khud (2 spans of 30 ft.) and Bhirl Khud 20 ft. span. They were all of Masonry arch type construction, and were built more than fifty years ago. The foundation of these bridges were not more than four feet below bed level. The embankment of the Kangra Valley Railway track runs down stream of the road, and very near to it. Water headed up due to the embankment, as the water way of the railway bridges was small. When finally the water found its way either by damaging the railway bridges or flowing over the embankment, it took away the P.W.D. bridges, by sudden force that was created due to abrupt release of water. The abutments and arch masonry toppled over, leaving few traces of masonry blocks. Either one or both the foundations had been scoured. During excavation of the foundation some of stone masonry blocks were found below the level of foundation of old bridges. This showed that either the old bridges had not been properly founded, or the beds of the nalas had been lowered, during half a century of the existence of these bridges.

Awa Khud bridge in mile 142/4 of A. P. K. Road.—This bridge is about 4 miles away from Palampur where the intensity of rainfall was maximum. It consisted of one span 58 ft. and one span 18 ft. with a total water way of 76.

The flood came with great force, and topped the bridge. The right abutment and pier were completely washed away. The steel girders of the superstructure were severely damaged, and carried away about a mile and a half downstream. Only a very small quantity of steel was recovered from the bed after the floods were over.

Beyond Awa Khud, only minor bridges mostly in Mandi State were damaged. These bridges had inadequate water-way, and were of a workmanship which was not of high order.

Ban Ganga bridge in mile 87/1 of J. H. D. Road. This bridge had one span of 80 ft. The height of roadway over the bed of nala was 55 ft. (see Plate II-a). The bridge was built in 1890, with both abutments founded on solid rock. The approach on right bank was founded on the same rock, and had a height of about 38 ft. It was protected on both sides by the retaining walls of dry stone masonry. The old bridge formed a bottle neck, but water-way seemed ample for normal floods. During this flood the water headed up and reached the top of the bridge. The bridge was not damaged but wing walls and approaches were washed away completely creating a gap of about 180 ft. (see photograph No. I).

Apart from damage to bridges, hill slides occurred at several places. At Bahmanu Nala in mile 168/5 near Jogindernagar, a huge mass separated from the parent hill, blocked the nala and created a lake about 30 ft. deep. The road was also blocked for a length of about 300 ft. The hill-side was composed of treacherous clay, mixed with small shaley stones, which came down with every shower of rain. The viscous material was very difficult to remove by manual labour, and was a constant trouble throughout the rainy season.

At one or two places, *i.e.*, Kotwali Bazar, Dharmsala in mile 100/7 J.H.D. Road, the whole road, along with breast and retaining walls slid about 50 ft. down the Khud and is standing there intact.

In Kulu Valley serious damages occurred at Bajaura and Mohal Khuds, where bridges in permanent streams were completely washed away. The R. S. Beams at Bajaura were severely twisted. The flood was so great that it washed away the stables attached to Bajaura Rest-House, and also threatened the main Rest-House Building. Similarly at Mohal Khud, the bridges over permanent streams were washed away. Both streams have a great width, but water generally flows in two or three well defined channels.

Restoration of Traffic. There were about 50 breaches throughout the length of the road, on which temporary bridges were constructed, and traffic was restored completely within a short period of one month and a quarter. H. E. the Governor of the Punjab, who visited this area in October 1944, was able to keep to his programme and travel up to Manali without any hindrance.

CHAPTER II

Physical Characteristics of Streams, Formulae Connecting Runoff with Catchment Area. Waterway for Bridges, etc.

Characteristics of Streams. The hill streams in Kangra Valley have steep slopes which cause high velocities. The data regarding slopes, velocities approximate discharges of various streams is given in Appendix II. While Chakki has a slope of 1 in 195, Awa Khud, Manjhi, etc., have a slope of about 1 in 32. The velocities worked out are 12.8 feet per second and 22 feet per second.

The nalas in Mandi State have very steep slopes, *i.e.*, ranging from 1 in 5 in smaller streams to about 1 in 15 in streams like Googly. The quantity of debris which moves along, in these beds is quite considerable, and often chokes the bridges. The bridges may have adequate waterway for floods, but are sometimes insufficient to pass enormous quantities of debris that move in the bed. During severe flood, big boulders about 8 to 10 feet in diameter, roll along in the stream and cause heavy devastation.

Streams in Kulu Valley. At certain places, the hills in Kulu Valley are made up of soft and friable strata which starts flowing when heavy rainfall occurs. This causes slips which block the road. The quantity of debris moved is out of all proportion to the discharge in the nala. For instance, on 29th June, 1945, a nala in Mile 16/1 of Kulu-Manali road having a width of about 25 feet brought about 150,000 cubic feet of sand, silt, and stones, in a period of about of two hours. The debris collected about 20 feet high on the road, and was spread in a length of about 250 feet. The rainfall during 24 hours was 2.4 inches, and the catchment area of the nala is half a square mile.

Similarly debris was brought in other nalas between Kulu and Manali. The quantity was however, not so great as in this. From enquiries made at site, the debris moved in layers of 4 or 5 feet and at a speed of about 3 to 4 miles an hour.

Waterway. For the design of permanent bridges over the streams, the determination of adequate waterway was of vital importance. The discharge of the stream was determined very carefully by taking cross sections of the nala both upstream and downstream, and measuring bed slope. The velocity was calculated by Manning's formula

$$V = \frac{(1.486)}{n} \times R^{\frac{2}{3}} \times S^{\frac{1}{2}}$$

Where R is mean hydraulic depth

and S is bed slope *i.e.*, $\frac{\text{fall}}{\text{distance}}$

(See page 41 of M. E. S. Hand Book Volume III)

The values of n which is co-efficient of rugosity, are given on page 39. The value of n for streams in Kangra Valley = 0.05. With a bed slope of 1 in 30, the velocity worked out to be 20 to 25 feet per second.

From the cross sections, the area of waterway at H. F. L. was determined, and it was proposed that waterway at bridge site should be more than this, and provide at least two feet clearance. The width of stream at bridge site has not been narrowed much, in order to avoid heavy afflux.

Calculation for velocity, discharge waterway ,etc., have been given for Manjhi and Awa Khud bridges in Appendix III for reference.

Discharge from catchment area. Attempts were made to determine discharge in the streams from formulae connecting runoff, and catchment area of the basin drained.

The most important formulae followed in India are :—

$Q = CM^{\frac{3}{4}}$ Col. Dickens' formula.

Where C averages 825.

$Q = CM^{\frac{2}{3}}$ Ryve's formulae

Thus formula is applicable near Madras. The maximum value of C = 675.

M. E. S. formula for N. W. F. P.

$$Q = 1{,}200\ M^{\frac{3}{4}}$$

Where M is less than $9\frac{1}{2}$ square miles.

For M $9\frac{1}{2}$ to 12,000 square miles.

$$Q = 2100\ M^{\frac{1}{2}}$$

Hyderabad State formula

$$Q = C.M.^{(0.92 - \frac{1}{14}\log\ M)}$$

Value of C = 1,700 for hill catchments.

These formulae are based on data for comparatively big catchment areas, where the probability of rainfall over the whole area is reduced. Moreover the intensity of rainfall is not as severe as in this area. The discharges worked out by these formulae, were low compared with discharges work out from data available at site of bridges.

A large number of bridge sites from Chakki to Baijnath have been inspected very carefully. The flood data of August 1944, and of floods in the past when these bridges were either overtopped or damaged has been collected from records. The values of discharge worked out from velocity and area of waterway have been correlated, with the area of catchment.

The results from various bridge sites differ considerably as shown in Appendix II. They however lie in an area of which the lower limit is given by formula

$$Q = 2{,}000\ M^{\frac{3}{4}}$$

and higher limit $Q = 3{,}000\ M^{\frac{3}{4}}$

Most of the points lie near the higher limit, while a few points lie on the lower limit. (See Graph Plate III). These points relate to bridge sites, where the stream has an easier slope, signifying that the stream has come out sufficiently away from Dhaula Dhar range. At some bridge sites, *i.e.*, Banganga, Manjhi, Awa Khud and Bhirl which were either damaged or entirely washed away, the value of discharge is given by the formula

$$Q = 3{,}500\ M^{\frac{3}{4}}$$

These sites are immediately at the foot of Dhaula Dhar range, where heavy intensity of rainfall occurred. The length of catchment basins is not much, and a very slight proportion of run-off must have been lost due to evaporation and absorption.

If compared with formulae already in vogue, these give extravagant results. This is as it should be, because the area is subject to severe intensity of rainfall. A rainfall of one inch per hour in one square mile produces a discharge of 645 cubic feet per second. As pointed out in Chapter I, two to three inches per hour is not an abnormal occurrence. When rainfall of heavy intensity continues for some hours in an area which is already saturated with monsoon rains, the losses due to sub-soil flow, absorption and evaporation are reduced considerably. The total quantity of rainfall is practically converted into surface flow which gives the run-off of the catchment.

The total rainfall on 4th and 5th August, 1944, was about 24 inches, and it is expected that a rainfall of about 5 inches per hour must have been recorded during the peak period. This gives a discharge of about 3,000 cusecs per square mile allowing about 10 per cent losses due to absorption, etc. The intensity of rainfall of 5 inches per hour may seem very high, but if compared to rainfall in San* Gabriel Area Los Angeles, America, where intensity of one inch per minute for one minute has been recorded, this figure will appear reasonable. The floods normally do not occur until rainfall has continued for three or four hours, and is widespread.

For Kangra Valley $= 3{,}000\ M^{\frac{3}{4}}$ will give reliable results, and is therefore safe to adopt for these catchments.

Twenty per cent increase may however be allowed for streams which just debouch from the range into the valley, and have severe bed slope *i.e.* about 1 in 30. This formula is not applicable in Kulu Valley where rainfall is less severe.

Waterway per square mile of catchment area

Lacey's formula

$$\text{Wetted Perimeter} = (2.67\ Q^{\frac{1}{2}})$$

or determination of waterway and length of bridge is inapplicable. The wetted perimeter worked out by the above formula gave figures much in

*Indian Concrete Journal, September, 1944, pages 170-171.

excess of bed width of the stream. The formula is however applicable for Chakki Stream, which is a Major stream having a maximum discharge of about 2,30,000 cusecs.

The waterway per square mile of catchment area for existing bridges, and new bridges varies from a minimum of 50 square feet to a maximum of 150 square feet.

Length of bridging per mile of road. The average length of bridges per mile is 115 feet from Chakki to Nagrota including Chakki bridge and 76 feet from Nagrota to Mandi. The average length of bridges per mile of Kangra Valley railway is 113 feet.

Rain Gauges. The rain gauges in this area are under the control of officers of the Civil Department, who are not interested in the intensity of rainfall. The observations are taken to determine the yield of crops, therefore they are interested in total rainfall during the season. The observations made at Dharmsala on 5th August and Palampur, etc., show that the junior officials, *i.e.*, Moharrirs, etc., who are incharge of these, are not very careful about these measurements. They are not properly trained and do not attach much importance to accuracy in these matters.

The Engineers are more concerned about the intensity of rainfall, as this produces floods, etc. It is therefore suggested that automatic rain gauges should be installed at different places in the area, and P. W. D. should look after them. The subordinates who are properly trained can record the rainfall, and higher officers can sift the data supplied by subordinates. Any inaccuracy can be checked when sifting and correlating the data from different stations. The initial cost and maintenance of these rain gauges will be more than met from savings on account of economic design of structures based on reliable and scientific data.

If this work is considered too much for P. W. D. it is suggested that Meterological Department may be asked to open more stations during the post-war period. The data will be equally useful to Hydro-electric, Irrigation, and Railway Departments.

Preservation of catchments. It appears that heavy grazing by sheep and goat and heavy fellings of timber, either for war purposes or indiscriminate felling by the local inhabitants to earn high profits, has caused considerable denudation in hills in Kangra Valley. Lesser quantity of rainfall gets absorbed, and therefore run-off from the catchments is considerably increased. In our opinion the floods of August, 1944, which caused so much damage were also a result of denudation of the hills.

If such floods are not to recur, the Government must close certain areas, and start afforestoration in all earnestness. The closing of areas and planting of trees must be started immediately by the Forest Department on a proper scientific basis.

CHAPTER III

Details of design and construction of Bridges

The bridges have been designed for Indian Road Congress Heavy Loading (See page 7 of I. R. C. Bridge Specifications 1937 Edition)

The clear roadway on all these bridges has been kept as 20 feet to allow two lanes of traffic. Ban Ganga bridge which is on an umetalled Road has a clear roadway of 13′ 6″.

For Major bridges, *i.e.*, 40 feet span and over reinforced concrete bridges would have been cheaper, as stone bajri sand and water are available in the bed of nalas. Timber for staging and shuttering is easily available in Kangra Valley and can be had at cheap rates. Due to war, steel for reinforcement was not easily available. Construction of R. C. C. Bridges would have delayed the work by one year. As it was imperative to have the bridges rebuilt before the monsoons of 1945 old girders from Railway bridges had to be used for important crossings. These girders are probably overstrong and heavy. The P. W. D had to pay high price for these. The weights of the steel girders without decking are given below:—

Name of bridge.	Weight of steel per span excluding weight of slab.
Manuni Bridge 20 ft. span plate girder	8 tons
Manjhi, Manuni and Naira birdges 40 ft. span plate girders.	16 tons
Goodly bridge 40 ft. span, plate girder	20 tons
Awa Khud bridge 120 ft. soan, Trussed girder ...	144 tons
Ban Ganga Bridge 150 ft. span 13′ - 6″ road way Trussed girder	155 tons

The steel bridges were fabricated and erected at site by the Bridge Department of N. W. R. These bridges have been provided with reinforced cement concrete slab to carry the roadway.

Arch bridges. For spans less than 40 feet arch bridges, with 1 : 3 : 6 cement concrete arch rings have been built. These were preferred to R. C. Slab and Tee Beam bridges as steel for reinforcement was not available. The arch bridges were more easy and rapid in construction than Tee Beam bridges, which involved the use of difficult form work and staging. The local masons and mistries under proper supervision were quite efficient in the construction of staging for the arches.

Details of work. The details of construction and design of all bridges starting from foundation upwards are given below:—

Foundations. The design of foundations of bridges depends upon total dead and live load including earth pressure, bearing pressure of soil, and depth up to which scour takes place. The total load coming on foundations can be calculated exactly, and bearing pressure can be assumed by examining the nature of bed, as well as from data available from older bridges.

The depth of scour is however difficult to determine, exactly by theoretical methods. It is equally difficult to determine by practical methods, as no observations can be taken during floods. Immediately after the floods, the scour holes get filled up and no reliable data can be collected.

The depth of non-silting and non-scouring flow can be determined by formula of Lacey and Kennedy.

According to Lacey

$$R=0.9\left(\frac{q^2}{f}\right)^{\frac{1}{3}}$$

Where R = mean hydraulic radius, or depth of non-scouring flow below H.L.F.

q = the average intensity of flood per foot run-off water way $=\frac{Q}{w}$

f = silt factor depending upon size of particled in the bed. For shingle, boulder bed f = 6.0 approximately.

Kennedy's formula (see M. E. S. Hand Book Volume III, 1935 edition pages 38 to 48)

$$D=\frac{(V_o)}{(m)}^{1.56}$$

where D = Depth of non-scouring and non-silting flow,

V_o = critical velocity which causes neither silting non-scouring.

m = Scour coefficient for the material for which bed is composed. For gravel and boulders varies from 8.0.to 3.5

Lacey suggests that for worst conditions, depth of foundation to be 2 R below H. F. L. this was impracticable as the depth of foundations, worked out was excessive. The foundation of bridges have been taken 3 or 4 feet blow R.

M. E. S. Hand-Book suggests that foundation should be taken to a depth of $1\frac{1}{3}$ D to allow for factor of safety. This gave reasonable results.

Calculations for waterway and of depth of foundation for typical bridges are given in Appendix III.

The depth of foundation as worked out by the formula seem to be ample, as shown by the records of previous bridges.

Practically all the bridges in Kangra Valley road were built between 1890-1900. During the earthquake of 1905 most of the office record, and plans were destroyed. It is not possible to know the depth of foundations of all the bridges which have stood the onslaught of floods for last 50 years or so. The plans of some of the bridges which exist, show that design level of the foundation was 10 to 14 feet below bed level. Plans of bridges constructed on Kangra Valley Railway, show that foundations were taken 10 to 16 feet below bed level. The foundations of piers were deeper then those of abutments. Generally a depth of 12 feet was considered ample.

When framing estimate for new bridges a design depth of 10 feet was provided for major bridges below bed level. The cross section of most of nalas reveals (See Cross Section of Awa Khud Plate IV) that there is a big different of levels in the bed of nala at right angles to the direction of flow. A deeper channel runs in the bed and carries low water. The bed at this place may be about 6 feet deeper than at other places. If the level of the lowest bed is a criterion, the depth of foundations taken in bridges built previously was about 6 to 10 feet below the lowest bed levels. The same practice was followed for new bridges.

During excavation, it was found that nalas were full of gravel and boulders. Some of the boulders were 4 to 6 feet in diameter and being water worn were very hard. As the foundations were taken deeper, the quantity of bajri was reduced, the boulders were wedged in each other and formed a compact mass. Going deeper meant either chiselling or blasting under water. At these places the depth of foundation was reduced. This depth was however much deeper than the foundation of the old bridges. At certain places during excavation fallen masonry of old bridges was met with which showed the depth of scour holes. Foundations were taken at least 4 feet below this level.

Excavation of Foundations. The foundations were dug at a slope of 1/2 horizontal to 1 vertical even under water. At certain places especially in lower 2 or 3 feet the foundations were taken vertically. No timbering was necessary to retain the sides of foundation. The sides consisting of boulders with interstices filled with bajri, were like a conglomerate rock, which could stand vertical. As the quantity of sand in the foundation was small, and foundations were not taken very deep the sand blowing effect was negligible, and did not give much trouble.

The most difficult work was digging of foundations in boulders bed, where jumper work blasting and chiselling had to be done. The average progress of digging foundations in upper strata was about 6″ to one foot depending upon the nature of soil. In the neighbourhood of designed level it was difficult to go more than 3″ to 4″ a day. Drilling of holes in hard stones was tedious and lengthy affair. It would take two men about two or three hours before a hole was completed for laying charge of blasting powder. A mechanical drill worked by an air-compressor could have accelerated the work.

Water. The quantity of water coming in the foundation pit varied in each nala. The quantity of water in nalas between Manjhi Bridge and Awa Khud was much more than bridges in Mandi State, except in Googly Khud bridge where quantity of water was quite as much as in bridges near Palampur. The quantity was however not excessive, as was anticipated before the work was started. Ordinarily one pumping set with 4″ diameter suction and delivery could dewater the foundations, 3 or 4 feet below spring level. At lower depths two pumping sets either both 4 inches, or one four inches and second two inches were required. In certain pits on account of shortage of pumps, water was taken out by means of tins. A gang of about 20-30 men working from early morning till midday could empty the pit, when excavation would start.

Pumping sets. Light and portable sets were very useful as they could be shifted easily from one place to another. The sets used were from 2″ suction and delivery, to 4″ suction and delivery, and were run by 4 H. P. to 8 H. P. prime movers.

The details of types of pumping sets, name of manufacturers, horse power, speed, cost of running, etc., are given in Appendix IV.

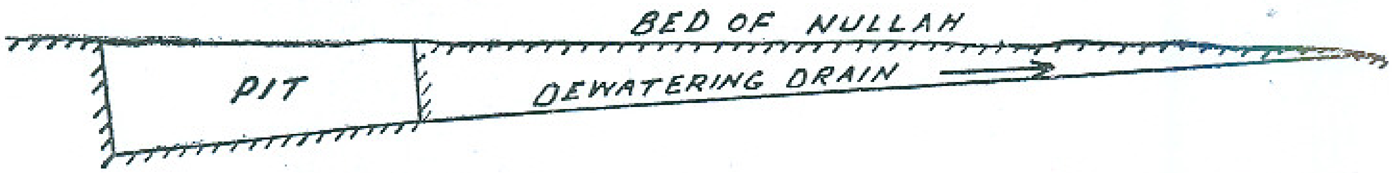
The appendix also gives analysis of cost of working of pumps per day.

Dewatering drain. Work on about 10 bridges had to be started simultaneously. Foundation of all these bridges were below water level. It was estimated that at least two pumping sets will be required in each foundation pit. During war, it was impossible to arrange for such a large number of pumping sets.

A very practical method of dewatering the foundation pit by means of a drain was suggested by Mr. L. A. Freak, Chief Engineer, Punjab, P. W. D., B. & R. Branch.

The bed of nalas had very steep slopes, ranging from 1 in 15 in Googly bridge, to 1 in 35 at Awa Khud bridge.

Fig: 1 a



If a drain is dug with a slope say 1 in 200, simultaneously with the excavation of the foundation pit, it will drain the water of the foundation pit. The length of drain will depend upon the depth of pit, slope of bed and slope of drain.

In actual practice it was found very difficult to dig this drain up to the level bottom of the pit. The quantity of excavation of drain was much more than the quantity of excavation of the pit. The bed consisted of shingle and boulders, and therefore digging of drain was a slow and laborious process. A big boulder in the drain anywhere in its long length would block it, and stop all work in the foundation pit until it was cleared. A via media was adopted. The drain was dug to a depth of about 4 feet and for balance quantity of work, *i.e.*, in last 5 or 6 feet pumping sets were used. By this method lesser number of pumps were employed and work also accelerated.

Foundation concrete. 1 : 5 : 12 cement concrete was specified for foundation of bridges. Where too much water was met with, a richer mix of 1 : 4 : 8 cement concrete was used. The foundation pits were dewatered by pumping sets before laying concrete.

Prominent and conspicuous springs were closed by gunny bags filled partly with cement sand mortar.

After dewatering the foundation, pumping was stopped. Concrete was laid in one foot thick layer. After about 15 to 20 minutes, water came up to the top of this concrete, when concreting was stopped. When initial set of concrete had taken place, pumping was again started. During pumping some of the cement mortar was washed away. Any spots where concrete was hungry, were finished again by mixing hungry concrete with additional cement sand mortar. Concreting was again started and similar operation continued until the first layer was concreted. A hole two to three square feet in area was left, in which foot valve of the pump was placed for suction of water. After the first layer had been completed the inflow of water was greatly reduced, and there was no difficulty in laying second or third layer. The hole for foot valve was plugged when concreting had been completed. The ideal method of laying concrete was by means of a Tremie pipe, but as there was no tools and plant for working it, and shifting it from position to position, ordinary method by laying it with hands was adopted.

As far as possible partially set concrete, *i.e.*, concrete which had been mixed and kept at the mixing platform for 10 to 15 minutes was used, so that mortar should not get easily separated by water.

Where too much water is met with in foundation, it is suggested that Ist layer of concrete may be slightly more rich, *i.e.*, 1 : 3 : 6 to counteract too much dilution of cement in water.

Abutments for girder bridges. These have been designed to carry dead and live load including impact, and horizontal pressure of filling behind the abutment. It is doubtful if the effect of impact can be felt in the masonry of abutment and it should be excluded. The load due to impact is however very small compared to the total dead load, and its effect is negligible. This load being vertical slightly helps to reduce the eccentricity due to the earth pressure.

Pressure of filling. Stone has been used for filling behind the abutments. It is assumed that angle of repose for this type of filling is 45°

$$P = wh\frac{(1-\text{sine}\,\varnothing)}{(1+\text{sine}\,\varnothing)} = 17 \text{ lbs.}$$

Where W=100 lbs, as stones boulder have been used for filling, and they will have certain percentage of voids.

This horizontal pressure may be considered low, but as practice in the past in this Division has been to work on this figure it has been adopted for design. This figure however seems quite safe. As the wings are built at right angles to the abutment, the space occupied by filling is very small. The abutment and wings are like a rectangular silo with three sides. The pressure of filling is therefore very much reduced. There is room for economy in the size of abutments if these pressures could be determined accurately.

Water Pressure. When the stream is running full, water must exert a horizontal pressure on the abutments. This pressure is counteracted by the mass of abutment, and pressure of filling behind the abutment. Some authorities suggest that pressure of filling behind should be neglected, as it may shrink. The effect of water pressure has been calculated in the design of abutment of girder bridges and it is found that resultant lies within the middle third, even if we neglect horizontal pressure of filling.

Abutment for arch bridges. In addition to forces and pressures given above for girder bridges, they have to withstand the pressure due to arch ring. The resultant of arch thrust, earth pressure due to back filling, and vertical dead and live load must pass through the middle third. If we neglect the pressure due to back filling the abutment becomes unsafe.

Pressure on abutment due to water flowing in the nala has also been neglected. This pressure is counteracted by the passive earth pressure. The passive earth pressure is given by the formula

$$P = wh\,\frac{(1+\sin\varnothing)}{(1-\sin\varnothing)}$$

The scrutiny of plan reveals that thickness of abutments at springing level can be expressed by the formula

$$T = (.16\,S + 2 \text{ feet})$$

where T = thickness and S is the span of the arch

The abutments have a back batter of 1 in 3 in most cases. In one or two cases, back batter of 1 in 4 has been found sufficient.

Piers. The piers have been designed for dead and live load including impact. Unequal loads from adjacent spans have been considered. The resultant of all forces passes through the middle third, and pier is safe for all conditions of the loads.

A batter of 1 in 24 has been provided on both sides to increase the thickness uniformly.

Pressure of flowing water. The pier is subject to a horizontal pressure due to flowing water. The pressure is given by the formula

$$P = KAV^2$$

Where P = Total pressure in lbs due to water current.

A = Area in square feet of vertical projections of the exposed part.

V = Velocity of current in feet per second

K = A constant depending upon shape of pier

For circular pier $K = 0.62$

(See I. R. C. Bridge Specification page 12)

As the pressure acts in the direction of the length of pier which is about 27 feet, its effect is negligible.

Shape of piers. The construction of piers in a channel disturbs the harmonious and uniform flow of the stream. Eddies are formed which create scour, and endanger the structure. On page 245 of Fowler's Engineering and Building Foundations are described experiments conducted by Crecy on shape of piers in a hydraulic laboratory. Piers with different shapes of starlings or noses, *i.e.*, triangular, elliptical, semi-circular, formed by two arcs of a circle, etc., were tried.

He found that the model which produced the least amount of eddies was elliptical, of which the small diameter was one-fourth the length (see Figure I).

Fig: II

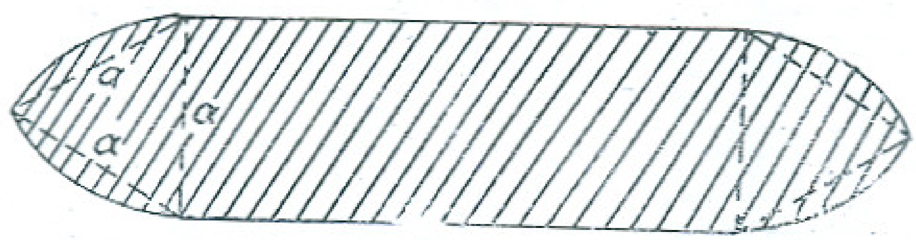
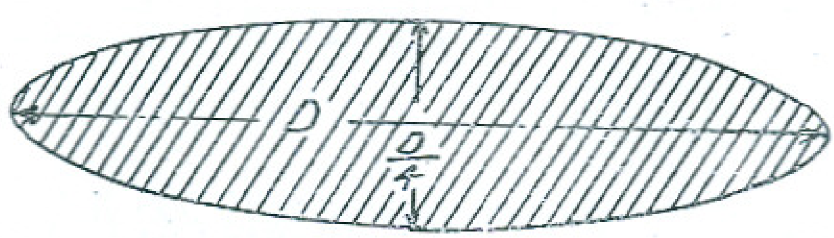


Fig: I

ELLIPTICAL SHAPE



This shape offers the least resistance, and occasions the least contraction. As it is not possible to make piers of this shape, the next best shape consists of a pier with sterlings formed by two circular arcs, tangent to the sides, and described on the sides of an equilateral triangle (see Figure 2).

This shape has been adopted on the downstream side of the pier. On the upstream side, semi-circular shape has been adopted. As huge boulders move with great force during flood, piers with sharp noses are likely to be damaged by impact of boulders.

Crecy has also proved by experiment that abutments having wings which are concave in plan (see Figure 3) create severe eddies, and should not be built on any account.

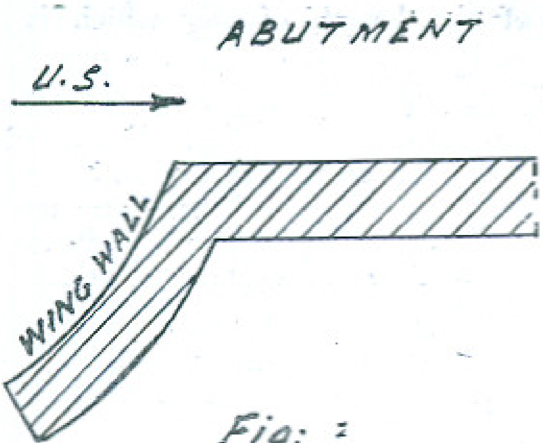


Fig: 3

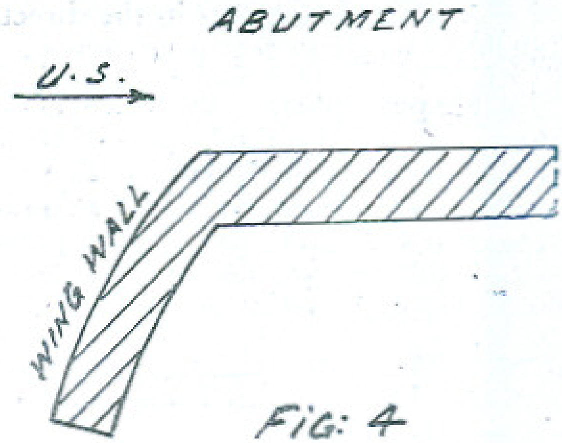


FIG: 4

The most satisfactory shape of abutment with wings is shown in Figure 4. These cause more or less streamline flow.

It is suggested that some experiments should be carried out in Hydraulic lahoratory of Irrigation Branch to determine the ideal shape of junction between abutment and wings.

Specifications. The piers and abutments have been made of C. R. masonry 1 : 5 with a hearting of cement concrete 1 : 5 : 12.

Plums have also been provided. On the outside face an average of 1½ feet thick masonry has been built, which acted as shuttering for hearting concrete. Bond stones have been provided in the masonry to bond the concrete and masonry. These are very necessary as concrete has a tendency to shrink. It is necessary that large bond stones should be used in corners, which are likely to break off due to impact of boulders, etc.

In the Googly Khud mile 162/1 the direction of flow keeps on changing, and at some period the flow may not be parallel to piers. In addition huge boulders 8 or 10 feet in diameter move in the stream at time of flood. The piers have therefore 1 : 3 : 6 cement concrete hearting which has been suitably reinforced.

Weep holes. Weep holes have been left in abutments and wing walls to allow drainage. They have been built through the hearting concrete as well as stone masonry.

Wing Walls. The foundations of wing walls have not been taken to the same depth as abutments. These are built near the banks of streams, where scours is not likely to be heavy. They have been built at right angles to the abutment, in order to reduce their length, and save filling behind abutment which is difficult to get in hills. The specifications followed in the construction of abutments and piers, have been adopted for wing walls.

Arch Bridges. For design of arches the I. R. C. Heavy loading was converted into an equivalent uniform distributed load including impact in lbs per square foot of decking area. As the arches are made of plain concrete, a cushion of earth one foot six inches has been allowed over the arch ring. This will reduce impact, and also distribute the knife edge load over a greater area. The dispersion of the concentrated load is very necessary, as it otherwise causes heavy stresses over the crown of the arch.

The arch rings have been designed for full dead and live load, and also live load on half the span. The latter produces greater eccentricity, and worst results near the springing of the arch. The arch rings have been designed, so that resultant passes through the middle third, and there is no tension at any section. Trials were made to reduce the thickness of arch ring, so as to reduce dead load.

It was proposed to use richer concrete, *i.e.*, 1 : 2 : 4 and have thinner arch rings. The analysis by elastic theory, and stress diagrams showed that tension occurred if sections were reduced.

If light reinforcement were used, the thickness of arch ring for 30 feet span would be 12 inches at crown and 18 inches at springing as against $2\frac{1}{2}$ feet thickness adopted. This reduces the dead load due to arch by more than half. The cost of the arch ring would have been practically equal to arch ring with 1 : 3 : 6 concrete, and as steel was difficult to get plain concrete arches were cast. These being more heavy, disperse the live load much better.

Thickness of Arch. Theoretically the arch should vary in thickness from crown to the springing. Thickness at springing should be 1.5 time to two times the thickness at crown depending upon the design of the arch. The secondary stresses in the arch caused by temperature, shrinkage of concrete and rib shortenings due to dead load, are directly proportional to the thickness of the arch at crown. In order to minimise these stresses, and have a more uniform stress distribution of load in the arch ring, the section at crown is always reduced. As the span of arches was small, and there would have been difficulty of shuttering, etc., in having a varying thickness, uniform thickness has been adopted.

The analysis of arch rings of spans 5 feet to 30 feet constructed in Kangra Valley shows that the thickness of arch ring can be expressed by the formula

$$T = .43\sqrt{S}$$

Where T = thickness of arch ring and S is span in feet.

Type of arch. Generally 120° arches have been constructed. Where waterway was inadequate 90° arches have been made. Both these types are most economical, when we consider the combined cost of abutment and arch ring.

The arches have been made of 1 : 3 : 6 cement concrete with a facing of either dressed stone or pre-cast concrete. The facing which was laid a day or two earlier, has acted as shuttering for concrete. As stresses in the arch ring are low, *i.e.*, about 5 tons per square foot leaner concrete could have been used but leaner concrete probably would not have been so uniform, or given such satisfactory results.

Centring. For centering of arches stone masonry pillars, were constructed to support the sleepers which carried the arch ring. At certain places, drums filled with stone, were used as pillars. The pillars were spanned by sleepers, over which a row of sleepers, more or less in the profile of the arch was laid.

The inequalities were made good by mud plaster. Over mud plaster a coat of 1/2 inch thick cement plaster was given. The cement plaster was oiled and greased so that it could separate easily from the concrete.

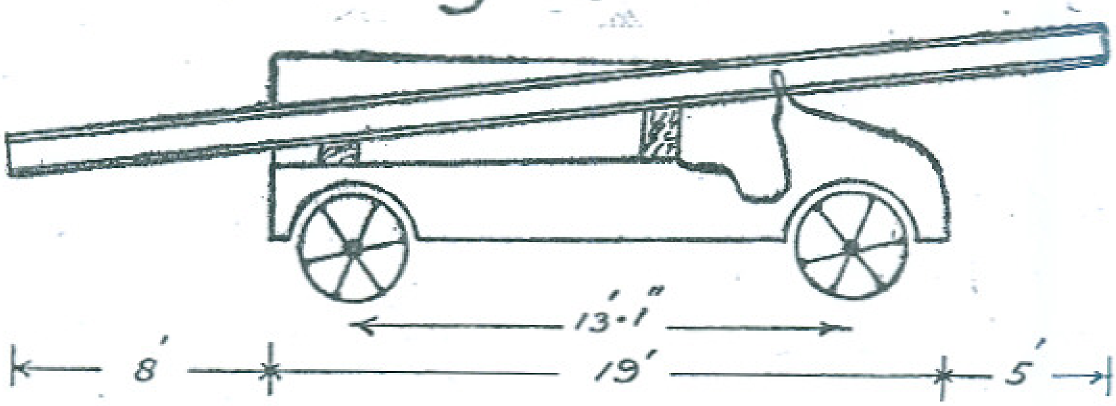
The centering was removed when concrete was 28 days old.

Superstructure for girder bridges. The alterations to members of old railway girder bridges, to suit for road purposes, were made in the N. W. R. Workshop at Jhelum. The members were catalogued and numbered in the workshop and then despatched by train to Pathankote where these were transhipped to Kangra Valley Railway and finally unloaded at Kangra and Nagrota Railway Stations.

Carriage of girders. The carriage of the girders to the site of bridges has been done departmentally by the P. W. D. trucks. Some of the members were 33 feet long and 4½ tons in weight. Previously at Ghambir Bridge on Chakwal-Talagang road, where a similar bridge was erected, these members were carried by a trailer attached to a lorry. Arrangements were made with Military authorities for carriage of these by their special vehicles fitted with trailers. Their expert staff after inspection of the road informed that the vehicles with trailers could not negotiate sharp curves on these roads.

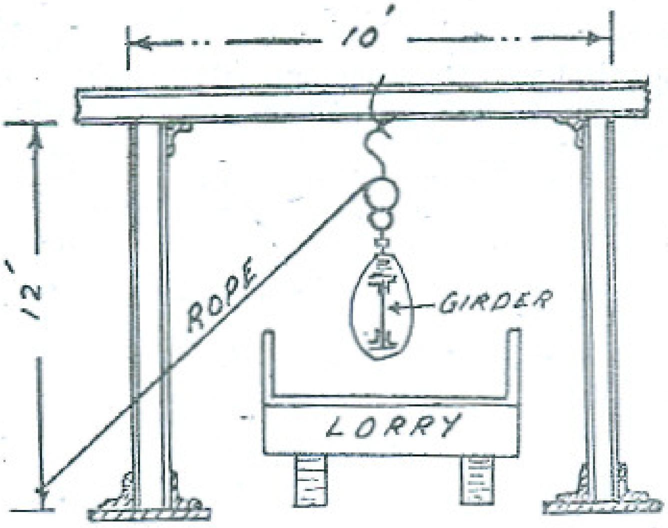
Ultimately it was decided to load the girders directly on P. W. D. trucks by dismantling the roof of drivers' seat.

Fig: 5



The girders projected about 5 feet beyond the bonnet, and about 7 or 8 feet behind the body of the truck as shown in the Figure 5. The lorry was very carefully driven by an experienced driver, and safely taken to the site of works. Near hair-pin bends the lorry had to be backed once or twice to negotiate the bend. An additional driver was given in the lorry, to help and guide the driver throughout the journey. As against the cost of 0-1-6 per maund per mile the work has been done at a rate of 4 pies per maund per mile.

Fig: 6

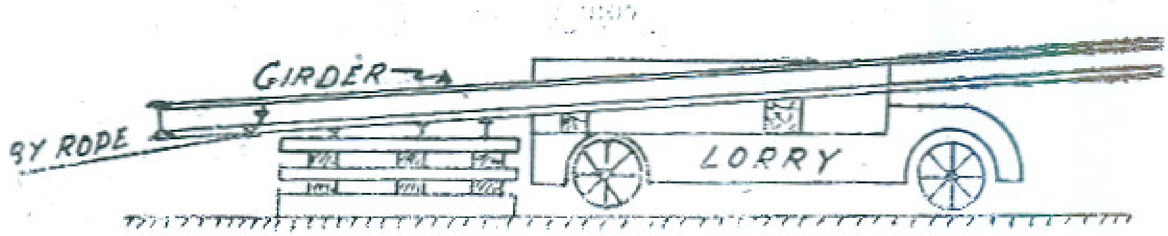


Loading of griders on lorries. A false staging about 10 feet wide and 12 feet high was erected as shown in Figure 6 at the railway station. In the centre of the horizontal beam a chain pulley block was fixed. This pulley block lifted the girder from the ground until it reached the top. The lorry was driven inside the staging, and the girder was gradually lowered in position. Timber blocks and sleepers were placed in the lorry to keep the girder at its correct position. The girder was tied firmly by slings of wire rope or chain so that it could not move during the journey.

Unloading. A similar arrangement was made at the bridge site unloading. Where work of unloading was not much, *i.e.*, bridges with plate girders a different method was adopted.

The rear end of the girder was tied by means of a chain sling to a tree nearby. The chain tying the girder, to the body of the truck was unloosend and the truck was started very slowly. The girder was pulled by the chain or wire rope tied to the tree until its one end rested on the ground. The truck was then stopped.

Fig: 7



A platform of timber sleepers having the same height as the truck was erected at the rear of the truck, and it was started again. The girder got unloaded and quietly rested on the platform of sleepers.

It was lowered from the sleeper platform by means of jacks. By adopting this novel method of transport arrangements, a saving of about Rs.15,000 was effected in the cost of transport.

Ban Ganga Bridge. The erection of this bridge created a special problem. The height of bridge was 55 feet above bed level. If staging was to be erected, it had to be kept in the bed up to end of May. It was feared that floods may come at any time and damage the staging, and girders supported on these. The original bridge which was standing intact was 80 feet span. The new bridge was to be built at angle of 26 degrees to the original bridge and was of 150 feet span as shown in the plan (Plate II-*a*).

The change in the alignment of the bridge was made for two reasons :—

(1) The alignment of approach to the bridge on the right bank was considerably improved. The approach was now practically straight. If the old alignment had been followed the approach would have been on a sharp curve with a radius of 50 feet.

(2) The approach road was shifted to a high ridge of rock, thus reducing its height considerably. The average height was only 12 feet as against a height of 35 feet if the original alignment had been followed. This resulted in a saving of about Rs.40,000 in the cost of approaches. In the interest of economy, it was further decided to make use of the existing left abutment, and extend it on the downstream side to take the bearing of the new girder.

It was proposed to assemble the girders of new bridge by making use of the old bridge as false work. Later on the new bridge was to be used for dismantling the old bridge. The old bridge was a weak structure and could just bear the weight of a single boom of the new bridge. The boom which was 155 feet long was supported during erection on the old bridge, and staging beyond the right abutment. The photograph No. II shows the old bridge, staging, etc., and the booms of the new bridge. After completion of the boom when it could carry its own weight, the bridge was relieved of its weight by supporting it on left abutment, right abutment and staging, etc.

Work on second boom was then started and it was similarly completed. This boom was also supported on the left abutment and staging.

Both the booms were connected by temporary cross girders at both ends, so that they were about 7 feet apart. The steel troughing of t' old bridge and cross girders were then dismantled and lifted from th position.

The two booms of the old bridge were then brought near each and lifted bodily from their original position and supported on th bridge. The members of trusses which were connected to each o bolts were disconnected and transported to a level piece of ground near the left abutment and dismantling of the old bridge was completed.

Slewing of the booms. Meanwhile temporary staging had been erected to serve as an approach to the abutment on the right (See photograph III). On this staging long line of rails were placed which were oiled and greased to reduce friction. These rails were at a higher level than the proposed level of the bottom of booms. Jacks were placed below the booms and these were lifted at both ends. Rails were placed below the booms, in continuation of the rails of the staging used as approach to the new abutment. The right end of the boom was gradually and slowly made to travel on the rails, with the left end kept in its original position.

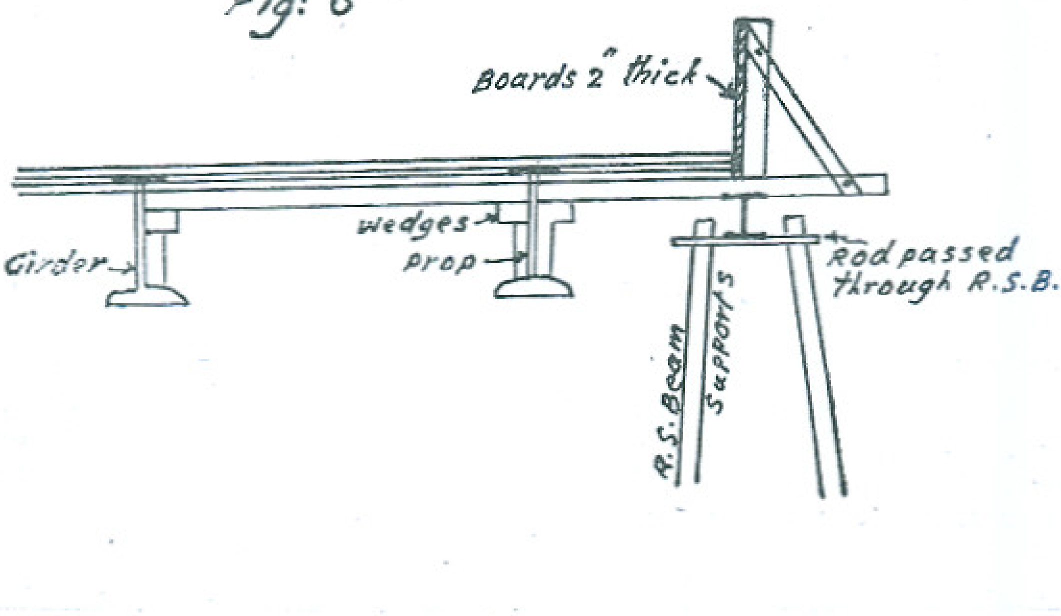
When the booms reached their true position they were lowered by jacks. The temporary cross girders were removed, and the booms separated and shifted to their correct position on the new abutment. The cross girders and longitudinal girders were then connected. The bed stone on the upstream end of the left abutment was to be built at the same place on which the old bridge rested. This bed stone was laid after jacking up the girders and supporting them on a platform of R. S. Beams. This difficult work of erection and slewing the bays in position was done skilfully and successfully by the experienced staff of the Bridge Department without any hindrance or difficulty.

Awa Khud bridge. The erection of girder for Awa Khud bridge was a much simpler affair. Special steel staging was fixed to support the members of the bridge during erection. This staging was brought from Jhelum Workshop, and was sent back after completion of the job. The cost of transport, erection and dismantling of this staging was a heavy sum.

The photograph IV shows Awa Khud Bridge with part of staging dismantled.

Reinforced concrete slab over girder bridges.
Detail of Shuttering

Fig: 8

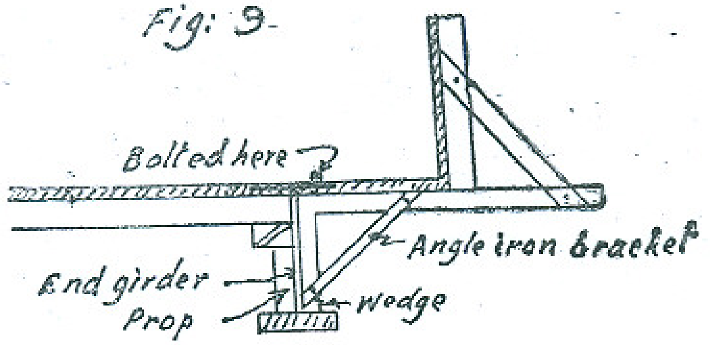


The slab over plate girder bridges is cantilevered out at both sides. The shuttering for space between the plate girders was an easy matter. Vertical props about 5″ × 5″ were placed on the lower flanges, about 5 feet centres. Over these 2½″ × 5″ battens were placed, so as to span across the space between plate girders. Wedges were placed below these battens so as to raise or lower them when necessary, 2″ thick wooden planks for shuttering were placed over these battens parallel to the plate girders.

The battens for cantilevered portion were supported on the outside on R. S. Beams running parallel to the plate girders, and on the inside end on props placed on the flanges. The R. S. Beams were supported on vertical posts made of steel beams (See Figure 8).

At Googly Bridge however, as there was danger of floods the

Fig: 9-



cantilevered portion was supported on brackets made of angle irons to be used for hand railing later on. The bracket was supported on the outer flanges of plate girder. Holes were made by cutting a rivet in the top flange and the horizontal angle iron bolted to this. It was also supported by diagonal angle iron which was securely wedged in on the lower flange as shown in Figure 9.

The bolts were cut after concrete had set. The angle iron frames were fixed about 5 feet centres.

For trussed bridges the battens for shuttering were supported on props placed on longitudinal girders. The fixing of shuttering was not a difficult affair. The striking of shuttering however proved a tough matter, as it had to be released from the underside of the bridge. Scaffolding was suspended from the booms on which men worked and removed the shuttering.

Timber used for shuttering was chirwood which was liable to warp. It was not planed. The planks were mud plastered, and then cement plastered before laying concrete.

Reinforcement. The reinforcement was fixed on the shuttering as per plans supplied to subordinates, and tied by means of G. I. wire. Precast concrete pellets were used to keep the bottom bars about ¾″ above the shuttering to give the necessary curve.

Concrete. For R. C. C. work 1 : 2 : 4 concrete has been used. The aggregate was screened and graded before use. For important bridges, the concrete mix was designed according to a method given in an article by Walsh in Concrete and Constructional Engineering of August 1934. It was proposed that the mix should have a strength of 3,000 lbs per square inch.

The formula for strength is

$S = 2{,}950\ (c/w - 0.34)$

Value of c/w = Cement water ratio for 3000 lbs
concrete = 8.2 gallons.

For easy workability the design mix was to have one part cement to about 5.25 parts of loose mixed aggregate by volume which is practically 1 : 2 : 4 concrete. The percentage by weight passing each sieve for standard grading is given below.

Size of seive No.	100	48	28	14	8	4	$\frac{3}{8}$	$\frac{3}{4}$
	2	9	21	26	33	44	65	100

The aggregate at Ban Ganga Bridge gave the following results when analysed by Tyler's Sieves

Size	100	48	28	14	8	4	$\frac{3}{8}$	$\frac{3}{4}$
Percentage passing each sieve								
(1) (*a*) Fine Aggregate	2.5	11.5	34.8	73.8	100	100	100	100
(2) (*b*) Coarse aggregate	0	0	0.0	0	0.7	28.2	73.2	100
(3) (*c*) Mix obtained by combining parts of coarse aggregate and one part of fine aggregate.	0.8	3.8	11.6	24.6	34	52	82	100

If two parts of coarse aggregate are mixed with one part of fine aggregate, the resulting mix has percentages shown in third row tabulated above. When compared to standard mix, this is deficient in fine sand size 14 downwards. Similar coarse aggregate is deficient in size above No. (4) and $\frac{3}{8}$ inch. In other words coarse aggregate is too fine, and fine aggregate too coarse.

A pocket of finer sand was searched, and this was mixed with the original sand half and half.

Similarly bajri was screened on $\frac{3}{8}$ inch sieve. The bajri which was retained on $\frac{3}{8}$ inch sieve was mixed with the original bajri in proportion of 1 : 2 The resulting grading was more or less similar to standard grading.

The aggregate in other bridges was graded in a similar manner.

The grading was deficient in fine sand at every place, which was difficult to get. Concrete for important bridges was mixed in a concrete mixer. About 8.5 gallons of water were used per batch. Each batch consisted of one bag cement 5 cft coarse aggregate, and about 2 to $2\frac{1}{4}$ cft of sand according to grading determined above.

Laying of concrete. In long bridges like Awa Khud and Ban Ganga where concreting could not be completed in one day, the bridge was divided into bays 31 feet long, and each bay was completed in one day. The concrete was started at the end of bridge away from the mixer. Wooden trestles or planks were placed over the reinforcement for labourers carrying concrete. The concrete was tempered rodded and speared so that it became one homogeneous mass and left no cavity. The surface was finished roughly, up to the top level of the slab. Immediately this had been finished 1" thick wearing coat was spread, so as to bind with the original concrete. The wearing coat was properly finished by wooden floats. On each day about 500 cft of concrete was laid. The wheel guard was concreted usually the next day. The slab was given a cross camber of about $1\frac{1}{2}$" and holes were left near the wheel guards for drainage.

Expansion Joint. The detail of expansion joint for Googly and Manjhi Bridges are shown below.

Expansion joints were also provided on Awa Khud and Ban Ganga bridges at the ends. The details of these are shown below (Figure 10 and Figure 11).

Fig: 10

Expansion Joint for Gugli Bridge

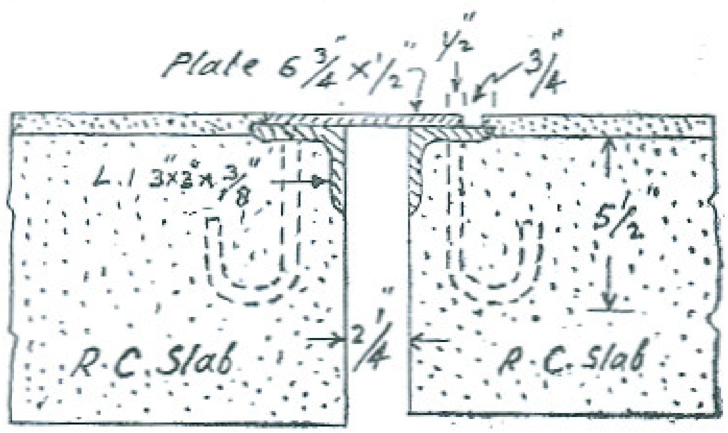
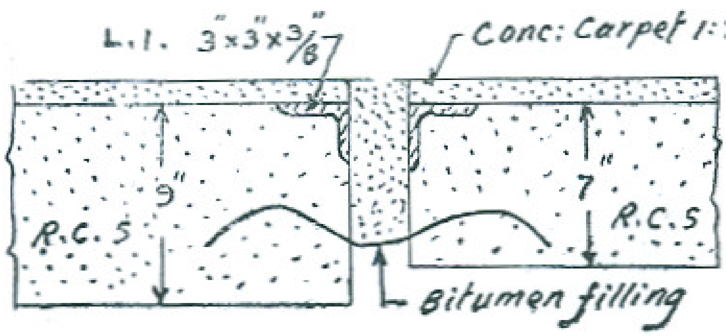


Fig: 11

Expansion Joint for Manjhi Bridge



The expansion joint shown in Figure 10 is much better in performance, as no dust, sand or grit can get lodged in the space between two slabs.

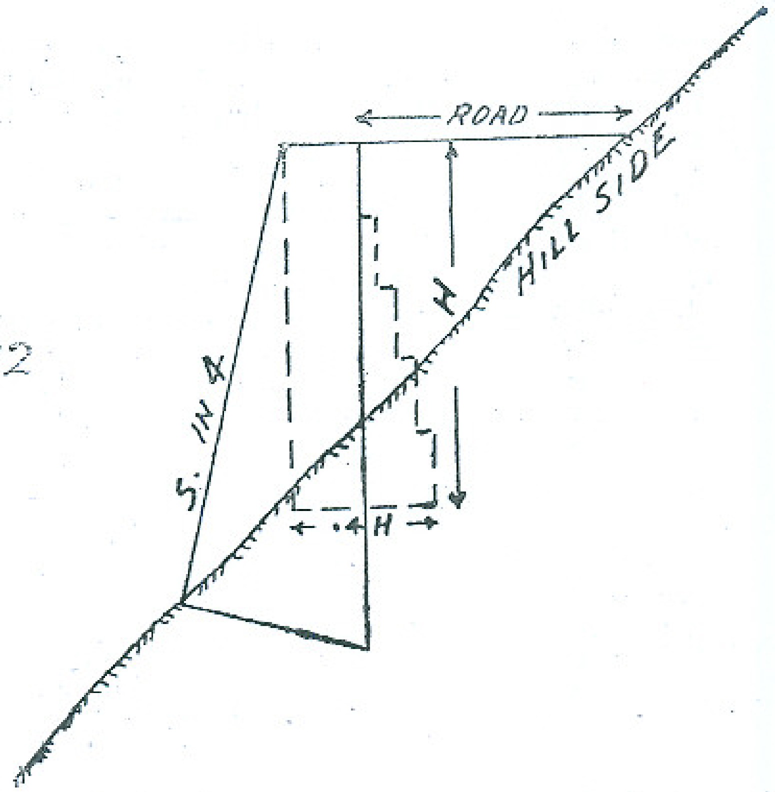
In case of expansion joint shown in Figure 11, the width of space between slabs increases when the slabs shrink. This space gets filled up with dust grit, etc., and has to be kept clean and filled with bitumen every now and then.

Retaining Walls. Where large hill slides occurred and road was either washed away, or in danger of washing away retaining walls have been constructed as per details shown in Plate VI.

These walls have a back batter of 1 in 4. If the walls having different heights are tested, it is found that they are safe for a pressure exerted by filling having an angle of repose of 45° *i.e.*, 18h lbs against a pressure of 30h lbs adopted in plains.

The pressure exerted by stone filling is less than sand and silt therefore they are safe.

Fig: 12



There has been a fetish of building retaining walls with sloping face. They are sometimes uneconomical as compared with a face vertical, and back stepped as shown in Figure 12. The side of the hill on which wall was to be built is sloping. For the same width of road formation the wall with the sloping face has to be taken very deep as compared to a wall with face vertical. The volume of masonry is very much less in the second case.

These walls can be built in the same manner as wing walls of bridges, having width equal to .4 H. Instead of having walls entirely vertical, we can give a slope of 1 in 12. Walls with this profile have been built at certain places. They have resulted in increased width of road, without any extra cost, which is a great boon in the hills.

At certain places, where walls were made of dry stone with this profile the dry stone masonry bulged out. This profile is not recommended where dry stone work is to be done, because contractors will seldom dress stones properly.

Bahmanoo Nala Mile 168/4. As explained in Chapter I, a huge hill slide occurred and blocked the road and nala. The soil was extremley treacherous and with slight rainfall the road would get blocked. When wet the soil was sticky and very difficult to remove with pick axes and shovels. The soil was mixed with stones which would keep rolling on without any warning and make the work of clearance of slips very difficult and slow.

From top of the hill to bottom of nala, the soil was soft and slippery. It was impossible to find a suitable and firm ground for foundations of retaining and breast walls. During winter, the road at this place was widened to 40 feet width, but the first heavy rainfall of monsoons caused heavy slips, and blocked the road entirely. The slip was however cleared in a few hours, and 9 feet wide road way was made for passage of cars and lorries. In the heavy rainfall of 12th July, the lake burst on the downstream side, and created a chasm about 40 feet deep in the bed of nala. Retaining walls protected by tar drums filled with concrete were built departmentally, and a causeway was built in the nala. The traffic was restored in about one week. The hillside was troubling throughout the monsoon on account of slips and many gangs were kept at this site to keep the road open.

Finding this state of affairs, Mr. Sondhi suggested that the loose hill side should be washed by sluicing it with a water jet under high pressure. In other words artificial rain of heavy intensity should be created to remove the loose debris.

A Haleford Trailer Pump fitted with V-8 engine was used for this purpose. The centrifugal pump had 4" suction and $2\frac{1}{2}$ inch delivery. The pressure produced in the jet on the average was 75 lbs per square inch.

The pressure could be raised to 100 lbs or 125 lbs per square inch, but the engine would get hot, if it was run at this speed.

The delivery pipe made of canvas was about 40 feet long, and was supported by 12 men. Two experienced men worked the nozzle, which produced the jet. The back pressure of the jet was so great, that it was difficult to hold the nozzle in position. The output of removal of debris was measured roughly. Where the soil was soft, the jet removed the soil at a rate of about 1 cubic foot per second. When the jet had been worked