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Improvements to Delhi and Lahore Stations

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1. DELHI STATION

1. **Introduction.** The necessity for improving the facilities provided for the travelling public arriving at and departing from Delhi Main Station and the conditions contributing to the state of congestion prevailing at Queen's Road at train times had long been realised. The 3rd and Inter class waiting halls in the East and West wings of the station building opened directly on to the Queen's Road, with the result that, at train times this thoroughfare became heavily congested. Tongas picking up and setting down passengers at the entrance, impeded the passage of through traffic on the road and of vehicles bound for the upper class entrance in the middle of the station building. The position was further complicated by the fact that Tonga, Taxi and Motor Bus stands were situated on the other side of the main road and they had to cross both traffic lines to get into the station premises.

At the upper class entrance and exit, conditions were no better, the circulating area for road vehicles being inadequate and having narrow gates opening directly on to the already congested Queen's Road, at right angles to the traffic lanes. This state of affairs made traffic control practically impossible and there was danger of accidents occurring.

Improved access to the station premises and effective police control of vehicular traffic bound for Delhi Main station, could only be obtained by realigning the Queen's Road. This necessitated throwing it back on to a portion of the Queen's Gardens, in a straight line parallel to the station building. It will be seen from Plate I that realignment provides easy access through wide openings and by separate traffic lanes to each of the three station entrances as well as to the inward and outward parcel office. The Tonga and Taxi stands are now situated within the station premises and are well under Railway Police Control.

The old 3rd class waiting halls and booking offices were very cramped and not nearly large enough to cope with the volume of the heavy traffic offering. Passengers had to enter and leave through a few small arch ways and no protection was afforded against the weather, while getting down from and in—to a conveyance. This exposed all the passengers arriving or departing by tongas or taxies, to considerable ɔnvenience in the rainy season and during the excessive heat of ımer. It was therefore proposed to add to the waiting halls as wn in Plate II, providing a covered porch under which tongas and es could pick up and set down passengers in comfort whatever the ther.

At the first and second class entrance, the concourse in which king and luggage offices were situated, was inadequate and gave rise onsiderable confusion and congestion at rush hours. Passengers and r luggage which was set down at the steps beneath the porch, ıained there on account of the congestion in the circulating area and consequent delay that occurred in buying tickets and waiting for ghment of luggage. To remedy this, the accommodation in the ion building was remodelled and rearranged so as to widen the con-rse by about 30 ft. as a first measure and eventually by more, it nd necessary. This involved considerable alterations to the station ding on the left as shown in outline in Plate III. It was also proposed t the barrier between the concourse area and the platforms should be oved further towards the luggage lifts, thus providing additional ulating area free of columns at the entrance to the various platforms. en the work came to be executed this was deferred. It is intended at ıter stage to remove various small pillars existing in the concourse and onvert it into a commodious hall free from obstruction.

The lavatories and bathing facilities attached to the 3rd class waiting s both East and West—were somewhat unsatisfactory. Those on the t were considerably spread out and cramped the circulating area in ıt of the parcel office adjoining and on the West they were too close to Vendors Shops alongside It was therefore necessary so rebuild h these as shown in outline on Plate I to a more modern and sanitary ıdard.

2. **Design of Porch and Overhang**. It will be noted from the cription given in Para 1, that many of the improvements that were ied out in the upper class and 3rd class entrances did not present any icate structural work. It merely meant rearrangement and rebuilding ɔetter standards but the provision of a wider porch and overhang for public vehicles for all the three entrances presented a very interesting blem of structural design. Although the scheme for improvements to upper class entrances was the first to be worked out, the alterations he Inter and 3rd class halls were carried out first as they were sidered to be more pressing. Work on the upper class entrances was erred and it was decided to carry out the improvements later in the owing stages :—

ge (I).

(*a*) The concourse would be extended at either end by shifting the existing upper class Booking and Enquiry offices and the Luggage office elsewhere in the building ;

(*b*) the existing porch would remain as it is except that its fl
would be raised by about a foot and ramped at either end
facilitate washing and drainage ;

(*c*) a low platform would be provided outside the porch along
length for the convenience of the passengers using tonga.

Some of the work, coming under this stage, has already been carr
out.

Stage (*II*).

Of the existing columns in the concourse every alternate one will
removed to improve the circulating area therein. The design of
arrangements for the removal of these columns which carry the waiti
and retiring rooms on the first floor and the actual execution of work
expected to present great difficulties.

Stage (*III*).

The existing porch would be extended at either end or rebuilt
the extended length and a lean to shelter provided for the tonga traffic,
shown in Plate IV.

3. **General considerations affecting Design.** It will be seen fro
Plate I that a shelter for the tongas had to be provided without encroac
ing on the roadway outside by columns which would have restricted t
road width and been a menace to traffic. The question was therefo
to design a cantilever shelter for this purpose. It would, of course ha
been easy to design this shelter if it could be kept at the same height
that of the porch and the floor level of the first floor but this would r
have given any protection either against the sun or beating rain. It w
absolutely necessary, therefore, to put the projection at a lower heig
than the main roof and yet design it as a cantilevered structure. T
alternative of designing the whole of this work in steel, was considere
but this would have involved very intricate connections and present
considerable difficulty in erection. Moreover, this would not ha
conformed to the architectural feature of the station building, which
built in brick. It was decided, therefore, to design it in reinforc
concrete. The main difficulty in design and later in execution was t
provision of cranked beams which were cantilevers in the projecti
shelter and a part of the front wall in their vertical length. T
anchor span formed the roof of the porch. In designing the bea
its corners had to be strong enough to resist the bending moments comi
on them. The main beam carrying the cranked beams is reinforc
with diagonal stirrups to resist torsional stresses. The gener
features of this design are shown in Plate V. Expansion joir
have not been provided as the maximum length of the structure
118 ft. only, against the 100 to 200 ft. apart which is permissib
depending on the range of temperature expected. Their provisio
would have necessitated duplicate cranked beams and columns, whic
besides being costly, may have marred the appearance of the structur
The alternative of an expansion joint above the level of top of colum

was considered and rejected, as special arrangements at column tops would have been necessary and the continuity of the deep lintel beam would have had to be sacrified. The design has proved to be satisfactory. No trouble due to temperature effects has been experienced so far.

4. Details of Design.

(*a*) *The slab.* The design of the slab, and cantilever beam is common to the Upper class and Inter and 3rd class entrances. Plate IV shows the key plan and typical cross sections for the upper class and Plate V for the Inter and 3rd class entrances respectively. The slab in the overhang, is designed for a live load of 30 lbs. per square foot. A heavier load will not have to be carried. The same live load has been adopted for the beams, columns and foundations. The slab on the portico, however, has been designed for a live load of 50 lbs. per square foot which is usually permitted for flat roofs. Since the slab panels on both sides of a beam are not likely to be simultaneously covered with live load for their entire length, it is customary to allow a smaller live load in the design of beams, columns and foundations than for the slab. In the present case the latter were designed for a live load of 30 lbs. per square foot. The slabs have temperature reinforcement at the top and mud or lime concrete terracing has been omitted to cut down the dead weight and obtain economy. There has been little or no serious cracking in spite of the direct exposure of the slab to heat and cold and it may be inferred that the temperature reinforcement used has been effective in distributing temperature stresses.

The design of the slab does not present any special features. A bending moment of $\frac{WL}{12}$ has been assumed both at the centre and over the supports as is usual for slabs carried on beams. For appearance, the under-surface of the slab is flat. The beams carrying the slab project above it and are designed as T beams, which are economical. The whole construction is in 1 : 2 : 4 ordinary grade concrete with the following constants :—

Cylinder strength of concrete after 28 days preliminary tests	2700 lbs. sq. inch
Cylinder strength of concrete works tests after 28 days	1800 lbs. sq. inch
Safe compressive stress in bending ...	750 lbs. sq. inch
Safe compressive stress in direct Compression	600 lbs. sq. inch
Diagonal tension	75 lbs. sq. inch
Bond stress	100 lbs. sq. inch
Modular ratio	17.8

(*b*) *The cranked beam.* The cantilever portion of the cranked beam is designed as a T beam. The adopted depth gives maximum economy,

taking the cost ratio of steel to concrete as 60. The mathemat
formula for economical depth of a T beam may be derived as follows :

The steel area required for a T beam to carry a bending mom "M" is $\frac{M}{fs(d-\frac{t}{2})}$ where it is the permissible stress in steel, d the de of T beam from top of slab to the centroid of tension reinforcement a t the slab thickness. In T beams the entire compressive stress is assun to be carried by the slab and none by the stem and hence the lever a of the resisting couple is approximately $d-\frac{t}{2}$. Assume the cost concrete as a unit, then the cost of steel is r units denoting the ratio of cost of a cubic feet of steel to a cubic feet of concrete by r. Now the volu of concrete in the stem is $b'(d-t)$ where b' is the width of stem. Theref the total cost of beam in concrete units per foot run = C = r × are steel + area of concrete = $\frac{rM}{fs(d-\frac{t}{2})}+b'(d-t)$.

This is a minimum when

$$\frac{dC}{dd}=O \text{ or } \frac{d}{dd}\left\{\frac{rM}{fs(d-\frac{t}{2})}+b'(d-t)\right\}=O.$$

$$\therefore \frac{-rMfs}{fs^2(d-\frac{t}{2})^2}=-b'$$

$$\therefore \left[d-\frac{t}{2}\right]=\pm\sqrt{\frac{rM}{fsb'}}$$

$$\therefore d=\pm\sqrt{\frac{rM}{fsb'}}+\frac{t}{2}$$

The stresses in T beams were checked by the usual method.

The bending moment at the end of the cantilever is, strictly speak transferred to the main R. C. beam resting over the columns produc torsional stresses in it. This torsional effect is counteracted by the sc portions between louvred openings which form the vertical legs of cranked beams and are held back by the anchor span *vide* Figure Plate V. An exact analysis of this torsional effect is extremely complic ed. To simplify matters, it was assumed that the entire bending mome from the cantilever is transferred to the vertical leg of the cranked bea which had to be designed for both an axial load and a bending mome The section is rectangular and the method used for the determination stresses is given in Appendix I. It will be seen from the calculations t the results obtained by the first trial are close enough and that the conc stress is satisfactory. It will be noticed that the tensile stress in the steel is low at 10,000 lbs. sq. in and the design is therefore wasteful. This cannot however, be helped due to the limitations imposed by the nature of construction and architectural requirements.

(*c*) *The anchor span.* This carries a bending moment arising from the cantilever at one end. It is monolithic with the vertical leg of the cranked beam on one side and is assumed to be freely supported on the wall at the other end. The anchor span derives relief of positive bending moment from the bending moment in the cranked beam due to the cantilever load. Only half of the B. M. due to the dead load has been assumed as affording relief for the reasons given in Appendix II, where the calculations are given. It will be noticed that the maximum unit shear is only 78 lbs. sq. in. and no shear reinforcement is necessary. Actually stirrups have been provided for the reasons given below.

At the support the beam is doubly reinforced and carries the full bending moment from the cantilever. There are two recognised methods of designing such beams :—

(*i*) Assuming that the steel and concrete work together as a composite beam and

(*ii*) Assuming that the steel carries the whole bending moment as in a steel beam and the concrete performs the functions of a web transmitting shear only.

In the present case the bending moment being very heavy the concrete will be overstressed if the former method is adopted. The latter method was therefore adopted and steel provided to carry the stresses. When the steel beam theory is used for stress calculations, it is conventional to tie the two layers of bars by stirrups for effective transmission of shear at a spacing of eight times the diameter of the main bars.

(*d*) *The Main Beam.* This beam is continuous over four spans. See Plate V Figure 3. Each span carries two cranked beams with the live and dead load taken by them. In addition it carries a uniformly distributed load of 780 lbs. per foot run.

This beam and the columns are monolithic forming a frame. The analysis of such frames bv the old classical methods is very involved. Some of the methods of analysis are reviewed in Appendix III to show how easy analysis by modern methods is.

The classical methods of analysis are : —

(*i*) the general method of indeterminate frames ;

(*ii*) the method of strain energy or least work ; and

(*iii*) the slope deflection method.

None of these is universally applicable with convenience to all problems but the one most suitable has to be selected for a given problem.

The slope deflection method is the more recent. Instead of solv for stresses and moments as the unknowns the slopes at joints and defl tions are taken as the unknown quantities. It can be shown that change of slope between any two points A and B of a beam is equal the area of the $\frac{M}{EI}$ diagram lying between these points and deflection B from the tangent to the Elastic Curve at A is equal to the moment the same portion of the diagram about B. The given beam is theref loaded with the $\frac{M}{EI}$ diagram, the resulting shear and bending mom diagrams giving the slope and deflection of the conjugate beam.

The number of the simultaneous equations to be solved in the classi method is very large and the process laborious. Any error in solving equations would result in the waste of considerable labour. Simp methods have now been evolved and one of the best is Professor Har Cross's Moment Distribution Method.

In this method all the joints (of a frame) are first considered locked against rotation, *i.e.* the ends of beams meeting in a point treated as "encastre." They are then released one by one and effect of such release on all the other members is considered step by ste

The method has been illustrated in Appendix III. It will be notic that the solution of complicated equations has been replaced by a sim arithmetical process. In our actual design a slightly simpler method adopted. It is given later in the Appendix III.

(*e*) *The Columns.* The intermediate columns are assumed to ca only an axial load and are designed by the formula :—

$$P = fc\ Ac + fs\ \ As.$$

The two end columns carry a bending moment arising from the r load. From the calculations which are given in Appendix IV it will seen that strictly speaking the weight of the column should not have b considered as it only helps to relieve the stresses which are a maximum the top. The design is for B. M. only and is completely unaffected the weight of the column.

(*f*) *Foundations.* In designing these, the columns were assumed fixed at the base with only a construction joint. At the base the conditic are intermediate between fixed and hinged ends. But the worst con tions have been allowed for.

A hinged base could have been provided but construction wo have been complicated without any compensating advantage.

Plate V shows the detail.

(*g*) *General Remarks.* There are several methods in use for solvi indeterminate structures. The principles involved are essentially the sa but the methods in use have been reviewed in some detail in the Appendices to stimulate interest in the subject.

Multi-storey buildings instead of being massive brick structures should be built in R. C. with sleek outline and an attractive frontage. The utility, economy and appearance of R. C. work have been considerably enhanced by the recent development of prestressed concrete. Its technique places concrete under initial compression transmitted by prestressed reinforcement, so that any tensile stresses merely serve to relieve the initial compression. It cuts down weight and secures economy.

The analyses of indeterminate frames has grown in importance with the development of welding of steel structures also.

5. Construction. The authors must apologise for the considerable space given to the mathematical features of the design. They offer as an excuse the fact, that in a structure of this kind, that is the most important part of the project.

The work was sanctioned for execution at a difficult time. The clouds of war were on the horizon, which later broke out and created unprecedented difficulties before the work was completed. In fact, when the work was actually put in hand, serious consideration had to be given to its postponement in face of the difficulties already being experienced with labour and material. The improvement was however long overdue and the need was crying and it was decided to proceed with the work in spite of some opposition.

The work was done by contract. That in itself was a mixed blessing. The contractor himself was a qualified Engineer, who had imbibed the high traditions of Contracting Firms during his stay abroad and was out to emulate them. He was however somewhat inexperienced and although his resources were good, the capital at his disposal was limited. Conditions in the country, which were worsening were new to him and to add to his difficulties, he had cut his quotation to the skin, in order to get a footing in the contracting world. Towards the ends he was caught in the whirl of rising prices and scarcity of labour and material and the work suffered. But, it must be said to his credit, that he pulled through with commendable grit.

In the introduction to the paper, the congestion at the station entrances, the Queen's Road serving it and in the concourse and waiting halls was touched upon. If the new Queen's Road could have been undertaken first, the progress of the work which encroached on the old narrow highway would have been smoother. But difficulty arose over the acquisition of the necessary land in the Queen's Gardens, which was not overcome till about a year later and then with the intervention of the Chief Commissioner, Delhi Privince. Later, material for concreting and asphalting the New Queen's Road, was subjected to controls and the Municipal Authorities struggled through the work which was being completed about the time, the other work on the improvements ready for opening.

While material for the main work was awaited the alterations in main station building and the rearrangement of the wash places lavatories were undertaken. The lavatories attached to the third waiting halls need mention. The ordinary flushing type of W. C universally misused by the passenger unused to modern sanitation. T were therefore built with one 9 inches dia. semi-circular channel with rests on either side, running through a number of compartments. At end was a high level flushing tank of 20 gallons capacity, which wo automatically and emptied itself at regulated intervals, flushing out high velocity the accumulation in the channel. The floor and dad the lavatories were made of monolithic terrazo with marble chips yellow ochre for colouring matter. The corners were rounded off there were no cracks in the floor, which could harbour dirt. arrangement worked very well for a whole year, during the author's s

For a work of this magnitude, steel shuttering and skilled lab with experience of R. C. work are essential. Unfortunately, when work came to be done, steel was scarce and Delhi and its surrounc had no labour with experience. Wooden shuttering had therefore to designed. So as to minimise the expenditure on shuttering, in spite of class material and workmanship in its manufacture, it must be desig for repetitive use. Otherwise, the unit cost of construction is un high. In our case, there were ten columns exactly similar, shuttering which could be designed for reuse. Three sets were therefore made, reuse four times, which was the maximum, shuttering in wood work c be expected to last. The wood selected was well seasoned C. P. T which was strong and absorbed little water in use. There was little no warping between reuse. The construction was battened and height was made up in three sections, with angle iron frames, top bottom, which could be bolted together. Each section was made in halves so as to reduce to weight in handling and positioning du progress of concreting.

Concreting of the columns and footing was done three and two a time, for each of the East and West waiting halls. The time sche was, two days for excavation and set up of the forms, 1 day in concre and seven days for setting. So that the five columns at each end, t 20 days. In the ten days taken for the casting of the last two colur shuttering for the beams and slabs in front of the three columns alre cast was in progress.

Some detail of the concreting would be of interest. The foot were a straight forward job and the concreting up to just below gro level was easy. As it was finished, the first section of the shuttering erected on guides firmly fixed in the ground, on which the two halves the square section were mounted and bolted together, any slight error the set up being corrected by sights taken from bench marks. The b in the bottom iron frame were then drawn tight so that there was displacement during concreting. The whole operation took about 15 minutes. When the set up was ready, the scum of the concrete already in position was removed with a trowel and fresh concreting started. No attempt had been made to level off the top of the first layer of concrete and there was a rough surface left to ensure good bond. The height of each section of shuttering was about 5 feet and concrete was deposited in position by men working inside the column reinforcement. Two men working inside placed and rammed the concrete. While concreting of the first section was in progress, the second section of the shuttering was taken up the scaffolding and placed on the side, ready for setting up. Working with a small 13 cubic ft. concrete mixer and plenty of labour, three columns took about 12 hours of continuous work.

The shuttering of the beams and slabs presented great difficulty, some of which were intrinsic in the design of the work and others were due to scarcity of material. The cranked beams projected underneath the slab in the portico, were carried vertically down the front and were continued into the cantilever, but this time above the slab which was underhung. To complicate matters further, the walling between the cranked beams, was itself a deep lintel of reinforced concrete, in which space was to be left for the insertion of louvres for light. The photographs at the end show these clearly.

If only steel and workshop capacity were available, it would have been a simple matter to design first class steel shuttering which could be easily bolted together and dismantled. As it was, massive kutcha pukka brick piers and scrap sleepers had to be put to use and the work of erection and dismantling was both slow and clumsy. The supporting columns were 2′ 3″ square 9′ 0″ centres, except those under the beams, which were made rectangular 3′ 6″ × 2′ 3″ so as to accommodate the beam forms and in addition supports independent of the beam forms, for the planking of the slab on top of the beams. For the inside face of the deep lintel 13½ inches kutcha pakka brick walling, carried on 10 inches deep sleepers bolted together and acting as beams was used. So far the shuttering if clumsy was easy enough. The real crux of the problem was the front face of the cranked beams and the overhung beams of the porch slab. The shuttering for these could not be directly supported and the following method was adopted.

A floor of stout planking was built over sleeper beams carried on brick piers, spaced 9′ 0″ centres as before. The floor was strong enough to carry the load without appreciable deflection.

Small pyramid shaped precast concrete blocks were recessed into the planking and had on their upper surface slight projections which fitted into corresponding recesses in the body of the shuttering supported on them. A hollow frame of wood, formed the outer-edge of the vertical faces of the deep lintel and vertical legs of the cranked beam. The two low sides of the frame, formed the vertical faces of the horizontal legs of cranked beams. This frame was supported on the floor mentioned above, by means of the precast. Concrete blocks, being prevented from lateral displacement by recessing the lower edge, into projections on the blocks. The blocks themselves were embedded into the slab concrete when it was cast and were integral with it. The slight projection above the top o! slab was hidden from notice and on the underside, it was covered b) plaster put on for appearance.

Great care had to be taken in building the frames, to make t rigid. This was done by means of angle iron stays across the length at the corners. Although precaution had been taken against la displacement of the forms, these were frequently checked for alignme the process of casting.

With the material available, centring had been improvised to en concreting being done, continuously. Over the planks forming the shutte of the slabs, a ¼ inch thickness of weak cement plaster was laid. This intended to provide a smooth unbroken surface for the underside of the s and minimise the absorption of the water content of the concrete by wood. It enabled also, final levelling to be done, and slight irregular in level being corrected.

Then came the setting up of the reinforcement. The design allo for 25 to 30 feet length of reinforcing rods but when supplies came t made, nothing more than 18 feet lengths could be obtained. This m 50 per cent more splicing and consequent wastage in the length overlap and hooks at the end. The reinforcement required was in case heavy on account of the spacing of the columns, and the spacing proportioning of the rods had been based on the longer length reinforcement and a given diameter of reinforcement. On account of extra splicing required and smaller diameters of bars available working space between the bars, was seriously curtailed, making casting more difficult. A slightly more workable concrete mix had there to be used, which meant more water and some loss of strength.

An even more important factor was the loss of rigidity, whic invaluable, particularly with unskilled labour. This was greatly fel the case of the cranked beams, which being unwieldy had to be caref centred and attended to, frequently during the progress of concreting. is usual, the reinforcement was supported in position, vertically precast concrete blocks, which were built into the mass of the conc and horizontally by wooden spacers, which were removed as concre progressed.

As every engineer with experience of R. C. Work knows, for s work, 3/8 inch rods are to be preferred to 1/4 inch rodding. There two reasons. The greater spacing of the former, makes casting tamping easier. But what is even more important is that 3/8 inch ro ing has greater rigidity than 1/4 inch rods and will not easily distort un the feet of the workmen, who will tread on it. The authors certai learnt this by bitter experience, on the work, for which only 1/4 i rodding was available.

The design visualised a monlithic structure and concreting had t done continuously from one end to the other. The beams, lintels, sl sunshades and facade were all cast in one piece. The quantit concrete involved was of the order of 3,000 cubic feet and the whole operation took nearly 16 hours with two concrete mixers and about 300 labour.

It is usual in concrete work, to proportion the grading of the aggregate and design the mixes. This was done, the mix being roughly 1 : 2 : 4 with a quantity of water slightly in excess of that required to allow for the rapid evaporation expected in the months of March to May, when the casting was done. Considerable difficulty was experienced in pouring the concrete of the 5′ 0″ deep lintel, which spanned the columns and for ease in depositing had to be somewhat wetter.

The disposition of the labour on casting needed care. The quantity of concrete in the lintel and beams was considerable and it was necessary to ensure that concreting progressed continuously, no part of it being left exposed long enough to acquire the initial set. The labour was divided into four batches, two being served by one mixer. One worked on the lintel, one on the longtitudinal beam and two on the slabs, some men taking short rest between intervals of work, while a small reserve of men relieved them.

II. LAHORE RAILWAY STATION

The circulating area in the old Lahore Station was very cramped, being only 45′ × 43′ and serious congestion resulted during rush hours. The available space was reduced further by the projection of the 10 ft. wide staircase of the central footover bridge which extended some 20 ft. into the concourse. Also, the space was cut up by 4 Cast Iron Pillars in front of the footover bridge supporting the roof of the concourse. The arches at the entrance were themselves narrow and restricted movement. Conditions were, if anything, worse than those in the concourse at Delhi Main described in the first part of this paper. The need for a larger concourse which had been felt for a long time was therefore pressing.

The only footover bridge for upper class passengers was 10 ft. wide and restricted the free movement of passengers from and to the platforms in the Station. It was also defective and structurally weak. It was therefore decided to separate the incoming and outgoing passengers by providing a bridge 8 ft wide for each direction from the concourse to the entrance to platform Nos. 4 and 5. Reconstruction of the portion beyond platform No. 5 leading to Platform Nos. 8 and 9 was considered to be unnecessary as the number of passengers using this part of the bridge was considerably less. Some inconvenience is still caused to passengers from other platforms going to No.2 to which there is no direct opening from the bridge itself. They have to pass through the concourse. But it was considered undesirable to obstruct the already narrow Platform No.2 with a staircase leading down to it. The work on these improvements is now completed and the reconstructed concourse has a magnificent appearance with plenty of circulating area for upper class passengers.

The Foot-over Bridge. This part of the work was done by Bridge Branch. To avoid the provision of columns on platform : and in the area between 4 and 5 the bridge is carried on knee tru The design of the bridge itselt is conventional in triangulated tr with an R. C. slab floor carried on channel beams. The arche the central colonnade through which the two branches of the br pass as shown in Plate VI were dismantled and rebuilt to prc greater room for a cooly with a head load, the under side of the br having to clear a fixed distance above the Rail level underneath. spans of the arches to be dismantled were filled with brickwork in before the arches were dismantled to ensure that the stability of colonnade as a whole was not disturbed by unbalanced arch thr

The Tonga Shelter. As in the case of Delhi upper class passer arriving in Tongas had to get in and out of the conveyance, witl shelter from the sun and rain, and it was necessary to provide at an awning. On the other hand, it was extremely important to pres the pleasing external architectural features of this fort type of sta building with turrets at the ends. This, coupled with the fact there was insufficient room available in front of the station buil for both a road way, a car park and certain incidental accommoda ruled out the possibility of providing the type of structure descr for Delhi.

The shelter was therefore designed as an awning of reinfo concrete with the roof consisting of a slab carried on channel sha reinforced concrete frames as shown in Plate VII. To ensure stab and uniformity of foundation pressure the centre of gravity of foundation area under each frame was made to coincide with the ce of gravity of the total load at the top. The frames themselves 25 ft. apart which also measures the span of the slab. To ha flat unbroken under surface and to cut down the weight in the const tion of this long span slab, it was decided to adopt light weight feat crete floor construction.

This construction is based on the principle that the concret tension below the neutral axis of a slab is useless and simply adc the dead weight and cost. It is therefore removed and replaced light weight material, small ribs being left at intervals to take reinforcement as shown in figure 6. Hollow tiles were previo generally used for this purpose.

Recently, however, hollow blocks made of rice husk and cer have been manufactured by Gammon & Co. under the name of feat crete. The blocks are $\frac{1}{8}$ in. wider at the top and are thus wec in between the ribs. They are generally 12 in. wide and 18 in. and of various thicknesses. Full information regarding them ca obtained from the pamphlet issued by the manufacturers.

The undersurface of the blocks and ribs is plastered to giv smooth appearance.

The overhang was designed for a live load of 25 lbs. per sq. ft. for the slabs, beams, columns and foundations. Typical calculations for one bay are given below.

Live load on slab = 25 lbs. sq. ft.—
Dead load of slab = 83 lbs. sq. ft.
Cement saw dust cover = 12 lbs. sq. ft.

Total = 120 lbs. sq. ft.

Bending moment for continuous slab both at centre and

$$\text{support} = \frac{WL}{12}$$

$$= \frac{120 \times 25 \times 25 \times 12}{12} = 75{,}000 \text{ in lbs.}$$

The ribs are spaced 16 in. apart, therefore bending moment in each small T beam $= 75{,}000 \times \frac{16}{12} = 1{,}00{,}000$ in. lbs.

Depth required if N. A. lies in the topping or coincides with its bottom $= \sqrt{\frac{100000}{137 \times 16}} = 6.75$ in.

Use 7 in. thick feather-crete blocks with a total depth of 9 in. for slab.

$$\text{Steel area required} = \frac{100000}{18000 \times .86 \times 7.6} = 0{\cdot}85 \text{ sq. in.}$$

Provide 2 bars 3/4 in. diam. with a steel area of 0.88 sq. in.

The pillars of the overhang were kept butting against the masonry wall of the new porch and were sloped off near the base on the inside to clear the offsets of the foundation for the wall. The frames of the shelter are thus absolutely independent of the wall and the torsion of the overhang does not affect the design of the wall.

The foundations are taken $6\frac{3}{4}$ ft. below the foot path level. Trial pits dug for ascertaining the nature of soil showed that there was heavy filling for a depth of 15 to 16 ft. below road level at the site. In fact when the foundations for the new porch wall and towers were being dug, an ancient well covered over, was discovered below. This well was filled up with earth and covered with a R. C. slab before putting the foundation load on it. Tests carried out at a depth of 6 to 8 ft. below road level showed that a foundation pressure of 1 T per sq. in. on the well consolidated filling will give a factor of safety of 2 on the yield point of the soil. This pressure was therefore adopted in designing the foundations. The details of overhang and porch are given in Plate VII.

The new porch is 30 ft. wide and its roof is carried on a front wall with 19 ft. span segmental arch openings in it and on the other side by the wall of the concourse. The concourse has three entrances one of remove any possibility of the horizontal thrust being effective so far down as foundation level. The foundations were therefore designed to take the horizontal thrust from the 24 ft. span three centred arch on one side only. This resulted in an economical design and did away with the unwieldy foundations that would have been required otherwise. The assumptions made have been fully justified by the results obtained, as no adverse effects have been noted on the stability of the tower. Large economies in design, as in this case, are possible if the problem is viewed from all angles and the various factors affecting it are fully considered instead of theoretically designing for the worst conditions which do not occur in practice.

The Concourse. As stated earlier the old concourse was only 43 × 45 ft. as against an area now of 104 ft. × 50 ft. with no more than 2 columns in front of the old masonry towers to obstruct the movement of the public using the station. This involved the dismantling and rebuilding of the wall carrying a part of the old station roof in front of the old foot over-bridge. The two Cast Iron columns immediately in front of the old foot over-bridge supported a three span arch and above them the roof of the over-bridge consisting of wooden battens and tiles. This roof was underpinned with steel work and the wall itself was dismantled. The columns and the girders were however left in position to serve as a centring for the main lintel of 50 ft. span. This lintel carries the wall which is seen in the elevation in Plate No. between the Clock towers.

During the progress of this work; the roof over the concourse had also to be underpinned. The roof of the new extended concourse was to be carried on the lintel already described, one R. C. frame consisting of two pillars and a girder and the front wall opening into the porch. The 50 ft. span beam in the centre and the two R. C. pillars were designed as a monolithic frame neglecting the restraining effect of the side slabs on the distortions of this frame under load. The method of analysing has already been given in the Appendices relating to Part I of this paper. The frame is so simple that the use of any special method is unnecessary and the bending moment can be calculated from the formulae given in all standard text-books on structual design.

The design of the beam needs some comment. As a rectangular beam in reinforced concrete carrying a heavy load it was 3 ft. × 5 ft. in section. In such construction involving long span, the Vierendeel girder is certainly cheaper but sufficient height could not be obtained for it, as otherwise, it might have been visible above the roof from the windows in the wall behind. It is considered that prestressed concrete would have enabled cheaper construction, by cutting down the weight, which could be made more economical still by designing it as an "I" Section, to achieve greater lightness.

On account of the difficulties experienced in obtaining the desirable lengths of rodding, the design was later altered from R. C. slabs to three inches troughing filled with cement concrete. The delay likely to have been caused if the original designs had been worked to, and the possibility

40 ft. span in the centre and 2 of 20 ft. span on the sides. The roo the porch consists of the same trusses as were provided on the old p with C. I. sheets on top. There are no special features in its de or construction.

The porch wall behind the awning was first built in front of the porch wall and the R. C. awning was constructed. There were no sp difficulties in this construction, except that the correct bar lengths not obtained and a lot of waste occurred in splicing the bars which only 18 ft. long. The top of the overhang was covered with a cer and saw-dust slab in the proportion of 1 : 7. This has not proved t entirely satisfactory as cracks have developed throughout the cove and water can percolate under it and thence into the R. C. slab feather-crete. The authors, therefore, do not recommend its provision

Against expansion and contraction effects, expansion joints been provided in the slab so that its central bay is separated from adjoining ones. It was not considered advisable to extend the expan joints into the frames as this would have necessitated the provisic duplicate verticals, which would be architecturally undesirable. harmful temperature effects have been observed except in the R. C. f in front which has cracked in two or three places on each side of expansion joints. This may be due to improper curing which is diff to carry out in this position and the concreting having been done in weather, The centring for the work was carried on pillars of brickv in mud. These carried timber decking of sawn sleepers on which laid earth, to bring it to correct surface. The feather-crete blocks then put in position, reinforcing bars laid and the concreting of ribs topping done. The frames were cast before casting the slab.

After the work on the overhang was completed, the roof of the p was dismantled and the back wall with 40 ft. and 20 spans lintels b The porch was then completed with a trussed roof over it.

The back wall of the porch takes also the roofing of the concou The brick masonry columns supporting the lintels are T shapped anc designing their foundations eccentricities due to loading were neglecte the foundations are made continuous and the columns are well tied to rest of the building by deep lintels.

The end towers in the front wall of the porch are subjected to thrusts from two sides at right angles to each other, in addition to overturning moments from the vertical loads. This introduced l complications in design and would have resulted in very heavy four tions. It is common knowledge, however, that pillars of arches buildings can stand unbalanced thrust even when of ordinary Sometimes a part of a series of arches on pillars is dismantled and yet whole colonnade does not give way. The reason obviously is spandrel masonry acting as a tie and neutralizing the effect of horizo thrust.

Moreover, the foundations under the front wall are continuous rails embedded in foundation concrete. These also will act as ties of considerable inconvenience to the public during the monsoons wl were then imminent made this necessary. Eventually, a plate girder o ft. span was used instead of R. C. lintel as originally proposed. Th inches troughing which was completely encased in concrete and fo the roof was precast in the Jhelum Workshops and delivered at site re for erection. The erection thus became much simpler but the cost of roof was much higher than the original design. A typical cross-sec of the concourse roof as finally constructed is shown in Plate No. VII.

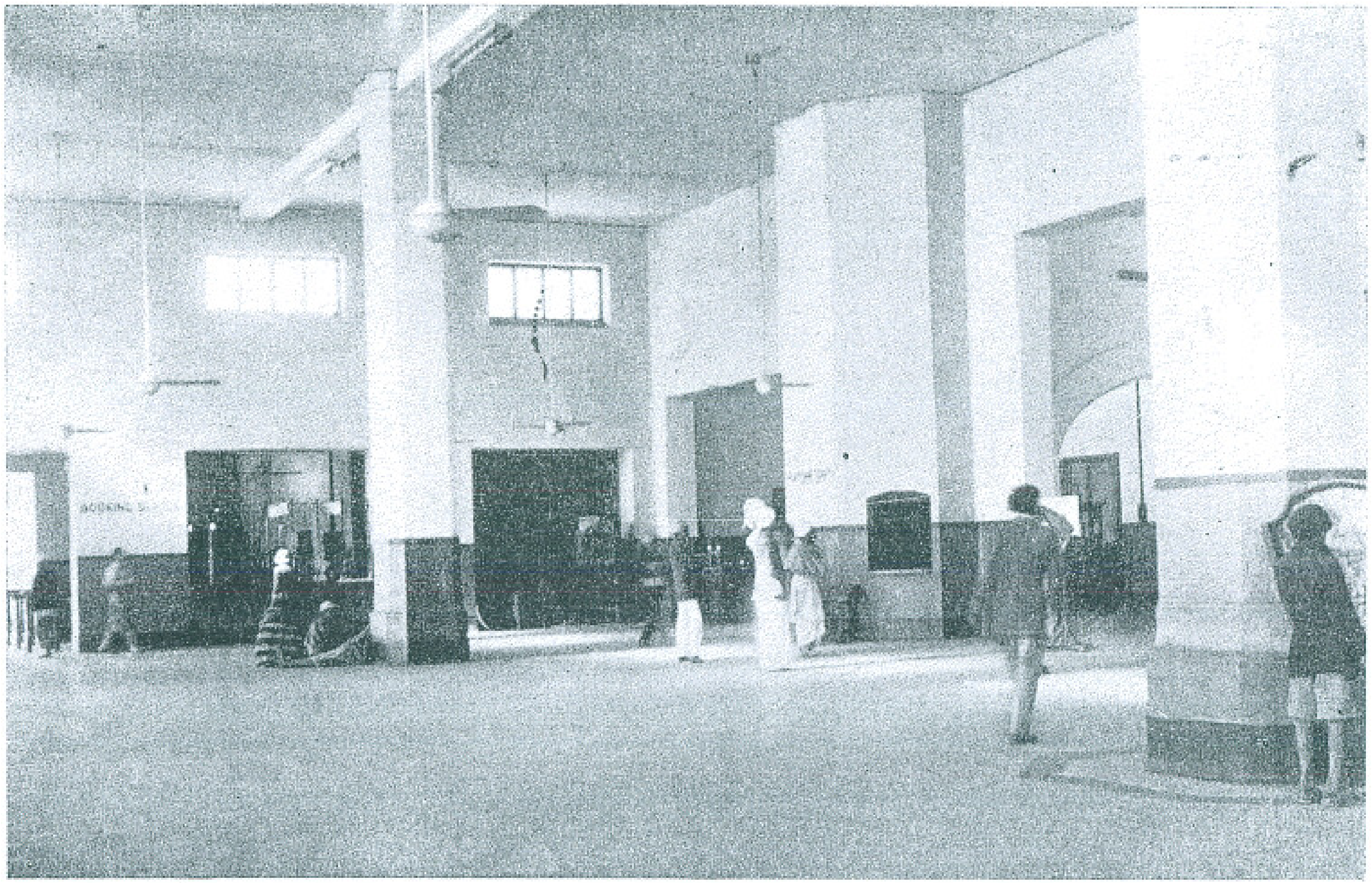
When the concourse roof was in place and arrangements were be made for providing terrazo dado to the walls, the old plaster was remo from the existing brick columns carrying the 40 ft. span lintel in front the foot overbridges. This brickwork was found to be in a rot condition and since the columns carried a long span lintel with a over it, it was decided to remove this rotten brickwork and put in s columns instead. This necessitated careful underpinning for the lintel, a the execution of the work was interesting.

Two steel columns supporting a girder were erected under lintel Hydraulic jacks were then placed in position near the ends the girder, as shown in the sketch, which exerted an upward press of 40 tons equal to the calculated reaction, relieving the load from masonry. Short pieces of R. S. J. S. were then inserted and wedged into position against the lintel, relieving the hydraulic jacks in turn, wh were then released. The lintel being supported, the bedstones of lintel were also supported on R. S. joists resting on columns erected both sides of the lintel. The masonry under the lintel was then disman ed and permanent steel columns put in. Hydraulic jacks were th placed on these columns and an upward force of 40 tons was appl under the bedstones. The 14 ft. long joists were then withdrawn a replaced by short pieces of R. S. J's equal to the width of lintel resting the top of the permanent columns and supporting the bedstones, be wedged tight into position. The jacks were then removed as well as temporary columns and girders under the span of the lintel. T permanent steel columns were then encased in concrete and faced w brickwork in cement, for appearance.

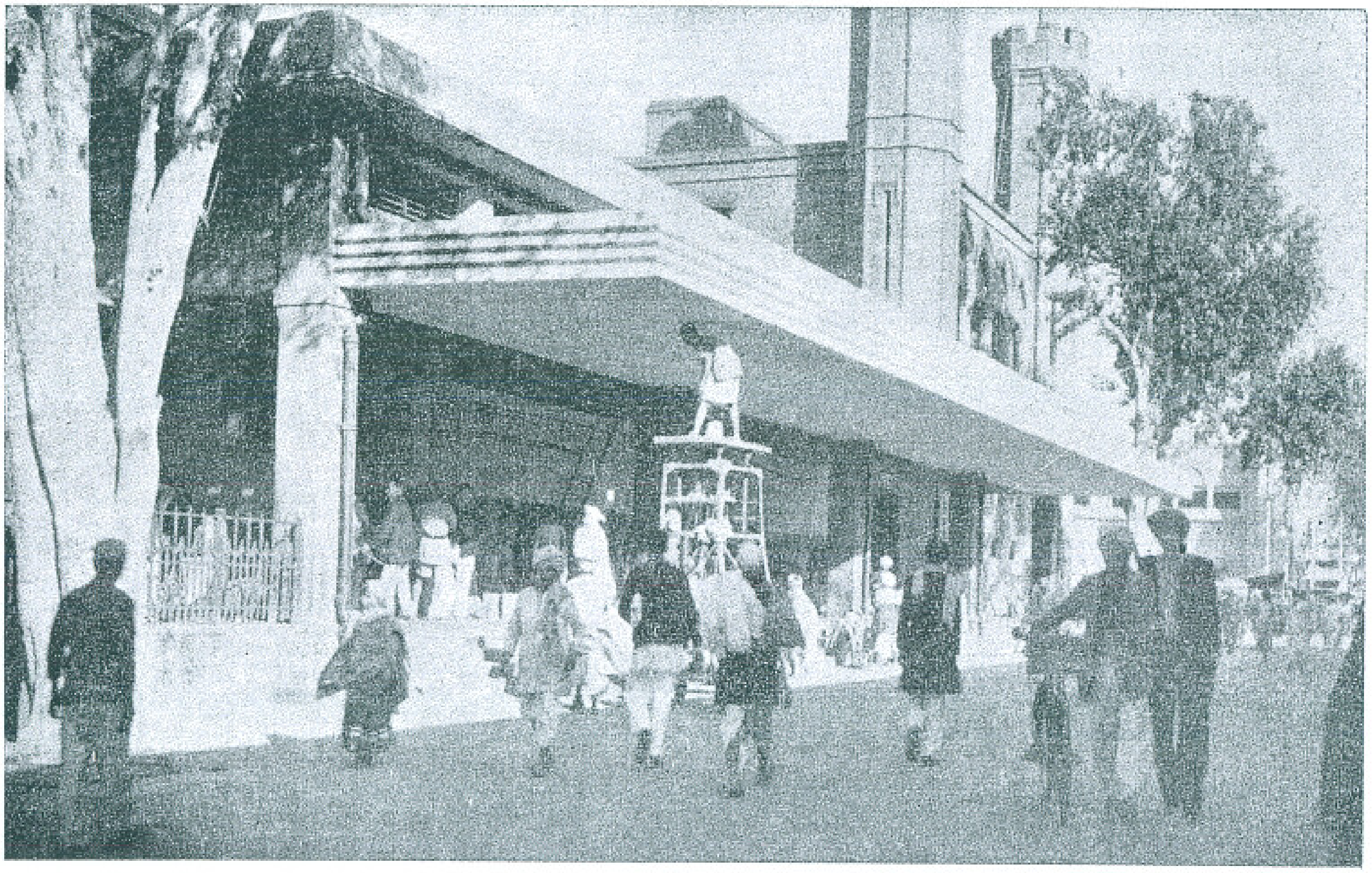
A soon as the load from the new concourse beams came on the 29 span lintels resting on the wall between the concourse and porch, cra appeared in the masonry, at the end of the R. C. lintels, extend upwards and downwards. Investigation showed that the bearing press under the lintels, was within the permissible value for brickwork cement and the appearance of the cracks has not been explained. Th may have been due, to the contraction of lintels, by a fall in t temperature, pulling the masonry with it, inwards. Proceeding on t latter basis, the pillars have been strengthened temporarily by a method hoop reinforcement.

Two plates on either side of the column are pressed together w hydraulic jacks. The plates are joined by tie bars, which we hydraulically prestressed to exert a predetermined compression and we welded into position under stress.

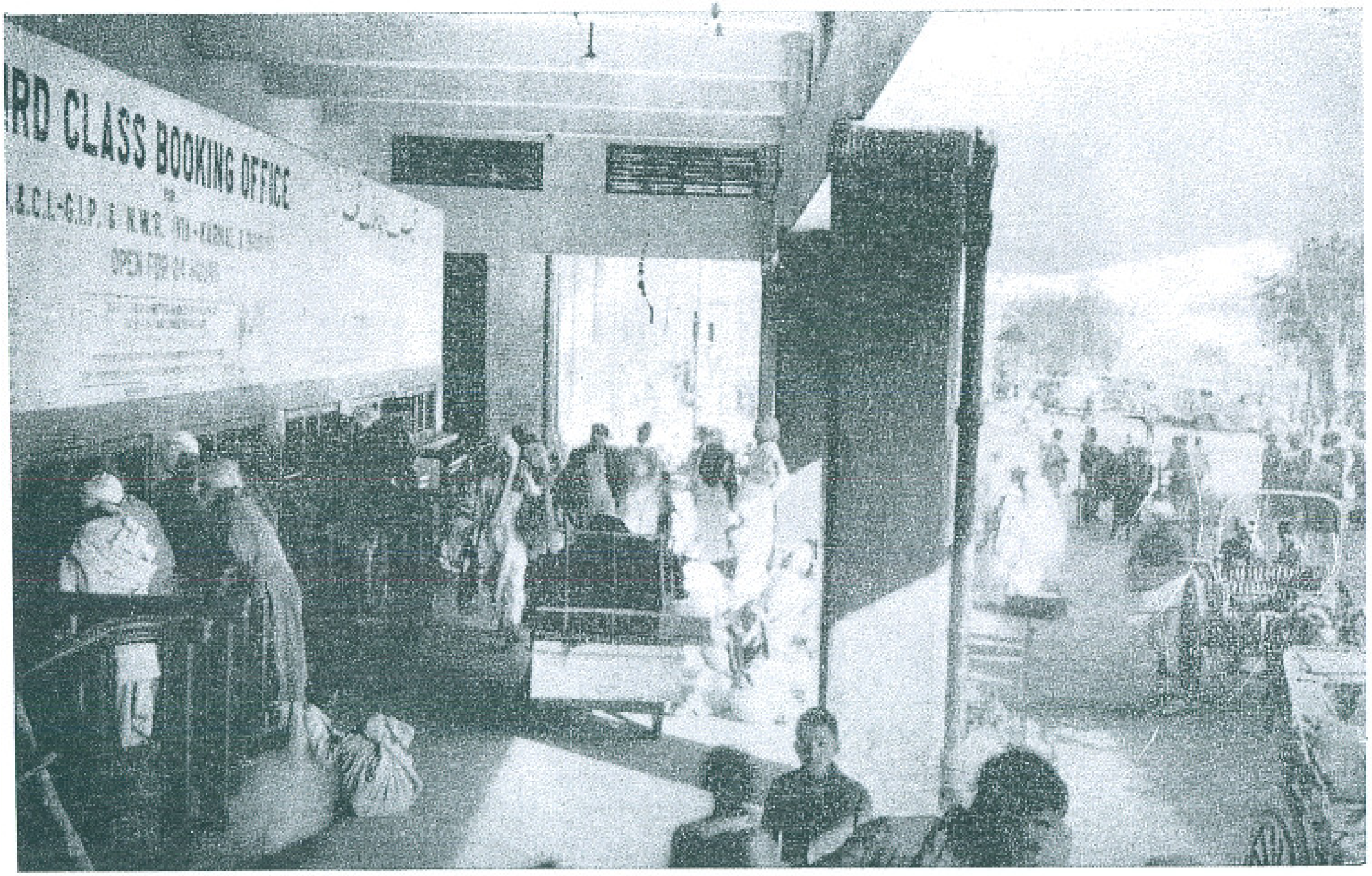
A view of the Porch and Concourse of Lahore Railway Station after Remodelling



Inside view of Concourse at Lahore Station after remodelling.



Another view of Inter and 3rd Class entrance at Delhi Station after Remodelling



Inside view of Inter and 3rd Class entrance at Delhi after Remodelling showing Booking Offices.