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Reinforced earth - a case history

La terre armée - une application

Suite aux premiers championnats juniors mondiaux de ski nordique terres à Ste. Croix en Suisse en 1975, Québec fut choisie pour la deuxième édition des championnats junior mondiaux, en février 1979. Le site choisi pour la construction du tremplin 70 m requis pour les jeux fut le Mont Ste. Anne a un endroit spectaculaire avec vue sur le fleuve St. Laurent. La mise en oeuvre de ce nouveau saut a amené la construction d'un mur en terre armée de 13 mètres de hauteur avec parements en béton prefabriqué, et ce sous des conditions de construction et d'accès qui créaient un veritable défi. Ce document décrit les événements qui ont conduit au choix de la terre armée pour cette utilisation unique et pour l'avancement des travaux.

BACKGROUND

Following the award of the Second World Nordic Junior Championships to Quebec, Canada for 1979, a site was selected at Mont Ste. Anne, in one of Quebec's most scenic ski resort areas, to develop the 70 metre ski jump required for this event. Site selection was governed by numerous factors peculiar to this competetive sport which are not related to the subject of this case history and are not discussed herein. When the appropriate site was discovered, it was noted that a spring emerged from a rock formation and discharged into an area located within the projected toe limits of a proposed side hill fill slope. This water source was estimated to have a minimum flow of less than 0.01 m^3/sec and a maximum flow in excess of 0.25 m^3/sec. To avoid obstruction of the water source and therefore to avoid the eventual erosion of the fill slope, it was considered necessary to construct a retaining wall to support a short section of the ski jump. Since timber crib walls were used for access road construction elsewhere in this general area, this type of wall was also selected originally to make up the total required difference in elevation of about 13 m for retaining the fill materials.

In the fall of 1977, two timber crib walls, were constructed approximately 6 m and 4 m high, separated vertically by about 2 m of fill and at a slight angle to each other in plan. (See Figs. 1 and 2.) During construction, a temporary stockpile, placed some 30 m above the level of the lower crib, failed during a heavy rainstorm. This transformed the material, which was primarily granular, into a semi-liquid mass which flowed down and over the crib wall. It came to rest on the slope in front of the crib and built up to cover about half of the crib face. Simultaneously, it covered the water source below. This did not create any immediate problem because the prevailing flow rates were minor. The impact and pressure caused by the additonal soil mass shifted the crib wall from the constructed 1 horizontal to 6 vertical (1h:6v) position through the vertical, and bulged the upper portion of the crib about 0.75 m beyond its vertical axis. Because of the pending precompetition and the approaching winter, it was not possible to resolve the problem immediately. Temporary repairs were undertaken to permit precompetition trials to proceed. Full scale reconstruction was scheduled for the summer of 1978, since it was believed that complete collapse would occur in the spring.

The ski jump was repaired without further complication and was ready for precompetition trials in March 1978. Then, during the spring break-up in April of 1978, water flow from the rock below the crib wall level increased substantially. Progressively, this washed away all the slide debris in front of the crib wall by the process of repeated erosion and subsidence. This process finally led to the anticipated classical slope failure, the surface of which passed through the lower wall and developed a 0.5 m shift of the upper wall and dish-shaped crack in the fill above the upper wall. Upper sections of the lower wall collapsed and were carried down the slope with the slide debris, whereas the lower section of the lower wall remained in place. (See Fig. 3.)

As a consequence, it was decided to rebuild this part of the ski jump using Reinforced Earth, (RE), leaving the crib wall in place only where it could serve as a berm in front of the RE mass.

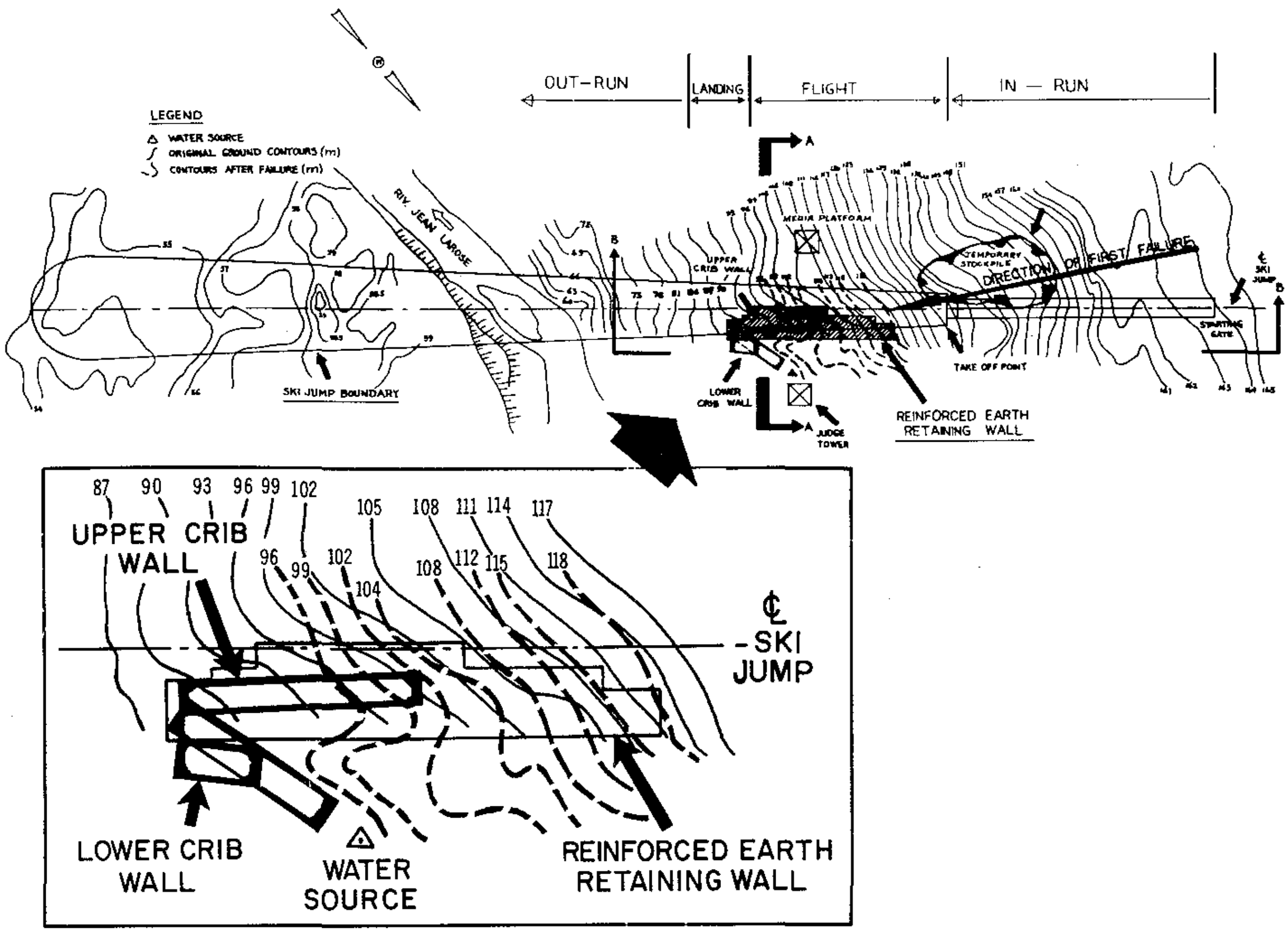

Fig. 1. Site plan of ski jump.

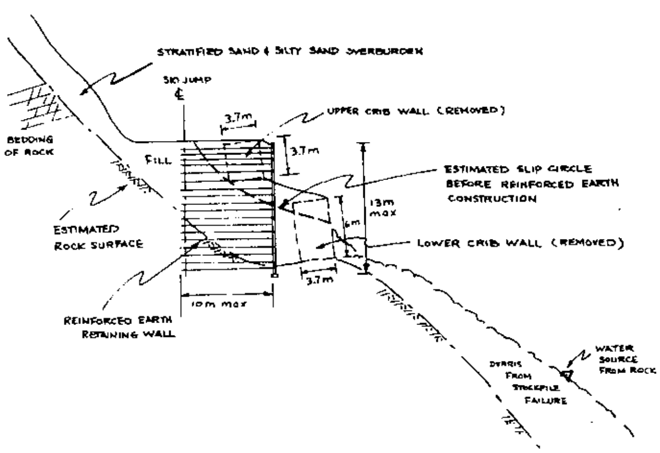

Fig. 2. Cross section through jump shows relative positions of removed crib walls and reinforced earth wall.

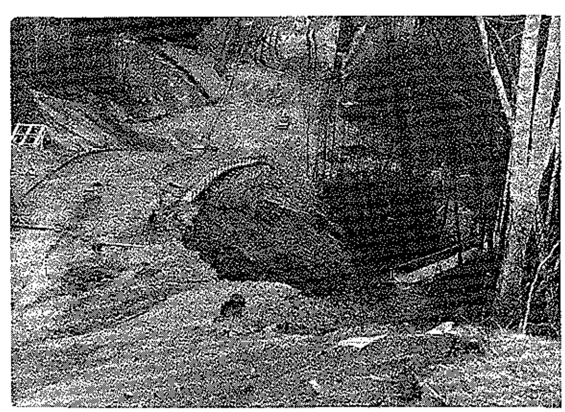

Fig. 3. Picture showing final slope failure in Spring 1978.

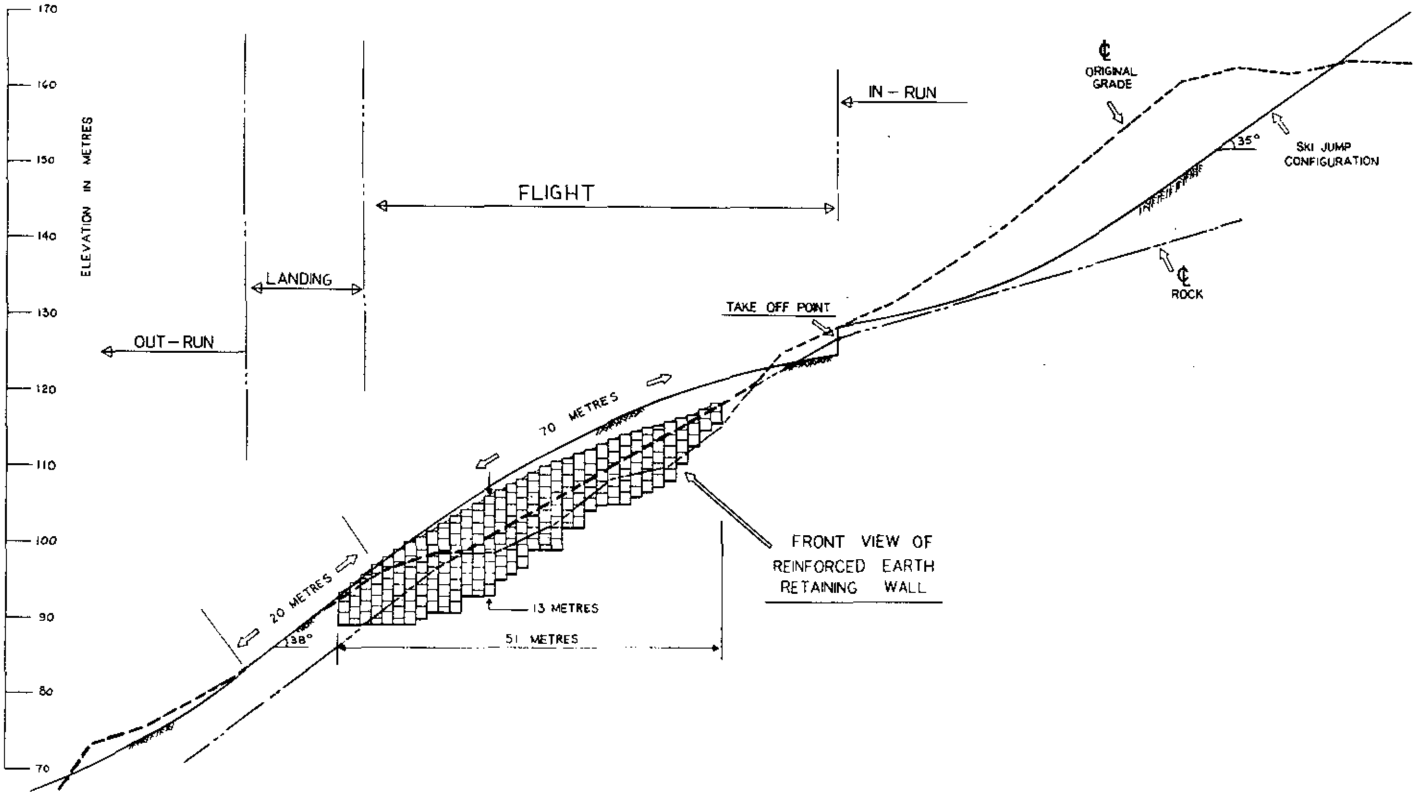

Fig. 4. Longitudinal profile (Section B-B) showing front view of the reinforced earth structure relative to the ski jump.

SITE CONDITIONS

Fig. 1 illustrates the general outline of the area and the relative positioning of the ski jump, the judges' tower, television platform and spectator observation areas. The top of the jump is located approximately 100 m adjacent to a public highway and 10 m above it. The bottom of the jump and spectators' area are accessible only via a steep and winding roadway constructed solely for access to the spectator - ski run-out area. Retaining wall construction was required about half way between these levels or about 60 m below the top and 50 m above the run-out.

The relative differences in elevation, configuration of the jump and the level of the retaining wall are illustrated in Fig. 4. Due to environmental and other restraints, access to the construction area itself was available only along the slopes of the ski jump, ie, directly from the top or bottom and via slopes as steep as 1.25 horizontal to 1 vertical (1.25h:1v). To appreciate the access problems at this site fully, one needs to consider only the fact that it was necessary to erect the judges' tower with a 200 tonne crane (equipped with an 84 m boom) to obtain a lifting capacity of 4 tonnes at full reach. An alternative to the crane, which was considered, was the use of a helicopter. However, one was not available economically in the required capacity.

Soil conditions at the site consisted of predominantly brown sand or silty and gravelly sand with local layers of clayey silt, clayey sand and silt. The overburden was underlain by limestone bedrock which was severely weathered within the upper 0.3 m and generally fractured with frequent bedding planes to the depth investigated. Bedding planes were found to be numerous during excavation but were favourably inclined, ie, opposite to the direction of any potential sliding plane or arc failure. The original and ground profile, soil and rock conditions are also illustrated in Fig. 4.

DESIGN CONSIDERATIONS

Several routine, but also some unusual design aspects needed to be taken into consideration as follows:

1. Because of the poor site accessibility, it was desirable to utilize locally available materials as much as possible within the RE mass. These materials consisted of blasted limestone mixed with sand, silty sand and gravel, but also clayey silt and silt layers. These layers would have to be disposed of outside the RE mass. Random sampling prior to and during construction indicated the range of grain size distribution to be generally within the limits of the RE specification. (See Fig. 5.) The results of direct shear tests confirmed the predominating frictional materials to be suitable for use within the RE mass.

2. Design could not count on any support value of any remaining sections of the crib wall. Foundation levels were therefore selected such that the crib would not fall within the limits of the stress influence of RE. Also, differential settlement would be dependent on the rate of change of overburden thickness. Transitions from rock to soil were made sufficiently gradual to accommodate the tolerance limit of 1% differential settlement stipulated for this case.

3. External stability of the wall with respect to sliding, overturning and overall slope stability had to be satisfied taking into consideration the relatively poor condition of the rock. This was checked by a geotechnical consultant. At the same time, it was considered prudent to maintain a rectangular configuration of the RE mass in view of the generally fractured condition of the rock as noted in the soil consultant's report. The seismic activity in the area was also a contributing factor considered in this decision.

4. Adequate drainage was required to handle potential seepage flows.

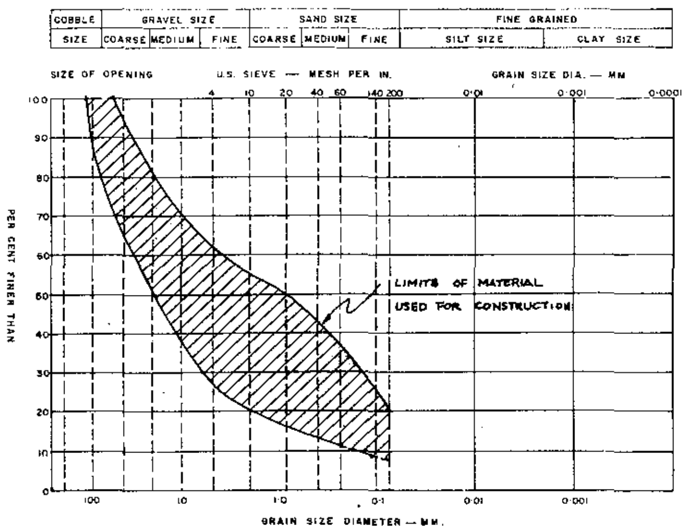

Fig. 5. Grain size distribution chart indicating the limits of material sizes used for construction.

5. The level of the concrete levelling pad needed to be flexible to allow for variations in rock quality encountered during construction.

6. Erosion protection in front of the wall was applied such that deterioration of the rock in that area was controlled. At the same time, use of a fabric filter blanket beneath the RE mass ensured that any unknown water sources could be controlled without the loss of fines.

7. Close liaision was maintained during construction to review any changes in design necessitated by field conditions.

8. Construction methods and schedules needed to be worked out with the contractor to facilitate construction within the schedule and with the equipment available. Approximately 6 weeks were available for construction, provided that design could be finalized and material supply for construction could commence within a 3-week period.

Internal design of the RE mass utilized the parameters determined from laboratory tests carried out by others. The results all fell within the usual requirements of the Reinforced Earth Company. In this regard it was realized that compaction would be achieved only by the movement of the backhoe, tamping with its bucket and the action of a very small bulldozer. Also, the cover in front of the wall could be maintained at the usual configuration with the exception of one area where the lower and undamaged part of the crib could serve as berm because of insufficient space. A typical design section is shown in Fig. 6. The total surface area of the wall facing was 500 m^2 and its average height was about 10 m.

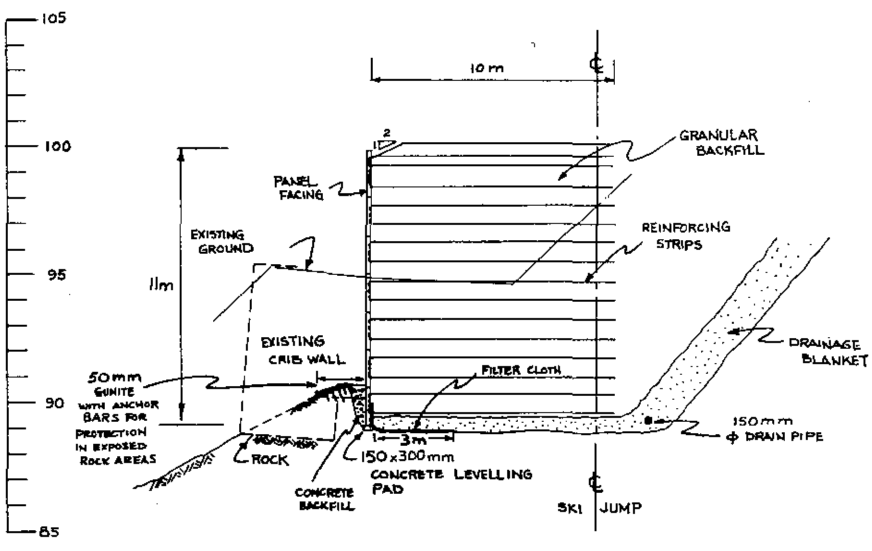

Fig. 6. Design section.

WALL CONSTRUCTION

Scheduling of the wall took into consideration all the factors peculiar to this site and was set to accommodate construction within the remaining construction time. The governing factor for scheduling was that only a single track mounted backhoe could be accommodated at the location of the wall. Thus, this backhoe (Caterpillar Model 235 with 1.5 m^3 bucket capacity) was to be used for excavation, setting of panels, backfilling and compaction. Initially, this process would be particularly slow and cumbersome since excavated materials needed to be stockpiled temporarily then brought back as backfill for the wall. It was anticipated that the time for this process would be shortened by utilizing excavated materials immediately as backfill in the wall. Another factor which would have some effect on the rate of construction was the individual delivery of each panel by skid which was to be winched up the 1.25h:1v slope by bulldozer. Panel delivery to wall location however could have easily been accelerated were it not for the space limitations at the wall location. The deciding influence on the rate of construction was therefore excavation and fill placement. On this basis, the erection rate was projected to be 15 to 20 m^2 of face area per day (or some 6 to 8 panels per day), probably the slowest construction rate ever projected for the RE system anywhere.

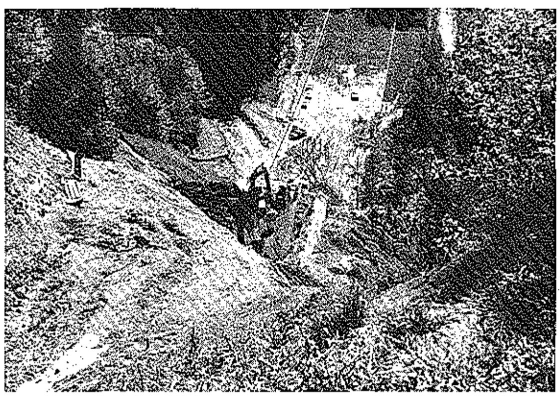

Fig. 7. Overall view of construction site.

Wall construction commenced August 20th 1978. The backhoe climbed to the construction level under its own power by making small platforms progressively with its bucket. It took about five hours to reach the construction level in this mode of operation. Excavation of the fractured rock to accommodate the strips proceeded without blasting and excavated rock broke into sufficiently fine fragments, generally less than 100 mm size, for use as fill. Placement of a coarse gravelly layer immediately above the filter fabric for drainage purposes was achieved successfully by utilizing the excavated rock first, and the mix of rock and much finer granular overburden next.

A timber skid was made for transporting wet concrete for the levelling pad, steel strips, precast concrete panels and other components up to the wall site. A Caterpillar D3 bulldozer with winch was set up at the wall site. It was tied to the backhoe so that it could winch up the loaded skid. Precast concrete panels, cast some 50 km away in Quebec City, were delivered to the bottom of the jump by tractor-trailer units. The unexpected refusal of truck drivers to deliver the precast panels via the winding access road with 25% gradient was soon resolved by limiting truck loads to half capacity or 10 panels per truck load.

The wall rose in horizontal lifts stepping up at the levelling pad as indicated by the rock and overburden conditions. Close liaison was maintained between the supplier/designer, the consultant and the contractor to determine where steps could be optimized and to maintain close control over the backfill material. In one instance the consultant noted a soft fill area; construction was immediately interrupted and approximately 10 m^3 of material was removed. The material was later checked and found adequate with respect to its grain size distribution, but as a precaution its source was not used thereafter as borrow material, because of the 'wet-of-optimum' water content at some locations. Rock levels were,

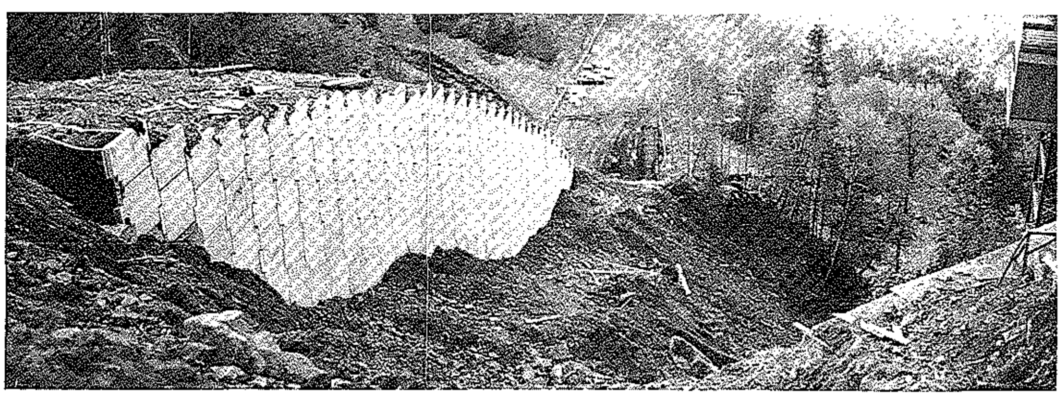

Fig. 8. Reinforced earth wall nearing completion.

of course, only roughly known in advance and actual stepping was subject to changes determined during construction. In fact, rock conditions were very close to those estimated beforehand requiring adjustments in only four steps. Fig. 7 illustrates the site condition during construction with the wall at its half point.

Initial delays due to poor weather, slow excavation, the refusal to truck the panels down the mountain etc. appeared to delay the scheduled completion at first, but subsequent good weather and reduced handling of fill improved the rate of wall erection to the point where it was completed ahead of schedule. the average rate of wall construction was, in fact, about 20m^2 of face area per day with the maximum recorded at about 35 m^2 per day, a very satisfactory rate of construction under the difficult circumstances. Fig. 8 shows the nearly completed wall. Note the cars in the out-run area and the judges' tower at the left as reference points.

To complete this case history, it is of interest to make a comparison of the cost involved to construct the RE wall and the estimate made for the alternative reinforced concrete wall. One other alternative was to relocate the ski jump but this solution was abandoned quickly in view of the work already completed at the first site and the schedule involved. Thus, compared with the nearest, feasible reinforced concrete retaining wall alternative, RE represented a saving of more than 60%. This saving contributed substantially to the fact that, in spite of all earlier problems, the overall project costs increased from the original estimate only by an amount equivalent to the rate of inflation experienced since the original budget was established.