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**REINFORCED CONCRETE HANGARS FOR
AIR FORCE STATIONS IN NEW ZEALAND**

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Introduction

Even in 1936, construction of Aerodromes in Great Britain was being pushed ahead at great speed in preparation for war, which even then seemed almost inevitable. In 1937 instructions were received to build aerodromes in New Zealand each capable of accommodating a squadron of Wellington bombers. Two sites were chosen, at Ohakea and Whenuapai, by the Air Staff, under Group Captain Cochrane (now Air-Marshal Sir Ralph Cochrane), in consultation with the Engineer-in-Chief, Public Works Department, represented by the Aerodromes Engineer (Mr. E. A. Gibson, now Director of Civil Aviation).

The writer at that time held the office of Chief Designing Engineer, and received instructions to prepare plans, specifications and contract documents for two hangars at Ohakea. Definite instructions regarding Whenuapai were not received until the Ohakea hangars were half completed as there was some difficulty in final choice of site between Kumeu and Whenuapai. It was known, however, that two similar hangars would be required at the Northern station (incidentally, this was a consideration of some moment, as it was possible to spread the cost of the rather costly steel centering over the four hangars).

As will be explained later, several preliminary designs were made both in structural steel and in reinforced concrete, and combinations thereof, but the choice finally lay with monolithic construction in reinforced concrete.

The large concrete hangars at the R.N.Z.A.F. Stations at Ohakea and Whenuapai, represent a type of some considerable interest. No information of similar hangars was available but after they were designed, information came to hand of very similar (but triple span) hangars at French Air Force Stations at Berre, France, and in Tunisia; there is also reference in German literature to this type being in use by the German Air Force. Again, the U.S.A.F. has used this type in San Diego, California, and Charles S. Whitney proposed the type in 1944 (1).

It is interesting, that in both German and French examples and in Mr. Whitney's proposal also, the spans were approxi-

mately the same as those adopted here. The Royal Air Force has also made use of concrete hangars, particularly for storage purposes, presumably on account of the very low fire risk attendant on this construction; they were, however, much smaller structures than those which are now about to be described.

The reinforced concrete "shell" type of construction for buildings has advanced in favour overseas, and there are numerous examples constructed with concrete shell roofs, two to four inches in thickness. Many of the applications are to small buildings, but one of the most interesting examples, a description of which was published when the design of the concrete hangars for the R.N.Z.A.F. was nearly complete, is that of the Hershey Sports Arena at Hershey, Pennsylvania, U.S.A. (2). The roof of this building covers an area 232 feet wide by 340 feet long, and incorporates arches of 220 feet span. This description afforded much valuable information, and some of the details were very helpful, particularly the use of lead bearing plates at the arch hinges. Certain of the methods of design for shell structures are patented, the most well known being the Zeiss-Dywidag system, which is employed in the design of the Hershey roof.

Shape and Size:

The layout and orientation of the hangars was carefully considered and designed at the same time as the whole aerodrome development, in proper relation to their functional service. Discussion of this aspect of the work is beyond the scope of this paper: in any case a description would not be of much value to the Engineering Profession because the layout of an Air Force Station is a military matter, and is governed very largely by the type of aircraft which it is proposed to use. For instance, the layout for a Station from which fighting aircraft are to operate, would not be on quite the same lines as for a Station where bombing aircraft are to be used. Suffice to say that the problem was solved in close consultation with the Air Department.

The span and depth of the main hangar floor area was decided by arranging cardboard models of aeroplanes for the most economic use of the space. Both end-opening and front-opening hangars were compared and models of several types of aeroplanes, with progressive increases of wing span, as compared with those to be immediately accommodated, were also applied to the design in order that the floor space might be proportioned so as to give the best possible efficiency for all types. The front-opening type of hangar suited the operating conditions, and was adopted in preference to the end-opening type. The

final proportions were 220 feet clear opening, by 170 feet depth, giving a floor area of 37,400 square feet. This main hangar space is surrounded by extensive annex accommodation aggregating approximately 20,000 square feet per hangar, giving a gross hangar floor area of 57,400 square feet (about 1½ acres) per hangar.

The clearance of the main doors was kept to a minimum, consistent with the anticipated development of military aeroplanes, and for these hangars is 25 feet, this point being decided by the Air Force Authorities. In contrast with this clearance, we have the interesting comparison in the case of hangars used for civil aircraft and for seaplanes where clearances of 35 feet and 40 feet are not uncommon.

Type:

While certain special circumstances eventually turned the balance in favour of the reinforced concrete type of hangar, the basic requirements of any construction were:—

- (1) High Fire Resistance.
- (2) Low Maintenance.
- (3) Long Life.
- (4) Proof against Bomb Splinters.

A very interesting review of hangar types was given by Dr. Ing. Mehmel, of the German Air Ministry, in a paper before the Institution of Civil Engineers and the Institution of Structural Engineers in London in April, 1937 (3). It was evident from that paper that the German Air Ministry had definitely departed from the conventional truss type of roof, and that German engineers had even passed the stage of designing roofs as rigid frame structures, and that they almost exclusively made use of the stressed-skin principle of construction such as is used very extensively for aeroplanes. In effect then, we have the most advanced design in steel evolved into a similar type to that used in concrete; and in that material known as the "shell" type of structure. In both stressed-skin type of steel, and the shell type of concrete, the roof covering itself acts as part of the stress-carrying structure. Comparative analyses of all factors are necessary before one can choose as to whether steel construction or concrete construction is the most suitable for a particular example.

In the case under discussion, the condition which finally decided the issue was the availability of reinforcing steel. It was not possible to obtain structural steel except after very considerable delay, and, moreover, it would not have been possible to have placed an order for structural steel until the design was fairly complete. On the other hand, with the use of concrete

construction, an order for about 80 per cent. of the reinforcing steel was placed within a week or so after the design was started, and this material came to hand in such time that no delay was caused by non-delivery of this main material of construction. In a word, the choice between concrete and structural steel is one of economics and expediency; the concrete construction takes rather longer to build, but that is sometimes largely neutralised by the fact that one can order materials early in the design. In a great steel-manufacturing country, such as England, one would expect structural steel to be used for the roof system, but in New Zealand, where steel has to be imported and materials for concrete are plentiful, there is a considerable argument for using reinforced concrete, which contains only 5% of imported steel, and 95% of materials found in the country. Possibly this argument was applied by the French authorities when they decided on concrete as their medium for hangars in Tunisia.

Preliminary estimates for various types of construction gave the following results:—

Type 1.—R.C. Structure; Arches tied at ground level	£
Type 2.—R.C. Structure; Arches tied at "Ceiling" level	67,000
Type 3.—R.C. Walls, steel roof and asbestos cement covering	65,500
Type 4.—End opening, R.C. Construction	63,300
It will be noted that the steel design was not far below the reinforced concrete shell type. Type 1 was chosen.	74,000

General Design:

The tied arch (Type 2) was not used for several reasons, one of the most important of which was that it would be difficult to use movable centering because of the ties and hangar rods, which would make translation of the centering difficult, if not impossible. With such large construction the cost and time that would have been involved if it had been necessary to use centering over the whole length of the arch barrel, or to take down and re-erect centering three times for each hangar, would have been very considerable, whereas with the design adopted, the centering, complete with formwork was erected once only, at the back end of the hangar and was pulled through by winches for each consecutive pour to its new position, and, indeed, was transferred from the first hangar to the second hangar without being dismantled.

This centering, which is shown in the illustrations, included 138 tons of structural steel. The foot of each leg carried a

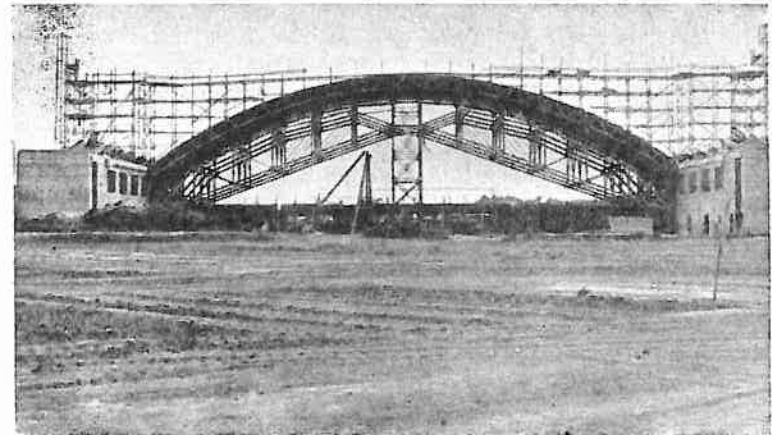


Fig. 1.—Showing centering and concreting hoists, etc.

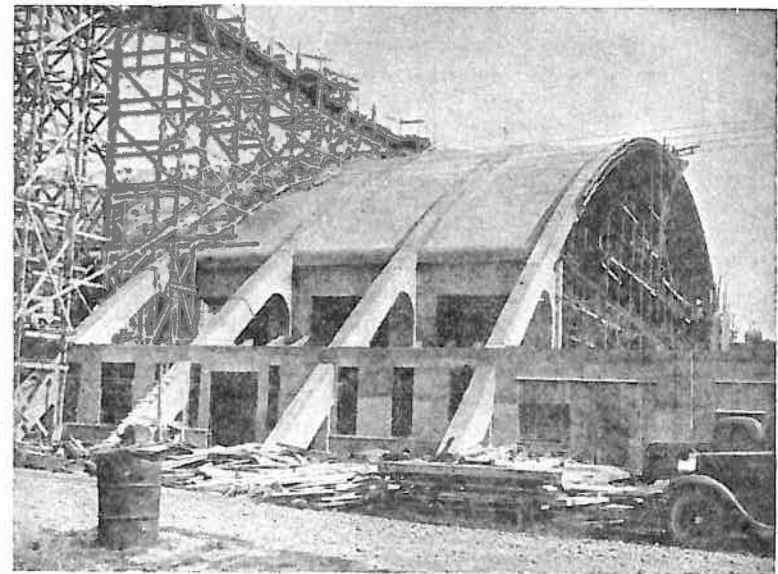


Fig. 2.—Side elevation showing falsework for rear gable and concreting scaffold. This remained in one position throughout.

steel shoe, which moved quite readily along greased rails under the pull of a winch. The formwork, which was supported by the centering, will be more particularly described later. The centering and formwork was first used for hangars at Ohakea, and later transferred for similar construction work at Whenuapai.

Another curious, but important disadvantage of the tied arch type was that it would have been necessary to have enclosed the whole horizontal plane of the ties with close mesh wire netting to prevent access of birds, whose droppings cause considerable damage to the finish of aeroplanes. On the other hand, an economic use of the arch vault is possible with this type of construction, making use of the otherwise waste space above the door clearance level, for storage purposes. This entails the construction of a suspended floor and a hoist, but this offers no great difficulty. Such a suggestion was made to the Air Department for the R.N.Z.A.F. hangars, but it was not called for by them.

Considerations of construction led to adoption of two-hinged arch ribs. This design enabled buttresses (skew-backs) to be built up from the ground each side, and left only the clear span between the walls to be carried by the arch centering. The buttress was initially lined through on the arch pressure line but in the final design it is deflected outside the line of thrust so that there is no possible danger of overturning of the buttress; and a small but definite load is transmitted through the inner column. The design incorporates a tie below the floor level, pre-stressed so as to take a major proportion of the arch thrust. This tie, at Whenuapai, was a considerable problem, because the foundation there was so poor that any attempt to pre-stress the tie would have resulted in the arch footings being pulled through the ground. A solution was found which incorporated a concrete strut between the skew-backs. When the steel tie was pre-stressed by expanding it by heating and inserting a pin, the contraction resultant on cooling, was taken through the concrete strut in compression. The thrust of the arch ribs finally relieved this concrete strut of its compressive stress, and all the thrust from the arch was taken through the steel tie, without displacement of the buttresses (or skew-backs). The pre-stressed ties were a feature of the French and German design also, but round rods were used, whereas the New Zealand design used a R.S.J. for each tie. The rods were pre-stressed during the decentering process, so reducing the amount of movement necessary for the centering, and minimising the bending moments in the ribs resultant from that process.

The arch ribs were first designed to follow the pressure

line resulting from their own weight, the weight of the intermediate slab and a wind load of 20 lbs per square foot of roof. Each rib was reinforced arbitrarily with about 1 per cent. of reinforcing steel near the intrados and the extrados. The reinforced concrete roof slab between the ribs (which were spaced at 20 ft. centres) was also designed to follow its pressure line, and it was reinforced in the ordinary way with the necessary steel to support it between the ribs. The slab is 4 ins. thick haunching down near the ribs, to an increased thickness of 6 ins. The design of this slab was tested at various points along its span, to ensure that sufficient shear resistance was provided between it and the arch rib so that all thrust developed in the slab itself would be progressively and safely transferred into the adjoining ribs. It will be seen from the plans that it was necessary to introduce inclined rods over the lower portion of the slab to augment this shear resistance, and these inclined rods were designed in much the same way as the inclined rods which are commonly used as shear reinforcement in reinforced concrete girders. It was necessary to stiffen the lower edge of the slab between ribs, and the beam necessary to perform this function was cast into such shape that it also forms a rainwater channel. It may be of interest here to remark that an approximate idea as to the necessary size and shape of this channel was obtained by means of a model on which was played water at such a rate as to represent the heaviest possible rainfall. Incidentally, it was found that this concrete construction was by far the cheapest method of making adequate provision for rainwater discharge from this large roof.

The provision of expansion joints at various points in the structure were studied very carefully. These should more strictly be called shrinkage joints, and two such joints were provided which completely separate the hangar and annexes at two planes transversely across the entire building. Moreover, the rear wall is similarly divided by vertical joints at approximately the quarter points. The amount of shrinkage expected in the concrete was only one factor in arriving at a decision as to the number of joints, but a very important one because, of course, on a careful study of this point depended whether or not the building would be free from shrinkage cracks. Further factors were, cost of the falsework for erecting the structure, and the size of the pour, that is to say the quantity of concrete which could be poured within a reasonable working time having due regard for the vagaries of the weather. It was evident that it would be necessary to incorporate at least one joint transversely across the hangar from considerations of shrinkage alone,

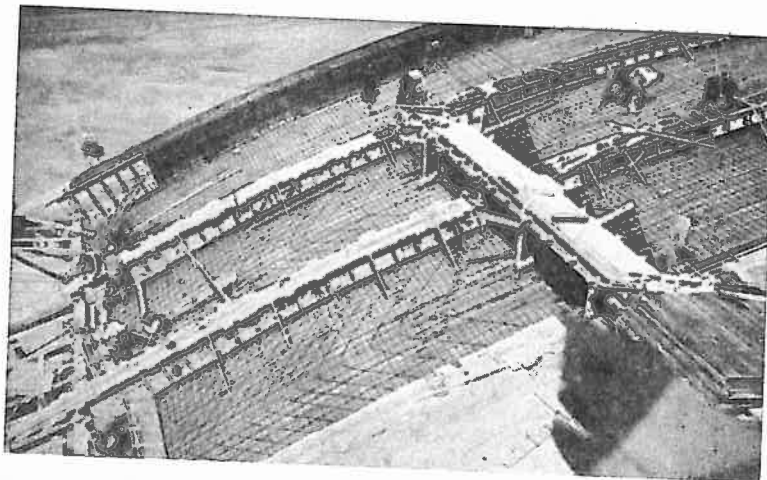


Fig. 3.—View looking down on roof showing reinforcement of slabs in place.

but it was obvious that the extra cost of falsework or centering as compared with that required when two joints were provided would be considerable—in fact 50 per cent. extra, and, moreover, the quantity of concrete per pour would be very considerable indeed, approximately 620 cubic yards. Shrinkage joints at the third points were finally decided upon and with that consideration the weight of steel in the falsework or centering was 138.5 tons, while the amount of concrete to be poured was 413 cubic yards. This concrete was poured continuously, and in general, the time taken was about 24-30 hours. The result has been very satisfactory as up to the present no cracks whatever have appeared and it is evident from the space that exists at the shrinkage joints that these have proved invaluable.

Just about the time that the design was approaching completion a description of the Hershey Sports Arena was published which bore remarkable resemblance at many points to the hangar design. The method of dividing that building at the shrinkage joints was carefully studied as compared with the method adopted for the hangars. For the Hershey building the roof was divided mid-way between ribs, and a special turned-up edge was provided to stiffen the thin slab, whereas the method adopted for the hangars was to split the roof at a rib, that is to say, where sections butt two arch ribs butt. The Hershey method was discarded on account of difficulties which would have occurred at the windows.

The three divisions of the building are each slightly different, the rear section comprising one "rear wall rib" two "standard ribs", and one "half rib"; the middle span comprises two "standard ribs" and two "half ribs"; and the front portion comprises one "half rib", two "standard ribs", and one heavy rib at the front of the building for supporting the gable. As a result of this there was some construction difficulty in getting exactly identical shapes for the three sections because it was necessary to judge the amount of deflection that would occur during construction; but it is considered that the results obtained were quite good, the respective difference in rise of the ribs being very slightly in excess of $1/16$ in. It was, of course, necessary to provide flush joints between the "half ribs" of adjoining sections because of the slight differential movement which took place when decentering. These shrinkage joints are flashed on the top surface, throughout their length, with copper flashings set in lead.

The rear wall of the hangar was designed against wind pressure and suction, and allowance was made at the junction with the arch barrel for the elastic movement of the arch under temperature conditions. It has already been stated that the rear wall was divided into four parts by vertical shrinkage joints. The junction of this wall with the arch rib was made with a grooved joint and pouring holes were left in the arch rib, use being made of 4 in. diameter galvanised downpipes as forms for this purpose. When the concrete wall was finally poured and vibrated so that the grooves were properly filled, a reinforcement cage was inserted through each of these dowels, and they were poured flush with concrete so that they now act as keys between the top of the rear wall and the arch rib, while allowing relative movement.

The front gable of the hangar is sheathed with steel sheets having a proprietary covering of bitumen and asbestos fibre, with aluminium paint on both surfaces. The sheathing is fixed to light steel channels, which run horizontally, and which are themselves attached to the vertical concrete-covered ties, which depend from the arch rib. These ties or hangers support at their lower end the outer flange of the wind truss, and the steel tracks on which the upper edge of the main doors are guided. The wind truss does not call for particular comment; it is of conventional design of structural steel and lies in a horizontal plane, its inner chord being supported by steel hanger rods depending from the second arch rib. The ends of the truss are supported against concrete bearings in the walls, so transmitting the thrust along these walls to the ground. The lower surface

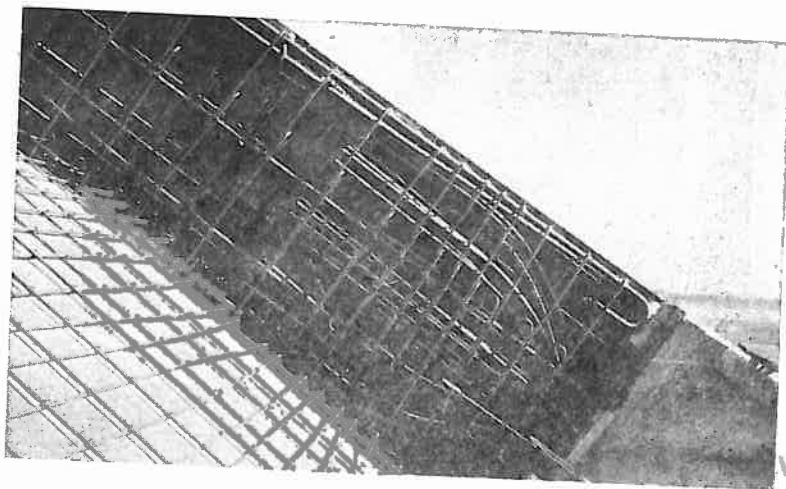


Fig. 4.—Mesnager hinge of front rib.

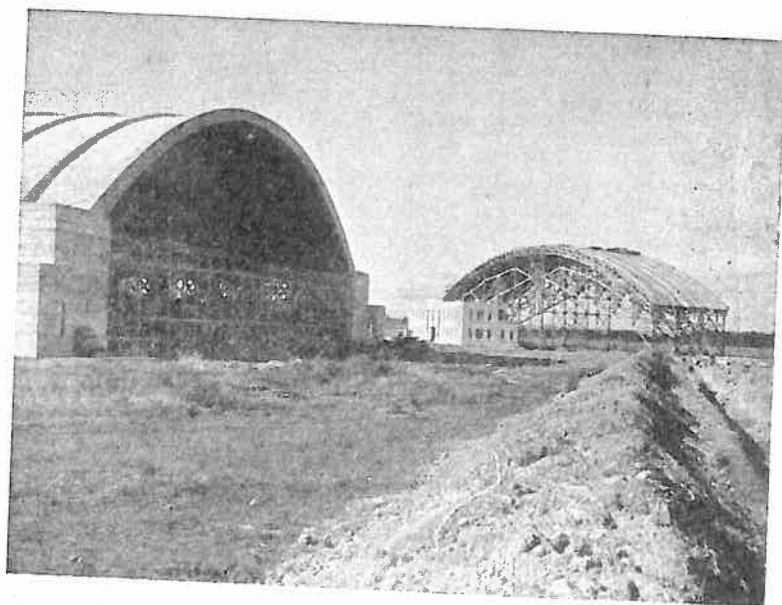


Fig. 5.—First hangar complete and centering being shifted by winches to No. 2 site.

of the truss and the vertical face of the second row of steel hangar rods is completely covered in with fine mesh wire netting as bird-proofing.

Doors:

The weight of each main door is supported on two wheels, one at each lower corner, and the doors are kept in a vertical position by means of guide rollers at the upper corners. All wheels and rollers are mounted on ball or roller bearings. One of the lower wheels is driven, being fitted with a roller chain geared down to a sprocket shaft arranged for manual operation. Each door leaf is 25 ft. high x 23 ft. 6 in. long. Their design was largely influenced by the requirements for splinter proofing against bombs. For hangar doors this is usually provided by steel construction, but owing to the difficulties attending the supply of steel, in this case reinforced concrete doors were adopted. The doors are 10 in. in thickness, and doubly reinforced with a heavy mesh of steel to a height of 15 ft. above ground level, above which level a window, glazed with wired glass, is provided. This construction may be compared with what is usually supplied in structural steel, viz; outside plate $\frac{1}{2}$ in. thick, inside plate $\frac{3}{4}$ in. thick, spaced 10 ins. apart and filled with ballast. The wheel assemblies were properly set up, and each door was cast in place, the total weight of each leaf being about 27 tons. In one hangar the doors proved difficult to traverse, but this was traced to inaccurate rail setting; when these were ground true, no further trouble was reported.

The floor was cast in 4 in. thick concrete in 10 ft. squares, alternate sections being cast so that excessive shrinkage at joints would be avoided. The floor was finished to grade to drains at each side of the hangar and along the front inside the doors. The hangar apron is similarly constructed, and it has a drain also along its outer edge. All openings into the main hangar space from the annexes can be closed by means of automatic fire doors constructed in accordance with the N.Z. Fire Underwriters' specification.

The annexes which surround the hangar on three sides are subdivided into a series of units for services connected with the operation of the Air Squadron. These services are inter-related in a very complex manner, and a considerable amount of planning was required before a satisfactory arrangement was reached. There was nothing of particular interest in the structural design of the annexes save that the rear annex is 40 ft. wide, and carries R.C. roof beams having a clear span of that amount.

It is beyond the scope of this paper to discuss the subdivision of the annexes but it may be mentioned that the various services of air conditioning, heating, water supply, sewerage, lighting of various kinds, etc., which were necessary, provided in themselves, quite complex problems. The heating of the annexes is carried out electrically. The lighting of the main hangar is by means of mercury vapour lamps.

Construction:

These hangars have been constructed at both Ohakea and Whenuapai, and in view of the very interesting foundation problem which was presented at the latter Station, it is proposed to describe the construction at Ohakea, and then to give some information as to the variations which were necessary at Whenuapai.

The hangars at Ohakea are founded on a very tight shingle overlaid with clay. The bearing power of the shingle was excellent and offered no difficulties; in fact, it is possible that the arches could readily have been supported by the country without the assistance of the pre-stressed ties which were provided. These were actually supplied having in mind the possible effect of an earthquake. The excavation for the underground steel joist ties between the rows of footings was a very dirty job, as a considerable amount of rain fell, and the clay became very sticky indeed so that it was not at all easy to keep the trenches, in which the joists were laid, to a reasonable shape. As the footings were cast the rolled steel joist ties were laid in position, being fitted at the middle of the span with a dummy plate with holes drilled, so spaced that the gap for pre-stressing would be kept constant. The footings were then concreted up and the dummy plates removed so that the joists could move as called for by the dictates of temperature. The pre-stressing of the ties was carried out quite readily, the Contractor employing flame throwers in the web of the joist. This proved to be a very fast method indeed, about eight or ten minutes being sufficient time to expand the joists the necessary $1\frac{1}{8}$ ins. for insertion of the pin, such that on contraction they would be pre-stressed to 104 tons. It is the writer's opinion that this pre-stressing could have been done quite as readily, and more cheaply, by placing some cotton waste saturated in fuel oil along the trough of the joist from end to end. When one considers that a temperature rise in the metal of only about 106° was necessary, it seems hardly necessary to resort to such special devices. This particular feature of the job, however, caused some uncertainty in the minds of contractors; it was something quite new, and they

found it rather difficult to believe it was quite so simple. The junction of the tie beams was made by means of a steel pin which was dropped into place as the pin plates came together with the expansion of the joists.

Vibrated concrete was used throughout the hangar construction, except in the actual roof slab where the slopes were so steep that vibration would have caused the concrete to slump away out of place. The vibrators employed by the Contractor were of two types, the electrically driven surface vibrator, and the spud type immersion vibrator. The surface vibrator was found of particular use in the early stages of the work. It was used during the casting of the concrete cover round the steel tie beams. Small holes were drilled in the tie beam webs and the concrete was conveyed to the underside of the beam. The action of the vibrator was such that the concrete flowed smoothly, and fully against the underside of the web as was indicated by the grout oozing through the small holes. This vibrator, however, was found in general to be too severe on the boxing for use in walls, and the Contractor made use almost entirely of the immersion vibrators, which were portable units, each driven by a small petrol engine. The mix was designed on the basis of Walsh's grading curves, and the result was a very high class, dense, concrete, throughout the work. The specification called for $4\frac{1}{2}$ bags (560 lbs.) of cement per cubic yard of finished concrete, with a minimum field test of 3,000 lbs. per sq. in. on a standard 12 in. x 6 in. cylinder at 28 days; works tests averaged 3,800 lbs. per sq. in.

A considerable amount of thought was given to the type of formwork which should be called for, and the specification set out alternatives for steel forms, timber forms, timber forms lined with prepared form paper, timber forms of plywood. The Contractor first chose to use timber forms lined with prepared form paper, but this gave results that were not very satisfactory. Due to the wrinkling of the paper on becoming dampened by the concrete, the surface was often disfigured, and although many attempts were made, it was found almost impossible to avoid these worm-like wrinkles on the surface. The use of form paper was therefore discontinued, and the Contractor used timber forms machined to even widths and put together with great care. As a result the concrete surface is of a very high class, but not so perfect as would have been attainable with the use of plywood forms. On the Whenuapai hangars the timber forms were used, machined to even widths, and it was specified that the horizontal lines of the forms should join up between shutters and, moreover, the horizontal construction joints were

concealed by forming at this point an inset or groove about 2 in. high x 1 in. deep.

During the construction of the annexes the steel falsework was being erected between the walls. It had been specified that each pour of concrete, between hinges, should be carried on in one continuous operation, and it was therefore most desirable that good weather should be experienced. The Contractor therefore kept in close touch with the Meteorological Office, and adjusted his times of pouring according to the advice of that office. Even so, on two occasions he was badly caught by a thunderstorm and some trouble was experienced with water on the roof slab. The ends of the main arch ribs are carried on Mesnager hinges, which are clearly shown in the photographs. The steel-bender on this job was a very fine craftsman, and the hinges throughout the length of each wall were in perfect line; this, in fact, applied throughout the work in regard to the steel bending and placing. The photograph shows fairly clearly the steel tray which contained sheet lead against which the concrete was poured. This detail was largely developed from that shown for the Sports Arena at Hershey, and acknowledgments are made to the Engineers of that structure. The number of rods, their angles, etc., for the Mesnager hinges was determined, and although at first sight the problem of placing them seemed to be a formidable one, nevertheless everything went much better than had been expected. Neither at the decentering nor since that time has any perceptible movement been noticed at these hinges. In any case, the angle of rotation of the hinge during decentering was computed to be 1 min. 29 secs. of arc, which, of course, is quite a small movement.

The steel falsework was of conventional design, and comprised two sets of inclined trusses bearing at their inner ends against a steel tower. The outer end of each truss, and also the legs of the tower were equipped with steel shoes which thrust against steel rails laid along the length of the main hangar. There were four trusses in line, and they were braced together in two directions to form one complete whole. The upper portions of the verticals of the trusses supported 25 ton jacks which were provided for the purpose of adjusting to proper profile the wooden formwork which was supported from the steel centering. These jacks were used for de-centering over the arch ribs and intermediate jacks of 5 ton capacity were used for de-centering the slab formwork. The information given in regard to the construction of the Sports Arena at Hershey was again drawn upon in regard to the proposed method of keeping control of the arch shape and a number of measuring

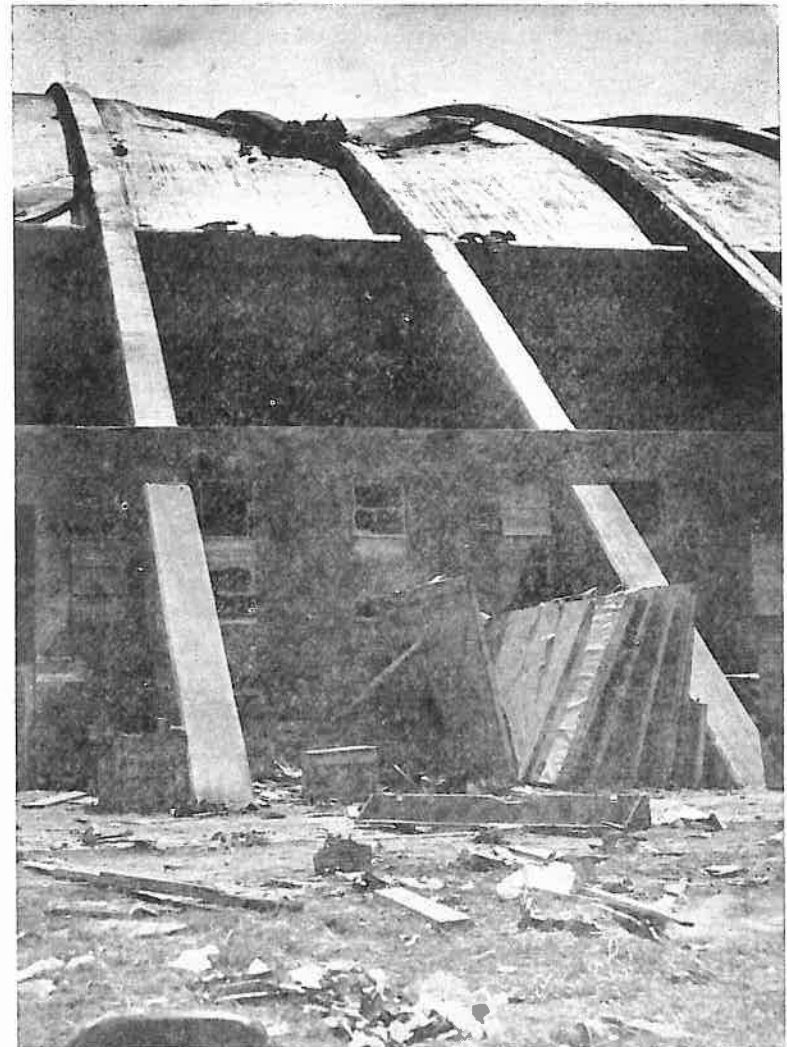


Fig. 6.—Side elevation showing damage to roof.

points were installed along the span of each steel truss. An ordinary steel tape was suspended from each point and loaded with a weight at its lower end, a post being set in the ground, and provided with a scale against which a pointer, fixed to the steel tape with a thumb screw, enabled deflections to be read directly. Two tapes of Invar steel were used as master tapes for correction of temperature extensions of the other tapes. According to the description of the Hershey Sports Arena construction, as the arch centering deflected from its true shape under load, men were sent up to adjust the jacks so as to bring the arch curve back to normal. This was found very difficult to carry out in practice and it was finally discontinued. In the first place it was objectionable to call half-a-dozen men down from the work of concreting, and in the second place, as the concreting proceeded overnight, it was considered to be quite a dangerous task for men to be clambering over the steel members in the darkness. A careful watch was kept on the deflections and generally speaking, the deformation of the centering was very largely that which one would expect from deformation of the simply supported steel trusses. It was considered after study that the amount of deflection was not such as would give rise to any serious condition in the arch ribs, the depth of which is considerable compared with the movements noted (The arch ribs are 2 ft. 0 in. deep at the crown, and 2 ft. 6 in. deep at the springings at eaves level. The movements were of the order of one inch).

The remaining sections were cast very true to shape by taking advantage of the initial information to set the centering above its final form by the amount of deformation that had been noted for the first section.

The first section of roof was cast without top forms and no great difficulty was experienced so long as the concrete was kept very stiff (about 1 in. slump) for the lower portions of the roof. Unfortunately, however, a heavy thunderstorm occurred, and a good deal of the concrete was washed down into the gutters, and set the Contractor back considerably. Due to this difficulty and also on account of the difficulty of screeding the surface of the steep slope of the roof, future sections carried top boxing for about 30 ft. from the hinges.

The roof slab was not vibrated, but the action of treading in by the heavy boots of the workers was adequate to give good concrete. The concrete in the arch ribs was vibrated. The top surface was screeded by means of a heavy plank attached to two ropes, which was raised and lowered along the screeding strips. A good surface was obtained by following up with hand

trowelling. Concreting was carried on symmetrically from each side of the arch until finally a junction was made at the crown. The Contractor set up his construction trestles on both sides of the hangar, and had a walkway for the men with the concrete skips right across the structure. This walkway was supported directly on the arch centering, but when the centering was withdrawn, it was then supported by the arch itself, and remained in position so as to serve for the whole of one hangar. The casting of one section was usually begun about 5 o'clock in the morning, and carried on all day and throughout the night, and finished the following morning, the men working in shifts. The concrete was cured by water service; it was kept thoroughly damp for seven days. Decentering was carried out first of all for the slabs, the forms for which were released by lowering the screw jacks which held them. The forms did not come away readily, but had to be stripped off progressively from the springings upwards. There was, however, no real trouble because incorporated in the formwork was a device by which a downward pull could be imposed on the form. When the slabs were completely self-supporting, the operation of de-centering the main ribs was proceeded with, and caused no difficulty. Sufficient draw or taper had not been allowed in the design of the ribs to enable the side formwork of the slabs to leave the concrete readily. This was remedied for the other sections by lining with several thicknesses of building paper, but the clearance was still rather on the tight side. In this connection, a rather interesting fact is worthy of record. In order to speed up the work, rapid hardening Portland cement was used for the second and third sections of the first hangar, the concrete for the first section having been made with ordinary Portland cement. When an attempt was made to de-center the slabs, and the side boxing of the arch ribs, it was found that the swelling of the timbers was such that this was not possible and it was necessary to wait nearly as long as would have been necessary with ordinary cement concrete before de-centering could be carried out. This is a point which should therefore be carefully watched in the design of the boxing by anyone who wishes to use rapid hardening cement, and ample taper provided.

As the falsework was drawn through to its next position, the electricians worked on the installation of the main lights in the hangar roof, and the Inspectors carefully looked over the finish of the interior surface. In order to facilitate this inspection together with any cutting out or patching which was necessary, a walkway was provided along the rear end of the falsework at such a height as to be convenient for men working

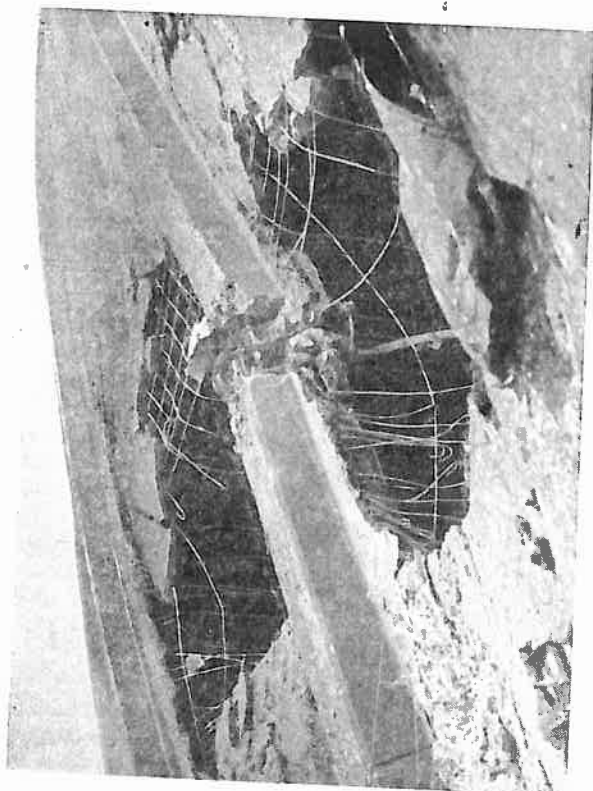


Fig. 7.—Damaged roof slab and arch rib.

on the underside of the roof structure.

During the roof construction a very heavy gale occurred with wind velocities recorded at 70 m.p.h., and during this time the Clerk of Works estimated the effect of gusts on the roof to be such that the deformation at the quarter points was about $\pm\frac{1}{4}$ inch.

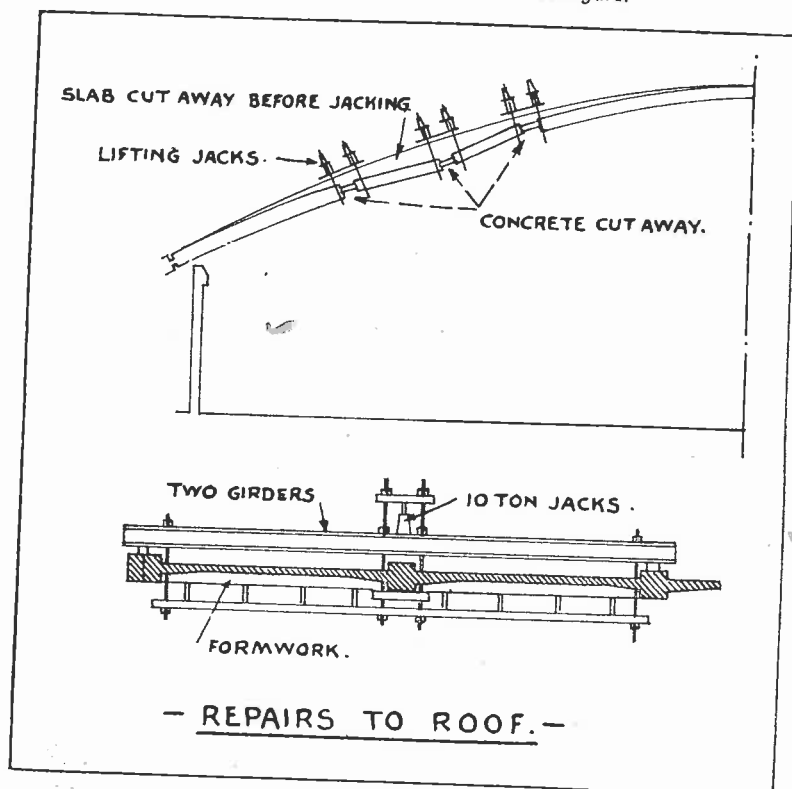
Special Notes on Construction at Whenuapai:

The site for the hangars at Whenuapai presented grave engineering difficulties to the construction of any permanent structure. Test pits which were sunk to determine the possibility of spread footings, showed that underneath poor clay a layer of black vegetable matter occurred, varying in thickness from 6 ins. to 6 ft., and varying in depth below the surface from about 6ft.

to 9 ft. Below this layer, the material was a soft grey pug of the consistency of soft putty, to a depth of 50 to 60 ft. below ground level. Careful samples of all these materials were taken by impressing them into a sharp-edged sampler which on withdrawal was sealed at each end with a heavy coating of paraffin wax. These sealed samples were then forwarded to the Engineering School at Canterbury College, and were carefully examined and analysed as to their physical properties. It was evident that it would be impracticable to found a building above the black vegetable material because of the variable thickness of the stratum and the variable depth at which it occurred. In any case the bearing properties of the overlying clay were very poor indeed. The properties of the grey pug were examined and the bearing capacity was estimated at about 300 lbs. per sq. ft.; moreover, the movement with variation of water content was very considerable (a small sample scooped into the lid of a small tin $1\frac{1}{8}$ in. diameter and placed on the office desk shrunk on drying over $\frac{1}{4}$ in. on the diameter). It was evident therefore that foundations for any permanent buildings would have to be taken to the layer of dense sand which occurred below the grey pug at 50 to 60 ft. below ground level. No other site was available, and this procedure eventually had to be carried out at considerable extra cost. Under the main footings, concrete cylinders were sunk, the excavation being carried out by open grabbing. No particular difficulties were found in sinking the cylinders, except after rain when they would hold up quite considerably due to the considerable expansion of the country against the outer walls. Concrete piles were driven underneath the more lightly loaded portions, and it is significant to note that it was quite usual for a pile to be driven to its full depth of 50 to 60 ft. in about 7 to 8 minutes. The excavation at the base of the caissons was belled out to a diameter of 7 ft. There was no difficulty with water, and the base was sealed up with a concrete plug for the necessary depth. The footings were then constructed over the tops of the cylinders, in similar fashion to construction at Ohakea.

The only other point of special interest at Whenuapai was the provision of a concrete trench around the rolled steel joist ties. This has already been mentioned, together with the reason for its adoption. Actually, however, quite apart from the structural necessity for this trench, the very great advantage of cleanliness was such that the extra cost to the Contractor was comparatively small, compared with the cost of timbering and maintaining open trenches in that country.

The procedure followed at Whenuapai in regard to prestressing was somewhat similar to that used at Ohakea, but the

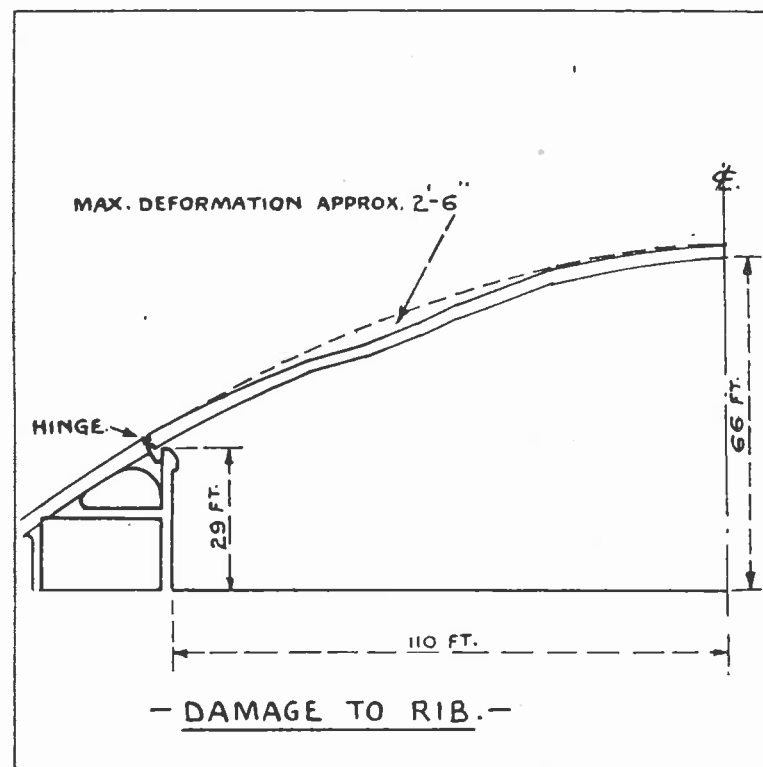


amount of extension which was necessary was $2\frac{3}{8}$ inches, as against $1\frac{1}{8}$ inches at Ohakea, because, in this case no reliance whatever was placed on the carrying capacity of the concrete footings and cylinders against lateral thrust. After connecting the tie by the insertion of the pin, the cooling and consequent contraction of the steel gave rise to a compressive strain in the concrete strut amounting to 0.3 ins., over its whole length, indicating a value of Young's modulus for the concrete of about $2\frac{1}{2}$ million lbs. per sq. in. (i.e. $n=12$).

Costs:

The cost of the Ohakea hangars was about £76,750 each, while the Whenuapai hangars, due to advances in costs of construction, and to difficult foundations, cost approximately £103,000.

These costs include very considerable sums to cover cost



of aprons, concrete floors, also workshop cranes and dope shop, ventilating plant and other special equipment.

The cost per square foot of nett area may not be of much value for estimating similar buildings, because of the special equipment and conditions, but for what it is worth it is given below (Total area hangar and annexes—57,400 sq. feet.).

Ohakea	£1.33 per sq. ft.
Whenuapai	£1.79 " " "

Damage to Roof Whenuapai Hangar:

During the war an aeroplane under test crashed into one of the hangar roofs. The engines fell through onto the floor, and the 1400 gallons of petrol exploded in the hangar. The force of the explosion blew out the front gable sheathing, and some of the windows, flames extending about 40 feet out of the building.

A rib was completely fractured about 30 to 35 ft. above the

hinge, the damage extending over about 50 ft. of the rib. The slabs on either side were shattered to the extent of 1,000 sq. ft. around two holes about 10 ft. in diameter. The maximum deformation of the damaged arch rib was about 2 ft. 6 ins.

The roof generally, however, suffered no damage, the structure adapting itself to the new conditions of loading with small deflections.

The concrete was cut away completely at the rib fractures, and the sound sections remaining were jacked back into line by an arrangement as shown, the whole then being concreted against formwork suspended from R.S.Js. bearing against the adjacent ribs. An internal steel scaffold was used to facilitate the work.

Undoubtedly, an ordinary trussed type of steel hangar would have been completely demolished by this very severe happening, and it is a most impressive object lesson of the very great latent strength of this type of construction.

Acknowledgement:

Acknowledgement is made to the Air Secretary and to the Engineer-in-Chief for permission to publish the foregoing information; also to Mr. J. T. Gilkison and Mr. Percy Laing who were in charge of construction at Ohakea and Whenuapai respectively, and who supplied notes on construction.

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DISCUSSION

MR. A. D. MEAD said the design of military structures presented novel problems possibly all the more difficult because similar works constructed abroad were usually kept fairly secret. Mr. Turner and his staff were to be congratulated on their excellent solution.

Mr. Turner had apparently treated the question of wind load in a somewhat casual fashion, mentioning 20 pounds per square foot. Was that the horizontal wind pressure? Was it applied only on the windward side or all over the roof? Wind loads on ordinary pitched roofs created a pressure on the windward side and a suction on the leeward side usually greater than the pressure on the windward side. That introduced rather curious stresses into the ordinary type of roof truss. The wind pressure of 20 pounds per square foot was quite a substantial proportion of the total load. For a 4 inch slab the dead load was 50 pounds per square foot. If there were a suction on the lee-

ward side there would be quite considerable bending moments in the rib during high winds, and it would have been interesting if Mr. Turner had added a wind pressure diagram and the resulting bending moment curve. Probably with the large size of the rib the 1 per cent of reinforcing steel that he had allowed would cover this. If one were to attempt to build a hangar of these dimensions out of light lattice steel work with a light roof covering instead of a solid slab, one might find the wind pressure bending moments form quite a considerable problem.

MR. I. R. ROBINSON asked was the wind loading of economic importance or did the fact that it was necessary to design for splinter proofing make the wind loading unimportant?

The shape of the hanger viewed from the front and from the sides was very unusual. Did Mr. Turner know if any experiments had been carried out in wind tunnels on what the wind pressure was on such shaped structures? It appeared that the use of wind tunnels for such purposes would become of increasing importance in the future.

MR. J. F. BRUCE commented that Mr. Turner had passed over rather hurriedly the point that it was not possible to obtain structural steel. That point was much in evidence in regard to Rehabilitation construction. No matter how important or urgent a work little could be done until the designers came to light with the quantities and specifications.

In the case of structural steel work it was impossible to make a very successful forecast of steel requirements, whereas with reinforced concrete one could quickly make an estimate of the steel required and gain a very close idea of the approximate amount of steel required from 1 inch diameter down.

MR. C. R. DAVIS, saying that Mr. Turner had stated that the roof slab was not vibrated on account of its tending to slump out of place when there were no top forms, but that later top forms were put in place, asked whether the later work was vibrated.

MR. R. A. CAMPBELL, congratulating Mr. Turner on his very fine design, said the saving in industrial buildings by adopting the arch form of design whether in steel or concrete, was not always appreciated. The cost had been given as £76,000. Did that include the cost of design, and did it include a proportion of the cost of the form work or not?

Mr. Campbell sought clarification of Mr. Mead's point about the 20 pounds per square foot wind pressure. Recent investigations in wind tunnels had proved that a structure of the form of this particular hangar would result in suction over the whole of the roof, not only on the lee side. When designing certain hangars the speaker had been in touch with an engineer

from the Old Country who had the latest figures on wind tunnel experiments of the National Physical Laboratory at Home. They had proved quite definitely that in structures of that kind there was no pressure on the roof at all. When designing these hangars he had arranged with Professor Calvert to make some metal models which he put into his tank at Canterbury College and two drawings were made showing the flow over the top. Over an arched surface there was a beautiful smooth flow. With two flat surfaces at an angle like the conventional pitched roof the flow was fairly smooth about half way up the windward side then a mass of eddies all the way across. He would like to know if Mr. Turner took pressure at one side and suction on the other, and if so how much.

MR. TURNER replying, said that the wind pressure was assumed to be from one side only and to give a pressure on the one side and a negative pressure on the lee side. He did not know of any wind tunnel experiments made at the time on this specific type of structure. There had, of course, been experiments on the more conventional gable type and from the results of those, modified to suit the curved shape, the wind diagram was drawn. It was certainly approximate but the magnitude of the wind stress was comparatively unimportant because of the considerable dead weight of the roof. The curved shape of course gave considerable reduction as it did in structures generally. With curved surfaces it had been used for many years to take $\frac{2}{3}$ of the projected area as effective compared to a flat surface. A similar sort of reduction was made in this case.

With regard to Mr. Robinson's question, splinter proofing was carried only to a height of 25 feet and it did not affect the roof design at all. As a matter of interest the splinter proofing was designed to resist a 500 lb. bomb exploded at 50 ft. distance.

In reply to Mr. Davis: In the first place the roof was cast without top forms. But what put the contractor right off this procedure was a heavy thunderstorm when he had partly finished the first section. There was a terrible mess. After that he provided top forms for about 30 feet or so from the eaves. Concrete was vibrated for the whole job, except for the roof slab which was very thin.

As to the cost of designing: The total cost was £76,750 for the Ohakea hangers and for Whenuapai £103,000. The latter were founded on caissons and that accounted for the extra cost to some extent although at that time prices generally had risen between 5 and 10 per cent. perhaps about 10%. The Departmental services for design and construction in the case of Defence works were a 5% charge on the total cost. The costs as given included this Departmental charge.