IMPROVING CEMENTED ROCKFILL DESIGN IN OPEN STOPING

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ABSTRACT

Cemented rockfill, CRF, is comprised of sized aggregate mixed with various types and amounts of binder materials. This type of fill with closely controlled specifications is employed for subsequent pillar recovery and improved ground support.

The goal of this study is to improve consolidated rockfill design for bulk mining methods, with Kidd Creek Mines, KCM, as a case study, from a functional and cost point of views. Cemented rockfill at KCM represents approximately 20% of the total extraction costs. Cost cutting initiatives however have to be mindful of the negative if not disastrous effects on grade, recovery, and ground stability that a decline in fill quality can produce. This dictates that any attempt to cut operational costs should be approached in a scientific and orderly fashion. This thesis describes consolidated rockfill improvement steps taken at Kidd Creek to obtain the highest quality fill at the lowest possible cost.

The main trust and achievements in this thesis include:

1. Extensive site investigation and mapping in drift driven through backfill have resulted in establishment of four distinct zones in a typical rockfill mass. Structural rockfill design steps using the information obtained form field mapping are then established and implemented at KCM with great success which will be described during this thesis.

2. The main body of this thesis contains significant amounts of laboratory test work, 1750 test specimen, on lower cost binder alternatives. Some of the results obtained from the test work have been implemented at Kidd Creek since late 1992 and have resulted in considerable savings and improved dilution control.

3.Quality control measures and techniques in three main stages, surface plant, during transportation, and most importantly during placement are also established.

4. CRF structural design optimization steps are identified through extensive site observation and consultation with other operations. This covers all the steps that should be taken from start to finish to achieve the highest quality rockfill at the lowest possible cost.

5. Extensive field experiments are also carried out to obtain in situ mechanical and dynamic properties of a typical rockfill mass.

This work is based upon field and laboratory studies undertaken within the KCM over a 5 year period. The work has resulted in establishing quality control measures, mix design improvement, and structural design implementation at KCM to achieve the required physical characteristics at the lower operational cost. The in situ and laboratory test results proved that a more economical and more stable fill mass was obtainable and this has resulted in 35% unit cost reduction for KCM rockfill system within last 3 years and a saving of around $4 million on binder cost alone at a rate of $1.3 million / year. The total unit cost has dropped from $12/tonne in 1991 to around $7/tonne in 1995.

RÉSUMÉ

Le remblai en enrochement consolidé se compose d'un mélange d'agrégats et de différents types et quantités de matériel liant. Ce type de remblai est produit selon des normes rigoureuses et est ensuite utilisé lors de la récupération des piliers et pour améliorer le système de support.

Cette étude a pour but l'amélioration du design du remblai en enrochement consolidé utilisé dans les méthodes d'extraction en vrac, à la mine Kidd Creek, au point de vue pratique et économique. Le remblai en enrochement cimenté utilisé à la mine Kidd Creek représente environ 20% du coût total d'extraction. Toute initiative visant à réduire ces coûts devraient tenir compte des effets négatifs ou même désastreux qu'aurait un remblai de moins bonne qualité sur la teneur, la récupération et la stabilité du terrain. Ceci démontre que toute coupure dans les frais d'opérations doit être effectuée selon une approche scientifique et ordonnée. Ce mémoire décrit les étapes suivies à Kidd Creek pour obtenir un remblai en enrochement consolidé de plus haute qualité à un coût le moins élevé.

Cette thèse a accompli les points suivants:

1) une investigation et cartographie détaillée des galeries construites dans le remblai en enrochement a permis d'établir quatre zones distinctes dans la masse du remblai. Les étapes requises dans le design du remblai structurel ont été établies et implantées avec succès à la Mine Kidd Creek grâce à l'information obtenue de la cartographie détaillée.

2) ce mémoire décrit un nombre important d'essais en laboratoire, les 1750 spécimens utilisés et les matériaux liant les moins coûteux. Certains de ces résultats ont déjà été implantés à Kidd Creek depuis la mi-1992 et ont produit des économies considérables tout en améliorant le contrôle de la dilution.

3) des mesures et techniques de contrôle de qualité pendant trois étapes majeures soit, la fabrication, le transport et le placement du remblai sont aussi établies.

4) les étapes d'optimization du design structurel du remblai en enrochement consolidé sont identifiées par des observations détailiées en consultant d'autres opérations. Ceci couvre toutes les étapes depuis le début jusqu'a la fin pour arriver à produire le meilleur remblai en enrochement au coût le plus bas possible.

5) des essais détaillés sur le terrain ont été effectués pour obtenir les propriétés mécaniques et dynamiques du remblai.

Ce travail se base sur des études en laboratoire et sur le terrain entreprises à la mine Kidd Creek, pendant une période de 5 ans. Ce travail a comme résultats l'établissement de mesures de contrôle de qualité, l'amélioration du design du mélange et l'implantation du design structurel à la mine Kidd Creek pour obtenir les caractéristiques physiques requises à des frais moins élevés. Les résultats des tests en laboratoire et sur le terrain ont démontré qu'un remblai plus économique et plus stable pourrait être produit. Ceci a permis une réduction de 35% des frais par unité pour le système de remblai en enrochement à Kidd Creek durant les trois dernières années, soit une économie de 4 millions $ en ce qui concerne le coût du liant à un taux annuel de 1.3 million $. Le coût total par unité a diminué de 12 $ / tonne en 1991 à 7 $ / tonne en 1995.

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1 INTRODUCTION

Rockfill system refers to the use of coarse rock for backfilling compared to the use of tailings in a hydraulic fill system. The rockfill system could be unconsolidated, consolidated and/or post consolidated for different applications.

RF is normally used as a void filler for providing passive support, by minimizing regional ground movement problems. Unconsolidated rockfill has a limited ground support capability and the free standing height capacity of this type of fill is negligible. Unconsolidated rockfill is used when the filled stope will not be exposed in future pillar recoveries since the stopes surrounding the filled stope have been previously mined and filled.

In some operations e.g, Geco Mine, after filling the stope with coarse aggregate, the stope/pillar interface is post-consolidated with percolation of cemented hydraulic fill into the rockfill at the edge of the stope. The filling starts prior to completely emptying the stope and with gradual mucking of ore, uncrushed waste is introduced. Once the stope is filled with waste, single pour point at the top introduces 30:1 consolidated tailings fill to consolidate the waste.

CRF is comprized of sized or unsized aggregate mixed with various types and amounts of binder materials. The aggregate is consolidated just before the entry into the stope, Fig. 1.1 and 1.2. This type of fill usually contains a moisture content below 5%. CRF with closely controlled specifications is employed for subsequent pillar recovery and improved passive / active ground support. CRF yields a higher strength fill with lower amount of cementing agents compared to cemented hydraulic fill. With equivalent binder contents, CRF will exhibit uniaxial compressive strength two to three times higher than those of hydraulic fill. It also has higher modulus of elasticity, cohesion and angle of friction compared to hydraulic fill.

A variety of fill types could be used to suit the different mining methods and

Figure 1.1: Slurry is added to aggregate before entry to stope.

Figure 1.2: Mixing of aggregate and slurry.

performance requirements of the fill. Consolidated rockfill should be employed if future exposure of the fill wall is expected. The material combination and the cement contents of the fill placed may vary, depending on different stope requirements. These combinations could be as follow:

CRF (CONSOLIDATED ROCKFILL). In the majority of the operations, sized rockfill aggregate is mixed with cement slurry, usually 5 to 6% by weight of aggregate at a pulp density of 50-60%. This type of fill exerts an active pressure on the contacted wall, providing not only ground support but also improvement of inherent strength of the walls.

Advantages: There is no drainage problem . If placed correctly, high quality fill is achieved.

Disadvantages: Segregation control is difficult. Quality can be variable. Access & layout is critical.

CSRF(CONSOLIDATED SAND ROCKFILL). This is a combination of CRF with varying amounts of sand added to it, usually 5-10%. With the same cement content as CRF, the cement sand slurry is introduced simultaneously with the CRF to fill the voids in segregated aggregate. This enhances the fill stability for both gravity loading as well as blasting vibration resistance during excavation of adjacent stope or pillars, only used at KCM.

Advantages. Raise layout is not critical. It has relatively good mobility and less segregation than CRF. It has a lower angle of repose than CRF, and it is denser than CRF.

Disadvantages. Relatively good access is required and there is slurry runoff control problem.

CSWF(CONSOLIDATED SAND WASTE FILL). Waste is left in place and consolidated by pouring a cement sand slurry mixture which percolates through it. The cement by weight of aggregate is around 18% and has a pulp density 55-60% for cement slurry and 65-70% for sand slurry. The richer amount of cementing agents in the mixture will consolidate the waste for improving ground support and also reducing ore dilution with overall cement content of around

5%)

Advantages. It is very mobile and good access is not essential. It saves having to remove waste and can consolidate specific areas of stope, such as individual walls.

Disadvantages. Bulkhead control is essential. The path of the sand slurry flow is difficult to control.

CSF(CONSOLIDATED SAND FILL). Consolidated sand fill is a cement-sand slurry mix, with a lean cement content, 5 to 10%. The cement sand slurry is placed after the majority of the stope has been filled with CRF in order to tightly fill the remaining void beneath the stopeback, thus providing roof support.

Advantages. It is very mobile, and has a low angle of repose. Good access is not essential.

Disadvantages. Bulkhead control is essential and there is slurry runoff control problem.

RF (ROCKFILL) This is sized or unsized waste, which is obtained from surface or underground. For cost savings without impairing the role of fill, RF is occasionally used in some selected openings. The unconsolidated fill, however, can only be regarded as applying a passive pressure on the wall to be supported, and has a limited effect on supporting ground.

Advantages. It is cheap, quick and simple.

Disadvantages. It is not consolidated, thereby, offers limited ground support.

Within last 10 years, the strength and stiffness which can be achieved with CRF have encouraged its increasing use as backfill. There are strong economic links between the strength of rockfill, the stable fill exposure dimensions, the possible stope dimensions and hence the profitability of a mining operation. CRF has the capacity to stand over exposures not economically possible with other fill types. The stiffness which can be achieved with rockfill offers some significant advantages to operations seeking to ground control subject to high stresses, control which is not possible with any other fill type. Rockfilling also provides an inexpensive method of disposing of development waste and or surface open pit waste.

A proper design of rockfill system is relatively complex and involves number of parameters which require appropriate attention in order to ensure a continous, efficient and low cost operation. A proper design of rockfill system can contribute greatly to the efficiency and profitability of the mine.

This thesis will critically review the main CRF operations around the world and together with over 15 years of documentation and experience on rockfilling system at KCM will identify the major parameters to be investigated for an efficient and low cost rockfilling operation.

Then based on extensive literature review and author's extensive site investigation which included, drift mapping, stope history and actual filling observations, four main parameters related to CRF design are identified and further investigated in this thesis. The four areas are identified due to their importance in achieving high strength fill and lack of available information on extent of their contribution in assuring a properly engineered CRF system. The four areas identified with the most strength improvement and cost saving potentials are: to improve engineered structural design process, establish quality control measures, increase usage of lower cost alternative binders, and carry out in situ testing for back analysis.

Structural Design: Structural design optimization is a very important part of the fill cycle. This is needed to predict, place, and obtain competent rockfill mass where it will be exposed in future mining. The ultimate goal is to minimize and control segregation, hence minimize dilution and mass and/or local failures.

Quality Control : A typical rockfill mass has much superior physical and mechanical properties compared to other fill systems if only properly controlled. Closely monitored and properly engineered quality control measures have to be established and followed by operations people. These are the measures taken to achieve the highest possible fill quality at the lowest possible cost.

Binder Alternative: Since binder usage is around 80-90% of a typical rockfill operating cost, excluding labour cost, the establishment of an optimum binder combination in any mine is a must and could yield the most financial and/or strength benefits.

In Situ Testing : Very little work had been done on the in situ behaviour of a consolidated rockfill mass. Optimum fill quality at the lowest possible cost can only be achieved by back analysis and continuous improvement of the existing fill system using actual in situ values.

The investigation of above parameters, as the thesis for Ph.D studies, were considered to be significant in advancing Canadian mining backfilling technology. The majority of laboratory and in situ experimentations have been unique to the Canadian mining industry, mostly due to lack of expertise and the substantial cost involved.

This thesis investigates the geomechanical behaviour of a rockfill mass which could be exposed on all four walls and also investigates cost and quality improvement potentials in four areas mentioned above. The thesis establishes design criteria for mass design optimization and reviews different fill mass in situ strength estimation techniques. This will enable the designers to evaluate and estimate the required in situ static and dynamic strengths of cemented rockfill, for any given condition and aim to achieve the highest possible strength at the lowest possible cost.

This thesis will present the first fully evaluated rockfill system in Canada. This involves improvement and optimization in preparation, transportation, and most importantly placement of the rockfill material to achieve the highest quality fill at the lowest potential cost at Kidd Creek Mines. However, the result of this work can be easily employed and implemented for any new or existing rockfill system.

This thesis has been the product of work conducted as an employee of Falconbridge Limited and a Ph.D student at McGill University. As such, the author readily recognises the assistance provided to him by other KCM employees and those of the other operations during author's site visits.

Most of the studies are carried out to improve the KCM fill system which is the largest rockfill operation in Canada. The emphasis in this thesis is on rockfilling large blasthole stopes, which is the mining method at KCM. Prior to this study, extensive research has been carried out in the mine in the field of rockfill mass design. Most of the related test results are either presented or referred to during this thesis.

The Kidd Creek minesite is located 27 km north of Timmins. The annual production of the mine is around 3.5 million tonnes of copper, zinc, lead and silver ore. The mining method is sub-level blasthole stoping. Typical stopes in the upper mine (above 2600 level) are 18 m wide and from 35 m to 60 m long depending on ore contacts. Stope heights range from 75 m to 105 m depending on footwall or hangingwall ore limits and ground conditions in the vicinity of the stope. Secondary recovery of the transverse pillars, 25 m in width is carried out between filled stopes. The consolidated rockfill quality has a major impact on the dilution and recovery of these pillars. The mining method in the central area of the mine (2800-4600 levels) is pillarless blasthole stoping and a typical stope is 15 m wide, 30 m long and up to 60 m high. The lower (below 4600 level) has the same method as the upper mine, using rib pillars. Stopes are however, only 15 m wide, 30 m long, and 35 m high.

Currently, KCM produces 3.4 million tonnes of ore annually. A total of 1.8 million tonnes of backfill is required to completely fill the mined out areas. Approximately 95% of the fill placed is consolidated material. Backfill is placed to provide both short and long term benefits. Short term benefits include: the complete recovery of ore, safe mining of adjacent stopes, and wall sloughage control. The long-term benefit is to stabilise the general mining environment. The quality of backfill has a major impact on recovered ore grade and the quantity of the fill delivered must be matched to the budgeted production rate. The fill method directly and/or indirectly effects: mining cost, mining rate, stope sequencing, and final grade due to controlled dilution. With the same amount of cementing agents consolidate rockfill mass yields a higher compressive strength, modulus of elasticity, cohesion, and angle of friction compared to any other fill type.

The main factors for employing a CRF system at KCM are as follow:

A: Initially the rockfill system was adopted as the backfilling method because of the substantial stockpile of ideal waste material produced from the open pit phase of mining. Approximately 55 million tonnes of rhyolite and andesite waste rock were produced during the open pit operation.

B: High filling capacity requirement at KCM, which only could be supplied by a CRF method. At KCM, approximately 1.8 million tonnes /year of placed fill is required to keep up with annual production of 3.5 million tonnes.

C: CRF with lower binder content achieves uniaxial compressive strengths two to three times higher than those of hydraulic fill. It has superior modulus of elasticity, cohesion and friction angle compared to hydraulic fill, hence provides a more active ground support role.

D: Unusual distance between mine and metallurgical sites at KCM eliminating the possibility of using classified tailings in a hydraulic fill system.

The objectives of this thesis consist of the following:

1: To review rockfill practices around the world with emphases on Canadian Mines, especially KCM in Timmins, Ontario.

2: To establish structural rockfill design optimization criteria. This is done through extensive site visits and employing different in situ testing techniques.

3: To establish quality control measures and techniques in three main stages, surface fill plant, during transportation, and the most importantly during placement.

4: To review material properties testing/ estimation techniques, for a typical rockfill system.

5: The main portion of this thesis examines lower cost binder alternatives. Much of the test work was aimed to get the lowest cost binder which performs as good or better than the existing binder at KCM.

At the start of this research thesis, there did not exist a general design procedure/ technique for assessing the properties and strength requirements of a CRF mass. Another major portion of the thesis is to establish the required structural design and in situ estimation technique criteria.

In this thesis the following are considered to be author's contributions to the field of rockfill design:

1: Improvement in rockfill design strengths estimation and requirement methods.

2: Establishment of rockfill optimization path.

3: Comprehensive in situ testing to find mechanical and dynamic properties of a CRF mass to be used for an engineered design. This is done through different monitoring techniques and underground observations. This area is probably the least understood part of a CRF mass.

4: Establishing new quality control measures and techniques.

5: The most important part of this thesis is the investigation into more effective and lower cost binder alternatives.

This thesis contributes greatly in understanding the steps to be taken to improve and even optimize any rockfill system with emphasis on KCM rockfill system. Chapter 2 reviews information from literature survey of other CRF operations. Following this, in chapter 3 the review of engineering structural design for an homogeneous rockfill mass is carried out. Chapter 4 describes the extensive site investigation program at KCM to identify parameters for a non homogeneous mass, to be further investigated. Chapters 5 to 8 report on the studies carried out on 4 main parameters that were identified in chapter 4. The parameters investigated in chapters 5 to 8 are : Structural design criteria, physical/ mechanical properties, quality control measures and extensive in situ testing program, respectively. Chapter 9 contains some capital and operating costing information for a typical rockfill system. Chapter 10 summarizes all the improvement made in main parameters and presents the steps to be taken to improve and/or optimize a typical rockfill system. Conclusion based on the experiments and findings of this thesis plot a course for the development and implementation of an improved rockfill system.

2 REVIEW OF CRF OPERATIONS

Considerable amounts of information regarding rockfill operations are available around the world. Due to variation in mining method technique, orebody shape and grade, each rockfill operation has its own advantages and disadvantages. It is hoped by critical review of such systems, major parameters effecting overall performance of CRF will be identified which are common to all rockfill operations. Such parameters are then further studied in depth for possible improvement throughout this thesis. This chapter covers the information from major CRF systems in the world, with emphases on Canadian CRF operations. The rockfill accounts for 30% of all the fill types in Canada, while it is only 6% when one includes all the mines in the world which use some type of backfill. This indicates that Canadian mining industry possibly has the most advanced technology on CRF.

Advantages of a CRF as compared with hydraulic fill system is the higher filling rate and strength with lower cost. It has higher compressive strength, cohesion, friction angle, modulus of elasticity with lower binder content. There is no drainage problem and it is a continuous filling system. This translates to average filling rate of 5000 tonne/ day and filling rate of up to 15000 tonnes/day at KCM. Studies in rockburst prone mines have clearly indicated that CRF plays much more active ground control role than hydraulic fill. (Quesnel,W.J.f, et el. 1989)

Almost every mine that used CRF system was employing blasthole stoping method in both the longitudinal and transverse direction. Stope size ranged from 125 m long by 3 m wide to as large as 61 m long by 24 m wide.

For all the fill systems, the inert materials commonly used were mill plant tailings, sand and gravel, waste rock and slag. In Quebec 61% of the materials used in all the fill systems was waste rock. Typically waste was crushed /screened to either minus 200 mm or 150 mm depending on the mine. CRF at Kidd Creek and Williams mine consisted of an aggregate crushed to 75 percent between sizes of 9.5 mm to 150 mm and 25 percent minus 9.5 mm.

Almost all operations were using some sort of binding material, cement being the most popular. Some operations used the combination of cement/ flyash and/or slag. Although the Portland cement/ Flyash combination allows one to obtain higher long term strength, in Canadian mines, flyash was seen only at Kidd Creek, Williams, and 5 Quebec mines. Even in the mines that flyash was being used, the replacement percentage was considerably lower than optimum replacement value of around 60%. Five mines in Quebec were using flyash in conjunction with the Portland cement, with replacement ranging from 8% to 50%. Chemical additives such as flocculent, accelerator, retarder were employed to improve the fill permeability, flowability of the slurry and the consolidation properties of the fill. The binder content was often more than usual 5% where higher free standing ability was required.

Unconsolidated fill was placed in stopes where no adjacent mining would take place. The waste was either dumped from surface into large raises directed to the underground levels where filling was taking place, or it was trammed directly to the stope to be filled from an underground development area. The raise were commonly between one to 2.4 meters in diameter. At Williams waste was transferred laterally from raise to raise by conveyor s in order to reach the lower stopes. Kidd Creek uses conveyors to take the material directly to the open stope to be filled.

The Geco method for transport of CRF was different in that it required that filling operations start prior to completely emptying the stope of its muck content. As muck was drawn at the drawpoints, fill was introduced at the top of a blasthole stope. This greatly minimised the wall sloughage. When the stope was filled with waste then 30:1 consolidated hydraulic fill through a single pour was used to post consolidate the aggregate.

Different mixing of aggregate and binder was used in different operations. In majority of operations slurry was piped from surface and then agitated in holding tanks located near the stopes to be filled. The slurry binder mixture was then pumped through 4 in. slurry lines and sprayed onto the aggregate as it was dumped from a conveyor into a baffled chute. The chute tumbled the materials and ensured full coating of the aggregate. In truck filling operation, the trucks were loaded with aggregate and sprayed with binder prior to dumping into the open stope.

Most of the mines were using 4 inch diameter perforated pipes which came in customized length. When filling with CRF water was needed only for cement hydration and aggregate coating and very little decanting was required. However, at the bottom of the stope 4 inch pipes were placed to decant excess water from mixing, or ground water.

Typical fill fences used in blasthole stopes included either a cable fence as shown in Fig. 2.1 or timber fill fence. The bottom portion of the stope was filled with higher cement content to act as plug. Kidd Creek used a 14 inch thick concrete bulkhead, Fig. 2.2, with holes to accommodate drain pipes. However, Kidd Creek has started using fill fences due to their ease and speed of installation.

The compressive strength of the fill required in mining operation varied over a wide range. This was from 1.4 Mpa to 7 Mpa.

Monitoring methods were very simple and crude. There were just few automatic control systems in use and most mines were monitoring manually.

There was little data regarding the quality control of the in-situ backfill with respect to its physical and mechanical properties and the above concern was clearly examined throughout this thesis.

The backfill cost accounted for 1.3% to 19% of the total mining cost. The capital cost for different rockfill operations varied considerably, ranging from $500,000 to $2.2 million 1991 Canadian dollars.

Operating cost, expressed in dollars per ton of fill, ranged from 4 to 15, depending on fill preparation, placement system and more importantly binder content.

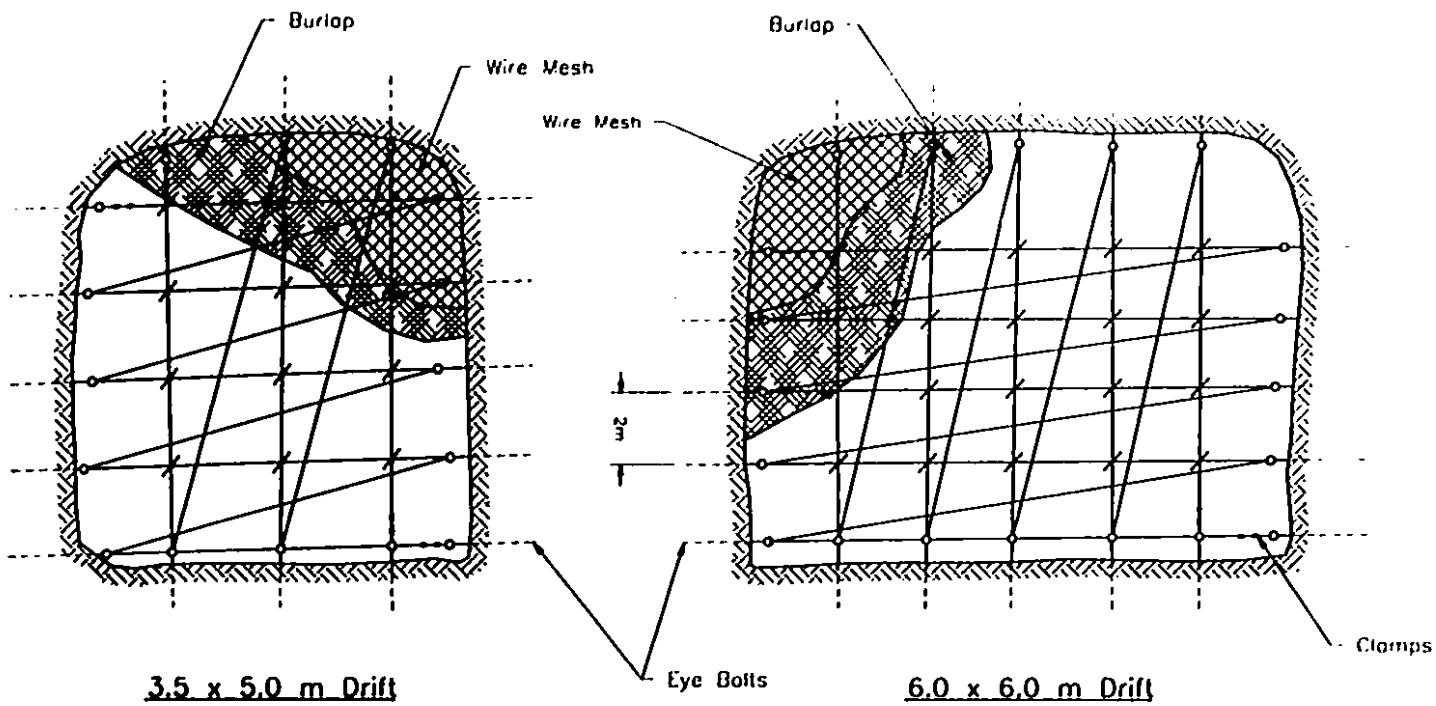


Fig: 2.1 Lac Shortt
Typical Fill Fences

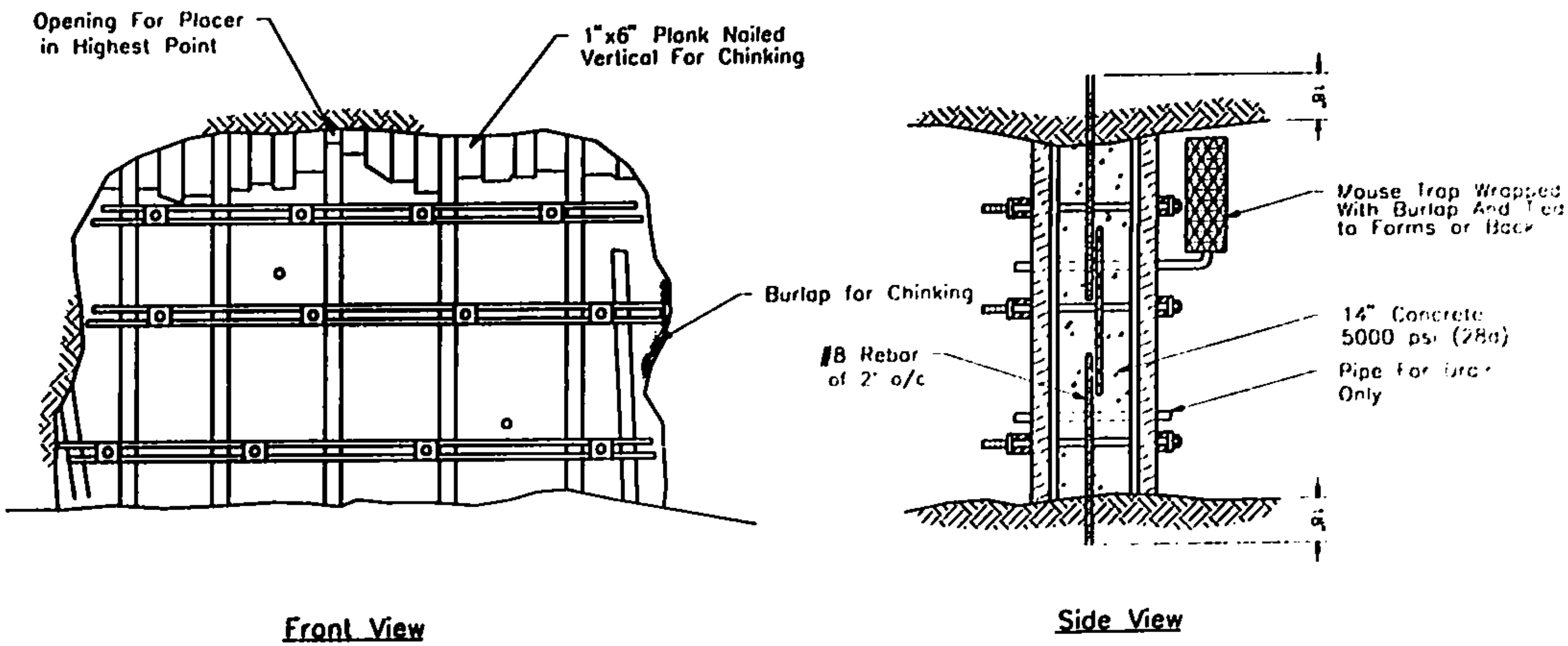


Fig: 2.2 Kidd Creek Mines
Standard Concrete
Bulkhead

In rockfill operations, material cost accounted for 43% of the total operating cost of which 80% was the binder cost. For Quebec Mines, the capital cost for all fill types varied in the range of 0.4 to 2.2. million dollars. Rockfill operating cost was between 3.8 to 9.5 dollars per ton of fill. This is compared to operating cost of 5 to 25 for cut and fill and 3.9 to 11 for hydraulic fill systems.

Rockfill was more widely used in Quebec. Approximately 40% of the mines surveyed either use rockfill alone or with hydraulic fill.

There were typical problems and concerns associated with CRF systems. They were as follow:

1: Segregation of aggregate

The main reason for fill failure in a CRF system is aggregate segregation and close control during placement is necessary to get the least segregation which then should minimize dilution. This is the biggest concern for any CRF system and the segregation phenomena is studied in depth in chapters 4 and 5 with recommendation to minimize the extent of the segregation.

2: Excessive fines at depth.

Aggregate breaks down while being transferred underground and close control on aggregate sizing could be difficult. This is covered in chapters 4 and 7.

3: Difficulty in in situ evaluation of placed CRF.

In most of the mines trial and error method was the only way to experiment with changes in the system. The physical and dynamic properties of CRF are extensively studied and results and recommendations are given in chapter 8.

4: High capital requirements.

The cost of pass and plant set-up could be very high. For example at Kidd Creek Mines, 2000 meters of vertical raises and 3000 meters of conveyor belt installation are used for aggregate delivery. This indicates that CRF system is most suited to large operations and bulk mining methods. Operating and capital cost information are given in chapter 9.

5: Frozen aggregate on surface, causing hangups and higher moisture content. The solution to this problem is covered in chapter 4 and 7.

2.1 ROCKFILL PRACTICES IN ONTARIO

Some of the main rockfill operations in Ontario are:

2.1.1: GECO MINE

The orebody is an easterly trending, sub-vertical zone occurring on the south limb of an easterly plunging folded sequence of highly metamorphosed Archean volcanic and sediments known as the Manitouwadge Synform. The orebody is a lenticular, continuous zone of mineralization extending from surface to a vertical depth of 975 meter along an easterly 35 deg.plunge. The average strike length of the orebody is 731 m. The width varies from 3 to 76 m, with the average being 20 m. The core consists of massive pyrite, pyrrhotite, sphalerite, chalcopyrite and galena. In addition to copper and zinc, the ore body carries silver and minor quantities of gold and tin. (Olav Svela, 1989).

MINING METHOD AT GECO:

Access to the underground orebody was first provided by No.1 shaft, which bottomed at the 746 m level. An internal shaft east of No.1 shaft, provided additional depth penetration as mining went deeper. Geco has three distinct orebodies: the main zone, which has provided the bulk of the ore; and the 4/2 and 8/2 zones. Blasthole mining has been the major mining method at Geco. Transverse pillars are 37 m wide and each side of the stopes and every third pillar is 46 m wide, boundary pillar. Transverse stopes and pillars are in the lower part of the orebody and are 10 m or thicker. Anything narrower than 10 m is mined longitudinally. (Olav Svela, 1989). This mine is currently producing approximately 1 1/2 million tons of ore, 4,100 tons/day, recovering Cu, Pb and Zn. The sizes of stopes are 30 m x 21 m x 60 to 200 m (L,W,H)

BACKFILL DESIGN AT GECO

Well fragmented, unconsolidated and uncrushed waste rock can be fed directly to stopes through finger raises. When blasting of the stope is underway and mucking has started a void will develop at the top of the stope. Waste is now passed through a finger raise which has been driven directly over the stope. As the void fills the raise will chock off until more ore is pulled. The waste continues to fill the void as it is created by mucking from the bottom of the stope and provides support keeping wall slough to a minimum. This technique has been found to minimize wall sloughage by providing continuous support at the walls of the stope. When the stope is emptied of ore it will be full of waste and if it is a pillar this will be the end of the filling cycle. Unconsolidated materials are used for backfilling areas where adjacent mining will not occur, such as in mined-out pillars where primary stopes have been previously mined and filled, or in stopes where unrecoverable pillars are left. If stopes are to be mined adjacent to this block then a single pour point at the top of the stope will introduce 30:1 consolidated tailings fill into the waste rock. Pouring continues until the stope is full. The standard bulkheads are installed prior to pouring the cemented tailing fill. For consolidation purpose, the backfill comes from the mill's flotation tailings. The coarse fraction is used in backfill, while the fines are sent to the tailings impoundment area. The backfill plant produces 300,000 tons of backfill sand per year in both consolidated and unconsolidated form.

The tailings fill plant area has a 200 ton cement silo, two mixing tanks and one surge tank. The cement ratio is obtained by setting weightometer scale to a calculated cement to classified tailings fill ratio of 1:30. A screw conveyor feeds the cement into the mixing tank. Cement is the only additive presently used. Density of the mixture is 64% and is checked with a Marcy scale.

Surface waste from a quarry is passed through three fill raises . Finger raises to these main raises feed waste directly to stopes. The waste is uncrushed but well fragmented.

A standard 5' thick reinforced concrete bulkhead is installed at each opening into the stope prior to pouring the consolidated tailings fill. Over 1000 of these bulkheads have been installed at Geco. Drain lines are installed with a mousetrap on the active side.

In summary Geco mine has a unique post consolidation method. This method allows immediate wall slough control, however post consolidated rockfill in general has much lower in situ strength. Portland cement was the only binder used in this operation and considerable savings could be realized by using lower cost but more effective binders such as flyash.

2.1.2 : WILLIAMS MINE

The mine hosts an ore body with proven and probable reserves of 34 million tonnes grading an average 6.2 gr of gold per tonne. Annual production in the mine is slightly higher than two million tonnes. Average daily production is 5,400 tonne. This mine produces approximately 500,000 oz of gold per year, making it Canada's biggest gold-producer. The production and operating costs in this mine is around $210 (US) per oz of gold.

MINING METHOD AT WILLIAMS MINE

Longhole open-stoping with delayed backfill is the sole means of production. Open stope blasthole method is carried out with drilling of 4 1/2" downholes and blasting to a Robbins slot raise. Main levels are established at 105-m intervals, and sub-levels every 25 m. Three main mining blocks exist in the mine. The top block, at 300m depth can provide 750 to 1,000 tonnes of muck per day. The second block, between 785 to 890 m depth, contributes the bulk of the daily tonnage of 2,500 tonnes. The third mining block, between 995 and 1,100 m depth, produces around 2,200 tonnes/day.

Stope dimensions run 25 m in height, 20 m along strike, and to a maximum width of 25 m. Any stopes in thicker zones are split into two panels to allow more efficient mining. The dilution factor is 8 to 9%, including material from backfilled stopes. The 42-inch-diameter slot raise serves a dual function as initial blast void and as backfill raise for mined out stope below.

BACKFILL DESIGN AT WILLIAMS MINE

Backfill is supplied from a quarry at surface called the C zone open pit and fed through raises to underground. Quarry rock is mixed with cement slurry on 26-tonne trucks at backfill station on the main levels. Straight quarry rock without cement is placed between two consolidated backfill stopes. Surface waste from the open pit is crushed to minus 6" resulting in a ratio of 75% +3/8" and 25% -3/8". This aggregate is stacked and picked up by front-end loaders as required and dumped into backfill raises. At the first underground level the fill may be passed to the upper zone or conveyed by permanent conveyor to raises which will pass it to the lower zone.

There are usually three areas being filled at one time. One area used to be filled by portable conveyors and the other two were using scoops which have been replaced by fill trucks. Portable conveyors carry the aggregate to the borehole or to the open stope. Baffled chutes are often used at the head end of the conveyor to further mix the slurry with the aggregate. Because of small size of the stopes all the stopes are presently filled using 26 tonne trucks. Almost 100% of the present filling at Williams are done using 26 tonne truck due to the small stope sizes.

When using LHD, a conveyor loads directly into the bucket and metered amount of binder is piped into the aggregate as it falls into the bucket, see Figure 2.3. The scoop then travels to the dump site which is usually a Robbins raise or it can be dumped directly into an open stope. It should be noted that the Robbins raises are drilled through several sub-levels in one lift and are used as both slot raises and fill raises. The choice of these two methods is based on an economics decision. Normally 40,000 tonnes of fill or less will result in using the scoop haulage method. Fill production is approximately 4,000 tons per day. Overall cost of placed fill is $10.0 per ton. The only fill used at this mine is CRF with cement/ flyash as the binder in ratios of 4 to 7% by weight.

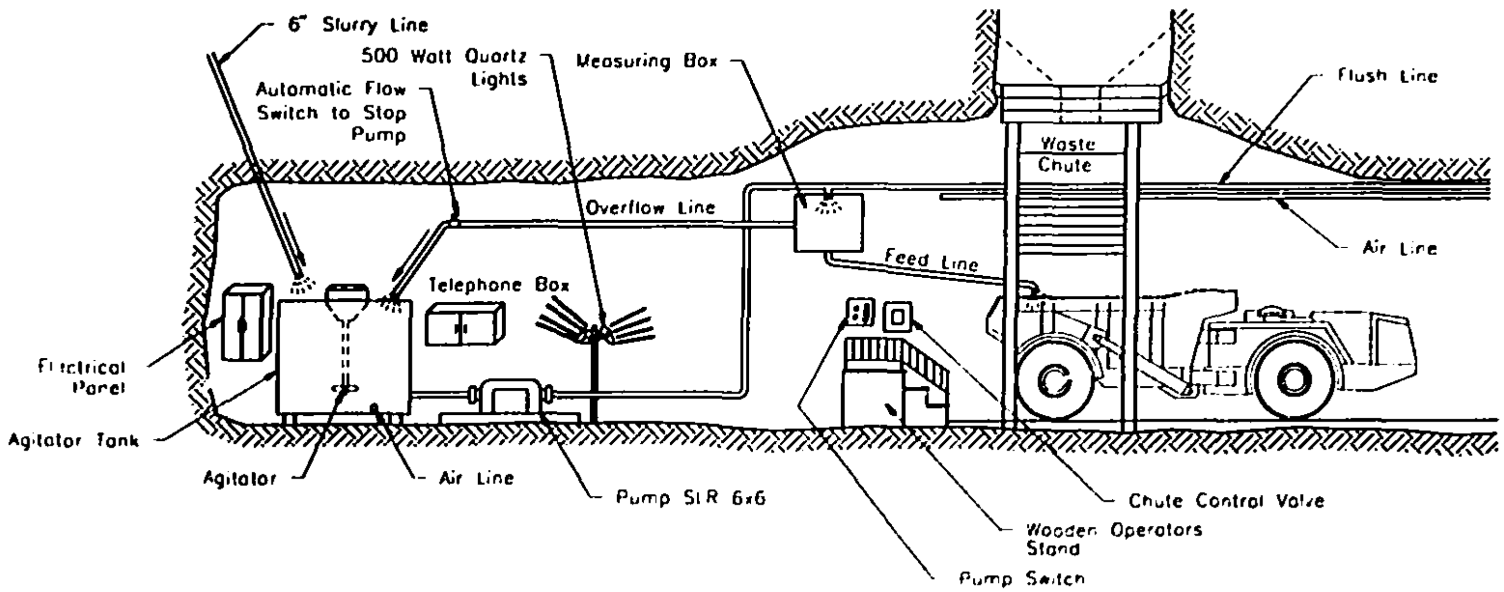


Fig: 2.3 General Arrangement of Underground Backfill Station

Standard fill fence provides excellent and clean drainage of the water. Where ground conditions have been deteriorated, for safety reasons, self forming bulkheads are poured using a 7% cement to aggregate ratio. Fences have drain pipes and fabrene for water control. Here, a hinged trap door allows access to both sides of the fence during the erection process. A drain pipe is placed through the concrete base of the fence and another pipe through the concrete sealing the sides of the fence to the rock walls. The fence is made of timber with welded mesh screen and filter cloth covering. It is secured to the walls and back using rebar and bullhorn attachments. Concrete is poured along the sides of the fence to prevent leakage at the walls.

In summary, the fill system at Williams mine is very similar to the system at Kidd Creek mines. The major difference is the smaller size of the stope to be filled at Williams, around 30,000 tonnes, that dictates 100% truck filling while Kidd Creek uses mostly conveyor for aggregate transportation. Although flyash was added to replace Portland cement, the percentage replacement was only 40% and as the result obtained from this thesis indicate the value could be as high as 60%.

2.1.3 : GOLDEN GIANT MINE

Golden Giant has reserves of 15 million tonnes at a grade of 10.85 gr gold per tonne. The production rate is 3,000 tonnes per day. The ore at this mine comes from three blocks : block 1, between 321 and 421 m depth, is essentially mined out, except for a 180,000- tonne pillar. Stopes in this block were mined to a 100-m height, but because of the ground movement problems, 50-m lifts were used for No.2 block. Stopes in block 2, between 446 and 546 m depth, are 50 m high, 20 to 25 m wide and 15m along strike. Block 3, between 571 and 921 m depth, is the most active block. From the experience in the upper two blocks, the optimum stope height of 66m has been employed in this block.

MINING METHOD AT GOLDEN GIANT

The mining method is blasthole stoping, blasting into slot raises. These raises are dropped using 4 1/2" diameter In-The-Hole (I.T.H) drills for the cut and 2 1/8" diameter for the square. A retreating panel sequence is being used rather than leaving pillars. The average stope size in the upper zone is 100m x 25m x 20m (H.L.W) while in the lower zone the average dimensions are 50m x 15m x 20m.

BACKFILL DESIGN AT GOLDEN GIANT

Backfill consists of a combination of quarry rock (90 to 95%) from surface and cement (5 to 10%) mixed underground at strategically located backfill stations. The fill plant has one cement silo with 150 tonne capacity. Cement ratios are approximately 3 to 4% but in the past ratios of up to 10% have been used. Cement is added as preset on a computer and is automatically monitored to stop when S.G. of slurry reaches 1.77 which corresponds to a density of 64%.

Surface waste from a quarry is crushed to minus 8" and passes underground through fill raises. Development waste is occasionally added to the crushed rock on a 50-50 basis. When this is done the crushed rock is screened to remove all fines below 3/4". This is due to the nature of the schist waste tends to provide excessive fines.

Self-forming plugs are formed by piling muck in drift or crosscut to act as a catch fence to stop the flow of C.R.F. as it pours out of the stope. The angle of repose of the fill is about 33 deg. The cement ratio is 4% but close monitoring by the operators can have the amount of slurry reduced to raise the fill density and lower the flowability. The small percentage of water in the fill percolates from fill and flows to sump areas.

All the filling at Giant is currently done by dumping the CRF directly into the stope or through a drop raise. Fill trucks holding 17 tonne of consolidated fill are used. Same as Kidd Creek and many other mines the aggregate is loaded into trucks and sprayed with slurry prior to dumping in the open stope.

In summary, the general aggregate size in this mine is considerably coarser than other rockfill operations. The lower cost binders were not being used in this operation. Both Golden Giant and Geco mines had higher than average pulp density for their slurry transportation due to limited horizontal slurry lines. Foe example, the pulp density at Kidd Creek mine only averages around 57% due to extensive horizontal line to cover the large orebody.

2.2 : QUEBEC ROCKFILL OPERATIONS

Extensive survey has been carried out on Quebec mines using CRF alone or in conjunction with cemented hydraulic fill and details are given elsewhere, Hassani, 1992.

Review of this detailed survey indicates that CRF is the most common fill type in Quebec. Due to the smaller size of stopes almost all the filling is done using truck to haul aggregate. The fill plant and fill methods are similar to the ones in Ontario and the only distinct difference is that the mines in Quebec have considerably higher consumption of lower cost flyash as their binder. In the five mines using flyash as Portland cement replacement, the flyash replacement varies between 10 to 50%. With the results presented in this thesis this could be brought up as high as 60% which translates to considerable savings.

2.3 : MOUNT ISA ROCKFILL SYSTEM

Mount Isa Mines, MIM, Limited operates two underground base metal mines, Isa and Hilton, at Mount Isa, a city located in north-west Queensland, Australia. Copper ore, at an average of 3%, is mined from the 1100 orebody, a large shallow dipping orebody with a north-south strike length of over 2000 m, a maximum width of 500 meters and a maximum height of almost 400 m. The orebody is mined over the full height using sub level open stoping.

The fill exposures are generally 40 m wide and can be in excess of 200 m high. The cost effectiveness of the orebody is significantly influenced by the filling costs which account for approximately 20% of the total mining costs.

At MIM both uncemented and cemented backfill are used in underground operations. Uncemented hydraulic fill in cut and fill stopes forms working floors and cemented fill are used to fill the voids created during open stoping where stable fill exposures are required during the mining sequence.

Three categories of cemented fill are used at MIM; cemented hydraulic fill (CHF), cemented aggregate fill (CAF) and cemented rock fill (CRF). CHF is produced by adding Portland cement and Copper Reverberatory Furnace Slag (CRFS) to deslimed tailings from the on site copper and lead concentrators. CHF contains typically 91% tailings, 6% CRFS and 3% cement by weight and forms a material which has the strength of a 6% cement CHF material.

Aggregate for CRF system is the local siltstone which is crushed and screened to produce +25 mm, -300 mm rockfill. The rock fill is transported along the surface via conveyors, choke fed down fill passes and along conveyors underground to the top of the stope being filled. The simultaneous placement of rockfill and CHF into a stope produces a combined fill which is termed cemented rockfill. (Ratio of RF/CHF is between 1:1 to 3:1 by weight). The characteristics of this fill vary within the stope due to segregation of the two fill constituents during placement. CRF is used exclusively in the copper open stopes.

Aggregate fill, AF, uses rejected material, -25 mm, from the heavy medium separation plant. This material is mixed with CHF at typical ratio of 25:75 AF:CHF by weight and transported hydraulically through pipes to the stope.

Some of the findings from reviewing backfill operation at MIM are:

- MIM CRF grading curve possessed the largest particle size and the greatest size distribution of all fills reviewed.

- MIM CHF and CRF contained 1.5% and 1.4% cement equivalent respectively, well below typical binder contents.

- MIM CHF possess the highest reported porosity value of 0.47. Mount Isa CAF and CRF, being well graded fills, possess significantly lower porosity values than most other reported fills (0.22 and 0.11 respectively).

- All three MIM backfills have strengths between 0.5 to 1.3 Mpa, although the strength of CRF could vary up to values grater than 2 Mpa as a result of variations in in-situ properties.

Summary of all above surveys are as follow:

- The distribution of the various fill categories around the world were: Paste fill (14%), HF (49%), Sand fill(9%), AF(23%) and RF(6%). This indicated that rockfill was still considered as a new technology and lack of in-situ information had limited it's application.

- Australian fills were dominantly HF and AF/RF mixes.

- KCM rockfill mass achieved the highest compressive strength of all the mines surveyed.

- Canadian fills were largely dominated by HF.

- The majority of binder contents were approximately 6% to 7% cement equivalent

- The majority of porosity values were between 0.30 and 0.50

- More than 50% of the fill strengths occurred between 0.5 to 1.3 MPa which suggests that the majority of the applications for backfill require design strength within this range.

- There was a notable spread in friction angle values. The mean value was 35.1 with a standard deviation of 9.3. Seventy percent of the values were between 30 and 40 degrees.

- The strength trend was: Rockfill > Aggregate fill > Sand fill > Paste fill >Hydraulic fill.

2.4 SUMMARY

This chapter indicated that similar operations have completely different rockfill application and strength requirements. This clearly indicates that a properly engineered structural design approach is needed. This approach should cover the strength requirement and estimation techniques which will be covered in chapter 3 and 8, respectively.

The review also indicated that most of the fill failure problems are due to segregation phenomena and lack of established quality control measures. Segregation effect can not be eliminated, however with proper structural design the segregation extent could be minimized. Both of these key parameters will be covered in chapters 5 and 7, respectively.

The review also clearly indicates that ultimate strength and operating cost are heavily effected by the type of binder being used. Binder cost translates to around 80% of the total material cost in a typical CRF system.The potentials for strength improvement and lower operating cost exist in almost all the mines. Chapter 5 clearly examines this potential improvement area.

3 STRUCTURAL DESIGN REVIEW

There are different and limited theoretical analysis from the field of rock mechanics and soil mechanics to allow one to structurally design for appropriate static and dynamic strength requirements of a CRF mass. All these analysis consider the entire CRF mass to be homogeneous and does not take into account the extent of segregation and different zoning in a typical CRF mass.This chapter will review the available techniques for strength requirement estimation for the homogeneous CRF mass and the following chapter will investigate the relationship of extent of the segregation to the decreased static and dynamic strengths.

Segregation phenomena is the main reason that operations use different safety factor numbers to account for in situ non homogeneity of the CRF mass. In most of the cases, the strength requirement is over and/or under estimated and in both cases the results could be very costly. Under estimation of the actual strength required will result in excessive dilution and major production delays and over estimation means much higher operating cost. For example an extra 1% binder requirement at KCM translates to increased binder cost of around $1.8 million/year.

For a rockfill mass to sustain not only the gravitational loading of the overlaying fill material, but also the dynamic loading applied during blasting of the adjacent areas the following should be considered:

1: Compressive strength required , confined and unconfined, with a given safety factor,
2: Shear strength,
3: Dynamic tensile strength,
4: Elastic Modulus,
5: Poisson's ratio,
6: Friction angle,

7: Cohesion and apparent cohesion.

All the above factors are needed to be able to estimate the static and dynamic strengths requirements and will be discussed in detail during this chapter.

3.1: STATIC STRENGTH REQUIREMENTS:

At this stage one just considers the gravitational loading of the fill due to it's own weight. The competent fill should have enough strength to prevent shear or tensile failures in the fill mass. Some of the typical failure models for estimating fill strength to sustain static loading are described in detail, Yu, 1992.

Due to tension cracks on the fill surface we could have simple wedge failure, Fig.3.1, along the plane of lowest resistance. Fill strength requirement to avoid the wedge failure could be calculated with eq. 1.

Fs = Shear strength/ Driving force

$$= (cB/\cos\beta + W\cos\beta \tan \phi) / W\sin \beta \qquad (1)$$

where:

Fs = static safety factor by shear failure

c = cohesion of fill (kN/ m2)

B = width of the fill block (m)

ϕ = friction angle of the fill

W = weight of the sliding wedge (kN)

β = dip of the sliding plane= $45^{o} + \phi/2$

Equation 1 assumes the following:

- No arching effect due to confinement,
- Gravity loading only,
- Failure starts at the toe of the moving block

Also the relationship between unconfined compressive strength, Qu , and cohesion, c, using Mohr Coulomb's strength theory is as follow:

$$\mathbf{Qu = 2c \tan (45 + \phi/2)} \qquad (2)$$

where: Qu = Unconfined compressive strength, kPa, and c= cohesion in kPa.

Equation 2 gives the required strength for a safety factor of only 1.

To make this more realistic, one could assume tension cracks parallel to fill face, Figure 3.2. A large test model indicated that tensile cracks were often seen at a distance equivalent to 1/3 the height of failure from the exposed face for a certain depth(Smith et al., 1983). Assuming that a vertical tension crack, developed at the middle of a block width, extends downward to intersect the shear failure plane, Fig. 3.2, one could calculate the safety factor using equation 3.

$$\mathbf{F= ((cB/\cos\beta) + W_f \cos \beta \tan\phi))/ \; W_f \sin\beta} \qquad (3)$$

where:

W_f= Weight of failure wedge per unit length (kN)

$W_f = B\, r_p$ (o.5 H- B tan ß /8)

r_p= fill density (kN/m3)

If the pillar around the fill is blasted and there is no confinement, the block is subject to it's own weight loading. The required compressive strength at the bottom of the fill block is as follows:

$$\mathbf{Fs = Qu / (r_p H)} \qquad (4)$$

Qu= Uniaxial compressive strength (kPa)

H= height of free standing block (m)

By knowing Qu we could use equation 2 to calculate cohesion, c.

However, if we consider arching effects on a confined fill by adjacent stope walls, we have much lower stress concentration at the bottom of the fill block. The vertical stress at any depth at this condition can be expressed by equation 5, (Terzaghi, 1961; Coates, 1981).

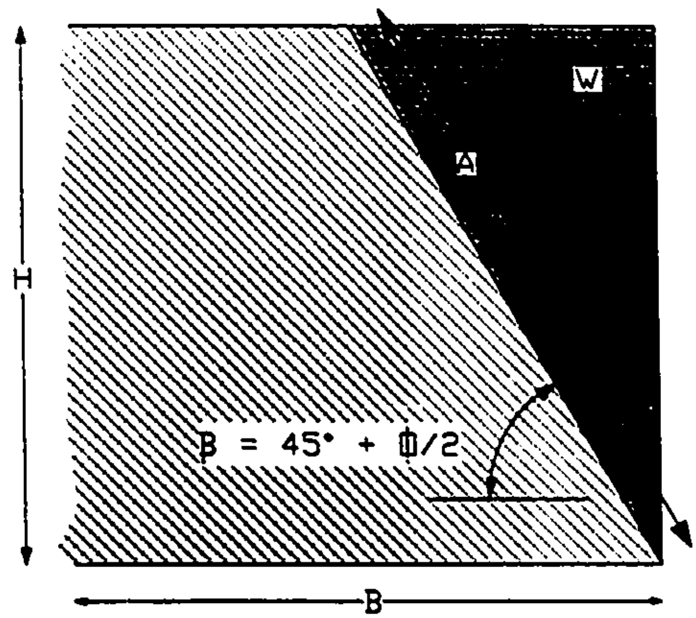


FIGURE 3.1.1: SIMPLE WEDGE FAILURE MODEL

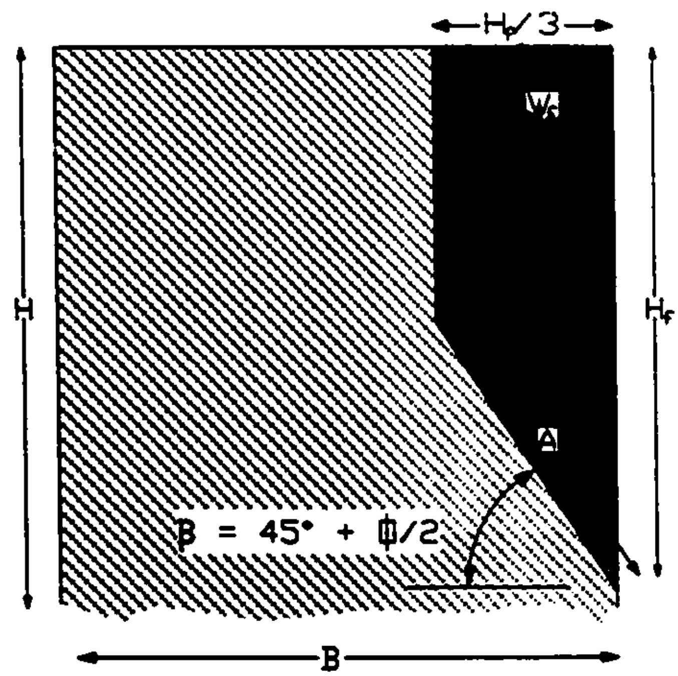


FIGURE 3.1.2: TENSION CRACK MODEL

$$Pv = D\,(r_p - (2c/B))\,(1 - e^{(-H/D)}) \quad (5)$$

where:

Pv = vertical stress (kPa)

D = B/2 K tan ø (m), K= ratio of vertical to horizontal stress, K= 1/ (1+ 2 tan2 ø)

r_p = fill density (kN/ m3)

c = cohesion of fill (kN/m2)

B= stope width (m)

H= depth below surface (m)

Following steps could be taken when considering arching effect, (Askew et al. , 1978)

1: Find vertical stress, Pv, from equation 5 and multiply it by factor of 1.25 for the peak stress, Qu.

2: Use Mohr-Coulomb's failure criteria, equation 2, to estimate the required cohesion from the measured friction angle.

3: Apply the selected safety factor , Fs= 1.3, for the required cohesion.

Over top of arching effect, we could consider a confined fill block with friction model, Fig. 3.3, more applicable for a CRF type of fill. In this case we have the frictional resistance induced between fill and walls to help stabilizing the block, Figure 3.3.

$$Fs = ((c\,L\,B / \cos ß) + W_f \cos ß \tan ø) / (W_f \sin ß) \quad (6)$$

where:

W_f = block weight minus wall friction component (kN)

W_f = B (Hm L rp - 2 ∂f)

Hm = mean depth to the sliding plane (m) = H - 0.5 B tan ß

∂f = frictional resistance per unit width (kN/m)

∂f = K tan ø D2 rp ((Hm / D) - 1 + e (-Hm/ D)) for c = o

L = strike length of an exposed fill (m)

B = width of the block (m)

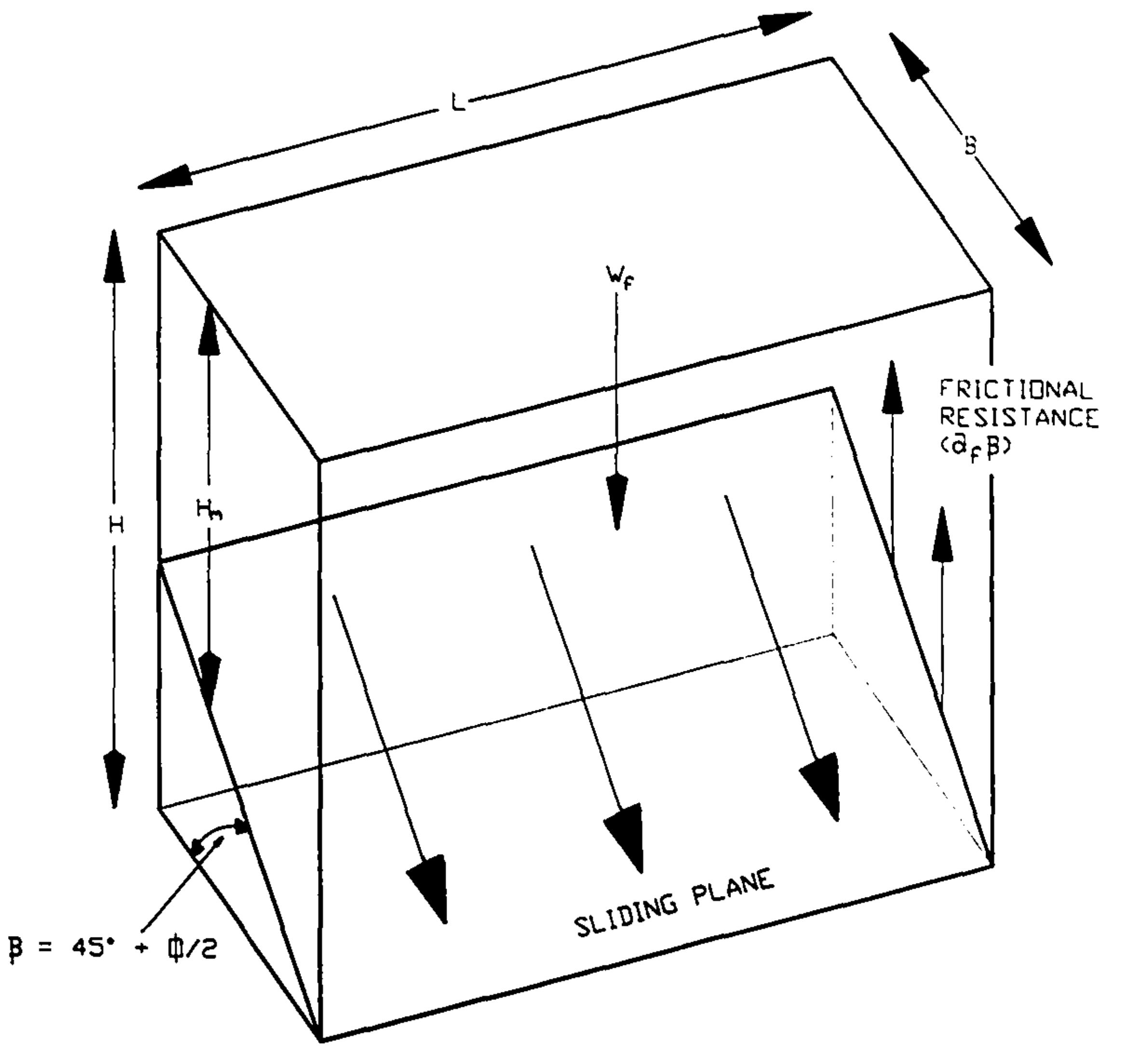


FIG: 3.1.3 CONFINED BLOCK WITH FRICTION MODEL

For example for block of H= 60 m. B=15 m. Fill density of 2000 kg/m3 or 19.6 kN/m3. ϕ = 36. and exposed fill face of 30 m long and assuming safety factor of 1. cohesion = 162 kPa. from equation 6: Compressive strength = 3.92. and c = 628 kPa from equation 2.

Up to now one has just considered the static loading of the fill and another very important factor is the dynamic loading/strength of the fill mass to be considered. especially for a rockfill system. since like concrete has a very low tensile strength.

3.2: DYNAMIC LOADING REQUIREMENTS:

Right after a mass blast. there is an instant release of confinement and the fill plug should be able to carry the sudden increase of stresses. Tests have shown that the sudden loading into the fill plug will induce double the deformation that would have been caused by gradual loading of the fill plug. This explains why the fill plug should be designed to carry at least double the maximum required static strength. There is also the blasting waves travelling through fill that cause internal fill failure. This is the reason that a factor of safety of 2.5 is used at Kidd Creek Mines.

Test drifts at Kidd Creek through a CRF mass showed that the fill had scabbing failure at an estimated Peak Particle Velocity of 300 mm/sec. (Yu. 1989)

To avoid internal spalling. the induced dynamic stress should be lower than the dynamic tensile strength of the fill material. Equation 7 gives an estimated dynamic tensile stress induced in fill. (Yu. 1992)

$$\partial_d = r_p \, P \, V \qquad (7)$$

where:

∂_d = dynamic tensile stress. kN/ m2 (kPa)

r_p = fill density. kN/ m3

P = compressional wave velocity of fill. m/sec

V= particle velocity at failure in fill. m/sec

3.3: STATIC AND DYNAMIC STRENGTH ESTIMATION:

1: If free standing fill mass is required use equation 4. Use at least a safety facto of 2 for a stage blasting and 3 for mass blasting. This will take care of both static and dynamic strength requirement of the fill mass to minmize dilution.

2: If the fill block is confined by both sides, and will stay confined, the strength requirement could be calculated from equation 6. Use at least a safety facto of 2 for a stage blasting and 3 for mass blasting.

3: Use lab testing to estimate the in-situ strength of the fill mass. The stope fill strength averages 63% $\pm$ 6% of the rockfill laboratory test results using 150 mm diameter and 86% $\pm$ 8% of results from 300 mm diameter cylinders. More than 150 samples were considered for above analysis.

3.4: FILL PLUG DESIGN

There is time that not only the free standing height is required, but also the fill plug will be undercut. This is to go after previously left high grade ore, or sill pillars. Here, the fill plug should be strong enough to take the static and dynamic loading. An study was carried out at Kidd Creek to obtain simple formula to estimate the minimum required thickness of the higher cement content/ stronger fill layer for various opening spans. To simplify the situation the followings were assumed:

1: The geometry of the vertical loading applied at the bottom of backfill is assumed to be the combination of a load increasing uniformly to the centre and a uniformly distributed load.
The height of the vertical loading:

$$H=(L/2)\tan(45^{o}+\phi/2).$$

where

L= mid span of the opening to be filled, and L/3 at the edge

ø = Angle of friction of backfill, 370

2: No loads other than gravitational weight are considered. The load being applied on the undercut of a CRF is solely supported by the footwall and hangingwall.

3: The in-situ strength parameters of CRF in stopes at KCM are as follows:

a: Bulk density of CRF, 1.8 tonne/m^3

b: Uniaxial compressive strength,

Sc10 = 6.9 Mpa for 10% binder

Sc7 = 4.9 Mpa for 7% binder

Sc5 = 3.5 Mpa psi for 5% binder

c: Modulus of rupture , tensile strength

St10 = 1Mpa for 10% binder

St7 = 0.8 Mpa for 7% binder

St 5 = 0.5 Mpa for 5% binder

d: Angle of friction, $\phi = 37^{o}$ The required fill plug strength with above assumptions could be calculated using two methods:

3.4.1: BEAM THEORY

The minimum thickness of a CRF required for a stable base can be found from the following equation which is derived from the flexture formula, and Terzaghi's vertical loading formula: (Figure 3.4.1)

$$\mathbf{H= 0.083\ (6\ M\ /\ S_t)^{0.5}} \qquad (8)$$

$$M= 0.097\ r_p\, L^3$$

where:

H = Minimum thickness of CRF. ft

S_t = Modulus of rupture. psi

r_p = Bulk density of CRF. pcf

L = Span of opening to be filled. ft.

3.4.2: ARCHING THEORY

When backfill is placed between the solid pillars. the load distributes around the low modulus component. fill material. into high modulus components. solid pillars. This phenomenon is called arching. Friction between a stope wall and fill in a stope helps to support the weight of the fill and reduces vertical pressure due to the weight of the fill. The extent of arching. reduction of vertical loading. depends upon:

- Stope geometry
- fill properties
- non-uniformity of fill
- stope dip
- stope wall roughness
- stope wall movement.

This theory is useful for analysing masonary arches. The arches are built of material which have the strength parameters of high compression and low tension. The use of the arching theory may not be perfectly applicable to the CRF. However. the arching theory allows us to check the result of the beam theory.

An arch will fail under the following conditions:

1: The blocks of arching materials could slip out of the arch as a result of the frictional re .istance at their contacts not being great enough with respect to shear caused by the loading on the arch.

2: The arch could fail by crushing at the relatively small areas of contact between the blocks.

3: If the load is high for the span . tensile stresses could open spaces between the blocks. permitting them to fall.

In the following analysis. only the first mechanism is considered. The other two cases are unlikely to cause any major instability in the CRF. Therefore. for an arch of CRF to be stable. the frictional resistance has to be larger than the vertical loading and the point of application of the resultant force should be within the third of the cross section of the arch. The analysis results in the following equation: (Figure 3.4.1)

$$\mathbf{H = 0.87\ L\ \tan \varnothing} \qquad (9)$$

H= 0.66 L at Kidd Creek.

where:

H= minimum thickness of the plug. m.

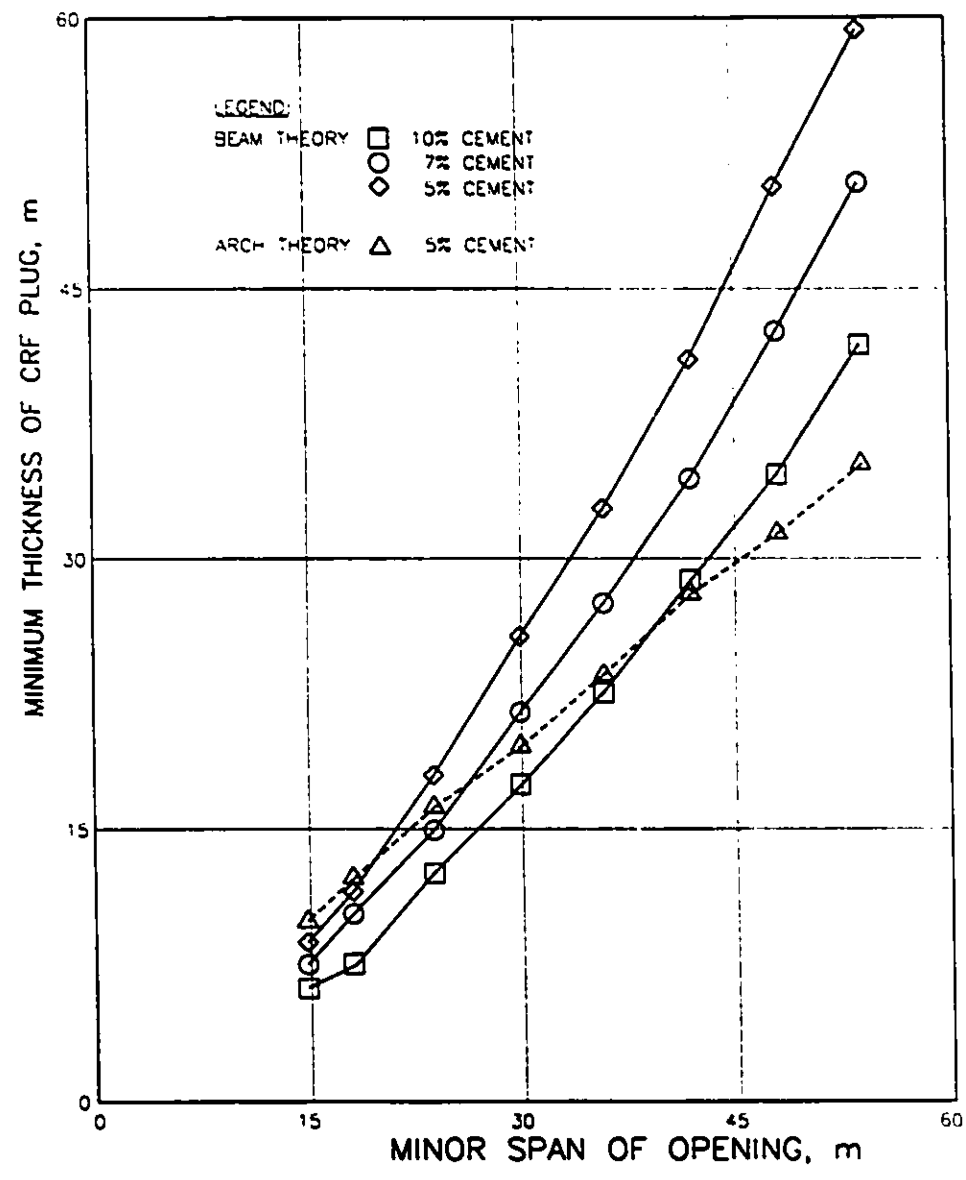
L = span of the opening to be filled. m.

ø = angle of internal friction

This indicates that if the fill plug is to be exposed in future mining the cement plug. higher than usual binder content. height should be at least 2/3 of the minor span to be exposed. Figure 3.4.1.

3.5 SUMMARY

The above review covers all the strength requirement estimation tools for a non segregated and homogeneous CRF mass. Underground observation and in situ testing have proven that such homogeneous mass does not exist and the effect of segregation on CRF strength is a key element that should be introduced in to strength requirement estimation methods. An ideal system will have minimized segregation and the strength requirements will be close to what is expected from a homogeneous mass. One should be very careful in only using the above theories for design



(Yu, Counter 1983)

Fig: 3.4.1 Minimum Thickness of CRF vs Span of Opening purpose and this chapter should just act as guideline for strength design estimation. The most important part of the design process is to determine the extent of segregation through extensive site observations and predict the zones being formed in the stope. Next chapter will try to use authors's experience in the field evaluation of structural integrity of such a non homogeneous CRF mass for future design and implementation purpose.

4 SITE INVESTIGATION

This chapter contain results from extensive site investigation, especially structural mapping of drifts driven through CRF. Due to operational needs and advantages, numerous drift have been driven through CRF mass at KCM. This allowed the author to extensively examine the structural variations in the placed CRF. The relationship between individual stope in situ mapping, combined with information from the initial stope design which was carried out by the author, is examined throughout this chapter. The aim of the site investigation is to study the extent of aggregate segregation for different filling conditions. This information was not available at the start of this thesis and is required for proper structural design of CRF mass and behaviour. The in situ investigation was carried out at KCM.

4.1: KIDD CREEK MINES

From onset of this thesis it became apparent that in order to investigate the critical parameters regarding CRF design improvement for extensive period of time, a site had to be selected. This was needed for the author to identify main parameters or problem areas, carry out the improvement projects, analyze results, and most importantly implement the apparent recommendations in the field for reasonable period of time to further i .vestigate the long term effects of changes made.

This became all possible at KCM, since the author was employed by the mine and held the major responsibilities in design and implementation of the CRF system for last 5 years. The author was responsible for both, backfill engineer and backfill general foreman positions at KCM. The author was responsible for all the design and implementation work carried out at KCM in last 5 years. The fill system at KCM is described below:

4.1.1: SURFACE PREPARATION AT KIDD CREEK

The production and handling of various aggregates for CRF needs rugged equipment due to the volume and size of the materials used in the fill system. Rockfill material is transferred underground from surface by dumping from a front-end loader, truck or conveyor into boreholes which are commonly between 1 to 2.4 meters in diameter. The coarse and fine aggregates are usually sized and mixed on surface before entering the raises going underground. This allows the removal or addition of different size fractions to meet the stope requirements. Different surface and underground facilities for handling and distributing waste rock and binding agents could be employed. For example at Mount Isa, the waste rock is quarried, crushed and screened to produce up to 1300 tonne per hour of minus 300 mm material with 5% passing 25 mm. Screen product is conveyed 2500 m along surface, drawn 550 m down a 2.4 m diameter raisebored pass and distributed by a multi-conveyor system into the desired stope. (Thomas et al., 1979).

At KCM preparation of fill material is accomplished at the surface backfill plant by crushing waste rock to a maximum size of 15 cm. Cement slurry is prepared at a slurry plant, located next to three silos with a total storage capacity of 1700 tonnes of cementing materials. A sand plant has the capacity of producing 200 tph(tonne/hr) of sand slurry at 65% pulp density. More details on these facilities for handling the backfill material are as follows:

4.1.2: CEMENT HANDLING FACILITIES

Cement must be transported from surface in slurry form, as the size of the operation precludes the handling of bulk cement underground. On average, 500 tonnes of cementitious materials will be required daily to meet the annual projected requirement of 1.8 to 2 million tonnes of backfill.

Bulk cementitious materials are stored in three silos within the slurry plant. Two 500-tonne silos are used to store flyash, while the third 700 tonne silo is used to store normal Type 10 Portland cement, for gold mine operations such as Golden Giant Mine, 150-tonne silos are used.

No cement production facilities exist in the Timmins area, and as a result, all cementitious products are supplied by either rail or truck from the Toronto area for cement, and Thunder Bay and/or Sheboygan, Wisc. for flyash . Railcars are unloaded in a special unpressurized unloading facility at the minesite. Vibrators are attached to the railcars, and the cements flow by gravity and pressurized pipeline to the top of the silos. Truck unloading is performed by 'blowing' the cement under pressure into the silos from a separate unloading point.

The cementitious material passes from the silos into a weighing hopper via either one of the three 15- cm airslides. The hopper has a capacity of 12.5 tonnes and the load cells used to weigh the cement or flyash are accurate to 10 Kilograms. The hopper is capable of discharging the binders into either of two mixing tanks in approximately 5 minutes.

Each of the mixing tanks has a rated capacity of 15.6 m3, with a four-blade impeller mixing the slurry. Water is fed into the mixing tanks by a 15 cm pipeline from either the recycled mine water ponds, or a fresh water lake. The water is measured by flowmeter, and the tanks are equipped with high level indicators, electronic pulp density readouts and manual pulp density scales to assist in the correct mixing of slurry. Pulp densities can range from 40 to 60% solids by weight, depending on the fill design, but typical operating ranges are between 50 to 55% solids.

The plant operator controls which mine (No.1 or No.2) the slurry is sent to by remote actuation of valves. Once underground, the slurry destination is controlled by the underground workers. Both tanks are capable of sending slurry to No.1 Mine, and each tank is capable of sending the slurry to either south or north zones of No.2 Mine. A batch takes approximately 15 minutes to mix and deliver to the underground fill location. See Fig. 4.1.1.

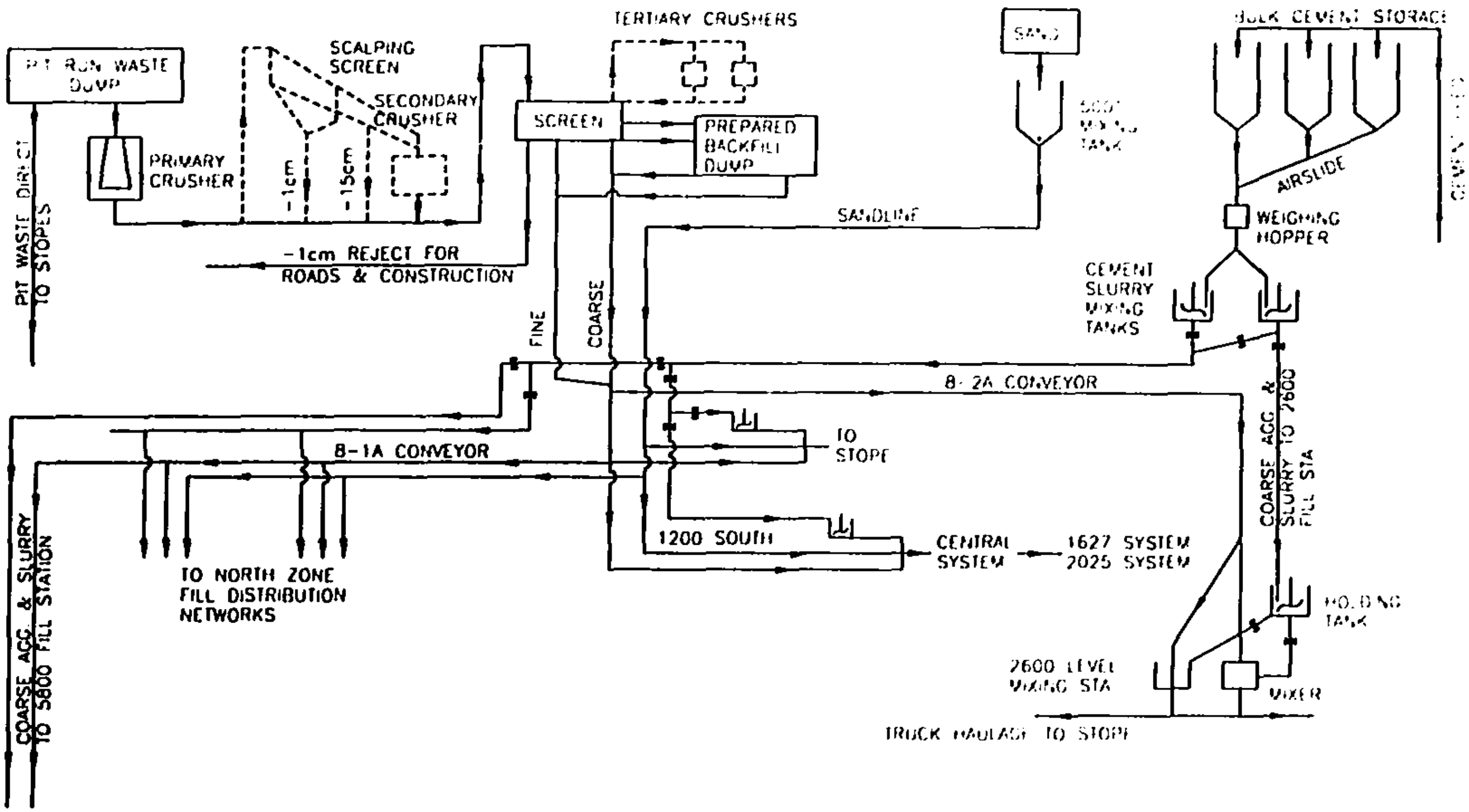


Fig: 4.1.1 Kidd Creek Backfill System – Schematic

4.1.3: AGGREGATE PRODUCTION FACILITIES

Waste rock, hauled by a fleet of 85 tonne Euclid dump trucks, is passed through the primary surface gyratory crusher (Allis-Chalmers 137 by 188 cm) and conveyed to the backfill plant. The magnitude of increased attrition on the aggregate with depth was not foreseen during the initial design of the plant. Initial testing indicated that aggregate with a gradation of 75% -15 cm to +1 cm, and 25% -1 cm by weight would, with 5% cement, provide a fill with a compressive strength of 7 MPa. Secondary and tertiary crushing of some of the aggregate provided the fines required when fill was being placed in the uppermost horizons of No.1 Mine. With increasing depth, the attrition of the aggregate as it passed through the raises, made secondary and tertiary crushing unnecessary. Sufficient fines for No.1 Mine are now produced by the primary crusher.

The aggregates are sent underground, via two silos, or are loaded into trucks and hauled to the prepared fill stockpiles for future use. There is access to the two silos from outside the plant to avoid sending stockpiled material back through the system. Some of the fines are rejected, as more fines are produced than required. They are used in construction or as roadway material.

4.1.4: SAND PLANT

Sand from a 150-tonne bin, is screened to -3 cm and fed into a 500-tonne mixing silo. A Marconajet mixing system slurries the alluvial sand in the bottom of the tank at a rate of 200 tph at a 65% pulp density.

The sand is sent underground via boreholes and 15 cm diameter steel pipe. The pipe networks follow the conveyorways, in a manner similar to the cement slurry line.

4.1.5: MIX DESIGN

Once an opening has been designed and a mining sequence decided, the backfill mix can be formulated. Formulation of the batch used for filling is as follows :

1. Estimate the feeder or belt capacity. Belt capacity at KCM for handling rockfill is approximately 720 tonne/hr, otherwise the rate of filling is dictated by the mixing capacity of the culvert, sand= 120 tph, CSRF=130 tph. Estimate 4 batches per hour since it takes approximately 15 minutes to deliver a batch of slurry to the desired stope.

2. Determine the cement content requirements, from anticipated free standing height, or span of fill, during pillar recovery. The overall cement content is approximately 5-6 %. Also indicate binder proportioning i.e., cement : flyash (if required).

3. Determine water required per batch. Experience has shown the following water requirements for the filling materials: binders(30%), aggregate(2.6%) and sand(27.9%), by weight of the material. The water content of the slurry may be altered for special angles of repose.

4. Establish the pulp density

5. Ensure that for a given pulp density and cement mass, the volume of the cement slurry will not exceed the volume capacities of the surface and underground slurry tanks.

SAMPLE CALCULATION:

CRF @ 720 tph = 180 tonne per batch

Binder @ 5% of aggregate = .05 x 180 = 9 tonnes per batch

50 / 50 cement/ flyash = 4.5 tonne of cement and 4.5 tonne of flyash per batch

Mixing water required = 180 x 0.026 + 9 x 0.3 = 7.4 tonnes

Pulp density for 9 tonne binder + 7.4 tonne water = 55%

Volume of water (7.4 tonne) = 7.4 m3

Volume of Portland cement (4.5 tonne) = 1.38 m3 (s.g.= 3.25)

Volume of flyash (4.5 tonne) = 1.64 m3 (s.g.=2.75)

Total = 10.4 m3

Before starting to fill the stope at KCM, a proposal is issued for comments. If approved, the operating procedures are issued to the production department. These include the aggregate tonnage, quantities of cementitious materials and amount of water per batch, plus any other special procedures to be followed.

In the slurry plant the use of Xycom computer system enables the operator to select any one of the four mixes that are already programed in the computer. The recipes of the mixes could be changed easily to suite the requirement. These recipes are designed by the backfill engineer for specific strength and cure time requirements.

4.2: UNDERGROUND FILL DISTRIBUTION

Fill materials can be distributed by several methods. The transportation of the aggregate in waste passes or fill raises from surface to underground is the most popular method for economical reasons.

Combinations of three transportation methods could be used to placed the fill in the desired stope underground. The methods are 1: Hydraulic transportation, for cement and sand slurries, 2: Conveyor transportation, used to transport the coarse particles, 3: Vehicular transportation, using truck or scooptram to haul the material directly to the stope.

To transport the rockfill aggregates underground using conveyor belts, generally the material is fed in fill passes and it ends up at main conveyor horizons underground. The materials are then moved to either another fill raise for lower levels or to another conveyor belt. Figure 4.2.1

For cement or sand slurry transportation, gravity loading through pipelines and /or boreholes is employed. The pipeline usually follows the path of the conveyor system, leading to a holding tank. The slurry is then sprayed on the aggregate just before entering into the stope, Figure 4.2.2.

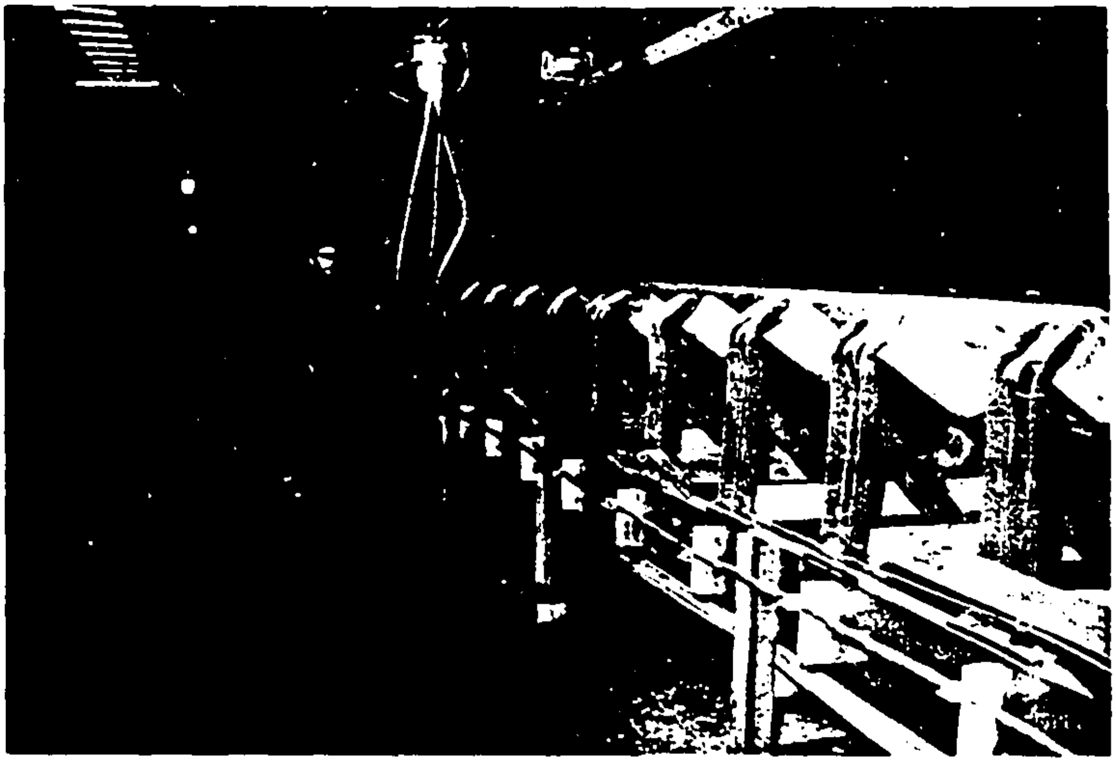


Figure 4.2.1: Aggregate transportation using conveyors at KCM.

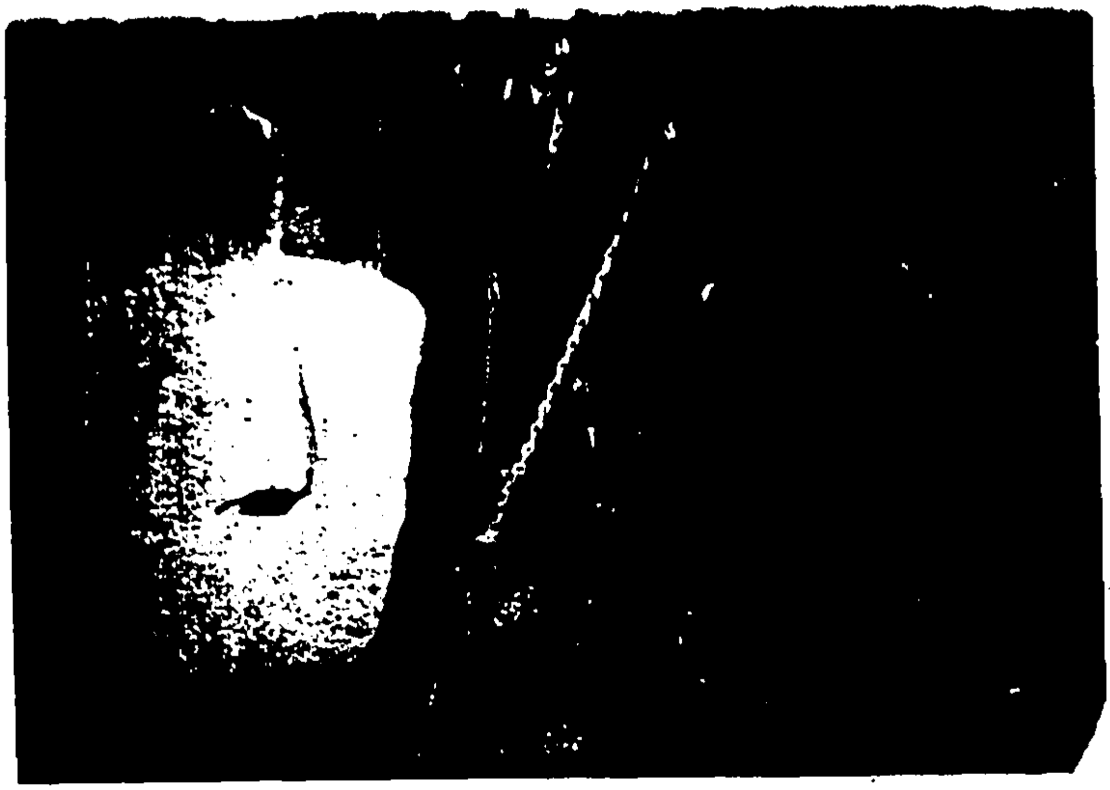


Figure 4.2.2: Slurry added to the aggregate just before entering the stope.

Transportation with scooptram and/or truck is the most expensive method for fill handling. It is employed in the cases where the drift size does not allow the installation of a conveyor system, or the required tonnage to fill the stope does not justify the expense of installing a conveyor system. For example at Williams Mine, 40,000 tonnes of fill or less will result in using the scoop and/or truck hauling method. The typical system for transporting fill material underground at Kidd Creek Mine is described below (Yu and Counter, 1983).

4.2.1: AGGREGATE DISTRIBUTION

At KCM, the aggregates pass via the silos from the surface backfill plant down to the 8-2A sublevel. From 8-2A, the aggregate is sent to one of two raises (Figs. 4.2.3 and 4.2.4). One raise carries course aggregate directly to the No.2 Mine backfill station on 2600 level. The other raise is jointed by two-finger raises to allow the aggregate to be distributed to three locations in No.1 Mine. One raise leads to the south orebody on 1200 level and a conveyor transports the aggregate at a rate of 600 tph to the stopes below the 1200 level. A second raise led to a temporary conveyor system in the south orebody on 800 level. All stopes in this area have been filled, and the system dismantled. The remaining leg of the pass leads to the No.1 Mine north orebody distribution conveyor on the 8-1A sublevel, 15 metres above the northern end of 800 level. From here, coarse aggregate (fed specially from 8-2A) can also be sent directly to the No.2 north orebody.

The 8-1A conveyor system is comprised of a 0.9 m wide, 360 m long belt, with a capacity of 1200 tonnes of aggregate per hour. All other conveyors operate at approximately 600 tonnes of aggregate per hour. Three different forms of conveyor are used at KCM. The first form is used in permanent installations such as on 8-2A, and 8-1A sublevels. Steel rails support the idler and return rollers, and the rails are supported by steel legs firmly anchored to the drift floor.

The second form also uses steel rails as roller supports, but the rails are suspended from the drift back by chains. The use of these conveyors is more flexible as a prepared floor is not required for their installation. These conveyors are classed as semi-permanent and have a longer

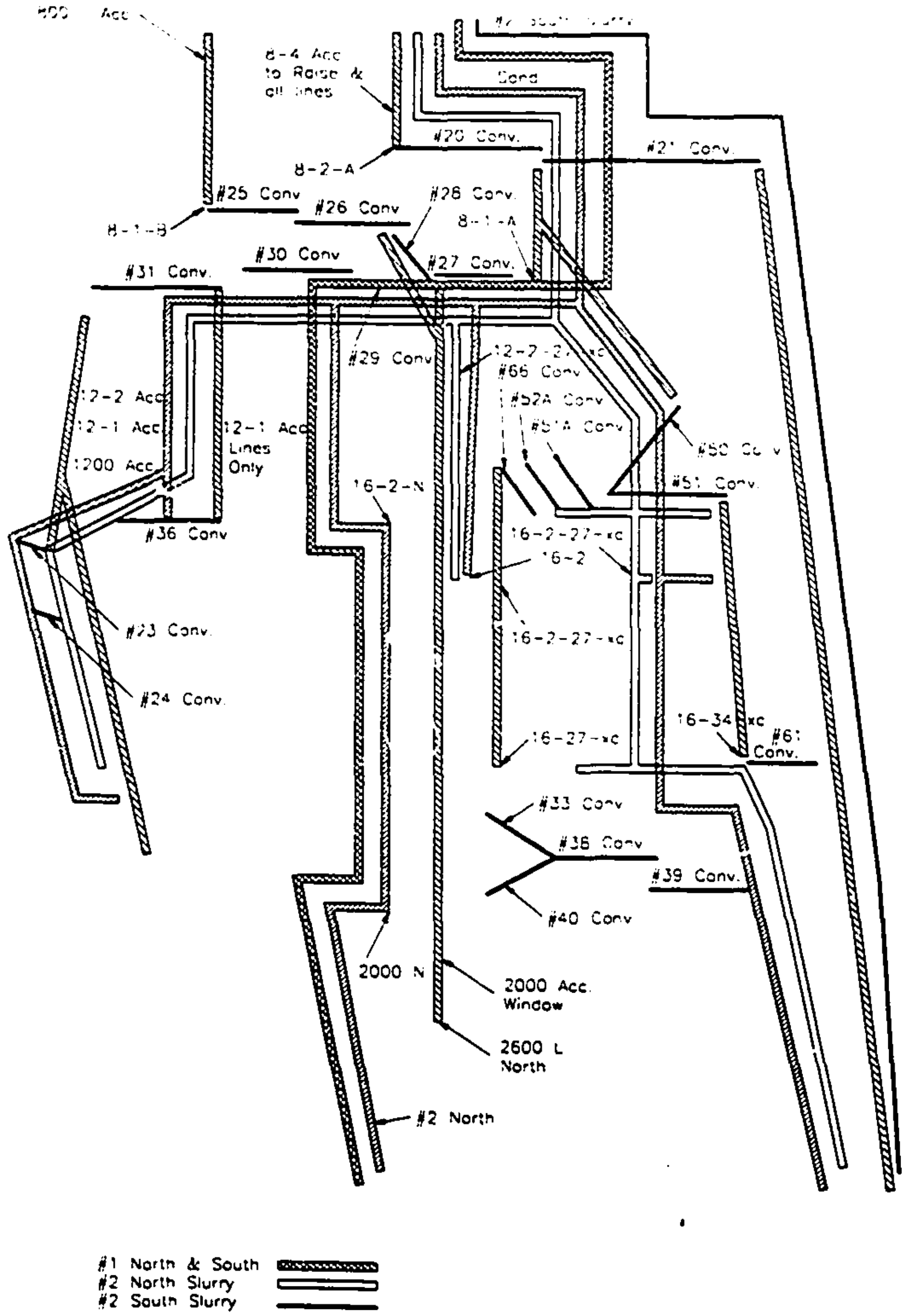


Fig: 4.2.3 Underground Fill Transportation at Kidd Creek Mines

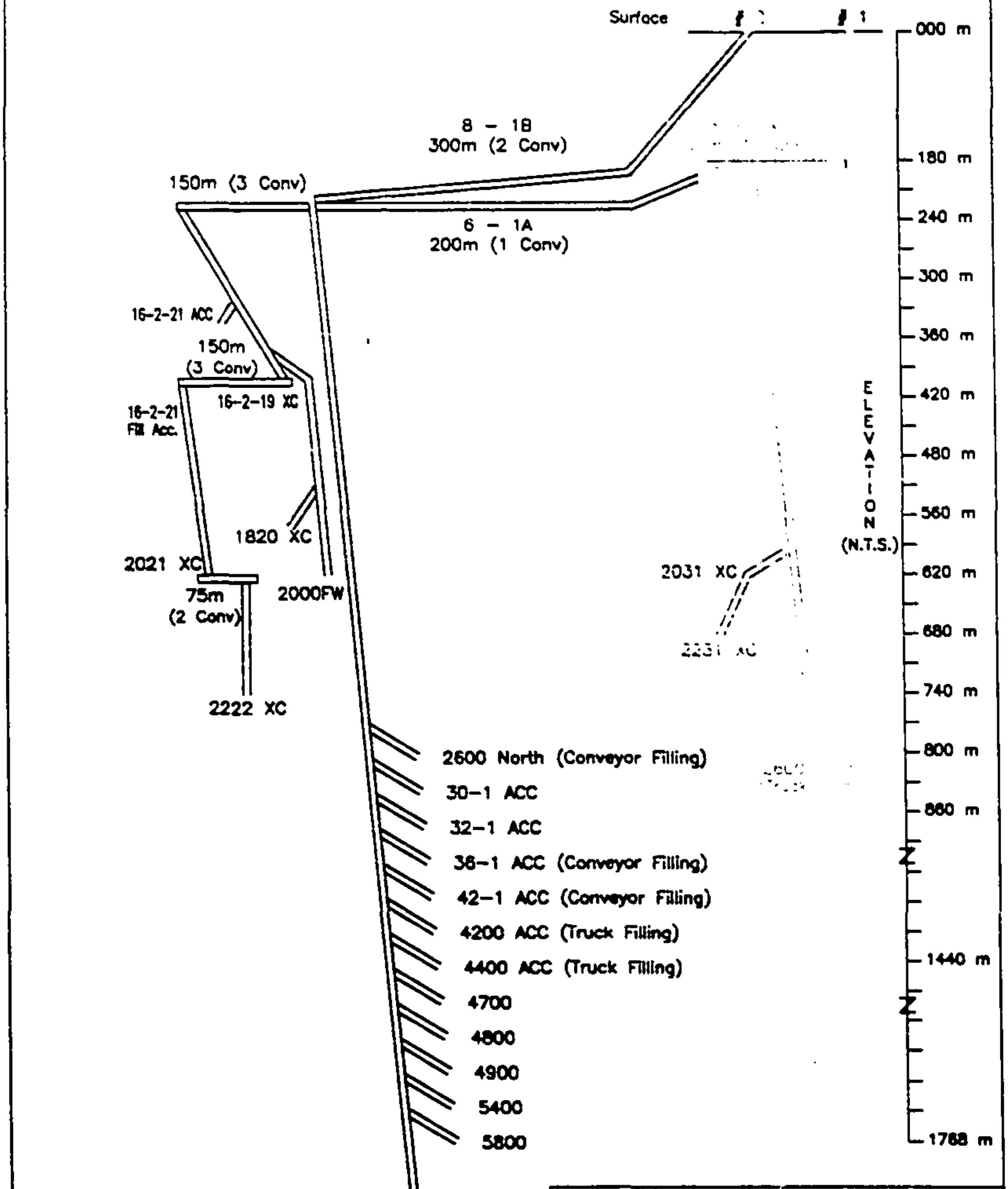


Fig: 4.2.4 Kidd Creek Mines Fill Transportation System service life than the temporary form.

The third form of conveyor is truly temporary, as the rollers are supported by wire ropes. The ropes are tensioned and supported by chains from the back. The use of wire ropes as supporting members eliminates many of the problems of handling rigid conveyor sections underground, and can be installed and dismounted easily.

4.2.2: CEMENT SLURRY DISTRIBUTION

All slurry is transported by gravity through 10 or 15 cm pipelines and lined or unlined boreholes. In most cases, the slurry lines run along the conveyorways and parallel to the raises. In all cases, the slurry lines end at a portable, skid mounted, holding tanks near the stope to be filled. The tanks, equipped with agitators, have capacities ranging from 5.7 to 11.9 m3. In the case of No.2 Mine backfill stations, these holding tanks are permanently installed. Pipe networks, also permanently installed, are connected into the distribution system whenever a particular conveyor network is to be used.

4.2.3: SAND SLURRY DISTRIBUTION

The sand is sent underground via boreholes and 15 cm diameter steel pipe. The pipe networks follow the conveyorways, in a manner similar to the cement slurry line. Near the opening to be filled, the sand line and the 10 cm diameter pipe from the cement slurry pump are joined and the mixed hydraulic fill is sent into the stope through a 20 cm borehole.

4.3: DESIGN PARAMETERS AT KIDD CREEK

The fill must be able to sustain not only the gravitational loading of the overlying fill material, but also the dynamic effects applied during blasting. If the stability of the fill exposure is to be maintained, it is necessary to consider both the static and dynamic fill strength. For example, after many years of research to find a suitable backfilling material at KCM, the result was the production of strong consolidated fill formed by mixing crushed aggregate from pit waste with a ratio of 20 parts aggregate to 1 part cementing agent (the size distribution of the aggregate is shown in Fig. 4.3.1). This produced a stabilized fill mass that would reduce ore dilution and the risk of ore losses during the mining of the pillars between and/or adjacent to the filled stopes. The aggregate/ cement ratio in different operations ranges between 30:1 to 10:1.

4.3.1: STATIC STRENGTH AT KIDD CREEK

Uniaxial compressive strength is one of the most important parameters to be considered when dealing with consolidated backfill such as CRF. The uniaxial compressive strength, U.C.S., requirement of the consolidated rockfill should be determined by defining the height and width of the most likely fill exposure. The uniaxial compressive strength in different rockfill operations ranges between 2.3 to 7 MPa.

At KCM it was anticipated that the exposed fill face would have a height of 120 m and length of 60 m and this required a fill having a strength of 7 MPa. To support the gravity loading alone, the fill required a compressive strength of 2.8 MPa for the above dimensions. A safety factor of 2.5 was applied to allow for additional blast loading and the reduction of fill strength if inadequate mixing occurs. Fig. 4.3.2 shows the relationships between the compressive strength and the cement content for 15 cm cylinders of consolidated rockfill at KCM. The compressive strength of these cylinders at 28 day curing time shows a relationship to the cement content by:

$$\mathbf{Qu = 1.5\ e\ 0.25\ C} \qquad \text{for} \qquad 2 \le C < 10 \text{ at KCM}$$

where: Qu is uniaxial compressive strength in MPa

C is Portland cement content by weight % of minus 4 cm aggregate.

The shear failure mechanism by gravity loading on backfill has been discussed elsewhere

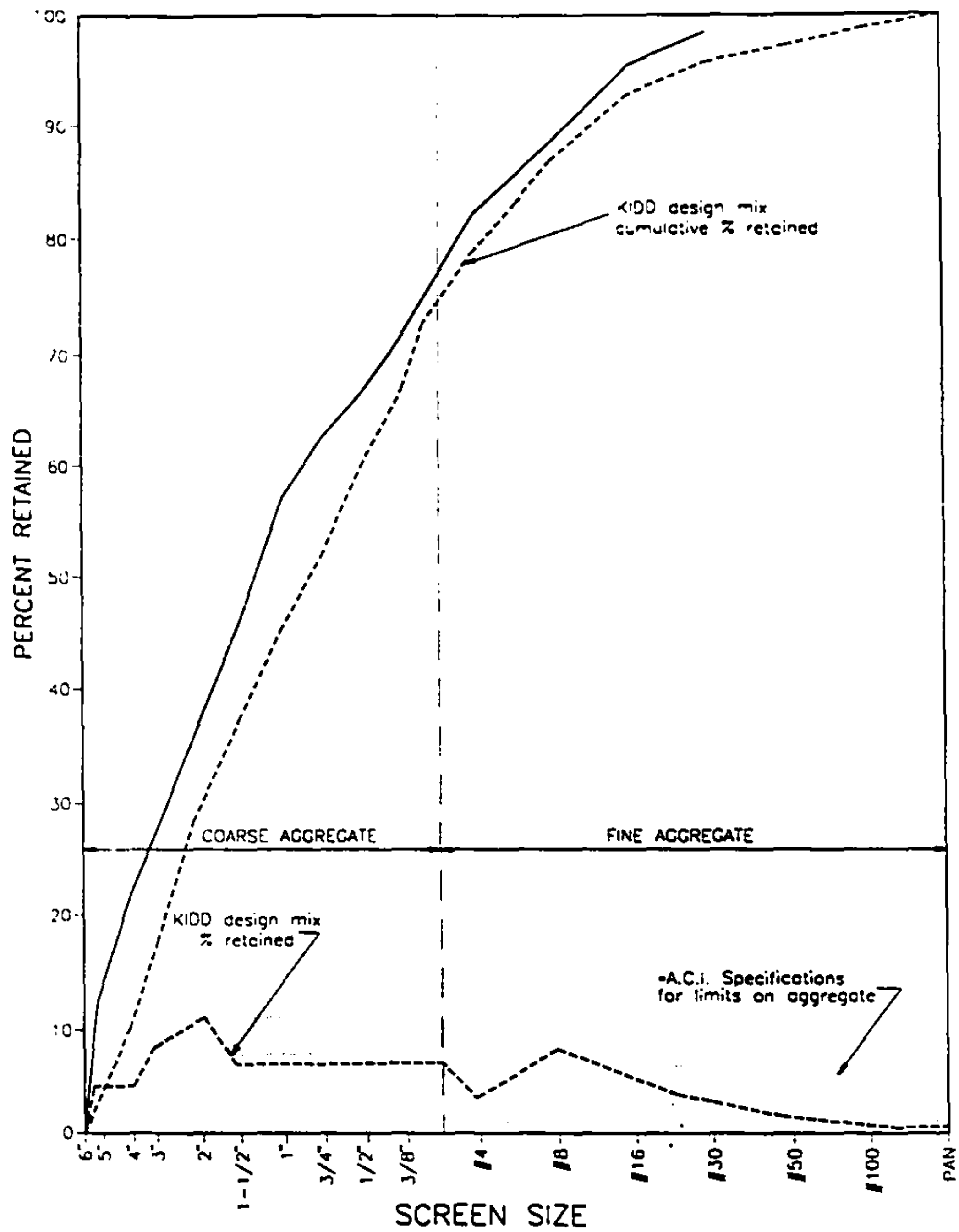


Fig: 4.3.1 Typical Size Distribution of Fill Aggregate at Kidd Creek Mines

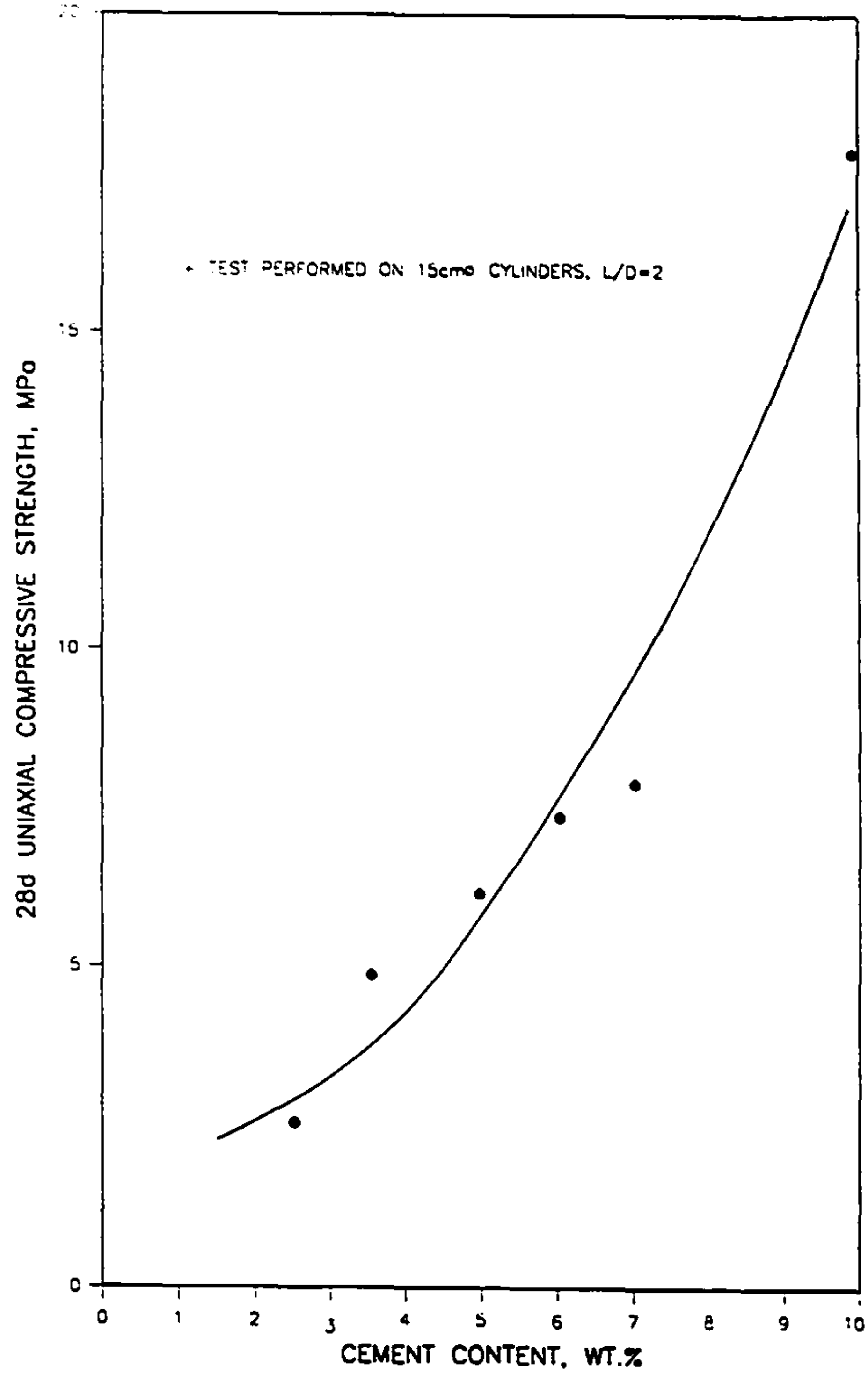


Fig: 4.3.2 Optimum Strength of CRF⁺vs Cement Content

(Askew et al., 1978) and can be expressed by the following equation using a wedge shear sliding model:

$$Fs = \text{Shear strength/ Driving force}$$

$$= (cL + N \tan \emptyset) / W \sin D$$

where : Fs= static factor of safety, c= apparent cohesion, L= length of the sliding plane, Ø= apparent friction angle of the plane, N= effective normal reaction of the plane, W= weight of the sliding block per unit width, and D= dip of the sliding plane.

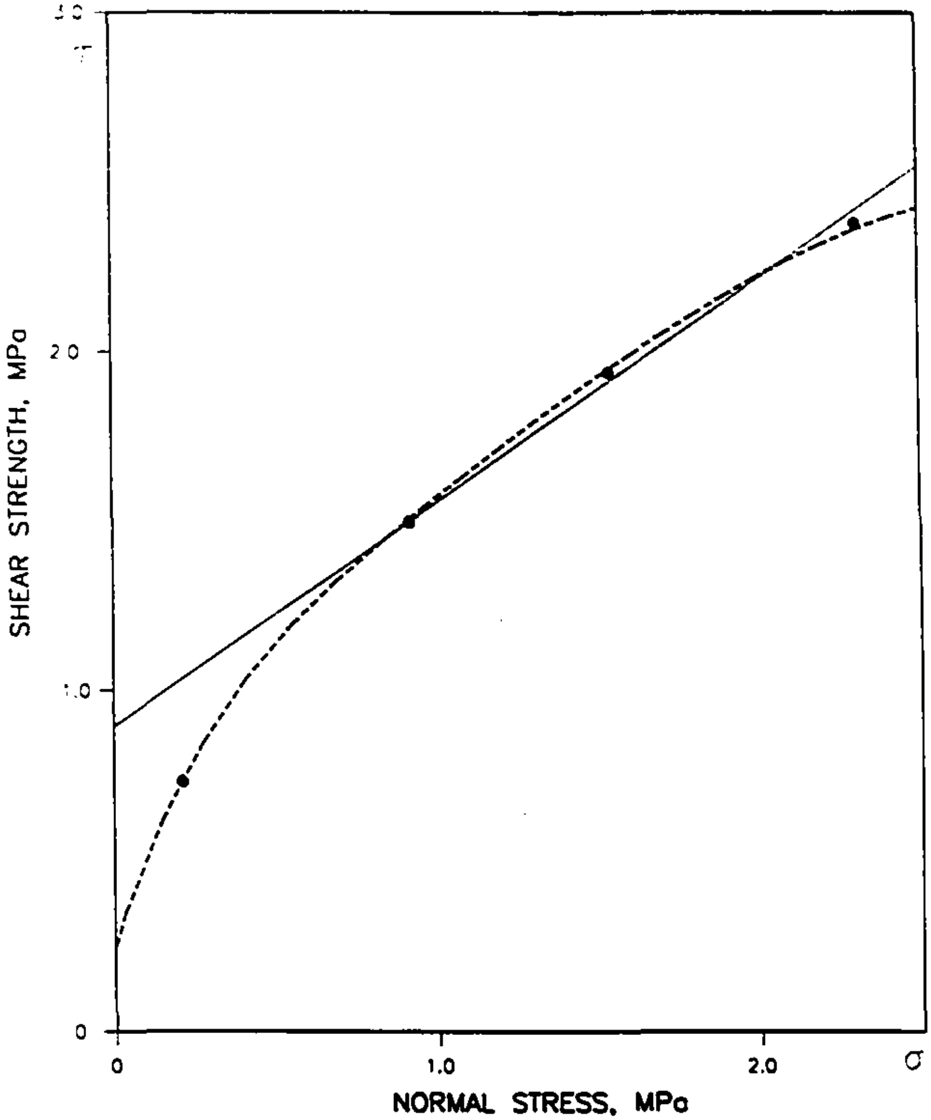
Cohesion and friction angle values of rockfill should be evaluated for stability analysis. The shear strength of CRF could be determined using the direct shear test. The results for KCM rockfill (Fig. 4.3.3) show the relationship between the shear strength and the normal stress. As measured from the graph, the apparent cohesion and the apparent friction angle are 1.1 MPa and 33 degrees respectively. This corresponds well with the observed angle of repose of the fill, normally 34 -37 degrees. Figures 4.3.4 and 4.3.5 show the angle of repose of CRF during filling and after drifting through fill, respectively. The cement addition to the fill mix increases the shear strength, apparent cohesion, and the modules of elasticity.

4.3.2: DYNAMIC STRENGTH

The parameters governing blast damage to consolidated fill are not fully understood yet. It is believed that the blasting vibration resistance may be closely related to the dynamic tensile strength of the fill. Hence, the dynamic strength can be related to the blasting vibration level in terms of peak particle velocity as shown in the following equation.

$$Fd = \text{Dynamic strength/Dynamic loading}$$

$$= T / dPV$$



(Yu, Counter 1983)

Fig: 4.3.3 Direct Shear Test on CRF Cylinders

Figure 4.3.4: Fill angle of repose while filling.

Figure 4.3.5: Fill angle of repose after drifting through fill.

Where Fd = dynamic factor of safety, T = dynamic tensile strength, say 5% of U.C.S

d = fill density, P = fill compressional wave velocity,

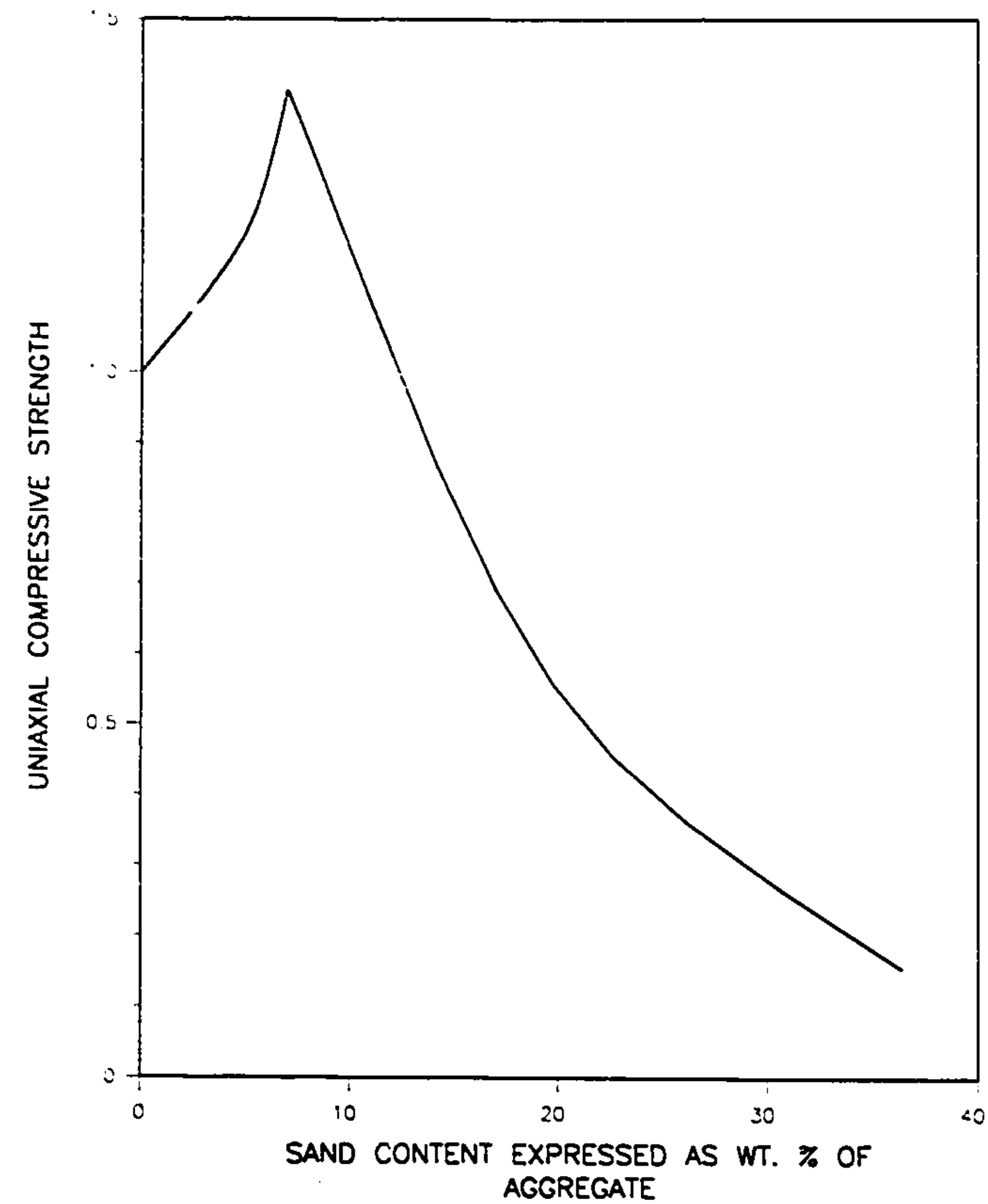
V = particle velocity at failure in fill.

CRF, being similar to weak concrete, can be strong in compression, yet weak in tension especially in the outer zone of the fill pile, or adjacent to stope walls where most of the segregated aggregates exist. The weak zone should be reinforced to enhance the tensile strength if the degree of blast damage is to be minimized. To minimize the effect of segregation without increasing the number of fill dump points, tests were conducted at KCM by adding sand into the CRF to act as a void filler. The results indicated that the addition of sand significantly increased not only the U.C.S at a particular 5% sand content, but also the dynamic strength of the CRF, Figs. 4.3.6 & 4.3.7. Another reason for the use of the consolidated sand rockfill, CSRF, is that the need for coating all aggregate with cement slurry becomes less critical. This implies that a lesser degree of mixing of cement slurry and aggregate is acceptable and the number of fill raises required to reduce segregation, therefore, becomes less critical .

Based on the favourable laboratory test results and evaluation of three stopes filled with CSRF, a sand slurry plant at KCM was constructed to provide hydraulic sand fill. However, due to the extra cost of the sand slurry, the use of CSRF has been limited to selective stopes where a more stable fill exposure is required.

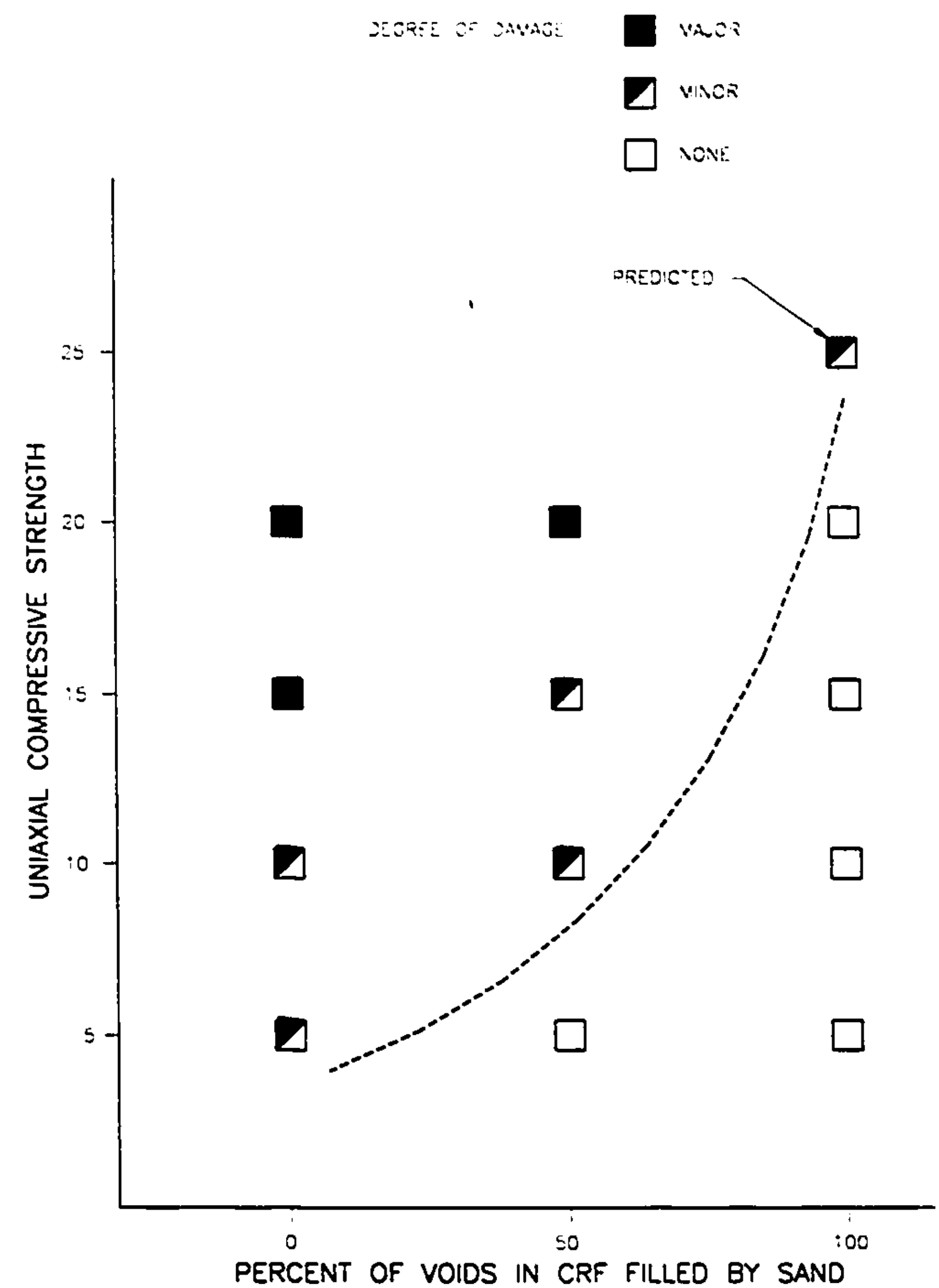
4.4: DRIFTING THROUGH CRF

The initial planning of N0.2 mine at Kidd Creek Mines involved the reestablishment of the undercut drifts of filled stopes to be used for future stope drilling sites to allow for almost 100% ore recovery. In practice, however, it was found that the consistency of the segregated fill hindered the reestablishment of the filled drifts. Due to segregation phenomena, Figure 4.4.1, drifting through backfill was only done when absolutely necessary to access stoping areas behind the filled



(Yu, Counter 1983)

Fig: 4.3.6 Compressive Strength vs Sand Content



(Yu, Counter 1983)

Fig: 4.3.7 Impact Resistance Testing with Schmidt Hammer

Figure 4.4.1: Aggregate segregation at surface stockpile.

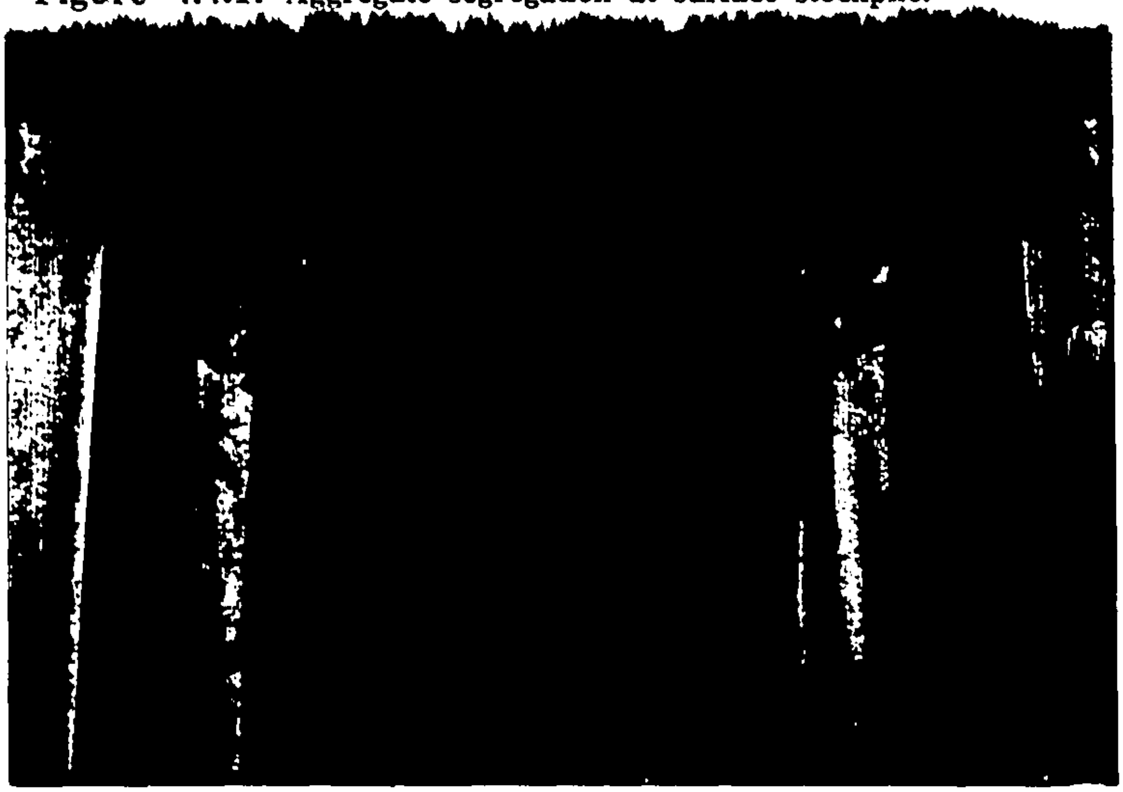


Figure 4.4.2: Conventional drifting through CRF.

stopes.

4.4.1: CONVENTIONAL DRIFTING

The difficulty associated with conventionally drifting, Figure 4.4.2, in backfill severely limited the lateral development advance per round. The main problems with conventional drill and blast method were:

1: Only 1.5 m rounds could be drilled and blasted with any degree of accuracy. However at present time at majority of the areas rounds up to 3-4 meters are taken with good success due to high quality fill and planned drifting through the strongest zone in backfill.

2: Holes tended to slough or became cratered when drilling, hindering effective loading and blasting.

3: Drill rods mudded very easily.

4: Only high density explosive could be used to blast because AN/FO releases ammonia when in contact with the cemented fill; and

5: Blasting caused excessive damage to the fill which required extensive timber support for ground support.

The alternative was to gain the required access by drifting around the backfill in solid rock. This increased development costs by requiring longer drifts in very poor ground. A second alternative was to drive between the backfilled crosscuts to gain access to the required stope for longhole drilling. The surrounding rock in this area was shattered rock due to earlier production blasting. This shattered rock created difficulties and costly drifting as well as poor ground conditions. Extensive ground support was required for each advancement.

4.4.2: MECHANICAL EXCAVATION

Since 1988, Kidd Creek Mines has started using mechanically excavating drifting machine, roadheader, to reduce requirements for drilling and blasting. Figure 4.4.3. Drill and blast method is still being used when drifting through competent CRF. The objective of using a roadheader machine is to access ore which was encircled by CRF. The use of roadheader allows for more direct access to crown, sill and vertical pillars were valuable ore is entrapped. Further, The drifts cut through CRF are more stable due to elimination of blast damage.

Various types and sizes of roadheaders were reviewed and the operational functions were compared. Dosco MK-2A was selected mostly due to the restricted size of the opening to be excavated. Wittchen, 1990.

The work cycle of the roadheader consists of two parts; excavation and ground support. The excavation portion comprised of cutting, loading and hauling the backfill while the ground support portion consist of erecting two-piece steel arches using the machine's cutting boom or shotcreting.

The actual cutting time per shift for drifting ranges from 18% to 23% on an eight-hour shift with approximately 20 m^3 of backfill being excavated. This translates to 1.4 m advance per shift.

Before the start of using shotcrete, supporting the excavated drift in backfill was the major time-consuming element in the work cycle. Hollybank two-piece steel arches of grade 50 steel on 1.2 m centres were installed. Figure 4.4.4.

some of the advantages of roadheader machine are:

1: Drifting with roadheader can achieve an average of 1.2 m advance per shift with only 1.5

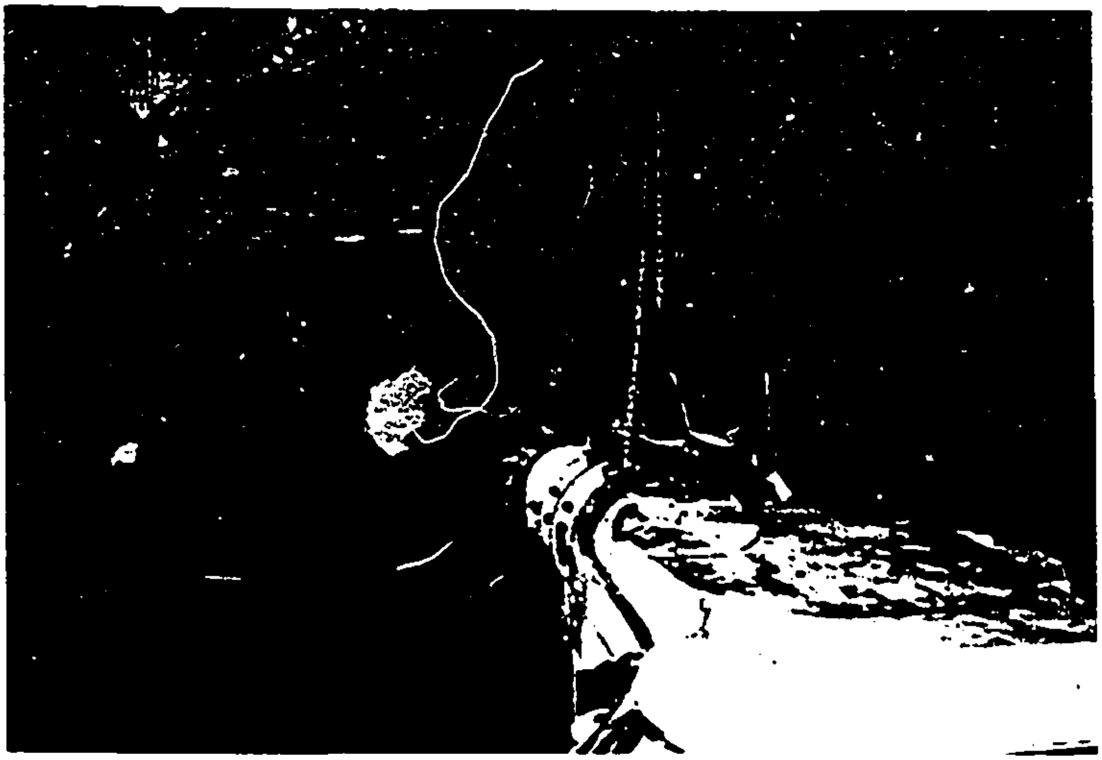


Figure 4.4.3: Mechanical excavation method to drift through CRF.

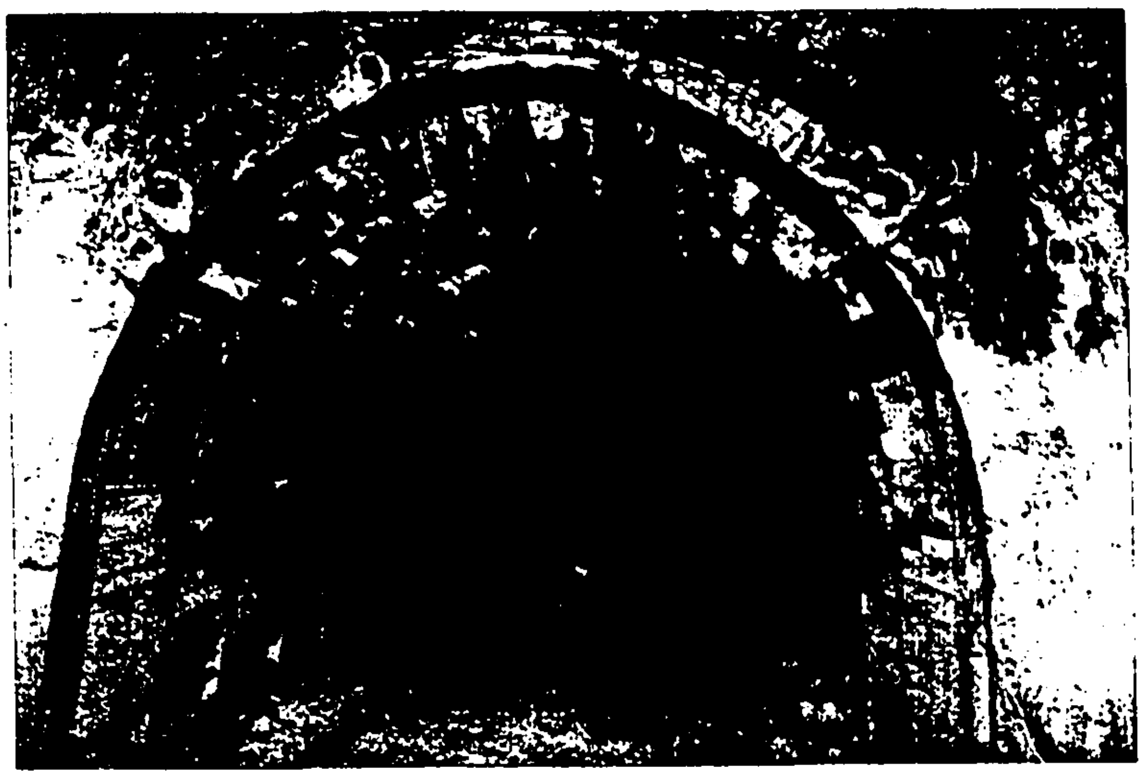


Figure 4.4.4: Hollybank two-piece steel arches for drift support.

hour cutting hours. With present use of shotcrete the advance rate has doubled and is comparable with conventional drifting method through CRF. The cost is also comparable with conventional drifting at around $1800/meter.

2: The backfill is more stable after cutting with the roadheader compare to drill and blast method.

3: The roadheader cuts through areas of less consolidated backfill with success and this is not as easy when drilling and blasting method is used.

4.5: ZONE MAPPING

Due to operational and some research needs at KCM, there are approximately 500 meters of drifting through CRF with reasonably good success. The mapping of these drifts to investigate the in situ behaviour of the CRF matched well with the main objective of this thesis. Around 300 meters of the backfill drifts were extensively studied by the author to investigate; the extent of segregation due to different fill methods and stope sizes, the effectiveness of lower cost binder alternatives and other parameters effecting overall fill quality such as success in mixing and attrition effect. The stopes that were studied were:

2020-L-PIL, 37 meters.
2021-M-ST, 27 meters.
2027-L-ST, 35 meters.
2027-M-ST, 38 meters.
2021-L-ST, 20 meters.
28-663, 28-664, and 28-644 stopes were also studied, 109 meters.

At least 5 more drifts that were conventionally driven were studied to distinguish different zones in a typical CRF mass.

To study each drift following steps were taken:

4.5.1: STOPE FILL HISTORY

Stope filling history was reviewed. The information obtained from the fill report were:

- Fill method, conveyor or truck.
- Number and orientation of the raise(s) used.
- Binder contents and recipes in different phases.
- Aggregate size and moisture content while filling.
- Any operational problems during filling.
- Daily tonnage placed at different fill elevation in the stope.

4.5.2: CRF DRIFT MAPPING AND OBSERVATION

Most of the drifts were observed during cutting process, however investigation was carried out after drift completion. At this stage following parameters were investigated.

1: Approximate fill cone location from fill trajectory and/or deflection off the walls.
2: Aggregate sizing at different locations in drift.
3: Aggregate blending, amounts of fines, midds or coarse particles.
4: Fill competency by observation and Schmidt hammer testing. Some unconfined compressive strengths values were obtained from core testing. This could be done only in competent zones.
5: Approximate binder content by observation and some exact values for binder content by chemical analysis.
6: Degree of success in mixing.
7: Attrition, aggregate breakdown effect.
8: Extent of impact damage.
9: Colour of the fill mass for estimation of amount of flyash in the mix.

10: Through roadheader cutting records the success of cutting in different locations in the drifts were also obtained.

4.5.3: DISCUSSION

From above investigation, four distinct zones were identified in most of the drifts studied and they were as follow:

ZONE A: The wall on which a collision occurred and below the impact zone was like a concrete mass and had the highest in-situ strength. This also included the zone 5 to 10 meters away from all the fill peaks observed, when the fill did not collide with the wall. This area had high binder content, 7- 8%, and high uniaxial compressive strength, approximately 5-8 MPa. The aggregate sizing contained around 90% minus 7.6 cm. Minimized segregation was observed, except some small zones of coarse aggregate which probably were formed during the mass flow after the fill build up slope was grater than the angle of repose of the fill . Excellent coating of aggregate was notices, almost 90 to 95% success, Figures; 4.5.1, 4.5.2, 4.5.3, and 4.5.4.

ZONE B: This area had medium binder content, 3-5%, and uniaxial compressive strength of 2-3 MPa. This area had a good blend of coarse and fine aggregate and covers anywhere from 10 to 25 meters away from the fill peak and/or impact point. Some segregation was noticed, especially after 20 meters away from the fill cone. Approximately 80% of the aggregate mass was coated with slurry, Figures 4.5.5 and 4.5.6.

ZONE C: At the stope boundaries the fill was highly segregated and had a low binder content of 1-2% and compressive strength of 1-2 MPa. The lower strength in this area is due to the lack of fine particles, which were needed to avoid point contact between coarse particles and achieving higher tensile strength. The extremely segregated and weak zone typically started around

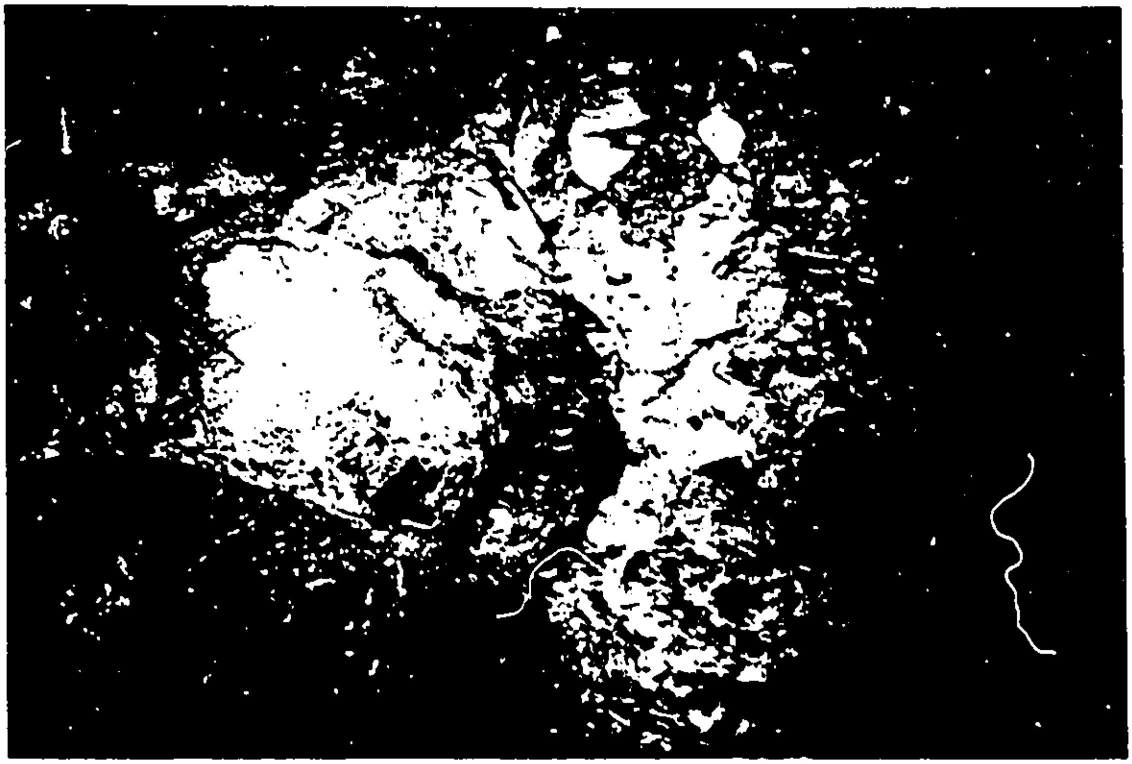


Figure 4.5.1: High strength CRF, zone A, 5 meters from cone in 2021-M-ST.

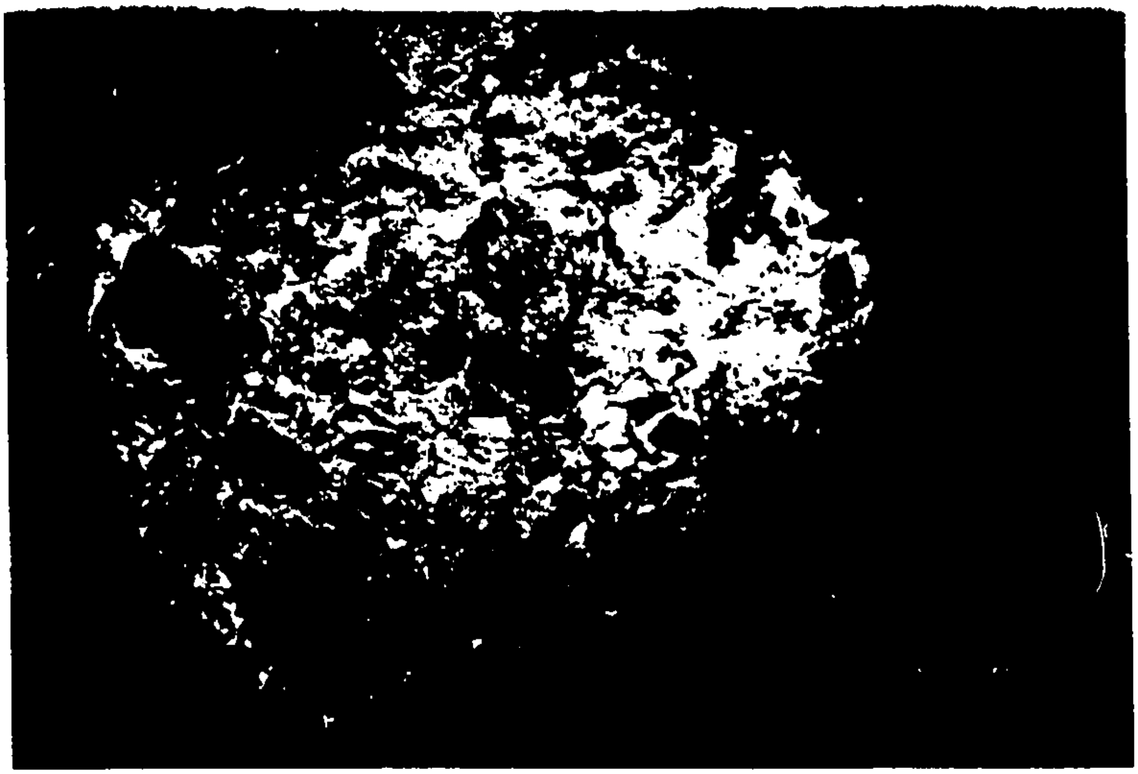


Figure 4.5.2: High strength CRF, zone A, 8 meters from cone in 2027-L-ST.

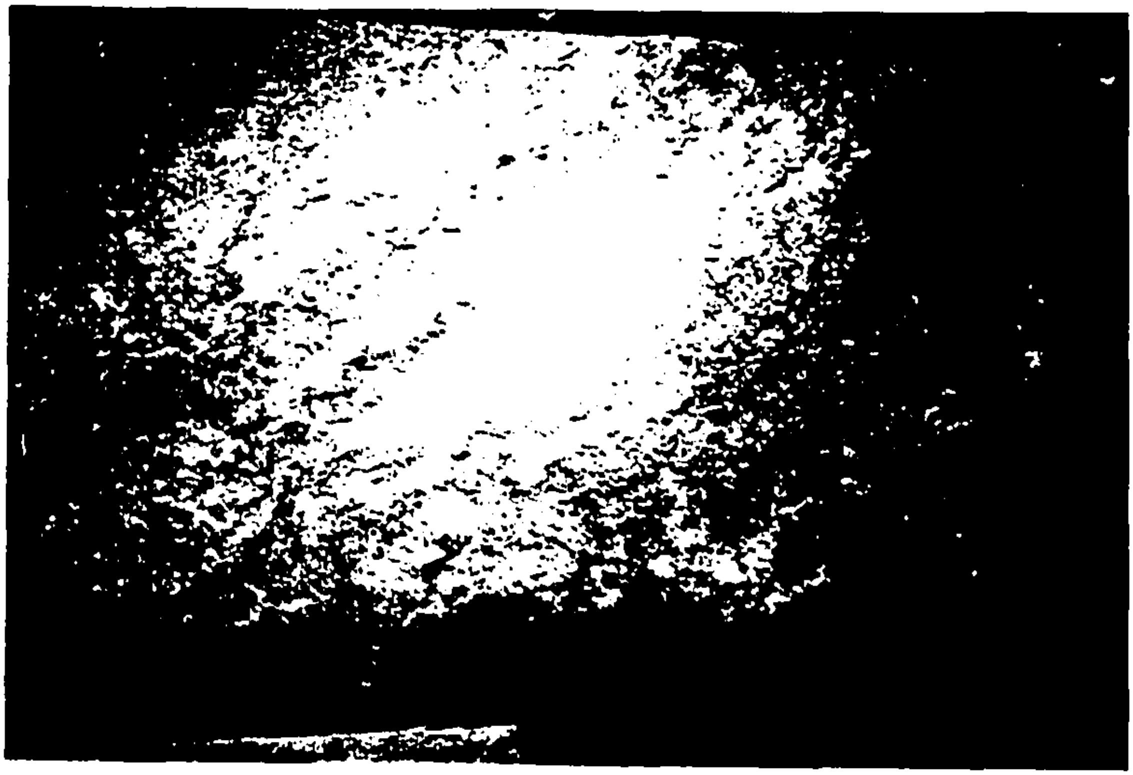


Figure 4.5.3: High strength CRF, zone A, at the cone in 2021-L-ST.

Figure 4.5.4: High strength CRF, zone A, at ore contact in 2021-M-ST.

Figure 4.5.5: Medium strength CRF, zone B, 17 meters from the cone in 2021-L-ST.

Figure 4.5.6: Medium strength CRF, zone B, 12 meters from cone in 2021-M-ST.

25-30 meters away from the fill cone. This zone contained almost all coarser aggregate and mostly the same size aggregate. Roadheader had great difficulty going through this zone and in some cases the drift did not advance any further when encountering this zone, Figures 4.5.7 and 4.5.8.

ZONE D: At the fill toe area the aggregate was also highly segregated and depending on fill raise orientation could be rich in binder content from the slurry running down towards the slope of the fill cone(s). Since the aggregate was mainly coarse the excess slurry did not increased the CRF strength considerably, Figure 4.5.9.

Some distinct findings in each drift were as follow:

2020-L-PIL (Figures 4.5.10, 4.5.11, and 4.5.12)

At start of the drift the fill looked competent, zone B, however started to segregate severely around 20 meters into the drift , zone C.

60% of the Portland cement was replaced with type C flyash with no observed problem.

This was a poor design, since the only wall to be exposed in future mining was against zone C type of material and resulted in 10-12% dilution in mining of 2019-J-PIL. The raise could not be angled properly towards this wall due to narrow stope back.

2021-M-ST (Figures 4.5.13, 4.5.14, and 4.5.15)

At start of the drift the CRF mass was like concrete, zone A, and it stayed competent throughout the entire drift with some segregation after 15 meters from start of the drift.

50% flyash replacement of Portland cement performed well.

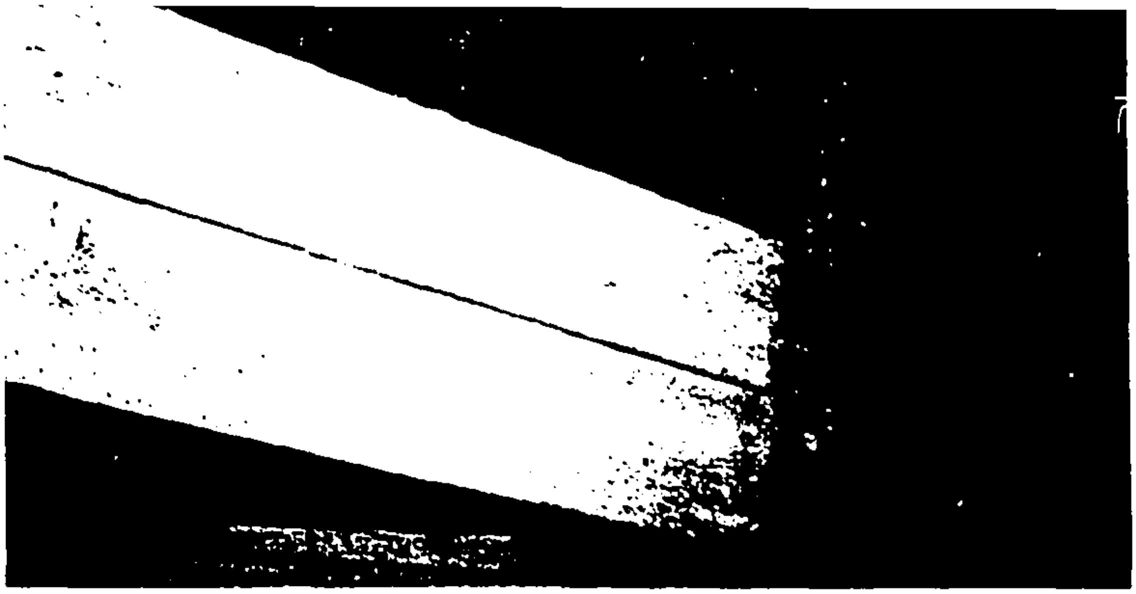


Figure 4.5.7: Low strength CRF, zone C, 21 meters from the cone in 2027-L-ST.

Figure 4.5.8: Low strength CRF, zone C, in stope drawpoint, 25 meters from cone.

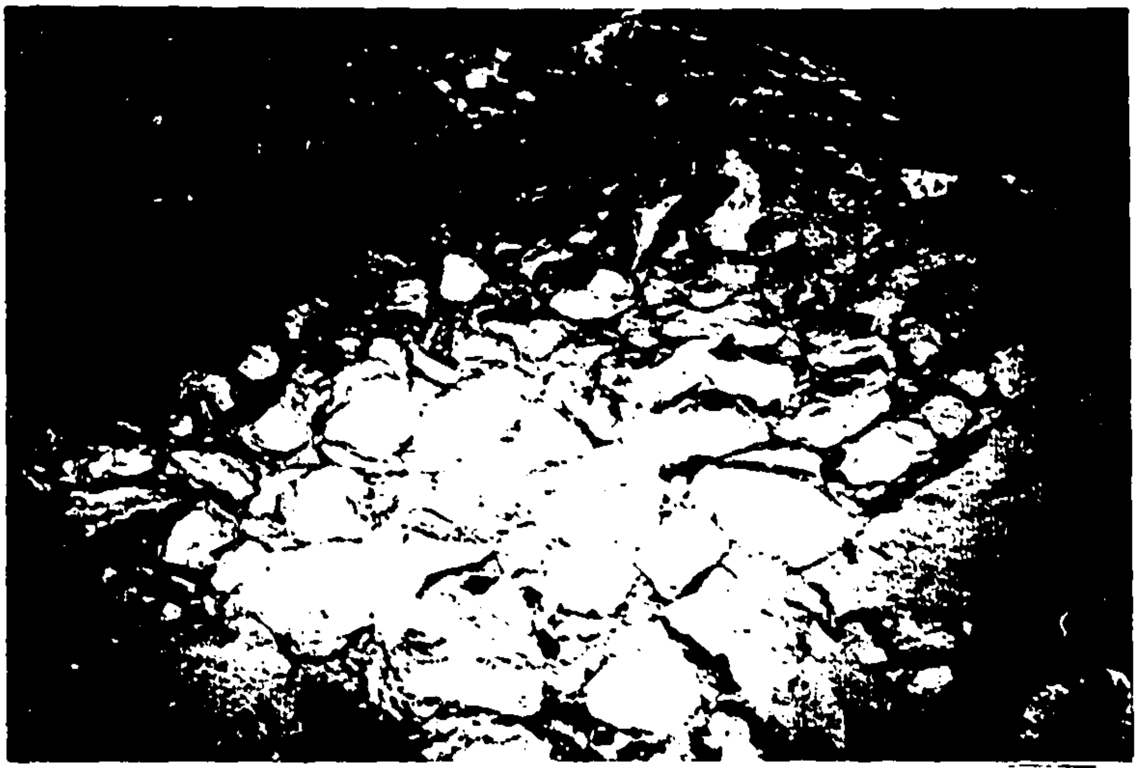


Figure 4.5.9: Low strength CRF, zone D, in stope drawpoint.

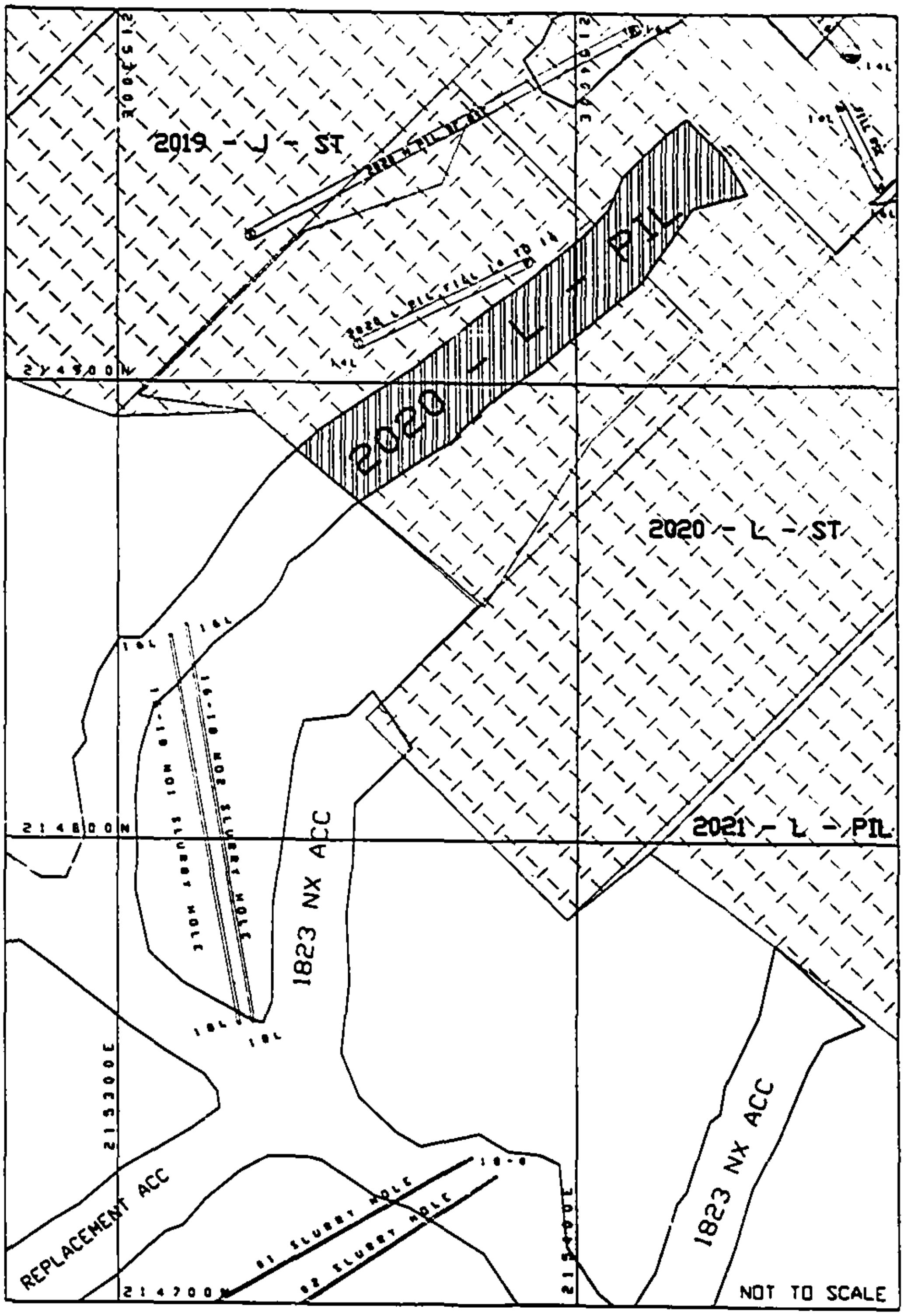


FIGURE 4.5.10: LEVEL PLAN SHOWING 2020-L-PIL ROADHEADER DRIFT

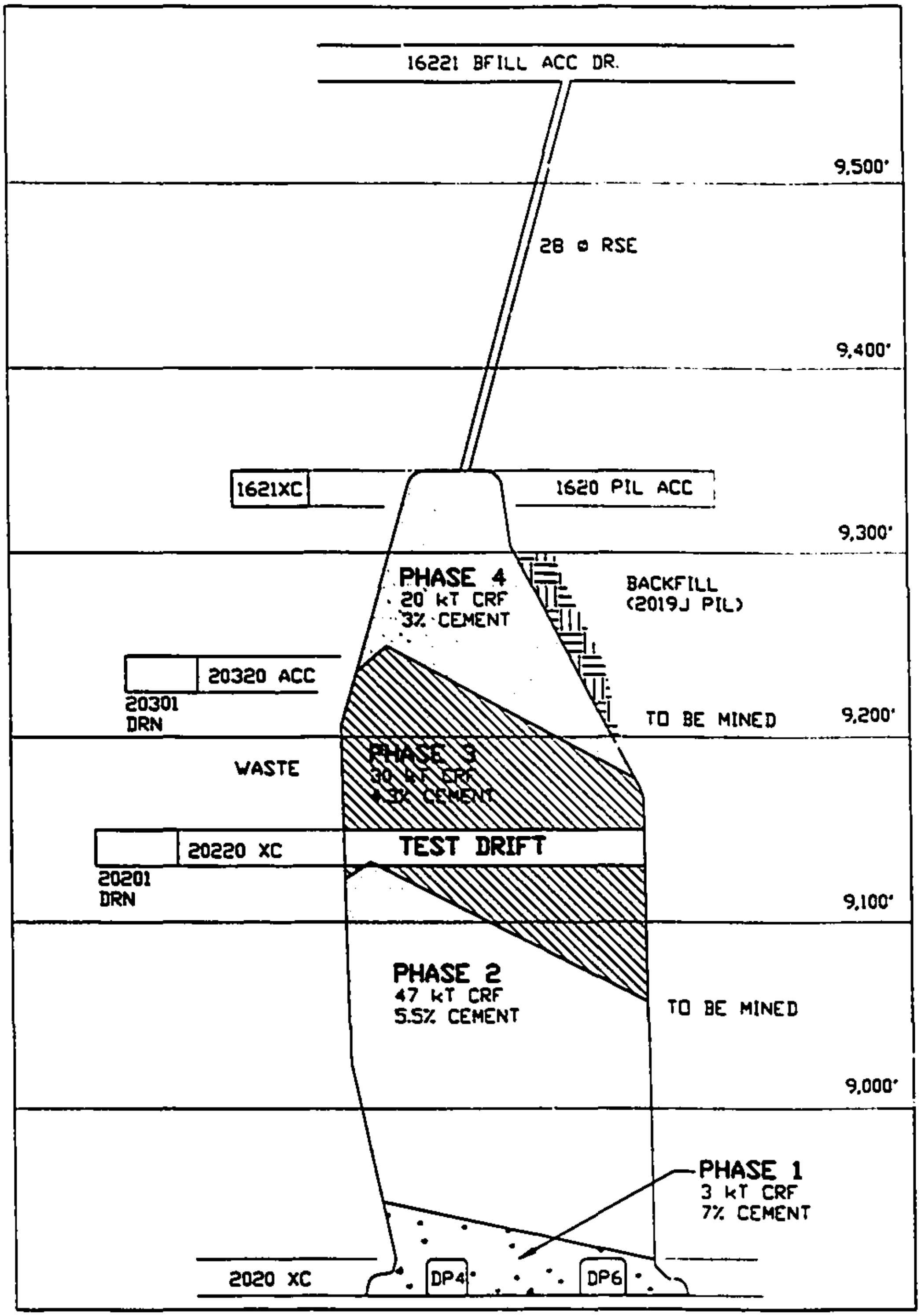


FIG: 4.5.11 2020-L-PIL LOOKING NORTH

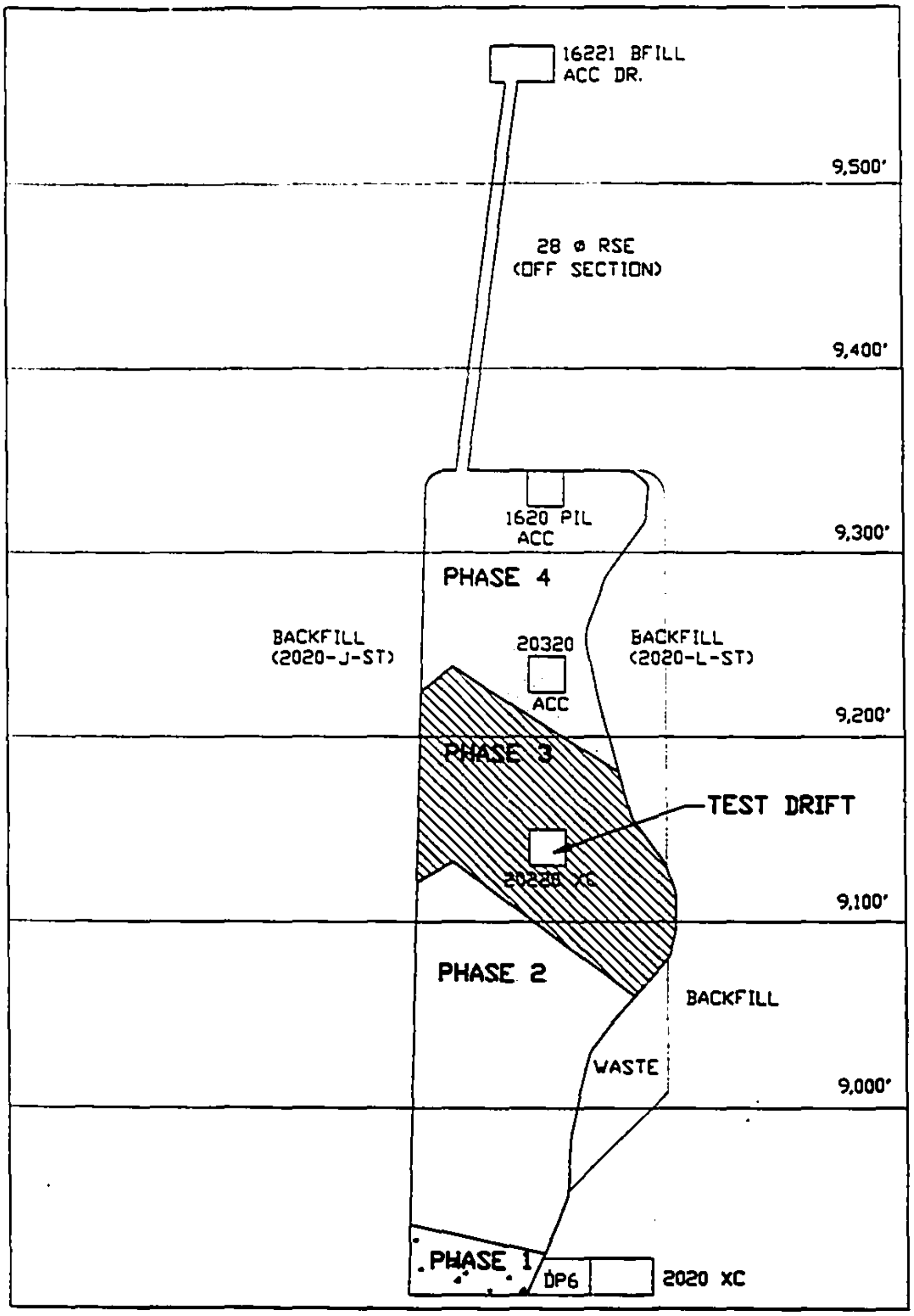


FIG: 4.5.12 2020-L-PIL LOOKING EAST

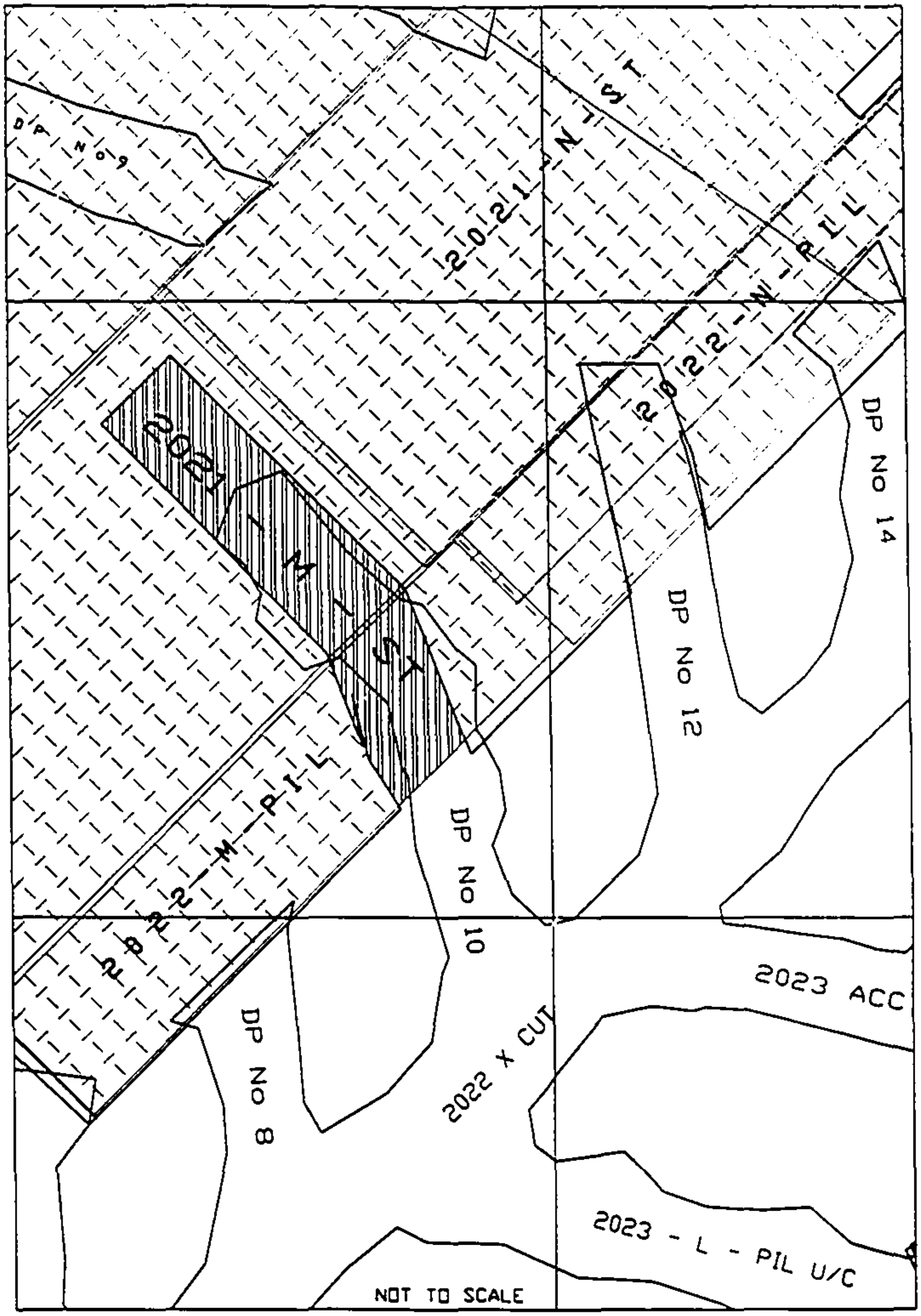


FIG: 4.5.13 LEVEL PLAN SHOWING 2021-M-ST ROADHEADER DRIFT

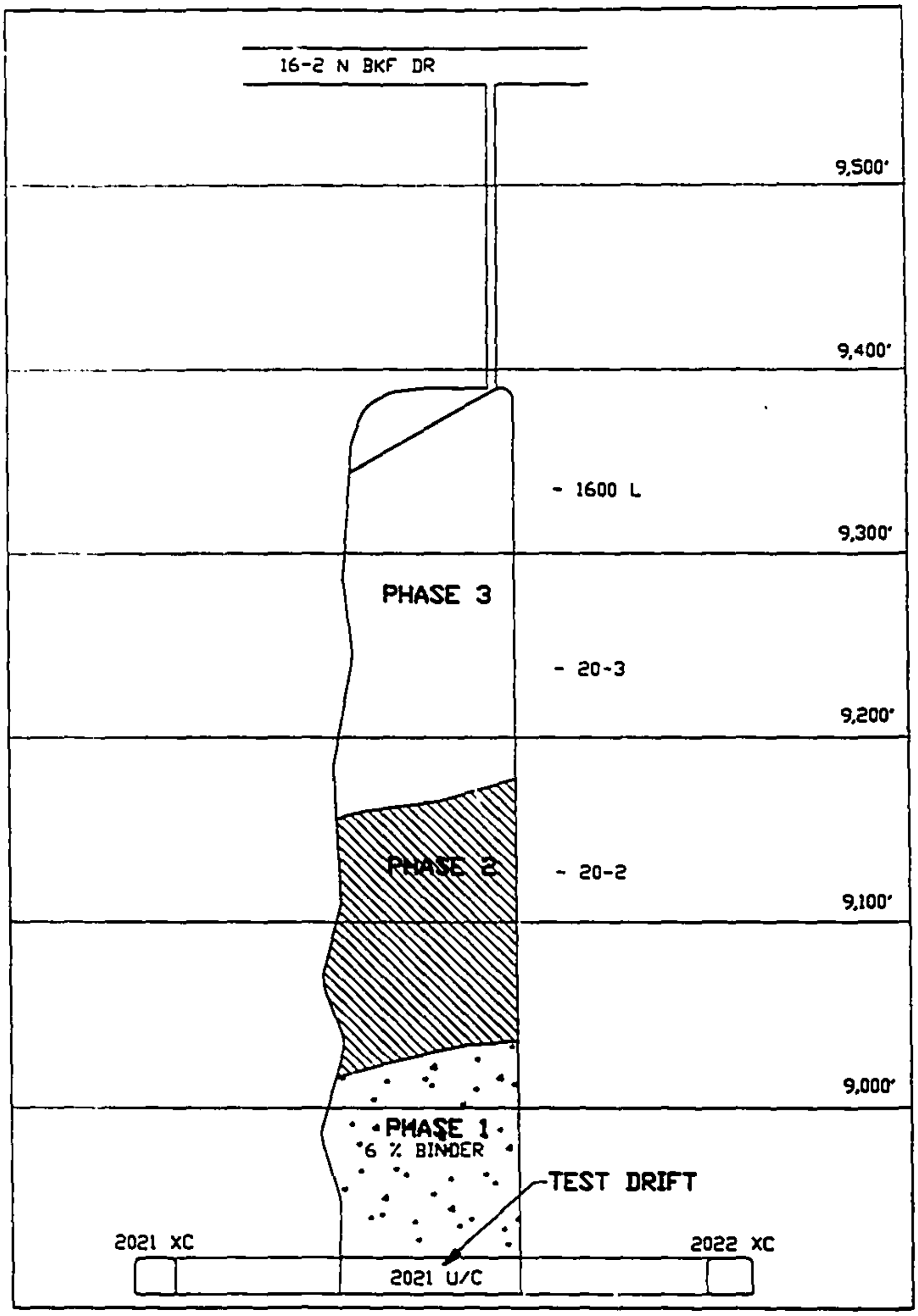


FIG: 4.5.14 2021-M-ST LOOKING EAST

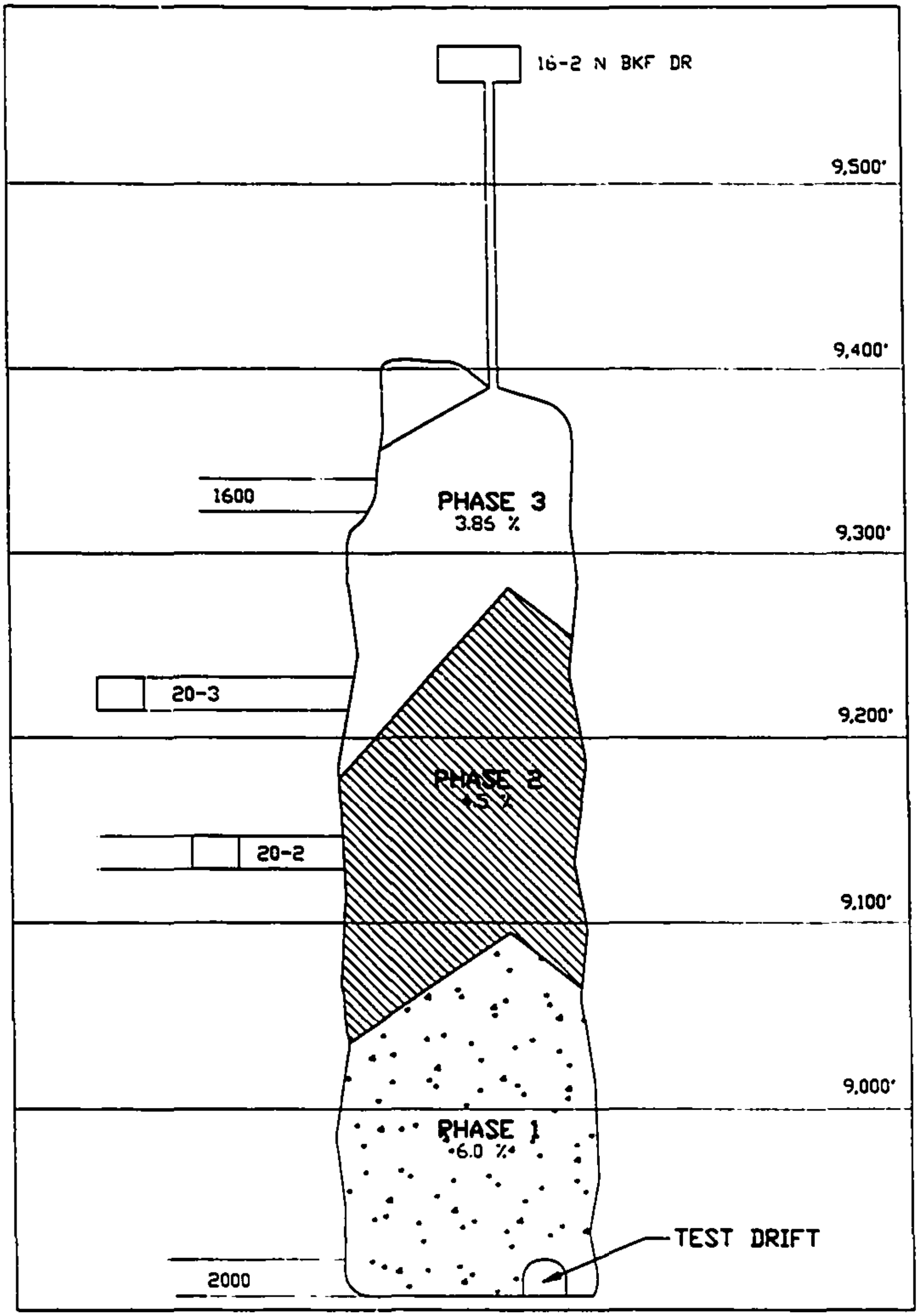


FIG: 4.5.15 2021-M-ST LOOKING NORTH

2027-L-ST (Figures 4.5.16, 4.5.17, and 4.5.18)

15 meters in the drift the CRF mass was very competent, zone A, and this was approximate location of the fill cone. Minor segregation was noticed 10-15 metes away from the fill cone.

This roadheader drift was placed in the perfect location in the stope since no weak mass, zone C, was observed.

2027-M-ST (Figures 4.5.19 and 4.5.20)

In general the drift was placed in zone B and C types of CRF. Zone A was not observed and the location of the roadheader drift was not ideal. Extensive segregation was observed on the north wall.

2021-L-ST (Figures 4.5.21, 4.5.22, and 4.5.23)

The roadheader drift was placed in good location and minor segregation was observed. The fill cone was approximately half way in the stope and one could clearly see the high strength material within the 10 meter radius of the cone and especially right at the cone.

28-663, 28-664, AND 28-644 STOPES (Figure 4.5.24)

These were the only drifts mapped that were filled using trucks. The fill was competent throughout the lengths of the drifts and zone C was not observed anywhere in the three drifts. Most of the drifts were in zone A and some areas it had minimal segregation, zone B. The areas at the fill cone was almost like concrete with very high compressive strength. The compressive strength at fill cone area using core testing technique averaged around 12 MPa. This clearly

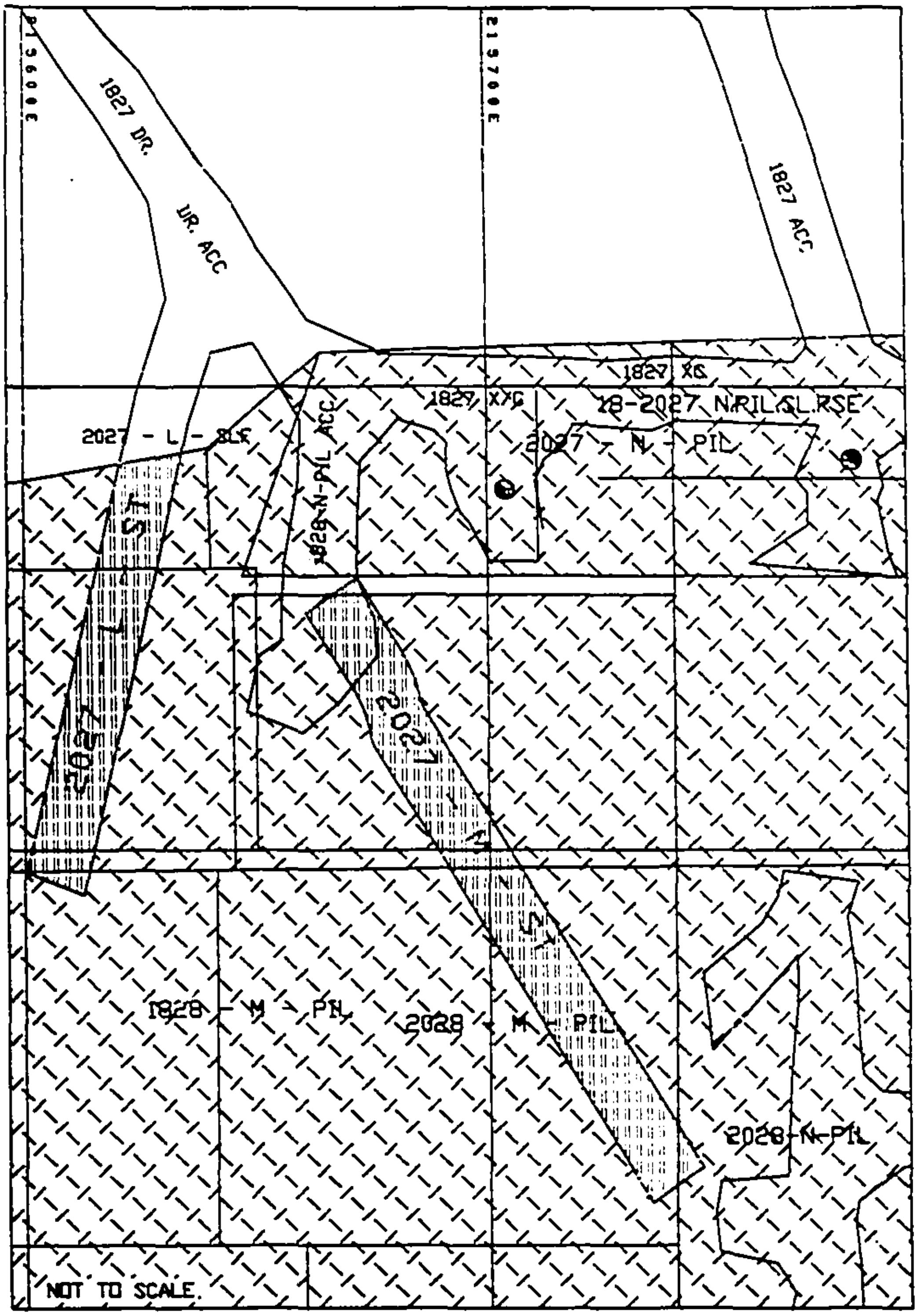


FIG: 4.5.16 LEVEL PLAN SHOWING 2027-L-ST AND 2027-M-ST ROADHEADER DRIFTS

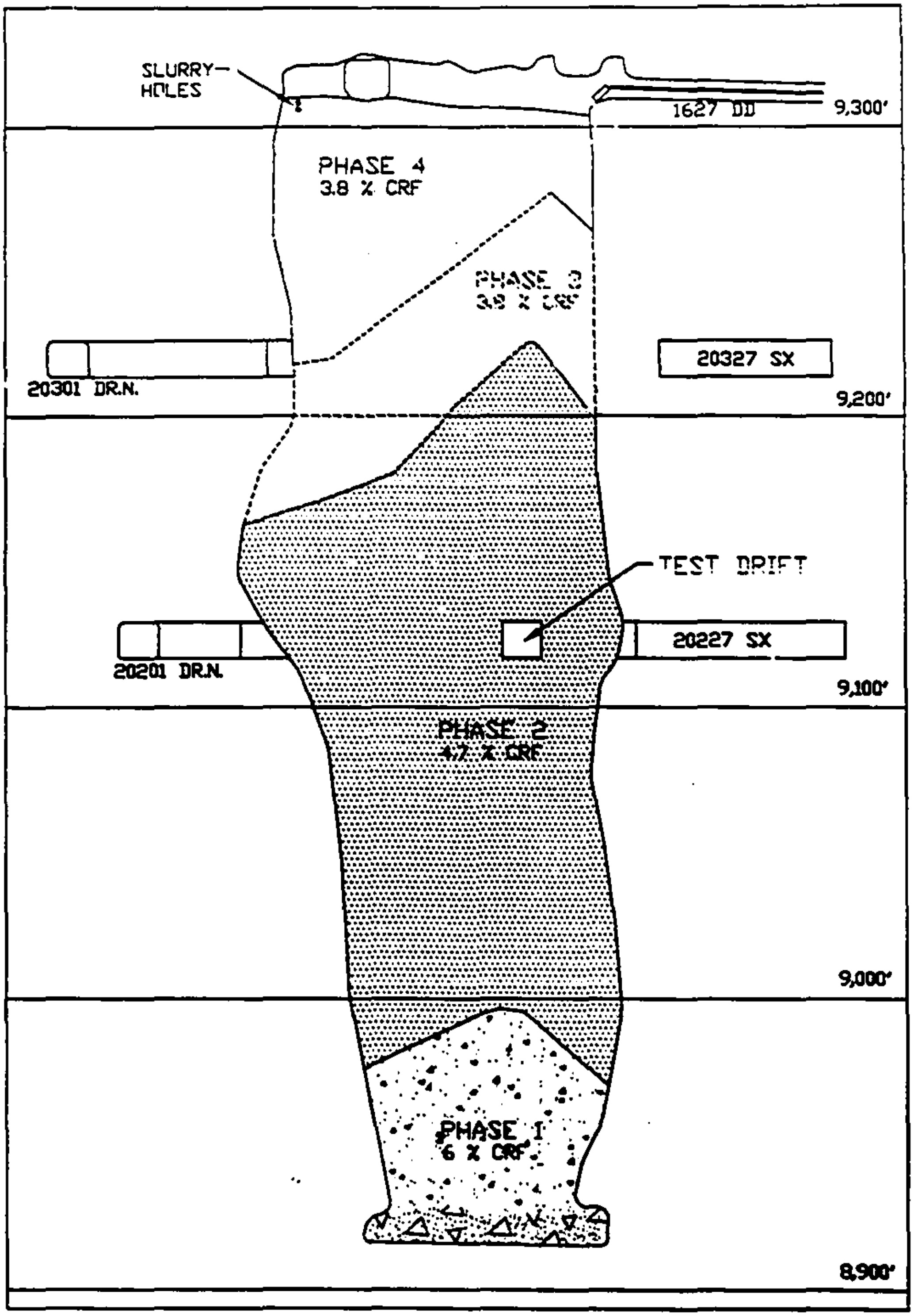


FIG: 4.5.17 2027-L-ST LOOKING NORTH

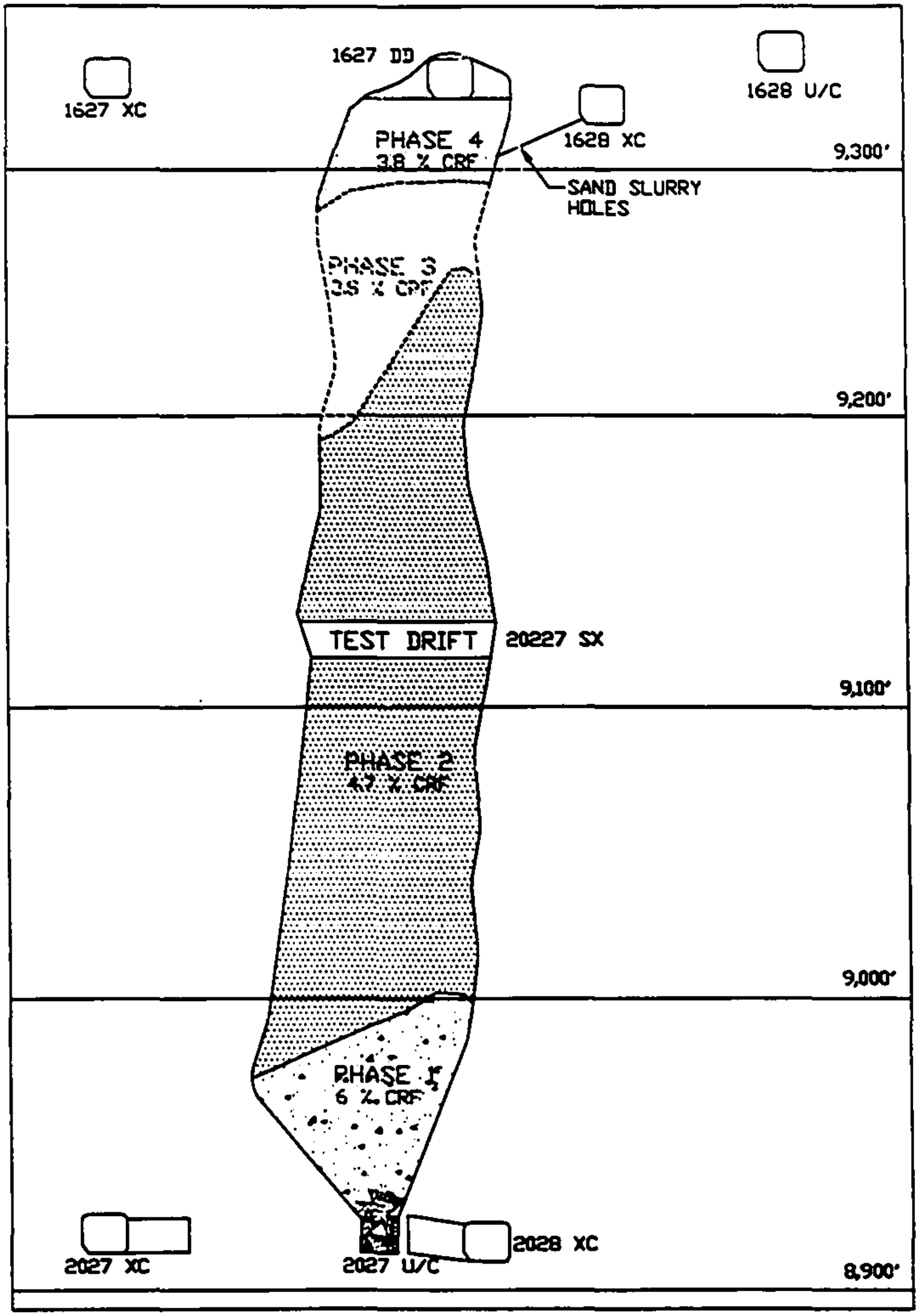


FIG: 4.5.18 2027-L-ST LOOKING EAST

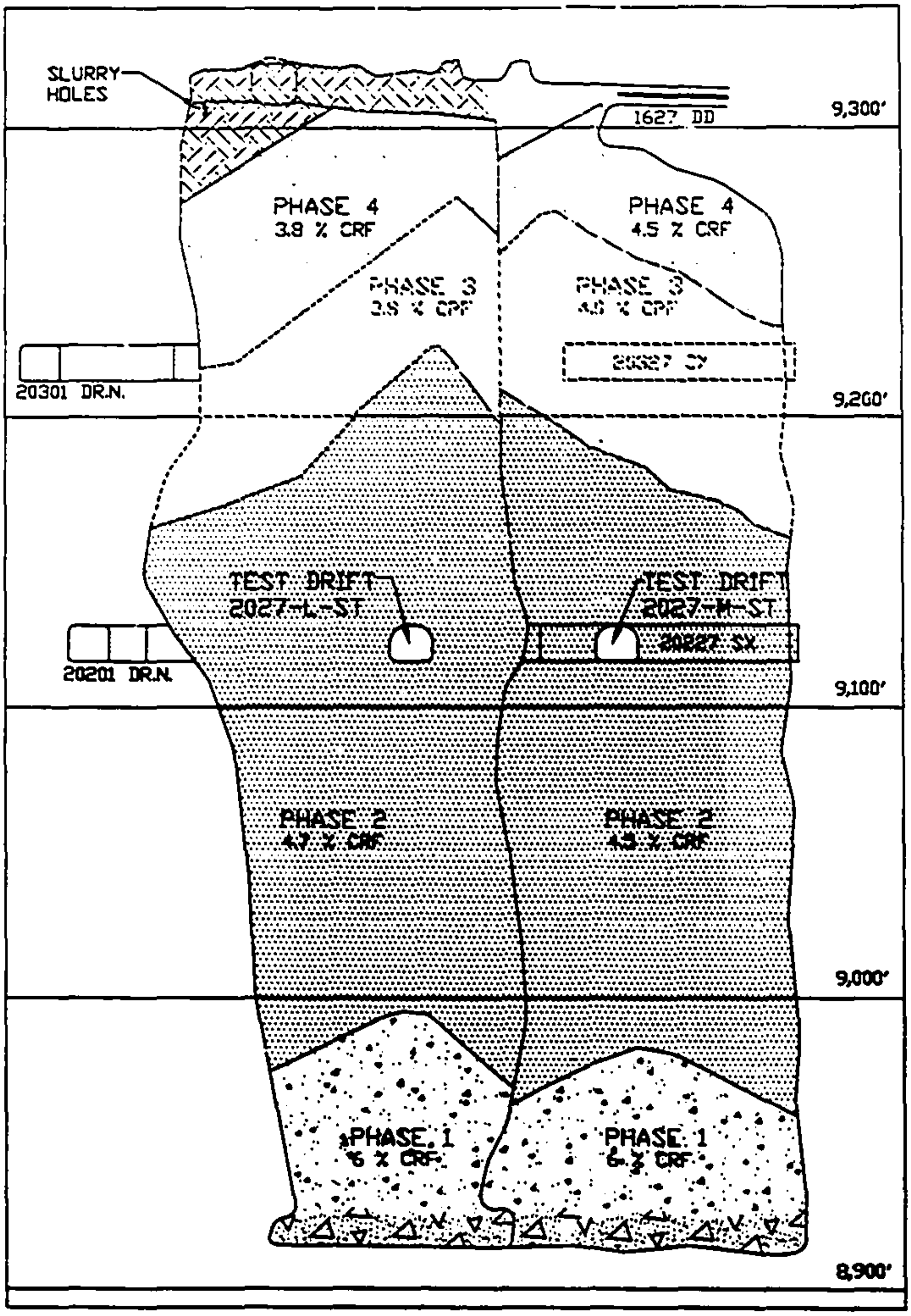


FIG. 4.5.19 2027-M-ST LOOKING NORTH

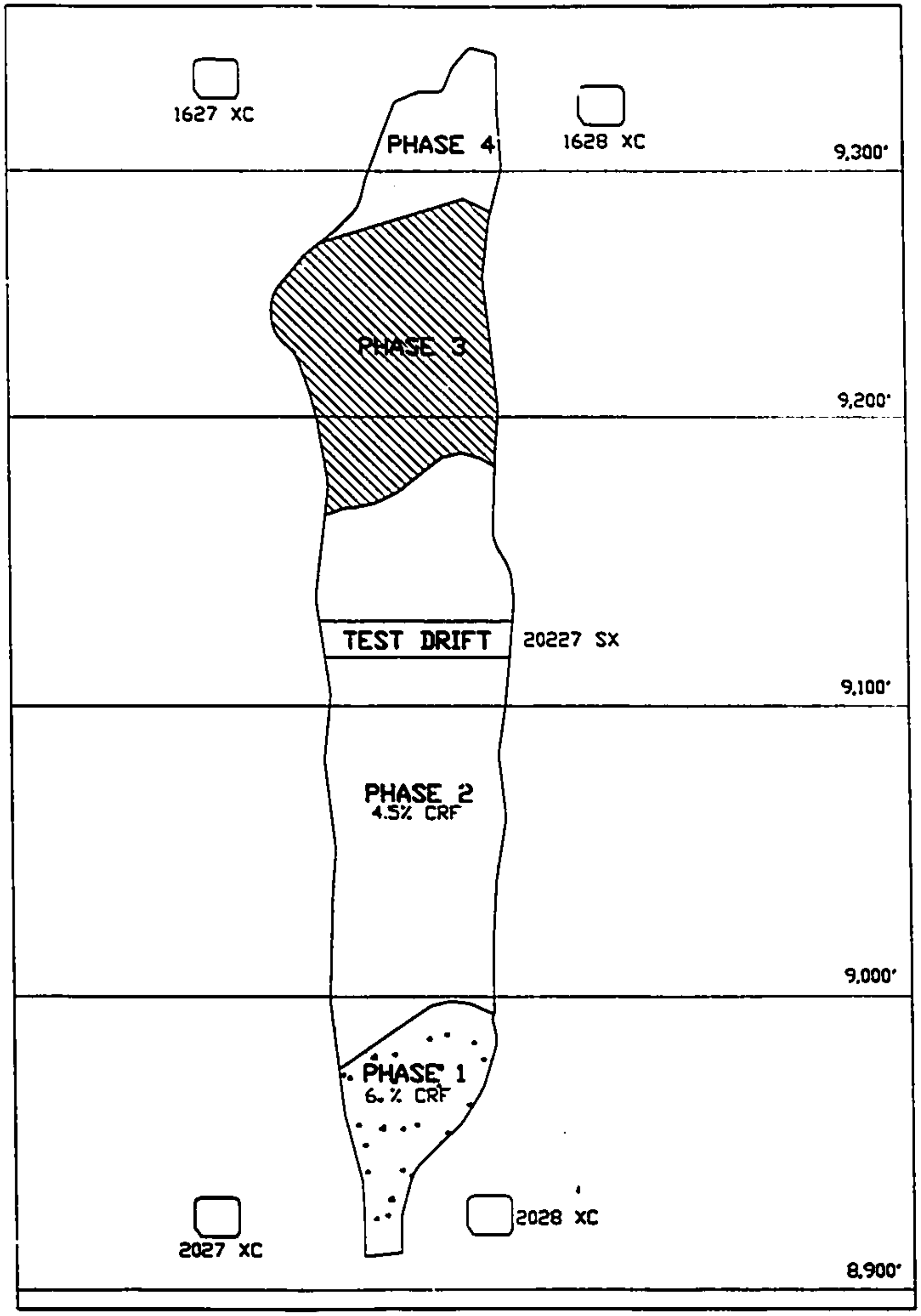


FIG: 4.5.20 2027-M-ST LOOKING EAST

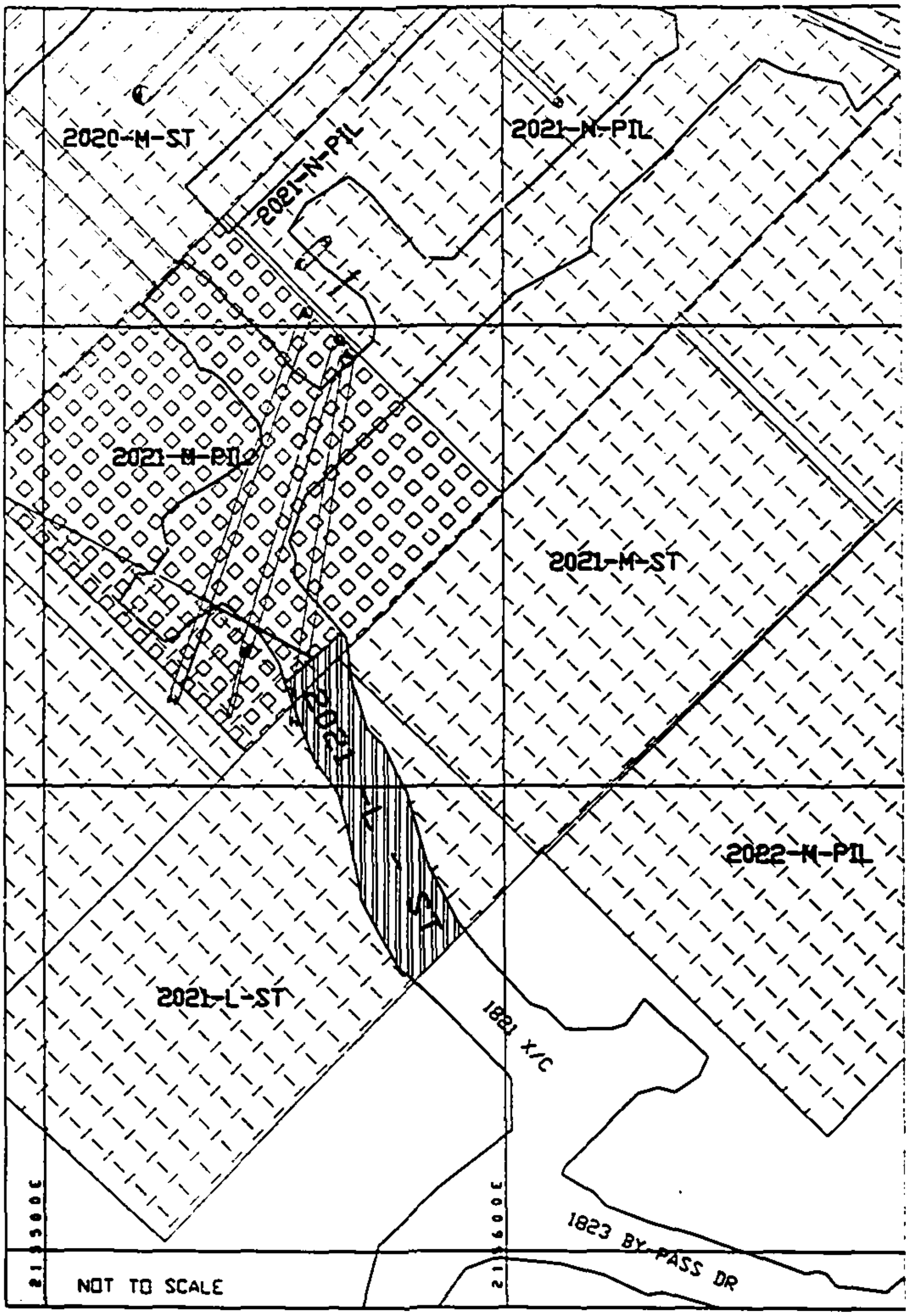


FIG: 4.5.21 LEVEL PLAN SHOWING 2021-L-ST ROADHEADER DRIFT

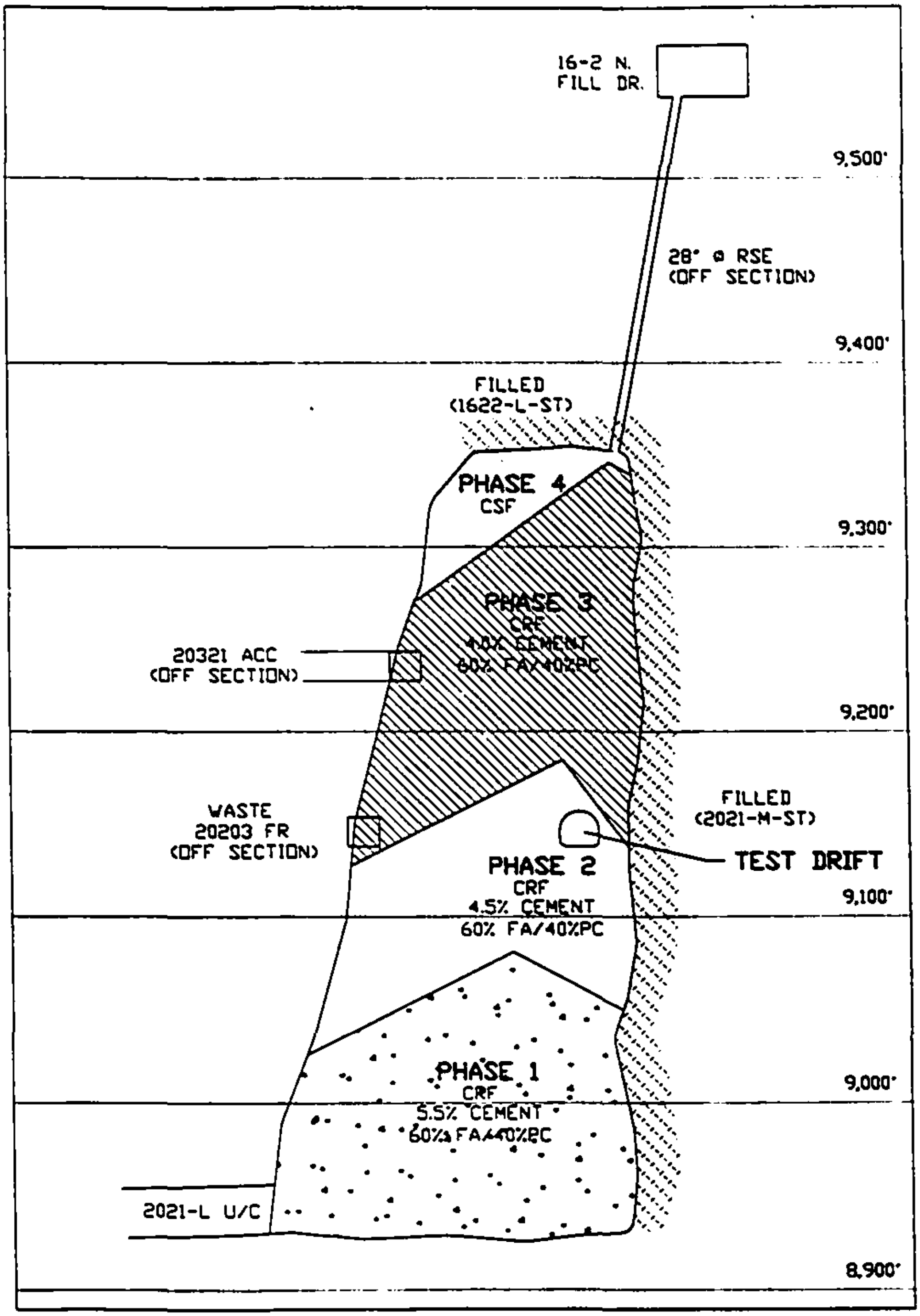


FIG: 4.5.22 2021-L-ST LOOKING NORTHWEST

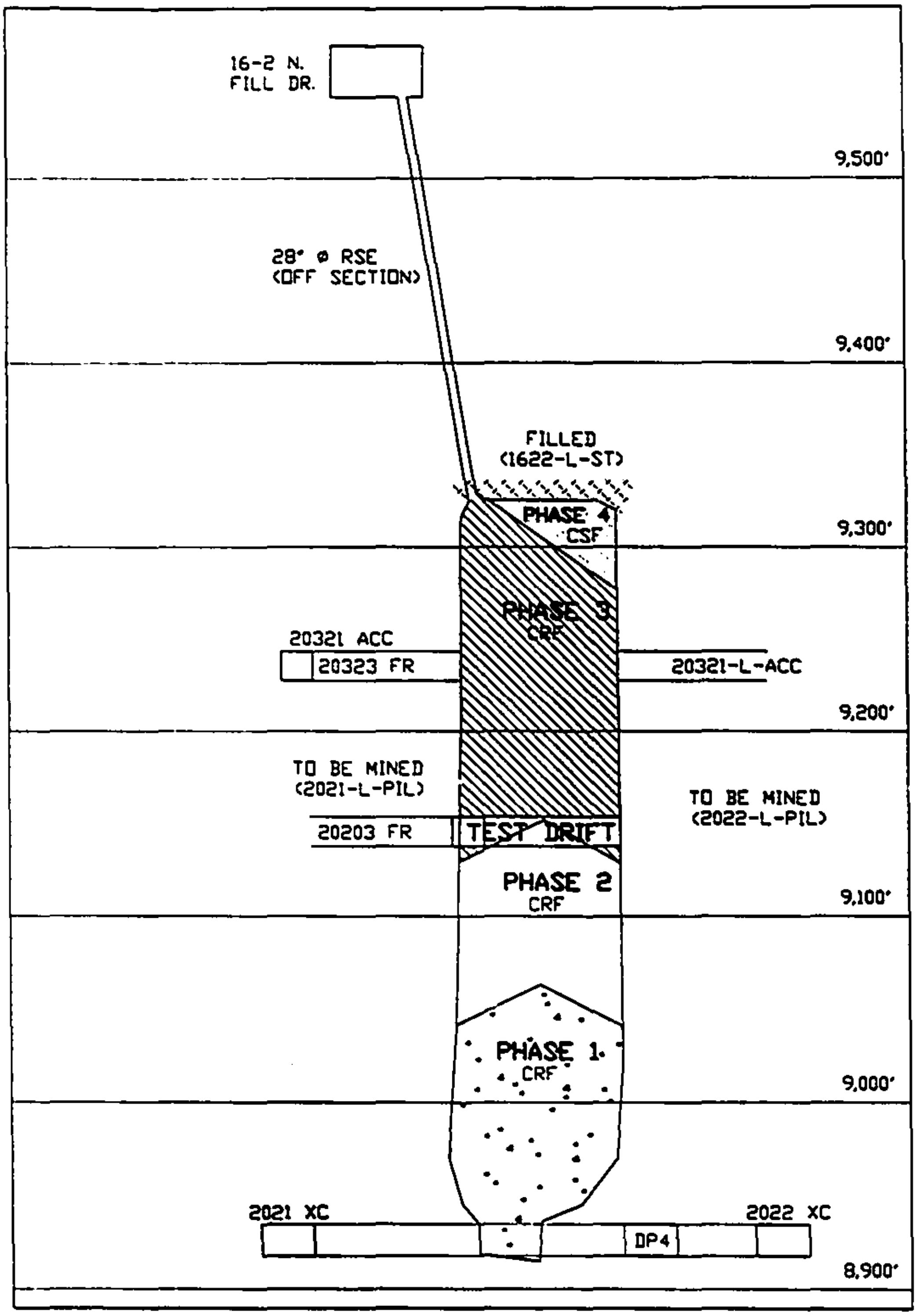


FIG: 4.5.23 2021-L-ST LOOKING NORTH EAST

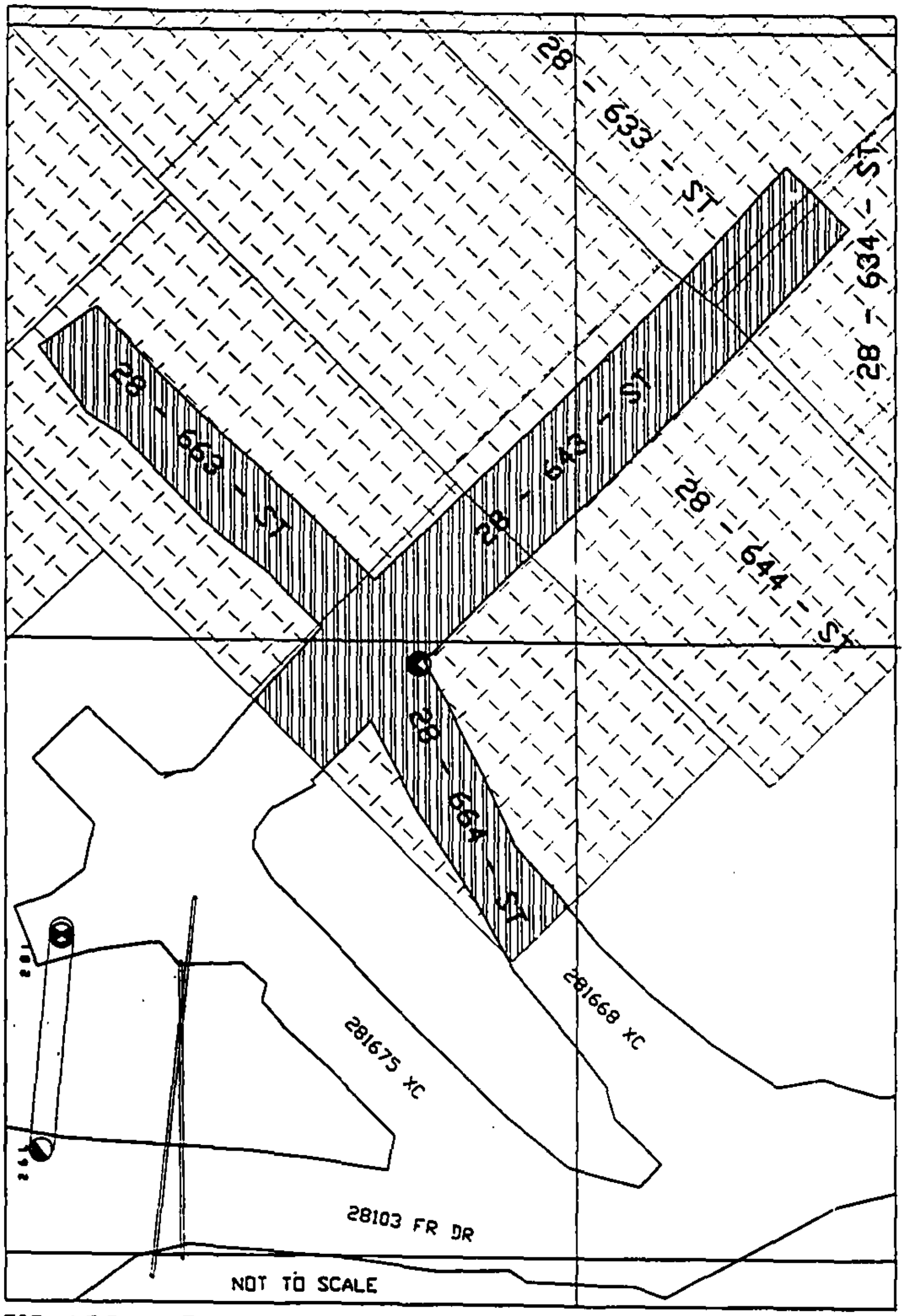


FIG: 4.5.24 LEVEL PLAN SHOWING 28-663, 28-643 AND 28-644 ROADHEADER DRIFTS (TRUCK FILLED) indicated that truck filling method could result in much stronger CRF mass due to minimized segregation and smaller size stopes. The roadheader crew did not face any problem when going through these drifts and averaged around 2 meters per shift which is comparable with conventional drill and blast method.

4.6 : SUMMARY

This chapter confirms that the most important factor in a properly designing CRF is to control and minimise segregation. The study identified four typical zones in a CRF mass and the mechanical behaviour of each zone were also identified. The drift mapping clearly identified that the smaller size stopes which were filled with trucks had considerably higher strength characteristics than compared to the larger stopes filled with conveyors. For larger stopes, when multiple fill raises were used to control segregation, the fill strength was comparable with the one's of the truck filled stopes. This indicated the importance of establishing design steps to optimize the structural integrity of CRF. Since each stope had different parameters such as: mixing method, aggregate sizing, fill rate, depth of the stope and others, it was concluded that proper quality control measures had to be established. This would result in better blending and mixing of the aggregate with slurry. The investigation also clearly identified that lower cost binder such as flyash performed extremely well and further investigation on binder mixes should be carried out, Figures 4.6.1 and 4.6.2. This chapter also shows the importance of in the field studies and the importance of having in situ methods to further understand the behaviour of the backfill. The following chapters will in detail investigate, CRF structural design, quality control measures, lower cost binder alternatives, and in situ testing techniques for CRF mass.

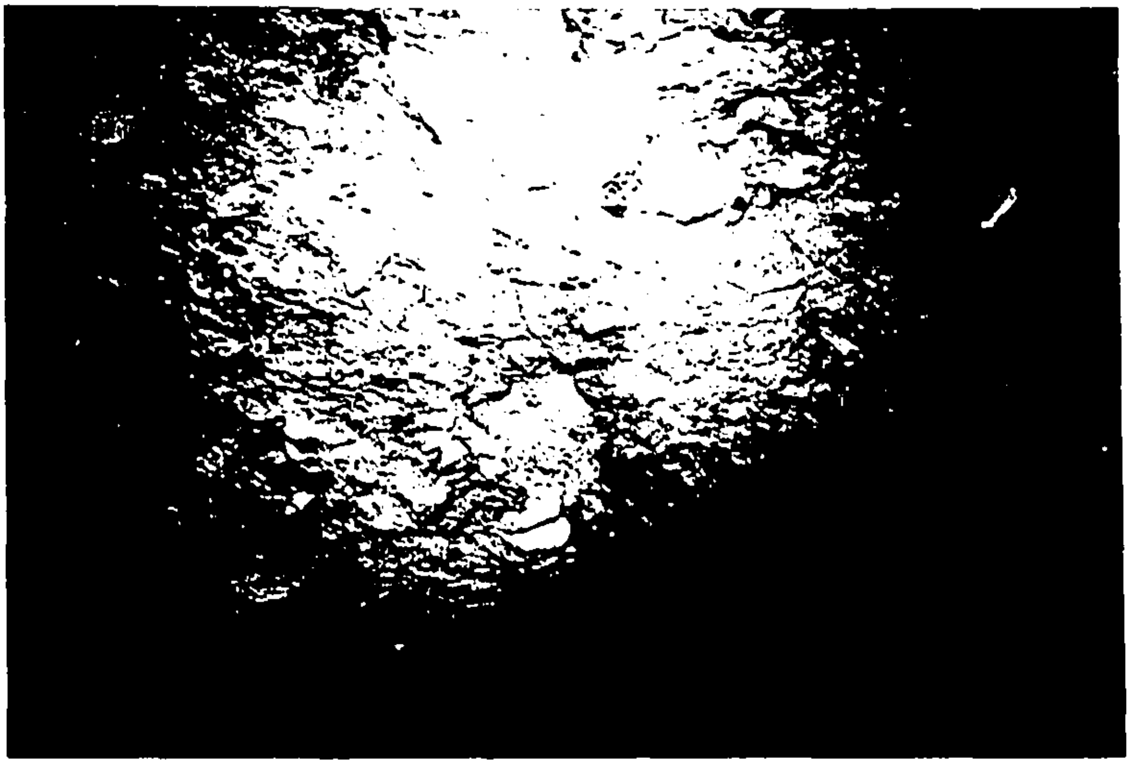


Figure 4.6.1: 60% flyash /40% Portland cement mix in 2020-L-PIL.

Figure 4.6.2: 50% flyash/ 50% Portland cement mix in 2021-M-ST.

5.0: CRF STRUCTURAL DESIGN

Last chapter concluded that the most important part of design for CRF mass is to minimize segregation . This chapter describes the parameters to be considered for minimizing segregation. Following parameters should be considered when structurally designing CRF; orientation of backfill raise(s), extent of segregation, degree of success in mixing the aggregate with slurry, the extent of impact damage due to the free fall height, and the size of the aggregates entering the stope. From the results obtained in chapter 4 and small scale physical modelling in this chapter, ideal fill set-up conditions to control segregation are also presented.

5.1: SEGREGATION PHENOMENA

Structural differences when filling a stope with consolidated rockfill are mainly due to segregation phenomena, Figure 4.4.1. Segregation of consolidated rockfill is unavoidable but it can be minimized if fill operations are well planned and closely monitored. Once the backfill material has left the belt or been dumped from the truck into the stope, limited control or remedies are possible. Close observation of the state of the fill during placement may need changes to for example, pulp density of slurry, amounts of fines in the aggregate or addition of extra cement slurry to lower the extent of segregation. As mentioned earlier, the fill in the raise attains a specific horizontal and vertical velocities which in turn determines the trajectory of the fill in the stope. When fill material leaves the fill raise, the trajectory of the fill may be such that it will collide with the wall of the stope above the floor. This collision will cause two things to occur:

1) A portion of the slurry which was coating the aggregate will be knocked off and run down the wall,

2) Due to the greater momentum of the larger particles, they will rebound further in a horizontal direction compared to finer particles.

Because of these two occurrences the area below the impact zone and on the same side of the stope which the collision occurred will tend to be the strongest side of the stope at points A & B on Fig. 5.1.1, case 1. If one considers a point above the level of impact zone, point B of case 1, four conditions could exist:

1) Excess slurry or water will fall short of the peak of the fill and have a tendency to flow down the slope of the fill in the direction of the fill raise. While the slurry is flowing down the slope it will carry some of the fines with it.

2) Due to having higher momentum the larger particles will rebound further horizontally than the finer particles after impact. This will lead to an accumulation of coarse particles on the wall of the stope. This would occur to a portion, rather than all of the coarse particles.

3) Upon impact other portion of the coarse particles will bury themselves rather than rebound which will cause some of the coarse material to remain in the backfill cone area.

4) Due to effect of friction and cohesion in the raise as well as air resistance in the stope, the finer particles neither leave the raise with the same velocity nor have the same acceleration in the stope as the coarser particles. This would account for the fines not travelling the same horizontal distance in the stope and thus magnify the effect of an increased percentage of coarse material on the side of the cone towards the wall which the backfill previously collided with.

Point C in Fig 5.1.1, case 1, represents a location where an accumulation of coarse particles is expected. The appearances of the backfill at this point can be like either coarse concrete, due to excess slurry washing a portion of the fines down from the fill peak towards this wall, or unconsolidated conglomerate pile with no excess slurry to fill the voids.

Roadheader drift investigation concluded that:

1) The wall on which a collision occurred and below the impact zone, point A, is like

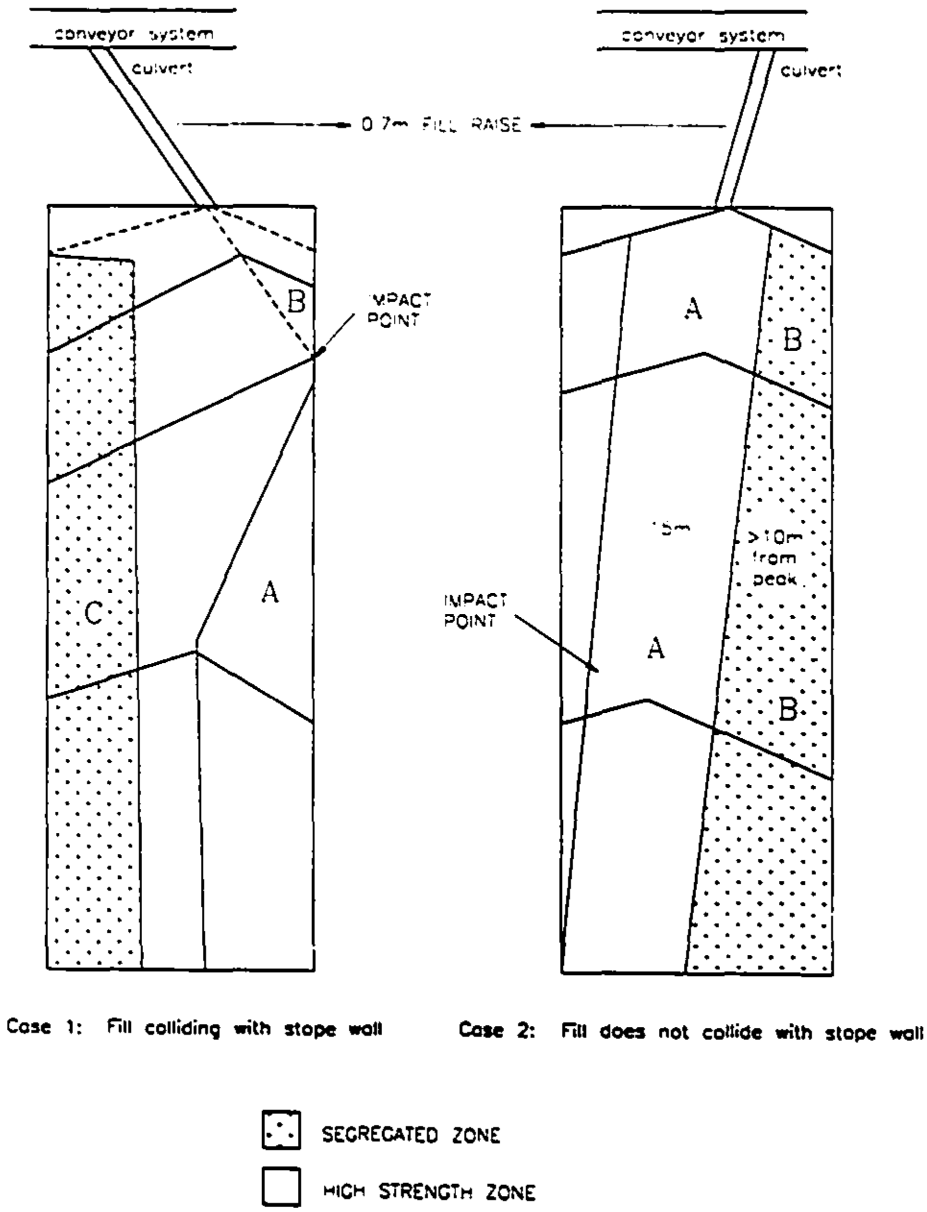


Fig: 5.1.1 Backfill Profile

concrete and has the highest in-situ strength, Figure 5.1.1.

2) The wall on which a collision occurred and above the impact zone is partially segregated, point B in Figure 5.1.1.

3) The wall opposite the one on which a collision occurred and below the profile of the fill at the impact zone is like coarse concrete or conglomerate, point C in Figure 5.1.1.

5.2: PHYSICAL MODEL TESTING

In early 1980's Kidd Creek Mines carried out several physical model testing programs to investigate the effect of raise (s) orientation on the extent of the segregation. The same was carried out by Inco Thompson at the early stage of the CRF system. The author was involved with the latest physical modelling test which was carried out by Inco Thompson. This was to repeat the test carried out at KCM in the past and utilizing author's experience for further investigation into parameters effecting segregation. Several fill raise orientations were investigated using the scaled plexiglass physical model to try to minimize the segregation occurring within the actual filled blocks. The trial was identical with the one carried out at Kidd Creek Mines.

In this test no moisture was added to the model fill material. This tended to exaggerate the extent of segregation, however the results verified the flow characteristics which occur within the stope. The coarse material was spray painted so it could be easily identified. A scaling factor of 40:1 was used for the model, raise and rockfill material. Cement slurry was not added to the material during the tests due to the scaling problems. In some tests water was added to simulate the slurry, Figure 5.2.1 shows the model set up.

Figure 5.2.2 shows the raise positioning and the results for each trial was as follow:

Figure 5.2.1: Physical modelling set up.

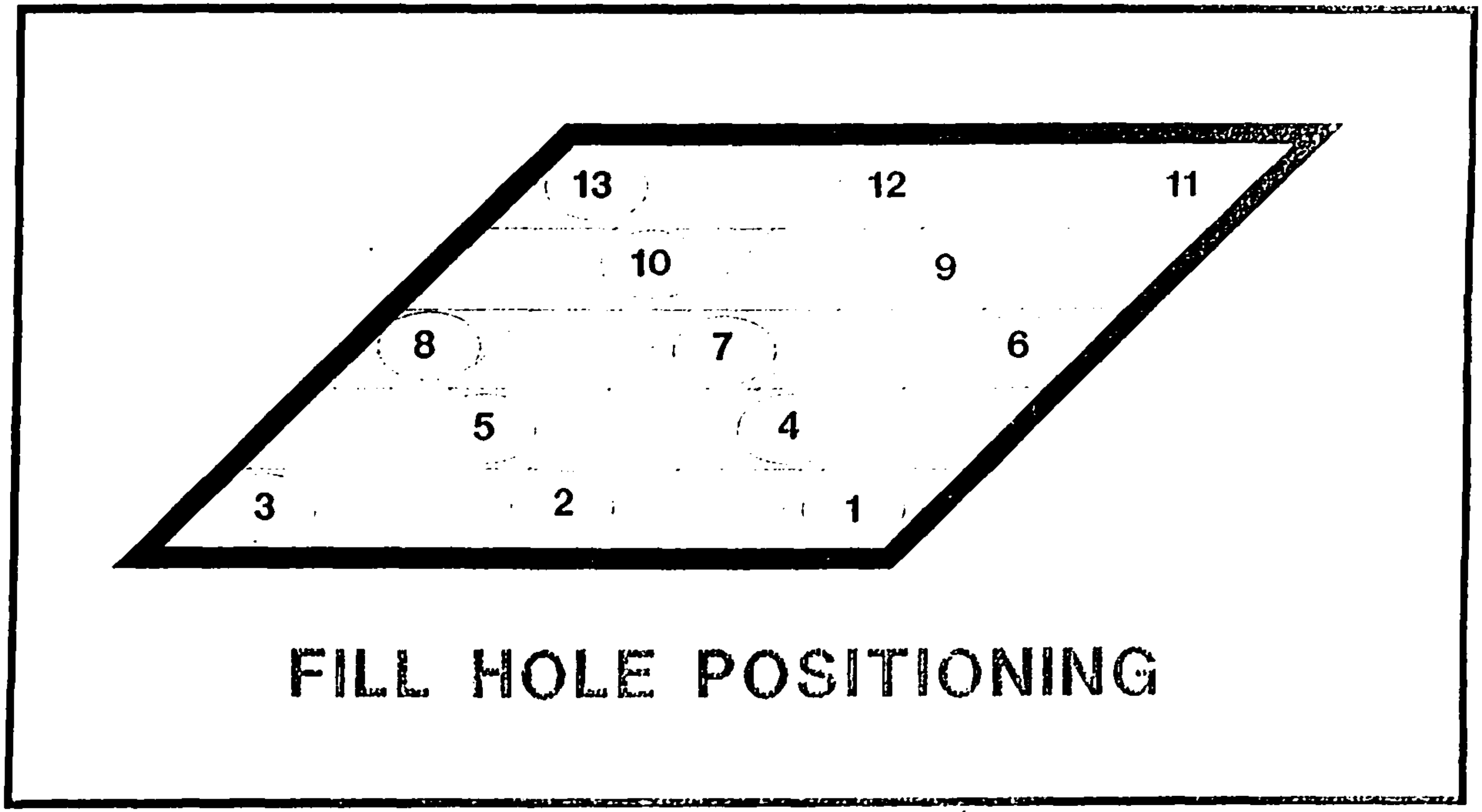


Figure 5.2.2: Fill raise positioning

Trial 1:

The majority of the coarse rebounded off the footwall and migrated towards the far end of the model, while the fines remained at the footwall. The method created the extensive segregation. This segregation action could be useful in a multi-raise system if used to place the coarse material in the center of the block rather than at the toe and walls, Figure 5.2.3.

Trial 2:

This set up produced a fairly homogeneous mixture as compared to the other trial methods. More coarse remained embedded within the impact cone than with any other method. However segregation was hard to control with this method and a large amount of coarse aggregate still migrated towards the walls, Figures 5.2.4, 5.2.5, and 5.2.6.

Trial 3:

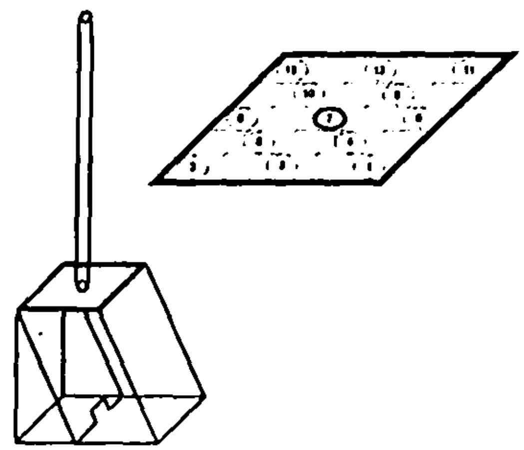
This method was intended to confine some of the coarse material between the impact cones and keep the fines at the walls of the model. By placing the fill raises at the edges of the model, the fines remained at the walls while most of the coarse rolled towards the center and toe of the model. Both of the raises were positioned so that the fill would hit as high on the stope walls as possible, causing the fines to remain at the walls. Fill raise #13 was placed close to the footwall so it could top-off the block. Placement through this fill raise produced considerable segregation due to the rebounding effect off of the walls. The interaction between the two fill cones reduced the chance of failure along the one of the plains of weakness, along the fill cone slopes. Although the method performed fairly well by burying some of the coarse aggregate between the two fill cones, the toe end of the model still contained a large amount of coarse aggregate, Figures 5.2.7 and 5.2.8.

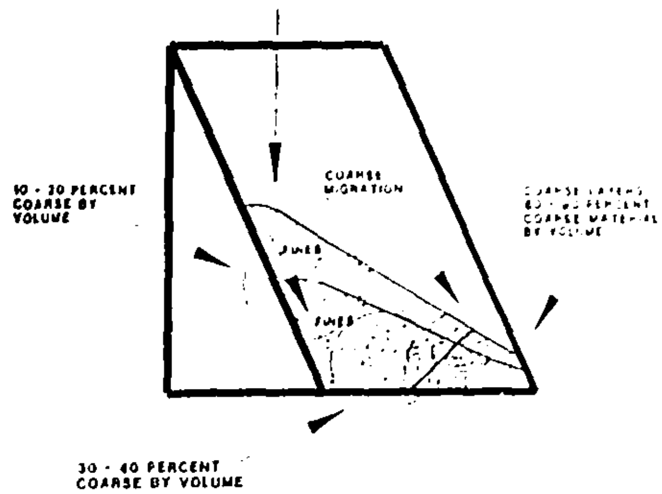
Trial 4:

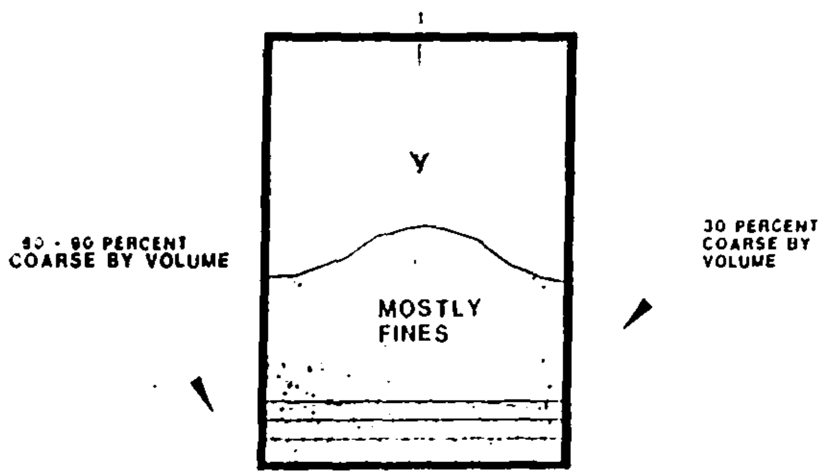
Figure 5.2.3

PHYSICAL MODEL TESTING

TRIAL #1 - HOLE 7 AT 90 DEGREES







FRONT VIEW

THE COARSE MATERIAL REBOUNDED OFF OF THE FOOTWALL AND MIGRATED TOWARDS THE BOTTOM END OF THE MODEL.

THE FINES SLID DOWN THE FOOT-WALL AND REMAINED WITHIN THE IMPACT CONE AND MIDDLE PORTION OF THE MODEL.

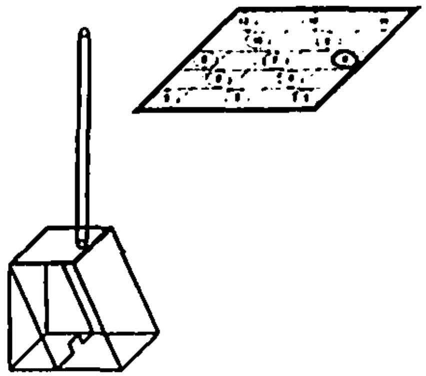
THE COARSE LAYERS COULD CAUSE DILUTION WHEN ADJACENT BLOCKS ARE MINED.

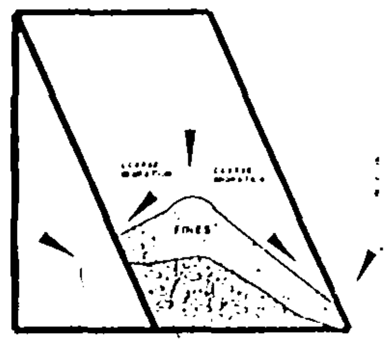
0- 15 PERCENT COARSE BY VOLUME WITHIN THE IMPACT CONE.

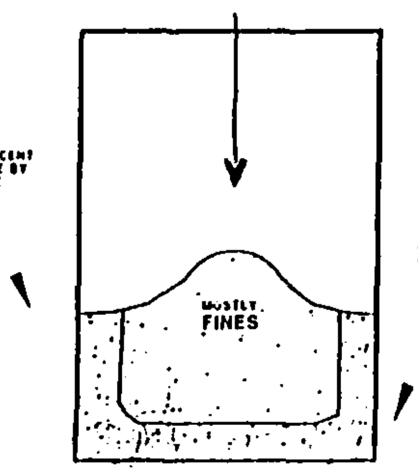
Figure 5.2.4

PHYSICAL MODEL TESTING

TRIAL #2 - HOLE 6 AT 90 DEGREES







FRONT VIEW

THE COARSE MATERIAL ROLLS DOWN THE IMPACT CONE TOWARDS THE WALLS OF THE MODEL.

THE IMPACT CONE CONTAINS MORE COARSE MATERIAL THAN BOTH TRIAL #1 & #4 METHODS SOME COARSE IS EMBEDDED WITHIN THE IMPACT CONE RATHER THAN BEING DEFLECTED AWAY TOWARDS THE PERIMETER OF THE MODEL.

ALTHOUGH MOST OF THE FINES REMAIN WITHIN THE IMPACT CONE, SOME MIGRATE TOWARDS THE WALLS OF THE MODEL.

THE AMOUNT OF COARSE MIGRATING TO THE MODEL WALLS MAY CAUSE DILUTION WHEN THE ADJACENT BLOCKS ARE MINED...
50 - 60 PERCENT COARSE IS TOO MUCH.

Figure 5.2.5: Extensive segregation in drawpoint, trial #2

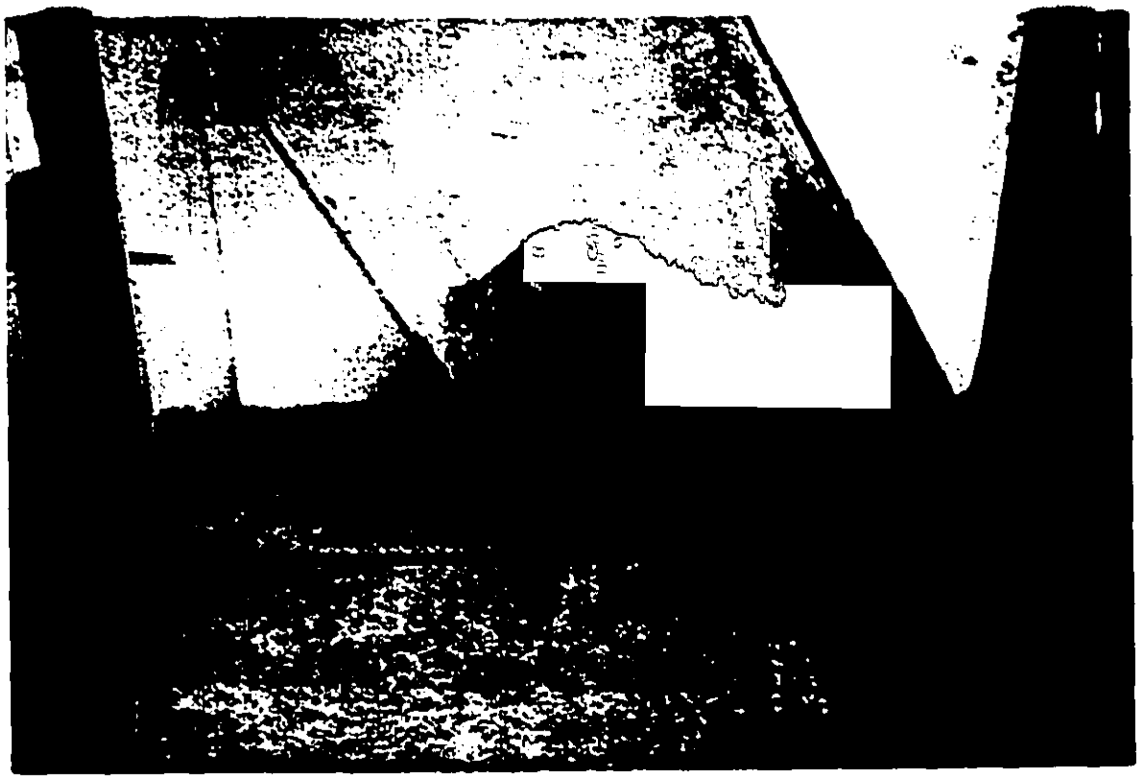
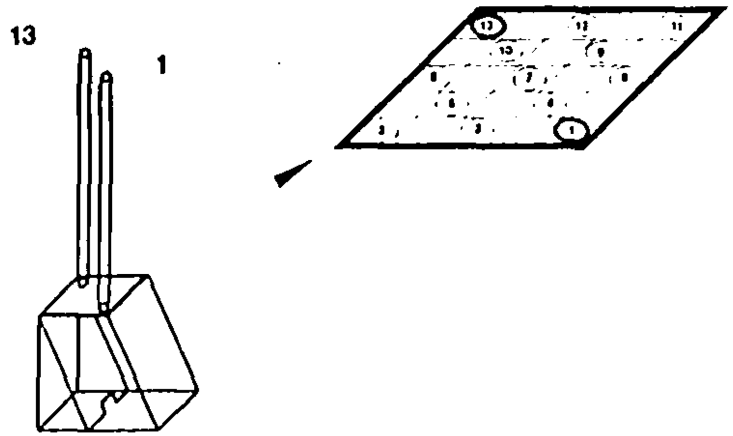


Figure 5.2.6: Segregated material on both footwall and hangingwall.

Figure 5.2.7

PHYSICAL MODEL TESTING

TRIAL #3 - HOLES 1 & 13 AT 90 DEGREES



HOLE #1 FLOW:

COARSE MIGRATES DOWN THE IMPACT CONE SURFACE TOWARDS THE END AND CENTER OF THE MODEL

FINES REMAIN IN THE IMPACT CONE, AND AT THE WALL OF THE MODEL.

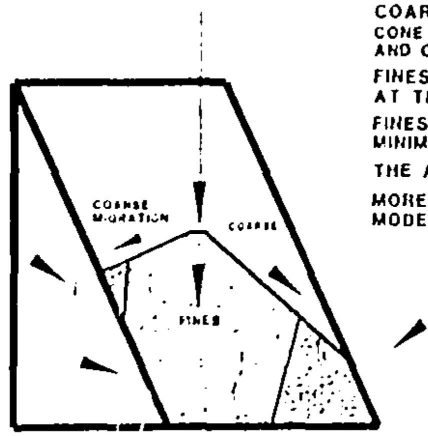
FINES ARE DESIRED AT THE WALLS TO MINIMIZE DILUTION

THE ANGLE OF REPOSE IS 42 DEGREES

MORE COARSE AT THE END OF THE MODEL THAN WITH TRIAL 4 METHOD

20 - 50 PERCENT COARSE MATERIAL BY VOLUME

0 - 5 PERCENT COARSE BY VOLUME



COARSE LAYERS 60 - 80 PERCENT COARSE MATERIAL BY VOLUME

HOLE #13 FLOW:

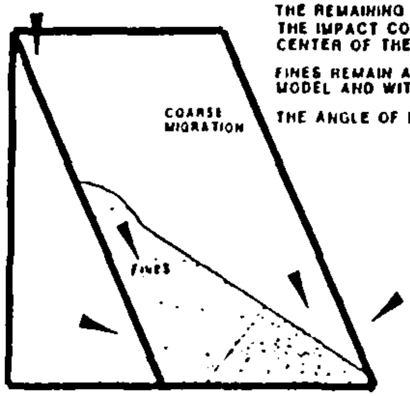
COARSE BOUNCES OFF THE FOOTWALL TOWARDS THE END OF THE MODEL.

THE REMAINING COARSE ROLLS DOWN THE IMPACT CONE, TOWARDS THE CENTER OF THE MODEL.

FINES REMAIN AT THE WALL OF THE MODEL AND WITHIN THE IMPACT CONE.

THE ANGLE OF REPOSE IS 33 DEGREES.

0 - 20 PERCENT COARSE BY VOLUME



COARSE LAYERS

80 - 90 PERCENT COARSE MATERIAL BY VOLUME

THE COARSE LAYERS WILL CAUSE DILUTION WHEN MINING ADJACENT ORE BLOCKS

HOLE #1 FLOW HOLE #13 FLOW

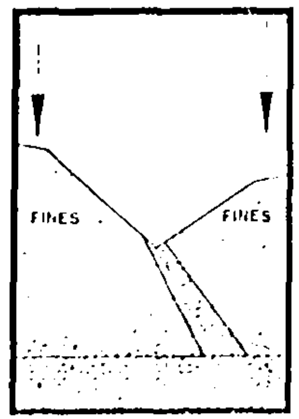
COARSE MATERIAL MIGRATES TOWARDS THE END OF THE MODEL AND TOWARDS THE CENTER OF THE MODEL.

COARSE ROLLS DOWN THE IMPACT CONES TOWARDS THE CENTER OF THE MODEL.

THE FINES REMAIN AT THE WALLS OF THE MODEL

THE COARSE BANDING AT THE WALLS IS NOT DESIRED, BUT THE COARSE MATERIAL TRAPPED AT THE CENTER OF THE MODEL IS DESIRED. COARSE LAYERS ARE AREAS OF LOW STRENGTH AND THEREFORE ARE NOT DESIRED NEXT TO THE WALLS OF THE ADJACENT BLOCKS. THE COARSE LAYERS WILL CAUSE DILUTION WHEN MINING ADJACENT BLOCKS

THIS FILLING METHOD REDUCES THE AMOUNT OF COARSE AT THE WALLS OF THE MODEL BUT DOES NOT REDUCE THE AMOUNT OF COARSE AT THE END OF THE MODEL.



FRONT VIEW

This method was investigated to determine the effects that an angled fill raise had on the segregation within the block. Due to the angle of the fill raise, the coarse material did not remain embedded within the impact cone but instead migrated towards the perimeter of the model. More fines accumulated at the footwall and a higher percentage of coarse material at the walls and toe of the model than the vertical raise method. The collision of the fill material with the impact cone caused a portion of the impact cone to be pushed towards the toe of the model, Figures 5.2.9 and 5.2.10.

Trial 5:

This method is similar to trial #3, except the fill raises are angled at 70 degrees. The intention of the trial was to place the material from raise #1 as close to the toe as possible, while fill from raise #13 was to be placed close to the the footwall. Both raises were situated close to the perimeter so that the fines would remain tight against the walls. The raises were positioned so that the fill would hit as high as possible against the stope walls. The interaction between the two raises reduced the potential of failure plains. The positioning of raise # 1 was ideal, placing the fill closer to the toe of the model producing a reduced amount of coarse at the toe than with trial #3, Figures 5.2.11, 5.2.12, and 5.2.13.

5.2.1: PHYSICAL MODEL TEST RESULTS

The test indicated that centrally located fill holes are sufficient as long as the length and width of the block is less than 30 meters. One should expect minimal dilution for smaller stope size if proper quality control measures are followed. If the dimensions are greater than 30 meters than the long rolling distance along the sides of the fill cone produce considerable segregation and reduced strengths at the perimeter of the stopes. When the length and/or width of the stope is greater than 30 meters additional fill raises will be needed, or the raise should be angled towards the stope walls so that the fine portion of the rockfill will remain at the wall where the higher

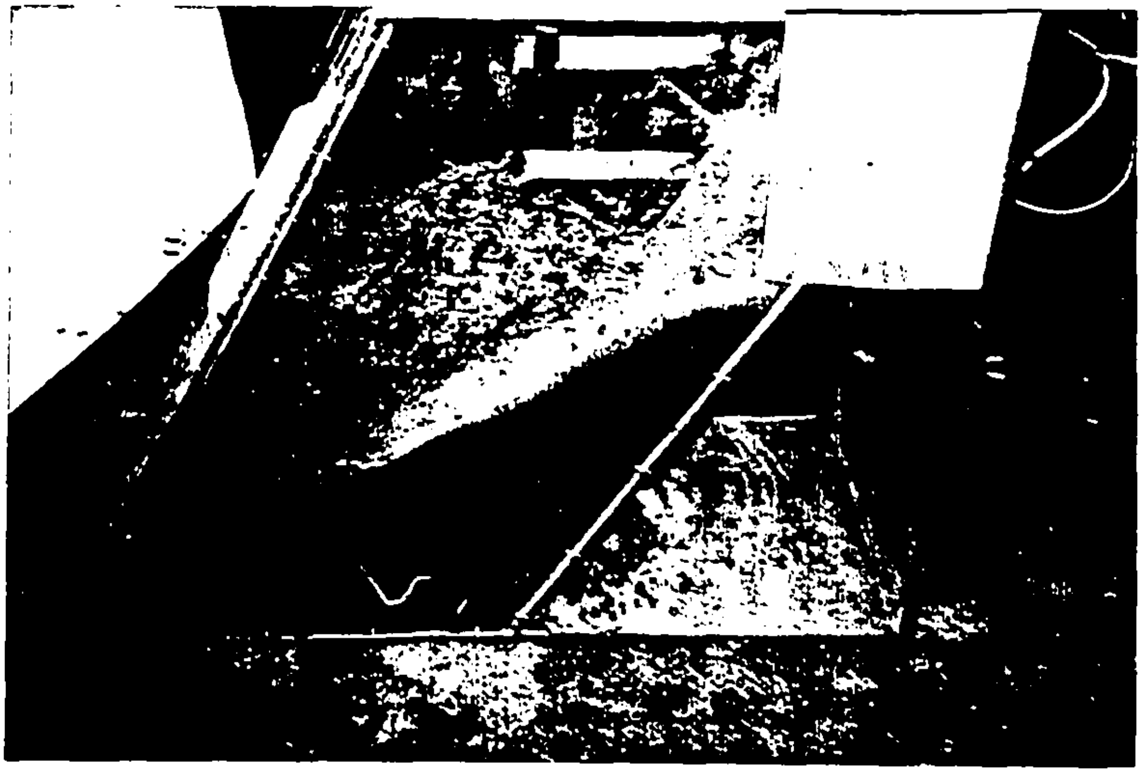


Figure 5.2.8: Fill raise #13 side in trial #3, footwall raise.

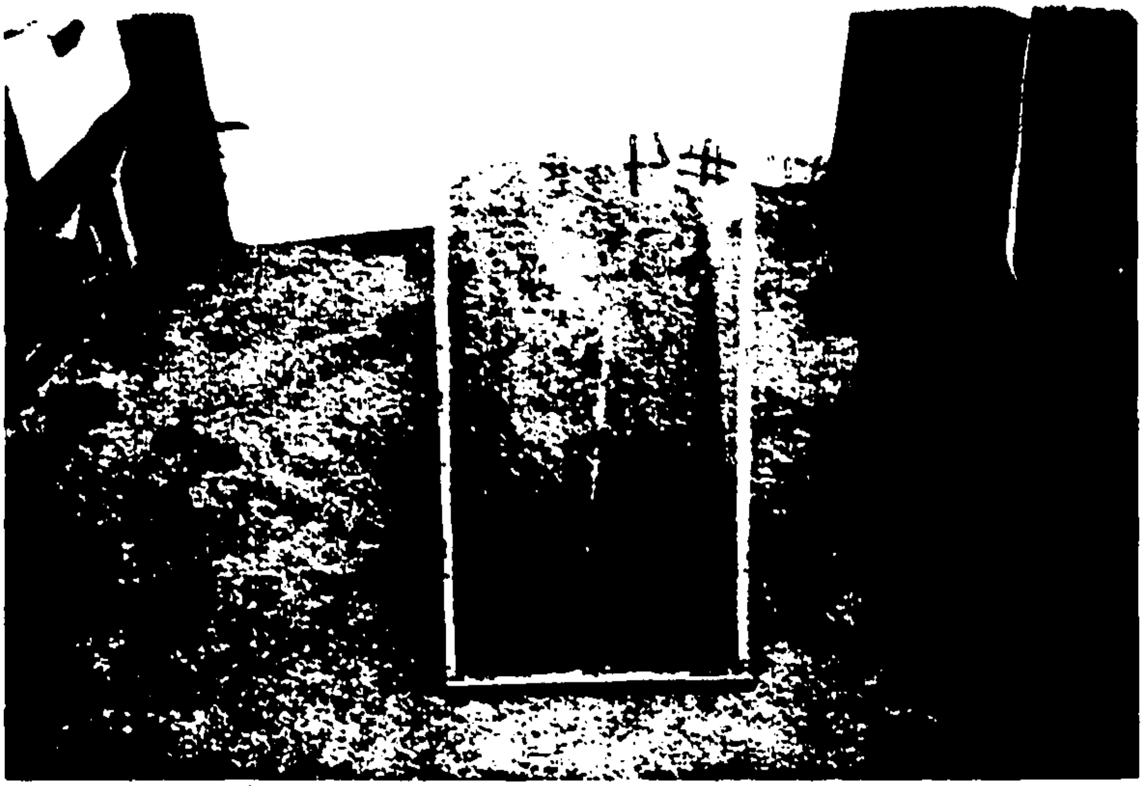
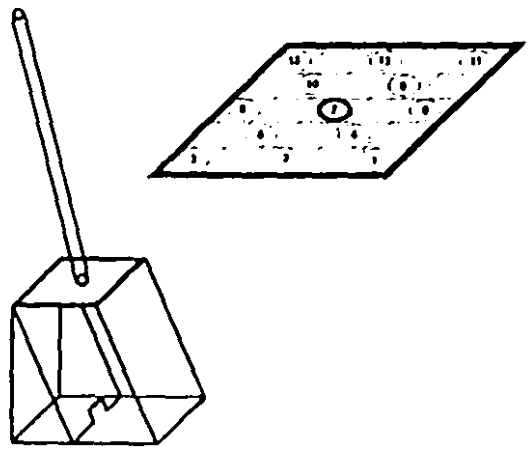


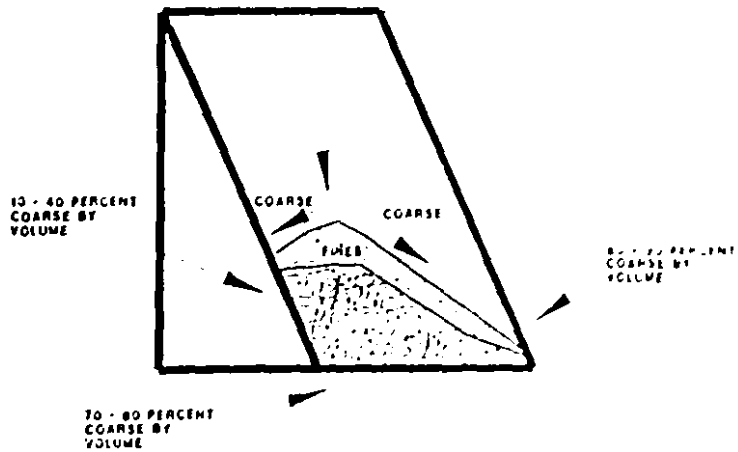
Figure 5.2.9: Segregated material in trial #4, front view.

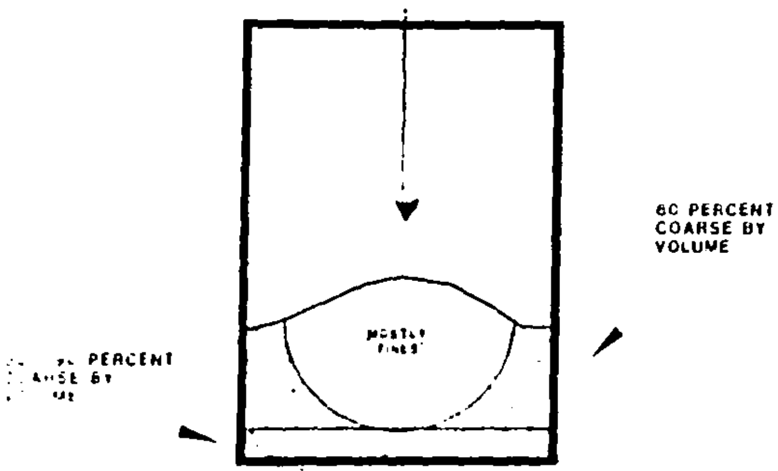
Figure 5.2.10

PHYSICAL MODEL TESTING

TRIAL #4 - HOLE 7 AT 70 DEGREES







FRONT VIEW

THE COARSE MATERIAL ROLLED AND FORMED A HORSESHOE SHAPE AROUND THE IMPACT CONE.

DUE TO THE ANGLE OF THE FILL RAISE, MORE FINES ACCUMULATED AT THE FOOTWALL OF THE MODEL THAN WITH A VERTICAL FILL RAISE.

DUE TO THE ANGLED FILL RAISE, THE COARSE MATERIAL DID NOT REMAIN IN THE IMPACT CONE BUT INSTEAD MIGRATDE TOWARDS THE PERIMETER OF THE MODEL.

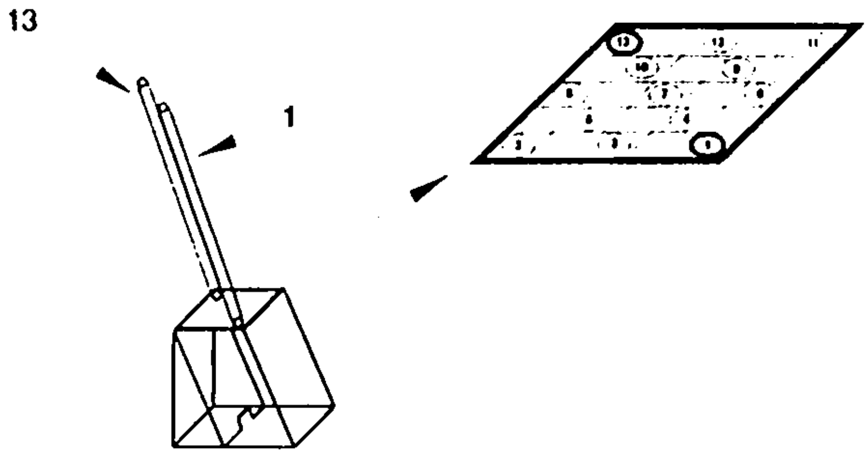
THIS METHOD PRODUCED A HIGH PERCENTAGE OF COARSE MATERIAL AT THE WALLS OF THE MODEL THE COARSE MATERIAL WOULD CAUSE DILUTION WHEN ADJACENT BLOCKS WERE MINED.

SOME OF THE FINES WITHIN THE IMPACT CONE WERE PUSHED TOWARDS THE MIDDLE-END OF THE MODEL...BUT THIS IS NOT ADVANTAGEOUS...THE FINES SHOULD BE AT THE WALLS OF THE MODEL.

Figure 5.2.11

PHYSICAL MODEL TESTING

TRIAL #5 - HOLES 1 & 13 AT 70 DEGREES



HOLE #1 FLOW:

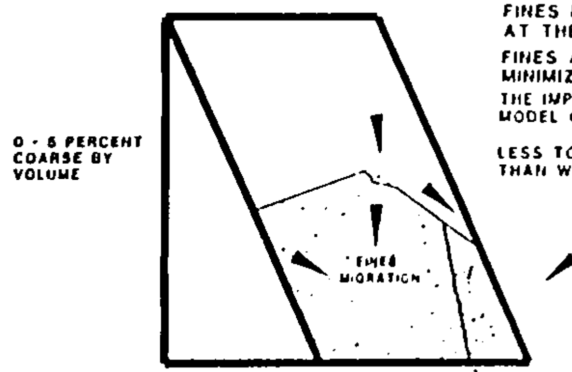
COARSE MIGRATES DOWN THE IMPACT CONE SURFACE TOWARDS THE END AND CENTER OF THE MODEL

FINES REMAIN IN THE IMPACT CONE AND AT THE WALL OF THE MODEL

FINES ARE DESIRED AT THE WALLS TO MINIMIZE DILUTION

THE IMPACT CONE AND FOOTWALL SIDE OF THE MODEL CONTAINED VERY LITTLE FINES

LESS TOTAL COARSE AT THE END OF THE MODEL THAN WITH THE VERTICAL RAISE METHOD

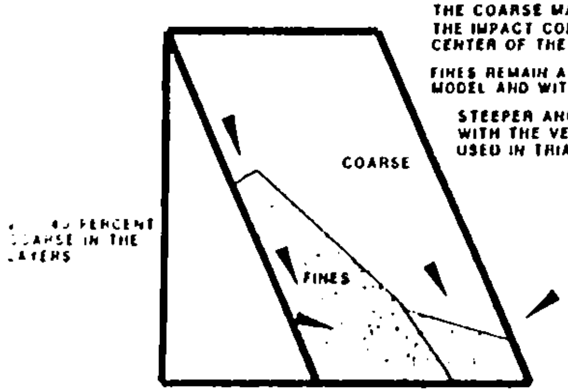


HOLE #13 FLOW:

THE COARSE MATERIAL ROLLS DOWN THE IMPACT CONE, TOWARDS THE CENTER OF THE MODEL.

FINES REMAIN AT THE WALL OF THE MODEL AND WITHIN THE IMPACT CONE.

STEEPER ANGLE OF REPOSE THAN WITH THE VERTICAL RAISE METHOD USED IN TRIAL 3.



THE COARSE LAYERS WILL CAUSE DILUTION WHEN MINING ADJACENT ORE BLOCKS

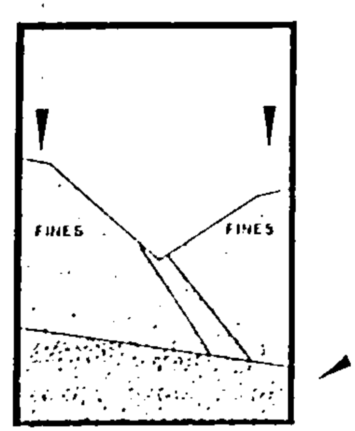
HOLE #1 FLOW HOLE #13 FLOW

COARSE MATERIAL MIGRATES TOWARDS THE END OF THE MODEL AND TOWARDS THE CENTER OF THE MODEL.

COARSE ROLLS DOWN THE IMPACT CONES TOWARDS THE CENTER OF THE MODEL.

THE FINES REMAIN AT THE WALLS OF THE MODEL.

THE COARSE BANDING AT THE WALLS IS NOT DESIRED, BUT THE COARSE MATERIAL TRAPPED AT THE CENTER OF THE MODEL IS DESIRED. COARSE LAYERS ARE AREAS OF LOW STRENGTH AND THEREFORE ARE NOT DESIRED NEXT TO THE WALLS OF THE ADJACENT BLOCKS. THE COARSE LAYERS WILL CAUSE DILUTION WHEN MINING ADJACENT BLOCKS



FRONT VIEW

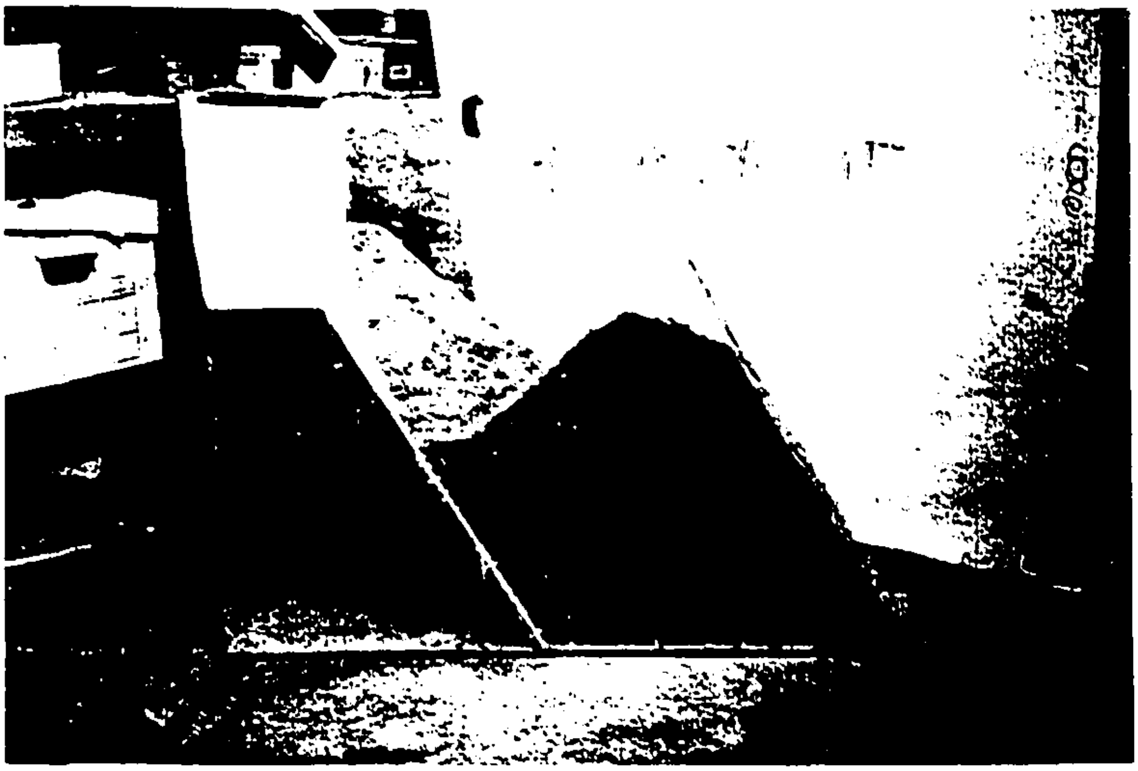


Figure 5.2.12: Fill raise #1 side in trial #5, 70 degree raise.

Figure 5.2.13: Fill raise #13 side in trial #5, 70 degree raise.

strengths is required. The fill raise should be angled so that the rockfill hits as high as possible against the stope walls for optimum results.

The coarse portion of the rockfill naturally migrates towards the perimeter of the impact cone, while the finer materials within the impact cone. Therefore if a fill raise is positioned so that the impact cone is situated directly against a stope wall, the coarse material will migrate towards the center of the stope where minimum strength is required, while the fine material remains at the stope walls producing high strength material which will be exposed in future mining.

The highest degree of segregation occurred when the fill raise was positioned so that the fill material hit the footwall. In this situation the majority of the coarse material rebounded towards the end of the stope, producing very fine, competent fill at the footwall and very coarse non-competent fill at the hanging wall side of the model.

The most homogeneous fill was produced when there was a centrally located fill raise. A portion of the the coarse material became embedded within the impact cone and therefore did not migrate towards the stope walls.

5.3: FILL METHODS

Considering the in situ information obtained from chapter 4 and the results of the physical modelling, following conditions could exist:

SET-UP 1: This is the ideal filling system with trucks if two accesses are available. This set-up will result in having the strongest fill mass at the stope boundaries which will be exposed in future mining, Figures 5.3.1, 4.5.8, and 4.5.9. All the coarse aggregate will end up in the middle of the stope and will cause no dilution problem.

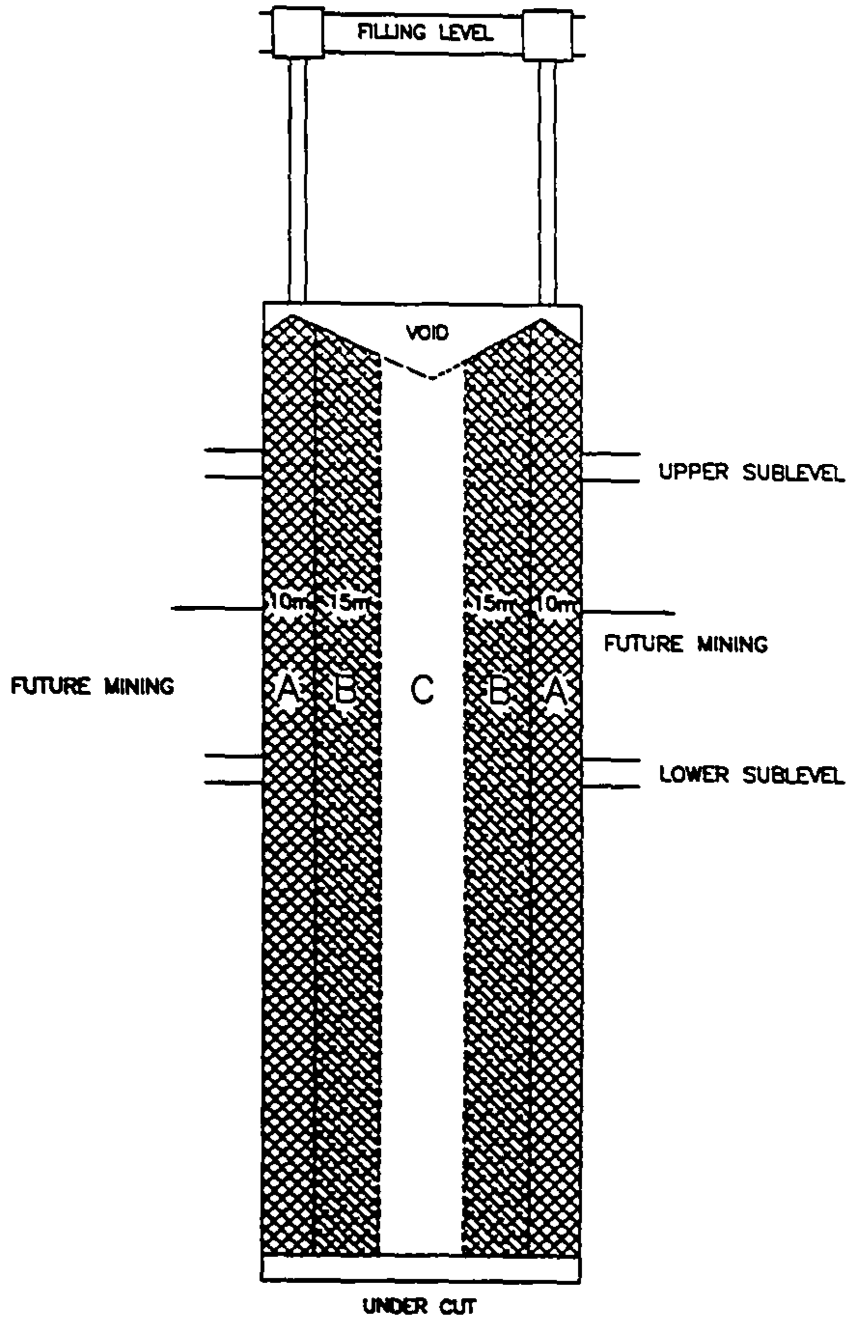


Fig: 5.3.1 Ideal Fill Set-Up With Trucks

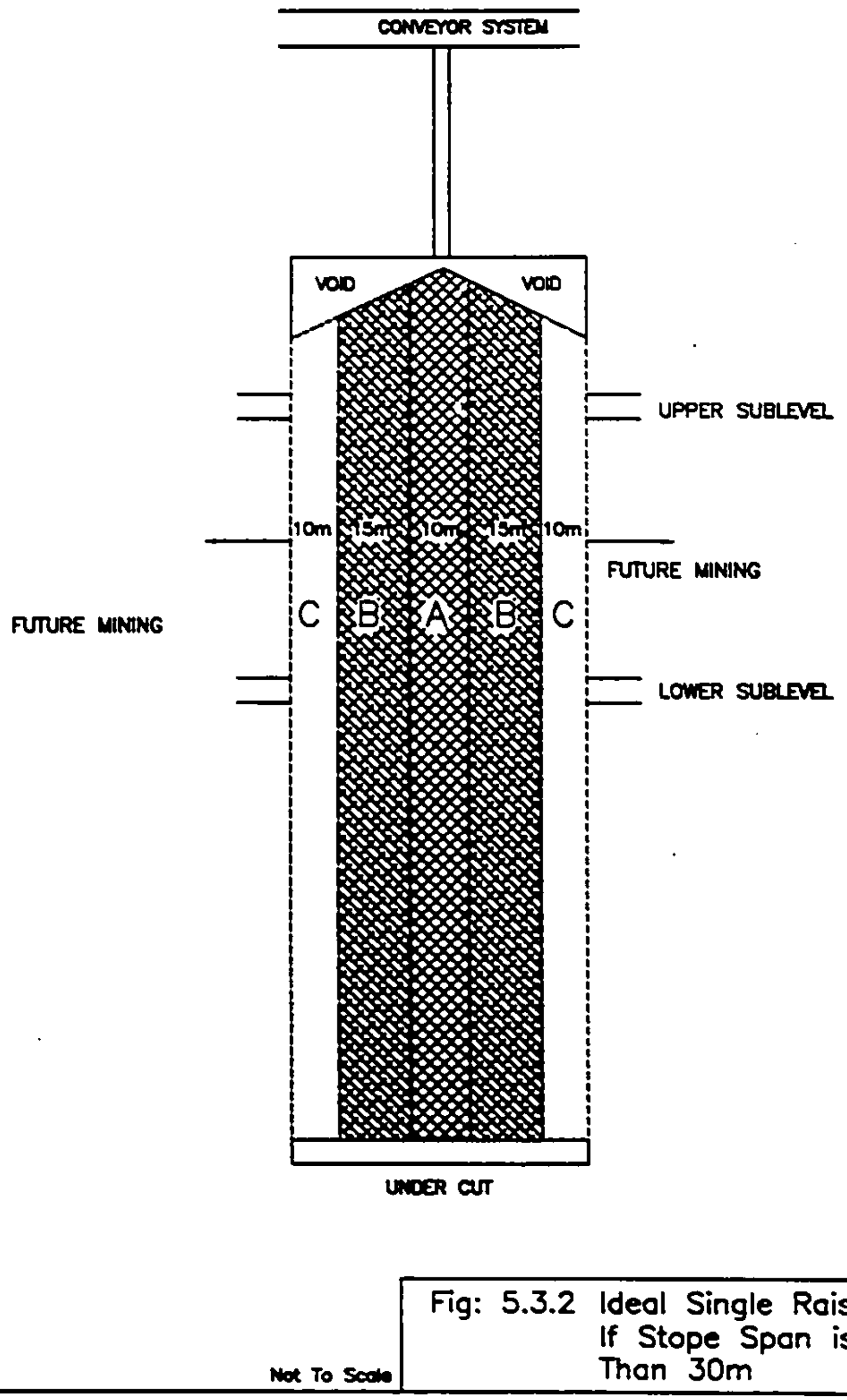


Fig: 5.3.2 Ideal Single Raise Set-Up If Stope Span is Less Than 30m

SET-UP 2: The single raise in this set-up could cause extreme segregation at the walls depending on the stope dimensions. Fill failure and dilution should be expected if the width or the length of the stope is grater than 30 meters, Figures 5.3.2, 4.5.8, and 4.5.9.

SET-UP 3: Ideal vertical fill/slot raise location if only one wall will be exposed in future mining, Figure 5.3.3.

SET-UP 4: This is the most frequent fill raise set-up used at Kidd Creek Mines, Figure 5.3.4. Backfill placement alternates between the two available raises on shift by shift basis, or approximately after every 3000 tonnes of placed fill. This practice results in a more uniform distribution of cementitious material and reduces coarse aggregate roll distance, hence creating a more horizontal fill profile. This will result in accumulation of all the coarser particles in the middle of the stope and away from the walls that will be exposed in future mining, Figures 5.3.5 and 5.3.6.

SET-UP 5: This set-up utilizes existing slot/fill raise and an inclined raise to ensure high fill quality at the desired wall, Figure 5.3.7.

The most critical portion of a backfilled stope is the wall (s) which will be exposed for future pillar recovery. The fill raises(s) should be placed such that the fill will collide as high as possible on the wall(s) which are required to stand. This will utilize the phenomena that the wall below the impact zone is the strongest part of the filled stope. This indicates the importance of having multiple raises if the filled stope is going to be exposed at more than one wall. By having multi raise system, the fill could have a more uniform distribution and a more horizontal, flat, profile. This would eliminate the existence of a rill surface which is a plane of weakness.

In the case where the fill material does not collide with the stope walls, fine backfill aggregates accumulate at the fill cone and have a high cement content, producing a high quality

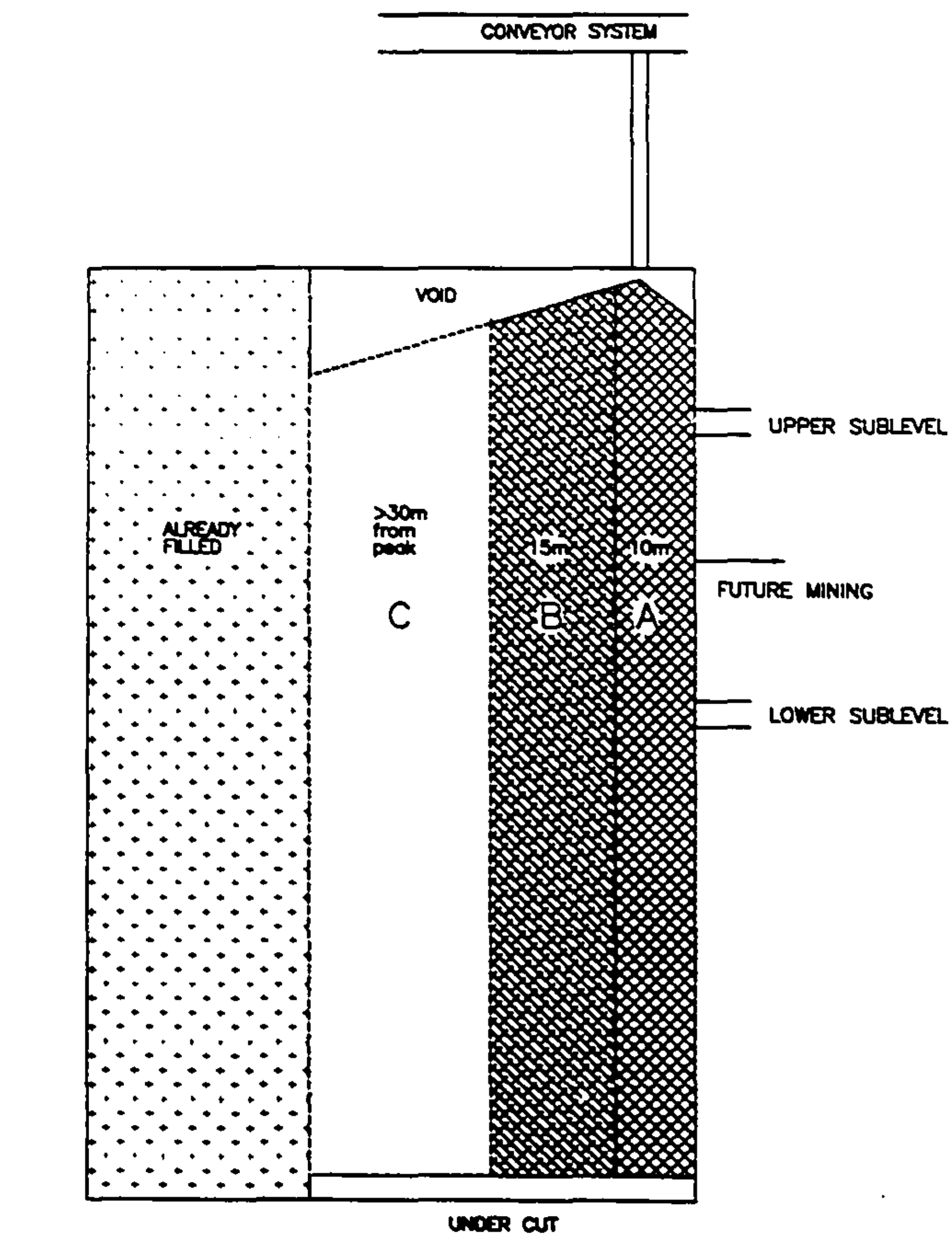


Fig: 5.3.3 Ideal Single Raise Set-Up If Only One Wall To Be Exposed In Future Mining

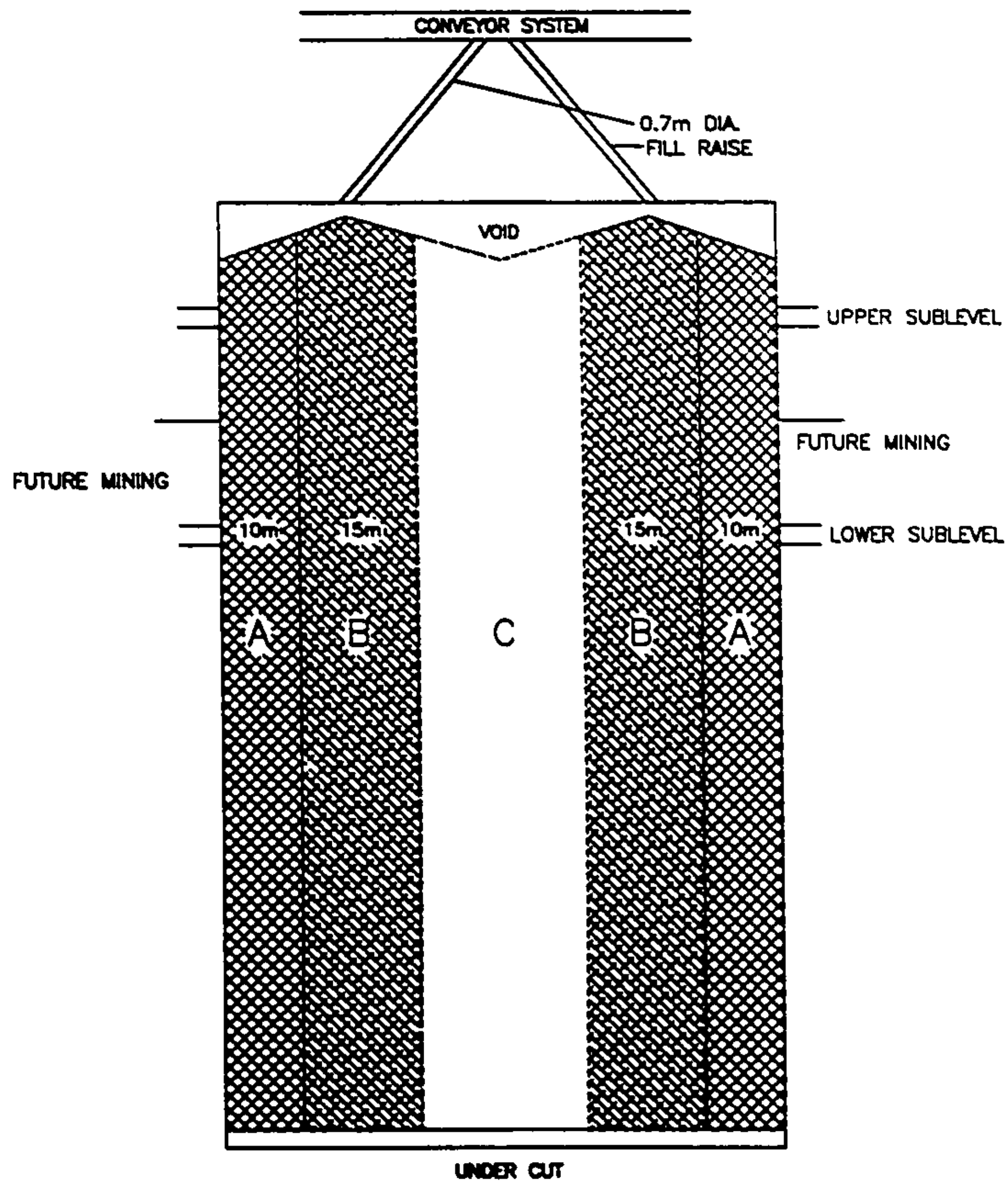


Fig: 5.3.4 Ideal Fill Raise Set-Up When At Least Two Walls Of The Stope Will Be Exposed In Future Mining

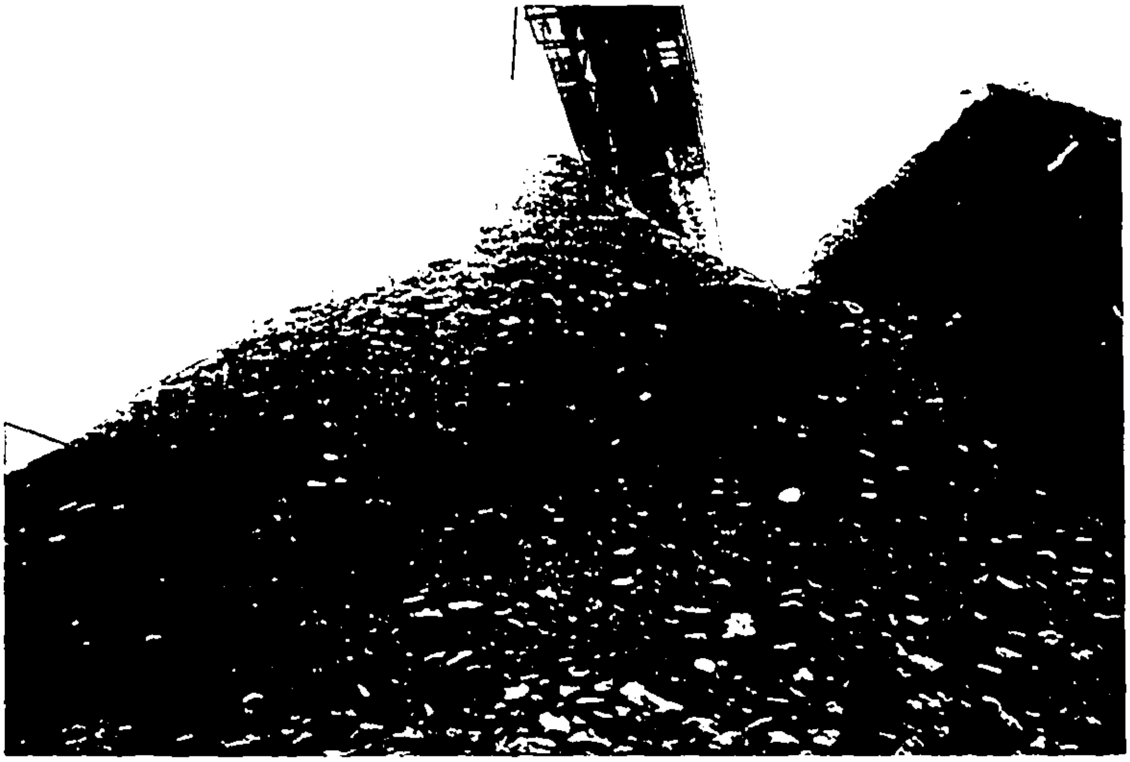


Figure 5.3.5: Multi raise system to direct coarse aggregate to center of stope.

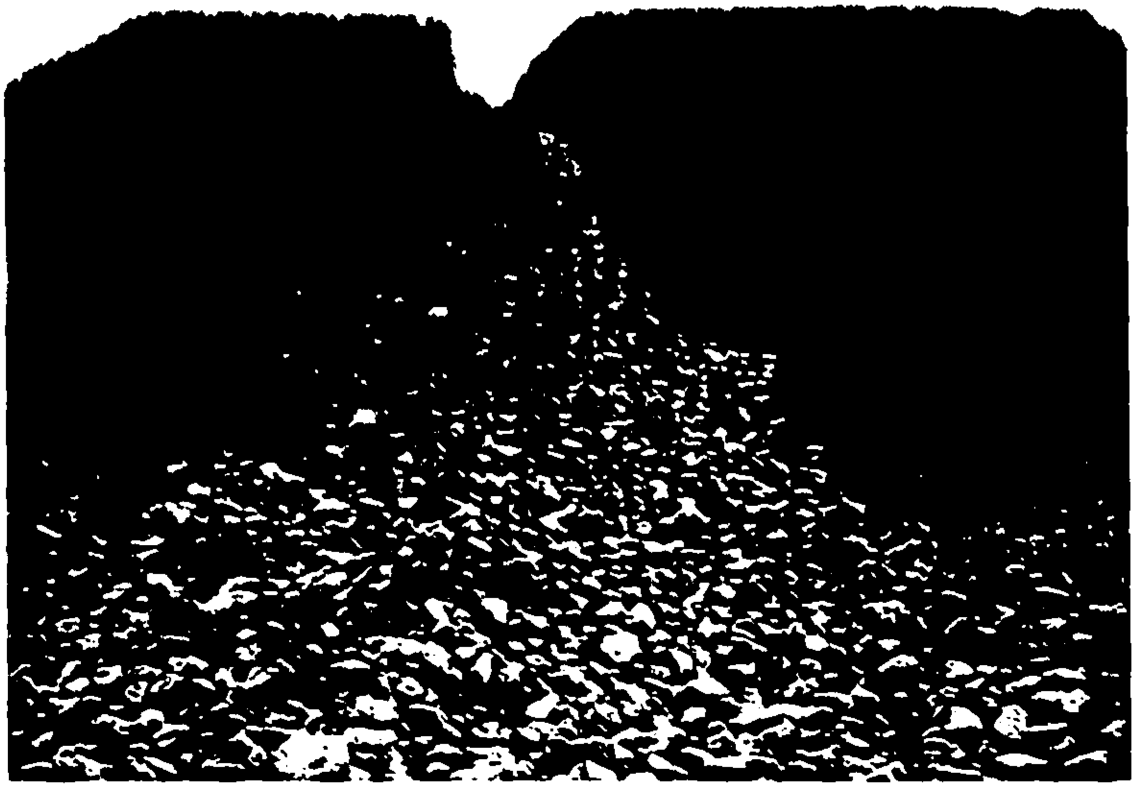


Figure 5.3.6: Coarse aggregate in between to dump cones in surface stockpile.

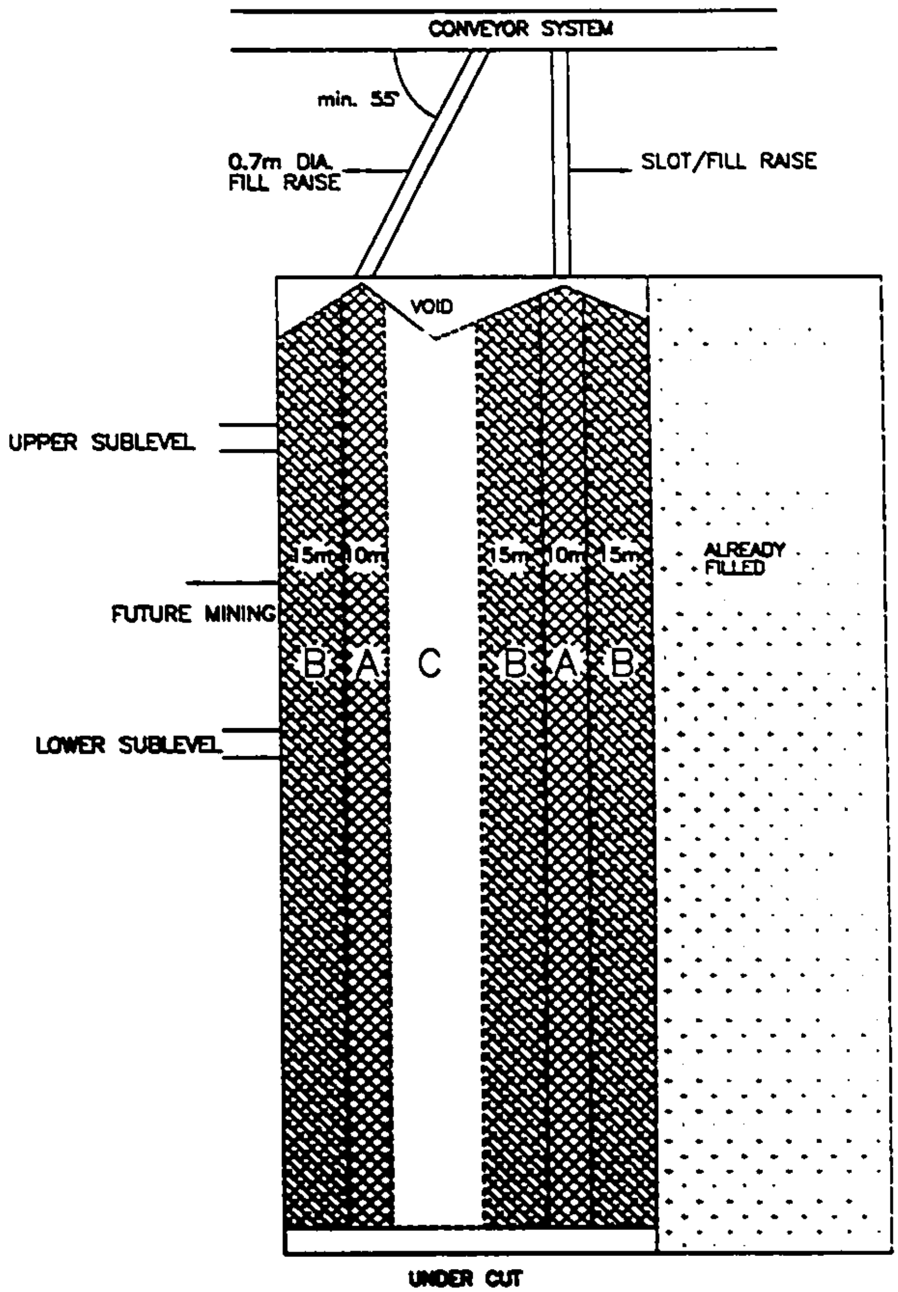


Fig: 5.3.7 Typical Multi–Raise Set–Up At KCM Using Existing Slot/Fill Raise fill, point A on Fig. 5.1.1, case 2. Because of the higher momentum of the coarser particles, they segregate towards the boundary of the stope resulting in a low cement content, poor quality backfill (point B, Fig.5.1.1). Observations underground show that coarser aggregate tends to gravitate towards stope perimeter and the finer material remains around the impact area. Since much larger strengths are obtained under the impact point than away from it down the rill, all dump points should be positioned around the perimeter of the stope scheduled for filling. The fill quality and segregation extent in boundaries of each stope are related to the rolling distance that the coarse particles have to travel before colliding with the walls. Lack of cement and fine fractions at the boundaries will have an adverse effect on the fill quality. Some portion of the coarse particles could, however, bury themselves in the impact zone due to the high impact velocity, especially when using high speed conveyor belts for placement. If the orientation of the fill raise (s) does not allow the fill material to collide with the stope wall, the dip of the fill raise(s) should position the cone area as close as possible to stope walls that are to be exposed in future pillar recovery, refer to Figures 5.4.1 to 5.4.4. This will translate in having higher cement content, higher fine fractions and less segregated material close to the perimeter of the stope .

As noted earlier, if all stope walls are to be exposed for future pillar recoveries, then multiple vertical raises yield the best possible distribution of the fill materials. The vertical raises would produce a more horizontal fill profile and any excess cement slurry will accumulate at the stope boundaries. If vertical raises are not possible due to the available access, then the number of inclined fill raises should be increased to produce a competent fill at the required stope walls.

To simulate the actual filling operation, several large scale dump tests were carried out at Kidd Creek Mines. The results revealed an adequate strength of over 6 MPa, with a peak strength of 10.3 MPa in specimens cored from 838-Stope. However, it was found from chemical analysis that the cores contained a high cement content, 7.3% as compared to the average of 5%. This indicated that the cement concentrations could occur in some areas of large fill piles, resulting in undesirable deficiencies of cement in other areas.

Results of the tests indicated that CRF caused a significant segregation of coarse aggregate in the outer zone of the fill pile. As it was explained before, the segregation would also occur in the area adjacent to stope walls even within a small rolling distance from the dump points. Such common phenomena caused a weak zone in the underground fill prone to blasting damage during pillar extraction. To minimize the effect of segregation without increasing the number of fill dump points, tests were conducted by adding sand into the CRF to act as a void filler. Consolidated sandfill, CSF, is essentially a cemented hydraulic fill using alluvial sand in place of normal tailings. The sand has 90- 95 % by-weight passing a No. 18 mesh (1.0 mm) screen, and has an average percolation rate of 15 cm/hr. This sand, when mixed with cement slurry, becomes an ideal material to fill the voids in segregated aggregate or waste.

On occasion, the angle of repose of the fill during placement has to be adjusted so that the resulting slope will allow ore to run as an adjacent pillar is blasted and mucked. Repose angle can also be adjusted so that tight filling is achieved in enclosed areas. The typical angle of repose for aggregate is 45 deg. and for CRF is 37 deg., which varies with the pulp density and cement content of the slurry introduced. For consolidated sand rockfill the angle of repose is between 18 to 25 deg., and for cemented sand fill and /or sandfill alone it is 10 degrees . This indicates that the main concern when using a fill raise should not be the final profile but the effect on the backfill quality after being placed with that specific fill orientation. The final fill profile can be altered as explained above, but dilution caused by the backfill may not be remedied.

The stope geometry plays an important role in decreasing segregation effect. The shape of the stope and the orientation of the fill raises dictate the rolling distance of the material after the impact with the fill cone. If the distance is kept small, such as in a small square shape stope with centrally located fill raise, then the coarse particles have a chance to rebound back to the vicinity of the impact area after colliding with the stope wall. This could result in a more uniform fill distribution and a more horizontal fill face in the stope. Should one or two walls are required to stand, the fill raise (s) should be placed such that the fill will collide as high as possible on the wall(s) of which is required to stand. This will utilize the phenomena that the wall below the impact zone is the strongest wall.

If the rolling distance is increased due to the enlarged stope geometry or a rectangular shape stope, then the material has no chance of travelling the same distance after colliding with the wall. This would lead to the formation of low density and weak materials at the wall of the stope which will be exposed at future pillar recovery. To overcome this effect, a multiple fill raise system is recommended to produce a competent fill foi larger stopes. Whenever possible, when four stope walls are required to stand, the backfill raises should be vertical and centrally located spaced at a distance of no more than the stope width. If the vertical fill raises are not possible, the number of the fill raises should be doubled, to ensure that the advantage of both case 1 and case 2, which were explained earlier, are utilized.

If possible the fill free-fall height should be kept as small as possible, by having pour points at different levels. This would minimize the amount of energy of the aggregates at the time of colliding with the fill cone which causes further breakage of the material. This energy would cause an increase in the already high fine fraction at the peak area and possible formation of impermeable layers which would lower the percolation rate of the slurry through the material. The fines also coat the coarser particles preventing proper bounding between them.

5.4 : EXAMPLE AT KCM

The following is an example of a specific stope filling proposal showing different mix design at different phases of the fill progress.

SUBJECT: 1828-P-ST FILL PROPOSAL

BACKGROUND

Mining of the 1828-P-ST has been completed and the stope plus the adjacent 2028-O-PIL above 1800 level are to be backfilled with consolidated rockfill. The 2028-O-PIL below 1800 level has been backfilled with consolidated rockfill. All fines in the aggregate are to be used, therefore sand will not be used as a void filler. Any loose muck left in the stope will be consolidated. Two raises are available for backfilling this stope, however, good quality (well slurry coated) backfill cannot be placed down both raises simultaneously.

PROPOSAL

Fill the 1828-P-ST by conveyor down a raise from 16227 Backfill Acc., DP #5 with consolidated rockfill in four phases, see Fig. 5.4.1. A cement batch mix of 40% Portland cement / 60% flyash is recommended for phases 1, 3 and 4. A cement batch mix of 60% Portland cement / 40% flyash is recommended for phase 2.

Overall Fill Tonnage : 150,000 tonnes aggregate
Overall Cement (Flyash) Content of CRF: 4.9%

PHASE ONE:

Fill the 1828-P-ST by conveyor down the 28-inch diameter raise from the 16227 Backfill Acc. until estimated tonnage is reached, see Figures 5.4.2, 5.4.3 and 5.4.4. Extra slurry in Phase One will consolidate the rockfill and muck left at the base of the stope, and will provide a good bond with the underlying backfill. Mixing water from the surface recycled water ponds is recommended.

Tonnage Estimate : 15,000 Tonnes Aggregate
Cement Content : 5.5 %

Run 180 tonnes of aggregate per batch of slurry.
(4.0 tonnes (8,800 lb) of Portland cement & 6.0 tonnes (13,200 lb) of flyash)
17,300 lb (2,075 USG) water per batch
Pulp Density of slurry = 56%

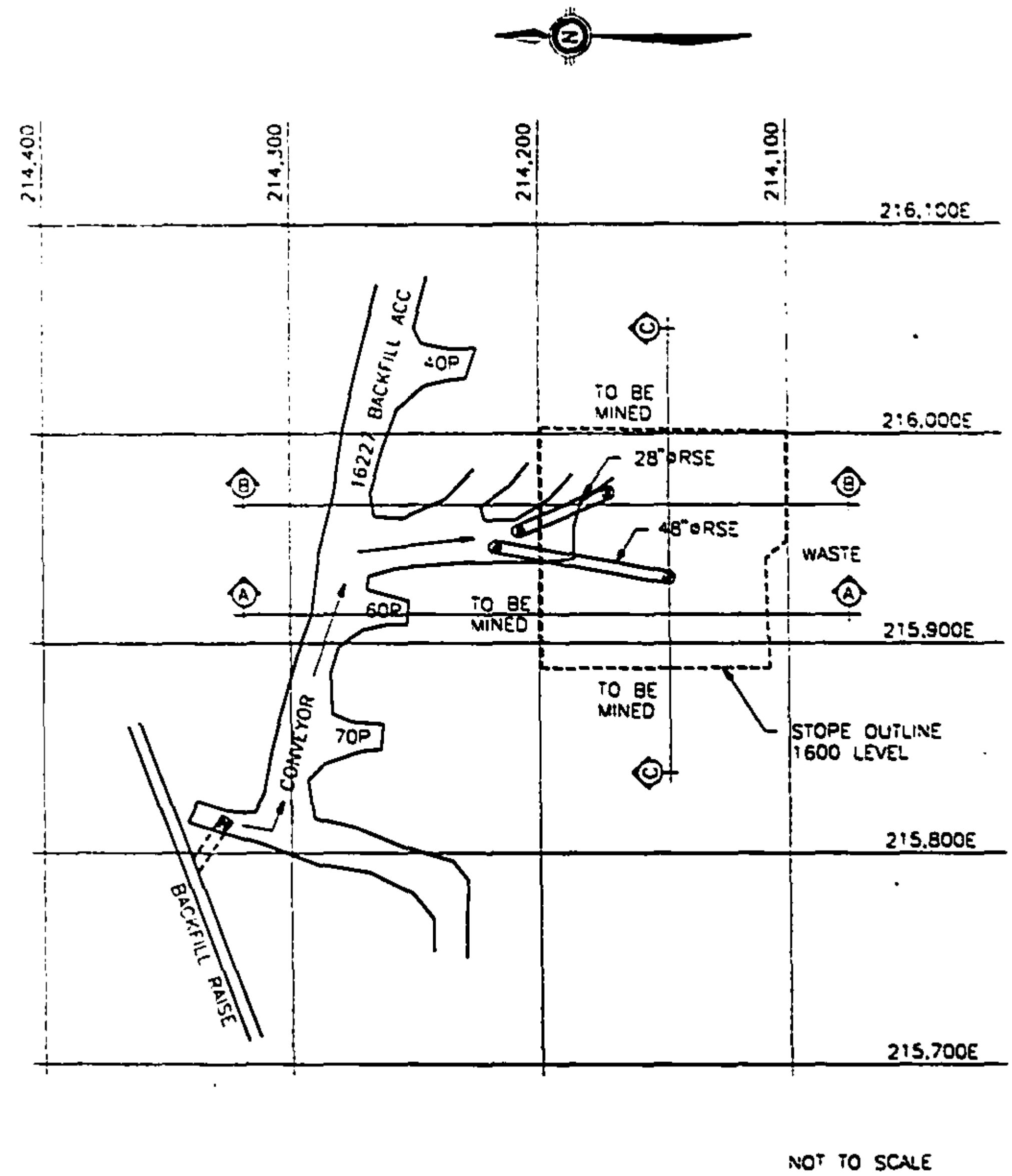


Fig: 5.4.1 Fill Plan 16–2 Sublevel
1828–P–ST/2028–D–PIL

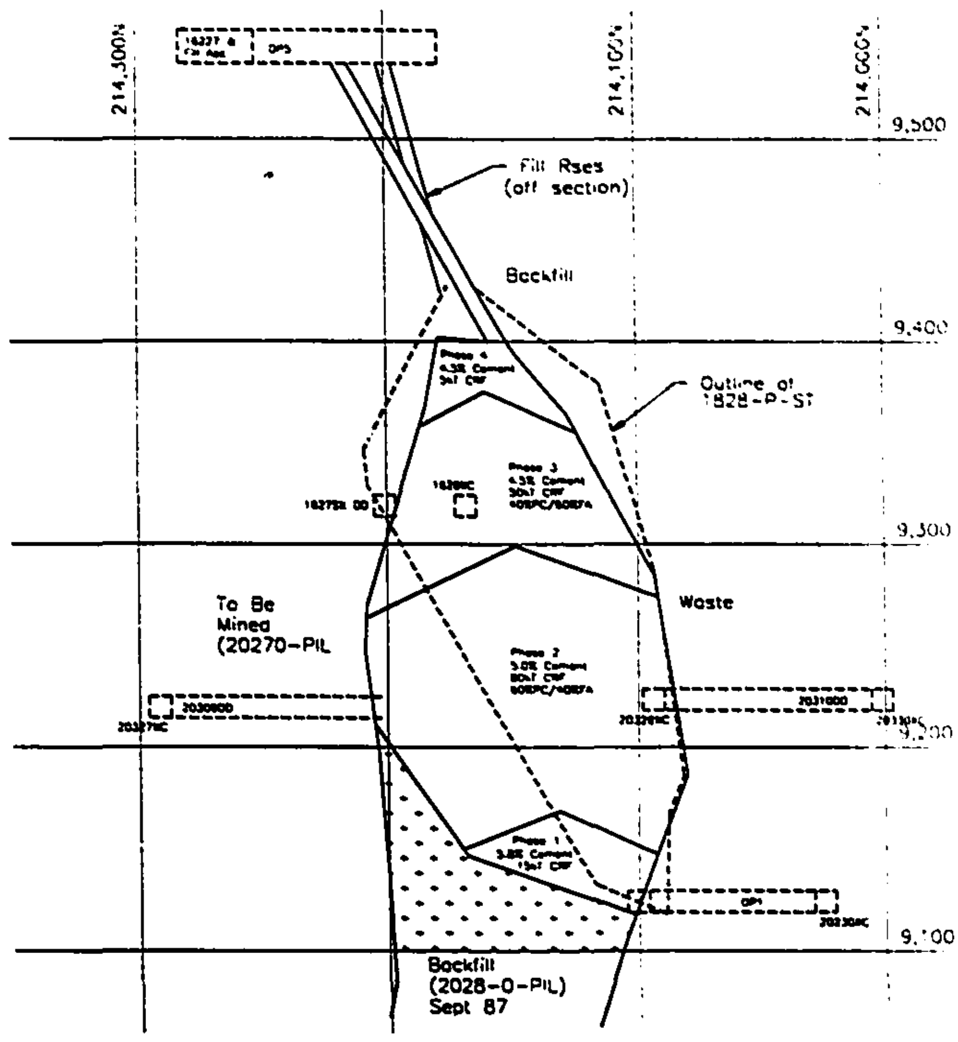


Fig: 5.4.2 Section A-A
Through 215.910E Looking
East 2028-0-PIL

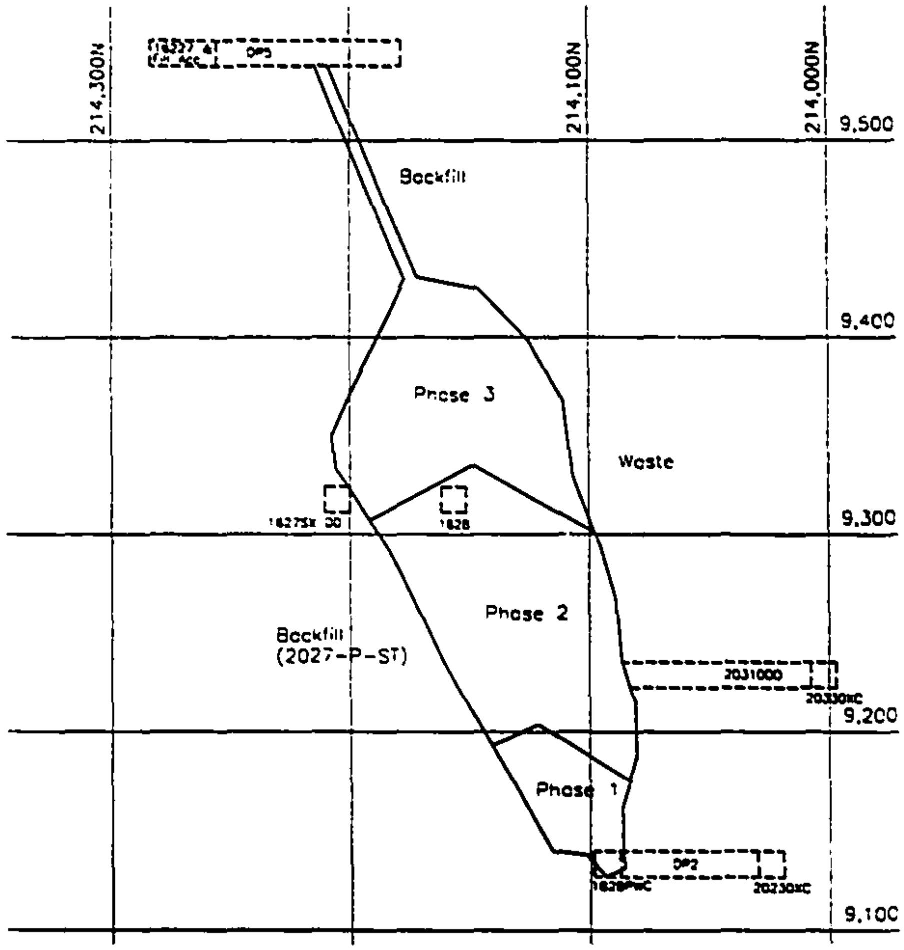


Fig: 5.4.3 Section B-B Through
215.965E 1828-P-ST
Looking East

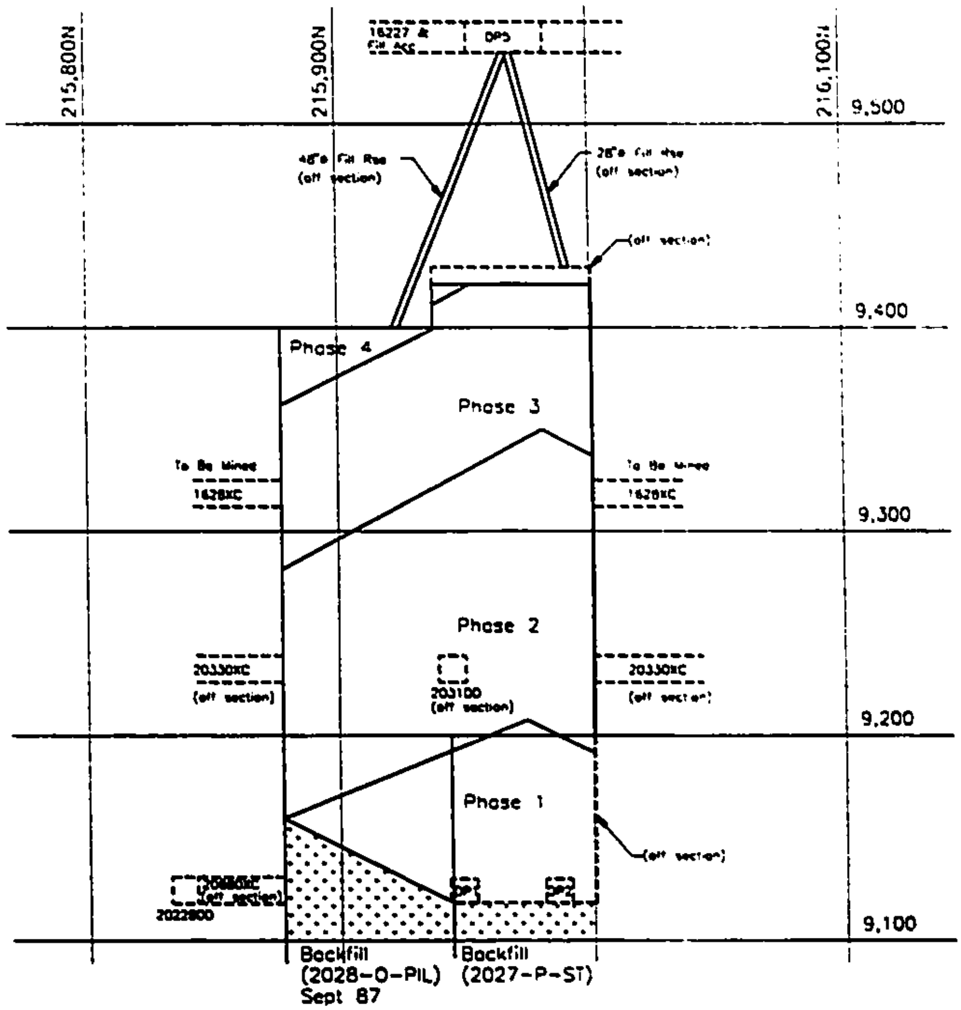


Fig: 5.4.4 Section C-C Looking North Through 214.150N
2028-0-PIL 1828-P-ST

Volume of Batch = 399 cubic feet

PHASE TWO:

Continue to fill the 1828-P-ST by conveyor down the 28-inch diameter raise from 16227 Backfill Acc. until the peak of the backfill reaches the 1600 level. The toe of the fill will be approximately 50 feet below 1627 SX DD footwall. Mixing water from the surface recycled water ponds is recommended. At the end of every shift, with the conveyor stopped, run an extra batch of slurry into the stope. The extra batch should wash slurry and fines down to the base of the fill cone in order to compensate for the long roll of aggregate. Since the 2027-O-PIL adjacent to the 2028-O-PIL is scheduled for mining in two months, a cement batch mix of 60% Portland cement / 40% flyash will be used to provide a faster rate of curing.

Tonnage Estimate : 80,000 Tonnes Aggregate
Cement Content : 5.0%

Run 180 tonnes of aggregate per batch of slurry.
(5.4 tonnes (11,880 lb) of Portland cement & 3.6 tonnes(7,920 lb) of flyash)
16,800 lb (2,015 USG) water per batch
Pulp Density of slurry = 54%
Volume of Batch = 378 cubic feet

PHASE THREE:

Continue to fill the 1828-P-ST by conveyor down the 28-inch diameter raise from 16227 Backfill Acc. until the backfill reaches the base of the raise. Mixing water from the surface recycled water ponds is recommended. At the end of every shift, with the conveyor stopped, run an extra batch of slurry into the stope. The extra batch should wash slurry and fines down to the base of the fill cone in order to compensate for the long roll of aggregate.

Tonnage Estimate : 50,000 Tonnes Aggregate

Cement Content : 4.5%

Run 200 tonnes of aggregate per batch of slurry.

(3.6 tonnes (7,920 lb) of Portland cement & 5.4 tonnes (11,880 lb) of flyash) 17,600 lb (2,111 USG) water per batch

Pulp Density of slurry = 53%

Volume of Batch = 391 cubic feet

PHASE FOUR:

Tightfill the 1828-P-ST by conveyor down the 48-inch diameter raise from 16227 Backfill Acc. until the backfill reaches the base of the raise. Extra batch into the stope or additional fine aggregate may be required to flatten the angle of the fill in the late stages of phase four. Mixing water from the surface recycled water ponds is recommended.

Tonnage Estimate : 5,000 Tonnes Aggregate

Cement Content : 4.5%

Run 200 tonnes of aggregate per batch of slurry.

(3.6 tonnes (7,920 lb) of Portland cement & 5.4 tonnes (11,880 lb) of flyash) 17,600 lb (2,111 USG) water per batch

Pulp Density of slurry = 53%

Volume of Batch = 391 cubic feet

5.5: SUMMARY

The in situ observations and model testing verified that segregation can not be eliminated.

however, it could be partially controlled if the fill raises are positioned properly. From the above information ideal fill methods for different fill conditions were established to allow the control of segregation. The most critical portion of a backfilled stope is the wall(s) which will be exposed in future pillar recovery. The fill raise(s) should be placed such that the fill will collide as high as possible on the wall(s) which are required to stand. The proper structural design will direct the coarse aggregate to the center of the stope which will never be exposed in future mining. Also, the segregated material could be designed to be at the walls against previously filled stopes or waste contacts.

6.0: BINDER ALTERNATIVES

Physical and mechanical strengths of a typical fill mass dictate the fill performance for specific requirements. As shown in the literature survey almost 43% of the total fill cost is the cost of the binder materials used. Obviously with higher binder contents, and much higher operating costs, operations could obtain higher strength fill mass, lower dilution and fewer operational delays. From the extensive site investigation in chapter 4, it was concluded that the lower cost binder, flyash , performed extremely well. This translated to significant financial saving at KCM without any strength loss and in most case, if properly used, could achieve a higher long-term strength. A major part of this thesis is to evaluate and implement new fill recipes at KCM. The new recipes could easily be used by other mines, especially in other rockfill operations. Of course, unlimited combination of the lower cost binder materials could be evaluated, however, with author's experience and cost considerations, the mixes with the most financial and strength improvement potentials are evaluated. Total of 1750 test specimens are used for this extensive laboratory work.

6.1: BINDERS AVAILABLE

Considerable savings can be achieved by minimizing the percentage of Portland cement used in any backfill mix by partially replacing it with other cheaper binder materials. A comprehensive testing program was conducted to evaluate the cementitious properties of different binders with or without addition of commercially available chemicals. The binders evaluated in this laboratory test program were:

6.1.1: PORTLAND CEMENT

Portland Cement is a hydraulic cement produced by pulverizing clinker consisting of hydraulic calcium silicates and containing one or more forms of calcium sulphate as an interground addition. In presence of water, calcium silicate hydrate (CSH) compounds form the gel that gives cement its hardening characteristics.

6.1.2: FLYASH

Flyash is a by-product from the combustion of pulverized coal in thermal power plants, and is removed by mechanical collectors or electrostatic precipitators as a fine particulate residue from the combustion gases. Commercial flyash can be classified into two types, Type F and Type C. Type F flyash, produced from bituminous coals, has a low lime content and possesses little cementitious value by itself; however in presence of Portland cement, it slowly combines with the calcium hydroxide released during the cement hydration process to form new cementing compounds.Type C flyash, produced from sub-bituminous or lignite coals, contains a higher lime content than Type F flyash, and possesses some cementitious qualities of its own. It will chemically react with water, even if there is a deficiency of Portland cement. Hence, the setting time is faster than Type F flyash, Table 6.1

6.1.3: BLAST FURNACE SLAG

Blast furnace slag, BFS, is a by-product of the steel industry and accordingly, the availability of BFS is limited in quantity and location. BFS results from the fusion of the calcium from the limestone with the siliceous and alluminous residues from the iron ore in the blast furnace. Major oxides in BFS are not free but combined. It is the physical state of BFS that is fundamental to its cementitious properties, Table 6.1

6.1.4: NON-FERROUS SLAG

Non-ferrous slags generally refer to copper, nickel, zinc and/or lead slags extracted from sulphide concentrates by pyrometallurgical treatment. The process includes three different

% BY MASS

	PC	BFS	TYPE C FLASH	TYPE F FLY ASH	CSF
SiO_2	19.6	38	35	45	94
AL_2O_3	4.3	9	20	20	1
Fe_2O_3	3.4	0.8	5	5	1
CaO	61.5	40	25	5	0.5
MgO	3.6	12	5	5	1.1
SO_3	3.4	-	2.5	1.2	0.5
S (Sulphide)	-	1.8	-	-	-
Total Alk. as Na_2O	1.1	0.5	1.4	0.5	0.1
LOI	2.4	-	0.4	6.0	2.5
Physical					
Blaine (cm2/g)	3.500	4.500	4,000	3.000	200.000
45μm (% ret)	10	2	12	17	-
SG	3.15	2.9	2.7	2.3	2.2

Table 6.1: Typical Chemical and Physical Compositions of Supplementary Cementitious Materials.

operations: roasting, smelting and converting. During the smelting and converting stages, different slags are produced. Smelter slag is either discarded without treatment or granulated with excess water. The chemical composition of non-ferrous slag consists of high iron oxide and lower calcium oxide, exactly opposite of the one of the ferrous slag .

6.2: PAST STUDIES AT KIDD CREEK

The replacement of Portland cement with ground blast furnace slag and Type C flyash had already been evaluated at Kidd Creek Mines before the start of this thesis. The studies carried out before implementing these materials in the KCM fill system are briefly described below.

6.2.1: CEMENT SLAG ROCKFILL, CSLRF

The high cost of Portland cement prompted the use of ground blast furnace slag as a partial replacement of cement. Cement /slag rockfill, CSLRF, refers to the consolidated rockfill incorporating a blended binder of Portland cement and ground blast furnace slag. Test results, Fig.6.2.1, indicated that at 85 days the strength of CSLRF with 2.5% cement and 2.5% slag by-weight as a blended binder attained an equivalent strength of a 5% Portland cement mix (Yu and Counter, 1983). The addition of 5% sand enhanced the compressive strength of the CSLRF by 45%.

To evaluate the curing process of cement slag rockfill in an underground environment, fourteen large test blocks, 60 cm by 60 cm by 100 cm high, were cast underground. The dynamic strength characteristics were determined by employing the impact method with a Schmidt hammer and by the detonation of blasting caps within the blocks. This allowed the evaluation of the degree of blasting resistance by the relative size of craters.

The test result confirmed the previous observation that blocks containing 15% sand possess a higher dynamic resistance than those with a lower sand content. After more than two

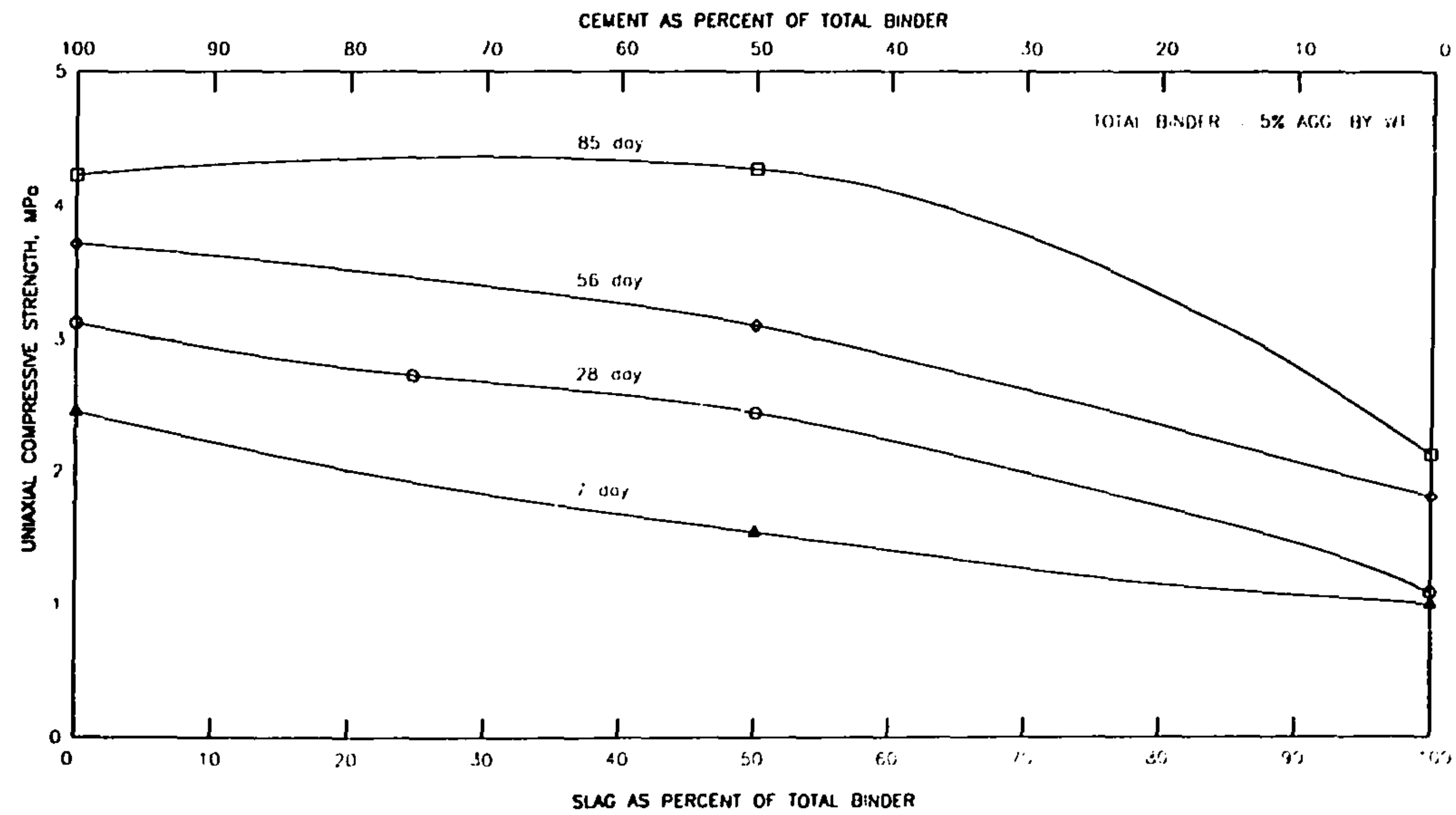


Fig: 6.2.1 Uniaxial Compressive Strength vs Slag/Cement Ratio years of testing, the following conclusions were made:

1: Ground blast furnace could replace up to 50% of the Portland cement in CRF without loss of its compressive strength provided that the fill was allowed to cure for three month or more.

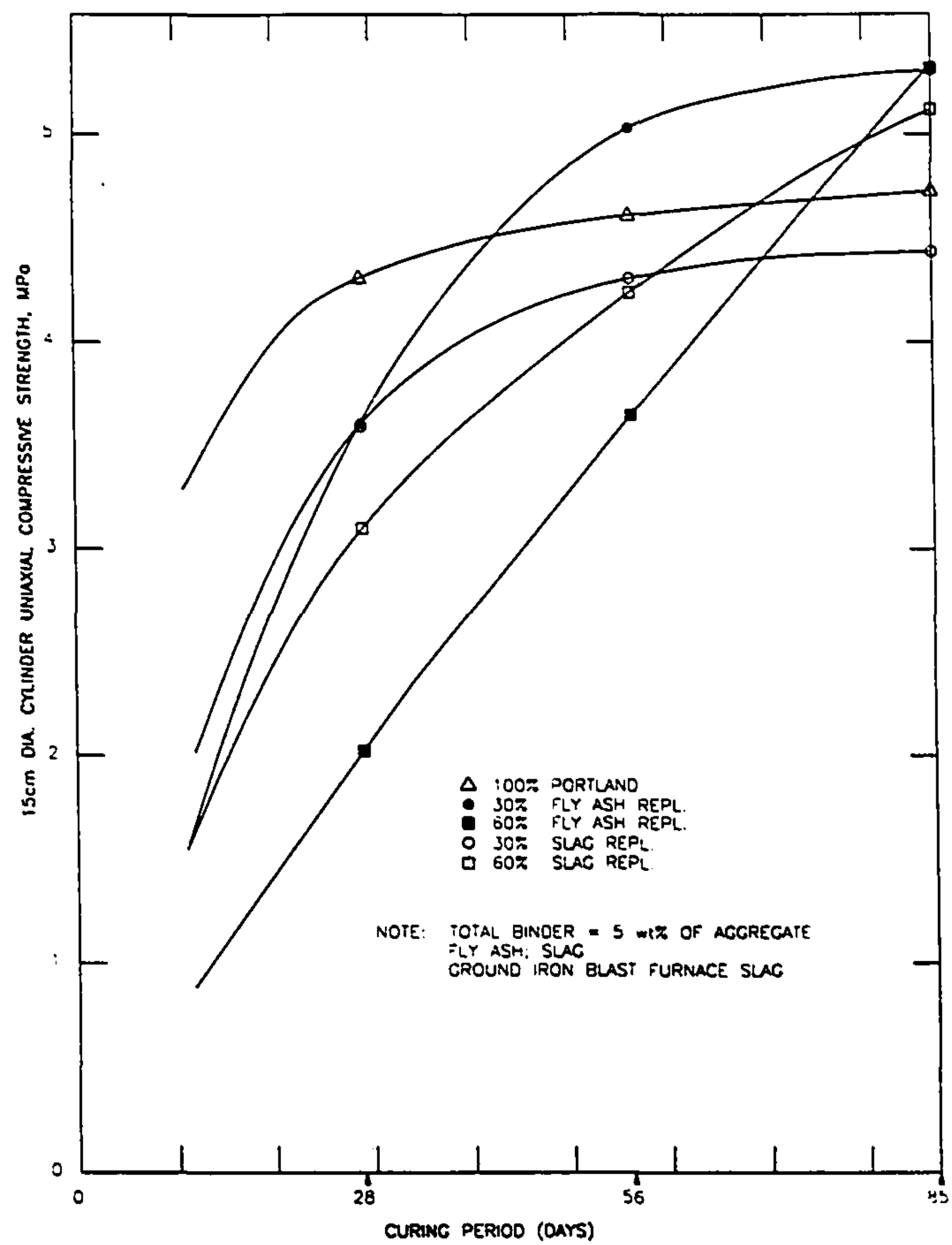
2: The mix of Portland cement and slag with sand slurry also produced an acceptable fill, but required longer curing periods to obtain the desired strength.

3: No operational problems were encountered in the use of blended cement and slag when filling a waste pass. This large scale test also revealed that the slow curing nature of slag eliminates the need of a retarding admixture, resulting in a further cost saving.

In Kidd Creek No.1 Mine, where a minimum three-month curing period was available before adjacent pillar recovery, up to 60% of the Portland cement was replaced by an equivalent weight of ground slag with satisfactory results. Due to a shorter curing period available in the No. 2 Mine, normally three weeks, only 33% replacement of Portland cement by ground slag was allowable. A total of 70,000 tonnes of ground slag was used in the backfill.

6.2.2: CEMENT/ FLYASH ROCKFILL

In the past Kidd Creek carried out test work on a partial Portland cement replacement by Type F flyash in consolidated fill. The results indicated, Figure 6.2.2, that flyash had the potential to be considered as a partial Portland cement replacement. However, the requirement of longer curing time periods and the inconsistent chemical composition of flyash impeded its use. Also, at that time, cement prices were low, and there was not a significant cost advantage to justify the use of flyash in the backfill. In 1983, however, the steadily increasing costs of Portland cement and ground blast furnace slag prompted the search for a lower cost cementing agent. Four flyash



(Yu, Counter 1988)

Fig: 6.2.2 Strength Development of Various Mixes samples from different sources were evaluated. Of the four samples, two were Type C ash, and two were Type F flyash (Yu and Counter, 1988).

6.2.2.1: LABORATORY WORK ON FLYASH

Three series of tests were conducted. The first test series evaluated the cementitious properties of one Type C flyash sample and two Type F flyash samples, in comparison with ground blast furnace slag. The second test series confirmed the strength development of Type C flyash. A total of over 200 test cylinders, 15 cm in diameter by 30 cm high, were cast for uniaxial compression tests after 28, 56 and 85 days of curing. The third test series compared the two Type C flyash samples. In addition, 60 cm cubes were cast for relative dynamic strength evaluation.

Results, Fig. 6.2.2, showed that Type C flyash exhibited higher strength than either of the two Type F flyash samples at every mix proportion and curing period. The compressive strength of the 30% Type C flyash was the same as that of the 30% slag mix at 28 days of curing, and became higher afterwards, surpassing the control mix after 43 days of curing. The 60% Type C flyash mix had a lower 28-day strength, but grew steadily and exceeded both the 60% slag mix, and the control mix after 73 days of curing. The second series of tests was to further determine the effects of varying the content of Type C flyash, and of slag on the strength development. There was no indication of any advantage of combining the flyash with slag. The third series of tests showed that the development of strength for both Type C flyash samples was very consistent.

6.2.2.2: STOPE TEST EVALUATION OF FLYASH

To fully evaluate the performance of flyash as a partial Portland cement replacement, a full scale stope test was carried out in 1984. This trail resulted in the followings:

1. Flyash could be pneumatically loaded into the silos from tanker trucks, in the same manner as Portland cement or slag, with no difficulty.

2. The behaviour of the flyash was similar to that of slag during the weighing, mixing,

delivery and clean-up phases in the cement slurry mixing plant.

3. No accumulation of undue scale build-up was noted with using flyash.

4. Type C flyash exhibited comparable cementitious properties as ground blast furnace slag.

5. The emanation of radon gas, in terms of Working Levels, from all tested flyash mixes was within the Ontario Ministry of Labour's guidelines.

6.3 : OTHER BINDER ALTERNATIVES

Considerable savings can be achieved by minimizing the percentage of Portland cement used in the present backfill mix by partially replacing it with other cheaper binder materials. A comprehensive testing program was conducted to evaluate the cementitious properties of different binders with or without addition of commercially available chemicals. The challenging aspect of this testing program was to match or exceed the strength of the present backfill using less expensive and/or local materials, such as KCM copper slag from the Timmins area. The main emphasis on this testing program was to increase the early and ultimate strengths of the present flyash/cement mix, but also to evaluate the cementitious properties of KCM copper slag for possible use as a binder.

The aggregate used in the cylinder tests was a mixture of washed gravel and alluvial sand at a ratio of 2 to 1. Sample preparation followed the C39-83b ASTM standard. The samples were all cured at 25 deg. Celcius tempreature and placed in standard curing tank. The maximum aggregate size was 1.8 cm. All the cylinder tests were carried out using 7.35 cm diameter, 14.7 cm long, cardboard moulds. Each mix contained 5% by weight of cementitious material. A water/cement ratio of 1.2 by weight was maintained in all sets unless mentioned otherwise. To reduce the effect of variation in the composition of Portland cement on the test strength, a blend of normal Type 10 Portland cement from four cement suppliers was used in all mixes. A minimum of three cylinders per set were tested. The binder alternative test program is shown in Figure 6.3.1. Figures 6.3.2 and 6.3.3 show the steps taken to prepare each set of samples.

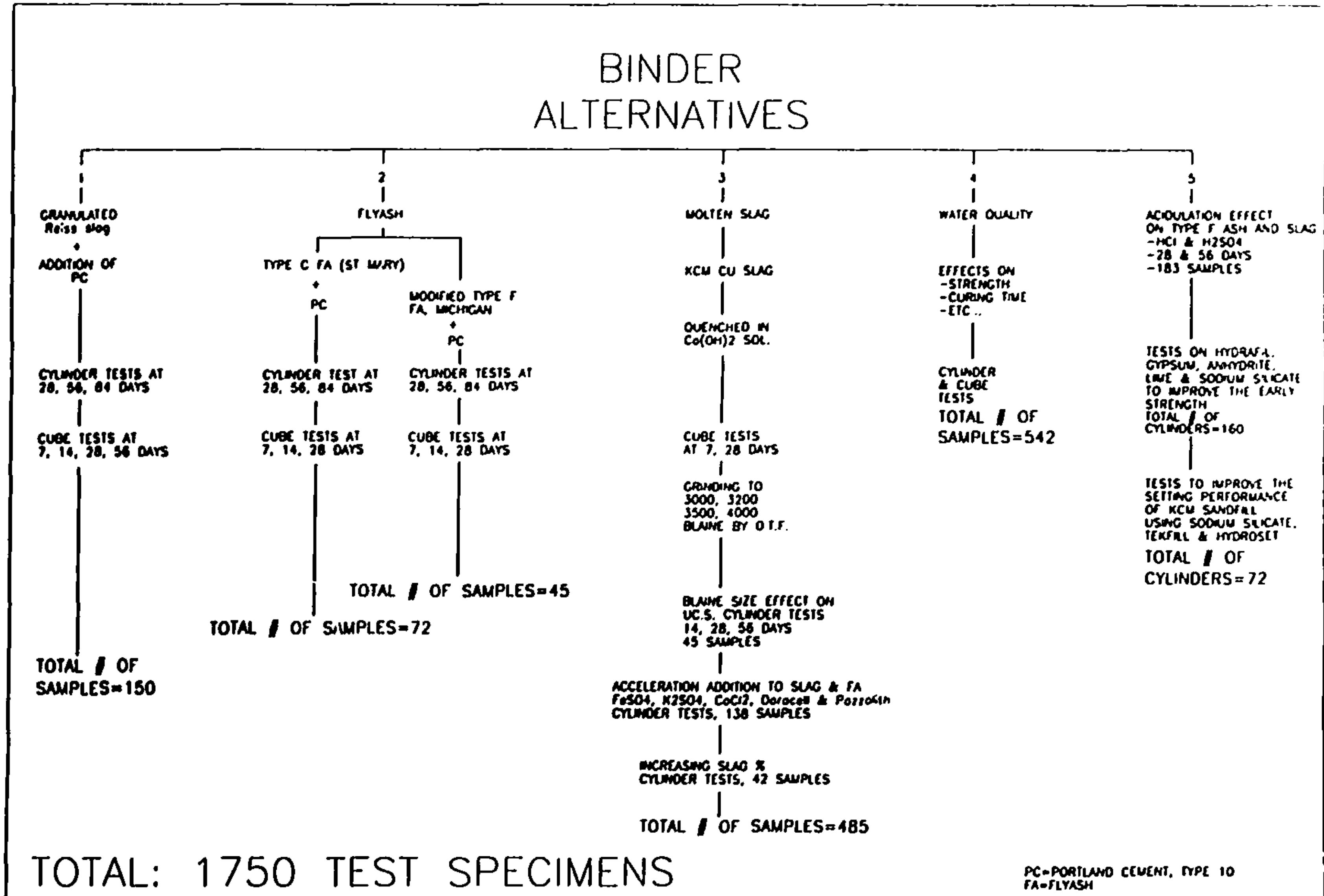


Fig: 6.3.1 Binder Alternatives Program

LABORATORY WORK CARRIED OUT AT KIDD CREEK MINES

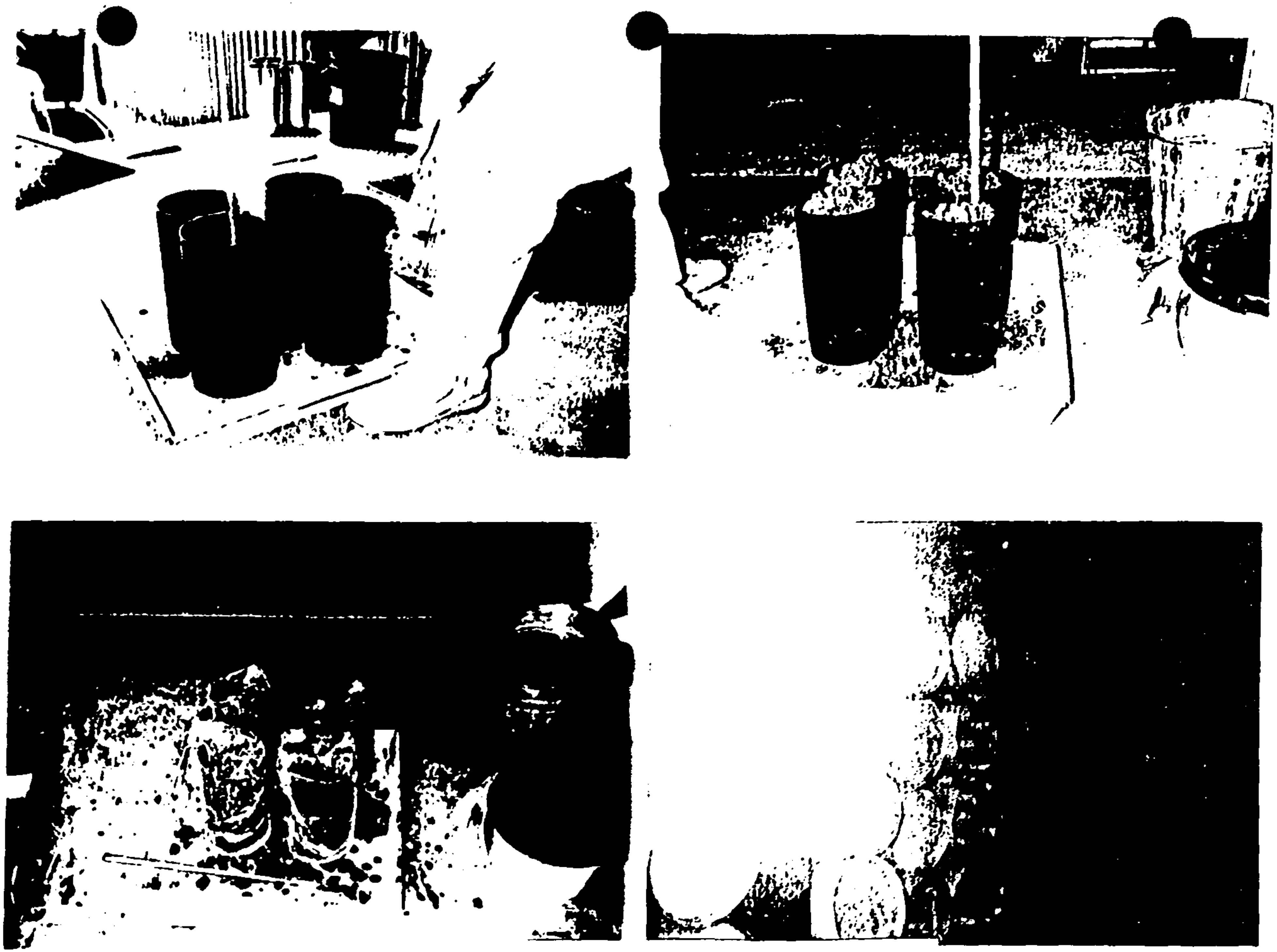


Table 6.3.3: Small Scale Cylinder Preparation.

Table 6.3.2: Small Scale Cylinder Preparation.

6.3.1: COPPER SLAG BLAINE SIZE TEST

OBJECTIVE: To study the effect of copper slag Blaine size on the compressive strength development of backfill material. A comparison between regular copper slag and requenched, lime-enriched, slag was also carried out.

The materials used in these sets were :

SET A : 100% P.C. (CONTROL)

SET B : 60% P.C. / 40% slag, 3000 Blaine, KCM regular copper slag which had Work Index value of 28.9, and D_{80}, 80% weight passing micron size, of 1960.

SET C : Same as set B but the slag used was 4000 Blaine.

SET D : Same as set B but requenched copper slag #1 (high lime content) of 3050 Blaine was used, Work Index = 48.7 .

SET E : Same as set B but requenched copper slag #2 (lower lime content) of 3214 Blaine was used, Work Index = 27.2, and had D_{80} of 893.

DAYS	**COMPRESSIVE STRENGTH, MPa**				
	SET A	**SET B**	**SET C**	**SET D**	**SET E**
14	1.9	1.82	1.65	1.75	1.56
28	2.15	1.94	1.92	2.0	1.7
56	3.2	2.15	2.2	2.4	2.0

- Set A (control) had the highest compressive strength values, Figure 6.3.4

- Sets B and D showed very encouraging results. The 14 and 28 day strengths of set B were 95 and 90% of the strength of the control set A respectively. However, the longterm strength of set B was only 68% of the control set A.

- Comparing sets B and C indicated that the finer slag of 4000 Blaine did not contribute in increasing the compressive strength.

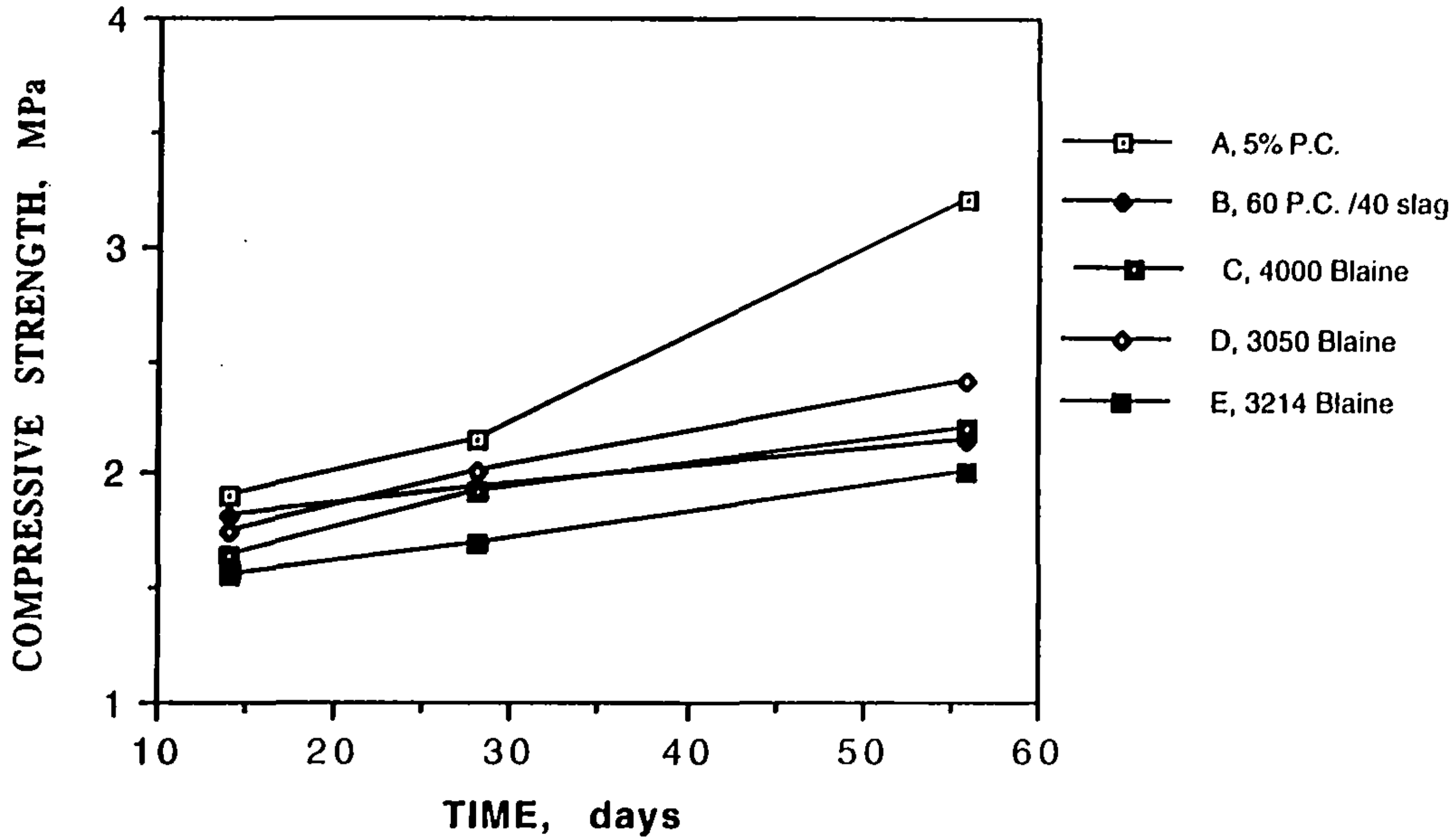


FIGURE 6.3.4: SLAG BLAINE SIZE TEST

- Control set had 48% increase in its strength from 28 to 56 day curing. The highest compressive strength increase rate for samples containing slag was 20% in set D.

- Set D, requenched lime-enriched slag, attained the highest longterm strength values in samples containing slag. The compressive strength of set D reached to about 75% of the control set.

CONCLUSION

The results indicated that the copper slag did contribute greatly to the strength development of the cement/slag mixes. The Blaine size finer than 3000 had no effect on the strength development of the samples. Use of lime solution with requenched slag slightly increased the compressive strength of the material.

6.3.2: COPPER SLAG CUBE TESTS

OBJECTIVE: To study the cementitious property of copper slag . This is to investigate the rate of increase in compressive strength of the concrete cubes with different amounts of cement and/or copper slag. Copper slag, 4000 Blaine, was used in this testing program.

Compressive strength after 28 days, MPa

SET A, 100% P.C.	**22.4**
SET B, 40% P.C., 60% SLAG	**14.2**
SET C, 40% P.C., NO SLAG	**4.6**
SET D, 60% P.C., 40% SLAG	**20.7**
SET E, 60% P.C., NO SLAG	**11.7**

- Set A, 100% Portland cement, showed the highest strength after 28 days of curing, Figure 6.3.5.

- Set D with 40% replacement of cement with copper slag obtained a strength of 20.7 MPa

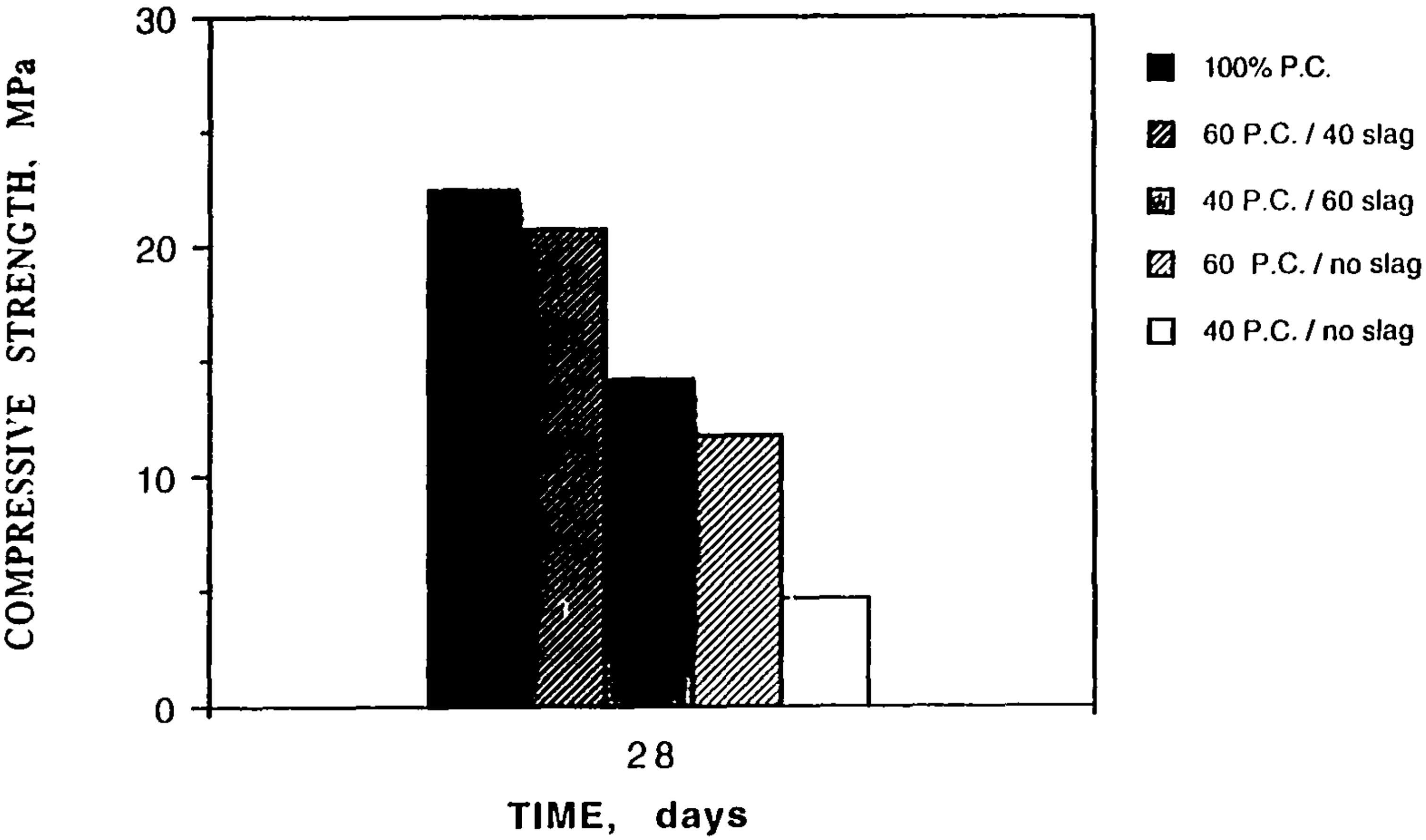


FIGURE 6.3.5: COPPER SLAG CUBE TEST

after 28 days which was 92% of the control set A.

- Since the slag curing process is much longer compared to Portland cement, the longterm strength of set D would surpass the strength of the control set A.

- An additional 20% replacement of cement in set B decreased the compressive strength by 45% compared to set D. This indicated that the partial replacement of cement with slag should not exceed 40%.

- Sets C and E containing no slag and low cement content had very low compressive strengths. Comparison of these results with sets B and/or D showed that the slag had a major contribution in strength development of the samples. The high water to cement ratio and high sand to cement ratio might have caused the low compressive strength of sets C and E.

CONCLUSION

The resulted indicated that copper slag has a moderate cementitious property if the replacement of the cement with slag does not exceed 40%. Replacement of 40% cement with slag, set D, decreased the compressive strength by only 8% and the samples containing slag will cure for a longer period of time and eventually will exceed the strength of the control set A.

6.3.3: $FeSO_4$ AND K_2SO_4 ADDITIONS, PART 1.

OBJECTIVE: To study the possibility of using accelerators such as $FeSO_4$ and K_2SO_4 to speed up the curing process of cement / slag mixes.

	Compressive strength, MPa
SET A, 5% P.C.	**1.16**
SET B, 60 P.C./40 SLAG	**0.31**
SET C, SET B + 10% $FeSO_4$	**0.23**
SET D, SET B + 20% $FeSO_4$	**0.21**
SET E, SET B + 10% K_2SO_4	**0.82**
SET F, SET B + 20% K_2SO_4	**0.61**

- Result from set E, 10% K_2SO_4, was encouraging. The accelerator had increased the compressive strength by 265% compared to set B without accelerator after 6 days of curing, Figure 6.3.6.

- Set F, 20% K_2SO_4, indicated that the amount of accelerator in the mix had passed the optimal percentage.

- $FeSO_4$ accelerator did not increase the early strengths of the mixes.

- K_2SO_4 showed good reaction properties with the P.C./slag mixes.

- A point load tester had to be used to determine the low early strengths.

-Since set F (20% accelerator) showed lower strength than set E (10% accelerator), it was concluded that the amount of accelerator in set F had exceeded the optimal dosage. The amount of accelerator at 5% and 15% of the binding material should also be investigated.

CONCLUSION

More detailed study of the type and the performances of different accelerators should be carried out to verify their potential future use. The longterm effects should also be monitored for material deterioration caused by chemical reactions of these accelerators. Other important factors for possible future use are the availability and the costs of these materials.

6.3.4: SLAG / ACCELERATOR TEST, PART 2.

OBJECTIVE: To study the effect of different accelerators on the compressive strength development of P.C. /slag binder. The accelerator will be used to speed up the curing process that was slowed down by replacing 40% of the cement with copper slag. Accelerators used in this study were $FeSO_4$, K_2SO_4, $CaCl_2$ and Daracell, a product of W.R. Grace Chemical Company. Various amounts of these materials were tested to find the optimum percentage of the accelerator in each case.

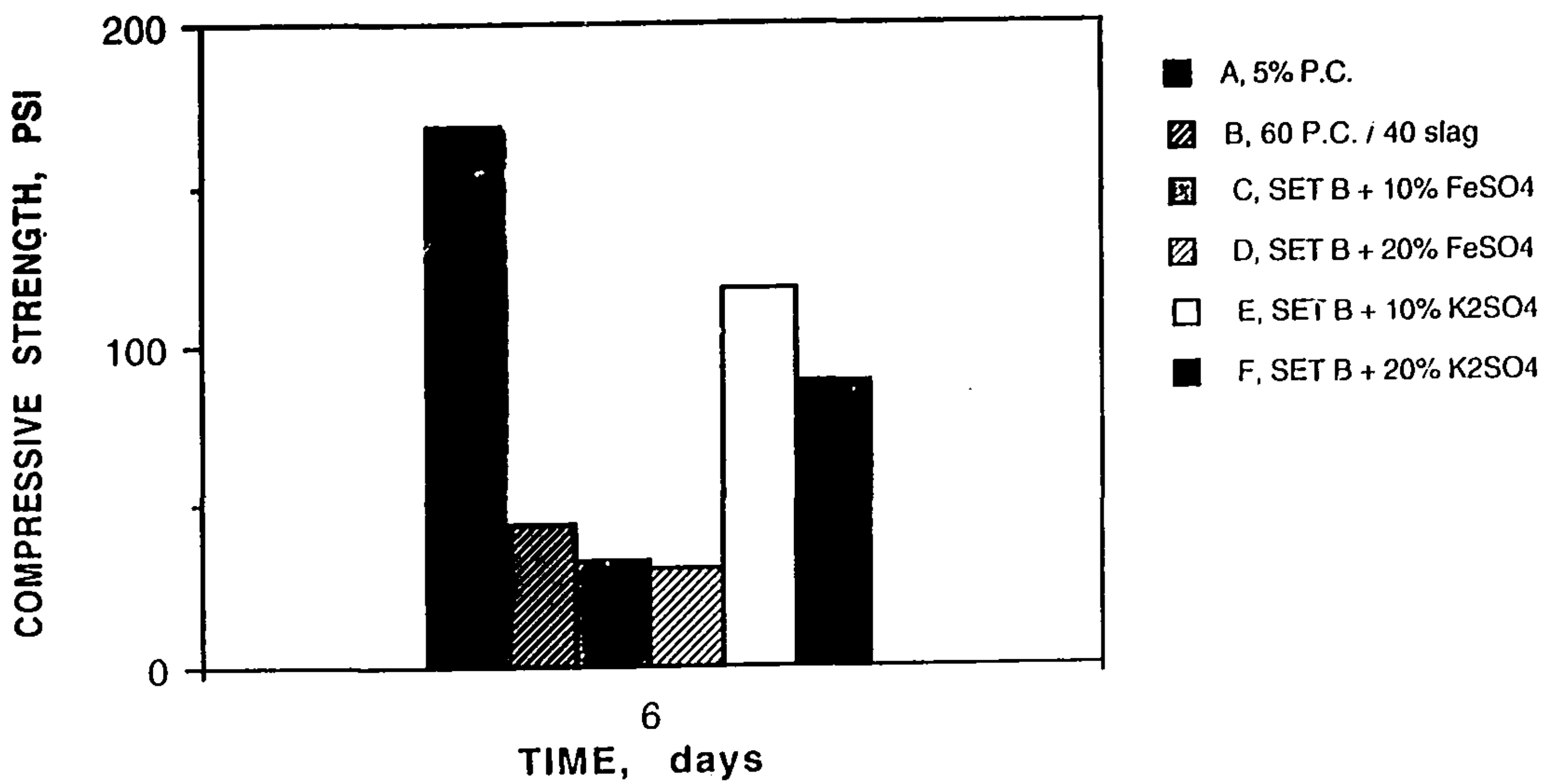


FIGURE 6.3.6: ACCELERATOR ADDITION, PART 1.

- Following table presents the results obtained for each set after 28 and 56 days of curing.

	COMPRESSIVE STRENGTH MPa 28 days	56 days
SET A: 5% P.C., CONTROL	2.37	3.18
SET B: 3% P..., NO SLAG	1.14	1.46
SET C: 3% P.C. + 2% SLAG	1.73	1.73
SET D: ADDING 5% K_2SO_4	1.75	2.42
SET E: ADDING 10% K_2SO_4	1.50	2.05
SET F: ADDING 15% K_2SO_4	1.87	2.34
SET G: ADDING 5% DARACELL	1.68	2.18
SET H: ADDING 10% DARACELL	2.81	3.30
SET I : ADDING 15% DARACELL	2.24	2.90
SET J : ADDING 5% $FeSO_4$	1.22	1.42
SET K: ADDING 10% $FeSO_4$	1.77	1.70
SET L : ADDING 3% $CaCl_2$	2.04	2.04
SET M: ADDING 5% $CaCl_2$	2.14	2.00

- Comparing sets B and C showed that the addition of slag to set C which had 3% cement content increased the compressive strength by 18%. This is 54% of the strength of the control set A with 5% cement content, Figure 6.3.7.

- Set H, 10% Daracell, showed the best results by achieving a 4% higher compressive strength than the control set A.

- There was a 51% increase in strength when the amount of Daracell was raised from 5%, set G, to 10%, set H. The strength decreased by 14% with increasing the Daracell concentration from 10 to

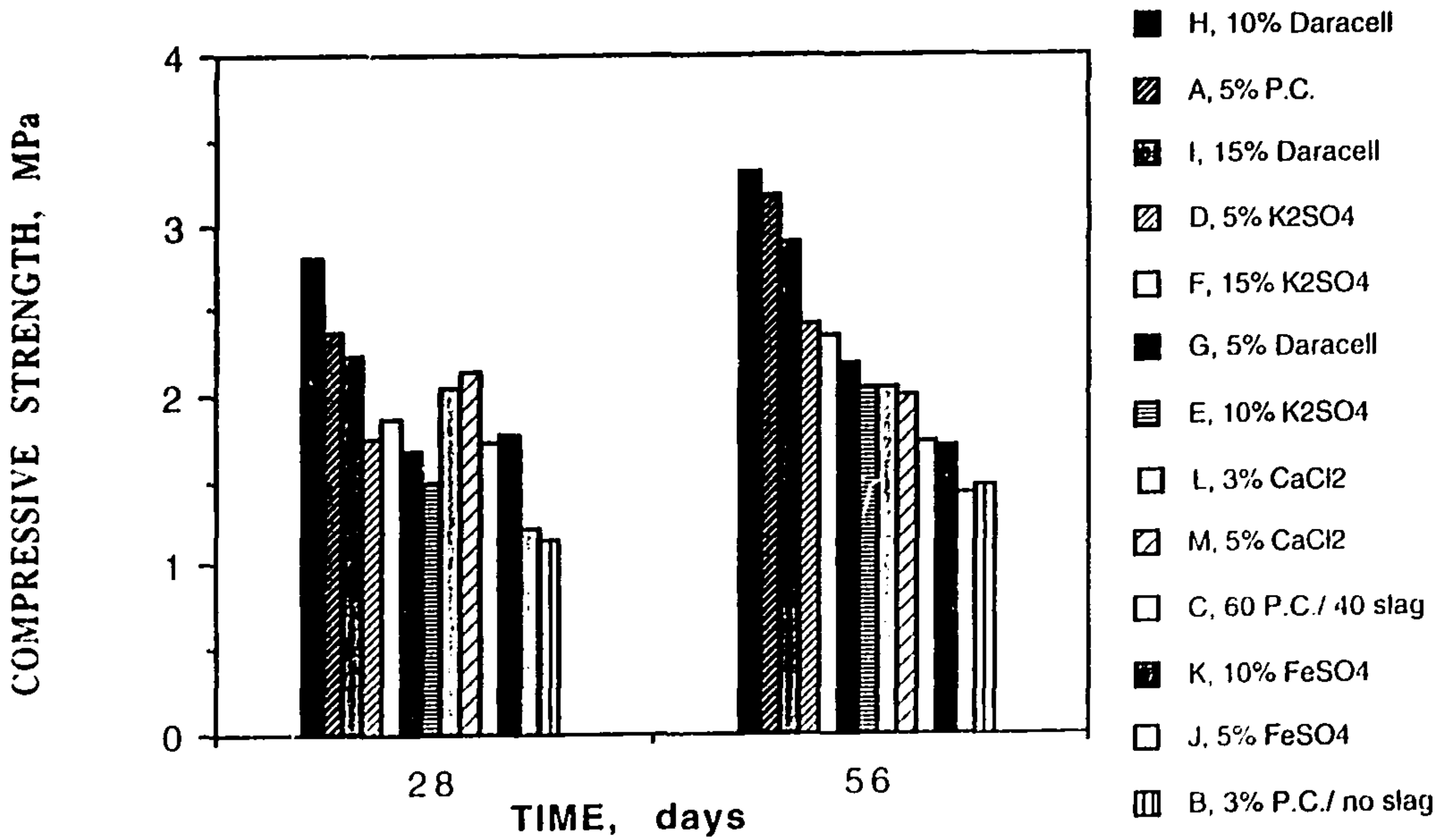


FIGURE 6.3.7 : ACCELERATOR ADDITION, PART 2.

15%.

- Sets D, E and F having 5, 10 and 15% K_2SO_4 showed a much better results after curing for 56 days of curing compared to 28 day results. Set D, 5% K_2SO_4, achieved 76% compressive strength of the control set A after 56 days of curing.

- Adding 5% $CaCl_2$, set M, to P.C./slag mix, set C, increased the strength by 16 % as compared to 19% when 3% $CaCl_2$ was added in set L.

- The strength of set M, 5% $CaCl_2$, was 76% of set H, 10% Daracell, and 90% of the control set A after 28 days curing. These values drop to 61% and 63% respectively after 56 days.

- Ferrous sulphate did not contribute to the compressive strength of the mixes after 56 days of curing. It even decreased the strength when 5% $FeSO_4$ was used .

CONCLUSION

Daracell and K_2SO_4 performed well with the P.C./slag mix. Daracell increased the compressive strength higher than the control mix. The extra cost of Daracell, however, could make it impractical for use in a rockfill system. Potassium sulphate obtained a much higher compressive strength after 56 days of curing. Ferrous sulphate did not perform satisfactory after 56 days of curing.

6.3.5: SLAG / ACCELERATOR, PART 3.

OBJECTIVE : The same as Test #4. The accelerators used in this study were Daracell, a product of W.C. Grace Chemical Company, Pozzolith 133-HE, a product of Master Builder Chemical Company, K_2SO_4 and $CaCl_2$. Various amounts of these materials were tested to find the optimum percentage of the accelerator needed in each case. These test results were then combined with the results of the the past tests to evaluate the performance of each accelerator.

- Following table presents the results obtained for each set at 28 and 56 days.

COMPRESSIVE STRENGTH, MPa

	28 days	56 days
A. 5% P.C., CONTROL	4.60	6.20
B. 60 P.C./ 40 SLAG	2.50	3.50
C. SET B + 7.5% DARACELL	2.83	4.50
D. SET B + 12.5% DARACELL	1.86	2.70
E. SET B + 5% POZZOLITH 133-HE	2.25	3.70
F. SET B + 10% POZZOLITH 133-HE	3.90	5.30
G. SET B + 2% CaCl2	3.01	3.40
H. SET B + 7% CaCl2	2.71	2.90
I. SET B + 2% K_2SO_4	1.98	2.06
J. SET B + 4% K_2SO_4	2.56	2.81

- Set F, 10 % Pozzolith 133-HE, reached 85% strength of the control set A, 5% P.C., after 56 days of curing. This set had 51% higher strength compared to control set B after 56 days of curing, Figure 6.3.8.

- Results of this test combined with the results of 6.3.4 indicated that the optimum percentage of Daracell in the mix was around 10%.

- There was a 40% decrease in strength when Daracell concentration was increased from 7.5 to 12.5% in sets C and D, Fig.6.3.9.

- The strength of set G, 2% $CaCl_2$, was 64% of set F, 10% Pozzolith 133-HE. Fig. 6.3.10 shows that the optimum $CaCl_2$ concentration in the mix was around 3%.

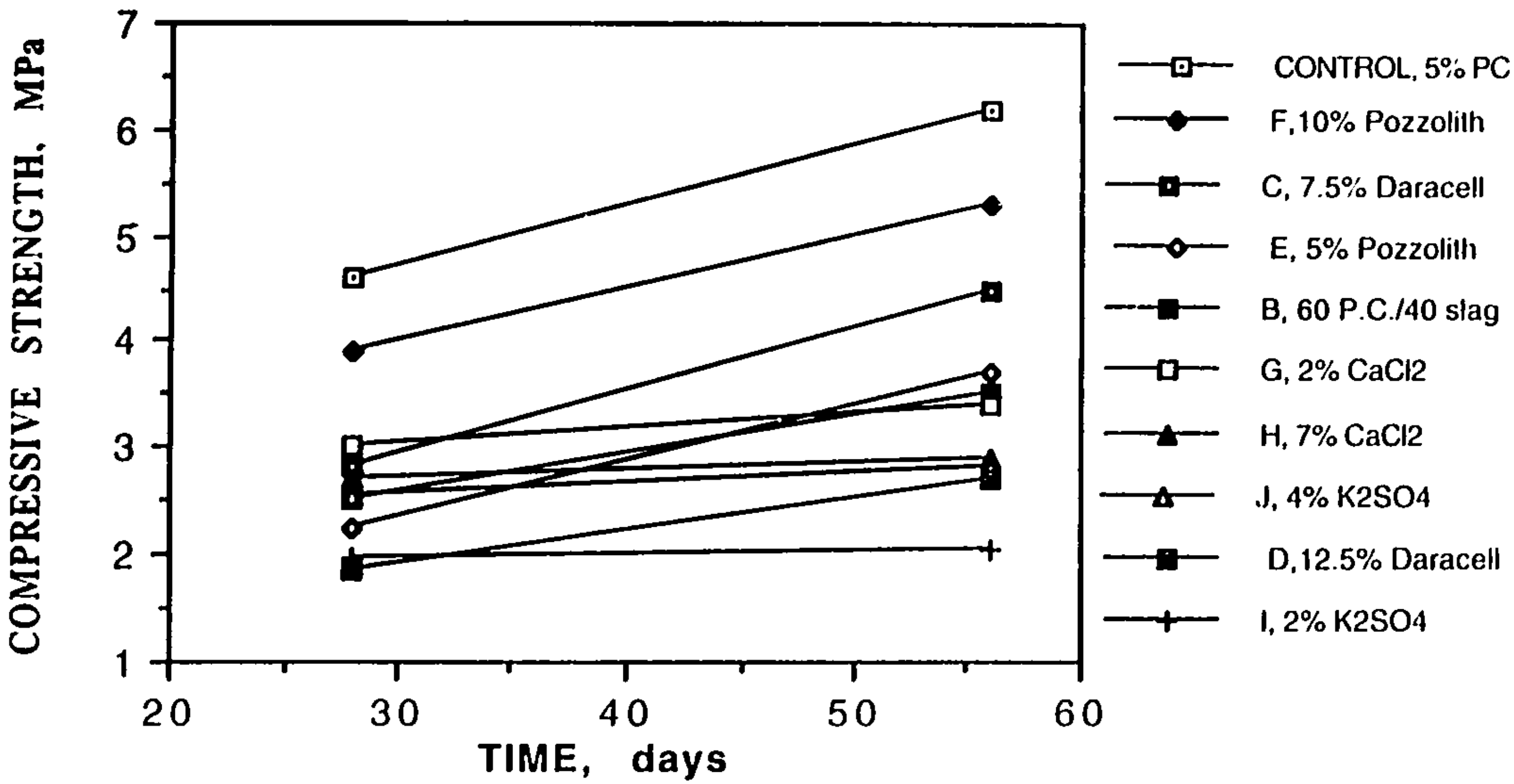


FIGURE 6.3.8 : ACCELERATOR ADDITION, PART 3.

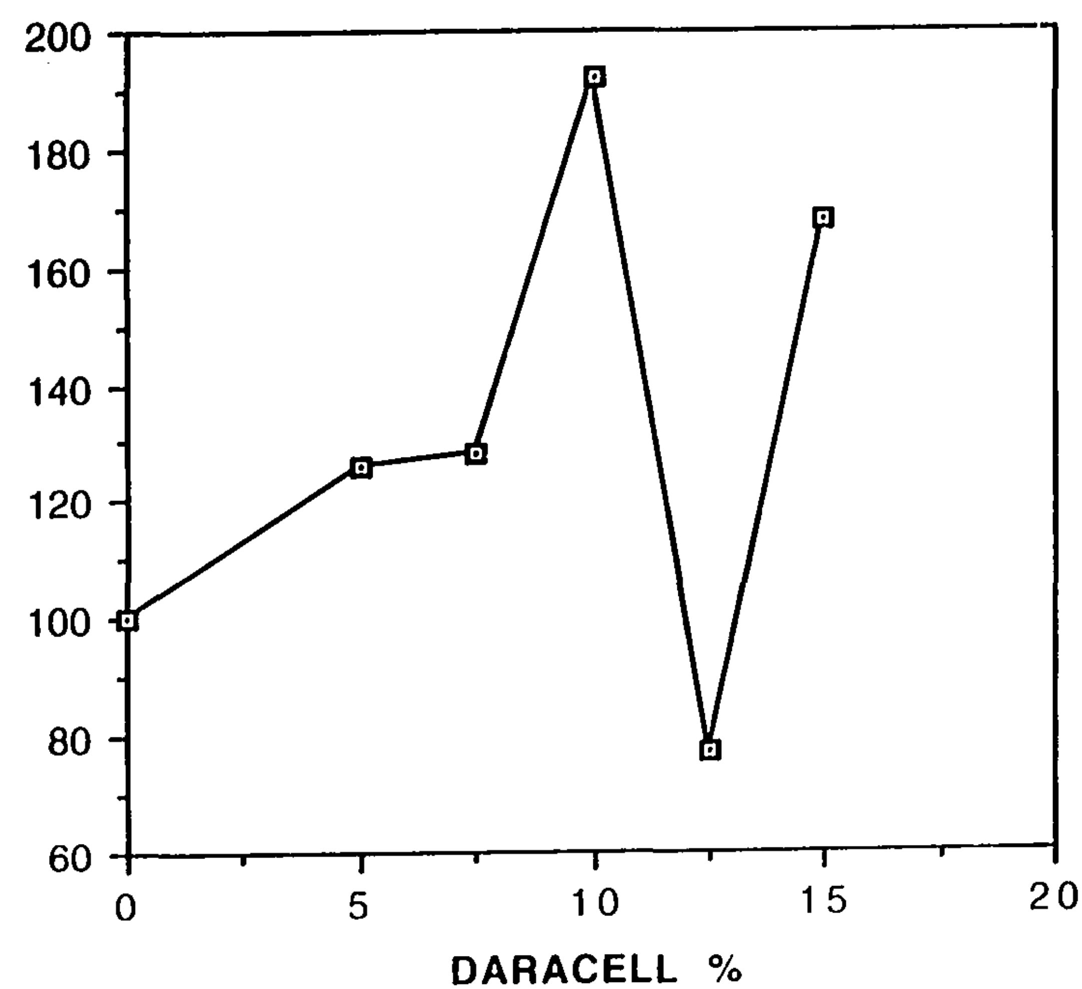


FIGURE 6.3.9 : OPTIMUM DARACELL CONCENTRATION

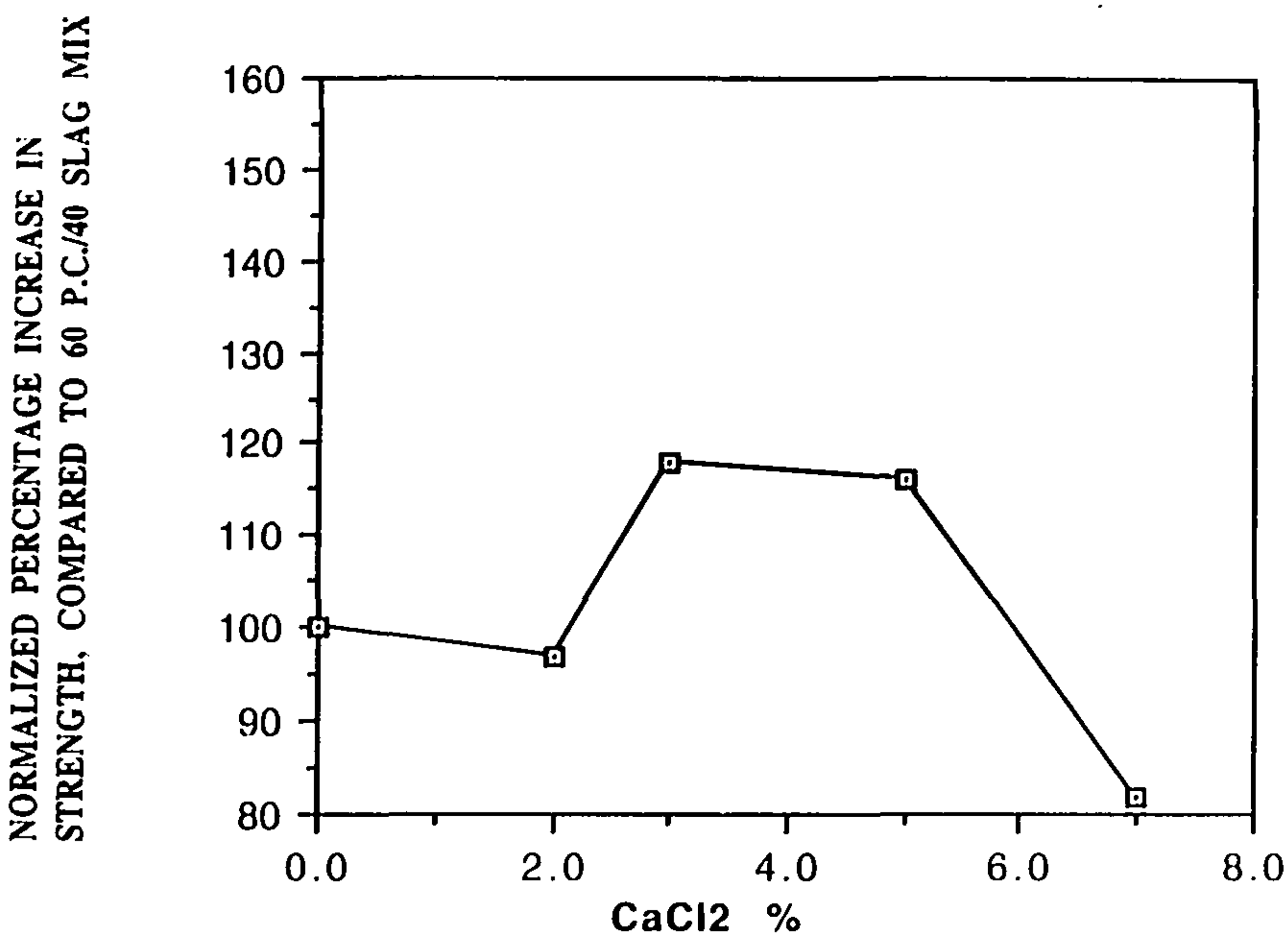


FIGURE 6.3.16: OPTIMUM CaCl2 CONCENTRATION

- 2% and 4% K_2SO_4 decreased the compressive strength of the samples considerably after 28 and 56 days of curing, Fig. 6.3.11.

CONCLUSION

All the results were compared with control set B, 60 P.C./ 40 slag mix. Daracell and Pozzolith 133-HE improved the strength considerably but the costs of these material were high and no savings could be obtained from using these in the fill system. Addition of 2% $CaCl_2$ increased the compressive strength by 20% compared to set B after 28 days of curing but the 56 day results were almost the same as the control set with no $CaCl_2$. This indicated that the $CaCl_2$ increased the curing rate in the first month and could be employed to achieve faster mining cycle. Addition of K_2SO_4 showed a very disappointing result and did not improve the compressive strength of the fill material. The longterm results of samples which contained both 2% and 4% K_2SO_4 were lower than the control set B, and no benefit would be gained by using this chemical.

6.3.6: ACIDULATED TYPE F FLYASH , PART 1

OBJECTIVE : This study involved the partial replacement of Portland cement with Type F flyash. Flyash normally has high iron and/or aluminium content. By treatment of the flyash with strong mineral acids the excess metals could be removed. The goal was to convert the oxides of excess metals into soluble salts that could be washed away. Removal of the undesirable metals would create a much greater surface area for the reaction of silica. Hydrochloric and Sulphuric acids at two different concentrations were selected for testing.

Acids weighing 1.2 times the weight of the flyash were used to acidulate the flyash. The acidulated solution was allowed to chemically react for 15 and/or 60 minutes before mixing with the rest of the materials. Before using the acidulated flyash in the final mix, the acid on top of the mix

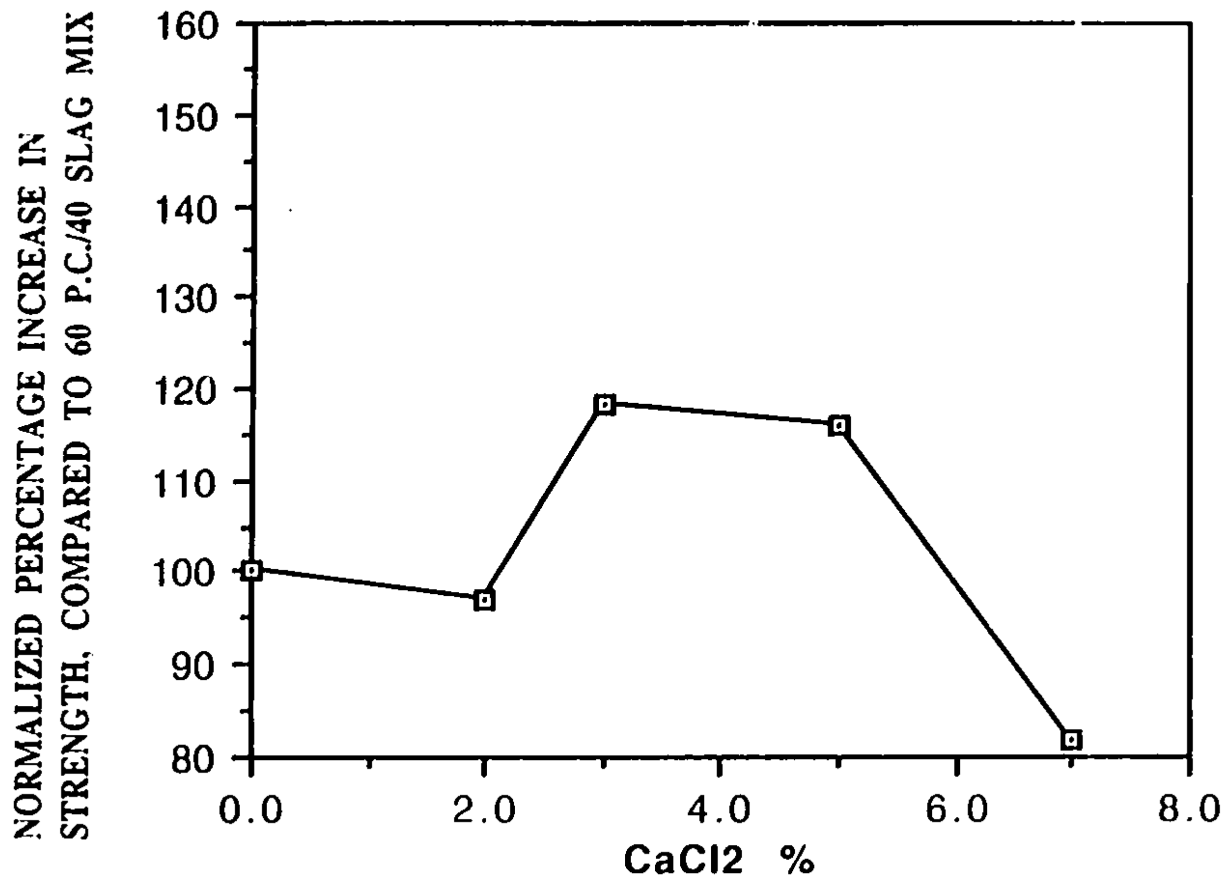


FIGURE 6.3.11 : OPTIMUM CaCl2 CONCENTRATION

was decanted. The amount of acid left in the mix was then calculated to find the final amount of water to be added to the mix. It should be noted that the moisture content in acidulated samples was about 1% higher than the one in unacidulated sets .This fact most likely caused the decrease of the strength in the acidulated sets due to the higher water content.

COMPRESSIVE STRENGTH, MPa

	28 days	56 days
A. 100% P.C.	1.86	2.46
B. 33% P.C., 66% FLYASH (F)	1.05	1.42
C. SET B+ 38% HCl, 60 MIN.	0.00	0.00
D. SET B+ 38% HCl, 15 MIN.	0.00	0.00
E. SET B+ 20% HCl, 60 MIN.	0.39	0.67
F. SET B+ 20% HCl, 15 MIN.	0.85	1.40
G. SET B+ 38% H_2SO_4, 60 MIN.	0.00	0.00
H. SET B+ 38% H_2SO_4, 15 MIN.	0.00	0.00
I. SET B+ 20% H_2SO_4, 60 MIN.	1.76	2.40
J. SET B+ 20% H_2SO_4, 15 MIN.	1.80	1.57

- Replacing 66% of the Portland cement with unacidulated Type F flyash decreased the compressive strength by 56% after 28 and 56 days of curing, by comparing sets A and B, Fig. 6.3.12.

- Acidulation of flyash with H_2SO_4 solution (20% concentration) for a time period of 60 minutes, set I, increased the compressive strength comparable to the control set A, and caused a 70% increase in strength compared to the unacidulated one, set B.

- Sets C, D, G and H with 38% concentration of both HCl and H_2SO_4 had no strength development even after 56 days of curing.

- 20% HCl solution, set E, decreased the compressive strength by 53% compared to the

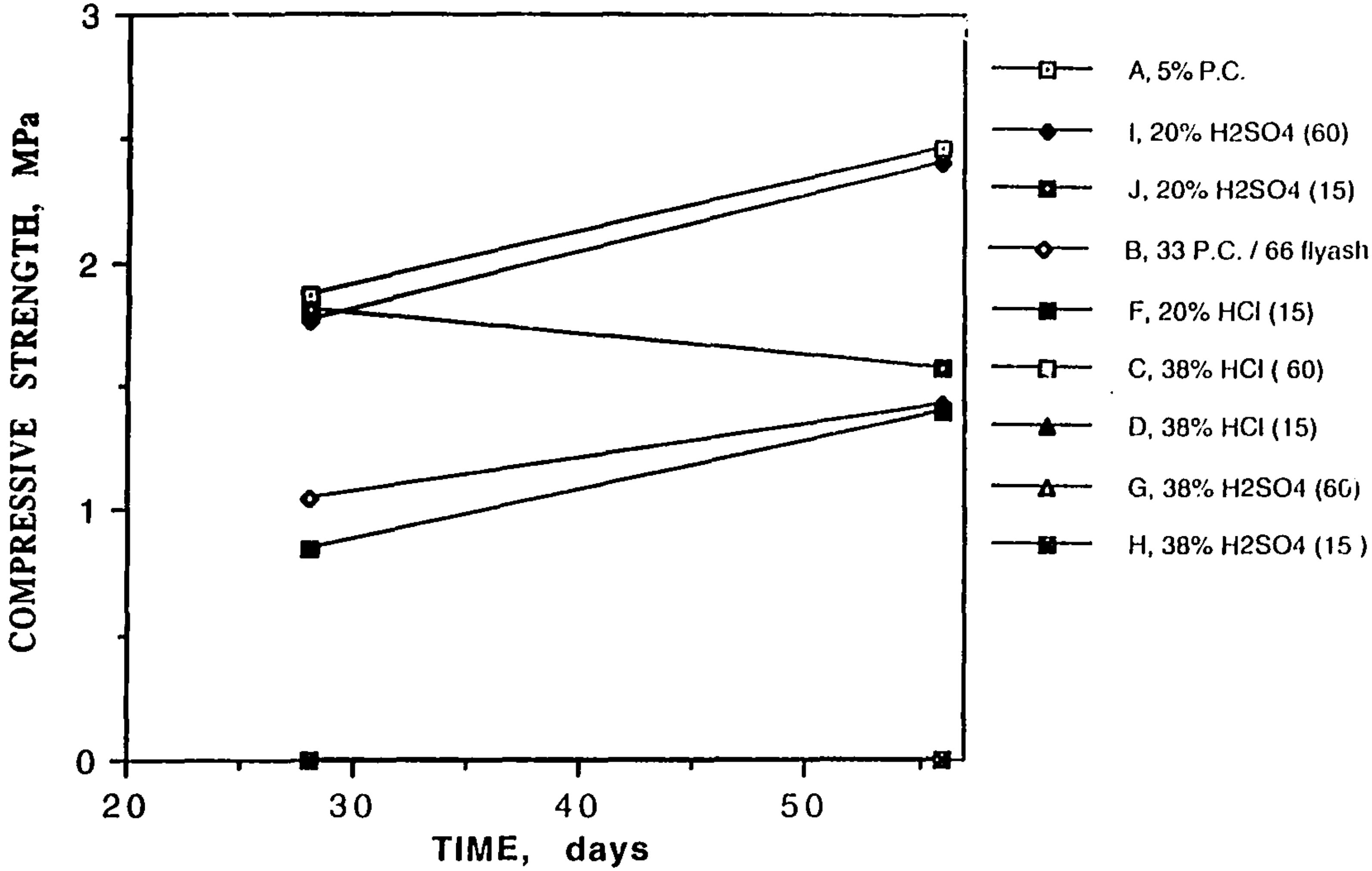


FIGURE 6.3.12 : ACIDULATED FLYASH TEST, PART 1.

control set B, when the flyash was acidulated for 60 minutes.

- Set J, 20% H_2SO_4, had 13% lower strength at 56 days compared to the strength at 28 days. This was the only set that did not increase in strength after 28 days of curing.

CONCLUSION

More tests on acidulated flyash with a lower concentration of acid should be carried out .

6.3.7: ACIDULATED TYPE F FLYASH/ COPPER SLAG, PART 2.

OBJECTIVE : To study the effect of acidulated Type F flyash and ground copper slag on cementation development.

COMPRESSIVE STRENGTH, MPa

	28 days	56 days
A. 5% P.C.	**1.72**	**2.60**
B. 33 P.C., 66 FLYASH(F)	**1.03**	**1.66**
C. 60 P.C., 40 SLAG	**1.57**	**2.03**
D. SET B + 20% H_2SO_4 . 30 MIN.	**1.16**	**N/A**
E. SET C + 20% H_2SO_4, 30 MIN.	**1.42**	**1.41**
F. SET B + 10% H_2SO_4, 30 MIN.	**1.10**	**1.60**
G. SET C + 10% H_2SO_4, 30 MIN.	**0.96**	**1.15**

- Replacing 66% of the Portland cement with unacidulated Type F flyash decreased the compressive strength by about 60% after 28 and 56 days of curing, by comparing sets A and B, Fig. 6.3.13.

- Control set for PC/slag mix, set C, obtained 91 and 78% of the strength of the 100% cement control set A, after 28 and 56 days respectively.

- Acidulated flyash with H_2SO_4 solution (20 and 10% concentrations) for a time period of 30 minutes did not contribute to the compressive strength as much as the samples used in 6.3.6.

- Both 10 and 20% addition of H_2SO_4 to the copper slag decreased the compressive strength by 44 and 31% respectively, compared to unacidulated samples of set C.

CONCLUSION

The preliminary results obtained from these tests indicated that the acidulated flyash did cause a very small increase in the strengths of the P.C./flyash mixes, but the acidulated P.C./slag mixes showed negative results. This could have been due to the short acidulation time; thus, further tests are deemed necessary to obtain conclusive results on the effect of acidulation of Copper slag and flyash.

6.3.8: ADDITIONAL COPPER SLAG TEST

OBJECTIVE : To study the cementitious property of copper slag. This is to investigate the rate of increase in compressive strength of 3-in. diam. cylinders with different amount of copper slag with a constant Portland cement content.

	COMPRESSIVE STRENGTH, MPa	
	28 DAYS	**56 DAYS**
A, 5% P.C.	**2.76**	**3.02**
B, 3% P.C., 2% SLAG	**1.80**	**2.30**
C, 3% P.C., 4% SLAG	**2.40**	**2.65**
D, 3% P.C., 6% SLAG	**2.20**	**2.40**
E, 3% P.C., 8% SLAG	**1.20**	**1.73**

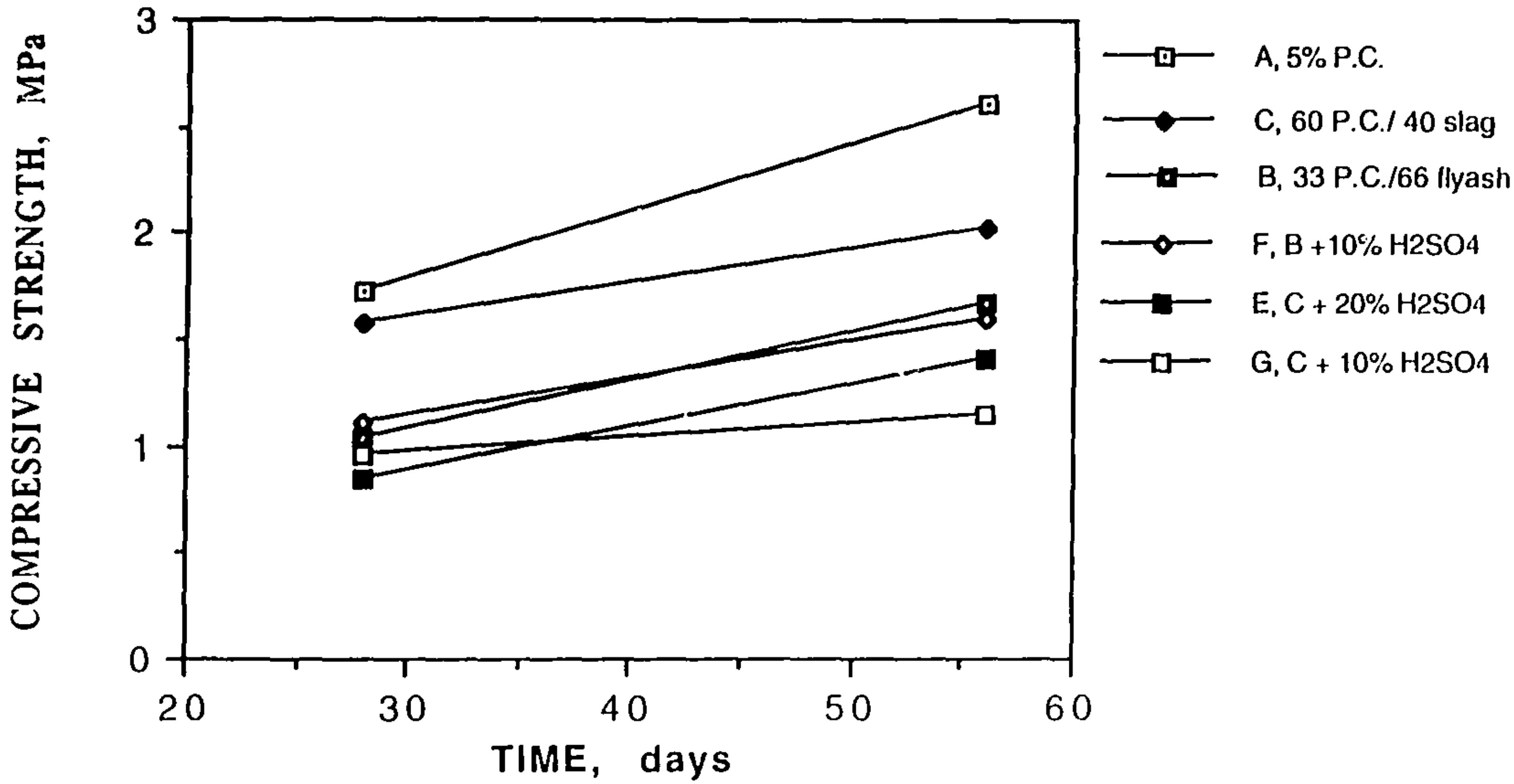


FIGURE 6.3.13 : ACIDULATED FLYASH TEST, PART 2.

F, SAME AS D WITH 20% LESS WATER	2.65	3.63
G, SAME AS E WITH 20% LESS WATER	1.70	2.04

- Set F, had the same 28-day compressive strength compared to the control set A. The 56 day compressive strength of set F surpassed the strength of set A by 20%, Fig.6.3.14

- 40% replacement of cement with slag decreased the compressive strength by 35 and 24% after 28 and 56 days of curing respectively, by comparing sets A & B.

- Doubling the amount of slag in the mix increased the compressive strength by 33 and 15% after 28 and 56 days of curing respectively, by comparing sets B & C. However, they were still lower than the control set A by 13% for both 28 and 56 day curing.

-Adding more slag in sets D and E did not contribute to the compressive strength, possibly due to the increased amount of water in the final mixes. Although water to binder ratio was kept the same as sets A, B and C, the mixes for sets D and E appeared soupy with approximately 20% excess water in the final mixes.

- Set F was the same mix as set D except for having 20% less water . This resulted in a strength increase of 51% after 56 days of curing, compared to sets D and F.

- Although set G had 20% less water than set E, it still appeared to have excessive water in the final mix. This fact might have prevented the development of the optimal strength of set G.

CONCLUSION

The results indicated that (1) : The copper slag had a moderate cementitious property if the replacement of the P.C. with slag was below 40%, and the compressive strength could be increased by increasing the amount of slag/P.C. ratio, (2) : The amount of water in the P.C./slag mix played an important role in developing cementation. Further work, therefore, should be concentrate on determining the optimal water content.

6.3.9: LIME, GYPSUM AND SODIUM SILICATE TESTS, PART 1.

OBJECTIVE: To investigate the effect of addition of lime, gypsum and sodium silicate on the

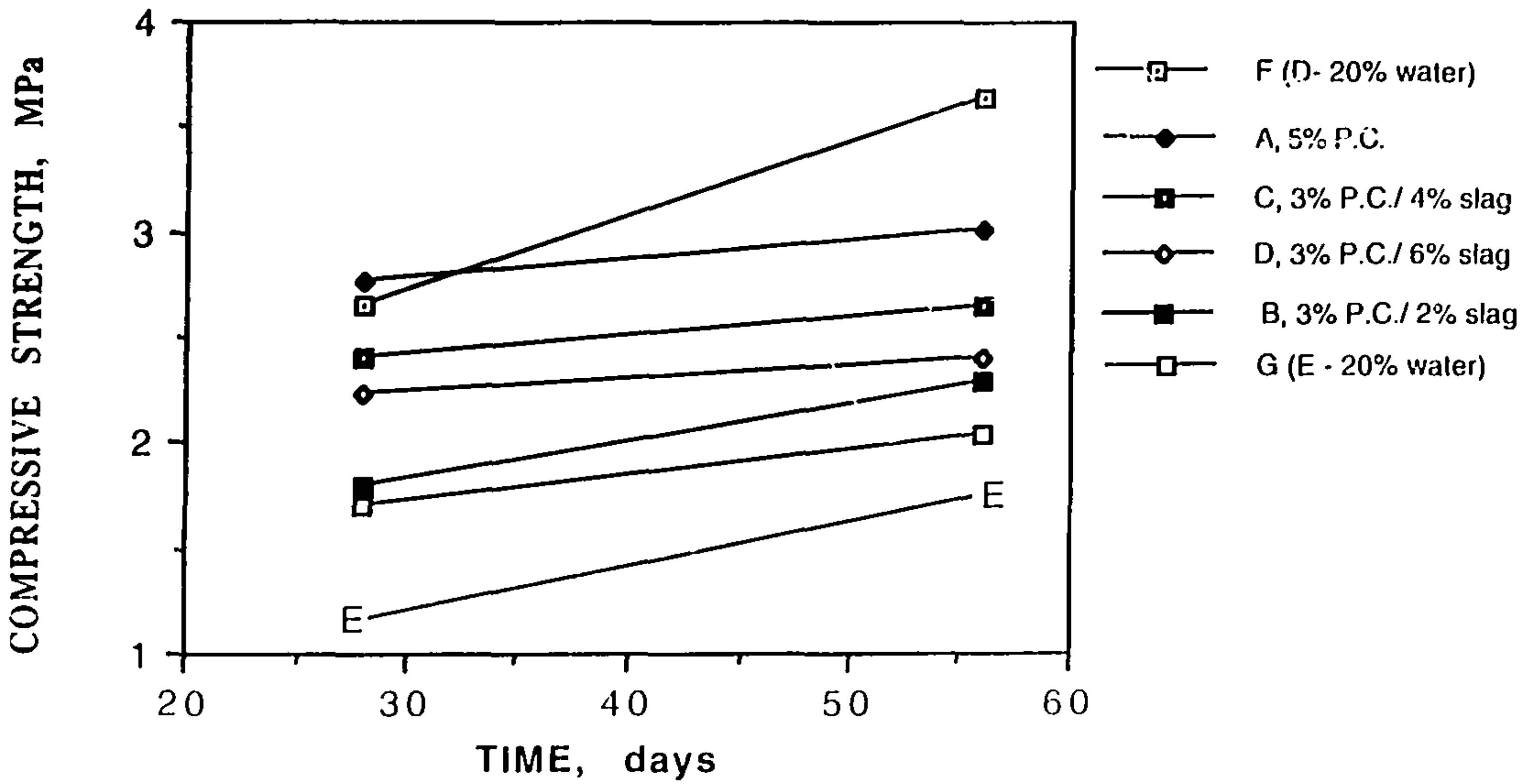


FIGURE 6.3.14: COPPER SLAG HIGHER CONTENT TEST

early strength development of P.C./flyash and P.C./slag mixes. The gypsum samples from Mosse River basin near Moosonee, Ontario, were provided by the Ministry of Natural Resources. Type C flyash was used in this study.

	COMPRESSIVE STRENGTH, MPa	
	28 DAY	56 DAYS
A, 5% P.C.	3.05	3.62
B, 2.5% P.C., 5% SLAG	1.90	1.82
C, 2.5% P.C., 2.5 FLYASH(C)	5.20	6.02
D, SET B+ 1% GYPSUM + 0.65% LIME	2.58	2.95
E, SET B+ 1.5% GYPSUM + 1% LIME	2.47	3.10
F, SET C+ 0.65% GYPSUM + 0.33% LIME	6.05	6.23
G, SET C+ 1% GYPSUM + 0.5% LIME	7.00	7.10
H, SET C+ 5% SODIUM SILICATE	4.45	4.12
I, SET C + 10% SODIUM SILICATE	3.33	4.26

- Set D had a 62% higher U.C.S, unconfined compressive strength, than the control set B. However, increasing the amount of lime & gypsum in set E compared to set D caused only a minor increase in strength, indicating that higher strengths could not be obtained with an additional gypsum and lime in P.C./slag mixes, Fig. 6.3.15

- Set F increased the U.C.S. by 3% compared to control set C. By increasing the amounts of lime and gypsum in the mix, set G, the U.C.S. was increased by 18% compared to control set C, which had a relatively high strength. This means that still there might be a possibility of obtaining a higher strength with additional increase in lime and/or gypsum content in samples containing flyash.

- Addition of sodium silicate to the mix was disappointing. The strength was decreased by 32% in set H and 30% in set I. Both sets H & I indicated that the addition of sodium silicate would not have any positive effect on the strength development. In other applications for which a more rapid increase in strength is needed, say within 2-7 days, the addition of this chemical might be useful.

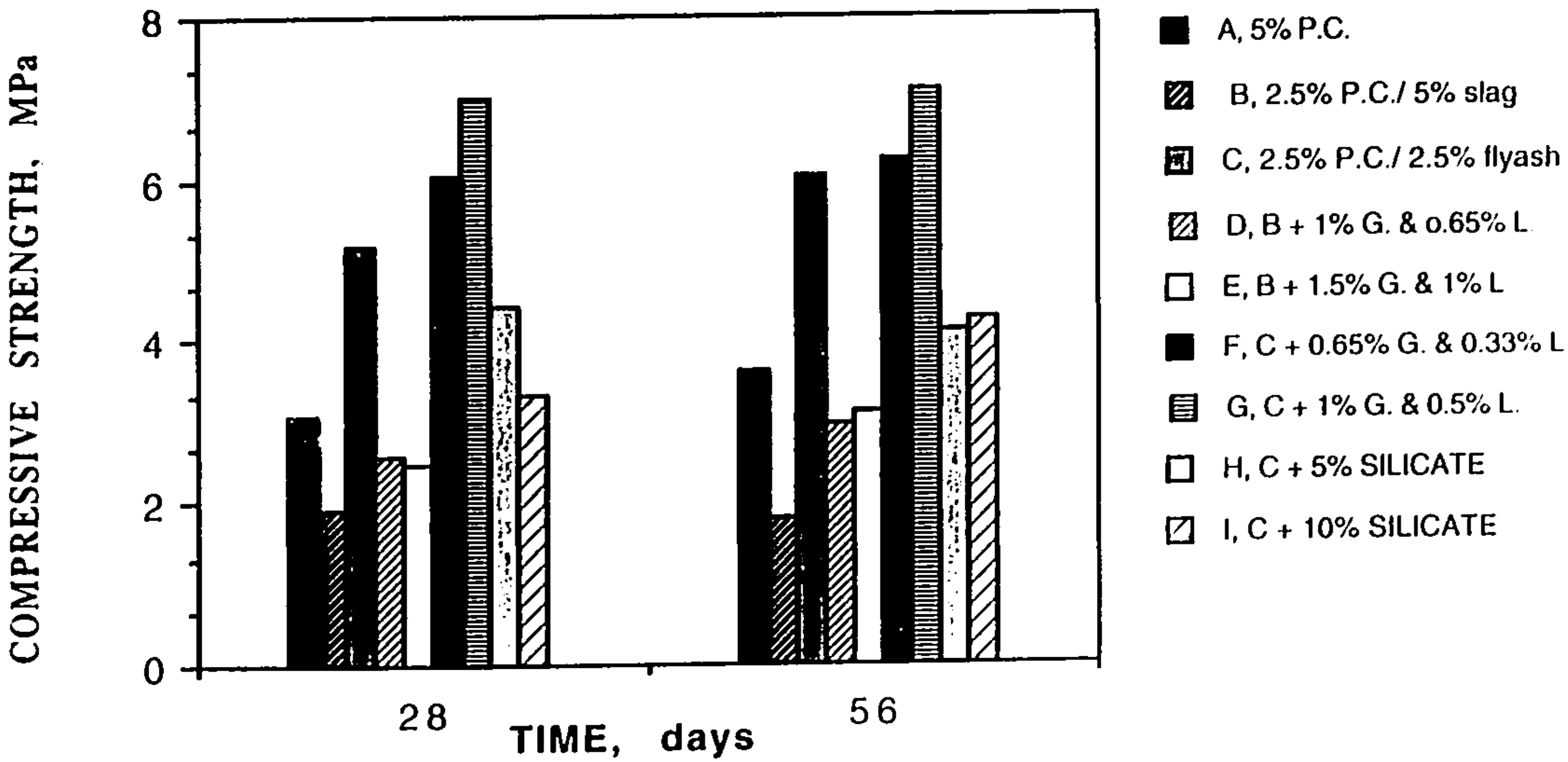


FIGURE 6.3.15 : ANHYDRITE AND GYPSUM ADDITION, PART 1.

- It was noted that the rate of increase in compressive strength from 28 to 56 days was much higher for control sets A, B and C, compared to other sets with the addition of chemicals.

- The test results indicated that the addition of chemicals to the tested binders would be beneficial in increasing the early strength of the fill for a faster mining cycle.

CONCLUSION

The rate of increase in compressive strength was much higher in samples containing KCM copper slag in set E by 70%, compared to control set B. The highest strength increase in samples containing flyash was obtained in set G by 17%, compared to set C. It should, however, be noted that the compressive strengths of the samples containing flyash were much higher compared to the ones with copper slag.

6.3.10: LIME, GYPSUM AND ANHYDRITE TESTS, PART 2.

OBJECTIVE: The same as 6.3.9 with an addition of anhydrite. The anhydrite sample was obtained from Cape Breton area.

SET	**COMPRESSIVE STRENGTH, MPa**	
	28 DAY	**56 DAYS**
A, 5% P.C.	3.15	3.25
B, 2.5% P.C., 2.5% FLYASH (C)	2.78	2.92
C, 2.5% P.C., 5% SLAG	1.45	1.75
D, SET B + 1% GYPSUM + 0.65% LIME	2.96	3.95
E, SET B + 1% GYPSUM	3.50	4.46
F, SET B + 3% GYPSUM	1.78	1.90

G, SET B + 1% ANHYDRITE + 0.65% LIME	4.02	4.93
H, SET B + 1% ANHYDRITE	4.90	4.80
I, SET B + 3% ANHYDRITE	2.13	2.55
J, SET B + 5% ANHYDRITE	2.20	3.15
K, SET C + 1% GYPSUM + 0.65% LIME	2.35	2.95
L, SET C + 1% GYPSUM	2.31	2.90
M, SET C + 3% GYPSUM	2.44	2.60
N, SET C + 1% ANHYDRITE + 0.65% LIME	3.13	4.00
O, SET C + 1% ANHYDRITE	2.70	3.23
P, SET C + 3% ANHYDRITE	3.15	4.15

- Sets D, E, G, H, N and P had all surpassed the strength development of control set A for both 28 and 56 days of curing. These increased strengths were achieved even with partial replacement of Portland cement with flyash and/or copper slag.

- The highest compressive strength was obtained with set G which had 38 and 52% higher strength compared to control set A after 28 and 56 days of curing respectively, Fig. 6.3.16.

- The rate of increase in strength was generally superior to control set A, by 20 to 30%, from 28 to 56 days of curing.

- The only sample which had a constant strength from 28 to 56 days of curing was set H, which had the highest 28 day strength and the second highest 56 day strength .

- Comparison of sets D and E showed that addition of lime decreased the strength by 12% at 56 days of curing, although, it caused an increase of 18% for the 28 day strength, Fig. 6.3.17.

- Comparison of sets E and F showed that the addition of more than 1% gypsum to the mix decreased the strength by 58%. This could be due to lack of enough water for the sample material to completely react, Fig 6.3.17.

- Sets G and H showed that addition of lime to the samples containing anhydrite decreased the 28 day strength by 18% and increased the 56 day strength by just 3%. This indicated that the addition of lime to anhydrite samples had no major effect in strength development.

- Increasing the anhydrite content from 1 to 3 and 5% caused a decrease in compressive strength

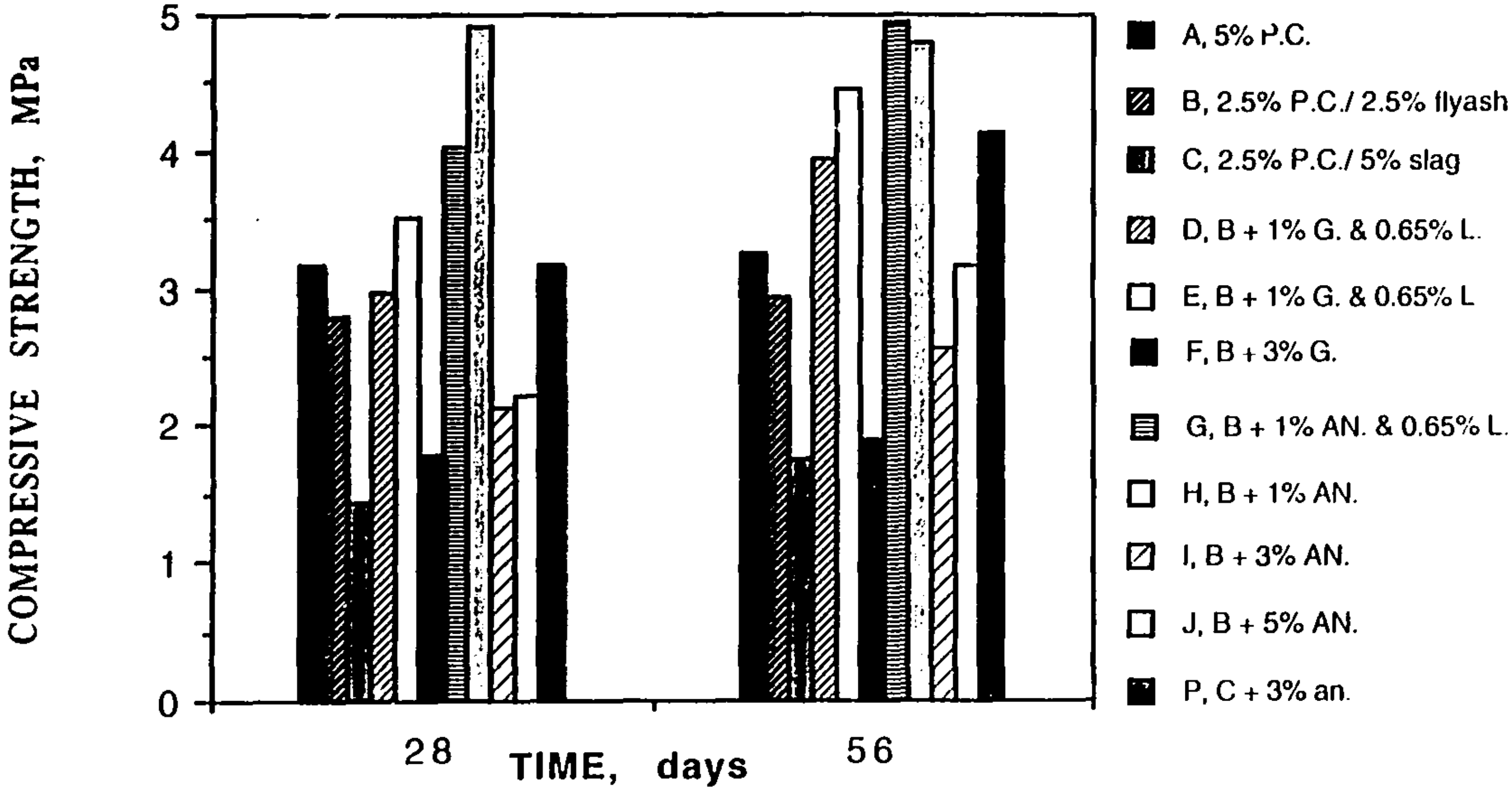


FIGURE 6.3.16 : RESULTS OF FLYASH MIXES

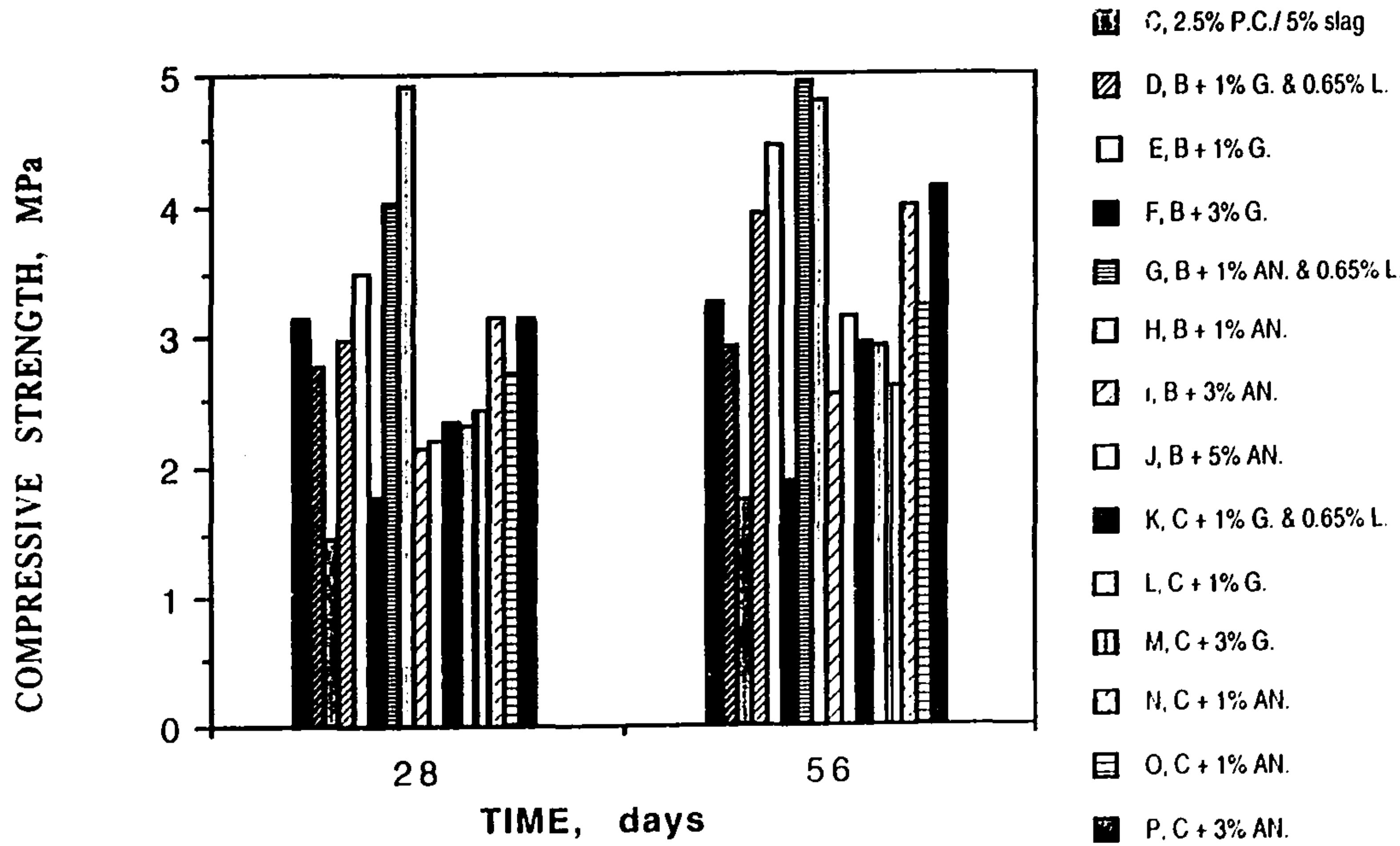


FIGURE 6.3.17 : RESULTS OF FLYASH/SLAG MIXES.

of the samples, by comparing sets H, I and J.

- The addition of anhydrite, gypsum and/or lime to control set C, P.C./slag mix, increased the strength in all mixes.

- The best result was obtained when using 3% anhydrite, set P, which increased the strength of the control set C by 217% and 240% after 28 and 56 days of curing respectively.

- Set N, with just 1% anhydrite and 0.65% lime had almost the same 28 and 56 day strengths as set P, with 3% anhydrite. Addition of lime to PC/slag mix caused a significant increase in the strength of the samples, by comparing sets N and O.

- All the slag/P.C. samples with addition of anhydrite had equal or better 28 and 56 day strengths compared to set A.

CONCLUSION

The results from these tests indicated that a small addition of anhydrite and/or gypsum significantly increased the strength of the backfill mix containing slag and/or flyash, especially with the copper slag mix.

6.3.11: SLURRY DISPERSANT TEST

OBJECTIVE: To study the effect of slurry dispersants on the strength development of cement/ flyash mixes. This test was repeated 3 time, total of 135 cylinders, and the average results are given below:

	COMPRESSIVE STRENGTH, MPa		
	14 Days	28 Days	56 Days
A,60% F.A./40% P.C	4.6	5.1	5.8
B, 60% F.A./40% P.C. + Hydrafil	9.1	9.9	11.6
C, 60% F.A./40% P.C. + Hydrafil - 5% binder	7.1	9.3	10.1

D, 60% F.A./40% P.C. + Hydrafil - **7.5% binder**	6.6	8.1	10.1
E, 60% F.A./40% P.C. + Hydrafil - **10% binder**	6.1	7.8	9.4

Figure 6.3.18 shows the highly sloped curves of the Hydrafil specimens. The control curve on the other hand is flatter and shows us that the curing stage for these cylinders is almost over.

Increase in compressive strength measured as percentage between Controls and test batches are presented below:

COMPRESSIVE STRENGTH, MPa

(Comparison to Controls are in **bold**)

	14 Days		28 Days		56 Days	
A) 60% F.A./40% P.C **(Control)**	4.6	**100%**	5.1	**100%**	5.8	**100%**
B) 60% F.A./40% P.C. + Hydrafil	9.1	**198%**	9.9	**194%**	11.6	**200%**
C) 60% F.A./40% P.C. + Hydrafil - 5% binder	7.1	**154%**	9.3	**182%**	10.1	**175%**
D) 60% F.A./40% P.C+ Hydrafil - 7.5% binder	6.6	**143%**	8.1	**156%**	10.1	**174%**
E) 60% F.A./40% P.C. + Hydrafil - 10% binder	6.1	**133%**	7.8	**153%**	9.4	**162%**

CONCLUSION

As observed in test batches "A" through "C", the percentage increase of compressive strength between 14 day and 56 day cure time has been increasing. This phenomena indicates that while most of the curing has taken place in the control cylinders before 14 days, in batch "E" most of the curing takes place after 14 days. Also, batch "E" Hydrafil - 10% binder continues to cure after 56 days.

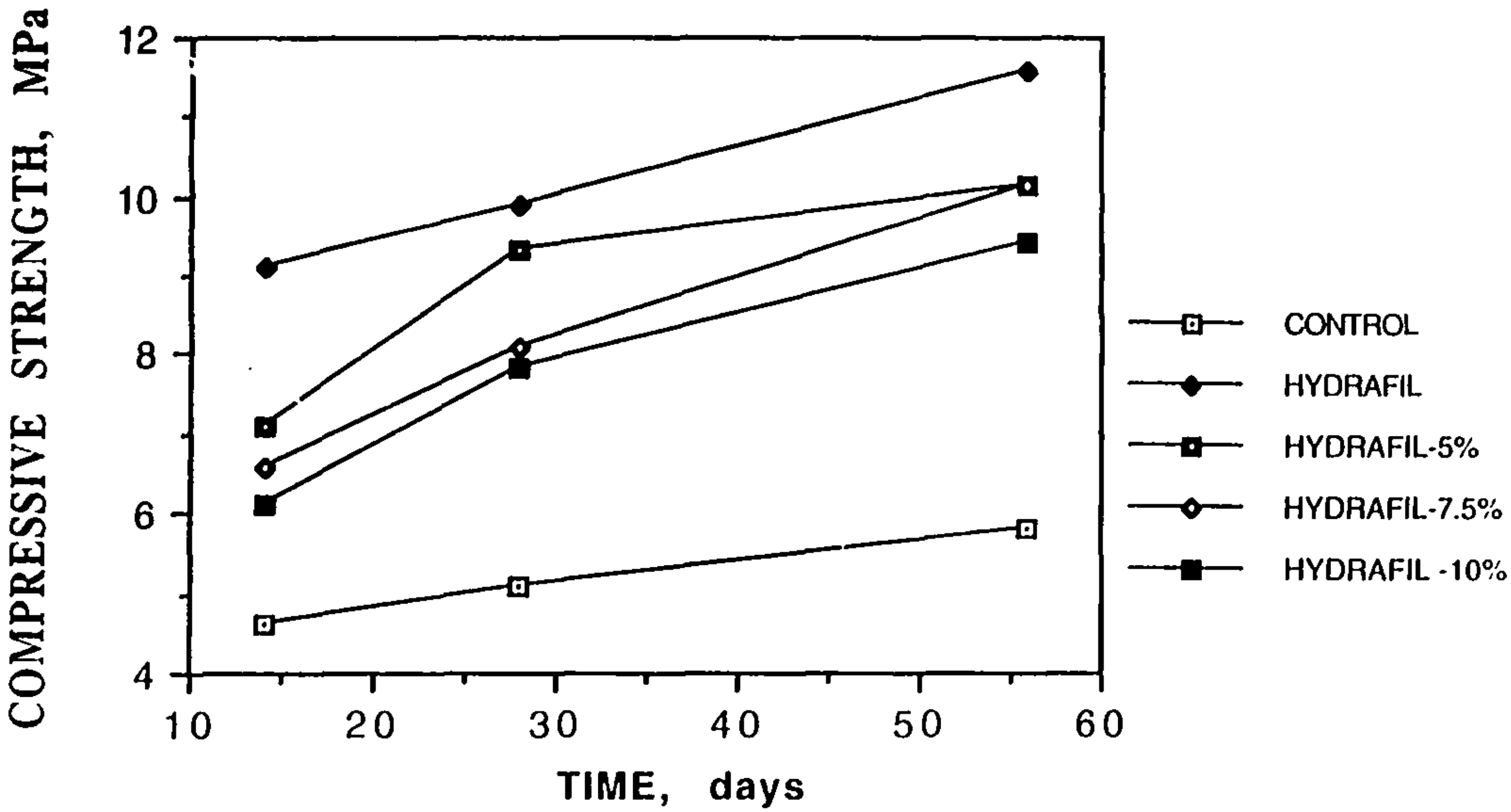


FIGURE 6.3.18: HYDRAFIL ADDITION RESULTS

6.4: LARGE-SCALE TESTING

Final step of implementing any new product underground, is to conduct large scale testing. These are actual rockfill material that are casted in 150 mm by 300 mm cylinders with various amount and types of binder material. Test procedure and photographs of actual testing are presented in this section.These tests are mostly done in Lafarge Canada's Belleville laboratory. Before implementing the use of new binder recipe, refer to section 6.3.11, at KCM, author spent a week at the Lafarge laboratory in Belleville to cast 91 cylinders of the new recipe mix. As it was mentioned before, the actual strength to be expected underground is 86% $\pm$8% of the the results obtained by large-scale test cylinders. The testing procedure is described below:

6.4.1: ROCK PREPARATION

Approximately 20 tonne of aggregate is required for a typical large-scale test . Once the material has been unloaded, it is passed through a 6" sieve or grizzly, Figure 6.4.1. At this point the loader operator tries to blend the coarse and fine fractions, as he passes material through the grizzly.

The material is placed in approximately one foot layers. The loader operator dumps the material close to the ground, and right next to the last dump. This prevents the coarse fraction from rolling off the bucket. Once the initial layer is placed, a second layer is placed, at right angles to the first. Once the material has been blended to the point that the pile is homogeneous, it is moved indoors and stacked following the same procedures as above. It was found that for best results the final sample of rock should be arranged in a pile that is about 15 feet wide and no more than 3 feet high. This width gives a good face to sample while mixing cylinders. The low height of the pile will ensure that the course fractions does not always roll out to the front of the working face. The pile is finally covered with a tarp to maintain the moisture as received.

6.4.2: CYLINDER PREPARATION

Prior to batching the rock material, the batch weight must be determined. A cylinder can be filled with rock to determine an approximate weight of required material. The visible gradation of the rock is often an indicator of the amount of rock required in each batch. A coarse sample normally will weigh about 300kg per cylinder. A well graded rock will require approximately 320 kg. The amount of binder and water used in the mix, will affect the cylinder weights. At 3% the cylinder will weigh about 300 kg and at 5% the cylinder will weigh about 320 kg. Once the amount of rock required has been determined, the weight of cement and water is calculated.

Normally the rock is sampled across the face of the pile, placed into 20 litre steel pails and weighed on the scale, Figure 6.4.2. One batch is placed into the hopper of the mixer and a second batch is prepared and left in the pails.

A portion of the water is poured into the mixer, Figure 6.4.3, through the discharge chute. The binder is added to the mixer while it is running, and the rest of the water is added . The slurry is mixed in the mixer for 2 minutes. The rock is then added to the mixer from the hopper.

While the batch is mixing, the next batch of rock is quickly placed in the hopper. The pails are refilled with another sample. This takes about 5 minutes. The batch that is mixing is discharged into the loader bucket, Figure 6.4.4. Some effort may be required to ensure that all the coarse does not discharge into one area of the bucket.

The loader is driven to the area chosen for casting the cylinders, Figure 6.4.5. A cardboard cylinder 18"x36" is fastened to the plate using four rubber tie down straps. The bucket is raised to the top of the cylinder, and it is kept virtually level. The material is shovelled into the cylinder by hand, Figures 6.4.6, 6.4.7 and 6.4.8. The person casting the cylinder must ensure great care is taken in filling the cylinder. A good blend of the rock fill must be maintained. Normally in filling cylinder the person casting will begin at one side of the bucket and work across the bucket,

Figure 6.4.1: Kidd Creek aggregate pile after screening

Figure 6.4.2: Weighing of aggregate for different mixes.

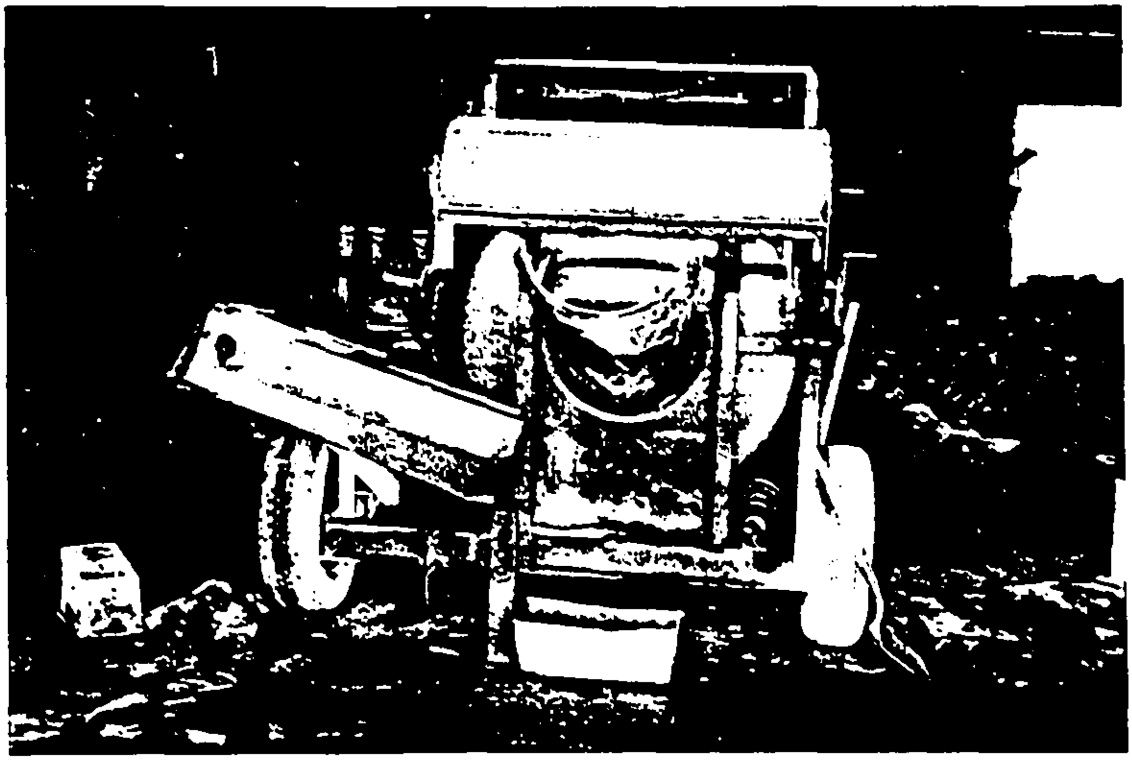


Figure 6.4.3: Concrete mixer used for the test.

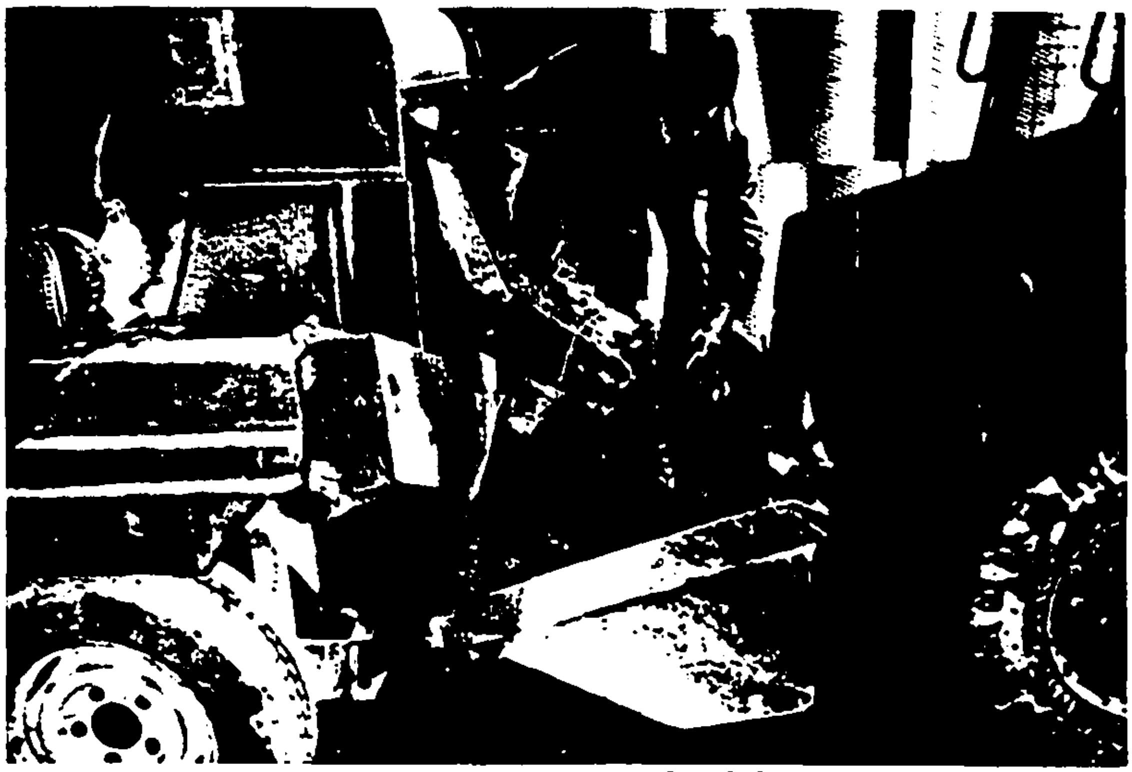


Figure 6.4.4: CRF after 5 minutes of mixing.

Figure 6.4.5: Mixed CRF ready to be casted.

Figure 6.4.6: Filling of the cylinder with final mix.

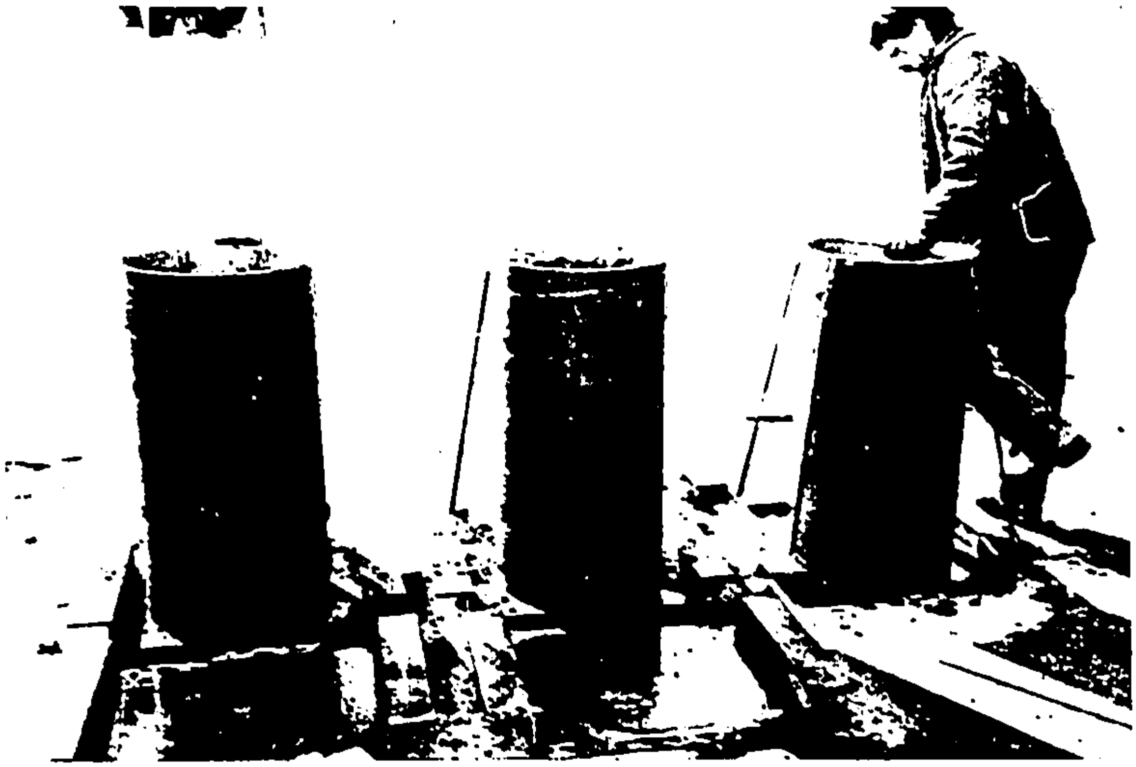


Figure 6.4.7: 3 cylinders per set are prepared.

Figure 6.4.8: Cylinder before capping.

taking the full depth, until reaching the other end.

The cylinders are weighed, covered with a tight fitting plastic bag, and taken to the curing area.

6.4.3: CYLINDER CAPPING

The cylinders are capped as soon as possible. The cap should be given enough time to cure to a strength that exceeds the waste rock.

The capping material is Portland cement mortar. Normally a mix of 1:3, cement/sand, is used, but sometimes a mix of 1:2 has been used. A high cement content gives a very smooth surface. The mortar should not be too wet otherwise shrinkage will occur.

The mortar is placed on top of the cylinder, and smoothed with a trowel. The 1/2" plexi glass capping plate is placed on top of the mortar. Care must be taken to apply the pressure evenly. The application of pressure at the edge will result in a convex cap. When the surface is level and plane, weights (bricks) are placed on top of the capping plate, and the cap is left to cure for 24 hours. On the following day, the capping plate is removed and cleaned.

6.4.4: UNIAXIAL COMPRESSION TESTING

At the desired curing time, the cylinders are removed from the curing room. This room had constant tempreature and humidity conditions, such as 25 deg. celcius temperature. The cardboard cylinder mould is removed and the cylinder is taken for uniaxial compression testing, Figure 6.4.9. The plate and the cylinder are placed into the frame of the compression machine using the forklift.

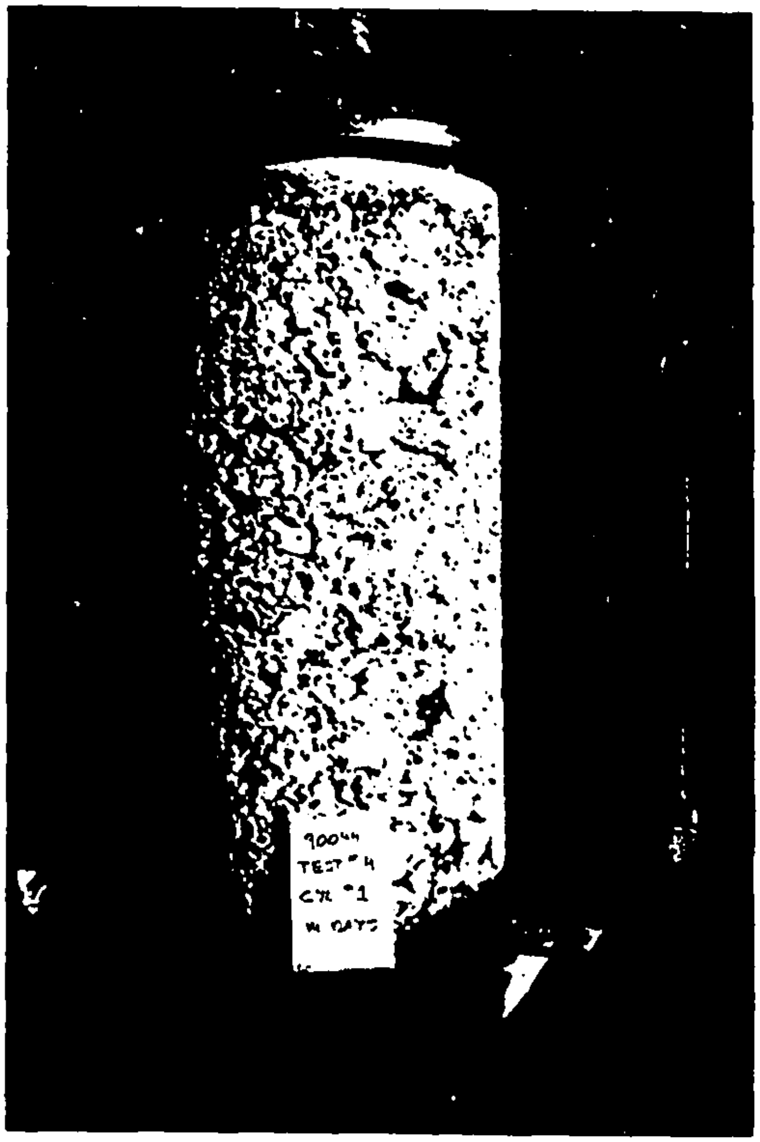


Figure 6.4.9: Compressive strength testing on final sample.

The cylinder and plate are directly centred under the upper platen. The cylinder is normally loaded at 10 KN/sec. This rate was kept constant for all samples. After failure the cylinder is photographed and results are recorded. The cylinder is removed from the lower plate, the plate is cleaned and oiled and the cylinder is discarded.

6.4.5: SIEVE ANALYSIS

As rock is taken from the pile for mixing, samples for the sieve analysis and hardness tests are taken at several locations throughout the pile. The sample size is approximately 2 - 20 litre pails each time.

A sieve analysis is completed on the sample. The sieve sizes used are as follows: 6", 3", 2.5", 2", 1.5", 1", 3/4", 1/2", 3/8", #4, #8. Occasionally the fines are split and a further analysis of the fines is completed.

6.5: SUMMARY

Using the results of small and large scale test programs, the KCM has substantially reduced the cost of cement rockfill over the past 4 years. This was done by both, substituting lower cost binders, and reducing the overall binder content by utilizing the higher strengths of the new binder mixes. At KCM, the old recipe of 60% Portland cement and 40% flyash at 5% binder by weight was changed to a new recipe of 40% Portland cement and 60% type C flyash at 4.5% binder. This was accomplished with no sacrifice in strength due to the hydrafil addition, section 6.3.11. The new recipe has resulted in 10% reduction in binder consumption and annual savings of $1.3 million on binder cost alone.

In addition, a locally available material, slag from the Kidd copper smelter was identified as having potential for possible future use as a binder substitute. Presently 3 different groups are doing further tests on KCM copper.

The addition of anhydrite, gypsum and/or lime to slag/P.C. and flyash/P.C. control mixes increased the compressive strength in almost every combination tested. The best result was obtained when 3% anhydrite was added to slag/P.C. control mix which caused strengths increase of 217% and 240% after 28 and 56 days of curing, respectively. The same strength increase was achieved when 1% anhydrite and 0.65% lime were added to the slag/P.C. control mix.

7:0: QUALITY CONTROL

A typical cemented rockfill mass has much superior physical and mechanical properties compared to other fill systems only when properly controlled. Closely monitored and properly engineered quality control measures have to be established and followed by operations people. These are the measures taken to achieve the optimum fill strength at the lowest possible cost. Chapter 6 indicated that with proper structural design one could expect high strength CRF mass where required, however, to have an acceptable fill product, the qualities of aggregate and binder materials have to be maintained. In addition to many other operational factors it is also equally important that the quantities of the materials used in the fill system should be held according to the design criteria. Consolidated rockfill attains the optimum strength when a properly blended aggregates which are well coated with binder material are placed in the stope.

In this chapter the quality control measures are established for three main stages in a typical rockfill system; 1) At the backfill plant 2) During transportation and 3) During placement underground.

7.1: QUALITY CONTROL AT THE FILL PLANT

Fill plant, same as concrete plant, should provide properly sized aggregates, by precise weight or volume, to be placed underground. The placed aggregate and slurry mixture should provide an adequate strength needed for the fill to provide the required support.

Most of the plants have a combination of different electronic and mechanical hardware to monitor and produce the required backfill materials. All the equipments such as pumps, valves, flow meters, density meters and binder measuring devices should closely meet with their designed requirements. The emphasis should be placed on weighting the right amounts of cementing agents, and obtaining the proper pulp densities for the slurries used underground.

To be able to produce a good quality fill mixture on surface all the devices for monitoring material consumption, rates of transfer, storage quantities, etc., should be accurately maintained, and the calibration checks on the devices should be carried out at a specified interval. If regular checks are not carried out the deviation of the designed quantity can easily occur without being noticed. For CRF, any extra aggregate or insufficient amount of binders can cause shortage of cement slurry to coat each solid particle, resulting in weak fill structure. On the contrary, an increased binder content or lower proportion of aggregate may produce a high strength fill but with higher cost.

Clean aggregate has to be used for filling to allow the cement slurry properly coating and sticking to the aggregate for consolidation purposes. Dirty aggregate which could contain: clay, oil or chemicals, prevents proper coating of the aggregate and delays the curing process.

Frozen aggregate could also adversely effect the strength of the final CRF mass, by introducing excess water into the system. To prevent this problem at KCM, liquid calcium at a rate of 0.8% by weight of the binder is sprayed on the aggregate before entering underground raises. This has completely eliminated the past problems with frozen aggregate during the winter months.

Another important factor for obtaining good quality fill is the quality of the water used in cement and/ or sand slurries underground. The same criteria of mixing water required for the preparation of concrete should be met in producing the fill. An example of failures experienced at Kidd Creek was related to the use of recycled underground waste water for mixing the backfill slurry. Upon close inspection of the failed backfill, it was observed that the slurry coating of the aggregate was still "green" and had not properly cured. Several possible causes for the lack of proper curing were examined, including possible separation of flyash from the backfill slurry mix, the quality and type of flyash being used, the ambient temperature effect on curing at the time of backfill placement, and the quality of the water used in the slurry.

After a comprehensive investigation, it was believed that the the quality of the water used was the most feasible explanation for the low strength of the fill. Test results indicated that the cylinders containing 60 % F.A. / 40 % P.C. binder, which were cast using underground recycled water had lost about 50 % of their compressive strength compared to the ones using drinking water. (Henning, 1988)

Test results also indicated that the samples using underground recycled water were much less cohesive and bled more, especially in the 60 % F.A. / 40 % P.C. binder mix, than cylinders cast with potable water. The report also suggested that the hydration of P.C. and F.A. was hindered by the recycled water, with the result that water intended for hydration was released as bleed water. Hydration of P.C. with the resulting release of calcium hydroxide is required to activate the cementitious properties of the flyash. A reduction in the degree of the P.C. hydration would therefore be more pronounced in the 60 % F.A. / 40 % P.C. blend than in the 100% P.C. mix. Contaminants found in the underground recvcled water which might have caused the reduction of the backfill strength included :

a) High concentrations of dissolved solids
b) Buildups of oil and grease.
c) Buildups of water treatment chemicals added to water.

The most important step of surface quality control is to conduct regular sampling of the final products. The quality of the water and binders, especially lower cost binders, could vary significantly and continuous small scale cylinder testing is required to verify and detect any longterm deviation in quantity and quality of the fill materials prepared by the fill plant.

When consolidated rockfill is used as the filling material, excessive fines created by the attrition of aggregates in a long raise will reduce the fill strength. Attrition would cause a great deviation from the original size of the aggregate on surface. The effect of attrition for different levels of the mine should be estimated and proper size aggregates for each level, if practical, should be produced on surface. The properly sized aggregate would produce an optimum fill density underground. To control the attrition effect, the initial aggregate size could be larger than the required aggregate size when being placed. The more common method for decreasing the adverse effect of attrition is to reduce the amount of fine fraction by screening in the surface plant. Adequate blending of coarse, midds and fine material in the backfill aggregate would translate to higher placed density of the fill, less voids in the fill and lower cement consumption.

The cementing agents should not be stored in bins or hoppers for a long period of time and also in a moist environmental conditions due to the moisture and time consolidation effects. The aggregate storage facilities should be closely controlled, and any possibility of adding water to the aggregate in raises should be avoided. Excess water in the slurry or aggregate will wash the cement paste and the cement coating off the aggregate, and flush it toward the lowest zone of the stope. This causes dilution of cement content in the fill pile resulting in a more heterogeneous fill mass. The control and prevention of such segregation would make a strong fill near the stope perimeter, which is desirable during pillar recovery.

A slight change of moisture content in the aggregate may affect the fill quality significantly. To prepare a normal batch of CRF at Kidd Creek, the amount of mixing water requires only 3.9 % by weight of aggregate. An increase of 1 % moisture content in aggregate results in an excess of 25% mixing water. Close observations of fill piles during stope filling, therefore, are essential to the adjustment of the pulp density of cement slurry for a proper mixing of fill materials.

7.2: QUALITY CONTROL MEASURES DURING TRANSPORTATION

The main quality control method related to conveyor transportation is to closely observe the sizing of the aggregates on belt by the underground operators . This is to see if the proper size aggregate, no excessive fine, is being placed in the stope. It also enables the operator to act rapidly in response to required process changes for example, having an extra batch of slurry if the material on the belt is too fine. Also, the weightometres and speedometers in connection with belts should be regularly calibrated.

Some of the factors effecting the transportation of aggregate underground are as follow :

7.2.1: ATTRITION

As mining progresses deeper it has become quite apparent that the aggregate used in the backfill suffers greatly with depth. In greater depth, excessive fines created by the attrition of particles passing through longer raises are found in the aggregate, and additional binder is required to coat the extra fines. The result of an improperly sized aggregate in the backfill becomes very costly, for example, each increment of 1% binder used at KCM increases the annual binder cost by \$1.7 million.

Crushing and blending should be utilized to produce a quality, graded product, suited to requirements at a certain depth. The attrition of aggregate in a fill raise was observed at KCM, and the results, in terms of the attrition ratio can be expressed by Eq.(1), (Bronkhorst, 1986).

$$\textbf{D50 at surface / D50 at } h = 1 + h/1100 \qquad (1)$$

where D50 = aggregate size corresponding to 50% passing & h= aggregate vertical travel distance (m).

By using the formula, the relative size of aggregate in terms of D50 at any depth can be found. For example, the attrition ratio at h = 900 m can be found from Eq. 1 to be 1.8. If the D50 of aggregate = 8 cm at surface, the D50 of degraded size after passing through a 900 m raise is 4.5 cm.

7.2.2: MOISTURE CONTENT

If the aggregate is either sent underground wet or becomes wet on route, the fines will coat the coarser particles, preventing bonding of aggregate from cement paste. Wet aggregate is caused by one or more of the following situations:

1. Ice crystals or snow mixed with the aggregate.
2. The temperature of the air occupying the voids passes its dew point, resulting in condensation on the aggregate.
3. Seepage into backfill raises.

At KCM during the winter season it was observed that poor coating and delayed initial curing were caused by frozen aggregate. To alleviate this situation, liquid calcium chloride is sprayed on the aggregate, at a rate of 0.8 % cement by weight, to lower the freezing point of the aggregate by 12 deg. celcius, and to provide additional heat for curing.

Constant monitoring of excess water seeping into backfill raises should be carried out by operation personal to try to get the aggregate to the final destination as dry as possible.

The transport of binder material from surface to underground is by means of hydraulic, transportation. The main quality control measure on the hydraulic transportation is maintaining proper pulp densities for cement slurry and keeping the pulp density as high as practical. Extent of the horizontal lines in the mines dictate the practical pulp density for slurry transportation.

Trucks and/or scooptrams are used to transport the development waste and/or sized fill materials from the fill raises directly into the stopes. Generally, development waste consists of coarse material used as fill aggregate. Blending of waste with fines, therefore, may be necessary.

If the material is transported through raises and is hauled by truck or scooptram, the amounts of fine in the aggregate should be closely monitored. If the material has a high fine content, differential settling problem could occur in the truck or scoop bucket.

Another aspect to consider with truck transportation is to make sure that the operators load the truck with the same amount of aggregate for each slurry batch. Since the only measuring quantity when filling the truck is by the volume of the box, obtaining a consistent aggregate load is essential.

7.3: QUALITY CONTROL IN PLACEMENT OF THE FILL

Two methods of filling a stope with rockfill are 1) To fill the stope with unconsolidated fill materials and then consolidate the outer edges with cement sand slurry for future pillar recovery , and 2) To pour the cement slurry on the rockfill material in a mixing culvert as it leaves the conveyor belt before entering the stope. The first method is suitable when a high degree of segregation has occurred, and where by consolidating the coarse particles at the walls of the stope, ore dilution can be minimized. The second method results in a much more competent and uniformly distributed backfill. The use of the second method at KCM has allowed the mine to drift into and drive raises through consolidated rockfill (Wittchen et al., 1989).

Some of the factors effecting the placement of backfill underground are as follow :

7.3.1: AGGREGATE AND BINDER SEGREGATION

Segregation of consolidated rockfill during backfilling is unavoidable, because the flow of fill down any stope is subject to differential settling. The degree of segregation is governed by the fill raise orientation and the opening geometry to be filled, and will differ for each stope as stopes are neither identical nor possess the same backfill raise orientations. It was shown in chapter 4 that a zone of fine aggregate tends to occur near the impact area, by consuming most of the cement paste and leaving a low cement content rockfill at the perimeter of the fill cone.

The segregation phenomena becomes more pronounced when stopes are filled by conveyors due to the impact velocity caused by the speed of the belt and the free fall. When a stope is filled by mobile vehicles, only the largest particles have the momentum to travel to the further stope wall. The rest of the material fill the stope by progressive slumping resulting in a more uniform product. The main factors which effect the extent of the segregation in stope are stope geometry, aggregate size, filling system (conveyor, truck), the orientation and dimensions of the fill raises. One of the best ways to control segregation is to have fill raises, collared 2-3 feet apart, directing fill to different parts of the stope and alternating fill through each raise on shift by shift basis. Refer to Figures, 5.3.4, 5.3.5 and 5.3.6.. This arrangement will direct the coarse aggregate to the the center of the stopes and not to the walls that will be mined against in future pillar recoveries. Detailed segregation control for different fill set ups is explained in chapter 5.

7.3.2: BACKFILL RAISE

The fill raise constitutes another important factor to be considered when filling a stope. The raise should be strategically located and oriented so that there is a uniform distribution of fill material. The choice of stope geometry could be such as to minimize the segregation phenomena by preventing the development of steep fill cones while filling the stopes. This is not followed in majority of the cases since alteration of the shape of the stope just for the purposes of filling is not a common practice. The general practice for filling an open stope involves using a single fill raise which might not be adequate when a larger stope is being filled. In a larger stope a second pour point should be available to minimize the rolling distance of the coarser aggregates toward the walls of the stope. The effects of stope geometry and fill raise orientation on the quality of the placed fill were discussed in chapter 5.

The orientation of the raise determines the location of the fill cone in the stope to be filled.

When material is in the fill raise, it attains a specific falling velocity which governs the trajectory of the fill into the stope. The trajectory of the fill into the stope can be predicted using the motion of projectile, Figure 7.3.1.

The material in the raise, however, encounters some frictional resistance, deviating from a freely falling body. It has been observed that in determining the initial velocity leaving the raise, a constant (K=0.42 for KCM) should be incorporated in the calculation of the falling velocity in the raise. The equations are illustrated below, together with an example to compute the horizontal trajectory of the fill in a stope.

For a given dip angle and length of fill raise above a stope to be filled, the horizontal trajectory, xh, of the fill material can be found from the following:

$Xh = Vh \times T$ & $Vh = Vv / \tan D = (2g \times L \sin D \times K)^{0.5} / \tan D$

$T = [-Vv + (Vv^2 + 19.6\ H)^{0.5}] / 9.8$

where:

Vh and Vv = horizontal and vertical velocity components respectively when discharging fill into stope, m/sec

L= length of the fill raise, m

D= dip angle of the fill raise, degree

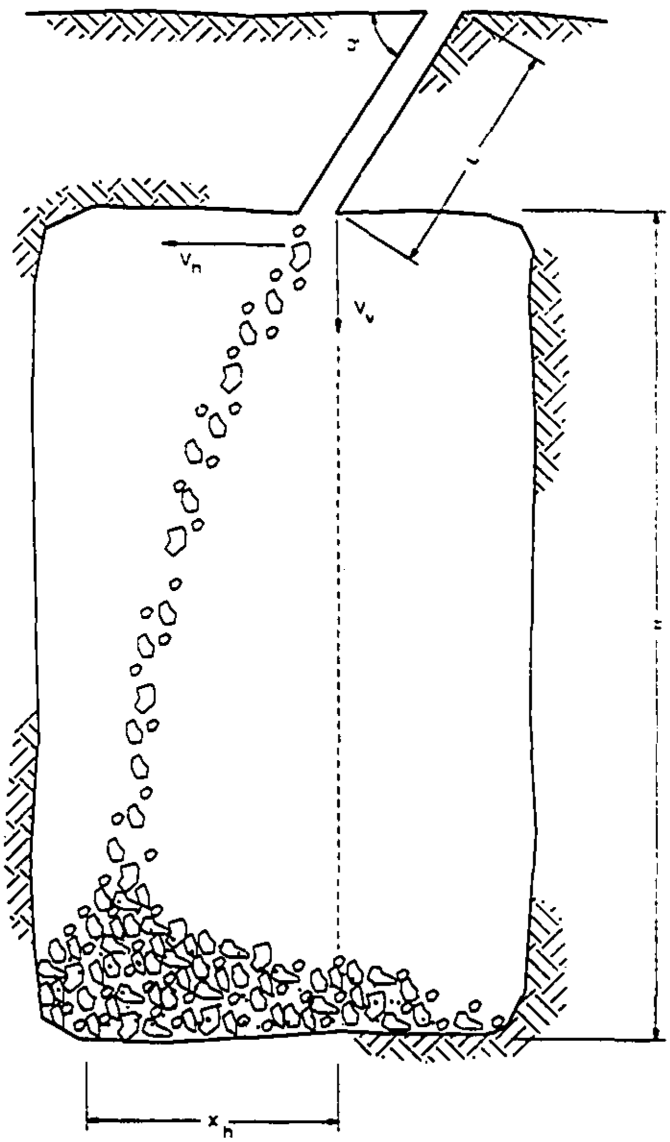
K= 0.42, resistance constant

g= gravitational acceleration, 9.8 m/sec2

T= time taken for the fill material to fall from the end of the raise to reach the top of the fill cone, sec

H= height of the free fall of aggregate in stope, m

For example, in a fill raise dipping at 60 deg. with an inclined length of 35 m, the



(Yu 1989)

Fig: 7.3.1 Fill Trajectory

horizontal velocity at the end of the raise, Vh, is equal to 9.1 m/sec. If the free falling height is 60 m in the stope, the falling time, T, is 2.24 sec. Thus the horizontal trajectory can be found to be 20.4 m.

In chapter 5, after a comprehensive study on the number of raises to be required, it was concluded that the use of two raises in an opening 18 m wide by 45 m long would be satisfactory without major segregation problem. Most of the mines with rockfill utilize single fill pour point, as the cost for having the second conveyor and fill raise are high. In a larger stope, a second pour point should be available to minimize the rolling distance of the coarser aggregates toward the walls of the stope.

7.3.3: MIXING

The key for producing a competent consolidated fill is to thoroughly coating all the aggregate with the supplied amount of cement slurry. If the material is not coated with slurry during the mixing process, it may never be properly coated since: (1) It is impossible to control the flow of slurry in the stope, (2) The slurry does not flow uniformly over the entire backfill cone, (3) The percolation rate of the slurry is variable due to differential settling, and (4) Slurry which is not actually used to coat the aggregate acts as a void filler. Since there is insufficient cement to fill all voids, some portion of the fill may remain unconsolidated.

Prior to starting the backfill operations at KCM, an extensive study on this subject was made, including full scale testing of a vibratory mixing conveyor ; the use of a slusher for mixing ; a baffled mixing slide, figure 4.2.2 ; and a drum mixer. The simplest system found to date consists of a baffled slide or chute, a spray header for the slurry which is pumped from the holding tank, and the conveyor carrying the aggregate.

In KCM the conveyor discharges the aggregate into a 1.2 m diameter steel culvert (slide), 2 to 3 m long, which is typically equipped with three baffles set at 45 degrees to the axis of the culvert , Figure 7.3.2. The slurry is sprayed on the aggregate as it enters the culvert. The tumbling action of the aggregate as it passes from baffle to baffle ensures that the aggregate receives a good coating of slurry. The mixed fill then falls through a 0.7 m diameter bored raise into the opening being filled. The dip of the raise is typically greater than 55 degrees. When an opening is directly accessible from a fill level, a mixing chute, 1 m wide by 4 m long, replaces the culvert. In this case the use of the slurry header and baffles is the same as in the culvert. The chute can be advanced with the conveyor out onto the fresh fill to tight fill the stopes. Mixed fill can also be hauled by teletram or scooptram when the tonnage of fill required to fill a location does not justify the expense of installing a conveyor.

In lower part of KCM, aggregate is passed to fill stations below the 2600 level. At the backfill station, 6.4 tonne Jarco dumping trucks are loaded from a feeder. The cementing agent is sprayed on the aggregate and hauled to the desired stope. The fill is poured from a drift access directly into the stope, and coating of the aggregate by the slurry takes place during dumping and rolling of the material in the stope. Conveyors are occasionally used to transport aggregate to some of the larger stopes.

In the earlier backfilling stages in KCM the aggregate from the raise used to pass into a 4.5 m3 metering pocket equipped with guillotine gates. The aggregate was then dumped from the pocket into a pivoting tipple chute, capable of feeding either of two 5 m3 redi-mix concrete mixer drums. Mounted discharge end to discharge end, the mixers operated singly or in tandem. Slurry was pumped to a measuring hopper equipped with an overflow outlet. When the hopper was full, small quantities of slurry would overflow into either mixer, signalling the station operator to shut off the slurry pump.The full 'shot' of slurry was fed by gravity through a 10 cm diameter pipe, lying within the tipple chute, into the mixer. The slurry and aggregate were then mixed until all of the aggregate was coated. The entire cycle took approximately 2 minutes. Low-profile trucks, 6.4 tonne capacity, would haul the fill to the stope, where it was dumped from a drill drift directly in the stope. An average circuit time from the mixing station to the dump point and back was about 7 minutes. As it took 1 minute to discharge the mixer, the total cycle time was around 8 minutes and four trucks could operate very efficiently with each station. Each station could prepare

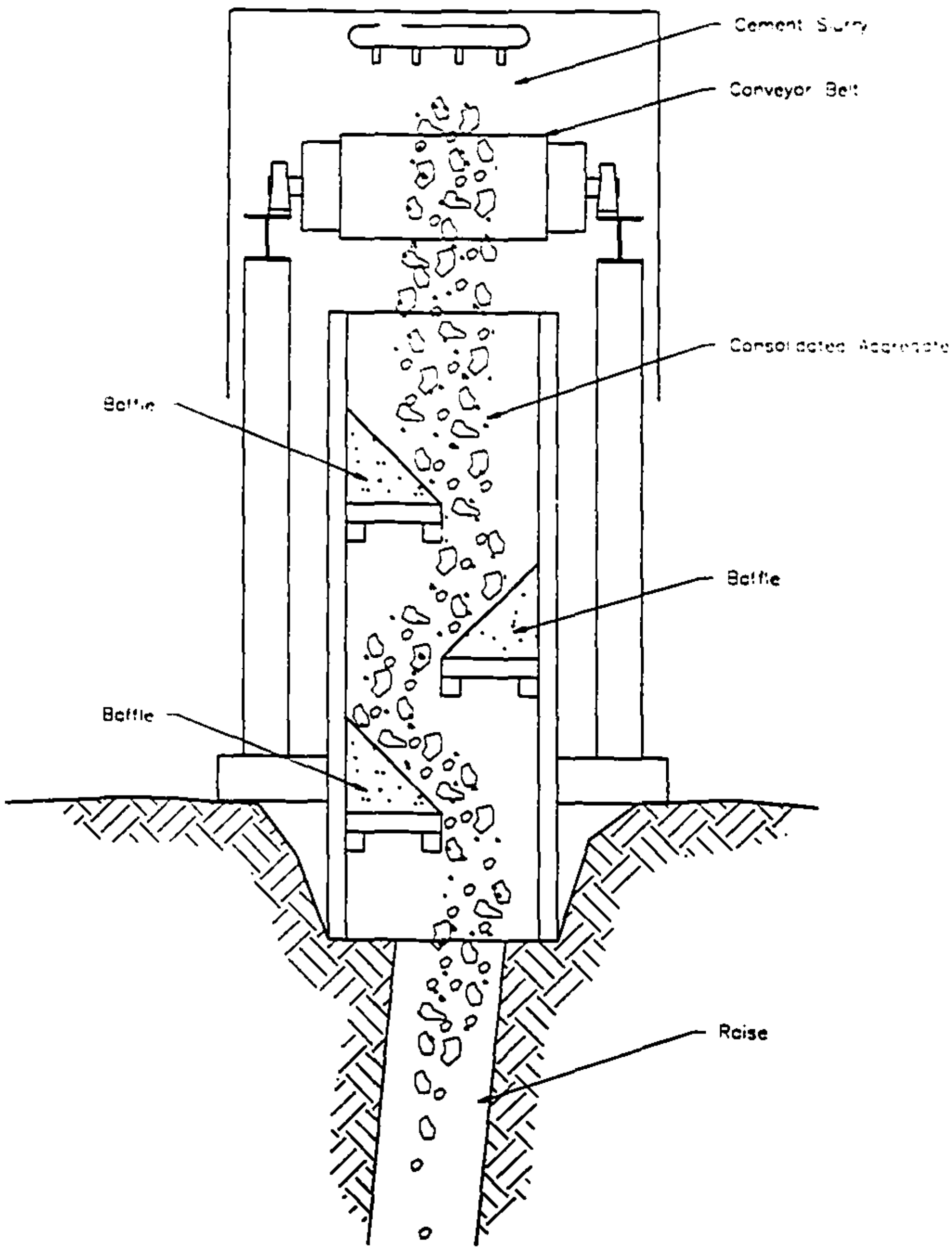


Fig: 7.3.2 Baffled Slide for Mixing of Aggregate with Cement Slurry approximately 3000 tonnes of backfill per day. The stations were also equipped with an aggregate bypass finger raise and slurry discharge pipe to allow unmixed fill or development waste to be placed in the event that both mixers were inoperable. The mixing process was regarded to be one of the best methods to provide a well coated fill, but was discontinued later as the demand of a greater quantity of backfill was required.

7.3.4: IMPACT DAMAGE

The impact damage is another important yet easily overlooked factor governing the fill stability. The impact damage can be divided into two categories:

1: When the backfill aggregate collides with the peak of the backfill in the stope, some of the aggregate will break. Should there be insufficient slurry present, the fresh surfaces may never be coated.

2: Should a collision of aggregate and cemented backfill occur after the final curing period, any broken bonds below the placed fill will not be recoated and thus a zone of broken bonds will exist.

To reduce the extent of impact damage, addition of a retarding additive in the cement slurry is necessary. At KCM an admixture was introduced at a ratio of 90 cc retarder per 45 kg Portland cement to delay the final set time so that a minimum of 60 cm thick buffer layer of consolidated fill was created. This allowed the shock of dumped fill to be absorbed by the plastic state surface layer preventing impact damage to the cured fill beneath. Another advantage of adding retarding agent to cement slurry was found to significantly reduce the rate of the cement scale build up in the slurry pipelines.

The retarding admixture is not needed if slag and/or flyash is used as a partial replacement for Portland cement, as the blended binder exhibits a much slower curing rate compared to the binder of Portland cement alone.

7.3.5: SUMMARY

The major quality control problem when filling with consolidated rockfill is the segregation phenomena during placement, which has been explained earlier. The segregation could be minimzed by having a proper engineered design for individual stopes. The next key step in maintaining a good quality backfill would be minimizing the water content of the mixes by having the highest possible pulp densities for cement slurry, around 60%.

Close observations of fill piles during filling are necessary for quality control. Daily inspection should be carried out to monitor the fill quality. As there is generally very limited access to evaluate the progressive filling of any stope, all possible accesses should be utilized, e.g., by having windows in all bulkheads above the floor of the stope. The daily records of the tonnages of the fill materials placed underground should be reviewed and corrected, if necessary.

To minimize blast damage to adjacent fill material, care must be taken to avoid blasting in the fill, by leaving a thin skin of ore when mining an adjacent stope, by preventing production holes intersecting the fill, and by stemming the holes should an intersection occur.

8: IN SITU EXPERIMENTS

As shown in chapter 4, consolidated rockfill exhibits considerable heterogeneity in physical and mechanical properties, Figure 8.1. In situ tests, therefore, are required to supplement measured results from laboratory tests. The following tests were carried out at KCM : 1) Measurement of stress change in pillars, adjacent to a filled stope, 2) Measurement of stope wall deformation around a filled stope, 3) Determination of blast vibration resistance characteristics of CRF, and 4) Pressuremeter and point load testing. This chapter also refers to some of the past studies at KCM

8.1: STRESS CHANGE AND STOPE CLOSURE MONITORING

The primary 40-641-ST at KCM was selected to carry out a monitoring program using extensometer and Irad gauges. Monitoring of this stope started after it was completely mined out. The adjacent stope, 40-631-ST, was also monitored while being mined and filled. The pillar was instrumented to determine wall closure, rock movement and seismic characteristics of the rock and consolidated rockfill.

Figures 8.1.1 to 8.1.3 show the stope layouts and instrument locations. Two Irad gauges (I-24 and I-25) were used to measure the horizontal rock stress changes. The Irad stressmeter installed into a 3.7 cm diameter diamond drill hole was used to measure stress change in the surrounding rock as a function of borehole deformation. Stress changes in stope walls were measured using two borehole extensometers, one at each side of 40-641 (GE-13) and 40-631(GE-14) stopes. A third extensometer to measure rock and fill movement could not be installed due to poor CRF strength. For exact orientation and anchor points of the Extensometers refer to Appendix A.

There are three main parts for a stress measuring device. These elements are, vibrating wire

Figure 8.1: HETEROGENEITY IN CRF, WITHIN ONLY 3 METERS OF 202L-L-ST ROADHEADER DRIFT.

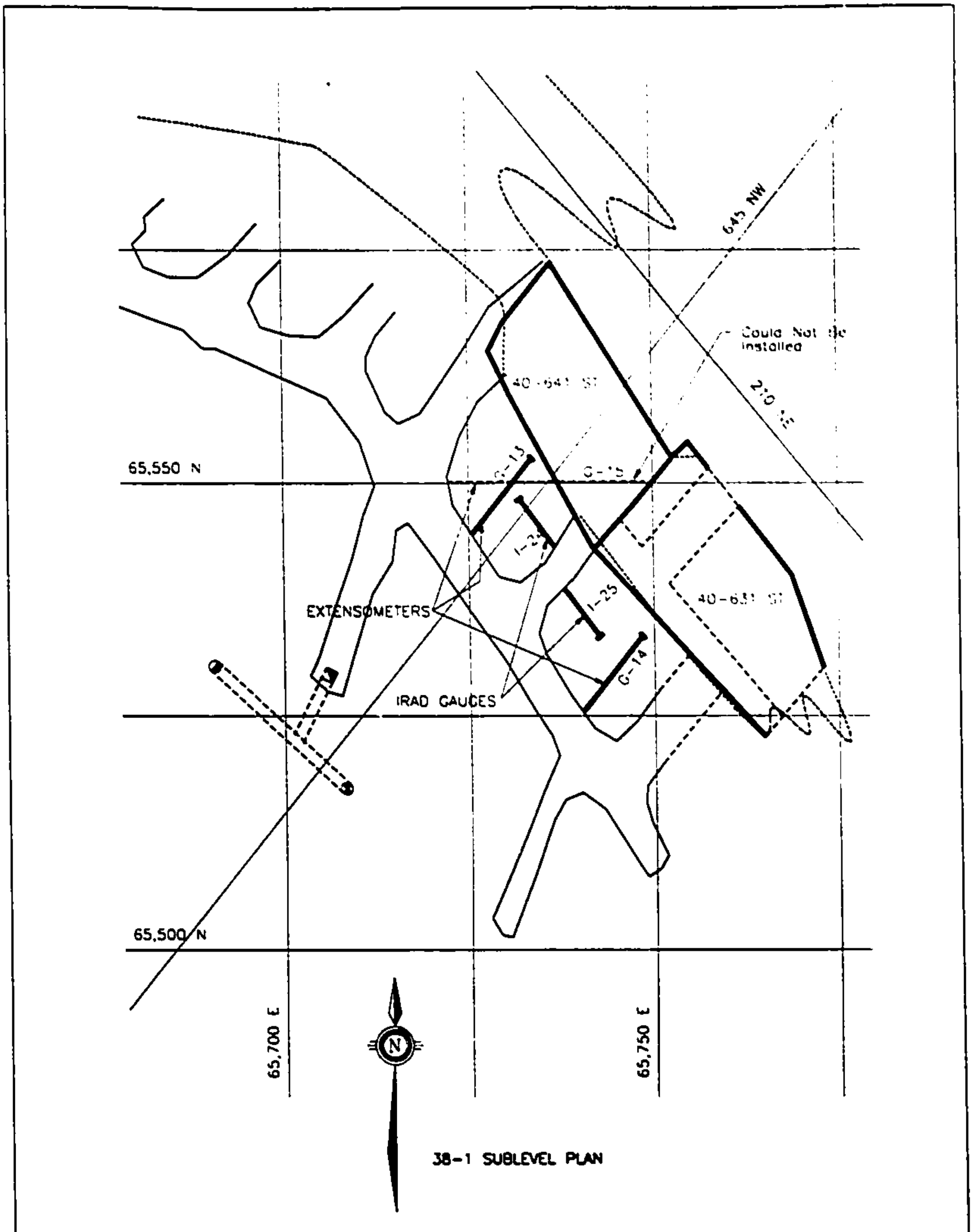


Fig: 8.1.1 Kidd Creek Level Plan

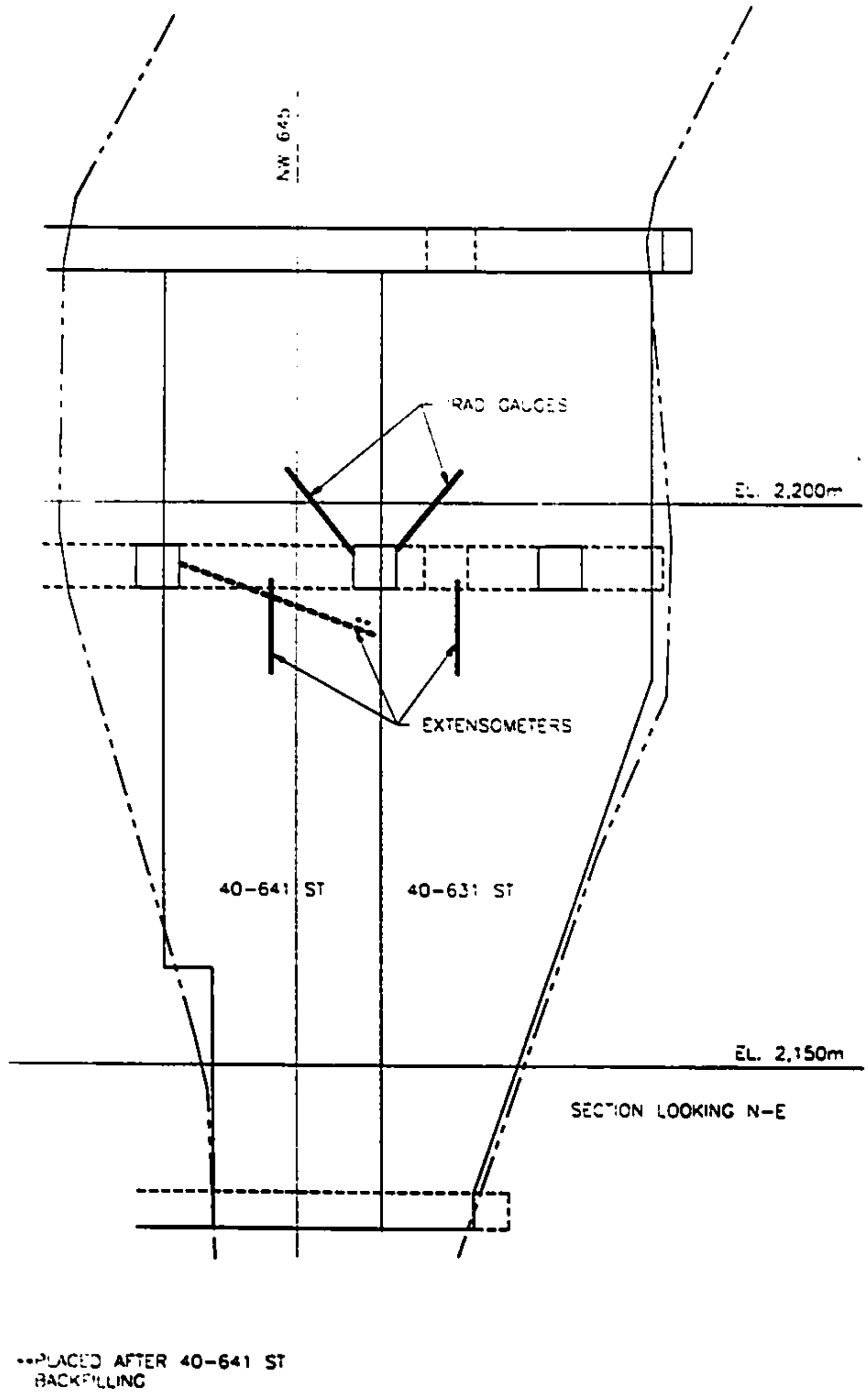


Fig: 8.1.2 Kidd Creek 40-631 & 40-641 ST Longitudinal Section

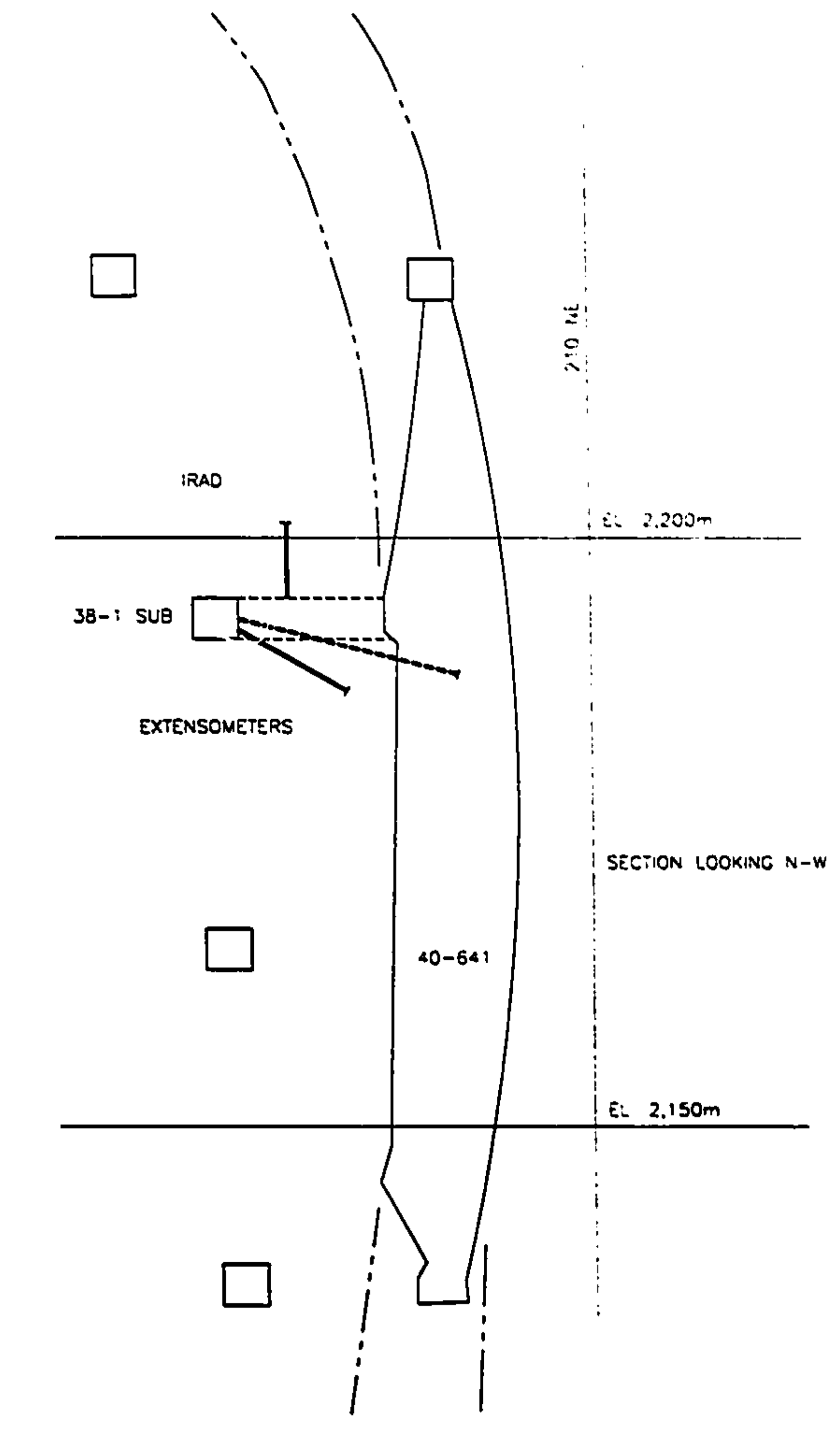


Fig: 8.1.3 Kidd Creek 40-641 ST
Typical Transverse Section

stressmeter, readout meters and setting equipment.

The vibrating wire stressmeter consists of a hollow steel cylinder which, in use, is preloaded diametrically across the sides of a 3.8 cm diameter borehole by means of sliding wedge and platen assembly. Stress change in surrounding rock cause small changes in the diameter of the cylinder which are measured as changes in the natural frequency of vibration of highly tensioned steel wire stretched diametrically across the cylinder walls in the preloaded direction. By calibration, changes in the wire period have been related to the magnitude of stress change for a range of rock types.

An extensometer measures the relative displacement between anchors set in a rock formation. When using the instrument a constant tension is applied to a wire each time a measurement is taken. From these readings the relative displacement of the anchors can be calculated and compared to previous readings to check for changes. A gauge for the tension measurements and a micrometer complete the measuring device which can detect changes of 0.05 mm for a total range of 203 mm.

8.1.1: WALL MOVEMENT MEASUREMENTS (40-641 & 40-631 stopes)

The results of the stressmeter monitoring program are presented in Tables 1to 3, Appendix A. In these tables, the values were converted to stress change using the stressmeter calibration equation, Appendix A. Total stress changes vs time in 40-641 and 40-631 stopes are presented in Figures 8.1.4 & 8.1.5, respectively.

Fig. 8.1.4 shows that while filling 40-641-ST, between days 35 to 130, the ground around the filled stope relaxed about 0.17 MPa (Table 1, Appendix A).The major inflection in the stress change with time occurred on days 132 and 165, corresponding to the major ring and/or slot blasts in 631-38-1 & 631-38-2 respectively. Appendix A shows the exact date for each blast. After each blast due to the increased stope dimensions, rock stress increased and the maximum

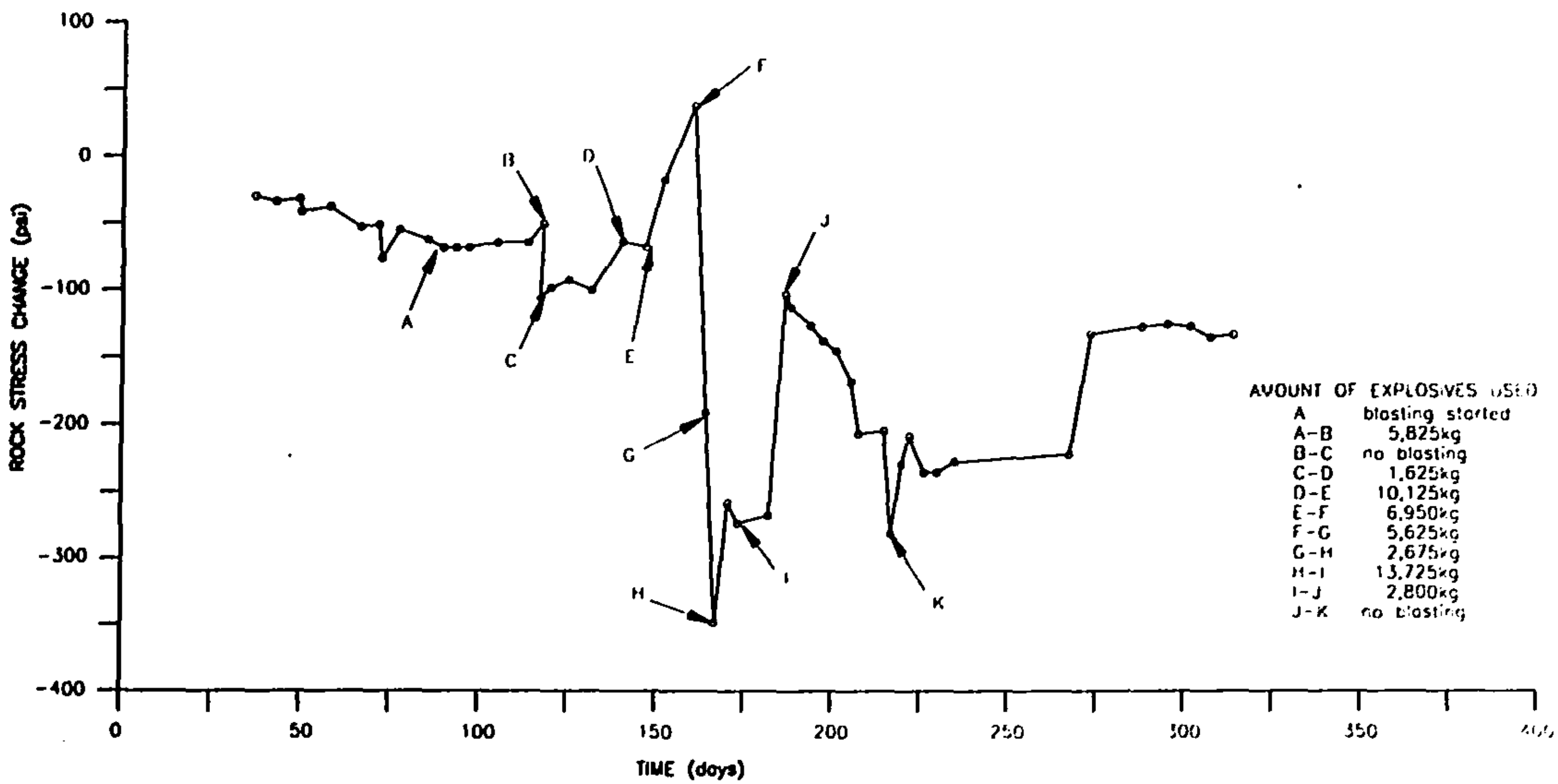


Fig: 8.1.4 Data From I-24 40-641 Stope

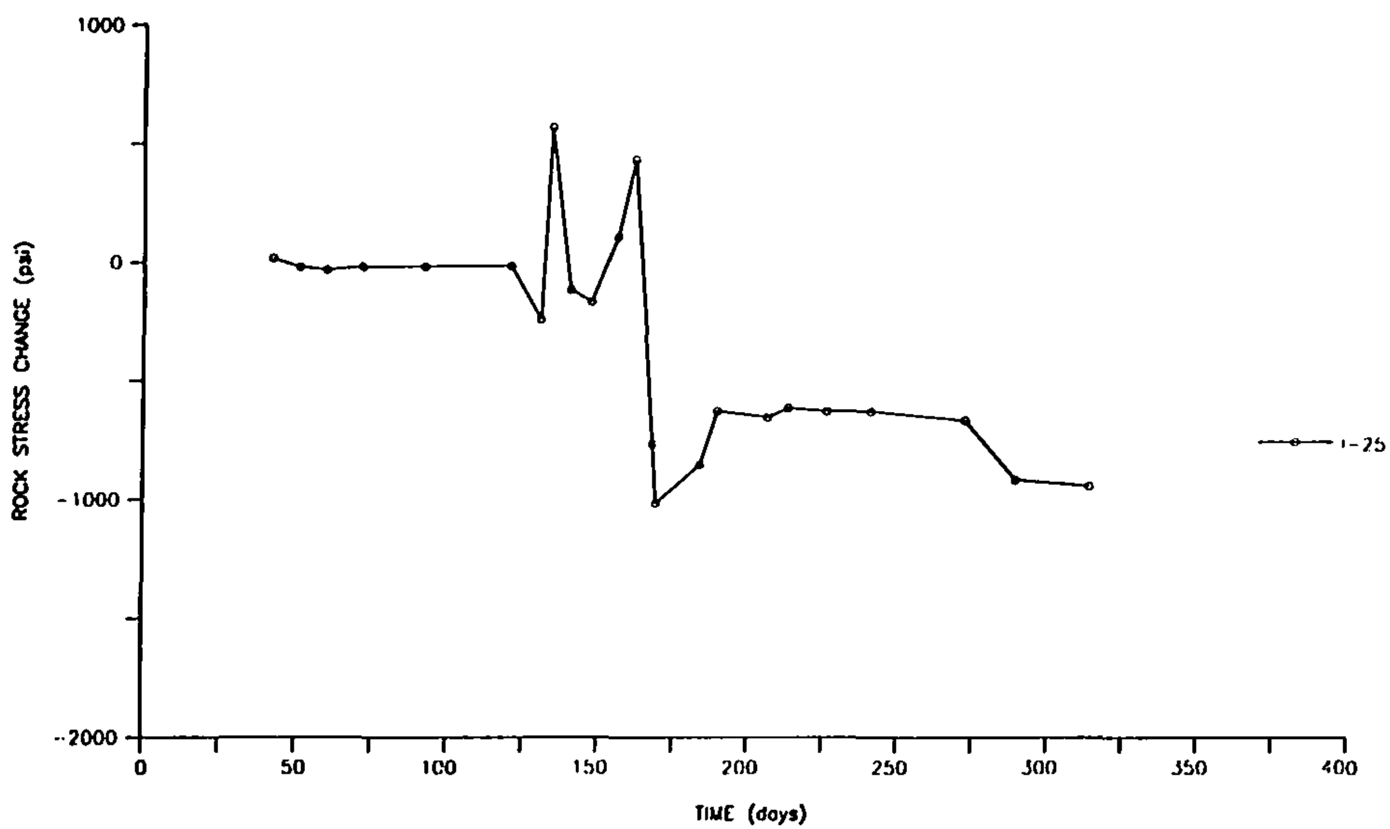


Fig: 8.1.5 Data From I-25
40-631 Stope

compressive stress change at I-24 stressmeter was 0.31 MPa, point F.

A sudden decrease in stress was noticed after the final blast for 631-38-1 X.C., point F-H, Fig. 8.1.4. This was probably showing a shift of abutment stress field surrounding the stope, from a zone of rock subjected to confining stress to one of stress relaxation. The maximum stress release due to the increased relaxation zone is shown by point H, with a magnitude of 2.43 MPa. Ring and slot blasting for 631-38-2 X.C. from point H to J was associated with an increase in compressive stress on the stope wall. The stope wall started to relax after the final blast at 631-38-2 X.C. Deformation occurred between blasts may be due to drawing of the ore and also progressive fracturing at highly stressed areas which resulted in redistribution of wall load. From point J to K there was no blasting and the mucking of 40-631-ST was ongoing. Filling of 40-631-ST started at day 278 and was completed at day 310. No stress change was observed during this period.

Stressmeter I-25, adjacent to the blasted stope, showed the same trend in stress change measurements as did stressmeter I-24, Fig. 8.1.5. There were noticeable changes in stressmeter readings at days 132 and 165, as shown in Fig. 8.1.4. The increased magnitude of the compressive stresses due to the blasting at days 132 and 165 and the relaxation process after blasting shocks, were the only major difference between I-24 and I-25 readings, Fig 8.1.6 . Maximum compressive stress change measured by 1-25 stressmeter was 3.94 MPa. This stress change occurred between days 130-140, when 10,000 kg of explosive was used for final slot blast at 38-1 sublevel, Table 3, Appendix A.

Another major slot blast at 38-2 sublevel using 13,000 kg of explosive caused 2.9 MPa compressive stress change adjacent to 40-631-ST wall, measured by I-25 Irad gauge. After this blast the complete relaxation of the surrounding area took place and the maximum relaxation value was 7.1 MPa at day 168. At the same day I-24 readings showed the highest relaxation value of 2.4 MPa which was 33% of the I-25 value. The maximum relaxation values were obtained 2 days after the slot blast, Table 3, Appendix A.

Filling of 40-631 ST started at day 278 and was completed at day 310. During this period

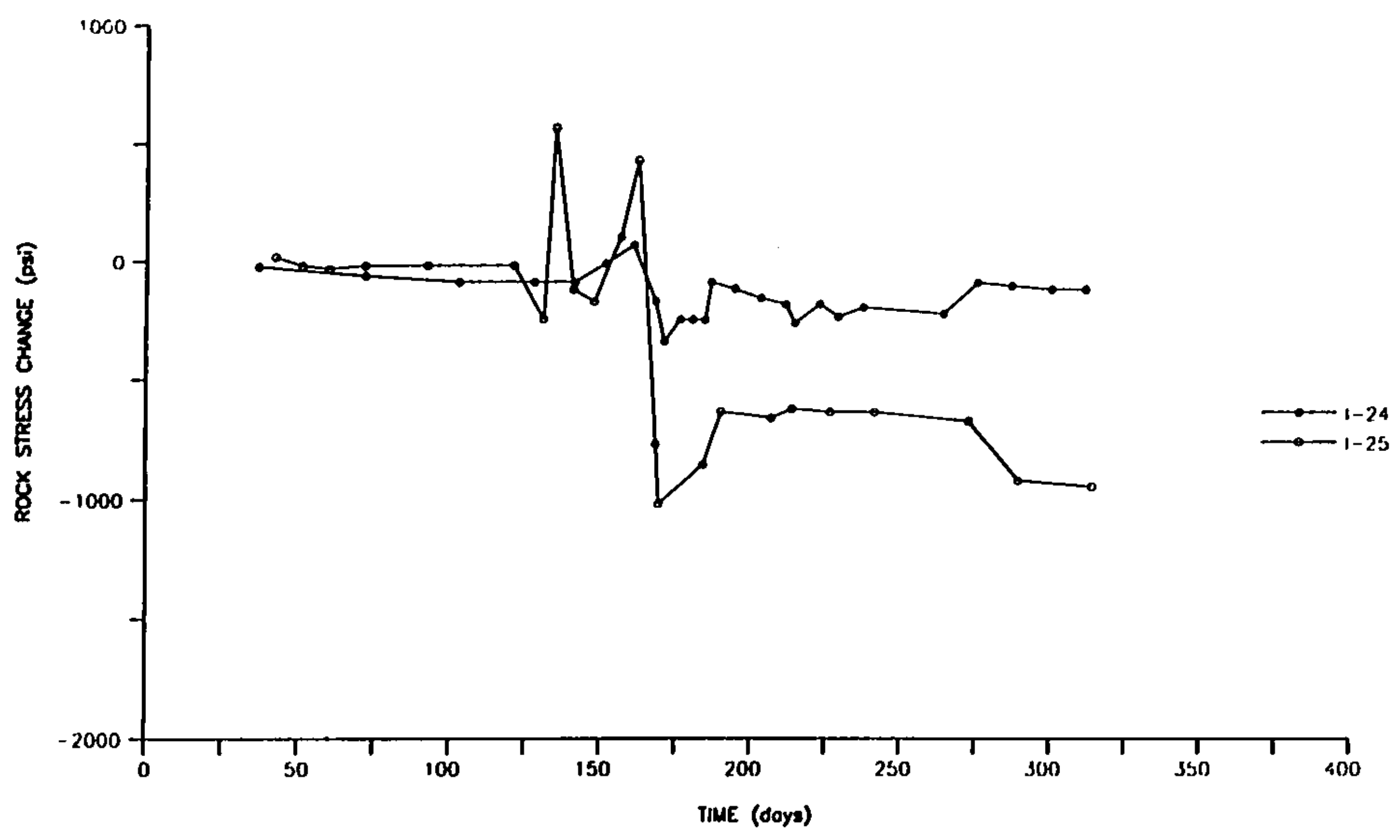


Fig: 8.1.6 Results of I-24 & I-25 Stressmeters the wall surrounding this stope relaxed for a magnitude of 1.73 MPa, Table 2, Appendix A. This could be the amount of redistributed stresses to the fill material.

8.1.2 : PILLAR STRESS CHANGE MONITORING (28-671 stope)

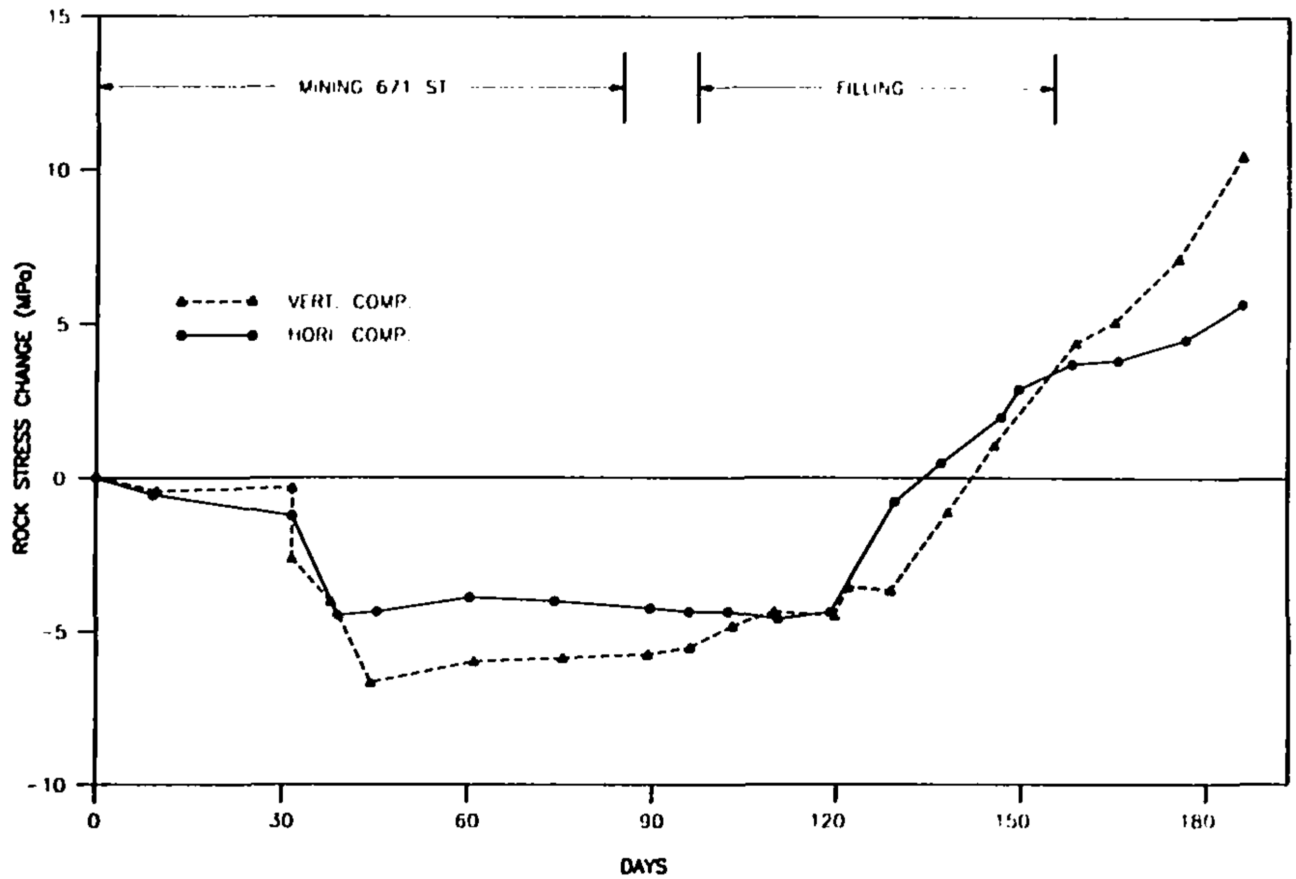
In another case study at KCM, change of pillar stress adjacent to the stope to be filled was monitored. This study showed the effect of fill on the stress distribution at the adjacent pillar, since no production blasting was conducted in the vicinity of the pillar during the monitoring period. The pillar had been virtually isolated from the lateral ground stress, as its two vertical walls had been separated by placed backfill and open stope. Two Irad stressmeters were installed in the pillar to measure the stress change in vertical and horizontal components, as backfill was being introduced into the open stope.

As shown in Fig. 8.1.7, both vertical and horizontal components showed a slow response in the early stages of filling, and continued to increase in the post fill stage. The vertical component appeared to respond faster than the horizontal one, indicating the effect of confinement, as a result of the decrease in the height to width ratio of the pillar, Fig. 8.1.8. (Yu, 1987)

8.1.3: STOPE CLOSURE MONITORING

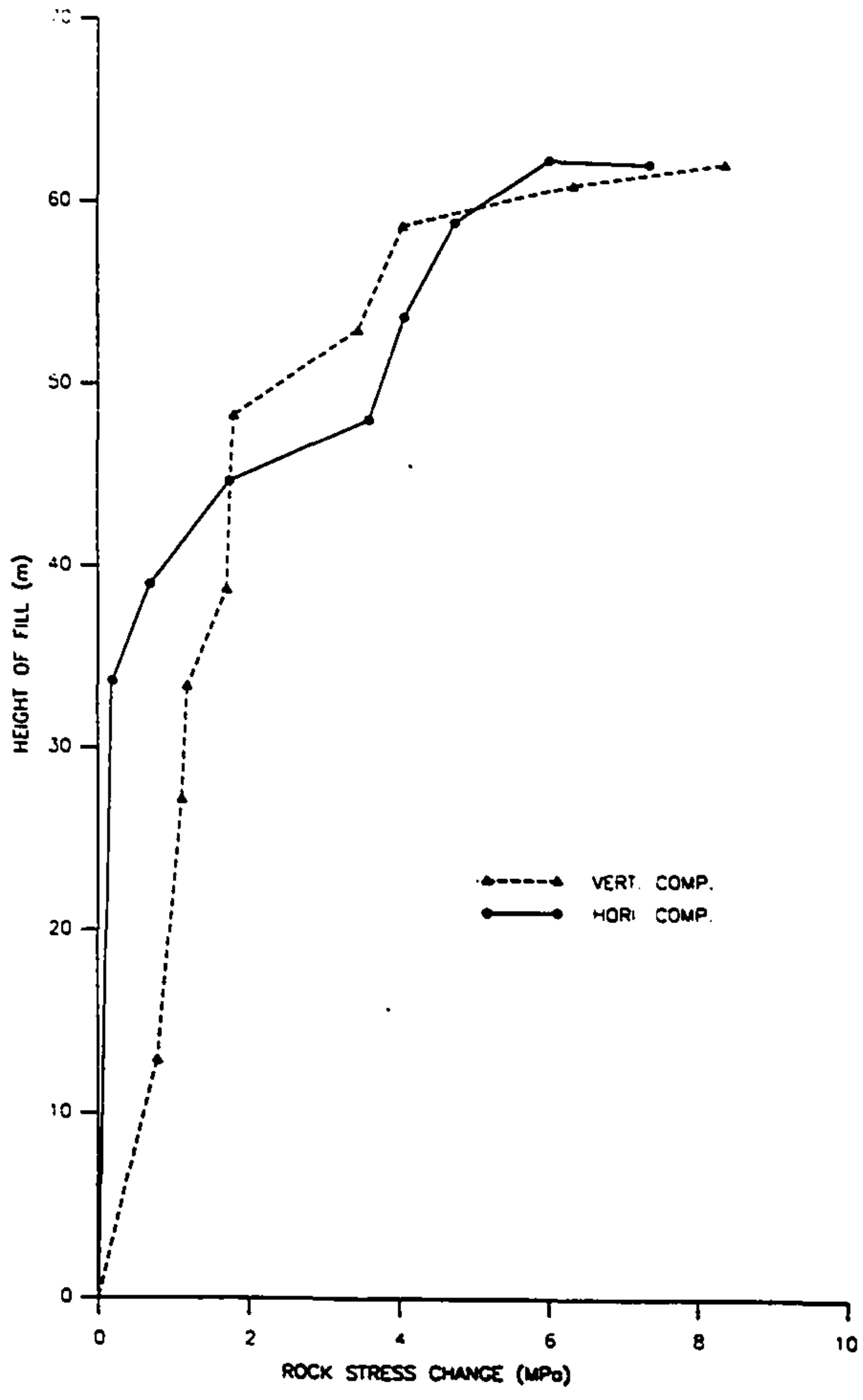
The total displacement vs time for G-13 and G-14 extensometer readings are presented in Figs. 8.1.9 & 8.1.10 respectively.

As expected, the stope wall expanded adjacent to the mined-out area and contracted in the solid ground. Fig. 8.1.9 shows that the stope wall relaxed with a slow rate up to day 104, point B. Blasting started at day 104, point B, and after each blast expansion continued throughout the wall opposite to the blasted area. Most of the recorded expansion in 40-641-ST took place at the



(Yu, 1987)

Fig: 8.1.7 Measured Rock Stress vs Stope Mining & Filling with CRF, 28-671 ST



(Yu, 1987)

Fig: 8.1.8 Effect Of Fill Height On Rock Stress Change, 28-671-ST

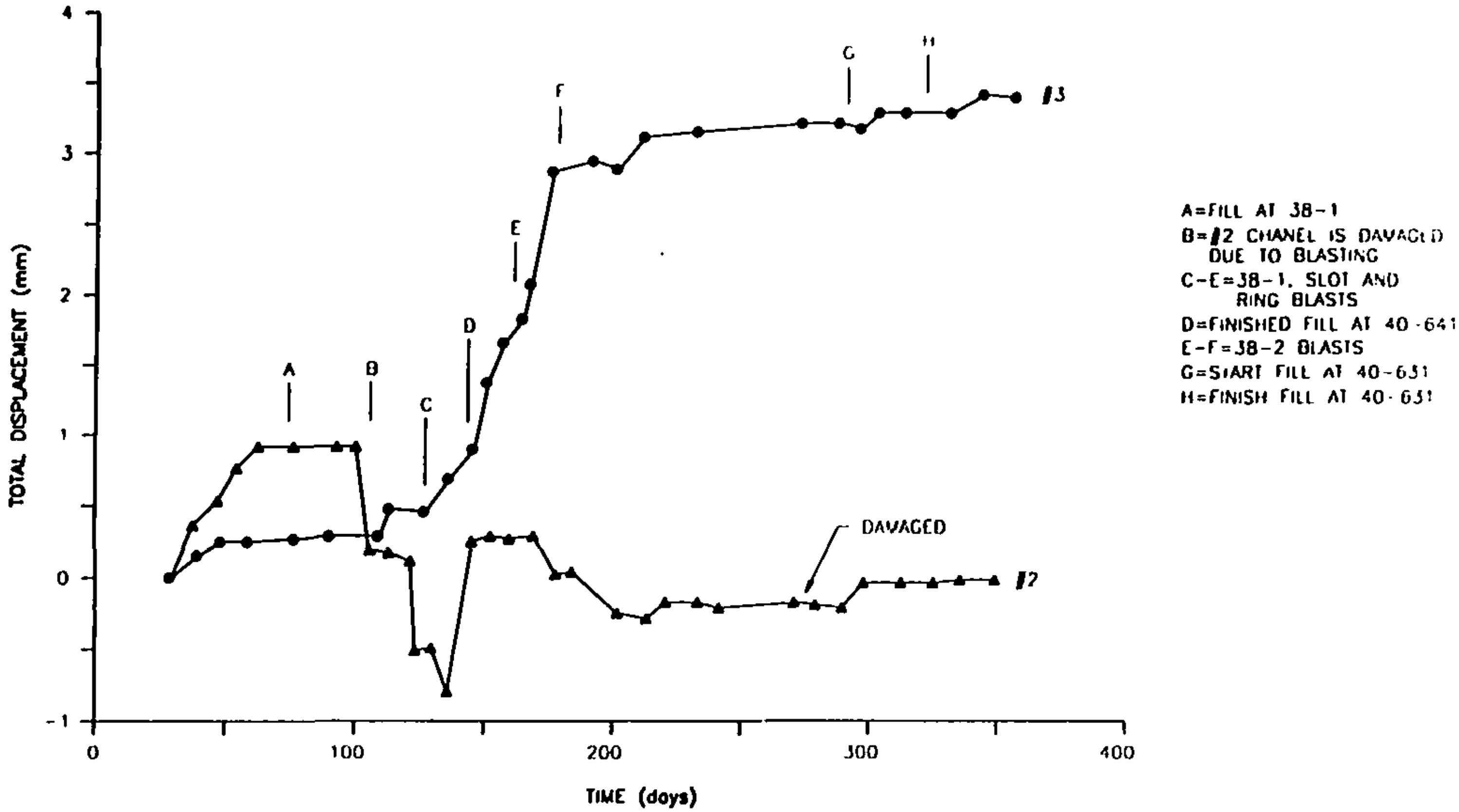


Fig: 8.1.9 Movement Measured In Ext. G-13, 40-641-ST time of major blasts in 40-631-ST . The maximum expansion in 40-641-ST, without blasting, would have been under 1mm. The blasting of 40-631-ST increased this value to 3.2 mm (ch.#3), measured by G-13 extensometer. The deformation occurring between blasts was due to drawing of the ore in 40-631-ST and also progressive fracturing of the the ground in the highly stressed zones. The sharp slope of the line between points C and F, Fig. 8.1.9, corresponded to the major slot and ring blasts at 631-38-1 and 631-38-2 sublevels. After starting to fill 40-631-ST at point G, Fig. 8.1.9, the line had the same slope as the one before blasting started. The wall expansion before blasting started was around 2.5 X 10-3 mm/day and it increased to 4.1 X 10-2 mm/day between points C & F. The expansion rate during blasting was around 17 times of the rate prior to blasting.

Maximum expansion measured by G-14 extensometer was approximately 33 mm, between points H and I. Fig. 8.1.10 indicated that about 80% of deformation happened between points B-G which was during blasting and mucking of the stope. The slope of the line in this period is around 0.34 mm/day, Table 5 in Appendix A. The G-14 results indicated that wall deformation related to mining of the stope included a period of:

A: Short period of compression

B: Rapid expansion caused by blasting

C: Slow time-dependent expansion related to ore drawing and mining.

D: Additional expansion after the fill has been placed and the mined-out area is extended.

Extensometer G-15 which was planned to monitor the movement through backfill during mining and filling of 40-631-ST., could not be installed due difficulties encountered in drilling through consolidated rockfill.

8.1.4 : STOPE CLOSURE MONITORING (838 stope)

In previous studies, measurements of wall deformation in the footwall of 838- ST., was

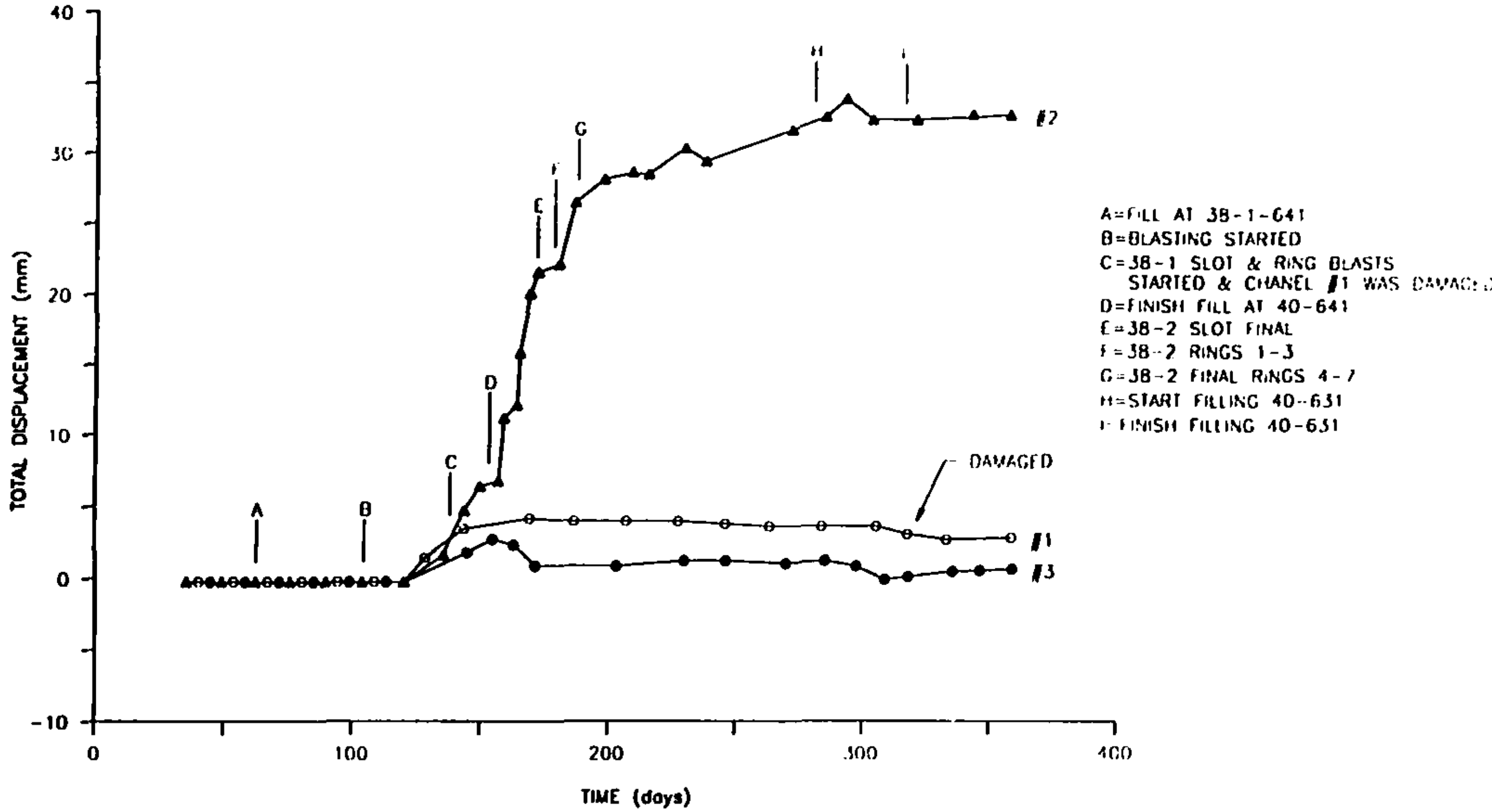


Fig: 8.1.10 Movement Measured In Ext. G-14, 40-631-ST conducted throughout its backfilling process over a four-month period. Two wire extensometers were used to measure the deformation in this stope, Figs. 8.1.11 & 8.1.12. Extensometer M-1 was located at the edge of the stope toward the adjoining rib pillar, and M-2 was installed opposite to the middle of the stope, normal to the wall face. Measurements had been taken throughout the filling period, and Figs. 8.1. 13 and 8.1.14 show the measured data. No production blasting was taken place in the vicinity of the stope; therefore, the measured data showed the sole effect of consolidated rockfill on the wall deformation.

The measured record revealed that, on commencement of backfilling in the stope, the outward displacement of the footwall halted and inward movement began for a contraction of 0.9 mm in the wire of M-2, over the backfilling period. The longest wire in M-1 also detected a contraction of 0.4 mm. These results clearly indicated that the placed consolidated rockfill developed an active pressure state compressing the relaxed stope wall. (Yu, 1987)

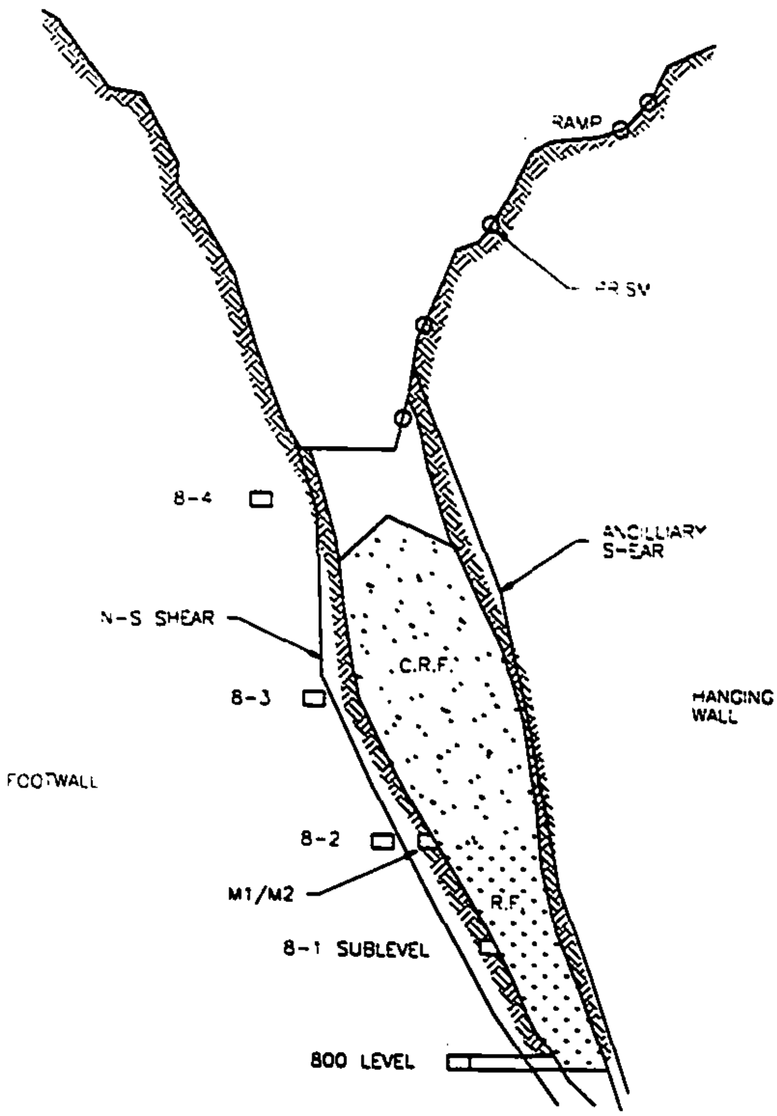
8.2: BLASTING VIBRATION MONITORING

When a pillar adjacent to a filled stope is blasted, the fill must be able to sustain not only the gravity loading of the overlying fill material, but also the dynamic effects applied during blasting. Stress waves generated by blasting can be degraded due to 1) Absorption of seismic intensity from the source, following the inverse square law, 2) Absorption of seismic energy in the medium during propagation according to an exponential decay, and 3) Partitioning of the waves at interfaces due to impedance mismatch. This study was carried out to quantify some parameters of seismic wave propagation into a consolidated rockfill mass.

- Major dynamic properties investigated in this monitoring program were :

A- Transmission of seismic waves in CRF

- To measure the transmission of seismic energy from rock to consolidated rockfill.



(Yu, 1987)

Fig: 8.1.11 East–West Section Through 838 Stope

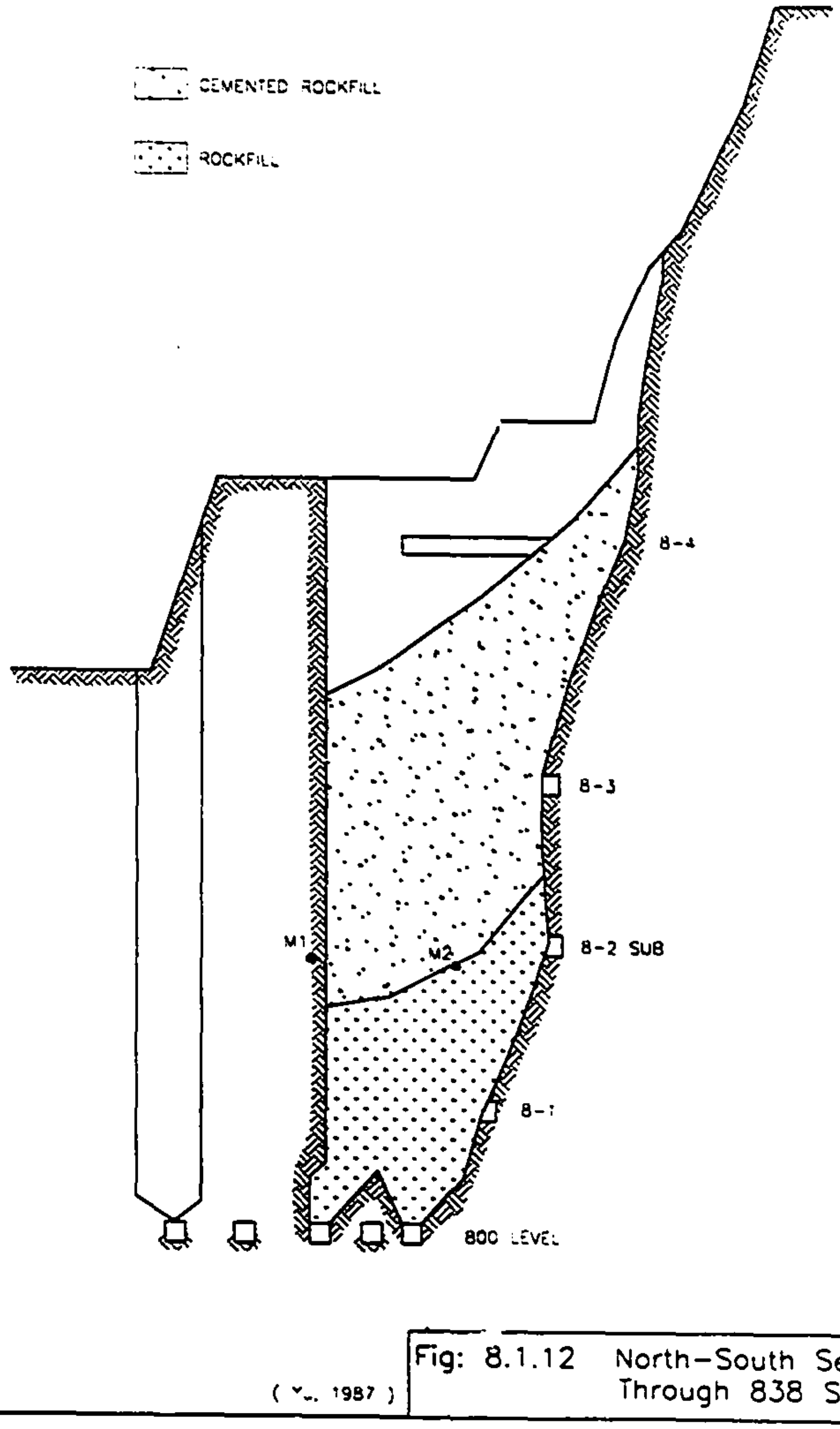
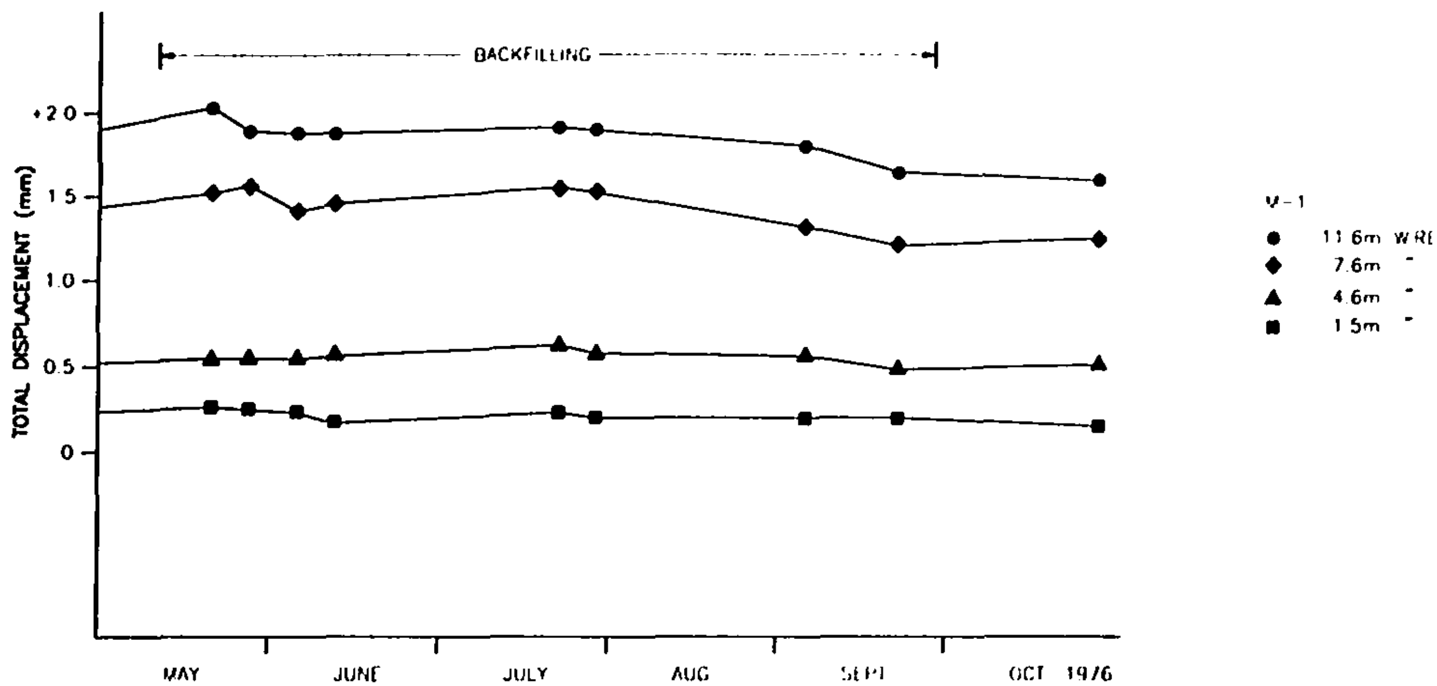


Fig: 8.1.12 North–South Section Through 838 Stope



(Yu, 1987)

Fig: 8.1.13 Displacement of Footwall Measured On Wire Extensometer M1

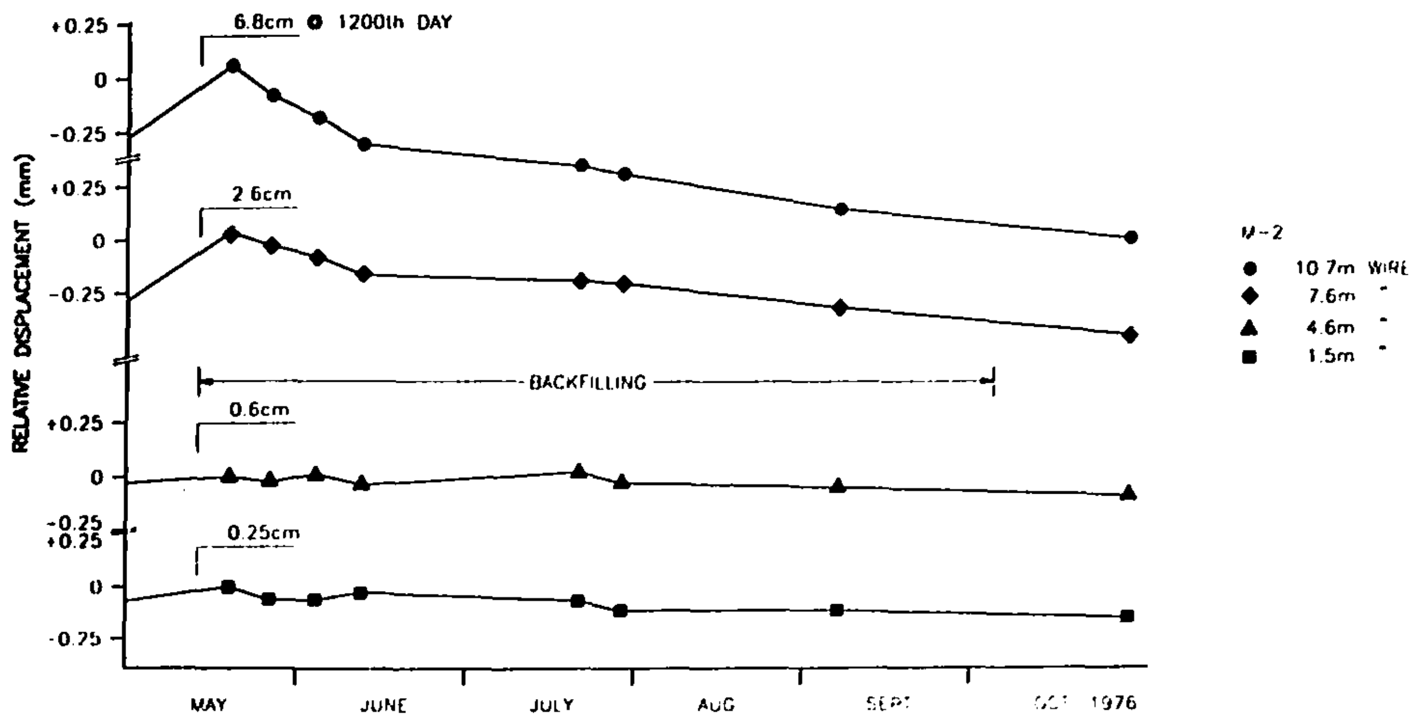


Fig: 8.1.14 Displacement of Footwall Measured On Wire Extensometer M2

B- Attenuation in CRF

- To measure the attenuation factor in terms of seismic magnitude.

C- Amplitude reduction through backfill

- Seismic energy reduction by having the wave passing through rock, fill, and back to rock.

PROCEDURE

- The general blast and monitoring locations are shown in Figs.8.2.1 and 8.2.2.

- Amounts of explosive employed in blasting were as follow:

- Blast A------------------1650 kg of Amex (21 holes)
- Blast B-------------------250 kg of Amex (2 holes)
- Blast C-------------------250 kg of Amex (1 hole)

Monitoring was conducted using two sets of triaxial velocity sensors and a seven channel F.M. instrumentation recorder. Analysis of the data was carried out with a signal analyser. Appendix B.

All the studies in the blast vibration monitoring used that the ' peak particle velocity ', P. P. V., of a vibration for assessing the performance of the media that the shock-waves were transmitted through. The P. P. V. is the rate of change of displacement and is proportional to the product of displacement and frequency.

RESULTS

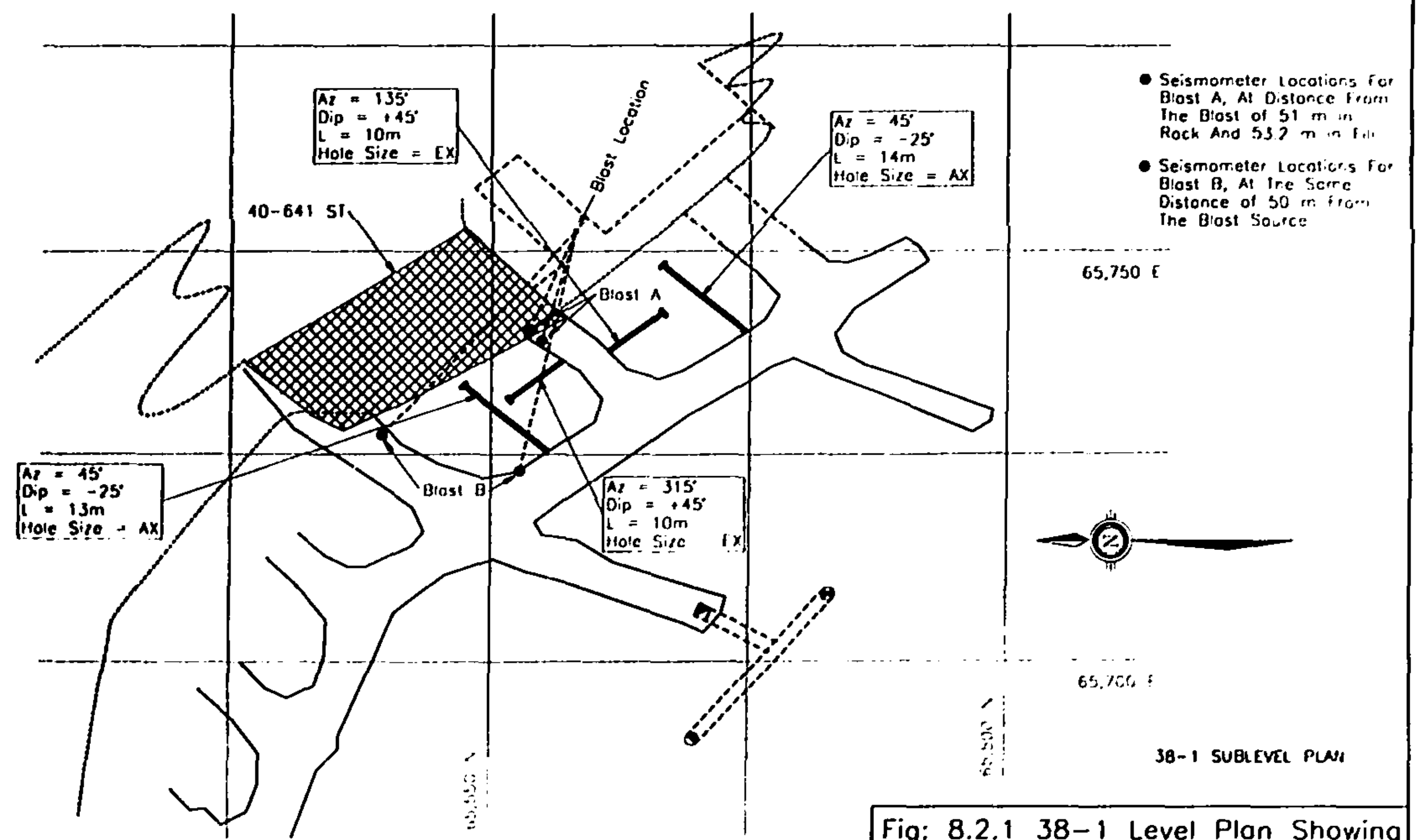
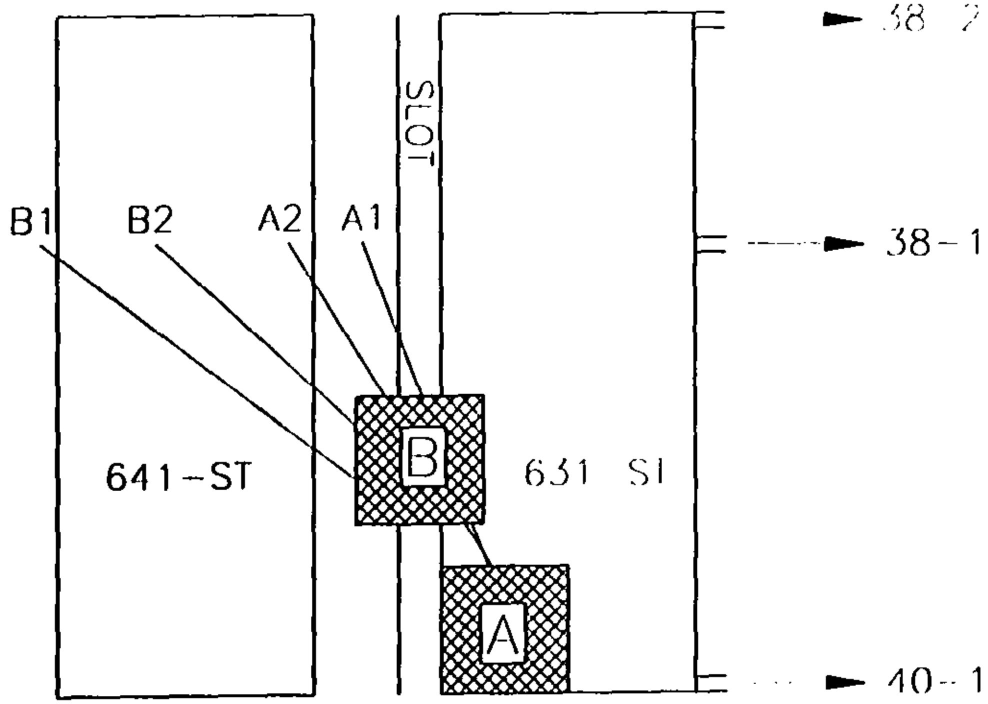


Fig: 8.2.1 38-1 Level Plan Showing Instrument Locations



A1 & A2 : Seismometer Locations For Blast A, In Rock And Fill Respectively

B1 & B2 : Seismometer Locations For Blast B, In Rock And Fill Respectively

Fig: 8.2.2 Longitudinal Section Of 40-641 & 40-631 Stopes

Readings in all tables were converted from voltage to velocity values by using the calibration number given for the monitoring instrument.

- The P.P.V. values were obtained by finding the square root of the square sum of the component readings from the three orthogonal components. Table 6, Appendix B.

BLAST A

This was a stope slot blast, and the amount of explosive used in the blast ranged from 12 kg to 84 kg per delay for a total 1650 kg of explosive. Ground vibration was measured using two sets of triaxial seismometers at a distance from the blast of 51 m in rock and 53.2 m in CRF respectively. The first set of readings (ch. 4-6) was taken after the shock fronts had travelled only through rock. The other set (ch.1-3) showed the reduction of P.P.V due to the waves travelling into fill (Table 6, Appendix B). Results, Fig. 8.2.3, showed that the transmission coefficient for particle velocity from the rock through backfill averaged 37%. The amount of energy transmitted through fill could have been much higher if the fill had lower void ratio and finer size material at the interface of two media. Past studies at KCM revealed that the transmission coefficient varied in the range between 25% to 73%.

BLAST B

This was a small blast compared to blast A with a total of 250 kg of explosive used. Both sets of seismometers were installed at the same distance of 50 m from the blast source. The sensor in the fill would detect the shock waves travelling the first 15 m in rock and the other 35 m in CRF. The sensor in rock would pick up the shock waves travelling through rock only. Measured results indicated that the P.P.V. at the fill sensor and the rock sensor were 1.7 i.p.s and 2.8 i.p.s respectively. From these two sets of data, the attenuation of seismic waves in the CRF was found to be 0.42 db/m which is comparable to the past measured data of 0.57 db/m in a competent fill and

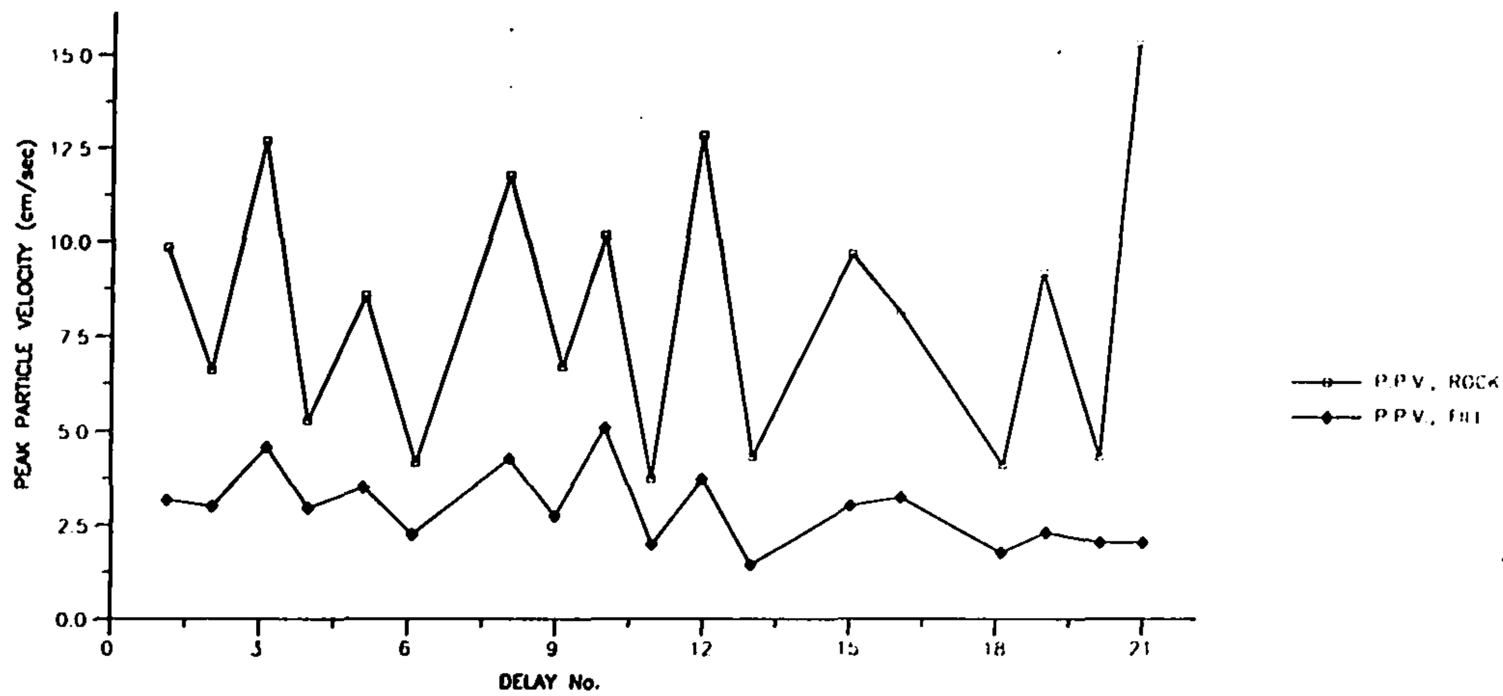


Fig: 8.2.3 Seismic Measurements In Rock And Fill From Blast A

0.95 db/m in a segregated zone. (Yu, 1989)

The seismic attenuation through rockfill mass was calculated using following information:

Va = Measured vibration in rock at a distance of 50 m (Ra) from the blast source = 2.8 i.p.s.

Vb = Measured vibration level in fill for shock waves travelling the first 15 m (Rb) in rock and the other 35 m (Rc) in CRF.

Vc = Vibration level at 15 m from the blast in rock, as derived from:

$= Va \ (Ra / Rb)^{1.8}$

Vd = Vibration level at the boundary, 15 m from the blast, in fill

= 0.37 Vc from the results of blast A = 0.91 i.p.s

The seismic attenuation, constant in CRF, can be found from:

$= 20 \log 10 ((Vb / Vd)) / Rc) = - 0.42$ db/m

BLAST C

Blast C was conducted to study the magnitude of amplitude reduction of shock waves through backfill. Similar to blast B, 250 kg of explosive was employed for this trial. The energy from the blast travelled through rock, fill, and then back to rock respectively. Blast A and B were monitored on 38-1 sublevel, but blast C was monitored on 38-2 sub-level. By comparing the magnitude of the wave amplitude in rock before and after travelling through fill the reduction factor could be obtained. Channels 4-6 were placed in rock and channels 1-3 were also in rock but the signals they read had passed through fill. The measured data, however, are not presented due to inconclusive results.

DISCUSSION

The results obtained from blast A could be used in future designs and results from blasts B and C, especially blast C, need more detailed investigation. This study showed that a considerable reduction of seismic intensity up to 37% was observed at the interface between rock and CRF, and the attenuation of seismic waves in fill was approximately 0.42 db/m. Also, from past studies it was observed that a competent consolidated rockfill mass was subjected to a scabbing failure at an estimated particle velocity of 30 cm/sec. This vibration level may be considered as a maximum allowable particle velocity in planning a safe production blast against fill. Stope fill failures were seen less severe for blast holes drilled in parallel to the fill than those drilled toward it.

In the process of analysing seismic data, the difficulty in reading the lower limits of seismic traces, resulted in trying another approach. The peak amplitude (not peak-peak) of seismic tracing for each shot, by assuming the sinusoidal vibratory waves, was compiled and presented in Tables 7 and 8 of Appendix B. The data was then used to calculate the transmission coefficient of seismic waves for blast A and blast B. The results matched closely with the peak-peak amplitude readings, as shown in Table 6, Appendix B. Blast energy calculation was also carried out for all 3 blasts and results are presented in Tables 9-12 in Appendix B.

8.3: IN SITU BACKFILL TESTING USING PRESSUREMETER

Due to the highly heterogeneous rockfill masses, in situ testing is needed to obtain the different physical and mechanical properties within placed backfill materials.

In-situ testing of rockfill was mostly carried out in the 12-2-35 test drift at KCM. The test drift 14 m long was driven in a stope which was filled earlier with consolidated rockfill. A longitudinal section of the drift is shown in Fig. 8.3.1.

8.3.1: PRESSUREMETER TESTING

The objective of this monitoring program was to evaluate the effectiveness of pressuremeter

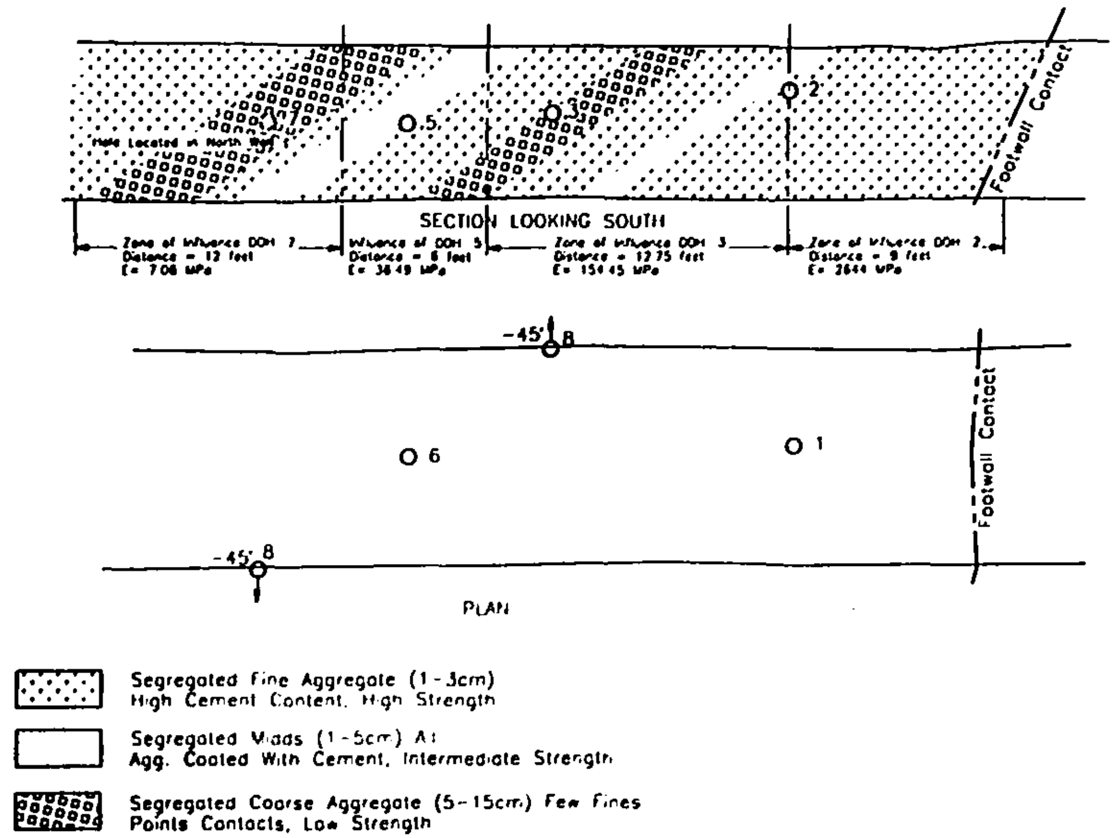


Fig: 8.3.1 12-2-35 Test Drift
Pressuremeter Test Hole
Locations

testing technique for in-situ measurements of consolidated rockfill material. A TEXAM pressuremeter, borehole device, was used to run an in-situ loading test. Trow Ontario Ltd. was involved in the series of testing work.The pressuremeter apparatus is made up of three components;

PROBE: The probe is a loading device which is inserted into the borehole at the desired test elevation. The probe is essentially a hollow metallic cylinder on which an inner rubber membrane is fixed in the central part. An outer protective sheath is mounted over this rubber membrane and extends over the length of the probe. It is this sheath that is in direct contact with the walls of a borehole when the probe is pressurized, Figure 8.3.2.

CONTROL UNIT: The control unit is a fibreglass case with a front panel on which all the various regulators, pressure gauges, valves, etc. are fixed. Within the control unit is a reservoir which supplies the water to central measuring cell. The volume variations during the test with a manual actuator to operate the piston are read on a sight-tube, Figure 8.3.3.

COAXIAL TUBING: Two tubes connecting the control unit to the probe are arranged coaxially and made of a semi-rigid material. The inner tube is used to apply water pressure to the central measuring cell, whereas the outer tube allows the application of the gas pressure to the guard cells. Expansion of the measuring tube when under pressure is negligible.

The pressuremeter test allows for the evaluation of rockfill quality at specific depth and location in the filled stope. An in situ stress/ strain curve is obtained by plotting the injected volume vs. pressure, as shown in Figure. 8.3.4. The CRF modulus of deformation, E, can be calculated using the given equation :

$$E= [(2 \times (1+ U) \times V \times dP) / dV]$$

U= Poisson's ratio, estimated at 0.33 ; V= Initial Volume of probe

dP= Pressure change dV= Volume change

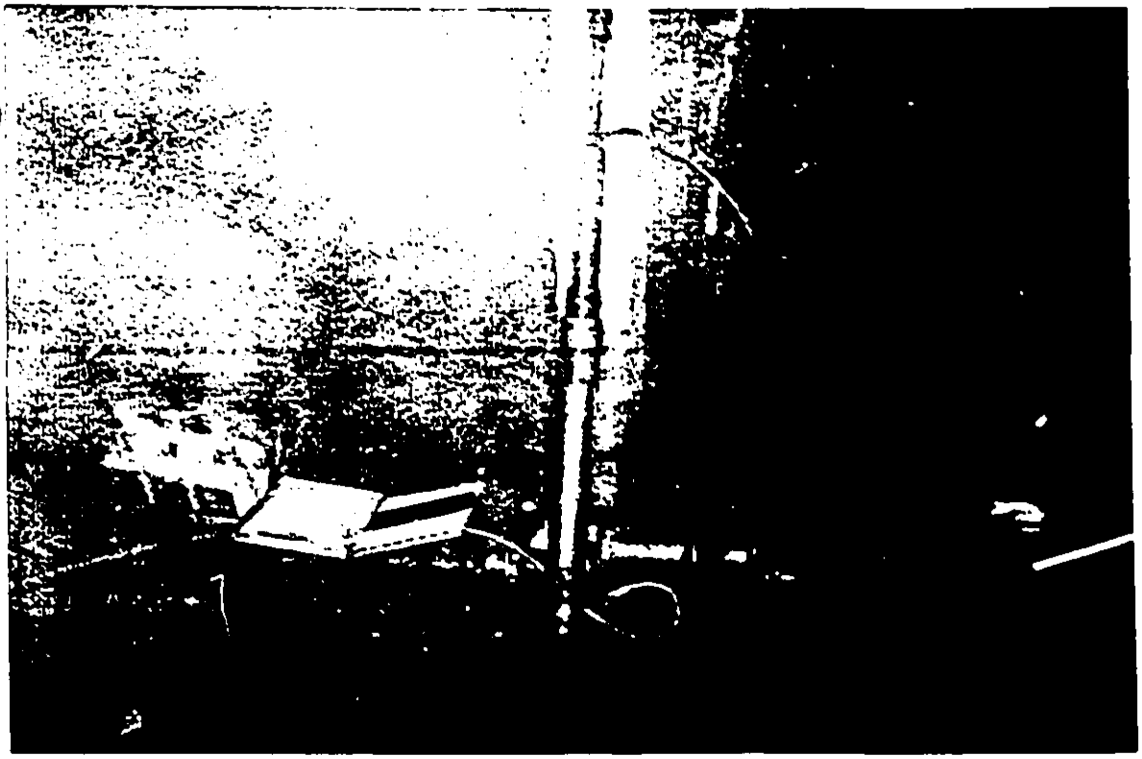


Figure 8.3.2: PRESSUREMETER PROBE

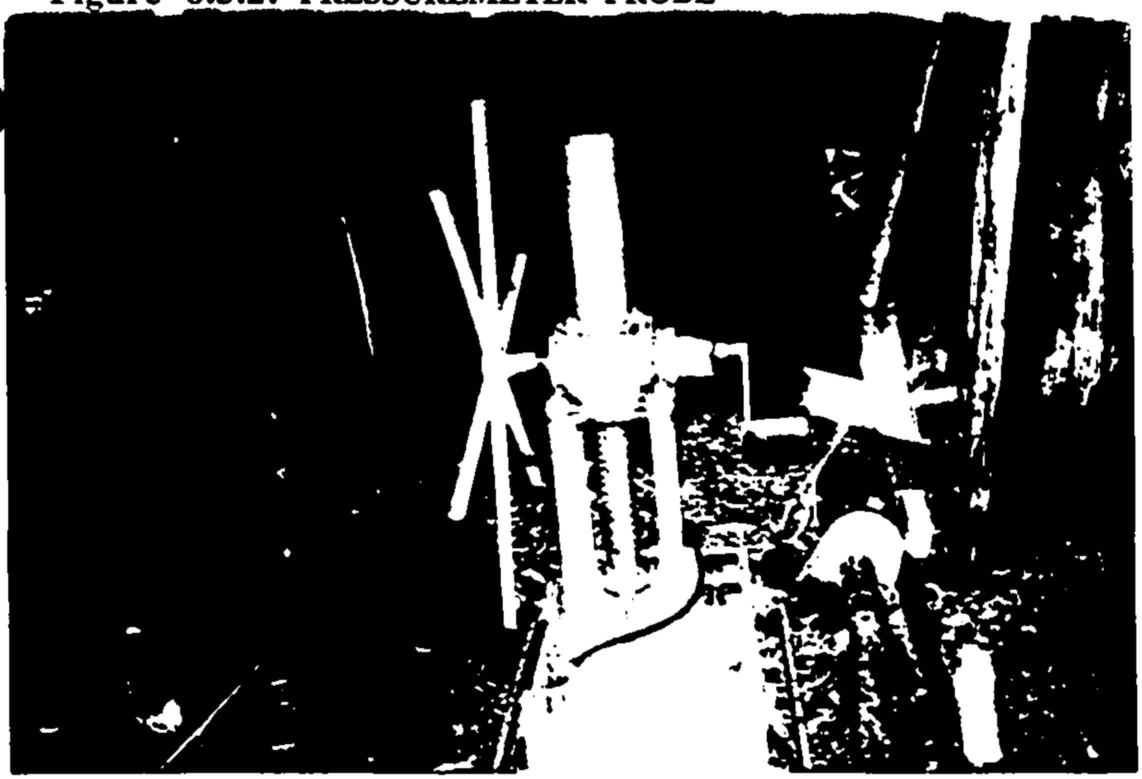


Figure 8.3.3: PRESSUREMETER CONTROL UNIT

The loading device has to be set at the desired level of testing such that the materials to be tested undergoes minimal disturbance. Once installed, the probe is submitted to an increasing pressure applied in equal increments. At each pressure stage the volume changes of the probe are recorded at specific time intervals. The pressure / volume relationship is then drawn up for subsequent determination of material properties. Figure 8.3.4 shows a typical pressure / volume curve obtained from a pressuremeter test.

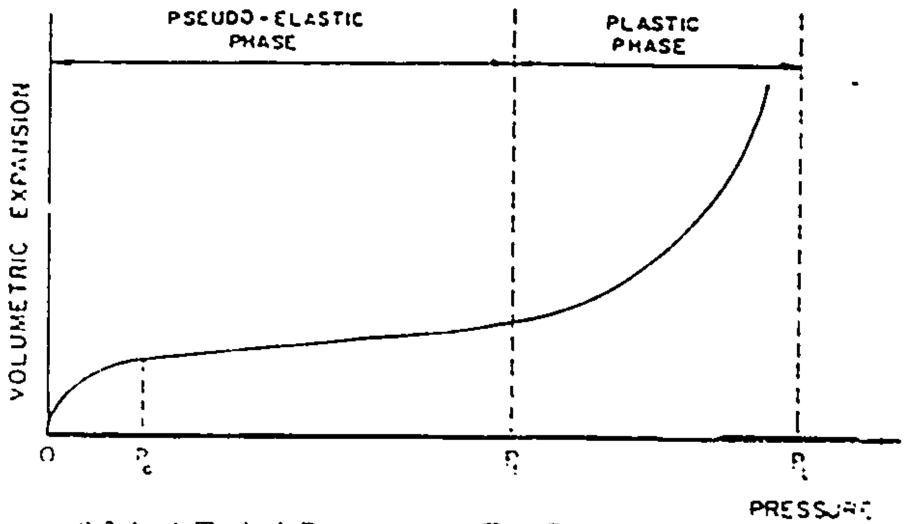


Figure : 8.3.4 : A Typical Pressuremeter Test Curve.

The typical pressure/ volume curve can be divided into three parts.

1) From P=0 to $P= P_o$

This portion of the curve corresponds to the seating of the probe against the wall of the borehole. The disturbance of the wall induced by drilling or driving the probe into place will have considerable influence on this portion of the curve. Also, the difference in diameter between the hole and the probe will affect the shape of the curve.

2) From $P=P_o$ to $P=P_f$

This represents the pseudo-elastic behaviour of the loaded material. The probe is in contact with the walls of the borehole and the loading is uniform all along its length. The linearity of this portion of the curve helps define the modulus of deformation of the mass under load which in turn can be used for settlement evaluation.

3) From $P=P_f$ to $P=Pl$

The pressure P_f by definition is the pressure at which the mass enters a plastic state. From this pressure on, the deformation of the mass under load accelerates up to a point where complete failure occurs. The pressure which defines failure is the limit pressure Pl.

For rockfill material, it is not always possible to get the pressuremeter probe into close contact with the sides of the borehole if the holes are uneven due to caving. Even if it is possible to insert the probe properly, sharp pieces of aggregate can puncture the sheath and membrane and cause the fluid to drain out of the probe. In the case of rockfill in situ testing, the probe must be placed inside a special device which can provide the required support for the sides of the hole and protect the membrane. The device is a slotted tube which has six longitudinal, 2 mm wide, slots cut through it. During testing the slots open up and allow the tube to expand with the probe. The slotted tube has an outside diameter of 63.5 mm and an inside diameter of 47.5 mm and accommodates a standard AX probe of 44 mm. The slots are of the order of one meter length. All the above information & materials could be obtained from Roctest Limited, Montreal, Canada.

A total of eight holes were drilled using a drilling and casing technique developed for this experiment. The drilling was conducted using a JV drill on a jack-bar setup, Figure 8.3.5. Two holes, one horizontal, Figure 8.3.6, and one vertical, were typically drilled from each setup to a nominal depth of 3 m. Drilling lines were tied into the footwall/backfill contact and were spaced at a spacing of 2.44 m. The probe was then placed at the test depth in each of eight boreholes. Precise locations are indicated on Fig. 8.3.1.

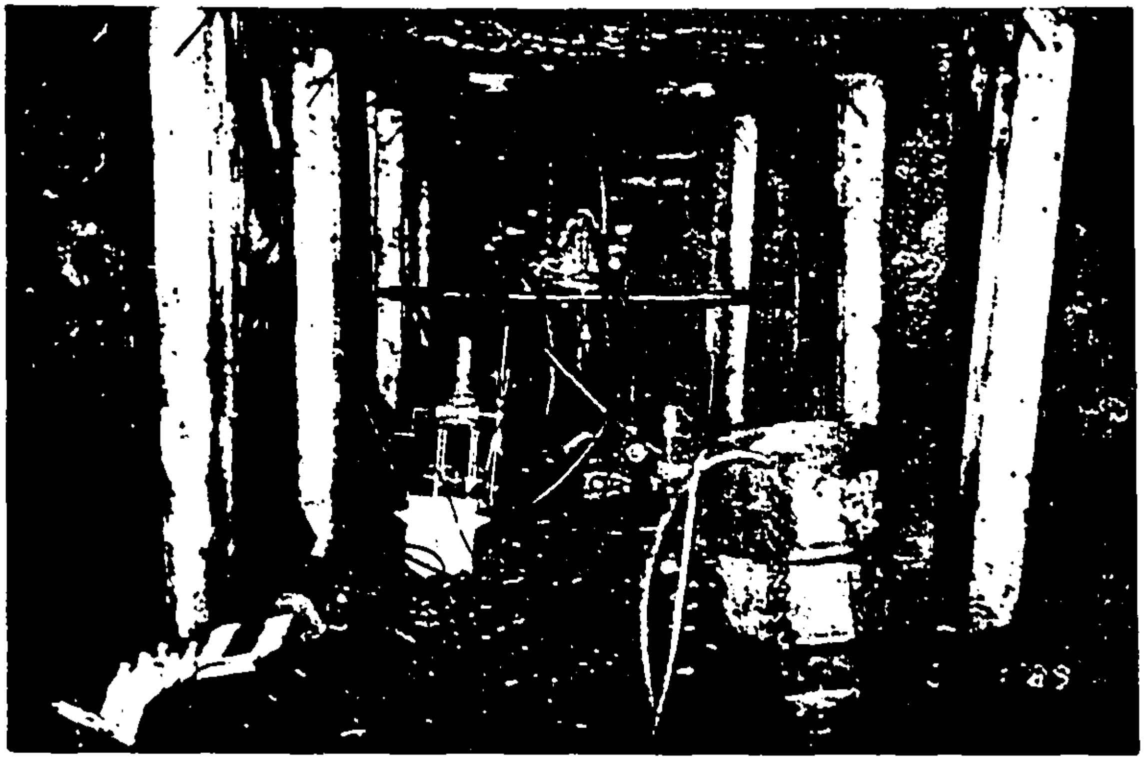


Figure 8.3.5: JV DIAMOND DRILL ON A JACK-BAR SET UP.

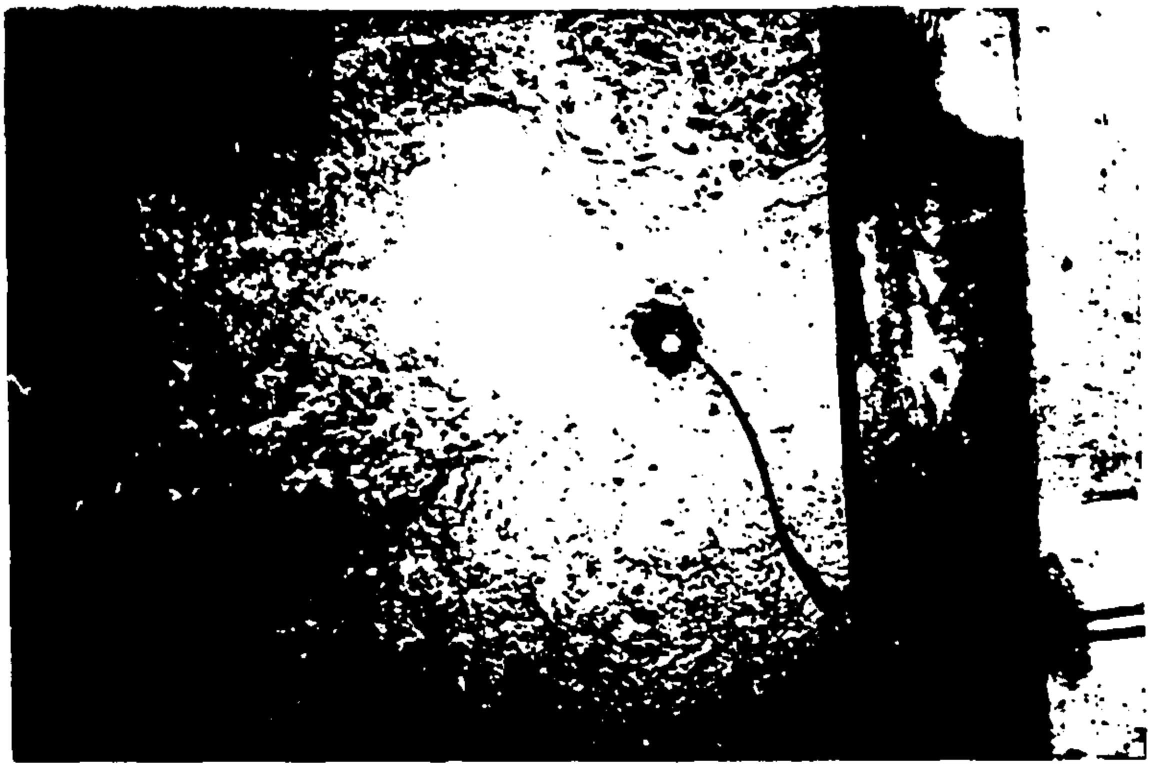


Figure 8.3.6: PROBE IN HORIZONTAL HOLES IN CRF

Two to three tests were normally conducted in each borehole with the exception of borehole #7 which caved on retrieval of the rods. In total 21 pressuremeter tests were conducted. Typical data relating to pressuremeter testing is graphically presented in the drawings 1& 2 of Appendix C and summarized in Table 1. The data indicated a general decrease in bulk modulus as holes moved away from the footwall. In fact, the only core which could be recovered was in the first drilling ring, closest to the footwall.

The bulk modulus data was typically in the order of 1.5 GPa to 4 GPa in the first drilling ring, adjacent to the interface between rock and fill. This gradually decreased to the 0.1 GPa to 0.2 GPa range in the second drilling ring, as the ring moved toward the hangingwall, and was further reduced thereafter to the 0.07 GPa to 0.04 GPa range. This latter range indicated that the tested material contained little cement.

A second trend was also noted. With the exception of borehole #6, the downholes (1, 4, 5 and 8) indicated a greater variation in data than did the horizontal holes. This was to be expected as the downholes are more likely to traverse various fill layers than are the horizontal holes which were targeted to specific layers.

8.4 : CORE SAMPLE TESTING

To compare the results obtained from laboratory testing and in situ borehole trials, the core recovered from borehole #2 was cut and trimmed to a height/width ratio of 2.5:1 and tested in an ELE-200 testing machine at a constant loading rate. A total of seven cores were tested . The individual stress/strain relationships are presented in Figures 8.4.1 to 8.4.7, respectively and the extracted data are summarized in Table 2. This phase of testing yielded a mean modulus value of 4.92 GPa and uniaxial compressive strength of 17.84 MPa. The mean modulus value obtained in the laboratory is in the order of twice that measured in the field. However, the laboratory data, which are related to the specific depths at which tests were conducted in the field, correlates well with the field results. This is illustrated in Table 3.

8.5: SEISMIC MEASUREMENTS

Two series of seismic measurements were carried out. The first series included field surveys and core sample measurements conducted by Trow Ontario Ltd. In the first field trial, the high attenuation of seismic waves in the 'porous' CRF, and the low magnitude of the seismic source made the first arrival of seismic waves indistinctive, resulting in poor results, Fig. 8.5.1

To obtain the seismic velocity, the second series of measurements were carried out by SIAL Geophysics Inc., using OYO McSeis 160, 24 channel stacking digital seismograph. The determination of P-wave velocity was conducted in the field by using three shotpoints, two hammer and one by blasting. Geophones were set up along a profile on the north wall and another one on the south wall of the test drift. The hammer shotpoints, SP-1 and SP-2, were at 5E on their respective lines and the blast was set off at the east end of the drift in CRF on the north wall, for more information refer to Figs. 8.5.2 to 8.5.6.

From the hammer shotpoints P-wave velocities of 3080 and 3610 m/s were obtained. At SP-4, closest to the blast, the P-wave velocity was rather low, probably because of nonrepresentative full body wave. The next interval, 7.1 m away, yielded a velocity of 2960 m/s. Average P-wave velocity in CRF from all three readings was 3220 m/s.

Laboratory seismic velocity measurements were conducted on core samples from borehole #2 using the PUNDIT instrument and the data is presented in Table 4. The average velocity of 3,080 m/s measured from core samples was in agreement with that of 3,220 m/s obtained from the field measurement by SIAL.

8.6 : PLATE-LOAD TEST

To develop a quantitative method for determining the in situ strength of consolidated rockfill, a series of plate-load tests were carried out on backfill material in the backfill test drift.

In the plate-load test, a steel plate, 10 cm x 10 cm x 2.5 cm thick, was used as a bearing plate, and load was applied with a 30 tonne capacity ram until the fill under the plate failed. Tests were conducted in chambers which were carefully prepared in the fill to be tested. Testing on the fill exposed was accomplished by holding the ram against a timber post to opposite drift wall while loading of the ram was in progress, Figure 8.6.1.

The test results indicated that the bearing plate-load test would provide a quick and reliable method for determining the in situ strength of consolidated rockfill, provided test sites be readily available. The ratio of ultimate bearing strength to uniaxial strength was found to be approximately 3.2 to 1.

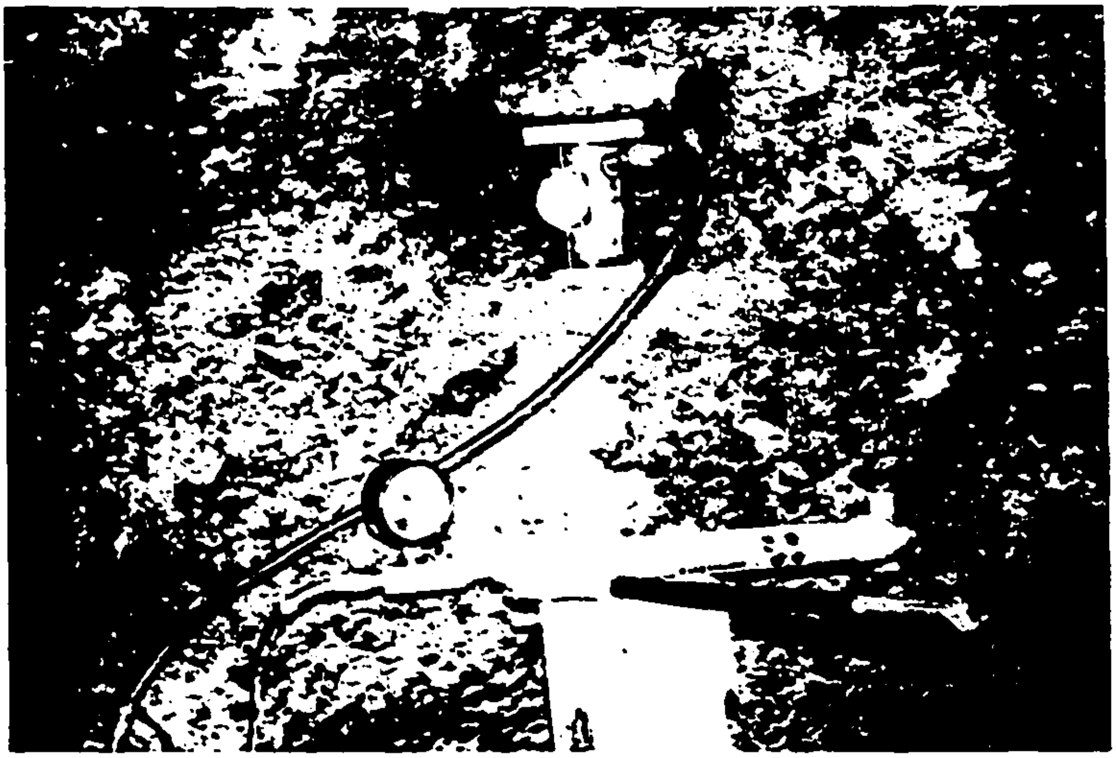


Figure 8.6.1: In situ testing using plate-load method.

TABLE 1

Summary of Pressuremeter Tests

Borehole #	Depth (m)	Bulk Modulus (MPa)
1	.92	> 4000
	2.14	1470
2	.92	3383
	1.83	2270
	2.75	2280
3	.92	96.0
	1.83	192.2
	2.14	144.3
	2.75	185.3
4	.92	37.98
	1.83	2.19
	2.75	30.18
5	.92	6.31
	2.14	44.87
	2.75	28.1
6	.92	28.20
	2.14	12.87
	2.75	7.91
7	2.14	7.06
8	.92	22.01
	2.14	8.05

TABLE 2

Summary of Laboratory Testing

Sample Depth (m)	Deformation Modulus (Gpa)	UCS (MPa)
.61	4.31	19.0
1.22	3.71	10.0
1.83	6.45	17.0
2.14	3.40	21.5
2.44	6.7	17.0
2.90	4.10	20.0
3.05	5.75	20.0
Average	4.92	17.84
Standard Deviation	1.25	3.59

TABLE 3

Comparison of Field and Laboratory Data

Field Data		Laboratory Data	
Depth (m)	Modulus (Gpa)	Depth (m)	Modulus (Gpa)
.92	3.38	.61	4.31
.92	3.38	1.22	3.71
1.83	2.27	1.83	6.45
1.83	2.27	2.14	3.40
2.75	2.28	2.90	4.10

TABLE 4

Summary of P-Wave Velocity Measurements

Laboratory Trials (PUNDIT)

Sample Depth (ft)	Distance (in)	Time (us)	Velocity (ft/s)
2	9.252	73.6	10,475
3	6.130	46.3	11,033
4	15.729	121	10,832
6	5.857	53.7	9,089
7	14.536	115	10,533
9.5	5.799	50.6	9,550
10	11.625	105	9,226
Average			10,106

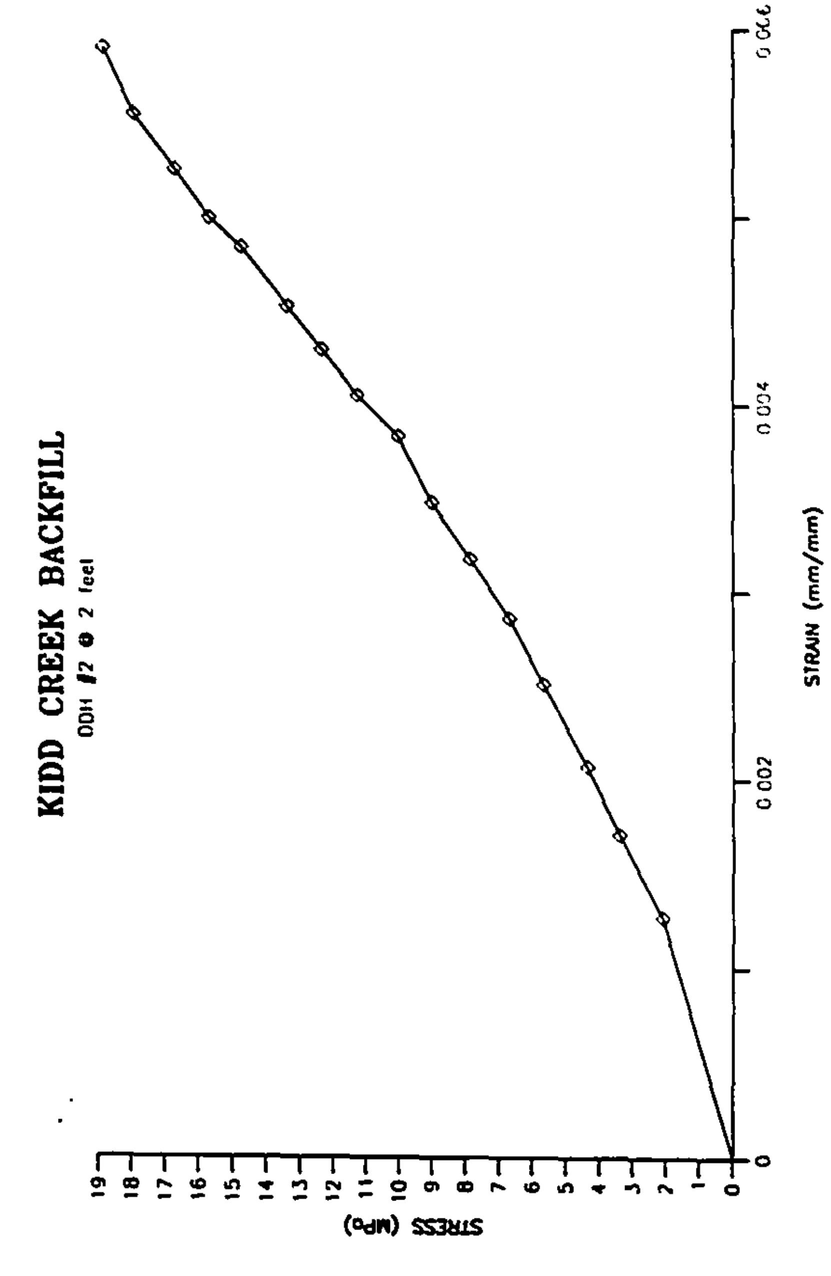


Fig: 8.4.1 Kidd Creek Backfill DDH #2 @ 2 feet

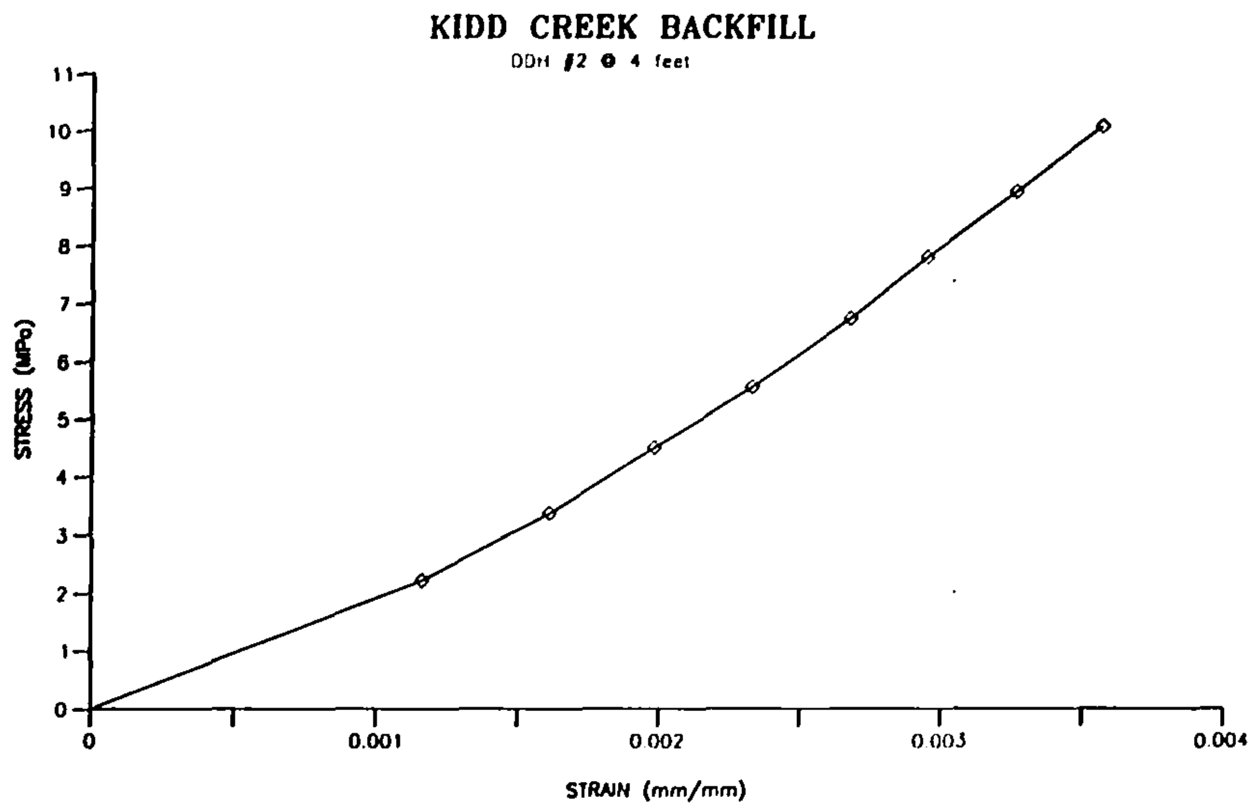


Fig: 8.4.2 Kidd Creek Backfill DDH #2 @ 4 feet

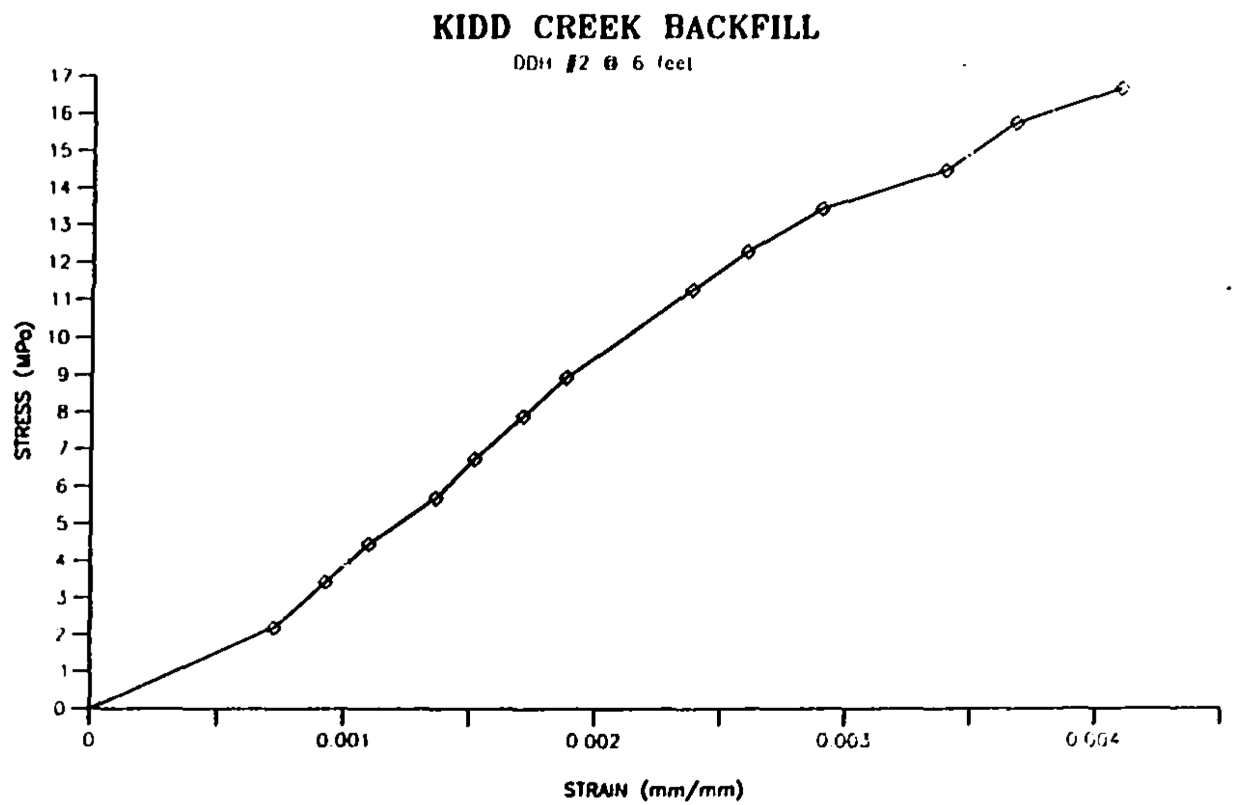


Fig: 8.4.3 Kidd Creek Backfill DDH #2 @ 6 feet

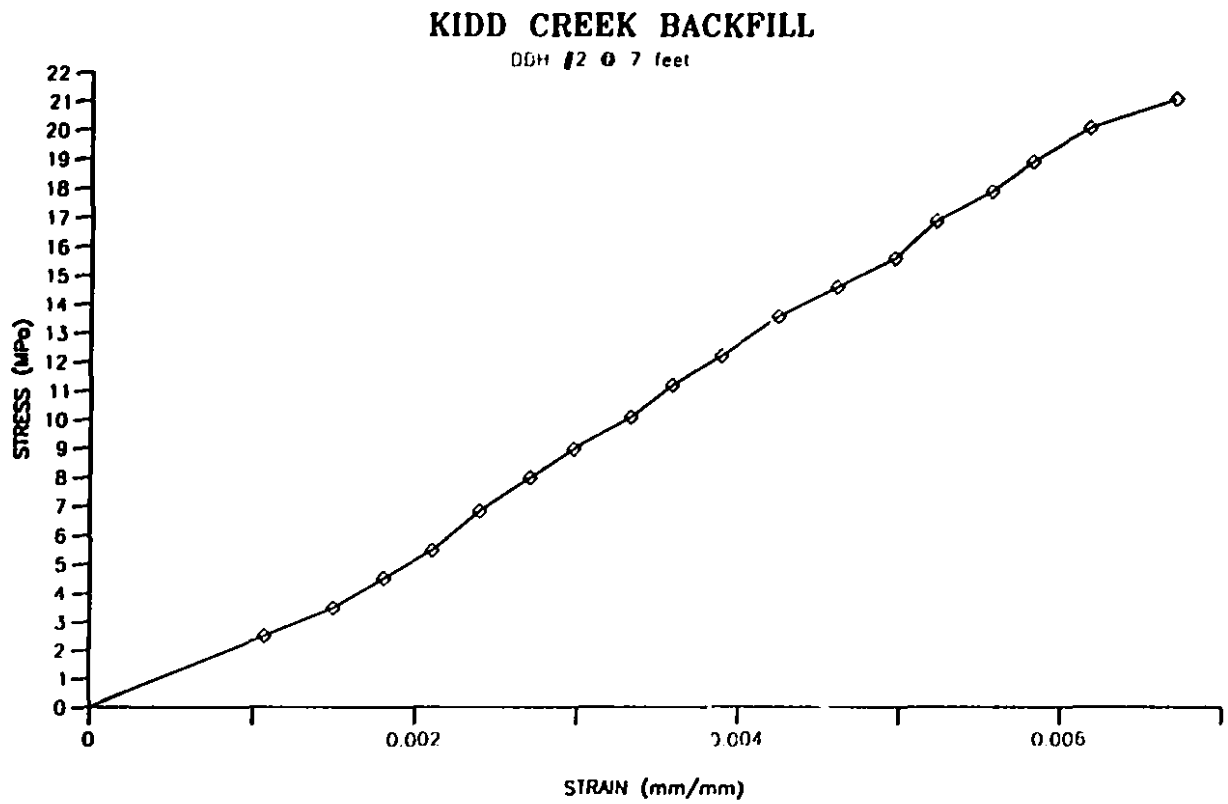


Fig: 8.4.4 Kidd Creek Backfill
DDH #2 @ 7 feet

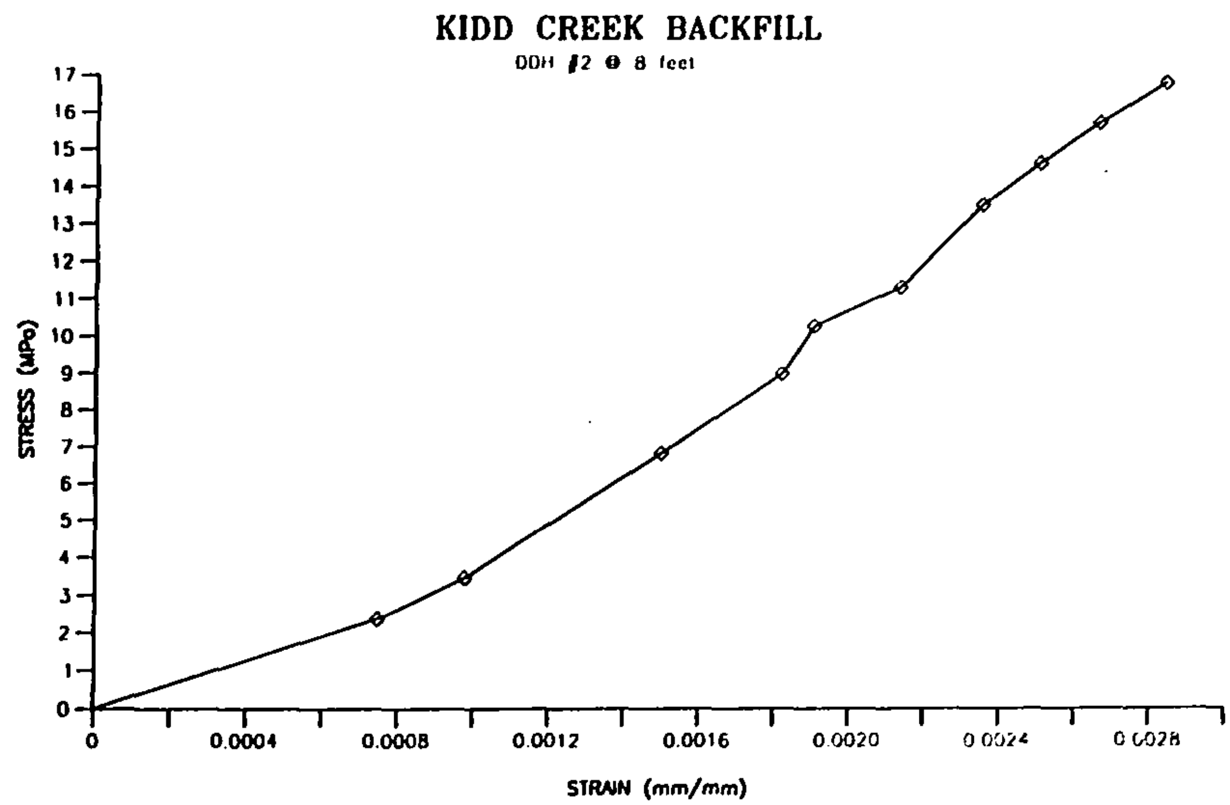


Fig: 8.4.5 Kidd Creek Backfill DDH #2 @ 8 feet

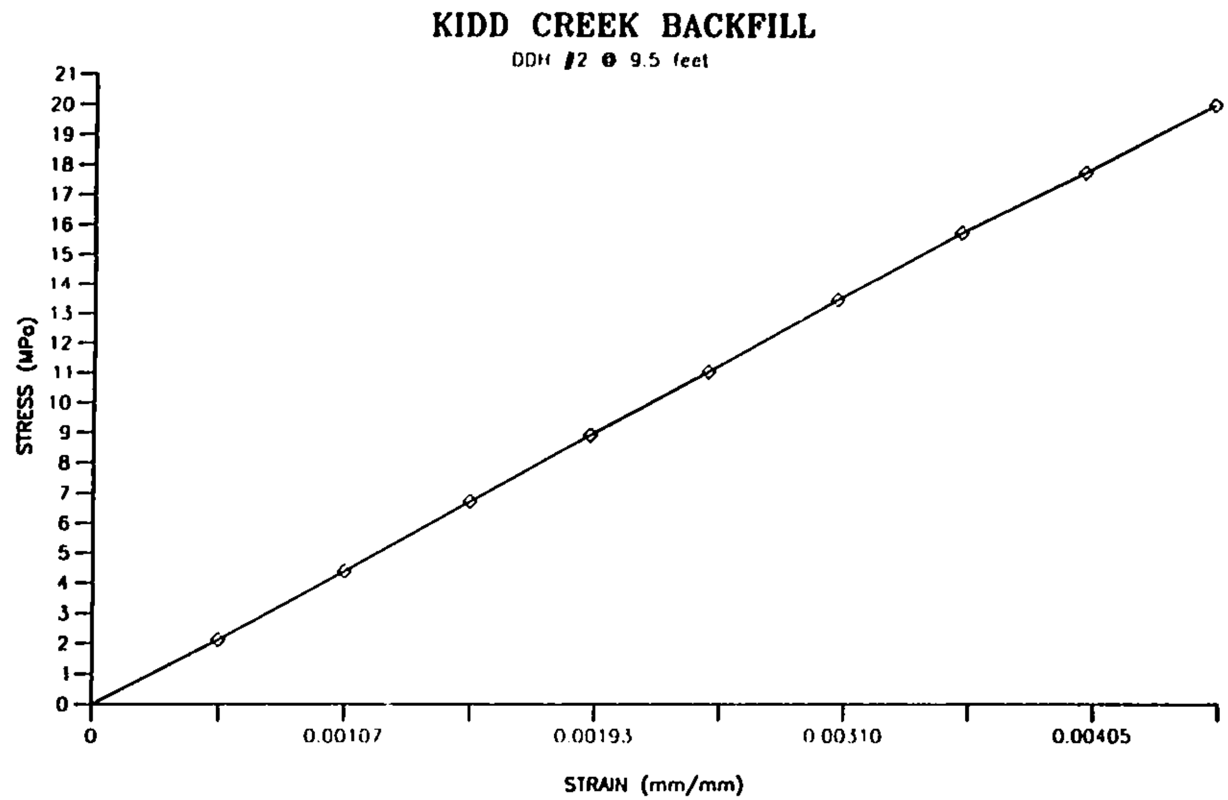


Fig: 8.4.6 Kidd Creek Backfill
DDH #2 @ 9.5 feet

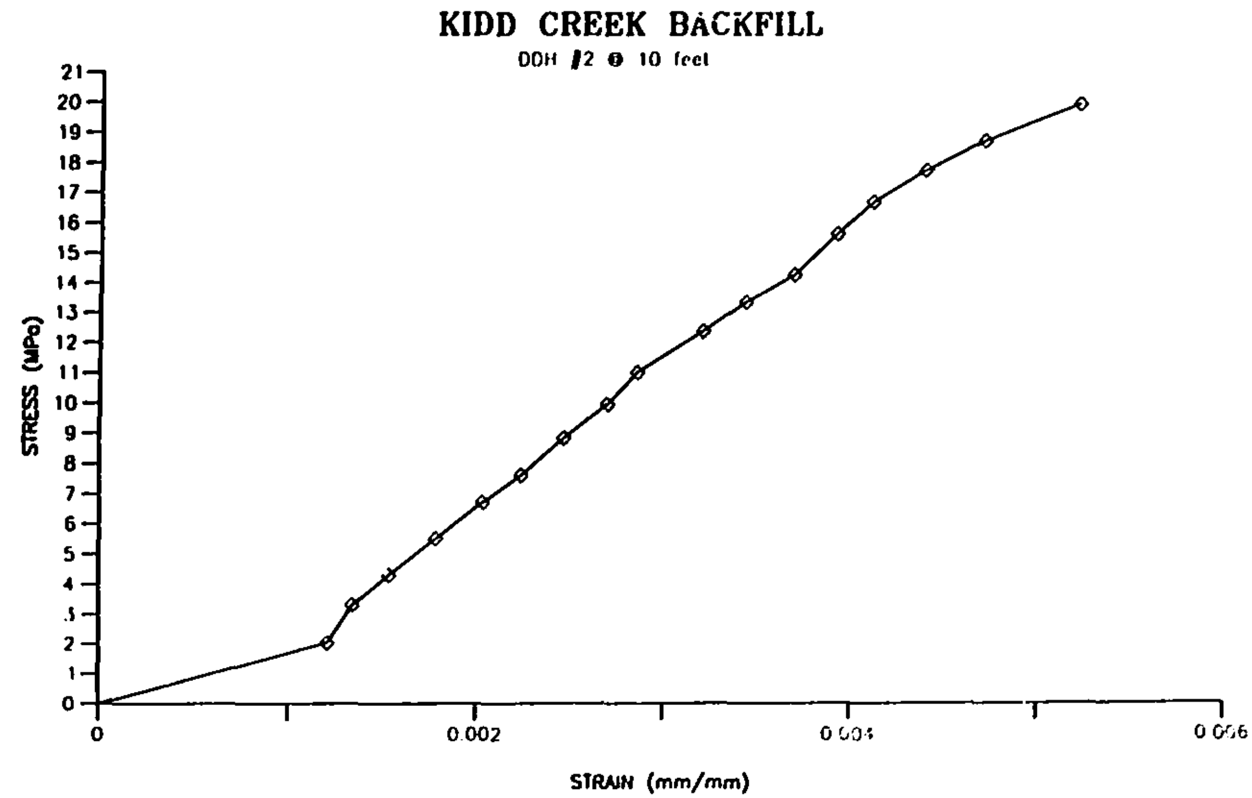


Fig: 8.4.7 Kidd Creek Backfill DDH #2 @ 10 feet

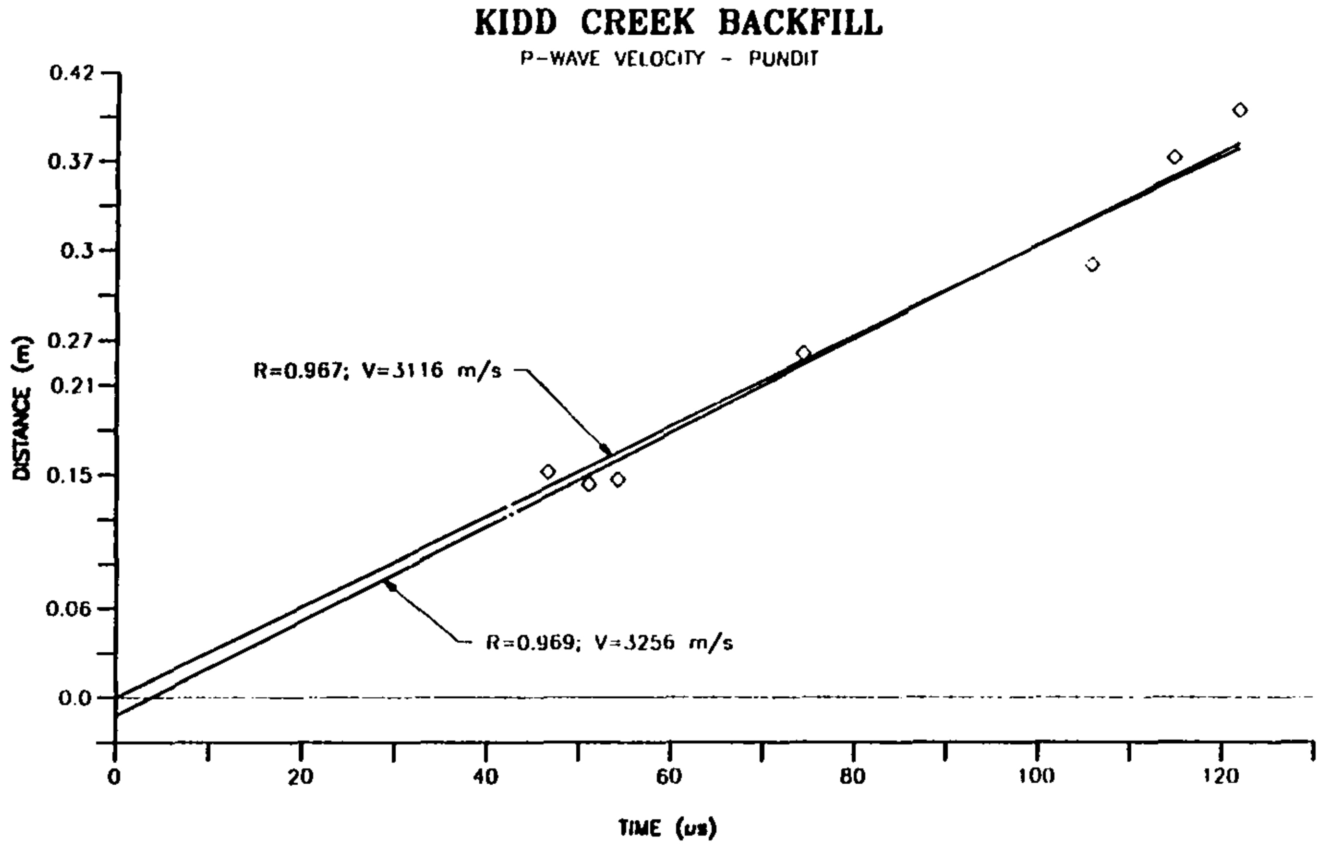


Fig: 8.5.1 Kidd Creek Backfill
P–Wave Velocity – Pundit

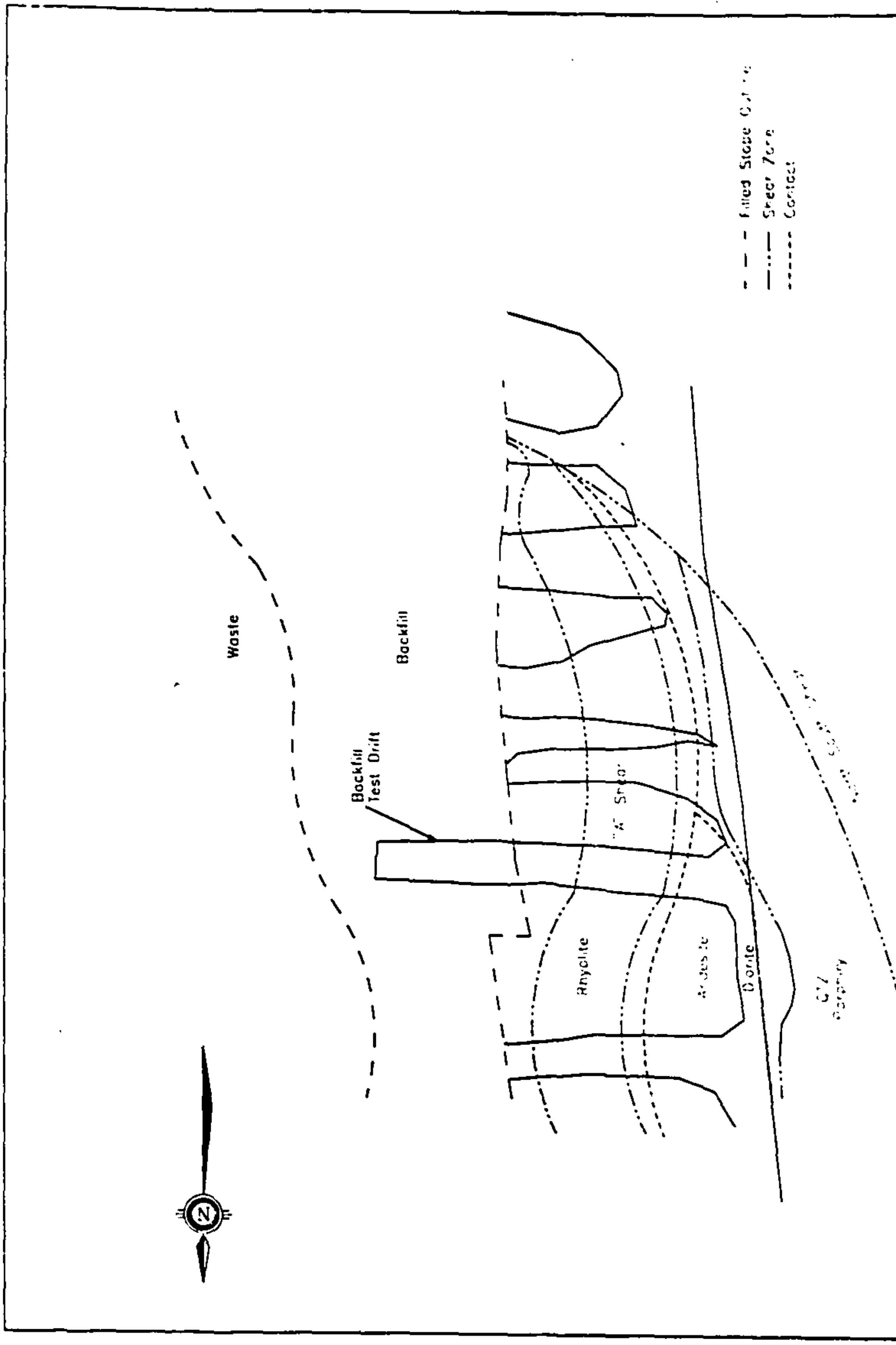


Fig: 8.5.2 12-2 Sub-Level

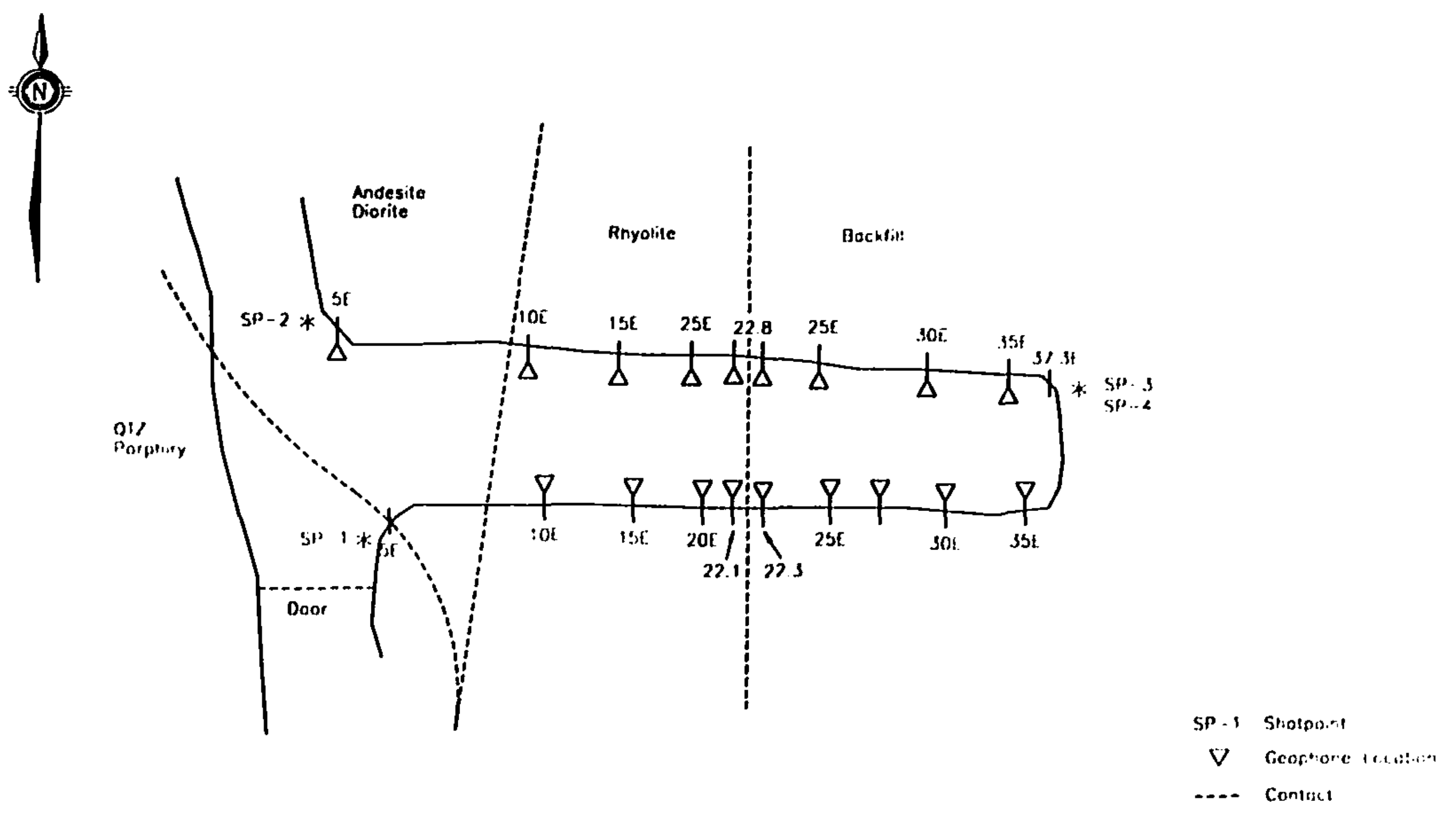


Fig: 8.5.3 12–2 CRF Test Drift

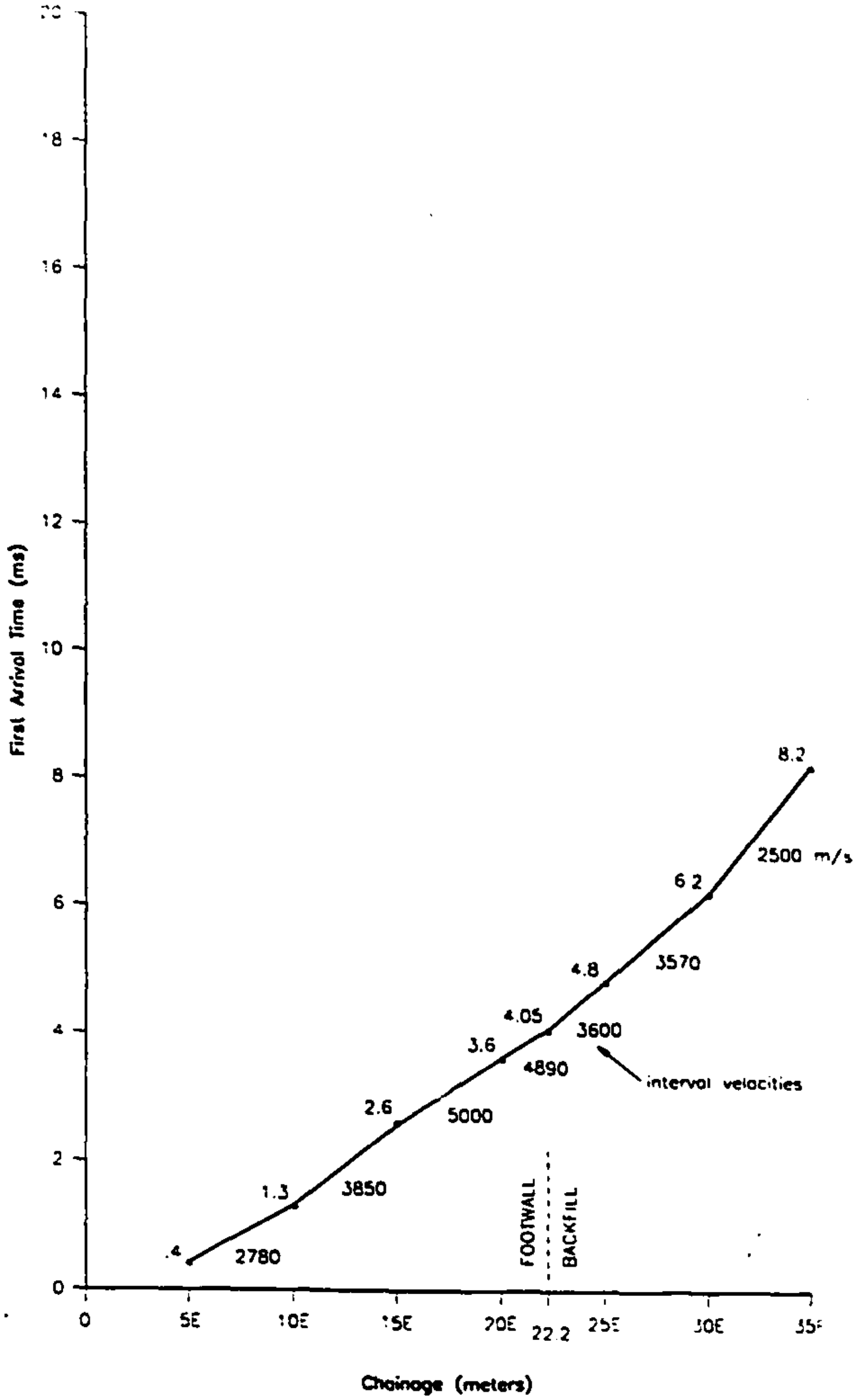


Fig: 8.5.4 12-2 CRF Test Drift
SP-1 South Wall
(Near 5E)

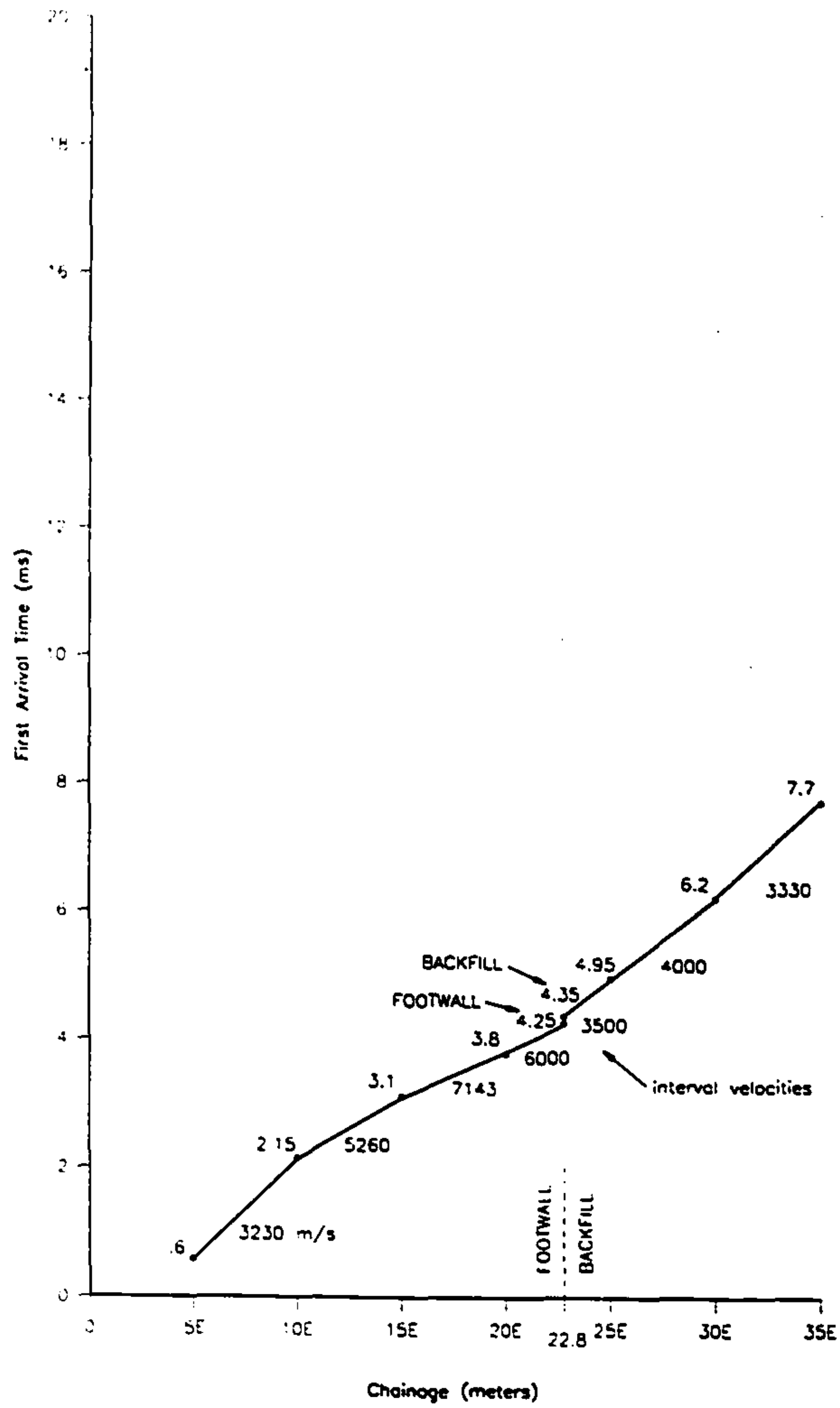


Fig: 8.5.5 12-2 CRF Test Drift
SP-2 North Wall

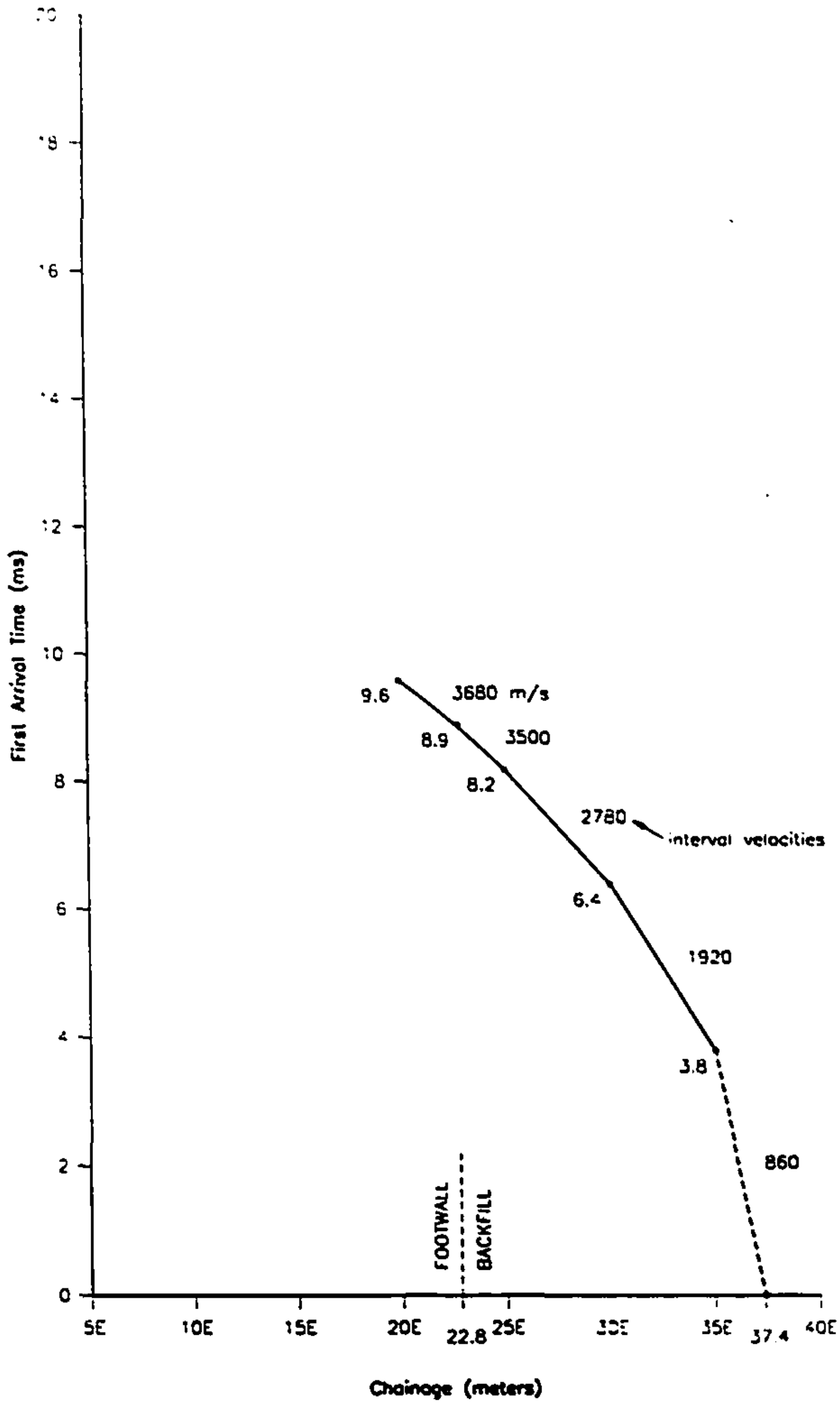


Fig: 8.5.6 12-2 CRF Test Drift
SP-4 North Wall

8.7: SUMMARY

In general, very little work has been done on the behaviour of a consolidated rockfill mass. The main reasons for the limited information are the degree of difficulty and the high cost involved in obtaining the in situ behaviour values. This chapter tried to review and repeat some of the past experiments at KCM and also investigated the new techniques that could be used for obtaining further information on behaviour of a typical CRF mass. Some of the typical values obtained in this chapter are the first for CRF mass and the results are as follow:

Bulk density:	1.88 tonne/m3.
Void ratio:	0.51 (ranges from 0.2 to o.55)
Moisture content:	3.5 %
Compressive strength :	1 to 17 MPa. Average 3-5 MPa
Elastic modulus :	0.6 to 4.5 GPa
Transmission coefficient from rock through CRF:	37%
Absorption coefficient in CRF :	0.41 db/m.
Poisson's ratio :	0.35
Friction angle :	37 deg.
Cohesion :	1.1 MPa
P- wave velocity:	3,150 m/sec

Above information was obtained from limited in situ work and one should use the values obtained with great care.

9.0: CRF OPERATING / CAPITAL COSTS

Chapters 4 to 7 concluded that high strength CRF mass is obtainable with proper structural design and quality control measures. However, economical consideration is as important as the strength requirements in selecting a fill type. The aim of this chapter is to breakdown the existing cost in fill operations and get average cost values for a typical fill system with emphases on cemented rockfill systems.

CRF has much superior strength characteristics compared to other fill types, but it also has a disadvantage of having a high capital cost. In this chapter cost modelling will be introduced and examined for both operating and capital costs. This chapter will also identify the high cost areas, such as binder cost, for all the fill types with emphases on CRF.

Three sources are used to collect the cost information to set up the cost models. First source is the survey done by Ontario Ministry of Labour on backfill in all Ontario mines (Campbell et al, 1987) and the second source is the survey done on all Quebec mines by Mr. D. Bois and McGill backfill group, which the author is a member of. The last source of data collection was through author's extensive site-visits to most of the backfill operations in Ontario.

9.1: COST BREAKDOWN

Traditionally unconsolidated fill was utilized in stopes to provide passive support by decreasing the ground movement around the filled stope. However, with addition of binding materials, higher pillar recoveries, lower mining cost, decreased ore dilution and improved ground control are possible. The total fill cost, capital and operating, depends primarily on the mining method and the role of backfill in the operation.

The operating cost compilation is based on 7 cost elements that can be compared from sites to sites. These are :

Material Cost: This item refers to the direct costs involved in the production of the backfill material required for the operation.

Haulage,S. (surface): This item refers to any kind of direct transportation cost involved in surface.

Slurry: This item refers to any direct cost related to the surface preparation of the fill material.

Haulage,U. (Underground): This item refers to any any direct cost related to underground work done to transport the backfill material to the stopes.

Bulkheads: This item refers to any direct cost related to the bulkhead construction.

Binding: This refers to the cost of any binding agent.

The cost survey included 15 Ontario mines and 18 Quebec mines. All the data from the Quebec mines are presented in Figures 9.1.1 to 9.1.3. However, the average cost for all the mines are given below:

	TAILINGS OR SANDFILL	**CUT AND FILL**
Cost in $/MT		
Material:	3.1+ 0.1 for 6 operations.	0.0
Haulage, Surface:	1.65 + 0.2 for 6 operations.	0.0
Slurry :	1.13 + 0.3 for 5 operations.	1.1 + .2
Placement:	0.5 + 0.2	8.7 + 4.2
Other:	0.1	0.55 + 0.0

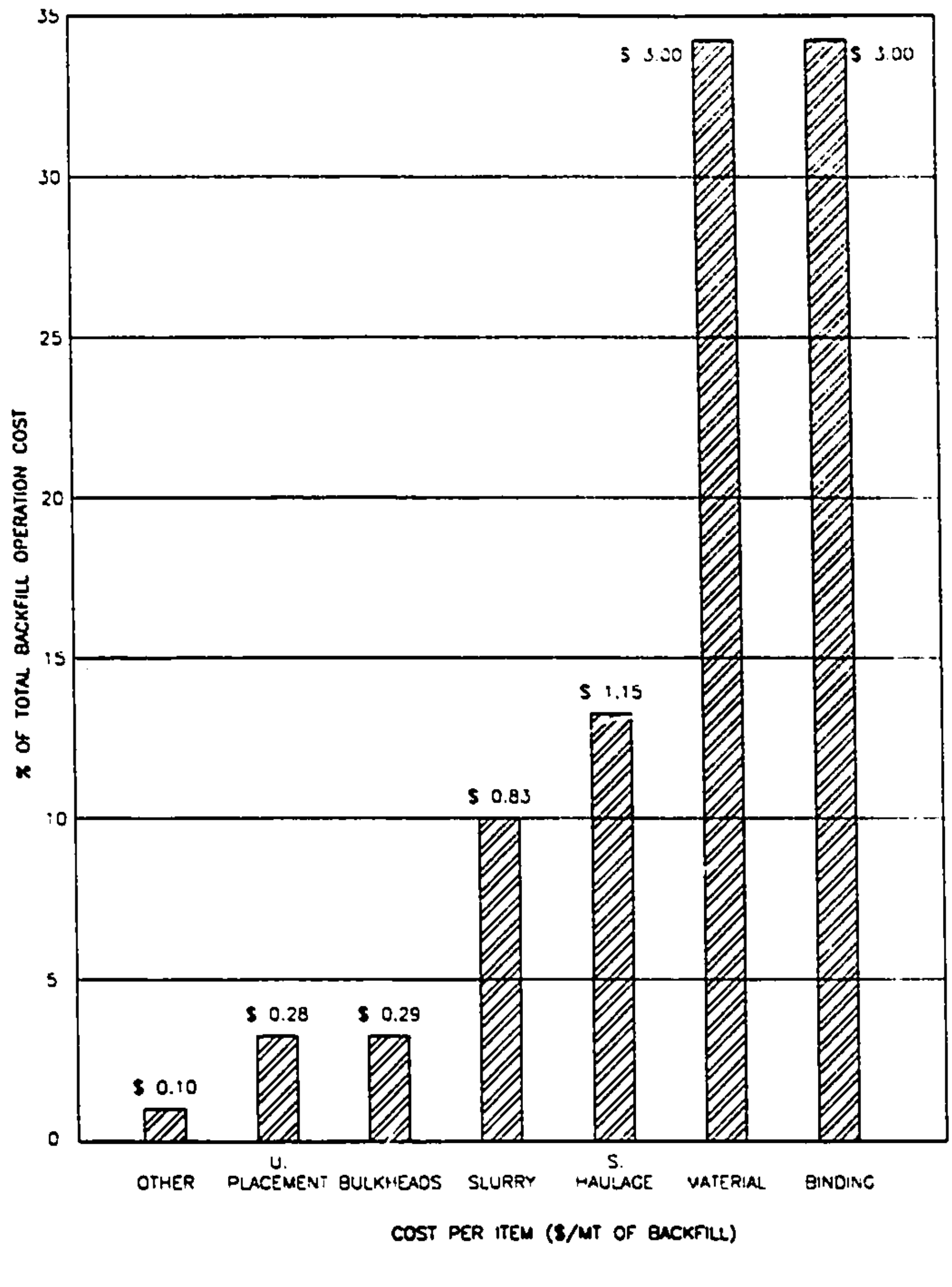


Fig: 9.1.1 Backfill Cost Distribution. Tailings Or Sandfill Operation.

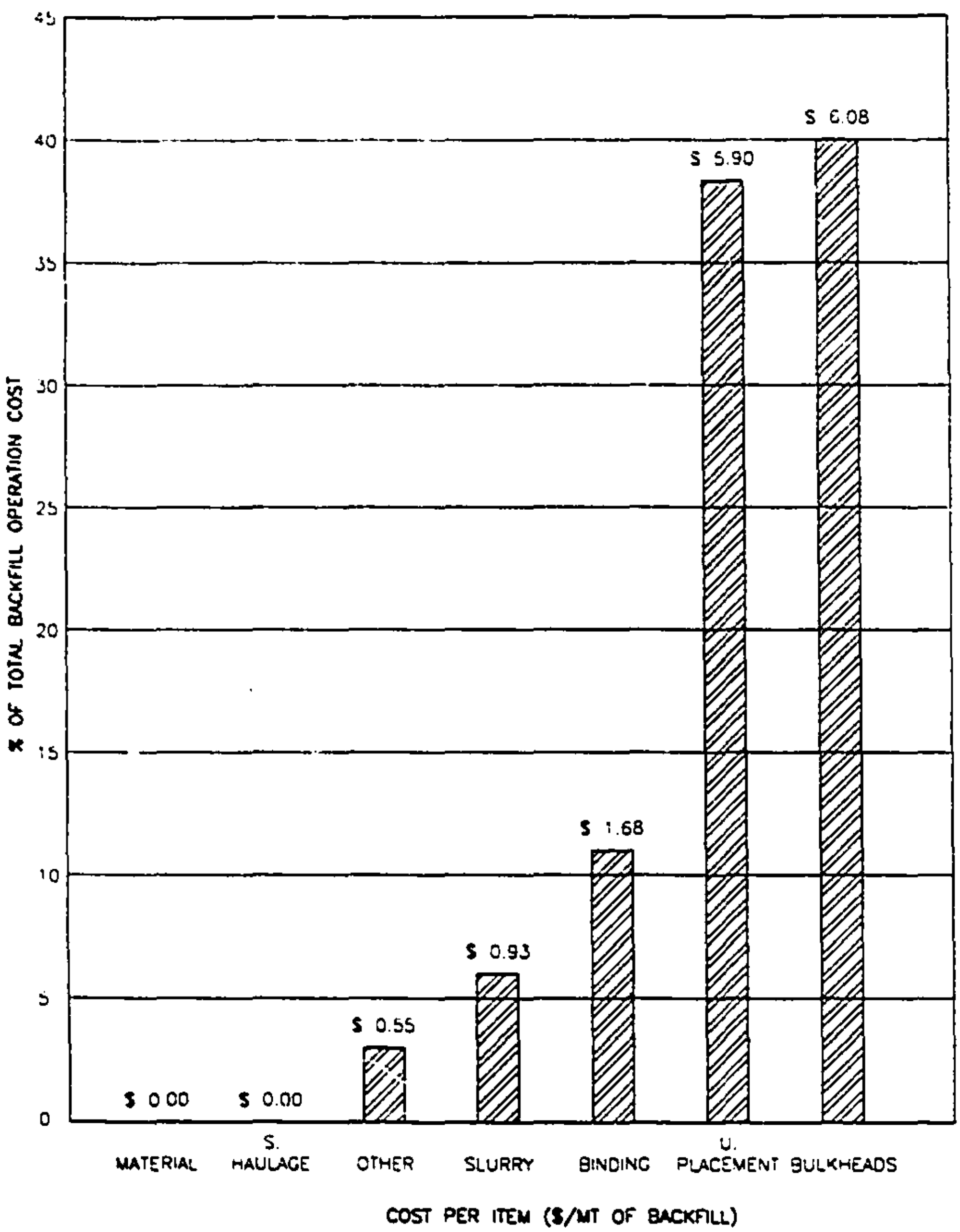


Fig: 9.1.2 Backfill Cost Distribution. Cut & Fill Operation.

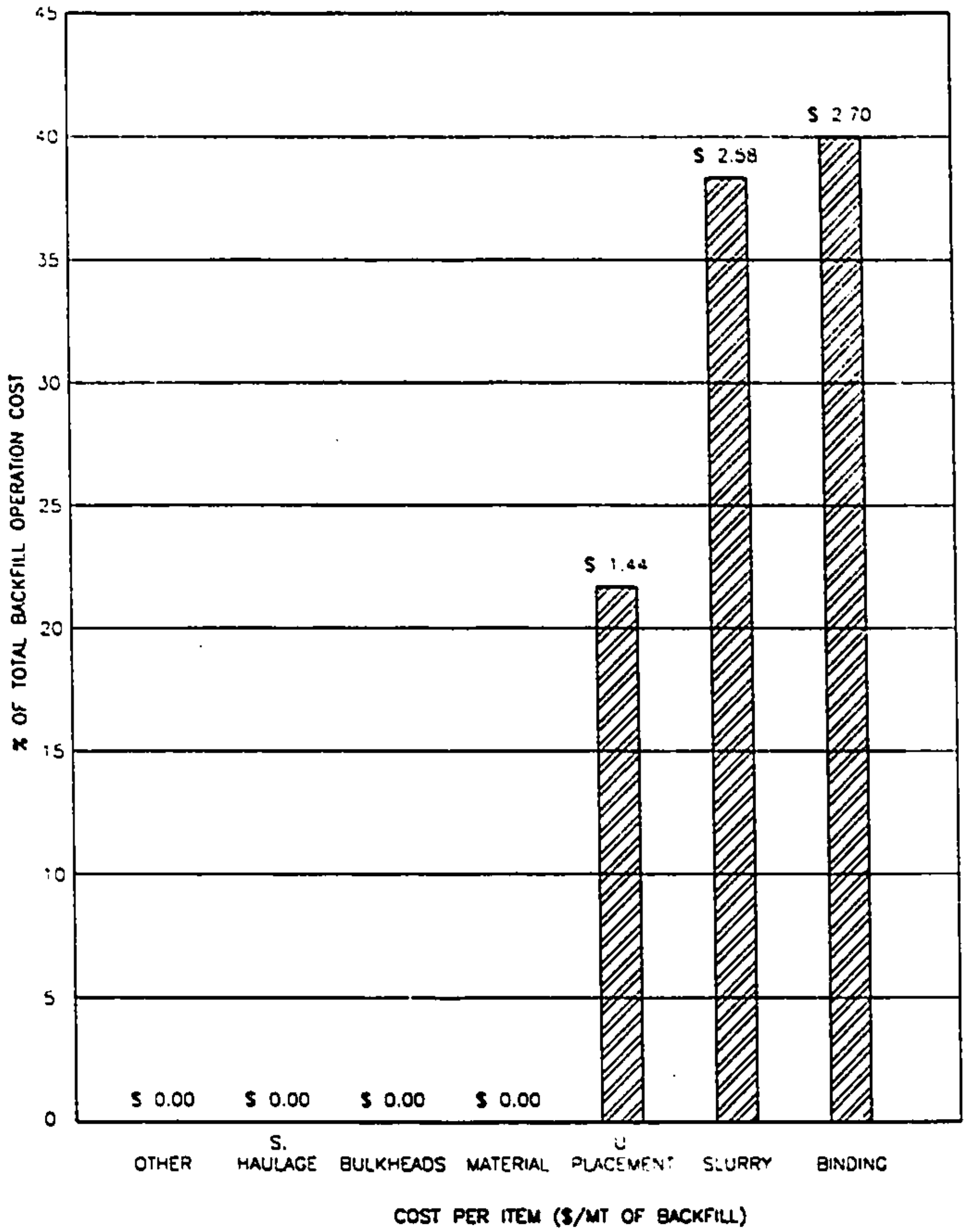


Fig: 9.1.3 Backfill Cost Distribution.
High Density Fill Operation.

Bulkheads:	0.56+ 0.17	10.8 + 3.7
Binding:	5.2 + 1.30 for 6 operations.	4.4 + 1.6
Total Cost: (MT):	**10.45 + 3.9**	**20.5 + 7.5**
Total Cost (M3) :	**15.2 + 5.0**	**39.8 + 13.1**

	CEMENTED ROCKFILL	ROCKFILL/ TAILING
Cost in $/MT:		
Material:	5.2 + 2.8 for 10 operations.	5.03 + 1.12
Haulage, Surface:	0.4 + 0.5 for 8 operations.	0.5 + 0.37
Slurry :	0.32 + 0.2 for 5 operations.	0.83 + .25
Placement:	2.5 + 0.1.4	2.82 + 1.2
Other:	0.38 + 0.2	0.3 + 0.19
Bulkheads:	0.3 + 0.15	0.31 + 0.15
Binding:	1.85 + 1.7 for 9 operations.	2.11 + 1.01
Total Cost: (MT):	**7.1 + 3.1**	**7.02 + 1.64**
Total Cost (M3) :	**13.8 + 4.3**	**14.45 + 3.45**

9.2: COST MODELLING

Using the data from the Quebec survey operating and capital cost models for all fill types were developed. Some part of the models were modified when Ontario mines were added to the data source. This study tried to model cost for 6 main backfill elements. These elements were: material, surface haulage, surface preparation (slurry), underground haulage, bulkheads, and binder cost. The degree of accuracy in each area is different due to the type and the amount of cost information received, however, the results show that the model estimation is within 20% of the actual operating cost in different operations. This model should be used by designer at the feasibility stage of selecting a fill type. The sample example at KCM, section 9.2.2, will show that the modelled cost is almost exactly the same as actual operating cost, except for the binder cost.

9.2.1: OPERATING COST MODELLING

The average results of the both Ontario and Quebec surveys, Figures 9.2.1 and 9.2.2, indicate following costs:

1: Backfill Material:

This depends where the aggregate comes from:

Surface pit waste: \$3.35 /MT (drilling, blasting, mucking)

Underground waste: \$7 /MT

Development waste: \$0

Surface stockpile : \$0

Surface stockpile if needs crushing: \$0.12 /MT

2: Surface Transportation.

Based on the transportation distance, as:

Distance > 1 Km

Surface transport (\$/MT) = \$1.08/MT + ((0.08/MT) x D) where D is in Km

Distance < 1 Km

Surface Transport (\$/MT) = \$0.2/MT + ((0.0008/MT) x d) where d is in m.

Both distances are total round trip distance.

3: Surface Plant Preparation:

The average value obtained was \$3.73/ MT of binder used.

As the average solid fraction in the slurry = 57%

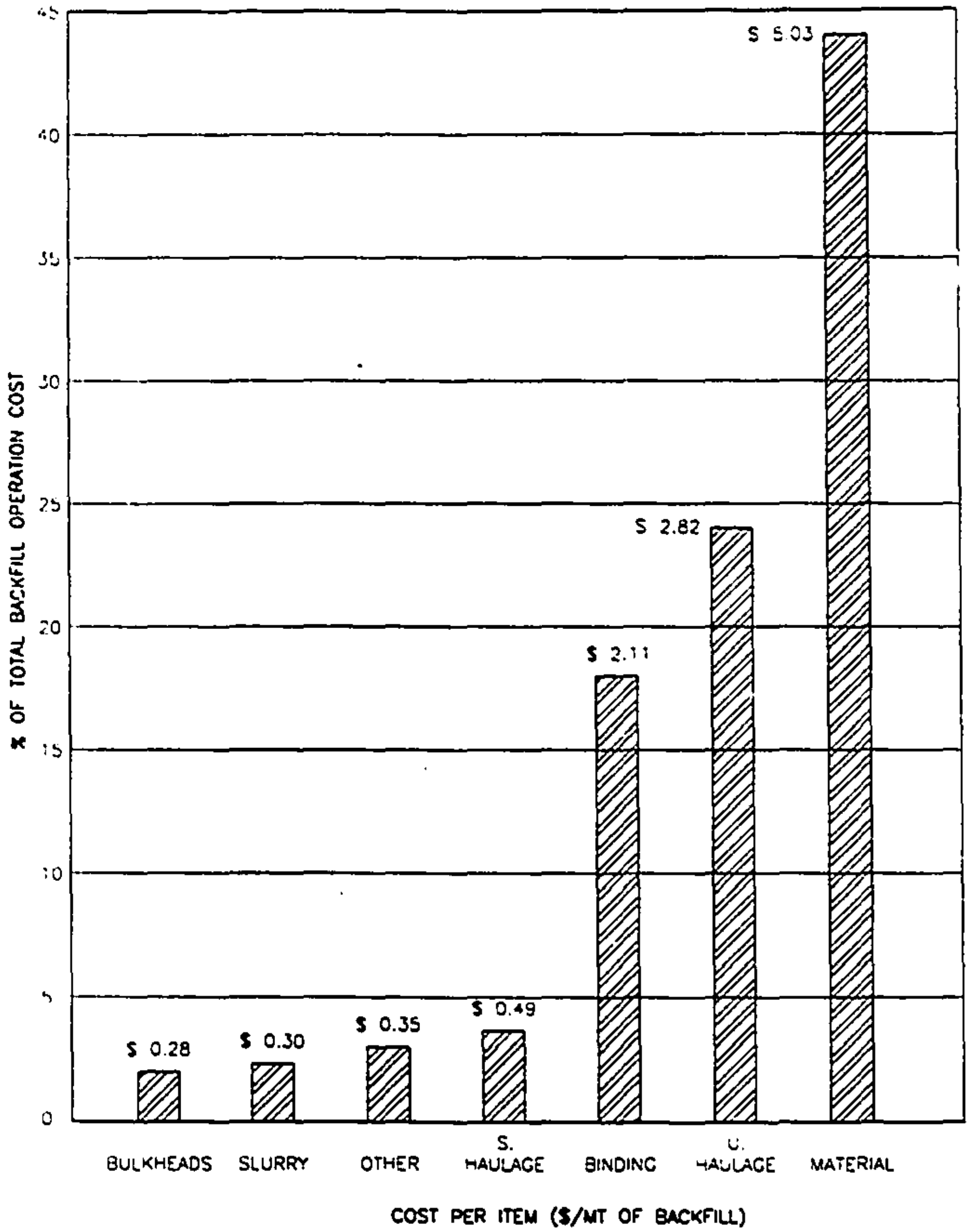


Fig: 9.2.1 Backfill Cost Distribution. Rockfill Operation.

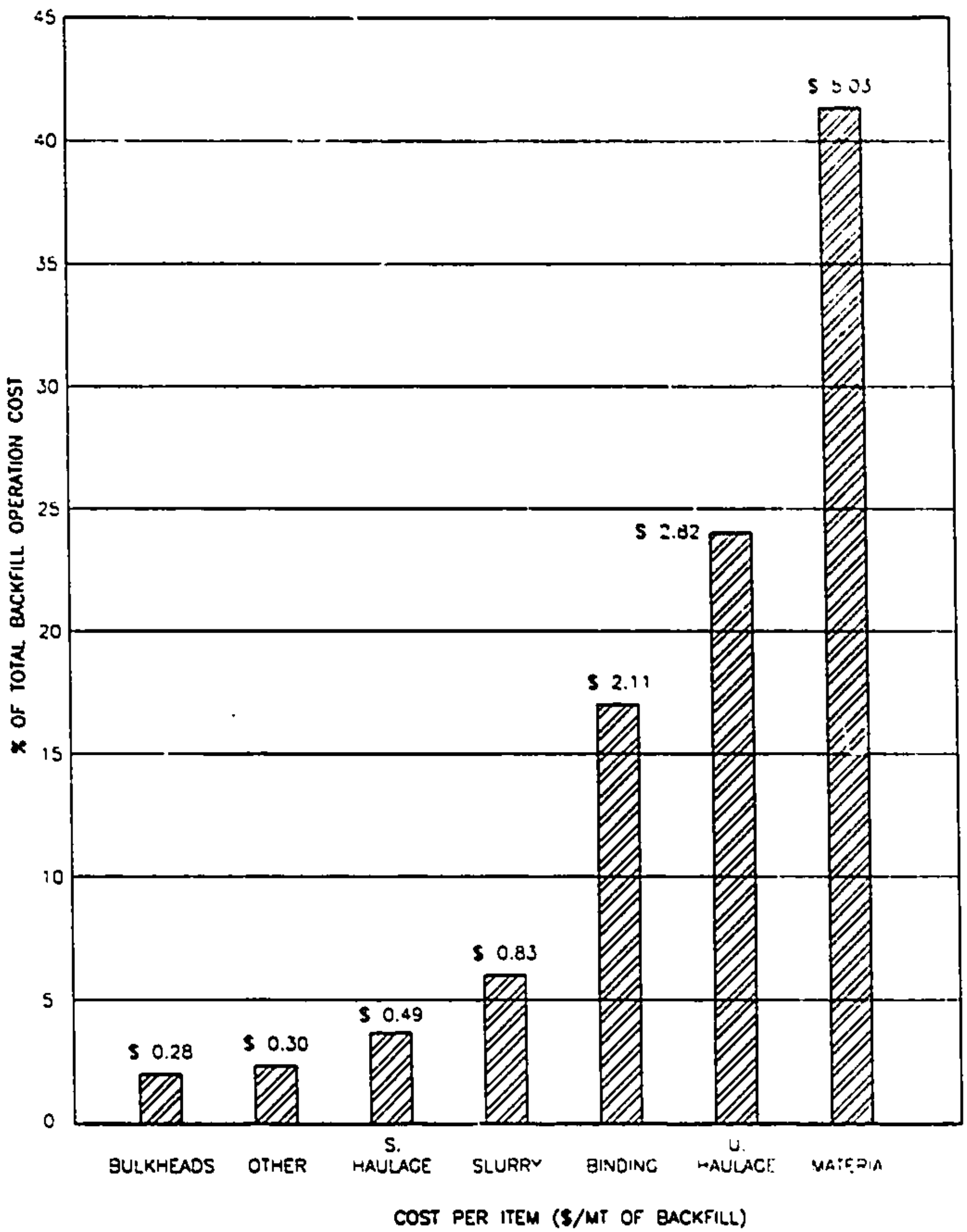


Fig: 9.2.2 Backfill Cost Distribution. Tailings & Rockfill Operation.

Slurry tonnage = Total Binding Tonnage / 57%

Surface Preparation Cost (\$/ MT) = 6.78 x (MT of slurry)) / (MT of backfill)

4: Underground Transportation

The average operation cost in different operations indicated:

U/G Transport (\$/MT) = (5.00 + (.01 x d)) 0.5 where d is total distance in meters.

5: Bulkhead Cost:

This depended mostly on the application and strengths required. The average cost was around \$3900 per bulkhead. The average cost was \$0.3/MT.

6: Other Cost

Other costs which included monitoring, dewatering and cleaning, can be estimated at:

Other cost average = \$0.35 per MT

7: Binder Cost

The cement cost is a function of the cement to solid ratio used for the fill. Following equation could give a good estimate of the binder % :

% Binder = 1.2 + (1.8 x Strength (Mpa)) Quebec Mines

% Binder = 1.2 + (1.6 x Strength (Mpa)) Ontario Mines

Depending on the mining method and mining sequence, small or large portion of the fill which will not be exposed in future mining, could be unconsolidated. This will significantly reduce the total amount of cement needed. The % of consolidated rockfill in the mines surveyed ranged from 30% to 95%.

The average cost of Portland, type 10, = \$103 per MT FOB.

The average cost of Flyash, Type C, = \$74 per MT FOB.

Average cement to flyash ratio was: 60% Cement and 40% Flyash.

Average cost (\$MT of Cemented fill) = \$93 x % binder

Major portion of the filling cost for a consolidated rockfill is the cost of binder materials which have to be added to the properly sized aggregate to produce the required fill strength. The cost and the amount of binder used in a stope could easily be calculated, but the calculations of the intangible benefits resulted from using consolidated fill such as, reduced mining costs, improved pillar extraction ratio and minimized ore dilution are much more complex to evaluate. The major economical saving for any consolidated fill system is to minimize the amount of the binding material required to produce the desired fill strength. The amount of binder material used for each stope may differ, and is related to the following : (1) Stope/ pillar grade and tonnages, (2) Stope/pillar mining costs, (3) Relationship between consolidated fill quality and % pillar recovery and (4) Production delays in case of fill failure.

Generally, for the recovery of a high grade pillar, every attempt should be made to minimize ore dilution caused by fill failure which could also cause production delays, by placing a higher than average cement content in the fill system. For a low grade pillar recovery where mining costs are important factors, the cement content of the fill is kept to a minimum.

Methods which yield a lower overall cement cost for a given fill strength are :

1) Increase mining cycle time with proper planning, which gives the binder material a longer curing time. The consolidated fill materials get stronger with time, although, with a decreasing rate. This would enable one to lower the overall cement content for the required fill strength. For example, consolidated rockfill using a blended binder of flyash and Portland cement at Kidd Creek Mines has 20 to 30% higher strength when the curing time has increased by a month, from 28 to 56 days curing intervals, estimated from laboratory results. The rate of

compressive strength increase depends mainly on the types of the cementing agents used.

2) Because of the lower costs of different local and/or commercially available pozzolanic materials, a portion of Portland cement could be replaced with these materials without sacrificing the strength and quality of the fill. In recent years pozzolanic materials such as flyash and slag have successfully replaced a portion of Portland cement in different fill operations resulting in significant savings.

9.2.2 : KCM OPERATING COST

Using the above model the cost for 7 main fill elements are estimated:

1: Material cost

95% surface stockpile that has to be crushed = $0.12/MT

5% development waste rock: $0/ MT

2: Surface Transportation cost

80% > 1Km

1.08 + (0.08 x 3) = $1.32 /MT

20% < 1KM, 800 m

0.2 + 0.64 = $0.84 /MT

This gives an average cost of $1.22/MT.

3: Surface Preparation

Pulp Density= 57% and 3.73/.57 = 6.55

1.9 million tonnes x 4.5% = 85,550 tonnes of binder

(6.55 x 85,550)/ 1.9 mil = $ 0.29 / MT

4: Underground Transportation

60% conveyors and 40% trucks.

Average cost for Conveyor = \$1.7/ MT

Cost for truck filling at average distance of 300 m = (5 + 0.01 x 300) 0.5 = \$2.82 /MT

Average cost = \$2.15 /MT

5: Bulkhead Cost: \$ 0.28/MT

6: Other Cost: \$ 0.35/ MT

7: Binder Cost:

At 2-3 MPa in situ strength the percentage of binder = 4.8%, however at KCM the binder percentage is at 4.5%. At 90% consolidated backfill and average binder cost per tonne of \$93:

Binder cost = (1.9 mil x .9 x .045 x 93) / 1.9 = \$3.76 /MT

Total Operating cost using the model = \$8.17/MT

Actual Operating Cost for 1994 : \$7.30/MT

Six out of seven items modelled are very close to actual cost. However, due to extensive binder research work at the mine, the average binder cost per tonne at is at \$75 compare to an average of \$93 for other operations. This is achieved by using the recipe developed by the author which is 60% replacement of Portland cement with Type C flyash and Hydrafil addition for achieving the required early strength. Considering this new value in the model :

At 90% consolidated backfill and average binder cost per tonne of \$75.2:

Binder cost = (1.9 mil x .9 x .045 x 75.2) / 1.9 = \$3.04/MT

Total Operating cost using the model = \$7.45/MT

Actual Operating Cost for 1994 : $7.30/MT

More case studies to compare modelled and actual costs are presented by Figure 9.2.3.

9.3: CAPITALIZATION COSTS

There are many cost factors that should be included in estimation of the capital cost of a fill system. This is varied considerably for different fill types and different operations for even the same type of fill. Some of the components of a typical rockfill system are: plant building, surface crusher, surface storage, surface handling and hauling equipment, binder storage silos, vertical passes, conveyor installation for fill delivery, backfill stations, tanks and agitators, slurry lines, electrical requirements, trucks or conveyors for fill delivery and others. See Figures 9.3.1 and 9.3.2.

It was very difficult to obtain average cost for all above since most of the information was not available or was not accurate. However using a limited information from Quebec mines a model was developed for capital cost estimation which is accurate within + 30%. The model is explained below:

1: Surface Plant

Capital cost to build a new slurry plant averaged $530,000 for a total backfill placement of 400,000 MT per year. This can be represented as:

Slurry plant capital costs: $66 x (total binder material used per year)

2: Surface Transportation

Surface transport unit : $0.45 x (average yearly tonnage) x (Dm /100) where Dm is the

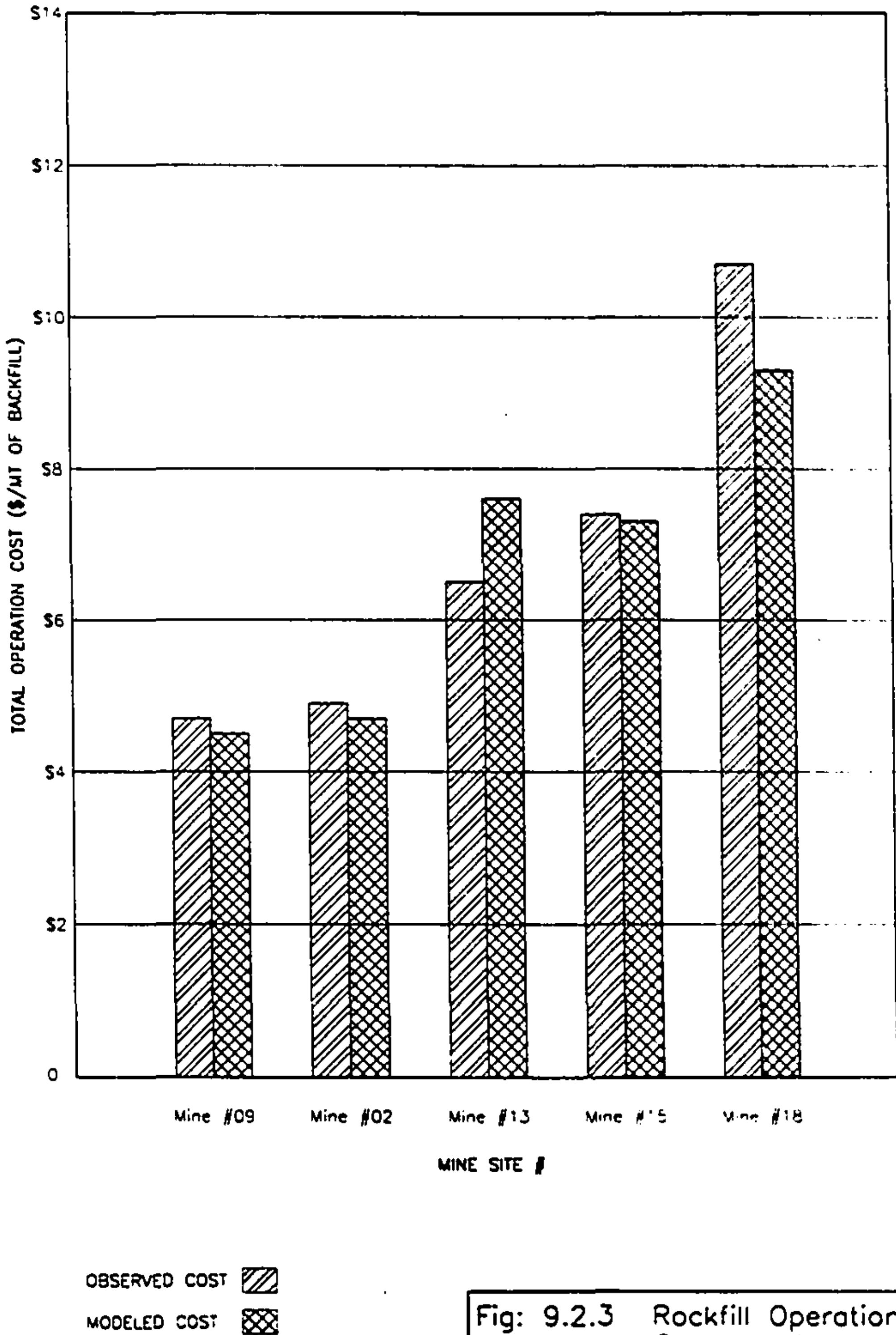


Fig: 9.2.3 Rockfill Operation Cost. Observed Vs Modeled.

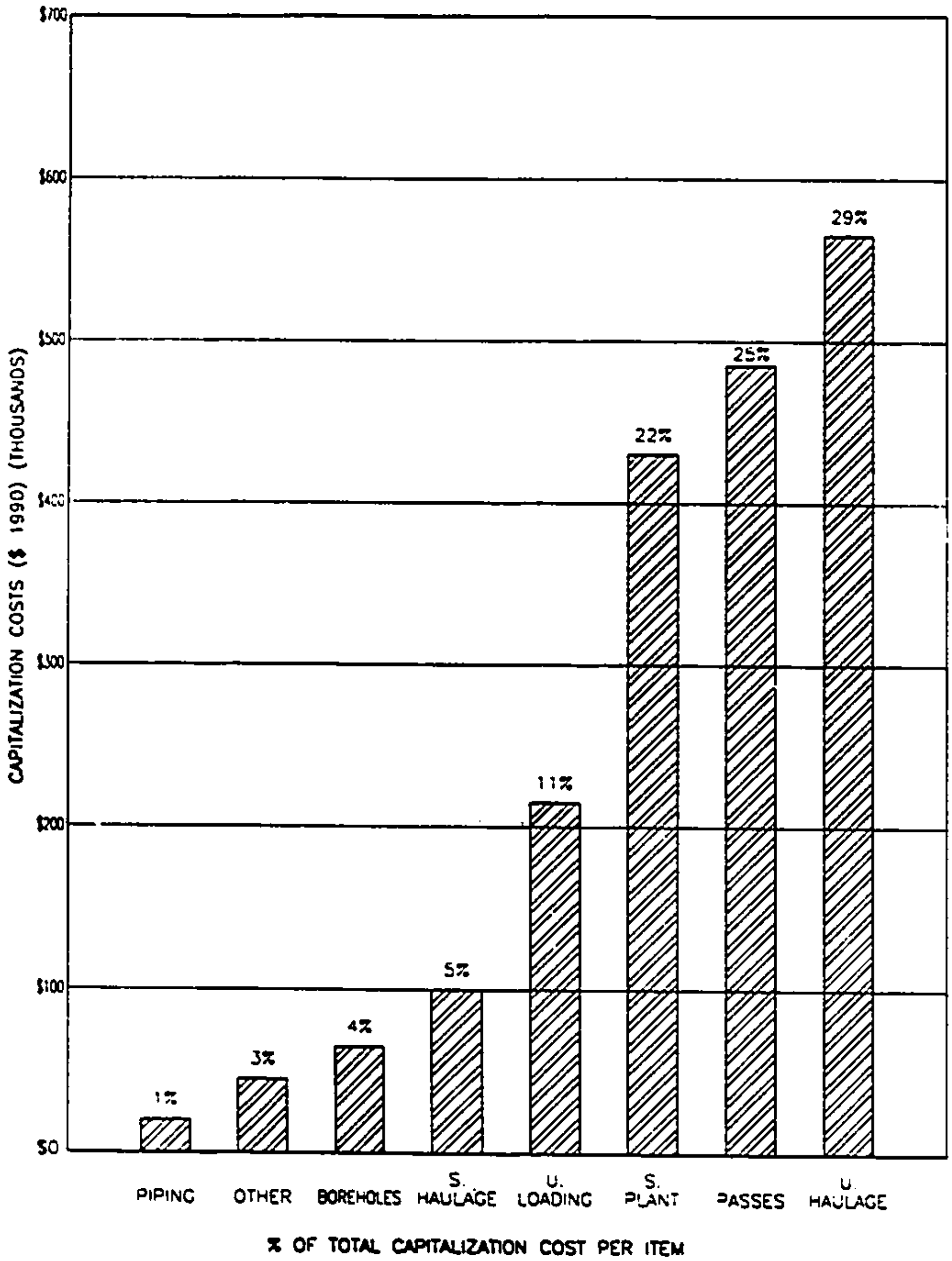


Fig: 9.3.1 Rockfill. Average Capitilization Costs.

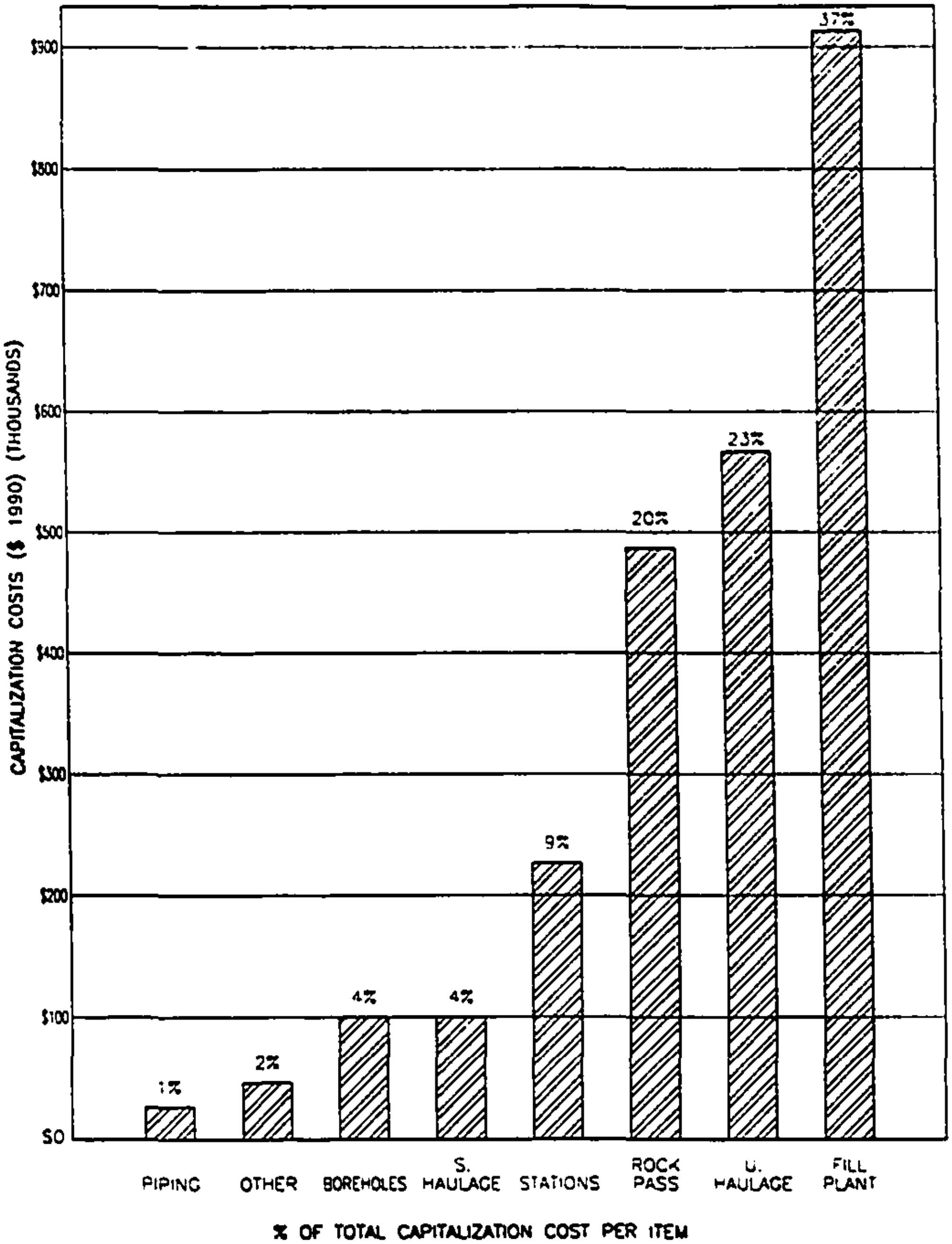


Fig: 9.3.2 Tailings & Rockfill. Average Capitilization Costs.

round trip in meters.

3: **Boreholes**: Average borehole cost was \$120 /meter.

4: **Piping**: Average piping costs = \$20/ meter

5: **Waste rock pass**: Average cost = \$925/meter

6: **Loading Stations**: Average cost was \$53.500 / loading station.

7: **Underground transportation unit:**

This was effected by the distance between loading station and stopes to be filled. The productivity of the trucks could be estimated from following relationship:

Productivity (MT/hour/unit) = { 1/[0.015 + (2 x 10-5 x Dm)]}

Then depending on the tonnes required per hour. the number of trucks can be calculated. For smaller trucks, less than 16 tonne, an average of \$250,000 per unit could be used. However for larger trucks, 26 tonne or more, and average cost of around \$400,000 could be used.

9.3.1: CASE EXAMPLE

Type of fill = Rockfill

Amount of Fill /year = 350,000 tonnes

Transport = 400 m round trip underground.

Percentage of consolidated fill at 5% binder = 70%

Waste rock pass = 1000 m

Boreholes = 2000 m (one as backup)

Piping = 2000 m

Loading stations = 6

Underground haulage trucks: daily tonnage of around 1500 tonnes is required to achieve 350,000 tonne of fill per year.

Considering the productivity:

1/ (0.015 + (2 x 10-5 x 400)) = 43 tonne/hr/unit x 6 (working hour) = 260 tonne/shift

Number of unit required = 1500 / 260 = 5.8 = 6 trucks

The Capital cost could be estimated as:

Slurry plant = 350,000 x .7 x .05 = 12,250 = \$66 x 12,250 =\$808,500

Surface transportation = 0.45 x 350,000 x 400/100 = \$630,000

Boreholes: 2000 m x \$120 /m = \$240,000

Piping: 2000 m x \$20 /m = \$40,000

Waste rock pass = 1000 m x \$925/ m = \$925,000

Loading stations = 6 x \$53, 500= \$321,000

Underground transport = 6 trucks x 250,000 = \$1.5 million

Other = \$ 51,000

Total Capitalization costs = \$4,464,500

For more information on averaged and modelled cost for other operations refer to Figure 9.3.3.

9.4: SUMMARY

As expected, the cost information from different fill types indicates that one of the biggest disadvantage in using CRF is the high initial cost. The high capital cost makes the fill type suited to only bulk mining methods where the rate of backfilling is a key element in maintaining the production. CRF, however, had the lowest operating cost and the highest strengths obtained. The higher static and dynamic strengths of the CRF would translate to significant indirect cost savings, such as, allowing faster mining rate, minimized dilution, improved ground control, lower mining cost and minor operational delays.

Operating and capital cost models presented in this chapter matched well with the actual costs at KCM and they could be great tools in estimating the fill costs.

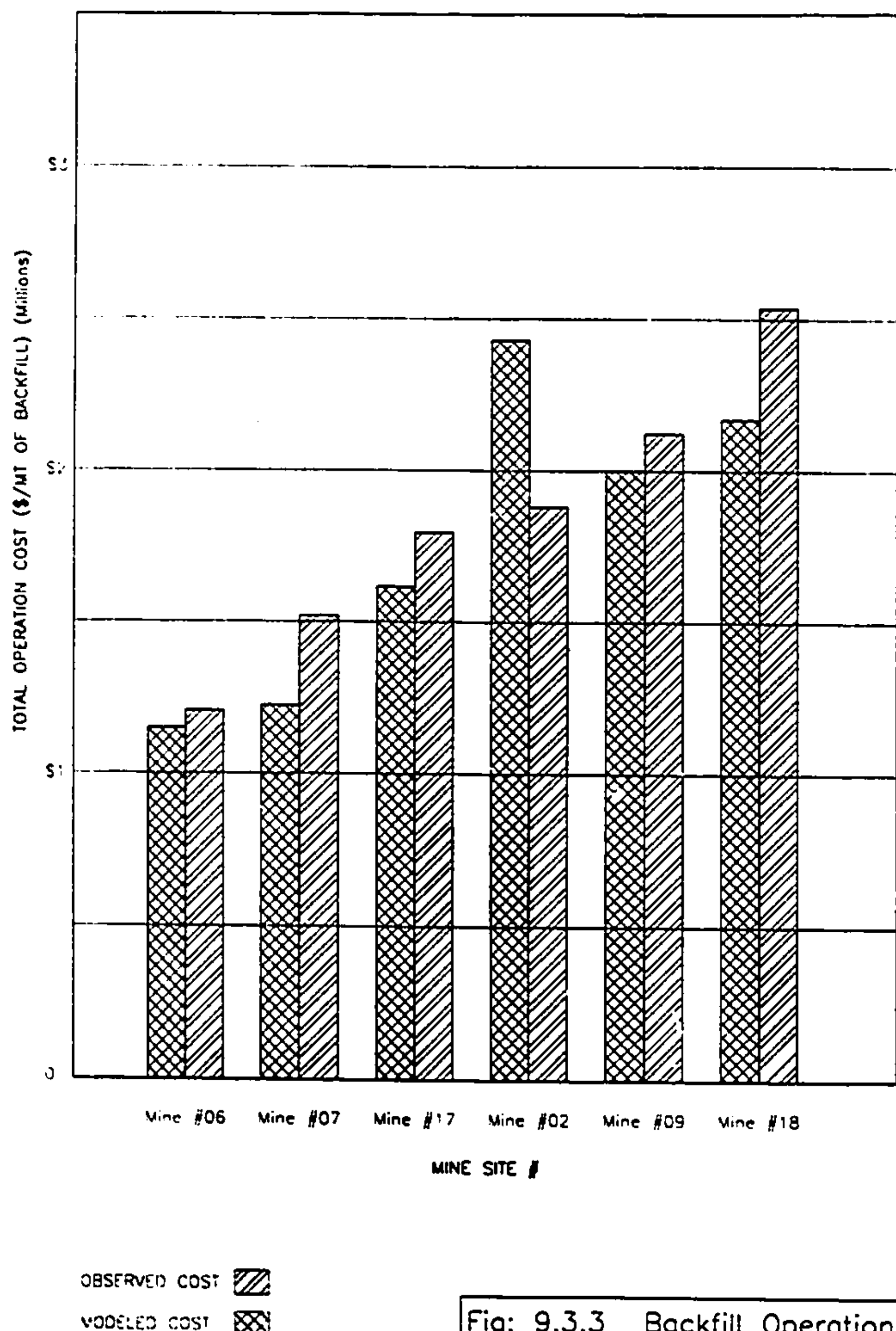


Fig: 9.3.3 Backfill Operation Cost. Observed Vs Modeled.

10: DESIGN OPTIMIZATION AT KCM

Cemented rockfill at the KCM represents annual cost of approximately 17 million dollars, 18% of the total extraction costs. Cost cutting initiatives however have to be mindful of the negative if not disastrous effects on grade, recovery, and ground stability that a decline in fill quality can produce. This dictated that any attempt to cut costs should be approached in a scientific and orderly fashion. This chapter summarizes the steps taken to ensure placing high strength CRF at lowest possible cost.

10.1: DESIGN SYSTEM

Optimum design of a CRF mass can be very complex and clear and logical path for the design improvement is required. This chapter presents a newly established CRF design system, Figure 10.1. The steps shown in this design system could and should be used at any stage of the CRF process to improve the existing and/or future rockfill systems.

Based on extensive literature review and extensive site investigation which included, filed mapping and actual stope history and filling observations, four main parameters effecting CRF design were identified and further investigated. The four areas were identified due to their importance in achieving high strength fill and lack of available information on extent of their contribution in assuring a properly engineered CRF system. The areas identified with the most strength improvement and cost saving potentials were: to improve engineered structural design process, increase usage of lower cost alternative binders, establish quality control measures, and carry out in situ testing for back analysis.

10.2: STRUCTURAL DESIGN

Structural design improvement is a very important part of the fill cycle. This is needed to predict, place, and obtain competent rockfill mass where it will be exposed in future mining. The ultimate goal is to minimize and control segregation, hence minimize dilution and mass and/or local failures.

To improve the design process more in situ observations and investigations were required. Actual behaviour and zoning of the placed CRF were required to allow a meaningful and properly engineered CRF design. After extensive drift mapping and underground inspections, chapter 4, four distinct zones in a typical CRF mass were identified.

ZONE A: The wall on which a collision occurred and below the impact zone was like a concrete mass and had the highest in-situ strength. This also included the zone 5 to 10 meters away from all the fill peaks observed, when the fill did not collide with the wall. This area had high binder content, 7- 8%, and high uniaxial compressive strength, approximately 5-8 MPa. The aggregate sizing contained around 90% minus 7.6 cm. Minimized segregation was observed, except some small zones of coarse aggregate which probably were formed during the mass flow after the fill build up slope was grater than the angle of repose of the fill. Excellent coating of aggregate was noticed, almost 90 to 95% success.

ZONE B: This area had medium binder content, 3-5%, and uniaxial compressive strength of 2-3 MPa. This area had a good blend of coarse and fine aggregate and covers anywhere from 10 to 25 meters away from the fill peak and/or impact point. Some segregation was noticed, especially after 20 meters away from the fill cone. Approximately 80% of the aggregate mass was coated with slurry.

ZONE C: At the stope boundaries the fill was highly segregated and had a low binder content of 1-2% and compressive strength of 1-2 MPa. The lower strength in this area is due to the lack of fine particles, which were needed to avoid point contact between coarse particles and achieving higher tensile strength. The extremely segregated and weak zone typically started around

25-30 meters away from the fill cone. This zone contained almost all coarser aggregate and mostly the same size aggregate. Roadheader had great difficulty going through this zone and in some cases the drift did not advance any further when encountering this zone.

ZONE D: At the fill toe area the aggregate was also highly segregated and depending on fill raise orientation could be rich in binder content from the slurry running down towards the slope of the fill cone(s). Since the aggregate was mainly coarse the excess slurry did not increased the CRF strength considerably.

After identifying the above zones and considering other key factors for a CRF mass design such as: orientation of backfill raise(s), extent of segregation, degree of success in mixing the aggregate with slurry, the extent of impact damage due to free fall height, and the size of aggregate entering the stope, the ideal fill raise orientation for different mining conditions were also established. The details are presented in chapter 5. This will assist the designer in obtaining the high strength fill, zones A and B, against the walls that future pillar recoveries are planned to be carried out. Having high strength fill, zone A, at all the walls to be exposed could be very costly mostly due to the number of raises required. The information could be used to design individual stopes keeping in mind: grade, dilution effect, operational delays, and overall stope sequencing.

10.3: BINDER ALTERNATIVES

Since binder usage is around 80-90% of a typical rockfill operating cost, excluding labour cost, the establishment of an optimum binder combination in any mine is a must and could yield the most financial and/or strength benefits. One of the easiest ways to reduce filling cost is to reduce the amount of binder used or increased use of lower cost binders. However, if the approach is not in a scientific fashion, it could have disastrous effect on the efficiency and profitability of the operation.

A comprehensive testing program, 1750 specimens, was conducted to evaluate the cementitious properties of different binders with or without the addition of commercially available chemicals, chapter 6. Increasing the early strengths of the flyash/P.C. mix and copper slag/P.C. mix through the use of different binder and/or chemicals were also extensively studied. This testing program resulted in the change of KCM backfill recipe which has resulted in 10% reduction in binder consumption and annual savings of $1.3 million on binder cost alone.

Using the results of the test program, the KCM did substantially reduced the cost of cement rockfill over the past 3 years. This was done by both, substituting lower cost binders, and reducing the overall binder content by utilizing the higher strengths of the new binder mixes. In addition, a locally available material, slag from the KCM copper smelter was identified as having potential for possible future use as a binder substitute.

At KCM, the old recipe of 60% Portland cement and 40% flyash at 5% binder by weight was changed to a new recipe of 40% Portland cement and 60% type C flyash at 4.5% binder. This was accomplished with no sacrifice in strength due to addition of 0.2% of Hydrafil, a superplasticizer, from Grace Chemicals.

Other mixes with potential were:

The addition of anhydrite, gypsum and/or lime to slag/P.C. and flyash/P.C. control mixes increased the compressive strength in almost every combination tested. The best result was obtained when 3% anhydrite was added to slag/P.C. control mix which caused strengths increase of 217% and 240% after 28 and 56 days of curing respectively. The same strength increase was achieved when 1% anhydrite and 0.65% lime were added to the slag/P.C. control mix.

Testing of $CaCl_2$ with slag/P.C. mix indicated that 2% $CaCl_2$ increased the early strength of the mix by 20%. This indicated that $CaCl_2$ could be used if a faster mining cycle is required.

10.4: QUALITY CONTROL

Under controlled condition, a typical rockfill mass has much superior physical and mechanical properties, compared to other fill systems. Closely monitored and properly engineered quality control measures were established at KCM which were followed by operations people. These measures were taken to achieve the highest possible fill quality at the lowest possible cost.

Quality control measures were established in plant, during transportation and placement. Some of the key findings were:

In the fill plant on surface, quality control measures should be taken to minimize the adverse effects of aggregate attrition and excess aggregate moisture content. Attrition would cause a great deviation between the original size of the aggregate on surface and the size of aggregate received underground. By estimating the attrition effect, crushing and blending could be utilized to produce a quality, graded product, suited to requirements at a certain depth. To control the attrition effect, the initial aggregate size could be larger than the required aggregate size for stopes at certain depth. However, a more common method of decreasing the adverse effect of attrition is to reduce the fine fraction by screening out the excess fines in the final mix underground. Adequate blending of coarse, midds and fine material in the backfill aggregate would translate to placing a higher density fill with less voids and, in turn, reducing cement requirements.

The moisture content of the stockpiled aggregate should also be closely monitored. A slight change of moisture content in the aggregate may affect the fill quality significantly. If the aggregate is sent underground wet or becomes wet on route, the result is that the fines will coat the coarser particles preventing the bonding of coarser aggregate and cement.

All the devices in the surface fill plant for monitoring material consumption, rates of transfer and also storage quantities, should be accurately calibrated and the calibration checks should be carried out in short and regular intervals.

The main quality control measure with hydraulic transportation is maintaining proper pulp densities for cement slurry. The water content in the slurries should be minimized to obtain the highest pulp densities possible. The quality control measure when using conveyor transportation method is to closely observe the sizing of the aggregates. This enables the underground operator to act rapidly if any process changes may be required, for example adding extra slurry if the aggregate on the belt is too fine.

When filling a stope, segregation of consolidated rockfill is unavoidable but it can be minimized if the fill operation is well planned and closely monitored. For each individual stope following factors should be considered when structurally designing backfill : orientation of backfill raise(s), extent of segregation, degree of success in mixing the aggregate with slurry, the extent of impact damage due to the free fall height and the size of the aggregates entering the stope.

The most critical portion of a backfilled stope is the wall(s) which will be exposed for future pillar recovery. The fill raise (s) should be placed such that the fill will collide as high as possible on the wall(s) of which is required to stand. This will utilize the phenomena that the wall below the impact zone is the strongest wall. In stopes where more than one wall will be exposed for future pillar recoveries multiple raises should be used. By having multi raise system the fill could have a more uniform distribution and a more controllable fill profile. The filling of the stope should alternate between the raises. This practice will result in a more uniform distribution of cementitious material in the stope and also a much stronger fill at the boundaries of the stope which is desirable for future blasting.

The fill quality and segregation extent in boundaries of each stope are related to the rolling distance that the coarse particles have to travel before colliding with the walls. If the orientation of the fill raise (s) does not allow the fill material to collide with the stope wall, the dip of the fill raise(s) should position the cone area as close as possible to stope walls that are to be exposed in future pillar recovery.

If all stope walls are to be exposed for future pillar recoveries, then multiple vertical raises yield the best possible distribution of the fill materials. The vertical raises would produce a more horizontal fill profile and any excess cement slurry will accumulate at the stope boundaries. If vertical raises are not possible due to the available access, then the number of inclined fill raises should be increased to produce a competent fill at the required stope walls.

The stope geometry plays an important role in decreasing segregation effect. The shape of the stope and the orientation of the fill raises dictate the rolling distance of the material after the impact with the fill cone. If the distance is kept small, such as in a square shape stope with centrally located fill raise, then the coarse particles have a chance to rebound back to the vicinity of the impact area after colliding with the stope wall. This could result in a more uniform fill distribution and a more horizontal fill face in the stope.

All the quality control measures are presented in chapter 7.

10.5: IN SITU TESTING

Very little work had been done on the in situ behaviour of a consolidated rockfill mass. Optimum fill quality at the lowest possible cost can only be achieved by back analysis and continuous improvement of the existing fill system using actual in-situ values.

Extensive in situ test program was carried out to 1: To establish physical and mechanical properties of consolidated rockfill and 2: To evaluate the potential techniques for short interval in situ quality control measurements. The test program included: ground stress, ground movement, blast vibration, and seismic measurements. Pressuremeter and core testing were also carried out. All the details are given in chapter 8.

Results indicated that the stope wall expanded adjacent to the mined-out area and opposite in the solid ground away from the opening. After each blast expansion continued throughout the wall opposite to the blasted area. Most of the recorded expansion took place after major slot and/or ring blasts. The data revealed that, after fill was placed, the expansion of the wall halted and inward movement began for up to 0.9 mm in the monitored stope. This implies that the placed CRF developed an active pressure state compressing the relaxed stope wall, as observed by the stress change.

Pressuremeter in situ testing indicated a general decrease in bulk modulus as holes moved away from the footwall, impact area. The modulus of deformation was typically in the order of 1.5 GPa to 4 GPa in the first drilling ring, zone A of the fill mass. This decreased to the o.1 GPa to 0.2 GPa range in the second drilling ring and was further reduced to 0.007 GPa to o.04 GPa range. The vertical holes indicated a greater variation in the data than the horizontal holes. This was to be expected as the vertical holes were more likely to traverse various fill layers than the horizontal holes.

10.6: SUMMARY

Improvement in all areas effecting CRF performance, especially the four areas mentioned above, were investigated at KCM and the results from the studies were implemented with great success. The extensive field work accompanied with author's experience and extensive consultation with other operations resulted in establishing CRF design system steps that are presented in Figure 10.1.

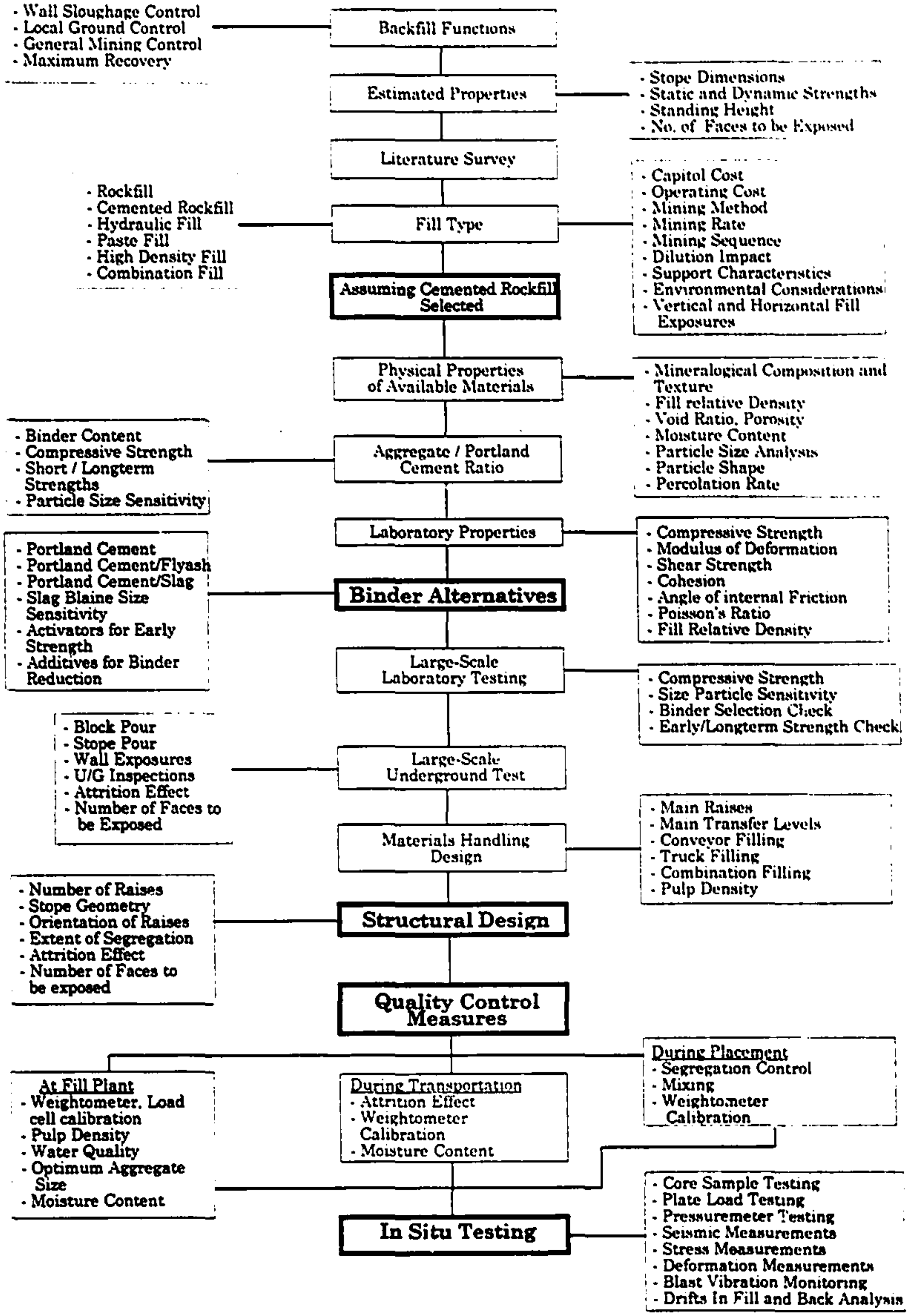


Fig: 10.1 CRF DESIGN SYSTEM

11: CONCLUSION

Optimum design of a CRF mass can be very complex and clear and logical path for the design improvement is required. This thesis presented a newly established CRF design system. Although most of the work to develop the design system was carried out at KCM, the steps shown in this design system could be used at any stage of the CRF process to improve the existing and/or future rockfill systems.

Based on extensive literature review and extensive site investigations which included, filed mapping, actual stope history and filling observations, four main parameters effecting CRF design were identified and further investigated. The four areas were identified due to their importance in achieving high strength fill and lack of available information on extent of their contribution in assuring a properly engineered CRF system. The four areas identified with the most strength improvement and cost saving potentials were: to improve engineered structural design process, increase usage of lower cost alternative binders, establish quality control measures, and carry out in situ testing for back analysis.

To improve structural design, after extensive tunnel mapping and underground inspections, chapter 4, four distinct zones in a typical CRF mass were identified:

ZONE A: The wall on which a contact occurred and below the impact zone was like a concrete mass and had the highest in-situ strength. This also included the zone 5 to 10 meters away from all the fill peaks observed, when the fill did not collide with the wall. This area had high binder content, 7- 8%, and high uniaxial compressive strength, approximately 5-8 MPa. The aggregate sizing contained around 90% minus 7.6 cm. Minimized segregation was observed, except some small zones of coarse aggregate which probably were formed during the mass flow after the fill build up slope was grater than the angle of repose of the fill . Excellent coating of aggregate was noticed, with almost 90 to 95% success.

ZONE B: This area had medium binder content, 3-5%, and uniaxial compressive strength of 2-3 MPa. This area had a good blend of coarse and fine aggregate and covers anywhere from 10 to 25 meters away from the fill peak and/or impact point. Some segregation was noticed, especially after 20 meters away from the fill cone. Approximately 80% of the aggregate mass was coated with slurry.

ZONE C: At the stope boundaries, the fill was highly segregated and had a low binder content of 1-2% and compressive strength of 1-2 MPa. The lower strength in this area is due to the lack of fine particles, which were needed to avoid point contact between coarse particles and achieving higher tensile strength. The extremely segregated and weak zone typically started around 25-30 meters away from the fill cone. This zone contained almost all coarser aggregate and mostly the same size aggregate. Roadheader had great difficulty going through this zone and in some cases the drift did not advance any further when encountering this zone.

ZONE D: At the fill toe area, the aggregate was also highly segregated and depending on fill raise orientation could be rich in binder content from the slurry running down towards the slope of the fill cone(s). Since the aggregate was mainly coarse the excess slurry did not increased the CRF strength considerably.

Above information helped to establish ideal fill set ups for different stope conditions, chapter 5. Some of the findings were:

1: When filling a stope, segregation of consolidated rockfill is unavoidable but it can be minimized if the fill operation is well planned and closely monitored.The most critical portion of a backfilled stope is the wall(s) which will be exposed for future pillar recovery. The fill raise (s) should be placed such that the fill will collide as high as possible on the wall(s) of which is required to stand. This will utilize the phenomena that the wall below the impact zone is the strongest wall. In stopes where more than one wall will be exposed for future pillar recoveries multiple raises should be used. By having multi raise system the fill could have a more uniform distribution and a more controllable fill profile. The filling of the stope should alternate between the raises. This practice will result in a more uniform distribution of cementitious material in the stope and also a much stronger fill at the boundaries of the stope which is desirable for future blasting.

2: The fill quality and segregation extent in boundaries of each stope are related to the rolling distance that the coarse particles have to travel before colliding with the walls. If the orientation of the fill raise (s) does not allow the fill material to collide with the stope wall, the dip of the fill raise(s) should position the cone area as close as possible to stope walls that are to be exposed in future pillar recovery.

3: The stope geometry plays an important roie in decreasing segregation effect. The shape of the stope and the orientation of the fill raises dictate the rolling distance of the material after the impact with the fill cone. If the distance is kept small, such as in a square shape stope with centrally located fill raise, then the coarse particles have a chance to rebound back to the vicinity of the impact area after colliding with the stope wall. This could result in a more uniform fill distribution and a more horizontal fill face in the stope.

The above information will assist the designer in obtaining a high strength fill, zones A and B, against the walls that future pillar recoveries are planned to be carried out.

To investigate the increased use of lower cost binders, a comprehensive testing program, casting 1750 specimens, was conducted to evaluate the cementitious properties of different binders with or without the addition of commercially available chemicals, chapter 6. Increasing the early strengths of the flyash/P.C. mix and copper slag/P.C. mix through the use of different binder and/or chemicals were also extensively studied.Using the results of the test program, KCM has substantially reduced the cost of cement rockfill over the past 3 years. This was done by both, substituting lower cost binders and reducing the overall binder content by utilizing the higher strengths of the new binder mixes. In addition, a locally available material, slag from KCM copper smelter was identified as having potential for possible future use as a binder substitute.

At KCM, the previous mixture of 60% Portland cement and 40% flyash at 5% binder by weight was changed to a new mixture of 40% Portland cement and 60% type C flyash at 4.5% binder. This new recipe resulted in 10% reduction in binder consumption and annual savings of $1.3 million on binder cost alone.This was accomplished with no sacrifice in strength due to addition of 0.2% of Hydrafil, slurry dispersant, from Grace Chemicals.

Also additions of anhydrite, gypsum and/or lime to slag/P.C. and flyash/P.C. control mixes increased the compressive strength in almost every combination tested. The best result was obtained when 3% anhydrite was added to slag/P.C. control mix which caused strengths increase of 217% and 240% after 28 and 56 days of curing, respectively. The same strength increase was achieved when 1% anhydrite and 0.65% lime were added to the slag/P.C. control mix.

Under controlled conditions, a typical rockfill mass has superior physical and mechanical properties compared to other fill types. Throughout this thesis properly engineered quality control measures were established. Quality control measures were established in plant, transportation and during placement.

Very little work had been done on the in-situ behaviour of a consolidated rockfill mass. Extensive in situ test program was carried out to 1: To establish physical and mechanical properties of consolidated rockfill and 2: To evaluate the potential techniques for short interval in situ quality control measurements. The test program included: ground stress, ground movement, blast vibration, and seismic measurements. Pressuremeter and core testing were also carried out. Some of the important values obtained were:

Compressive strength range:	1 to 17 MPa
Elastic modulus range :	0.6 to 4.5 GPa
Transmission coefficient from rock through CRF:	37%
Absorption coefficient in CRF :	0.41 db/m.
Poisson's ratio :	0.35
Friction angle :	37 deg.
Cohesion :	1.1 MPa
P- wave velocity:	3,150 m/sec

Improvement in all areas effecting CRF performance, especially the four areas mentioned above, were investigated at KCM and the results from the studies were implemented with great success. The extensive field work and the experience gained at KCM, resulted in establishing CRF design improvement steps presented in Figure 10.1.

12: RECOMMENDED FUTURE WORK

1: Segregation phenomena in a rockfill mass provides weak zones with little cement, zones C and D in the thesis. Research into improving the present techniques for obtaining better control of segregation should be carried out. The techniques for reinforcing the weak zone, by post consolidation should be further investigated.

2: Pressuremeter testing demonstrated an ability to provide in situ data and responding to significant changes in different zones of CRF. Future work on the system will be to improve it's capability of assessing failure criteria for CRF. Another area of improvement should be increasing the length of the monitored hole to obtain more representative strength values.

3: Applications of geophysical borehole logging to determine a number of physical properties of CRF should be considered.

4: The damage criteria by dynamic loading (e.g., production blasts, rockbursts) on CRF are not well defined and should be further investigated. Also, a standard method of evaluating the dynamic strength of CRF products is necessary.

5: Every mining operation should evaluate the addition of dispersant and superplasticizers to the backfill mix. The tests will determine if these chemicals would allow a reduction in overall use of binder material.

6: Each mining operation should evaluate the cementitious properties of different available binders for use in the fill system. Considerable savings could be achieved by partially replacing Portland cement with local and/or cheaper binder materials. The replacement of the Portland cement should be carried out without sacrificing long term and short term strengths and quality of the fill.

7: Testing indicated that small additions of anhydrite or gypsum to backfill mix in most of the operations could increase the strength of the fill significantly. The addition of anhydrite or gypsum would probably result in obtaining equal or better short term and long term strengths compared to the most of the present mixes, even with partial replacement of Portland cement with lower cost binders. The adequate early strength would not limit the mining operation to a specific mining cycle for allowing the fill to cure. This could translate into a considerable saving in the costs of the binders used in the system. It is recommended that a comprehensive test program be carried out for different operations to evaluate the behaviour of anhydrite or gypsum in the present and/or a more economical mix.

8: There is considerable lack of automation in Canadian mines using CRF and more work in this area is needed, for example different conveyors transporting aggregate could be ran from one central location.

9: Use the obtained in situ data related to physical and mechanical properties of CRF to simulate the CRF mass stability, 2D or 3D, for different stope conditions.

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APPENDIX A

Layout of Irads and extensometers:

NAME	AZ	DIP	LENGTH	SIZE	ANCHORS
G13	45	-25	13	AX	5 ,10 ,12 m
I-24	315	+45	10	EX	
I-25	135	+45	10	EX	
G-14	45	-25	14	AX	5 , 8 , 13 m

G-15 was not installed due to low RQD of fill.

INST. No.	I - 24				
		T	STRESS	TOT. STR	
DAYS	last read	cur. read	CHANGE	CHANGE	REMARKS
28	2984	2979	-18.4438		To = 2984
35	2979	2977	-7.4036	-25.8474	
42	2977	2976	-3.7074	-29.5548	
47	2976	2975	-3.7111	-33.2659	
49	2975	2973	7.4335	-40.6994	
54	2973	2973	0.0000	-40.6994	
56	2973	2974	3.7186	-36.9808	
63	2974	2969	-18.6307	-55.6114	
68	2969	2970	3.7337	-51.8778	
70	2970	2963	-26.2151	-78.0928	
75	2963	2969	22.4814	-55.6114	
84	2969	2967	-7.4787	-63.0901	
89	2967	2966	-3.7450	-66.8351	
91	2966	2966	0.0000	-66.8351	
96	2966	2966	0.0000	-66.8351	
104	2966	2967	3.7450	-63.0901	
112	2967	2967	0.0000	-63.0901	
117	2967	2971	14.9422	-48.1479	
118	2971	2956	-56.3465	-104.4944	
119	2956	2957	3.7831	-100.7112	
119	2957	2958	3.7793	-96.9320	
120	2958	2954	-15.1402	-112.0722	
126	2954	2960	22.6873	-89.3849	
132	2960	2958	-7.5471	-96.9320	
139	2958	2967	33.8419	-63.0901	
146	2967	2966	-3.7450	-66.8351	
152	2966	2981	55.7799	-11.0552	
153	2981	2980	-3.6925	-14.7476	
154	2980	2980	0.0000	-14.7476	
159	2980	2995	54.9994	40.2518	
161	2995	2995	0.0000	40.2518	
166	2995	2934	-228.9562	-188.7044	
168	2934	2895	-154.0268	-342.7312	
173	2895	2917	87.6462	-255.0850	
175	2917	2914	-11.8350	-266.9199	
180	2914	2916	7.8940	-259.0259	
182	2916	2915	-3.9450	-262.9709	
187	2915	2957	162.2596	-100.7112	
189	2957	2954	-11.3609	-112.0722	
196	2954	2950	-15.2018	-127.2740	
201	2950	2947	-11.4420	-138.7160	
203	2947	2945	-7.6475	-146.3635	
208	2945	2940	-19.1870	-165.5505	
210	2940	2930	-38.6690	-204.2195	
215	2930	2930	0.0000	-204.2195	
217	2930	2911	-74.5721	-278.7915	
221	2911	2924	51.1800	-227.6116	
223	2924	2929	19.5034	-208.1082	
228	2929	2923	-23.4161	-231.5243	
231	2923	2923	0.0000	-231.5243	
238	2923	2924	3.9127	-227.6116	
269	2924	2926	7.8134	-219.7982	
275	2926	2949	88.7141	-131.0841	
288	2949	2951	7.6164	-123.4678	
295	2951	2952	3.8024	-119.6654	
303	2952	2951	-3.8024	-123.4678	
309	2951	2949	-7.6164	-131.0841	
314	2949	2949	0.0000	-131.0841	
	2949	2951	7.6164	-123.4678	

TABLE 1

DAYS	LAST READING	CUR. READING	STRESS CHANGE, PSI	TOTAL STRESS CHANGE
28				
35	2268	2268	0	
42	2268	2270	16.7383	16.7383
47	2270	2268	-16.7383	0
49	2268	2267	-8.3858	-8.3858
54	2267	2267	0	-8.3858
56	2267	2266	-8.3969	-16.7827
63	2266	2264	-16.8271	-33.6098
68	2264	2264	0	-33.6098
70	2264	2264	0	-33.6098
75	2264	2264	0	-33.6098
84	2264	2264	0	-33.6098
89	2264	2265	8.4191	-25.1907
91	2265	2264	-8.4191	-33.6098
96	2264	2264	0	-33.6098
104	2264	2265	8.4191	-25.1907
112	2265	2263	-16.8495	-42.0401
117	2263	2265	16.8495	-25.1907
118	2265	2265	0	-25.1907
119	2265	2267	16.8049	-8.3858
119	2267	2263	-33.6543	-42.0401
120	2263	2264	8.4303	-33.6098
126	2264	2238	-222.8747	-256.4845
132	2238	2339	824.6632	568.1787
139	2339	2252	-703.6947	-135.516
146	2252	2246	-51.5667	-187.0827
152	2246	2280	286.8529	99.7702
153	2280	2282	16.4755	116.2457
154	2282	2282	0	116.2457
159	2282	2319	297.1503	413.396
161	2319	2319	0	413.396
166	2319	2180	-1196.1104	-782.7144
168	2180	2155	-240.0361	-1022.7505
173	2155	2159	38.9668	-983.7836
175	2159	2161	19.4023	-964.3813
180	2161	2171	96.2087	-868.1726
182	2171	2170	-9.5611	-877.7337
187	2170	2196	244.3587	-633.3749
189	2196	2195	-9.2381	-642.6131
196	2195	2193	-18.5142	-661.1273
201	2193	2192	-9.2761	-670.4034
203	2192	2192	0	-670.4034
208	2192	2194	18.5396	-651.8638
210	2194	2196	18.4889	-633.3749
215	2196	2194	-18.4889	-651.8638
217	2194	2192	-18.5396	-670.4034
221	2192	2191	-9.2888	-679.6922
223	2191	2195	37.0791	-642.6131
228	2195	2194	-9.2508	-651.8638
231	2194	2194	0	-651.8638
238	2194	2193	-9.2634	-661.1273
269	2193	2190	-27.8665	-688.9938
275	2190	2190	0	-688.9938
288	2190	2165	-236.7441	-925.7378
295	2165	2164	-9.6408	-935.3786
303	2164	2164	0	-935.3786
309	2164	2163	-9.6542	-945.0328
314	2163	2163	0	-945.0328

TABLE 2

DAYS	TOTAL STRESS	TOTAL STRESS
	CHANGE, PSI	CHANGE, PSI
	I-24	I-25
28		
35	-25.8474	
42	-29.5548	16.7383
47	-33.2659	0
49	-40.6994	-8.3858
54	-40.6994	-8.3858
56	-36.9808	-16.7827
63	-55.6114	-33.6098
68	-51.8778	-33.6098
70	-78.0928	-33.6098
75	-55.6114	-33.6098
84	-63.0901	-33.6098
89	-66.8351	-25.1907
91	-66.8351	-33.6098
96	-66.8351	-33.6098
104	-63.0901	-25.1907
112	-63.0901	-42.0401
117	-48.1479	-25.1907
118	-104.4944	-25.1907
119	-100.7112	-8.3858
119	-96.932	-42.0401
120	-112.0722	-33.6098
126	-89.3849	-256.4845
132	-96.932	568.1787
139	-63.0901	-135.516
146	-66.8351	-187.0827
152	-11.0552	99.7702
153	-14.7476	116.2457
154	-14.7476	116.2457
159	40.2518	413.396
161	40.2518	413.396
166	-188.7044	-782.7144
168	-342.7312	-1022.7505
173	-255.085	-983.7836
175	-266.9199	-964.3813
180	-259.0259	-868.1726
182	-262.9709	-877.7337
187	-100.7112	-633.3749
189	-112.0722	-642.6131
196	-127.274	-661.1273
201	-138.716	-670.4034
203	-146.3635	-670.4034
208	-165.5505	-651.8638
210	-204.2195	-633.3749
215	-204.2195	-651.8638
217	-278.7915	-670.4034
221	-227.6116	-679.6922
223	-208.1082	-642.6131
228	-231.5243	-651.8638
231	-231.5243	-651.8638
238	-227.6116	-661.1273
269	-219.7982	-688.9938
275	-131.0841	-688.9938
288	-123.4678	-925.7378
295	-119.6654	-935.3786
303	-123.4678	-935.3786
309	-131.0841	-945.0328
314	-131.0841	-945.0328

TABLE 3

-STRESSMETER CALIBRATION:

To relate stressmeter reading to a value for rock stress, following equation was used.

$$\sigma_r = \frac{((422400)^2 * (1-(To/T)^2)}{9.4 - 0.5 * 10^{-6} * Er}$$

where:

σ_r = stress change

TO= initial meter reading

T = later meter reading

Er = Young's modulus of the rock = assumed , $11.5*10^6$

which gives a value of change in the uniaxial stress in a block of rock.

631 STOPE BLASTING

SLOT BLASTS:

38-1 SLOT FINAL- MAY 20- DAY 140

38-2 SLOT FINAL- JUNE 16- DAY 156

U/C BLASTS:

U/C STARTED - APRIL 23 -DAY 113

U/C FINISHED - APRIL 27 - DAY 117

RING BLASTS:

38-1 RING 1 TOES - MAY 14, DAY 134

38-1 RING 1 FINAL- MAY 22, DAY 142

38-1 RINGS FINAL - JUNE 5, DAY 156

38-2 RINGS SOUTH OF SLOT ,1,2 & 3 - JUNE 19, DAY 170

38-2 FINAL , JULY 2, DAY 183

Extensometer G-13

DAYS	TOTAL DISPLACEMENT	TOTAL DISPLACEMENT
	CH.#2, mm	CH.#3 , mm
28	0	0
35	0.35	0.12
42	0.52	0.19
47	0.65	0.19
49	0.7	0.21
54	0.75	0.21
56	0.8	0.22
63	0.84	0.24
68	0.85	0.26
70	0.87	0.27
75	0.87	0.27
84	0.88	0.28
89	0.88	0.28
91	0.88	0.28
96	0.87	0.28
104	0.87	0.28
112	0.14	0.48
117	0.12	0.49
118	0.1	0.49
119	0.1	0.5
119	0	0.51
120	-0.02	0.49
126	-0.52	0.47
132	-0.5	0.63
139	-0.83	0.84
146	0.19	1.37
152	0.25	1.52
153	0.25	1.62
154	0.25	1.62
159	0.24	1.8
161	0.26	1.8
166	0.23	2.08
168	0.23	2.22
173	.05	2.88
175	.04	2.89
180	0.03	2.89
182	0.01	2.87
187	-0.17	2.93
189	-0.24	2.93
196	-0.25	2.9
201	-0.26	3.03
203	-0.26	3.01
208	-0.27	3.11
210	-0.27	3.09
215	-0.26	3.11
217	-0.2	3.2
221	-0.21	3.12
223	-0.21	3.2
228	-0.21	3.14
231	-0.21	3.14
238	-0.21	3.15
269	-0.22	3.16
275	-0.21	3.16
288	-0.21	3.16
295	-0.2	3.18
303	-0.07	3.26
309	-0.07	3.26
314	-0.07	3.26
321	-0.09	3.25
334	-0.07	3.27
341	-0.07	3.33
349	-0.07	3.38
356	-0.07	3.39

TABLE 4

Extensometer G 14

DAYS	TOTAL DISPLACEMENT	TOTAL DISPLACEMENT	TOTAL DISPLACEMENT
	CH.#1, mm	CH. # 2, mm	CH. # 3, mm
28	0	0	0
35	-0.08	-0.09	-0.09
42	0.01	0.03	0.01
47	0.01	0.02	0
49	0.01	0.02	0
54	0.01	0.02	0
56	0.01	0.03	0
63	0	0.01	0.02
68	-0.01	0.02	0.02
70	-0.03	0	0.02
75	-0.03	-0.02	0.02
84	-0.03	0.01	0.01
89	-0.03	0.01	0.01
91	-0.03	0.01	0.01
96	-0.08	0.01	0.01
104	-0.16	0.01	0.01
112	-0.16	0.18	0.05
117	-0.16	0.18	0.05
118	-0.03	0.17	0.05
119	-0.03	0.19	0.04
119	1.23	0.21	0.05
120	1.27	0.92	0.27
126	0.76	0.95	0.29
132	3.46	1.05	0.53
139	3.34	4.34	1.81
146	3.35	6.17	2.87
152	3.34	6.29	2.84
153	3.27	6.56	2.84
154	3.26	11.16	2.27
159	3.28	11.55	2.23
161	3.36	15.24	1.96
166	3.33	19.47	1.14
168	3.35	21.1	1.13
173	3.34	21.13	1.13
175	3.33	21.49	1.13
180	3.33	21.61	1.12
182	3.31	26.07	1.05
187	3.31	26.62	1.06
189	3.33	27.07	1.03
196	3.33	27.61	1.04
201	3.33	27.62	1.04
203	3.32	27.8	1.05
208	3.31	27.93	1.05
210	3.3	27.87	1.04
215	3.29	28.31	1.04
217	3.28	28.63	1.04
221	3.32	28.78	1.04
223	3.28	29.03	1.06
228	3.28	29.13	1.04
231	3.25	29.7	1.04
238	3.29	29.04	1.04
269	3.28	30.72	1.02
275	3.28	30.94	1.02
288	3.29	32.16	1.01
295	3.08	33.47	0.94
303	3.08	31.98	-0.14
309	2.87	31.97	-0.09
314	2.93	32.01	0.24
321	2.93	32.01	0.23
334	2.86	32.01	0.21
341	2.86	32.01	0.21
349	2.86	32.01	0.2
356		32.01	0.2

TABLE 5

APPENDIX B

1: BLAST MONITORING B1 TO B8

PEAK PARTICLE VELOCITY CALCULATION, METHOD # 1

Blast	Delay	Time	V4	V5	V6	PPV	V1	V2	V3	PPV	
			mv	mv	mv	i.p.s	mv	mv	mv	i.p.s	**BLAST A**
A	1	41	50	87	48	3.9	27	22	16	1.3	Channels 4-6 in rock was
38-1	2	55	58	33	35	2.6	21	21	19	1.2	51 m from blast
	3	89	63	103	75	5	41	27	15	1.8	
	4	105	42	25	33	2.1	21	25	12	1.2	Channels 1-3 in backfill was
	5	136	38	78	41	3.4	31	18	15	1.4	53.2 m from blast
	6	169	20	38	16	1.6	12	20	12	0.9	
	7										Transmission coef.= 0.37
	8	230	67	87	70	4.6	32	32	16	1.7	
	9	283	37	48	42	2.6	15	23	12	1.1	
	10	345	51	76	66	4	43	32	19	2	
	11	411	24	29	21	1.5	14	17	8	0.8	**BLAST B:**
	12	510	59	113	64	5	34	21	15	1.5	Channel 4-6 in rock
	13	588	18	44	9	1.7	10	11	11	0.6	Channel 1-3 in fill and 40 m
	14										from blast.
	15	771	57	79	29	3.8	17	26	13	1.2	
	16	821	45	75		3.2	17	29	12	1.3	Absorption coef.= 0.39
	17										
	18	994	19	41	10	1.6	10	16	10	0.7	**BLAST C:**
	19	1051	35	82	48	3.6	15	18	10	0.9	Channel 4-6 in rock 62.5 m
	20	1176	23	43	10	1.7	9	20	9	0.8	from blast
	21	1278	51	156	44	6	17	10	10	0.8	Channel 1-3 in rock but
											signal passing through back
B	1	28	67	29	53	3.2	24	26	32	1.7	61.3 m from blast
38-1	3	79	49	31	34	2.4	23	30	31	1.7	Amplitude reduction=11%
C	1	37	123	82	48	2.7	101	54	76	2.4	*PPV is in inches/sec.
38-2											

TABLE 6

PEAK PARTICLE VELOCITY CALCULATION, METHOD # 2

BLAST	DELAY	TIME	ch# 4	ch #5	ch#6	PPV , ROCK
38-1	NO.	mv	mv	mv	mv	ips
	1	41	30	92	28	3.5
	2	55	52	30	32	2.4
	3	89	70	115	64	5.2
	4	105	30	25	30	1.7
	5	136	40	102	62	4.4
	6	169	20	20	15	1.1
	7	....	......	......	......	....
	8	230	82	118	45	5.3
	9	283	55	30	28	2.4
	10	345	52	60	70	3.7
	11	411	25	30	20	1.5
	12	510	108	150	68	6.9
	13	588	18	48	9	1.8
	14	....	......	......	......	....
	15	771	52	102	25	4.1
	16	821	63	70	22	3.4
	17	....	......	......	......	....
	18	994	25	82	41	3.3
	19	1051	44	82	40	3.5
	20	1176	22	40	12	1.65
	21	1278	40	138	40	5.2
						P.P.V =3.4
B- BLAST						
38-1	1	58	58	31	40	2.7
	3	50	50	31	28	2.3
						P.P.V= 2.5
C-BLAST						
38-2	1	PEAK TO PEAK	120	110	43	2.92
		37	121	100	40	2.82
						P.P.V= 2.9

TABLE 7

PEAK PARTICLE VELOCITY CALCULATION, METHOD #2

ch #1 mv	ch # 2 mv	ch#3 mv	PPV FILL, ips	PPV RATIO % FILL/ROCK	STRESS RATIO FILL/ROCK %
22	20	8	1.05	30.00	15.00
20	31	15	1.4	58.33	29.17
32	27	18	1.6	30.77	15.38
11	18	18	0.84	49.41	24.71
22	24	11	1.2	27.27	13.64
10	12	10	0.65	59.09	29.55
......	......	......	....	....	....
18	22	8	1	18.87	9.43
33	20	9	1.4	58.33	29.17
32	21	13	1.41	38.11	19.05
11	13	10	0.69	46.00	23.00
33	20	11	1.4	20.29	10.14
8	11	8	0.55	30.56	15.28
......	......	......	....	....	....
11	30	15	1.24	30.24	15.12
11	30	11	1.18	34.71	17.35
......	......	......	....	....	....
10	13	10	0.67	20.30	10.15
10	18	10	0.8	22.86	11.43
8	11	8	0.55	33.33	16.67
11	10	8	0.59	11.35	5.67
			P.P.V= 1 ips	34%	17%
20	21	18	1.2	44.44	22.22
21	21	30	1.48	64.35	32.17
			P.P.V= 1.34 ips	54%	27%
90	61	77	2.31	79.11	39.55
32.5	27	30	0.9	31.91	15.96
			P.P.V= 1.6 ips		

TABLE 8

SUMMARY OF PEAK PARTICLE VELOCITY RESULTS

BLAST A, METHOD #2
channels 4-6 are in rock ,51 m from blast
channels 1-3 are in fill
53.2 m from blast
P.P.V IN ROCK =3.4 ips
P.P.V IN FILL =1.01 ips , 34 % OF ROCK
FILL STRESS= 17.2 % OF ROCK STRESS
BLAST B, METHOD #2
channels 4-6 are in rock
channels 1-3 are in fill
P.P.V IN ROCK = 2.49 ips
P.P.V IN FILL= 1.34 ips, 54% of rock
= 46 % absorption or reflection
BLAST C, METHODS 1 & 2
channels 4-6 in rock
62.5 m from blast
channels 1-3 in rock but signals
pass through fill
P.P.V IN ROCK =PEAK= 2.9 ips
P.P.V IN FILL = PEAK= 1.34 ips
P.P.V OF FILL= 46% OF P.P.V OF ROCK
54% REDUCTION
BUT
P.P.V IN ROCK AT DELAY TIME= 2.82 ips
P.P.V IN FILL AT DELAY TIME = 0.9 ips
P.P.V IN FILL = 32 % OF P.P.V IN ROCK
= 68 % reduction
ROCK DENSITY = 2.9 T/m3 FILL DENSITY = 1.9
P-WAVE VEL. OF ROCK =6 km/s for fill assumed 4.5 km/s

SUMMARY OF TABLES 6, 7 8

BLASTING ENERGY CALCULATION

BLAST A	DELAY NO.	TIME ms	CH #4 mv	CH#5 mv	CH#6 mv	AREA FOR ROCK	E ROCK Mpa/s
38-1	1	41	8	7	4	11	1845.25
	2	55	4	5	4	8	1342
	3	89	10	15	9	20	3355
	4	105	1	1	2	2	335.5
	5	136	4	6	2.2	10	1677.5
	6	169	1	3	1.3	3	503.25
	7						0
	8	230	10	8	7	15	2516.25
	9	283	3	5.6	2.7	7	1174.25
	10	345	8	9	3.5	19	3187.25
	11	411	1	2	1	2	335.5
	12	510	12	20	10	25	4193.75
	13	588	1	2.6	0.8	3	503.25
	14						
	15	771	6	7.5	4.5	11	1845.25
	16	821	4	6.3	2	8	1342
	17						0
	18	994	1	2.2	0.6	2.5	419.375
	19	1051	3	7.5	2.7	8.5	1425.875
	20	1176	2	2.4	1.3	3	503.25
	21	1278	4	16.5	3.8	17	2851.75
B BLAST @ 38-1	1	28	14	4	10	18	3019.5
	3	79	7	6	6	11	1845.25
C BLAST @ 38-2	1	37	28	27	15	21	3522.75

TABLE 9

BLASTING ENERGY CALCULATION

DELAY NO	TIME ms	CH#1 mv	CH#2 mv	CH#3 mv	AREA FILL	E FILL Mpa/s	E FILL/E ROCK %	
1	41	2	2.5	0	3.2	273.408	14.82	
2	55	1.5	2.8	2.2	3.9	333.216	24.83	
3	89	7.5	3.1	1.2	8.2	700.608	20.88	
4	105	1.5	1	1.2	2.1	179.424	53.48	
5	136	4.7	3.7	1.2	6.1	521.184	31.07	rock density= 2.85 t/m3
6	169	0.8	1.4	0.8	1.8	153.792	30.56	fill density= 1.9 t/m3
7						0		
8	230	4.6	5.1	2.2	7.2	615.168	24.45	p-wave at rock= 6 km/s
9	283	1.7	2.5	0.8	3.1	264.864	22.56	p-wave in fill = 4.5 km/s
10	345	6	5.4	2.1	8.4	717.696	22.52	
11	411	1.4	1.4	0.8	2.1	179.424	53.48	
12	510	4.5	7	1.8	8.5	726.24	17.32	
13	588	2.5	1	0.7	2.8	239.232	47.54	
14						0	0.00	
15	771	2	3.7	1.6	4.5	384.48	20.84	
16	821	2.1	3.5	0.8	4.2	358.848	26.74	
17						0		ENERGY = AREA *DENSITY* P-WAVE VEL
18	994	1	1.9	0.9	1.9	162.336	38.71	
19	1051	1.3	1.9	0.8	2.4	205.056	14.38	
20	1176	0.7	1.8	0.6	2.7	230.688	45.84	
21	1278	1.1	0.8	1.3	1.9	162.336	5.69	
							27%	BLAST A
1	28	4	4	7	9	768.96	25.50	AREA FILL=56% OF AREA ROCK
3	79	6	6	9	12	1025.28	55.50	ENERGY OF FILL = 27% OF ROCK
								BLAST B
								AREA FILL = 80% OF AREA ROCK
1	37	80	26	32	45	3844.8	110.00	ENERGY OF FILL= 41% OF ROCK
								BLAST C
								AREA FILL= 214 % OF AREA ROCK
								ENERGY OF FILL= 110 % OF ROCK

PEAK TO PEAK P AND S-WAVE MEASUREMENTS

BLAST A	DELAY NO.	TIME ms	P-wave ch #4 rock, mv	P-wave ch #5 rock,mv	P-wave ch#6 rock,mv	P-wave ch #1 fill, mv	P-wave ch#2 fill,mv	p-wave ch#3 fill, mv
38-1								
	1	41	67	170	47	40	33	25
	2	55	50	110	30	40	32	38
	3	89	78	204	52	65	55	28
	4	105	20	88	32	42	38	25
	5	136	62	152	32	55	40	20
	6	169	23	76	20	28	20	20
	7	......	....	....	....	....	....	....
	8	230	68	173	70	62	52	30
	9	283	30	92	22	30	28	20
	10	345	75	150	59	82	50	38
	11	411	13	42	22	22	20	10
	12	510	80	222	47	34	30	25
	13	588	28	88	22	15	20	20
	14		....	....	....	....	....	....
	15	771	56	158	35	32	50	25
	16	821	50	150	40	40	27	20
	17	..	....	....	....	....	....	....
	18	994	24	80	22	18	18	20
	19	1051	50	162	30	28	35	20
	20	1176	30	88	27	10	5	10
	21	1278	85	285	80	10	20	5
B BLAST @ 38-1								
	1	28	110	52	90	25	45	60
	3	79	75	35	40	30	60	60
C BLAST @ 38-2								
	1	37	242	200	82	68	57	60

TABLE 11

PEAK TO PEAK P AND S-WAVE MEASUREMENTS

DEL NO	S ch #4 rock	S ch#5 rock	S ch #6 rock	S ch#1 fill	S ch #2 fill	S ch#3 fill	P.WAVE ROCK mv	P-WAVE FILL mv	P-WAVE FILL/ROCK %	S.AVE ROCK mv	S.AVE FILL mv	S-WAVE FILL/ROCK %
1	79	52	98	40	43	30	188.67	57.56735	30.51	136.19	65.95	48.42
2	68	60	66	40	42	40	124.50	63.78087	51.23	112.16	70.46	62.82
3	130	102	152	70	32	22	224.51	89.63258	39.92	224.52	80.05	35.65
4	48	47	63	20	41	15	95.75	61.91123	64.66	92.098	48.02	52.14
5	75	70	80	60	30	20	167.25	70.88723	42.38	130.1	70.00	53.81
6	40	42	33	22	35	20	81.88	39.7995	48.60	66.731	45.92	68.82
7	....	....	....	....	....	....	....	....	....	....	....	....
8	133	67	140	38	55	30	198.60	86.3018	45.00	204.4	73.27	35.85
9	72	52	81	30	40	20	99.24	45.65085	46.00	120.2	53.85	44.80
10	96	60	130	40	60	10	177.78	103.286	58.10	172.38	72.80	42.23
11	42	48	41	12	32	20	49.16	31.36877	63.81	75.822	39.60	52.22
12	120	110	137	60	40	20	240.61	51.77837	21.52	212.77	74.83	35.17
13	33	22	22	20	20	20	94.93	32.01562	33.72	45.354	34.64	76.38
14	....	....	....	....	....	....	......	......	....	.....	....	....
15	65	68	90	32	50	25	171.25	64.41273	37.61	130.19	64.41	49.48
16	93	60	50	30	55	20	163.10	52.23983	32.03	121.45	65.76	54.15
17												
18	32	32	20	22	30	20	86.37	32.37283	37.48	49.477	42.24	85.37
19	73	75	90	28	30	12	172.17	49.08156	28.51	138.04	42.76	30.97
20	45	40	23	20	40	15	96.81	15	15.49	64.452	47.17	73.19
21	100	93	88	30	20	20	307.98	22.91288	7.44	162.46	41.23	25.38
									....			....
									TOT= 704			TOT=926
									AVE= 39%			AVE= 51%
1	100	55	68	50	33	30	151.34	79.05694	52.24	132.85	67.00	50.43
3	47	50	50	47	57	30	91.92	90	97.91	84.906	79.74	93.91
									AVE= 75%			AVE= 72%
1	100	80	91	170	110	150	324.48	107.1121	33.01	157.1	251.99	160.40

TABLE 12

APPENDIX C

TROW

DILATOMETER/PRESSUREMETER TESTS

BOREHOLE Nº :-	2
TEST Nº :	3
DEPTH :	3'
HOLE SIZE :	NW

PROJECT Nº : S03365R

SITE : Kidd Creek Backfill

SURFACE ELEVATION :

TEST ELEVATION :

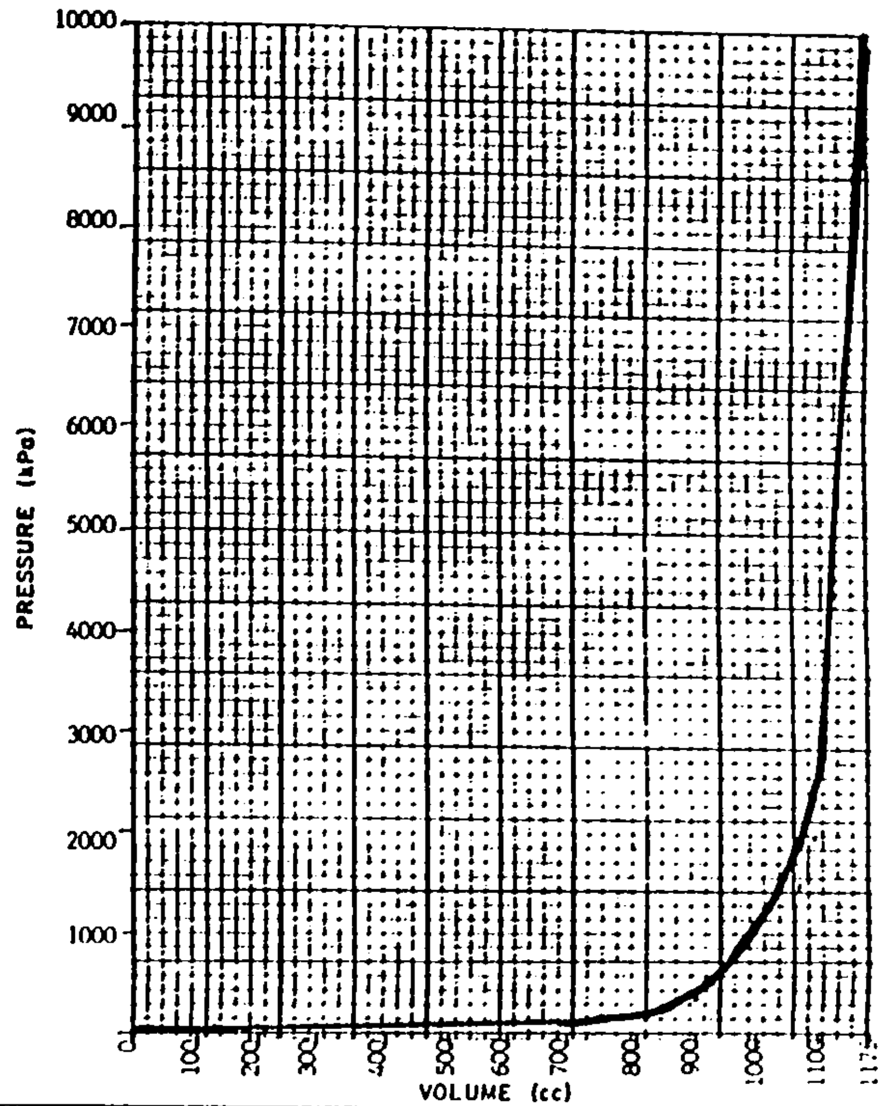
PRESSURE	READINGS 1 min.	2 min.	3 min.	INJECTED VOLUME (cc)	CORRECTED VOLUME (cc)
25				150	
30				200	
50				250	
60				300	
70				350	
75				400	
80				450	
100				500	
[illegible]				550	
112				600	
121				650	
[illegible]				700	

VOLUMETRIC CALIBRATION :

OBSERVATIONS ΔV = 1110 − 1038 = 72 cc ΔP = 7000 kPa

V = 1154 cc

(1110 + 1153.4) (11/1000 · 84 × 10⁻²) = 3.30 GPa



Drawing No. 1

TROW

DILATOMETER/PRESSUREMETER TESTS

BOREHOLE Nº : 2 (cont'd)
TEST Nº : 3
DEPTH : 3'
HOLE SIZE : NW

PROJECT Nº : S03365R
SITE : Kidd Creek Backfill

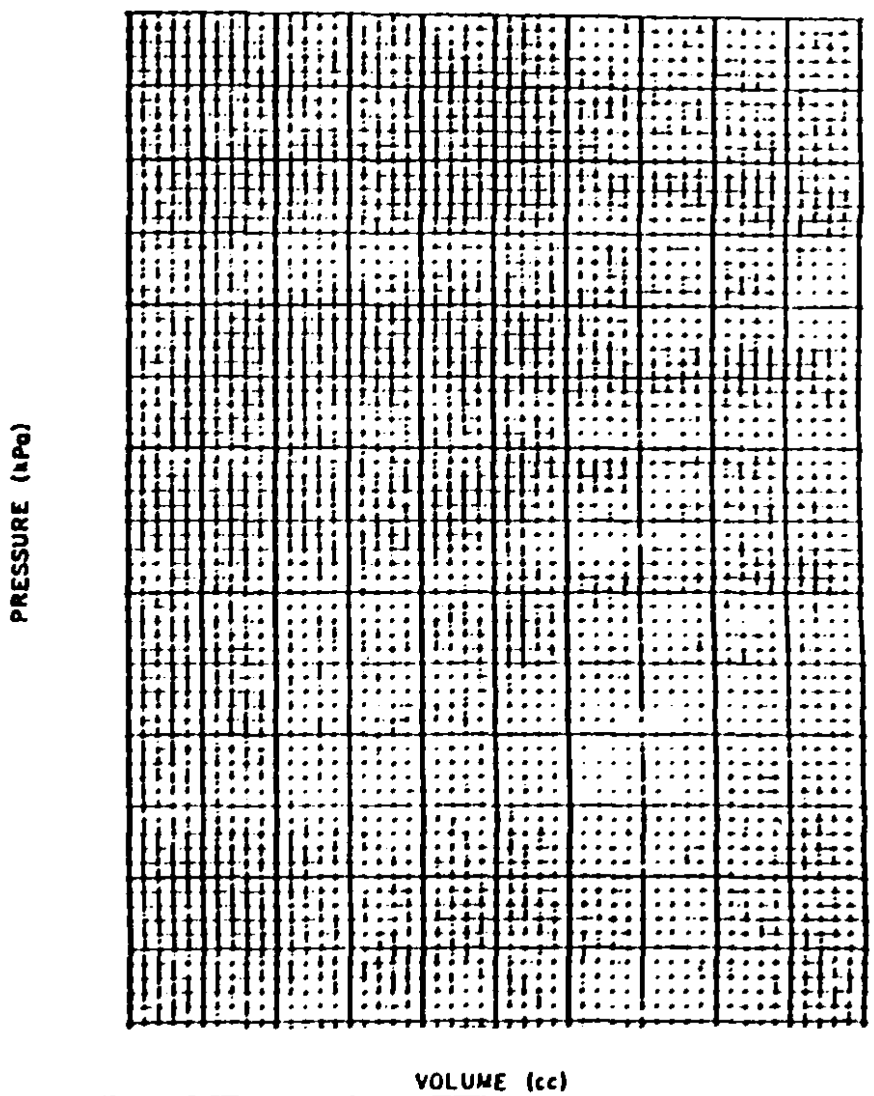
SURFACE ELEVATION :

TEST ELEVATION :

PRESSURE	READINGS 1 min.	2 min.	3 min.	INJECTED VOLUME (cc)	CORRECTED VOLUME (cc)
150				750	
175				800	
230				850	
330				900	
1500				1061.9	
2000				1096.8	
3000				1098	
4000				1109.1	
5000				1119.2	
6000				1129.8	
7000				1138.7	
8000				1151.6	

VOLUMETRIC CALIBRATION :

OBSERVATIONS :



Drawing No 1
cont'd

TROW

DILATOMETER/PRESSUREMETER TESTS

BOREHOLE Nº : 2 (cont'd)	PROJECT Nº : S03365R
TEST Nº : 3	SITE : Kidd Creek Backfill
DEPTH : 3'	
HOLE SIZE : NW	

SURFACE ELEVATION :

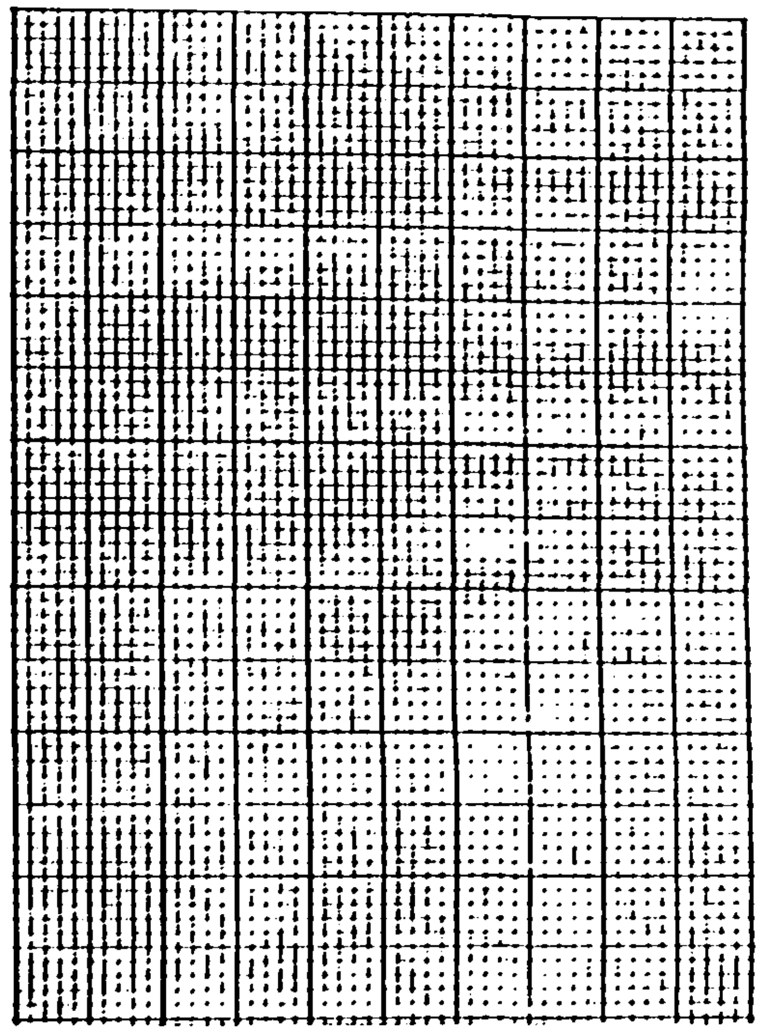
TEST ELEVATION :

PRESSURE	READINGS 1 min.	2 min.	3 min.	INJECTED VOLUME (cc)	CORRECTED VOLUME (cc)
[illegible]				1160.6	
10000				1170.0	

VOLUMETRIC CALIBRATION :

OBSERVATIONS :

PRESSURE (kPa)



VOLUME (cc)

Drawing No. 1 cont'd

TROW
DILATOMETER/PRESSUREMETER TESTS

BOREHOLE Nº :	2
TEST Nº :	2
DEPTH :	6'
HOLE SIZE :	NW

PROJECT Nº : S03365R
SITE : Kidd Creek Backfill

SURFACE ELEVATION :

TEST ELEVATION :

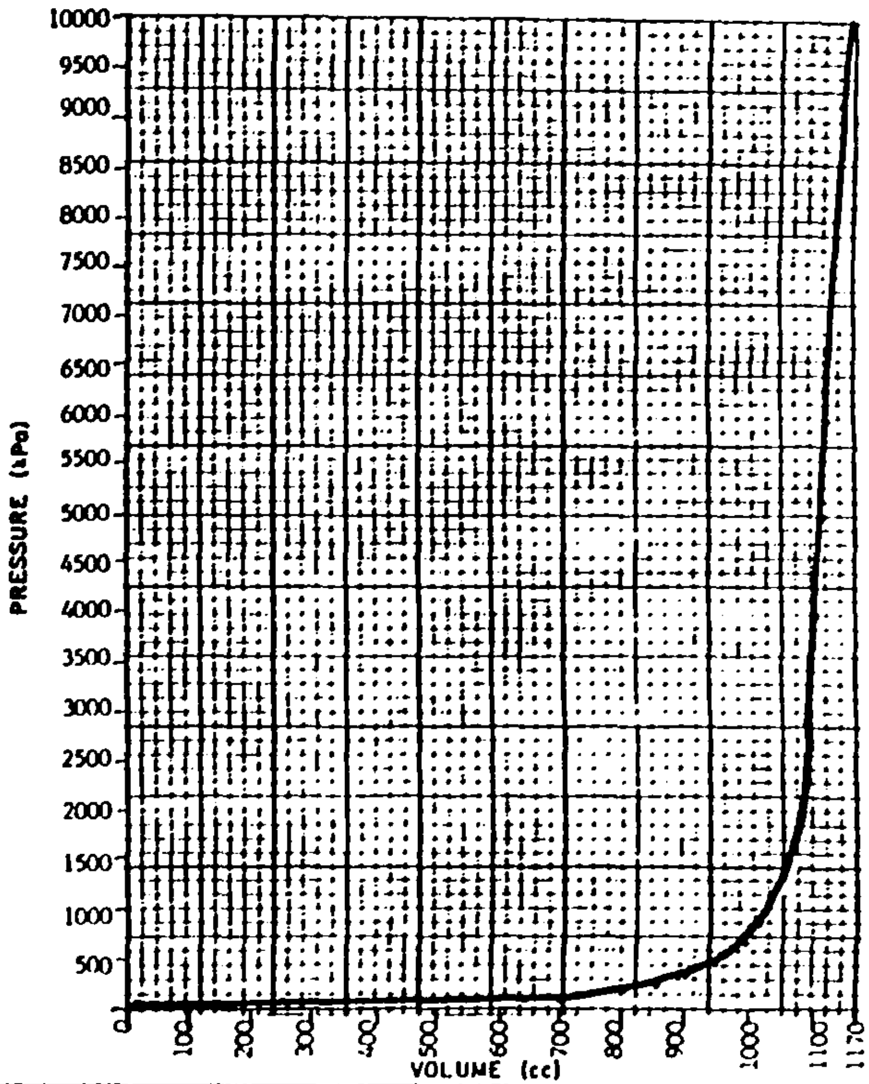
PRESSURE	READINGS			INJECTED VOLUME (cc)	CORRECTED VOLUME (cc)
	1 min.	2 min.	3 min.		
0				50	
25				100	
30				150	
50				200	
60				250	
70				300	
78				350	
87				400	
98				450	
106				500	
112				550	
120				600	

VOLUMETRIC CALIBRATION :

OBSERVATIONS ... ΔV = 1164.6 − 1075 = 89.6 cc ΔP = 8000 kPa

... 1119.8 cc

... (...) $\left(\frac{1}{89.6/8000 \; \cdot \; 84 \times 10^{-2}} \right)$ = 2.27 GPa



Drawing No. 2

TROW

DILATOMETER/PRESSUREMETER TESTS

BOREHOLE Nº : 2 (cont'd)
TEST Nº : 2
DEPTH : 6'
HOLE SIZE : NW

PROJECT Nº : S03365R
SITE : Kidd Creek Backfill

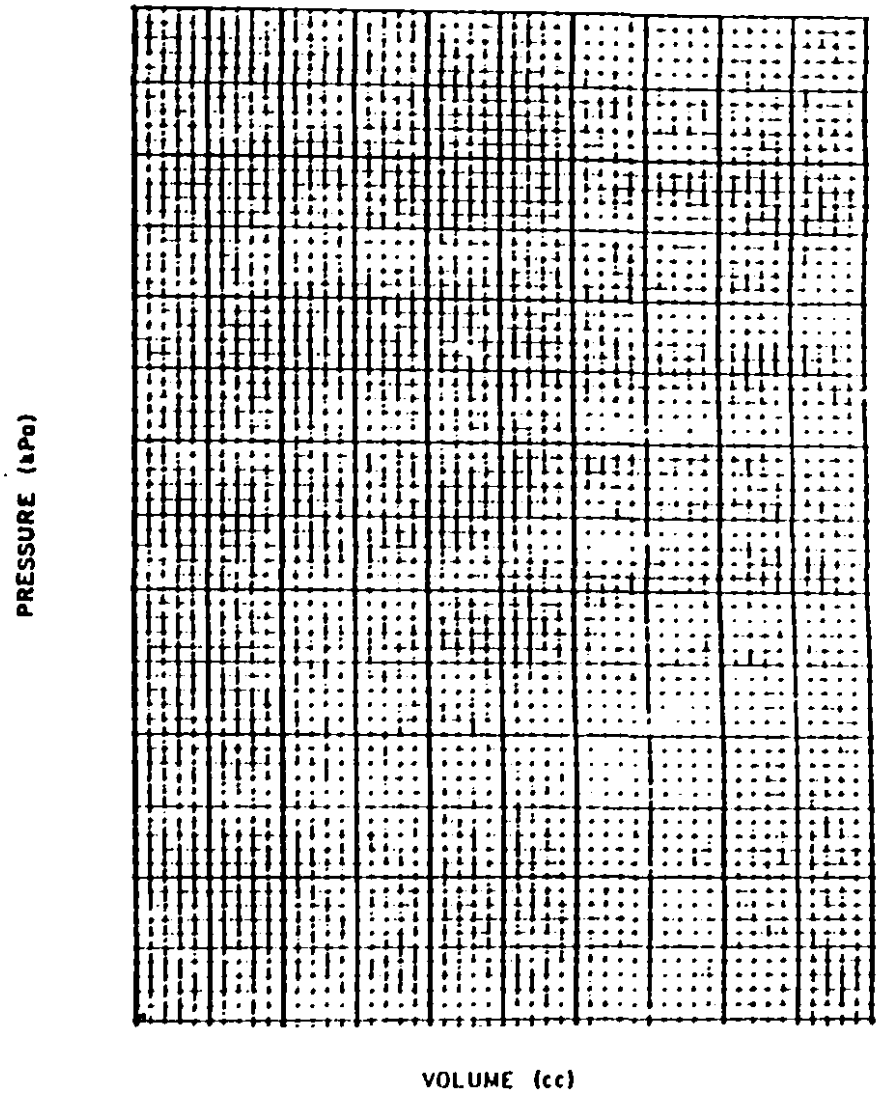
SURFACE ELEVATION :

TEST ELEVATION :

PRESSURE	READINGS			INJECTED VOLUME (cc)	CORRECTED VOLUME (cc)
	1 min	2 min.	3 min.		
125				650	
130				700	
150				750	
175				800	
230				850	
325				900	
430				950	
675				1000	
1000				1035.7	
2000				1075	
3000				1092.6	
4000				1104.8	

VOLUMETRIC CALIBRATION :

OBSERVATIONS :



Drawing No. 2
cont'd

TROW
DILATOMETER/PRESSUREMETER TESTS

BOREHOLE Nº : 2 (cont'd)
TEST Nº : 2
DEPTH : 6'
HOLE SIZE : NW

PROJECT Nº : S03365R
SITE : Kidd Creek Backfill

SURFACE ELEVATION :

TEST ELEVATION :

PRESSURE	READINGS 1 min.	2 min.	3 min.	INJECTED VOLUME (cc)	CORRECTED VOLUME (cc)
5000				1116.9	
6000				1126.1	
7000				1135.6	
8000				1145.1	
9000				1154.4	
10000				1164.6	

VOLUMETRIC CALIBRATION :

OBSERVATIONS :

PRESSURE (kPa)

VOLUME (cc)

Drawing No. 2
cont'd