

CANADIAN FORCES EXPERIENCE IN SLIP-FORMING AIRFIELD PAVEMENTS

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A. THE COLD LAKE PROJECT

Introduction

In October 1971 the decision was taken to construct a new 10,000 ft. x 150 ft. runway at Canadian Forces Base Cold Lake, Alberta. The requirement was generated by a planned re-structuring of the Canadian Forces pilot training program and its completion immediately became an urgent operational priority. Unfortunately due to financial and other constraints it had not been possible to carry out any preliminary engineering surveys or advanced planning. In short, a typical "rush job".

Preliminary Engineering

A basic engineering layout, illustrated in Figure 1, was hastily completed and No. 1 Construction Engineering Unit was tasked to carry out the necessary topographical, foundation soil and construction material surveys. A mean task under the winter conditions then prevailing. The site was covered with dense light brush and by late October with several inches of snow. The survey was carried out by bulldozing the brush and snow ahead of the survey crew. Foundation soils were sampled concurrently to stay ahead of the advancing frost line which penetrates anywhere from 7 to 10 feet during a typical winter at Cold Lake.

While the field work was in progress design alternatives and cost estimates were studied and the construction and performance histories of existing pavements were reviewed. The first 8,000 ft. of Runway 12 was constructed in 1954. It was extended to 12,600 ft. in 1958-59. The construction consisted of 3-1/2" of HMAC (Hot Mixed Asphaltic Concrete) on a 9" crushed gravel base course on 48" of pit run granular subbase for a total depth of 60-1/2 inches. It was designed by the Norman W. McLeod or MOT method in which both the spring reduced subgrade strengths and the requirements for frost protection are considered. In this instance the total depth was determined by the requirements for frost protection. Frost does penetrate the subgrade but there is sufficient overburden to prevent serious heaving or distortion of the surface. The riding qualities of asphaltic portions of this runway over the years had been fairly good. In 1968, six years after the first overlay the Riding Comfort Index was 7.2, not a bad ride. The roughness of the rigid pavements was another story, the RCI being about 5.4.

Periodic surface plate load testing verified that the bearing capacity of the existing construction exceeded design values and was more than adequate. Its LCN exceeded 100. Subgrade and infield drainage were considered good and there was no evidence of serious differential frost action.

From the operational and performance point of view the main problem lay in the asphaltic surface itself. It had weathered poorly and had become very extensively cracked at an

early stage in its life. By 1962 the surface was generating a considerable amount of debris which is a serious and unacceptable hazard to high performance Military aircraft. These conditions led to the application of a nominal 2" HMAC re-surfacing in 1962. Slurry seals were applied in 1966 and 1969, followed by another 2" HMAC overlay in 1972. As would be inferred from the foregoing the performance of the 1962 overlay was no better than that of the original surface. Virtually all cracks in the original surface had reflected into the overlay within two years of its placement. The reflection cracking was not unanticipated but the rapidity of its development and rate at which the cracks continued to widen with age were not fully anticipated.

There is little doubt that the problems experienced with the asphaltic surfaces at Cold Lake were due to deficiencies in the original surfacing. In the era when they were constructed, Engineers designing mixes tended generally to concentrate too much on laboratory Marshall stability and too little on other important aspects such as VMA, air voids and insitu densities etc. all of which have a major influence on durability. Another factor about which very little was known at that time was that certain asphaltic cements, particularly some of those manufactured in Western Canada, were extremely prone to low temperature thermal shrinkage cracking. Remaining records do not identify the source of the cement used in the original construction but from its field performance it would certainly be rated as a "bad actor". We could construct a more durable HMAC surface today that would be much less prone to low temperature cracking but we could not state with confidence that it would not crack. The climate at Cold Lake is severe with temperatures of minus 40 degrees C being quite common during the winter.

In reviewing the performance of existing rigid or PCC pavements at Cold Lake most of which were seventeen years old it was observed that, with the exception of some localized joint spalls, the occasional pop out and the odd random crack, the pavements showed no evidence of any serious structural distress or physical deterioration. Virtually no maintenance had been done on these pavements. Their main deficiency from a performance point of view lay in their riding qualities. All of them would have to be rated as unacceptably rough for runway service. This problem was not unique to Cold Lake. It has been our general experience with PCC pavements all across Canada. The riding qualities of any pavement will of course deteriorate with age but the problem was that we had not been achieving good initial riding qualities and with normal deterioration they were becoming unacceptably rough. This was a serious deficiency because once a rigid pavement becomes rough there is very little one can do to restore it.

The estimated unit pavement cost of a conventional HMAC pavement at Cold Lake was \$8.60 per sq. yd while that of a 12" thick rigid pavement was \$13.07, about 1.5 times the cost of flexible pavement. This higher initial cost however could be readily justified on the basis of future maintenance costs and the costs associated with closing the runway and deploying Squadrons and support activity to other airfields while the maintenance is being carried out. This justification however was totally dependant on one very important consideration. If we were going to realize the inherent benefits that lay in the superior durability and lower maintenance costs of a rigid pavement we would have to achieve significant improvements in initial riding qualities.

While there are many factors that can contribute to initial roughness in a rigid pavement we concluded that most of the problems we had been experiencing stemmed from the inexperience of our own site supervisory personnel and Canadian contractors generally in constructing this type of pavement. It is one thing to prepare a detailed design and write a rigid specification. It is quite another to achieve the specified result in the field. We were confident that with careful planning and extra effort these problems could be overcome.

From our general knowledge of the aggregates available in the Cold Lake area, mainly glacial fluvial deposits, we knew that we would have difficulty producing a base course with the high inherent stability necessary to support fixed forms. This led us to consider slip forming very seriously. We knew that slip forming was not without its problems but what probably tipped the scale was the requirement to complete the construction in one construction season. The construction schedule was going to be tight and it did not appear that it could be achieved using fixed form placing methods.

The detailed design of the pavement and related facilities was more or less routine. The major problem was that only one experienced design engineer was available for assignment to the project. It is a matter of record that he logged 300 hours overtime in 8 months to get the project out.

The 12-inch slab thickness was derived more from design policy than design procedure. The policy requires however that in each case the thickness be analysed by published PCA design procedures against an array of traffic and wheel loadings that the pavement in question might reasonably be required to support during its design life.

Original Design

As previously mentioned the soil and material surveys were carried out during the winter of 1971-72. On the basis of admittedly limited sampling we concluded that a pit located approximately 7 miles from the airfield contained sufficient material to meet our requirements. The design was therefore based on conventional methods and, for the rigid sections, consisted of a 12-inch non-reinforced slab with a 25 ft. joint spacing on 6 inches of crushed granular base course on 18 inches of granular subbase course on 6 inches of compacted subgrade. Tenders were called in September 1972 and on 6 October the contract was awarded to Eastbrooke Sand and Gravel Co. Ltd., the lowest bidder in the amount of \$6,635,730.00. Table 1, taken directly from the contract schedule of quantities, illustrates the scope of the project. The construction schedule called for site preparation and earth work to commence immediately with processing and stock piling of aggregates to run concurrently and to proceed until terminated by frost conditions.

Material Problems

Shortly after the pits were opened up and production got underway it became apparent that there was not going to be sufficient coarse material from the above source to fulfill project requirements. This was a serious development as the only known alternate source involved the construction of several miles of haul road, a river crossing and a 27 mile truck haul. While under the terms of the contract this would be the contractor's responsibility, for understandable reasons it was not one that he was willing to assume, nor could the Department within allotted funds. Moreover, to develop and deliver aggregates from this source would inevitably have delayed completion of the project beyond the target completion date of November '73.

Foundation soil borings had revealed a deposit of coarse, clean sand at the northwest end of the runway, much of it fortuitously in cut sections. We had not previously considered this as suitable construction material. We believed that it would be too unstable and difficult to handle. Some of it was to be used in infield grading and the surplus wasted.

Revised Design

On re-examination it was concluded that this material could, albeit with some construction difficulty, be utilized as a subbase material. It was also concluded that we were going to need a covering base course of much higher inherent natural stability than could reasonably be produced from the sources available. Tests on material from the aforementioned sand deposit indicated that it could be satisfactorily stabilized by the addition of about 6% fine silty sand that was also locally available and 5% portland cement. It was reasoned that the material thus stabilized would, once placed, confine and stabilize the sand subbase course. Some construction and compaction difficulties could of course be readily foreseen. The revised design then, consisted of 18 inches of sand subbase course, a 6-inch soil cement base course and a 12-inch slab, for the same total thickness of 36 inches.

Detailed cost estimates were made on all additional materials and operations that would be generated by the revised design and those that would be reduced or eliminated. These calculations indicated that the revised design could be constructed at about the same cost to the contractor, including the cost of the cement. The next problem of course was to convince the contractor that these calculations were entirely fair and reasonable. After some lengthy and detailed discussion he agreed and accepted a revised schedule of

quantities and unit prices that resulted in a net decrease of \$44,900.00 in the bid value of the contract. The largest extra cost item was of course the cement but this was offset by a major reduction in truck hauling, the increased production rates and efficiency that could be achieved in the operation, and the stable all-weather working platform provided by the soil cement base (SCB). These factors proved to be of major benefit to both the contractor and the Department in completing the paving on schedule and within budget. It is also believed that the SCB got us around some serious problems we would probably have encountered in attempting to deliver concrete and operate the slip form paver on a crushed granular base course of rather marginal stability.

Soil Cement Plant and Laydown Procedure

The SCB aggregate was taken from both runway excavation and borrow at the site of the work and stockpiled during the winter of 1972-73. A 500 T/Hr. Cedarapids continuous mix plant was used to mix the aggregate with 5% by weight of Portland cement, and water as required to achieve optimum moisture content. A 300 p.s.i. 7-day compressive strength was specified and easily achieved, (i.e. average 500-600 p.s.i.). As might be expected for a material of this type, durability test results were marginal. All specimens did however continue to gain strength during the freeze-thaw, wet-dry testing.

The SCB was placed in a rather innovative manner, utilizing the same Guntert-Zimmerman slip form paver that was later used for placing the concrete. For SCB work, the spud type vibrators were removed and the weighted one-ton vibrating compactor beams were added and pulled behind the paver. The SCB was set to stringline references and there was therefore no necessity for fine grading or trimming. At times the vibrating compactor beams did not achieve the specified 98% of modified Proctor density and supplementary compaction with air on the run pneumatic rollers was required. The sand subbase was quite tender and the tandem trucks hauling material rutted the subbase excessively, leaving a highly variable cross-section of SCB. As the haul was short this problem was rectified by using self-propelled scrapers which did not seriously rut the surface. The contractor averaged about 7,000 sq yds of a 6" thick SCB a day, with a good day's production being about twice the average. The SCB was all in place before any PCC pavement was slip formed and it is believed that most of the shrinkage had occurred before PCC paving commenced. Shrinkage cracks were fairly regular, spaced at about 10' to 20' intervals. SCB cracks did not result in any cracking in the PCC pavement, however it was noted that PCC transverse working joints did influence sympathetic cracking in the SCB.

Plant and Laydown Procedure

The PCC paving sub-contractor was J.W. Vickery Enterprises, Inc. from Dixon, California. He used a Johnson-Ranger twin-drum concrete batch plant rated at 650 cubic yards per hour. The paving train consisted of a Guntert-Zimmerman slip form paver set for 25' wide construction lanes, a C.M.I. pipe float and a Guntert-Zimmerman combination transverse broom cure applicator machine. Dump trucks transported the concrete from the central mix plant to the slip form paver. The concrete slump averaged about 1-1/2". If it approached 2" we experienced metering and edge slumping problems and if it dropped below 1", spreading and finishing problems. The paver was set to form the edge slightly high to compensate for settlement prior to set. As long as the slump was held within the aforementioned limits edge slumping was not a problem on this project. The paver was referenced to taut steel wires on each side with four elevation and one horizontal alignment sensors. The paving pattern, dictated by the schedule and available curing time, was the alternate lane method.

Metal keyway inserts were used to help support the free standing edges and these proved to be highly beneficial. Some problems were experienced in starting a lane where the ends must be firmly anchored. The inserts were lap spliced with rivets at the end of each roll. This caused the occasional problem as the splice passed through the forming attachment. We also discovered the obvious, they won't work on the inside of a curved lane.

The average daily production was about 2,500 cubic yards, and the best daily production was 4,000 cubic yards on a nominal 10 hour shift. Placing of the rigid pavement on this project comprising 200,800 sq. yds. was completed in 26 working days.

Logistics Problems

The supply of cement proved to be a logistics problem due to the rate of production possible with the slip form method. During the pre-construction planning the contractor rejected trucking from Edmonton as a primary source of cement and chose rail cars from Winnipeg, due to the quantities involved and the relative availability from the two sources. He planned on a production of 5,000 cu. yds per day @ 600 lb. of cement per cu. yd, for a total of 1,500 tons of cement a day. About 900 tons storage was planned at the batch plant with the rail cars acting as the remaining required storage.

As production got underway however the rail company saw fit to impose very stringent speed limits on trains hauling the 100 ton cement cars over portions of the track between Edmonton and Cold Lake. Apparently this interfered with other traffic as well as the delivery of cement. The contractor was forced to augment his supply by direct truck delivery from two sources in Edmonton, with the result that cement from three sources was being randomly mixed at the plant. Apparently our on-site people were aware of this but didn't see it as any problem. In retrospect it is considered that it was and probably accounted, at least in part, for some of the erratic and generally low 7-day strengths that were achieved earlier in the project. Subsequent testing indicated that the Winnipeg source tended to produce slightly lower 7-day strengths. It is quite probable that consistency was also being affected by this variable.

Off loading the 100-ton rail hopper cars proved to be a real bottleneck. Even though this operation was double shifted to 24 hrs. per day, it was never possible to achieve full plant capacity over a 10-hour shift without running out of cement, which of course would force a shut-down.

Off loading was accomplished by dumping into two under track hoppers that had been constructed for the purpose and elevating with two 32 ft. by 8 inch air slide augers into 25-ton semi trailer tankers. The haul to the plant was approximately 3 miles. The problem was in getting the cement out of the rail cars. When the cement would flow freely a truck could be loaded in about 25 minutes but endless problems were encountered with arching of the cement or lumps hanging up in the discharge chute. These had to be broken up by hand. Clearly, better techniques were called for but were not achieved on this project.

Paving was brought to a complete halt in September when rail deliveries were interrupted by a nation-wide rail strike experienced in Canada that year. As is usually the case in such circumstances we lost 3 weeks of absolutely perfect paving weather on account of the strike. Immediately following the strike the weather turned bad. There is no question that were it not for the rail strike the project would have been completed ahead of schedule. As it was we were able to complete the paving, but joint sealing and completion of some electrical work had to be suspended by the onset of inclement weather and the runway could not be opened to traffic until the following year.

Mix Design

Concrete aggregates were crushed during the winter of 1972-73 in preparation for the 1973 construction season. The coarse aggregate was split into 2 size ranges, 1-1/2" to 3/4" and 3/4" to #4, and the contractor chose to make this split on a 50-50 basis by weight. Sand was taken from borrow and stockpiled during the early months of 1973.

By the general terms of the contract the responsibility for the initial mix design and the processing of aggregates accordingly is assigned to the contractor but is subject to approval by the Resident Engineer. Clearly, with the benefit of hindsight, we were remiss in not monitoring this activity more closely. The contractor had virtually completed crushing and stockpiling the aggregate by the time it became apparent that the aggregates he had prepared were going to produce a fairly harsh mix and require a cement factor somewhat higher than planned to achieve the specified 7-day flexural strength of 550 p.s.i. This did not become really apparent until paving was under way and we started getting a number of low flexural test results. It was here that we made our second mistake. In our concern over the low strengths, which were ranging from 490 to 540 p.s.i., and with a view to improving the workability of the mix, we ordered the cement

content increased to 640#/cu. yd. at a water/cement ratio of 0.41. The contractor had planned on 580#/cu. yd. at a w/c ratio of 0.46. The reduction in w/c ratio did improve our 7-day strengths marginally but we were getting past the point of diminishing return. Our strengths were being limited by the marginal quality of the coarse aggregates. The C.A. did meet specifications, but was somewhat dirty and contained some encrusted and weak particles. While the additional cement appeared to have a minor beneficial effect on workability, it had another very adverse effect that we should have foreseen. It increased the shrinkage factor of the concrete to the extent that a significant footage of longitudinal shrinkage cracking developed after the project was completed.

The problem of marginal workability persisted throughout the job. Because the aggregates had all been produced and production equipment removed to other projects we were left with little margin in which to adjust aggregate proportions without the specter of running out of one size. There is little doubt that the fine aggregate which had a fineness modulus of from 2.9 to 3.0, right on the upper specified limit, also contributed to the harshness of the mix. Because of the harshness and poor workability of the mix the slip form paver did not produce an entirely satisfactory surface and considerably more hand finishing and floating than is desirable was necessary. The CMI pipe float proved to be beneficial in achieving an acceptable finish. The finish did, however, contain numerous "bug holes" that were later filled with an epoxy mortar to prevent entrapment of water and subsequent freeze-thaw damage.

The foregoing has been set forth in some detail as it points out some of the pit falls in producing aggregates without early and thorough investigation of resulting mix properties. This project certainly demonstrated what I am sure others with more experience have long since known, slip forming requires a good workable mix of constant consistency.

Uncontrolled Cracking

Prior to paving the runway, operations had been confined to the arming pad and a short taxiway where daily production was limited by the confined areas. As soon as work on the runway got under way and daily production increased we experienced uncontrolled transverse cracking. This was clearly due to inadequate sawing capacity and late sawing. All paving was immediately suspended until adequate sawing capacity was on site. The contractor solved the problem by producing a four blade span saw powered by two 30 HP motors. This proved to be adequate but the independent hand operated saws previously used were kept on site in reserve.

In all about 100 cracks developed in the first lane paved. Fortunately this was an outer lane. By the terms of the contract we could have insisted on removal and replacement of all cracked slabs but for a variety of practical reasons were reluctant to do so. The contractor made a proposal to repair these cracks using the "Hunt Process Co." method of epoxy injection. The process involves sealing the cracks with a proprietary wax-like compound. Injection ports are formed in this seal on 12-inch centres by the prior placement of masking tapes. Starting at one edge of the slab, low viscosity epoxy is injected at a pressure of up to 100 p.s.i. until the epoxy extrudes from the adjacent port. The gun is then applied to this port and the process is repeated across the slab.

The proposal was accepted under the following stipulations:

- (a) Adjacent non-working dummy joints were to be deepened to 6 inches.
- (b) The inside edge and key way of all cracked slabs were to be liberally coated with an asphalt bond breaker.
- (c) That full penetration of the epoxy would be demonstrated by coring following application.
- (d) Adjacent dummy joints would have to be observed to be working before placement of the adjacent slab would be permitted.

The repair method was successful. Adjacent dummy joints did open prior to placement of the filler slab and after two winters none of the repaired cracks showed any signs of opening up. Obviously we significantly reduced load transfer by aggregate interlock at the dummy joints that were deepened, but since we were dealing with an outside slab we weren't too worried about this. We were more concerned about propagation of the crack into the adjacent lane.

Texturing

As previously mentioned, final texturing prior to application of the curing compound was carried out with a Guntert-Zimmerman combination texturing and compound applicator machine. The machine drew a 10 ft. wide stiff nylon bristled broom transversely across the surface, advanced 10 feet and repeated the process. It produced an extremely harsh surface. The specified depth of surface texture was 0.035 inches as measured by the sand patch method. This was readily achieved. Because of the inherent harshness of the mix the broom tended to tear the surface leaving numerous sand particles dislodged and weakly bonded to the surface. Visually the texture appeared extremely harsh and sharp, so much so that the operational staff, upon observing it, became alarmed at the effect this was going to have on tires. We were confident however that much of the initial harshness would disappear with a short period of weathering and sweeping that the surface would inevitably receive. This has proven to be the case and we now have what I would judge to be an excellent coarse textured surface. About 90% of the "touch and go" operations are now conducted on this runway and there has been about a 20% increase in tire wear. On the other hand it continues to provide excellent braking action under all conditions, even when thinly coated with ice or hoar frost. There have been no reported incidents of hydro-planing or loss of braking action since the runway was commissioned.

Longitudinal Cracking

As previously mentioned, some time after completion some longitudinal cracking began to appear, at first in one of the outer lanes but later in interior lanes as well. These cracks would run through anywhere from 3 or 4 to 15 or 20 contiguous slabs and then stop. It was first noticed in the late fall but much of it developed during the course of the first winter when the runway was under snow. In all about 3,000 feet of this cracking has developed, virtually all of it during the first winter.

While there is a natural tendency for cracks of this nature to propagate themselves, the situation does seem to have stabilized. During the course of last winter, 1975-76 (the third winter after placement) the cracks have propagated through four additional slabs, two of which were partially cracked at the start of the winter. No new crack locations have been observed. With the exception of the cracking due to late sawing, previously discussed, no transverse cracks have appeared in this runway.

When the cracking first began to appear we were at a loss to explain it. Many theories and hypotheses were advanced but none could completely explain it. A consultant was engaged to make a thorough examination of the phenomenon including a review of all quality control test data. He was not able to find any specific defects that would explain the cracking. About all that we were able to discover is that in all cases the cracking occurred in those slabs containing the rich (640#/cu. yd) mix. Not all slabs containing this mix cracked but in all cases the cracking was confined to this mix. It is hypothesized that the cracks were initiated at the joints in the underlying SCB. Densities in the SCB tended to be a bit low right at the longitudinal joint between lanes. This would form a natural plane of weakness for a shrinkage crack in the SCB. Minor surface irregularities at the SCB joints would tend to increase friction or drag forces at the SCB/slab interface in the transverse direction. The SCB was exceptionally smooth in the longitudinal direction. It is believed that the foregoing was sufficient to initiate longitudinal cracking in some of those slabs containing the richer mix and consequent higher shrinkage stress. Once initiated of course the crack would tend to propagate across transverse joints into adjacent slabs. It may also explain why we did not get any transverse cracking.

It was realized that we were pushing the limit in designing for a 25 ft. joint spacing in a 12-inch slab in this climate, particularly with the aggregates we would be using. The decision was taken however on the basis of the satisfactory performance of a similar pavement containing similar aggregates, at CFB Edmonton and to take full advantage of the reduction in construction effort afforded by a 25 ft. lane.

While any cracking is undesirable, certainly aesthetically, the cracks at Cold Lake are not yet creating any real problems. There is very little horizontal movement, no vertical movement and no spalling of any consequence. They have all been routed and sealed. There is always the danger however that slabs thus cracked will tend to section up with the formation of mid-slab transverse cracks.

Joint Sealants

As previously mentioned, the runway and all other pavements were completed in the fall of 1973 but joint sealing was not completed until July '74. A two component JFR sealant was used in the holding areas and buttons, and the remainder was sealed with a single component non JFR hot poured rubberized thermoplastic material supplied under ASTM Spec. D 1190. The reservoirs were sawn 1/2 inch wide by 3/4 inch deep. As most of the shrinkage had occurred by the time the sealant was applied, the sub contractor elected to use a 1/4 inch backing rod to conserve sealant but this was not specified.

About two months after completion, certainly before any significant drop in temperature had occurred, it was observed that the bond between the hot poured sealant and the concrete was very weak. Once started, the sealant could be pulled cleanly from the joint in long strips with little difficulty. The contractor was put on notice that the bond was not satisfactory and that in those areas where it was not, the sealant would have to be removed and replaced. By the following spring we had virtually a 100% bond failure. A consultant was engaged to study and report on this failure and he came up with some interesting findings. He found that the product did meet the bond test requirements of ASTM D 1190 but only on the permitted second trial at elevated pouring temperatures. He also found that the asphalt recovered from the compound had a very high wax content. He also discovered that if the wax content of some sealants supplied by other manufacturers and which had proven to perform satisfactorily in the field was increased to the same value the bond strengths of their products were similarly degraded.

Performance to Date

The performance of the pavement itself has to date been entirely satisfactory. Neither the transverse cracks which were repaired with epoxy resin or the longitudinal cracks which were routed and sealed are presenting any real problem. The joint sealant which failed in bond has been removed and replaced and the new sealant seems to be performing normally. The "Riding Comfort Index" (RCI) as measured with a Wisconsin type road meter the year following completion was 7.1 but this figure is suspect. It is our judgement that the RCI would be nearer 7.5. Definitive friction values are not available but wet braking action is reported as excellent.

Conclusions

While the foregoing has dwelt at some length on the problems that were encountered during the construction of this project, it should not be inferred that the project was a disaster. Far from it. It was highly successful, it was completed on target within budget and its performance to date has been excellent. While the effort has no doubt been dwarfed by the achievements of other agencies, we feel that two years from conception to completion is a pretty good record. We don't believe that we could have achieved either the excellent riding qualities or the target completion date had we elected to use the fixed form method. This was our first slip form project. We made some mistakes and learned some lessons. It is hoped that others may benefit from our experience.

B. THE BAGOTVILLE PROJECT

Introduction

In 1973 while the Cold Lake project was under construction attention was directed to the up coming requirement to rehabilitate the existing 10,000 ft. by 150 ft. main operational runway at CFB Bagotville. The basic layout and scope of the project are illustrated in Figure 2. Unlike Cold Lake the history of Bagotville goes back to World War II when it consisted of the basic triangular runway layout associated with the training fields of that era. The airfield was decommissioned at the end of the war but was rehabilitated in 1952 to again serve as a training base for fighter pilots. At that time runway 18-36 was considered to be the main runway but Runway 11-29 was also rehabilitated by application of a sandwich overlay. Bagotville was subsequently designated as a NORAD Base from which all weather interceptors were to be operated and it became apparent that Runway 11-29 would have to be developed as the main all weather runway. It was extended westward to 8,000 ft. in 1955 and to 10,000 ft. in 1961. Bagotville is currently one of Canada's main NORAD and Operational Training Bases from which both CF101 (Voodoo) and CF5 (Freedom Fighter) aircraft are operated.

Existing Construction

As might be inferred from the foregoing, the construction history of Runway 11-29 is too complex to be presented in detail here. Essentially it is as follows. The subgrade is a free draining sandy material and no subgrade drainage is provided. The buttons are 10-inch and 12-inch thick rigid (PCC) pavement. The remainder of the runway is of typical flexible (HMAC) construction. To improve the profile of the old wartime pavement a heavy sandwich overlay was applied. The thickness of the crushed granular sandwich base course varies from 5 to 10 inches. The initial surfacing was 3-1/2 inches of HMAC. The post war extensions consisted of 3-1/2 inches of HMAC on a 9-inch crushed granular base on from 12 to 18 inches of pit run gravel subbase course.

In all cases the original asphaltic surfaces have been resurfaced with 2 inches (nominal) HMAC overlays at least once and in most cases twice (depending on their age) since originally constructed.

Past Performance

The performance of the flexible pavement sections of this runway had in many respects been similar to that of Cold Lake. The riding qualities have remained fairly good. There were a few more dips and humps or distortions of profile but it is believed that these were "built in". The main problem was the very extensive thermal shrinkage cracking and our inability to effectively seal or maintain these cracks. The average useful life of a nominal 2 inch HMAC resurfacing was no more than seven years, with considerable crack maintenance between overlays.

Performance of the 10-inch and 12-inch PCC pavements in the 29 Button was considered acceptable. The ride was a bit rough but there was no evidence of serious structural distress. The performance of the 12-inch PCC in the 11 Button constructed in 1961 had not been satisfactory. There was considerable random cracking and some corner breaking. Joint and crack spalling was extensive. The surface was badly crazed, surface scaling and disintegration were quite advanced. All these defects can be attributed directly to the poor quality of the initial construction and clearly demonstrate the penalty that is inevitably paid for inadequate on-site construction supervision and quality control.

Drainage

Pavement foundation and infield drainage at this site was rated only fair. The subgrade for the most part is fairly clean coarse and in some places gravelly sand, but one significant lens of dirty, slightly frost susceptible sand had been identified. With the exception of some mild frost action which had been detected in the aforementioned area, the runway was relatively free from frost heaving.

Structural Capacity

Plate load testing carried out in 1964 and 1973 indicated that pavement was structurally adequate.

Planning Considerations

In planning the rehabilitation, various options or strategies were considered.

They were:

- (a) Apply a nominal 4" thick* HMAC levelling overlay and carry on as we had in the past.
- (b) Remove the existing and construct a completely new HMAC surface.
- (c) Utilize the existing construction as a foundation or base for a new rigid PCC surface.

Clearly option (a) was the cheapest, at least initially, but was not necessarily the most economical in long term as will be shown, nor was it considered an entirely satisfactory solution operationally. Within 3 years at the outside all the cracks in the existing surface would be reflected through the overlay and would start generating debris again. Option (b) had some promise but in respect to a new HMAC surface the comments on the Cold Lake project would apply equally here. We were confident that we could build a better HMAC pavement but were doubtful that we could get a serviceable life of more than ten years before it would require an overlay. Option (c), the most expensive in terms of initial cost, appeared to have significant advantages from an operational point of view:

- (a) If properly constructed of quality concrete it would, based on the performance record of similar pavements, significantly reduce future maintenance effort and cost.
- (b) Virtually eliminate the problem of uncontrolled cracking and the surface debris resulting therefrom.
- (c) Significantly reduce future disruption of flying operations.

Performance records of existing PCC pavements meeting the quality criteria indicated that if we could construct a smooth good riding surface initially the useful service life of a rigid overlay could reasonably be taken as 20 years. When the foregoing factors, i.e. construction costs, future maintenance costs, deployment costs, etc. were analysed over a 10 year time frame, the following present cost picture emerged:

- (a) Option (a) - \$2,844,000.
- (b) Option (b) - \$2,977,000.
- (c) Option (c) - \$2,529,000.

It should be pointed out that when these estimates were made we did not have reliable figures on the actual cost of deploying Squadrons etc. to other air bases. These costs subsequently proved to be very much higher than we had estimated, which would render the actual costs of options (a) and (b) significantly higher than indicated.

The foregoing may look a bit like "re-inventing the wheel" but it formed the basis of the argument on which the decision to construct the more costly rigid overlay was based.

* The profile was considered beyond restoration by a single 2" HMAC overlay.

Jet Engine Damage

There is one unknown but possibly significant cost not included in the foregoing tabulation. It is the cost of repairing engines damaged by the ingestion of pavement debris. We know the overall cost of repairing engines but there are many sources of foreign object damage (FOD) and these can rarely be identified after the fact. Suffice to say that when engine damage is being sustained and pavement debris is present, the debris becomes highly suspect.

Design Considerations

A grid level survey taken on the existing surface the preceding year was transformed into profiles and cross sections which were studied in detail to determine where corrections in either profile or cross section were called for. Tentative finished grades were set during this process. The tentative grades were then checked both longitudinally and transversely to ensure:

- (a) That nowhere would the thickness of the overlay be less than 11 inches, 10-1/2 inches being taken as the minimum a slip form paver with 10-inch rigid side forms could handle.
- (b) That everywhere variance in longitudinal profiles were within acceptable limits and that adequate surface drainage or crown was achieved.

Adjustments were made as and where necessary. This is pretty much a judgement process in which a compromise is reached between variance in surface gradient and overlay quantities. Final grades were then checked to determine where they lay more than 13 inches above existing surface to delineate in plan the areas that were to be pre-padded. Padding was to be carried out with a fine graded HMAC. These areas were outlined on the contract drawings.

Because of the previously described condition of the PCC pavement in the 11 Button we elected to overlay this as well rather than fair into it as we did for the 29 Button. We arbitrarily elected to apply a nominal one inch HMAC bond breaking course on the rigid pavement to prevent the reflection of mismatched joints or cracks into the new surface. To date this has worked perfectly. Not a single crack has developed in these areas.

Core samples from some of the older sections of the HMAC surface had previously been tested for loss of stability on immersion. The results indicated that there were no serious problems in this regard. Previous plate load tests indicated the existing surface would provide subgrade reaction or "k" values well in excess of 500. A value of 250 was used in the design to allow for some possible long term degradation of the existing asphalt surface.

Runway 11-29 is the only runway at Bagotville from which routine jet operations and training can be safely carried out. The cost and operational disruption caused by temporary deployment of squadrons and support services to other bases made it essential that runway down time be minimized and it was absolutely vital that the project be completed and the system be back in operation before the onset of winter. It was for these reasons that we elected to stay with the 25 ft. wide paving lane in spite of our experience at Cold Lake. A 37-1/2 ft. lane with a sawn longitudinal joint was considered briefly but was rejected for practical reasons. Runway 11-29 had an offset crown which would probably have caused some problems with a 37-1/2 ft. paver. Moreover it didn't appear that we could count on delivering cement to the site (which involved a road haul of 130 miles) in sufficient volume to take advantage of a larger plant. It would also be more difficult to accommodate the variance in crossfall or crown with a paver of this width.

We knew that good quality aggregates were available at Bagotville and that there should be little problem in getting the desired 550 p.s.i. 7-day strengths. Accordingly we were more confident about a 25 foot joint spacing on this project but, as a hedge, reduced the transverse joint spacing to 20 ft.

Scope of the Work

The overall scope of the completed design may be seen in Figure 2 which illustrates the layout, and Table 2 taken from the contract schedule of quantities.

Construction Schedule

The contract was awarded to the low bidder, Les Construction du St. Laurent Ltd. on 29 Apr 1974. Les Construction du St. Laurent Ltd. subcontracted the mixing, placing, finishing, curing and joint sawing of the PCC pavements to Vickery Enterprises Ltd. of California, the same contractor who did this work on the Cold Lake project.

The construction schedule called for all work to be completed and the airfield back in full operation not later than 01 Nov 1974. Penalty clauses for late completion were in the contract.

Construction

With but one major exception very few engineering or construction problems were encountered on this project and these were of a minor or routine nature and expeditiously resolved. It is a credit to Les Construction du St. Laurent Ltd., Vickery Enterprises, other Sub-contractors and the DCL Resident Engineer and his staff who organized and cooperated to bring this project in ahead of schedule and within budget.

The major exception previously alluded to, and which was entirely a DND responsibility, was the discovery on the morning that slip form paving was to commence and while the paver reference wires were being set out that there were errors in the original grid survey on which final design grades had been established. Fortunately for the Department, but not for him, the Senior Design Engineer happened to be on a routine liaison visit to the site at that particular moment. The contractor must certainly have appreciated the gravity of the situation from the Department's point of view, and the Department will be forever grateful for the manner in which he cooperated. He immediately supplied an extra survey crew and the able assistance of his paving train foreman to assist Mr. McLean, the Senior Design Engineer, in carrying out an in-field and on the spot revision of design grades. Between them they developed a technique of using wooden blocks to represent the thickness of the overlay. These were set out on the existing surface and, working with levels and a good deal of "eye-balling", revised grades were established. Anyone who has actually done this type of work in the relative peace and quiet of a design office will appreciate the problem. To do it in the field with a slip form paving train just a few yards behind you must have been a frightening experience. Mr. McLean estimates that he worked 24 hours a day (20 hours actually working, 4 hours worrying about it) for several consecutive days to complete the task. It is a credit to both him and the contractor who co-operated so diligently that the total delay or stand time chargeable to this error was limited to 12 hours.

The foregoing has been set out in some detail as it points out the necessity of having reliable, accurate and verified data on existing grades before a project of this nature is launched. The potential consequences of proceeding without this should be self-evident.

The same basic concrete plant and paving train were used on this project. Plant cement silo storage was increased to 1850 tons as no other on-site storage was available and all cement had to be delivered by truck 130 miles over fairly congested highways from Quebec City. Storage capacity proved adequate.

The major difference was in the method of finishing and texturing the surface. The same machine was used but the nylon broom was replaced with a spring steel comb. The tines were 0.08 inches wide by 5 inches long by .028 inches thick and were at 3/4 inches on centre. The comb was approximately 10 ft. long. Following burlap dragging the comb as drawn transversely across the finished but still plastic surface. The specified depth of groove was 3/16 inch but this was difficult to achieve without undue tearing of the surface and dislodgement of aggregate particles. It is believed that longer and possibly stiffer tines may have produced a deeper groove with less tendency to tear, but this is

conjecture as they could not be obtained in time to try them. The initial appearance of the finished texture was a bit disappointing but its performance and braking action have not been. Bagotville receives exceptionally heavy and intense rain showers during the summer months and the Base had been experiencing some hydro-planing incidents on the previous asphaltic surface. There have been no reported incidents of hydro-planing or loss of directional stability since the new surface was commissioned. It is also reported that surface icing is more easily removed from this surface than it was from the old HMA surface. Apparently the grooves cause the ice sheet to break up and flake off more readily. The excellent riding qualities coupled with the absence of harshness have prompted many favorable comments from pilots landing on this runway for the first time.

Performance to Date

A detailed inspection of this runway after a year in service revealed only two randomly cracked slabs. The cracks occurred in the aforementioned "silty" subgrade area and are attributable to mild frost action. There is no faulting or spalling at these cracks.

At this writing it is reported that a bond failure is developing in the joint seals. The sealant employed was a non (JFR) hot poured rubberized thermoplastic, similar to that product used at Cold Lake. The problem has not yet been investigated and more detail is not available. Suffice to say we have drastically revised our Sealant specifications.

Conclusions

In all, this project was eminently successful. The slip formed overlay on the runway itself was completed in 20 working days. The project was completed and the airfield re-opened to unrestricted flying one month ahead of schedule. Both the Department and the contractor derived significant economic benefit from this achievement. The RCI, measured in 1975, was 7.8. The quality of the concrete and finish, the riding qualities and braking action are excellent. Its long term performance remains to be proven but it is anticipated that it will be vastly superior to the asphaltic surface it replaced.

TABLE 1.
SCHEDULE OF QUANTITIES *
COLD LAKE

<u>Item</u>	<u>Description</u>	<u>Unit</u>	<u>Quantity</u>
1	Site Clearing and Grubbing	Ac.	450
2	Removal & Disposal - Pipe Drains	Lin. Ft.	1,540
3	Removal & Disposal - Drainage Structures	Ea.	4
4	Common Excavation and Earthwork	Cu. Yd.	1,300,000
5	Subgrade Compaction	Sq. Yd.	302,150
6	Granular Subbase Course	Ton	332,595
7	Granular Base Course	Ton	110,864
8	Bituminous Prime	Imp. Gal.	27,400
9	HMAC Pavement:		
	(i) Binder Course	Ton	11,974
	(ii) Surface Course	Ton	7,184
10	P.C.C. Pavement (12"), including Asphalt - Concrete Joints	Sq. Yd.	201,264
11	Sealing PCC Pavement Joints:		
	(i) Jet Fuel Resistant Sealant	Lin. Ft.	18,260
	(ii) Non-Jet Fuel Resistant Sealant	Lin. Ft.	113,175
12	8" dia. Perforated Pipe Sub-drains, c/w Filter Backfill	Lin. Ft.	38,655
13	Non-Perforated Pipe Storm Drains, Common Backfill:		
	(i) 12" dia. pipe	Lin. Ft.	9,958
	(ii) 15" dia. pipe	Lin. Ft.	5,075
	(iii) 18" dia. pipe	Lin. Ft.	2,675
	(iv) 24" dia. pipe	Lin. Ft.	430
14	Drainage Structures:		
	(i) Catchbasins	Ea.	75
	(ii) Manholes	Ea.	83
15	Installation - Cable Duct:		
	(i) 4 - 4"	Lin. Ft.	2,490
	(ii) 6 - 4"	Lin. Ft.	1,010
	(iii) 8 - 4"	Lin. Ft.	560
16	Sealing Over Edge Sub-Drains	Lin. Ft.	38,155
17	Draining Structures - Altering Tops	Ea.	6

* Taken directly from the contract documents.

TABLE 2.
SCHEDULE OF QUANTITIES *
BAGOTVILLE

<u>Item</u>	<u>Description</u>	<u>Unit</u>	<u>Quantity</u>
1	Removal & Disposal - Asphaltic Concrete Pavement	Sq. Yd.	51,551
2	Removal & Disposal - Concrete Pavement	Sq. Yd.	4,155
3	Common Excavation & Earthwork	Cu. Yd.	58,455
4	Subgrade Compaction	Sq. Yd.	55,444
5	Granular Subbase Course	Ton	33,226
6	Granular Base Course	Ton	29,822
7	Bituminous Prime	Imp. Gal.	10,169
8	Bituminous Tack Coat	Imp. Gal.	4,299
9	Hot Mix Asphaltic Concrete Pavement:		
	(i) Asphalt Cement	Ton	1,562
	(ii) Mix & Place	Ton	26,035
10	Portland Cement Concrete Pavement:		
	(i) Cement	Ton	15,587
	(ii) Mixing & Hauling	Cu. Yd.	51,960
	(iii) Placing, Finishing & Sawing	Sq. Yd.	161,709
11	Sealing Portland Cement Concrete Pavement Joints:		
	(i) Jet Fuel Resistant Sealant	Lin. Ft.	8,166
	(ii) Standard Sealant	Lin. Ft.	93,770
12	Installation - Cable Ducts:		
	(i) 2 - 4"	Lin. Ft.	100
	(ii) 4 - 4"	Lin. Ft.	100
	(iii) 6 - 4"	Lin. Ft.	170
13	Drainage Structures - Altering Tops	Ea.	6

* Taken directly from the contract documents.

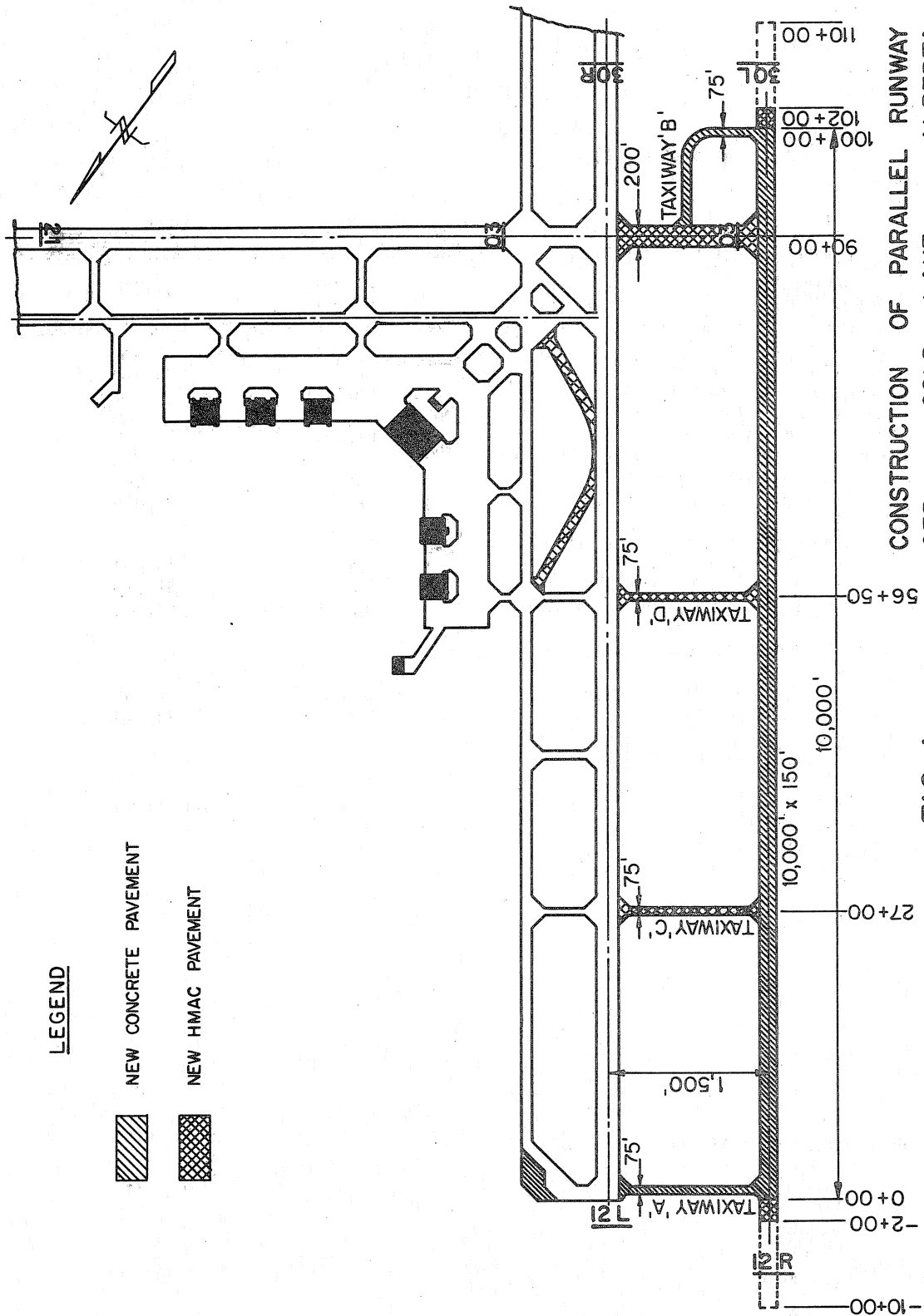
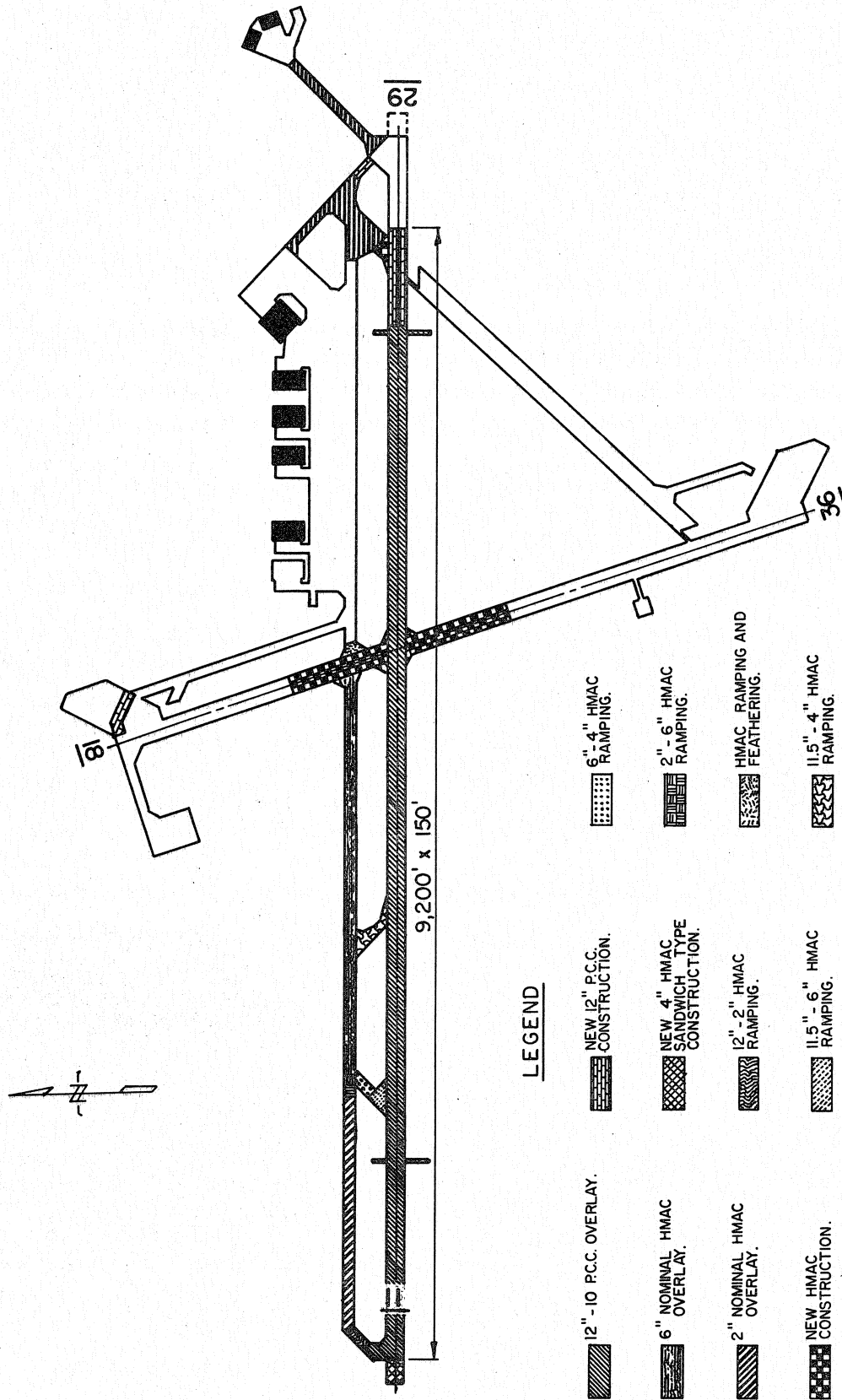


FIG. 1



OVERLAY RUNWAY 11 - 29
TAXIWAYS REHABILITATION
CFB BAGOTVILLE QUEBEC

FIG. 2