A. G. BENNETT

Conference on

RE-SHAPING CITIES USING UNDERGROUND CONSTRUCTION

Melbourne, 1974

The Institution of Engineers, Australia
National Conference Publication No. 74/5

An Assessment of the Potential of Underground Construction in Urban Planning

by

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SUMMARY. In this paper, an examination is made of existing and potential uses of underground construction in urban planning, and also of various critical factors which will affect the extent to which the sub-surface will actually be used in the future. It is concluded that without an improvement in present technology, the use of underground construction in urban areas will be limited. However, there are already many situations in the urban context where subterranean construction can be justified. Continuing developments in sub-surface excavation technology will further extend the range of applicability of the underground planning concept.

1 INTRODUCTION

Tunnels and other forms of underground excavation have long been used by man in his efforts to adapt to life on earth, and the need to satisfy a basic requirement for water was the motivation which provided the first examples of this type of construction in pre-Christian times. Since then, and from the beginning of the present century in particular, subterranean construction has been used at an increasing rate to provide life-support facilities, and this is a trend which will continue. The reason for this is clear. It stems from increasing population concentrations in urban areas, and theirneed for water, transport, communications, and other similar types of facility. Also, the adoption of traditional forms of surface construction to cope with these needs is becoming more difficult in many cases, because of the growing cost of land, and the problem of finding space for the multiplication of essential services. Hopefully, these trends will not continue indefinitely; and there is already evidence that population growth in some technologically advanced countries has begun to slow down. Nevertheless 56% of the Australian population now live in the capital cities, and a stable situation is not likely to be reached before the end of the century, based on even the most optimistic estimates.

In addition to the physical and logistical problems associated with growth, there is increasing community concern to improve the quality of the environment in which people live, work, and play. While subterranean construction alone cannot solve the problem, it can make a contribution towards this aim. For example, the undergrounding of a freeway system will reduce noise pollution, and may obviate the need to break up closely knit urban communities. Of course the environment of the freeway user may not be improved, and as pointed out by Hoch (Ref.1), "'bargain basements' seem to correspond to a generally held notion of lower quality as one descends". However, this is a situation in which the overall community interest must inevitably prevail.

Recently, a considerable amount of attention has been given to the whole question of underground construction, and the results of a wide ranging examination of the subject are set out in a report prepared by the American Underground Research Council (Ref.2). This report identifies opportunities for alleviating national problems through a deliberately increased use of the underground, and recommends that many life support facilities should be transferred to the sub-surface. Specifically, it suggests that substantial social, economic, and environmental benefits would accrue in the short term from the undergrounding of urban freeways and electrical distribution systems, provided that certain requirements are met. These are

(i) a large research programme is put in hand to develop a new sub-surface technology which would close the gap between current surface and underground capabilities and costs.

(ii) a political decision is made to promote the transfer to sub-surface space, and institutional arrangements created in the private and public sector to execute and implement the programme.

In addition to the possible uses of underground construction mentioned above, the report suggests that in the long term, subterranean space is a vital resource which has the potential to resolve many of the problems now existing in the urban area. A similar conclusion is also reached in References 3 and 4.

In this paper an examination is made of existing and potential uses of underground construction in urban planning, and also of various critical factors which will affect the extent to which the sub-surface will actually be used in the future. These factors include the technology and economics of underground construction, and also the way in which benefits are evaluated. The necessity of using a comprehensive or system-wide approach in assessing benefits and adverse effects is emphasized.

2 UNDERGROUND EXCAVATION TECHNOLOGY

It is not proposed to include here an extensive review of the present state of underground excavation technology. However, some discussion of the subject is required, in order to provide a background against which existing and possible future applications of underground construction can be viewed.

Present day rock tunneling techniques consist of the 'conventional' drilling, blasting and mucking procedure, and excavation by means of tunnel boring machines (also known as moles). The essential technology of the former method was developed in the 17th century, and by the 19th century, excavation rates of about 2 m per week were achieved. By the use of vastly improved equipment and explosives, average excavation rates increased to about 35 m per week during the first quarter of the 20th century, and rates of about 80 m per week are currently being achieved (Ref. 5). Tunnels of up to 17 m diameter have been constructed using blasting, and much larger excavations have been formed in the same way for underground power stations. For example, one of the chambers in Churchill Falls Hydro-Electric Scheme in Canada has a clear span of about 25 m, is 286 m long, and rises to a maximum height of 47 m.

Underground excavation in soft material presents special problems, and these increase with the size of the excavation. Temporary or permanent support of the ground is required as soon as or shortly after the excavation is made, and very poor conditions could make a large underground excavation quite impracticable. Tunnels up to 14 m diameter have been constructed in soft soil using the shield method of tunneling. The excavation rates which can be achieved in shield work depend very much on the nature of the ground, and are greatest when material can be displaced by a bulkheaded shield. In Ref. 6 a comparison is given of excavation rates in two 9.5 m diameter tunnels, where progress of 9 m per day was achieved in silt, and only just over 1 m per day where the tunnel face was in mixed rock and granular material. A recently developed method of coping with poor ground where water ingress may also be a problem is by ground freezing (Ref.7). However this system does not necessarily lead to lower excavation costs.

Undoubtedly, the most significant advance which has occurred in the last 20 years is the large scale development of the mole. It would seem likely that further development centred around these machines is possible, and this is one of the major factors leading to optimism concerning the possibility of reducing excavation costs and increasing excavation rates beyond current levels. The mole was developed initially for use in 'soft' rock, and has been most successful in material such as sandstone or shale having an abrasion resistance less than about 5 on Moh's scale. Construction rates of up to 300 m per week have been attained in this type of rock, and tunnels of more than 11 m diameter excavated. Bored tunnels of relatively small diameter have been made in granite, but because of the hardness of this rock, not all have been economically successful (Ref.8). A problem associated with use of the mole in favourable conditions is the difficulty of transporting excavated material to the disposal area at a sufficiently great rate. Because of this, a mole cannot always be utilized at maximum capacity. This will be an even more pressing problem when higher excavation rates are achieved in the future.

Although moles can now cope moderately well where there is a variety of material types in the one tunnel by changing cutters, fitting shields etc., severe difficulties can arise where the ground in the tunnel face is mixed in character. A recently developed technique which could be of importance in this context is a method which enables tunnels to be driven through granular soils above or below the water table using a mechanical tunneling machine within which the face is supported by a thixotropic slurry (Ref.9). One factor which has contributed much to the flexibility of tunnel boring machines in fault zones and other types of poor ground is that systems can be devised which allow precast concrete liners, shotcrete and rock bolts, to be quickly and readily placed.

It has been estimated that improvements in existing techniques and technology should enable 1970 mole excavation rates to be doubled by 1980 (Ref.5). However, a much greater improvement than this will be required if underground construction costs are to anything like approach construction costs aboveground. The changes required are probably innovative rather than developmental in nature, and research is at present underway on various aspects of the problem. Some of the possibilities being examined which could improve the performance of the mole are chemical softening of rock, rock melting probes, high pressure water jets to produce kerfing in hard rock, high pressure water cannons, and a laser beam device that produces kerfs and thermal cracking (Ref.10). Muck removal systems which transport muck at a high rate through pipelines, and tunnel linings consisting of rapid hardening fibre reinforced concrete are also being investigated (Ref.11). The fibres (consisting of 40 mm-75 mm length needles of steel or fibre glass) are mixed into the concrete itself, and this obviates the need for steel mesh reinforcement. Other areas where improvements in technique are required are the excavation of vertical and inclined shafts, and of non-circular holes. At the moment, only the more conventional excavation methods can be used in these situations.

3 EXISTING AND POTENTIAL USES OF UNDERGROUND CONSTRUCTION

(i) Power Stations

One of the most common uses of underground construction has been the underground hydro-electric power station. Often, the use of underground rather than surface construction can be justified on economic grounds alone. In other cases it can be justified on the basis of safety (e.g. from rock falls) or on grounds of national security.

Because of the rapid and continuing depletion of oil and natural gas resources, greater reliance in the future will have to be placed on the use of coal and nuclear power as energy sources. However, both of these forms of electric power generation can have adverse environmental effects, and new means of minimizing and controlling this is being sought. In the case of nuclear power, the partial or complete underground siting of power plants has been suggested as a means of achieving these aims. Four small European reactors have been constructed partially underground already, and a study has been made at the California Institute of Technology (Ref.12) of the feasibility of undergrounding large nuclear power plants. This study confirmed the feasibility of the proposal, and indicates that if suitable rock is available, the estimated cost penalty associated with underground siting could be 5% to 10% of the total plant cost. Benefits resulting from undergrounding include a substantial improvement in containment against radiation hazards, and the possible siting of power plants closer to load centres.

The combination of both pumped storage and nuclear power at underground power centres has also been put forward as a means of ensuring that full use is made of power plants during off-peak hours (Ref.13). In this system, an upper surface reservoir is used to store water pumped from an underground chamber containing reversible pump-turbines. During on-peak hours, the water generates power as it passes through the turbines on its way to the storage chamber.

(ii) Storage Caverns

The use of unlined underground rock caverns for the storage of petroleum products below the water table has been a common practice in Scandanavia for some time now (Ref.14). This system can be used for a variety of grades of fuel oil, and operates on the principle that the inward pressure gradient resulting from the higher density of the water will prevent egress of oil from the storage cavern. In Sweden, where rock conditions are exceptionally good for underground installations, the capital cost of underground storage is found to be less than for surface storage when the volume exceeds about 30,000 m^3, and operating costs smaller for volumes in excess of 15,000 m^3. These smaller operating costs result from factors such as reduced insurance and land requirements.

Very large underground deep-freezing plants for the storage of food have also been constructed in Sweden. These plants are found to be economical on the basis of initial and operating costs, and the system has many technical advantages over the surface alternative.

It would seem reasonable to suggest that storage caverns could also be put to uses other than those outlined above, e.g. temporary storage for storm water run-off, waste disposal, and storage of natural gas.

(iii) Water Supply and Sewerage Systems

Water supply and sewerage systems are essential services for a city, and tunnels have long been used and will continue to be used for these purposes.

Underground sewage treatment plants have been constructed in densely populated areas where the available space is limited. Favourable aesthetics have helped to justify the high cost.

(iv) Buildings

These have been placed underground for use as car parks, civic centres and manufacturing plants. In the latter case, it may turn out that even though the capital cost is higher than for an above-ground plant, operating and maintenance costs make the subterranean alternative a better investment (Ref.15). A number of university buildings have also been located underground in Australia and North America for aesthetic and functional reasons.

(v) Utilities

To date, extensive use has not been made in Australia of the undergrounding of low voltage electrical cables because of the high capital cost involved. For new housing estates having average population densities, this form of distribution is two to three times dearer than overhead distribution. One way in which these costs could perhaps be lowered is by the use of common utility tunnels which would contain services such as natural gas, water, telephone cables and electricity. It has been suggested that this type of tunnel could also include provision for transporting mail in pneumatic tubes, or even the movement of goods in urban areas (Ref.16).

(vi) Transportation

The extended use of sub-surface transportation is likely to be one of the most significant future developments in the use of underground construction. Notable recent examples of how the underground can be utilized to solve central urban transport problems are discussed in Ref.17, which also describes the use of a third dimension of development consisting of one or more transportation levels.

Schemes such as these reflect the scarcity and high cost of land in urban areas. This latter point is also illustrated in the figures set out in Table I, which gives very rough order of cost values for freeway construction in an Australian capital city.

TABLE I
ORDER OF COST FIGURES (\$ per km x 10^6) FOR FOUR LANE FREEWAY

Location	Inner	Suburban	Outer
Land	10	1	1
Construction	4	3	2
Total	14	4	3
Tunnel (bored)	26		
Tunnel (cut & cover)	13		

Another well known form of subterranean transportation system is the underground railway. A considerable number of these were built during the first half of this century and several are currently under construction in various parts of the world.

4 ECONOMICS OF UNDERGROUND CONSTRUCTION

A critical factor in comparing undergrounding with the equivalent above-ground alternatives is the cost of sub-surface construction.

There are a number of reasons why surface excavation costs are lower than underground excavation costs. Above-ground work can be tackled simultaneously on a number of fronts, and there is relatively little restriction on the movement of machinery. Also, much larger equipment can be used to remove spoil, and no ground support is normally required for shallow excavations. It is difficult therefore to see how underground costs will ever be as low as those for surface work, although the marginal cost for excavation of large holes in sound rock could be similar to the above-ground value.

For tunnels constructed by conventional drilling and blasting procedures it is found that the excavation cost per cubic metre decreases with increase in size of the tunnel. The cost versus diameter relationship takes the approximate form

$$C = a + b/D^2$$

where C = tunnel cost per unit volume
D = diameter of excavation
a,b are constants

For medium strength rock at depth under good average tunneling conditions, typical order or cost figures for a tunnel between 1.5 and 2.5 km in length would be C = \$120 per m^3 for D = 3 m, and

$C = \$35$ per m^3 for $D = 7$ m. These values include allowance for a concrete invert along the full length of the tunnel, and assume a full concrete lining, steel rib supports and rock bolts over 20% of the tunnel length. If the tunnel were fully lined with concrete, the costs would be approximately $C = \$135$ per m^3 for $D = 3$ m, and $C = \$50$ per m^3 for $D = 7$ m. Where the ground encountered consists of crushed rock or unconsolidated sediments, these costs could increase by a factor of about two, and by about four if in addition there is a wet heading. (A useful and comprehensive collection of tunnel cost data relating to American conditions can be found in Ref.2).

It is probably still true to say that the mole has not yet had a significant general effect in reducing the costs of excavating tunnels, although in specific instances, substantial cost savings have been achieved. It is clear however that the mole is competitive in the smaller range of tunnel sizes in rock having a compressive strength which does not exceed about 120 MPa. At present, it is not economical for tunnels having a diameter greater than about 10 metres. This is because although unit costs using the mole decrease with increasing tunnel size, they do not decrease as fast as for conventional tunneling methods (Ref.18). The decision to use a tunnel boring machine is based on the consideration that the high capital cost of the machine is traded off against reduced labour costs. These latter costs are essentially proportional to the time taken for excavation, which depends in turn on the capacity (and hence cost) of the machine. Because of this situation, there is normally an optimum advance rate for the mole which will minimize excavation costs, and also a minimum tunnel length for which the use of a tunnel machine can be economically justified.

In assessing the likely effect of future changes in tunneling technology on costs, it is necessary to examine the break up of costs between labour, capital equipment charges, and consumables. With conventional tunneling methods, the largest cost element is labour costs. This situation remains the same with machine tunneling, although the proportion is smaller (Ref.19). It is apparent therefore that if tunneling rates can be significantly increased without a corresponding increase in machine costs, the net result will be substantial improvement in the economics of tunneling. A similar improvement could result if the whole process were automated. In this case the critical factor would be the capital and maintenance costs of the mole, rather than its rate of progress. It has been stated (Ref.2) that a threefold increase in tunneling rates and reduction in construction costs could be achieved within ten years if a sizeable investment in research and development were put in hand.

The cost of excavating large underground openings is very much dependent on the quality of the rock encountered. If it is free from faults and has tight joints at large spacing, no temporary supports will be required during the excavation phase apart from a nominal amount of rock-bolting and shotcreting. However, where the rock is of poorer quality, it may be necessary to line the excavation progressively as excavation takes place, and this will add to the cost. An order of cost figure for an underground powerhouse excavation of 10,000 to 15,000 m^3 size in medium strength jointed rock requiring some staging but with no substantial stability problems is \$50 per m^3. If heavy support is required, or the rock is difficult to break, this figure could increase substantially.

It has been pointed out in Ref.20 that the costs of excavation in mines are much less than for underground powerhouse chambers. The suggestion is made that by using mining techniques such as the room and pillar method, the excavation cost of large underground chambers might be reduced by a factor of 5 to 10. A further but much smaller reduction in cost would result if the excavated rock were sold for construction purposes. The conditions under which it is maintained that these low excavation costs would be obtained are:

(i) the rock is stable and relatively uniform in character, so that it can be worked in a way which requires little or no special wall or roof support

(ii) work is done concurrently on several excavation faces at a high production rate

(iii) the excavation is large enough to require several years work, so that equipment does not have to be written off before its useful life is reached.

Naturally, the number of suitably located sites which would have rock of the required quality is limited, and this is a matter which can only be decided by detailed geological investigation. However, it would seem that the concepts set out above are worthy of further examination.

Another factor which would assist in reducing the cost of tunnels, shafts and underground chambers is the removal of financial risk as far as possible from work carried out by contractors. Ways in which this might be achieved are:

(i) the use of schedule of rates contracts in which all conditions likely to be encountered are catered for

(ii) a more detailed geological examination, the complete results of which are made available to tenderers.

5 THE BENEFITS AND POTENTIAL OF SUBTERRANEAN CONSTRUCTION IN URBAN AREAS

The possible greater use of subterranean construction in urban areas is a technical means by which the goals or objectives of urban communities might be achieved. These objectives are not always easy to define, and will vary for different groups within the community. Nevertheless, it is essential that goals be formulated as part of a systems or overall approach, if planning is to be carried out in a rational manner. It is also important to realize that the priorities given to these goals will change as a society becomes more affluent. For example, concern with the quality of the environment has a relatively low priority in many developing countries, because of a greater preoccupation with basic requirements such as food and shelter. In contrast, the attitude in more developed countries is exemplified by the way in which alternative programmes of investment in public works are now evaluated. Traditionally this has been done using a cost/benefit analysis, in which a heavy reliance is placed on the quantification of benefits and costs. By giving a monetary value to items such as safety, pollution effects and reduced travel times, it is possible to make an allowance for some of the relevant social and environmental factors. However it is clear that there are definite limits to the extent to which these criteria can be included in this way, and other means of evaluating the comparative merits of surface and sub-surface alternatives have been adopted. The method currently used for handling problems involving 'intangible' elements is to quantify benefits and costs as far as possible, and then use some form of ranking or weighting procedure in order to make a final choice. Such a procedure is obviously crude, and must be regarded as an aid only to decision making. It does however represent a distinct advance in guiding the selection process.

A detailed analysis of the type just mentioned was used by the Underground Research Council in evaluating the benefits to be obtained from subterranean construction, and care has been taken to reject economic criteria as the single measure of merit. A total of nineteen social, economic and environmental factors were identified (see Table II) and each of these items considered in relation to the undergrounding of liquid and solid waste disposal transportation, communications, energy distribution, water resources, production systems and industrial structures, and also shelter.

TABLE II

ECONOMIC, SOCIAL & ENVIRONMENTAL FACTORS CONSIDERED IN RELATION TO UNDERGROUND CONSTRUCTION

Economic Benefits	Social and Environmental Benefits
Direct costs Time Energy Pollution Safety Reliability Material resources	Opportunity time Public health Safety Job opportunity Attractiveness Aesthetic quality Environmental quality Shelter quality, availability and density reduction Mental health Ethnic quality Conservation of natural resources

After allowing for adverse as well as beneficial effects, it was concluded from this analysis that the largest gains to the environment and to man himself accrue from the potential transfer of

(i) transportation
(ii) production systems and industrial structures
(iii) human shelter and housing
(iv) storage

However, the important point is also made that some transfers to the sub-surface have low social and political feasibility. When this is allowed for, transportation and electrical energy distribution are reckoned to be the most feasible functions for transfer to the sub-surface, and the ones which would show the highest benefits. It should be emphasized again that all of these conclusions are based on the assumption that the cost differential between surface and sub-surface construction is eliminated by a very substantial research and development program.

A number of general comments on the subject of sub-surface construction will now be made.

(i) The minimal surface disturbance involved with subterranean construction leads to a significant improvement in the immediate above-ground environment. However, care will need to be taken that the spoil removed does not degrade the environment in some other location. In the future, it is likely that environmental benefits alone will often provide justification for the use of sub-surface space.

(ii) The degree to which use of the underground will be required beyond the year 2000 is closely tied up with the growth of population in urban areas. This in turn is greatly affected by the changed aspirations of married women, which has had a significant effect in reducing birth rates. Decentralization policies which are now being vigorously promoted in Australia will probably also play a part in reducing the concentrations of population in urban areas. It is of interest to note that in the USA, fertility rates have now fallen below the zero population growth rate.

(iii) It would seem unlikely that a sudden and substantial shift to the underground will occur in the USA as recommended by the Underground Research Council. This does not seem to be politically feasible. Rather, it is probable that a transfer of politically 'safe' functions such as transportation will occur incrementally.

(iv) The high cost of land and lack of space in the inner areas of cities will often in itself provide sufficient justification for placing transportation and parking services underground.

(v) It is difficult to imagine that justification exists anywhere in Australia for the provision of underground shelter in which people should be asked to live on a permanent basis. However it is plausible to suppose that workers would be prepared to work in subterranean plants and industrial complexes. There is some evidence that psychosomatic problems are greater in below-ground plants than in those above-ground. However, this is an area where more research is required.

(vi) The long term effects of the growing shortage and increase price of oil are difficult to predict. There is no energy crisis in the sense that alternatives do not exist, but energy will undoubtedly cost more in the short term and probably also in the long term than it does today. As far as buildings located underground are concerned these are in a thermally stable environment in which little energy is required for heating and cooling, and the requirements for lighting are not significantly greater than is required in surface plants. With regard to transportation, the rising cost of energy should hopefully tend to result in more people using public transport, since the per capita energy requirements are only 1/5 to ½ that used by motor vehicles. This would not support any trend to underground construction if public as well as private transport were to move to the sub-surface.

(vii) If a break through in sub-surface excavation technology could be made, there are undoubted benefits to be obtained from placing underground much of the flows of people, goods and services which currently occur at the earth's surface. In this way, conflicts with human, social and business activity could be avoided. However the problems of exit and entry to these subterranean systems should not be minimized, from the point of view of both cost and technical requirements.

6 CONCLUSION

Clearly, without an improvement in present technology, the use of underground construction in urban areas will be limited. This is primarily because of the high costs and difficulty currently associated with many types of underground excavation work. Nevertheless, adoption of underground construction can be justified in many situations, when total costs and overall community amenity are taken into consideration. The present significant and continuing developments in sub-surface technology will further extend the range of applicability of the underground planning concept.

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Terraspace—a Hidden Resource

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SUMMARY. The increasing urbanization makes ever greater demands on use of the underground space. At present many problems occur because of insufficient planning, registration or investigation for subsurface constructions. In connection to a Swedish R&D project, led by the author, this paper discusses facts and goals within the fields of knowledge to be covered in planning and utilizing the subsurface resources. These fields are 1) estimation of future subsurface demand, 2) current and future tunnelling technology, 3) geology and soil mechanics, 4) hydro-geology including technical and echological problems, 5) man's reaction to underground stay, 6) cost-benefit analyses, 7) recording and visualizing methods, 8) legislation, 9) liability for damages, and 10) relations between underground and above-ground planning. The present main task for subsurface planning should be to call the attention of planners and decision-makers to the hidden underground resources, and to work for regular evaluation of subsurface location for different functions in all phases of planning.

1 INTRODUCTION

Several antique cultures have, in pace with a developed urbanization, quite consciously claimed subsurface space for different kinds of plants. Therefore, in a historical perspective, subsurface constructions are a sign of an economically and technically highly developed culture.

Perhaps the great number of subsurface constructions in the heavily extended Western industrial society at the present day are also a proof of a highly developed culture. The future will tell.

The indisputably most intensive population development and urbanization in the history of the earth now taking place, constantly make increased demands on the subsurface utilization. A central role is played by the hunt for minerals and other subsurface resources claimed by the growth in population, the industrialization and the increase in the standard of living connected thereto. At an ever increasing degree the need of water, energy and better communications causes subsurface location of production and distribution plants and transport routes.

"Terraspace", here used as a slogan and a futurist designation for underground space, has, however, also come to be a "hidden resource" in high-density urban areas, partly to supplement and substitute space above ground and partly to overcome topographic and other ground obstacles but also to conceal sensitive, offensive or disturbing functions.

According to the limitation given by the subject for this conference, my presentation will only comprise the demands on the subsurface area that can be made by the total urbanization. Furthermore, objects and functions are limited to those of a considerable dimension (more than 20 sq feet sectional area) and which are relatively isolated from surface constructions. The limitation is made in full awareness of the fact that this distinctive subsurface constructing is of small extent compared with the consumption of subsurface space from normal on-ground buildings, pipes and cables.

The inquiry to its member countries that OECD prepared before the Advisory Conference on Tunnelling in Washington in 1970 gives an initial basis for estimating the extent and rate of development of subsurface construction. The result of the inquiry tells that within 16 OECD countries 136 % more subsurface volume (excluding mines) is expected to be built during the seventies than during the sixties.

An attempt to analyse the figures indicates that the most accelerated tunnelling is particularly connected to those areas where heavy concentrations of population lead to lack of area and communication difficulties (Fig. 1).

A hypothesis concerning possible future urban subsurface construction in the entire world has been performed based on more or less official studies of future population development in urban areas and of economic growth, two factors which seem to have great influence on urban subsurface construction. The hypo-

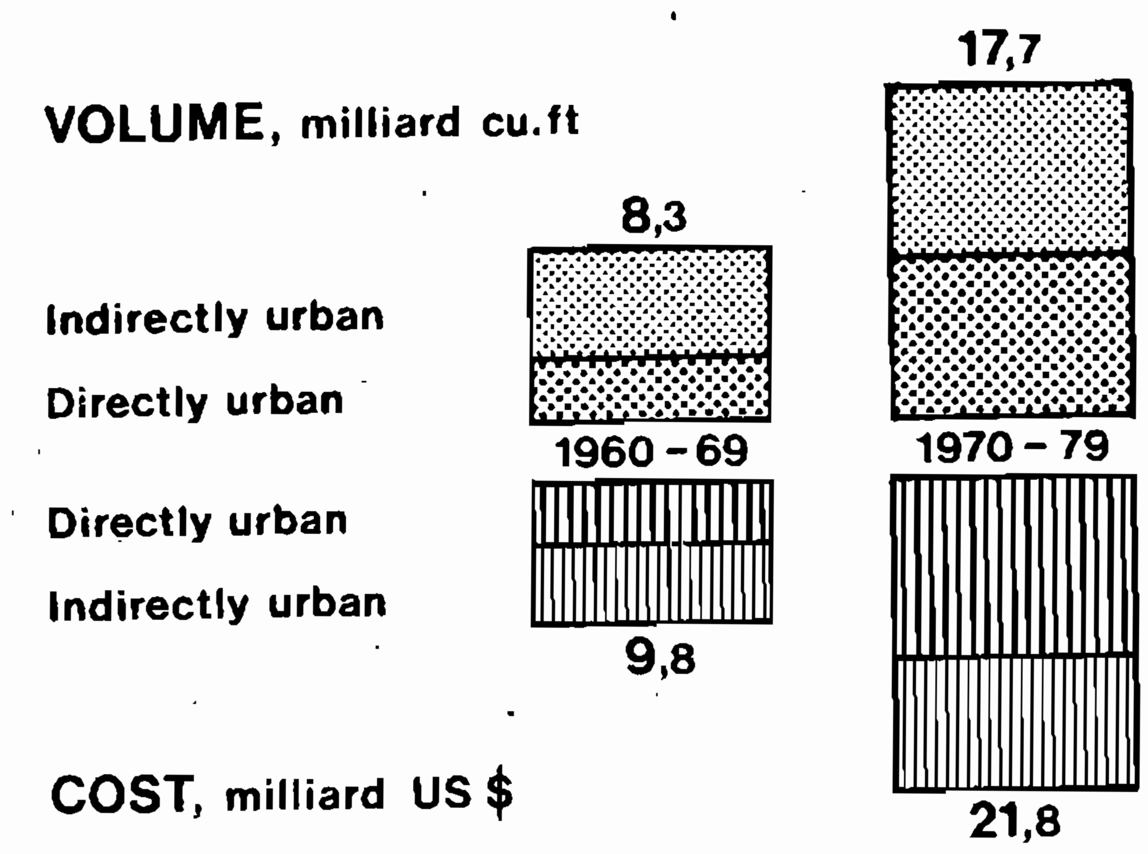


Fig. 1 Constructed and estimated urban tunnels in 16 OECD countries

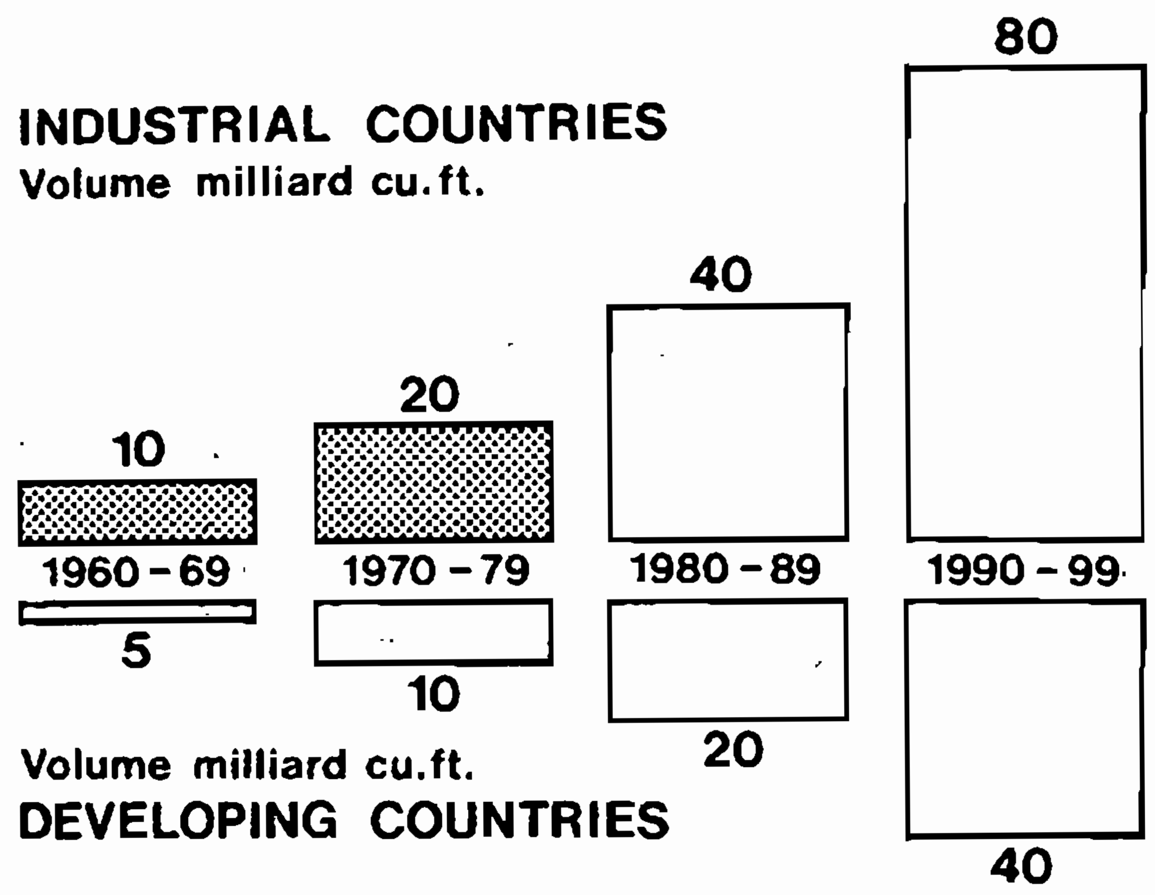


Fig. 2 Attempt to predict future urban tunnelling volume

thesis indicates (Fig. 2) that urban subsurface construction might double every tenth year in industrial countries as well as in developing countries, and that the developing countries will have a subsurface construction volume which is half as large as that of the industrial countries.

In the OECD material the different countries show fairly great variations in both extent and development of subsurface construction. In this connection it is of particular interest to study Australia and Sweden in the international perspective and in relation to one another. Fig. 3 shows general relations in area and population. The geological conditions in Australia and the Fenno-Scandinavian area have a similarity as they are old eroded formations.

The small European countries, Sweden and Norway, show (Fig. 4) a considerably larger tunnel volume per capita than many industrial countries, which depends on a relatively complicated topography and the occurence of good, firm rock utilized throughout centuries for mining and other cavity constructions. In Sweden subsurface construction (excluding mining) is expected to grow from 595 million cu ft during the sixties to 1,310 million cu ft during the seventies, an increase of 120 %.

If the OECD statistical material gives a completely correct picture, Australia would be comparable with a number of Central-European countries, including the United Kingdom. During the sixties the country would have built 175 million cu ft (mining excluded) and is expected to accomplish 105 million cu ft during the seventies, which means a reduction. With regard to the uncertainty of the statistical material I dare not make any farreaching conclusions.

However, certain tendencies appear in the comparison between the two countries which I would like to point out. Mining has a dominating position in the two countries and the main part of the subsurface plants is located in hard rock in both countries.

Australia seems to have a similar technical orientation as Sweden as regards subsurface constructions and a fund of experience with close connection to the mining industry. Also the urban structure and the economic standard in the two countries are fully comparable.

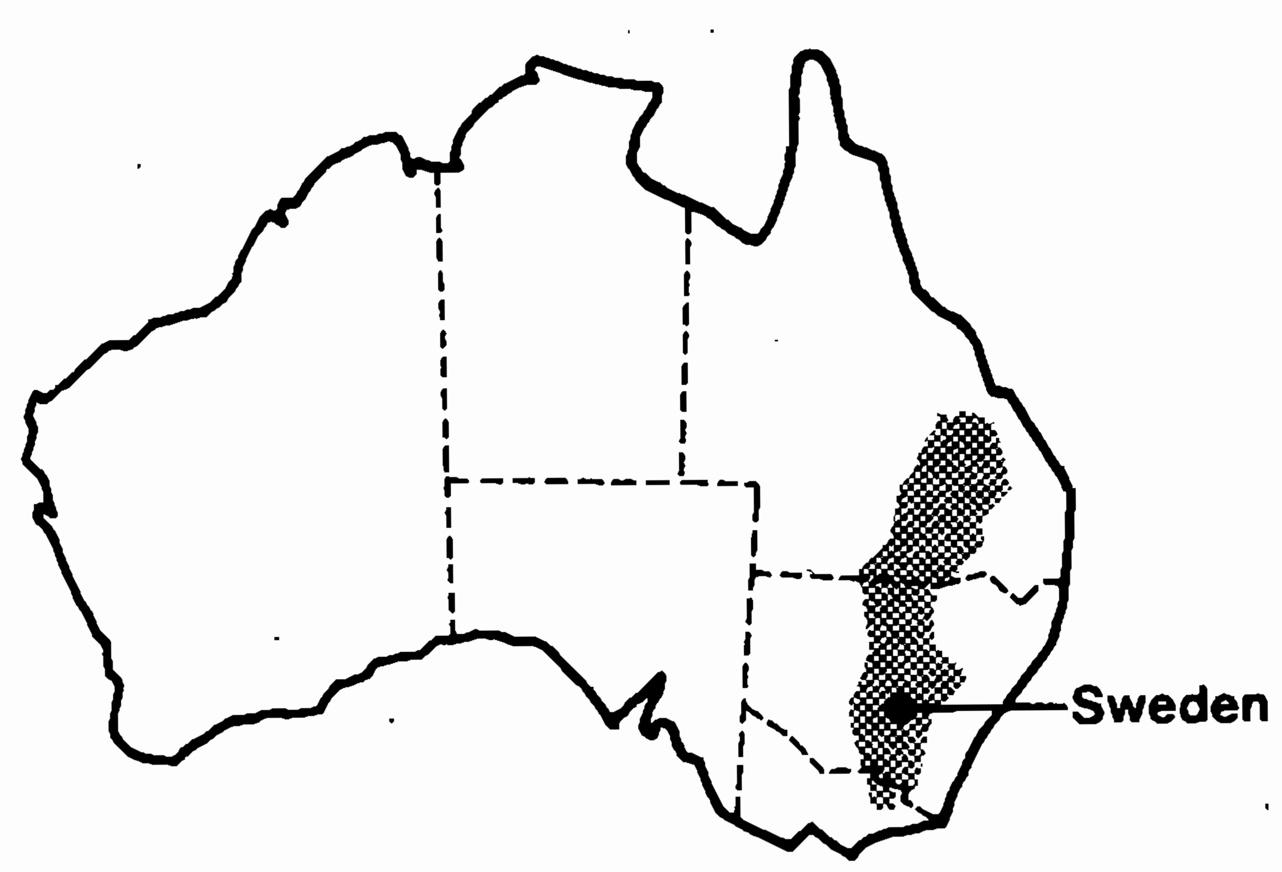
All these conditions indicate that approaches to problems and experience concerning subsurface construction in our country can have relationship with Australia in spite of the large geographical distance

2 PROBLEMS

The subsurface construction in urban areas is mainly concerned with adapting a number of different functions to the medium which "Terraspace" offers, but also with securing subsurface plants against disturbing influence from the surroundings. In addition, the realization of subsurface plants requires that the surroundings, mainly plants and buildings on the surface, do not suffer disturbances or damages.

The adaptation of functions to subsurface location requires great familarity with the characteristics of the subsurface area and with the available technology for tunnelling. Moreover, an engineering boldness and creative intuition should be added. I venture to say that the technology in our country stands on a high level in these connections and that comparatively many types of objects have already been given subsurface solutions and that new objects are continously tested under ground.

In spite of a considerable knowledge of "Terraspace" as a medium, disturbances and damages often occur on ground, plants and built up areas as a result of subsurface construction. Due to the topographic situation and the land-use on the surface, some parts of the subsurface space become so attractive that crowding occurs.



	Australia	Victoria	Sweden
Population	12,960,000	3,550,000	8,120,000
Area, sq. miles	2,970,000	88,000	170,000
Pop./100 sq. miles	435	3,980	4,680

Fig. 3 Population and area of Australia and Sweden

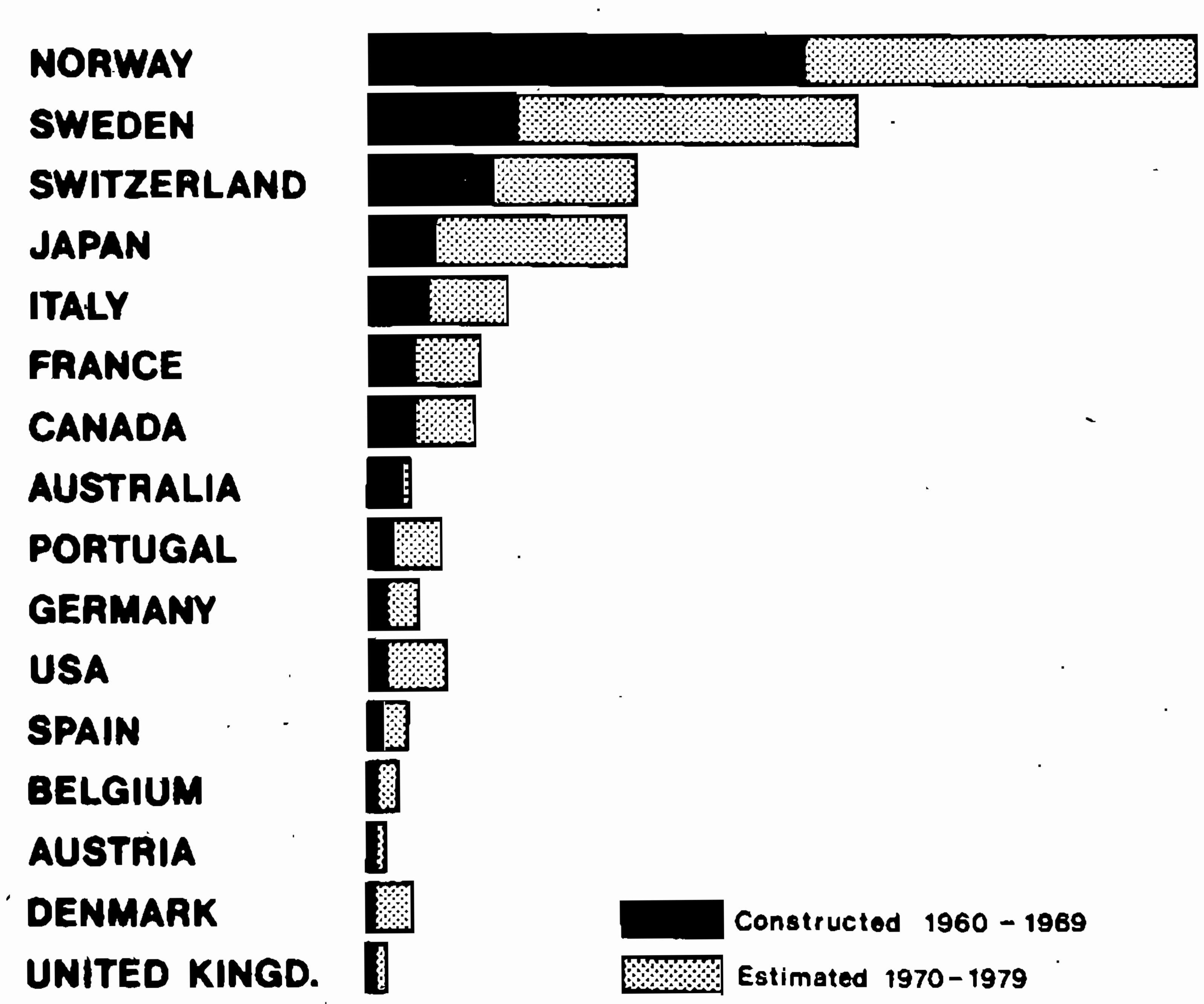


Fig. 4 Volume of tunnels per capita

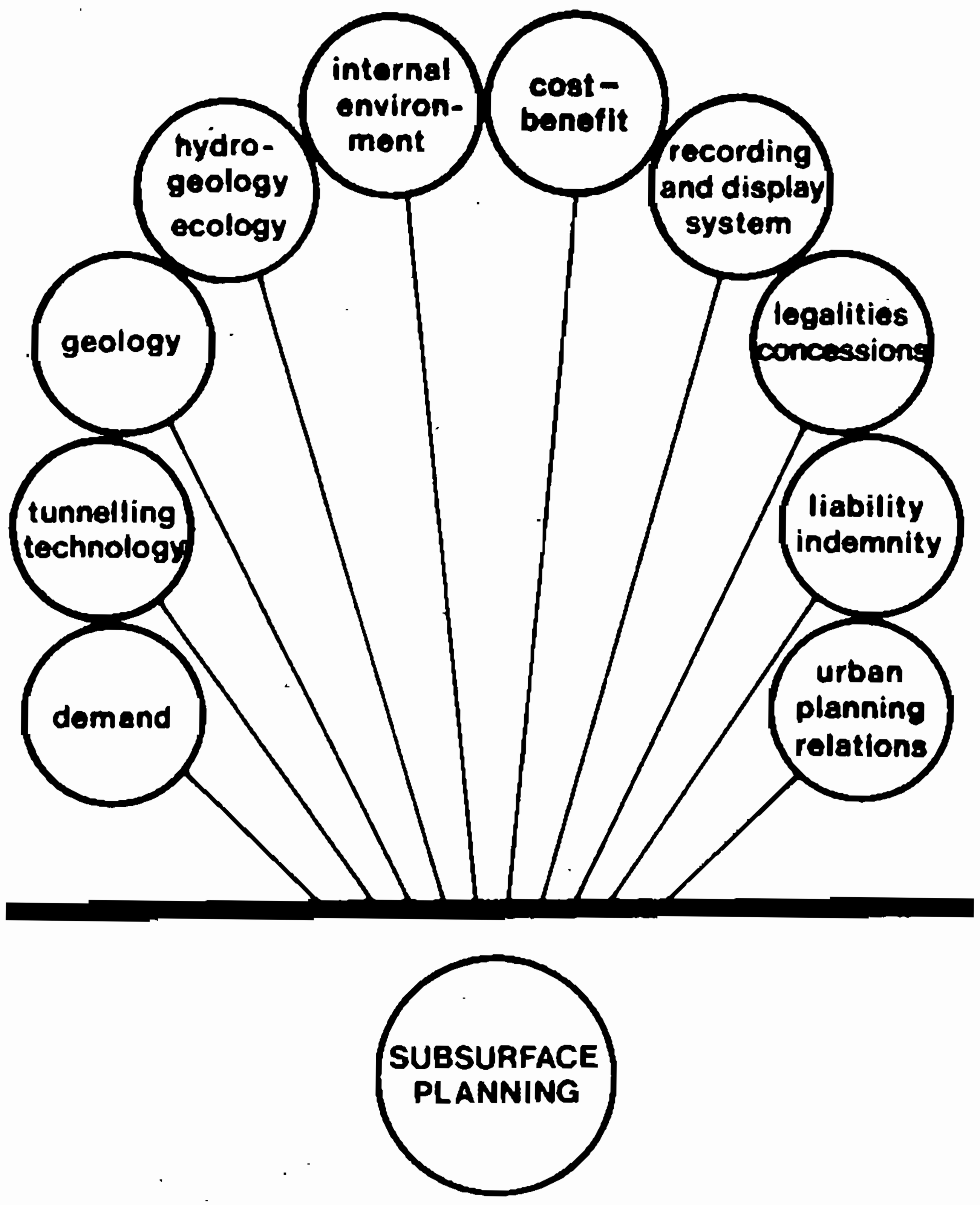


Fig. 5 Fields of knowledge affecting subsurface planning

The reasons for mishaps are three: 1) insufficient registration of the subsurface conditions including existing plants, 2) insufficient planning of the utilization of the subsurface, and 3) insufficient coordination of preparations, decisions and realization.

Because of the insufficiencies pointed out, the Swedish focal agency, IVA's Committee for Subsurface Construction, and the highest planning authority, The National Planning Administration, have worked to call attention to the subsurface construction and to achieve necessary stipulations in the current revision of the Swedish building legislation. Furthermore, it has been found necessary to increase the exchange of knowledge between town planners and subsurface constructors and to give increased information on subsurface construction to decision-makers and contractors, particularly government authorities and local authorities. Conferences similar to the type being held in Melbourne have been carried out and different types of informative publications have been prepared or are being prepared.

As part of the efforts to achieve a better coordination in the subsurface construction and an integration of the subsurface construction in the total urban building, BFR, The National Council for Building Research, has assigned to an investigation group to carry out and R&D study on subsurface planning during this and next year. The main purpose is with a general picture to collect and present the factors which affect the planning of the subsurface area utilization. The measures taken harmonize completely with the recommendations decided at the OECD conference on tunnelling in 1970.

The approach to the underground use depends on the environment in which the construction has to be executed. The US, for example, has an extremely heavy urban development and a strong pressure on a continuous strong technical and economic expansion. The attitude towards subsurface construction is also affected by clearly pronounced federal support in order to force subsurface space to unload surface space and reduce the environmental disturbances on the surface.

The Swedish apprehension of subsurface construction as drawn up within the IVA Committee for Subsurface Construction, rather markedly differs from that of most other countries. Subsurface construction in Sweden has been subjected to critical examination also from the wide public and from medical and psychological experts. The subsurface space is thus regarded not only as an important future resource for different needs for which the technological development within the field should be stimulated. Subsurface location of places of work as well as transportation routes must also be carefully tested from social and environmental viewpoints and with regard to the influence on the human being.

The R&D study on subsurface planning which I mentioned earlier and which from now on will be the basis for my presentation, partly takes up chapters on the knowledge so far of human reactions to staying under ground, partly chapters on alternatives in the total urban structure which put less pressure on subsurface localization of important human activities.

The R&D report intends to map out the level of knowledge within the fields shown in Fig. 5 thereby covering problems which are relevant for subsurface planning. The result of the investigation work intends to give methods for subsurface planning which are appropriate with regard to the demands which can be made from society and the individual. The study is assumed to be the basis for legal and administrative regulation and for continued research and international exchange of experience.

3 DEMAND FOR SUBSURFACE SPACE

The planning of subsurface construction requires that the demand for space is estimated partly on a short-range basis for immediate projects and partly on a long-range basis to make reservations for the future. Compared to estimates for more normal urban needs, estimates of subsurface space contains more parameters to be considered, mainly: 1) the functional need, 2) the urban structure in which functions shall be integrated, 3) choice of site on or under the surface.

The general motives for subsurface location can be collected under the headings:

- Protection	- Location
- Connection	- Economy

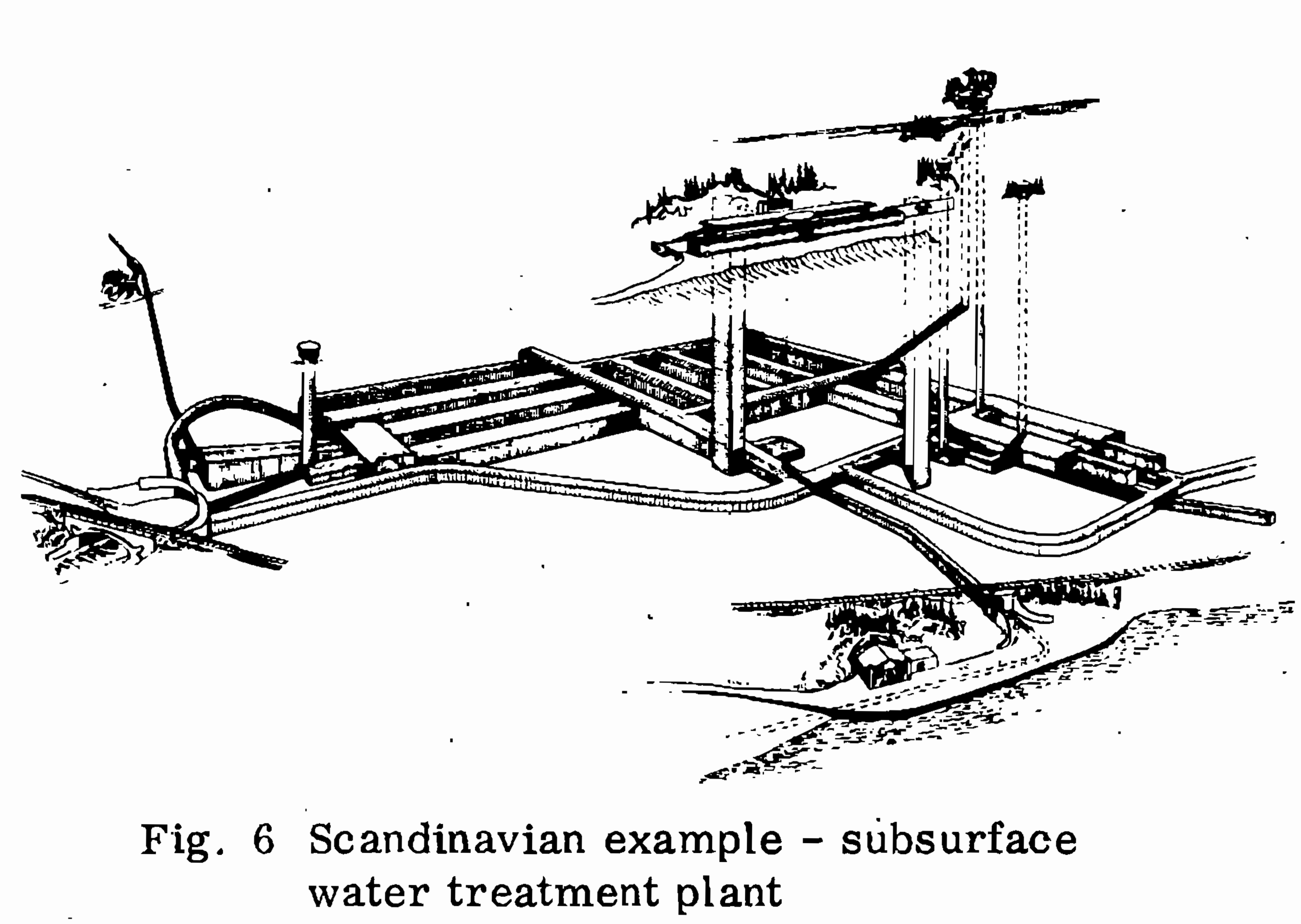


Fig. 6 Scandinavian example - subsurface water treatment plant

TABLE 1 EXAMPLES OF MOTIVES FOR SUBSURFACE LOCATION OF DIFFERENT FUNCTIONS

	TRANSPORTS															STORAGE									TERMINALS			PROD PROCESS				DEFENS INSTALL		
	fresh water	storm water	sewage water	oil	gas	heat	electricity	telephone	garbage	liquid wastes	solid wastes	bulk goods	piece goods	car traffic	rail traffic	storm water	oil	gas, liquid	hot water	pass storage (archives)	act storage (goods)	vehicles	solid wastes	liquid wastes	transports	telephone	electr (transformer)	industry	water treatment	sewage treatment	power plants	underground shelters	headquarters	others
PROTECTION																																		
climate						●			●		●	●	●	●	●	●		●	●	●	●	●			●			●						
damage	●			●	●	●	●	●							●		●	●		●	●		●	●		●	●	●			●		●	●
insight				●	●	●	●	●	●	●	●	●	●	●	●	●	●	●		●	●	●	●	●	●		●	●		●	●	●	●	●
disturb ext			●	●	●		●		●	●	●	●	●	●	●	●		●				●	●	●	●		●	●		●	●			
disturb int			●	●	●		●	●		●	●			●	●			●		●			●	●				●			●			
war damage				●	●		●	●									●	●		●	●		●	●		●	●	●			●	●	●	●
safety ext					●					●	●						●	●					●	●			●	●			●			
safety int	●						●	●									●	●					●	●										
CONNECTION																																		
shortest way	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●																			
diff in level	●	●	●	●		●			●	●	●	●	●	●	●																			
water course	●		●	●	●	●	●	●		●	●	●	●	●	●																			
other obstacle	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●																			
LOCATION																																		
suitable site	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●
lack of space	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●
ECONOMY																																		
time gained	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●
cost	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●
quality	●	●	●	●	●	●	●	●								●	●	●	●													●	●	●

The functions which, at any greater frequency, have so far been planned and carried out under ground can be distributed according to the following:

- Transports
- Terminals
- Storage
- Production, processing
- Defensive installations
- Special needs

In Table I different motives and functions have been put against each other. Location and economy appear as motives for all types of functions, while other motives appear only for specific functions.

Among special functions which are not covered by given headings the following can be mentioned: geothermal plants, mineral extraction, premises for cultural, religious and funeral purposes and museums, demonstration and educational facilities with connection to subsurface construction.

The inclination to choose subsurface localization for different functions is affected by a number of factors. Available statistics show that the following factors have a great effect on the demand for subsurface space in a given area or country.

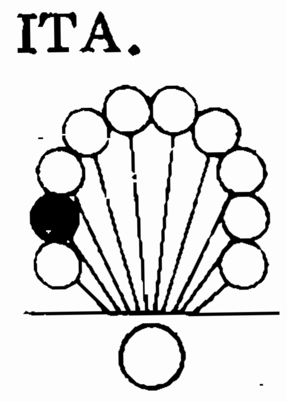
- Topography
- Geology
- Urban structure
- Urban density
- Environmental demands in the urbanization
- Technical standard within subsurface construction
- Economic standard
- Social standard
- Laws and administration in the urban building

In principle, the need value, NV, can be expressed as the total of the value of the individual factors (f) and their importance for the choice of subsurface site (k)

$$NV = k_1 \times f_1 + k_2 \times f_2 + k_3 \times f_3, \text{ etc.}$$

Already in this connection can be stated that the need within given natural conditions is affected by a thorough knowledge of the conditions for subsurface construction and a conscious planning.

The methodology of a short-range estimate ought to be based on inquiries with known and presumptive interested parties, the data of whom are compiled into a requirement projection. A long-range estimate demands in addition a more comprehensive general knowledge of how the motives for subsurface location of different functions are affected by the general inclination to choose subsurface location. Such a knowledge requires an extensive analysis of a great number of urban areas - according to my opinion an urgent project within the newly established ITA.



4 TUNNELLING TECHNOLOGY

Knowledge of the technique to carry out a subsurface construction plays a central role in the planning. Location and design are strongly affected by cost relations between different techniques. Different methods also cause different effects on the environment from a subsurface plant under construction as well as in operation.

The costs for the execution of cavities are often determinative for the choice of technique. However, sometimes the demands of avoiding disturbances to the environment are guiding factors for the technical execution.

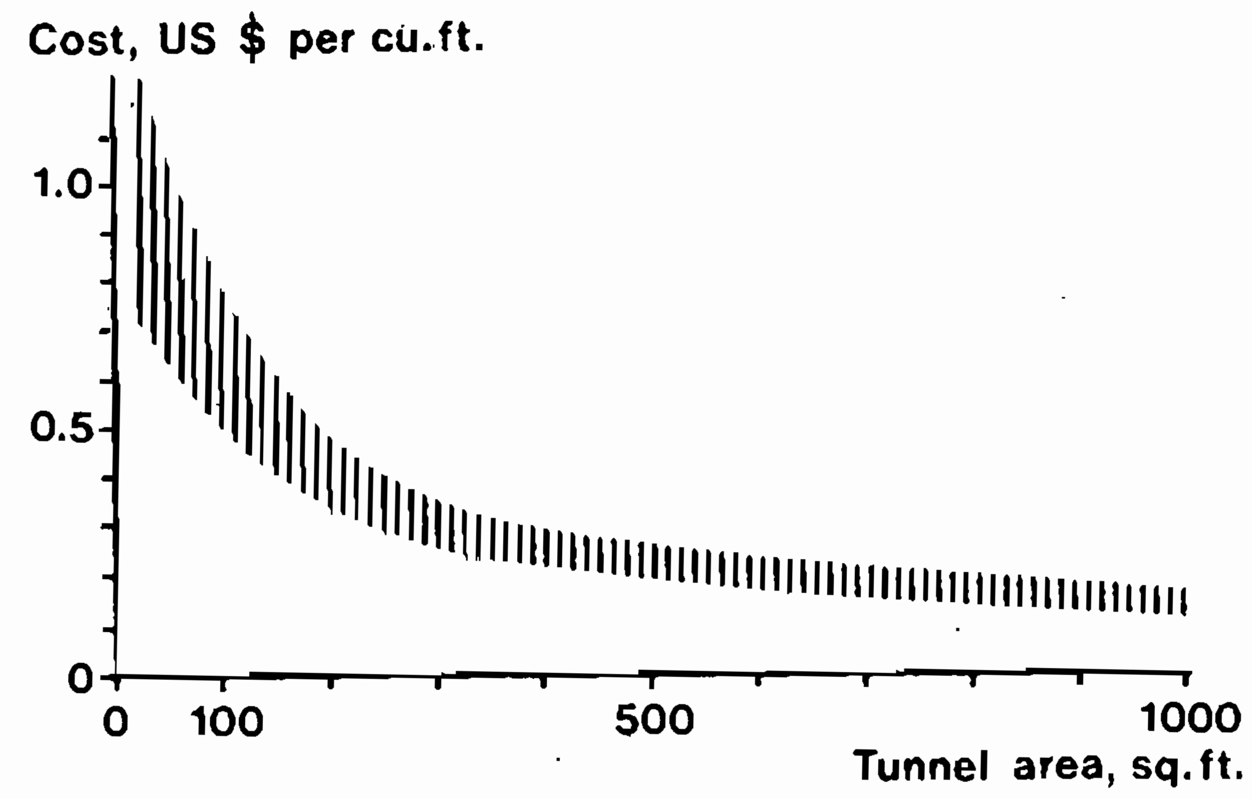


Fig. 7 Hard rock tunnelling costs in Sweden

The geological, geotechnical and hydro-geological conditions must be well known to make a technical and economic calculation accurate enough.

Generalized ground conditions, size of cavity and tunnelling technique are variables in a cost classification which can be utilized for rough estimates of the construction costs when choosing between different alternatives in a subsurface planning (example Fig. 7). Different methods to pay particular consideration to the environment, such as built up areas sensitive to disturbance, shall also be considered as a variable in a cost pattern. In this connection the entire construction process of the subsurface plant must be included, thus also transports, storages and fillings.

In Sweden, subsurface construction in hard rock through boring and blasting dominates completely. An ever developing technique and organization of work have strongly increased the capacity (Fig. 8). Well developed methods of loading calculation and standardization of permissible vibration have made boring and blasting possible also in a sensitive environment. Full face boring has been tried but test projects have indicated a tripled cost in granite compared to the usual technique. Cut-and-cover dominates in soil, but boring and blasting in frozen or petrified ground has been used.

The range of possible methods for tunnelling in solid rock is great (examples: fluid erosion, jet piercing, laser beam, nuclear energy), but according to Swedish experience, mainly boring and blasting is commercially useable in our rock. The usual methods for other geological regions are not expected to be applied in Sweden at any considerable degree.

5 GEOLOGY, SOIL MECHANICS

Urban subsurface plants are often located close to the surface to obtain short ways for connection and construction. In doing so the plants are located in a shallow rock zone, the characteristics of which have so far been little known. The zone from the ground surface down to the rock is regularly tested at soil mechanical tests for surface construction, and large rock depths are known through the mining technique. But the zone in between has so far been a no man's land, which the tunnel constructor must now investigate.

In this connection it is important to create, from the beginning, a system for which subsurface data should be collected for different stages in planning and projecting works involving subsurface plants. The main principle should be

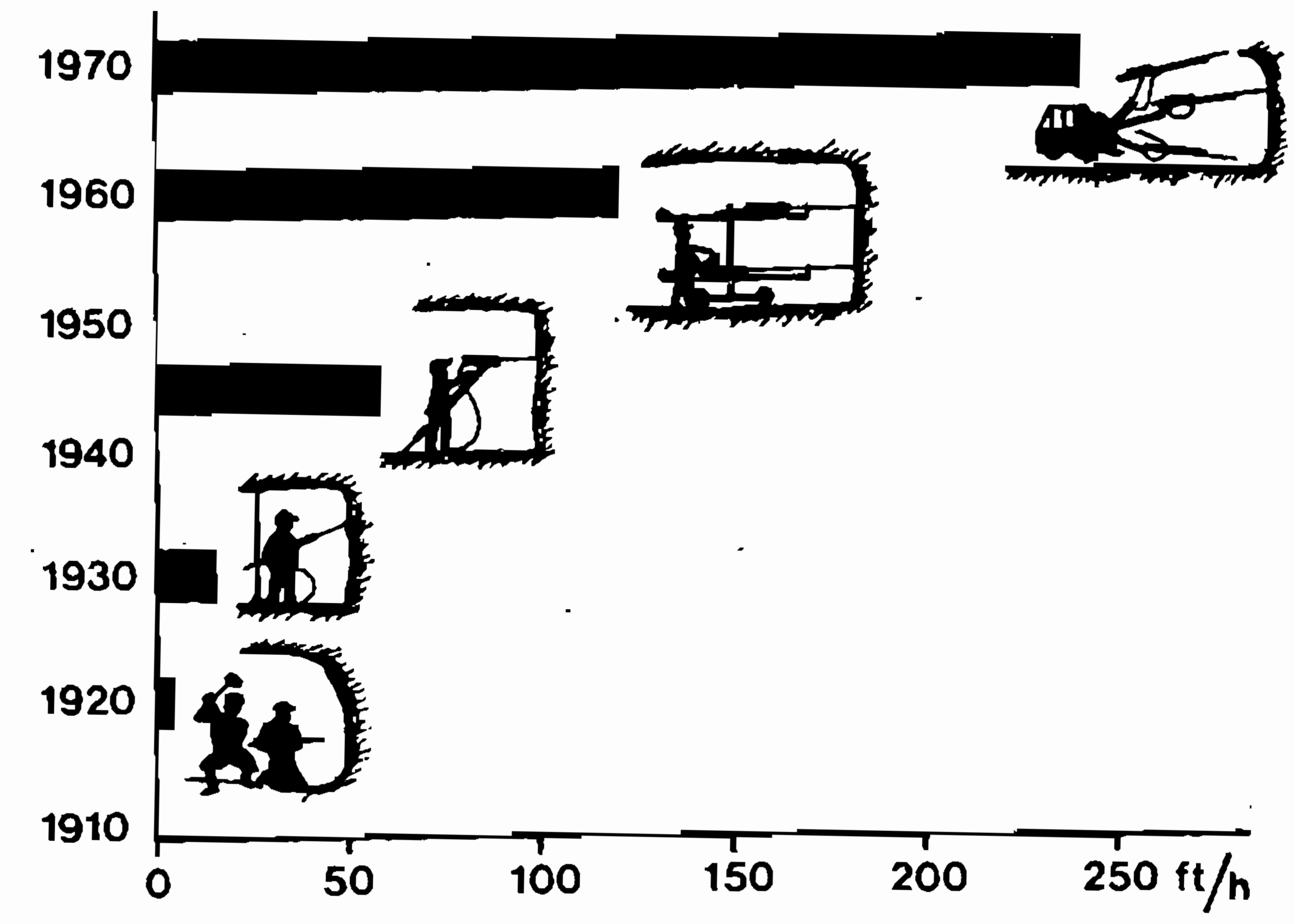


Fig. 8 Development of tunnel drilling

- general, indicative investigations at a reasonable price for long-range planning (for example master plan)
- elaborate investigations for immediate feasible planning

Data information should be collected centrally from pre-investigations, detailed in-situ investigations and completed plants.

For planning purposes it is necessary to translate the information from investigations into a form which is directly applicable when alternate locations for subsurface plants are to be compared and connected to the entire surface planning. This can be done through a map presentation of areas with different ground cost indices, either stated in unit prices or in relation figures to the best ground in the entire area.

The goal for geological studies in connection with urban planning should be to obtain a basis for finding suitable sites and areas for subsurface plants which can be reserved for long-range needs.

6 HYDRO-GEOLOGY, ECOLOGY

In Sweden's loose layers of clay there are today problems with settlements and rotten wood piling due to groundwater lowerings in a great number of large cities, and in many areas leaky tunnels are the cause of the groundwater loss (Fig. 9). In spite of the fact that tunnels are most often built in hard rock where the problems are possibly less than in soft rock or in soil, leakages through slits are considerable. As a thumbrule it has been mentioned that a zone of a mean width of 500 meters above the tunnel will have its groundwater balance disturbed.

Just as regards the geology, a system should be worked out as to what hydro-geological investigations should precede planning and projecting on different levels. Elaborate hydro-geological preinvestigations give possibilities of predicting the extent of for example settlements, so that suitable preventive measures can be

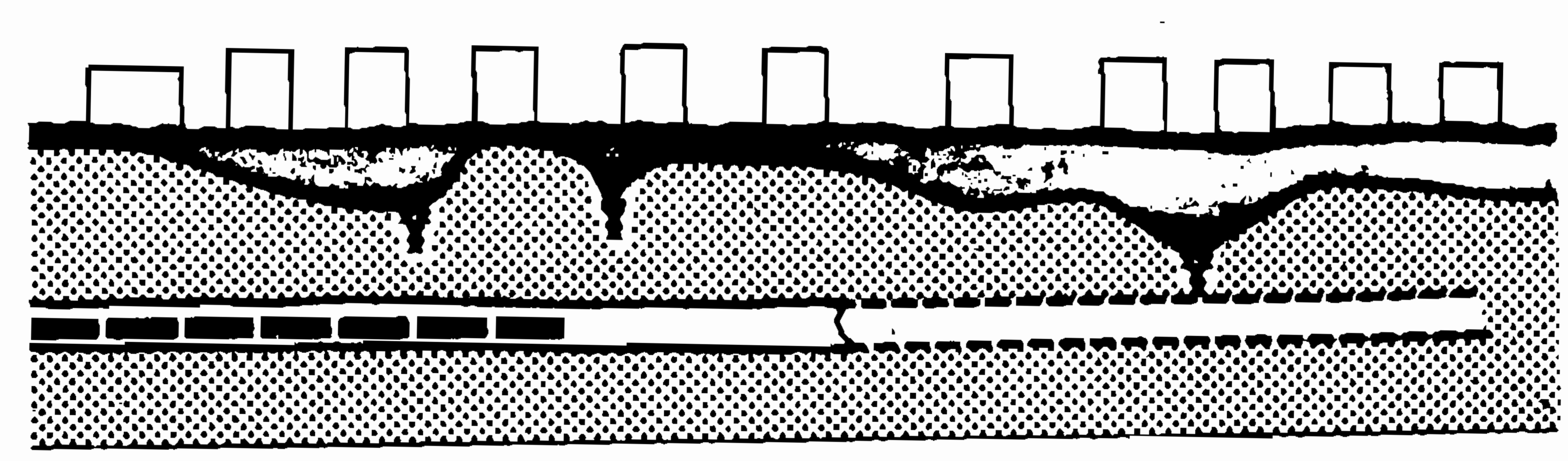


Fig. 9 Water break-through at Stockholm subway

prepared. For planning purposes hydro-geological information should be mapped where the sensitivity of different areas to subsurface construction can be estimated and preventive measures can be cost determined.

The biological consequences of subsurface construction in rock and groundwater lowering are minor. The vegetation is mainly dependent on the water in the surface. Subsurface construction in soil can, however, considerably affect the biological part of the echo system.

7 INTERNAL ENVIRONMENT (HUMAN REACTION DURING STAYS UNDER GROUND)

An increased knowledge of medical and psychological reactions to man's staying under ground has, in the Swedish consideration regarding subsurface construction, been found particularly important. The debate on location above or under ground of mainly public transportation has strongly emphasized this need.

This knowledge has a strong effect on the location of plants but also on demands for design and equipment of subsurface spaces, for example

- Temperature
- Humidity
- Ventilation
- Lighting
- Noise level

A classification of subsurface plants with regard to internal environment can be made according to these criteria and according to how long time people have to stay in the plant.

Earlier Swedish investigations in underground factories have shown that stays under ground did not cause any fysiological inconveniences if the climate was correctly adapted, while psychological reactions of importance occurred.

An example from the Swedish debate is related to the planning of a subway to a new residential area in Stockholm. Criticism arose against the fact that the public commuter was to stay under ground at least 1 hour per day in order to go to work (Fig. 10). According to the critics the public traffic should have priority to the less efficient car traffic when choosing the best location - above ground.

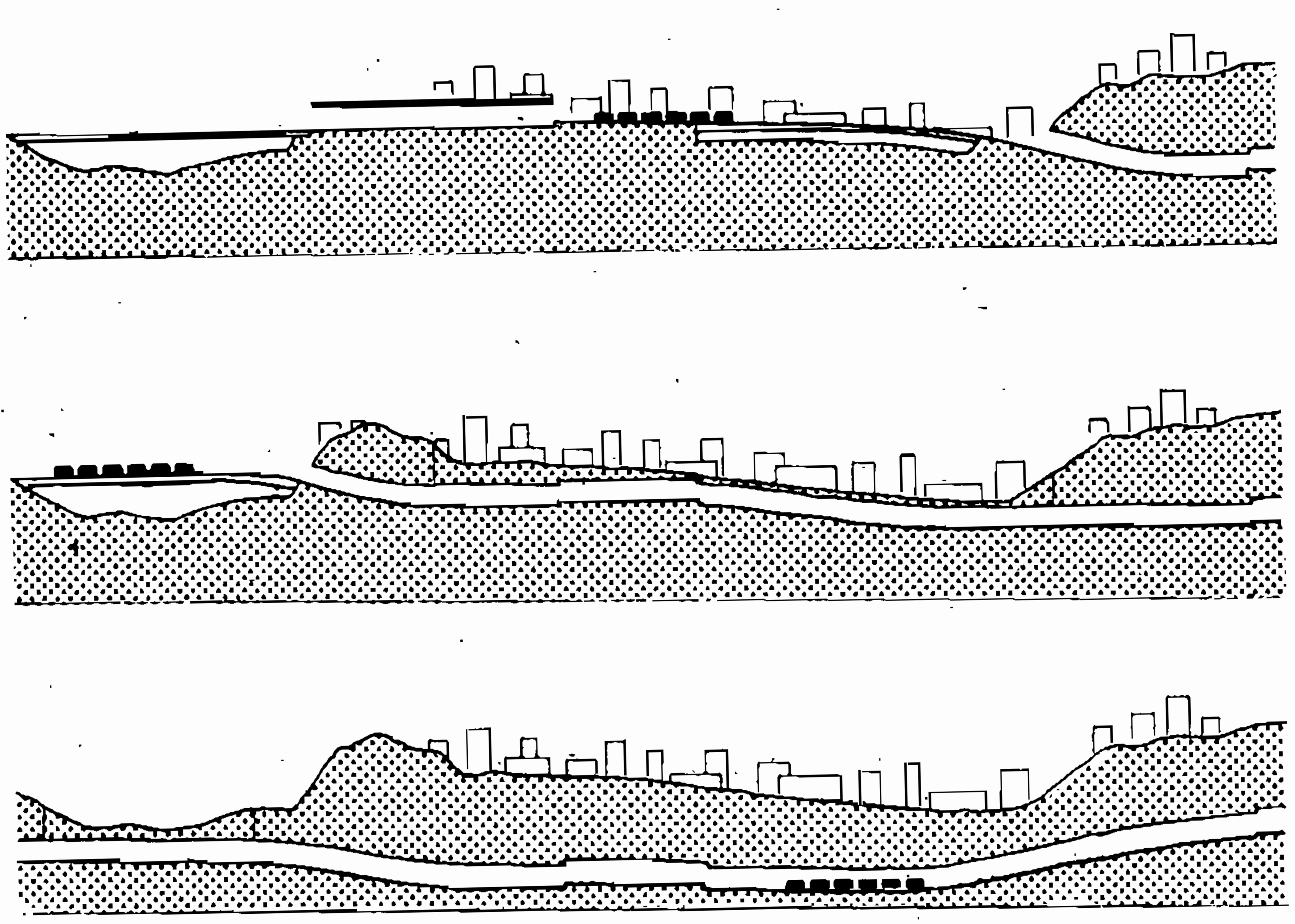


Fig. 10 Opinion against underground travels forced investigation of subway on different levels

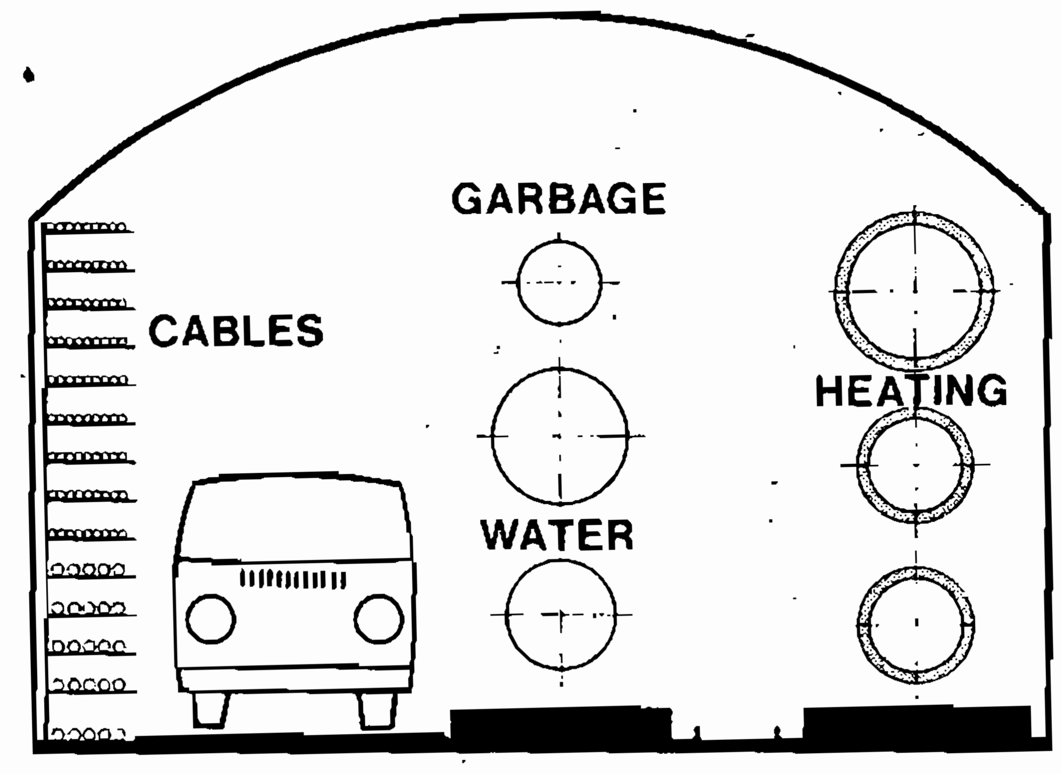


Fig. 11 Multi-purpose tunnel in Gothenburg

Also the workman's conditions during the actual construction has been paid special attention to in our country and comparatively great efforts have been made to make these conditions satisfactory.

For these reasons our R&D study on subsurface planning has included a special expert group consisting of scientists within the fields of psychology, ergonomics, medicine and behaviour science. In addition, in a reference group there are representatives of the national authority on labour welfare questions.

8 ECONOMY (COST-BENEFIT)

The coming into existence of a subsurface plant is today most often the result of an optimization of initial costs, i.e. a sub-optimization for society. In many cases, however, the economic optimization coincides with the result from a more complete cost-benefit analysis. There are also examples that the social benefit has been overrated so that larger tunnel space than the totally optimal has appeared favourable. To ensure a public confidence in subsurface construction, the benefit part of the analysis must be given great consideration.

As an example of performed cost-benefit analyses, multipurpose tunnels for district heating, water, sewage, electricity, and telephone cables in central business districts and new residential areas in Sweden (Fig. 11) can be mentioned. Even if traditional surface location of individual pipes and cables would be cheaper in some cases, the tunnel alternative has been chosen. Tunnels mean radically reduced conflicts with street traffic and building erection, and an unloading of the land-use close to the surface. In addition, profits are made regarding maintenance, inspection and safety in operation.

A complete cost-benefit estimate of a proposal for tunnelling in urban areas, for example a traffic route, often requires a very wide analysis where also all factors pointed out under "Internal environment" including human reactions shall be included. Future development with tunnels as well as other solutions (including that no measure is taken at all) must be estimated with a correct horizon of time. Just as in other cost-benefit analyses we meet with the problems of finding appropriate discounting rates for both costs and environment values and of weighing cost against quality.

9 RECORDING OF SUBSURFACE PLANTS

An extremely urgent task in the planning preparations is to inventory and registrate existing subsurface plants. In this connection also the subsurface parts of buildings on the surface should be recorded in a surveyable way. Uniform rules for registration should be worked out. The illustration of location and shape of subsurface plants makes great demands as there are no points of reference in the subsurface area. There is a need of an easily comprehensive way of describing subsurface plants during the planning stage as well as for reports to decision-makers and parties concerned.

The mutual location of different objects and the location in relation to the built-up area above ground can, besides in traditional drawing methods, be stated in physical models or perspectives. By utilizing data determination of the objects and data perspective programmes, a visualization of extensive plants can be made at low costs.

10 LEGALITIES, CONCESSIONS

In Sweden, as well as in most other countries, there is no common legislation which controls the planning and granting of permits for subsurface plants. Legislation is today mostly tied to a certain field of application, and not to subsurface localization (Fig. 12).

A current investigation of a new Swedish building legislation suggests that need and location of subsurface spaces shall be stated in key plans for entire communities, for densely populated areas and in detailed plans. In addition, it is suggested that subsurface plants shall require building permits. Important questions to be considered in legislation are the right to the subsurface and the possibilities of limiting this right. Restrictions of the right to build surface buildings deeper than 7 meters below the surface have, in some cases, been applied in our country.

11 LIABILITY, INDEMNITY

Characteristic for the situation in Sweden is that there are no prejudicial decisions regarding responsibility when damages occur and that a number of newly established laws have not yet created a clear case law. Prevailing laws stipulate in principle that during a risky or

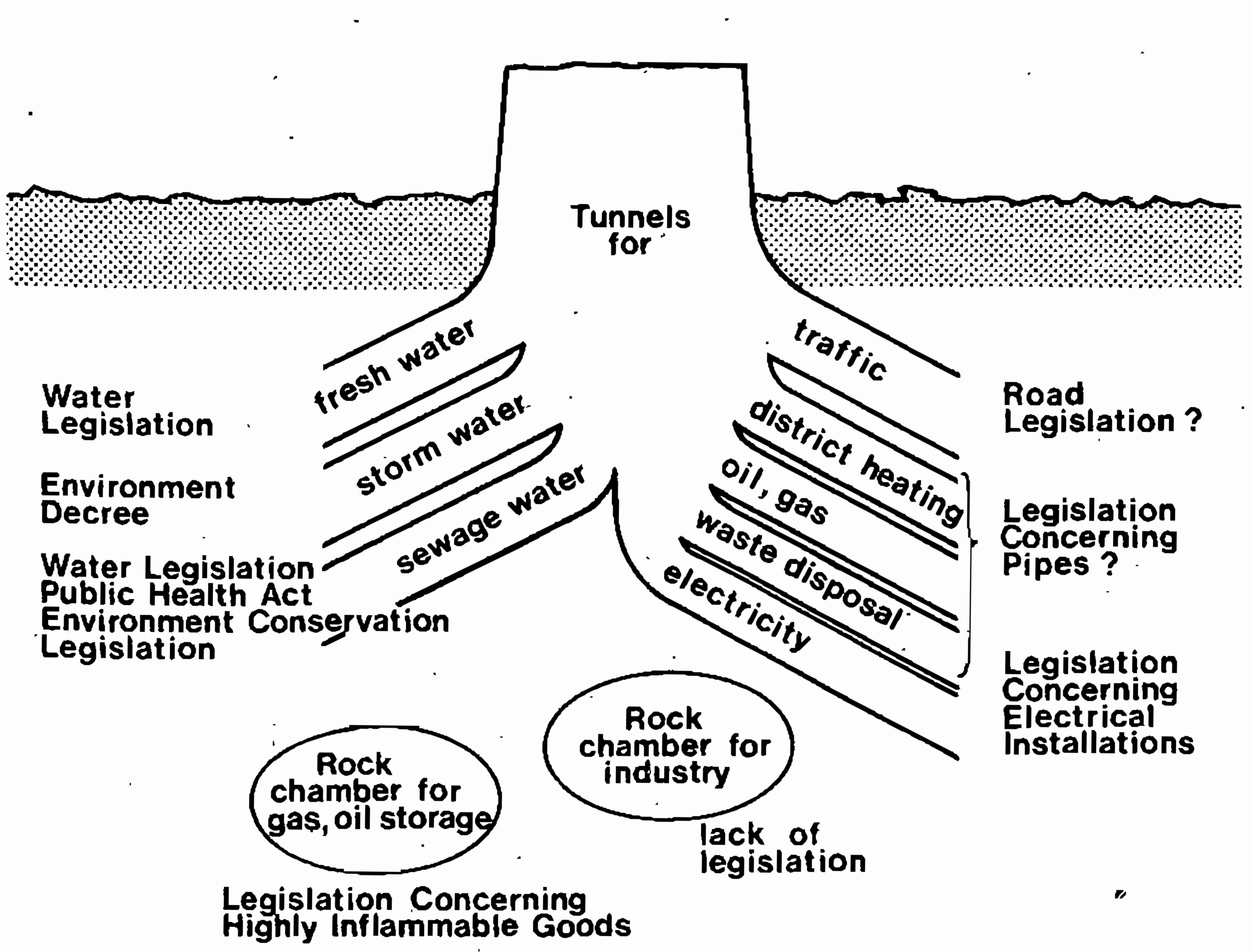


Fig. 12 Existing Swedish legislation on subsurface plants is complex and contradictory

extensive ground construction (such as for instance subsurface construction) the person who is executing or is having the work executed is strictly responsible for damages to the adjacent land. The damages need not have occurred through negligence but it is sufficient to prove a clear connection between construction and damage for receiving compensation.

Damages during blasting have been legally treated and in these cases the customer and the contractor were jointly and severally responsible for the damage, that is compensation can be demanded from any one of the two. For damages through groundwater lowering, after the actual construction, the tunnel owner is strictly responsible.

Experiences from various cases of damage show that small property owners are mostly the losers and that they are not able to claim their right towards for instance government and local authority interests.
Through more careful mapping of geological, geotechnical and hydro-geological situations before and after construction of the plant the responsibility for damages occurred can be easier to trace.

12 URBAN PLANNING, RELATIONS BETWEEN SURFACE AND SUBSURFACE

Most of the urban subsurface constructions are closely connected to the urban planning on the surface. The demand and the location of tunnels and other subsurface facilities are caused and decided by the surface utilization. Subsurface constructions promote the surface utilization but also give restrictions related to disturbance or demand of protection. Town plans and other urban plans have to a very low extent been carried out with aspects on or demand for subsurface use. As a rule, urban planning includes a long-range requirement on land and supply. Corresponding plans for subsurface space are often missing.

It is quite clear that urban planning has sometimes been executed with so little foresight that human activities have unnecessarily been forced under ground. There is also the risk that subsurface location of supply or communication routes does not only solve an immediate space problem but becomes an incentive to increased land-use on surface, which in turn creates new space problems and new demands for subsurface localization.

The attention to the subsurface use and the activities in subsurface planning have to be related to a predicted demand. A normal routine in urban planning should however also include an impartial evaluation of the possible subsurface location for different purposes. In existing high density urban areas subsurface solutions could be demanded in order to give the greatest possible satisfaction and freedom of action in a long range. In new urban areas the task may be to avoid future dilemma by choosing densities and structures in the surface planning which take good care of the subsurface resources but maintain the freedom to choose between underground and above ground.

There is reason to believe that different urban structures give different tendencies to utilize the subsurface. Already the general basic types for urban structure seem to have different pressure on the subsurface utiliza-

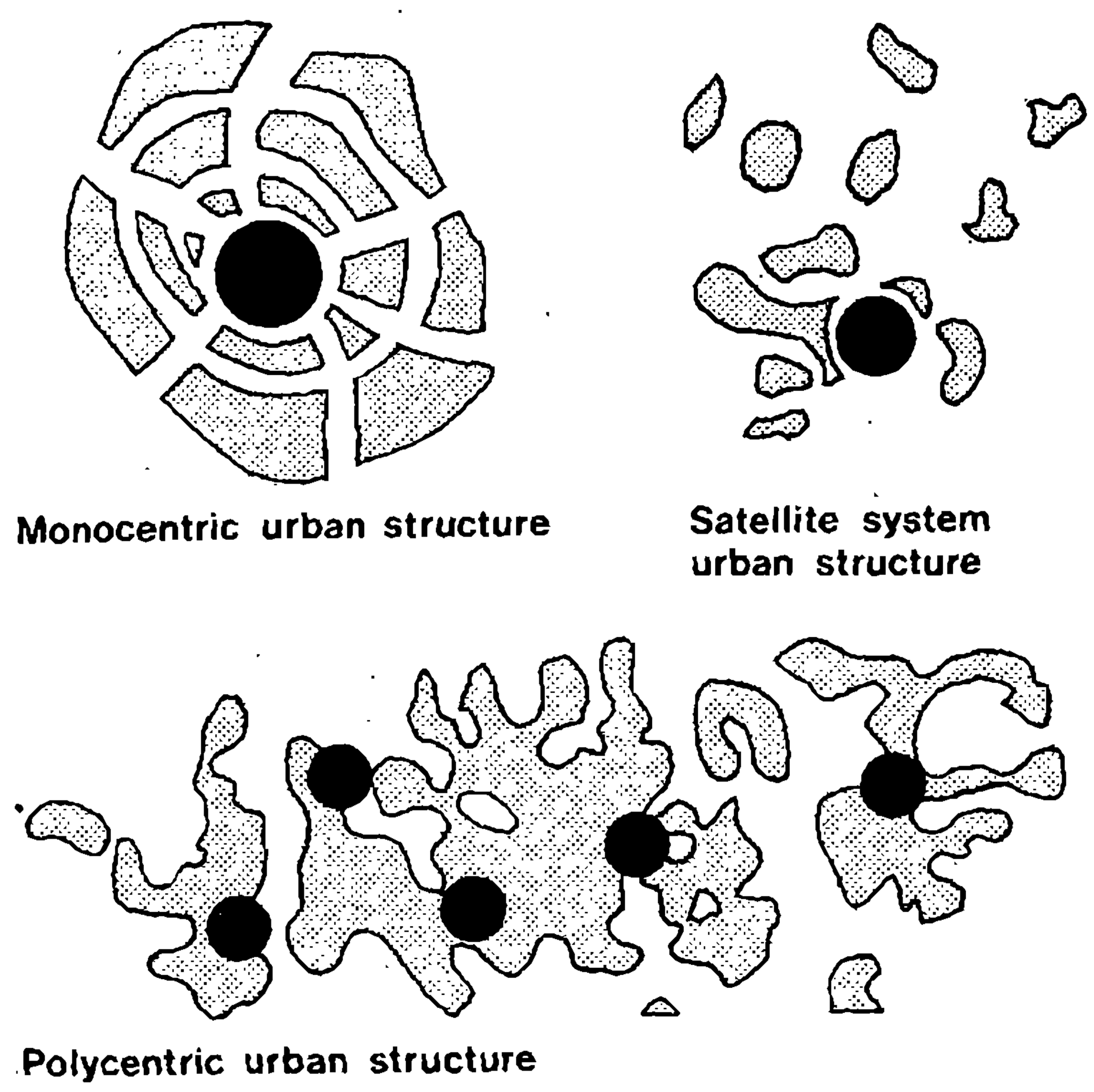


Fig. 13 Different urban structures influence the demand for subsurface space

tion in the centre. The monocentric urban structure (Fig. 13) causes high pressure from the urban local area and from surrounding regions with growing density in land-use and population. This structure seems to require more and more subsurface use in an unregulated expansion. The structure with one centre but distributed satellites still gives a high pressure from the surrounding region, but a lower pressure from the local suburban areas. An expansion seems to give possibilities for a more free choice of localization under ground or above ground of important supply and communication functions. The polycentric structure distributes the pressure to several points and will probably be still more flexible for the use of the subsurface area.

13 SUBSURFACE PLANNING - CONSLUSIONS

A main task in the field of subsurface planning is to call attention to "Terraspace" as the "hidden resourse", to inform and activate urban planners, technicians and responsible decision-makers.

In a national perspective it is required that available knowledge of subsurface construction is collected, analysed and made available for the planning. When so required, the knowledge shall be broadened and deepened within the fields which have not been sufficiently penetrated.

The need of subsurface space must be estimated especially with regard to necessary future reservations. As the need is closely linked to the choice between location above or under ground, it is necessary to make complete cost-benefit analyses of the possible objects. For long-range needs the demand of freedom for future choice should be observed.

The subsurface planning requires a systematic mapping with suitably weighed accuracy of existing plants as well as the ground conditions.

Social, environmental and human aspects on the subsurface utilization demand great attention, not in the least to prevent a lack of confidence between technology and society.

Planning methods, legal regulation and administrative handling of subsurface plants shall be seen as an integrated part in the total urban planning.

Pilot Tunnel Investigations into Aspects of Excavation and Primary Support for Melbourne Underground Rail Loop Tunnels

by

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SUMMARY. Two pilot tunnels were constructed for the Melbourne Underground Rail Loop to assess behaviour of primary support, construction methods and provide exposures for future tenderers. The Adderley Street pilot tunnel was a small drift driven through basalts, sandy clays and into weathered Silurian mudstones, whilst the Treasury Gardens pilot tunnel was a full size section of running tunnel constructed in Silurian strata. Together, these structures enabled assessment of rib, rock bolt, and shotcrete support systems, particularly in regard to ground settlement problems.

Results from extensometer and slopemeter installations showed the effectiveness of a shotcrete skin in conjunction with ribs or rock bolts in developing a rock arch and limiting ground movement over the tunnels to a very few millimetres for most circumstances. Other results obtained included information on rock modulus and blasting vibration characteristics. Construction techniques including shotcrete ground support and excavation by a small tunnelling machine were assessed. The results from the pilot tunnel investigations were then further tested in a small preparatory contract before commiting major works with the shotcrete-rib method.

1 INTRODUCTION

Among the major early decisions facing John Connell - Mott, Hay & Anderson, Hatch, Jacobs, the Principal Consultants to the Melbourne Underground Rail Loop Authority (MURLA) in 1971/72 were those associated with methods of excavation and primary (or "temporary") tunnel support.

The tunnelling phase of the Underground Rail Loop comprises 10,150 metres of running tunnel, 1,280 metres of platform tunnel and 350 metres of escalator or concourse tunnels ranging in excavated span from 7 metres to 10 metres.

The majority of tunnelling (90%) is in rock with the remainder in mixed face or wholly soft ground. The soft ground tunnelling is confined mainly to one contract, yet to be let. On the other hand, the five contracts planned for award from 1972 to 1974 all lie either wholly or predominantly within a rock or mixed ground zone. Additionally, the proposed tunnels are significantly larger than any previously driven in the Melbourne area and are driven in close proximity to each other and to substantial urban structures.

It was thus of some importance to clarify early the particular problems associated with the varying rock and mixed ground types to enable the early contracts to be proceeded with in an orderly, economical and safe manner. As a result, the Authority approved an early recommendation by the Principal Consultants to construct two preliminary underground structures to assess the effectiveness of the methods of support proposed. In addition to this, the preliminary structures made it possible to evaluate methods of construction, and significantly to expose underground strata for examination by future tenderers. In this regard it should be noted that apart from diamond drill cores there is virtually no exposure available in or near the City area to enable visual examination of the rock mass at tunnel depth.

These structures were designed, instrumented and supervised by MURLA's Principal Consultants in conjunction with the designs and specifications then being drawn up for the first major contracts. Thus, time was at a premium and an extended and intensive instrumentation programme was not appropriate in this context. Rather, engineering judgment was necessary in using the limited time to the best advantage. This paper describes the programme and discusses some results.

2 GEOLOGICAL BACKGROUND

The geology of the City area has been described elsewhere (Ref. 1) and for the purposes of this paper in relation to the MURL works can be most simply considered as two distinct areas:

(a) The area to the east of Elizabeth Street (along La Trobe and Spring Streets to the tunnel portals adjacent to Wellington Parade). All structures in this area are within the Silurian sedimentary bedrock of the city. The two main factors influencing tunnel design and construction methods in this area are the many discontinuities arising from intensive folding, jointing, faulting and dyke intrusions, together with the variable and deep weathering (Ref. 2).

(b) The area to the west of Elizabeth Street (along La Trobe Street to the portals beyond Spencer Street). Structures in this area lie in one or a combination of Silurian sedimentary strata, the Werribee Formation (sandy clays), Coode Island Silts and weathered Older Basalts. A deep alluvial pocket at Elizabeth Street penetrates the alignment of the upper tunnels.

The basic relationships of the various formations and the general location of the pilot structures are shown in Fig. 1. This profile was developed by the Mines Department as a result of previous investigations of the route carried out by the Railway Construction Board in the years 1961-1971.

boring machine operation.

In addition to this application of relatively light ribs placed and preloaded immediately on exposure of the ground, the Principal Consultants also had access to results of overseas advances in ground support by the so-called "New Austrian Method" using shotcrete (Ref. 4). However there appeared to be serious cost problems in applying shotcrete in this manner in thicknesses sufficient for sole support, as well as major difficulties in applying shotcrete in the required quantities to a machine bored tunnel.

For these reasons what might be termed a modified "New Austrian Method" using only a thin shotcrete skin in conjunction

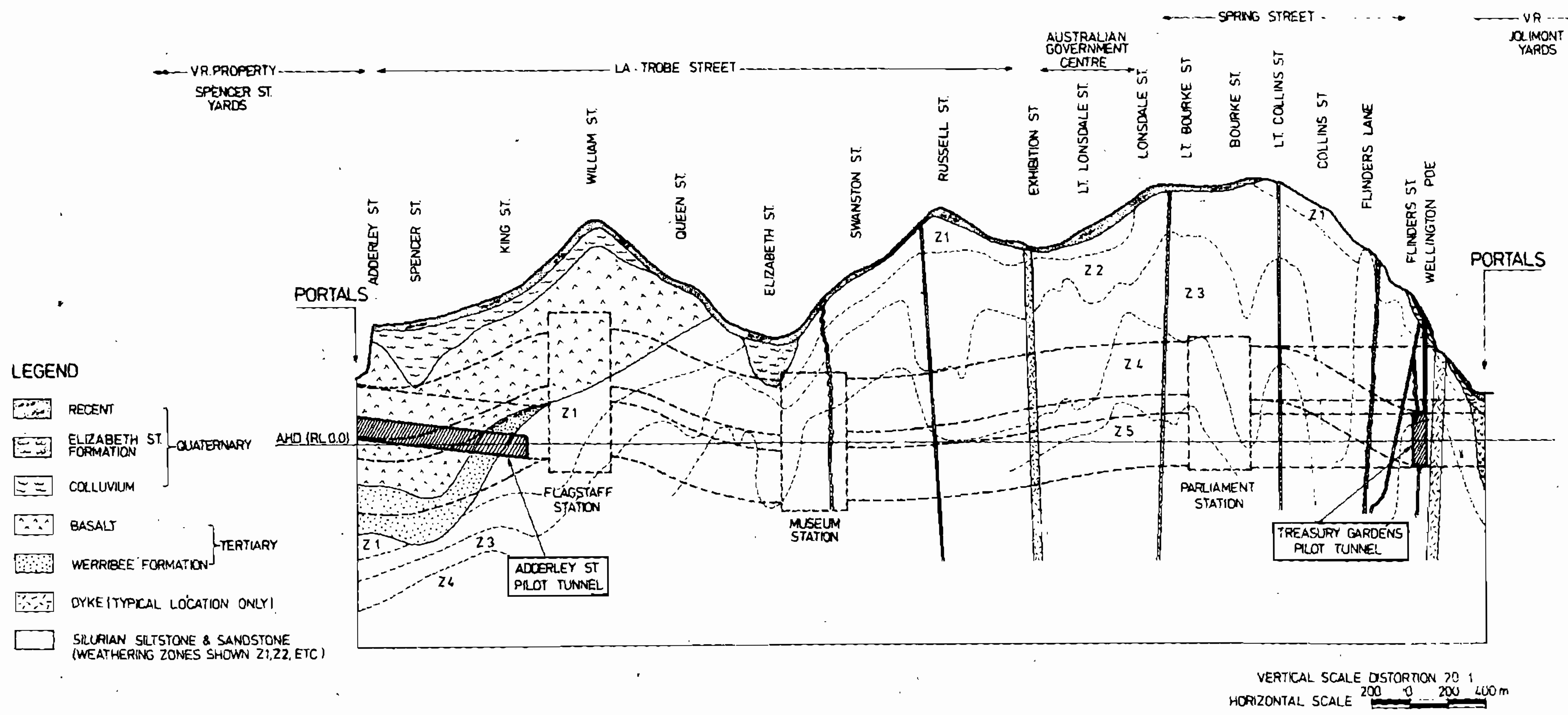

Fig. 1 Simplified Geological Profile along route Melbourne Underground Rail Loop

3 PRIMARY SUPPORT CONSIDERATIONS

The basic requirements of primary ground support in the MURL project arise from three major considerations:

(i) That of careful ground control of excavation to avoid undue ground movement in the zone of the neighbouring tunnels (either before or after the excavation of the nearby structure). The close proximity of the four adjacent 7 metre excavated diameter tunnels is notable in this respect. Typical separation of running tunnels is 0.5 diameters vertically and 1.2 diameters horizontally.

(ii) That of limitation of ground subsidence both at the surface and adjacent to the foundations of the urban structures along the route of the Loop.

(iii) That of systematic full support in even the best of the ground in order to maintain safety and organized excavation procedures due to extremely variable ground competency of the entire route. Typical local experience of the support problems in this type of ground are given in Ref. 3 where light preloaded ribs and mesh were successful with a full face

with steel ribs was proposed to generally meet the circumstances of the MURL project. This method appeared to be applicable to both conventionally excavated and machine bored tunnels, and the opportunity was taken to make both quantitative and qualitative tests in the two pilot structures. Shotcrete was therefore introduced on a trial basis into the Adderley Street pilot tunnel and then further exploited in the Treasury Gardens pilot tunnel in conjunction with either bolts or ribs.

4 ADDERLEY STREET PILOT TUNNEL

(a) General

This tunnel was located to penetrate the three major formations - Basalt, Werribee Formation and Silurian bedrock - to be encountered in the major tunnelling Contract to be let for the western end of the Loop. It was planned as a bottom heading of one of the future running tunnels. Because of the possible difficulties to be encountered in the Werribee Formation a rectangular section of 2.7 x 2.6 metres was chosen to facilitate forepoling if required. A cross section of the tunnel is given in Fig. 2.

Fig. 2 Adderley Street Pilot Tunnel

Portalling commenced in November, 1971 and tunnelling for a distance of 413 metres was completed in September, 1972. The work was carried out by the day labour forces of the Railway Construction Board. Data on ground movement was obtained from multipoint and singlepoint extensometers and from slopemeter installations. Further details of these installations are shown in Fig. 3.

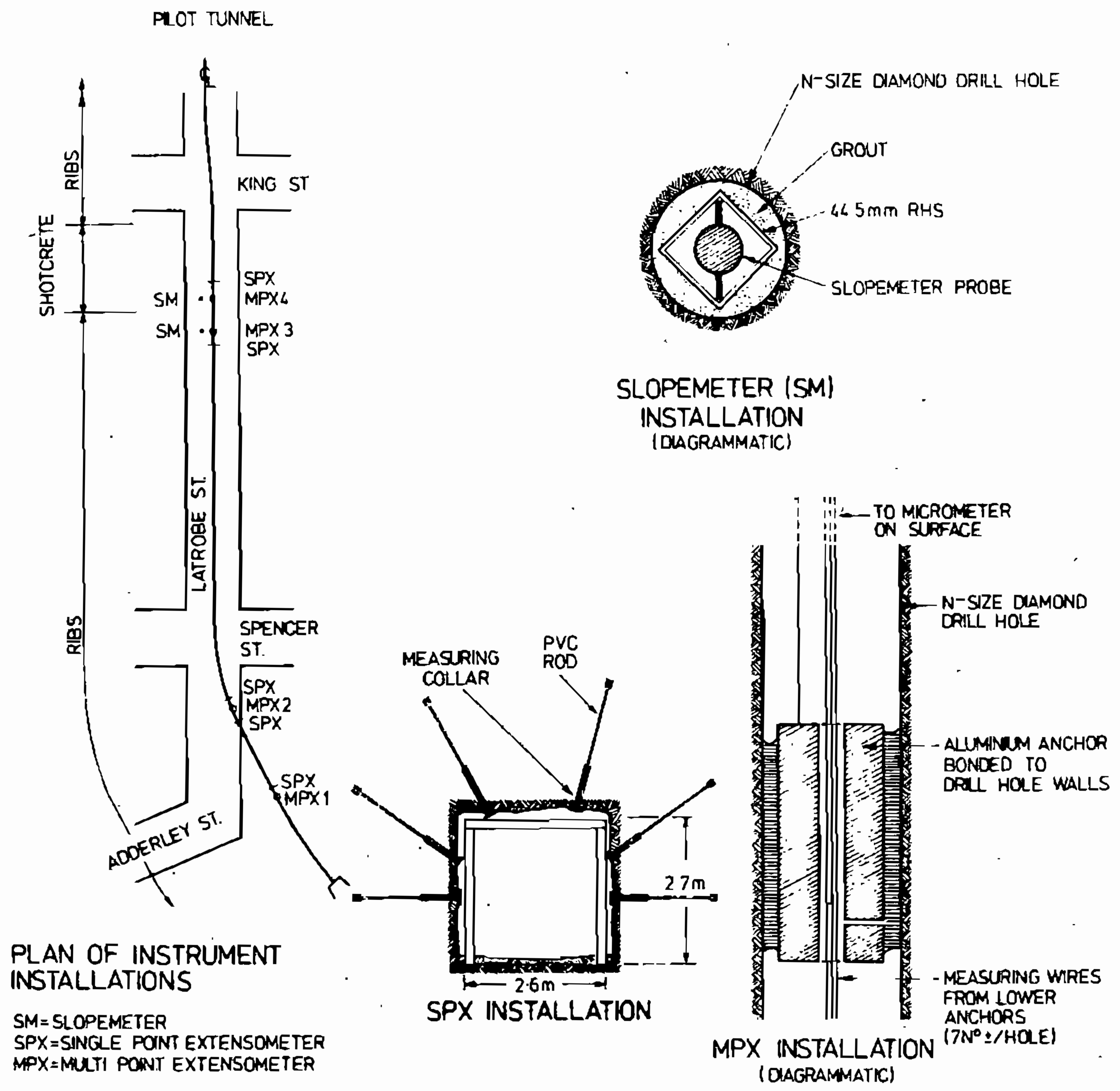

Fig. 3 Monitoring Installations Adderley Street Pilot Tunnel

(b) Multipoint Extensometer (MPX) Installations

The MPX technique adopted was similar to that reported by Bennet and Peck (Ref. 5). The aluminium anchors were bonded in holes drilled from the surface at various elevations above the tunnel at locations generally determined by rock quality as predicted from core recovery. The anchors in a drill hole were connected with stainless steel wires to a Potts Mark II Extensometer head mounted at the surface. The first three MPX installations were above rib supported areas of the tunnel and the fourth was above a shotcreted zone. Results can be briefly summarized as follows:

(i) Perceptible upward anchor movements occurred as the tunnel face approached each installation.

(ii) The final magnitude of downward deformation at tunnel crown varied from 4 to 12 mm.

(iii) The zone of deformation extended from 1.2 H to 1.7 H above the tunnel of height H.

(iv) The rock mass stabilized when the full face had advanced from 3H to 33H beyond respective installations.

(c) Slopemeter (SM) Installations

Associated with two of the above MPX installations, a group of 4 SM tubes was installed. The slopemeter (Soiltest Type C-350) is an electrically monitored inclinometer in a waterproofed probe. For borehole use the probe runs in either diagonal of a square tube grouted in the borehole to measure lateral movements in either plane.

The nearer tubes, 0.76 m from the tunnel, showed a maximum deflection of 11 and 15 mm away from the excavation walls, whilst another 1.68 m from the wall recorded movement of 5 mm away from the wall.

In all cases the outward movements were of the same order as the downward crown movements recorded by the MPX. A typical SM result is given in Fig. 4.

(d) Single Point Extensometers (SPX)

To further investigate the zone of deformation about the tunnel, series of SPX were installed from inside the tunnel. These consisted of PVC tubes of various lengths anchored by suitable bonding to the ends of drill holes of appropriate lengths, and installed immediately upon excavation of the tunnel face. Movements of the anchor point relative to the collar were measured with vernier calipers. Although cheap and simple in concept, results tended to be variable due to installation and measuring problems in the weathered rock mass.

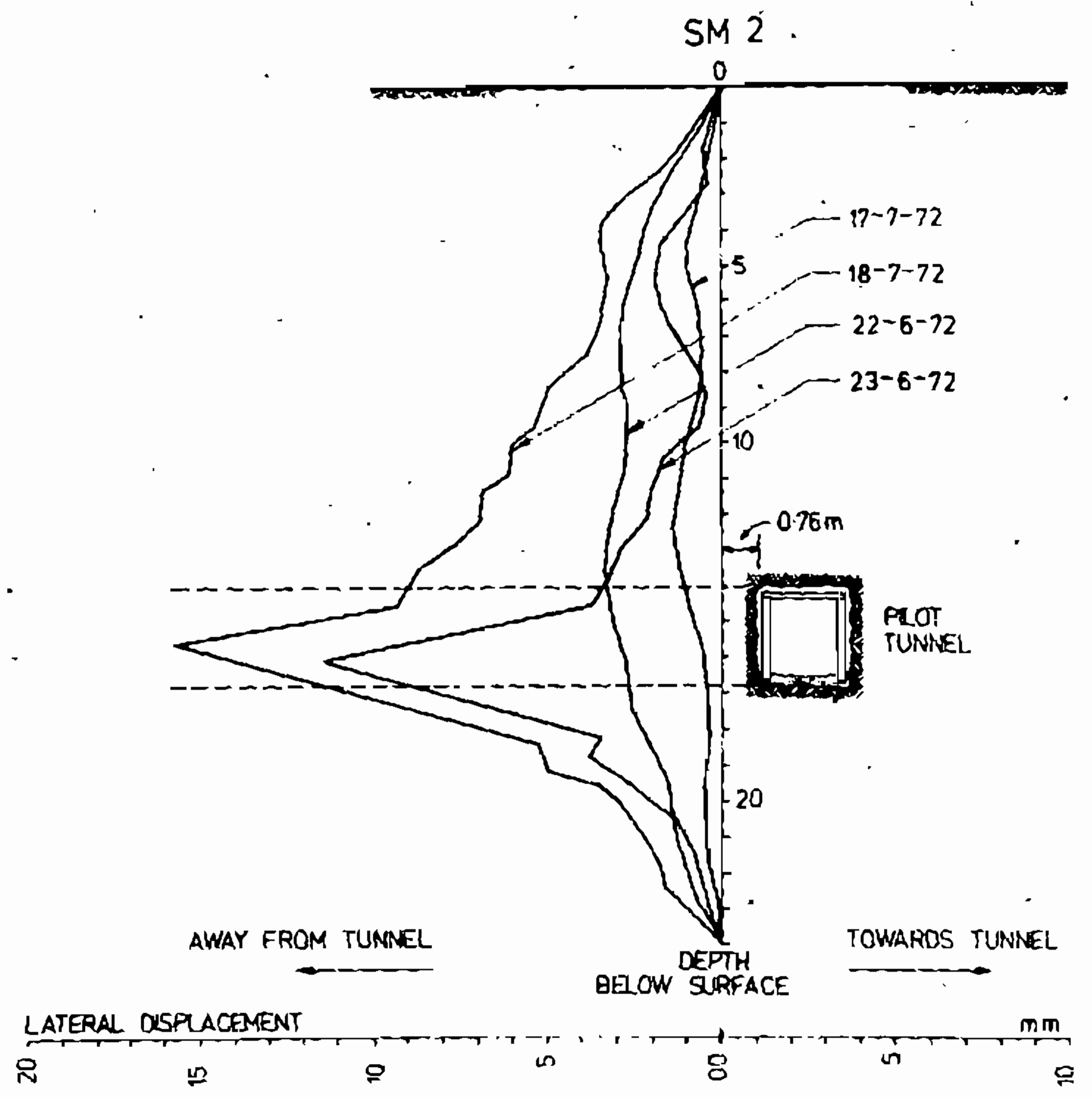

Fig. 4 Typical Slopemeter Results
Adderley Street Pilot Tunnel

Results indicated that the time to reach stability corresponded with a face advance of 3 H beyond the station, and correlated with nearby MPX results. The effect of shotcrete in providing more immediate rock support and less deformation of the tunnel crown is illustrated in Fig. 5, based on work by Askew (Ref. 6).

5 TREASURY GARDENS PILOT TUNNEL

(a) Scope

The Treasury Gardens pilot tunnel of 7 m diameter (D) was excavated in Silurian mudstones and sandstones by heading and bench methods after breaking out from a 2.75 m diameter shaft at a depth of 26 m. Portion of the tunnel was supported by steel ribs blocked off a 50 mm shotcrete skin, and the remainder by 150 mm shotcrete with mesh and rockbolts. The work was carried out for MURLA by the construction forces of the Melbourne and Metropolitan Board of Works. Fig. 6 illustrates the pilot tunnel with Alpine F6-A miner in service.

A major aim was to assess whether shotcrete applied to a full size tunnel as soon as practicable after excavation could sufficiently minimize adjacent ground movement on rock discontinuities. (The controlling effect of joints on the stability of a tunnel in a similar Silurian rock mass has been discussed by Bennet in Ref. 7).

Instrumentation consisted of 3 MPX, 9 SM tubes and surface precise levelling. Some load cell work and mechanical strain gauging was also performed.

(b) Multi-point Extensometer Installation

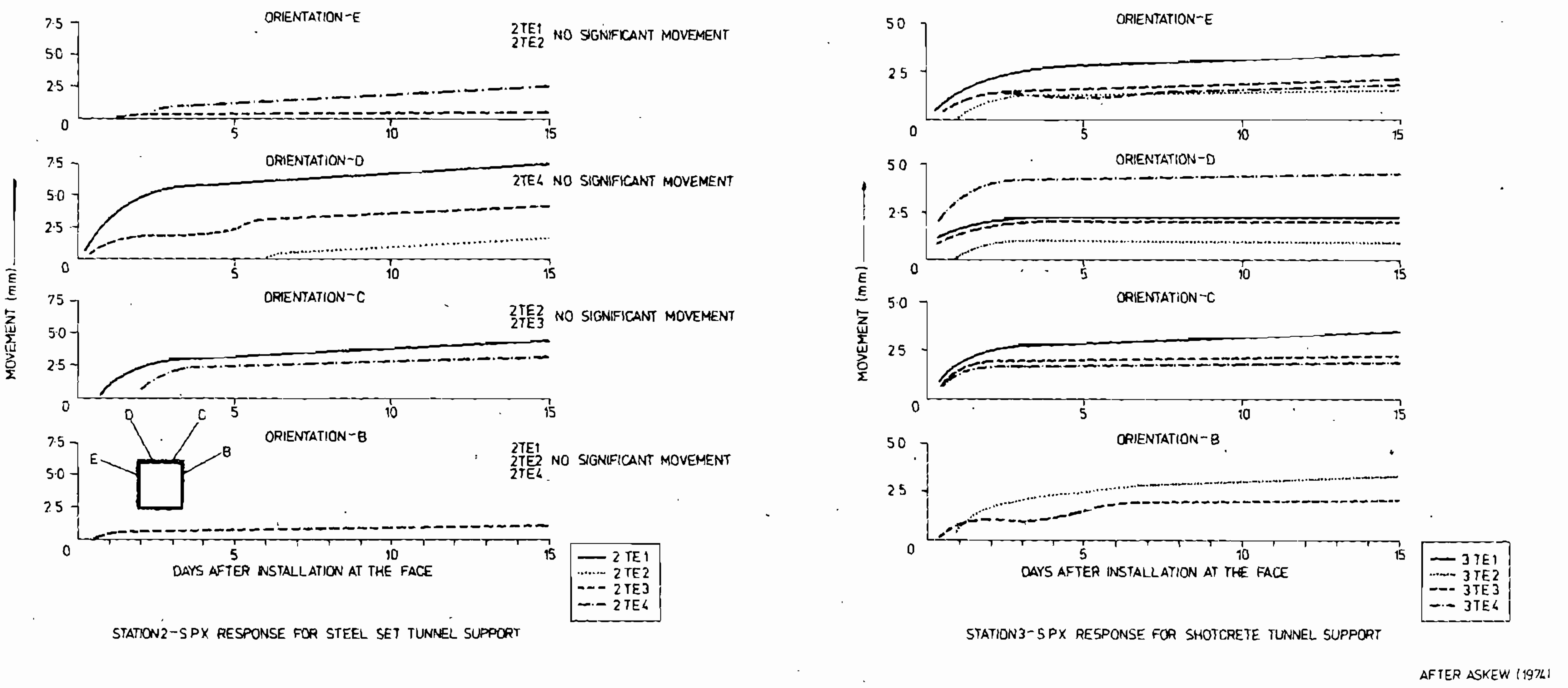

Fig. 5 Typical Spx Results Adderley Street Pilot Tunnel

(e) Other instrumentation

Additional checks were performed to determine loads on tunnel ribs using hydraulic load cells and by mechanical strain gauging. No substantial loads were measured. Detailed geological logging was carried out by the Mines Department as tunnelling proceeded.

The multipoint extensometer installation from which the following results were observed is given in Fig. 7:

(i) Anchors 1.5 D ahead of the advancing face moved perceptibly upward, and then moved down towards or below their original position as the face passed.

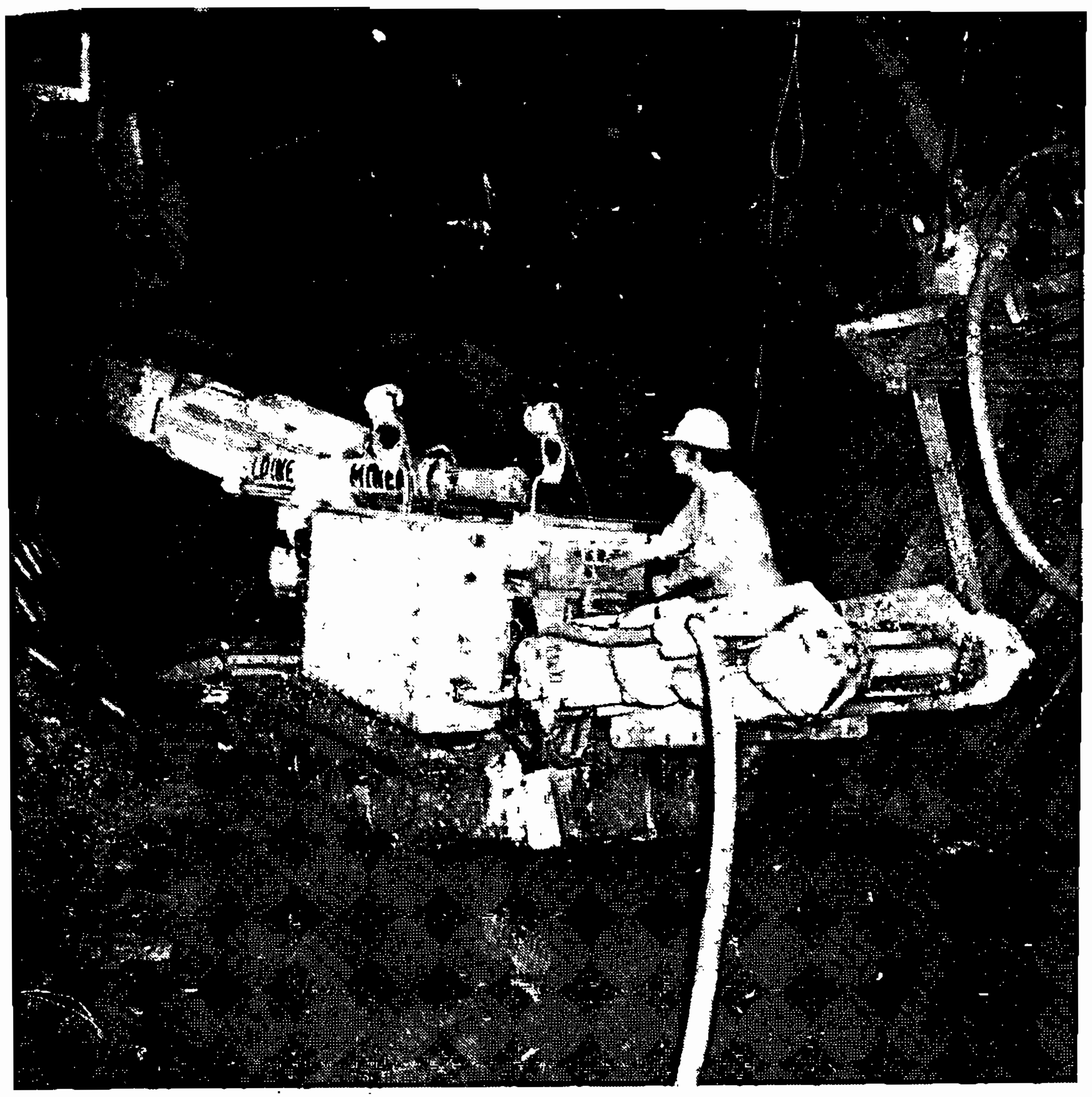

Fig. 6 Treasury Gardens Pilot Tunnel with Road Header Machine

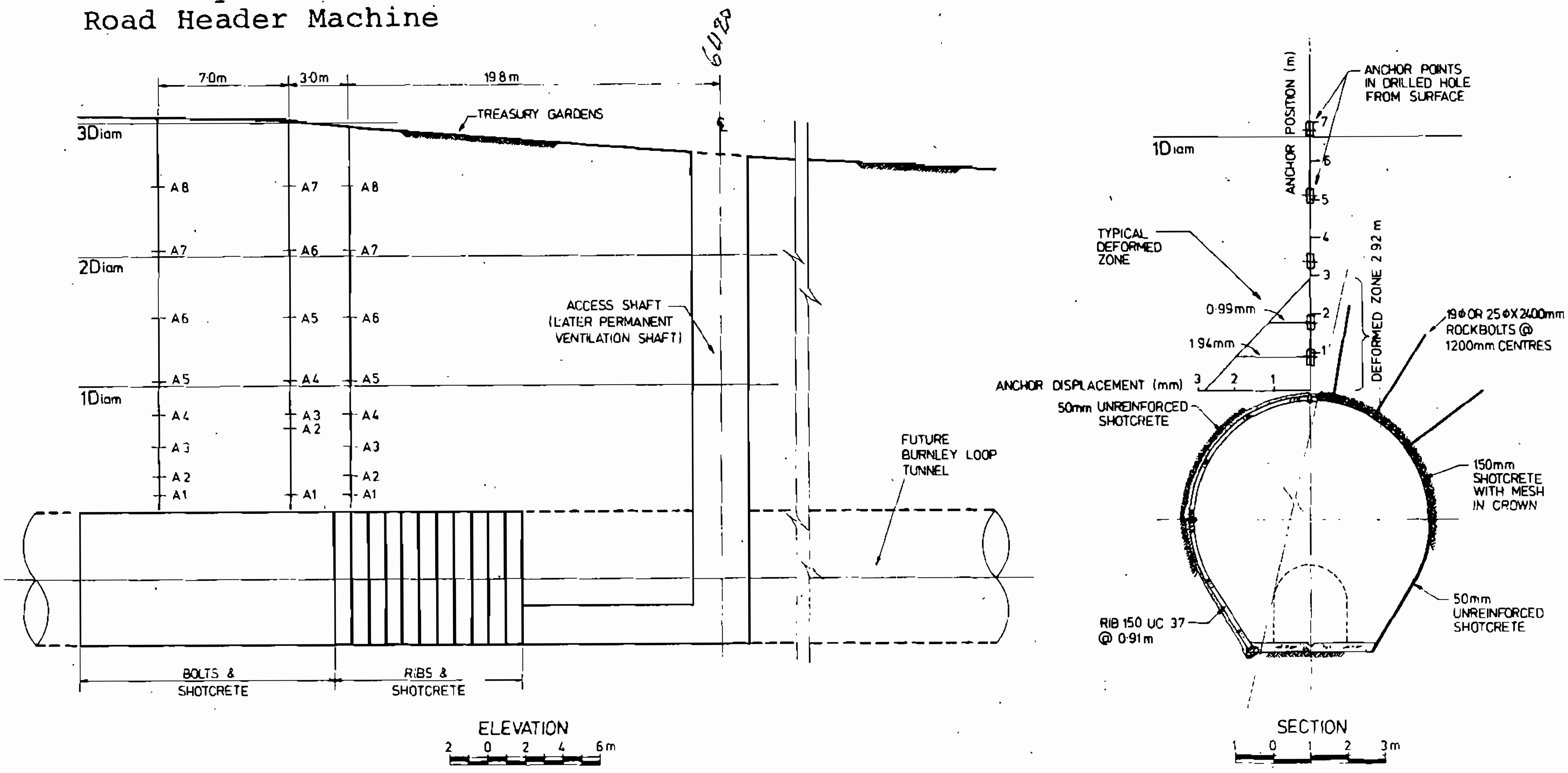

Fig. 7 Multipoint Extensometer Installation Treasury Gardens Pilot Tunnel

care should be exercised in planning programmes involving such an instrument that is more subject to malfunction than the more robust and simple MPX, SPX, load cells and mechanical strain gauges.

(d) Other Instrumentation

Load cells and strain gauging showed loads on the steel tunnel ribs consistent with the height of deformed ground measured above the tunnel.

6 OTHER RESULTS FROM PILOT TUNNELS

In addition to the essentially rock mechanics investigation described above, pilot tunnel construction enabled other significant areas of investigation, and these are summarized below:

(a) Static Modulus of Elasticity

The static rock modulus was determined parallel and normal to the bedding in the Silurian strata using jacking forces up to 100 tonnes and a 610 mm diameter jacking plate. This investigation was carried out by Coffey and Hollingsworth Pty. Ltd. as sub-consultants.

(ii) Anchors showed a tendency to stability when the face had advanced 2 D beyond the installation, and this was confirmed in later work.

(iii) The height of the measured zone of deformation was less than 0.5 D, and the maximum observed deformation at the crown was 3 mm.

(iv) No detectable surface subsidence occurred.

(c) Slopemeter Installations

The malfunction of the instrument restricted the full benefit of this portion of the programme. The results available showed lesser lateral deformations than for the smaller Adderley Street pilot tunnel and to this extent corroborated other results.

For future reference it is noted that

(b) Dynamic Modulus of Elasticity

Dynamic elastic parameters were determined using a Bison seismograph by Maunsell Geotechnical Services as sub-consultants.

(c) Rock Bolt Investigations

A number of types of rock bolts and anchorages were tested. As a result 25 mm diameter bolts with a chemical anchor and a working load of 6 tonnes have been specified for future contracts in comparable rock.

(d) Blasting Vibration Investigations

Data regarding ground vibrations from blasting was obtained and various measuring instruments assessed. As a result, firm specification limits were chosen and appropriate instruments were ordered to be available for the first of the major tunnelling contracts.

(e) Electric Detonator Investigations

The safety of electric detonation in various circumstances in the city area was investigated and specific precautions prescribed. An alternative method of millisecond firing without electric detonators was developed for special situations.

(f) Shotcrete Investigations

Rapid hardening structural shotcrete had not been introduced to Victoria at this stage, and the pilot tunnel programme offered the opportunity to introduce and assess various types of plant and accelerators and to develop a mix using local materials. Specification requirements for strength and time of application were also developed from this work and a new technique was introduced to local industry.

(g) Mining Machine Investigations

Although a full face tunnel boring machine has been successfully applied to the Melbourne Silurian (Ref. 3) other more versatile methods of mechanical mining were necessary for the range of structures involved in the Loop Project. The Authority therefore took the opportunity to make a full scale test of a small "road-header" type mining machine - "Alpine F-6A - thus marking the introduction of this type of machine to civil engineering work in Australia. As a result this type of machine has since been introduced to at least three different projects in Australia, and at the time of writing, three machines are in use by one MURLA contractor.

7 SUBSEQUENT DESIGN AND CONSTRUCTION

Concurrently with the construction of the pilot tunnels described above, design work commenced on the first of the major tunnelling contracts of the Loop. In order to further develop the concepts applied in the Treasury Gardens pilot tunnel before major contracts were committed, the first work let to contract in November, 1972 was a relatively small "preparatory contract" of short duration. This comprised two portals at Jolimont, 460 metres of running tunnel and two Y-junctions. (Fig. 8).

The "shotcrete-rib" concept was the predominant method of support throughout, and geomechanics work was continued to supplement the earlier results.

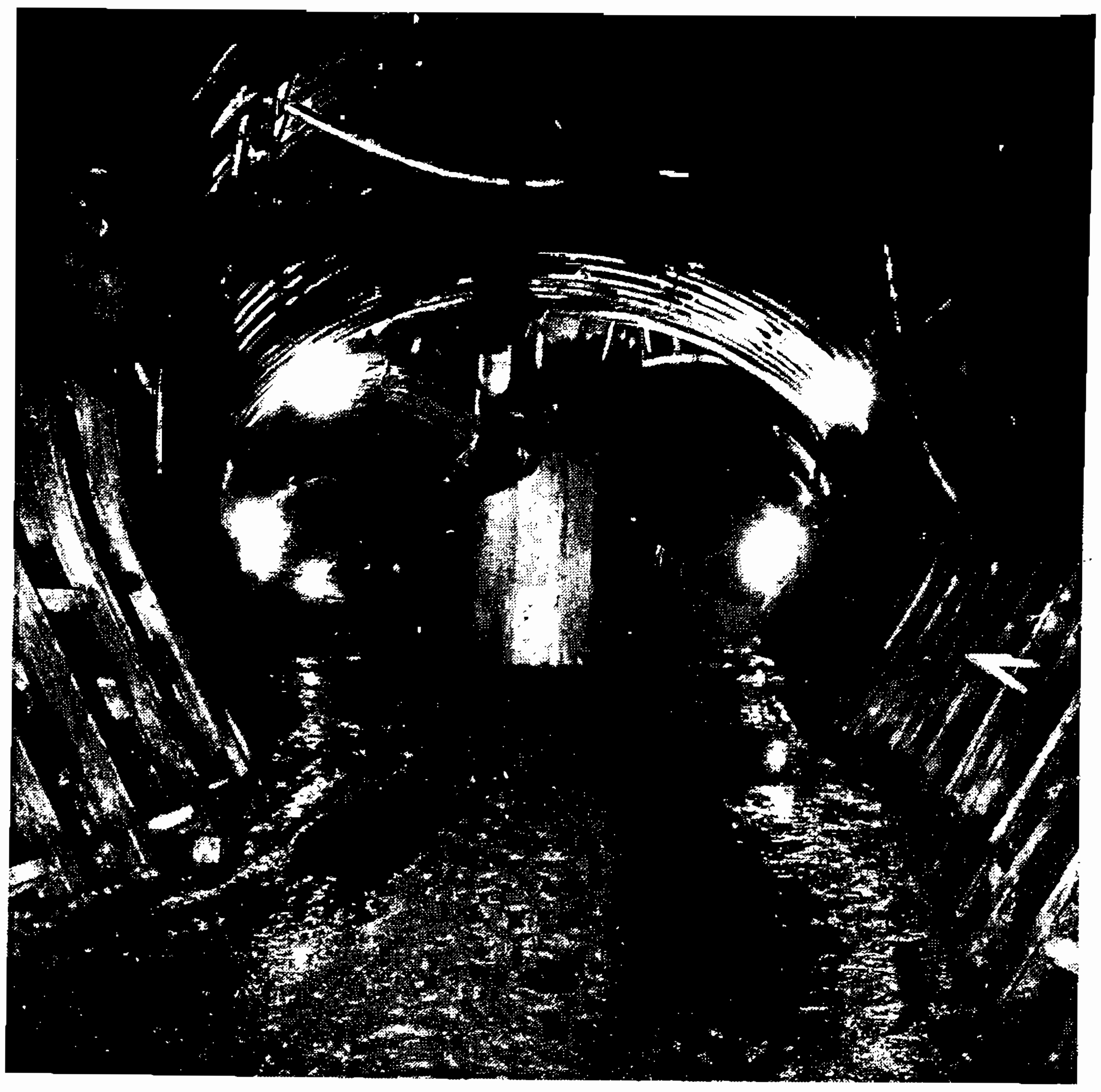

Fig. 8 Contract 311 (Preparatory Contract) Y-Junction

8 CONCLUSIONS

(a) The pilot tunnel investigations described have shown that a thin shotcrete lining in conjunction with normally blocked steel ribs is generally effective in minimising ground movements during the excavation of large diameter tunnels in variable Silurian sediments. Later tunnelling experience has supported this.

(b) As a corollary to this conclusion, it is evident that the thin shotcrete layer promotes an early arching action. This results in a smaller rock load from the deformed zone to be taken by the main ribs than might be predicted from classical theory.

(c) The pilot tunnel studies also yielded both qualitative and quantitative data on rock moduli, shotcrete equipment and accelerators, rock bolt anchorages, blasting vibrations and measuring equipment, the potential problems of electric detonators and other construction procedures proposed for the main tunnelling contracts, including the application of "road header" type tunnelling machines.

(d) The rock exposures not otherwise available to prospective tenderers were also a most valuable result of this investigation work which had a nett cost of approximately $0.5 million for two structures described. This sum represents less than 1% of the cost of the tunnelling works on the Loop.

9 ACKNOWLEDGEMENTS

The authors wish to acknowledge the permission of Mr. F.G. Watson, Director of Engineering and General Manager of the Melbourne Underground Rail Loop Authority to publish this paper, and to the following for assistance and advice:

Mr. W.E. Bamford, University of Melbourne, for consultant advice regarding all aspects of the rock mechanics investigation. Mr. J. Neilson and the geological staff of the Victorian Mines Department for continuing geological assistance. Staff of the Railway Construction Board and the Melbourne and Metropolitan Board of Works in connection with the construction of the two pilot tunnels.

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Selection of a Machine Method for the Excavation of Service Tunnels in the Melbourne Metropolitan Area

by

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SUMMARY. The Melbourne and Metropolitan Board of Works is one of the urban authorities in Australia that has been involved in an extensive machine tunnelling programme. The Board has already used both soft ground and rock tunnelling machines to excavate small diameter tunnels and it is proposed to expand the use of this method in the future.

This paper discusses the considerations and investigations to be made in deciding to use a machine method for the excavation of a small tunnel in either the soft wet sands or clays or the soft silurian mudstones that occur extensively around Melbourne. It also includes details of the machines already used and of the tunnels excavated.

It has been found that provided the ground conditions are fully investigated, the tunnel design suitable and a properly selected machine is used, then the machine method has advantages of economy, greater progress, better tunnel conditions, improved safety and less disturbance to the community and despite some disadvantages, it is likely that there will be an increase in the use of small diameter machines in the future.

1 INTRODUCTION

Melbourne, in common with many other similar cities throughout the world, is facing the problem of expanding and replacing vital services. By 1970 the population of Melbourne had reached 2½ million and the urban area extended over 1942 square miles. The population increase and city sprawl have been factors which have added to the problem of providing urban services. One of the services vital to the densely populated community is the sewerage system, and as the Melbourne and Metropolitan Board of Works the authority responsible for Melbourne's sewerage system, provided only minimal system expansion in the decade up until 1950, it has been necessary to proceed with a scheme of major amplification and extension during the last two decades. If the growth of the city continues at the currently predicted rates with a population of more than 4 million by 2000, then the expansion of the sewerage scheme must continue on a major scale for the next 30 years.

The current extensive works programme and the knowledge of the forward programme have been factors which have induced the Board's engineers to develop and use high speed machine tunnelling methods which are safe, economical and suitable for both soft and hard ground conditions which exist in Melbourne.

The Board has already constructed many miles of small diameter tunnels with more than 4 miles being constructed using a machine. These tunnels are generally 7 to 8 feet in diameter.

Apart from the advantages of speed and economy, the machine method also saves on the extent of labour required and it minimises and centralises community environmental disturbance.

For the method to be successful, the tunnel design must be appropriate and the ground conditions must also be suitable. The pre-construction ground investigation must be adequate for a proper choice of equipment to be made. The equipment necessary for machine tunnelling is expensive and involves a considerable capital outlay.

2 TYPES OF TUNNEL

This paper deals primarily with small diameter sewer tunnels as constructed in Melbourne. Although the Board of Works has used a variety of machines, including rotary, oscillating and free-arm types, it has been found that rotary machines are most suitable in the small diameter tunnels for use in Melbourne conditions and this paper will refer primarily to this type of machine.

3 DESIGN REQUIREMENTS

If a tunnel is to be constructed by machine, then there are both essential and desirable features regarding its design.

3.1 Tunnel Shape

For rotary machines and for shielded machines as used in soft ground, the cross-sectional shape of the tunnel must be circular and the tunnel must be of uniform size over a considerable length. With soft ground machines a change in size involves a major re-build of equipment and consequently is very expensive. In rock conditions, a diameter change, while possible, is still economically undesirable.

With free-arm machines, the tunnel shape can be varied. However, these machines cannot normally be operated in service tunnels of the 7 or 8 feet diameter size.

3.2 Tunnel Length

To warrant setting up a tunnelling machine and the necessary associated equipment, a minimum length of tunnel is necessary. This length is normally established from an economical assessment and varies considerably according to conditions.

The length of tunnel which warrants the use of a machine, varies according to whether the tunnel is in soft ground or rock and is also dependant upon the availability of equipment and the relative cost of excavating the tunnel by alternative methods.

In the case of tunnels constructed by the Board the minimum length of soft ground tunnel that warrants the use of a machine is approximately 3,000 feet and approximately 5,000 to 10,000 feet for a rock tunnel. In both cases, lengths of 15,000 feet or more are desirable. The specific length of the tunnel is usually controlled by many other factors and cannot normally be varied specially to suit the use of a tunnelling machine.

3.3 Tunnel Alignment

It is probably self evident that a straight tunnel on a uniform grade is the most desirable for machine construction. Such a situation however, is so rare in an urban situation that it might be considered unique and it is therefore necessary to establish the extent and form of variation from a straight alignment which can be constructed by machine.

Experience has shown that for service tunnels using a machine with accessory equipment suitable for high speed tunnelling, the excavation can proceed around a curve with virtually no slowing of production, providing the radius of the curve is approximately 800 feet or greater. It is also important to realise that with machine excavation, especially a shield type machine, it is not possible to immediately correct the alignment, but the tunnel must be brought back to the correct position gradually.

This has shown the need to introduce tolerances into the design which not only specify the permitted deviation from position, but also the rate at which the tunnel may vary in alignment and grade.

This approach has meant that the construction engineer can reduce the construction tolerance which he allows for the machine being out of position. In a concrete lined tunnel, this also reduces the thickness of concrete with a consequent substantial cost saving.

3.4 Tunnel Position

The position of the tunnel must be fixed by the designer, and its relation to structures and improvements as well as its depth and specific position in the soil strata is of great importance if a machine method is to be used.

While a machine provides safety protection in a tunnel by eliminating the amount of work that men must carry out at the face, it is also a fact that any ground collapse around the face of a machine is difficult, hazardous and expensive to correct.

In the normal urban situation, as it exists in Melbourne, service tunnels are comparatively shallow with 30 to 100 feet of cover over the crown and it is thus desirable that the tunnel be kept clear of above-ground improvements, particularly in areas of variable, unknown or soft ground conditions.

Apart from the aspect of safety, it is important to realise that the use of a machine is justified by its economy resulting from the high rate of tunnel excavation. If the tunnel is located in a position which leads to delays in passing under valuable structures, or correcting ground faults, then the anticipated saving can readily be lost, even though the problem area of the tunnel may be less than say 5% of its length.

It is also important that a tunnel which is to be excavated by machine has convenient access points which will accommodate the surface working sites necessary to carry out the work.

With a machine, these working sites are required 2-3 mile intervals along the tunnel. They must be adequate in size and suitably situated to permit shift work if the full capacity of the machine operation is to be realised.

4 GEOLOGICAL ASSESSMENT

4.1 Soft Ground Conditions

In Melbourne, the post-war spread of the city has been to the east and south-east and covers substantial areas where the ground consists of saturated sands and clays.

This development has necessitated the provision of services and due to the generally flat topography the sewerage mains have been at a tunnelling depth. In choosing a method for these conditions, it has generally been necessary to use a ground stabilizing technique in conjunction with the use of a machine. The system most commonly used has been to pressurise the tunnel with compressed air, however, due to the escalating labour cost involved in compressed air work, current trends are towards the use of more sophisticated machines with adequate face control in place of or partially in place of the compressed air. In either circumstance, it is necessary to first establish the general nature of the ground, the position of the water table and the suitability for excavation by machine.

It is desirable to establish the permeability of the various strata and the sensitivity of the ground to a loss of the fine cementing material. Sufficient investigation must be carried out to ensure that the conditions are reasonably uniform and that there are no zones of material which cannot be tunnelled by machine. If there are known hazard areas, such as ironstone bands, then it is possible to choose a machine which may have a special ability to cope with such hazards, however, if the hazards are not predicted, then the whole method can readily fail.

Where the tunnel is of a considerable length and requires forced ventilation, it has also been found necessary to investigate the oxygen demand of the ground to ensure that there is no risk of a de-oxygenated atmosphere occurring in the tunnels.

4.2 Rock Conditions

Since the economical success of a rock tunnelling machine is dependant upon the speed of tunnelling which is in turn dependant upon the specific ground conditions encountered, it is essential that proper knowledge of the ground is obtained prior to making a committment to adopt a machine method and certainly prior to selecting a specific machine. This is particularly important with a rock tunnel as there is a considerable variation in the types of machine available and suitable for different rock conditions, and it is not normally possible to convert a machine from one type to another. It is therefore necessary for the geologist to investigate the ground and supply the construction engineer with the details of:

(a) Rock type;

(b) Rock Weathering;

(c) Rock strength;

(d) Rock hardness;

(e) Rock abrasiveness (usually the percentage of quartz present);

(f) jointing including the number, type and spacing of joints as well as details of any cementing material in the joints;

(g) faults and other features such as folding which may effect the rock stability;

(h) the ground water conditions. It is also necessary for the geologist to provide an assessment of the reliability of the formation and the risk of encountering short lengths of quite different materials.

This information can be obtained from surface mapping, drilling and testing, test shafts and siesmic investigation and represents a more extensive investigation than that needed for conventional tunnelling. This geological information can them be used to assist in determining:

(a) ground stability and hence the design of the support portion of the machine;

(b) the shape and configuration of the cutter head;

(c) the type and design of the cutters;

(d) the power, torque and thrust to be incorproated in the machine;

(e) the propel system of the machine.

In Melbourne, the silurian sedimentary mudstones and siltstones contain extensive minor faulting with a high degree of jointing and weathering and whilst the rock strength, which varies from approximately 500 psi up to 15,000 psi, is within the range suitable for machine tunnelling, there are many rock faults involving hard intrusions and soft unstable conditions.

Associated with the programme of tunnelling in Melbourne, the Board has adopted a policy of recording accurate logs of the rock tunnels and where possible, the soft ground tunnels, and also of using an on-site field laboratory to enable strength testing of samples from the tunnel to be carried out rapidly. This site testing assists in the prediction and assessment of cutter wear and performance and together with the log, enables an appraisal to be made of the original geological survey. From the experience gained to date, by the Board, too much emphasis cannot be placed on the importance of carrying out a full geological investigation and of using the information to set up each job individually.

5 ENVIRONMENTAL EFFECT OF MACHINE TUNNELLING

An attractive feature of using a machine method for the construction of tunnels in an urban situation is the desirable environmental effects that are achieved. There is ample evidence that the community is becoming increasingly concerned and aware of environmental disturbances, and the use of machines has enabled some objections to be satisfied.

Of benefit is the reduction or elimination of blasting in densely populated areas. This avoids causing damage to property and avoids disturbing residents in the vicinity of the tunnel and means that the tunnelling process can proceed on a continuous basis. The use of a machine usually means that working sites are widely spaced and hence there is a reduced number of persons affected by the operation. It necessarily follows however, that those residents who are close to a working site do suffer some disturbance and also they suffer for a considerably longer period.

An undesirable feature of tunnelling machines arises from the need to use large quantities of oil and the risk of its spillage in the tunnel with the consequence of possible pollution being carried from the tunnel with the water. Special facilities must be installed to overcome this problem.

6 SELECTION OF A TUNNELLING MACHINE

The actual selection or design of the machine for the specific conditions anticipated is very important. Although there are many machines which are similar in layout, each tunnel has specific requirements and it is normal to adapt, modify or specially design machines for the particular conditions.

6.1 Soft Ground Tunnelling Machines

The features to be considered when selecting a soft ground tunnelling machine (see Figure 1) are as follows:

6.1.1 Ground support.

The shell or shield of the machine must be adequate to support the tunnel under all conditions and must not deflect so as to jamb the cutter head if a rotary machine is used and must enable the tunnel support to be placed safely and conveniently within the tail of the shield. While the normal thrusting load with a machine is low, there are situations requiring large, out of balance thrusting loads with consequent high loads on the shell. In many cases it may also be necessary for the head of the machine to be specially designed to provide the maximum amount of ground support to the face without building up an excessive torque resistance.

6.1.2 Cutter Head and Cutters

The cutter head, including the cutters has to excavate the ground and transfer it back to the conveyor without undue disturbance to the ground and without blocking up and stalling. On machines used by the Board, the cutter head has been fully plated with three or four slots to permit entry of the spoil and with the cutters mounted on the edge of each slot. The types of cutters used have included chisel bits with tungsten carbide tips, large picks with hardened ends and point attack bits for soft ground containing hard bands.

On soft ground machines, the cutters have a substantial life and do not form a major cost item. In terms of controlling the tunnel face and preventing "over break", it is desirable to be able to control the size of the openings through the cutter head. The Board is currently purchasing an 8 foot diameter machine which has a cutter head with controllable apertures and it is anticipated that this control will lessen the need for compressed air. It is, of course, necessary that the cutters be mounted so that they can be changed from the

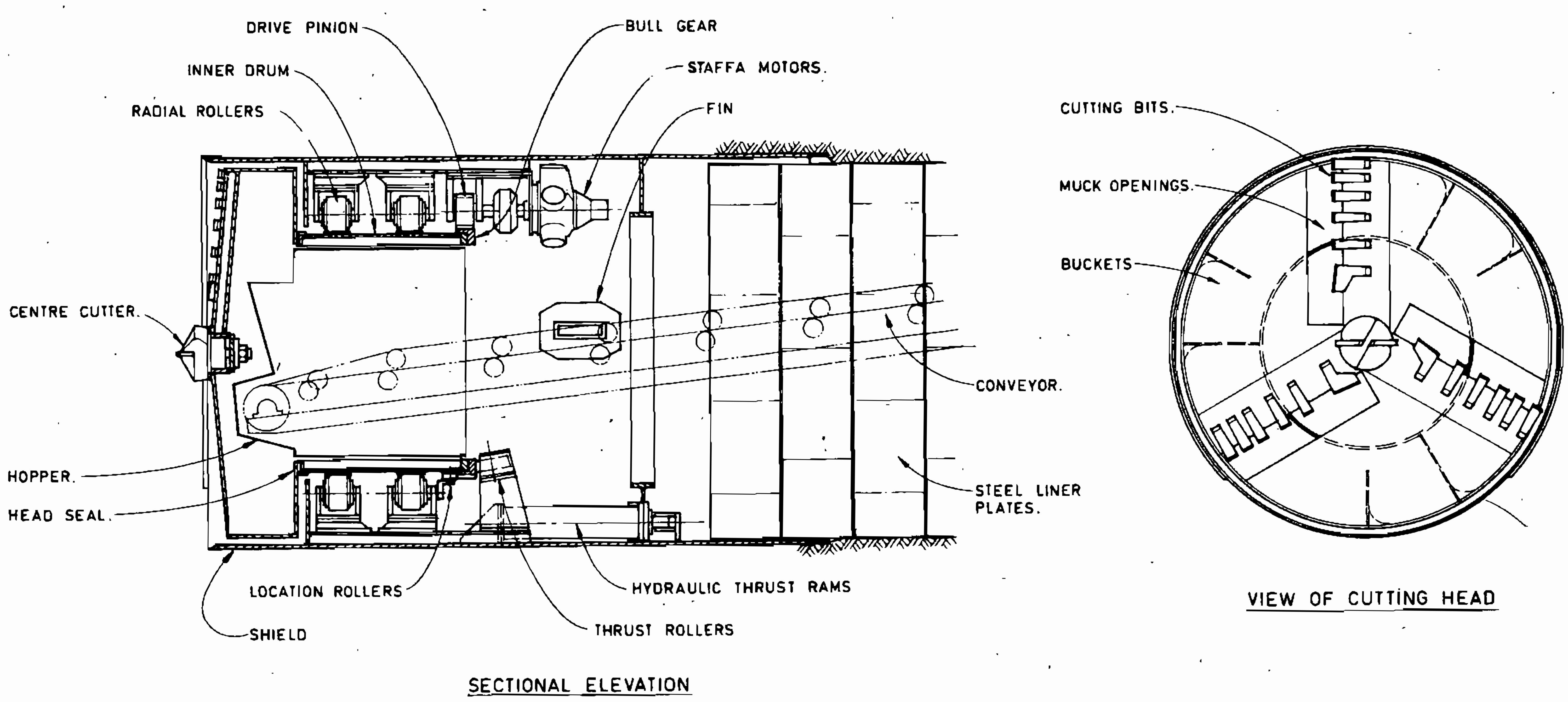

Fig. 1 Soft Ground Tunnelling Machine

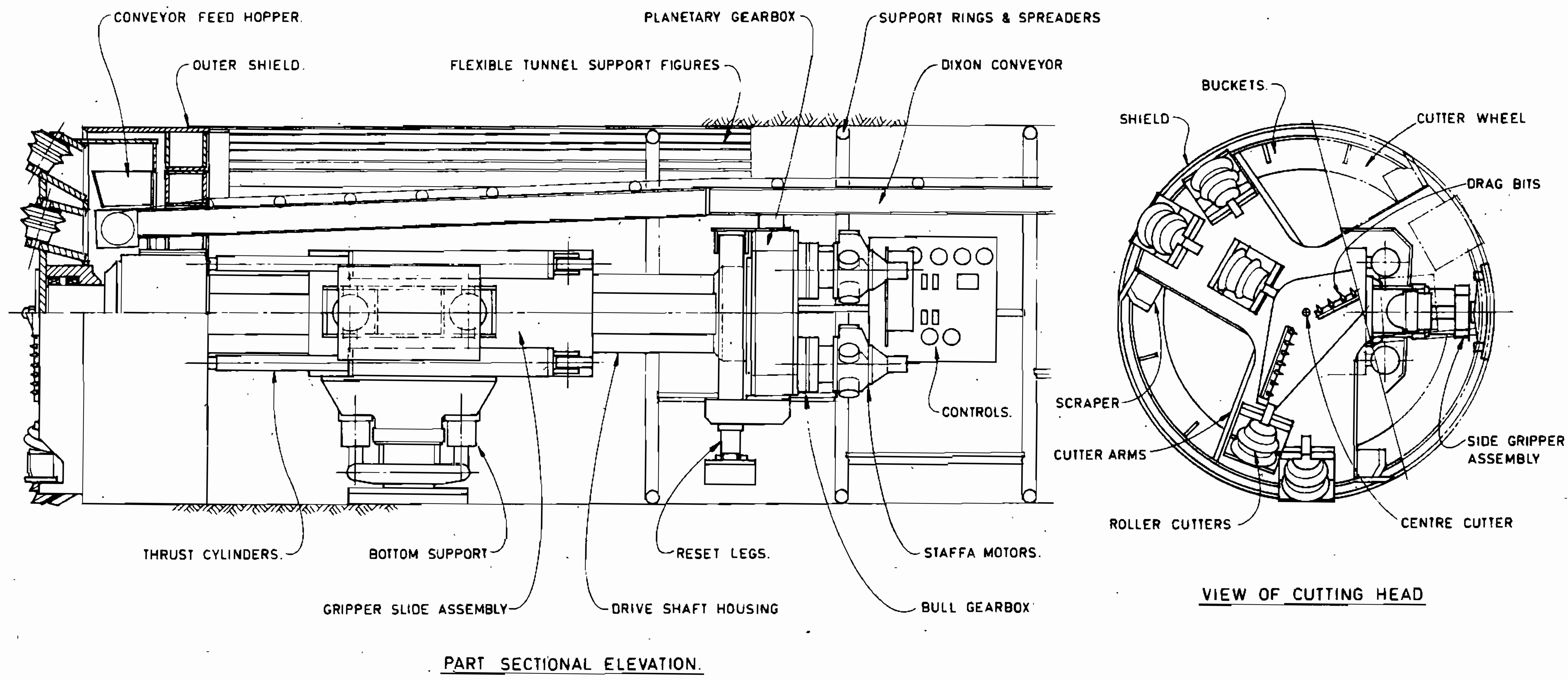

Fig. 2 8ft. Diameter Caldweld Rock Tunnel Boring Machine rear of the cutter head. While not essential, it is desirable that the cutter head be able to rotate and excavate in either direction. This reduces the risk of stalling, provides a means of correcting body roll and assists in the steering of the machine.

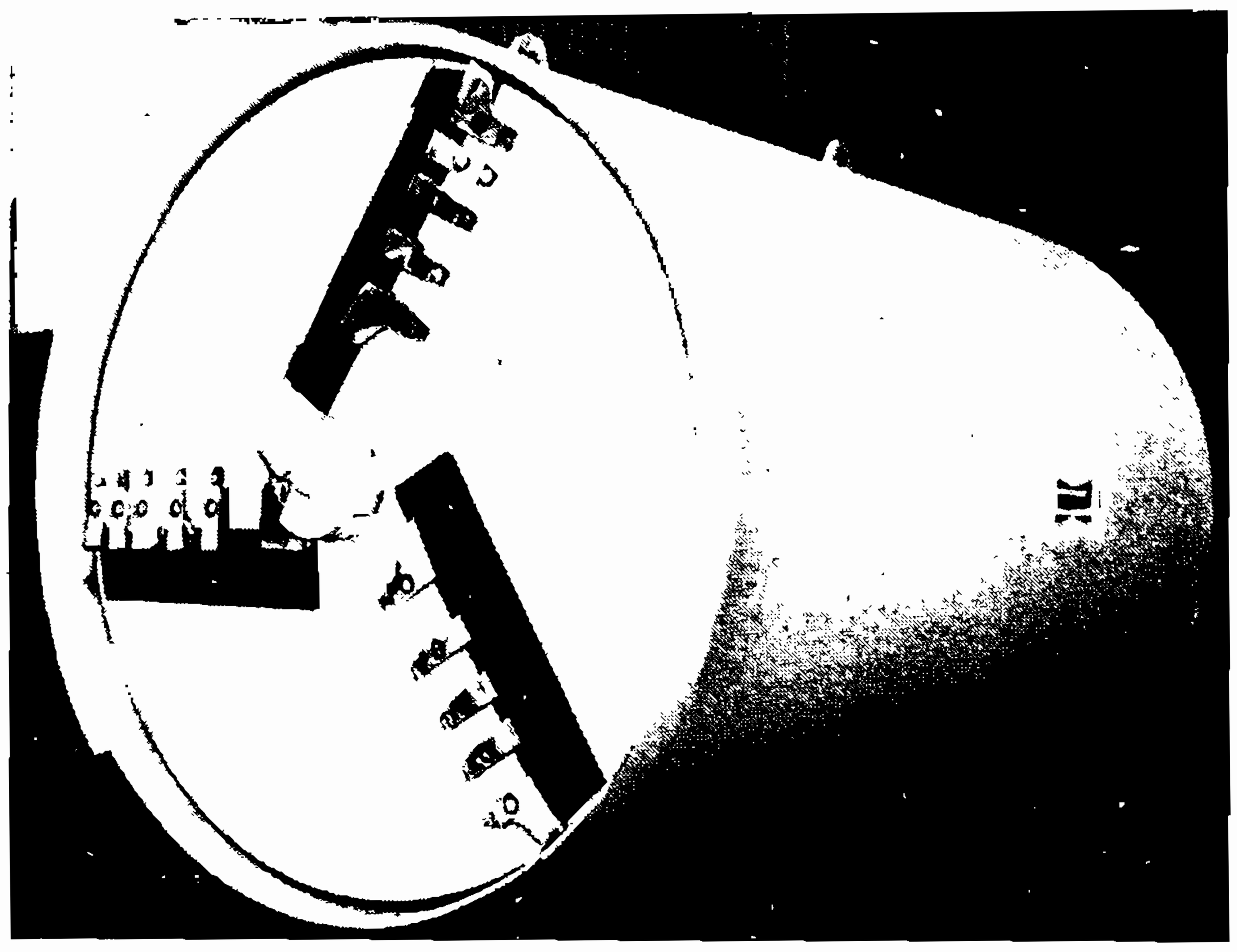

Fig. 3 Soft Ground Tunnelling Machine

6.1.3 Machine Power

One of the greatest hazards to a machine in a soft ground tunnel is the risk of jambing the cutter head due to "heavy" ground. It is therefore essential that the maximum torque which can be applied to the cutter head be sufficient to prevent stalling. With machines that the Board has used or designed, it has been found that a maximum torque of approximately 2,000 x (the face area in square feet) ft. lbs. is sufficient. A small diameter soft ground machine requires a thrusting capacity of approximately 20 to 30 tons per foot of circumference, which is less than the thrusting capacity required by a conventional shield. The thrusting capacity of the machine is normally limited by the restricted space which is available. The whole drive system of the machine must be strong and robust and must be able to accept wear from operating in sandy, abrasive material without causing break-down or failure. This naturally leads to an optimum use of high quality materials and heavy components consistent with the space available.

In a soft ground machine it is most common to see an electro-hydraulic power system. This involves the transmission of electric power along the tunnel with electric motors driving hydraulic pumps located on the "power pack" at a point behind the machine and with hydraulic motors actually driving the machine cutter head. Hydraulic power is also required at the machine for the various jacking systems. While this system has the advantage of providing small power units at the machine and also provides a ready means of speed variation and an overload control, it does mean that the power pack is bulky and in a small tunnel the hoses or pipes needed to transmit the high pressure oil from the power pack to the machine are space consuming. These problems can be reduced by using high pressure and high efficiency hydraulic equipment. A careful selection of both pumps and motors should be made on the basis of size, efficiency, maximum operating pressure and reliability.

6.1.4 Machine Speed.

With an hydraulically powered machine, the speed of rotation of the cutter head can normally be varied. The maximum speed depends upon the cutters, the type of ground, the power available in the tunnel, the maximum torque required and the physical space limitation of the machine. For chisel bits in abrasive ground, the maximum linear speed of the cutters should desirably be less than one hundered feet per minute or approximately 4 rpm for a small diameter machine. In clay conditions, the maximum speed can be doubled or approximately 8 rpm.

6.1.5 Primary Tunnel Lining

In selecting a machine it is important to point out the need for selecting a suitable primary tunnel support. It is not proposed to discuss details of various support systems in this paper, however, the system chosen, apart from being suitable to support the tunnel must be suitable for rapid handling and must also be readily and quickly erected so that the economy derived from the machine excavation is not lost by delays with the tunnel support system.

There are, of course, many other features important in the design of the soft ground machine. These will not be discussed in this text as they do not affect the success or failure of the machine and are not important in selecting a machine method for a particular tunnel.

6.2 Rock Tunnelling Machines

These comments are made particularly in relation to the selection by the Board of an 8 feet diameter Calweld rock tunnelling machine. This machine was selected for use in typical silurian sedimentary rock as exists in Melbourne and the paper does not attempt to discuss the hard rock machines operating in an unsupported tunnels. The Calweld machine adopted includes features which were either developed or found necessary on larger diameter tunnels excavated using a Robbins tunnel boring machine. Details of the Board's 8 foot diameter rock tunnelling machine are shown on Figure 2.

6.2.1 Tunnel Support

The nature of the ground typical to Melbourne, and experience gained on other tunnels has shown the need for providing a system which enables the

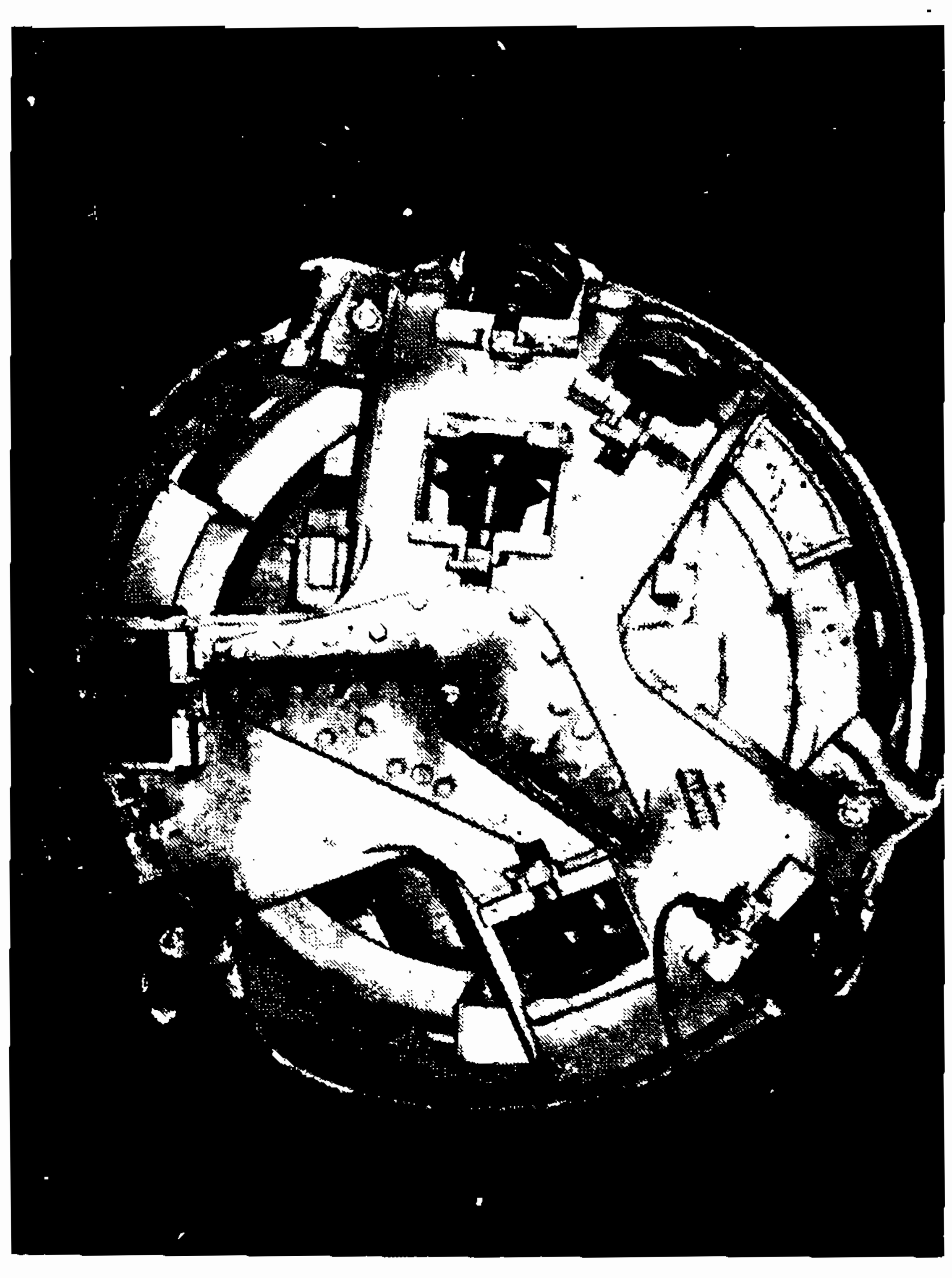

Fig. 4 8ft Diameter Caldwell Rock Tunnel Boring Machine

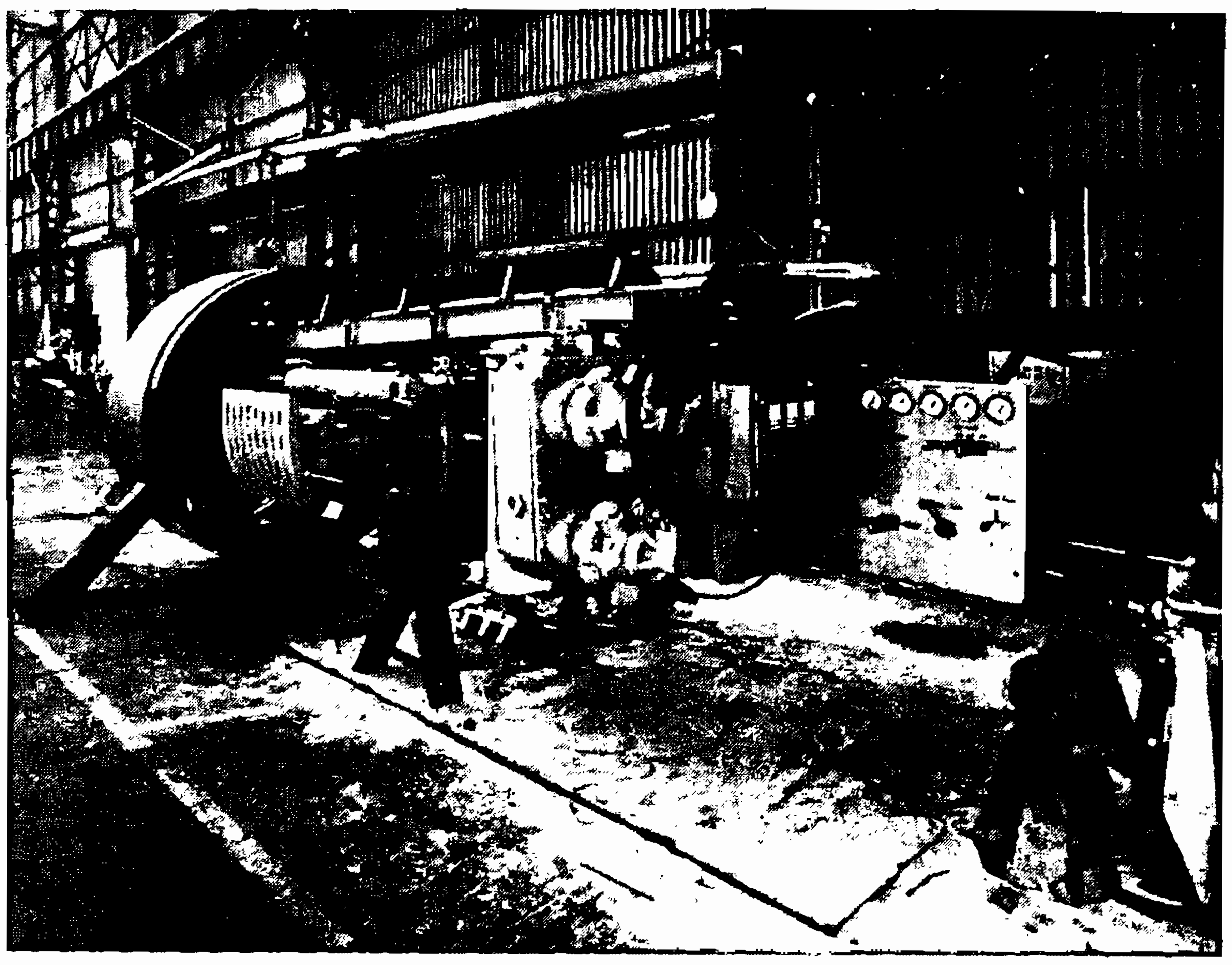

Fig. 5 8ft. Diameter Caldwell Rock Tunnel Boring Machine

support of the crown of the tunnel to be built within the confines of the machine. This means that the primary tunnel support must be placed in position within the machine area. This has lead to the Board using a machine with a full shield and with trailing flexible fingers to support the ground until the primary lining is positioned. In this situation, some compromise is necessary as systems in which the machine provides complete support have disadvantages in the steerability of the machine and in the restriction on space and the limitation on production.

6.2.2 Cutter Head and Cutters

The cutting system is the vital part and usually decides whether the machine is able to excavate the rock. The machines used by the Board for service tunnels and larger have had a cutter head with a flat profile with the gauge cutters mounted on a radius of 6 inches. Every effort has been made to keep the distance from the face of the cutter head to the leading edge of the supporting shield to a minimum. On the Board's 8 foot diameter machine, this distance is approximately 18 inches and it is believed that the larger this dimension, the greater the risk of loss of ground from above the cutter head. The Board has used both drag bits and disc cutters. There is little doubt that in soft and suitable conditions, the drag bits are economical, however, these cutters are unsuited to hard conditions or rock with hard bands, and the tearing action of the drag bits tends to cause overbreak in areas of fractured rock. Disc cutters, although more expensive and more difficult to maintain, provide a more reliable means of excavating the full range of ground conditions and enable consistent progress to be achieved even in varying ground. It is important to choose the cutter layout carefully paying attention to the need to produce even cutter wear and not overload the cutters. It is also important to prevent vibration of the cutter head, particularly transverse vibration as this produces rapid cutter wear and failure. The small diameter machines are of necessity, cramped for space and this means that very heavy and bulky cutters are difficult to handle when being replaced as mechanical handling methods are impossible. The cutter head must be designed to permit through access for maintenance. The Board's machine has not excavated sufficient tunnel to comment on the performance of the selected cutters and cutter layout, however, this information should be available at the date of presentation of this paper. Perhaps one of the major causes of wear and failure of the disc cutters is the failure to clear the rock cuttings from the face with the result that the cutters are forced through the loose cuttings in the invert of the tunnel causing overloading and overheating. It is therefore considered essential that the cutterhead incorporate scrapers which clear the cuttings and assist in discharging the cuttings into the spoil buckets for depositing on the conveyor.

6.2.3 Machine Layout

The layout of the Calweld machine incorporates certain basic features which have been fitted into the limited space. The need for access at and through the cutterhead meant that the drive system could not be built into the cutterhead and hence a machine using a centre drive shaft and kelly bar system was adopted. The four hydraulic motors are mounted at the rear and drive a 14 inch diameter shaft through multi-stage gear boxes and the shaft is connected direct to the cutterhead, with a ring fedder system.

This enables the cutterhead to be driven at speeds of up to 10 rpm with a maximum torque of 180,000 ft/lbs. The machine is thrust forward with a force of up to 160 tons by gripping the tunnel wall with horizontally opposed grippers operated with a force of up to 250 tons.

The machine has been made long enough to permit the primary tunnel lining to be placed immediately behind the grippers. In the small tunnel this configuration provides the maximum space for assembling tunnel supports, however it has also meant that the machine is not well balanced with insufficient weight forward of the grippers, leading to overcut and lack of steerability. It may be necessary to have the grippers mounted towards the rear of the tunnel support being placed in the more restricted space forward of the gripping system In either case, some compromise of the individual optimum features is necessary. Additional hydraulic legs are provided on the machine to assist in steering and for support while re-gripping, and to correct the position of the machine if it rolls.

TABLE I

SMALL DIAMETER TUNNELS EXCAVATED BY MACHINE

	Caulfield Intercepting Sewer, Section 5	Mentone Intercepting Sewer, Sections 1 & 2	Gardiners Creek Main Relieving Sewer Sections 7 to 10
Diameter of Machine	7'-4"	7'-4"	8'-0"
Length of Sewer	5802 feet	14,445 feet (total)	26,500 feet (total)
Machine Type	Soft Ground	Soft Ground	Soft Rock
Machine Make	Mitsubishi	M.M.B.W.	Calweld
Length of Machine	13'-8"	11'-10"	17'-0"
Weight of Machine	20 tons	20 tons	30 tons
Max. Head Torque of Machine	39,000 ft.lb.	110,000 ft.lb.	180,000 ft.lb.
Max. Head Revs. of Machine	5.2 r.p.m.	5.5 r.p.m.	10 r.p.m.
Max. Pushing Force of Machine	320 tons	400 tons	160 tons
Max. pushing Rate of Machine	4 inches/min.	4 inches/min.	6 inches/min.
Max. Gripping Force of Machine	-	-	250 tons
Max. Power of Machine	60 H.P.	125 H.P.	375 H.P.
Head Cutters	Tungsten Carbide Tipped Cutters	Tungsten Carbide Tipped Cutters	Roller Disc and Drag Cutters
Excavation with Machine	3588 feet	8650 feet to June 1974	2600 feet to June 1974
Average Rate of Excavation	138 feet/week	211 feet/week to date	200 feet/week to date
Max. rate of Excavation	251 feet/week	303 feet/week	330 feet/week
Type of Support	16" wide steel liner plates	16" wide steel liner plates	3" pipe sets at 4'-0" spacings with wire mesh lagging
Geology	Sandy clays and silts with bands of limestone (max. 2 ft.)	Sands and clayey to silty sands with bands of ironstone (max. 6")	Slight to medium weathered silt stone (some highly weathered zones)

In selecting or adopting a machine layout and power system for a small tunnel, the key considerations are:

(a) provision of sufficient access areas to all necessary locations, in particular, to the areas for placing support and to the cutterhead.

(b) use of an adequate shield or support system to prevent loss of ground either at the head or to the rear of the cutter head.

(c) use of robust reliable and efficient components to avoid breakdown and reduce maintenance.

(d) provision of adequate power.

(e) selection of suitable cutters and proper scrapers.

(f) the use of an adequate conveyor system to dispose of the spoil.

The rock tunnelling machine, as with the soft ground machine, requires a layout of back-up equipment to facilitate the prompt disposal of spoil, the provision of support material and the ventilation of the tunnel. This equipment is of course, important in the tunnelling operation, however it is not a major consideration in adopting a machine method or in choosing a specific machine.

7 SAFETY

The use of a tunnelling machine to excavate a small tunnel involves a number of important safety considerations. Improved safety to the tunnellers and to the tunnel is obtained from:

(a) the lack of need to store or use explosives in the tunnel.

(b) the tunnel lighting can be substantially better in a tunnel without explosives.

(c) the improved support which is readily placed in the machine excavated tunnel.

(d) there is no unsupported area of tunnel where the men are working.

There are also some additional safety hazards in a machine tunnel. These include:

(a) the extensive amount of machinery with belts and moving parts and, in particular, the head of the machine. A system must be used to ensure that the machine is not operated while men are actually in the head;

(b) the oil and electrical equipment produces a potential fire hazard and stringent tunnel operating procedures are required to control this risk. The oil also produces a risk of men slipping and falling on smooth surfaces.

8 MACHINE PERFORMANCE

Tables I and II set out details of tunnels excavated by the Board's machine.

9 CONCLUSIONS

The science of machine tunnelling is still in its infancy in Australia and the technique of using small diameter machines is even less developed than for larger machines. It is therefore premature to make decisive conclusions regarding machine use for service tunnels, however it is possible to enumerate the current trends in this area.

From the tunnelling operations carried out by the Board of Works, it has been established that small machines can function satisfactorily and economically. It is essential that the geological conditons be carefully investigated in detail, that the design be prepared having regard to the machine requirements and the machine be carefully selected for the particular tunnel.

In comparison with the conventional methods, the following advantages may then be achieved when using a machine method:

(a) reduced costs;

(b) reduced labour content. With the current inflation of labour rates the machine method should become even more economically attractive in the future;

(c) a greater rate of tunnelling progress. The rate of tunnelling by machine is approxi-

TABLE II

DETAILS OF SMALL TUNNELS PROGRAMMED FOR CONSTRUCTION BY MACHINE

	Length feet	Diameter feet	Ground type
East Oakleigh Branch Relieving Sewer	3000	7 or 8	Soft - wet sand and clay
Hobsons Bay Main Relieving Sewer	Up to 22,000	7	Soft - wet sands and clay
Eumemmerring Creek Main Sewer and Hallam Valley Main Sewer	22,000	8	Soft - wet clays and sandy clays
Dandenong Valley Trunk Sewer Section 5	15,000	8	Rock - Mudstone
Eltham Main Sewer	Up to 13,000	8	Mudstone and sandstone

mately double that achieved by conventional methods;

(d) improved working conditions in the tunnel, especially in soft ground tunnels;

(e) improved safety with protection from tunnel collapse or movement;

(f) reduced disturbance to the residents along the line of tunnel;

(g) reduction of property damage near the rock tunnels by elimination of blasting.

While this is an impressive list of advantages, the use of machines has some undesirable features as well. These include:

(a) greater investigation has to be carried out;

(b) a substantial financial outlay is necessary at the beginning of the job. It is thus important that a correct assessment is made;

(c) some pollution is possible from the oil associated with the machinery in the tunnel;

(d) the machinery in the small tunnel poses a potential safety hazard.

At this stage, it is fair to say that the use of small diameter machines is at an interesting and promising stage with great potential and it is hoped that, for the benefit of the urban community, this potential is exploited in the future.

Down-Hole Investigation Techniques for Underground Construction

by

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1 INTRODUCTION

A significant portion of the effort and expense of investigation programmes for tunnels is devoted to the obtaining of core samples for study by those involved in the design and construction of these structures. This paper evaluates three down-hole techniques which can be incorporated into a boring programme for relatively little additional cost, and enable quantitative data to be obtained on soil and rock properties.

The three techniques to be discussed are borehole pressuremeter testing, crosshole seismic testing, and electric well-logging. The borehole pressuremeter is a means of performing in-situ strength and deformation tests on borehole walls from which can be obtained the shear strength and Young's Modulus of the material being tested. In weathered rock materials where poor core recovery has not provided suitable specimens for laboratory testing, pressuremeter testing may be the only available method of obtaining design parameters. Crosshole seismic testing permits evaluations to be made of the Young's Modulus of very large samples of rock masses. Electric well-logging offers the possibility of supplementing the data obtained from a drilling programme and enables assessments to be made of whether down-hole electrical geophysical techniques are capable of filling in gaps between cored holes.

Each technique produces data with its own significance which would be relevant to certain problems in the design of tunnels whether constructed by machine boring, conventional tunnelling or by cut and cover methods.

2 BOREHOLE PRESSUREMETER TESTING

The use of borehole pressuremeter devices has increased markedly over the past decade in an effort to obtain reliable strength and deformation characteristics of both soils and rocks. The technique involves the insertion down a borehole of a cylindrical probe containing on it a reinforced membrane sheath which can be expanded against the sides of the borehole by pumping water into the gap between the sheath and the body of the cylinder. A graph is obtained which relates the volumetric expansion of the sheath (i.e. the radial expansion of the borehole) to the applied internal pressure. From this 'pressuremeter curve' it is possible to calculate both the shear strength and Young's modulus of the material being tested. Correlation between the parameters obtained and the core recovered at the level of the test enables an extension of results to other boreholes where core recovery has been made.

The use of pressuremeter testing is particularly relevant in Australia due to the extensive weathered rock profiles which exist in both urban and rural areas where tunnelling may be contemplated. Over the past twelve months, the authors have used the Menard Pressuremeter to obtain design parameters at a number of tunnel and excavation sites. In Melbourne and Canberra, test data have been obtained from weathered basalt, sandstone and mudstone at locations where core recovery has been difficult, and no other suitable test method has been available. One particular model of the Menard Pressuremeter has been used, involving the expansion of a 75mm probe approximately 450mm in length.

The information obtained from each particular test can be illustrated by reference to the pressuremeter curve presented in Figure 1, which was obtained from a test at 14m depth in weathered mudstone at an excavation site in Canberra. The internal pressure was increased in increments of 550 kPa, with readings for each increment being taken at 30 seconds and 1 minute after increasing the pressure. Figure 1 shows two curves - the pressuremeter curve which plots the 1 minute volume readings against the applied pressure, and the creep curve, which plots the difference between 30 and 60 second volume readings against pressure.

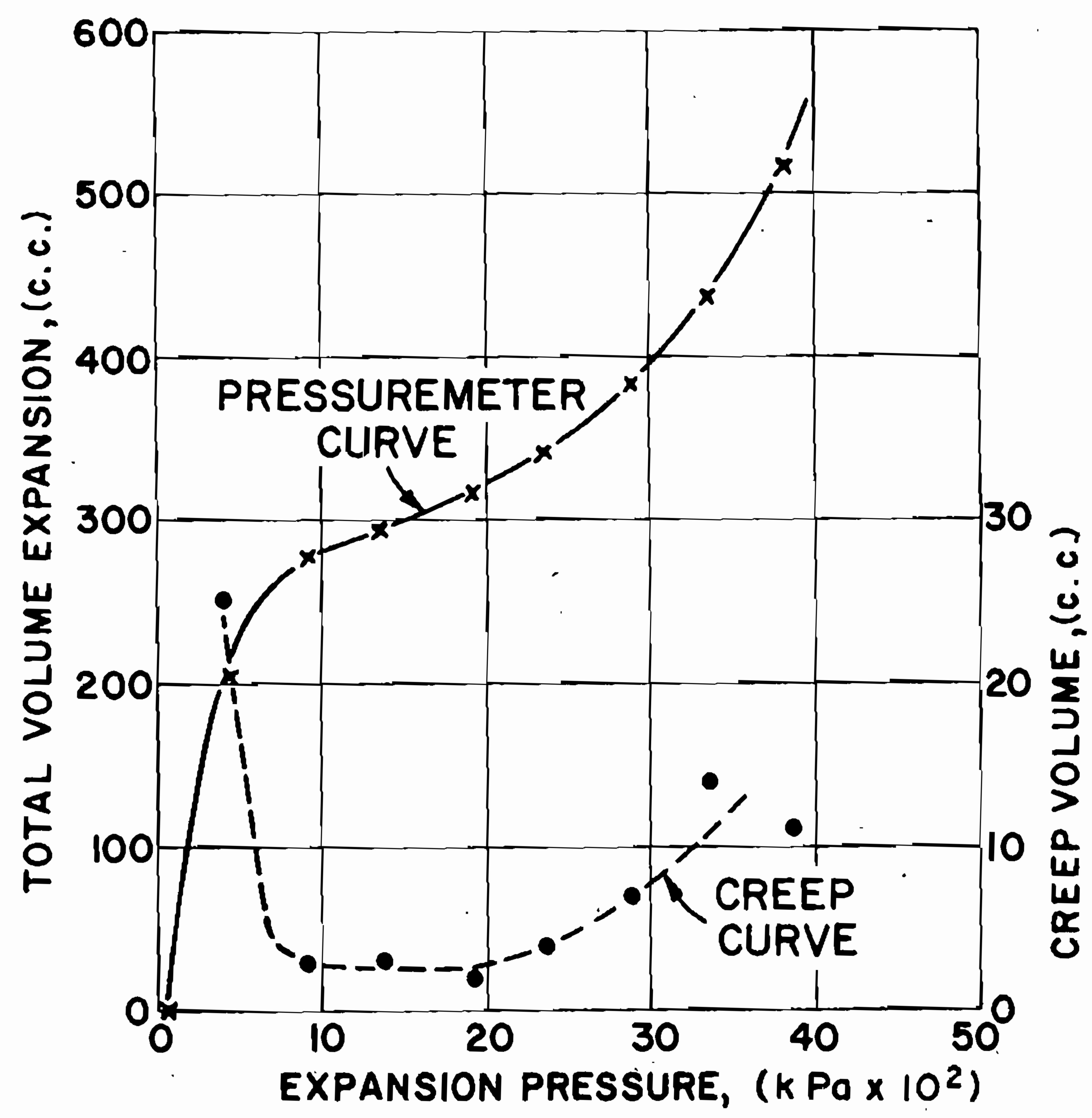

Fig. 1 Typical pressuremeter test data

The illustrated pressuremeter curve defines three stages in the test:

(i) the initial stage (0-650 kPa), during which slack in the borehole is taken up and the in-situ horizontal stress is equilibrated

(ii) the elastic range with low creep readings, over which an elastic modulus can be obtained (650-2300 kPa)

(iii) the plastic range with significant creep, which is a prelude to failure at an estimated 4600 kPa.

From the above results, it is possible to calculate the following important parameters:

in-situ horizontal stress	650 kPa
Young's Modulus	65 MPa
Yield stress	720 kPa
Unconfined compressive strength	1430 kPa

At the same site, a total of 19 tests were performed in one borehole over depths from 6.5m to a maximum of 35m. Core recovered from the borehole indicated that conditions varied from completely weathered to moderately weathered mudstone over short intervals. A detailed plot of the elastic moduli obtained as a function of depth is illustrated in Figure 2. This figure gives a valuable indication of how the elastic behaviour in weathered rock profiles varies with depth, and is of particular relevance in the design of support in both cut and cover and bored tunnel excavations.

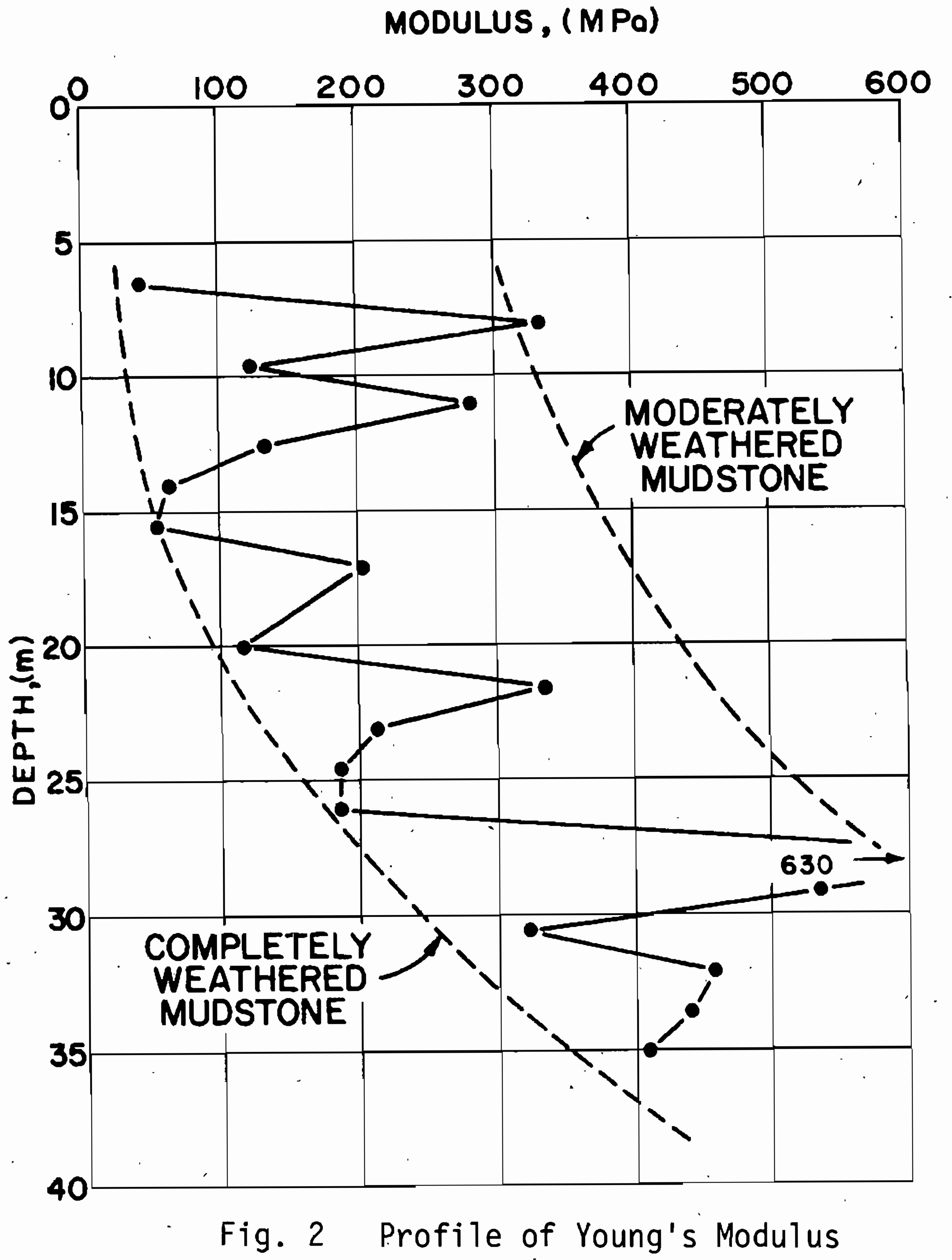

Fig. 2 Profile of Young's Modulus

A general trend of increasing modulus with depth is noted, with two boundary lines being drawn to represent the variation in modulus for the extremes of completely and moderately weathered mudstone. The rapid variation in modulus over short distances is quite consistent with observations of core quality, and in fact gives a quantitative measure to the variations in strength of the core. Of the tests represented on Figure 2, six were taken to failure while the remaining thirteen were beyond the capacity of the instrument to achieve failure. The unconfined compressive strengths obtained ranged between 1100 and 2650 kPa, while the ratio of modulus to shear strength varied between 75 and 180 with an average value of 110.

While the pressuremeter used by the authors is ideally suited to the range of stresses relevant to weathered rock profiles, the equipment has also been used to determine the elastic modulus for slightly weathered mudstone of high strength at a tunnel site east of Melbourne. Rock moduli ranging between 2.9 and 41 GPa were obtained, although the accuracy of measurement is limited for rocks with such high moduli.

In the detailed design of tunnel walls for cut and cover, or tunnel linings for bored tunnels, modern computer techniques allow sophisticated analyses to be performed to assess stresses and deflections in the support systems. An integral part of such analyses is the determination of elastic properties of the soil or rock mass surrounding the tunnel. The pressuremeter provides these basic paramteres in a form readily usable in theoretical analyses.

An illustration of the type of analysis possible can be given by reference to the problem studied by the authors of a buried footing founded on variably weathered rock with relatively high applied stresses. A large number of pressuremeter tests were performed to assess the variation of elastic modulus both in plan and with depth. Using the parameters obtained, a finite element analysis was performed to determine the differential settlements across the footing developed due to the variable ground conditions. It was further possible to insert the elastic limit obtained from the pressuremeter curves to assess what effect plastic strains in the rock mass might have on both settlements and redistribution of stress across the footing.

The form of analysis outlined above can be readily adapted to the solution of specific tunnel design problems. The authors have in fact used the technique to assess the lateral deflections of U-framed box structures under lateral loads applied during service of the tunnel. The value of such analyses are restricted by the accuracy of the input data, and for this reason the parameters provided by in-situ pressuremeter testing can be of considerable significance in achieving economical designs.

3 CROSSHOLE SEISMIC MODULUS DETERMINATIONS

A comparatively simple and inexpensive method of evaluating Young's Modulus of in-situ rock masses is to measure the velocity of compressional waves produced by an explosive charge placed down a borehole. The velocity is obtained by measuring the wave travel time from the energy source in one borehole to the receiver in a second borehole. Since the measurements can be made at or near the level of a proposed tunnel on a very large sample that includes rock defects such as joints and bedding planes, seismic methods can provide in-situ data to supplement laboratory testing of core and block specimens.

Laboratory measured values of Poisson's Ratio μ and the unit weight of the rock material γ are combined with the compressional wave velocity Vp to evaluate Young's Modulus E from the following relation-ship, where g is the acceleration of gravity:-

$$E = \frac{Vp^2 \gamma}{g} \frac{(1+\mu)(1-2\mu)}{(1-\mu)} \qquad (1)$$

The use of equation 1 in evaluating Young's Modulus has been criticized on the grounds that an assumed value of Poisson's Ratio may differ from the actual in-situ value by an amount large enough to cause the Young's Modulus to be over-estimated by up to 100 per cent. However in the authors' experience the differences between laboratory measured values and in-situ values of Poisson's Ratio have only a modest effect on the value of Young's Modulus for the range of values of Poisson's Ratio frequently encountered in rocks. For example, if the laboratory mean value is $\mu = 0.25$ for core specimens of a particular rock type, and the true in-situ value for the rock mass is $\mu = 0.30$, then the calculated modulus value from equation 1 would be 11 per cent too high. Such an error is not regarded as excessive in the estimation of elastic modulus.

If the proposed tunnel is at shallow depths it may be possible to run surface traverses in which both compressional waves and shear waves are generated and their velocities measured. Where both shear and compressional wave velocities are measured, the in-situ Poisson's Ratio can be calculated and this can be used in equation 1 in lieu of the laboratory determined value. However, in down-hole seismic measurements where a simple energy source such as a small explosive charge is used, it is difficult to measure the shear wave velocity. Since shear waves travel more slowly than compressional waves, which are the main type of seismic waves generated by small explosive charges, the first shear wave arrivals are not always distinguishable from second and later compressional wave arrivals.

The authors have carried out crosshole seismic modulus determinations at the sites of two proposed pressure tunnels in tightly folded jointed strata east of Melbourne. At the first site in sandstone of high strength the crosshole seismic technique gave a mean modulus of 20.8 GPa which compares well with laboratory modulus values of 22.4 GPa for the tangent modulus in the static tests.

At the second site in steeply dipping mudstone of high strength, the designers of a pressure tunnel wished to assess the degree of modulus anisotropy with respect to the bedding. In order to make measurements in the fresh to slightly weathered strata a number of crosshole seismic measurements were made. The down-hole receiver used was a hydrophone since most measurements were being made below the watertable. Where the hydrophone was located above the water-table the borehole was filled with water from a water tanker to ensure continuity of the receiving medium.

A profile of the test site is shown as Figure 3. Because of the steep bedding dip the crosshole seismic shots could only be made at angles ranging from 90 to 40 degrees with respect to the bedding. For measurements made at smaller angles to the bedding the seismic source was located down a borehole and a surface geophone was used as the receiver. Surface seismic traverses were run over the areas where the surface geophone was used so that the thickness and velocity of the overburden layers could be ascertained to permit calculation of the compressional wave velocity in the underlying fresh mudstone.

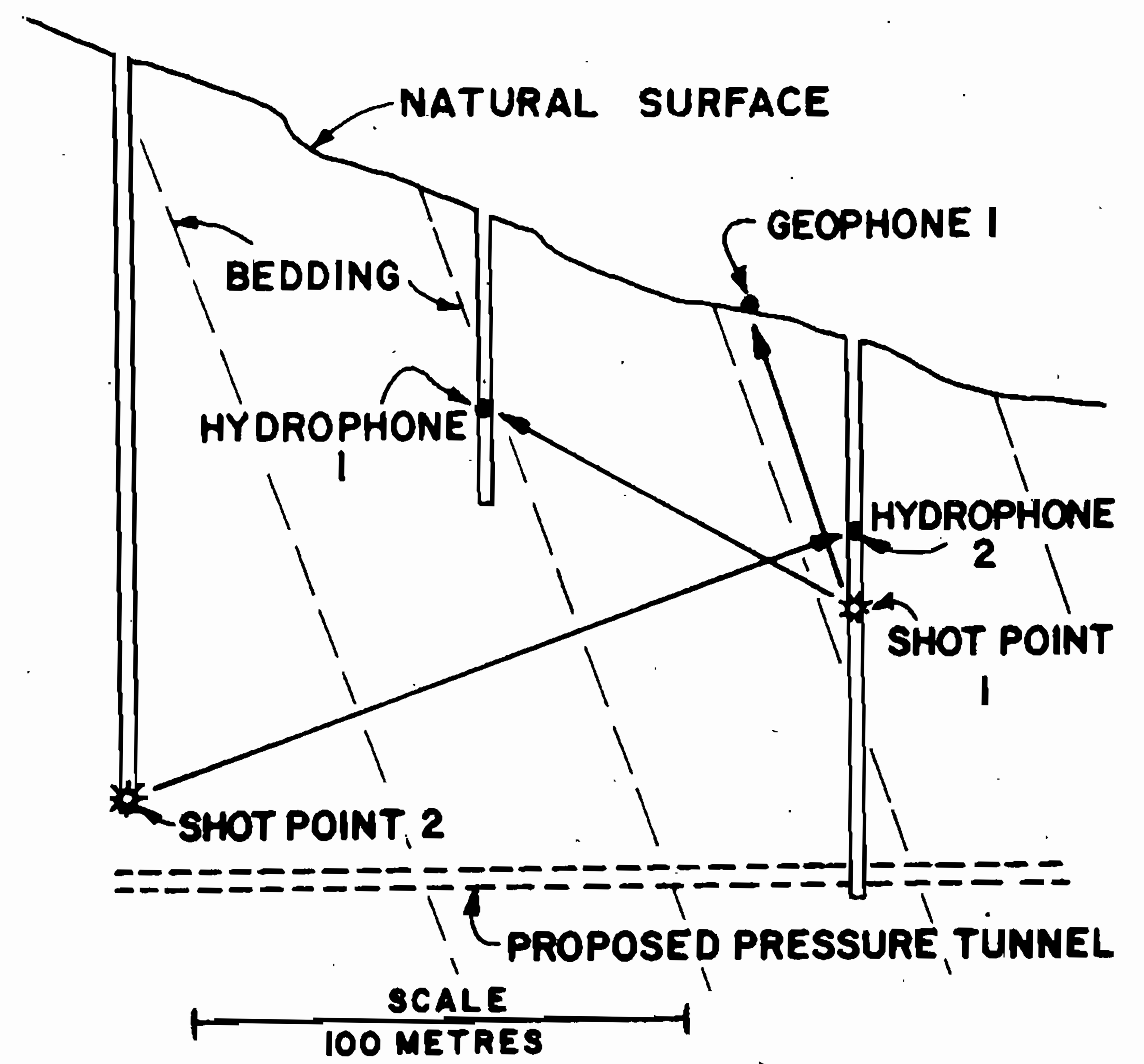

Fig. 3 Typical cross-hole seismic shots

A total of thirteen seismic measurements were made at various angles to the bedding. The Young's Modulus was calculated from equation 1 using the laboratory determined values of $\mu = 0.25$ for fresh mudstone, $\mu = 0.29$ for fresh to slightly weathered mudstone, and $\mu = 0.33$ for slightly weathered mudstone. A plot of the calculated modulus values versus the angle to the bedding at which the seismic measurements were made is presented in Figure 4.

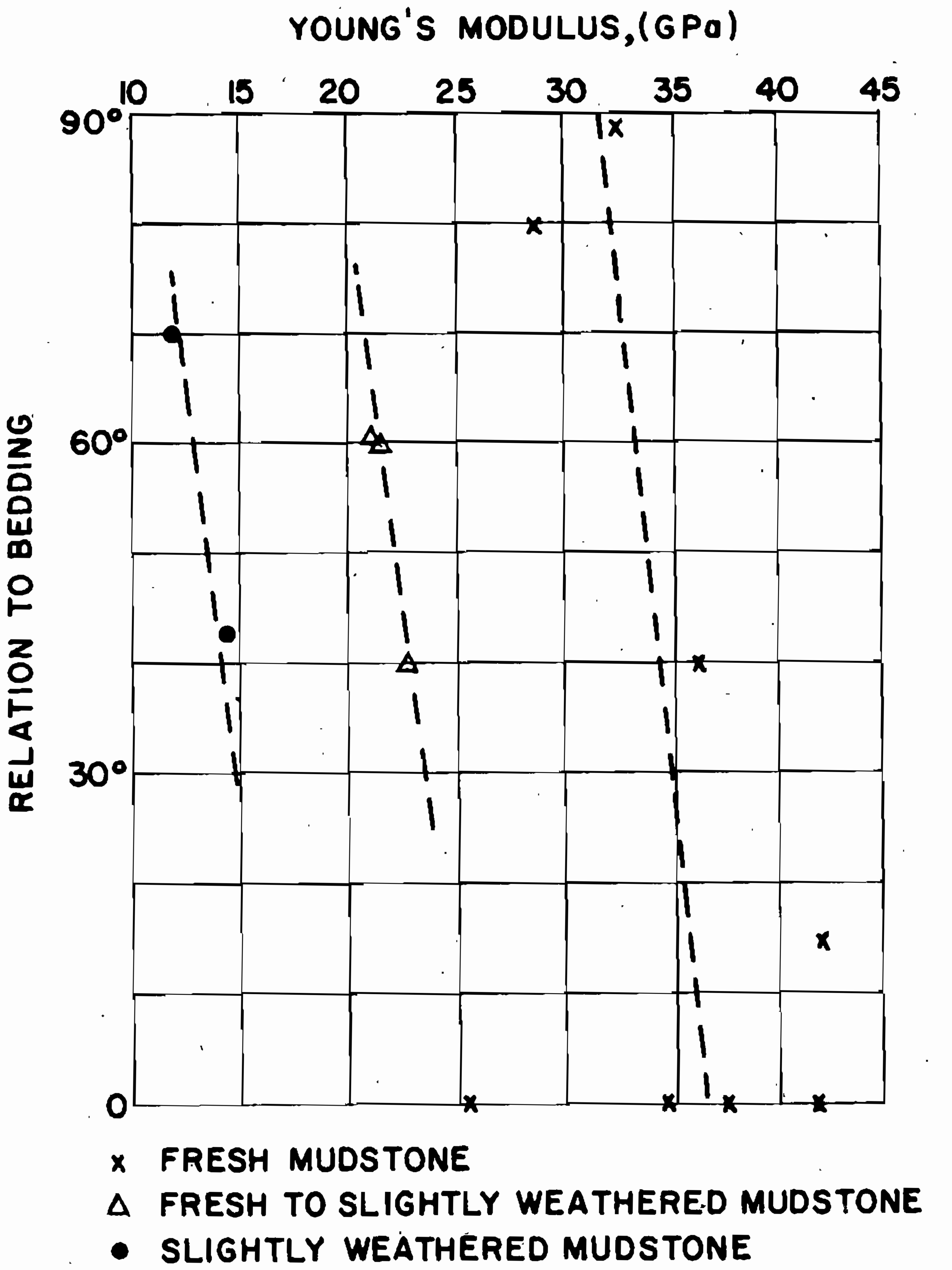

Fig. 4 Effect of bedding on modulus determined by seismic methods

The graph shows a small variation of modulus with the angle made with the bedding - the highest modulus values being measured parallel to the bedding. However the graph indicates that the rock modulus is reduced considerably by slight weathering of the mudstone and hence weathering is the more significant variable for the site.

Borehole pressuremeter testing of the weathered mudstone layers was carried out in some of the boreholes used in the seismic testing. Six tests were performed in mudstone logged as slightly weathered and gave a mean modulus value of 16 GPa for the two seismic tests performed in similar material.

4 ELECTRIC WELL LOGGING

Standard commercial well-logging techniques for the Petroleum Industry have reached a high level of sophistication in the 40 years since the first down-hole geophysical measurements were made in oil wells. Techniques currently available include resistivity, induction, sonic velocity, neutron and density logging which in combination provide fairly precise information on the nature of various rock formations and the depths to the various interfaces. Whereas very few core samples are taken in the drilling of petroleum and water wells and much of the data about the formations penetrated by these wells is derived from well-logging, in civil engineering projects it is usual to fully core rock formations. Hence very little use has been made of well-logging techniques in engineering investigations.

One of the simplest and comparatively inexpensive well-logging methods involves the lowering of a probe containing several electrodes down a borehole and measuring the variation of electrical resistance as the probe passes through the various rock and soil layers. The authors have been evaluating the resistivity method of electric well-logging in conjunction with drilling programmes for tunnel investigations.

With the exception of clay minerals and metallic sulphides most mineral grains are insulators and electrical conduction in the majority of rocks is essentially through the interstitial water which is usually present and which invariably contains some dissolved salts. Consequently the resistivity of a formation generally depends on the resistivity of the contained electrolyte and is inversely related to the porosity and degree of saturation.

In fresh rocks which have low porosities conduction takes place along the discontinuities and, other factors being constant, resistivity should be controlled by the degree of fissuring and the number of clay seams encountered. This is illustrated by Figure 5 which compares the rock quality designation (R.Q.D.) of a rock core with the specific resistance measured down the hole with a borehole resistivity probe, using an electrode separation of 40 cm in a standard electric well-logging array known as the "Short Normal". The bore is located in steeply dipping fresh to slightly weathered mudstone of high strength with most joints being clay-free. The specific resistance generally follows the rock quality designation but the clay seam that occurs between 20.75 and 21.0 m and its associated band of moderately weathered mudstone produces a marked reduction in specific resistance.

While the results included on Figure 5 would indicate a fair correlation between R.Q.D. and specific resistance, the authors have been unable

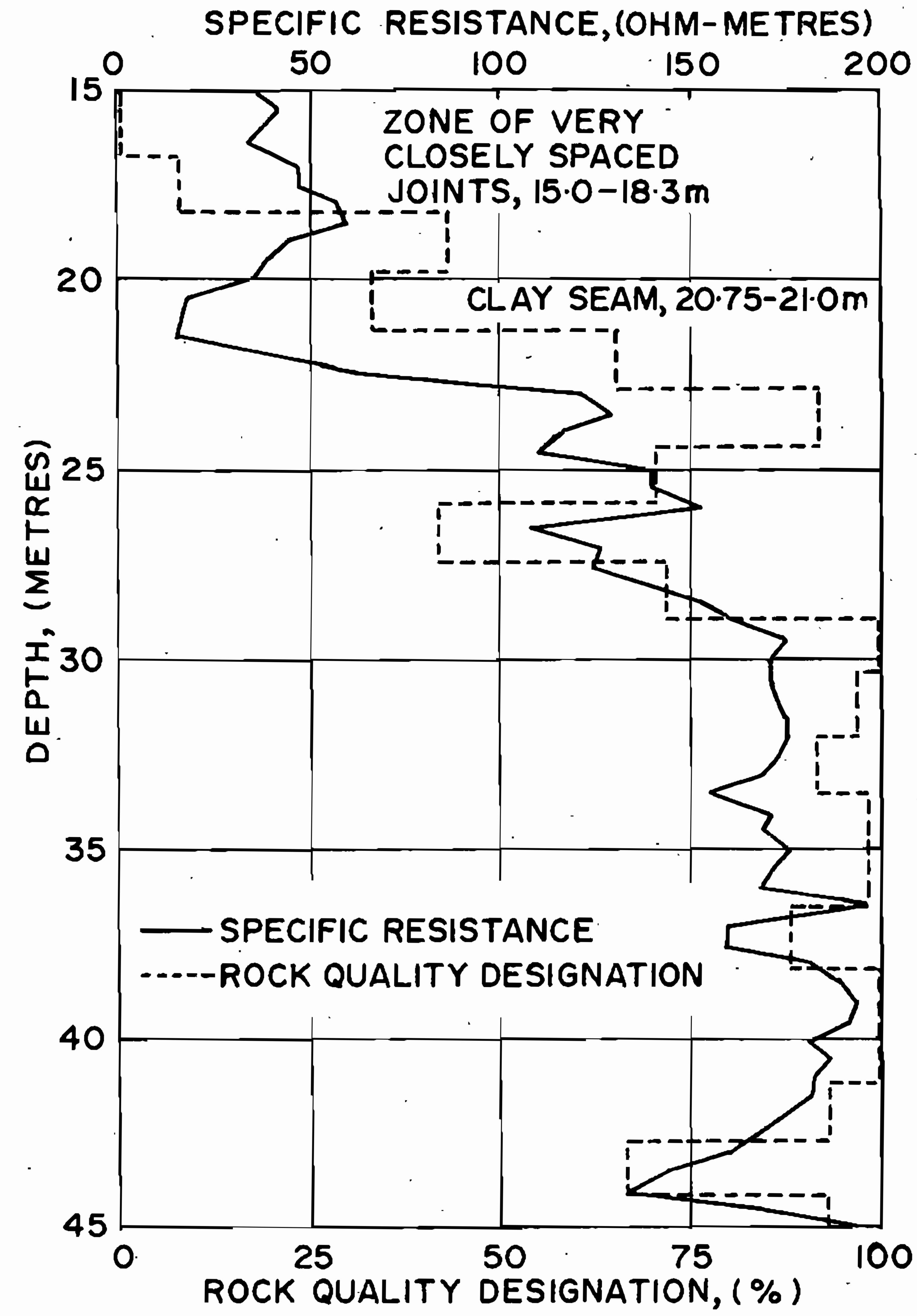

Fig. 5 Electric well log correlation with R.Q.D.

to maintain such correlation for well-logs obtained at a number of sites. Boreholes in similar geological conditions to that used for Figure 5 have provided correlations which are excellent over part of the hole but poor elsewhere. Similar experiences have resulted from attempts to apply the electric well-logging technique in soft sediments of Tertiary and Quaternary age. There are two probable reasons for this, the first being the relatively minor contrasts in specific resistance which exist between layers with significantly different engineering classification properties.

The second and more important reason however, is the dominating effect which groundwater salinity has on specific resistance. This was shown during an investigation which was performed using surface resistivity techniques in an effort to assess the effect of groundwater salinity on resistance. A calibration was attempted between specific resistance and soil classification for various groundwater salt concentrations. Typical results indicated that a specific resistance of 6 ohm-metres represented 50% passing the 200 sieve for a salinity of 1 gm per litre. As a comparison, a specific resistance of 1 ohm-metre represented 50% passing the 200 sieve for a salinity of 6 gm per litre. It must be concluded that meaningful results can only be obtained for extremely uniform deposits, in which groundwater salt concentration can be mapped, or under conditions of uniform groundwater salinity which may permit interpretations of the soil profile to be made. Such effects are relevant both to electric well-logging and surface resistivity techniques.

The authors' experience with the electric well-logging technique is sufficient to suggest caution in its use for engineering purposes until considerably more experience has been gained of the factors which influence the measured resistance. The technique in principle remains a useful one for rapidly assessing ground conditions between cored holes, however application of the technique to tunnel investigations is obviously still in the developmental stage.

5 CONCLUSIONS

Conventional site investigations of tunnel alignments concentrate on the recovery and visual classification of core obtained from the level of the tunnel. Increasing demands are however being placed on field engineers to obtain reliable soil and rock parameters, so that these can be fed into computer analyses to aid in the design of tunnel linings. In this paper, the authors have presented some aspects of their experience in using three down-hole investigation techniques which have to date received limited attention in tunnelling investigations in Australia. The techniques indicate means by which quantitative measurements of soil and rock properties can be made in-situ, without the additional demands of sampling, transportation and laboratory testing.

Of the three methods discussed, the best results have been obtained using the Menard Pressure meter and the cross-hole seismic technique. Both provide estimates of in-situ Young's Modulus, the first over a relatively limited area surrounding the probe itself, with the second being averaged over the distance between boreholes. Because of the different nature of the test methods, each has its own particular advantage depending on the particular sub-surface conditions at the site. The Pressuremeter provides additional information on the in-situ horizontal pressure in the ground and the in-situ shear strength. When supplemented by surface seismic traverses in which both shear and compressional wave velocities are measured, estimates of Poisson's Ratio can be made. Taken together, both techniques provide the means by which reliable tunnel design parameters can be obtained.

The electric well-logging technique obviously requires further developmental work before it can be used widely in tunnelling investigation. In particular, a careful control on stratigraphy changes and groundwater salinity is required before meaningful results can be obtained. The rapidity with which well-logging can be performed is the main advantage which the technique appears to offer, although close correlation at some cored boreholes is obviously necessary.

The additional information provided by investigation methods such as those discussed in this paper should permit tunnel designers to obtain a clearer picture of the variations in sub-surface conditions, and their impact in both design and construction.

Model Tests for the Design of Underground Railway Stations—Eastern Suburbs Railway, Sydney

by

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SUMMARY. Kings Cross and Martin Place Stations of Sydney's Eastern Suburbs Railway are located underground in densely developed business areas. The 9 m to 21 m rock cover at both sites is horizontally bedded Hawkesbury sandstone with varying structure, degree of weathering and mechanical properties.

The paper describes photoelastic model tests, which were carried out to provide design estimates of the surface subsidence during excavation of the stations, loads on rock pillars and steel columns, and the range of possible rock stress conditions around the caverns. In addition to photoelastic patterns, strains and displacements in the models were observed during the tests.

In the case of Martin Place Station, a finite element model was used together with photoelastic models; where comparable both methods gave similar results.

The models simulated rock layers with different mechanical properties. One of the models simulated different degrees of jointing and bedding in the rock above the station cavern.

Both the Kings Cross and the Martin Place investigations included studying alternative excavation sequences and/or construction procedures and their effect on subsidence and on loads on rock supports. In the case of Kings Cross Station, excavated under a 15 storey building, a construction procedure was investigated and adopted which provided for an early erection and preloading of the steel columns which support the roof of the station. Observations during excavation of the station showed the maximum vertical movement and subsidence gradients were in general agreement with those predicted.

1 INTRODUCTION

The Public Transport Commission of N.S.W. is constructing the Eastern Suburbs Railway, an underground extension of the Sydney Metropolitan railway system. In excess of 4 km of the line is being built through the heart of the city of Sydney. The Commission retained the Snowy Mountains Engineering Corporation as its consultant for the engineering investigation and design of the civil engineering works associated with the project.

The City of Sydney is fortunate that it has good quality sandstone as a foundation material generally close to the natural surface and that major buildings are typically constructed on highly loaded column footings. To facilitate access to railway stations, it is essential that the rail level be as close to the surface as possible and in this situation large footing loads can have a significant effect on underground structures. The majority of underground construction consists of unreinforced concrete-lined single-track tunnels with an excavation width of 5 m and cover averaging 12 m. Only in isolated circumstances, when footings were very close to the tunnel crown, precautions were required to support the footings during construction and provide tunnel lining sufficient to carry the imposed loads. Underground stations are of the order of 168 m long and 24 m wide and excavations of this size can have a significant effect on the performance of buildings above them.

This paper briefly deals with model studies for the design of Kings Cross and Martin Place Stations, the two underground stations on the Eastern Suburbs Railway that are located in the central area of Sydney.

The main studies were concerned with surface subsidence during construction of the Kings Cross Station excavated under a 15 storey hotel, and with the loads on pillars and columns which support the roofs of the stations. Two-dimensional photoelastic models of transverse cross sections of the stations were used with some supplementary analyses done by the finite element method. Investigations of the effect of rock structure, properties, pressures in-situ and of alternative construction procedures on the subsidence, on pillar and column loads, and on rock stresses around openings were included in the studies.

Subsidiary analyses were done of pillar loads in a longitudinal section; of isolated foundation loads adjacent to a single tunnel; and of subsidence during excavation of single and twin tunnels.

2 GEOLOGY, ROCK PROPERTIES, ROCK PRESSURE

(a) Geology

Both stations are situated in the flatly bedded Triassic Hawkesbury Sandstone Formation. An extensive geological investigation was carried out at both stations. The mineralogical composition, structure and properties of rock varied greatly at each site making it necessary in the physical and mathematical modelling to revert to considerable simplifications of site geology.

The geological assessment of the vertical sequence of rock formations for the central area of the Kings Cross station near the Crest Hotel provided the following data for modelling:

1.5 m to 3 m of fill and loose soil,

TABLE I

MECHANICAL PROPERTIES OF SANDSTONE AT KINGS CROSS AND MARTIN PLACE STATIONS

Sandstone Station	Sandstone Type	Compressive strength lb/sq.in. (MPa)	Tensile strength Vertical lb/sq.in. (MPa)	Tensile strength Horizontal lb/sq.in. (MPa)	Deformation modulus, 10^6 lb/sq.in.(MPa) Short time loading	After creep	Increased load and creep	Plate bearing	Accepted in analysis
		After 24 hrs immersion in water							
KX	A	1700 (12)		120 (0.8)	0.28 (1 900)	0.08 (550)	0.11 (800)	-	0.05 (350)
KX	B	3800 (26)	30 (0.2)	170 (1.2)	0.3 (2 100)	0.15 (1 000)	0.2 (1 400)	-	0.1 (700)
		After storage in 65% RH atmosphere							
KX	B	3600 (25)	64 (0.4)	510 (3.5)	-	-	-		
MPL	C	Not tested, taken as for B							0.15 (1 000)
		Preserved in approx. in situ moisture condition							
MPL	D	3800 (26)	90 (0.6)	360 (2.5)	0.7 (4 800)	-		load 0.3 (2 100) unload 0.7 (4 800)	0.25(1 700) -0.5(3 400)
		After 24 hrs immersion in water, another batch of samples							
MPL	D	5000 (35)		330 (2.3)	1.0 (6 900)				

acting as dead load

7.5 m to 9 m of weathered and closely jointed sandstone (extending down to 6-7 m above the crown of the station)

from 6 m above the crown down, 'fresh' sandstone; the upper 3 m (Sandstone A in Table 1) were mechanically weaker than the underlying sandstone (B in Table 1).

The scheme of rock formations for the Martin Place Station was similar:

up to 9 m of loose soil and weathered rock

about 9 m of sandstone of moderate strength (Sandstone C in Table 1) similar to Sandstone B at the Kings Cross Station

fresh Sandstone D in which most of the station cavern is located.

Depending on the position along the station, formation D extends from about the level of the crown down to below the floor of the station.

(b) Mechanical Properties of Rock

A series of mechanical tests were carried out on drill core samples comprising uniaxial and triaxial compression, direct and indirect tension, modulus of rupture, direct shear and creep tests under compressive stresses of different magnitudes.

The average rock properties observed in core tests at both stations and in plate bearing tests at Martin Place Station are shown in Table 1.

The last column in Table 1 shows the long term in-situ moduli values which were accepted for the model investigation, taking into account the laboratory and plate bearing tests results, the geological information about rock structure in-situ and the anticipated stress paths of the critical rock areas around the station during construction. Since an estimation of the in-situ modulus of the rock is always difficult, model tests were carried out to check the effect of structure and moduli of the roof rock.

(c) Rock Pressures

For the analysis, the vertical pressure was taken as equal to the weight of overburden. No measurements of the horizontal rock pressures in-situ were carried out. The absence of any signs of tensile stresses in the roof of the drives or of excessively high compressive stresses in roofs points to the existence of moderate horizontal pressure in the area. Tests were carried out during the investigation to assess the effect of horizontal pressure on subsidence and on pillar and column loads.

3 KINGS CROSS STATION

(a) Description of the Station

Kings Cross Station platform cavern is located to the north of and parallel to William Street and extends from Brougham Street to Bayswater Road, crossing Victoria Street and Darlinghurst Road. Under the Crest Hotel (Kings Cross Centre) the basement and footings of the Hotel are 6 m to 9 m above the roof on the down line side of the station. The platform cavern consists of two outer sections

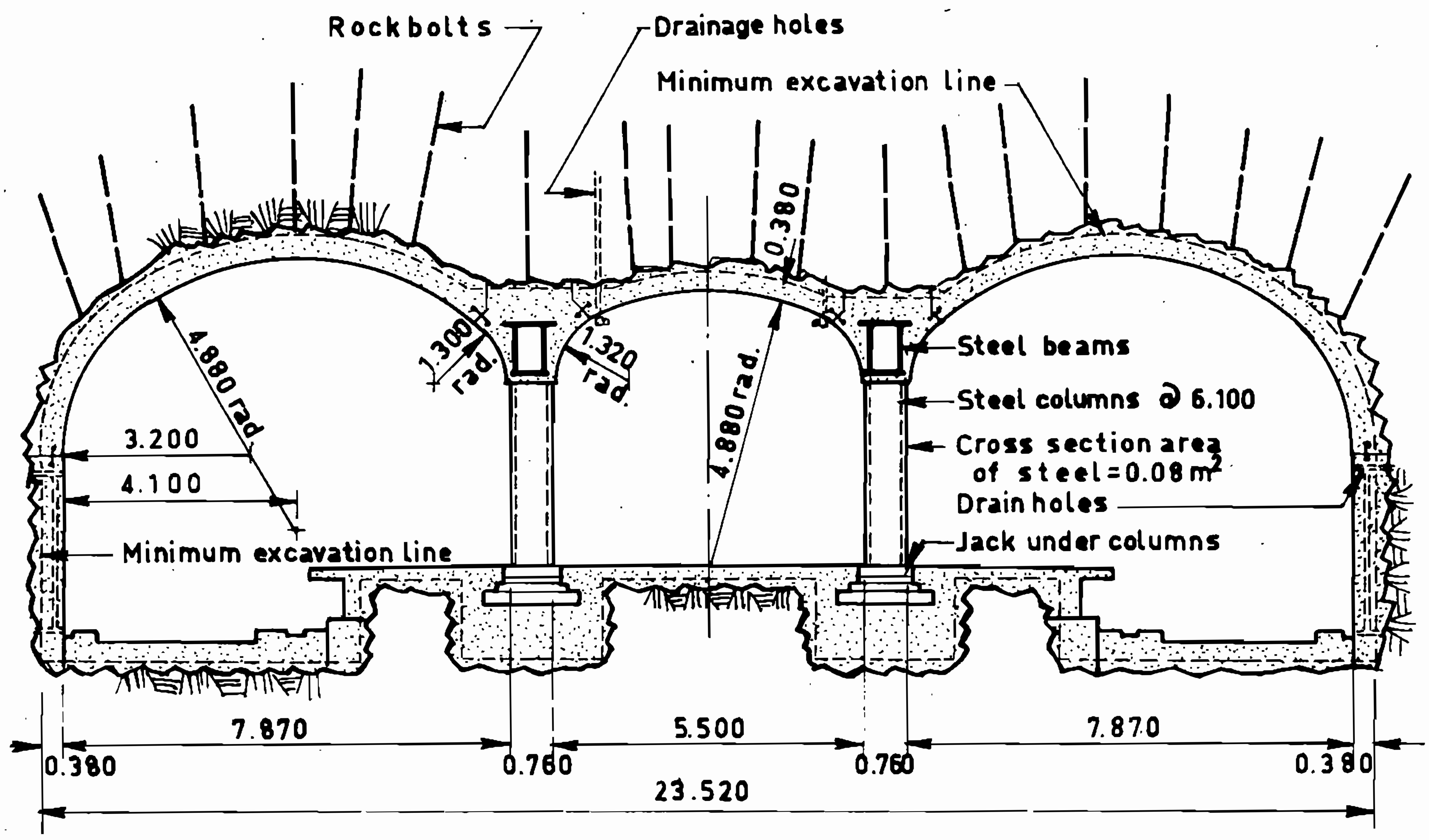

Fig. 1 Kings Cross Station - Typical transverse cross section

with rail tracks and platforms and of a central platform section for service rooms and escalators. A typical cross section of the station is shown in Fig. 1.

The station was excavated by conventional hard rock tunnelling methods. Parallel drives were excavated along the length of the station leaving rock pillars between the drives as construction support. The roofs of the drives were reinforced with rock bolts 2.5 m long and spaced generally at 1.2 m centres.

The main permanent structural support of the load of rock above the station and of overlying streets and buildings is provided by steel-concrete beams and walls running along the length of the station at the intersection of the central roof arch with the outer arches. The beams are supported by steel columns encased in concrete and spaced 6.1 m apart.

The beams also serve as abutments for the concrete linings of the roofs of inner and outer drives. The concrete linings are provided mainly for protection, water control and appearance, but have also been designed to carry a nominal load of 3 m of loosened rock above an arch.

Rock pillars, 15 m and 18 m long, were left in the middle platform section (the western pillar is under the Crest Hotel). Escalator shafts, inclined at 30° up, were excavated through and above the pillars. The bottom parts of the pillars served later to support the escalators. The main structural support in the escalator shaft areas is provided by 0.9 m thick concrete walls running on each side of the shaft.

(b) First Design Proposal

(i) Description of the tests

Initially, it was proposed to construct the Kings Cross Station by excavating a 7.9 m wide central drive and two 5.7 m wide outside drives with 2.1 m wide rock pillars left between them. Roof beams, steel columns and the concrete roof arch in the central drive were to be erected next, to be followed by excavating the rock pillars and completing the lining of outside drives with concrete.

Two models were tested to investigate this proposal, both of the two-dimensional type as the shape of the station cavern allowed it. One model was made of gelatine-glycerine mix which exhibits photoelastic fringes under its own weight (Model KX-1); the other (Model KX-2) was made of plastics, with the gravity load and horizontal pressure being simulated by external loading. For both models the depth of cover above the crown of the station was taken in the tests as being 15 m.

Model KX-1 simulated only the excavation of three drives.

During the test horizontal and vertical cuts were made progressively in the model simulating clay seams and joints to represent geological conditions. All rock above the station right up to the surface was initially represented as intact homogeneous material, and finally, as completely weakened by horizontal and vertical joints and clay seams.

Three construction stages were simulated in the test of Model KX-2: excavation of the three drives as in Model 1, erection of columns and excavation of rock pillars.

The model represented only the rock areas from 6 m above the crown to 60 m below the floor of the station. The top 9 m of soil and weathered rock were assumed to act solely as a dead load on the underlying more sound rock.

The 'soft' Sandstone A (3 m to 6 m above the crown) was simulated by a strip of Araldite D resin plasticized by adding 15 pcw of Thiokol LP-3; for Sandstone B unplasticized Araldite D was used. Based on the results of initial tests on rock samples available at this time, the ratio of the moduli of the materials was taken as being approximately 1:4.

The model columns were equivalent to continuous walls in the prototype. They were dimensioned to produce the same sum of axial contraction, and roof and floor penetrations as the beam-columns structure.

The 'subsidence' movements in the model were observed by measuring changes in the distance between horizontal reference lines inscribed on the model and a thin hair tensioned between two pins which were located at distances from the axis of the station equal to those of the prototype (approximately +20 m and -20 m).

(ii) Test results

- Rock stresses: The tests showed that compressive stresses in the rock around the station drives would be too low to cause rock failure but tensile stresses up to about 0.7 MPa were observed along the central roof arch, and up to 0.35 MPa in the roofs of outside drives at low horizontal pressures. The possibility of instability of the roofs of the central drive and of the drive under the Crest Hotel was one of the considerations for modifying the construction procedure.

- Pillar and column loads: A method, widely accepted in the mining industry for the computation of loads on pillars in flat-lying deposits, is the so called 'tributary area' method or theory (Ref. 1).

According to this theory the load on a pillar is approximately equal to the weight of the column of rock above the pillar contained within its tributary area. This area is defined as the area between the mid-width points of openings on each side of the pillar.

In the Kings Cross Station, according to the tributary area theory, the pillars would have to support rock spans of about 9 m in width. In the model tests, with a depth of 'sound' rock in the roof equal to 15 m, the observed pillar loads were approximately 70% of the tributary theory loads; with 6 m of 'sound' rock in the roof, the load ratio was about 90% in the gelatine model test and 95% in the plastic model test.

During excavation of a pillar in Model KX-2, approximately 80% of the load carried by the pillar was transferred to the nearest column. The effect on the other column was small. After excavation of both pillars the column loads were approximately 18 MN per column, i.e. the columns carried approximately 90% of their tributary rock load.

Subsidence: The maximum subsidence, occurring approximately at the axis of the station, was predicted to be between 20 mm and 25 mm (Fig. 2). The Crest Hotel is situated in the area of the relatively high subsidence gradients. A differential settlement of the hotel footings of up to 10 mm was predicted. With the distance of about 7 m between the footings, the subsidence gradient in the hotel area would be about 1:700.

About half of the total subsidence was associated with the excavation of the three drives and the other half with the excavation of rock pillars. The axial contraction of the pillars accounted for about three-quarters of the subsidence above the pillars during the excavation of the drives (the remainder was apparently due to the additional compression of the rock above and below the pillars) and for about two-thirds of the subsidence at the axis of the station.

The axial contraction of the columns accounted for about four-fifths of the subsidence which occurred at the axis during excavation of the pillars.

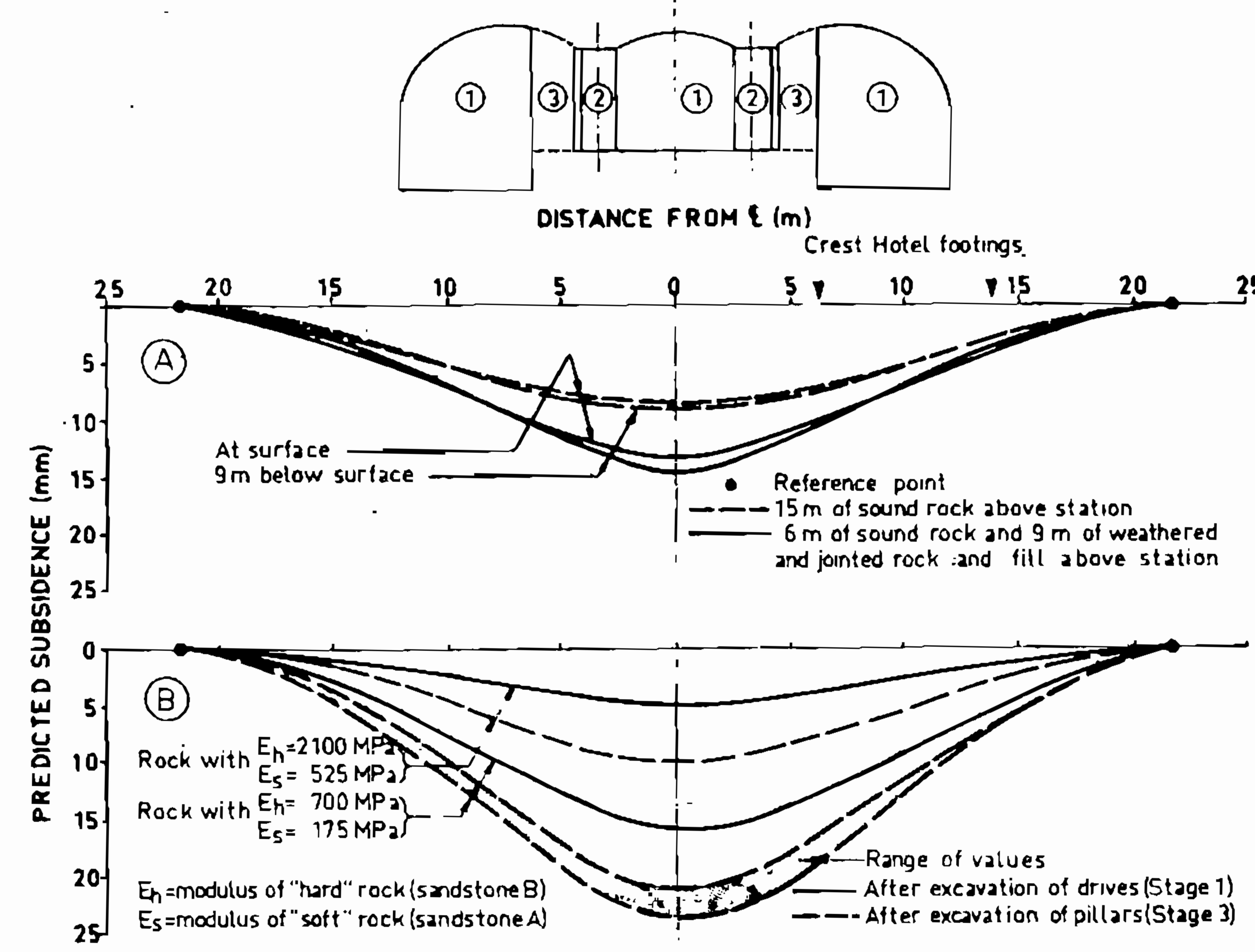

Fig. 2 Kings Cross Station - Initial Design Proposals
A. Model KX1
B. Model KX2

Cutting up the top 9 m of overburden to reduce the depth of sound rock cover from 15 m to 6 m increased the subsidence in Model KX-1 by about 50% (Fig. 2), roughly in proportion to the accompanying increase in pillar load.

The horizontal rock pressure was found to have only small effect on the subsidence and on the pillar and column loads. Compared with the subsidence values predicted by the tests, the subsidence and subsidence gradient at the Crest Hotel were considered excessive; furthermore, it was possible that the actual subsidence could even increase more if the rock modulus in-situ was lower than 700 MPa assumed in the estimates; some loosening up of the rock above and below columns would normally occur during excavation of the central drive, and would increase the subsidence when the rock becomes retightened by the column loads. Some concern was also felt about the stability of the roof of the 7.9 m wide central drive.

In view of this it was decided to investigate an alternative construction procedure which would minimise subsidence, and reduce the width of unsupported roof arches during construction of the station.

(c) Second Design Proposal

The main stages of the new construction procedure were (Fig. 3):

Stage 1 Excavation of two 4.5 m wide column drives with 3 m wide pillars left between the drives

Stage 2 Erection of roof beams and steel columns in the drives and concrete walls at escalator shafts; preloading of columns and walls to about half of estimated final load; encasement of steel columns with concrete

Stage 3 Removal of the pillar between column drives

Stage 4 Placement of roof lining in the central arch

Stages 5 and 6 Excavation of outside drives to the full width of the station

Stage 7 Placement of lining in the outside drives and construction of platforms

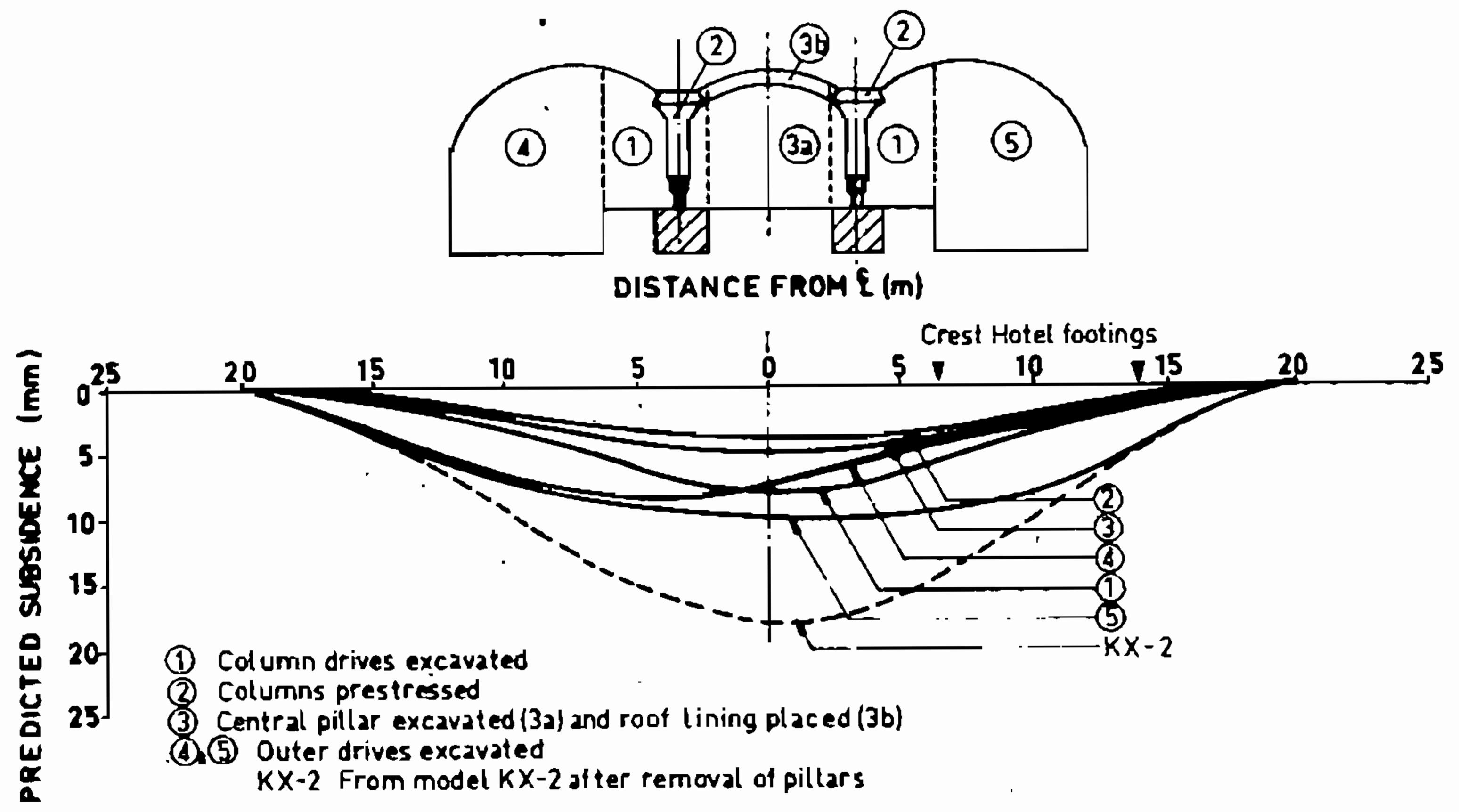

Fig. 3 Kings Cross Station - Second Design Proposals Model KX-3

(i) The model

A new two-dimensional plastic model was built (Model KX-3), similar to Model KX-2, on which the main construction stages of the new procedure were reproduced step by step (Fig. 3).

The modulus of sandstone was assumed to be 700 MPa in the prototype and of the Sandstone A above the roof as being 350 MPa. The cross section area of steel in the columns (As) was taken as 0.08 m^2. Fine screws with pads were used to preload the model columns. An aluminium arch simulated the reinforced concrete roof lining of the central drive.

(ii) Test results

• Rock stresses: The tensile stresses in the roof of the central drive were much lower than for the first design proposal (only 0.2 MPa, maximum for horizontal pressure equal one third the vertical pressure). This was partly due to different shape of openings and partly due to preloading of columns which produced compressive stresses in the roofs of the drives.

• Loads on central pillar and columns: Before preloading of the columns the central pillar carried about 90% to 95% of its tributary rock load. After preloading of columns the average vertical stress in the pillar became about the same as before excavation of the column drives.

The column loads at the completion of construction were up to 22 MN for the assumed depth of cover of 18 m, i.e. about 25-30% higher than for the first design proposal and about 10% greater than their tributary load.

The test results showed that the creep of rock in-situ would cause an increase of load on the columns and in the arch but the increase should be within the safety margins.

• Jacking up movement: To obtain the desired prestress of the columns, the total jacking up movement in the model was made equivalent to about 12 mm in the prototype; however because of the approximate nature of data on rock properties in-situ, it was recommended that the jacking system should be designed for movements up to 25 mm.

• Subsidence: Fig. 3 shows the estimates of the subsidence at the level of the footings of the Crest Hotel. In plotting this figure, an assumption was made that during preloading of the columns the rock will behave as if it has a deformation modulus of 1 400 MPa, i.e. the same value as in drill core tests and twice as high as the one that occurs during excavation. (Preloading of the columns creates in the roof and floor the conditions of triaxial compression under which inelastic deformations in the rock were thought to be less likely than during excavation.)

For the new construction proposal, the estimated maximum subsidence relative to the points located at $\pm$19 m from the axis of the station was about 10 mm, the differential settlement of the footings of the Crest Hotel was about 5 mm and the average slope between the footings was about 1:1500. All three values were about half as high as for the first proposal.

The axial contraction of the central pillar accounted for about 65% of the subsidence during the excavation of the column drives; and the axial contraction of the columns for about 55% of the subsidence during the excavation of the pillars and of outside drives.

• Stresses in the central arch: The predicted loading of the central arch, caused by excavating the outer arches, was below the nominal load of 3 m of rock for which the concrete roof arches were designed.

(d) Observations during the Construction of the Station

The procedure investigated in Model KX-3 was accepted for the final design of the station, except for the areas of the escalator shafts mentioned in Section 3(a), where short sections of the central pillar (one of them under the Crest Hotel) were in part left in place to support the escalators, and 0.9 m thick concrete walls were used instead of roof beam and columns structures. Both walls were prestressed to approximately the same load as that of the beam-column structure.

Twenty-two precise levelling points were established in the pavement of Victoria Street and in the foundations of buildings along this street, including nine points on the footing columns of the Crest Hotel. Forty-seven points were established on the surface along the centre-lines of the railway tracks in the outside drives of the station. The levels were measured with an accuracy to 0.3 mm. Only changes in the levels of 0.6 mm or more were considered significant.

Fig. 4 shows subsidence movements along Victoria Street transposed onto a plane perpendicular to the station, i.e. each precise levelling point is plotted according to its distance from the axis measured normal to it. A comparison of subsidence along Victoria Street with subsidence in the model is also given in Fig. 4. Fig. 5 shows subsidence observations along the line of the railway tracks.

As can be seen from Fig. 4, the subsidence profile after excavation of the column drives was similar to that observed in Model KX-3 but the magnitude of subsidence was about 0.7 times that of the 'predicted' values. It appears from this that the effective deformation modulus of 'hard' sandstone B was about 1 000 MPa, as compared with 700 MPa accepted for design purposes. The maximum slope at the Crest Hotel footings was similar to that in the model (1:1850 against 1:1500).

Surface movements during the preloading of columns and concrete walls were much smaller than

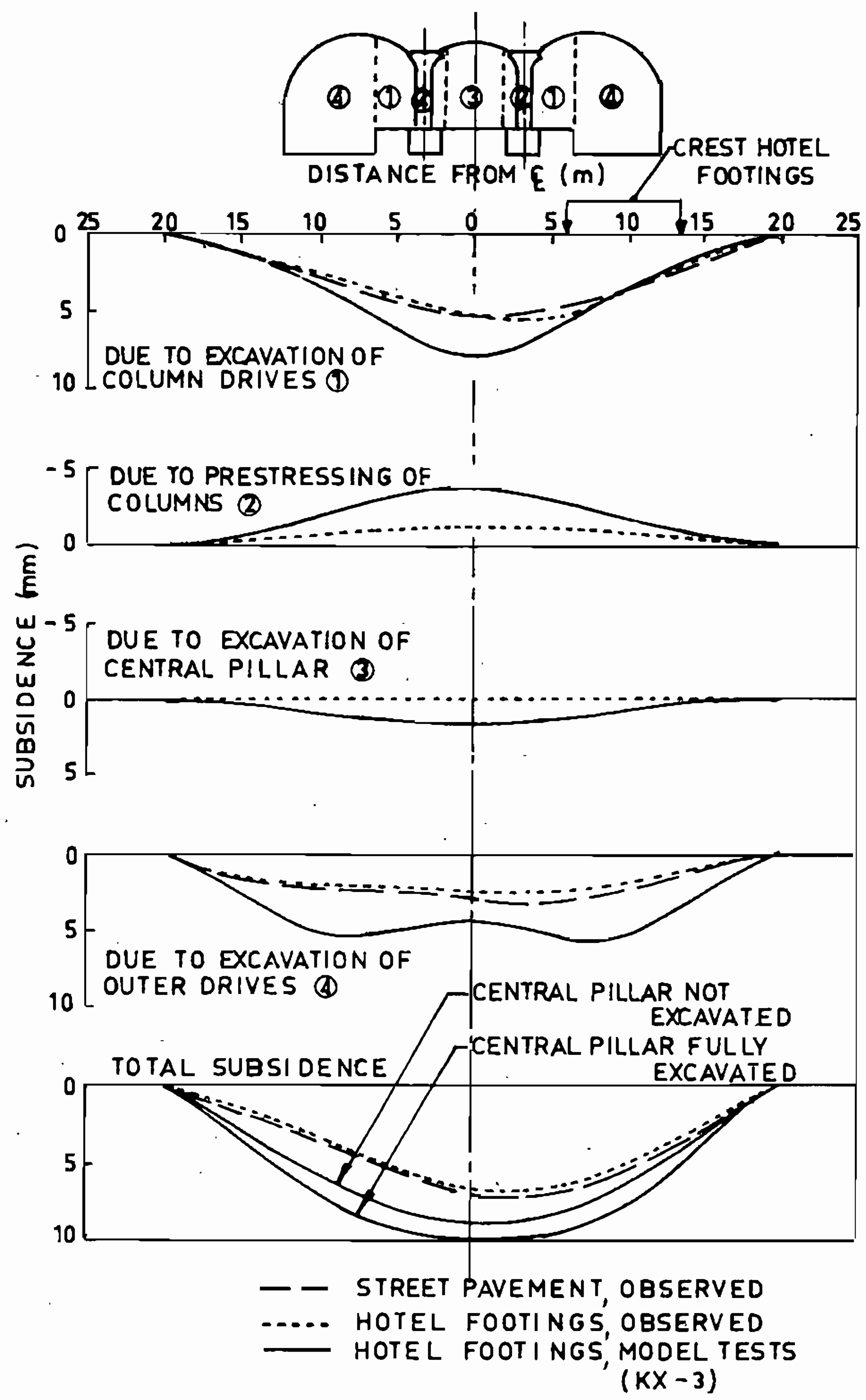

Fig. 4 Kings Cross Station - Predicted and observed subsidence

those predicted from model tests. The probable reason, not allowed for in the estimates, is that during the preloading of columns, the rock in the central pillar is unloaded and is likely to behave as material with a higher deformation modulus than during excavation of the column drives (analogous to the rebound modulus in the plate bearing tests or during unloading of sandstone specimens in compression tests). The possibility of a higher modulus of rock in the roof and the floors of the drives during the preloading of columns compared with deformation modulus during excavation was mentioned earlier. The observed jack opening displacements were up to 12 mm.

The absence of subsidence during the 'excavation of central arch' is due to the fact that -

(i) in the area of the Crest Hotel where subsidence was observed, most of the central pillar at the time of observations, was left unexcavated, and

(ii) the concrete walls, erected in these areas instead of columns, are twice as stiff as the columns.

The same factors would also reduce subsidence during the completion of the excavation of outside drives. The in-situ subsidence was about half as high as that computed from model tests for beams-and-columns frame work. The generally smoother deflection troughs than predicted during this stage as well as during the first stage of construction are consistent with a less flexible roof above the station, when compared with the 6 m thick rock roof assumed in the model studies. Presence of 9 m of weak material near surface would also tend to smooth out the subsidence curve.

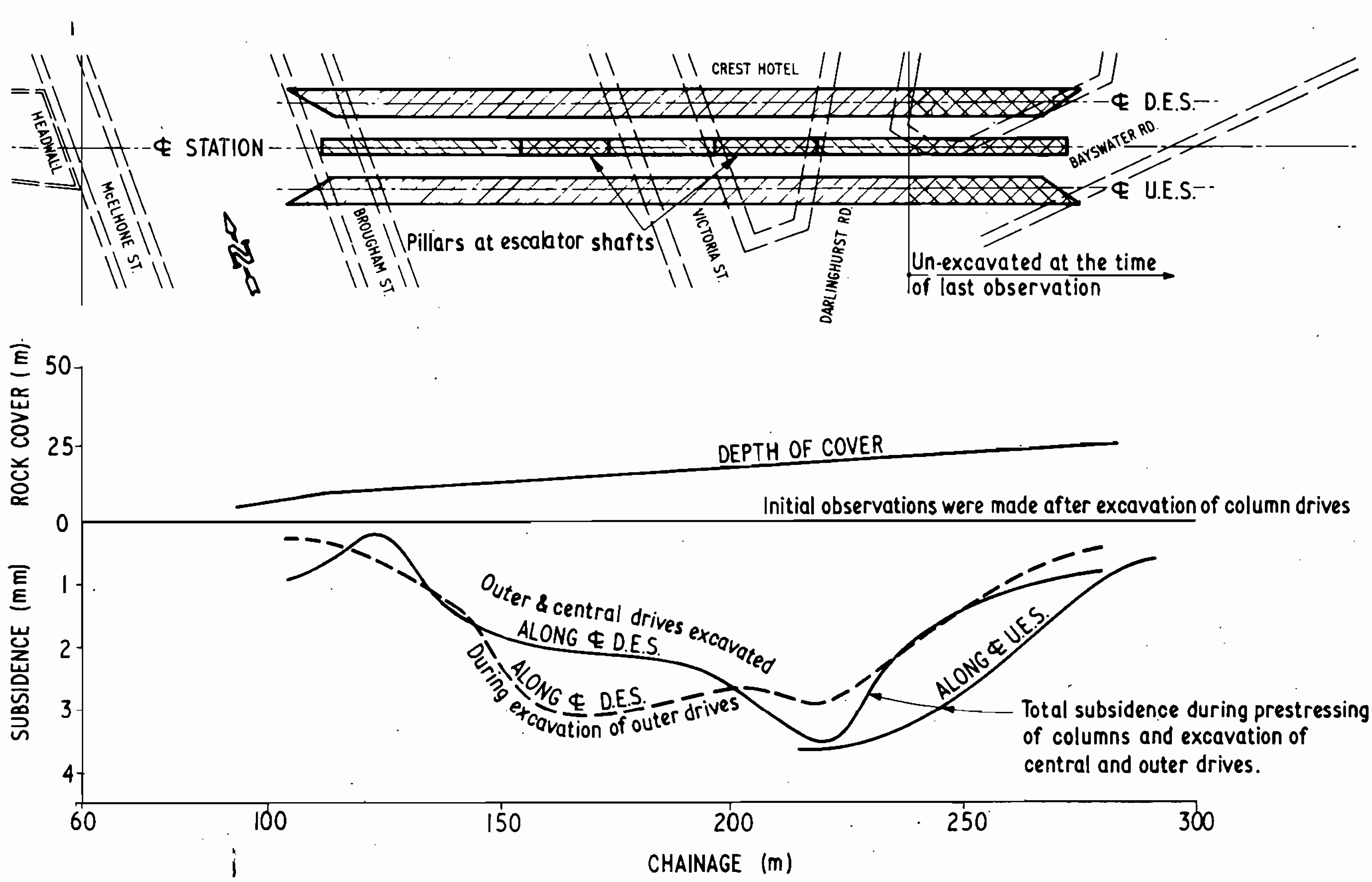

Fig. 5 Kings Cross Station - Subsidence along centreline of tracks

4 MARTIN PLACE STATION

(a) Description of the Station

The Martin Place Station runs approximately east-west along the line of Martin Place between Elizabeth and Macquarie Streets. The station is level and the cover above the station roof ranges from about 15 m at the west to 22 m at the eastern end. The layout of the station is similar to Kings Cross except that it is slightly wider, 25.7 m, and there are no rock pillars left along the platform. A pair of steel column and beam frameworks 8.5 m apart support the roof and the columns are 6.2 m apart.

At the eastern end three rail tunnels, the single track City Inner and City Outer lines and a double track storage tunnel cross at right angles to the station. At the closest point the excavated invert of the City Outer is about 2 m above the crown of the station.

The design problem at Martin Place Station is rather different from Kings Cross. The station is situated completely within the building alignment of the street and there are no structural loads above, although on either side of the station the basement levels of buildings come to within 10 m of the tunnel crown. For the construction of the station concourse most of the rock at street level had to be excavated to a depth of about 10 m above the roof of the station, so settlement of the natural surface was of no major concern. The critical design load for the station support is the full mass of the rock above but, as this rock is present only until the concourse excavation takes place and the load then decreases substantially, material could be saved by estimating column loads accurately.

(b) Photoelastic Model Tests

(i) Objectives and scope of the tests

A number of small size drives had already been excavated through the site of the station and, to fit with existing drives, different excavation sequences were considered for different areas of the station. A specific objective of the photoelastic tests was to establish whether the loads on pillars and columns would vary significantly with the excavation procedure.

Two excavation procedures were investigated:

- the excavation of three drives similar to the initial proposal for the Kings Cross Station with a 11.6 m wide central drive, 4.3 m wide outside drives and 2.7 m wide pillars between them, and

- four drives with two 2.3 m wide side pillars and one 3.7 m wide central pillar (Fig. 6).

(ii) The models

The design of the models was similar to that of Model KX-3. Three models were tested: Model MPL-1 and MPL-2 simulated the excavation procedures with two and three pillars, respectively. The moduli of the 'hard' and 'soft' sandstone areas in both models were in the ratio of approximately 1.3:1. Model MPL-3 was geometrically similar to the Model MPL-1, but had moduli ratio of 2:1 approximately. This model was used to check the effect of axial stiffness of the pillars, with the pillars made initially wider than required and then cut to a narrower width during the test.

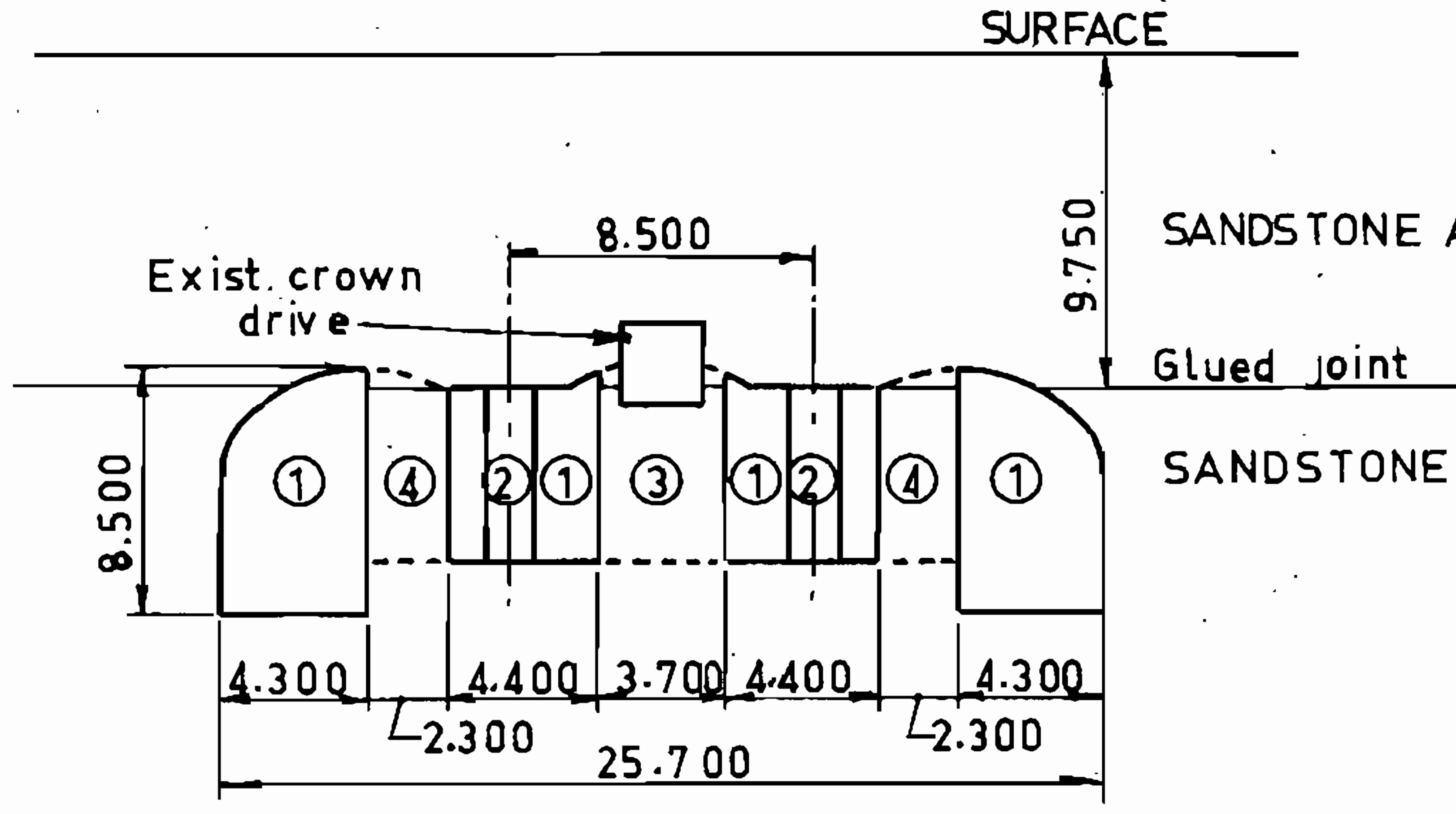

Fig. 6 Martin Place Station - Details of Model MPL-2
(All dimensions are for prototype)

(iii) Test results

For steel cross section area of 0.2 m^2 the loads could reach up to 80% of the tributary load for the two-pillar sequence and up to 90% for the three-pillar sequence. With the cross section areas of steel of about 0.13 m^2 and the modulus of sandstone B equal to 2 750 MPa the column loads were up to 70% of the tributary rock load for the two-pillar sequence, and up to 80% for the three-pillar sequence. The column loads in Model MPL-3 were only about 5% higher than in Model MPL-1.

(c) Finite Element Method Investigation

At the time of the design the finite element method was being introduced into engineering practice in Australia and it was decided to see what agreement could be found between the physical and mathematical models of the station. A one-half cross section of the station was modelled by a mesh of linear strain triangular elements, with boundaries of the mesh being set at the surface and a distance equal to the half opening below and to the side of the opening. Each step of the construction sequence of Models MPL-1 and MPL-3 was followed, as specified in the contract documents. The modulus of the rock was taken as being 3 400 MPa. Two different steel column stiffnesses were tested to check the sensitivity of column loads to column stiffness. The flexibility of the finite element method was very useful when during the tendering for the construction contract two alternative excavation sequences were proposed. With very little effort the model was altered and the alternative sequences examined. The first alternative (ALTA) involved excavating only the centre arch prior to placing the steel column; the second alternative (ALTB), which was similar to the procedure adopted for the Kings Cross Station but without column prestress, gave column loads about 20% higher.

Table 2 sets out the predicted column loads from photoelastic and finite element model studies. Although there were differences in some of the physical quantities, for example, the finite element model used only one modulus of sandstone, the column loads were compatible.

TABLE II

MARTIN PLACE STATION - COLUMN LOADS IN PHOTOELASTIC AND FINITE ELEMENT MODELS

MODEL	Rock Modulus		Rock Pillar Width		Column Area	Load on Column as % Tributary Rock Load
	Below RL 10 MPa	Above RL 10 MPa	Centre m	Outside m	m^2	
Finite Element S*	3 400 3 400	3 400 3 400	- -	3.0 3.0	0.21 0.37	67 100
Finite Element ALTA	3 400 3 400	3 400 3 400	- -	3.0 3.0	0.21 0.37	69 102
Photoelastic MPL-1*	3 400 3 400	2 400 2 400	- -	2.7 2.7	0.26 0.45	60-70 55-65
Photoelastic MPL-3*	3 400 3 400	1 700 1 700	- -	2.7 2.7	0.225 0.145	65-70 60-65
Finite Element ALTB+	3 400 3 400	3 400 3 400	4.9+ 4.9+	2.4 2.4	0.21 0.37	85 127
Finite Element ALTB*	3 400 3 400	3 400 3 400	4.9* 4.9*	2.4 2.4	0.21 0.21	86 128
Photoelastic MPL-2*	3 400 3 400	2 400 2 400	3.7+ 3.7+	2.3 2.3	0.26 0.145	73-78 70

\+ Crown Drive * No Crown Drive

Note: In ALTA and ALTB most of the cross section of outside drives is excavated after erection of columns.

(d) Estimates of Support Load by Tributary Area and Load Redistribution Methods

The pillar and column loads obtained from both the Martin Place and the Kings Cross model tests are compared in Fig. 7 with the tributary area loads. The 'observed' column loads are also compared with the computed column loads assuming that, when a rock pillar is excavated, the load carried by it is distributed to adjacent columns or rock supports in inverse proportion to the distances to them. As can be seen the tributary area method gives accurate estimates of the rock pillar loads; for estimating column loads the load redistribution method was better but the tributary load method was still within about 30% of model values. Because of creep of rock it is expected that with time the prototype column load may become close to the tributary area loads. On this basis the actual design loads at Martin Place were taken as 90% of tributary area loads before excavation of concourse and 100% of tributary area loads after concourse construction.

5 OTHER DESIGN STUDIES

A finite element model study was made of the eastern end of Martin Place Station where the rock cover was greatest and the loads are concentrated by the three tunnels crossing the station a short distance above the station crown. In several instances where large footing loads were adjacent to the single track tunnel sections finite element models were used to determine the effect on the tunnel.

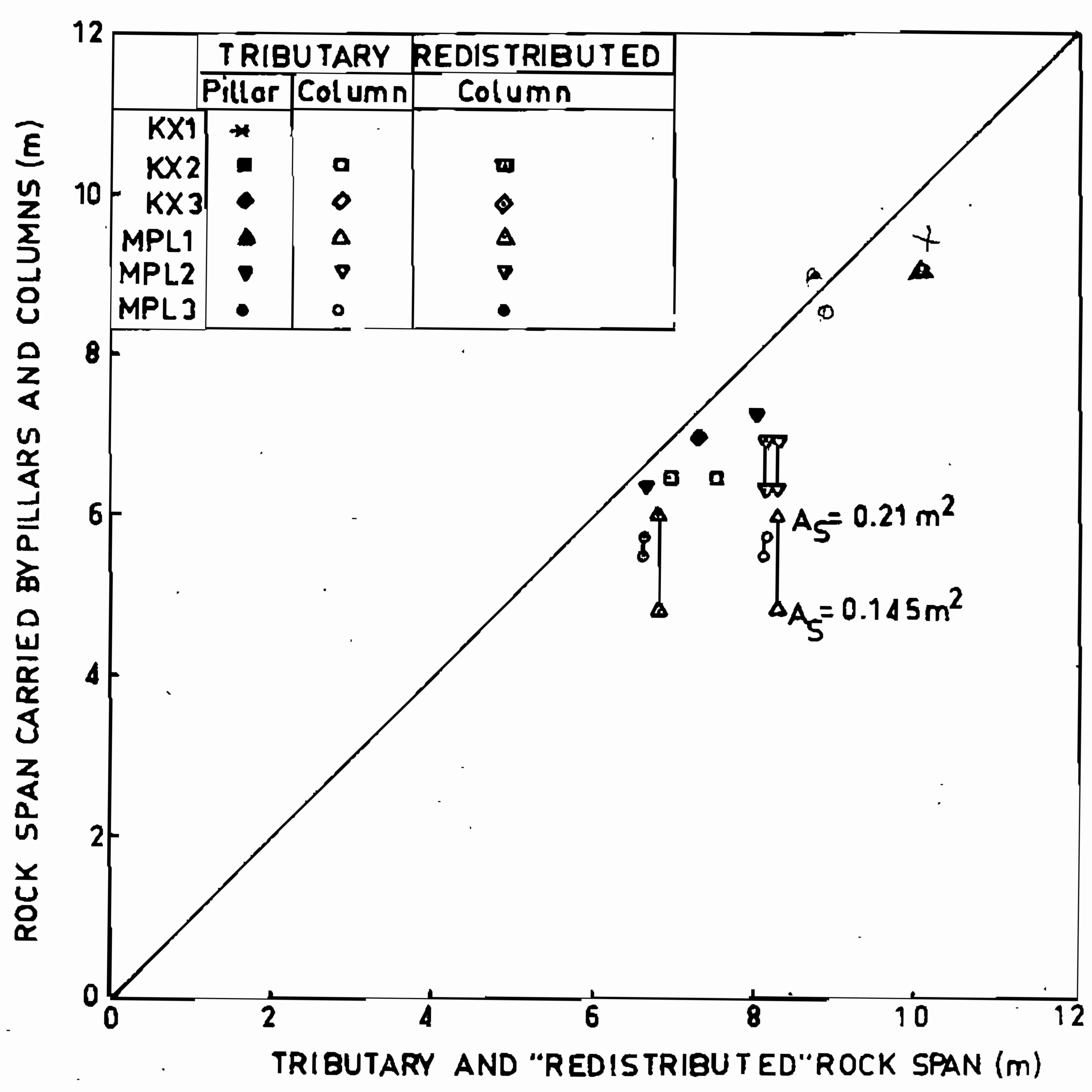

Fig. 7 Pillar and column loads - comparison of loads predicted from model study with loads computed using tributary area and load redistribution methods

Note: For Kings Cross the excavation is assumed to be in rock of modulus 700 MPa and steel column area is 0.08 m^2. For Martin Place the rock modulus was 2 800 MPa and the steel column areas are indicated on the graph.

6 CONCLUSIONS

(a) Physical and mathematical modelling provided a useful tool in the design of the Kings Cross and Martin Place railway stations, particularly in development of construction procedures best suited to each project's requirements.

(b) Despite the variability of rock conditions and the difficulty of estimating the properties of rock mass from laboratory and small scale in-situ tests, subsidence observations at the Kings Cross Station were in general agreement with the behaviour expected from model tests.

(c) Preloading of columns reduced the subsidence by approximately 8-12 mm although its effect on the surface uplift was smaller than the model tests led to expect; preloading provided a practicable method of subsidence control for a potentially difficult situation.

(d) The axial contraction of pillars and columns according to the model tests would account for 50-70% of the total subsidence observed in-situ, and in fact accounted for about two-thirds of the total subsidence observed in-situ. In the design of future underground stations in Sydney an initial estimate of the expected subsidence could be taken as twice the expected axial contraction of pillars and columns.

(e) According to physical and mathematical model tests, the loads on pillars and un-prestressed columns can be computed using the tributary area theory if the depth of sound rock above the roof of the stations does not exceed about one-half to two-thirds of the cavern width. The loads vary comparatively little with the sequence of excavation or construction procedure; rock structure and magnitude of the horizontal rock pressures in the area also have only minor effect on pillar and column loads.

(f) The observed gradients of subsidence were only one-third to one-half as high as could have been expected from the maximum subsidence according to the data of the National Coal Board and were generally smaller than are usual in mining for the same ratio of maximum subsidence to depth of cover. This is thought to be attributable to 'elastic beam' behaviour of the layer of sound rock immediately above the railway stations and to the relative stiffness of the fresh sandstone in the walls of the caverns.

7 ACKNOWLEDGEMENTS

The authors wish to thank the Public Transport Commission of N.S.W. and the Snowy Mountains Engineering Corporation for permission to publish this paper.

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MODEL TESTS FOR THE DESIGN OF UNDERGROUND RAILWAY STATIONS - EASTERN SUBURBS RAILWAY, SYDNEY

by: G. Worotnicki, M.I.E. Aust.,
R.J. Vincent, B.Sc., B.E., M.I.E. Aust.

E R R A T A:

p. 39 Column 2:

Caption to Fig. 2 should read:

Fig. 2 Kings Cross Station - Initial Design Proposals
A. Model KX1 - Excavation of drives (Stage 1; "sound" rock with E = 700 MPa)
B. Model KX2

p. 39 Column 2, second paragraph, from 3rd line to 9th line should read:

...
pillar and column loads. The subsidence values predicted by the tests, in particular the subsidence and subsidence gradient at the Crest Hotel, were considered excessive; it was possible furthermore that the actual subsidence could exceed the predictions:- the rock modulus in situ, could be lower than 0.1 x 10^6 psi assumed in the estimates; some loosening ...

p. 43 Fig. 7 to be replaced by new drawing attached.

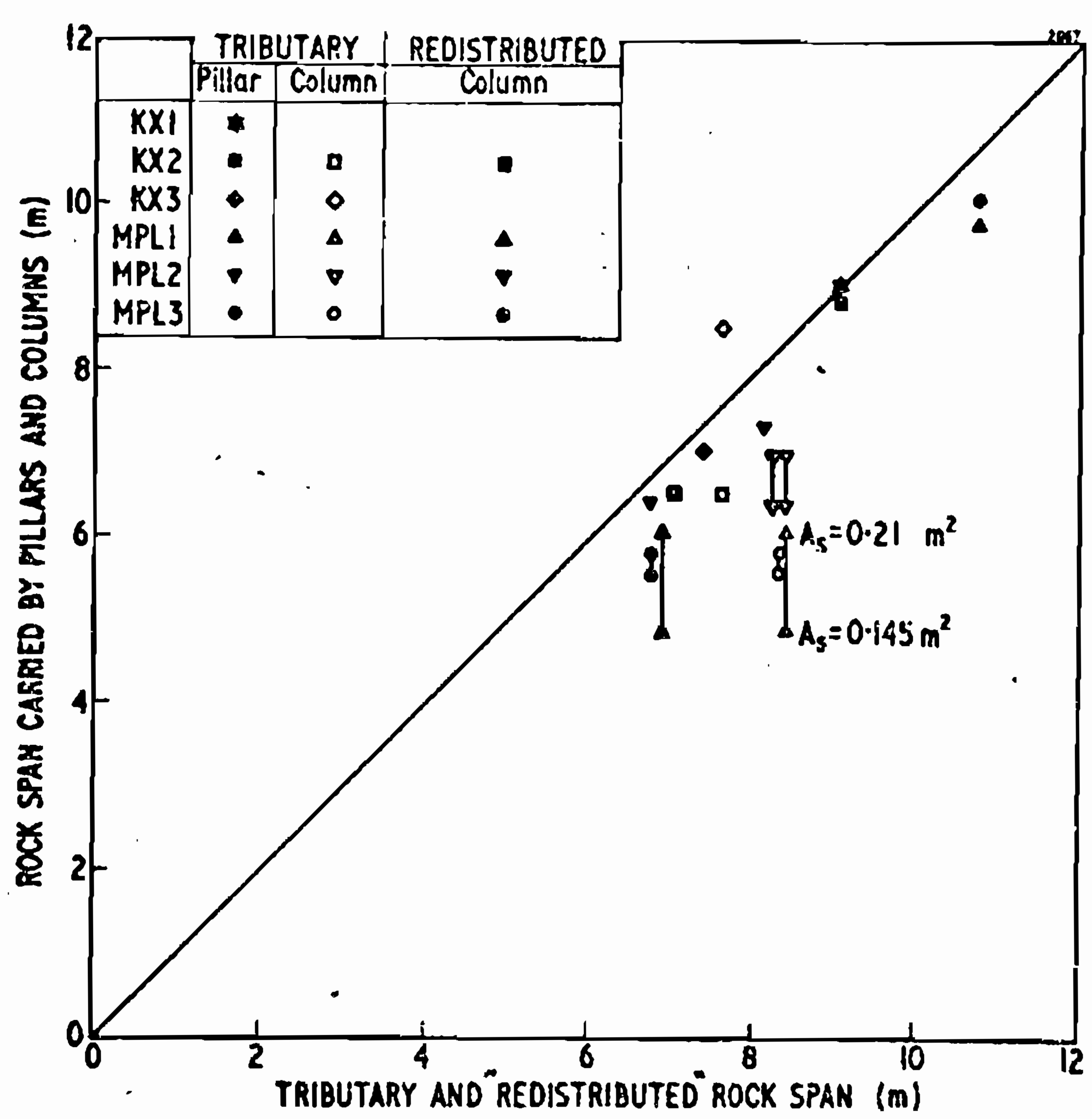

FIG. 9:- COMPARISON OF PILLAR AND COLUMN LOADS PREDICTED FROM MODEL TESTS WITH LOADS COMPUTED USING TRIBUTARY AREA AND LOAD-REDISTRIBUTION METHODS.

Factors Affecting the Underground Thermal Environment

by

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SUMMARY. Accepting the hypothesis that to survive on the Earth with reducing energy resources, we will need to explore other means of providing shelter for our activities and one of these will be subterranean accommodation, this paper attempts to point out some of the advantages and items of concern regarding human thermal comfort underground. The areas of main concern are shown to lie in the fields of volumetric occupation, ventilation and thermal earth connection. Since no model or analogue at present exists, with which to predict behaviour, recommendations are made concerning the measurable factors to be observed in a prototype.

1 INTRODUCTION

Cities are for people; living, producing, enjoying themselves; built for the advantages which only a community can generate; and the image of the city is the serried blocks of buildings huddling together for mutual protection.

Buildings - shelters for various purposes - are as old as homo sapiens himself, Morris (Ref. 1) suggests that the earliest shelters were "home bases" and territorial markers but there is no doubt that these shelters were soon regarded as convenient protection from the worst features of an otherwise not unfavourable climate and were used later on excursions into harsher climates. Gibbon (Ref. 2) records that, at a much later date, the pre-Roman tribes of Central Europe migrated Southward in Winter and Markham (Ref. 3) correlates the growth of the first civilisations (= living in cities) with the Northern 21° C. (70° F.) annual isotherm where the least sophisticated of permanent shelters sufficed the year round. As Broadbent puts it: (Ref. 4).

> "No building, anywhere, was ever built if people were satisfied with things as they stood. People wanted to do certain things, and if the climate of the open site was right, they didn't need a building. But sometimes the site was inhospitable......"

To live and survive throughout the year, anywhere on the Earth's surface, has been possible only by the building and warming of shelter. But temporary and portable shelters are, of necessity, surface buildings and in this image the development of building has continued ever since.

Surface shelters are costly items to build, condition, maintain and demolish not only in terms of money or labour but also in expenditure of material and energy resources. Wyatt (Ref. 5) makes a plea for the inclusion in the assessment of materials of the criteria of re-use potential and use of energy in manufacture. The costs of providing comfortable thermal environments are increasing and the arguments of the energy crisis are too well known to require elaboration here.

Of the alternative futures, the most promising is that of reducing the energy demand for thermal protection by sheltering underground. After all, the "Basket-weavers" of Utah (Ref. 6), the cave dwellers of the Pyrenees and Tunisia and the opal miners of Coober Pedy found (and still find) shelter underground for thermal reasons.

Utudjian (Ref. 7) suggests that there are three main factors which promoted underground development:

(a) war (Fall-out shelters),

(b) protection from the extremes of climate and

(c) the functional necessities of the situation (e.g. improving existing urban communication).

His views are stated in the constitution of G.E.C.U.S. (Group d'Etudes et de Coordination de l'Urbanism Souterrain) which he founded in 1933; that the subterranean development of cities should be confined to services, transport and storage - sewers, traffic tunnels, underground parking and cellars (Ref. 8) so as to gain space on the surface. Only in the higher latitudes of Norway and Sweden (Ref. 9) does he visualise that there would be any gain or reason for living underground. His correspondant, Moenaert (Ref. 10) at least does not dismiss the possibility and suggests shopping centres and entertainment complexes underground.

Considering the difficulties which we experience in exploiting ("developing") the surface, this paper is based on the contention that soon we shall have good cause to experiment with underground living. In doing so, the factors which affect human thermal comfort and the conditions, provisions and effects which contribute to these factors require consideration.

2 BASIC CONSIDERATIONS

Before continuing to the discussion of the factors which affect the underground thermal environment it is necessary, perhaps, to state briefly the nature and objectives of thermal control and the naturally occurring thermal environment.

(a) Human requirements

The main aim is to provide a thermal environment which will allow people to dispose of their metabolic heat comfortably. The rate of rejection is dependant on a number of factors which are well understood and documented (Ref. 11) and is of the order of 117W for a person at rest, to 6 or 7 times this rate during strenuous exercise. There are four factors which influence comfortable transfer:

(i) air temperature,

(ii) humidity,

(iii) air movement and

(iv) radiation exchange.

Comfort may be achieved by adjustment and control between the factors to give a Corrected Effective Temperature of the order of 19°C in Winter to 24.5°C in Summer (Ref. 12).

(b) Natural thermal climate

Any locality on the Earth's surface, between the Polar Circles (67° Lat.) is subjected to two principal periods of thermal input variations, diurnal and annual, the former being superimposed on the latter. The irradiation of the surface features, the effects of the atmosphere and the elevation of the Sun (Ref. 13) produce the daily and seasonal variations of air temperature and hence climate and weather (Ref. 14). The typical resulting graph of temperature v. time is shown in Fig. 1 (Ref. 15).

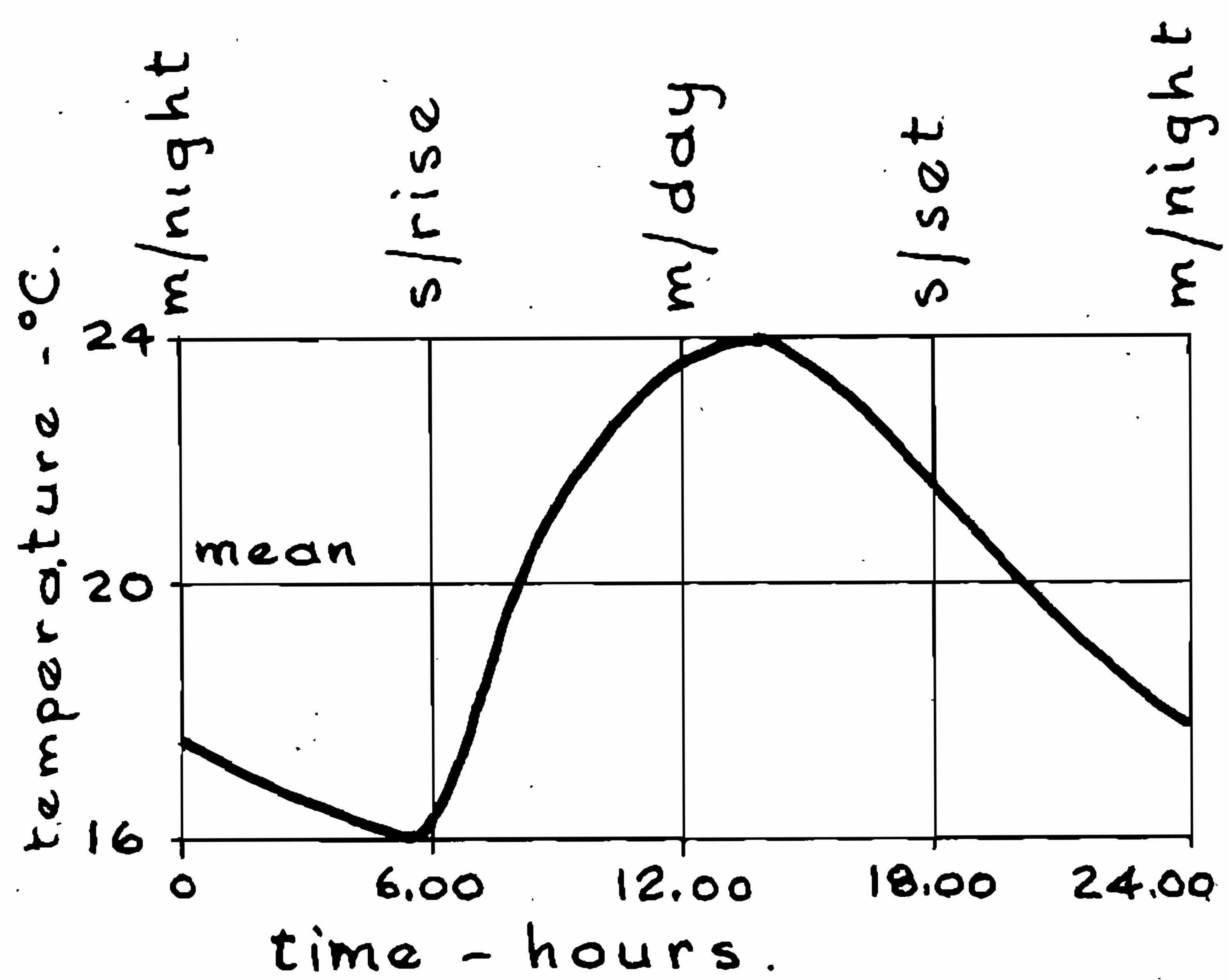

Fig. 1 Form of diurnal air temperature variation. (Typical, Sydney, vernal equinox).

The mean daily temperatures plotted on an annual scale for Sydney is shown in Fig. 2 with the graphs of daylight and sunshine for comparison (Ref. 16).

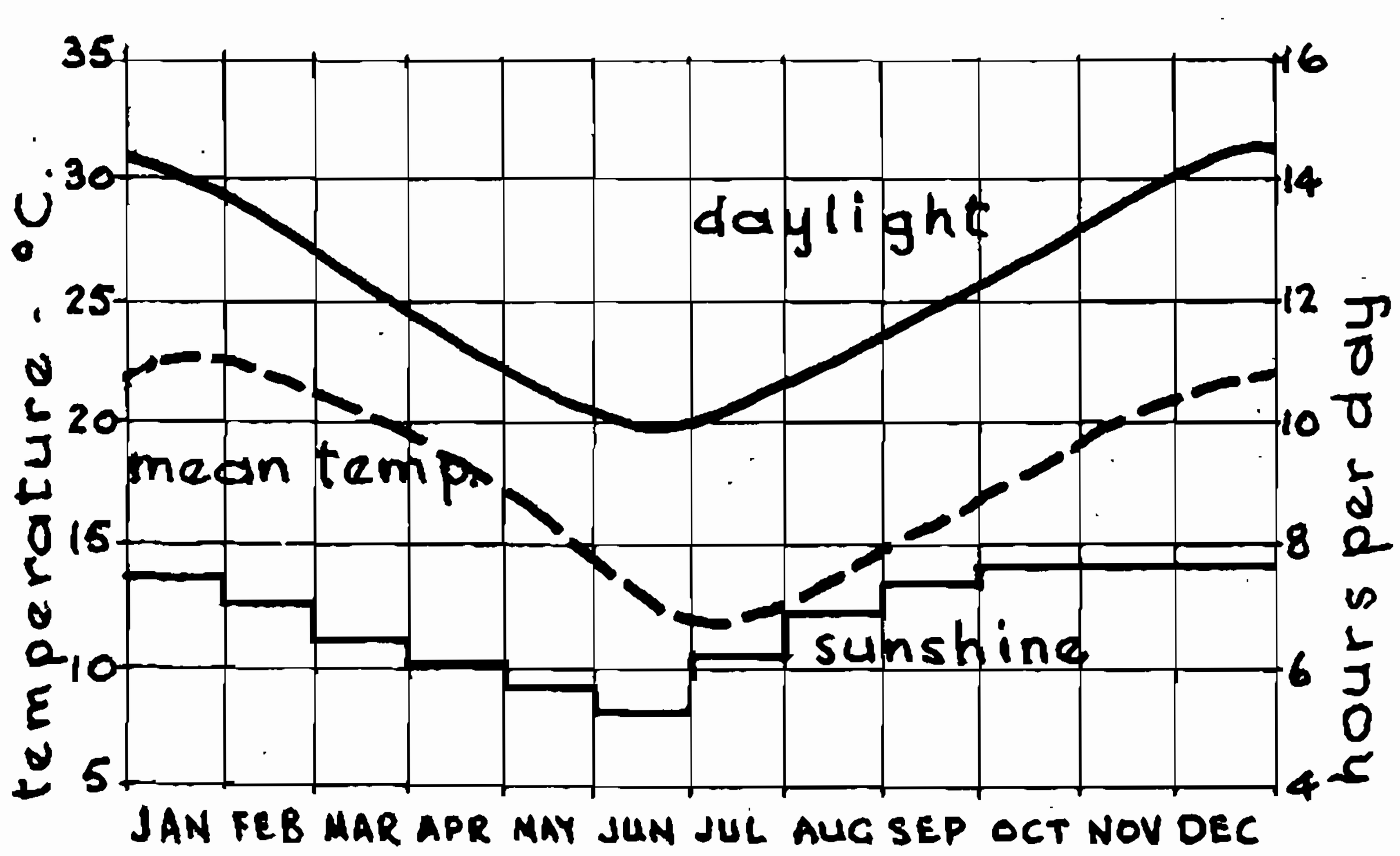

Fig. 2 Daylight, sunshine and temperature in Sydney.

3 UNDERGROUND THERMAL CONDITIONS

The locating of living spaces underground, ipso facto, removes them from the vagarity of surface weather. If this were the only benefit to be sought there would be no reason for going underground (except for factors proposed by Utudjian and mentioned above), for this scale of benefic can be achieved in surface shelters. Underground shelters possess a far greater degree of thermal inertia than can be exhibited by any surface shelter.

(a) Underground effects of surface variations

Muncey (Ref. 17) defines thermal inertia as the property of structures to resist fluctuating thermal change. This is shown to be due to the massiveness of the structure and the period of fluctuation. The mass properties concerned are thermal resistance and thermal capacity whose product gives a time constant:

$$R.C = t_c = \frac{x^2}{\kappa} \qquad (1)$$

where x is the thickness of the material and κ is its diffusivity, (Ref. 18),

($\kappa = \frac{k}{c.\rho}$, where k = thermal conductivity, c = specific heat and ρ = density of the material), (Ref. 19).

It will be seen from Equation 1 that for a homogeneous material the time constant varies as the square of the thickness and that the time constand relates physical properties with dimension.

In a regularly varying heat flow there is an impedance resulting in varying degrees of temperature amplitude decrement and time-lag, dependant on time constant, thickness and frequency of variation as may be shown also on the electrical analogue computer (Ref. 20).

Due to the earth's mass properties, the effects produced by this impedance may be summarised in Table 1 (Ref. 21).

The process is one in which mass by its physical properties damps-out, or "averages", thermal variations and in the process delays the transfer of thermal changes. Fig. 3 illustrates the effects of mean, annual surface insolation (as air temperature) on the temperature at depths underground. (Ref. 22).

It should be noted from Fig. 3 that falling temperature gradients from the surface indicate heat flow into the ground in Summer and vice versa in Winter. At depths down to about 1 m. the diurnal variations are damped out; at 3 m. the effects are those produced by long spells of constant weather but at 8 m. and below only the seasonal effects may be observed, reduced to less than 1° C variation and with 6 or more months time lag.

This almost constant temperature represents the annual mean at the surface, i.e. the annual isotherm.

(b) Geothermic heat.

The heat loss from the cooling of the earth results, at present, in an average temperature gradient of 1° C per 33 m. near the surface. (Ref. 24). Although this effect is small, highly variable and dependant on tectonics (geol.), it may be a determining factor in the optimum depth of shelters

TABLE 1.

TEMPERATURE VARIATION, AMPLITUDE AND THE LAG RELATED TO DEPTH FROM THE EARTH'S SURFACE.

Depth below surface	Period of temperature wave			
	Diurnal		Annual	
	Amplitude	lag	Amplitude	lag
0.00 (Surface)	1.0	0.0	1.0	0.0
5 - 10 mm	0.9	0.25	1.0	0.0
50 - 100 mm	0.3	3.0	0.99	48.0
1m	No significant effect.		0.5^+	3 weeks$^+$
6m				14.5 weeks$^\times$
8m			0.03^+	20.5 weeks$^+$
21.6m (70.8 ft)				1 year$^\times$

Amplitude = 1.0 at the surface
lag in hours (except as stated)

+ after Carson (c.f. Riehl, pp. 264-5)
x after Kelvin (c.f. Small, P. 17)

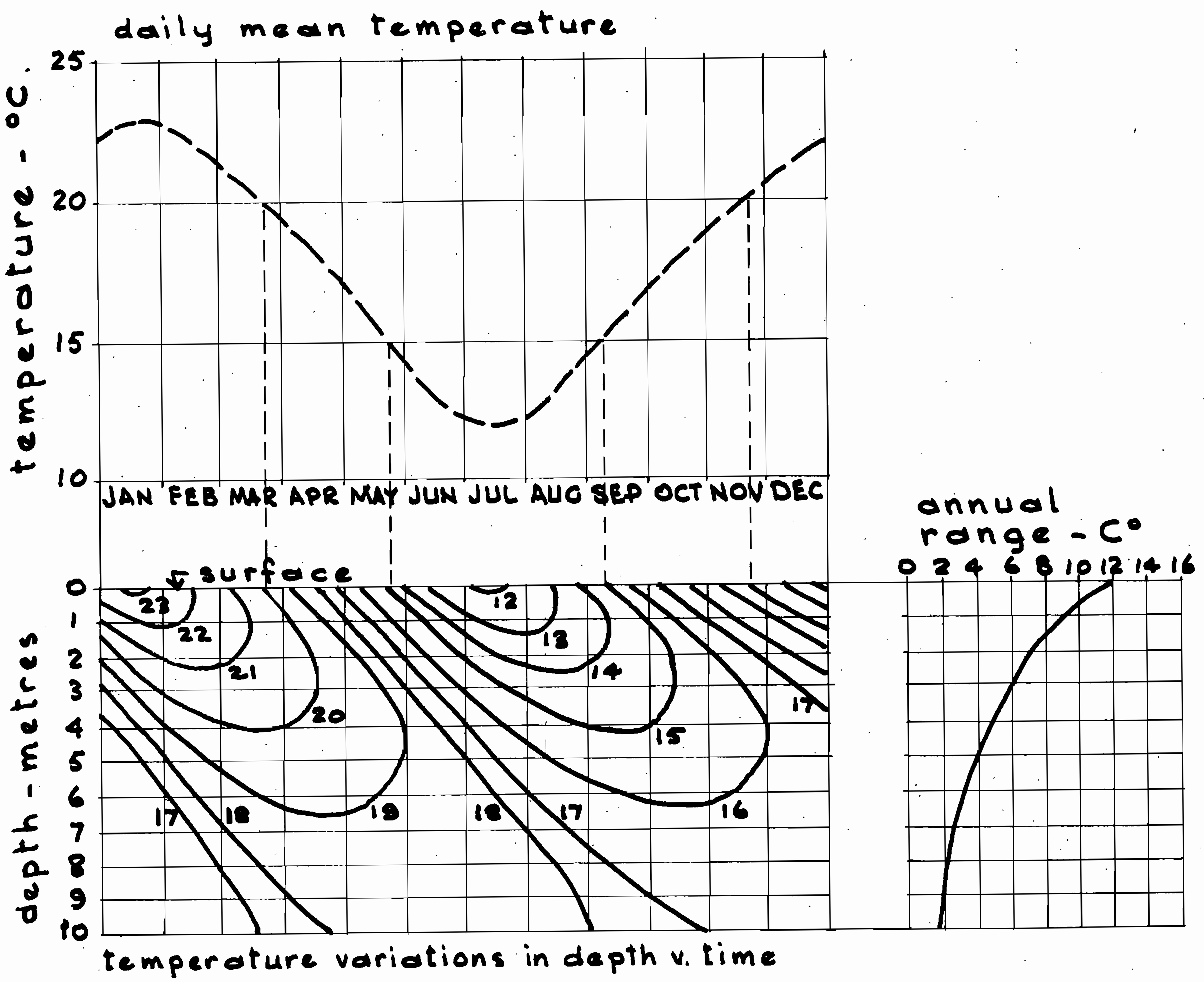

Fig. 3 Typical temperature gradients in the ground (exterpolated from Riehl (Carson) and Kelvin).

in specific localities and with reference to the feasible, economic introduction of artificial heating.

4 UNDERGROUND COMFORT FACTORS

Although the undisturbed underground situation may present conditions of uniform temperature, requiring little modification, the establishment of subterranean living quarters will disturb these conditions and generate problems in the achievement of human comfort. These problems are discussed and conclusions made here, in the light of four factors of comfort already mentioned (Para. 28).

(a) Air temperature

The air temperature in a cavity or gallery would be derived from the surrounding earth mass. But human occupation requires replacement of the air ventilation. Now, ventilation is a short-circuiting of mass thermal properties and the introduction of immediate, "high" frequency surface conditions. However, the configuration of the inlet and exhaust air routes could permit heat exchange and reduce the difference somewhat.

Heat would be contributed, also, to the gallery by human, lighting and mechanical heat production. The very fact that the air has to be moved requires work to be done on it, thereby raising its temperature. The result of these factors, as with surface shelters, would be a nett heat flow from the gallery because human activities, in the main, generate heat.

The air temperature will be governed, to some extent, by the rate of heat exchange with the earth mass. Thermally insulating coverings of low density will have the effect of promoting rapid changes of temperature (Ref. 25) and is to be frowned upon, whereas more direct thermal connection of the surface to mass proper would ensure the damping of transients.

(b) Humidity

The underground environment in most locations is damp. Relative humidity is determined by vapour pressure and temperature. The lower underground temperatures experienced in relation to instantaneously higher surface temperature (with high humidity) in summer might result, due to ventilation, in condensation on surfaces thermally well-connected to the colder earth mass. But this is a limiting case and the effect could be absorbed in the supply system by heat exchange with the earth mass.

Higher values of Relative humidity will compensate for lower temperatures in assessing comfort (Ref. 26).

(c) Air movement

This factor deals with the motion of air past the body thereby removing heat, particularly by the evaporation of moisture. It is not connected with ventilation. McGuinness and Stein (Ref. 27) indicate that it is only at air temperatures approaching skin temperature when other modes of body heat rejection become difficult, that air movement becomes increasingly important. Vigorous air movement, also, is a simple way of restoring comfort during strenuous exertion, as Angus shows (Ref. 28).

(d) Radiation exchange

Surfaces in good thermal contact with the earth will be at a fairly uniform temperature day-and-night and throughout the year. The body's heat loss by this route will be reasonably constant depending mainly on the proportion of the skin not covered by clothing.

High illumination levels may be a benefit.

(e) Underground comfort balance

The balance of these factors to achieve comfort will be different underground from what is more usual on the surface. While air replacement and, hence, air temperature will still be the most important factor it will be less so than in surface shelters due to the increased effect of the other factors. If the temperature of incoming air can be stabilised by heat exchange methods then the factors of air movement and radiation exchange gain in importance.

Since both air temperature and surface temperature (radiation exchange) will depend for their effectiveness on the heat storage of the surrounding earth mass it will be necessary to set limits to the proximity and volume of galleries. The effects of upper galleries on those below would also require investigation and regulation.

5 CONCLUSIONS

Buildings are built in the surface tradition on the assumption that energy for making materials and for building and energy to combat the exposure to the elements will always be available. When a surface shelter has ceased to be useful or economic it is pulled down and the cycle repeats, with less economy and more energy. An underground space cannot be "pulled down". Renewal becomes a matter of up-dating and development a case of extension.

There is a need to "prove" underground shelters, i.e. all aspects, particularly thermal aspects, must be measured and assessed so that the benefits of the situation may be exploited without over-stressing the possibilities. There are so many variables in the art of finding protection for our ways of living that the best answer, probably, is to try by testing a full-size, real-time scale model. In doing so, attention should be directed towards the assessment of limiting criteria for the following so that advantage may be taken of the underground situation.

(a) The percentage of underground volume occupied in relation to the regulation of temperature afforded.

(b) The effects of surface finishes and their thermal connection with the earth mass.

(c) The regulation of ventilation to reduce the incidence of variations from which shelter is sought.

(d) The illumination and power expenditures both as consumers of energy and as contributors to the underground energy balance.

(e) The concentration of people with their heat output (sensible and latent).

(f) The zoning of activities - living, education, production, recreation spaces - in relation to the surface (and each other), for degrees of thermal control.

Whilst cities built under transparent domes would have serious thermal faults (such as greenhouse effect which would require vast quantities of energy to correct), they possess the advantage of a popular visual image - the super colossal, heavenly-perfect tent!

Underground cities lack an image and they will not be considered seriously until one is created and popularised in the comic strips and paper-backs of science fiction.

The image generated from purely thermal considerations is of inter-connected spaces disposed three-dimensionally within the solid earth; of shafts, "roads" and galleries where the hollowed space is the activity space. There will be no question of excavating a huge cavern and covering its floor with buildings and streets, as in a domed city. Since there will be no weather, there will be no need for facades to exclude it; since there would be no need for vehicles, there would be no need for streets. We shall have moved from the strife and bustle of Nelson's "monuments and bell-jars" (Ref. 29) to the enjoyment of the continuous interior.

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Urban Highway Tunnels

by

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1 INTRODUCTION

The traffic problems of the world's cities continue to increase. With the world's impending oil shortage, there is perhaps a danger that people will delay highway development on the assumption that "there won't be any cars in fifty years' time". There are various reasons why this is a highly unlikely proposition. For one thing, the future shortage of oil is largely promulgated by assumptions of increased consumption. As oil becomes more highly valued, the price mechanism will ensure that it is used where it has particular advantages - and road vehicles will surely be higher in that particular league than static power plants: that is unless and until vehicles driven by other means such as batteries become more viable. Whichever happens, highway development will remain essential.

It is interesting to consider what would have happened if the planners of the last century, faced with the urban traffic chaos caused by horse-drawn vehicles, had delayed highway work on the brilliantly correct assumption that "there won't be any horses in fifty years' time".

The recent history of road planning in London is instructive. Studies revealed a clear need for a new network of freeways in the Greater London area, despite the existance of an extensive and efficient public transport system. Everyone agreed that the freeways would improve the quality of life by removing frustrated through traffic from the existing roads, and the economic return was adequate.

However, when the "motorway plan" was developed and published, a mounting wave of resistance substantially prevented its implementation. Some of the criticism is selfish or illfounded but there is also a solid core of resistance based on experience or existing urban freeways. Many communities have been partly demolished and split by surface roads and as to elevated highways, the Hammersmith Flyover is now closed at night to allow people to sleep.

The London plan of the 1960's proposed no tunnels except at two river Thames crossings. It was assumed, correctly, that the construction cost of highway tunnels would be several times that of equivalent surface roads.

During 1973 the Greater London Council commissioned a study (Ref. 1) to investigate whether tunnels could play a larger part in their highway network, and the conclusion was affirmative. Others have also urged that the time is ripe for more subterranean roads and development (Ref.2).

2 THE NEW SITUATION

(a) The value of land continues to rise; so much so that for urban motorways, the costs of acquisition and compensation are now often of the same order as the cost of construction.

(b) The introduction of tunnelling machines capable of boring increasingly hard rocks, and of clay digger shields and other devices, together with more economical designs of tunnel linings, are reducing the real cost of tunnelling.

Until quite recently tunnelling was more a traditional craft than a science, for good historical reasons. Even now, with advanced methods of soils investigation and geological assessment, and the increased use of such techniques as finite element analysis in design, there properly remains a reluctance to say just what can be done safely and economically without confirming the assessment with actual tunnelling experience.

Thus in the Silurian rocks of Melbourne the extensive sewer tunnelling of three to four metres diameter led to recognition that it would be possible and sensible to construct six or seven metre diameter tunnels for the Melbourne Underground Rail Loop. The construction of the Loop will doubtless produce many problems, but already in the construction of junction chambers and stations it is becoming clear that it is not absurd to think in terms of driving ten metre diameter road tunnels in Melbourne. Without the earlier experience, such a conclusion would have been foolhardy no matter how many boreholes and calculations were produced to justify it.

Nevertheless, in the world's cities the current tunnelling programmes are revealing these new possibilities, and the factors mentioned earlier are lowering the real cost of tunnelling.

(c) Perhaps the most potent factor favouring road tunnels is the increased appreciation of and resistance to the environmental effects of surface and elevated urban highways both during and after construction. The situation has been reached in many cities in democratic countries where the construction of badly needed roads will just not be tolerated by the local inhabitants.

This is the field in which it is difficult or impossible to put a price against the disadvantages of the surface solution. Compensation arrangements in some countries now include free sound-proofing and double-glazing, compensation for "injurious affection" and various other grants. But what price a church which is no longer conveniently sited for its congregation? The cost to the community of delays and disruption during construction can be estimated, but seldom are. The lasting effects of noise, pollution, visual intrusion and severance caused by a new surface highway are almost impossible to quantify in financial terms, but informed public opinion is undoubtedly putting an increased value on them.

3 THE TUNNEL SOLUTION: GEOMETRIC STANDARDS

(a) Traffic space

The urban highway tunnel should accommodate carriageways constructed to those standards of geometry, clearance and sight distance found necessary for roads built at ground level for a similar design speed.

In Britain a recommended width of 7.30m for unidirectional 2-lane carriageways and 11.0m for unidirectional 3-lane carriageways has been adopted for urban roads with a minimum vertical clearance of 5.1m. Based on the study of traffic movement in a number of British road tunnels, the corresponding traffic capacity for these standards adopted in a tunnel should be about 4900 vehicles/hour and 6200 vehicles/hour respectively.

To provide paved verges or laybys in tunnel would considerably increase the size and cost of the tunnel. Although statistically the measure of incidences of breakdowns in tunnel is similar to the surface road, a number of 2-lane tunnels as long as 2½km are operating satisfactorily without a breakdown lane or laybys but these tunnels do have facilities for rapid removal of broken down vehicles. With a 3-lane tunnel the problem is correspondingly reduced and breakdown lanes may not be considered necessary in tunnels; but it would be desirable to provide laybys at intervals of 1½km to 2km whenever practicable in very long tunnels.

A minimum horizontal clearance of 1.0m for all heights above the carriageway is recommended in current British standards for urban roads, but whilst it remains unproven that reduction in lateral clearance would cause a reduction in safety or in the maximum capacity, a clearance of 0.6m behind the kerb is considered an acceptably safe standard for urban highway tunnels.

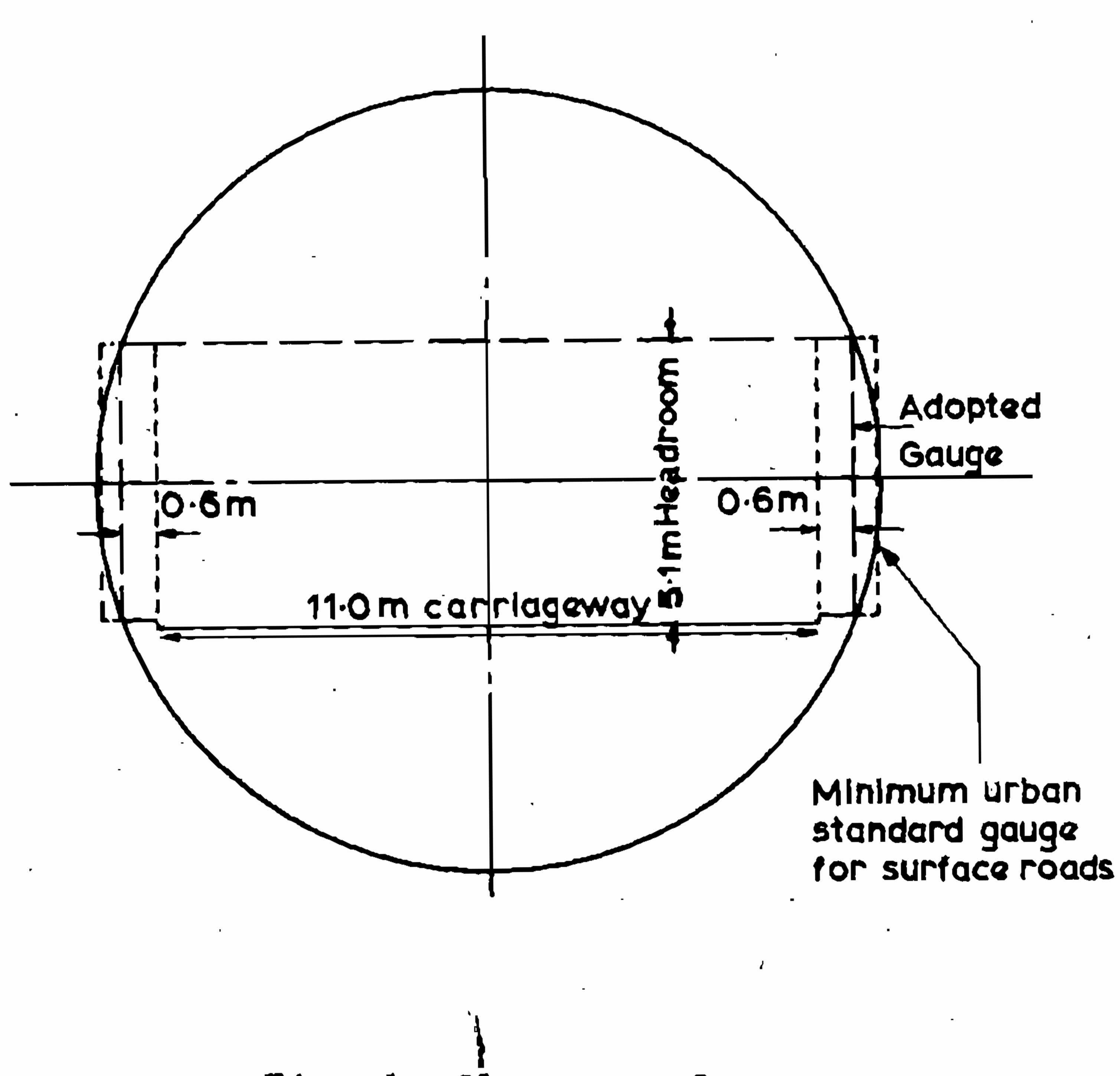

Fig. 1 Clearance Gauge

Adopting these standards the tunnel shape is then required to accommodate a "traffic space" as shown on Fig. 1, and defined as a rectangle formed by the overall width of carriageway (multiples of 3.65m traffic lanes) plus lateral clearance of 0.6m on each side and the height of 5.1m above the carriageway.

Although these can be considered desirable standards, many tunnels are operating satisfactorily with reduced lane widths and clearances. Even small changes in tunnel size particularly in unfavourable ground conditions can mean significant differences in cost and by way of example, Fig. 2 indicates the variation in cost as percentage differences from the adopted gauge for different combinations in a 2-lane circular tunnel.

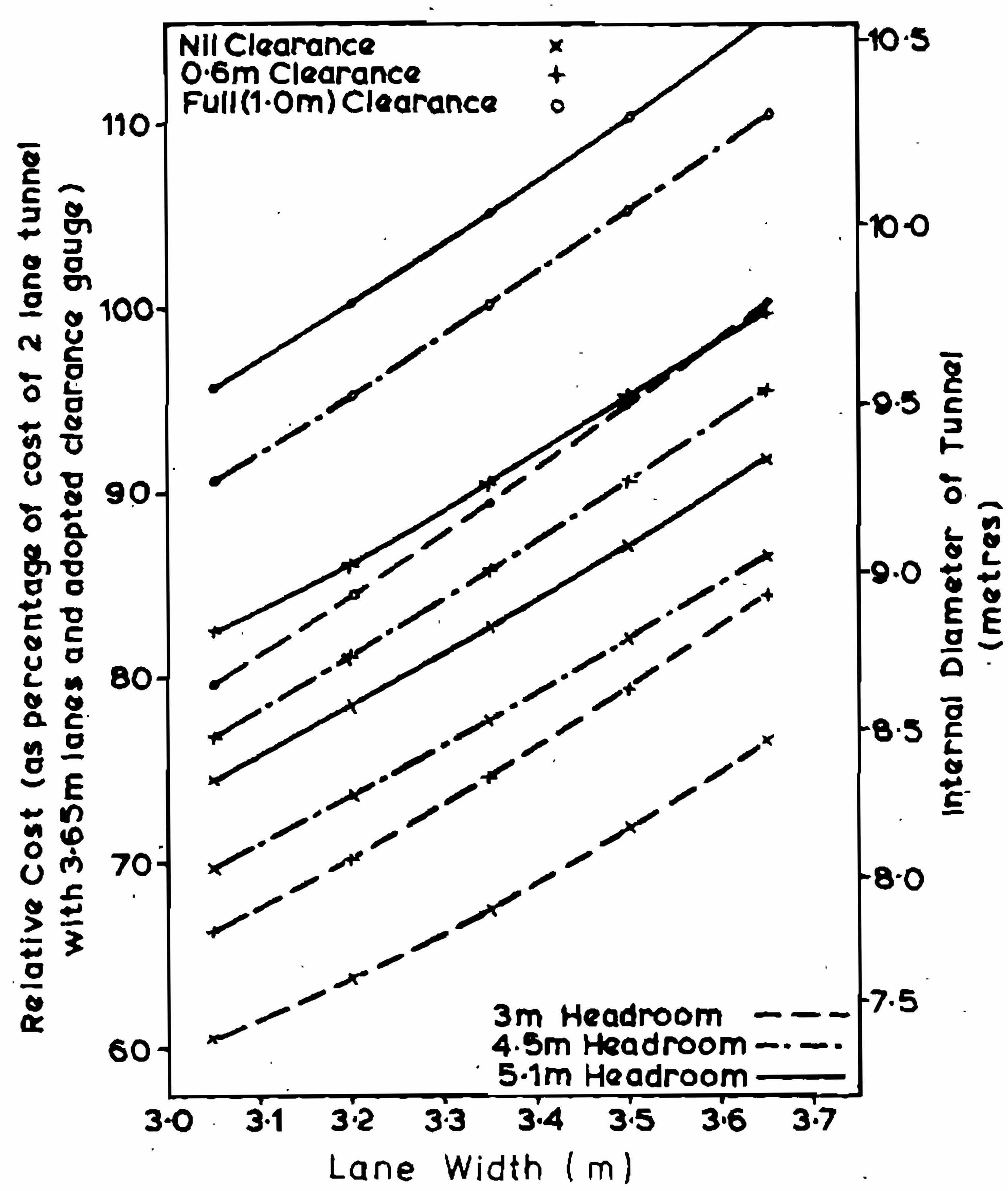

Fig. 2 Cost Difference for Change in Clearance Gauge

(b) Ventilation space

Artificial ventilation is considered essential for an urban road tunnel longer than about 400m and consideration must be given to the provision of space for air ducts.

Longitudinal ventilation induced by the piston effect of the traffic flow boosted by fans outside the traffic space can provide sufficient air speeds for dense traffic in tunnels not longer than 1km without provision for air ducts and in circular tunnels or tunnels with an arched roof adequate space is available outside the traffic space for mounting fans and equipment.

For tunnels longer than 1km semi-transverse or fully-transverse systems of ventilation are required. In both systems longitudinal ducts of substantial size are usually required with blowing fans to distribute fresh air uniformally along the tunnel. Whilst in the semi-transverse system vitiated air is extracted from the traffic space at widely spaced intervals, the fully-transverse system additionally requires substantial longitudinal ducts into which the vitiated air is drawn at close intervals along the tunnel.

The circular 2-lane tunnel is acceptably efficient in that the void outside the traffic space is usually no more than adequate for ventilation purposes in long tunnels and is therefore an efficient shape in all ground conditions. Circular tunnels to accommodate more than two lanes may become less efficient in the use of the void outside the traffic space and departure from the fully circular shape can be considered where ground conditions allow this to be an economical alternative, Fig. 3.

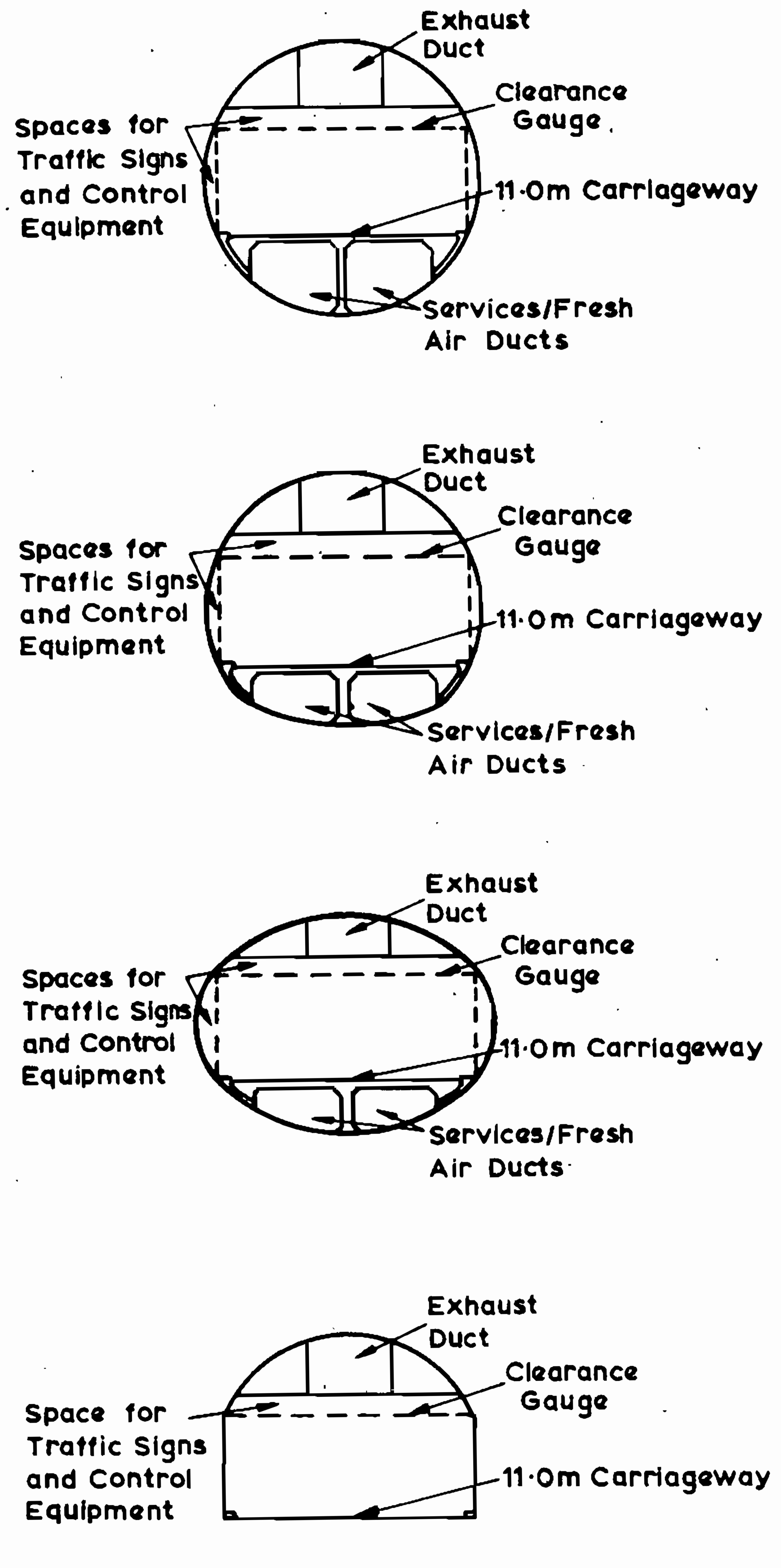

Fig. 3 Alternative Cross Section

(c) Intersections and junctions space

Joining two 2-lane tunnels requires a certain minimum distance between them and over a short distance the merging or diverging route must be accommodated in a space wider than four lanes. Such junctions would be very difficult and costly to construct by tunnelling methods, but by arranging for them to be constructed by cut-and-cover methods at major work sites, they provide many incidental advantages, including access for tunnel construction, assembly of shields or moles, and finally, incorporation of permanent major ventilation stations and other services. In addition to these chambers, a widening to provide an additional lane to the through route is required for acceleration or deceleration in the region of 240m long and 90m long respectively.

As a generalisation it is preferable that the need for tunnels larger than for three lanes should be avoided by siting intersections so that cut-and-cover or open-cut construction can be used.

4 THE TUNNEL SOLUTION: TUNNELLING TECHNIQUES

The alternatives in shape of tunnel to accommodate the required traffic and ventilation space follow from consideration of the type of ground and construction techniques available.

Although cut-and-cover construction for shallow tunnels and the use of rectangular sections may be economical where the land above the tunnel is relatively free of services and buildings, this is very often not the case in the urban situation. Cities that have embarked on cut-and-cover construction for rapid transit schemes have experienced severe traffic congestion during construction and much damage to services and buildings.

For driven tunnels over the past ten or fifteen years, there has been rapid improvement in tunnelling techniques and economy in lining design.

(a) Tunnelling in soft ground

The principal feature of excavating within the protection of a shield is likely to remain the safest and most economical technique in soft ground. Full-face rotating or part-face cutting heads can be mounted within a shield with the potential for rapid excavation.

Although with very few exceptions shields are circular in shape, there is no reason in principle why these cannot be of flat bottomed or near elliptical shape if economy in road tunnel shape will result. The development of expanded concrete block linings has substantially reduced the cost of lining tunnels in favourable soft ground.

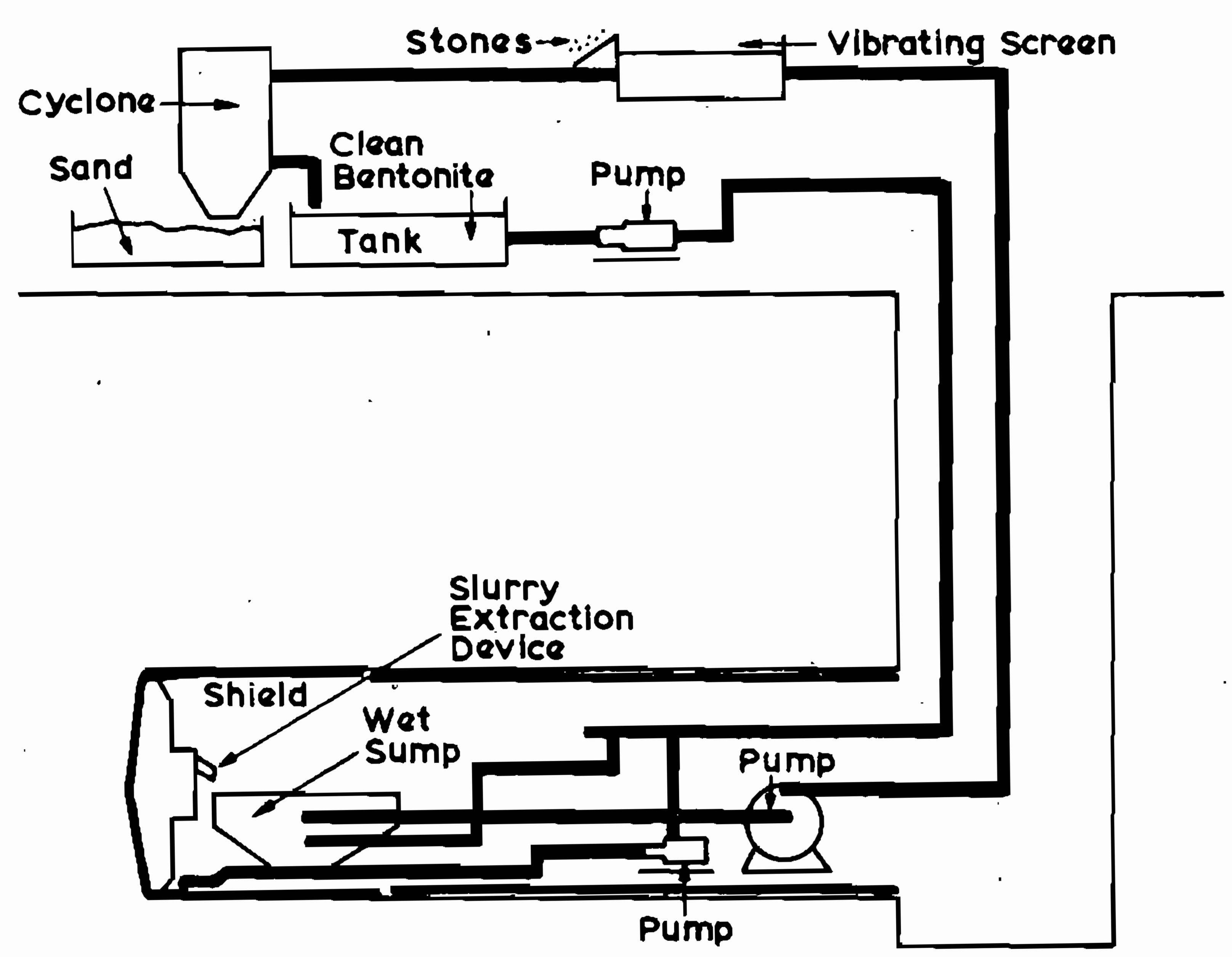

Fig. 4 Bentonite Shield Process

The development of various tunnelling machines which combine face support with mechanical excavation now offer comparable economies in unstable ground. Machines with adjustable face plates have been successful in some circumstances and the bentonite shield method has worked well in water bearing sands and gravels Fig. 4.

In mixed faces, e.g. a combination of clay, silt, and sand or gravel in the presence of water, tunnelling is likely to remain very difficult and expensive unless new techniques become available. The costs rise steeply to the extent that it is necessary to use available techniques of compressed air working, ground water lowering, freezing or the wide range of chemical treatments of the ground.

(b) Tunnelling in rock

This subject is very adequately covered in other papers at this conference and elsewhere. Suffice it to say that the continuing development of "moles", of drill and blast methods, of grouting techniques, rock-bolting and of sprayed concrete are all contributing to a reduction in the real cost of construction. It is increasingly possible for major rock tunnels to be driven under cities with no important adverse effect on the population. Fig. 5.

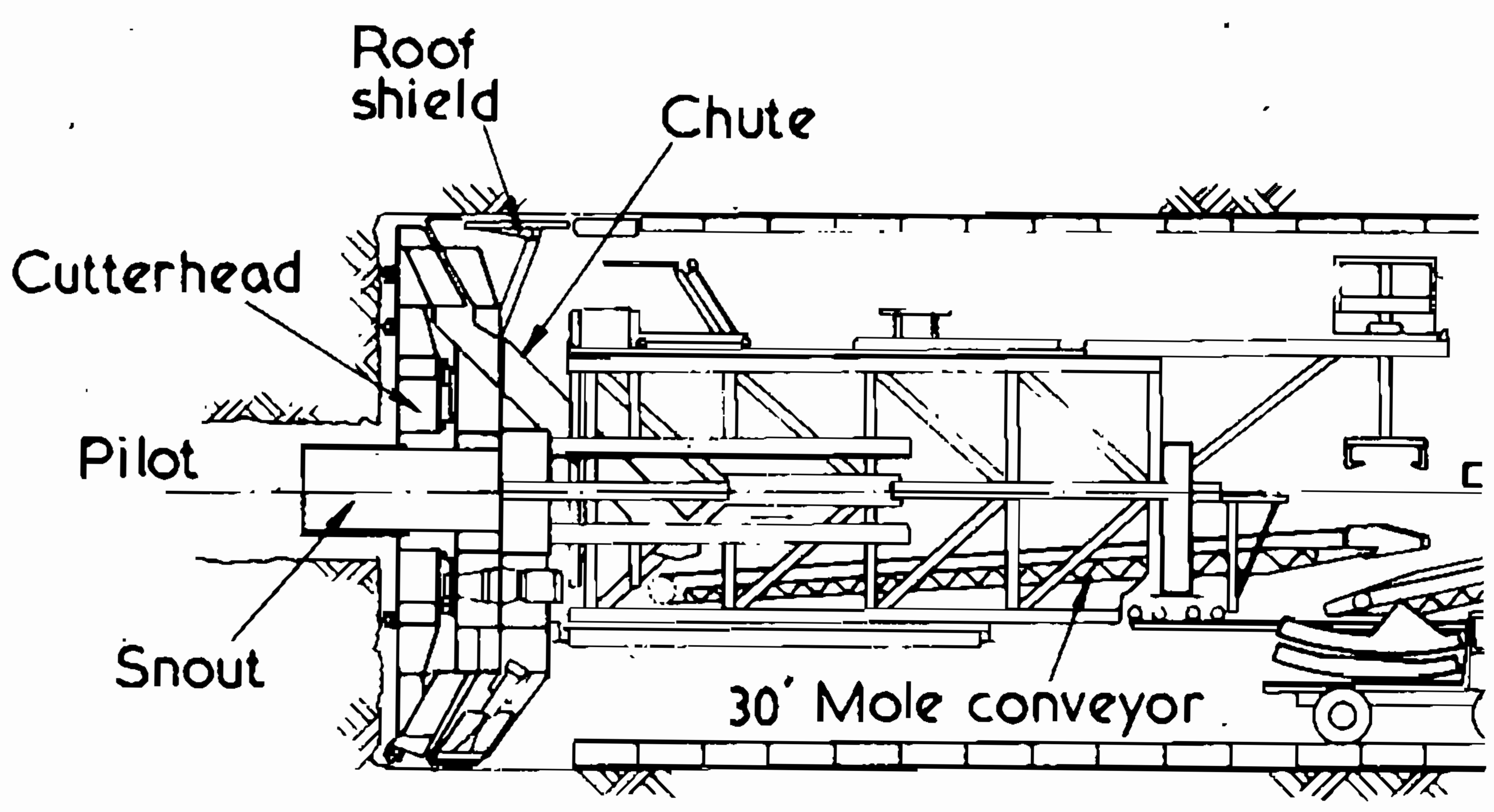

Fig. 5 Typical 10m dia 'Mole' in Segmental Lined Tunnel

5 THE TUNNEL SOLUTION: GENERAL CONSIDERATIONS

Having established the geometric requirements of road tunnels and briefly reviewed current tunnelling techniques, how should these be applied?

After World War II, the Americans led the world in highway engineering. Their great interstate highways have become a model which was not only applied in other countries, but also coloured the thoughts on urban freeways. If the main freeways in a city are to be pushed through on the surface, then there is every advantage in concentrating the difficulty to a few superhighways, six to twelve lanes wide, all lanes built simultaneously. This has been the pattern of recent urban highway development, the most difficult decision of all being which route to take.

If tunnels are to play a substantial part, planning strategy must be revised. For example, when the new intercity highway reaches the outskirts (and property values rise), instead of feeding the traffic into ring-roads, or carrying new surface highways into the heart of the city, it may be better to drive tunnels to selected destinations where existing distributer roads can best cope with the additional traffic.

Road tunnels will increase the ability to allow the organic growth of a city without spoiling its environment, or demolishing valuable property and historic buildings, and at the same time, meeting the reasonable desires of preservationists. One advantage of road tunnels is that they allow a planned phased investment in urban road development. If additional highway capacity is required along a particular route, a single tunnel can be taken through. It might be a one-way 2-lane tunnel used in conjunction with a surface one-way system. Provision can be made for future duplication without sterilising adjacent property. Indeed, the second tunnel if built, might be better placed some distance away to meet some special or changed requirement or to take advantage of some geological feature.

Unless tunnel solutions are carefully considered at the planning stage, the best will not be made of the opportunities they offer. On the one hand, tunnels can be aligned free of most of the restraints which apply to surface and elevated roads, but on the other hand, geological and topographical features must be taken into account in an entirely different way. A hill, far from being an obstacle, may be the means of taking through traffic between cheap surface inter-sections.

6 COST COMPARISONS

The cost of a highway tunnel depends principally on the type of ground and groundwater conditions encountered, but will also depend upon the ventilation requirement. The other costs, road decks, architectural finishes, etc. will vary but without affecting the total cost significantly.

Taking the ideal case of an easily excavated but homogeneous self-supporting rock as a base figure, the following table indicates a range of multiplying factors for the relative cost of primary construction in progressively more difficult ground conditions. Fig. 6.

Ground Conditions	Competent Soft rock	Impermeable Soft ground	Fissured rocks
Type of Lining	Nominal No water problem	Expanded Concrete	Insitu Concrete
Cost Ratio	1·0	1·1 – 1·2	1·2 – 1·6

Ground Condition	Mixed Soft ground	Sands/gravel (Bentonite Shield)	Unstable Soft ground (Comp. Air)
Type of Lining	Bolted Concrete	Cast Iron	Cast Iron
Cost Ratio	1·6 – 3·0	2·3 – 2·6	3·0 – 4·0

Fig. 6 Primary Construction Cost Ratio's (Excavation and Structural Lining)

Isolated special difficulties requiring for instance freezing or extensive underpinning, might locally increase the factor to as much as ten times the base cost.

The time was when each tunnel was an adventure, the final cost of which could hardly be estimated at the outset. Recent experience is that with thorough soils investigation and careful study by experienced engineers, estimates for the cost of tunnels have proved to be as accurate as those for other types of construction.

Primary construction (excavation and structural lining) will account for more than half the overall cost of any road tunnel and may vary as indicated above between the most favourable and the most difficult ground conditions. These variations demonstrate that there are considerable advantages to be gained from following a route through the most favourable ground whenever possible.

Ventilation costs are the next most important variable. A longitudinal system previously referred to but limited to tunnels not longer than 1km, is the most economical to instal and to operate, but even when it is desirable to provide a fully transverse system, this is unlikely to represent more than 15% of the total cost. The cost of secondary construction (road deck, finishes, tunnel services and lighting) is predictable and will not vary much between schemes for a given length of tunnel.

For tunnel schemes, land costs are relatively small and unless a scheme includes more than a minimum of cut-and-cover construction, the land required for approach ramps and ventilation buildings is unlikely to add more than 5% to the total construction costs. Cut-and-cover construction usually allows the land to revert to profitable or desirable use on completion of construction.

The recently completed Kingsway tunnels and associated roads at Liverpool in Britain, comprising twin 2-lane sub-aqueous tunnels approximately 2.5km long and the equivalent of about 13km of dual 2-lane highway at and above ground to urban motorway standards, has provided the opportunity to compare the construction costs of urban highways above and below ground within one scheme.

Although actual costs are constantly changing these cost differences are considered to be typical and have been used to compile Fig. 7. This attempts to show as a first approximation how the cost of underground construction may compare with the surface or elevated road for a range of urban land costs. The Kingsway tunnels are an example of tunnelling in competent soft rock but requiring a segmental lining to combat ground water difficulties. Fig. 7.

Description	Relative Constr. Cost	Relative Construction & Land Costs		
		$A 0.25M/ha	$A 0.5M/ha	$A 1.0M/ha
Road at surface	1·0 *	1·0	1·0	1·0
Road on viaduct	7·0 *	2·1	1·6	1·4
Road in retained cutting	2·5 *	1·3	1·2	1·1
Road in tunnel				
good soft ground	12·0	2·3	1·3	0·90
rock (with lining)	14·0	2·7	1·5	1·05
subaqueous rock	17·5*	3·4	1·9	1·3
waterbearing sands/gravel	26·0	5·0	2·8	1·9

* Kingsway tunnels

Fig. 7 Dual 2 Lane Urban Motorway Cost Ratio's

This table is based on the Liverpool work and it would be wrong to attempt to apply it in other specific cases. It is also a gross oversimplification; for instance it favours the "surface" solution if a scheme can be designed entirely on the surface, whereas any modern urban highway would entail frequent grade separations and in practice is therefore over substantial lengths elevated above ground. It does, however, serve to illustrate how the land costs affect the overall result.

The broad conclusion to be drawn is that if land costs for a highway scheme are of the same order as construction costs, then a very careful look at tunnelling alternatives is merited. If geological and topographical conditions are suitable the tunnel solution may now be justified on straightforward economic grounds. If the environmental penalties of a surface solution are high, then tunnelling may be justified even if geological and topographical conditions are not particularly advantageous.

7 CONCLUSIONS

Increased use of tunnels in urban highway networks would have the following advantages:-

1. Less land purchase, demolition and compensation.
2. Less disruption during construction, the isolated working sites being of limited area and carefully positioned.
3. Better alignment of highways, freed from many constraints applicable to surface or elevated roads.
4. Phased investment as it is not so necessary to construct initially to meet the full forecast traffic needs.
5. Improved environment, reducing visual intrusion, severance, noise and pollution.

The previously universal disadvantage of the higher cost of tunnelling is now disappearing, and has disappeared where good tunnelling conditions exist, and property values are high.

There are already several hundred kilometres of highway tunnels in the world, mostly used at full capacity much of the time, and demonstrating that they are a practical proposition for heavy traffic.

From the road user's point of view, driving in a tunnel is not as pleasant as his idea of "the open road", but if it offers him a quicker, safer journey, there is no doubt that he will prefer it to driving in congested city streets.

From the point of view of the rest of the community, the advantages of transferring traffic from the surface streets into the regulated environment of highway tunnels are very considerable.

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Reshaping India's Capital City—Proposed Underground Railway

by

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SUMMARY. Over last 3 decades, Delhi's population has experienced phenomenal 422% growth, its vehicles increased 20 - fold. Existing road-based transport system is totally inadequate to meet transport needs of estimated 5.7 million people in 1981. Rapid transit system including 36 Kms. of underground/elevated railway is, therefore, planned. The assessment of travel demands and techno-economic feasibility studies comprising of soil investigations, system design, methods of construction, costs and benefits are described. Rectangular tunnel box and cut and cover method are considered suitable. The proposed underground railway would help reshape the development and land-use pattern and afford much-needed relief to transportation problems. The capital-intensive system may not seem apparently justified financially, but an integrated view including savings in land costs and social benefits makes it viable.

1 HISTORY & GROWTH OF DELHI

The origin of the city of Delhi has been traced back to the epic period of Mahabharata (1000 B.C.). The real growth of Delhi, however, started after 1911 when it was declared as the Capital of British India. With the dawn of independence in 1947, Delhi the capital of the second most populous country became an important national and international centre on the world map. The changes in its population and economic structure since then have been phenomenal. The pressure of population and force of functional economy ushered in years of sustained and accelerating growth and development. The rapid growth of the city enveloping an area of 435 Sq.Km. brought many problems in it's wake - socio-economic, physical and administrative. Over the last three decades while the national population increased by 60% and urban population by 136%, Delhi's population has experienced phenomenal growth of 422%(Table I) thus making it the fastest growing city in the whole of India & perhaps in the world.

TABLE I

POPULATION GROWTH IN DELHI

Year	Population	Percentage Growth over preceding decade.
1901	208,575	-
1911	232,837	11.63
1921	304,420	30.70
1931	447,442	46.98
1941	695,686	55.48
1951	1,437,134	106.58
1961	2,359,408	64.17
1971	3,629,000	53.80

The population envisaged for 1981 is 5.7 million and 9.7 million in 2001. This high rate of growth has resulted in overstressing City's resources due to increased demands and the strains of growth are seen in all sectors including transport.

2 THE TRANSPORT PROBLEM

(a) Existing Pattern

The movement within the city has become a task by itself for the average citizen. The phenomenal increase of motor vehicles on the road (which have multiplied 20 times during the last three decades) compared with the limited increase in the capacity of road net-work has resulted in road congestion and long delays, further reducing effective capacity of the system. With 215,703 motor vehicles; 600,000 cycles and 16,538 slow moving vehicles, travel times on some roads of the city have increased upto 58% between 1963 and 1969. Road safety has also suffered, the number of accidents having increased from 1127 in 1956 to 7192 in 1970.

At present Mass Transport System is provided only by buses. The modewise distribution of intra-urban trips is indicated in Table II.

TABLE II
MODEWISE DISTRIBUTION OF INTRA URBAN TRIPS

Mode	Percentage Trips	
	1957	1969
Bicycles	36.0	28.01
Mass Transport(buses)	22.9	40.80
Private Cars, Motor Cycles & Scooters.	11.1	23.77
Hired Modes	13.6	5.23
Slow Vehicles.	16.4	2.19

Due to the restricted availability of private cars and the low income level, virtually all the increase in transit trips has to be borne by cheaper means of mass transport. This has become all the more imperative on account of the oil shortage and energy crisis.

(b) Assessment of Future Travel Demands

To determine the intra-urban travel demands and pattern of movement, the urban area of Delhi was divided into 57 zones. 12,100 households out of 602,000 were interviewed to collect the data regarding origin, destination, purpose, mode of transport, time of trip, fare paid, waiting time etc. of all the trips performed on the previous day as well as particulars of activity status, income, size of family, age, sex and number of vehicles owned. The survey data was systematically analysed with the help of electronic computer. Multiple regression analysis was then carried out to determine the future travel demands. It was estimated that for the year 1981, 4316921 trips with an average trip length of 12.16 Kms. would have to be catered for by the Rapid Transit System.

(c) Identification of Proposed Corridors

To meet these travel requirements, a network of 135 Kms. of Rapid Transit Railway System including 36 Kms. of underground/elevated railway & 99 Kms. of surface system (along the existing railway network) was identified as shown in Figure I.

The East-West and North-South underground corridors meet at the centre of the prestigious shopping-cum-business centre of the Capital-Connaught Place. The interchange station is also planned to provide an underground shopping arcade at the mezzanine level.

3 SOIL INVESTIGATIONS

As a part of the techno-economic feasibility studies, soil investigations were conducted along the proposed routes of underground/elevated corridors. The work comprised of field boring, sampling, field and laboratory testing. This provided information on sub-soil stratification, physical and engineering properties of soil & rock, chemical composition of soil & water, depth of sub-soil water and permeability characteristics of the strata. The results of investigations were made use of to determine the alignment of routes, design parameters for the underground structures, proposed construction methods and to indentify engineering problems likely to be encountered during the construction.

(i) Field boring

Deep boreholes were sunk at an average interval of 500 metres. The boreholes were generally located on the footpaths, traffic islands or berms of the roads to avoid interference to road traffic and to keep away from the proposed alignment to prevent subsequent drainage and other problems that might be caused to the tunnel section by these holes. Before commencing the boring, exploratory pits were excavated to a depth of about 2 m to locate and avoid damage to underground services like sewers, water mains, drains, electric and P&T cables etc.

Boreholes in soil, 150/200 mm dia, were sunk by shell and Augur method and were lined with casing pipes to prevent collapse of sides. In rock, boreholes were drilled with rotary drills using diamond bits to enable recovery of core samples. The boreholes in soil were mostly 30 m deep, unless rock was met with earlier, in which case boreholes were terminated at 16 to 22 m. The deepest borehole sunk was to a depth of 70 m to ascertain presence of rock to the east of river Yamuna.

(ii)Sampling.

Disturbed soil samples were collected at every one metre interval and were mainly used for geological logging. These were also used for soil testing where undisturbed samples could not be collected. Undisturbed soil samples were generally recovered at 3 m intervals with open samplers of 100 mm internal diameter.

(iii)Testing

Field tests included standard penetrometer test, field permeability test, dynamic probe test and cluster pumping test. Laboratory tests comprised of natural moisture content, Atterberg Limits, bulk density, particle size distribution, unconfined compression, triaxial compression, consolidation, permeability, Proctor Density, Direct Shear; Rock porosity, crusing strength and Elasticity modulus. A summary of some important properties is given in Table III.

TABLE III

SUMMARY OF IMPORTANT SOIL AND ROCK PROPERTIES

Property and Unit	Typical Value	Minimum	Maximum
Bulk Density gm/cc	2	1.6	2.2
Natural water content %	18	17	22
Unconfined compression kg/cm^2	0.85	0.112	1.403
Cohesion kg/cm^2	0.325	0.0	0.65
Angle of Internal friction, degrees	23.5	0	47
Coefficient of Permeability cm/sec	10^{-2}–10^{-3}	2.27×10^{-7}	6.42×10^{-3}
PH of soil & water	7.50	7.00	8.4
Rock hardness on Moh's scale	7.0	3.0	7.5
Rock crushing strength in kg/cm^2	1300	475	1350

Soil Profile of Line No. 1 is shown in Figure 2.

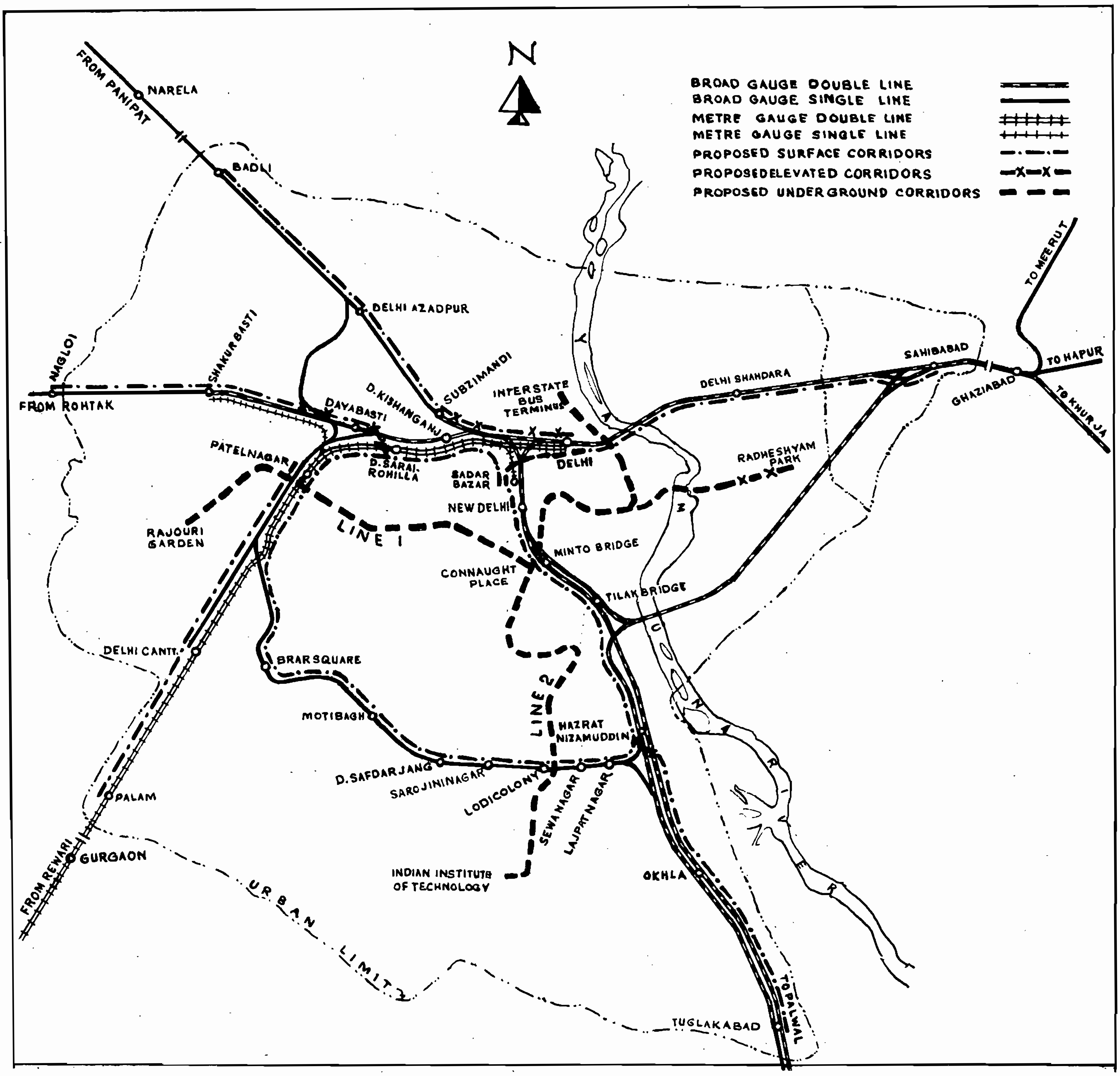

Fig.1 Proposed Rapid Transit System Network

(b) System Design

The underground system is proposed to have the same track gauge (1676 mm) as the Indian Railways Broad Gauge lines for ease in manufacture of rolling stock, transport of materials, connection with depots etc.

The system design has been done keeping in view the stringent safety requirements (as any accident inside the tunnel may result in catastrophic consequences due to large volume of passengers and the confined space), need for careful water-sealing as the tunnel is mostly located below ground water level and a high standard of cleanliness & ventilation including air-conditioning required.

On account of reasons detailed in (c) below,"cut and cover" method of construction is considered most suitable. A rectangular tunnel section has therefore been adopted.

The box is designed to cater for dead loads, earth cushion , road loading, lateral & vertical earth pressures, earthquake stresses and surcharge due to buildings, crane working etc. The effect of internal live load on account of trains is negligible. In view of the relatively thin sections as compared with the overall dimension of the structure, the individual members have been regarded as flexible and the structure designed as on elastic foundations. The central support consists of a row of columns designed with hinges at either end. Controlled concrete of 250 kg/cm^2 strength is proposed to be used. The box would be water-proofed all round to avoid seepage of water into the tunnel as this would interefere with the track-circuiting, power supply system etc. and hence

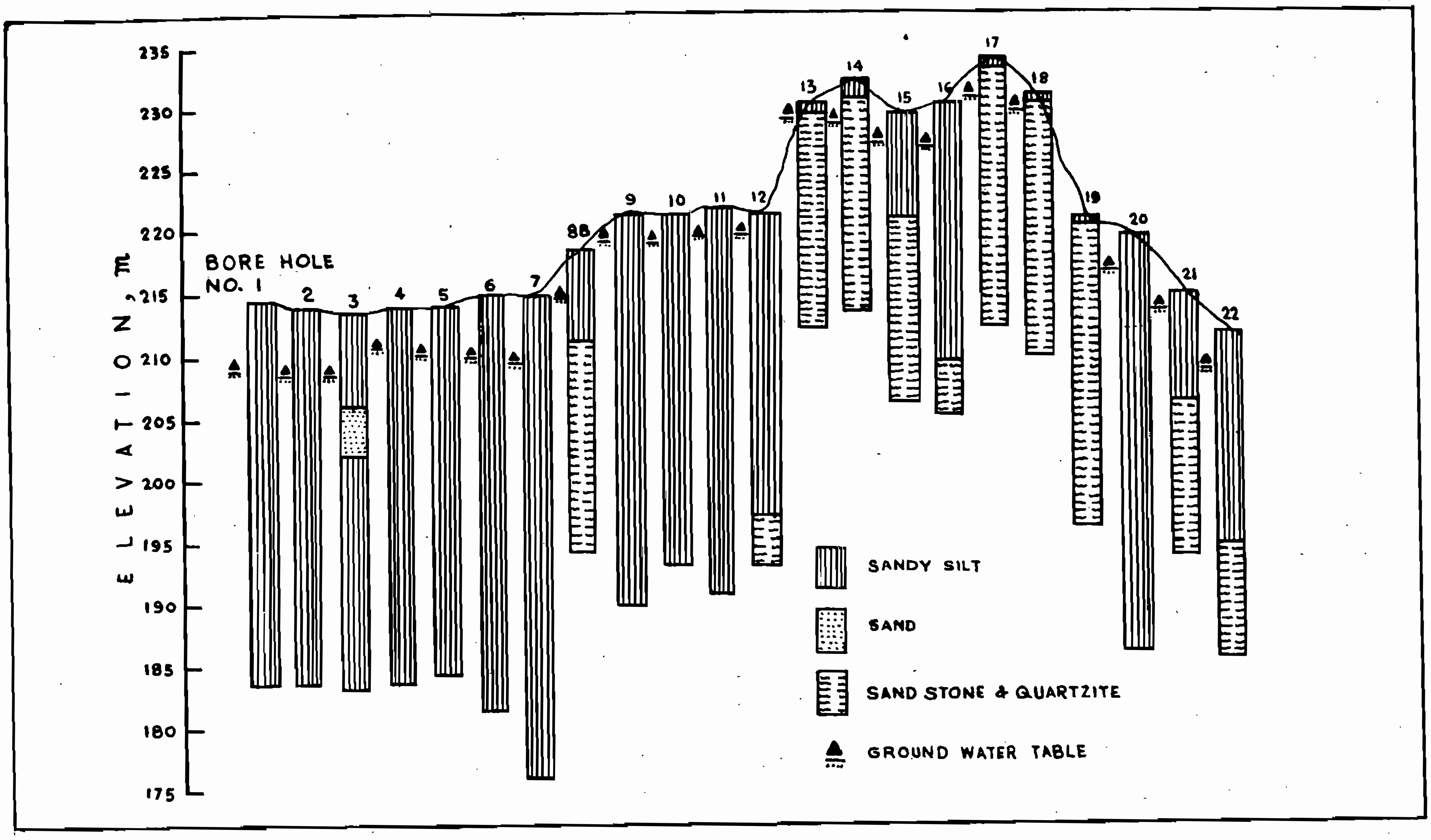

Fig.2 Soil Profile of Line No.1

affect the safe operation. To cater for any unforeseen seepage, waste water from washing of station premises & tunnel, and from lavatories etc., the external water-proofing is supplemented by an elaborate drainage system with sumps and automatically actuated pumps at intervals of about 1 km.

(c) Method of construction

Cut-and-Cover method is preferred on account of the following reasons:-

(1) It is considered desirable to keep the stations as close to the surface as possible to reduce the cost of escalators and time of access to/from the surface. It has been possible to do so, despite the wide variations in ground level (upto 26 m) by adopting a ruling gradient of 1 in 50. The depth of construction is generally about 12 to 14 m below the surface.

(2) The nature of soil varies widely from silt and sand to hard quartzite rock both along the lengths of the corridors as well as along the depth. For example, in the case of Line 1, out of 11 Kms. lengths, rock is encountered for 3.55 kms. in 4 stretches, at varying depths starting from ground level itself. Tunnelling in such diverse strata would require entirely different methods and equipment and would therefore be uneconomical.

(3) The alignment has been proposed mostly under the roads. In some cases at points of change in alignments, a few buildings are involved but as these are not high-rise buildings, it will be cheaper to rebuild them after constructing the subway than tunnelling below them.

In cases where the alignment passes under narrow and/or busy roads, temporary decking would be provided to allow the road traffic to pass uninterrupted. This is specially applicable at station sites where it will be necessary to occupy greater width of the road during the construction.

In the Delhi strata, soil generally contains gravel in most places. It may therefore not be suitable to use sheet piles to support the sides of excavation. Moreover, this method is not favoured on account of excessive noise and vibrations which would not be acceptable in the busy commercial and residential city areas. Diaphragm walls and bored-piles-and board methods are, therefore, considered more suitable. These methods are also better for stretches with partly soil and partly rocky strata. Both the bored piles and diaphragm walls can be anchored into rock, below which excavation could be continued without supporting arrangement. The diaphragm wall is also considered most suitable to keep out the ground water from the excavation area.

(d) Dewatering

On account of the high ground water level (2 to 4 m below ground) and the sandy nature of strata which is prone to " flowing" action, it will be necessary to lower the ground water by well point system. For greater excavation depths, double stage system would be necessary. Freezing of ground water by chemical injection is not considered suitable for the Delhi climate where temperatures rise above 40°C.

(e) Costs and economics.

The Civil Engineering cost for one Km. of double line underground corridor including one station (as the stations are located 1 km. apart approximately) works out to Rs.90 million. Taking into account the electrical, signalling & rolling stock costs the total cost is expected to be around 125 million per route Km. The underground railway system is undoubtedly capital intensive. Though its fare structure may not be as low as that for the buses, it is certainly much cheaper than the private cars/taxis. However, the economic comparison between the railway and roadways is really not on a fair basis as the cost of constructing the roads and other fixed structures is never taken into account when calculating the cost of service. Unlike this, the railway system always takes into account all the capital costs.

(f) Social Benefits

However, it is not right to take only the cost basis in isolation. Today, the profit-making possibilities of rapid transit systems are remote from the minds of those concerned with urban transportation. The social benefits which accrue from a rapid transit system far outweigh the direct cost benefits but their monetary value cannot be computed accurately, for example:

i) The underground railway provides the fastest means of transport and is approximately twice as fast as the buses. The average speed which is an important measure of " quality of service ", may be as much as 35 KMPH in the case of underground railways and remains constant in the peak hour whereas the average speed of road based transport system is about half this value and usually reduces in the rush hours. The saving in travelling time amounts to a great social benefit.

ii) It has been estimated that a single line of underground railway transporting two thousand persons every ninety seconds during rush hour traffic will carry as many passengers as nine lanes of motor buses with 75 persons in a bus every 30 seconds, or 26 lanes of private cars with four passengers in a car every 4 to 5 seconds, or even as many as 58 lanes of cars with 2 persons every 4 to 5 seconds. On a comparative basis the underground railway virtually requires no street area against 27 metres required for buses or 139 metres for cars.

With the saving in space affected by underground system, the city area can be used more intensively for other purposes such as offices, residential accommodation etc. This in turn keeps the city more compact and provides valuable savings in other services like water supply, electricity, telephones etc. Removal of land from the " end-uses" of urban life threatens to eviscerate the central areas of the cities and adversely affects the various activities.

iii) Another indirect social benefit the underground system provides for a city is the use of railway stations as the best possible air-raid shelters in case of war.

iv) In terms of energy consumption, this is the most economical form of transport (the energy consumption being 410 BTU per tonne Km. against 1410 for road transport) and would result in substantial savings in oil requirements.

These benefits are, of course, such that it is not possible to assess their exact monetary value but nevertheless they are very significant.

4 CONCLUSION

Considering the phenomenal population and economic growth of Delhi, an underground rapid transit system railway is the best solution to the intra-urban transport problems. The Techno-Economic Feasibility Studies have resulted in the need for providing a 135 Km. network including 36 Km. underground railways. The underground system though costly and capital intensive provides the best solution as the scope of road improvements is limited. In addition, it also contributes to many social benefits like saving in travelling time for commuters, saving in valuable city land and helping to solve the energy crisis.

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Tunnelling and the Environment; Recent Trends in the Use of Immersed Tubes for Highways and Rapid Transit

by

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SUMMARY. The paper deals with the various aspects of immersed tube tunnelling with emphasis on the advantages of such construction in the design of a crossing across a water barrier for both vehicular traffic and rapid transit railway system.

1 INTRODUCTION

This Conference has been organised to study the use and potential of underground construction in urban areas. As such it is throwing together specialists in urban development and underground construction. The subject I propose to talk about concerns a specialised form of underground construction -- immersed tube tunnels. Over the last twenty years or so I have been involved in various capacities including planning, design construction and operational management of several immersed tubes tubes, all of which have been located in urban areas. Having long espoused the benefits of such underground projects, it is gratifying to find that at long last authorities are losing their fear of putting anything below ground level -- the environmentalists have taught us that we don't have to see something to believe that it exists.

At this stage I should describe what I mean by an "immersed tube". There is none in Australia at this moment, so I cannot just quote an example. Most of the existing tunnels are in Europe, particularly in the low-lying areas of Holland and Belgium, and in North America, mainly in the United States. Others are located in Cuba, Hong Kong, France and Germany, and Japan. Essentially, an immersed tube is a tunnel built in sections in a dry dock, placed in a prepared trench on the bottom of the sea or river, and joined together to form a traffic artery. Most of the immersed tubes constructed to date have been for road traffic, but now rail rapid transit systems are utilising them, and utility companies will do so in the future. Probably the immersed tunnel best known to you as engineers and planners is the recently opened Cross-Harbour Tunnel in Hong Kong. Some 1850 m in length, with 4 traffic lanes, this tunnel links the island with the mainland of China, providing no marine restrictions to the many ships that use the port.

Why are there not more immersed tunnels in use today? Firstly, they do not provide the answer to all crossing problems. They are particularly appropriate at locations for water crossings where there are low-lying banks or shore lines, where soil can be dredged, and where marine traffic requires a certain navigation clearance. Normally they are not economical where the crossing site has high banks -- in such a situation a large span bridge would probably be more suitable. In terms of cost, they are often but not always more expensive than large span bridges, and have similar or lower operating and maintenance costs. In terms of aesthetic, there is little visual for a tunnel structure, and some people and environmentalists may prefer that; whereas, for a bridge, all are visible and to some people it reveals man's mark on the landscape.

The second reason why there are not more immersed tunnels in use today has to do with the level of expertise available. The construction of immersed tubes requires a greater level of skill, know-how and experience than above ground structures.

The third reason, I feel, has to do with the reluctance, until recent times, with which public authorities have treated underground projects in general, and underground traffic arteries in particular. Now we appreciate that our crowded urban areas must capitalize on space available, and quite often we go underground in a literal sense. In the process we preserve the air space for things that cannot economically be put below the ground surface.

As an indication of the suitability of tunnels vs. bridges, Fig. 1 shows a hypothetical crossing with a navigational clearnace of 500 m wide by 40 m above water. The required depth below water level is 15 m. This brings the depth of the roadway of a tunnel under the water surface to less than half that of the height for an elevated bridge deck above water surface. Therefore, in the case of low banks, the length of a tunnel structure is much less than that of an elevated bridge structure, considering the lengths of the approaches on either side of the shores in both cases. The greater cost of expropriation, maintenance, etc. for a bridge and the reduction of the indirect costs by less transportation work, capitalized time saving etc. for a tunnel, often make the construction of a tunnel more attractive and desirable than bridge building. This becomes more pronounced with the increasing requirement of the navigational requirement, say of the order of 600-1000 m in length. On the other hand, as mentioned earlier, in the case of steep and high banks, with harder river bed foundation, the elevated bridge is a more logical and economical solution.

The remainder of the paper will discuss in general terms, without being too specific or technical, the various aspects of the immersed tube tunnelling with particular emphasis on the modern trends and developments. The floor will then be open to the questions that you might have, either of a technical or general nature.

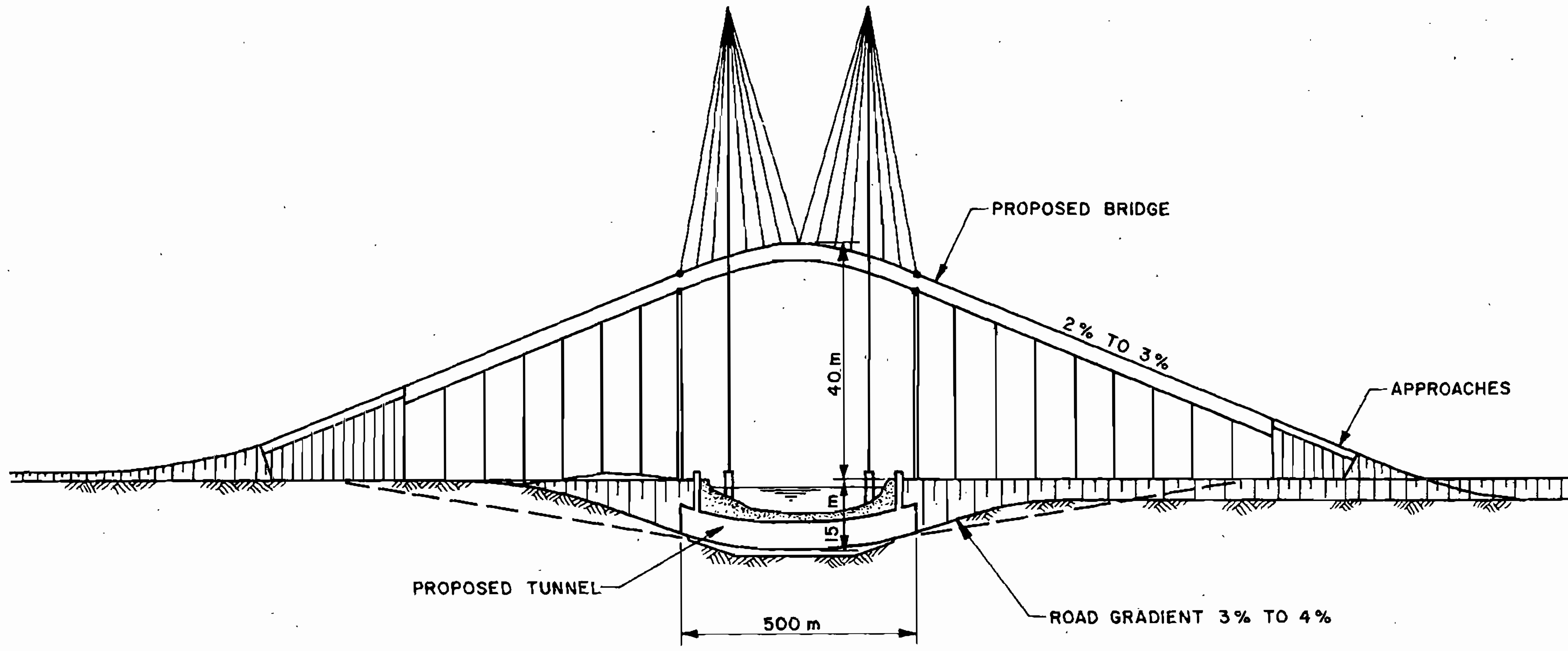

Fig. 1 Comparison between hypothetical bridge and immersed tunnel

2 BRIEF REVIEW OF IMMERSED TUBE DEVELOPMENT

The two basic methods for tunnelling under a waterway have been the bored or shield technique, and the immersed tube method. Perhaps the first subaqueous tunnel was the shield driven Thames Tunnel started in 1825. The shield method has remained expensive and dependent on the uncertainties of geological conditions. The major rival to this technique has been the immersed tube concept. This method permits tunnels to be placed at the limiting minimum depth, independent of geology, thus significantly reducing the length of below ground approach required over that of bored tubes, It applies itself well to sites with poor foundation material, as the tunnel is usually lighter than the material it displaces. Recent developments in prefabrication and joining of units have also kept use of compressed air and divers to a minimum.

The first major tunnel built by the immersed tube technique was the Detroit River Railroad tunnel, which was opened in 1910, and is typical of subsequent east-coast American tunnels. These tubes are generally circular using concrete for strength and sinking ballast, and a steel shell as concrete form and waterproofing membrane. The steel shell is fabricated on slipways and launches, then concrete is added to the floating unit until it can be sunk at its desired position. Other tunnels built using similar methods are the Baltimore Harbour, Maryland USA (1957), Hampton Roads, Virginia (1957), and the Mobile River, Alabama (1972) which is shown in Fig. 2.

sections. The only drawback is that extra width is usually required to provide for ventilation ducts. The circular tunnel normally has extra space available for ventilation ducts requirements above and below the reading. The circular tube is more structurally attractive and efficient than the rectangular shape and can result in very thin walls.

The Nieuwe Maas tunnel at Rotterdam (1941) was the first application of the reinforced concrete rectangular cross-section design. The major problem with this type of tunnel was to make it bear evenly on the subsoil. A sandjetting apparatus was developed to inject sand into the space between the underside of the tunnel and the trench bottom, while the tunnel sat on adjustable temporary supports. This method has until recently been used for most other rectangular tunnels.

The second tunnel of this type was the Deas Island tunnel at Vancouver, Canada, engineered by my former firm, and the first to use a rubber seal to make the initial joint between units. This was also the first application of a bituminous compound to replace the all-steel shell as waterproofing.

The Havana Harbour Tunnel (1956-1958) was the first tunnel to be prestressed longitudinally and trasversely. The L. H. Lafontaine tunnel at Montreal, Canada designed by our firm, was the second such tunnel At the time it was the world's largest prestressed concrete structure.

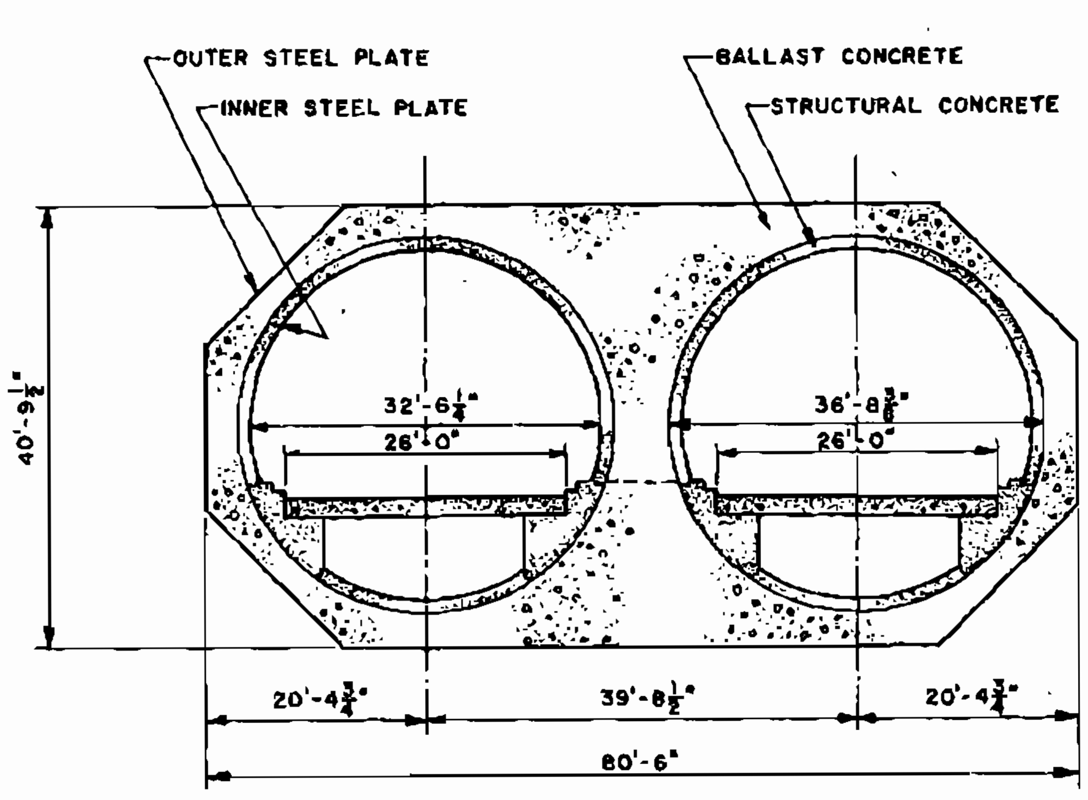

Fig. 2 Cross-section of Mobile River tunnel

In Europe the rectangular immersed tunnel is popular due to its ability to accomodate wide tubes, as required by modern highways, in one cross-section resulting in a significant reduction in dredging costs over equivalent circular cross-

In 1960 the one prefabricated element of the Rendsburg tunnel was placed in position on a screeded gravel mattress, marking the first successful attempt at placing a rectangular tunnel without sandjetting. This success was repeated with the placement of the Hong Kong Cross Harbour Tunnel on a screeded gravel bed in 1972.

Today we have more than forty major immersed tube tunnels scattered around the world serving both vehicular traffic and the rapid transit railway systems. With more attention oriented towards this field of construction, the immersed tube tunnelling is becoming popular and gaining a prominence in the design of crossings.

3 TUNNEL DESIGN AND CONSTRUCTION

The structural design of the immersed tunnel does not usually create much of a problem, and is clearly dictated by the method of construction. Transversely, the cross-section is analyzed as a rigid frame for various loading cases. Computer programs can be used beneficially for this purpose. From the moment of launching to the time they rest in their final positions and covered by the backfill, the tunnel structures go through a number of stages of different loadings and stress conditions. All these cases must be investigated, including the possibilityof a ship being sunk on the top of the tunnel.

Longitudinally, the tunnel units are analyzed as a long beam resting on an elastic foundation and being subjected to the imposed loading and imposed deformations due to differential settlements. The effects of differential temperature and the backfill are also taken into account. In addition, where seismic intensity and activity may require an earthquake analysis, the design must conform to such an investigation.

An important area of the tunnel design which differs from the normal engineering design is the calculation of the buoyancy characteristics of the prefabricated tunnel units which include the floatation and stability requirements. The cross-section is usually designed and proportioned so that it would float with some freeboard and safety against sinking but not high enough that it would require too much ballasting to sink it down. Our experience with Deas Island and Lafontaine tunnels indicates that a 2-3% reserve buoyancy while floating the unit is satisfactory. Due to the vital role of the tunnel crossing and its importance, extreme care is exercised in selecting the materials and construction techniques to ensure the adequate serviceability of the tunnel. The structure must not only be structurally sound but als be water-tight, durable, and capable of resisting the abnormal environmental conditions which, in most cases, are highly corrosive.

The construction technique of the immersed tunnels plays a vital role in the successful completion of the crossing and is often dictated by the proven methods and past experiences. It is one of the difficult tasks among the heavy civil engineering works and requires highly technical performance in the casting procedures and sinking and jointing methods.

In the construction of a concrete submerged tunnel crossing, the cost of fabricating, casting, sinking and founding of tunnel units can be divided approximately into 40-50% in the actual fabrication of the structure and 50-60% in the method of casting (for example within a casting basin or on slipway), placing and foundation preparation. The saving in using the most economical cross-section is relatively small compared to the saving that could be achieved by devising more efficient and economical casting, sinking and founding techniques.

Virtually all immersed tunnels are built by similar methods which can be summed up as follows:

1. Excavating a trench underwater to an accurate profile.

2. Constructing the tunnel units in suitable lengths with water-tight bulkheads at each end of the units either in a dry dock or on slipways.

3. Flooding the casting basin or launching the units from the slipway and then towing them afloat to the tunnel site and sinking them in the correct location by means of a sinking plant.

4. Joining the sunken section to the previous one by some mechanical means to form a temporary water-tight joint.

5. Removing the bulkheads and completing the sealing of the joint in the dry from the inside of the tunnel.

The above procedure has been commonly used to date because of the satisfactory procedure and also because any new technique devised, if not proven in reality, can cause the uncertainty that it may not work as well as envisaged.

The above construction procedure has been used in various ways with some difference in technical details. The methods employed in the construction of many major submerged tunnels are reported and can be found in many journals, reports and construction news.

4 ENVIRONMENTAL CONSIDERATION

With the rapid development of the urban areas and their traffic, more roads, bridges and tunnels are being built to satisfy the growing demand of transportation. This puts enormous pressure on the environment about which we are all concerned, and which we believe should be preserved or improved for better quality of life. For any major project, such as an immersed tunnel, an assessment of the environmental impacts created by the project must therefore be made with due regard to all implications.

There are certain areas of impacts which are generated commonly by any type of crossing, whether bridge or tunnel. Such areas include noise, air pollution, visual and aesthetic, traffic, land and economic, etc. Should a decision be taken to build a crossing, the impact on the above areas must be assessed and attempt made to improve upon the adverse effects. I do not intend to go into the detail of these impacts, but I would like to emphasize only those areas which are created by immersed tunnelling only.

The most important one is the dredging of the river bottom and disposal of the dredged material. Much of the concern over the actual dredging-disposal process is related to the direct destruction of benthic (bottom dwelling) organisms. Such organisms are known to play an important role in the aquatic ecosystem, and may include commercially valuable species such as lobsters, oysters and clams. Although the direct effects of dredging on benthic organism may appear to be obvious, there is little information available that permits the prediction or assessment of the overall extent, significance and duration of the effects. There are also some potential indirect effects on biological communities which are attributed to the environment such as changes in bottom geometry and bottom substrata which subsequently changes the water velocity and current patterns and the salinity gradients. Within the current state of knowledge, it is not always possible to definitely assess such effects or judge whether they are of an adverse or beneficial nature.

Apart from the concern over the dredging-disposal process, which is mostly directed to the water quality and aquatic organisms, there is no other serious impact caused by tunnelling. Ecological impact is negligible as the areas are in all cases almost totally industrial. An immersed tube tunnel, being underwater and underground, has no visual intrusiveness except for the ventilation towers which, in most cases, blend quite well within the background of tall structures on each shore. ~~This~~ offers the advantage of preserving the open sea or river area completely unobstructed.

5 RECENT TRENDS AND DEVELOPMENTS

The technology and practice of constructing immersed tube tunnels is relatively young and has limited application. A total of approximately 40 immersed tunnels of significant size have been constructed, most of them within the past 30 years or so, and almost 50% in the last two decades. This continuing uptrend in the use of immersed tube tunnels for a crossing design, either for vehicular traffic or for rapid transit railway system, is very much expected, partly due to the benefits of such construction and partly due to the surge of underground construction.

By the time this conference will be over, I am hopeful, if not optimistic, that we, planners and engineers, will be thinking more seriously about reshaping our cities using more underground construction which, in my opinion, will inevitably play the most important role in the future development of our crowded urban areas. As an answer to the acute problem of transportation in a major city, today more cities are building or getting ready to build an underground mass transit railway system. Such systems will utilize the full benefit of an immersed tube tunnel for the crossing under the harbour or the river. To substantiate this statement, examples can be made of the Metro Rotterdam tunnel, the San Francisco Bay tunnel, the Tamagawa Tunnel in Japan, and the newly proposed Hong Kong Mass Transit railway tunnel which is currently in the design stage by our firm. That more cities are actively considering the use of such construction can be judged by the number of feasibility studies that are currently being conducted.

The recent trend in increased use of this construction for vehicular crossing as well can be attributed partly to the advantage it offers over a bridge from the view point of the environmental impacts. An interesting example of this can be made of the recently proposed Baltimore tunnel for which we are undertaking the initial design work. The proposed interstate highway crossing is located near the Fort McHenry, which is a historical place and, as such, the proposal of a crossing of the harbour became a very sensitive issue right from the beginning to the local people and the environmentalists. After an extensive study of the environmental impact, an immersed tunnel, for the crossing, was favoured by the authorities in place of an elevated long-span bridge, although the cost of a tunnel will be higher than that of the bridge. This is just an example, but nevertheless demonstrates how the environmental issue is becoming increasingly important.

The wide acceptance of this technology among the planners and engineers has been aided substantially by the improved construction technique and the developments in underwater construction methods. More sophisticated marine plant and equipment are making dredging, screeding and sinking methods safer and easier to carry out with more control in the accuracy.

Founding of the tunnel units in a pre-dredged trench has been simplified considerably by the development of a screeded crushed stone mattress foundation. In this method, a crushed stone bed on the bottom of the trench is screeded and grades by a screeding plant to an accuracy of ± 40 mm upon which the tunnel units are laid in their proper positions. This method is less expensive than sand-jetting under the units which requires temporary adjustable support of the units, and has been successfully used for the first time in the Rendsburg tunnel and subsequently in the Hong Kong Cross Harbour tunnel. Due to its simplicity, we are proposing again to use this in the Mass Transit Railway tube in Hong Kong.

Much improvement has been made on the casting procedure. Usually, the horizontal construction joints, in the case of reinforced concrete tunnels, are not desired due to the potential weakness of the water-tightness of the tunnel cross section. This was first eliminated successfully in the construction of the Metro Rotterdam tunnel by adopting a single pour construction technique. The tunnel units were built in small segments which were fabricated in one continuous pour operation eliminating any joint. This was achieved by devising a collapsible and movable steel form work, Fig. 3, and using floating type vibrators. It is now quite possible to undertake a similar technique of casting procedure without much difficulty.

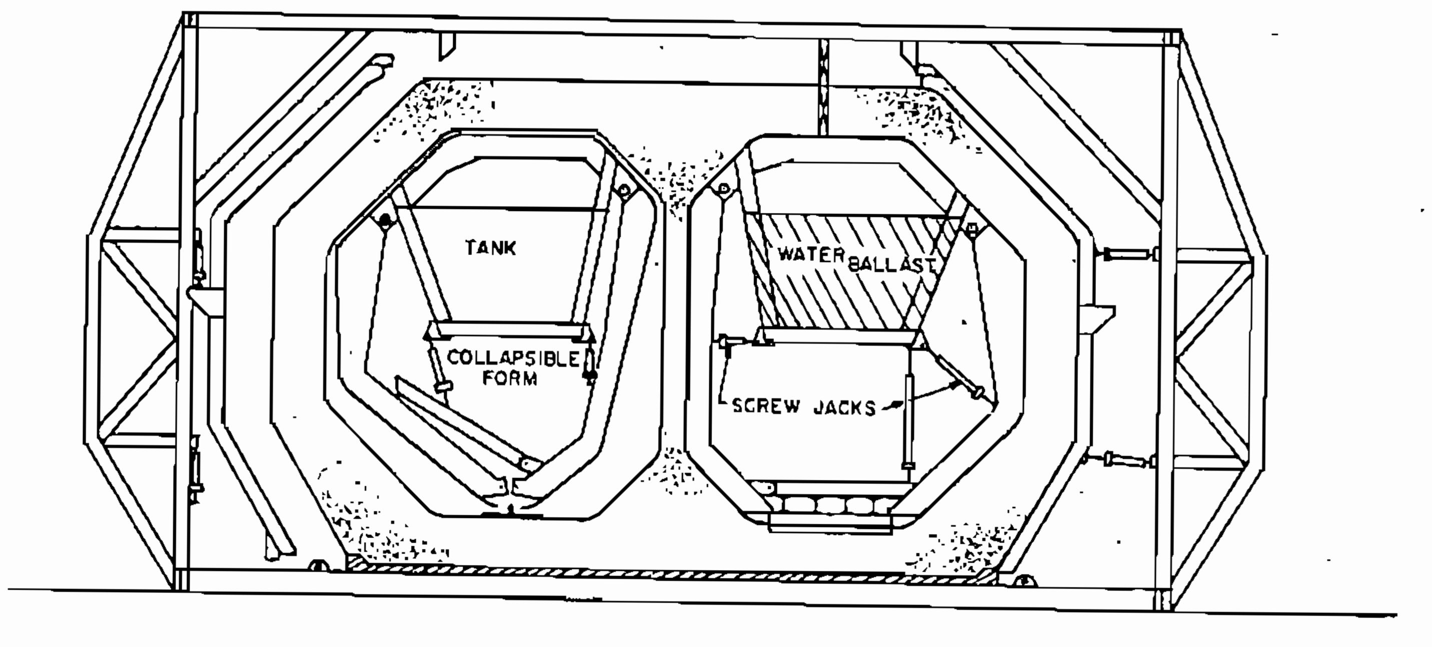

Fig. 3 Casting forms used in Metro Rotterdam tunnel

More developments are taking place in general on various products and accessories that are required for an immersed tube construction. Better quality rubber gaskets with proven performances, waterproofing system, and rubber seals are all adding to a better and safer tunnelling construction.

6 CONCLUSIONS

The Immersed tube tunnel is a specialized form of underwater construction by which a crossing of water course or harbour can be designed so as not to interfere with navigation and which would have no visual intrusiveness. The benefit of this construction with due regard to its environmental impact has been realized and has lead to the construction of numerous immersed tunnels in the last two decades.

With improved construction technique and past experiences, the immersed tube tunnel is no longer a difficult engineering undertaking, and is gaining wide acceptance and popularity among engineers. Tomorrow's transportation need for the crowded urban areas will utilize more underground construction and thus more immersed tube tunnels may well be seen to serve as the vital links between the shores.

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Urban Design and Underground Architecture in Metropolitan Centres, with Special Reference to Brisbane's C.B.D.

by
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SUMMARY. The role of public open space in the centres of towns and cities in the 20th C has changed with the increasing demand for car storage space. A number of historically important urban redevelopments are cited in overseas countries and three spaces in Brisbane's C.B.D. examined in greater detail. Design criteria are proposed and conclusions stress the need for a more subtle approach to the design of underground facilities.

1 INTRODUCTION

Public open space in the centre of towns and cities has long been a basic land use component. With few exceptions these small scale spaces have evolved from open air markets or social gathering places but through a process of formalization (Ref. 1) have become known as squares, places, plasze, piazzas and plazas in the urban centres of Europe. In Australian towns and cities, the names commonly used for these spaces are squares or gardens, although the term plaza has crept into the planner's vocabulary in recent years with increasing North American association.

Up to the 19th C the development of public open space in urban centres was concerned with paving, embellishment and the aesthetic qualities of surrounding buildings. During the 20th C a change of emphasis occurred with the increased demand for car storage space in 'western' city centres.

As the cost of C.B.D. land rose prohibitively in relation to car storage space and as engineering techniques in waterproofing and mechanical ventilation advanced to the point where underground construction became increasingly feasible, the space below public open space began to be viewed with new interest.

While the space below streets has long been exploited for underground railways, the greater dimensions of the average square has proved more attractive to planners for the storage of cars.

With the unprecedented expansion of cities over the past two decades has come numerous noteworthy examples of redevelopment using multilevel planning solutions.

2 EXAMPLES IN OVERSEAS CITIES

One of the earliest redevelopment schemes involving the use of a central public square for underground car parking was completed in 1942 in San Francisco (Ref. 2). The four storey garage with a capacity of 1700 cars demonstrated how unobtrusively a major facility could be inserted into the urban landscape.

In the last decade, while many car parking garages have been constructed under squares or gardens, a new trend has become evident, and that is that the underground space used is tending to exceed the curtilage of the space above. This is making possible direct linkage with surrounding land uses and opening new opportunities for imaginative urban design.

One of the most interesting examples of this type of redevelopment is the new Shinjuku Station Plaza in Tokyo (Ref. 3). During the 60's the station and adjacent plaza space was completely reconstructed to integrate shopping promenades and pedestrian malls at lower levels, with bus bays and vehicular circulation space at street level. Continuity between levels has been maintained by the use of a large opening in the plaza surface to allow spiro-helical car ramps to connect the upper and lower level road surfaces. Even the large ventilation shafts to the car park below have been designed as environmental sculpture and the redeveloped plaza is highly expressive of life in the late 20th C.

Another outstanding example is Sergels-Torg in Stockholm. A major revitalization programme in the C.B.D. is now nearing completion with the multi-level Sergels-Torg traffic intersection and lower plaza providing a dramatic focus for the redeveloped business centre. Shopping arcades have been incorporated in the scheme at the lower level which also provides access to the adjacent tunnelbahn or underground railway. Mechanical staircases connect the main levels, to encourage greater use of the pedestrianised spaces and to increase traffic flow generally. Service roads, a bypass road and an underground railway line have been inserted at various lower levels as unobtrusively as the large service ducts that have been built below street level throughout the C.B.D.

A new parking garage nearing completion in Zurich has been chosen as a final example to illustrate another trend which is apparent in European cities. Parkhus Urania in Muhle Strasse has been inserted into a sloping site adjacent to a small centrally located park which in the process has been extended visually over the landscaped roof of the new facility. The curvilinear roof of the new car park has been landscaped and perforated in a number of places to allow sunlight to penetrate to lower levels while the approach paths and ramp have been curved graciously through a landscaped plaza. Within a short space of time the growth of trees and shrubs will transform the parking garage and it will become an outstanding example of good environmental management.

These examples of central urban redevelopment, although constructed some decades apart, nevertheless highlight aspects of the trend toward the multiple use of public open space and of the growing awareness in urban communities of the need to provide imaginative and sympathetic solutions to environmental problems.

3 PUBLIC OPEN SPACE IN CENTRAL BRISBANE

Brisbane, with a population of approximately 0.9 million in 1974 has a central area of approximately 4.1 km^2 with a T-shaped core (shown shaded in figure 1) of approximately 0.7 km^2.

The working population of the core in 1970 was 48,000 (Ref. 4).

Within the core are three public open spaces at different stages of development.

(a) King George Square, recently enlarged and redeveloped with a multi-level car park below ground level, is on the boundary of the retail zone of the core.

(b) Anzac Square, where additional space has been acquired for an extended square over Adelaide Street with commercial space and car park below ground level, lies between the retail and office zone of the core.

(c) Queen's Park Gardens, enlarged and landscaped in 1963, is on the southern periphery in the State Government precinct.

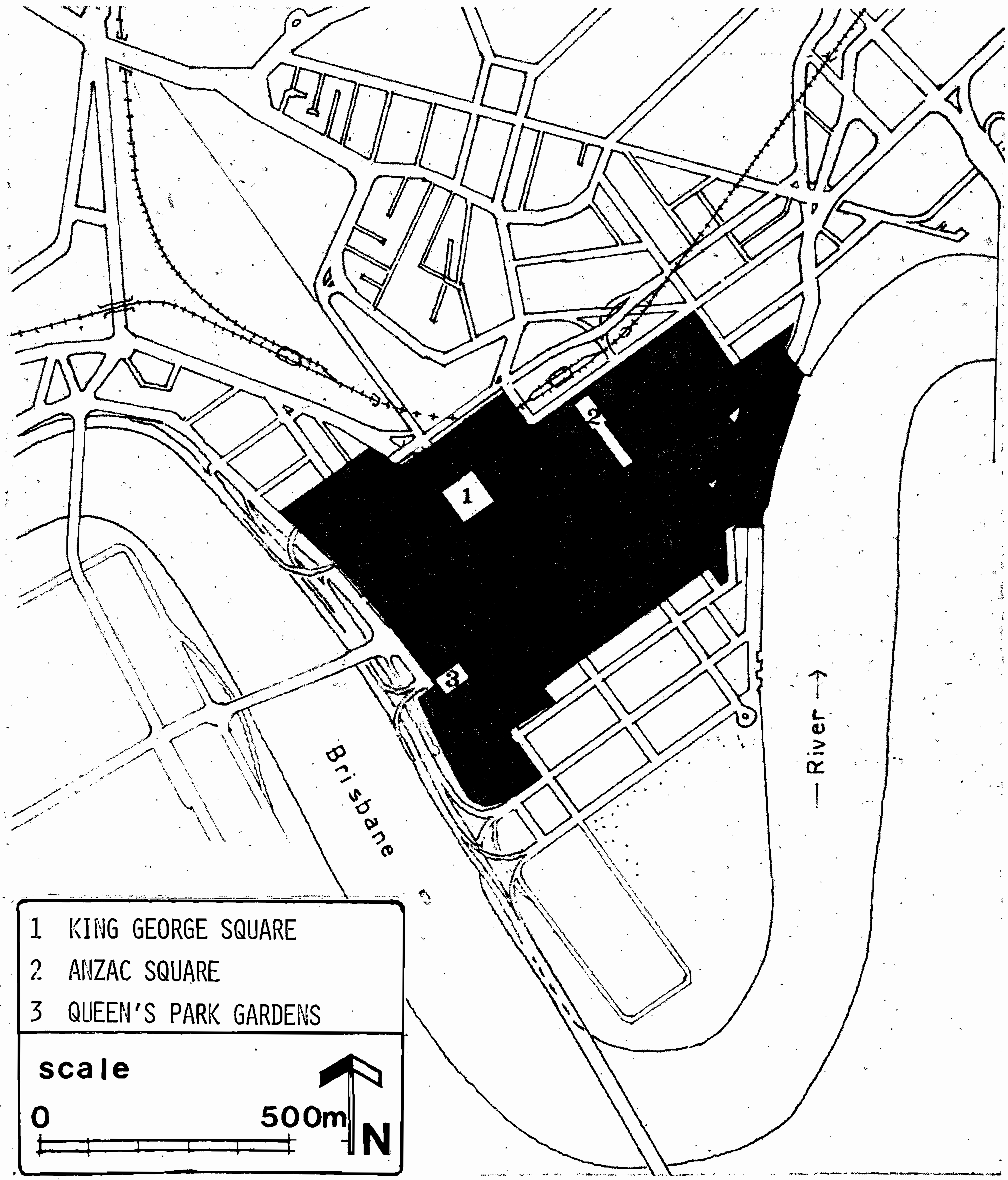

Figure 1. Public Open Space in Brisbane's C.B.D. core.

(a) King George Square

Located to the east of the City Hall, King George Square was extended in the mid-sixties by the City Council's expropriation of shops, offices and a theatre.

In redeveloping the square a multi-level car parking garage was built below ground with car access ramps from Roma and Adelaide Streets. (Ref. 5)

Before construction commenced argument over the eventual surface level of the square was lively. There were expressions of concern that it was set too high above Adelaide Street to be seen from the footpath and that the imposing approach steps to the City Hall would vanish as a result. Others feared the loss of well-established palm trees which were to be removed from the site. Indeed the final product confirmed these fears.

Arrangements for pedestrians are barely adequate, in that there is no clear definition between vehicular circulation and pedestrian space apart from the exit staircase in Adelaide Street. Once a driver becomes a pedestrian he has to be very vigilant between his car and the staircase exits.

Pedestrian tunnels under Ann and Adelaide Streets have not been provided and there is constant conflict between vehicles and people at the Adelaide Street car entrance and exit.

Land uses surrounding the square include a hotel, two churches, two banks, assorted shops and the City Hall. For most of the week the churches attract few pedestrians while the reserve bank, although providing an imposing building facade facing the square is a socially negative land use that would have been better located elsewhere.

On the northern side of the square two tall office buildings have been erected since 1968 which cast unwanted shadows over the square in winter.

The square itself has been landscaped to provide a venue for formal gatherings as well as for less formal use and is partly paved and partly covered with turf. Planting boxes provide an inadequate amount of root space for the trees that have been planted in them.

Notwithstanding these criticisms both the square and the car park below are well used, and some of the original design deficiencies associated with vehicular circulation in the car park have been rectified.

(b) Anzac Square

Anzac Square, situated between Central Station and the General Post Office, and central to the office zone within the core, was also extended by the resumption of properties between Adelaide Street and Queen Street over a period between 1967 and 1970. Although construction has not yet commenced, plans for the redevelopment of the square have been published in the press (Ref. 6) and a model of the scheme placed on public view in the City Hall.

The proposal is for an extended square which is to have a new plaza between Ann Street and Queen Street, bridging Adelaide Street.

Included in the plans is a landscaped square with a lower level shopping arcade between Adelaide Street and Queen Street. A car park is to occupy the whole area below the square connected by a tunnel under Adelaide Street.

A pedestrian tunnel under Queen Street leading to the G.P.O. has been included in the design and there is an existing pedestrian tunnel under Ann Street leading to Central Station.

Critics of the scheme fall mainly into three groups: those who are concerned with the preservation of the war memorial and shrine at the northern end of the square; those who wish the existing square to remain a green park with its valuable 'bottle trees' left undisturbed; and those who question the design and functional aspects of the proposal.

With regard to the latter, the items which need closer examination are: the lack of direct linkage between the plaza level and Ann Street; the lack of visual integration between the upper and lower plaza levels; and the fact that the redevelopment of Central Station on the northern side of the Square proposes tall office buildings that will produce shadows during the midday lunch period in winter. The tall buildings (approximately thirty storeys) that are being proposed on the eastern and western sides of the enlarged square will also cast shadows for part of the day, but these are not likely to be as negative in effect as the shadows from the buildings on the northern side.

The entrance to the car park is to be located in Adelaide Street away from the high pedestrian flows expected in the area. Although this location is logical for traffic movement it is within a potentially flood prone section of the street.

Fewer car parking spaces (529) will be provided than in the King George Square car park (925) and parking space for the public could be even more limited should the two large insurance companies with adjoining headquarter sites and responsible for the financing of the redevelopment of the square be allowed to preempt space for their own use.

Environmentally the new square should be enhanced by the preservation of the Post Office building at the southern end and by the extensions to the plaza space proposed adjacent to the square between the new Commonwealth Government buildings on the western side. The inclusion of shops under one end of the square should make for greater economic viability as well as increase the social vitality of this part of the C.B.D.

(c) Queen's Park Gardens

Queen's Park Gardens, located in the government precinct, is surrounded by old buildings of great historic value to Brisbane. Although they are not all outstanding works in themselves they are nevertheless representative of their type and style as high quality government buildings. Only the Public Library on the southern side has been extended in a style out of context with the original neoclassical facade, and, indeed, with the remainder of the buildings surrounding the square.

It is likely that the Gardens will be redeveloped to accommodate motor vehicles in this part of the C.B.D. fringe, close to the Riverside Expressway. When this is done it will provide an opportunity to formalize the layout of the existing gardens which do not at present reflect the architectural style of the surrounding buildings.

In such a redevelopment, the use of a horizontal plane would complement the neo-classical flavour of the precinct, while the landscape elements such as staircases, planting boxes and street furniture could reflect the quality of the surrounding buildings if not their traditional details. The design of the substructure should therefore be subservient to the broader environmental design intentions.

4 SUGGESTED DESIGN CRITERIA

From the foregoing study it can be seen that in redeveloping public open space in metrocentres, a number of design criteria should be considered by the design agencies charged with redevelopment. The following list is therefore set down as a guide.

(a) The space which results from the redevelopment should not become too obviously the roof of a building below ground level.

(b) Activity generators such as shops, restaurants, post offices, etc., if not included in the surrounding land uses, should be incorporated where possible in the redevelopment.

(c) Where shops are included in the space on the floor below square level, an opening in the square should be made to integrate the two levels visually.

(d) Vehicular access to car parking space below the square should be set away from the square and approached via short tunnels to avoid unnecessary pedestrian vehicular traffic conflict points on the footpaths surrounding the square.

(e) Some of the car stalls on the first floor below square level should be eliminated to allow for deep planting boxes for large trees or tree groups.

(f) Internal floor space arrangements of car parking garages should be considered in relation to future change of use. Ramped floors for example may not be useful for other purposes than parking.

(g) Vehicular movement inside car parks should be separated from pedestrian movement. Raised footpaths leading to exits should be provided, and vehicular crossings avoided in the design.

(h) Direct linkage with existing or future underground railway concourses should be considered at the design stage and passageways allowed for if not built.

(i) The need for flood controlled entrances must be examined.

(j) The design should go beyond a simple solution to technical problems and should make use of latent urban landscape opportunities.

5 CONCLUSION

The tendency to make greater use of the space below public open space in the centre of cities and metropolises has been examined briefly in historical context and in greater depth in relation to three spaces in Brisbane.

An attempt has been made to illustrate the need for environmental appraisal in addition to the traditional analytical investigations necessary in redevelopment schemes.

It is appreciated that the design criteria listed may not be applicable to all new public facilities in the C.B.D. and the need for refinement and greater research in this field is evident. To produce not only well designed but also imaginative solutions to urban problems of the type discussed is perhaps the most difficult task facing planners today. It should be remembered that it took many centuries to develop the aesthetic and ambient qualities of the famous spaces of European cities. To expect to emulate these achievements in the contemporary city centre is perhaps utopian. But there are also many excellent contemporary examples of redeveloped public open space using underground construction techniques in other countries on which we can draw. By studying these achievements and by reappraising the work done in Australia to date we should be better equipped to maximise the opportunities for imaginative urban design in the future.

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Kappala Underground Sewerage Works, Stockholm

by
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SUMMARY. One of the major projects for collecting and treatment of sewage constructed in Sweden, the Käppala Sewerage Works is presented. The scheme comprises 60 km of trunk mains, siphons, pumping stations and treatment plant, all built in rock. Design capacity for the first stage, now in operation, is 450,000 persons and a future extension to 1 million is possible. Total cost on completion (1970) was A$28 million. The underground location of the works was found economically viable and offered several advantages as ease of obtaining right-of-way and avoidance of disturbing encroachments on the urban environment.

1 INTRODUCTION

The capital city of Sweden, Stockholm was founded sometime before the 13th Century on a small island commanding the outlet of Lake Mälaren at the outlet into the Baltic. The City with its surrounding suburbs forms the largest metropolitan area in Sweden with a population of nearly 1.5 million.

The lake and the sea divide the land into thousands of islands, peninsulas and straits, thus forming a beautiful and extraordinary archipelago. The lake is the main source of fresh water for the whole region and the whole archipelago is by tradition a sacred and favourite resort area of the citizens.

It is therefore, natural that the increasing sewage pollution of these waters would cause public concern. This pollution became significant during this century and the situation deteriorated rapidly after the second world war when a great population influx to the region occurred.

The area north of Stockholm is scattered into several separate communities of various sizes. In the 1950's all of them faced the task of solving their sewage disposal problems. It was therefore natural (even if not readily obtainable) to look for a co-operation for a joint solution.

It was finally possible to create a joint organisation for 9 suburbs north of Stockholm with the aim of collecting all their sanitary sewage to one common treatment plant. The most suitable place for discharge of the treated effluent was at the eastern extremity of the island Lidingö where the outfall could be located in the main stream of water coming from Lake Mälaren. It was accordingly decided to locate the treatment works at that spot and the small village Käppala selected to harbour the treatment works was honoured by lending its name to the co-operative scheme. It was henceforth called the Käppala Association.

2 MAIN FEATURES OF PROJECT

The common sewerage system comprises trunk sewers, about 60 km of gravity tunnels, two inverted siphons in rock under sea straits, three pumping stations and a treatment plant also located in rock.

At the time of design the future population was estimated at 540,000 persons. As constructed the tunnel system has a maximum capacity of about 9m^3/s or 1 million people. The treatment works can be extended with a second stage, similar to the existing and of equal capacity.

3 IMPLEMENTATION OF PROJECT

The design of sewers, tunnels, pumping stations and treatment works was carried out by consultants. The works were constructed by contractors. Project administration and supervision of contracts was handled by the Association itself.

VBB (member firm of SWECO) was entrusted with the design of the downstream part of the tunnel system containing the two inverted siphons, the main pumping station and the treatment works.

4 PLANNING OF TUNNEL SYSTEM

Construction of a major trunk main system is a built up area using the cut-and-cover method is costly and causes considerable disturbance to the surroundings. Progress can be hampered by the need to obtain permits and easements in public and private lands. In this case the problems with a 'surface' system were further increased as the area to be served was intersected by numerous hills and valleys.

It was thus found advantageous to construct the main sewers as underground tunnels wherever possible. The advantages of tunnels were:-

- Ease of obtaining right of way. Upon application the Association was granted general permission by the Court of Water Law to construct tunnels notwithstanding any objection of affected land owners.
- Avoidance of costly and disturbing encroachments on the urban environment.
- Strategically located tunnels facilitated the connection of existing main sewers by gravity through access tunnels, shafts and bore holes.
- The geology of the area plus the available contractors' experience and equipment made tunnelling economically viable.

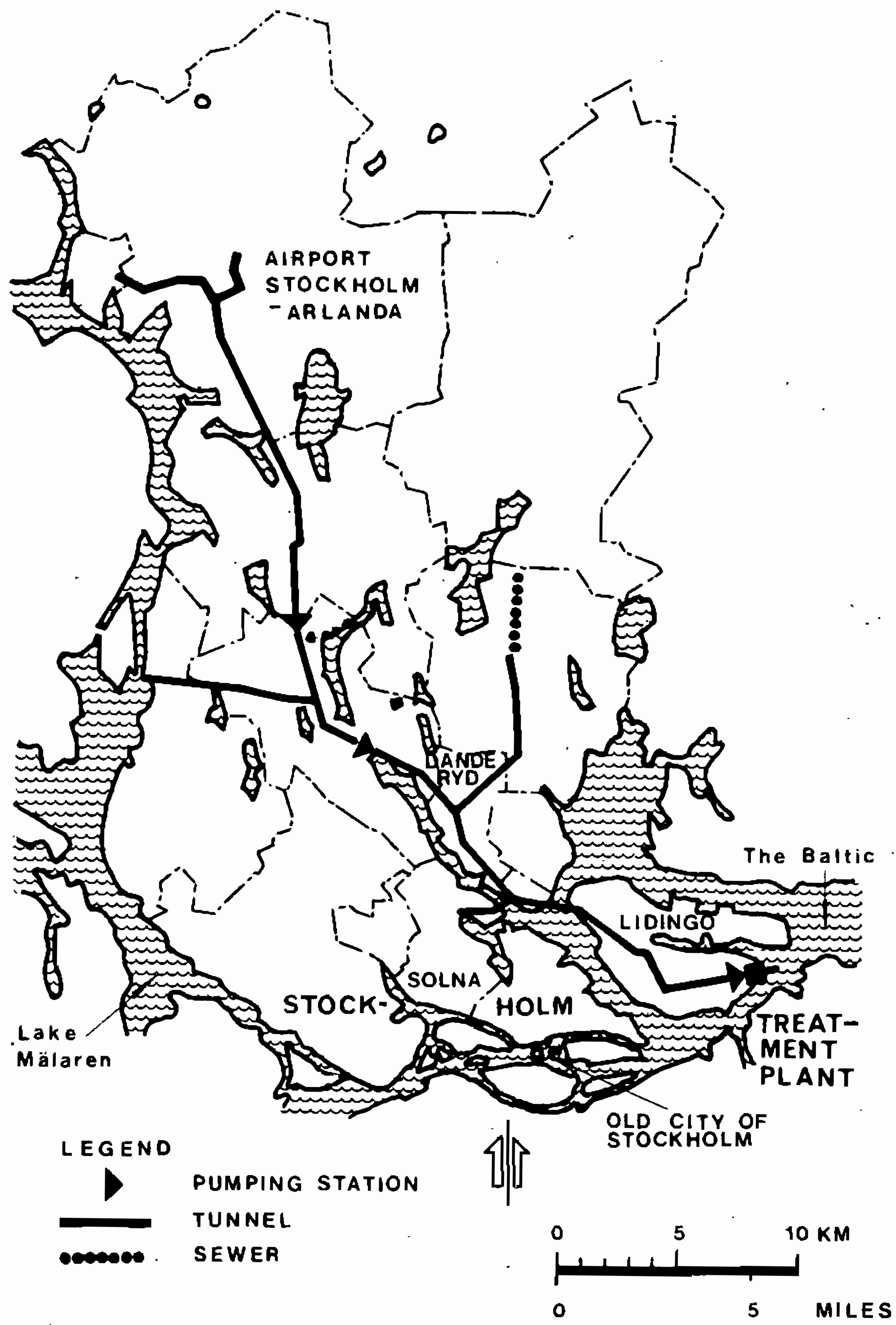


Fig. 1 Greater Stockholm area, the Käppala scheme

(a) Design Features of Tunnel System (Fig.1)

Starting from the treatment plant the main tunnel across the island of Lidingö, with a cross-sectional area of $8m^2$, starts 18m below sea level at the main pumping station. It crosses the island in a north-westerly direction to the strait between the island and the mainland. The strait is then crossed with an inverted siphon. On the mainland the cross-sectional area is $7m^2$ up to a point where another inverted siphon crosses another strait from the direction of Solna. From this point on the area is $5m^2$. Upstream of the dividing point at Danderyd the tunnels have an area of $4m^2$, which was considered the minimum practical size. The total tunnel length is about 60 km.

The tunnels are designed as gravity sewers with unlimited walls and roof, and a concrete slab V-shaped bottom (Fig.2). The gradient varies between 0.75 and 1.0 per thousand. From a strictly economic point of view it would have been better to make the slope even less. However, this slope was required to make the tunnels self-cleansing at small flows. The design capacity is reached when the water level touches the roof; normally the water level is lower. The masimum design flow was $7.4m^3$/sec. The flow was assumed to include municipal sewage and groundwater infiltration without large amounts of stormwater. As will be mentioned subsequently, the real capacity turned out to be considerably higher. Sewage in the eastern branch flows by gravity the whole way to the plant. In the western branch the sewage is lifted twice in pumping stations.

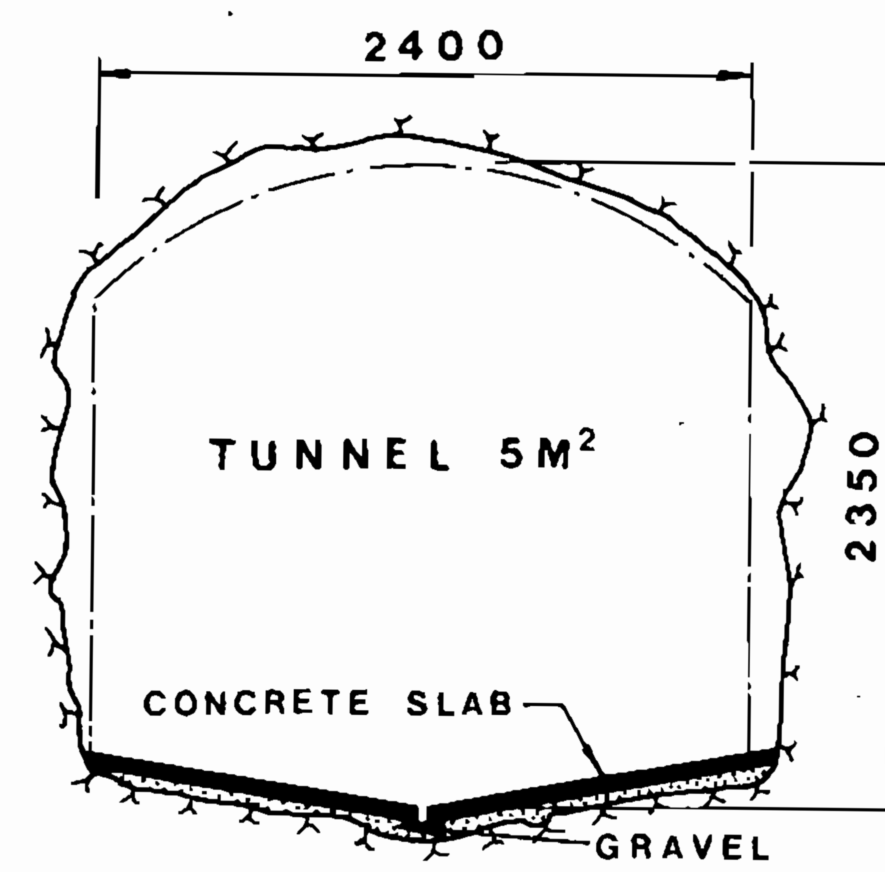


Fig. 2 Typical cross-section of sewage tunnel

(b) Planning of Gravity Trunnel

The planning of the tunnel system commenced with surveying, calculating, and co-ordinating existing maps to obtain fixed points from which to work. By seismic surveying of soil depths at critical sections a preliminary idea of the position and quality of the bedrock was formed. The method was time-saving and economical and has, as a matter of fact, turned out to agree well with later investigations. In areas with thin tunnel cover and in places where fissure zones were suspected the investigations continued with drilling from the surface. The most critical sections were specially investigated by diamond rock drilling and by an examination of drilled cores. The discovery of complex conditions often resulted in repositioning of tunnels. Tunnel levels were also lowered repeatedly as a result of discovery of successive weak zones, both to secure sufficient thickness of the roof and to protect exposed structures against effects of blasting. However, this lowering was only carried out when an estimate had shown the costs for reinforcement works and for damage, would be higher than the added costs of construction and future pumping.

A special problem to be considered was the risk of oxygen deficiency in the tunnels once in operation and bad odours escaping via the connecting sewers. To obtain air renewal and to prevent leakages to the outside, the system was designed to be kept under a slight vacuum by means of exhaust stacks containing fans at selected points.

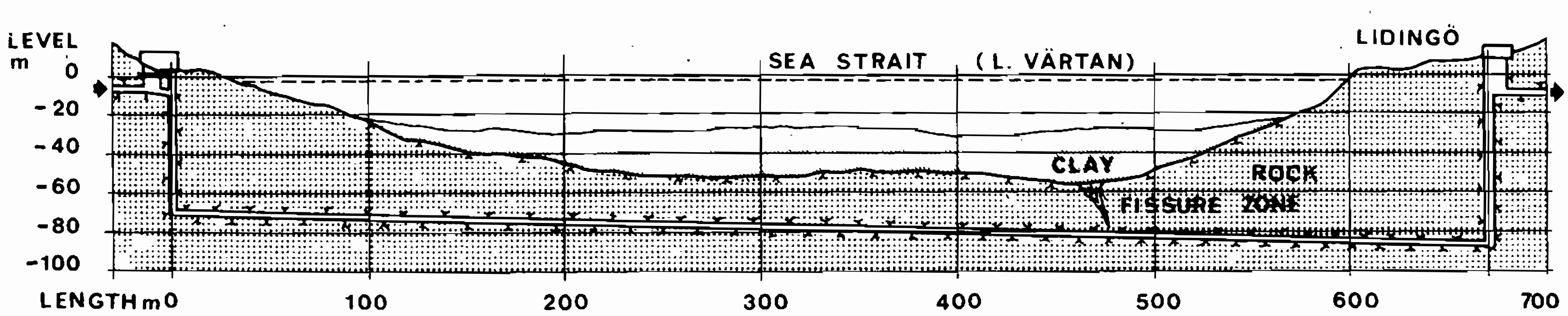


Fig. 3 Inverted siphon

(c) Design of Inverted Siphons (Fig.3)

The inverted siphons, and especially that connecting the mainland with Lidingö, presented the greatest difficulties in planning. With guidance from the seismic profiles a fissure zone was studied in detail by diamond rock drilling down to 90m below the sea level. The tunnel was finally placed 25m below the bedrock, i.e. 80m below sea level (Fig.3). The total length of the depressed tunnel is 650m. It is connected withthe tunnels on the normal levels through vertical shafts. This long siphon conveys raw sewage and is impossible to inspect during operating; consequently, its operation must be as unimpeded as possible. The siphon was made with a smaller area than the connecting tunnels to obtain a higher velocity; itis circular with a diameter of 2.4m and is lined with concrete. The upstream shaft has been provided with an intake for large amounts of sea water, by which device the siphon can be flushed regularly. As a final precaution the whole siphon can be taken out of operation and emptied for inspection and maintenance in less than 24 hours by means of a permanently installed pumping station using an ejector in the deep end shaft.

5 CONSTRUCTION OF TUNNELS

Due to the size of the project the construction of the tunnels was divided into several stages. The first stage started 1958 and the most remote parts were finalised in 1970. The tunnels were attacked through access tunnels and shafts; the intervals between these varied but were on average, about 4km. The rock was excavated by drilling and blasting only, while the muck was generally loaded with overhead loaders and transported by rail, in the access tunnels and outside the tunnels, trucks were used. Normally, the work was carried out in two shifts between 6 am and 10 pm with five working days per week. The average progress fora tunnel face was about 100m and sometimes up to 130m per month.

The rock in the area normally consists of fairly intact granites and gneisses. The tunnel work as a whole could therefore be completed without excessive difficulty. As has been mentioned above, the level and quality of the bedrock in areas judged critical had been investigated before construction started. When passing exposed structures the ground vibrations were tested with vibrographs. This made it possible to adjust the rounds to keep vibrations sufficiently low.

In spite of the bedrock investigations, the tunnel construction work had, however, to be interrupted twice because of thin roof cover. This occurred in areas judged non-critical and therefore had not been investigated. In one such case, there was a considerable landslide of soil into the tunnel fortunately without any danger to workers. In this particular place the direction of the tunnel had to be altered. In the depressed tunnels under the sea - which work was judged somewhat hazardous - operations proceeded smoothly. This fact and the relatively few unexpected incidents during tunnel construction can be cited to prove that the amount of preliminary investigation was justified.

The problem which turned out to be the most unforeseeable, and which influenced the total cost considerably, was the water inflow. To maintain the existing groundwater level, the tunnels had to be very tight, the permissible inflow being set to 1.5 litres/sec/km (0.2 imp.gal/sec/mile). A considerable length of the tunnels had to be sealed by grouting to cope with this rigorous requirement. As subsequent grouting means disturbing construction work, grouting in advance might be a better solution for the future.

(a) Liability for Damages

According to the permission granted by the Court, the Association was liable for damage to other people's properties, in the first place structures and wells. To establish the extent of such damage, structures closer to the tunnel centre than 75m were inspected in advance by an impartial inspector; also wells within 300m from the tunnel were kept under observation. To protect itself against unforeseen occurrences, the Association signed a damage insurance policy with a well-known British insurance company. As it later turned out, this was a wise precaution as the compensation paid, mainly for dried-up wells and cracked chimneys was considerably in excess of the insurance fees.

(b) Design v. Actual Capacity of Tunnels

The capacity of the tunnels had been designed utilizing a filled cross-section i.e. no over-break, and Manning's formula. Roughness co-efficients had been chosen on experience from numerous hydro-electric power tunnels constructed in Sweden and was kept somewhat on the conservative side.

As a result of the natural constructional over-break and the Association's rigorously maintained principle in supervision of the works - no rock was allowed inside the theoretical cross-section at any time (i.e. excess area on one side of the tunnel was not allowed to compensate for minor intrusion on the other)-the actual capacity turned out to be about 30% higher than the design value. This higher capacity should cater for a future population of about 1 million.

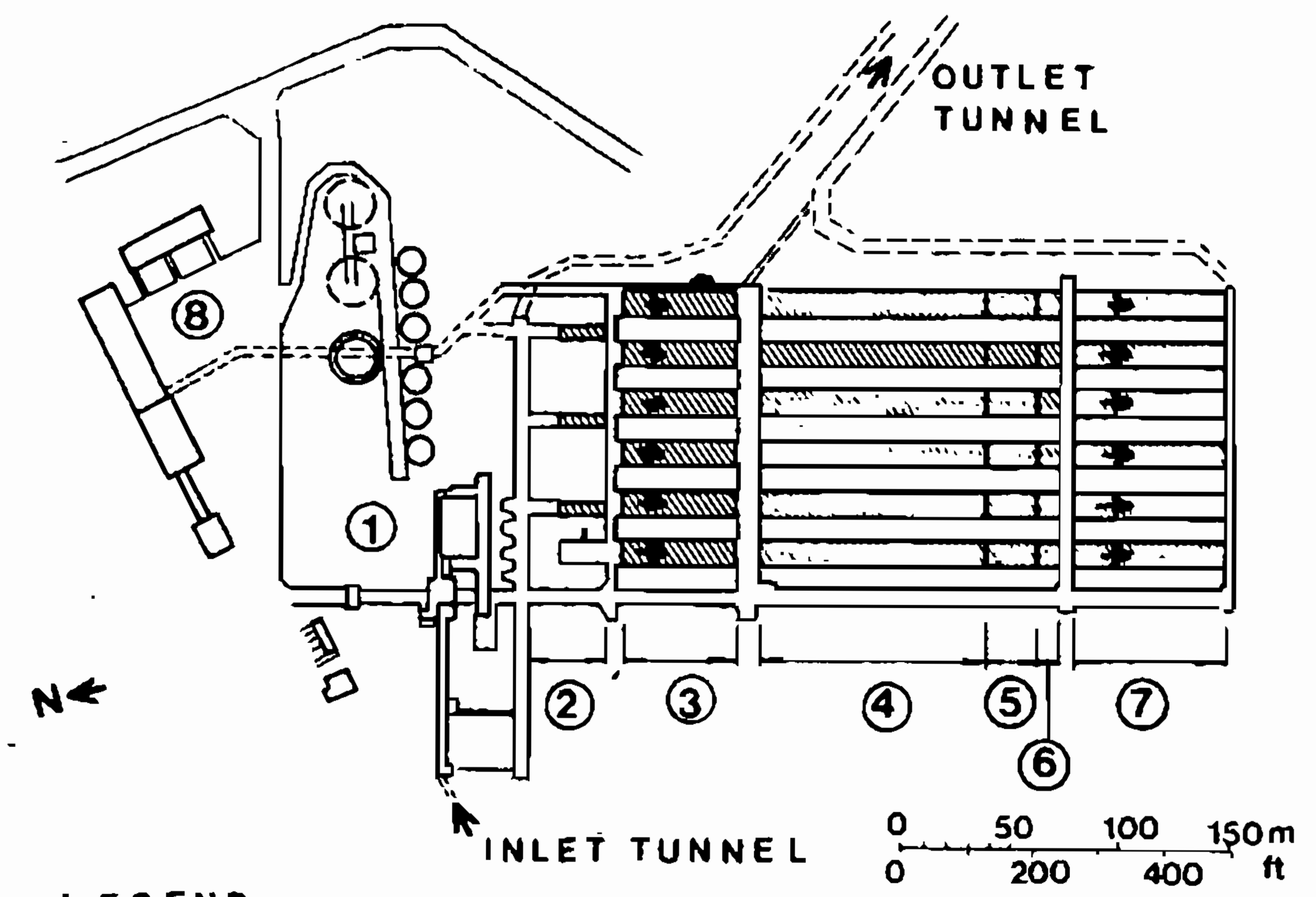
6 PLANNING OF TREATMENT PLANT WITH MAIN PUMPING STATION

The plant was located adjacent to the most suitable receiving waters available. The reasons for placing the treatment plant underground in rock were:-

- Shortage of suitable land in the vicinity
- Strong environmental pressure from the community selected to harbour the large, common plant for the Association.
- Once completed the extra investment for the underground location gives the advantages of an indoor plant; outdoor operation can be difficult during Swedish winters (mean temperature in January - -5° Celsius).
- Engineering know-how of similar works close at hand; the City of Stockholm already has three treatment works placed in rock, the first (in the world) constructed in the early 40's.

(a) Design Features of the Treatment Plant

It was required that the plant would have an efficiency of 95% BOD removal at average D.W.F. The first stage, now in operation, was designed for a D.W.F. of 2m^3/sec (40 MGD) corresponding to 450,000 equivalent units of population. The plant can be extended to a capacity of about 1 million people (Fig.4).



LEGEND

① PUMPING STATION
② GRIT CHAMBERS
③ PRIM. SED.
④ AERATION
⑤ LAMELLAE SED.
⑥ FLOCCULATION
⑦ FINAL SED.
⑧ OUTDOOR BUILDINGS

Fig. 4 Treatment Plant

The main tunnel reaches the site of the plant 18m below sea level. After screening, the sewage is lifted to the plant in a pumping station, which has a capacity of $10m^3$/sec (350 cu.secs) at a head of 22m. The sewage is conveyed to three aerated grit chambers and then to six equal-sized tunnels, each containing a primary sedimentation tank, activated sludge aeration tank, lamellae sedimentation tank, flocculation chamber and final sedimentation tank. The activated sludge is separated in lamellae sedimentation tanks. Before final sedimentation the sewage flows through flocculation chambers where chemicals are added for phosphorous reduction.

The outlet consists of a tunnel, utilised as a chlorine contact tank, and a wooden-stave pipe in the sea. The effluent is discharged at a depth of about 50m. The total detention time in the plant is about 10 hours at average flow.

After thickening the sludge is digested in anaerobic digesters and de-watered on vacuum-filters; the dried sludge is used for soil improvement. The gas produced is utilised for heating.

Apart from the fact that the tanks are separated by walls of rock. the general layout is very much the same as for an outdoor plant. Consequnetly the tanks inside the tunnels are normally provided with a slab of concrete on the bottom and concrete lining on the walls to obtain the desired true and smooth surfaces.

The plant is furnished with extensive, and in many cases novel, equipment for automatic remote control and operation, including a computer.

The only visible partsof the plant are auxiliary buildings for administration, control and operation, sludge dewatering, chemicals and power supply. The treatment works including the main pumping station and the digesters are all below ground in rock.

As the space within the hill selected was restricted it was desired to obtain as much free room inside as possible without requiring costly rock reinforcement. The plant lay-out consists mainly of six parallel tunnels, in which the tanks are located. Geological surveys determined the direction of the tunnels. The quality of the rock was judged to be good and the original lay-out with a tunnel width and a rock pillar width of 10m was altered to a more space saving pattern of 12 and10.5m respectively. The six main tunnels are 300m long and have a cross-sectional area of up to $120m^2$.

The advantage of an indoor location - all weather protection of process, equipment and personnel - demands extensive ventilation measures which must be considered at an early stage. As alterations are difficult and costly in a rock plant, careful planning is required. When the tunnels are excavated there must be enough space for activities involved in the final construction, equipment installation and operation.

7 CONSTRUCTION OF TREATMENT PLANT

The main pumping station and outfall was constructed in the years 1960 - 63. Work on the treatment plant started in 1965 and ended in 1969. All construction work, excavating and concrete casting was entrusted to one contractor; the work comprised about $250,000m^3$ of rock blasting and $17,000m^3$ of concrete.

As a result of the high degree of mechanisation the main part of the blasting was done with only four men per shift; two men drilling, one loading and one scaling. Four drilling machines were used, two mounted on trucks, and supplied with hydraulic telescopic arms. Tunnels with cross-sections up to $70m^2$ were excavated in full face heading; larger areas were divided into tunnel and bench. The muck was loaded with front end loaders and transported by trucks. One of the major problems for the contractor was to get rid of the muck, which was his property according to the contract. The main part was crushed at the site to be sold as building material. Part of the crushed material had to be dumped into the sea.

The predicition about the good rock quality proved to be right, there was no need for any extensive reinforcement. Originally walls and roofs above floor and water levels were to be uncovered. However, to eliminate all risk of falling stones it was finally decided to secure the rock surfaces with gunite.

8 COSTS

The total costs for the project were:-

	Million S. Crowns	A$Million (1A$ = 6.50 Sw.C)
Tunnels & Sewers	86	13.2
Pumping Stations	17	2.6
Treatment Plant	82	12.6
Total	185	28.4

The finance was obtained by loans raised by the Association. The member communities contribute their share of the annual costs in relation to population and quantity of sewage discharge.

9 CONCLUSION

The Käppala underground sewerage works, Stockholm, is one of several schemes for collecting and treatment of sewage which have been designed and constructed in Sweden.

Despite Sweden's comparatively low population density and lack of large metropolitan areas like for instance Melbourne and Sydney it has been found advantageous to place facilities such as public transport, electricity, telecommunication, gas, water, drainage and sewage underground in tunnels, excavated in rock.

Factors reinforcing this trend have been favourable rock conditions and ample resources of experience and capacity to execute such works and the experience gained from the comparatively large number of underground works for hydroelectric power, defence etc. in Sweden.

The Käppala sewerage works is an example of a project normally constructed on the surface or by surface techniques but which was found economically viable to construct underground in rock and which also resulted in the minimum disturbance to both the urban and natural environment.

Service Tunnels in City Areas

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SUMMARY. The aim of this paper is to acquaint the reader with the problems associated with the design and construction of relatively small service tunnels in cities using tunnelling and cut and cover techniques. The paper is based upon experience gained during the design and recent construction of two cable tunnels for the Postmaster-General's Department by the Australian Department of Housing and Construction. The tunnels were constructed in Castlereagh Street and Pitt Street, which are two of the main thoroughfares in the City of Sydney.

1 REASONS FOR CONSTRUCTING THE TUNNELS AND REQUIREMENT FOR NEW TUNNELS

The Postmaster-General's Department has had an extensive network of below footpath cable tunnels in the Sydney City area for many decades. In the past there was ample space in a tunnel which was large enough for a man to walk in to install the relatively few cables necessary. The rapid growth of the density of the working population over the last 20 years has resulted in a large increase in the number of cables installed in these tunnels.

In many instances the capacity of the tunnels has been reached, and in others cable routes are now required which are not catered for by the existing network. The high capital cost of installing trunk cabling necessitates selecting the shortest possible route.

The P.M.G. has used cable tunnels in preference to below footpath ducts or buried cables for the following main reasons :-

(a) Cables in a tunnel are relatively safe from disturbance due to accidents, excavation, vandalism etc.

(b) It is relatively easy to install additional cables as required. Full lengths of cable can be used as joints do not have to be located to suit surface pits etc. Jointing of cables is a time consuming task, each joint (which can only be worked on by one man at a time) taking several days to complete. (There are frequently 2,000 or more pairs of individual wires to be joined and tested). This activity can progress unimpeded by surface activities.

(c) Loss of operation of cables due to flooding or high water table is practically eliminated. In addition a visual inspection of the cables can be made as required.

The cable tunnels constructed in the past were placed immediately below the footpath and were of brick construction with vertical walls and arched roof, and on earth or rock floor.

Sub-floor drainage and concrete floors were later added. The tunnel dimensions inside were approximately 4 feet wide by 5 feet 6 inches high to the top of the arch. Access and ventilation was by means of footpath gratings and manholes. The ever-increasing risk of catastrophic damage by large scale flooding due to burst water mains, or explosion caused by a large petrol spillage or a gas main leak or, vandalism, has resulted in the P.M.G. requiring a new approach in the design of more recent cable tunnels.

The Department of Housing and Construction has constructed for P.M.G. several cable tunnels over the last ten years using both direct tunnelling and cut and cover methods. The two most recent tunnels which have been completed are discussed in this paper and are referred to as the "Castlereagh Street Tunnel" and the "Pitt Street Tunnel". Brief details of these tunnels are as follows :-

(i) Castlereagh Street Tunnel

This tunnel, approximately 530 feet long, was constructed between Martin Place and Richard Johnson Square, crossing Hunter Street (Refer figures 1, 2, 3). It was a two level structure for 380 feet leading up to a 3 level cable chamber which linked up with two smaller tunnels from Pitt Telephone Exchange. The section from the cable chamber to Richard Johnson Square was a single level tunnel. Provision was made at the intersection with Hunter Street for a single level tunnel extension to Chifley Square, which will ultimately lead on to a planned tunnel in Elizabeth Street. This tunnel was constructed using cut and cover techniques.

(ii) Pitt Street Tunnel

This tunnel was needed to connect the new City South Telephone Exchange in Castlereagh Street (between Bathurst Street and Liverpool Street) with existing footpath cable tunnels in Pitt Street. Planning for additional cable tunnels in Pitt Street in the future was allowed for when considering the need for this tunnel.

It was constructed mainly by direct tunnelling beneath an existing 3 storey brick building (Stafford House) which had previously been acquired by the P.M.G. The section of work in Pitt Street was done using cut and cover techniques (Refer figures 4, 5, 6). Extensive underpinning of Stafford House was necessary to enable the construction of this tunnel to proceed.

2 PLANNING FOR UNDERGROUND WORKS

During the planning stages of these works all relevant authorities were consulted and asked to state their requirements. In addition much effort was put into foreseeing potential problems and hazards, and these were also discussed with the appropriate authorities. As far as was possible the Department of Housing and Construction attempted to allow for as many methods of construction as possible in its planning, so that the various tenderers could offer many different methods for executing this work.

It is not considered practicable at this stage to assess the total community costs involved in works such as these. To attempt to do this would be a daunting task, as it would involve assessing the relative benefits and disbenefits of telecommunications and vehicular and pedestrian traffic affected by the works. Fortunately the authorities concerned with administering surface traffic co-operated to the fullest extent and it was possible to evolve a design and method of construction, which they considered caused minimum interruption and dislocation. Once the project was under way the contractor was obliged to liaise with the relevant authorities and to obtain their approval for any changes in the method of work.

3 DETAILS OF THE INVESTIGATION AND RESEARCH NECESSARY TO DETERMINE THE EXISTING SERVICES IN THE VICINITY OF THE TUNNELS

This was found to be the most time consuming operation in the design of these tunnels. The feasibility of the desired tunnel route could not be determined until all known services and their approximate locations had been determined.

It was usually expedient to make personal visits to the various authorities in order to get the most accurate up-to-date information. In addition it is necessary to learn as far in advance as possible of any future works the authorities may be planning in the vicinity of the tunnels.

The most difficult problems to overcome were generally those connected with stormwater and sewerage services, because both of these are dependent on gravity drainage and usually have house connections from buildings on either side of the street.

In many instances, services were so closely spaced that it was difficult to gain access through them for construction purposes.

In some instances the services were very old and the authorities concerned had only an approximate idea of where they were located. It was often necessary in an instance such as this to hand excavate pits to gain accurate information.

Test bores were put down along the centrelines of all tunnels except that portion of tunnel beneath Stafford House. Hand augering was generally used in overburden except in areas where for reasons of safety or where apparently conflicting services occurred hand excavated pits were dug.

4 SURFACE PROBLEMS IN OLD HIGH DENSITY CITY AREAS AND METHOD USED TO RESOLVE THE PROBLEMS

Conducting the site investigations did not present any major problems as it was possible to carry out much of the work in off peak times and in any case each investigation location was usually only occupied for a few hours.

The main problems encountered during the construction of the two projects were :-

(a) Pedestrian traffic;

(b) Vehicular traffic;

(c) Noise nuisance;

(d) Possible restrictions on retail traders.

(i) The Pitt Street site proved to be the most difficult as far as pedestrian traffic was concerned, as the full 12 feet width of the footpath over a distance of some 90 feet had to be opened up for the duration of the contract.

The method used was to construct in the footpath during the weekends a series of structural steel frames supported on the building footings on one side, spanning across the future excavation and supported on the road side on soldiers potted into rock via holes augered through the road pavement. This framework was covered with removable panels of plywood (refer figure 7). Thus the excavation could proceed at night by lifting sections of the panelling until head height was obtained underneath, at which time day work was commenced.

(ii) Vehicular access along Pitt Street, a one way street, was restricted from four 9 feet wide lanes to two lanes of 11 feet past the excavation with a total ban on stopping some distance either side of the excavation. An "A" Class hoarding was erected around the excavation. These arrangements continued until the completion of the work.

In Castlereagh Street, a one way street, half the road was closed between Martin Place and Hunter Street. This restricted traffic from four lanes to two lanes with no stopping applying for the full length. Around the closed section of road an "A" Class hoarding was erected with the exception of a Construction Zone, the treatment of which will be dealt with later on in the paper. Where the tunnel crosses Hunter Street hoardings were not used. The technique adopted was to place soldiers and beams in the pavement at night and plate over with steel plates until the future excavation was covered. Excavation was then carried out as a tunnelling operation under the steel plating with mucking out from both ends. This caused very little disturbance to the traffic.

(iii)It is considered that noise is a surface problem and will be treated as such. As both excavations were in sandstone and the use of explosives was not permitted, extensive use was made of compressed air equipment. In order to minimise the noise being emitted, silenced compressors together with muffled jack picks were used with some degree of success. The use of mufflers on the jack picks is not popular because of the icing problem and there is a tendency not to use them. One source of noise emission about which complaints were made was the air motors fitted to the belt conveyors. Petrol driven motors were found to be more acceptable. It might be noted that less complaints came from buildings which were air conditioned and did not have windows which opened than from non air conditioned buildings. Pedestrian traffic over the plywood panelling in Pitt Street caused complaints to be received from the owner of an adjacent high fidelity sound equipment shop. When the panels were able to be left in their permanent position they were covered with a layer of bituminous cold mix which proved quite successful.

(iv) While every endeavour was made not to restrict retail trade in adjacent premises, two complaints were received, one from each site. On the Pitt Street site the complaint was concerned with noise affecting clients listening to Hi-Fi equipment. After litigation it was agreed to work night shift until the tunnel under the building had advanced sufficiently to enable an acoustic curtain to be hung at the entrance to the tunnel. This agreement was in the form of an undertaking given to the Court and agreed upon by all parties rather than a court order as action by others could have been forthcoming to restrain night work. No further action took place nor were further complaints received.

The complaint from the Castlereagh Street site was made by a tobacconist, the only retailer on the side of the street where the excavation was being carried out. The nature of this complaint was that the hoardings obscured the view of his shop from passing trade. In the area in front of his shop the top part of the hoarding was removed and replaced with wire mesh so that there was minimum loss of visibility. It is not known if this proved successful, as it has not yet been determined if the tobacconist has suffered any loss of trade.

5 VARIOUS REQUIREMENTS LAID DOWN BY LOCAL AUTHORITIES AND UTILITIES

The authorities and utilities involved in the two projects either directly or indirectly were:-

Council of City of Sydney;
Parking Advisory Committee;
Department of Government Transport;
N.S.W. Police Traffic Branch;
Metropolitan Water, Sewerage and Drainage Board;
Australian Gas Light Company;
Sydney County Council;
Postmaster-General's Department;
Electricity Commission of N.S.W.;
Hydraulic Power Company;
Department of Labour and Industry.

Both projects were carried out wholly or partially in public roads vested "in fee simple" in the Council of City of Sydney and as a consequence Council laid down the conditions under which the work was to be carried out. The main requirements were:-

(a) Council would carry out at cost all pavement reinstatement;

(b) Temporary reinstatement to original levels including sealing of road was to be performed by the contractor;

This temporary reinstatement was to be left for some 3 months before Council undertook final reinstatement. Responsibility for maintenance of the temporary work was not accepted by Council.

(c) Type and location of all hoardings to be approved by Council. The original intention of the contractor was to use hoardings to cross Hunter Street closing half the road at a time. This was not permitted hence the plating over tunnelling technique was used.

The Parking Advisory Committee, Department of Government Transport and the N.S.W. Police Traffic Branch all had vested interests in the areas where the projects were being carried out and hence it was necessary to obtain their approval to any proposal. These authorities main concern was for traffic flow of private and public transport. Their requirements were that areas of works were kept to a minimum, that the least amount of parking space was taken up and that buses could safely manoeuvre. Some of the requirements impinged on the economics of the work.

The MWS & DB, AGL, SCC, PMG and EC and the HPC all had services in the area. Some of the services had to be permanently diverted because of the tunnels, while temporary support during excavation and concreting operations sufficed for others. All of the service authorities insisted that any alteration to their service be carried out by their own resources.

The Department of Labour and Industry became directly involved with the Pitt Street work because of the tunnelling operation under two lift shafts. During this operation the temporary removal of the lift counterweight stops was found to be necessary.

The D.L.I. would not permit the lifts to function with the counterweight stops removed. This involved the manufacture of a temporary counterweight stop which would become permanent when the tunnelling was completed.

6 TUNNELLING AND CUT AND COVER TECHNIQUES

In a congested city area the least disruptive method of construction is generally tunnelling. There are many other factors however, which influenced the actual method of construction adopted. In the areas of the city where cable tunnels have recently been constructed, the depth from pavement level to sound rock generally varies between 3 to 9 feet. The choices available in deciding the construction method are therefore to either keep the tunnel as shallow as possible using the cut and cover technique, thus minimising rock excavation, or lowering the tunnel to a depth which gives an adequate rock cover above the soffit and use a direct tunnelling technique.

In the case of the Castlereagh Street Tunnel the relatively short length between cross connections with existing shallow installations, coupled with the need to locate the tunnel so that as far as practicable gravity drainage could be used, dictated the choice of shallow construction using cut and cover. In the early stages of design it was hoped to precast much of this tunnel but the interference of existing services precluded this possibility.

There was little choice available in the construction method for the Pitt Street Tunnel. To prevent undue problems for the P.M.G. in the installation of cables, every effort was made to keep the soffit of that portion of the tunnel beneath Stafford House as high as possible within the limits of practical design and construction.

Broadly the work was carried out by the cut and cover technique with the exception of the tunnel under Stafford House (Pitt Street project). This was constructed by tunnelling with the use of steel sets.

The nature of the strata was similar on both projects consisting of a clay shale overburden (OTR) of up to 10 feet thick, then sandstone laminated with thin lenzes of shale and clay. The sandstone on the Castlereagh Street site was harder with fewer lenzes. It became necessary to make limited use of explosives in certain areas of the Castlereagh Street site to successfully excavate the rock.

In using explosives the following technique was adopted to reduce noise and vibration.

Holes 1-1/2 feet to 2 feet apart approx. 2-1/2 feet deep inclined at 1 in 10 were drilled in a dog leg pattern making alternate rows of 4 and 5 holes.

One row of holes was loaded with 1 inch dia. gelignite between 1" to 1-1/2" long, then with sand. A plug of clay wrapped in building paper was placed in the top 6" of each hole.

Once a set of holes had been fired the next set was then loaded and fired. All blasts were covered with heavy conveyor belting and two blasting mats.

Measurements of the blasting were taken by the Council of the City of Sydney without any adverse effects being recorded.

Where the cut and cover method was used the procedure was to first mark out the area to be excavated by means of lines painted on the pavement. The pavement was then cut with a diamond saw, hoarding erected, and excavation commenced. In all cases shoring was used to retain the O.T.R. This consisted of either soldiers anchored into rock by raked rock bolts or soldiers braced by struts across the excavation. Close timber walling was placed between the soldiers. Excavation in O.T.R. was carried out one foot wider on each side of the excavation than the actual tunnel structure. Excavation in rock was nett without shoring with the exception of an occasional rock bolt, where close jointed rock was encountered.

Prior to tunnelling under Stafford House, the following work was required :-

(a) Underpinning of two of the columns at the front of the building;

(b) Underpinning of the strip footing at the front on the building;

(c) Underpinning of the brick footing at the rear of the building;

(d) Underpinning of the lift well.

This work was done by cut and cover technique and once completed the tunnelling operation commenced.

The tunnel was driven from the front of the building working on two levels - the top level being some four feet in front of the level below. Steel sets were placed at 3 feet centres. The semi circular section being installed in the top level first, followed by the vertical post in the level below, once excavation at that level had proceeded far enough. Hardwood timber blocking was placed between the top of the sets and the edge of the excavation. Initially a set spacing of 4 feet was envisaged but because of the poor quality of the rock especially after exposure to the atmosphere the spacing was changed to 3 feet.

Once the tunnelling operation had been completed concrete placing commenced. Transverse construction joints were placed at 23 feet intervals along the tunnel. Longitudinal construction joints were used at the junction of the floor and walls and at the spring line of the arch, making three concrete pours between transverse construction joints. All concrete for the tunnel section was pumped.

7 COMMENTS ON THE IMPACT UPON THE COMMUNITY DUE TO METHODS OF CONSTRUCTION USED

The following comments are made in the light of observation recorded during the period of the contracts.

It was noted that once the hoardings were erected a change occurred in each area. The most noticeable change was the slowing down of the vehicular traffic while the pedestrians tended to move through the area more quickly.

The slowing down of the vehicular traffic is understandable as the width of the roadways had been restricted, however, this also applied to the footpaths with the opposite result. The reasons put forward for the improved flow of pedestrian traffic are that the hoardings obstructed the view, prevented the crossing of the road, except at certain locations, and limited the area available for congregation, while the nature of the work being undertaken did not make conditions conducive for conversation.

The hoardings once erected became one of the most popular places for people to plaster up graffiti.

The graffiti was continually removed, however, within a short period it was renewed with the result that the areas suffered visual pollution.

There is no doubt that the works increased the noise level in both areas even though all precautions were taken.

8 CONSTRAINTS PLACED ON THE PROJECTS DUE TO OUTSIDE CONSTRUCTION ACTIVITIES

When tenders were called for the work in Castlereagh Street, a building was under construction and a Construction Zone had been created in front of the building. It was anticipated that this zone would be removed within 6 months of the commencement of the contract. As the tunnel excavation passed through this Construction Zone and would have blocked all access to the building site this section of work was delayed. When the Construction Zone was still in use some 12 months later and the date of removal could not be determined a decision was made to plate over and tunnel underneath, in other words a similar technique was used as for crossing Hunter Street. The main factor influencing this decision was that the client could not tolerate the delay in completion.

The change in technique resolved the problem of access to the building site and was carried out as a variation to the contract.

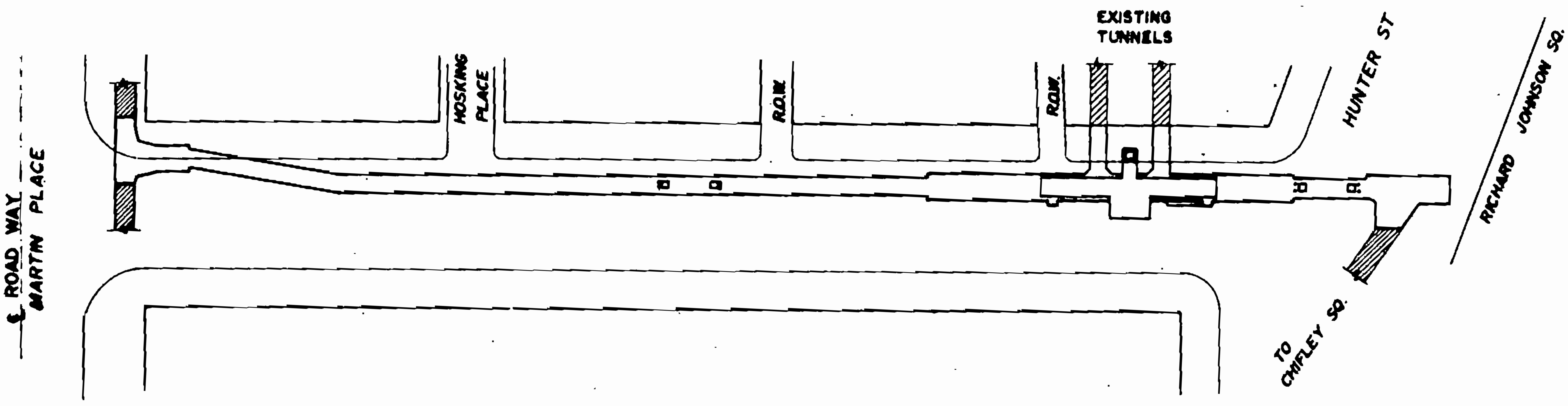

Fig. 1 Plan Castlereagh St. Tunnel.

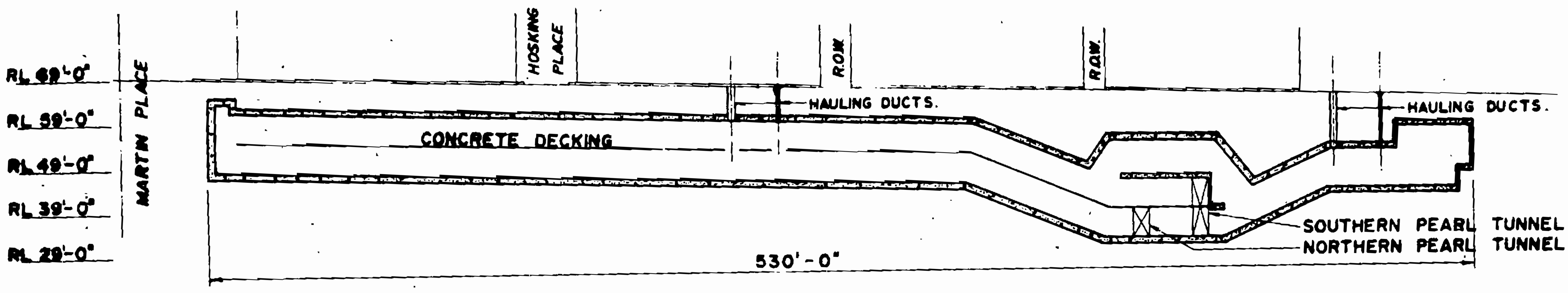

Fig. 2 Longitudinal Section Castlereagh St.

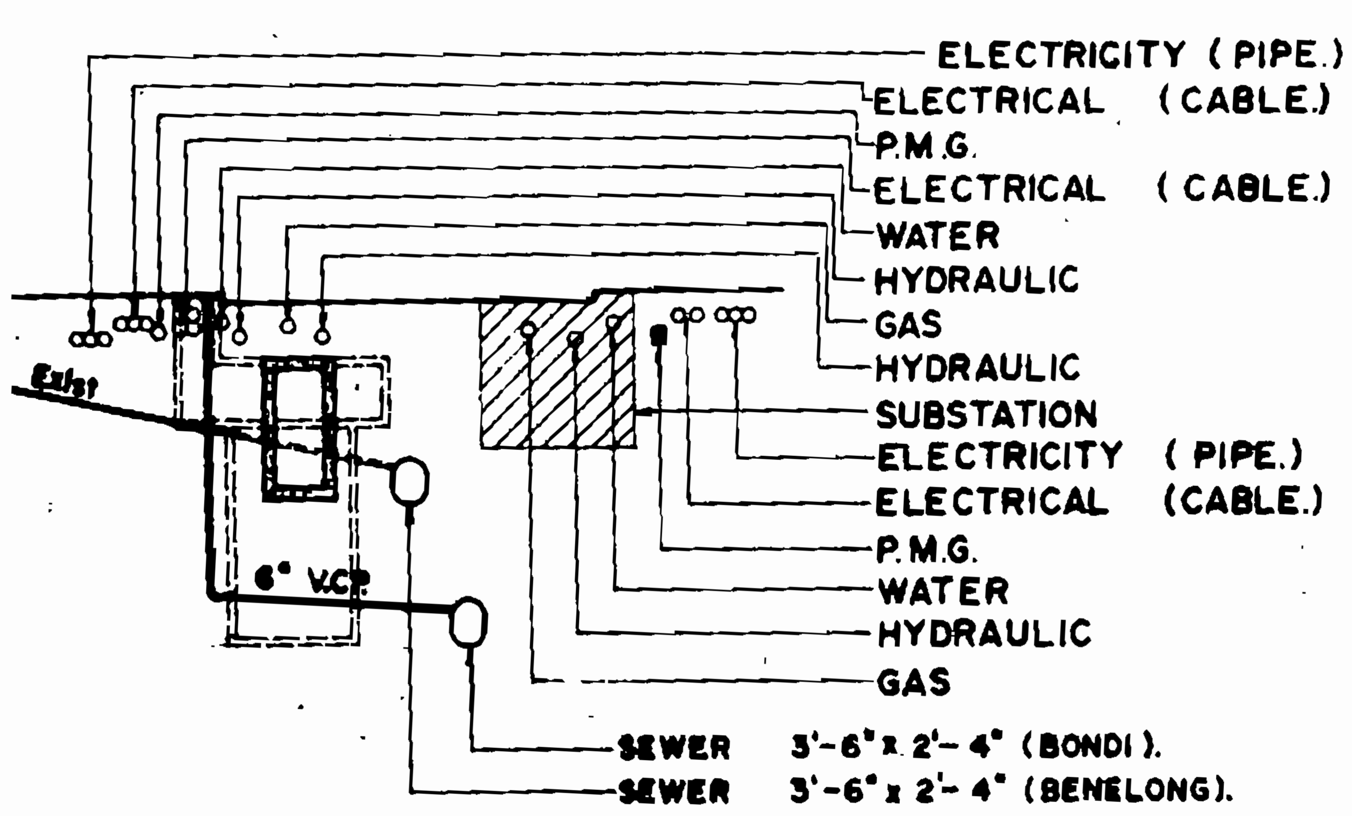

Fig. 3 Typical Cross Section Castlereagh St. Tunnel.

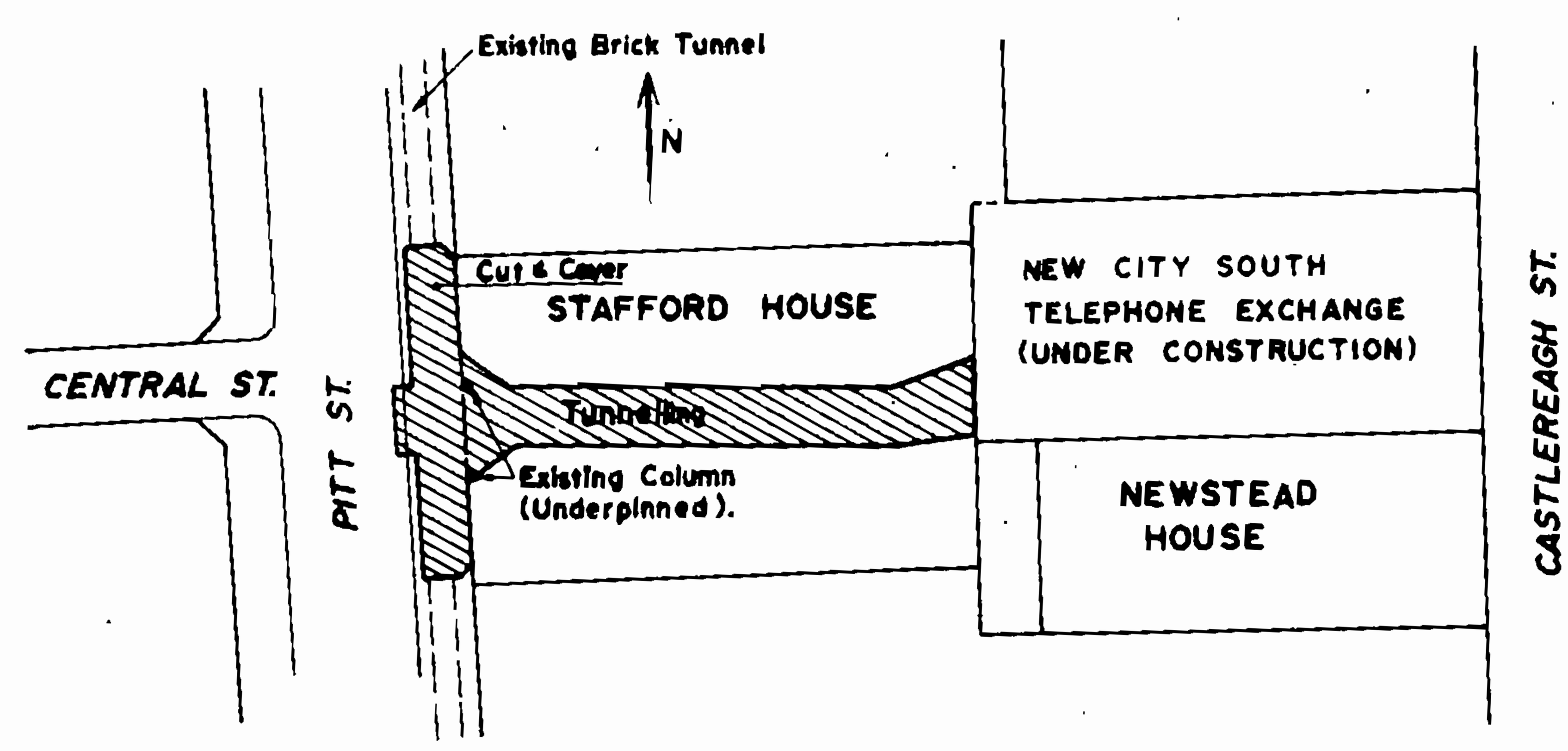

Fig. 4 Plan Pitt St. Tunnel.

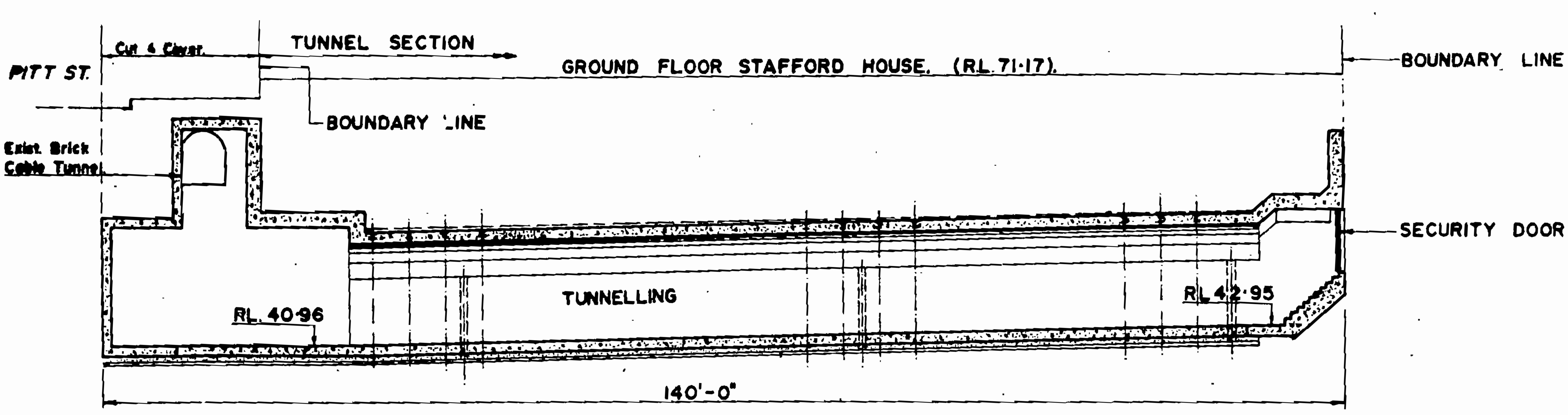

Fig. 5 Longitudinal Section Pitt St. Tunnel.

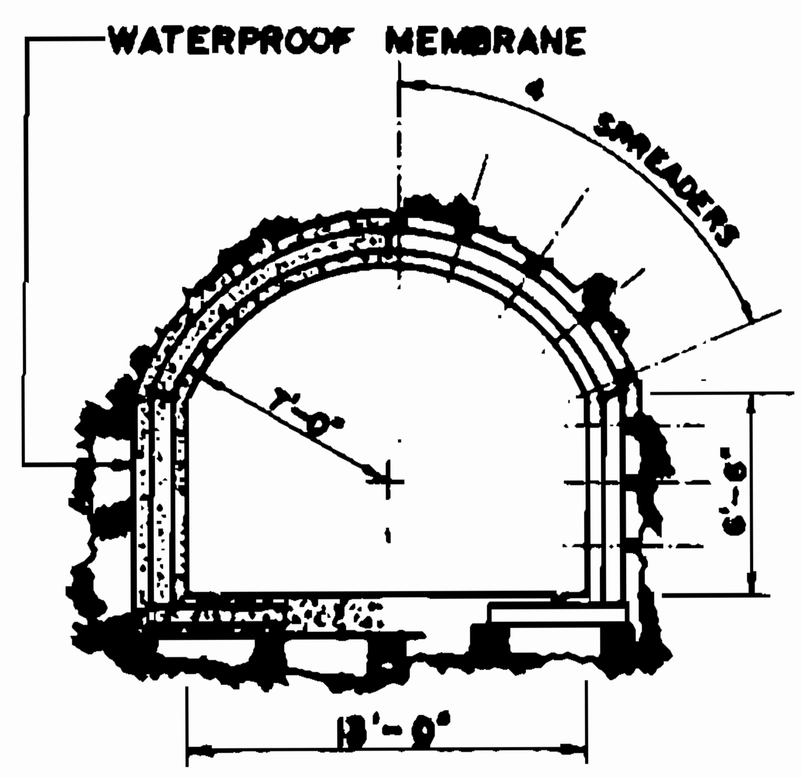

Fig. 6 Typical Cross Section Pitt St.

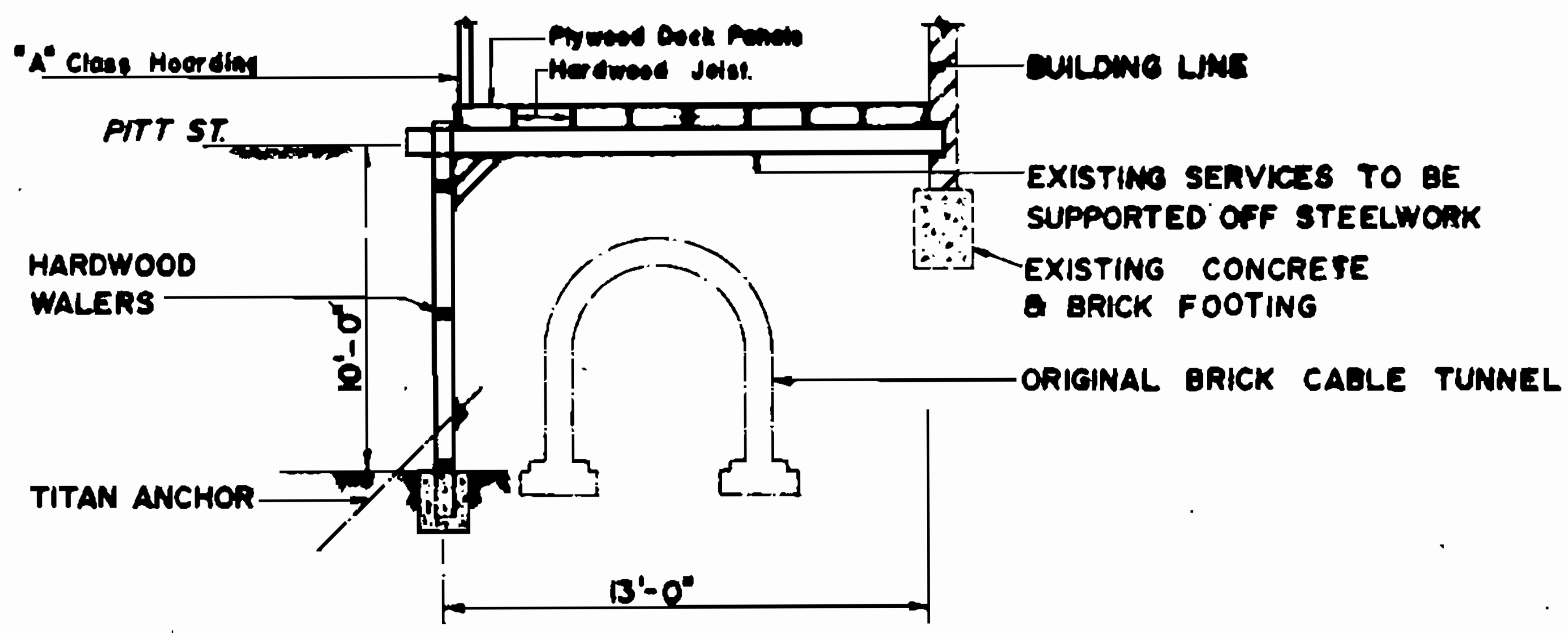

Fig. 7 Framework and Temporary Footpath Pitt St.

9 CONCLUSION

Whether cut and cover or tunnelling technique is used, there are problems some of which are common to both.

Those common to both are :-

(a) Disposal of excavated material;

(b) Noise;

(c) Material delivery;

(d) Site compound and storage of material.

Those pertaining to cut and cover technique:-

(i) Disruption of traffic;

(ii) Disturbance of access to local tenants;

(iii) Position and location of existing services both known and unknown;

(iv) The necessity for temporary support or relocate existing services;

(v) Backfilling and compaction.

Those pertaining to tunnelling technique:-

(i) The nature of the strata through which the tunnel will be driven;

(ii) The greater depth in order to avoid services;

(iii) Placing of concrete lining;

(iv) Access for other trades while tunnel is being built i.e. plumber, electrician, ventilator.

As can be seen from the problems that have been previously enumerated whether tunnelling technique or cut and cover is used depends upon the circumstances. There is no doubt that given certain conditions only one of the techniques would be a viable economic proposition. However, it is up to the Engineer and Planner to determine what is the marginal case and in doing so to fully consider all aspects.

Surely today, with the accent being placed on the quality of life, it is not unreasonable to occupy our minds in contriving methods which will have the least disruptive influences on the community as a whole.

Tunnelling Legislation (Victoria)

by

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1 INTRODUCTION

During 1961 a Bill was introduced to the Parliament of the day which amended the Victorian Mines Act (1958) and brought shaft sinking and tunnelling within the jurisdiction of the Mines Department.

The Bill resulted from a formal request by the Attorney General, who wrote to the Minister of Mines and expressed concern at the inadequacy of legislation at that time with respect to shafts and tunnels.

The Bill was debated at length by the Parliament and was eventually modified and passed and became law.

The Act to amend Division 2 of Part III of the Mines Act places the Victorian Mines Department in a unique position relative to other States. The New Zealand Mines Department exercises full jurisdiction over tunnelling operations but generally in Australia, particular operations need to be individually gazetted as mines before jurisdiction can be exercised.

Accordingly a variety of controls exist in Australian States.

2 TUNNELLING

Tunnelling is a very important aspect of civil construction activities and this has been recognized for hundreds of years. Tunnels are generally used for railway passages, roadways, canals, aqueducts, sewers or for drainage.

(a) General Description - The method used to establish a tunnel and the rate of advance depend on the nature of the ground in which it is established and this is generally pre-determined by test bores or shafts along the proposed line of establishment.

Where necessary, the roof and walls are supported by framed timber sets, arched masonry, steel ribs or concrete which may or may not be reinforced. Temporary support is currently being achieved by "shotcrete" skins with buttress ribs. The lining or support is normally carried over the floor to form a circular or horseshoe shaped structure.

Tunnels are normally established on a gradient to allow for drainage which is effected by side drains or by a culvert. A sewer may be constructed down the middle of the tunnel with inlets from gutters on either side.

Ventilation is provided by fans at the main openings in conjunction with ducting and incremental booster fans. Vertical shafts at intervals from the tunnel to the surface allow shortened ducted lines and exhaust points.

Broken rock may be removed by trains or rail-mounted trucks, by self-propelled trackless equipment or by conveyor belts. Tunnel advance is effected by the cyclic drill and blast method or by the continuous boring machine method. Shields may or may not be used, depending on the nature of the ground.

Good lighting is essential and this may be supplied by fixed installations or by portable lamps.

These are the basic features of all tunnels but there are many supplementary services and complementary practices associated with the activity.

(b) Historical Background - The record shows that the first tunnel was built by a Babylonian King in 2180 B.C. So much for the record. In fact, galleries and headings were driven in chalk by prehistoric man.

Early tunnels were generally established as aqueducts. Several of these have been renovated and continue to supply water to established communities. Athens currently obtains part of its water supply via a tunnel, 15 miles long, established by Hadrian around the year 115 A.D. and a tunnel established on the Isle of Samas in 687 B.C. continues to carry water supply after being renovated in 1882. The Romans built many tunnels but when that empire declined tunnel building also declined and was not resurrected for a thousand years.

Early tunnels were established in hard rock using primitive hand tools and fire setting methods. When the practice of tunnelling was revived during the seventeenth century, the construction methods were mechanized to a degree and many of the current practices were introduced with varying success. For example, a tunnel boring machine was designed, built and put to work a hundred years ago.

The advent of explosives, steam power, percussion drills, compressed air, track haulage, ventilation systems and geological knowledge advanced the technology of tunnelling until now the practice of tunnelling is a leader in modern construction activities.

(c) Tunnelling in Victoria - Tunnels have been established in Victoria for sewerage purposes, water supply, railway trains, communications, irrigation improvements, hydro-electric schemes and pedestrian traffic.

There are literally dozens of small bore pot and drive operations and other similar type construction activities in Victoria. They may not be individually significant but together, they represent a considerable amount of work.

In addition to these, there are nine major tunnels currently being established, or scheduled to be established in the near future, together with the associated ancillary construction works.

(i) Thompson-Yarra Tunnel Extensions
Two headings: 3.8 x 3.6 metre and 3.7 x 3.7 metre-horseshoe

(ii) Mentone Intercepting Sewer
One heading: 1.8 metre - circular

(iii) Dandenong Valley Trunk Sewer
One heading: 3.7 metre - circular

(iv) Gardiners Creek Main Relieving Sewer
One heading: 2.4 metre - circular

(v) Eumemmerring Creek Main Sewer
One heading: 2.4 metre - circular

(vi) Dartmouth Dam Headings
Four headings: 7 metre, 6.3 metre, 2.0 metre and 5 metre - circular

(vii) Lake Merrimu Project
One heading: 2.8 x 2.8 metre - horseshoe

(viii) Yarra East Main Sewer
Several headings: 2.3 x 1.7 metre-horseshoe

(ix) Melbourne Underground Rail Loops
Four headings (loops): 7 metre-circular and 6.9 x 6.9 metre-horseshoe

The total designed length of these major operations is in excess of 65 kilometres.

The establishment methods vary from drilling and blasting in hard rock, tunnel boring machines in soft to medium hard rock, and shields in soft ground. Alpine miners are also being used in soft to medium hard ground and are becoming increasingly popular

The tunnels generally are constructed for public purposes and are essentially government undertakings, financed by public money. Government Authorities and Instrumentalities generally conduct the geological investigations, prepare designs, and carry out preliminary work. Contracts are then let, and private contracting firms generally construct the tunnels under supervision. There are exceptions to this general practice and one notable exception is the Melbourne & Metropolitan Board of Works which has constructed several major tunnels.

3 DEFINITIONS

The majority of people accept that a tunnel is a horizontal, or slightly inclined, open ended subterranean passage way. They also accept that a shaft is a vertical, or near to vertical opening from the surface to a point underground. A shaft may connect to a tunnel or other underground working, or it may not.

Notwithstanding general acceptance, there are degrees of understanding and the term "tunnel" has been loosely applied. Excluding other meanings for tunnel such as a chimneystack, or a snare for game the term is often applied to adits, mine drifts, drives and cross cuts, and to cut and cover operations. This may lead to confusion.

(a) Technical Definition - it is important to establish clear definitions and one proposal put forward at a previous meeting of the Tunnelling Committee defines tunnelling as -

> "the construction by any method of a covered cavity of pre-designed geometry whose final location and use are under the surface and whose cross-sectional area is greater than two square metres"

This is a good definition, albeit a broad one which could well serve as a basis for detailed extensions. Technically, it could define a chamber or a covered trench but this is of no real significance.

The important fact is that engineers need to set down a generally acceptable definition, standardize the definition and then stick to it. By doing so, clarity will result and misunderstandings will be avoided.

(b) Legal Definition - The Victorian Mines Act defines a "Quarry" as -

(i) any place in which any operation is or has been carried on for the purpose of obtaining any mineral from the earth for commercial purposes and includes works machinery and plant used in such operations but does not include any place where gold or tin is sought or obtained by any method of mining;

(ii) any shaft which is more than fifteen feet in depth or in which explosives are used; and

(iii) any tunnel which is more than fifty feet in length or in which explosives are used.

The definition continues -

The provisions of Division 2 of the Act so far as applicable and with such adaptations as are necessary shall extend and apply to all quarries and every such quarry shall be deemed to be a "mine" within the meaning and for the purposes of the Division.

Accordingly, a tunnel is a quarry and a quarry is a mine.

I am not certain why a tunnel cannot legally be a tunnel and a shaft cannot legally be a shaft and perhaps they could. However, if they were, a lot more specifics would be needed and the rules would be cumbersome.

The draughting of legislation is a specialized activity. The aims and purposes are to produce uncomplicated statements which can be readily interpreted in the event of litigation.

With some practice and an acquired knack, legally draughted statements and definitions can be generally understood and appreciated.

4 LEGISLATION

For thousands of years men have made laws to regulate the affairs and to secure life and property of members of a society. The majestic succession of our western legal system stems from the early Greek law-givers, but the Greeks did not invent law, codes of law existed in Babylonia, when the Greeks were savages, and the Mosaic Law of Israel is ancient legislation.

The ancient laws were not democratic. They established order, but were primarily made for the purpose of enforcing the will of a god, a monarch, or a select group of leaders or priests.

The modern laws are democratic and stem from Lycurgus, who governed Sparta in the ninth century B.C. The Romans learned from the Greeks, and in turn the codes of Gaius and Justinian evolved to form the basis of modern statutory law.

Whenever the Greeks and Romans conquered a city, the law-givers set about their tasks as soon as the soldiers had finished theirs. In this way, law and order were soon re-established.

(a) Mining Legislation - When the Romans occupied Britain in order to exploit the mineral wealth of that country, they introduced mining legislation. The legislation was based on the Roman philosophy of engineering efficiency, and was comprehensive. It dealt with land use and included warden's courts. It replaced the Phoenician system which was mainly applicable to small mining operations.

Mining legislation prevailed from the Roman era, but it was not designed to cope with the frantic activity associated with the gold rushes. An attempt was made to control the situation by civil law but this failed, and the Eureka Stockade revolt is only one example of the futility of inappropriate legislation.

The Goldfields Act which came into operation in 1858 was based largely on Californian legislation. It was not adequate and as a result of a Royal Commission on the mining industry, the Mining Statute was proclaimed in 1865.

The Mining Statute (1865) is the foundation of all the mining laws of Australasia and it still remains embodied essentially in its original form. The Statute gave jurisdiction to the Chief Justice and the decisions and judgements made gave the law a firm and settled basis.

Mining law in Australia was practically made by the decisions of Mr. Justice Molesworth and the Supreme Court of Victoria.

The Statute defined the duties of the Department of Mines which included Miner's Rights, Mining leases, Leases of Reservoirs, Business Licences, Licences to Search, and Water Race Licences. Boards were set up to regulate the Statute and later Acts and Regulations increased the scope and the activity of the Department and the Statutory Boards and Committees.

Over the years, the variety of Acts and Regulations have been amended and consolidated to produce the principal legislation in current use.

(b) Tunnelling Legislation - Legislative jurisdiction over mining activities is more or less uniform. Jurisdiction over tunnelling activities is not.

In New South Wales, tunnelling is controlled by the Department of Labour and Industry under the Scaffolding and Lifts Act.

In Northern Territory there are no tunnels and the question of jurisdiction has never been raised.

There has only been one occasion in Papua-New Guinea where a project was inspected under the Mines and Works Ordinance.

In Queensland, the Mines Regulation Act does not apply automatically to tunnels, but there is a provision by which the Governor-in-Council by Order-in-Council may apply the Act to any excavation and this has been applied to a number of tunnelling projects.

In South Australia, an Amendment to the Act allows any civil project to be proclaimed a mine for a period of two years with renewal periods on an annual basis. Otherwise, control is exercised by the Engineering and Water Supply Authority.

In Tasmania, there has always been the ability to proclaim tunnels and other civil engineering constructions. Due to irregularities and inconsistencies it became necessary to make an overall proclamation to bring all tunnels for water or sewerage purposes under the Mines Regulation Act.

The only jurisdiction exercised in Western Australia is the relationship between constructing authorities and contractors.

Victoria exercises control under the authority of the Mines Act in a similar manner to New Zealand.

(c) The Mines Act (Part III - Division 2) - The Mines Act is a consolidation of several shorter Acts and Amendments. It is subdivided into five parts and each part is further subdivided into Divisions. There are seven Divisions within Part III. Division 2 applies particularly to "regulation and inspection of mines and mining machinery".

Within the Division there are eleven sub divisions headed -

1. Regulation of employees
2. Regulation of Mines
3. Mining Managers
4. Engine Drivers and Boiler Attendants
5. Boilers
6. Plans of Mines
7. Compensation to Employees and Actions for Injuries
8. Inspection
9. Inquests
10. Miscellaneous provisions
11. Regulations

These subdivisions apply generally to shafts and tunnels and should be understood. It has been the practice of officers of the Mines Department to apply the spirit of the legislation to shafts and tunnels rather than the letter of the legislation. The ability to do this is available in the definitive statement - "so far as applicable and with such adaptations as are necessary".

The clause enables limited discretion to be exercised judiciously.

However, the application of the spirit of legislation depends on co-operation to a large extent and there have been a number of difficulties in recent times and some problems have arisen. Accordingly, it has become necessary to be less general and more specific in the application of the rules.

(d) The Situation Prior to 1961 - There was little authority exercised over the humane aspects of tunnelling in Victoria prior to 1961 and in retrospect the resultant carnage was inevitable.

During the post war years the tunnel construction business boomed and millions of man hours and millions of dollars were spent in the establishment of tunnels for various purposes in Victoria and New South Wales. Modern techniques were introduced from overseas and our knowledge of the subject was advanced dramatically from a simple appreciation to a professional understanding which was evident in later programs.

These were boisterous days. There was plenty of work, plenty of money, plenty of labour and plenty of spirit. These were the days which produced the yardstick of "a man a mile" as a fair average. A modicum of control was exercised by the various Authorities over their contractors and this fluctuated with needs on the one hand and conflict of interest on the other. There was also some domestic conflict between walkers and engineers and between men from various countries in close association.

There are men who look back on these days with affection. There are others who look back in fright and I am one of them. In time, public pressure was exerted to introduce better controls and one result was the Explosives Act.

(e) The Explosives Act (1960) - The Explosives Act is comprehensive legislation which was introduced to control the manufacture, transport, sale, storage and use of explosives. The Act was administered by the Explosives Department which, at that time, was under the jurisdiction of the Chief Secretary. The Department later became a Branch of the Mines Department. The aspect of use presented difficulties in mining and allied operations as the qualifications and experience of officers of the Explosives Department were directed to the chemical and physical properties of explosives and safety in manufacture, carriage and storage. They did not have nor were they required to have a knowledge of the use of explosives, which can only be gained by practical experience in mines and works where explosives are used. Accordingly, the supervision of "use of explosives" in mining and allied operations was referred to the Mines Department.

In conjunction with this, when the Regulations of the Mines Act were redrafted, the Committee which drew them up recommended that consideration be given to bringing shaft sinking and tunnelling within the scope of these Regulations. A short while after, when several men were killed by a premature explosion in the Jindivick tunnel, it was further recommended by the Coroner, and later again by the Chief Inspector of Explosives as a corollary of the Explosives Act, in terms of "use of explosives"and other matters.

(f) The Mines Act Amendment (1961) - Following the introduction of the Explosives Act in 1960, the pressure to amend the Mines Act increased and this was implemented in 1961.

The passing of the Explosives Act made it essential to control explosives wherever they were to be used and this need was extended to give legal effect to competent persons to supervise and check other conditions which could be equally dangerous, such as a weak roof, faulty support, bad ventilation etc.

The Amendment defined shafts and tunnels as quarries, which was a legal definition as distinct from a technical one. By further definition a quarry was deemed to be a mine. Accordingly, shaft sinking and tunnelling activity automatically came within the scope of mining legislation, insofar as the legislation referred to the supervision of hazardous operations. In particular, and as a consequence of the Amendment, the concept of "Mine Managers" was introduced to tunnelling operations -

> "Every mine or quarry shall be under a manager who shall be deemed the mining manager of such mine or quarry - and who shall be responsible for the control management and direction of the mine or quarry".

This concept is not new in the general sense as qualified mine management was introduced at the beginning of the century. However, it was new in relation to quarries, shafts and tunnels.

(g) Application of the Mines Act Amendment - Since 1961, Officers of the Victorian Mines Department have regularly inspected civil construction activities.

They have concerned themselves with hazards peculiar to underground workings such as heavy ground and ground support, inflows of water, noxious fumes and poor ventilation, electricity in wet places, explosives handling and blasting, work in confined spaces with poor conditions underfoot, broken rock and exposure to falling materials, poor lighting, features of hoists and winches plus associated gear, dangers to the public and general standards.

They have not concerned themselves unduly with management in its various forms or levels or the ralationships between authorities and contractors.

Perhaps they should have been more concerned with management apart from recognizing nominated persons. The legislation allows that they should, and law makers should not be denied. However,there have been difficulties in this and compromise has been necessary.

The lack of a pool of skilled mining labour and the inability of contractors generally to train workmen effectively has been one factor; the shortage of experienced engineers and experienced supervisors has been another, and the lack of appropriate training for tunnel managers has led to compromise situations.

5 PENDING LEGISLATION.

The Mines Act and Regulations were framed to apply principally to underground metalliferous mining operations although there are provisions to allow applications to a wider range of activities in general terms. In principle, the bulk of the Regulations are applicable to tunnelling operations, but there are some anomalies.

(a) Construction Regulations - On the 15th May, 1962, the Industrial Safety Advisory Council recommended the formation of an Inter Departmental Committee to investigate the necessity for making regulations to cover all excavation work. This was in accordance wtih a recommendation made by the Board of Inquiry appointed in 1958 to enquire into and report on industrial safety in Victoria.

The Inter Departmental Committee met from the 13th November, 1962 to the 17th May, 1963 and reviewed existing statutory provisions governing the use of safe methods in shaft sinking and tunnelling, trenching operations and excavations associated with building construction, civil engineering, and other works.

A report was submitted to the Industrial Safety

Advisory Council in 1963 and amongst other findings contained in the report it was suggested that an amended Mines Act was the most suitable instrument for exercising control by statute over civil construction excavations generally.

During March, 1965, it was proposed that the Mines Department be the Statutory Authority.

A Conference on "Safety in Construction Excavations" was held on the 2nd September, 1965 between representatives of the various interested bodies and a report was submitted embodying the proposals. The various bodies and authorities engaged in construction activity were sent the amended proposals to which they agreed, subject to minor variations.

The proposed Construction Regulations were drafted by the 22nd March, 1967.

On the 23rd July, 1968, the State Mining Engineer recommended that a new Construction Excavation Act be prepared to facilitate the introduction of the proposed Regulations.

The recommendation was considered, particularly the aspect of trenching which presented difficulties and confused the issue generally. After review, it was agreed that the least complicated and simplest manner to introduce the regulations was by modification to the Mines Act. It was further agreed that the long term objective should be to introduce a separate Act to be entitled the "Construction Excavation Act".

The machinery of introduction of the regulations would be by four steps:-

(i) introduction of shaft and tunnelling regulations.
(ii) expansion of the General Operating Regulations to include regulations for trenching and other excavation work.
(iii) preparation and introduction of the proposed Act when the regulations have proved effective.
(iv) transfer of all enactments and rules concerning construction excavations from prevailing Acts to the new Construction Excavation Act.

The proposed legislation will relate to the safety and welfare of persons engaged in excavations, the prevention of injury to the public, the control of general nuisance factors and the avoidance of damage to buildings and other installations.

(b) General Operating Regulations - A considerable amount of work has been done to prepare guidelines and rules which are specifically applicable to tunnel construction work. Officers of the Mines Department are currently analysing these rules, in conjunction with existing regulations and other relevant legislation, to produce sensible, practical and appropriate Statutory Rules and Regulations for shaft sinking and tunnelling.

It is intended to produce a set of General Operating Regulations in three parts -

Part I	-	Underground Mining
Part II	-	Shaft Sinking and Tunnelling
Part III	-	Surface Mining

It is expected that this work will be completed and implemented within one year. A considerable amount of work has been done, but a lot of work still needs to be done.

The provisions of the Mines Act (Part III) which do not apply specifically to shaft sinking and tunnelling are also being studied in detail and modifications are being drafted in amendment form to allow sensible application. Modifications will include aspects of tunnel management in terms of educational standards, operating experience, types of operations and areas of authority and responsibility. The basic philosophy will be the same as for mine management and quarry management but the details will differ and will recognize differences in techniques.

6 TUNNEL MANAGEMENT

The establishment of large tunnels requires up-to-date engineering practice and a high level of operating and technical skill.

It is inevitable that advance in technical skill is accompanied by an increase in responsibility and a recognition of status.

Registration, certification and licensing of responsible men and women is a hallmark of status, and the recognition of the ability of the person so endowed.

(a) Certification - Registration and certification is more than an academic qualification. It is a recognition of technical training combined with practical experience which enables men to exercise competent judgement in their daily tasks. It signifies that the person so recognized has been judged by his peers and has passed the tests of individuality, talent, training, experience, responsibility and integrity.

An industry that recognizes the need for certification has come of age. The recognition of this need by the tunnelling industry is a long time overdue.

Tunnelling is one of the oldest of occupations. Other much younger occupations have recognized the need for many years and certification of doctors, lawyers, marine officers, aircraft pilots, mine managers, surveyors, and nurses to name a few has been accepted without question. Of these, mine management is the occupation more closely aligned to tunnel management than any other and the registration and certification of mine managers in the State of Victoria has been an accomplished fact for seventy years.

The ability to certificate a tunnel manager has been within the scope of the Mines Act for several years. This ability has not been applied to any extent in the past, but it is now intended to pursue the matter, resolve it and apply it.

(b) Management - It is apparent on reading the minutes of the early meetings of the Victorian Mining Managers Board that there were problems and there was doubt.

There are supporters for every philosophy and there are also opponents. No matter what concept is introduced, there will be people who will oppose it, or who will offer alternatives. In a democratic system the majority will decide the merits of the matter and the concept will succeed or fail by the wishes of that majority. Once the issue is decided, it is then necessary to legalize the decision to enable the matter to proceed without impediment.

It is important to obtain acceptance of the main principles by the industry as a whole as conflict is unrewarding and can be destructive. Recognition on a broad basis is necessary in addition to local acceptance to facilitate reciprocity both interstate and overseas.

It is also important to standardize all details and to avoid the setting of standards based on "high flown" ideals. It is necessary to provide the appropriate training courses at proper levels and to provide for progression between levels.

The initiators of the concept of certificated management overcame their problems before we were born, and as we have a wide range of background knowledge to call upon - we should have little difficulty in overcoming our problems. The groundwork has been done. All that is needed is to settle the details.

(c) Training - It is important for all levels of management engaged in tunnelling activities to be adequately trained and instructed in all aspects of mechanized tunnelling operations.

Several years ago, tunnel supervisors made most of the operating decisions and were essentially a law unto themselves. The main criterion was speed of advance at all costs and many men paid for this philosophy with their lives. Nowadays, the majority of decisions concerned with tunnelling are made by qualified engineers, but in many respects the engineering qualification is not appropriate to the activity and some post graduate training is necessary.

There are several civil engineers associated with tunnelling who believe that the activity is their particular forte. This is not necessarily so. Civil engineers are not trained in geology, drilling and blasting, ground control or ventilation to any extent. This is the domain of the mining engineer. Civil and mechanical engineers will need to study these aspects of underground construction before becoming efficient tunnellers.

Appropriate training is one of the criteria of management certification and courses of instruction have been designed and will be available next year. The courses relate to various levels and range from basic technical training to supplementary specialised instruction in shaft sinking and tunnelling principles and practice.

The other criteria are practical experience, first aid training, explosives and blasting knowledge and experience, and a knowledge of appropriate legislation.

(d) Qualification - Three levels of technical qualification are necessary to cover all aspects of legal management.

Requirements of the top level include an engineering degree or diploma, plus appropriate supplementary training and experience. Requirements of the intermediate level include a technical engineering certificate, plus appropriate supplementary training and experience. Requirements of the lower level include supervisory training and experience. Application of these levels relates to a formula which classes operations in terms of number of men employed, production, size, location, capital involved and complexity.
At present, the number of men employed is the most important factor.

The importance of practical experience cannot be overstressed. I know of no trade or profession, excepting some branches of engineering, wherein grass roots experience is not a prerequisite to occupational status.

It cannot be denied that knowledge and experience may be acquired in a variety of ways, with little formality at times. This aspect needs to be considered by every group of peers who judge others. The paths to examination should be uncluttered and examination should be comprehensive and searching but not inflexible. Efficacy, born of integrity and intelligence must be recognized and evaluated. Efficacy should never be denied, no matter what the parameters may be.

(e) The Mine Managers Board - It is often said that Government is a collection of boards and Committees. This is a truth. The lawmakers evolve legislation in committee and generally delegate a board or a committee to administer that legislation. The nominated group of people gather the necessary officers to give effect to the matter and proceed on the guidelines laid down in the legislation.

One such group of people is the Board of Examiners for Mine Managers. Government boards are generally similar in composition. The members are drawn from industry and government with no particular bias. The Mine Managers Board is typical. There are three members. Two of these are practicing mine managers and the third is the Chief Inspector of Mines who represents government. The secretary and all secretarial services are supplied by the Mines Department who also provide the venue for meetings and meet all other costs, including member's fees. It conducts its own affairs without impediment and sets its own standards within the limits of legislative ability.

In order to cope with the added responsibility of tunnelling activity the complement of the Board will need to be increased to four to include a member who is experienced in tunnelling. Provision for this has already been made, but an additional member has not yet been appointed.

7 COMMENT AND CONCLUSION

Tunnellers have a long and noble history, written not in books and manuscripts, but established in subterranean passages in many parts of the world.

Notwithstanding technological advances, tunnelling is a hazardous activity and many lives have been lost. Tunnel boring machines have lessened the danger to some extent but not entirely. The machines are not always appropriate and many tunnels will need to be established by other methods.

Skill and experience and discipline will always be necessary and many lives will depend on the actions of one man and his knowledge.

The need for competent technical managers and supervisors engaged in tunnelling activities is paramount. There is no substitute for effective co-ordination and control by a responsible person vested with proper authority.

Rules for the safety of persons and for the protection of surface installations are also necessary. The rules need to be sensible, practical and appropriate but they must be firmly established and must be enforced for the common good.

TABLE I

MAJOR TUNNELS - VICTORIA

	Name	Length (metres)	Section (metres)	Rock	Method	Commence	Complete
Thompson-Yarra	Western Extension	5183	3.8 x 3.6	Mudstone	Drill/Blast	1973	1975
	Swingler Access	300	3.7 x 3.7	Shales etc.		1973	1974
Mentone	Intercepting Sewer	2683	1.8 dia	Sandstone	Machine	1973	1975
Dandenong Valley	Trunk Sewer	8335	3.7 dia	Siltstone	Machine	1974	1976
Gardiners Creek	Relieving Sewer	7225	2.4 dia	Sandstone	Machine	1974	1977
Eumemmering	Main Sewer	6810	2.4 dia	Sand/Clay	Machine	1974	1976
Dartmouth	Upstream Heading	340	7.0 dia	Granitic	Drill/Blast	1973	1974
	Downstream Heading	352	6.3 dia	Greisses		1973	1974
	Access Tunnel	350	2.0 dia			1973	1974
	Pressure Tunnel	500	5.0 dia			1974	1975
Lake Merrimu	Project Stage II	3960	2.8 x 2.8	Slates etc.	Alpine Miner	1974	1976
Yarra East	Main Sewer	3850	2.3 x 1.7	Sandstone	Drill/Blast	1974	1975
Melbourne	Northern Loop	6560	7.0 dia	Silurian	Drill/Blast	1972	
Underground	Burnley Loop	6590	also	Clays and	Apline Miner	1972	
Railway	Caulfield Loop	6591	6.9 x 6.9	Sediments	Machine	1972	
	City Circle Loop	6475					

TABLE II

FATAL ACCIDENTS - (COMPARATIVE)

	1961	62	63	64	65	66	67	68	69	70	71	72	73	Total
Mines	-	-	-	-	-	-	-	-	2	2	-	1	1	6
Shafts and Tunnels	-	-	1	-	1	-	-	4	1	4	1	1	5	18
Quarries	4	2	4	1	1	3	1	3	-	-	1	1	3	24
	4	2	5	1	2	3	1	7	3	6	2	3	9	48

Notes: There is no specific ratio of accident severity, but on average figures, fatal accidents represent ten percent of serious accidents numerically.

Tunnellers often present themselves as rugged individualists bending nature to their will. In fact, tunnelling is necessarily a team effort and the best results are obtained by the best teams. A rugged individualist who is not a good team member is generally a poor tunneller.

Tunnels are generally Government undertakings and it seems to me that better co-operation between members of Government bodies would achieve better results. The frictions that exist are probably due to misguided loyalties but they are not necessary, are wasteful, and are generally not in the best interest of achievement.

The importance of this work, the capital involved, the labour content and the services provided makes tunnelling an extremely significant activity. It will become more important in the future as the need for more underground road and rail traffic increases.

8 ACKNOWLEDGEMENT

The Author thanks Officers of the Mines Department and members of the tunnelling industry for their encouragement and support in preparing this paper. Particular thanks are due to Messrs. J.O. Challender and R.F. Murrell.

9 REFERENCES

The references used were of a general nature including various Memoranda, Departmental Reports, Acts and Regulations and a publication titled "Tunnelling in New Zealand" (Ministry of Works).

Contracting Practices for Underground Construction

by

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SUMMARY. Disputes and unnecessary costs attendant on construction of underground works may be minimized by careful consideration of pre-contract construction planning, the interaction of site conditions with design and construction, special handling of insurance, special forms of contract and of contractor compensation, and the manner of contract administration. A special report by the U. S. National Committee on Tunneling Technology gives guidelines for improved contracting practices for underground construction.

1 INTRODUCTION

It has been said in the United States that tunneling is a process begun by geologists and completed by lawyers. This reflects the common observation that underground construction is increasingly beset by dispute and contention, especially for urban projects, where there is a strong interaction between the new construction and existing works and facilities. The resulting cost and delay, plus the diversion of owners', engineers', and contractors' senior personnel to the arena of disputes, is a substantial deterrent to the realization of much-needed underground works.

In the belief that unnecessary squabbling is against the interests of all parties, the U. S. National Committee on Tunneling Technology has undertaken to prepare a set of guidelines for improved contracting practices for underground construction. The recommendations, scheduled to be published in September, 1974, are based in part on a world-wide survey of present practices, including some valuable input from Australian sources. It is gratifying to observe that in some respects, current Australian practice is in advance of that commonly prevailing in the United States. On the other hand, many of the areas that have given us difficulties have not been widely encountered as yet in Australian works, but may be anticipated as greater development of the urban underground proceeds.

In addition to actual disputes, much unnecessary cost is incurred in underground projects when the work is not organized with recognition of some of the special conditions commonly encountered in carrying out such projects. The following discussion of common problem areas and recommended approaches to their resolution is based on the author's personal experiences as well as on the Contracting Practices Study.

2 PRE-CONTRACT CONSTRUCTION PLANNING

One of the greatest sources of unnecessary cost in urban underground work is delay associated with third party agencies. The most effective attack on this problem is for the Owner to become much more involved in pre-contract construction planning than normally. Many matters usually left to the Contractor can frequently be better undertaken by the Owner, for two reasons. First, particularly if the Owner is a public agency, he may have a superior bargaining position with respect to third party agencies. Second, the Owner certainly has more time available, and can get started on protracted negotiations sooner than the Contractor. Consideration should be given to Owner provision of:

Rights of access to construction sites, not only for permanent work, but also for necessary work on adjacent sites (such as underpinning existing structures and constructing temporary detour routes).

Negotiation with regulatory agencies of reasonable requirements for maintenance of vehicular and pedestrian traffic and access to adjacent buildings, and amelioration of onerous restrictions on construction operations (such as anti-noise ordinances and environmental protection regulations).

Negotiations for contractors' working and storage areas, and areas for stockpiling and disposal of excavated materials, (particularly if available sites are limited), and clarification of use, access, and haulage restrictions.

Advance relocation of utility services that are in conflict with permanent new construction, and replacement of old, fragile utility lines with new conduits that can be safely supported during construction.

Advance demolition of structures to be removed, and in critical cases advance underpinning or support of adjacent buildings that will be affected by the new construction.

Preliminary negotiation of construction permits, at least to the stage of establishing project-wide agency agreements.

Advance procurement of critical long-lead permanent materials and equipment.

In general, the Owner will benefit by arranging in advance for work pertaining to existing facilities that will impede the main construction contract, and by conducting negotiations with third parties wherever his bargaining power or available time is superior to the Contractor's.

3 SITE CONDITIONS

A distinguishing characteristic of underground construction is that the actual ground conditions are never fully disclosed until they are uncovered during construction. That disputes may arise as to whether the actual conditions differ from those contemplated by both parties when they entered into the construction contract is obvious and inevitable. It is less widely appreciated that many owners pay heavily, through contingencies included in the contractors' tenders, for apprehensions concerning what conditions may be encountered, and whether and how the contractor may be compensated if unexpected difficult conditions are encountered.

To reduce these contingent costs, which the owner would pay whether or not the difficult conditions actually occurred, many underground contracts include a "latent conditions" clause giving the Engineer authority to determine that the actual ground conditions differ materially from those that were anticipated, and to fix additional compensation. This practice is to be encouraged. Unfortunately, some owners then proceed to include elaborate exculpatory clauses attempting to absolve themselves of all responsibility for the accuracy of subsurface information furnished with the tender documents. Such clauses merely result in the restoration of contingencies in the tenders. Furthermore, if the contractors feel that the Owner's subsurface explorations have not been thorough, they will surely increase their tenders accordingly.

In the long run, it is in the Owner's interest to have thorough subsurface investigations made, and to disclose all factual information obtained without disclaimer clauses. Some experienced tunnel owners go further and furnish professional interpretive reports by geologists and soil engineers, with a proviso that these are opinions, and the contractor may form a different opinion from the same factual data. The principle is that the maximum disclosure will elicit the minimum contingency in tenders, and that the owner will pay less for actual latent conditions than for imaginary ones.

Another distinguishing characteristic of underground construction, particularly in urban areas, is the need to support and protect overlying and adjacent structures and utility services. Frequently the need for such work depends to a considerable extent on the contractor's choice of construction methods and the care exercised in the execution of the work. If, as is normally the case, the Contractor furnishes public liability insurance, the design of underpinning or support measures for such cases can properly be left to the Contractor. The owner and Engineer should, however, provide designs for cases of structures that will clearly be affected regardless of methods or workmanship, and cases where advance negotiation with the building owner (and perhaps advance construction) is necessary to avoid serious delay in construction schedules.

It is desirable for the Engineer to stipulate minimum criteria for contractor design of structures for support of excavation, street and sidewalk decking, and all temporary works affecting public safety. This places all tenderers on an equal footing, and precludes the possibility of award to a contractor who bases his tender on a flimsy bracing system and a trusting insurance broker.

A third important characteristic of underground construction is that the design cannot be fully separated from consideration of construction methods. It is obviously wasteful for the Contractor to provide a temporary excavation support system (for either mined tunnels or cut-and-cover work), independent of the permanent support system designed by the Engineer to resist the same ground pressures. Nonetheless, it is desirable to give contractors as much freedom as possible in the choice of construction equipment and methods. A partial resolution of this dilemma is for the Engineer to specify a minimum amount of temporary support to be provided by the Contractor, and to incorporate these support members into the design of the permanent structure. A more thoughtful approach is for the Engineer to design the permanent structure in such manner that its elements can be installed as excavation proceeds, thus providing both construction and permanent support. The use of precast or prefabricated tunnel linings, in lieu of temporary ribs and lagging with cast-in-situ concrete, is a common example.

It is frequently necessary in underground work for the Engineer to specify restrictions on contractors' operations that are inherent in the design assumptions. One example is the restriction of dewatering, to preclude settlement of compressible soils and damage to existing structures for which no underpinning has been provided. Another is restriction of groundwater control, removal of bracing, and placement of backfill until sufficient permanent structure has been installed to resist ground and groundwater pressures, and to counteract buoyancy.

The common thread of these measures is that serious difficulties may be avoided if the Engineer carefully thinks through the interaction of design and construction, and clearly stipulates in the tender documents all essential restrictions on construction operations.

4 INSURANCE

One of the most controversial developments in underground construction is the concept of Owner-Furnished Co-ordinated Insurance (known in the United States as "wrap-up insurance"), under which the Owner provides insurance coverage for all of his employees, engineers, and construction contractors. Generally Workers' Compensation, Builder's Risk, and Public Liability insurance for the construction contractor, and Professional Indemnity Insurance for the engineers, are included. The concept has been applied to a number of multi-contract urban underground projects in the United States. The advantage to the Owner are:

A uniform, high level of insurance expertise may be applied to the management and control of safety and risks on the overall project.

A co-ordinated safety program, co-operatively involving the insurers, owner, engineers, and all contractors, may be implemented. If compressed air work is involved, a central compressed air medical facility serving all contractors is desirable.

A single insurance group handles all claims, minimizing coverage disputes between insurance carriers, promoting prompt resolution of claims, and minimizing third party complaints of uneven treatment.

It promotes an improved pre-construction survey of existing conditions, as a defense against claims of damage resulting from construction.

The ramifications of a co-ordinated insurance programme are wide, and any such programme should include certain precautions. First it is essential to maintain the Contractor's incentive to promote safety. One method is to provide that Workers' Compensation records be kept individually for each contractor, and that he receive all premium refunds earned by him through retrospective rating of his actual experiencê record against that used in establishing standard premium rates. Anticipation of such refunds would enable a contractor with a good claim-loss record to maintain a competitive advantage in tendering.

The Owner's assumption of the cost of settling third party damage claims does not encourage the contractor to minimize such damages. Therefore, a relatively low deductible per occurrence should be included in the public liability coverage to restore the Contractor's incentive. Nonetheless, the Owner and Engineer must be more thorough and specific in designating protective construction for existing structures, and in stipulating limitations on construction procedures, such as dewatering.

It is also essential that the Owner and Engineer clearly agree on the basis for evaluating the need for protective work. Absence of such agreement promotes "defensive engineering", in which the designer may increase the owner's expenditures for protective works in order to reduce the engineer's exposure to errors and omissions claims.

Despite these precautions, there remain substantial objections to owner-furnished insurance on the part of many contractors. Their principal objections are:

The Contractor is not represented in the assessment of liability and the negotiations of settlements. His experience record may be capriciously impaired by the Owner's insurance carrier.

The Contractor's normal insurance program for other ongoing work is disrupted. His reduced premium volume with his own insurance carrier, plus the cost of administrating separate plans, increase the Contractor's overhead.

Coverage for completed operations, and other interfaces with the Contractor's insurance, are difficult to co-ordinate.

Subcontractors and suppliers may not give any credit for owner-furnished insurance coverage. Conversely, they may not exclude the project from their own coverages, creating problems of "contributing insurance".

Without attempting a final resolution of these viewpoints, it is perhaps fair to say that owner-furnished insurance deserves careful consideration for complex urban underground construction projects, but that it should in no case be adopted without careful consideration of its consequences and the necessary precautions discussed above.

5 CONTRACT FORM

Many difficulties arise from attempts to force contracts for underground work into standard formats, such as Lump Sum or Schedule of Rates. The former is ill adapted to the uncertain site conditions inherent in underground work. However, a pure Schedule of Rates form may be equally bad, since significant variations in quantities from those set out in the Contract Schedule may cause large variations in the total cost. This invites tenderers to play games with the Engineer's estimate of quantities, placing high rates on items which they think will over-run. In addition, tunnel contractors generally engage in "financial balancing" (or "unbalancing", in the Owner's view), placing high rates on early work items to secure an early return of invested working capital. If actual ground conditions differ significantly from those anticipated by either the Owner or the Contractor, some bizarre situations can easily arise, with unexpected financial disaster or bonanza to either party.

The provisions for compensation should match as closely as possible the manner and time in which the Contractor incurs costs. Separate lump sum items for mobilization and for setting up major items of construction plant (which generally form a relatively large part of the total cost of an underground project), will limit the Contractor's need for "financial balancing" and his inclusion of unreimbursed financing costs in other payment items. Partial payment for permanent materials, and such items as structural steel supports, upon delivery to the site, avoids similar distortions in the Contractor's cash flow.

Work susceptible to accurate estimating, and dependent on the Contractor's selection of equipment and methods, may best be compensated for on a Lump Sum (or fixed rate per unit tunnel length) basis. This might apply, for instance to excavation, removal and disposal of excavated material, and tunnel lining.

Work for which the quantities (or even the need) are uncertain should be handled on a unit quantity rate basis. Examples are temporary ground support, handling "excess water", and consolidation grouting. Where there is a fair factual basis for estimating these items, simply calling for tender rates on the Engineer's estimated quantities may be satisfactory. Where the work is more speculative (e.g., from uncertain geology), it may be preferable to utilize "split quantity" items, calling for separate tender rates on successive increments of similar work, or to provide contingent items to be utilized if certain conditions are encountered. It is necessary in such cases to stipulate clearly the basis for comparing tenders and awarding the contract. Some owners stipulate the rates that will be paid for contingent items that are not included in the tender comparison, to preclude the uninhibited quotation of high rates for such items.

(It is worth noting that European practice generally involves an elaborated breakdown of the payment schedule, including many separate items for plant, equipment, and procedures to be utilized if required or directed. The principle seems to be to try to anticipate every condition that may be encountered, and to provide an agreed method of dealing with the condition and compensation for it.)

Recent developments in underground construction have brought forth an array of new equipment and techniques, such as tunnel boring machines, shotcrete and other innovative techniques of ground support, slurry-displacement diaphragm wall construction, and soil and rock anchors. Design and construction concepts are increasingly interrelated in these new developments. However, the Engineer may hesitate to stipulate (or to preclude) a specific technique without any way of knowing the capabilities of the contractor who will offer the low tender. It has therefore become increasingly attractive for the Owner to call for tenders on the basis of an official design, generally based on techniques available to a wide range of contractors, but to permit the submission of alternative tenders based on contractor-proposed designs and techniques. The requirements of the design, and the basis for comparison of tenders on the official and alternate designs, must be clearly included in the advertisement for tenders. This system is commonly used in Europe, but has had relatively little use in the United States.

The complications of uncertain site conditions, alternative designs and construction techniques, and third party influences on the work have led to increased consideration of cost-reimbursable types of construction contract, "Cost-plus-percentage-of-cost" is generally disparaged by owners as promoting inefficiency. "Cost-plus-fixed-fee" is perhaps better, but more difficult to adjust to changed conditions.

One useful variation is "Cost-plus-incentive-fee", which includes a base fee associated with a target project cost, with provisions for increase or decrease of the fee depending on whether the final cost is under or over the target. This requires agreement on the method of adjusting the target cost for changed conditions.

Another variation is "Cost-plus-award-fee", which also includes a base fee and a variable incentive component. A percentage of the variable portion is awarded to the Contractor on the basis of periodic review by the Owner of his performance in meeting the Owner's objectives, which may include cost, time of completion, and quality of the completed work.

A final alternative is "Design-Construct", or "Turnkey", in which the Owner provides only a functional description of the desired facility, and engages a single organization with responsibility for both design and construction. Though popular in the United States with private industry for process plant work, where minimizing construction cost is secondary to advancing early completion and production, this concept has found little favor in the public works field, where governmental regulations have traditionally required public bidding on a definite design furnished by the Owner. An obvious disadvantage is that after selection on the basis of a very limited design development, the turnkey team has little incentive to economize on the subsequent development of the design. In general, turnkey is suitable for underground projects only where construction time has an unusual priority over construction cost.

6 GENERAL PROBLEMS

A number of related problem areas have been addressed by the Contracting Practices Study, and a brief review is in order here. Fortunately, two of the most vexing areas in United States practice appear to be well addressed by standard Australian General Conditions of Contract.

The first of these is Escalation of Costs, which is a particular burden in underground work owing to the generally long duration of the contract, which not infrequently may extend over three to five years. The Standard Clauses for Rise and Fall of Costs address this matter, and are much to be encouraged. Unusual classifications of labour, materials, and equipment may require special modification or amplification of the standard clauses.

The second area is provision for resolution of disputes by arbitration instead of litigation. This is uncommon in the United States, but a Standard Clause in Australia. It is perhaps worth a comment that a substantial increase in the volume of underground work may impose a strain on the number of qualified arbitrators who may be available for such cases.

One practice, perhaps more common in Australia than in the United States, and deserving further attention, is prequalification. Apprehension over receipt of a low tender from an incompetent contractor inhibits innovative design and admission of novel construction techniques. Although it is legally possible in the United States for a public agency Owner to disqualify a low tender on the basis of "irresponsibility", the opprobrium attached to this procedure makes its use extraordinary. Limiting tenders to contractors prequalified on the basis of experience and performance of both the organization and the key personnel available for the project, tends substantially to upgrade the quality of underground construction work, and is therefore to be encouraged.

It is of interest to observe that in Europe, where the general practice is to limit tenders to a few invited contractors, there is very little litigation on construction projects, simply because a contractor who establishes a reputation for litigation will not be invited back. United States contractors, who know that public agencies have little power to disqualify the lowest public tender, have little inhibition about engaging their clients in litigation, and some seem to do so with considerable enthusiasm.

Some difficulties have arisen in public works projects where the lowest tender either exceeded the maximum amount of funds available or exceeded the Engineer's Estimate by a greater amount than permitted by agency regulations. This situation causes unnecessary expense to contractors in preparing useless tenders, and embarrassment to the Owner and Engineer. Where such limitations prevail, difficulties might be averted by announcing both the Engineer's Estimate and the Maximum Funds Available with the advertisement for tenders.

One final comment may be obvious but needs emphasis: the form in which the contract is drawn is less important than the manner in which it is administered. If the Owner and Contractor desire to settle their differences, they will rarely be frustrated in so doing by the wording of the contract. Conversely, if they are bent on disputation, the contract will not compel agreement. In this regard, it is important that the Owner's site representatives be knowledgable of the conditions that may be encountered in underground work, and that they be delegated sufficient authority to make appropriate adjustments in the contract if the anticipated conditions do in fact change. A rigidly administered contract, in which no change can be authorized except by a Contracting Officer who never goes near the construction site, is an invitation to dispute. Owners who gain a reputation for rigid contract administration (and there are, regretably, quite a few such in the United States) also gain a reputation for attracting inordinately high tenders for their work. After all, as Shaw observed, the difference between a flower girl and a lady lies not in how she acts but in how she is treated.

7 CONCLUSIONS

Obviously, not all of the procedure discussed above will be applicable to any particular project. Nonetheless, owning agencies contemplating extensive underground construction projects can benefit from considering these general principles:

Thorough pre-contract construction planning can significantly reduce unnecessary delays and costs.

Contingent costs for unanticipated site conditions can be reduced by full disclosure of all available subsurface information, and by provision for contract changes if differing site conditions are encountered.

Provisions for support and protection of adjacent existing structures, and the interaction between design and construction, deserve thorough consideration in the design stage.

Where multi-contract projects impinge on a large number of third party properties, consideration should be given to an owner-furnished co-ordinated insurance programme.

The method and timing of contractor compensation should be tied closely to the way he incurs costs. Provision should be made for equitable compensation for contingent work, whose need and extent may not be reliably forecast at the outset of the work.

In some cases, alternative tenders and cost-reimbursable types of contract deserve consideration.

The manner of contract administration is more important than the form in which the contract is drawn.

The U. S. National Committee on Tunneling Technology report on "Better Contracting for Underground Construction" includes extended discussions on many of the topics covered in this brief summary. While it would be presumptuous to suggest adoption in Australian practice of specific recommendations developed under different conditions overseas, it is respectfully suggested that study of overseas experience may enable Australian designers and constructors to forestall difficulties encountered elsewhere on underground construction.

8 REFERENCES

Better Contracting for Underground Construction, a report prepared by the U. S. National Committee on Tunneling Technology -- National Research Council, 2101 Constitution Avenue, Washington, D.C. 20418, U.S.A.

AN ASSESSMENT OF THE POTENTIAL OF UNDERGROUND CONSTRUCTION IN URBAN PLANNING

KEY WORDS: underground construction; urban planning

ABSTRACT: An examination is made of existing and potential uses of underground construction in urban planning, and also of various critical factors which will affect the extent to which the sub-surface will actually be used in the future. It is concluded that without an improvement in present technology, the use of underground construction in urban areas will be limited. However, there are already many situations in the urban context where subterranean construction can be justified. Continuing developments in sub-surface excavation technology will further extend the range of applicability of the underground planning concept.

REFERENCE: TOAKLEY, A.R. An Assessment of the Potential of Underground Construction in Urban Planning. Conference on Reshaping Cities Using Underground Construction, Melbourne, The Institution of Engineers, Australia. October 21-22, 1974. Preprints of Papers. pp. 1-6.

TERRASPACE - A HIDDEN RESOURCE

KEY WORDS: demand for tunnelling; Swedish tunnelling; subsurface planning; tunnelling; urban planning

ABSTRACT: The increasing urbanization makes ever greater demands on use of the underground space. At present many problems occur because of insufficient planning, registration or investigation for subsurface constructions. In connection to a Swedish R&D project, led by the author, this paper discusses facts and goals within the fields of knowledge to be covered in planning and utilizing the subsurface resources. These fields are 1) estimation of future subsurface demand, 2) current and future tunnelling technology, 3) geology and soil mechanics, 4) hydro-geology including technical and echological problems, 5) man's reaction to underground stay, 6) cost-benefit analyses, 7) recording and visualizing methods, 8) legislation, 9) liability for damages, and 10) relations between underground and above ground planning. The present main task for subsurface planning should be to call the attention of planners and decision-makers to the hidden underground resources, and to work for regular evaluation of subsurface location for different functions in all phases of planning.

REFERENCE: JANSSON, B. Terraspace - a Hidden Resource. Conference on Reshaping Cities Using Underground Construction, Melbourne, The Institution of Engineers, Australia, October 21-22, 1974. Preprints of Papers. pp 7-14.

PILOT TUNNEL INVESTIGATIONS INTO ASPECTS OF EXCAVATION AND PRIMARY SUPPORT FOR MELBOURNE UNDERGROUND RAIL LOOP TUNNELS

KEY WORDS: railroad tunnels, investigations, underground support, shotcrete, ribs, instrumentation

ABSTRACT: Two pilot tunnels were constructed for the Melbourne Underground Rail Loop to assess behaviour of primary support and construction methods, and provide exposures for future tenderers. The effectiveness of a thin shotcrete skin in conjunction with ribs or rockbolts in limiting rock movements over the tunnels for most circumstances was demonstrated. In addition to investigation methods of support, excavation conventionally and by small tunnel machine was assessed. Further assessment of the shotcrete-rib method was made on a preparatory contract prior to a full commitment to this method on major contracts.

REFERENCE: NEYLAND, A.J. and BENNET, A.G. Pilot Tunnel Investigations into Aspects of Excavation and Primary Support for Melbourne Underground Rail Loop Tunnels. Conference on Reshaping Cities Using Underground Construction, Melbourne, The Institution of Engineers, Australia, October 21-22, 1974. Preprints of Papers. pp.15-21.

SELECTION OF A MACHINE METHOD FOR THE EXCAVATION OF SERVICE TUNNELS IN THE MELBOURNE METROPOLITAN AREA

KEY WORDS: machine tunnelling, mechanical tunnelling, small diameter tunnels; tunnelling

ABSTRACT: The Melbourne and Metropolitan Board of Works is one of the urban authorities in Australia that has been involved in an extensive machine tunnelling programme. The Board has already used both soft ground and rock tunnelling machines to excavate small diameter tunnels and it is proposed to expand the use of this method in the future. This paper discusses the considerations and investigations to be made in deciding to use a machine method for the excavation of a small tunnel in either the soft wet sands or clays or the soft silurian mudstones that occur extensively around Melbourne. It also includes details of the machines already used and of the tunnels excavated. It has been found that provided the ground conditions are fully investigated, the tunnel design suitable and a properly selected machine is used, then the machine method has advantages of economy, greater progress, better tunnel conditions, improved safety and less disturbance to the community and despite some disadvantages, it is likely that there will be an increase in the use of small diameter machines in the future.

REFERENCE: SMITH, N.B. Selection of a Machine Method for the Excavation of Service Tunnels in the Melbourne Metropolitan Area. Conference on Reshaping Cities Using Underground Construction, Melbourne, The Institution of Engineers, Australia, October 21-22, 1974. Preprints of Papers. pp. 22-30.

DOWN-HOLE INVESTIGATION TECHNIQUES FOR UNDERGROUND CONSTRUCTION

KEY WORDS: borehole; field tests; pressuremeter tests; resistivity; rock; seismic surveys; tunnels; Young's Modulus

ABSTRACT: The use of three down-hole site investigation techniques are discussed with reference to tunnelling projects. Specific examples are given of results obtained from pressuremeter testing, cross-hole seismic traverses and electric well-logging. The application and limitations of the methods are briefly assessed in relation to conventional investigation practice.

REFERENCE: PECK, W.A. and WALKER, L.K. Down-Hole Investigation Techniques for Underground Construction. Conference on Reshaping Cities Using Underground Construction, Melbourne, The Institution of Engineers, Australia. Preprints of Papers. pp. 31-35.

MODEL TESTS FOR THE DESIGN OF UNDERGROUND RAILWAY STATIONS - EASTERN SUBURBS RAILWAY, SYDNEY

KEY WORDS: design; finite element; photoelasticity; stress analysis; subsidence control; underground structures.

ABSTRACT: Photoelastic and finite element model studies on underground openings close to the surface and subject to large external loadings are described. The model studies indicated that excessive deflection was probable beneath a multi-storey building and recommended an excavation procedure to minimise subsidence. Measurements during construction confirmed the model predictions. Model studies also confirmed tributary area and load redistribution methods for excavation support load estimations.

REFERENCE: WOROTNICKI, G. and VINCENT, R.J. Model Tests for the Design of Underground Railway Stations - Eastern Suburbs Railway, Sydney. Conference on Reshaping Cities Using Underground Construction, Melbourne, The Institution of Engineers, Australia, October 21-22, 1974. Preprints of Papers. pp. 36-44.

FACTORS AFFECTING THE UNDERGROUND THERMAL ENVIRONMENT

KEY WORDS: comfort factors; diffusity; geothermics; mass; shelter; thermal analogues and models; thermal comfort; thermal inertia

ABSTRACT: Living underground is seen as a definite possibility. The main thermal factors influencing this situation are shown to be the volume of subterranean space occupied, ventilation and the thermal connection of bounding (internal) surfaces to the earth mass. Recommendations are made regarding the parameters influencing thermal comfort in a test situation for the establishing of criteria.

REFERENCE: CONNER, J. Factors Affecting the Underground Thermal Environment. Conference on Reshaping Cities Using Underground Construction, Melbourne, The Institution of Engineers, Australia, October 21-22, 1974. Preprints of Papers. pp. 45-49.

URBAN HIGHWAY TUNNELS

KEY WORDS: cost comparisons; feasibility; geometric standards

ABSTRACT: In the past, highway tunnels have been a costly expedient limited to difficult situations such as river crossings. In recent years several factors have made highway tunnels increasingly attractive including escalating land and property values in cities; advances in tunnelling technology which have substantially reduced the real cost of tunnels and increased resistance to the environmental effects of surface and elevated urban highways both during and after construction.

The paper reviews these factors, examines the limitations of highway tunnels, and emphasises the importance of considering a tunnel solution at an early planning stage so that maximum advantages can be taken, free of many of the constraints which limit the alignment of surface roads.

REFERENCE: BARTLETT, J.V. Urban Highway Tunnels. Conference on Reshaping Cities Using Underground Construction, Melbourne, The Institution of Engineers, Australia, October 21-22, 1974. Preprints of Papers. pp. 50-54.