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SESSION 6

Maintenance and repair

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Session 6 - MAINTENANCE AND REPAIR
Keynote address

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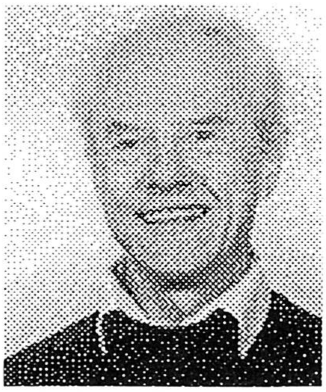

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Summary

The maintenance and repair processes on underground structures show the peculiarity that one of the structure faces is in contact with the ground. It is not visible, and not easily accessible. The main risk concerns water inflows and damages they are likely to produce. Another risk is the possible voids and decompressed zones, at the contact between ground and lining. The condition of the structure must be followed up on a regular basis, by means of non destructive (sonic, electric, magnetic, deformation measurements) or destructive (boreholes, cores and diagraphies) investigation methods.
ITA (International Tunnelling Association) published in 1991 a recommendation of the 'Damaging Effects of Water on Tunnels during Working Life'. This Association is preparing now the publication of a guide on repair methods.

Our session is devoted to the maintenance and repair of underground structures. Our paper is drafted on behalf of the International Tunnelling Association and its Working Group #6 on 'Maintenance and Repair of Underground Structures'.

1. Which structures are actually concerned?

Many purposes can be served by the underground infrastructures: These can be sewerage collectors, pressure gas ducts, water ducts; technical galleries for the passage of electric cables and tubes; pressure water ducts to supply hydro-power plants; metro tunnels, railway tunnels, road tunnels; underground caverns: metro stations, hydro-power plants, liquid gas or hydrocarbide storage caverns, Olympic pools, etc.

These structures are quite differing regarding their excavation mode, the composition of their external and internal structure, their inner equipment. But all have a common character: They are built in the subsoil (or under water for immersed tunnel caissons).

Regarding maintenance, their common characteristics are that they are submitted to an aggressive surrounding medium, situated on one side only of the structure wall, and that a free access is not possible to the rear side of this wall, to examine it or repair it.

Therefore, we will not consider here the structure parts which are not in direct contact with the ground. For these latter - either structures or equipment - we consider that they are subject to the usual maintenance and repair techniques developed for the structures in the open, bridges or buildings for instance.

Regarding this, it can be noticed that bridge foundations, located in the ground and non totally inspectable by their nature, could be assimilated to underground structures.

Moreover, we have to distinguish the structures that cannot be inspected by the personnel (tubes, ducts, and galleries under 1.5-2 m in diameter) from those that can be. We will pay particular attention to these latter. The first ones call for very special investigation and repair techniques, which are beyond our present purpose.

Among the various underground structures, if considering the construction methods, we must differentiate the excavated tunnels, the cut-and-covers and the immersed tunnels. In the following this distinction will be emphasised only when necessary for the proper understanding of repair modes that can be considered.

To summarise, we are concerned here by:
- the long galleries, collectors and tunnels (road, metro, rail)
- either excavated, or cut-and-cover, or immersed caissons
- and we are interested only in the structure elements in contact with the surrounding ground (or water)

2. Which is the extent of the problem, which are the risks that may be feared?

We must distinguish the old structures from the newest ones.

2.1 The old structures

In the 19th century, then in the first half of the 20th century, the excavated structures generally were lined with masonry or brickwork. The construction proceeded by excavating small galleries supported by wooden piles and planks. Between the masonry vault and the ground, the contact was not always very close, because voids were filled in with stones, mortar poured without extrusion, even perishable materials, such as wood. The encountered disorders then can be of two types:

- the ground, badly confined by the masonry, is getting decompressed, destabilised. Thus the masonry itself rests wrongly on the ground and can break up, resulting in possible crumbling or collapsing;

- the underground water circulation induces a progressive dissolution of the mortar bonding the masonry elements; the joints get weaker; the water inflows inside the gallery are increasing. In addition of the inconvenience brought by these water inflows to all inner structures and equipment, such dissolution can lead to the decay of the structure.

To summarise, for the old structures:
- walls are in masonry or brickwork;
- the contact with the ground is not always well ensured;
- the lining is not quite waterproof and the joints can get dissolved.

2.2 The recent structures

Essentially since the second half of the 20th century, they are lined with framed concrete, either unreinforced or reinforced, or with prefabricated segments for the shield-driven tunnels. Also, during the construction process, precautions are taken on the one part to ensure a good confinement of ground, and on the other part to guarantee a proper inner waterproofing.

To summarise, for the recent structures:
- the walls are in framed concrete, or made of prefabricated, generally concrete segments;
- the contact to the ground is properly ensured;
- the lining is theoretically waterproof, except in some weak points (construction joints, shrinkage cracks).

2.3 Which damages can be feared most generally?

2.3.1 The first enemy is water.
As a first reason, due its only presence: On a carriageway, it can make the surface sliding; on overhead lines, it can induce short-circuits. By its presence in the air loaded with acid ions released by the combustion engine vehicles, it is a source of a particularly severe corrosion. For instance, attacks to aluminium frames, ruined in less than 10 years, have been recorded in the French-Italian Mont-Blanc tunnel. In winter it can freeze and produce stalagmites and stalactites, both of them likely to cause damages or accidents.

Then, as a chemical agent weakening the lining: The water circulation, if water is very pure or corrosive to the lining, can widen the newly formed cracks and induces an increased output; this phenomenon worsens progressively. It can dissolve the ground behind the lining (gypseous ground or weathered dolomite).

In some extreme cases this could lead to a long-range decay of the structure.

It should be noticed, however, a strong difference with the bridge structures; the underground walls generally are not in pre-stressed concrete and often are unreinforced. They do not induce the same fear of cable or reinforcement corrosion.

Finally, if ideally the aim *a priori* is to obtain a properly waterproofed structure, it can be admitted that waterproofing cannot be total, and has not to be. Relevant recommendations have been published on this.

2.3.2 Then, the possible voids between the ground and the lining
They have already been mentioned: On the one part they can host water circulation (see above). On the other part, at their location, the lining does not lean upon the ground. This can have two consequences:

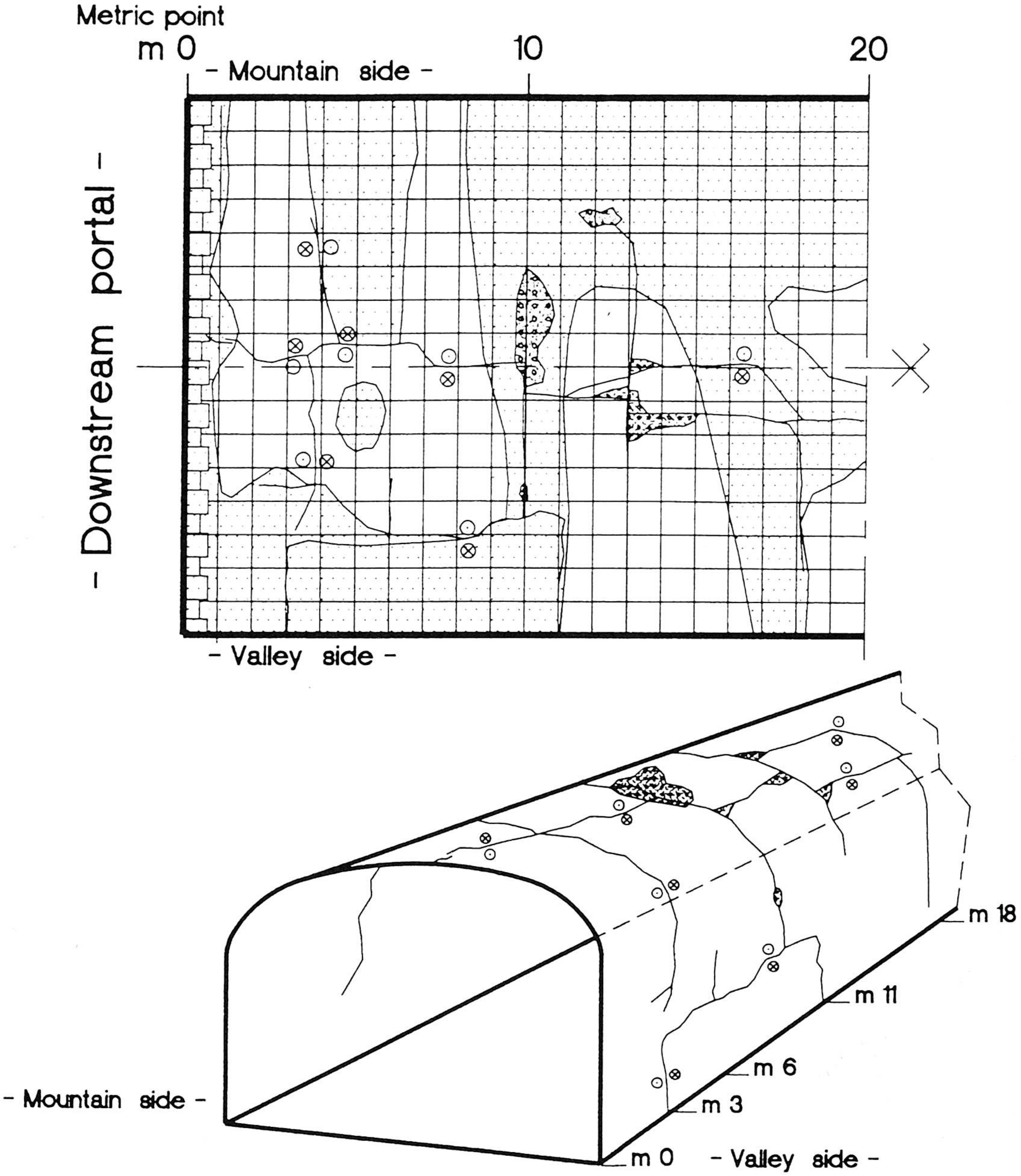


Fig. 1 Representation of cracking affecting a concrete tunnel portal

First, the unconfined ground, which is also submitted to stresses from the ground load, is then submitted to very unsymmetrical efforts (the stress deviator is high). It may get weathered, loose its self-supporting qualities, and the volume of voids in the contact zone with the lining may expand.

Also, in this zone, the lining itself is not confined. If submitted to high tearing efforts it can be led to break up.

The permeability of a rock mass can be increased by several powers 10 near the tunnel. This is the case for drill-and-blast tunnels, where no special precautions have been taken and where rock is cracked up to 1 to 2 metres in depth. This can be encountered on depths of about one radius in some deep tunnels where the stresses near the wall exceed the rock strength, especially if the breaking comes with dilatancy. As the opening of cracks is more marked when their longitudinal orientation is parallel to the wall, permeability develops mainly in the longitudinal direction. We must therefore keep in mind that the ground at the periphery of certain tunnels can be considered as a longitudinal drain.

Therefore it is essential to check, already at the construction phase, that no voids are left between the ground and the lining; then, during the structure working life, that possible water movements and dissolution are not likely to produce some.

2.3.3 Lastly, the ground itself can press increasingly on the lining with time (case of some grounds)
Most deformations in the lining, especially towards the cavity inside, correspond to comprehensive ground-lining movements, which must also be monitored very closely. They can correspond to insufficiencies in the lining bearing capacity, which did not exist when the structure was put into operation. They can result from a progressive deterioration with time of the geotechnical ground characteristics, due to the decompression caused by the construction process (softening phenomenon). The ground pressure, especially on the invert, can also augment if ground is likely to swell when saturated with water (swelling ground).

3. The main investigation methods

We differentiate the direct, visual establishments and the indirect establishments. These latter obviously concern the hidden parts of the structure, including the contact zone with the ground.

3.1 The direct establishments

They concern first the water inflows, then the cracks, lastly the lining deformations.

3.1.1 Water inflows
They can be seen immediately, except when too small and therefore corresponding to water quantities that evaporate as they reach the intrados. They correspond either to wet spots, or to seeping or flowing along cracks, or to seeping or flowing along construction joints.

3.1.2 The lining deformations
They generally are convergences, i.e. displacements of the lining inwards the structure. If there is any suspicion about their occurrence, they must be measured and their evolution followed up carefully.

According to the geological and geotechnical nature of the ground, adequate investigations will be conducted using non destructive geophysical investigations and boring and coring, until the phenomenon is fully understood and the remedies defined.

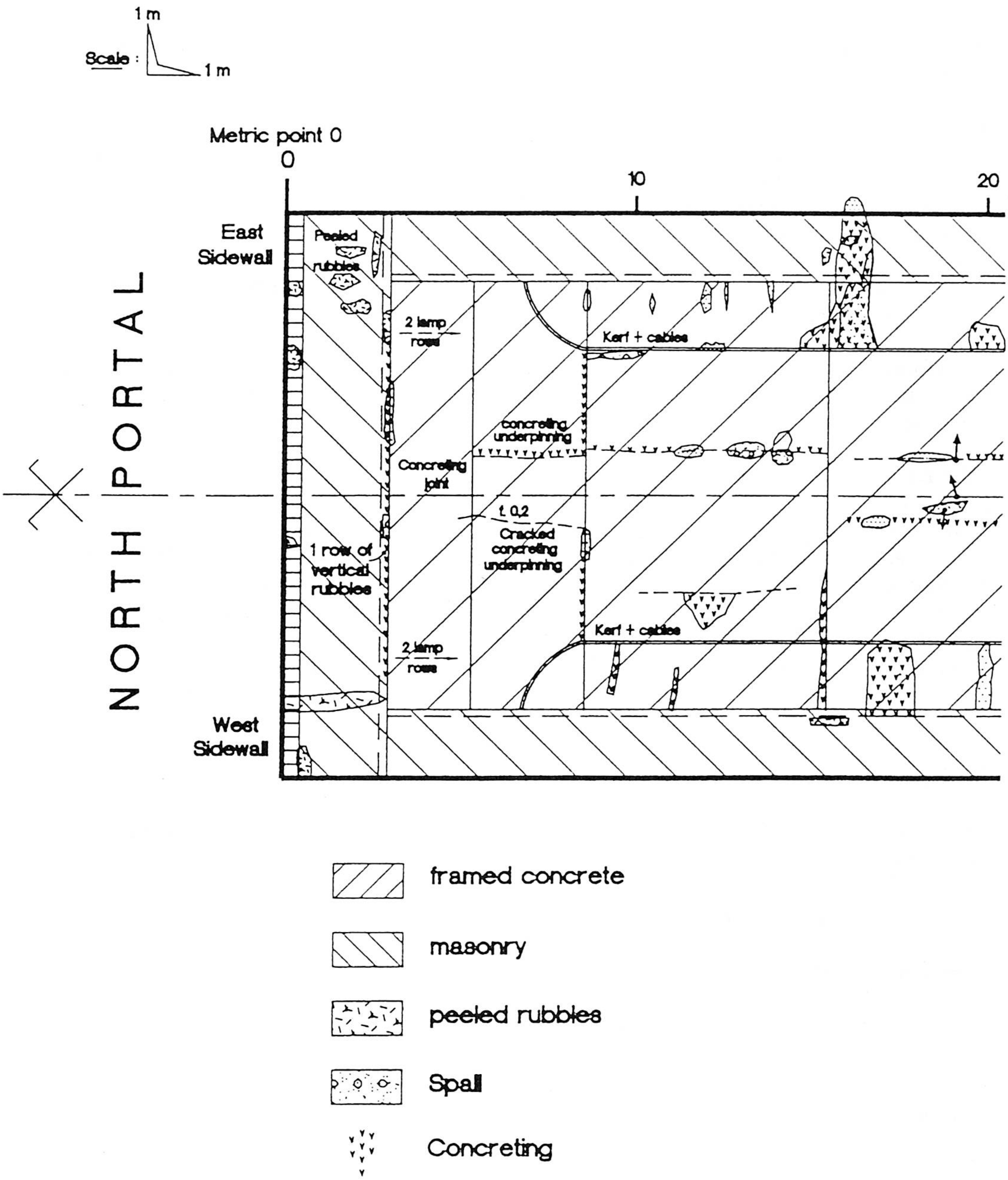


Fig. 2 Excerpt from the intrados survey of SISTERON tunnel, France

3.2 The indirect establishments

As soon as the direct establishments let assume that disorders are likely to occur, or as soon as they are feared in one particular zone, it can be wished to know if voids exist behind the lining; or which is the ground nature; or if the lining is reinforced and how.

We must state that here the term 'lining' means both the inside lining of the structure and the preliminary support installed during the construction process. It can show a most varying composition according to the ground nature. It can be made of steel ribs and sprayed concrete for instance. If no detailed and accurate drawings are available, the nature of ribs is not known, neither their distance or sprayed concrete thickness. Unfortunately this happens frequently.

The geophysical methods - radar, electric measurements, sonic measurements - aim at answering these questions.

4. QICW evaluation (Quality Index for Civil Works)

This is an example of what is currently done in France to quantify the quality condition of structures.

The Ministry of Equipment, Transportation and Tourism (Road Directorate) launched in 1992 a process of evaluation and synthesis of costs to maintain and reinstate the public national road patrimony.

The road network was classified into six categories, for which the NRQI (National Road Quality Image) is measured each year. An identical process is conducted for the Civil Works patrimony, assessed at 18,400 bridges with span over 2 m.

The Road Directorate charged CETU (Tunnels Design Centre to the Ministry of Equipment) of performing this task for tunnels and of assessing the cost of reinstatement of the existing patrimony whilst distinguishing civil engineering parts and equipment, taking account of the major role of the surrounding ground in this type of structure.

4.1 Method fundamental

The Quality Image for Civil Works (tunnels) differentiates portals, lining, and auxiliary objects. The type of lining is an essential criterion to evaluate the condition of the structure, insofar as the possible failures that may affect it are specific. The auxiliary objects include the ventilation ducts, excavated technical rooms, shafts and galleries, by-passes, along with the waterproofing, drainage and sewerage elements.

The tunnel classification is based on a general quotation of their structure through the evaluation of the condition of their constitutive parts.

• Class 1 correspond to the apparently good condition structures, which only require the usual maintenance processes, as defined in the national instruction on survey of civil works (leaflet #40)
• Class 2 gathers those structures which require a specialised maintenance process or minor repairs
Class 2: no emergency

Q.I.C.W. - Tunnels

QUOTATION METHOD
CIVIL ENGINEERING

Class	Structures	Water inflows
1	Good condition	Dry
2	Specialised maintenance required	Unendangering water
2E	Evolution must be checked	Strong water output must be checked
2S	People's safety endangered	
3	Important disorders	Weakened stability
3U	Urgent intervention	Urgent intervention

Q.I.C.W. - Tunnels

QUOTATION METHOD
EQUIPMENT

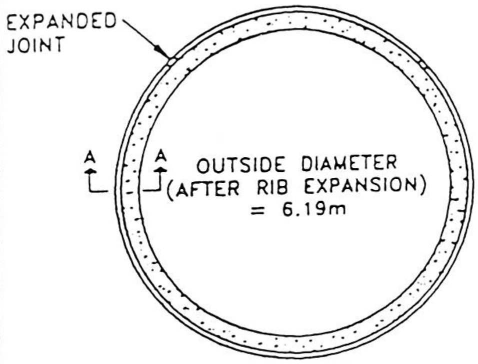
CLASS Average working life = A.W.L.	INDEX E	S	U
1 Age < A.W.L.			
2 Age < A.W.L. + 5 years	Evolution must be checked	Safety involved	Urgent intervention
3 Age > A.W.L. + 5 years	Evolution must be checked	Safety involved	Urgent intervention

Fig. 3 Q.I.C.W. - Tunnels. Quotation method: civil engineering and equipment

COUNTRY CANADA

Classification	
Classification	WATER STOPPAGE
Method	CRACK INJECTION
Outline of tunnel	
Type	SUBWAY
Tunnelling method	TBM
Length of service	10 YEARS
Lining materials	(Primary) STEEL RIBS/ WOODEN LAGGING
	(Secondary) CAST-IN-PLACE CONCRETE
Degree of leakage	DRIP
Location of leakage	SIDEWALLS
Presence of frost	NO

Cross-section of tunnel (Add geological information if available) GLACIAL TILL



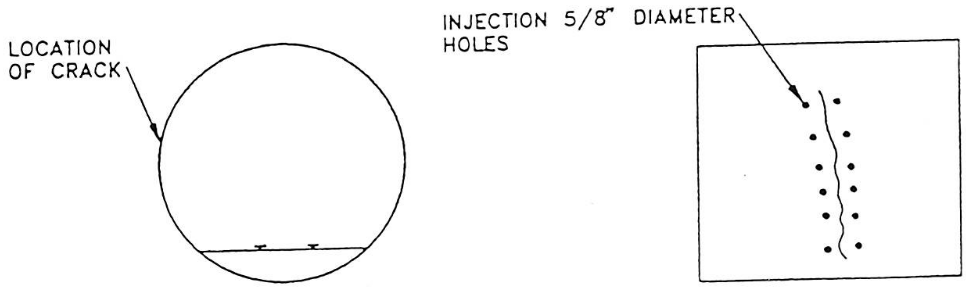
Special conditions in the choice of method
- LIMITATION OF CLEARANCE
- SMALL TOOLS ONLY NEEDED

Description of trouble caused by water leakage
PREVENTION OF DETERIORATION OF ELECTRICAL EQUIPMENT

Materials used
CHEMICAL GROUT

Outline of method
INJECTION HOLES ARE DRILLED AT 30° ANGLE TO THE CRACK. HOLES ARE DRILLED ON BOTH SIDES OF THE CRACK AND THEN ARE INJECTED WITH A CHEMICAL GROUT THAT EXPANDS AND SEALS CRACK WHEN IT COMES INTO CONTACT WITH WATER.

Schema of method



Evaluation of method (workability, cost, etc.)
METHOD IS TIME CONSUMING AND EXPENSIVE

Post-repair Inspection Observations
CRACKS HAVE MINOR LEAKS AFTER 10 YEARS

Fig. 4 Example of measures against water leakage in tunnel

Class 2E: emergency to prevent the quick development of more important disorders by evolution (E)
Class 2S: emergency to guarantee safety (S) to the users
• Class 3 corresponds to the structures the general stability of which is likely to be endangered, thus require repair works
Class 3: no emergency
Class 3U: emergency to perform the works due to unsafe conditions to the users, or risk of quick evolution of the disorders.

It should be noticed that repair works for tunnels Class 3 must often be undertaken only after investigations - boring and auscultation - for a better adaptation to the frequently poorly known local geotechnical conditions. The findings of the QICW evaluation are recorded on a document peculiar to each structure, stating the functional or background characteristics that may affect its condition, the dates of detailed inspections, and the allotted synthesis class.

5. Repairs: the ITA working programme

ITA published in 1991 a 'Report on the Damaging Effects of Water on Tunnels during their Working Like' (T&UST, Issue No.1, 1991). This report was elaborated by Working Group #6 on 'Maintenance and Repair of Underground Structures'.

This document is currently being updated, complemented and extended.

It will consist of two parts:

5.1 Methods for counteracting water flows through tunnel walls

5.1.1 Introduction
Setting the scene. Mention water problems report. Outline scope, methodology and contents of report. Presumes, failures of design, workmanship or materials, and identification of a problem.

5.1.2 Objectives of counteractive measures
Differing tunnel uses have differing infiltration/tolerance limits. The effects of infiltration are different for various ground conditions and linings. In some cases exfiltration is a problem. For each tunnel, therefore, the objective must be determined before design of remedial work commences. "Absolutely dry. Free of dripping. Drippings permitted, total inflow restricted". WG has distinguished four main categories of remedial work for inflows: list only, refer to following.

5.1.3 Categories of counteractive measures
Four categories:
- conduction - where it is acceptable to allow controlled drainage of the water towards the tunnel invert, for channelling along the tunnel towards a sump for disposal
- surface sealing - measures undertaken at the inner surface of the lining
- lining reinstatement - measures undertaken to establish or re-establish the impermeability of the lining
- elimination at source - measures undertaken outside the lining.

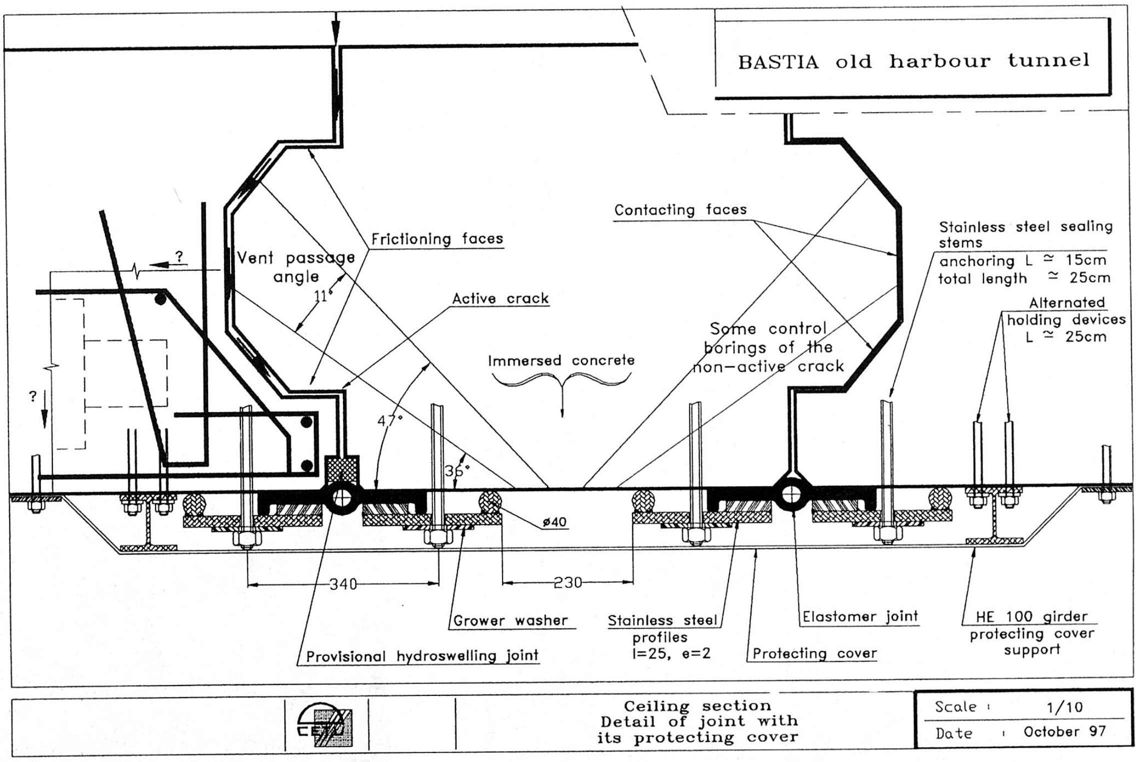


Fig. 5 BASTIA old harbour tunnel. Ceiling section: detail of joint with its protecting cover

These apply differently to different types of lining:
- masonry and brickwork
- sprayed or cast-in-situ concrete
- segmental steel, cast iron or concrete

Also depend on space availability. Unlined tunnels a special case, only first and last categories apply. Application discussed under separate headings. Sometimes used together.

5.1.4 Conduction methods

Free-standing inner shell

Channelling (with or without external drainage)

Membrane and inner protective lining

5.2 Study on performance of applied materials for repair of damage to tunnels linings (excluding sealing of water leakage)

Once the draft of the first Part was well advanced, it was judged that the other main causes of damages affecting the tunnel linings and their repairs should not be disregarded. Among these:

5.2.1 Attack of concrete and reinforced concrete linings by acids resulting from moisture and exhaust gases of vehicles

5.2.2 Consequences of a fire

A recent, impressive example is the fire of a lorry-transporting train in the Channel tunnel. The lining was partly damaged or destroyed on a length of 500 m. The repair work consisted of curing the damaged concrete, using a pneumatic hammer and pressure water jets; the reinforcement was kept; it was re-covered with sprayed concrete. The temperature was presumed to exceed 1000°C.

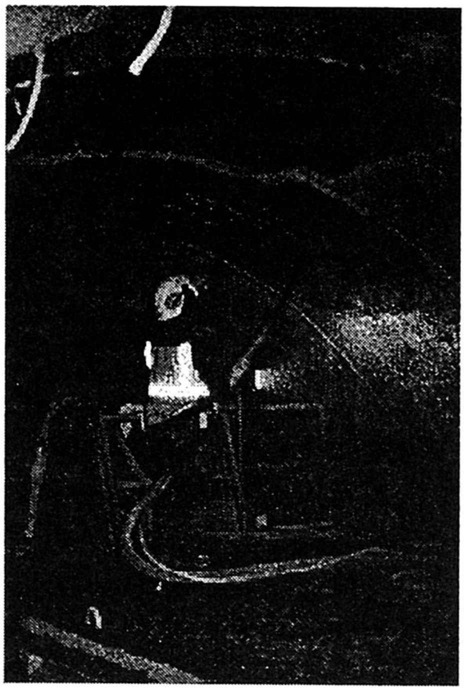


Fig. 6 Channel tunnel, after the fire of 18 November 1996
Repair of lining with sprayed concrete

The PIARC (World Road Association) Road Tunnels Committee defined the temperature curves to be considered in case of a fire in a road tunnel (see PIARC Publication on Fire and Smoke Control, 1998). This Association made an agreement with ITA, so that ITA investigate the damages likely to be caused to the structures by fire and how to cope with them. These two topics will be developed by the ITA Working Group #6 from 1998.

Other possible causes of damages to linings will be considered, e.g. 'Qualification of Engineers for Carrying out Underground Inspections and Assessment of Repair Needs'.

5.2.3 Repair cost

The conditions the tunnels are submitted to are so differing that it is impossible to quote an average repair cost. We can mention two examples.

• Boston Road Tunnels (see Henry Russel - ITA WG#6)
The reinforced concrete linings were attacked by acids from the vehicle exhaust gases, dissolved in seepage water. The cost of repair works was: 4700 ECU per linear metre of tube.

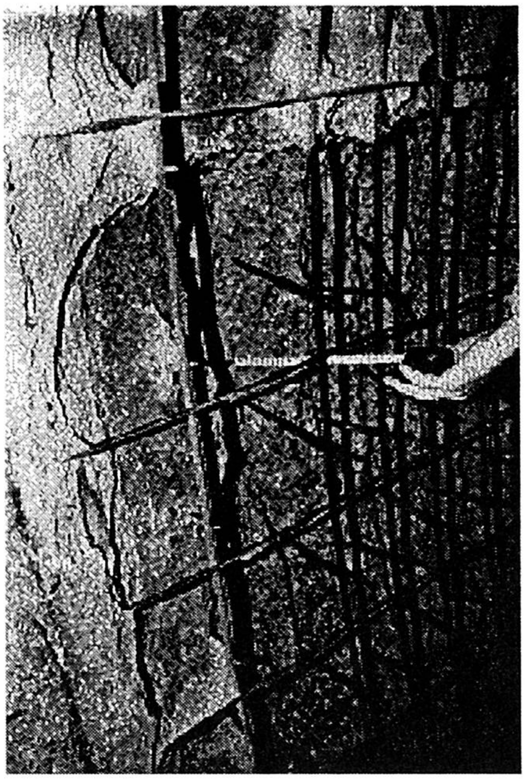


Fig.7 Channel tunnel, after the fire of 18 November 1996.
After curing of the concrete surface spalling, the reinforcement is maintained.

• The 40 tubes of road tunnels on the Nice and Menton belt motorway (see RGRA, Special Issue #1, 1991): Their total length is 19 km. They are lined with unreinforced framed concrete. The ventilation - if any - is longitudinal. Fifteen to twenty-five years after their opening they need civil engineering repairs assessed 2200 ECU per linear metre of tube in average. The expenses will be distributed over 15 years. **This gives 150 ECU per metre and per year, i.e. about 1% of the construction cost of the structure.**

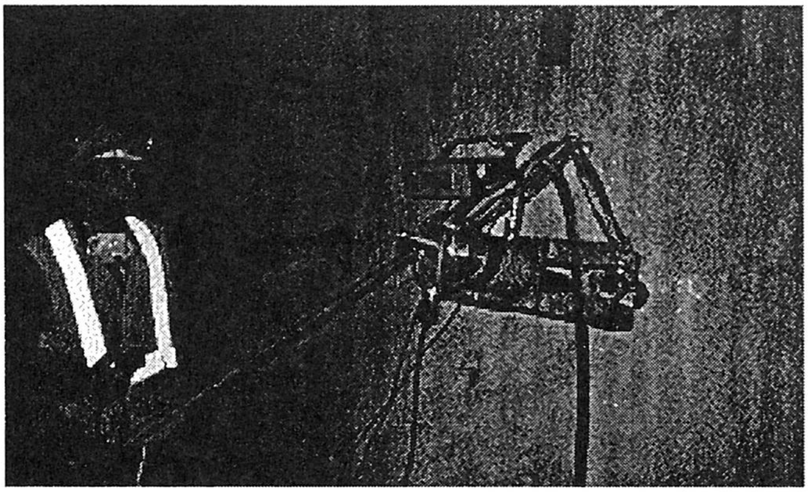


Fig. 8 After sprayed concreting, tearing tests on pins to test the equipment securing conditions

The assessment method of the quality of a lining, as used in France (QICW, Quality Index for Civil Works) also leads to the necessity **to plan a civil engineering maintenance budget, every year, of about 1% of the structure construction cost.**

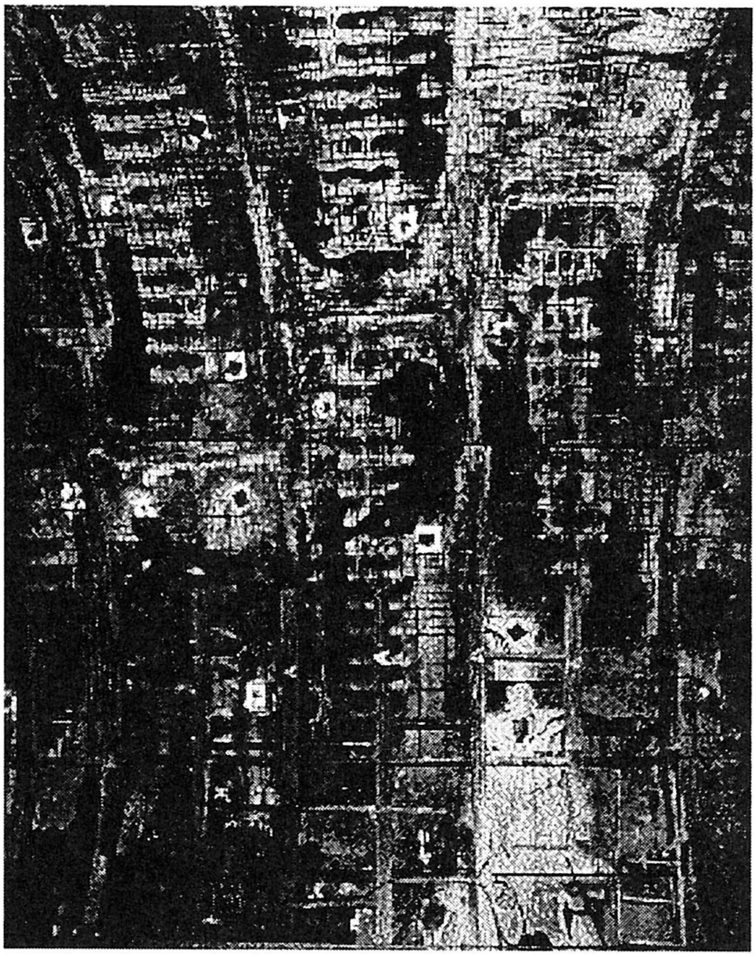


Fig. 9 Channel tunnel, after the fire of 18 November 1996
Rings 19797 to 800, south side. Middle of Zone 3, the most damaged.
The ribs are being removed, after reinforcing with systematic bolting.

6. Conclusions

The cruciality of corrosion problems on the underground structure linings is rather badly known for the modern tunnels, which are too recent. The efficiency of the curing watertightness devices, even if good on short-range (1 to 3 years) must always be estimated on long-range, i.e. 10 years and more. There now, the results are fairly poorer. Everything , therefore, must be undertaken, already at the design stage, to ensure an excellent watertightness, adapted to the peculiar needs of the structure. Fortunately concrete is increasingly improving, increasingly waterproof: For road tunnels with framed linings, most countries have initiated a wide use of polyvinyl continuous waterproofing sheets. The oldest are only about thirty years old and their integrity shows no doubt yet (but for how long?).

Suitable investigation means are available to understand the nature of a lining on a zone of restricted surface: Core borings, boring diagraphies generally bring the required information.

On the other part, if a long length of structure must be investigated quickly, non destructive methods must be called for. Their capacity to give the precise location in space of a possible incident is still restricted. Whilst they are invaluable to make a first diagnosis on the ground condition behind the lining, they must always be complemented with core borings in the doubtful zones.

It is the art of engineers, as expressed in their recommendations, to define the construction methods, then the repair methods, which will remain efficient several tens years after their achievement. But at this moment, unfortunately, the original designers generally are no more here to judge it.

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Repair of the Limfjord Tunnel, Denmark

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Poul Hededal, born 1960, got his civil engineering degree at the University of Aalborg. Since 1984 he has been employed in the consulting firm, RAMBØLL. He is now head of a department for design and maintenance of bridges and major concrete structures

Summary

By the early 1990's, after 25 years in service, leakages and the associated deterioration in the Limfjord Tunnel had reached a stage where an overall repair of the structure was needed.

A multiple phase repair strategy has been adopted to address both structural problems as well as maintenance and durability problems in the tunnel.

Phase 1 (completed in 1994) included a longitudinal post-tensioning of the tunnel structure to eliminate structural problems and facilitate a durable repair of the severely deteriorating areas in the tunnel, and phase 2 (completed in 1995) included injection of cracks and construction joints to stop leakages through tunnel walls and ceilings.

Phase 3 includes renovation of tunnel walls and -ceilings by repair of areas with deteriorating and chloride contaminated concrete and corroding reinforcement and installation of a new tile-covering on the tunnel walls.

Due to the large areas of deteriorating concrete and the extend in depth of the repair works in phase 3 special considerations concerning technical and practical aspects has been given to both the process of removing the concrete and to the following rebuilding processes.

1. Introduction

The Limfjord Tunnel in northern Jutland, opened in 1969, connects the cities of Aalborg and Nørresundby on each side of the Limfjord. At the same time the tunnel forms part of the continental highway, E45, connecting Scandinavia with the rest of Europe.

At present a daily average of nearly 48,000 vehicles pass through the tunnel; however, prognoses indicate that the traffic intensity will reach the capacity of the tunnel within the next 10-15 years.

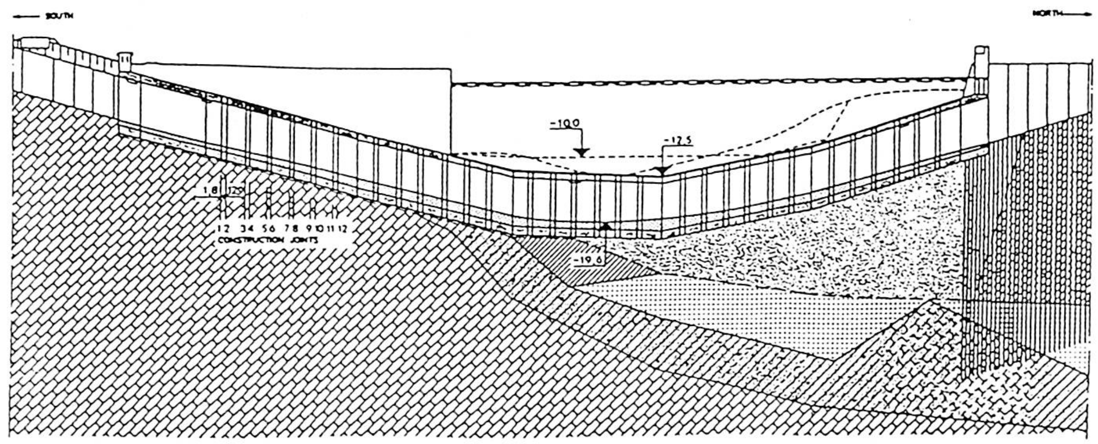
Throughout its nearly 30 years in service the Limfjord Tunnel has suffered from ingress of salt-laden water causing premature deterioration of the concrete and contamination with chlorides, which has led to extensive corrosion of the embedded reinforcement.

By the early 1990's the leakages and the associated deterioration had reached a stage where an overall repair of the structure was needed to ensure and extend its lifespan.

2. The Tunnel Structure

The tunnel is a reinforced concrete structure with a total length of 945 m, of which 510 m is immersed precast tunnel units and 43 m is in-situ cast tunnel..

The immersed part consists of five 102 m long precast reinforced concrete units joined together to form a monolithic structure. The units were cast in a dry dock in 12,8 m long sections, separated by 1,8 m wide gaps into which reinforcement bars protruded. The concrete for these gaps was poured after the sections had shrunk.



The units were waterproofed with a 2 mm butyl membrane which, as described in the introduction, has never been fully effective.

The tunnel has a typical box type cross-section with two separate tubes each carrying 3 traffic lanes of 3.5 m and 2 pavements of 0.75 m.

3. Previous Repairs

Leakages were observed in tunnel walls and ceilings shortly after the tunnel was put into service.

The leakages were primarily located in the immersed part of the tunnel at construction joints between the 12,8 m sections and the 1,8 m gaps where cracks extend in full depth through walls, top and bottom.

To stop seasonal movements of these cracks and make it possible to stop leakages by injection a longitudinal post-tensioning of the tunnel was established in 1993-94 (J. Vejlby Thomsen, P. Hededal, Post-tensioning of the Limfjord Tunnel, Denmark, IABSE Symposium, San Francisco 1995), being the first and very important step in the overall repair of the structure.

In 1995 cracks and leakages were injected to stop further ingress of water.

4. Repair of Tunnel Walls and Ceilings, Phase 3

Phase 3 of the tunnel repair strategy, starting in early August 1997 and lasting for 10 months, comprises the following main activities on tunnel walls and ceilings:

- Removal of damaged concrete
- Placing of new concrete
- Supplementary injection of cracks
- New tile-covering

4.1 Conditions in the Tunnel before Repair

Measurements have confirmed that the structural concrete in large areas of both walls and ceilings inside the tunnel is severely contaminated with chlorides.

High chloride contents - well beyond the level that is generally recognized to cause corrosion on the embedded reinforcement - are found in the concrete, especially in areas where cracks and casting joints in the structure have allowed ingress of salt-laden water from the Limfjord and in the lower parts of the walls exposed to splashes of salty water from vehicles passing the tunnel in the winter season.

Extensive corrosion of the embedded reinforcement have been observed in these same areas - in some areas corrosion of the reinforcement has even led to delamination of the concrete causing hazardous areas with the potential risk of pieces of concrete falling from walls or ceilings on passing vehicles. Since 1996 removal of such hazardously delaminated concrete has been part of the routine maintenance procedure in the tunnel.

Besides contaminating the concrete with chlorides, the excessive ingress of water though cracks and casting joints has locally caused severe leaching and degradation of the concrete accelerating the delay of the structure even more.

4.2 Removal of Concrete

Removal of chloride contaminated and deteriorating concrete is carried out by hydro-demolition (water-jetting), i.e. making use of water jets with extreme high pressure to chip of or cut the concrete.

The water jetting technique is well known in Sweden and Germany but has not yet been fully recognized in Denmark.

In the actual case, however, water jetting seems to be the only viable demolition method of the following reasons:

- Large areas of concrete (approximately 12,000 m^2) have to be removed to an average depth of 80-100 mm - in some areas even up to 250 mm depth which favours a high capacity, high effective, and rational method

- Generally the reinforcement in both the tunnel walls and the ceilings is very congested and consist in some areas of up to 4 layers, which, in combination with the expected depth to sound concrete, makes conventional demolition by jack-hammering extremely difficult and time consuming

Besides being the most efficient demolition method the water jetting method is also advantageous from a more technical point of view:

- the method leaves the demolished concrete surface very rugged and uneven but without micro-cracks enabling a very effective bond to shot-crete repairs and casted repairs

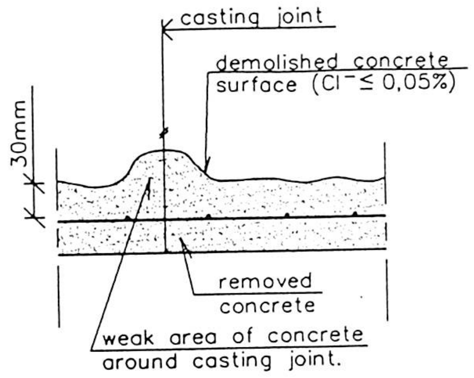
- corrosion products and chlorides are removed from exposed reinforcement to an effect that is comparable to sand-blasting

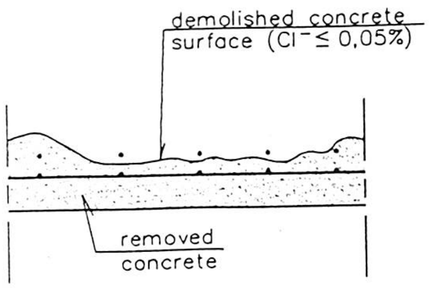
The actual extend of removed concrete is governed by three factors:

- the depth of the reinforcement
- the amount of chlorides in the concrete
- the concrete strength

Generally concrete is removed until the amount of chlorides in the concrete is less than 0.05% by weight of concrete, however, never deeper than 30 mm behind the deepest layer of reinforcement unless weak areas of concrete facilitate deeper demolition.

Two typical demolition profile is schematically shown below.





Due to the large extend of concrete needing to be removed and the amount of reinforcement being exposed in this process, static considerations have necessitated guidelines on how the demolition work must be carried out, imposing the contractor to split up larger continuous demolition areas - especially on ceilings - into smaller sections, and replacing concrete in one section before removing damaged concrete from adjacent sections.

4.3 Replacing Concrete

Concrete is replaced either by

- pumping high-slump concrete in a singled-sided form
- shot-creting

depending primarily on the depth of the removed concrete. Generally shot-crete will be used only in depths up to 70 mm.

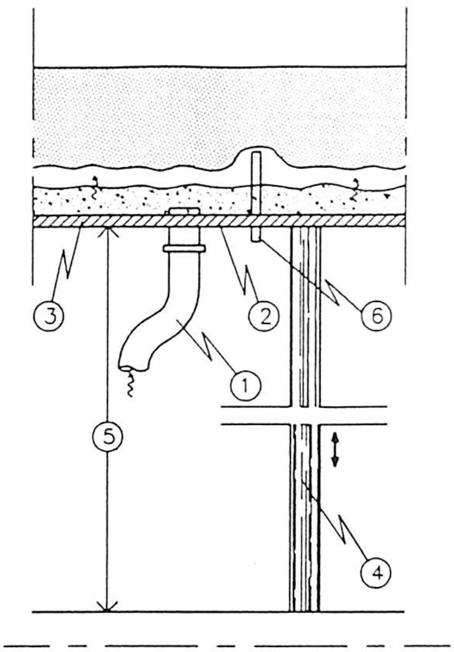
The concrete specification can be summarised as follows:

- 340 kg/m³ low alkali, sulphate resistant Portland cement
  60 kg/m³ fly ash
  16 kg/m³ micro silica
- 886 kg/m³ fine aggregate (0-2 mm)
- 856 kg/m³ coarse aggregate (2-8 mm)
- water-cement ratio (equivalent) = 0.36

Furthermore the concrete contains a variety of additives to enhance pumping ability, bond strength, frost resistance, casting properties and to minimize shinkage of the concrete.

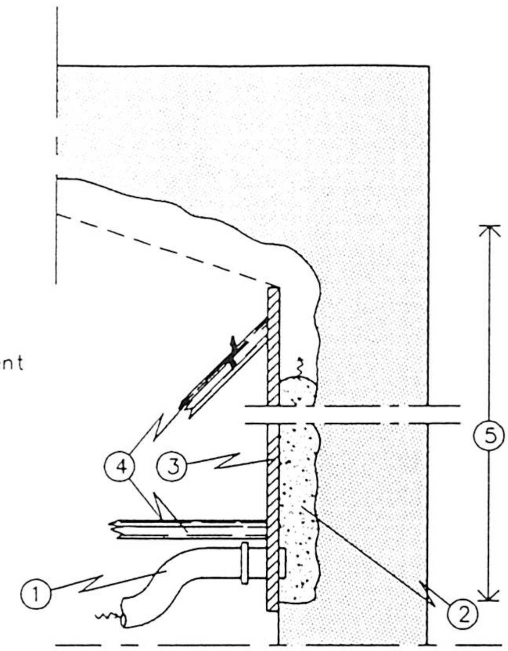
The concrete specification has been determined from numerous laboratory tests and large scale tests on mock-ups of typical areas on tunnel walls and ceilings.

The principle of pumping high-slump concrete is schematically shown on the illustrations below.



Note:
Reinforcement
not shown.

Principle - replacing concrete on tunnel ceiling



Principle - replacing concrete on tunnel wall

① Concrete supply, maximum pumping pressure: 1 atm
② High-slump concrete
③ Form, adjustable in height to comply with varying casting heights, see ⑤. The stiffness and capacity of the form is designed to meet specified requirements on straightness of wall/ceiling at maximum pumping pressure, see ①
④ Scaffolding - supporting the form, the scaffolding is accordingly designed to be adjustable in height, and to meet specified stiffness and capacity criteria at maximum pumping pressure. Furthermore, both form and scaffolding is designed to be easily relieved, moved, adjusted and re-established in another location
⑤ Casting height varying from approximately 4,5 meters to 5,5 meters
⑥ Ventilation tubes to avoid air being enclosed in voids during casting

The quality of all castings are controlled by close examination - both on site and in laboratory - of concrete cores and by testing the bond strength between old and new concrete in locations pointed out by the supervisors.

It is worth noting that shot-crete tests have been carried out parallel to the tests of the pumping method, however, for depths beyond approximately 100 mm and a dense reinforcement arrangement the pumping method especially on the tunnel ceilings has proven to be far superior to the shot-crete method.

Typically the shot-crete method leaves voids behind congested reinforcement and fails to obtain acceptable bond to old concrete when casted in thick layers.

4.4 Supplementary injection of cracks

As previously mentioned cracks and leakages in tunnel walls and ceilings were injected in 1995, however, at that time without removing any concrete. Therefore, in some areas leakages would not be precisely located, hence, injection was not completed, allowing ingress of water to continue.

Having removed damaged concrete the injection work will be completed in these remaining areas, a thorough inspection of all demolished surfaces will be performed and supplementary injection will be carried out to ensure that all leakages are effectively tightened.

4.5 Tile-covering af Walls

After completion of the concrete repairs and water-blasting of remaining parts of the walls a new tile-covering will be installed on the walls.

The tile-covering serves three main objectives:

- From a maintenance and technical point of view the tiles must be easy to clean and durable to withstand the environmental and traffical impacts in the tunnel
- Furthermore from an operating and economical point of view the tiles must have a light colour so that the required illumination in the tunnel can be obtained without excessive use of lights and electrical power
- From a traffic safety point of view the tile-covering must reduce reflections from car-lights but must at the same time form a distinct contrast to traffic ahead in the tunnel

Since water from the Limfjord will continue to diffuse through the tunnel walls, the tiles are mounted in a relatively thick layer of frost resistant mortar and separated by 6 mm wide mortar joints.

The adhesive strength between the tiles and the mortar and between the mortar and the concrete surface is systematically controlled after installation.

5. Concluding Remarks

The phase 3 repairs concludes the structural repairs in the tunnel.

Future works will amongst others include installation of a new lighting system and fire protection of the tunnel ceiling.

A primary objective in the whole repair process on both structural components and technical installations is to generally enhance the safety standard in the tunnel.

The decision on fire protection of the tunnel ceiling, for example, is a direct result of this "enhanced safety approach".

Similarly, in connection with the phase 3 repairs new fire protective cable ducts for future cabling of the technical installations in the tunnel have been established in tunnel walls and ceilings.

The total renovation of the tunnel is expected to be completed in 1998, at a total cost of approximately Dkr. 100 mio., of which the phase 3 repairs amount to approximately Dkr. 35 mio.

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An effective and eternal maintenance method for concrete tunnel structures

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Summary

This paper presents the results of study to confirm that a Cement Crystal Increasing Material is effective to make concrete itself impermeable and to suppress every kind of damage and deterioration, by increasing cement crystals in pores and voids of concrete body. It is a quite different method from ordinary ones by coating and injecting resinous materials which have properties that are different from concrete to the surfaces and into the cracks as physical barrier. The effects of application with the material was verified by permeability test and SEM (Scanning Electron Microscope) survey with concrete specimens and cylindrical test pieces from the concrete linings of some tunnels in service by applying this material.

1.Introduction

For many concrete lining tunnels, it has been a keen requirement to establish rational, effective and eternal maintenance method to give them tough and longer lives against damage and deterioration caused by water leakage from cracks or construction joints, frost damage, salt injury, AAR, and so on. Ordinary repair methods which have been applied so far to such concrete linings are to coat the surface with resinous materials as physical barrier, or to inject them into damaged places. However, these materials do not improve impermeability and other characteristics of concrete body itself. And it is fundamental disadvantages that they have tendency to deteriorate themselves in short period, which causes peeling from concrete with the passage of time, especially they are not adequate to apply for waterproofing to the areas against high pressure water leakage from back side of the lining.

The starting point of every damage and deterioration of concrete is caused by the movement of watersolution in cracks, pores and voids, and by various chemical reactions through the solution. From this viewpoint, we applied a Cement Crystal Increasing Material (Xypex Concentrate: the code XC) which makes concrete lining itself impermeable and suppresses all causes of water leakage or deterioration lastingly and improves many characteristics of concrete itself. We verified the repairing effects which we confirmed on some tunnels. The report on one of them is written in page 5 and page 6.

2.Cement Crystal Increasing Material XC and Principle of Cement Crystal Increasing

2-1. Cement Crystal Increasing Material XC

XC is an inorganic powder form material which includes some kinds of catalysis compounds in it. The main chemical composition is shown in Table 1. And assistant materials used in repair methods by applying XC as main material are inorganic, cementitious materials which do not prevent cement gel ~ crystals increasing function with XC. They are shown in Table 2.

Table 1. Chemical Composition of XC

Component	Specification Range(%)
SiO_2	39~46
CaO	25~34
Al_2O_3	4~5
MgO	3~5
Na_2O	4~7
Fe_2O_3	1.5~3

Table 2. Assistant Materials

Name of Material	Code	Character of Material
Modify	XM	Sub-Cement Crystal Increasing Material
Pach'n plug	XP	Fast Setting Hydraulic Mortar
Extra plug	XE	Quick Setting Hydraulic Fine Mortar
Mild Pach'n plug	XMP	Non-shrink Fine Mortar
Gamma Cure	XG	Curing Agent of Crystalizing Accelerator

2-2. Mechanism of Cement Crystal Increasing by XC

The Catalysis Compounds (Calcium complex L: Ca, a kind of acid as a Chelate Ligand combined with Ca) compounded in XC exist as the size of complex molecule and they would diffuse extremely, naturally in deep and wide through solution in pores and voids of concrete or cement compound materials. The indispensable minimum density of the complexes in concrete to show the effect of impermeability by cement crystal increasing is under 1 ppm.

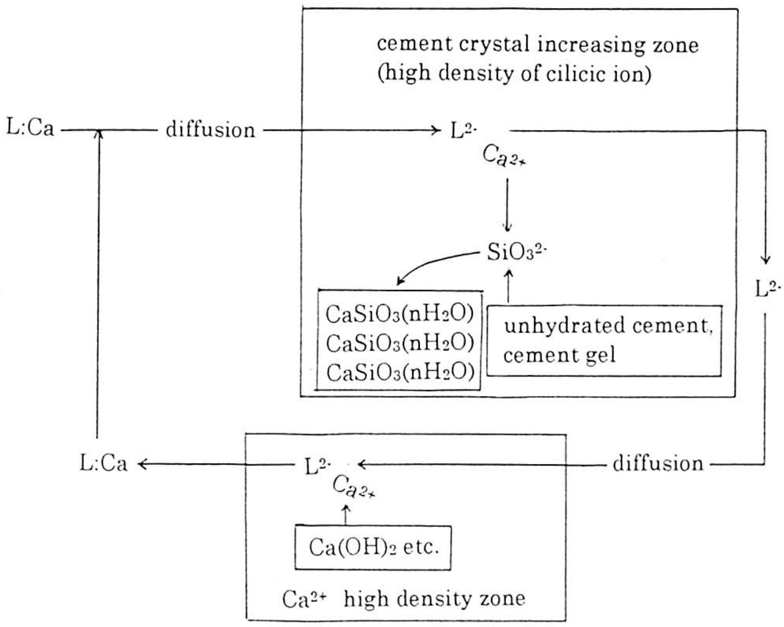


Figure 1. Basic Mechanism of Cement Crystal Increasing

It is well-known that there exist a lot of silicic ions (Si^{2-}_3, Sio^{4-}_4, Si_2

O^{6-}_{7}) not only in young concrete or mortar but in those of over 20 years. Those silicic ions are combined with Ca^{2+} which are carried as Calcium Complexes (L : Ca) and produce cement gel ~ crystals. The basic mechanism of cement gel ~crystals increased by L: Ca is shown in Figure 1. This reaction continues repeatedly in concrete or mortar as follows.

L: Ca combined with Ca^{2+} diffuse though the solution in pores and voids of concrete or mortar and within the high density zone of silicic ions, Ca^{2+} of L: Ca combine with silicic ions and precipitate as Calcium silicate CaSiO3 (main gel ~ crystal of cement) due to the difference of solubility products of them. After splitting off Ca^{2+}, the complex ions L^{2-} diffuse in concrete or mortar again. And when L^{2-} ions come into the high density zone of Ca^{2+}, from Ca $(OH)_2$, L^{2-} combine with Ca^{2+} and diffuse as L: Ca again. These reactions occur repeatedly and increase cement gel ~ crystals with passage of time. This is the basic mechanism of cement gel ~ crystals increasing. As the result, these reactions improve the concrete lining itself to be an impermeable concrete and suppress every cause of deterioration.

3.Impermeation Method of Tunnel Concrete Lining by XC

Impermeation effect of concrete by XC exercises as follows. Calcium Complexes L: Ca compounded in XC diffuse through solution in concrete, and with its catalysis function, cement crystals are increased. Therefore, it is the main point of this impermeation method to diffuse L: Ca into concrete effectively. And the basis of this method is to place and to hold XC directly and steadily on the concrete surface. The impermeation method by XC has two basic and concrete methods. One is Coating method and the other is Plugging method, as shown following.

3-1. Coating Method

This is the most basic method and the application process is shown below. After removing dirt, laitance, efflorescence or other impurities which cause to prevent Calcium complexes' permeation into concrete surface by high pressure water blasting, XC mixed with water (W/XC=35%) is applied coating (standard quantity of coating is 1.0 kg/m^2). Before and after XC coating, Curing Agent XG should be sprayed to accelerate crystal increasing. XC mixed with water is like cement slurry. Coated XC slurry needs 10 ~ 12 hours to set. Therefore, plugging method shown as follows should be applied before XC coating to some areas where XC slurry coat may be swept away before it is steadily set, for example, they are water leaking cracks or construction joints.

3-2. Plugging Method

Concrete linings which need to repair are mostly aged long time. They have few Calcium ions (Ca^{2+}) and Silicic ions (SiO^{2-}_{3},.....), inside the cracks and near around the surface. In these cases, it is the preventing factors for cement crystal increasing. This Plugging Method with XC as main material is the method which is devised to decrease such preventing factors and increase the crystal increasing reactions.

In case of applying XC to water leaking cracks, basic point is to make a small chipping slot (standard size of section is 2~3cm wide × 3~4cm in depth) along the crack, and plug it with XC as main material.

According to the size of the crack and degree of water leakage, we should choose to apply double

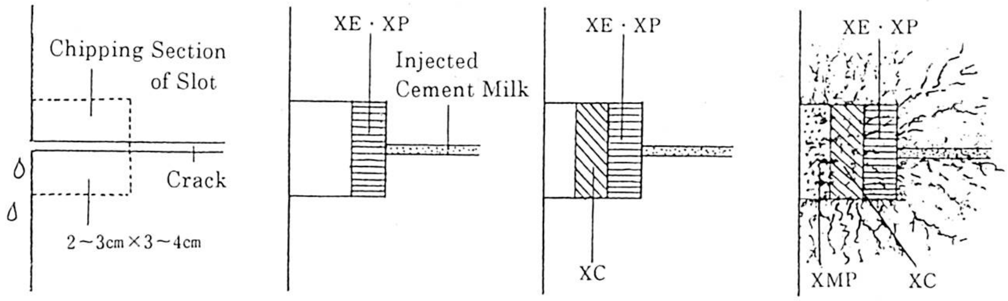


Figure 2. Rough Sketch of Triple Plugging Method

plugging or triple plugging method. Basically, triple plugging method as shown in Figure 2, is applied commonly to the cracks with water leakage. Aims of each application process of plugging method to help crystal increasing are explained in Table 3.

Table 3. Aims of Each Process of Plugging Method

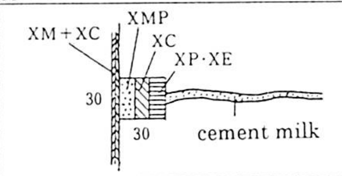
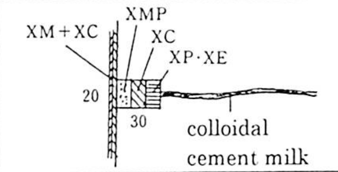
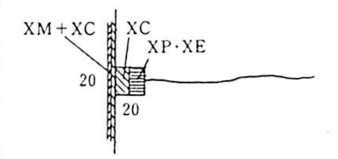
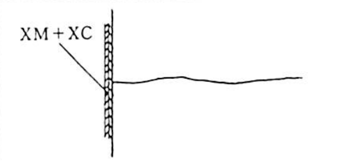
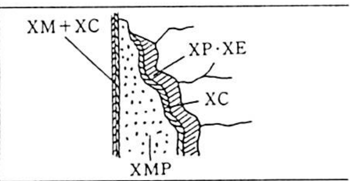
Application Process	How to Increase cement Crystals
Make a chipping slot along the crack	Make a new breakage of section ・To increase permeability of L : Ca ・To make easy to combine with Ca^{2+}, SiO_3^{2},…
Coat XC srurry on the inside surface of the slot	・To make easy to permeate and diffuse L : Ca earlier from inside the slot
Plug the slot with the mixture of fast setting mortar XP and quick setting mortar XE, as first layer	・To reduce the quantity of water leakage ・To fasten the crystal increasing effect by new contact surfaces of chipping slot and plugged XE and XP, instead of the old surfaces of the gap of crack ・To make crystals inside of XE and XP.
Plug crystal increasing material XC	・To supply and diffuse L : Ca continuously inside of XE, XP and concrete, and to all contact surfaces of them around XC.
Plug non-shrink mortar XMP as final layer	・Finishing plug to protect XC layer

The main material of this Plugging method is XC itself. Steadily plugged XC increases cement crystals inside of concrete, the other materials and all contact surfaces around XC itself. This is the special feature of this method. In case of big cracks, as shown in Figure 2, inorganic cementitious materials such as cement milk etc. are injected into them. And XC increases crystals inside of such injected materials and in the contact surface between the material and concrete, and impermeate the cracks completely.

4.An Example of Repair of Railway Tunnel

N.Railway Tunnel of JR is located at the northern and mountainous region of Japan, 110m long and was constructed in 1962. The thickness of the concrete lining is 50cm, and with passage of time water leakage through cracks and deteriorated construction joints of the lining have been increased, in addition, it is located in such cold region that the damages of the lining have been accelerated in winter by freezing damage by water leakage from the cracks. Any repairing method by physical barriers such as resinous materials were unusable no longer for these heavy damage by freezing. Therefore, 1993 repairing work was conducted by applying XC as main material to suppress water leakage and freezing damage, deterioration of lining. Repairing methods applied according to the degree of the damage of the lining are shown in Table 4.

Table 4. Repairing Method of Lining

Degree of Damage	Application Method	
crack (more than 5mm in wide) with water leakage	·inject cement milk ·3 plugging method ·coating method	
crack (5mm~0.3mm wide) with water leakage	·inject colloidal cement milk ·3 plugging method ·coating method	
crack (less than 0.3mm wide) with light water leakage	·2 plugging method ·coating method	
crack (less than 0.3mm wide) no water leakage	·coating method	
sectional caving damage (10cm or thereabouts in depth)	·3 plugging method ·coating method	

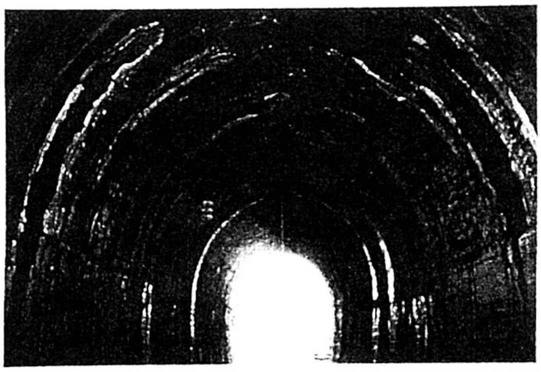


Figure 3. The surface before repair

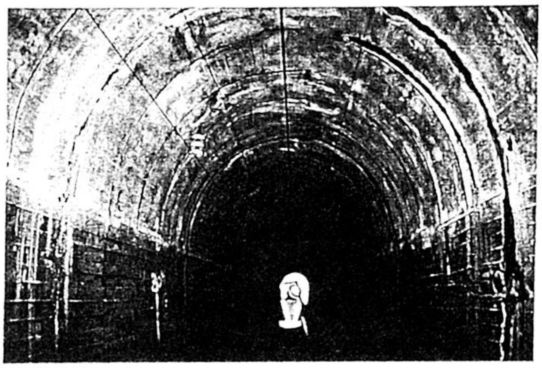


Figure4. The surface after repair

The final process of coating on the whole surface of the lining was double coating method which applied XM on XC coating.

To confirm the effects of application, one year later after the repair 2 concrete cylindrical cores (core No.1 and No.2) from applied area and another one (core No.3) from untreated area were collected as comparing specimens (the size of each specimen is 86 mm in diameter ×65 mm in length), and permeability test (water pressure 5kgf/cm^2) was conducted. The results are shown in Table 5.

Table 5. Results of Permeability Test

Core No.	Collected Area	Coefficient of Permeability(10^{-10}cm/sec.)
No. 1	Applied area	Non - leakage of water
No. 2	Applied area	11.6
No. 3	Untreated area	182.7

The samples for SEM were sheared through 5cm below end surface of cores No.1 and No.3, and structural improvement of the samples was examined by SEM photos. They are shown in Figure 5 and Figure 6. Each photo is 1000 times enlarged photo of inside the pore of thereabout 100 μ m in diameter in size of hardened cement structure of each sample. Figure 5 is the SEM photo of core No.1 and Figure 6 is the core No.3. In Figure 5, increased cement crystals in the pore are observed.

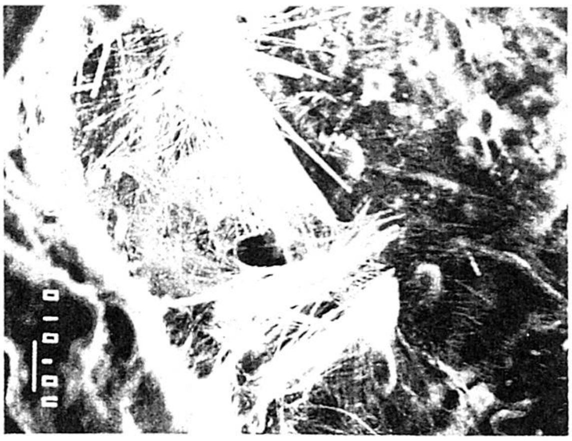


Figure 5. Inside the pore of core No.1

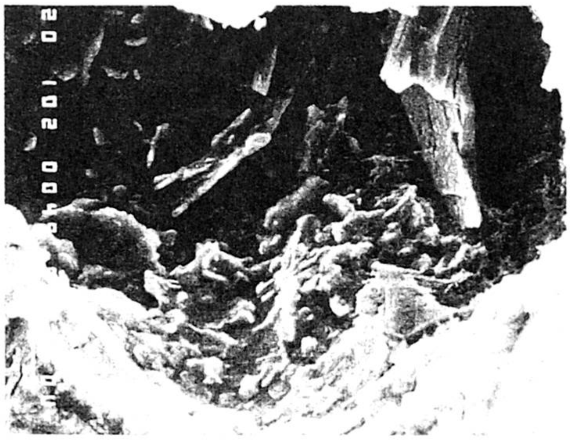


Figure 6. Inside the pore of core No.3

4 years have passed since this repair was conducted, now no water leakage and deterioration by freezing damage are observed, and the result of repair are going on quite satisfactory.

Conclusion

It is confirmed that the structural improvement of concrete by applying XC impermeates the concrete itself not by XC coating onsite as a physical barrier but by the cement crystals increasing function of XC.

It is considered that impermeation of concrete linings themselves by XC suppress not only freezing damage but other causes of deterioration. It is necessary to confirm the effects through further observations and studies of some more concrete lining repairs.

The impermeation effect of XC by increasing cement crystals is an eternally repeating mechanism of chemical reactions. The authors are going to conduct further experimental studies to confirm that the effect would be exercising eternally.

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Reconstruction Of The Dewey Square Tunnel

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Summary

This paper discusses preliminary design and construction staging plans to reconstruct the existing Dewey Square Tunnel in Boston as a part of the Massachusetts Highway Department (MHD) Central Artery Project. This highway tunnel, part of the existing Central Artery I-93 alignment, is to be upgraded to satisfy current highway standards. The design includes extensive structural modifications and a program of inspection and material testing of the existing tunnel.

1.0 Introduction

The existing Dewey Square Tunnel (DST) is a six-lane, 701 meters (2300 feet) long, two barrel highway tunnel in downtown Boston. It was constructed in the early 1950's by the cut-and-cover method, and forms the southern downtown link of the existing Central Artery (I-93).

The overall MHD's Central Artery/Tunnel (CA/T) project is a large transportation improvement project which includes removing the existing elevated artery and replacing it with a new, wider downtown expressway tunnel. Part of this effort requires the renovation of the existing DST and the addition of a 131 meters (430 feet) long tunnel extension to the existing south portal. The existing six lane DST currently serves both northbound and southbound traffic. The reconstructed DST will carry southbound traffic only. Refer to Figure 1 for a plan view of the existing traffic flow.

The existing DST, one link in the overall highway alignment, features 1950's design standards which are now unacceptable as a part of the upgraded highway system. For example, the southbound lanes have a 48.3 km/hr (30 mph) design speed due to a tight horizontal curve and a limited sight distance. The roadway super-elevation is only 1.5%, and there is no additional vertical clearance inside the tunnel to allow a roadway cross slope for higher design speed.

Initially, conceptual highway plans focused on two overall schemes to address these problems:

- Reuse of the existing tunnel with as few modifications as possible,
- Demolish and replace the existing tunnel with a new tunnel.

Both schemes had drawbacks. In the first option, the MHD and the Federal Highway Administration (FWHA) were reluctant to accept the existing tunnel "as is", since billions of dollars were being spent elsewhere to build the CA/T to current AASHTO Interstate Standards. The second scheme satisfied the agencies concerns but would have required preparation of a new Environmental Impact Statement (with a resulting delay to the project of several years), and over $100 million dollars in additional construction costs.

Due to difficulties with options 1 and 2, a third option was examined. This idea focused on the renovation of the existing tunnel. Overall, the existing tunnel exterior envelope will be maintained. However, the existing center wall will be shifted to the west to create two main traffic flows, three lanes for the I-93 Southbound and one or two lanes for the I-90 Collector. In addition, a divider wall between the I-93 Southbound traffic and a Northbound on-ramp (Ramp RT) will be provided. This renovation will achieve current AASHTO design criteria in the tunnel, resulting in improved vertical clearance, improved sight distances, and horizontal roadway curves with larger radii, thus improving roadway safety.

2.0 Construction Staging and Structural Analysis

The basic tunnel section is comprised of structural steel frames at 1.5 meter (5 feet) on center with a reinforced concrete encasement for the base slab, exterior and interior walls. The exposed steel roof beam supports a reinforced concrete slab above. The frames geometry and member sizes vary throughout the tunnel. The existing structural geometry will be revised, when the center column is relocated.

The existing DST renovation and the new 131 meters (430 feet) tunnel extension will be phased in two parts to maintain 2 to 3 southbound lanes within the tunnel at all times. Figure 2 illustrates the typical tunnel cross section and phases of construction.

The sequential procedure for modeling of the existing DST frame sections is as follows:

- Create a computer model of the existing geometry, member properties, and loading to determine the existing 'locked-in' stresses and deflections. The invert base slab with an embedded wide flange beam was modeled as a composite section, whereas the roof and wall column sections are non-composite.
- Install the new support columns based upon the revised tunnel alignment.
- Remove the existing center support column.
- Revise the geometry, apply new loading combinations, and reanalyze.
- Determine the existing corner connection capacity for each load combination and compare this limiting value with the fixed end moment from the frame analysis.

- Limit the capacity of the corner connections to this moment limit value and reanalyze the frame, if required. This analysis is an interactive process and has to be continued until convergence is achieved.
- Compare the final member stresses to the existing member capacities and design reinforcement as required.
- Document all stresses and deformations for the final configuration.

We have performed the preliminary analysis using both the STAAD-III and the ANSYS finite element programs, with the results being in consistent agreement. The ANSYS program will be used for the final analysis since the program can perform the convergence procedure automatically.

The results from analysis described above indicate some overstress of the roof beams and the base slab at areas of maximum moments. This overstress typically occurs in areas where the existing tunnel has the maximum width. At areas of roof beam overstress, structural steel reinforcement can be added to the exposed beam. However, strengthening of the invert base slab, consisting of a steel beam encased in concrete, is more complex. Two methods currently under review include:

1. Reinforce the top compressed flange of the beam,
2. Reducing the design negative moment by creating a moment connection between the invert beam and the support column.

3.0 Highway Realignment and Traffic Staging

The tunnel currently features a minimum 4.3 meter (14'-3") clearance from the top of the roadway to the bottom of roof beams. A new clearance of 4.4 meter (14'-6") plus additional space for the super-elevation is needed.

The traffic staging design was based on these points:

- Major work begins only after northbound traffic is diverted to a new tunnel under Atlantic Avenue.
- At least three lanes of southbound traffic flow at all times.
- The reconstruction work is done in two phases. In phase 1, three lanes of southbound traffic are diverted into the existing northbound barrel to provide a work zone in the existing southbound barrel and the area of the new tunnel extension. During the second phase, two lanes of the southbound traffic are diverted into the reconstructed southbound barrel and one lane is placed on the surface (see Figure 2). This will allow for the completion of the existing tunnel renovation and the new tunnel extension.

Software was developed to help complete the layout design. A tunnel profile generator was prepared to help set the alignment. CADD tunnel profiles are generated from a tunnel geometry database and superimposed upon the existing tunnel geometry. To determine which alignment fits in the available space and which doesn't, the geometry database is adjusted and profiles regenerated.

4.0 Tunnel Inspection and Evaluation

In order for the design to work as planned, it was necessary to verify the condition of the existing tunnel. Previous inspections indicated that the tunnel was generally in good condition. In order to proceed with the design, a three phased inspection program was developed and implemented.

The first inspection phase consisted of a visual examination of the entire tunnel and the development of a detailed work plan for the final inspection. This visual inspection consisted of the concrete roof slab, concrete walls, and roadway invert slab. Concrete deterioration including spalls, cracks, and delamination along with visible reinforcing steel corrosion were cataloged by location in a database program. The steel roof beams, although exposed to view, were not easily accessible and were not inspected during this phase. The roof beams had accumulated 40 years of soot from automobile exhaust, much of it a by-product of leaded gasoline. This hazardous lead material needed to be removed prior to handling and examination of the roof beams.

The second inspection phase included sampling and testing of steel coupons from the roof beams, and concrete cores from the existing walls and roadway invert slab. The results indicate that the structural strength is equal to or greater than that specified by the original design drawings.

The first two phases of inspection were used in preliminary design. The third inspection program was developed for use in final design. This phase of inspection featured some of the same elements as phase two, but to a much larger extent. Test pits were created to remove concrete and inspect connections between girders and columns and to retrieve steel coupons, rivets, and bolts for testing. Preliminary results from this ongoing testing program indicate that the existing tunnel is in good condition. The tunnel's major structural elements are shown to be capable of resisting future loading conditions.

Acknowledgments

The authors acknowledge the Massachusetts Highway Department, the Federal Highway Administration, and their Management Consultant, Bechtel/Parsons Brinckerhoff, for the organizations' support in preparation of this paper.

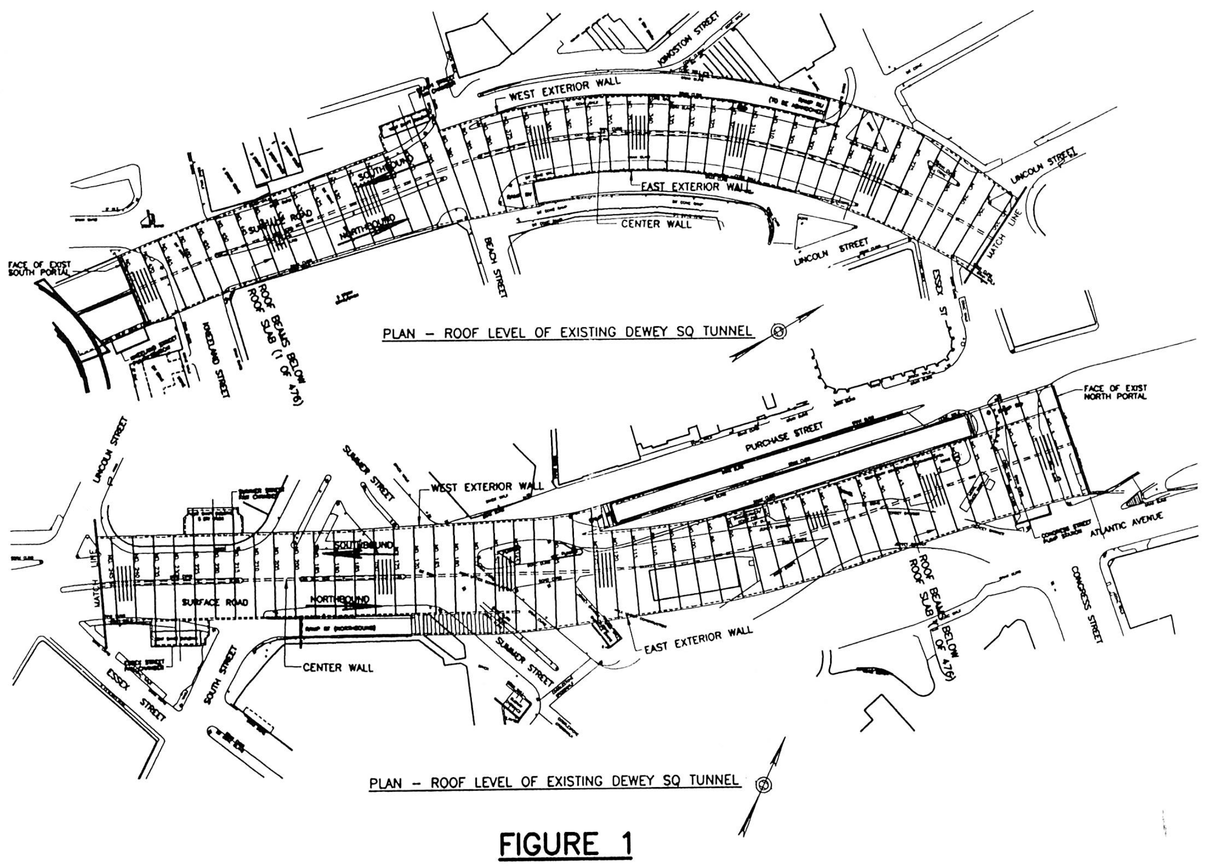
WEST EXTERIOR WALL
KINGSTON STREET
EAST EXTERIOR WALL
CENTER WALL
SOUTHBOUND
NORTHBOUND
SURFACE ROAD
FACE OF EXIST SOUTH PORTAL
BEACH STREET
LINCOLN STREET
ESSEX ST
MATCH LINE
KNEELAND STREET
ROOF BEAMS BELOW ROOF SLAB (1 OF 476)
PLAN - ROOF LEVEL OF EXISTING DEWEY SQ TUNNEL
LINCOLN STREET
SUMMER STREET
WEST EXTERIOR WALL
PURCHASE STREET
FACE OF EXIST NORTH PORTAL
ATLANTIC AVENUE
CONGRESS STREET
MATCH LINE
SURFACE ROAD
SOUTHBOUND
NORTHBOUND
CENTER WALL
EAST EXTERIOR WALL
ROOF BEAMS BELOW ROOF SLAB (1 OF 476)
ESSEX STREET
SOUTH STREET
SUMMER STREET
PLAN - ROOF LEVEL OF EXISTING DEWEY SQ TUNNEL

FIGURE 1

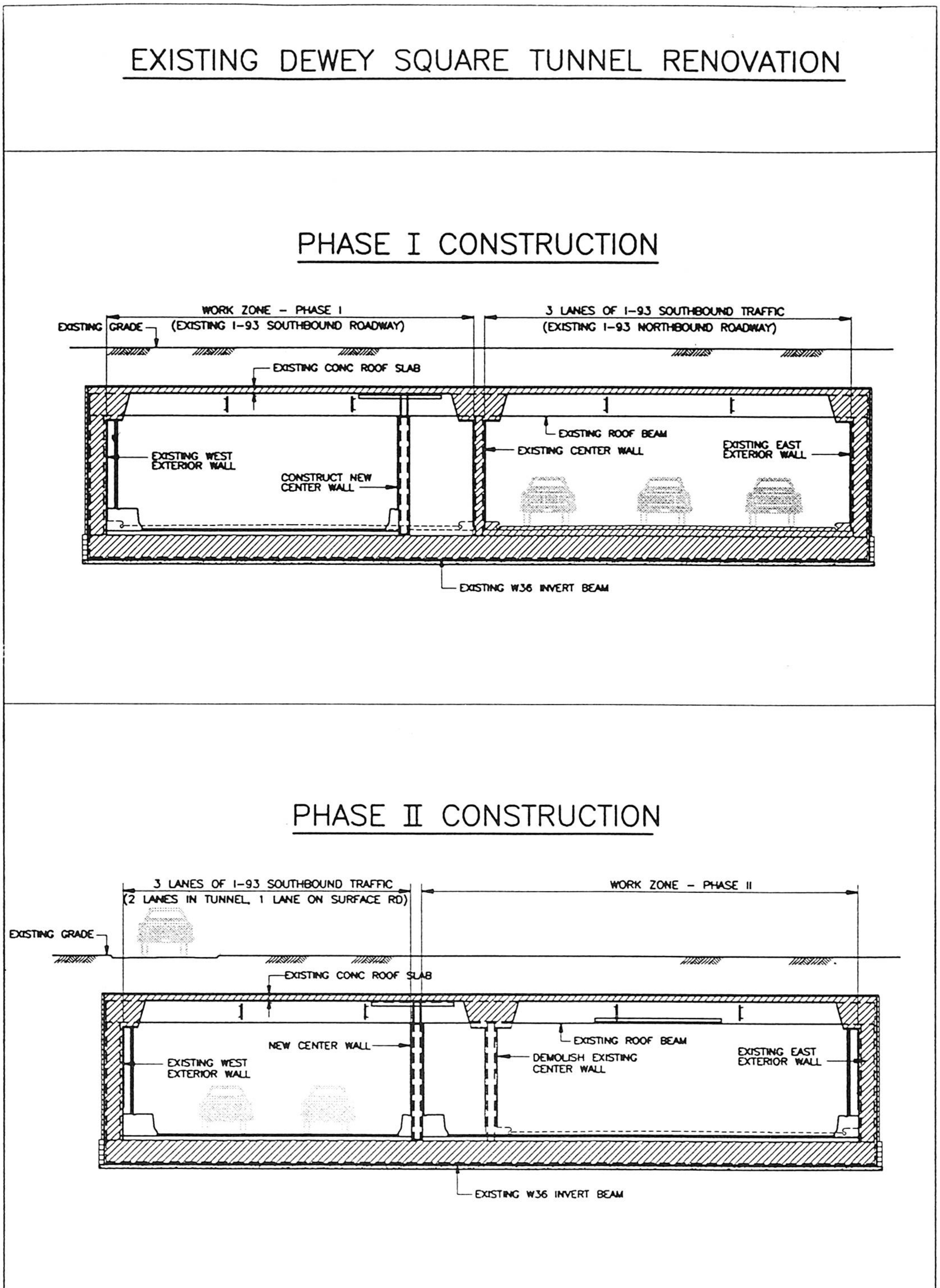
EXISTING DEWEY SQUARE TUNNEL RENOVATION
PHASE I CONSTRUCTION
WORK ZONE – PHASE I
(EXISTING I-93 SOUTHBOUND ROADWAY)
3 LANES OF I-93 SOUTHBOUND TRAFFIC
(EXISTING I-93 NORTHBOUND ROADWAY)
EXISTING GRADE
EXISTING CONC ROOF SLAB
EXISTING ROOF BEAM
EXISTING CENTER WALL
EXISTING EAST EXTERIOR WALL
EXISTING WEST EXTERIOR WALL
CONSTRUCT NEW CENTER WALL
EXISTING W36 INVERT BEAM
PHASE II CONSTRUCTION
3 LANES OF I-93 SOUTHBOUND TRAFFIC
(2 LANES IN TUNNEL, 1 LANE ON SURFACE RD)
WORK ZONE – PHASE II
EXISTING GRADE
EXISTING CONC ROOF SLAB
NEW CENTER WALL
EXISTING WEST EXTERIOR WALL
EXISTING ROOF BEAM
DEMOLISH EXISTING CENTER WALL
EXISTING EAST EXTERIOR WALL
EXISTING W36 INVERT BEAM

FIGURE 2

The Prediction of the Costs of Maintenance

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Summary

During the life time of a construction the costs for operation and maintenance exceed the initial construction costs by a factor of at least 3 to 5. More and more investors and owners realise this fact and put more emphasis on the real life cycle costs of a building. They want in particular to know the future costs for maintenance and the factors determing them. This knowledge is requested already in the phases of investment studies and preliminary design. Two approaches are being presented and illustrated in this paper. For obtaining reliable results the two approaches should be applied in combination. During the design phase the bottom up approach can also be used to evaluate different options for the construction with regard to the respective to the maintenance costs to be expected. Therefore the presented instruments are supporting the investor in choosing the most suitable strategy.

1. The significance of the prediction of the costs of operation and maintenance

Owners, planners and users of constructions and buildings are realising more and more that the costs for maintenance and operation are at least as important for the economy as the initial construction costs. Investigations prove that the aftermath of the initial investment may vary between 1 and more than 20% per year. The lower values of 1 to 10% are standing for appartment and office buildings, values in the range of 10 to 20% for public infrastructures, utilities and industrial buildings and the the values over 20% resulting from schools and hospitals. In a life time cycle view these aftermath costs for operation and maintenance exceed the initial construction costs easily by a factor of 3 to 5.

Up to now, very often only the annual costs of the initial investment for the construction, i.e. interests and redemption charges have been analysed already in the early phases of planning. But the decisionmaking bodies and investors need facts and figures of the life cycle costs including the costs for operation and maintenance in order to properly assess the chances and risks of a planned investment. One major problem consists in predicting the annual costs of operation and maintenance already in the early design phase.

2. Two approaches for the prediction of the costs of maintenance

In this paper the prediction of the costs of maintenance for constructions in the design phase (or in an early phase of operation) is explained in more detail. We distinguish between the top down and the bottom up approach.

2.1 Top down approach

The expected costs for maintenance for a specific construction are predicted by comparison with similar constructions. The experience gained from these similar constructions must be tranponded to the construction in question. The problem is to define and find data of similar constructions and to define the rules for proper transponding of these data. The top down approach consists of the following 4 steps:

2.1.1 Define similar constructions and subsystems
The construction for which the costs of maintenance should be determined must be subdivided into construction parts or subsystems for which data gained by experience are available. In some cases global data of more or less identical constructions are available and therefore no subdivision is necessary.

2.1.2 Data of similar constructions and subsystems
For each identified subsystem data must be found comprising investment or construction costs and costs of maintenance. Best results are obtained, when the two subsystems to be compared coincide very well and when long term records such as information about the type of maintenance work executed and its costs for each year are available. But even if you dont find such an ideal situation, do not hesitate to look for information and data everywhere - including in other countries. Remember that different subsystems have quite different needs of maintenance. It is therefore more important to distinct and define the subsystems adequately and make some extrapolations based on sound engineering judgement than to apply gained good records at the „wrongly“ defined subsytem.

2.1.3 Characteristic values
For each of the defined subsystems a curve of characteristic values must be calculated. The characteristic value of one year is defined by the quotient of costs of maintenance of this specific year and the initial construction costs. One of the main traps in using characteristic values is the problem of the index-linking of currencies. We strictly adhere to the principle that the characteristic value of the year „x+n“ is defined by the maintenance costs of the year „x+n“ divided by the construction costs in the currency of the year „x+n“. The construction costs which you will normally find in the records are the costs in the year „x“. They must be indexed to the year „x+n“. We strongly propose to use strictly such normalised characteristic values which are neutral to inflation. Note that these indexed construction costs at „x+n“ do not correspond to the real construction costs you would face, if you wanted to build in the year „x+n“. Normally the latter would be much higher, due to increased environmental and other requirements.

2.1.4 Maintenance costs
Since the curves of the yearly characteristic values for each subsystem gained in the third step are normalised with regard to currency values and indexes, it is easy to calculate the expected costs for maintenance over the years: Just multiply the characteristic value for one specific year with the construction costs of the respective subsystem and you have the maintenance costs for this subsystem for the specific year (of course expressed in the same currency value as the construction costs).

2.2 Bottom up approach

The expected costs for maintenance of a construction are being calculated by summing up the costs for every single maintenance task to be performed over the years for each subsystem. The problem consists in identifying all individual maintenance tasks, their return period and the costs for each execution. The bottom up approach consists of the following 4 steps:

2.2.1 Define maintenance units
The construction for which the maintenance costs shall be determined must be divided in its main parts and each part must be further divided in maintenance units. This definition of the maintenance units must be made strictly in view of the performance of the maintenance tasks, i.e. together with step 2. The maintenance units are not identical to the construction units and are also different to the subsystems defined in the top down approach.

2.2.2 Define maintenance tasks
For each maintenance unit all maintenance tasks from surveillance and tuning up to replacement and to partial reconstruction over a period of approximately 30 years have to be identified. It is important to define the maintenance units as per step 1 and the maintenance tasks as per step 2 from a very practical point of view. For each defined maintenance task the moment of its first occurrence, its interval and its costs have to be determined. Keeping in mind the great variety of the maintenance tasks it is obvious that the intervals vary from several times per year to once every 30 or even more years.

2.2.3 Calculate yearly maintenance costs
By summing up all data of the step 2 the maintenance costs can be calculated year by year for each maintenance unit, for each system or main part and finally for the entire construction.

2.2.4 Smoothening over 5-year periods
Due to the fact that the timing of the maintenance tasks, based on intervals as per step 2 (such as 1, 2, 5, 10, 15, 20, etc. years), you will automatically get peaks in the maintenance costs calculated as per step 3. In reality the timing of the execution of the different maintenance tasks is determined by the condition of the maintenance unit and can be chosen up to a certain extent (preventive maintenance). In order to reflect this fact, the results as per step 3 are smoothened over 5 year periods.

3. Application for a complex road system

Both approaches have been recently applied for a complex road system in Switzerland. It consists mainly of a tunnel system in an urban area and it is still in the design phase. In the following the main characteristics of the two apporaches are illustrated.

3.1 Illustration of top down approach

In view of the road system still in the design phase and considering the data and information sources which have been available to us , we have defined the following subsystems:

- tunnel construction comprising excavation, lining and inner construction in concrete and steelwork;

- roadwork;

- electromechanical systems including electrical and ventilation systems, systems for monitoring and control, sanitary systems;

- buildings comprising ventilation caverns, air intakes and outlets, transformer buildings, control centers, warehouses, etc.

For each subsystem data from different countries from more than 100 tunnels, from different types of roads and different types of buildings as well as all kinds of electrical, sanitary and ventilation systems have been evaluated. Out of these informations the following characteristic value curves have been calculated (see figure 1). The result of the calculation of the expected maintenance costs is integrated in figure 3.

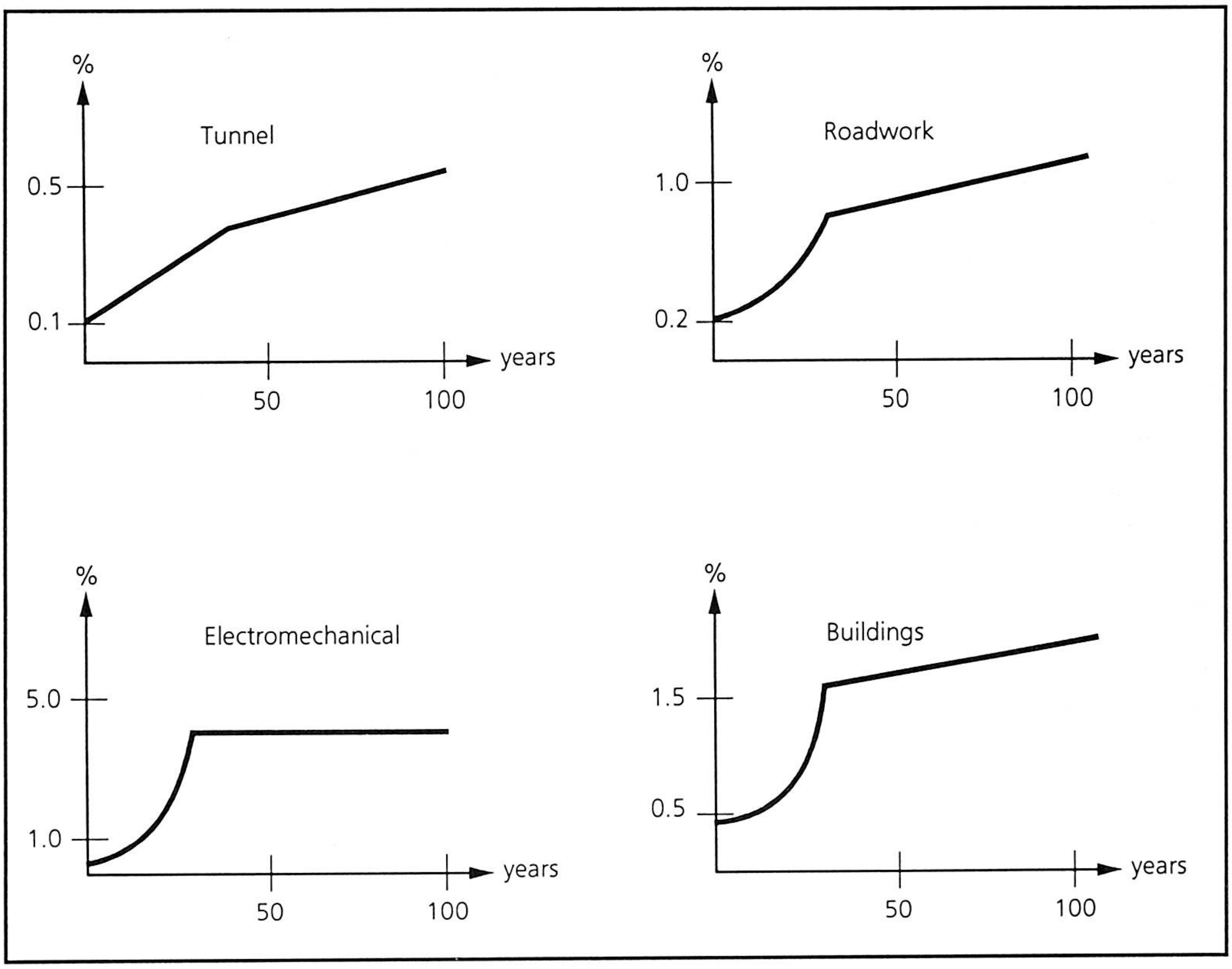

Fig. 1 Characteristic value curves for top down approach

3.2 Illustration of bottom up approach

The entire road system has been divided into approximately 20 main parts and a total of roughly 50 maintenance units. A total of 150 maintenance tasks have been defined. In addition to the cost estimate also an estimate of the duration has been made for each maintenance task. This would allow to predict also the interference of maintenance work with the traffic flow. The results of the bottom up approach are illustrated in figure 2.

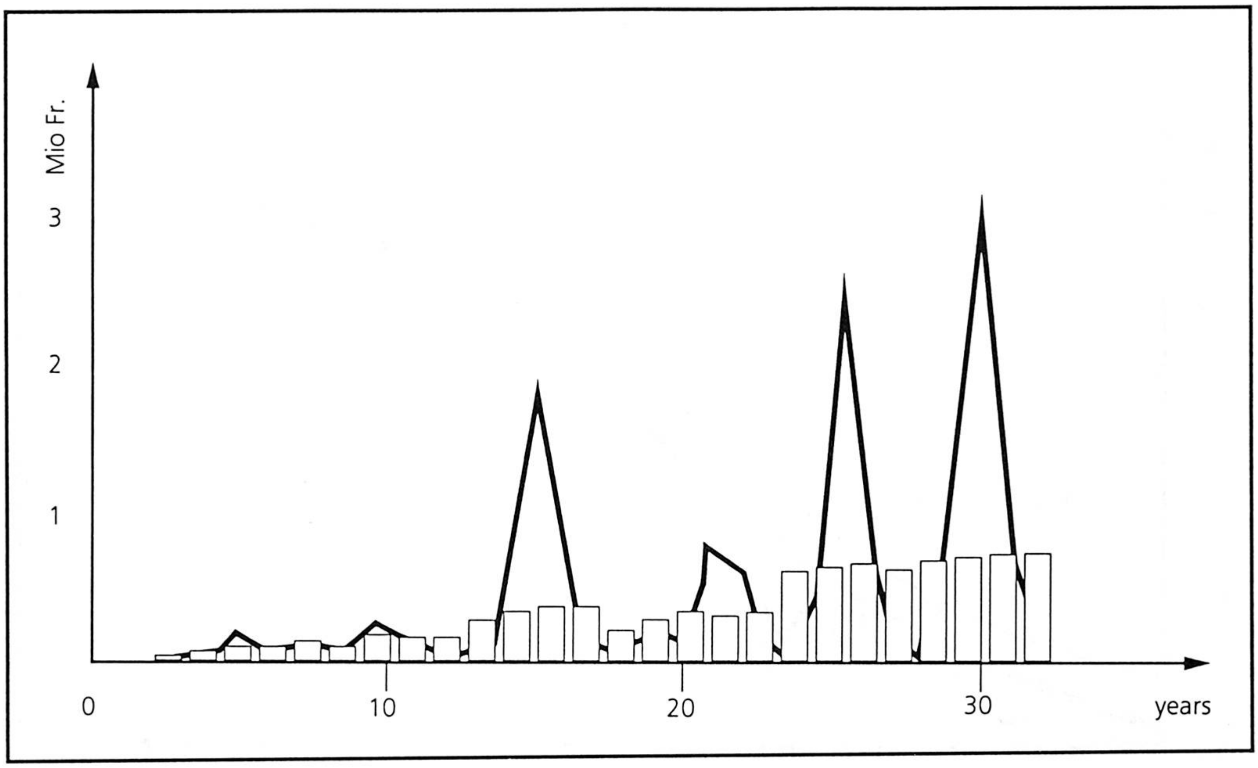

Fig. 2 Annual maintenance costs with 5 years period smoothening for bottom up approach

3.3 Prediction of the costs of maintenance for complex road system

In figure 3 the results of the two approaches are presented in one single graph. Generally speaking both approaches are fitting pretty good. Obviously the bottom up approach gives more accurate figures in the short to medium range i.e. up to 20-30 years, because it better takes into account the start up phase when the systems are new. It also covers the period when most of the electromechanical systems have already been completely renewed at least once. On the other hand the top down approach gives a better forecast of the longterm costs beyond the age of 50 years, because it reflects the growing maintenance costs when the civil engineering parts of the road system and the tunnel construction must be renewed.

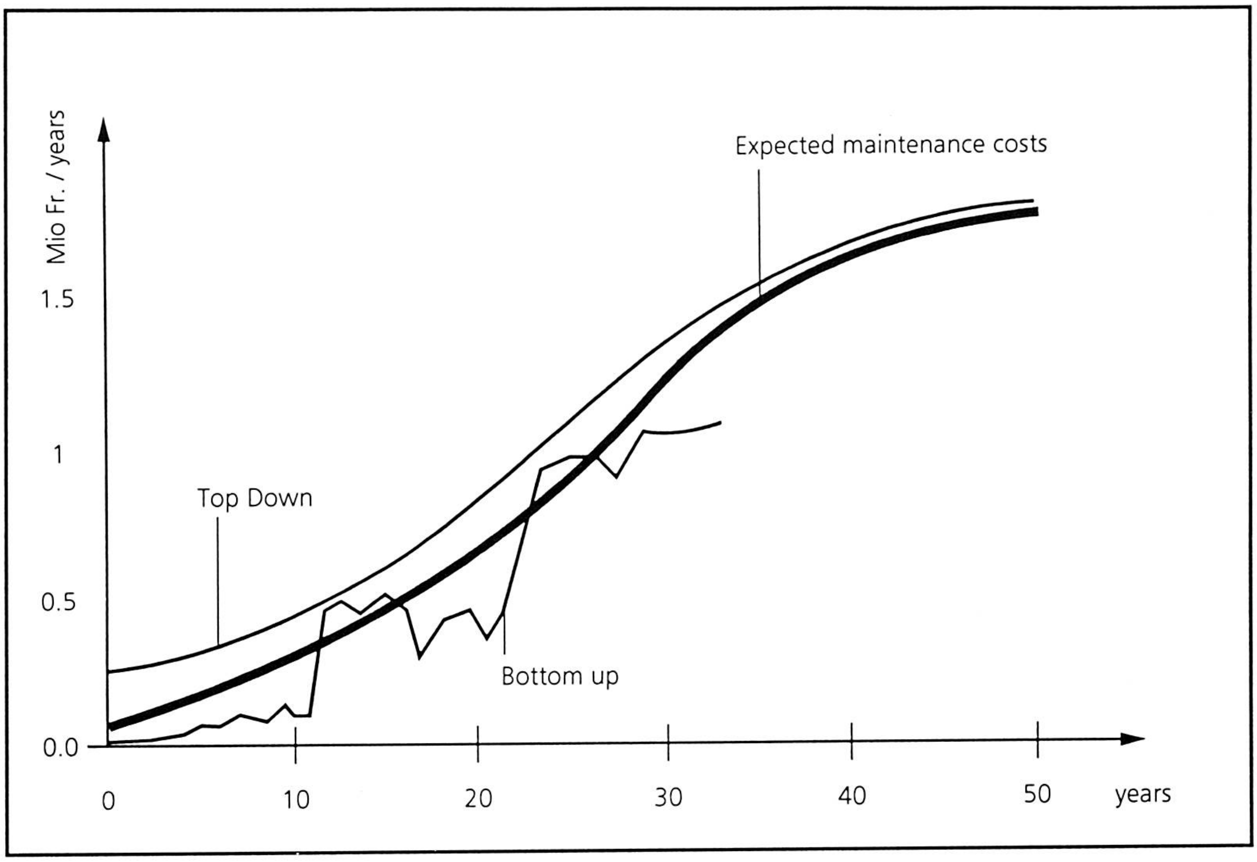

Fig. 3 Prediction of the costs of maintenance with both approaches

4. Outlook

The results of this early analysis of the maintenance costs to be expected can be used by the investor for decisionmaking. During all phases of design and realisation the presented tools, especially the bottom up approach, allows to determine the consequences of different design options. E.g. a design strategy to minimum maintenance costs or any other strategy is possible. It goes without saying that the same tool can also be used during the phase of operation for the ongoing rolling planning of all maintenance activities including budgeting, workforce planning and minimising the interference of maintenance with operation.

Laser Scanning for Tunnel State Assessment

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Summary

In this paper a technology to obtain a maintenance and safty inspections in tunnls is presented. The measurements are carried out with a laser scanner. This measurement method is fast and relieable and does not obstruct the normal traffic or work in the tunnel.

In addition to the surface covering visual and thremal image, distance measurements can be obtained. All the data is stored on a normal PC. That means the tunnel is always ready for inspection in the office.

1 The Technology

1.1 The Scanner

The time available to carry out survey work on track and tunnel systems is strictly limited. In addition, the project requirements are getting more demanding. The TS 360 scanners was developed as a tool for maintenance and engineering professionals in order to carry out fast, accurate and reliable inspections of the total track and tunnel surface.

Figure 1 shows the measuring principle. A rotating scan mirror directs the laser beam outward. The reflected beam energy is collected by the same scan mirror and led though the optical system to the register electronics. The optical signals are digitized and stored. The scan mirror rotates 200 times per second and with each rotation 2.500 individual measurements are taken. Each measurement comprises of different informations.

While the scan mirror rotates 360° the measuring vehicle moves ahead a distance which corresponds to the width of the laser beam. Thus, a surface-covering recording is obtained.

Typically, to record 1 km with this multi spectral data collection system will take approximately 10 minutes.

2 Maintenance and Quality Conrol

2.1 The image and thermal scanner

The image and thermal scanner allows to chart structural damages even when the damaged sections are not visible on the surface. This type of measurement is primarily used for periodical structural surveillance, but also for approval acceptance of new constructions.

Figure 2 shows the approval acceptance for a new tunnel in Stuttgart. It was a two-channel measurement with a thermography image and a visual image. Both show the same section of the tunnel. The thermography image shows a striking anomaly on the wall at the left-hand side, which indicates that water has intruded into the concrete of the lining. After drilling a boarhole at the indicated place at a depth of 70 cm a strong jet of water poured out of the hole. By pressing in 300 litre of material the problem could be solved before the tunnel got under traffic.

It is notable that the visual image shows no trace of water on the lining. Hence, this anomaly could only be detected using the thermography laser scanner.

Figure 3 shows another example for the quality control in a new concret tunnel. The lower section shows a very homogenose heat distribution which means that there are no major demages in the lining. The upper image shows a very inhomogenos heat distribution. That is an example for what we call in Germany a Monday concrete. The thermal image is always good for getting an overview of what is going on in a tunnel. Based on that knowlege refurbishment work or additional local measurements can be planed much more costeffective.

2.2 *High resolution image scanner*

The high-resolution scanner is used for charting small surface structures. Cracks in the walls larger than 0,3 mm can be charted with close surface coverage (Figure 4). If these surveys are repeated at regular intervals, objective results on cracks and crack development can be obtained and repair works can be planed and calculated.

3 *Clearance Control*

Besides other features, it is the surface covering measuring techniques, which allows new solution to known problems. One of these "classical" problems is concerned with "measuring the clearance" or determining the place of the narrowest points if an excess load has to be transported.

3.1 Clearance Control for Tunnels and open tracks

To determine if an excess load can be transported over a certain railroad track, it is important to know the distances to all intruding objects on the track and to know the profile of the tunnels in relation to the rails. To enable a quick decision if the excess load is transportable by train, the required data of the distances were measured with the laser scanner and stored in a database. Figure 5 shows the clearence map of a tunnel in Scotland. The profile of the excess load is indicated on the bottom left.

3.2 *Clearance Control during Tunnel Construction*

A very similar problem is the outbreak control during tunnel construction. In this case one has to determine whether a concrete tube with a given cross section and given position will fit into the outbreak after the blasting.

Common to all these problems is that they call for measuring the distance between a given theoretical profile and the actual tunnel surface. This distance is termed "Clearance". The theoretical profile may be a vehicle shape or cinematic envelope shown in the cross section perpendicular to the track axis. For the case of outbreak control it is the cross section of the concrete tube to be placed in proper position.

Due to the fact that the measured distances are available for every point on the registered surface, the result may be represented as an issue oriented map. With the measured clearance distances displayed by colors on a map showing the vault surface, a Clearance Map is obtained. For the Clearance Map presented here, only certain critical values of the clearance are displayed. The remaining portion of the surface shows the visual image as a kind of "background layer".

The profile plot in Figure 6 shows the measured profile together with the theoretical profile at a specific place in the tunnel. This theoretical profile has been used to generate the Clearance Map. The origin of the coordinate system is in the virtual track center.

A portion of the Clearance Map at the entrance of the eastern tube of the Hallandsås Tunnel in Sweden is shown in Figure 7. The background layer displays a black and white image of the developed tunnel surface. The bare rock covered with shotcrete is well recognizable. The tunnel crown with a ventilation tube can be seen in the center of the figure.

There are two classes shown: "Close Clearance" means that the surface approaches the theoretical profile between 20 to 0 cm, "Obstruction" means that the surface intrudes the given theoretical profile.

Using colors in the Clearance Map several classes with "Close Clearance" or "Obstruction" can be highlighted. This feature enables the quick and convenient discovery of critical zones in the tunnel or open track under inspection.

The advantages of using a scanner-based method for outbreak control are evident:

- The Clearance Map immediately displays the places where problems may arise, either with the help of drawings or as colored map.
- Due to the availability of the visual image the places where problems may arise can easily be recovered in the object itself, eventually intruding objects can be identified.
- The additional volume that has to be broken out can be calculated from the digital data.

4 Conclusions

The laser scanning technique enables a surface covering measurement of a tunnel or open railway track. Clearance maps, crack charts and tunnel state surveys are systematically and reproducibly obtained. The measurements are carried without disturbing the normal traffic.

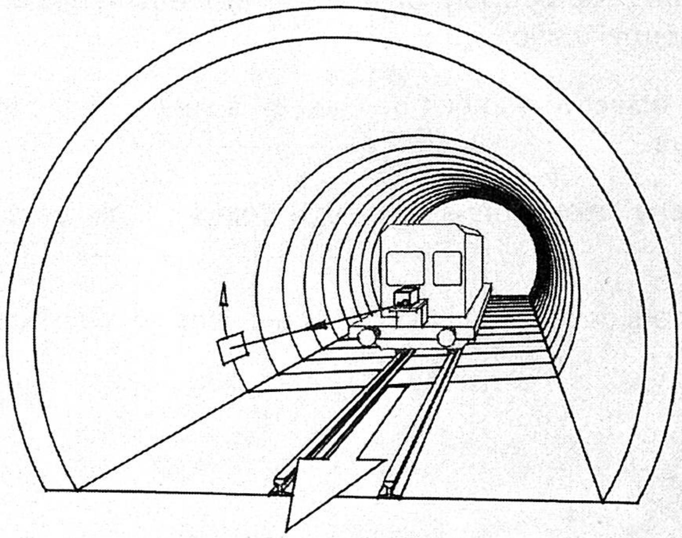
The results are objective and can be used for a surveying the trend of the state of a tunnel over years.

The thermography laser scanner is able to detect anomalies that are not visible by human eyes.

The laser scanners described in this paper are used all over the world for projects.

Tunnel Scanner TS 360

TS 360 scanner



- **TS 360 B**
  high resolution visual image scanner
  2500/5000 pixel/scan line
  measurement of the reflectivity of the surface

- **TS 360 BT**
  2 channel scanner, VIS/IR image
  2500 pixel/scan line
  measurement of the surface temperature; intensity of the emitted radiation

- **TS 360 BP**
  2 channel scanner, VIS image/profile
  2500 pixel/scan line
  measurement of the distance using the traveltime of light

Amberg Measuring Technique

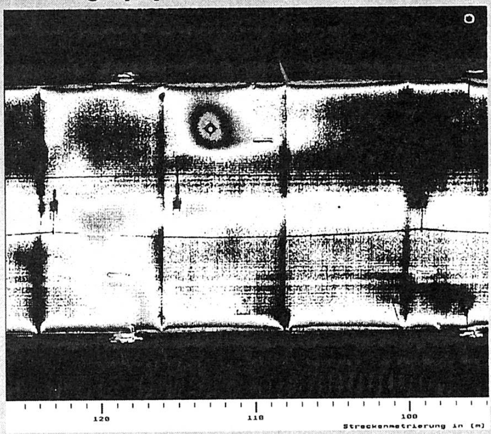
1

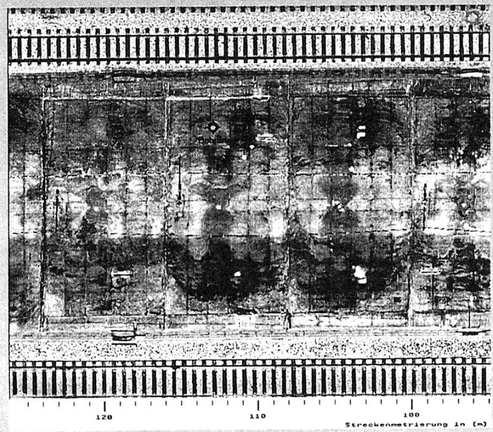
Figure 1: Measuring Principle

Tunnel Scanner TS 360

Stadtbahn Stuttgart

❐ Thermography / endoscopic investigation





Thermography shows a striking thermal anomaly on the left side wall. Water has intruded into the concrete of the lining.

The visual image shows no traces of water on the lining surface.

Drilling: **at the depth of 70 cm a strong jet of water exits from the drillhole.**

Amberg Measuring Technique

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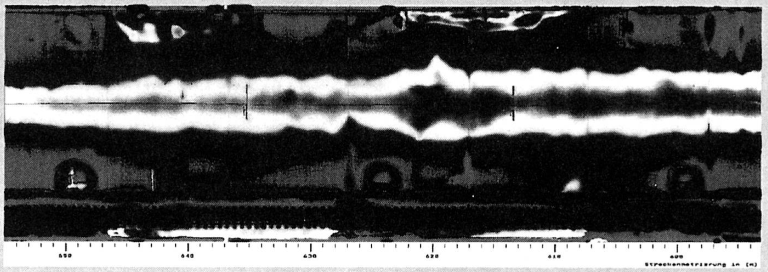
Figure 2: Acceptance approval

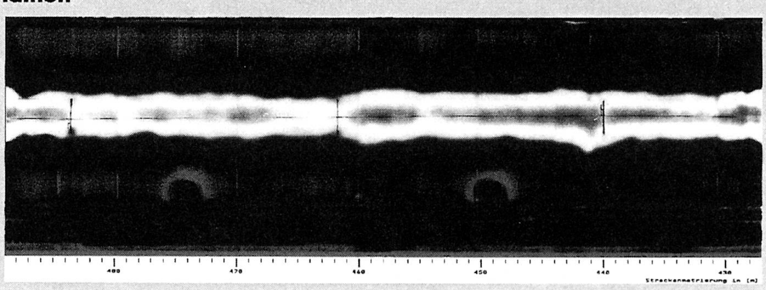
Tunnel Scanner TS 360

Fuchsenwinkel Tunnel

- **Problem section**



- **Normal condition**



Amberg Measuring Technique

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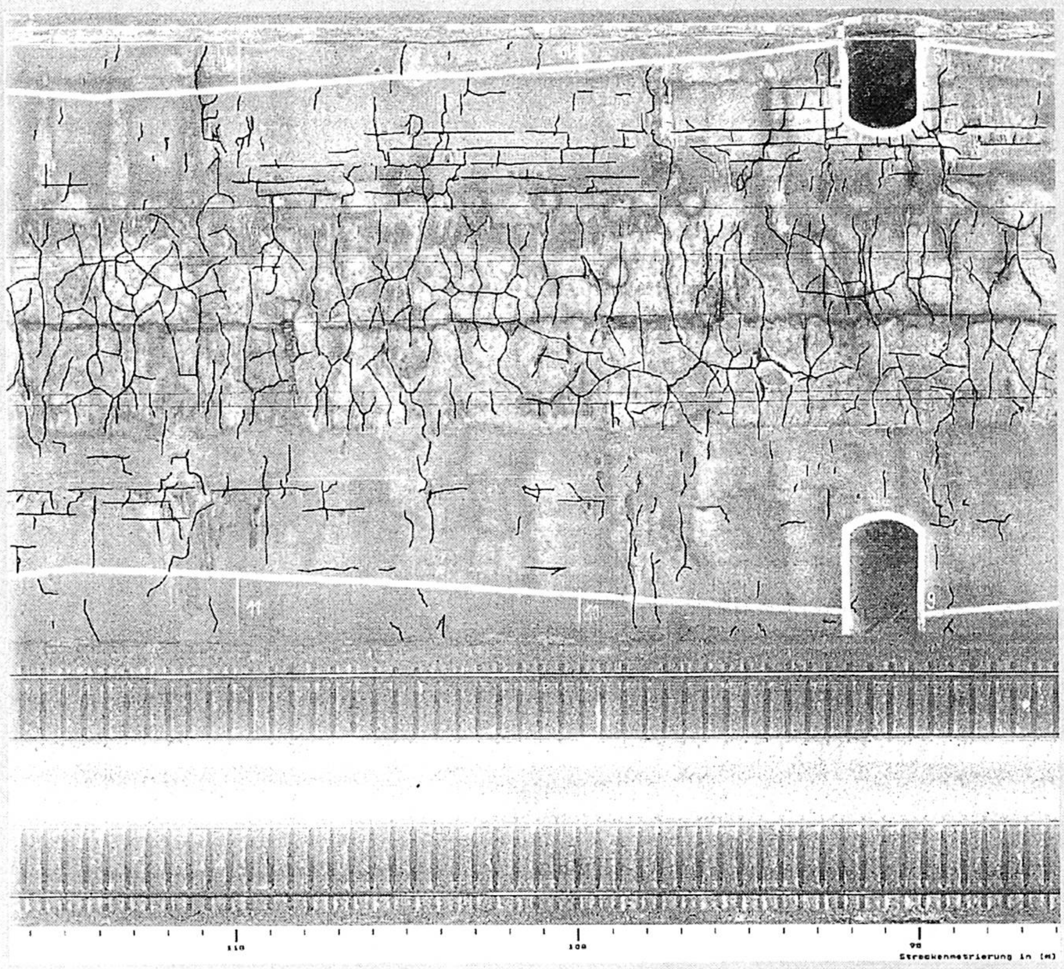
Figure 3: Quality control

Tunnel Scanner TS 360

Crack chart

- **Rattenberg tunnel (taking of evidence)**



Amberg Measuring Technique

5

Figure 4: Crack chart

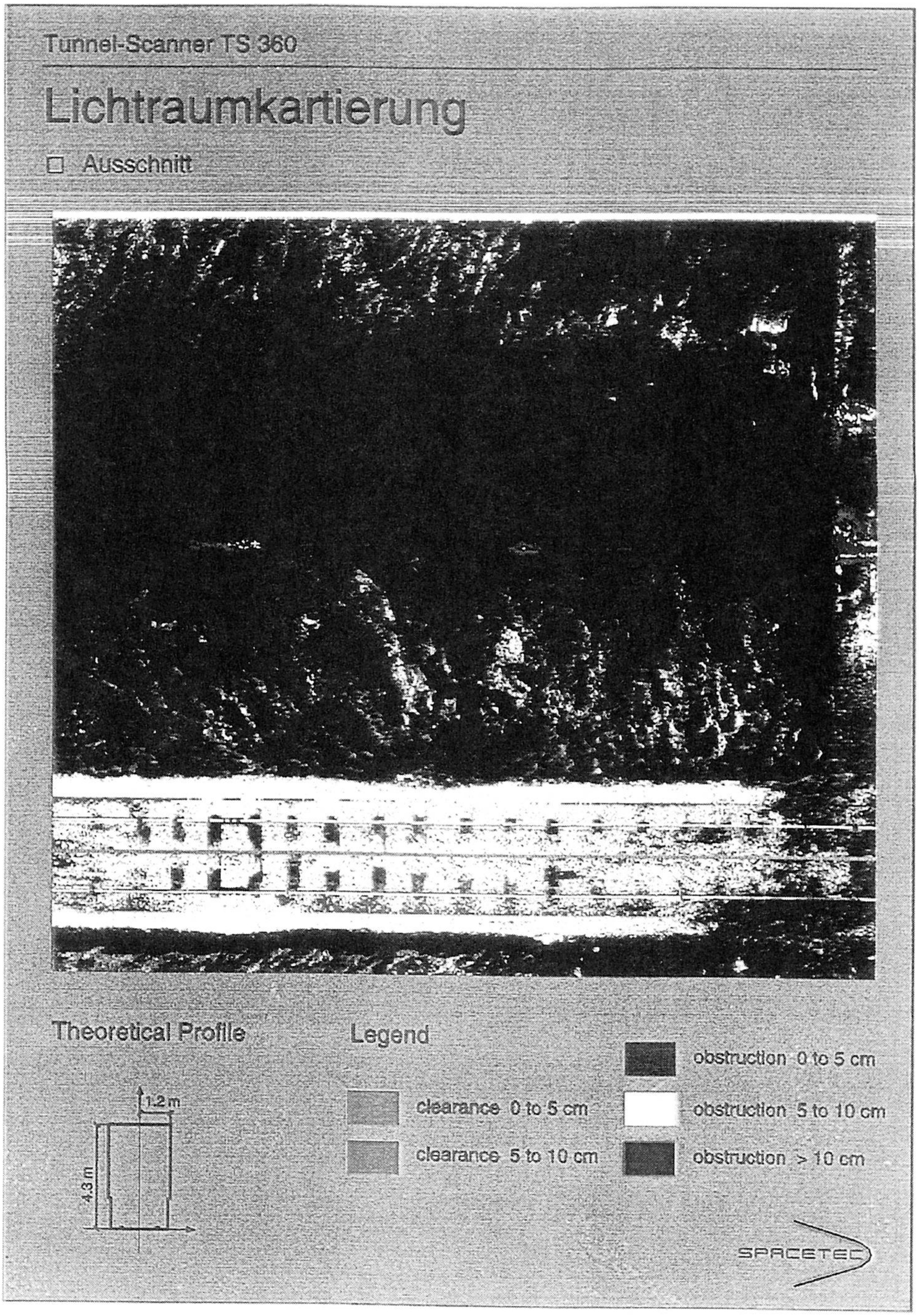


Figure 5: Clearence Map

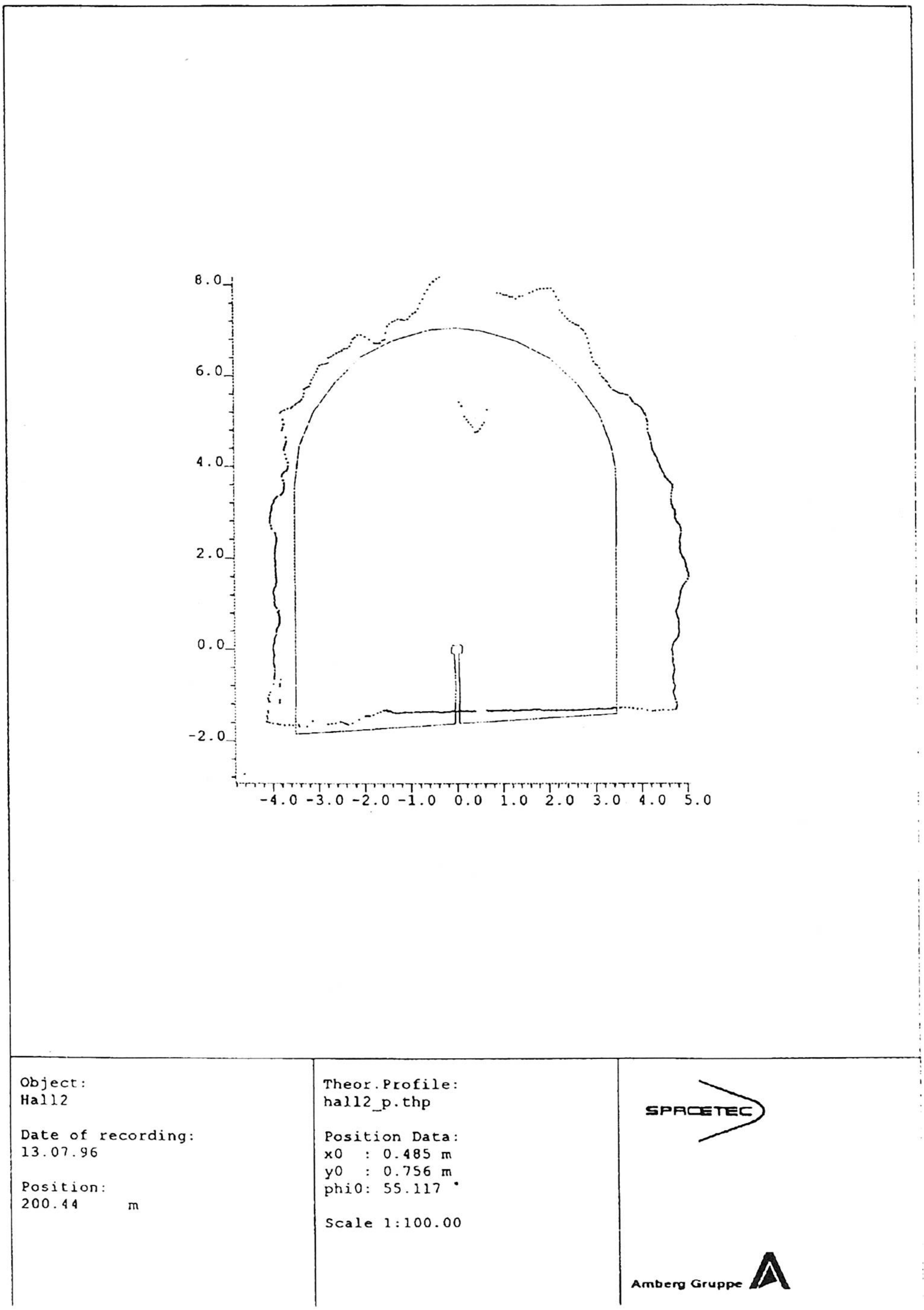


Figure 6: Cross section

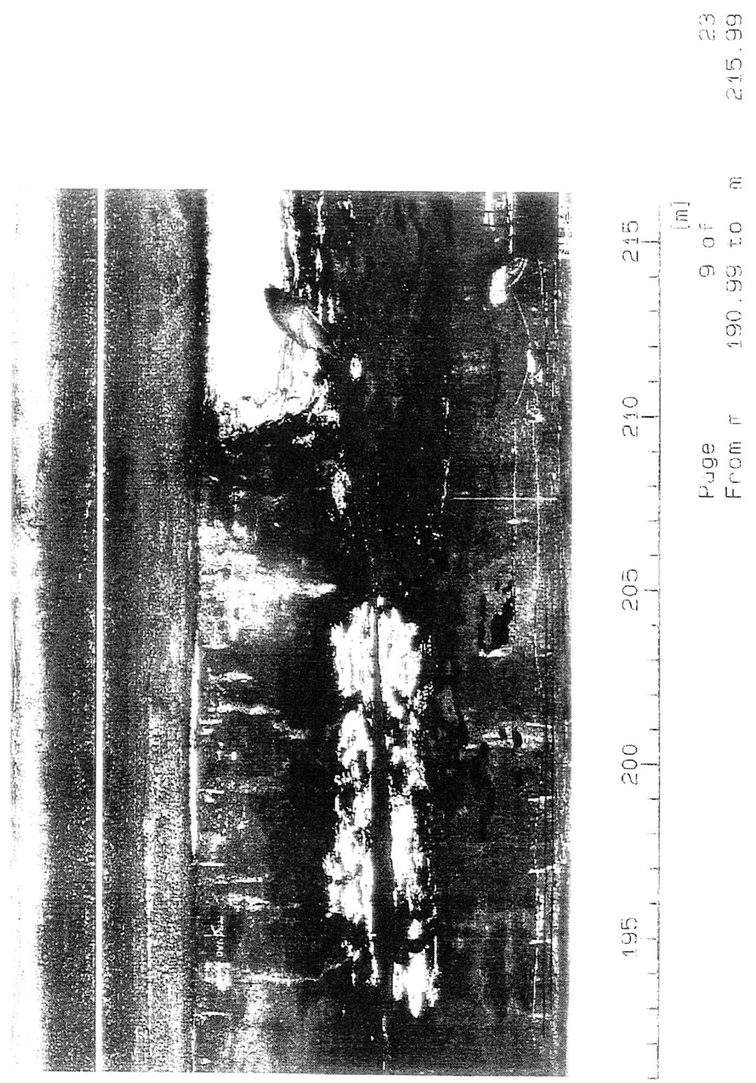


Figure 7: Clearence Map

Concrete Pavements in Tunnels

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Johan Silfwerbrand, born in 1958, received his Ph.D. in Civil Engineering from KTH in 1987. His research interests are in concrete repair, creep and shrinkage, concrete pavements and overlays, and steel fibre reinforced concrete structures. He is active in ACI, CEB and RILEM committees.

Summary

Road tunnels will be more common in the future. Concrete pavements constitute an interesting alternative to asphalt pavements, since they have high bearing capacity, wear resistance, and fire safety as well as low maintenance costs and a bright colour. Two concrete pavement solutions have been proposed to be used in Swedish road tunnels. The main alternative consists of a 190 mm thick, high strength, jointed concrete pavement placed on 300 mm unbound layers. Unsorted crushed-rock masses must be removed. The concrete pavement thickness is less than the corresponding one in the open, since the tunnel floor constitutes a stiff subgrade and the thermal gradient is low due to the absence of solar radiation. In the paper, factors that may influence the choice of pavement solutions are discussed. Finally, research needs are outlined.

1. Introduction

The amount of road tunnels constructed underground is expected to increase greatly in the coming years. The main reason is environmental demands that force new roads to be placed underneath the ground surface. In Sweden, long road tunnels are either under construction or planned both in Stockholm and Göteborg. The tunnel surrounding leads to special conditions for a pavement; both advantageous and disadvantageous. A stiff subgrade and a more constant climate without rain and snow are two advantages. Disadvantages are lighting needs both day and night, consequences after fires and other accidents, possible drainage problems, and difficulties and costs connected with repair and maintenance works.

Concrete pavements constitute an interesting alternative to more traditional asphalt pavements. Concrete offers higher wear resistance, higher bearing capacity, improved fire safety, lower maintenance costs, and better light conditions. All these advantages are especially important in a tunnel.

2. International Experience

According to a Swiss handbook [3], concrete pavements in Swiss road tunnels have usually a thickness varying between 120 and 190 mm. Both plain jointed concrete pavement (PJCP) and continuously reinforced concrete pavements (CRCP) occur. The concrete pavement is resting on an unbound gravel base. Between tunnel floor and gravel, there is a cement or asphalt bound layer of varying thickness. The purpose is to adjust the uneven tunnel floor and to protect the gravel layer against erosion. In short tunnels and in the exterior parts (0-1000 m from the opening) of long tunnels, the pavement thickness is the same as the thickness of the pavement outside the tunnel. In Austria, the concrete pavement thickness does not change when the road is going through a tunnel. That gives an increased safety against damage in the tunnel. It is valuable, since repair and maintenance costs are higher in the tunnel.

During the 1970s, a couple of Norwegian road tunnels were constructed and provided with concrete pavements, see, e.g., Silfwerbrand [5]. Both PJCP and CRCP were used. The concrete thickness varied between 100 and 180 mm. Very thin PJCP cracked considerably. In a recent Norwegian road tunnel, the concrete pavement thickness is 220 mm. The concrete pavement is resting on three unbound gravel layers with a total thickness of 200 mm. The concrete thickness includes an extra thickness in order to enable a couple of millings. Millings are needed in Norway to solve rutting problems that occur, since both cars and lorries are allowed to use studded tyres during winters.

3. Proposed Concrete Pavements in Swedish Road Tunnels

3.1 Design Requirements and Precautions

The following design requirements established the basis for the design:

- Design period: 40 years.
- Number of 100 kN standard axes during the design period: 60 millions.
- Distance between wheel edge and pavement edge: > 0.5 m.
- The proposed pavement is suitable for the interior parts of the tunnel, i.e., parts located > 800 - 1000 m from the tunnel openings.
- Unsorted crushed-rock masses and debris, that also might be connected to frost heave risks, should be removed.
- The removal of crushed-rock masses and debris should result in a solid and even tunnel floor.
- The tunnel should be provided with an adequate drainage in order to prevent water flow, if any, to undermine the pavement.
- Use of high strength concrete (HSC) with characteristic flexural strength equal to 7 MPa.
- Proposed concrete pavement thickness constitutes a minimum value, not an average.
- Proposed concrete pavement thickness includes an extra thickness amount enabling milling twice with a total milling depth of 30 mm.
- The concrete pavement should be provided with doweled transverse joints every fifth meter.

The consequences of a pavement damage might be more serious within the tunnel than in the open. This fact was considered by increasing the safety against failure. The pavement design followed the current Swedish pavement design practice, see, e.g., Petersson [4] or Silfwerbrand [6]. It is a mechanic design method based on Eisenmann [1]. Stresses due to traffic loads and thermal gradients are superimposed and compared with an available flexural strength that is fatigue dependent.

The tunnel pavement has two advantages compared to the pavement in open: (i) the rocky tunnel floor constitutes a stiff subgrade (especially compared to clay subgrades) leading to reduced traffic stresses and (ii) the interior tunnel climate means thermal gradient with less magnitude and less duration leading to reduced thermal stresses. According to the Swedish pavement design practice, the thermal gradient is assumed to be 0.06 °C/mm during 5 % of the year, 0.04 °C/mm during 20 %, and 0 during the rest of the year. In the tunnel, the thermal gradient is assumed to be 0.02 °C/mm during 5 % of the year and 0 during the rest of the year. The assumed thermal gradient was based on temperature measurements in a concrete pavement located in a Norwegian tunnel (Hakvåg [2]).

3.2 Concrete Pavement Solutions

The proposed concrete pavements are shown in Fig. 1. The main alternative consists of a 190 mm thick HSC pavement on an unbound gravel base. The second-hand alternative consists of 180 mm HSC and 80 mm lean concrete on an unbound gravel base. The difference in concrete thickness is very limited. The reason is that a lean concrete layer only to a minor extent contributes to the load carrying capacity in a case of a concrete pavement on a stiff subgrade like a rocky tunnel floor.

Comparison calculations in Silfwerbrand [5] show that the tunnel pavement might be 25 % thinner than corresponding pavement in the open.

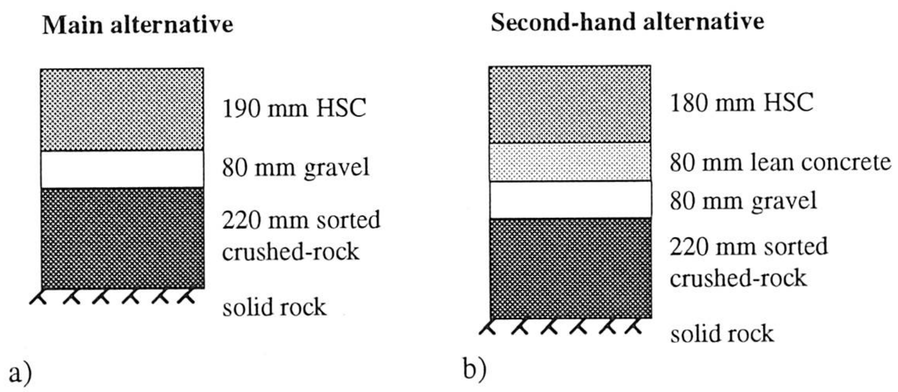

Fig. 1 - Proposed concrete pavements

4. Factors That May Influence the Proposed Pavement Solutions

4.1 Effects of Remaining Unsorted Crushed-Rock Masses

In order to save time and money, the contractor might wish to keep the unsorted crushed-rock masses instead of removing them. It will cause two problems: (i) the stiffness of the subgrade is reduced leading to increased traffic stresses and (ii) the risk of frost heave is increased. The second problem is the most serious one, especially in countries like Sweden with minimum winter temperatures considerably below zero. If the masses cannot be removed, first-class drainage and insulation are needed. The stiffness problem can be solved by increasing the concrete thickness with 10-20 %. Thick layers of crushed-rock masses might cause increased deflections under heavy vehicles. In order to prevent pumping effects, the second-hand alternative (Fig. 1b) with a lean concrete layer underneath the concrete pavement ought to be chosen in that case.

4.2 Effects of Tunnel Ventilation

The effects of the necessary tunnel ventilation has been a major concern in Swedish discussions on tunnel pavement design. In Austria, e.g., the ventilation is considered to influence the climate substantially also in the interior parts of the tunnel. That means that the concrete pavement within the tunnel would be exposed to thermal and moisture movements like the pavement in the open. The proposed concrete pavement solutions are based on the assumption of a rather small thermal gradient. It is in turn based on Norwegian measurements. Intensive ventilation might increase the thermal gradient but the gradient will hardly reach the values obtained in the open, since these gradients are results of solar radiation. However, thorough measurements of thermal gradients in tunnel pavements exposed to intended ventilation conditions are desirable.

4.3 Effects of Uneven Subgrade

According to the requirements, the tunnel floor should be solid and even. However, it might be difficult to succeed. What happens if the rock shoots up in a point under the pavement slab? Will it cause stress concentrations? The problem has been studied using a model of a concrete slab resting on a Winkler foundation (elastic foundation) (Fig. 2a) and solving the well-known differential equation (see, e.g., Timoshenko & Woinowsky-Krieger [7]) by using the finite difference method. A "rigid" point support was created by giving one of the springs a stiffness equal to 1000 times the ordinary spring stiffness.

Flexural stresses due to a point load at different locations were computed. Results for slabs with point support were compared with results for slabs without point support. If the point support happened to be located directly under the load, the flexural stress under the load is reduced considerably. Other locations of the point support only caused minor differences in flexural stresses. The maximum stress increase found was limited to 3 % (Fig. 2b).

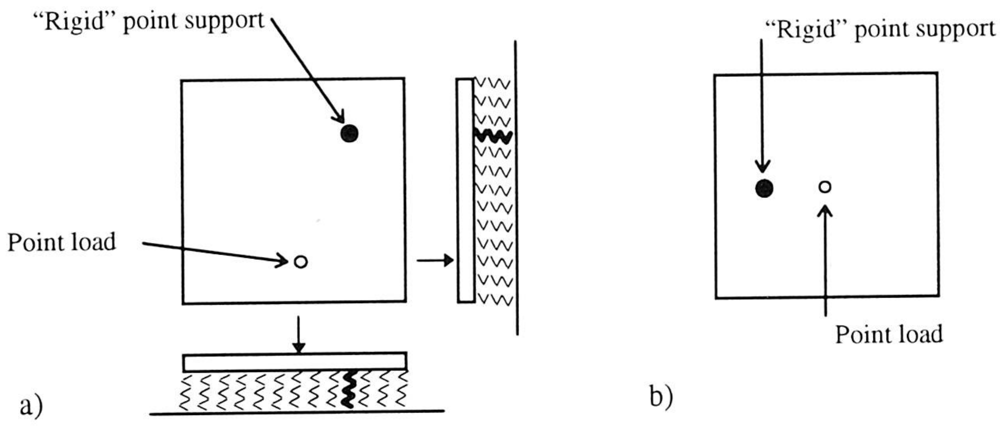

Fig. 2 - Model of concrete pavement on Winkler foundation (a) and example of load and support locations giving 3 % increased flexural stress (b).

4.4 Effects of Rigid Pavement Edge

In Stockholm, it has been suggested that the power supply should be provided by electric cables within concrete conduits along the exterior edge of the pavement. Structurally, the concrete conduit would constitute a continuous rigid support for the concrete pavement (Fig. 3a). Will this continuous simple support cause any stress increases? The problem has been studied using a model of a concrete slab resting on a simply supported edge and the remaining parts on a Winkler foundation (Fig. 3b).

Flexural stresses due to a point load at different locations were computed. The conclusion is that the continuous support did not influence the flexural stresses due to the point load.

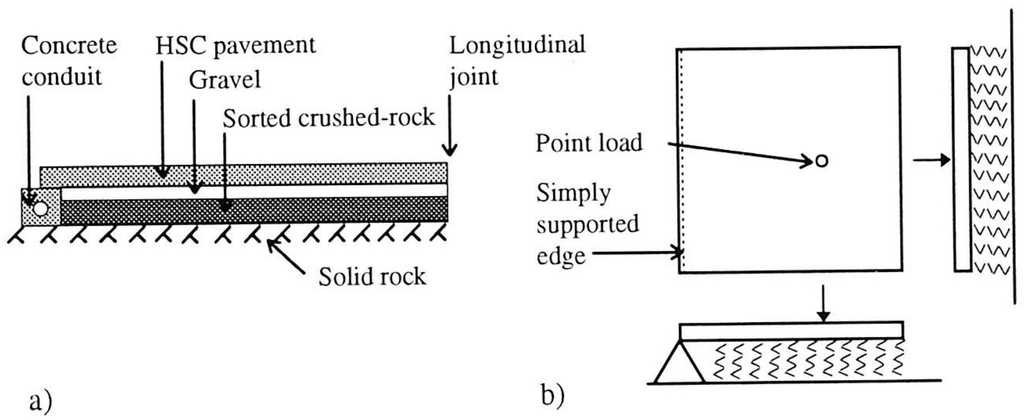

Fig. 3 - A longitudinal concrete conduit is placed on the exterior edge of the pavement (a) and model of concrete pavement with a simply supported edge and remaining parts on Winkler foundation (b).

5. Research Needs

The calculation method used to calculate traffic stresses at a certain distance from the road edge is approximate but has a large impact on the calculation result. Does the calculation method agree with reality? Do current calculation methods overestimate the real traffic stresses close to the pavement edge? Stiffness values of unbound layers are needed to calculate traffic stresses. Can stiffness values estimated by falling weight deflectometer measurements on thick unbound layers on soft subgrades be used also for thin unbound layers on solid rock? Is the constant and thereby favourable climate in the inner part of the tunnel destroyed by ventilation? More research is required to answer these questions.

6. Concluding Remarks

Road tunnels will be more common in the future. Concrete pavements constitute an interesting alternative, since they have high bearing capacity, wear resistance, and fire safety as well as low maintenance costs and a bright colour. Two concrete pavement solutions have been proposed to be used in Swedish road tunnels. The concrete pavement thickness is less than the corresponding one in the open, since the tunnel floor constitutes a stiff subgrade and the thermal gradient is low due to the absence of solar radiation. Unsorted crushed-rock masses should be removed before paving. Otherwise, the pavement thickness ought to be increased with 10-20 % and special care has to be taken to prevent frost heave. Calculations show that an uneven tunnel floor only negligibly effects the pavement stresses.

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Rehabilitation of Two Existing Tunnels In West Malaysia: A Case Study

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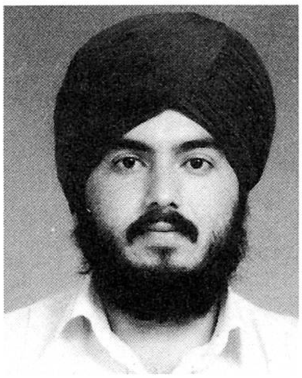

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Summary

The Meru and Menora tunnels situated along West Malaysia's North-South Expressway were recently rehabilitated after more than 10 years in service. As part of the rehabilitation scheme an investigation programme was carried out to determine the nature and extent of defects in the ten year old concrete lined tunnels. Two main types of deterioration were investigated namely cracking and faulty expansion joints. The most widespread type of defects observed in the lining of the two tunnels was cracking. After having carried out the investigation programme a rehabilitation scheme was implemented to reinstate the existing defects. This paper highlights findings of the investigation programme including details of the overall rehabilitation scheme. It also makes recommendations for future maintenance and suggests long term monitoring of functionality and durability of the tunnel structure based on local Malaysian practice.

1. Introduction

The North-South Expressway linking the entire length of Peninsular Malaysia from the northern border with Thailand and Singapore in the south was completed in 1995. The entire road network measuring approximately 850km was designed to serve increased user demand and exceptionally high annual traffic growth rate. With her present 8% rate of economic growth, the need for a modern infrastructure of road networks is essential for Malaysia's development. In 1988, the Government of Malaysia awarded a Concession to finance, design, construct, manage, operate and maintain the entire stretch of the North-South Expressway. The Concessionaire is expected to operate and maintain the highway for 30 years before handing it over to the Government by mid 2018. The highway consists of a two-lane dual carriageway with an emergency lane on each bound for certain stretches and a three-lane dual carriageway with an emergency lane on each bound for the rest. Almost 79.5% of the expressway pavement is paved with asphalt whilst the rest is made of concrete. Between the Jelapang Toll and Changkat Jering Toll, the expressway was built on hard rock. Close to the Jelapang toll, two tunnels were built after boring through hard granite rocks. The Meru Tunnel (north bound) and Menora Tunnel (south bound) were constructed between 1984

Fig. 1 Route map

and 1986. The Meru Tunnel measures approximately 821 metres whilst the Menora Tunnel is almost 832 metres in length. The climate of the site is tropical with a relative humidity range of between 70% to 100%. Average temperature of about 27°C is experienced throughout the year. The climate is generally characterised by a small seasonal variation of temperature, high relative humidity and pronounced wet and dry seasons. The average yearly rainfall is approximately 1,914mm. The months between January and March are generally marked by dry spells followed by moderately wet months of April and May. June, July and August are moderately dry months whereas September to December are marked by rainy seasons. After almost 10 years in service , a number of defects were observed in the concrete lining of both the tunnels. Cracks were observed in the concrete linings along the length of the tunnels. Severe signs of seepage and leaching were apparent in the cracked concrete linings.

Expansion joints which are spaced at every 6 metres along the length of the tunnels also showed signs of aging. Apart from the above defects, a faulty drainage system and fire hydrant system was also noted by the Maintenance team during a routine inspection in 1995. Following the routine inspection, a detailed investigation was carried out to determine the extent and type of defects in the tunnels.

2. Assessment Programme

A preliminary investigation was carried out to identify the existing defects and their extent in the different locations of the tunnel. A photographic survey was carried out to classify the different defects according to their severity. Existing drawings and past records related to the construction of the tunnels were reviewed to obtain useful data for detailed testing. At this stage, a preliminary scheme was drawn to ascertain the types of test and method of conducting these tests to suit site conditions. A traffic management scheme was prepared to ensure minimal disruption to traffic flow during the course of the investigation works.

2.1 Visual Survey

A detailed visual inspection carried out on the cracked concrete did not reveal signs of rust or corrosion of rebars. Localised break-outs were carried out to confirm the absence of rebar corrosion. Leaching of cement through wet cracks was evident in cracks located above the spring line of the tunnel. A detailed crack mapping exercise was carried out to document the extent and pattern of cracks in the lining. This was done by dividing the entire length of the tunnel into chainages and marking individual cracks according to their chainage. The total length of cracks measured for both the tunnels was close to 8500 metres. Most of these cracks were between 0.3mm to 3mm wide.

A detailed inspection was also carried out to determine the extent of damage in the existing expansion joints. It was noted that most of the sealant in the expansion joint was loose and damaged. A separate inspection was carried out to check on the efficiency of the existing fire hydrant system. The inspection confirmed that a section of the existing fire hydrant system in the north bound tunnel was faulty. Further inspections were carried out on the existing drainage system along side the kerbs. This was to ensure that water collected behind the concrete lining could be effectively conveyed into the drainage system via the existing weep holes.

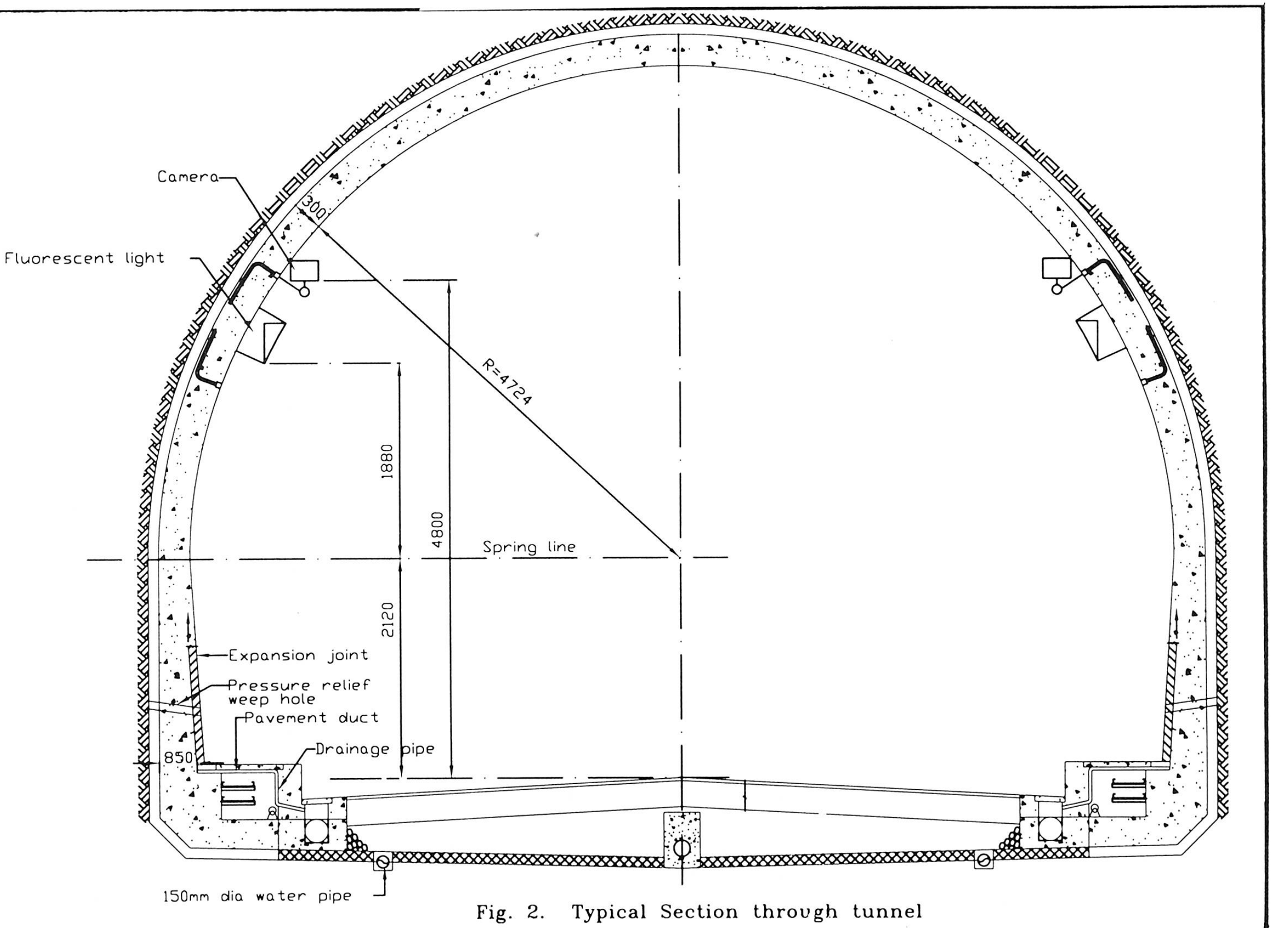

Fig. 2. Typical Section through tunnel

2.2 Detailed Tests

Based on the findings of the preliminary investigation a series of test were conducted to determine in-situ condition of concrete in the tunnel linings. As a means to determine surface hardness, Rebound Hammer test was carried out at selected locations after installing a mobile working platform. One lane of the dual carriageway was closed to traffic during the testing works. In order to determine the extent of cracks in the concrete a number of 50mm diameter core samples were extracted at cracked locations of the concrete. In most core samples the depth of cracks were observed to be in excess of 300mm. In some core samples the cracks were observed to have a series of smaller width cracks branching from the main crackline. Most of the cracks examined under a magnifying glass showed signs of contamination by dirt. An attempt was made to determine the depth of cracks using the indirect method for Ultrasonic Pulse Velocity test in areas where core sampling could not be conducted. This test served as a complementary test to compare sound concrete with cracked concrete by non-destructive methods. A hammer test was also conducted to check for weak spots in the concrete. This test was performed as a guide to differentiate between "ring" sound (often indicative of good concrete) and a 'hollow" sound for loose or cracked concrete. Carbonation test was carried out on selected locations by spraying phenolphthalein indicator on freshly exposed concrete. At most test locations the depth of carbonation was noted to be close to 5mm.

3.0 Rehabilitation Scheme

Following the detailed investigation programme a rehabilitation scheme was prepared to reinstate the existing defects in the tunnels. The scope of the rehabilitation programme was focussed primarily in the following areas:-

i) Repair of dry and wet cracks in the tunnel linings.
ii) Reinstatement of faulty expansion joints
iii) Rehabilitation of the drainage systems
iv) Upgrading of the fire hydrant system

3.1 Repair of Dry and Wet Cracks

The appearance of cracks in the tunnel lining of both the tunnels was generally of concern to most motorists using the tunnels. However, from the detailed investigation carried out during the assessment programme it was noted that the pattern of cracks was typical of map cracks as shown in Fig. 3. The dry crack pattern was typical for both tunnels and core samples obtained from cracked concrete generally gave details of depth of cracks. Fig. 4 shows typical crack depths as seen in six 50mm core samples obtained from the tunnel linings. A detailed crack mapping exercise was carried out to locate the extent and severity of the cracks according to established chainages along the total length of the tunnels. The cracks were further divided into two main types; dry and wet cracks. Almost seventy percent of the 8500 metres length of cracks measured for both the tunnels fell under the dry crack category. Most of these cracks were contaminated with dirt particles. The remaining cracks were typically wet owing to their depth and location which was close to faults in rocks. The thickness of the tunnel lining varied between almost 850mm at the base

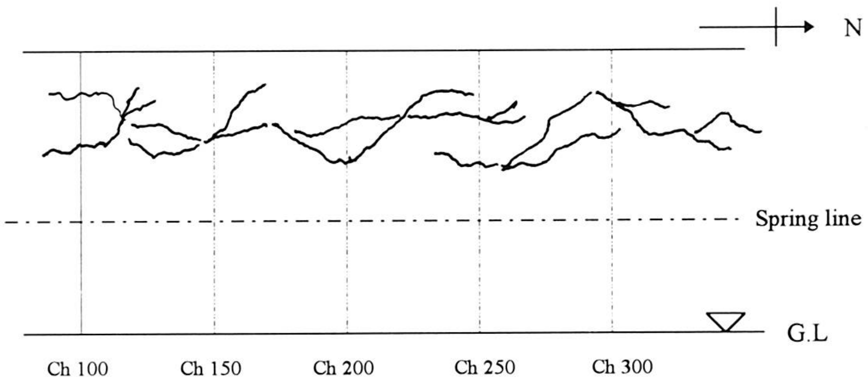

Fig.3 Typical map cracks observed in linings of north bound tunnel

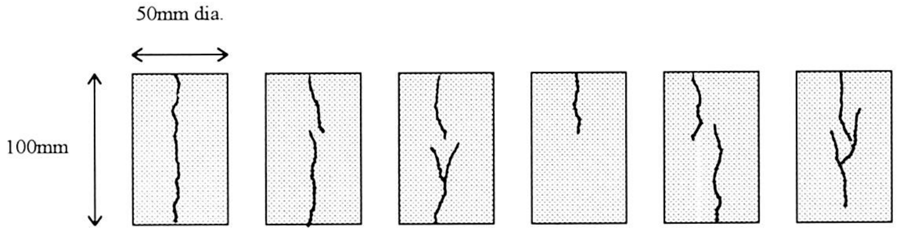

Fig.4 Section through 50mm diameter core samples

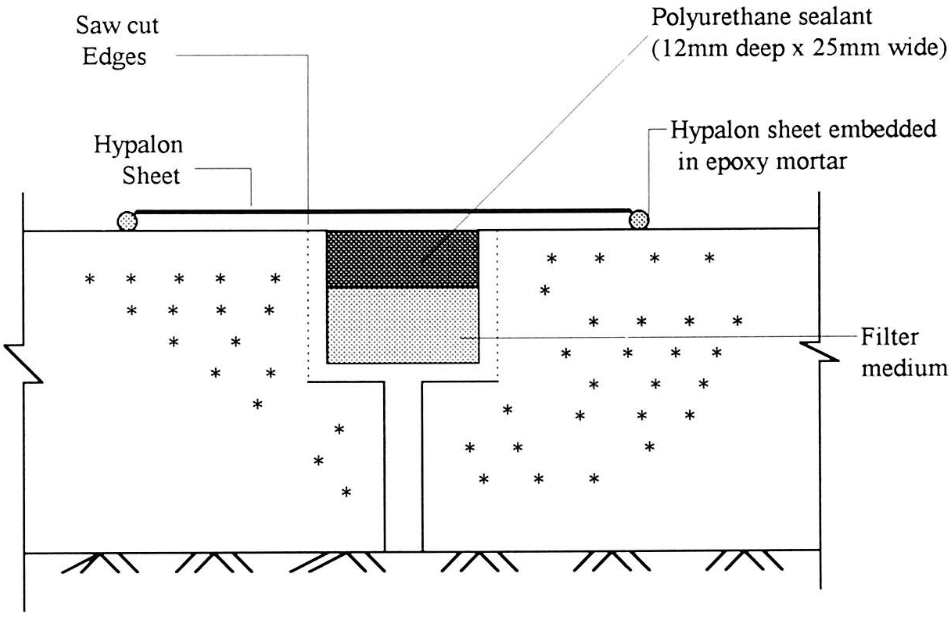

Fig.5 Section through expansion joint

of the tunnel to nearly 300mm at the crown. Most of these wet cracks were contaminated with lime salt deposits.

The dry cracks were first cleaned with a high pressure (1000 p.s.i) water jet using potable water. This was to ensure that the cracked surface was properly prepared prior to the crack repairs. A temporary seal was applied along the length of the cracks leaving masking seals along regular intervals. The temporary seal is an epoxy compound which serves as a barrier to prevent the flow of epoxy from the interior to the surface of the concrete during injection. As soon as the temporary seal was cured, the masking strips were removed exposing cracked concrete at regular intervals which served as entry ports for epoxy injection. The epoxy injection system used for sealing the dry cracks consisted of a low viscosity two component (resin & hardener) epoxy resin which is injected into the cracks using a high pressure, high speed, electromechanical handgun. The primary advantage of this electromechanical system is its ability to mix accurately the two components of the epoxy i.e resin and hardener in the desired ratio eliminating the possibility of inaccurate mix ratios. The epoxy resin is dispensed into the crack via a rubber nipple mounted onto the nozzle of the hand gun. The rubber nipple is pressed against the entry port to allow the entry of the epoxy into the crack. As soon as the epoxy is seen emerging from the adjacent entry port, the flow of the epoxy is temporarily halted by rubbing dry candle wax into the oozing port. Epoxy injection is continued until it appears in the successive entry port. The above steps are repeated until all the entry ports are injected with the epoxy.

The procedure for repairs to wet cracks differs a little from the dry cracks. For wet cracks, 8mm diameter holes are drilled along regular intervals of the crack length to serve as entry ports. Six mm diameter copper tubes approximately 75mm long are inserted into the holes. A temporary epoxy seal is applied to the surface of the cracks to seal off the crack surface between entry ports. The procedure for injection is repeated from the lower most entry port until all successive entry ports are completely injected using the hand gun. The sealing of the cracks using epoxy ensures that the cracks are structurally bonded and free from leaks.

3.2 Reinstatement of Faulty Expansion Joints

The main defect observed in faulty expansion joints was loose and hardened sealant which had debonded from the existing groove. Some of the joints were affected by seepage of water from behind the tunnel lining. The defective sealant was completely removed to ensure the affected expansion joint was free from loose and debonded material. A chase measuring 25mm wide and 50mm deep was cut along the length of the faulty expansion joint using a diamond saw. The joint was cleaned with a high pressure water jet to prepare the surface for remedial treatment. A filter medium was then installed along the length of the expansion joint to convey trapped water into an adjoining uPVC pipe which was connected to the existing drain. The surface of the expansion joint was then sealed with a 12mm thick polyurethane sealant to prevent seepage of water from the filter membrane. As a protection to the exposed sealant, a hypalon sheet was installed along the length of the joint as shown in Fig. 5.

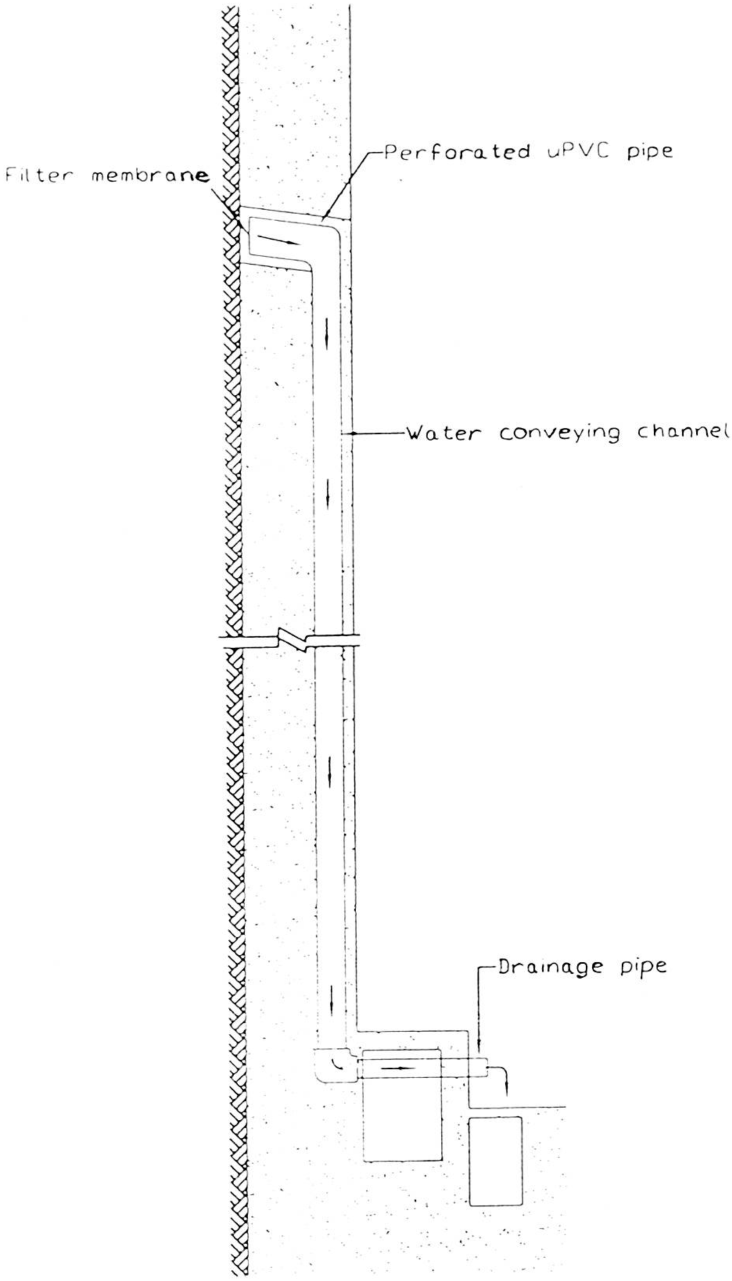

Fig. 6 Section through expansion joint showing drainage system

3.3 Rehabilitation of the Drainage System

In expansion joints where excessive seepage of water occurred, additional water relief holes were drilled into the existing concrete lining. A 30mm diameter uPVC perforated pipe was inserted into the concrete lining after cutting a chase. The perforated section of the uPVC pipe was covered with a geotextile to prevent entry of contaminants into the perforations. Water collected in the uPVC pipe was then directed into the existing drains which were located on either sides of the tunnel. The side drains were thoroughly cleaned to ensure these were free form blockages. Fig. 6. shows a typical section of the modified drainage and water relief system.

3.4 Upgrading of the Fire Hydrant System

Based on the findings of the inspection programme it was confirmed that the fire hydrant system located at the north bound tunnel was not functioning to its original serviceability level. In order to ascertain the "most probable" location of the fault, a pressure test was performed prior to the repairs. The fault was believed to have lied at a joint between the existing 150mm diameter pipe and a "Y" flange close to the elbow connection near the hose reel. Since the affected joint was buried underground, a section of the kerb had to be hacked to gain access to the faulty joint. The faulty joint was removed and replaced with a new pipe and joint system including new fittings. A final pressure test was conducted to check on the efficiency of the reinstated fire hydrant prior to backfilling with new concrete.

3.5 Suggestion for Future Maintenance and Repair

The once held belief that concrete is a durable material and requires minimum maintenance is a thing of the past. Recent cases of premature deterioration of concrete in existing structures which have been reported locally also involve other civil engineering structures such as bridges, dams, jetties and tunnels. The need for periodic inspection to check for defects related to premature deterioration is essential in order to overcome repetitive and costly repairs to rectify such defects. Recent advances in the field of concrete testing and assessment has enabled specialist engineers to diagnose defects related to concrete strength and durability. Maintenance personnel should be trained to monitor functionality and durability of concrete in existing structures on a routine basis. Data obtained from the monitoring can be analysed to identify the most cost effective maintenance strategy.

4. Project Management and Traffic Control

The rehabilitation scheme commenced in July 1995 and was completed in October 1995. During the sixteen weeks period proper traffic control measures were taken to ensure smooth flow of traffic through both the tunnels. Safety cones and traffic signage were placed at designated spots within the tunnels. Safety cones were placed at 7m apart spacing in the tunnel and at 4m spacing for a distance of 150m before the entrance and 150m after the exit point. Blinkers were placed at 7m spacing to ensure motorists could slow down well in advance of the working area in the tunnel. Rehabilitation works were carried out simultaneously in both the tunnels by closing each half of the tunnels in turn thus ensuring a free flow of traffic in both directions. However, during the course of the works a total of five working days were lost due to heavy flow of traffic during a festive season in middle of October and a minor accident which occurred outside the north bound tunnel. The rehabilitation works were carried out between 7am to 7pm on weekdays. During the course of the works, the client's project management team liaised closely with the specialist contractor to ensure smooth progress of the works. Quality control procedures stipulated in the contract were implemented by the client's representative.

5. Conclusion

This paper has attempted to highlight findings of an assessment programme carried out in two concrete lined tunnel of Malaysia's North-South Expressway which had to be rehabilitated after 10 years in service owing to the presence of defects. Both dry and wet cracks which were discovered during the assessment stage were reinstated using a proprietary epoxy injection system. A modified version of the expansion joint was introduced to provide additional protection to the sealant in some faulty expansion joints. Additional relief holes were introduced in leaking expansion joints to convey trapped water into the existing side drains. Faulty fittings in a section of the fire hydrant system for the north bound tunnel were replaced with new fittings. The overall rehabilitation scheme was successfully completed in sixteen weeks under tight supervision and a well planned traffic management system. Early indications are that no new cracks have developed in the tunnel linings and all sealed cracks do not show signs of leaking.

6. Acknowledgements

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Rehabilitation Work for the Upheaval Disaster at Underground Station Caused by the Groundwater Rising

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Summary

An abnormal rise in the groundwater level due to a spell of heavy rain resulted in ground upheaval of up to 1.34m over a 100m section at an underground station operated by the East Japan Railway Co. in the early morning of 12th October, 1991. The underground station is located in a cutting protected by U-shaped retaining walls, which are approximately 12m high and weigh approximately 120tf/m.
The railway line passes through tunnels on either side of the station.

In the restoration work, the elevation of the station structure was reduced by approximately 50cm by lowering the groundwater level. The space underneath the retaining walls was filled with low-strength non-segregating underwater mortar to ensure the stability of the lowered structure. As the station structure itself was undamaged, it was decided that it should be retained in use. In sections where sufficient lowering could not be achieved, the slabs were demolished and the ground was excavated for installation of new U-shaped slabs. As a measure against buoyancy the structure was fixed down with ground anchors to provide it with an adequate resistance even if the water level were to rise as far as the ground surface in future.

1. Introduction

During the night of 11th October 1991, damage was caused by the rain due to Typhoon No. 21 at Shin-Kodaira station on the Musashino line, which is located in a cutting (U-shaped retaining walls) with tunnels on either side. Rainfall had been almost continuous up to this point since 6th October, and the precipitation during this period was 227mm. Prior to this, a cumulative rainfall of 724mm had been observed between 1st August and 30th September, which was more than twice the average rainfall for these two months in the Musashino area. As a result of this rain, the groundwater level showed a sharp rise (to approx. 2.5m below the ground level), and the uplift (buoyancy) due to the groundwater caused upheaval (max. 1.34m) of the U-shaped retaining walls over a distance of 100m. The upheaval, in turn, resulted in a maximum opening of 70cm at the tops of the expansion joints in the retaining walls.

Large quantities of groundwater and earth flowed into the station through these openings, inundating the railway tracks. A major disruption of the passenger and freight traffic resulted during the subsequent two months until the restoration work was completed (Figure 1, Photos 1 and 2).
The report below is concerned with the causes of estimated disaster and design / execution method for the restoration work.

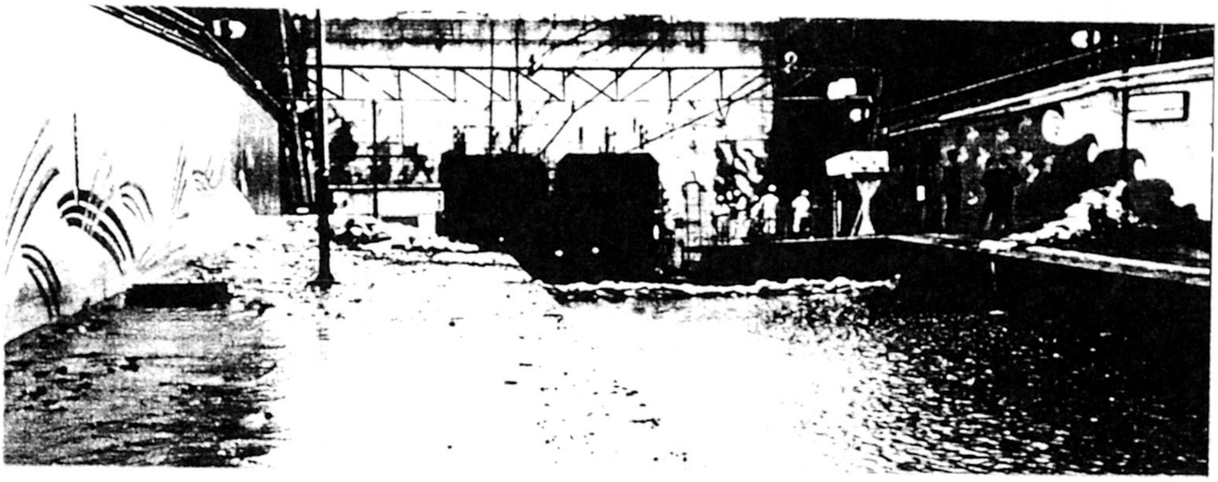


Photo 1 Conditions of Damage

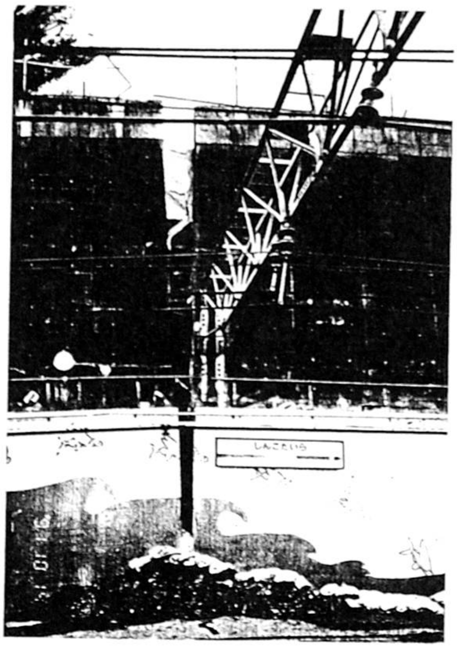


Photo 2 Conditions of Damage

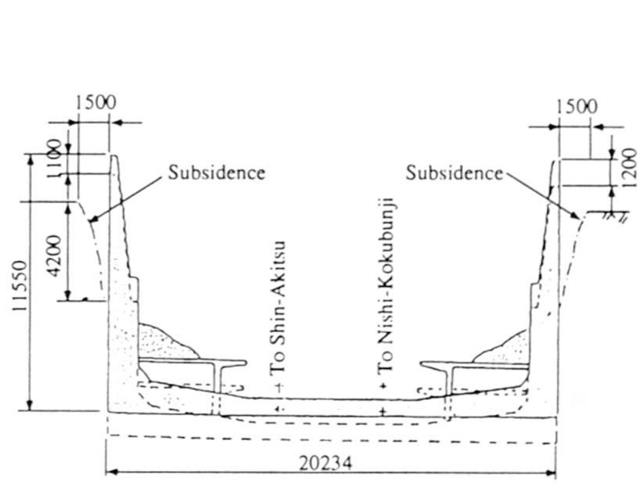


Fig. 1 Conditions of Damage

2. Shin-Kodaira Station

The Musashino line was constructed with the aims of improving the means of transportation by rail to central Tokyo and the connection between the satellite cities , as well as of promoting the development of the nearby areas. It was planned as a new circular line along a 20 to 30km radius from the center of Tokyo to supplement the existing circular line (Yamanote line) and runs from Nishi Funabashi to Fuchu-Honmachi. The construction work, implemented by the Japan Railway Construction Public Corporation, was begun in November 1966, and took approximately eleven years to complete before the opening of the last section in March 1976.
Shin-Kodaira station, opened on 1st April, 1973, is located in a cutting sandwiched between the Kodaira Tunnel (2,563m) on the Nishi-Kokubunji side, and the Higashi-Murayama Tunnel (4,380m) on the Shin-Akitsu side. It was constructed by the cut and cover method, using temporary earth retaining walls with H-shaped soldier piles. There are two side platforms on either side of the two railway tracks, each 6.0m wide and 140m long. The station is used by approximately 15,000 passengers per day (1990).

3. Geological Conditions

The Musashino Plateau, the diluvial plateau on which Shin-Kodaira station is located, is known as the largest plateau in Japan. Around Shin-Kodaira station, the Musashino gravel layer has a depth of 20m, below which are found alternating layers of clay containing shells and of sandy gravel. The Musashino gravel layer is covered with around 5m of Kanto loam soil. Groundwater is found under rather different conditions in the Musashino gravel layer and the sand layers further down. The Musashino gravel layer has a high void ratio and permeability and contains large quantities of groundwater. The groundwater in this area flows south-westwards towards the edge of the Musashino Plateau, crossing below the station (Figure 2).

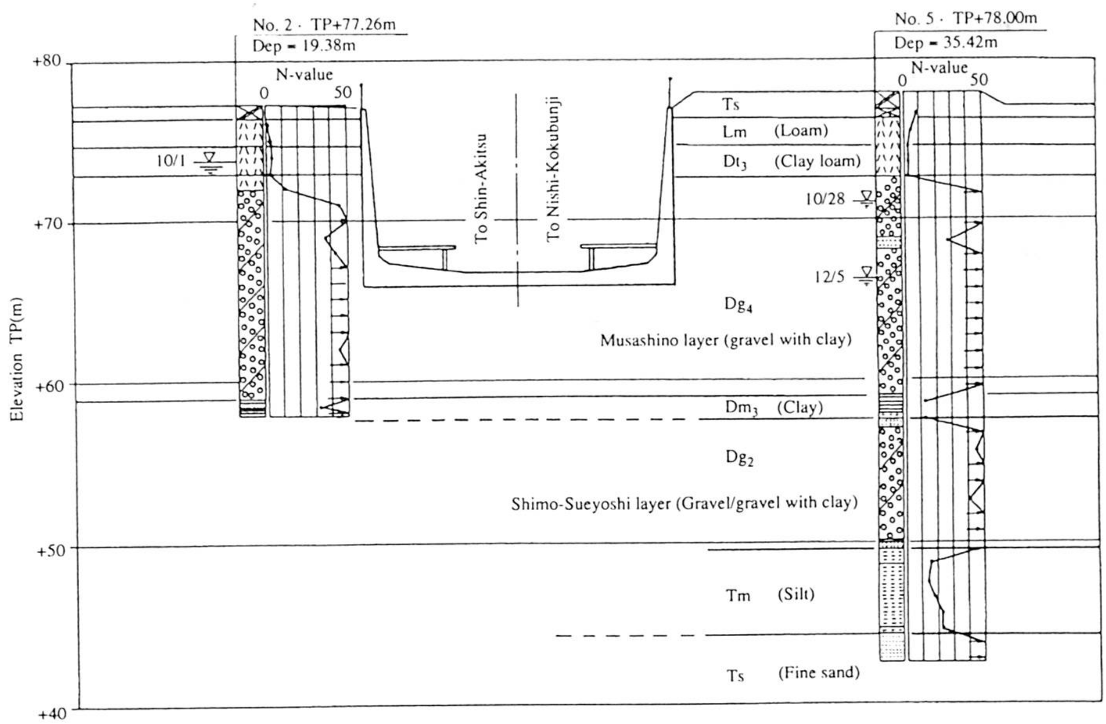


Fig. 2 Geological Profile

4 Restoration Work (outline)

4.1 Original Design

The U-shaped retaining walls that underwent upheaval were 11 to 12m high and were approximately 30m wide in the section supporting the bridge with the station building and 20m wide in the ordinary section. The ordinary section was divided into eight blocks with expansion joints at intervals of 10 to 15m.
As the original calculation documents were not available, calculations were made back from such factors as the weight of the structure (w=117.21 t/m) for the original U-shaped retaining walls approximately 20m in width and 12m in height, and it was estimated that the weight of structure would balance with the buoyancy when the groundwater level was 6m above the bottom of the slabs (5m below the ground surface). A slight margin would have to be allowed in this figure in view of such factors as the friction between the side wall concrete and the soil.

4.2 Estimated Causes of the Disaster (Buoyancy due to an Abnormal Rise in Groundwater Level)

Water quality analysis of the water entering the U-shaped slabs containing the track and platforms showed that it contained no organic components and was of a clearly different composition from that of the water in the nearby river. Therefore the water was judged to be groundwater and not sewage or river water.
Figure 3 illustrates data on groundwater levels gathered by Hosono of the General Center for Fire Prevention Science in Kodaira-Nakamachi, located approximately 1.1km east of Shin-Kodaira station. These observations continued for 24 years between 1968 and 1991. They show an annual cycle with the annual low water level around April rising through summer and autumn to reach the maximum. The highest groundwater level in the 23 years up to the preceding year was approximately 4m below ground level in 1974, but on 13th October 1991 the groundwater level exceeded this record by a further 1.5m, reaching 2.48m below ground level.

Investigation of the relation between the groundwater level and rainfall over these 23 years confirmed that the amount of rainfall over the preceding several months combined with the rainfall over the immediately preceding days produced a rapid rise in groundwater level. A relationship between cumulative rainfall over the preceding 60 and 90 days and the rise in groundwater level was also confirmed. Using 116 years of the precipitation data from the Central Weather Agency in Otemachi, Tokyo (27km away) the period of recurrence of the cumulative rainfall seen in the 60 days and 90 days surrounding the disaster was studied. In either case the recurrence period was 100 years (Tables 1 and 2). Calculations based on actual water level data put the recurrence period at 66 years.

The balance point between the self-weight of the structure of Shin-Kodaira station and buoyancy due to groundwater (neglecting the effects of friction with the soil) is reached when the groundwater level rises to approximately 5m below ground level. Thus it can be inferred that the continuing rain from mid-August to early October raised the groundwater to a level unprecedented in the life of the station and caused buoyant upheaval of the U-shaped retaining walls.

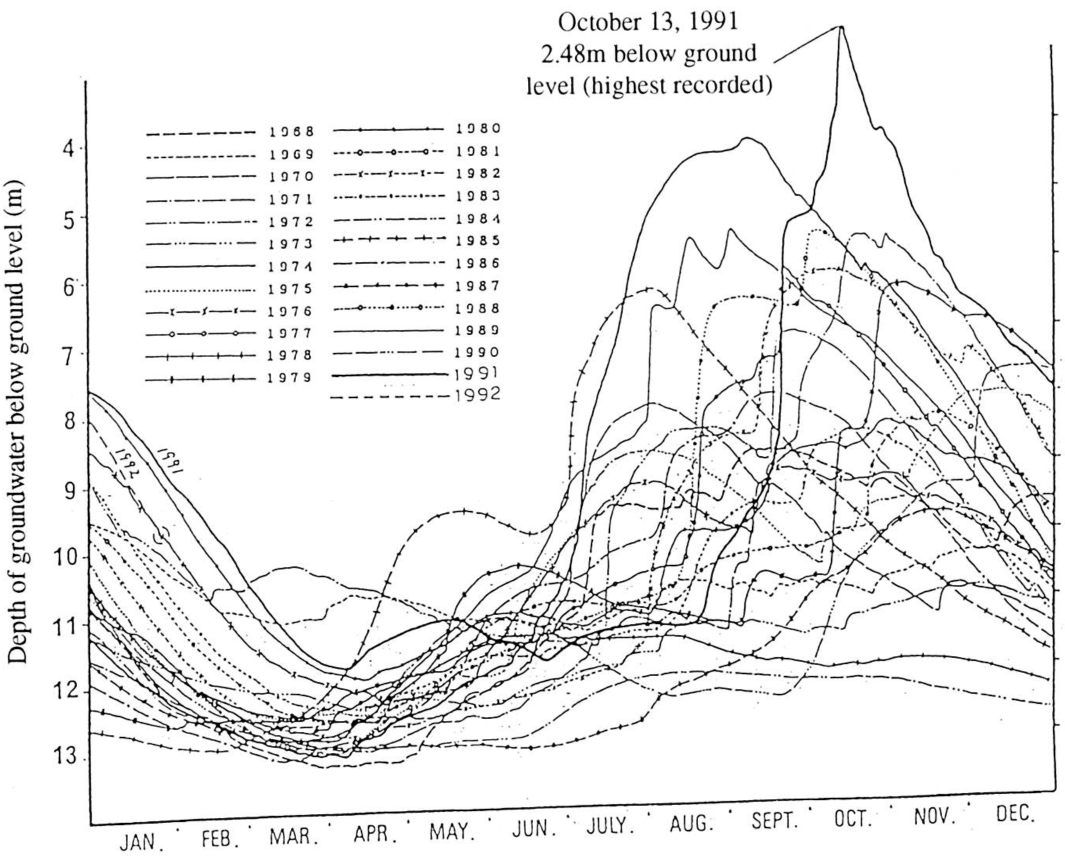


Fig. 3 *Changes in Water Level Over Time in a Well at Kodaira-Nakamachi (produced by Hosono committee member)*

Order	Year	Month	Day	Rainfall (mm)	Exceeding probability (%)	Recurrence period (years)
①	1991	10	17	1036.5	0.60	166.5
2	1938	7	15	958.0	1.25	79.8
3	1958	11	9	957.0	1.26	79.1
4	1941	8	3	940.9	1.47	68.0
5	1911	8	10	894.9	2.26	44.3
6	1929	11	2	867.6	2.91	34.4
7	1925	10	1	842.8	3.66	27.3
8	1921	10	12	785.5	6.16	16.2
9	1966	7	9	785.0	6.19	16.1
10	1924	10	23	740.9	9.17	10.9

Table 1 *The years With the Ten Highest 60-day Rainfall Records, and their Recurrence Periods*

Order	Year	Month	Day	Rainfall (mm)	Exceeding probability (%)	Recurrence period (years)
1	1938	9	3	1281.0	0.45	221.8
②	1991	11	8	1217.0	0.79	127.3
3	1941	8	14	1153.4	1.36	73.7
4	1929	12	1	1073.7	2.66	37.6
5	1958	11	18	1070.0	2.74	36.5
6	1921	10	20	1047.6	3.30	30.3
7	1911	8	21	1032.8	3.73	26.8
8	1925	10	18	978.3	5.81	17.2
9	1910	10	21	956.9	6.89	14.5
10	1920	11	1	938.5	7.96	12.6

Table 2 The years With the Ten Highest 90-day Rainfall Records, and their Recurrence Periods

4.3 Restoration design

Although there was little damage to the U-shaped retaining walls themselves, the gradient of the railway line was altered from 8 per thousand to 21 to 35 per thousand by the upheaval. This gradient posed difficulties in the operation of freight trains and had to be corrected. It was decided to lower the existing displaced U-shaped retaining walls as far as possible, in order to restore the station as quickly as possible and secure adequate platform space.

4.3.1 Investigations on Restoration Method

The alternatives for the restoration methods included reconstruction, lowering and rail-level alteration. Since an early reopening of the Musashino-line was desired in view of the importance of the line as a major freight and commuter route, the lowering plan was adopted, and it was decided that where adequate lowering could not be achieved, the track sections of the lower slabs should be demolished and removed for construction of new shallow U-shaped slabs. As a measure against future rises in the groundwater level, the structure was to be fixed down with ground anchors to raise its safety.

4.3.2 Lowering of Structural Frame

Reduction of the buoyancy through lowering of the water level was chosen as the method for lowering the structural frame, as this was judged to be the quickest and most efficient method. Deep wells were arranged to lower the water level, seven inside the U-shaped retaining walls (ℓ =7m, Ø=650mm, lifting pipes: 100A, high lift underwater pumps: 15kw) and nineteen surrounding the outside of the retaining walls (ℓ =20m, Ø=700mm, lifting pipes: 100A, high lift underwater pumps: 15kw). Pumping began gradually on 22nd October and continued until 30th November, after the completion of the new slabs. There had been no past cases, however, where complete lowering of the structural frame was achieved after an upheaval as large as 1.3m and there were concerns as to whether adequate lowering of the water level could be achieved through use of deep wells. (Figure 4).

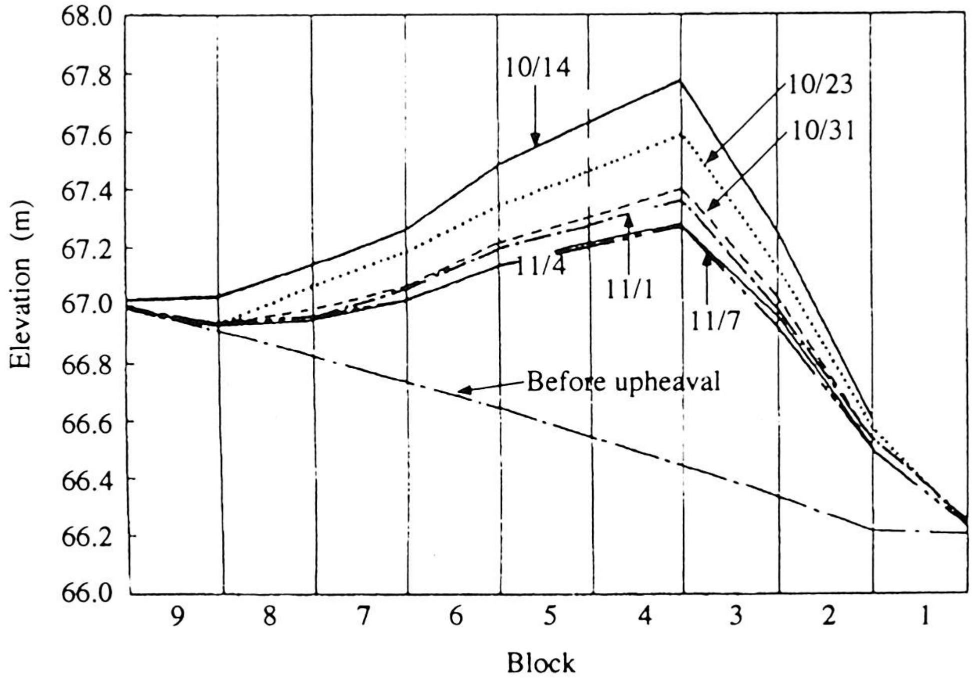


Fig. 4 Elevation of Lower Slabs (Slab Crests) : Measurement Results

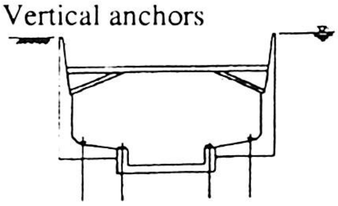
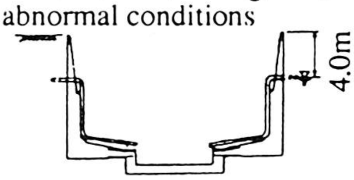
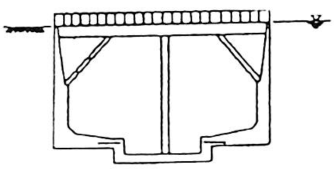
	Measure	Remarks	Evaluation
Alternative 1	Vertical anchors 	• Need for construction of reliable vertical anchor • Short execution period	○
Alternative 2	Water level lowering under abnormal conditions 	• Maintenance of drainage equipment required to retain its durability • Piping required within the station structure	△
Alternative 3	Additional load 	• Reinforcement of side walls required for placement of additional load on U-shaped retaining walls • Stable appearance • Long execution period	△

Table 3 Comparison of Measures Against Buoyancy

4.3.3 Design of Structural Frame

For the final form of the structure after the restoration work, the following three alternatives (Table 3) were considered as methods for ensuring stability, to prevent repeated upheaval of the restored structure in the event of future rises in the groundwater level up as far as the ground surface level.

1) Fixing to position with earth anchors
2) Prevention of buoyancy under abnormal conditions through drainage
3) Provision of additional load on structure

The use of earth anchors was selected out of the three methods as the most reliable and the fastest to construct.

The analysis model of the structural frame is shown in Figure 5. In the analysis, four rows of vertical anchors were installed to provide resistance to buoyancy and struts were installed on the upper parts of the structural frame. Through a comparison of the construction periods required, roller bearings were selected for use in the connections between the old slabs and sections where excavation was to be conducted below the slabs.

The analysis results are given in Table 4. It was confirmed that the stress generated in the lower slabs would not exceed the resistance stress. At the connections between the old and new slabs, taking into account the fact that complete roller bearings and pin bearings cannot be created in concrete structures, reinforcement was provided with steel rebars and anchors to prevent opening and dislocation. The details are given in Figure 6. To ensure early generation of strength, the concrete strength was set at $\sigma28=360kgf/cm^2$ in the mix design.

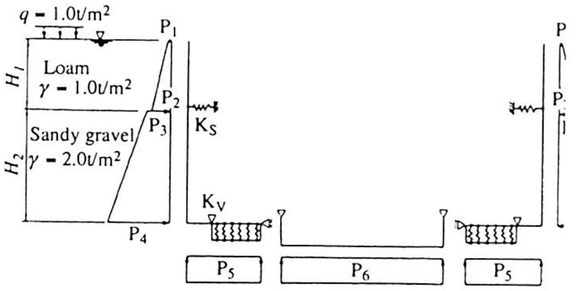


Fig. 5 Analysis Model

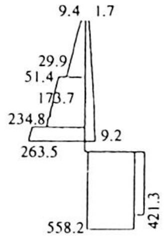
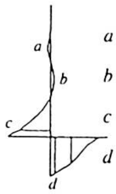
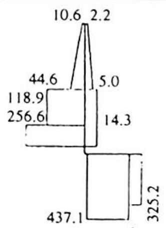
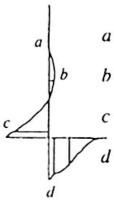
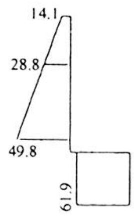
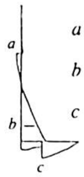
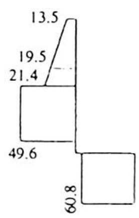
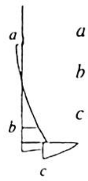
Block	2			3			4	5	6	7
	Resistance stress	Stress generated		Resistance stress	Stress generated					
Moment (t·m)			a = 6.01 b = -6.81 c = 72.85 d = 76.65			a = 6.48 b = -14.14 c = 49.51 d = 52.32	6.94 -12.90 49.66 52.00	7.91 -11.57 51.06 53.35	3.51 -14.34 78.86 82.69	3.82 -14.34 81.05 84.38
Shear force (t)			a = -11.46 b = 25.88 c = -35.46			a = -11.46 b = 25.88 c = -35.46	-15.87 23.66 -14.54	-15.87 22.75 -21.36	-11.22 29.63 -37.29	-11.63 29.29 -37.89

Table 4 Structural Stress

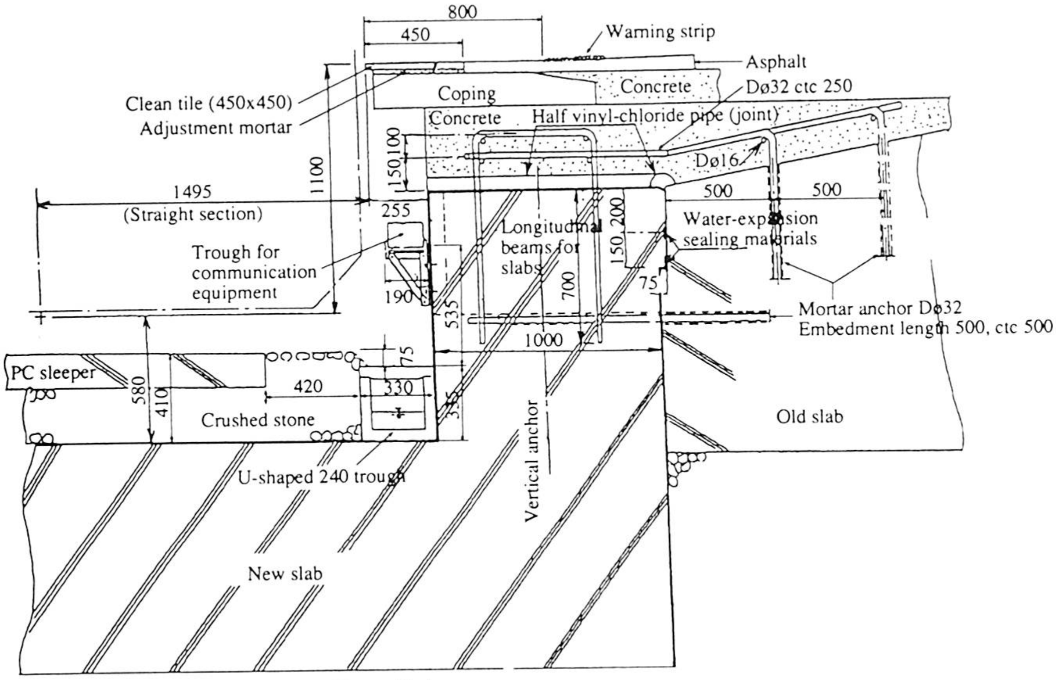


Fig. 6 Connection (Details)

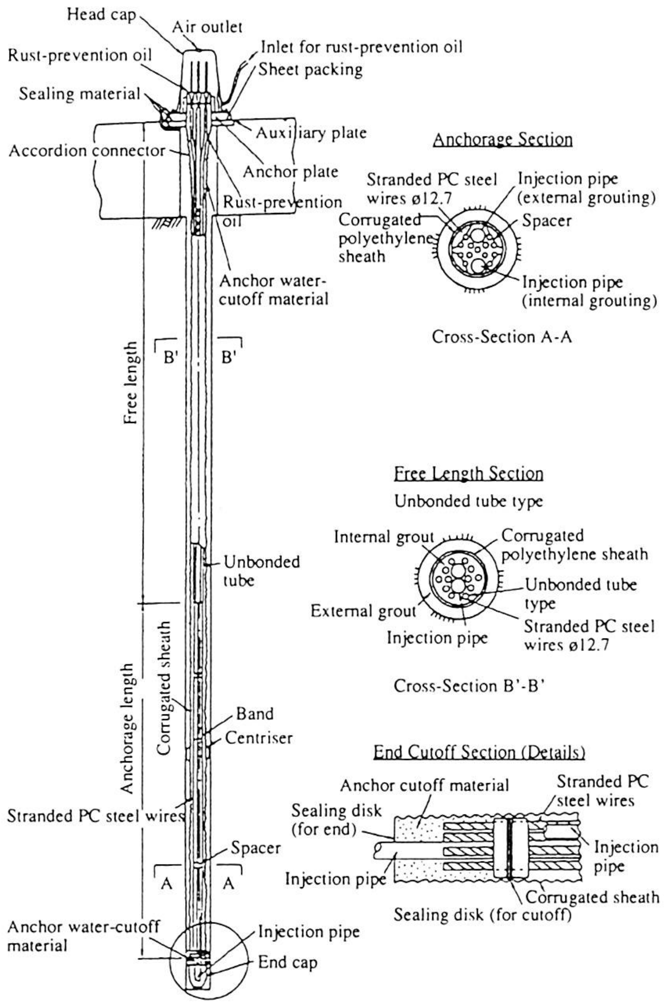


Fig. 8 Anchor Tendon

4.3.4 Design of Vertical Anchors

Permanent anchors aimed at prevention of uplift have been used in construction of buildings and, in recent years, have also been applied to railway facilities.

The VSL method was selected as the method for construction of the permanent anchors, which were made of twelve stranded PC steel wires, Ø12.7mm in diameter. The anchors were given an anchorage length of 10m in the gravel layer and a free length of 5m. The design anchoring force (per anchor) was 134tf for the side wall anchors (pitch: 4.0m) and 92tf for the anchors for new slabs in the middle (pitch: 2.0m). The details of the anchor tendons are given in Figure 7.

4.3.5 Design of Struts

After the completion of the restoration work, the U-shaped retaining walls would comprise a composite structure consisting of the original L-shaped sections and the newly constructed U-shaped sections. It was decided that struts should be provided in the L-shaped sections which were liable to destabilization through overturning at times of reses in the water level.

The struts were to be approximately 19m long. As a measure relating to the buckling stress intensity, angle braces with hinge metals at both ends were installed at points approximately 4.4m away from the two ends of the struts to reduce the buckling length. The struts were to be connected to each other with longitudinal beams to prevent sideways movement due to vibration and to ensure structural stability. The members used for the struts and angle braces were H 400x400x13x21 H-shaped SS 400 steels galvanized by hot-dipping.

4.4 Execution

The cross section after restoration and the restoration procedure are shown in Figures 8 and 9, respectively.

4.4.1 Lowering of Groundwater Level

Deep wells were used to lower the groundwater level. In this method, the permeability coefficient provides the basis for the design. Valid values for the permeability coefficients were not available at the initial stage of the design, but a value of $k=7.69 \times 10^{-3}$ (m/sec) was obtained by a deep well pumping test and used in the planning of the water level lowering.

The required reduction in the water level was calculated by the Theis' method and used for the determination of the number and pumping capacity required of the deep wells. Nineteen deep wells were created on the periphery of the U-shaped retaining walls and seven inside the retaining walls to lower the water level. During the water level lowering, inflow of earth was expected in the parts below the ends of the old slabs, which had been raised by the buoyancy of water. It was found that as buoyancy reduced, the old slabs would be supported from their ends by this earth, in which case they would be ruptured by the combined action of the weights of the old slabs themselves and the lateral earth pressure from the side walls. For this reason, temporary supports were constructed inside the U-shaped retaining walls to provide reinforcement (Photo 3).

The permeability coefficient calculated after the lowering of the water level from the reduction in the water level and the number of days required was $k=7.0\sim 8.0 \times 10^{-3}$ (m/sec), indicating that the value obtained through measurements in the deep well was close to the actual value.

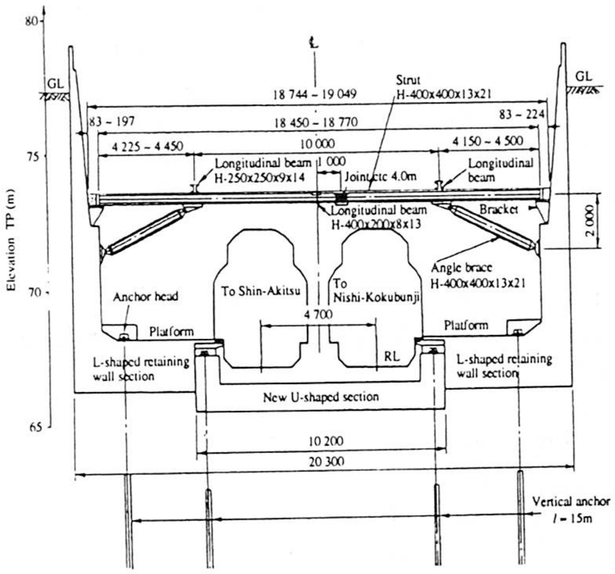


Fig. 8 Restoration Work: Typical Section

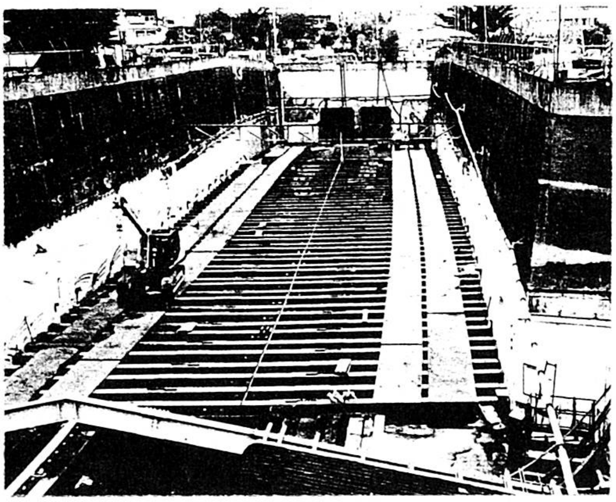
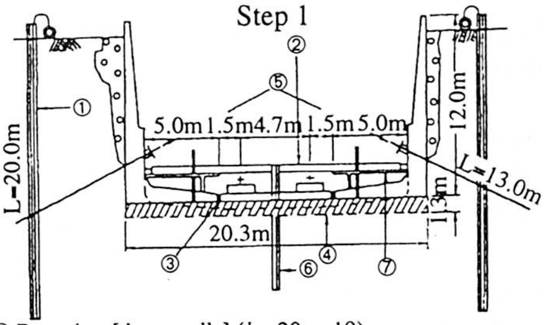
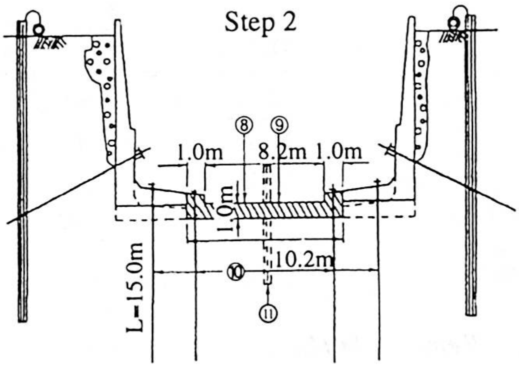


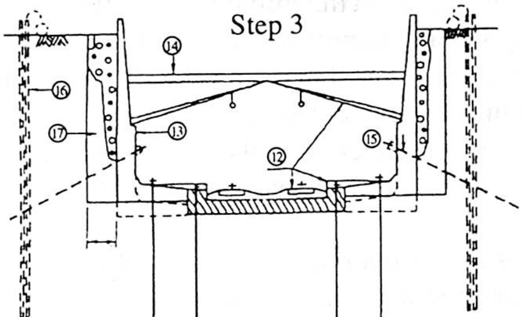
Photo3 Temporary supports



① Pumping [deep wells] (l = 20, x 19)
② Temporary supports (Blocks 2 to 8)
③ Coring of lower slabs for drainage (ø 50 mm)
④ Concrete grouting below slabs (coring ø 150 mm, x 24)
⑤ Removable horizontal anchors (ctc = 2.5 m, x 70)
⑥ Pumping at centre of slabs (l= 5 m, x 7)
⑦ Removal of supports and platforms



⑧ Demolition of reinforced concrete slabs (width: 10.2 m, depth: 0.8 to 1.0 m)
⑨ Construction of new reinforced concrete slabs
⑩ Vertical anchors (x 150)
⑪ Removal of pumping works at centre of slabs



⑫ Restoration of tracks, platforms and station building
⑬ Repair of retaining walls and tunnels
⑭ Construction of supports (Blocks 2 to 7)
⑮ Removal of horizontal anchors
⑯ Removal of pumping works
⑰ Grouting behind retaining walls

Fig. 9 Procedure for Restoration Work

4.4.2 Concrete Grouting Underneath Slabs

The progress of the water level lowering and lowering of the structural frame was monitored, and concrete was injected into the space underneath the slabs once a drop was observed in the rate of lowering. Utmost care was taken in the determination of the timing of the concrete grouting underneath the slabs, as this had a major effect on the subsequent full-scale restoration work. The grouting was injected through Tremie pipes installed on the slabs. The uniaxial compressive strength of the concrete was set at σ=80kg/cm^2, bearing in mind that a part of the concrete would need to be removed at a later stage. The water level was still above the level of the slabs at the time of the concrete grouting., which as a result took place under pressurized conditions.

To ensure satisfactory filling performance and resistance to segregation under water, non segregating under water concrete was used in the grouting, with the mix proportions determined through tests. The mix composition is given in Table 5.

Cement	W/C	S/a	Unit contents			
	(%)	(%)	W	C	S	G
Ordinary	83.3	45	250	300	720	902

Chemical admixtures (kg/m^3)			Slump flow (cm)
Non-segregation agent	Fluidiser	Air-entraining agent	
3.0	10	0.6	60 x 60

Table 5 Grouting under Slabs: Concrete Mix

4.4.3 Removable Horizontal Anchors and Demolition of Lower Slabs

Before demolishing the lower slabs, seventy removable horizontal anchors 13.0m in length (Ø= 12.7mm, nine-stranded PC steel wires) were first installed at 2.5m pitches to stabilize the L-shaped sections. The design anchoring force was 100t.

The lower slabs were demolished after the removal of the obstructions such as the temporary supports and platforms. As the old slabs below the platforms were to be retained as they were, they were isolated from the demolition section by concrete coring along the boundary. Concrete coring was selected as it was possible to use a large number of coring machines at one time, allowing a reduction in the time required. After isolation, the concrete in the demolition section was crushed with giant breakers and removed from the site.

4.4.4 Slab Concrete and Vertical Anchors

After creating a shallow sump on the Nishi-Kokubunji side of the station, slab concrete was placed and vertical anchors were installed according to the predetermined sequence. The mix for the slab concrete is given in Table 6.

Nominal strength	Cement	W/C	S/a	Slump	Unit contents(kg/m^3)				Chemical admixture
(kg/cm^2)		(%)	(%)	(cm)	W	C	S	G	(kg/m^3)
360	Ordinary	42.1	42.0	8±2.5	157	373	749	1,060	3.79

Table 6 Slabs: Concrete Mix

Besides the 150 anchors used in the main anchoring work, four were installed for measurement control and two for pullout tests, adding up to a total of 156 anchors. Thorough care was taken in the quality control of the anchors, with standard tests (pullout tests) carried out on three anchors and strength confirmation tests on five of the 150 anchors. Checks were also made on the rest of the anchors, the results of which were found to be satisfactory.
Stronghold anchors were used in blocks 1, 8, 9 and 10, while VSL anchors were used in blocks 2 to 7. Water-expansion sealing materials with high cutoff performance were used at the connections between the old and new structural frames to guard against a return of high groundwater (Photo 4 and Figure 10).

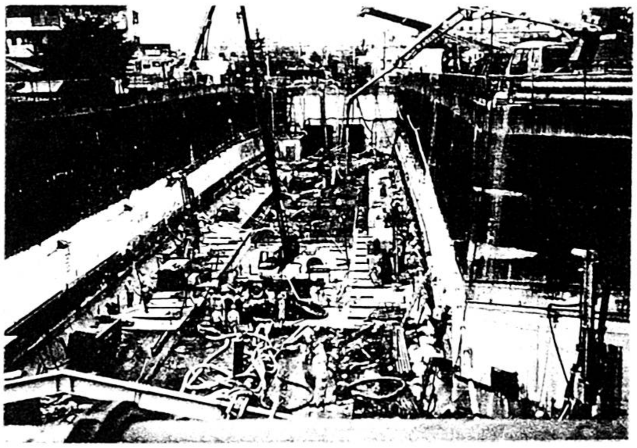


Photo 4 Restoration Work

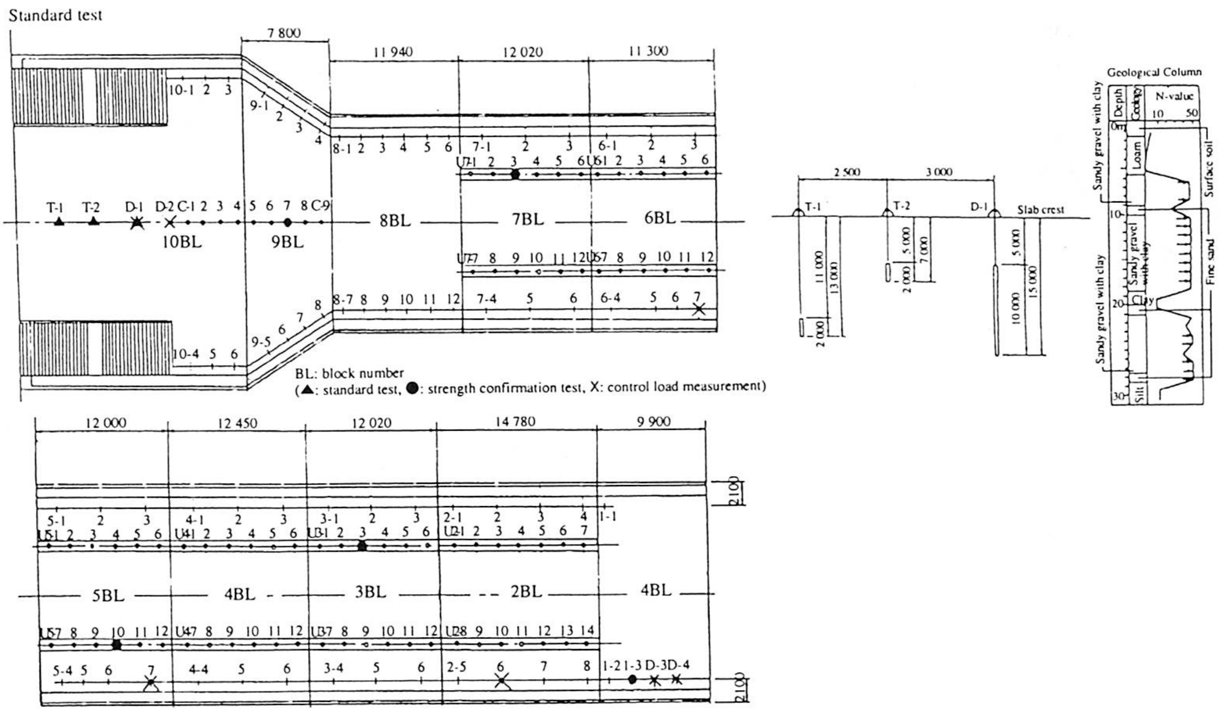


Fig. 10 Vertical Anchor Test Sites

4.4.5 Control Load Measurement on Vertical Anchors

To allow control of the tension in the vertical anchors, load measurement equipment has been installed at seven sites and measurement has been taken on these since the completion of the restoration work. Some of the measurement results are shown in Figure 11.

Forces larger than the design anchoring force were applied as the tension in the vertical anchors to make allowances for such factors as creep. The tension applied initially varies according to positions of the anchors. It was 144tf per anchor in the ordinary side wall section and 106tf per anchor in the central section.

The initial values given above are those at the time of the completion of the restoration work. A slight lowering can be observed in the tension both in the side wall section and the central section after the passage of one year.

The design anchoring force was set at a value that would provide sufficient resistance when the groundwater level reached the ground surface level. The groundwater level being lower, the tension is smaller than the design anchoring data at present.

4.4.6 Full Scale RestorAtion Work and Strut Construction

Full scale restoration of the platforms, rail tracks and station building was implemented along with the restoration of power and communication equipment. At the same time, liquid containing suspended water glass (LW) was injected into the ground behind the U-shaped retaining walls to fill the gaps left by the outflow of earth and for cutoff of water flowing towards the station structure. Struts were also installed at 4.0m pitches as reinforcement for the retaining walls in preparation for future abnormal rises in the groundwater level, thus completing the restoration work (Photo 5).

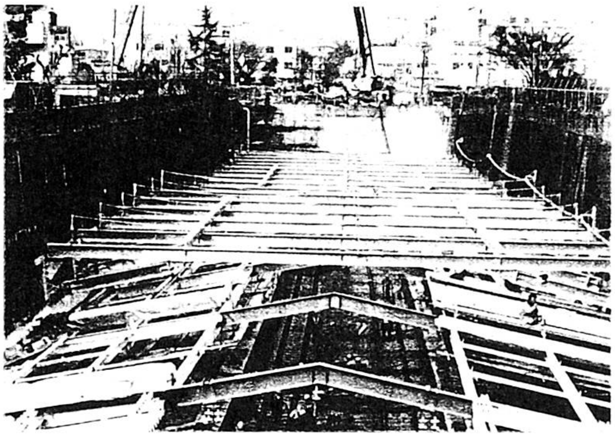


Photo 5 Installation of Struts

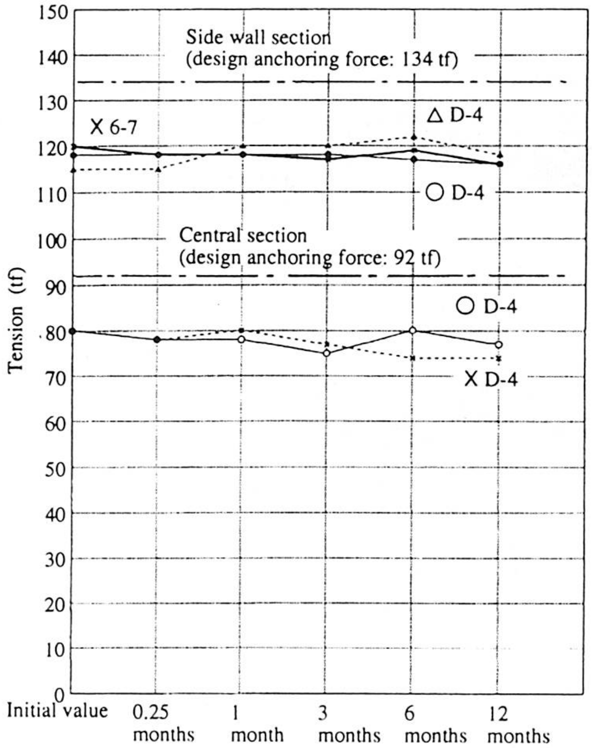


Fig. 14 Vertical Anchor Load: Measurement Results

5. Conclusion

The damage due to ground upheaval caused by Typhoon No. 21 led one to recognize anew the dangers of the natural force of buoyancy. The total damage on the Musashino Line, a major artery for commuter and freight traffic, is estimated at ¥8,600 million. The line was reopened after two months of round-the-clock restoration work, with the co-operation of the agencies concerned and the local residents.

Finally, the authors would like to take this opportunity to express their gratitude to all those who helped in one way or another. We hope this report will help to some degree in preventing a recurrence of this disaster.