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SESSION 4

Security and environmental aspects

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Risk analysis as a tool to determine design load due to explosion in a tunnel.

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Elisabeth Ahlenius, born 1953, received her Master of Science degree from Technical University of Gdansk in 1977 and from Royal Institute of Technology in Stockholm in 1983. During the last two years worked with the risk analysis of the construction and operation phases of tunnel systems for road and rail traffic and pipe lines.

Summary

This paper presents a method for estimation of the design load due to explosion in a tunnel. The method is based on risk analysis together with cost-benefit comparison and was applied in particular for the load bearing structure in Ringen in Stockholm. Risk is defined as a measure of the probability and the severity of an explosion and is expressed in terms of cost. The analysis takes into account risk connected with the accidents or adverse events during the normal operation stage of this tunnel. The stages of the analysis were: to determine causes of an explosion, estimate probabilities, consequences and calculate the risk. Two alternatives of the tunnel were studied: one structure with normal and one with increased resistance. For each alternative, the yearly cost of risk was determined and the savings in risk were compared with the cost for increase of the resistance.

1. Scope

Ringen is the name of a system with 3 links for road traffic planned to be built around the city of Stockholm. The whole system will consist of the roads and the tunnel sections. The tunnels will be constructed as rock tunnels with concrete sections with the length of about 300 m at each end. One link will contain a submerged section. The total length of tunnels in Ringen including the ramp tunnels will be about 30 km. The traffic flow is estimated to 50000 vehicles per day in average.

When designing a road or railway tunnel, the question about load due to explosion has to be dealt with. Explosion is an accidental situation and one of the design situations for the tunnel structure.

According to the special regulations established for Ringen [6], the value of this load is treated by assuming a static pressure of 70 kPa.

According to the Swedish Regulations "Tunnel 95" [1] the value of the load caused by explosion is given as a static load with a value which depends on the distance between the place of the explosion and the end of the tunnel. The value of the pressure above atmospheric pressure (which is 100 kPa) is assumed to vary between 150 kPa at the end of the tunnel and the max. value of 500 kPa at the distance of 2 km from the end of the tunnel.

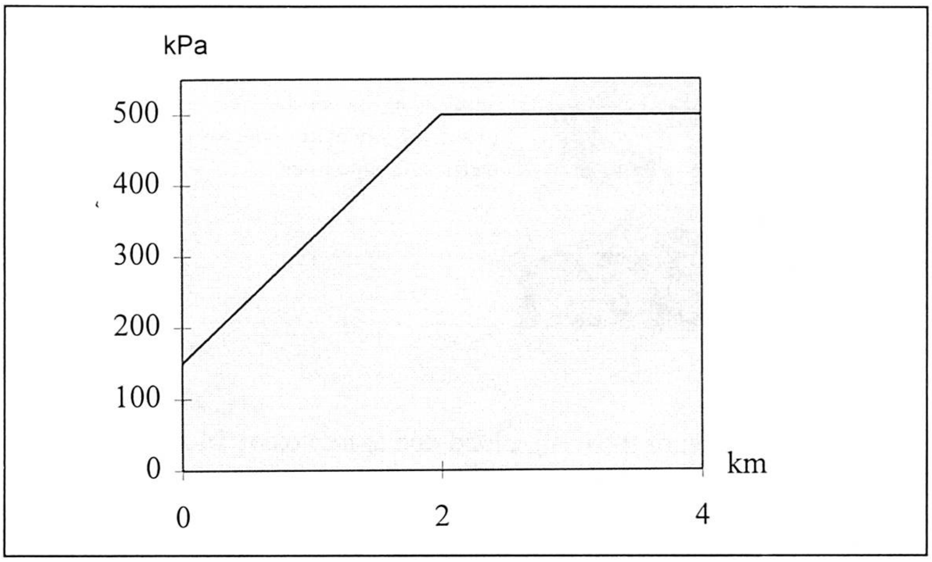

Fig. 1 The pressure above atmospheric pressure (100 kPa), [*1*]

As an alternative, "Tunnel 95" gives also the opportunity to use risk analysis to identify other kinds of the accidental loads or other values of these loads.

The Local Authority decided to use this option.

2. Risk analysis

Risk analysis is an estimation and a valuation of risks. In this analysis, risk was expressed in terms of cost, as:

$R = P \times C$

where

$P =$ annual probability of the damage to the construction,

$C =$ cost of the damage to the construction.

The object for this work was the estimation of risks connected with the normal use of the tunnels. The whole system of tunnels was treated as one unit with given length and traffic flow. The given traffic flow corresponds to a situation expected in the year 2005.

The risk analysis was performed in the following stages:

- identification of causes,

- estimation of probabilities of causes,

- quantification of consequences,

- calculation of risks.

Fault trees were used to calculate the probability of the explosive events.

The identification of causes, the estimation of probabilities of causes and probabilities of damage required experience in a large range of technical disciplines as: explosives, transport, concrete structures and rock engineering, hence the analysis involved experts in all these domains.

3. Causes

The scope for the analysis was that an explosion can occur during operation of the road tunnel, which means that an explosion is caused by dangerous goods transported in the tunnel.

Dangerous goods is a general name for a wide range of substances. According to the classification used in ADR (European Agreement Concerning the International Carriage of Dangerous Goods by Road) [2], ten classes of goods have been defined.

Some of the classes contain substances which can explode with different magnitudes. Only certain kinds of explosives (classified as class 1) and oxidising substances (which are class 5) can explode with such a magnitude that it will cause damage to a concrete or rock structure.

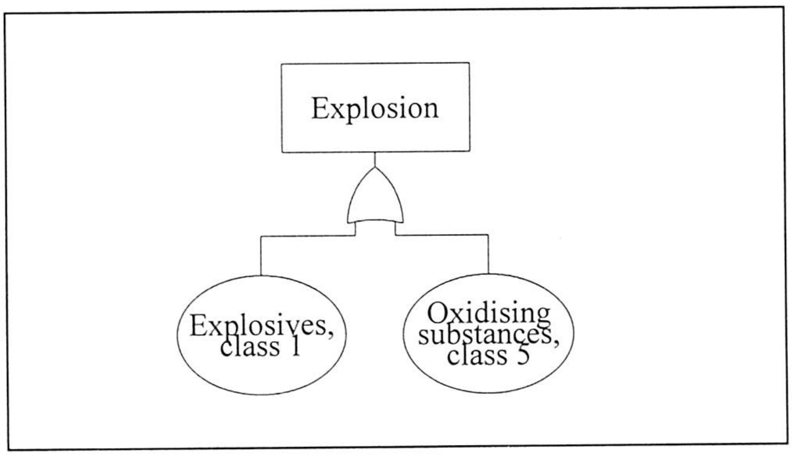

Fig. 2 Causes for an explosion

Other kinds of dangerous goods, such as bottled gases, can explode and cause danger for people but the pressure will not cause damage to the load bearing structure of the tunnel.

4. Probabilities

4.1 General

Assessment of the risks is based on frequency estimates for the occurrence of different categories and load sizes of dangerous goods.

The probability of the damage was based on the following:

1. The estimation of the total transport and the transport of goods.

2. The assumption about the probability of occurrence of goods which can explode in transport, based on the available reports concerning the road traffic in general [4], [5] and especially on the studies performed for Ringen. The assumption about the distribution among different kinds of dangerous goods was that dangerous goods participate in transport in the same proportion as they are produced. Both explosives and oxidising substances were assumed to be 1% of dangerous goods.

3. The assumption about the probability of occurrence of different load sizes.

Transported goods were divided into 4 load groups. The following assumption about the probability distribution of load sizes was used:

Load group	Load size	Probability of occurrence in transport
Load group 1	< 30 kg	20%
Load group 2	30 - 300 kg	20%
Load group 3	300 - 1000 kg	30%
Load group 4	1000 - 15000 kg	30%

Table 1. Probability of occurrence of different load sizes in transport.

4. The estimation of the probability of initiation, based mainly on the Health and Safety Commission´s report [4] about hazard aspects of transport of dangerous substances, which contains statistics about road accidents and events reported in Great Britain over 40 years.

The probability of initiation of goods which can explode was calculated separately for explosives (class 1) and for oxidising substances (class 5) and for all load groups (group 1- 4) as shown in Fig. 6.

4.2 Detonation of explosives

All explosives are thermodynamically unstable. They remain inert until they receive sufficient energy to initiate. In principle, detonation of explosives can be caused by impact, fire or unsafe explosives which means that badly packed, manufactured or out of specification material may explode during normal transport.

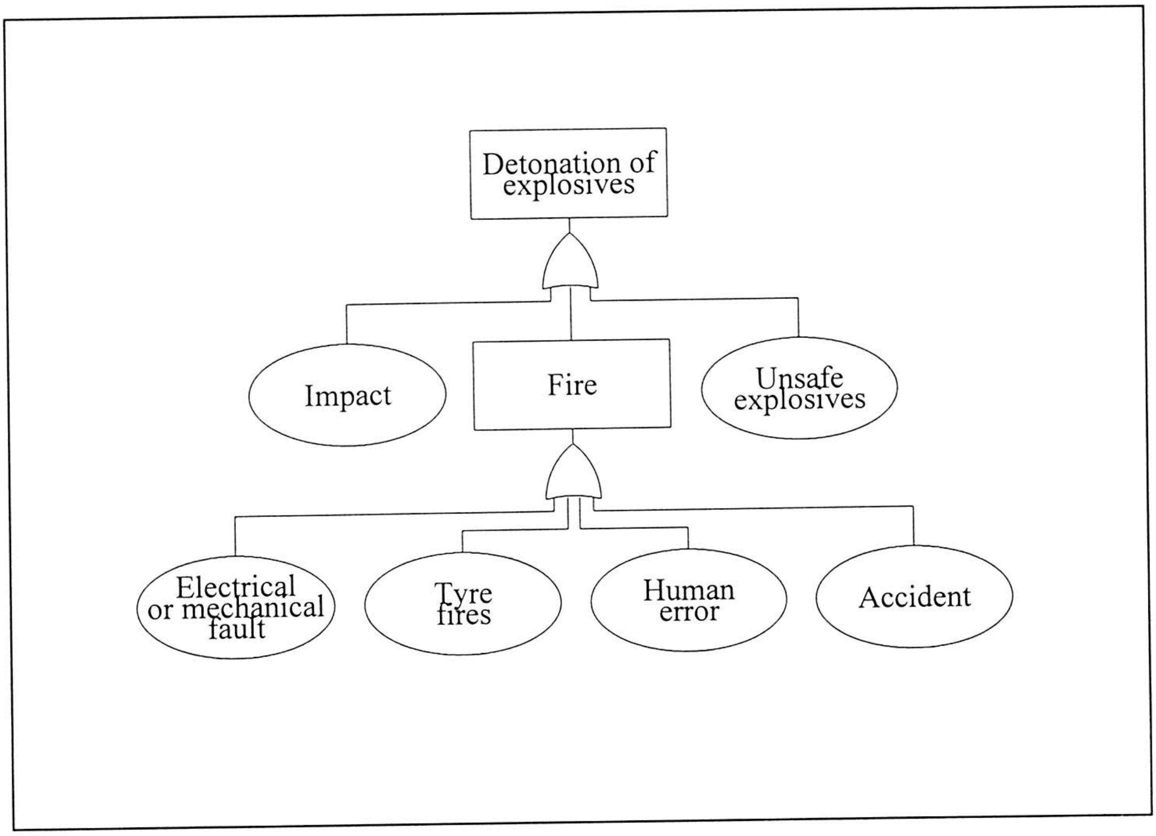

Fig. 3 Detonation of explosives

Estimation of probability of detonation was done separately for each group of explosives. The assumption was that only small quantities of explosives (load group 1) can be transported without any restrictions and larger quantities (load group 2, 3 and 4) have to be transported with escort, which reduces probability of explosion caused by road accidents and fire. The probability of road accidents involving one vehicle only was assumed to be reduced by 50% and collisions totally eliminated if the transport will be escorted. Probability of explosion caused by fire in a vehicle loaded with explosives was reduced by 90% for transport with escort.

The explosion caused by impact was assumed to occur in 2% of collision type of road accidents on basis of statistics about reported accidents and results from laboratory tests.

Fire can be caused by:

- ignition by electrical or mechanical fault in the cabin or engine
- tyre fires,

- human error,
- collisions.

Tyre fires are the dominant cause and correspond to about 80% of cases that a fire arises.

The frequency of unintentional initiation of unsafe explosives was estimated as one to one thousand million of vehicle km.

4.3 Explosion of oxidising substances

Oxidising substances are not dangerous by themselves. They can explode if they escape and come into contact with flammable substances, and there is an ignition source. The pressure will be of the same magnitude as for explosives.

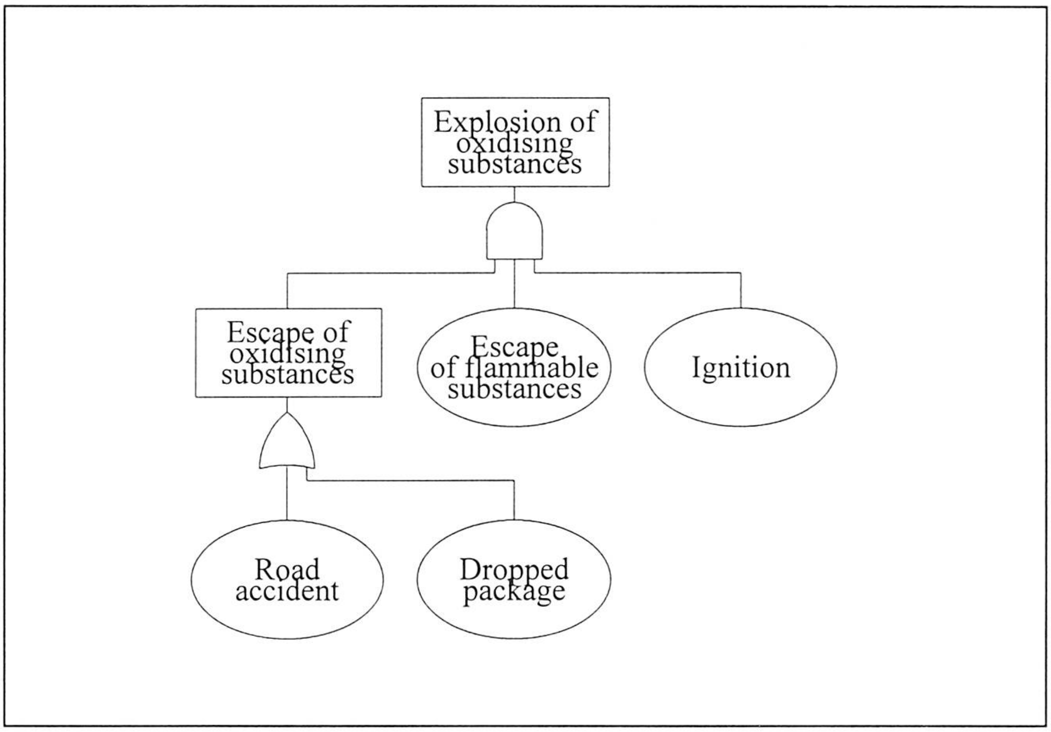

Fig. 4 Explosion of oxidising substances

The limitation is that escape of both oxidising substances and flammable substances has to occur at the same place and the same time. This analysis was based on the assumption that both these events have to occur within the distance of 1 meter and in less than 5 minutes.

Escape of oxidising substances can be caused by a road accident with oxidising substances involved or because a package is dropped from a vehicle during normal transport. The assumption was that 90 road accidents and 100 incidences of traffic such as a blow-out of tyres or a dropped package can be expected in Ringen annually.

The escape of oxidising substances was assumed to occur in 70% of road accidents and events of dropped packages. The escape of flammable substances happens in 10% of road accidents.

The probability of ignition in a tunnel was assumed to be 25%.

4.4 Summary

The result of the calculations is presented in the following table.

Goods	**Probability of explosive events**
Explosives, class 1	3 events per 100 000 years
Oxidising substances, class 5	5 events per 100 000 years

Table 2. Probability of explosive events

5. Consequences

An explosion in a tunnel would cause a pressure with a magnitude varying with the distance from the place of the explosion.

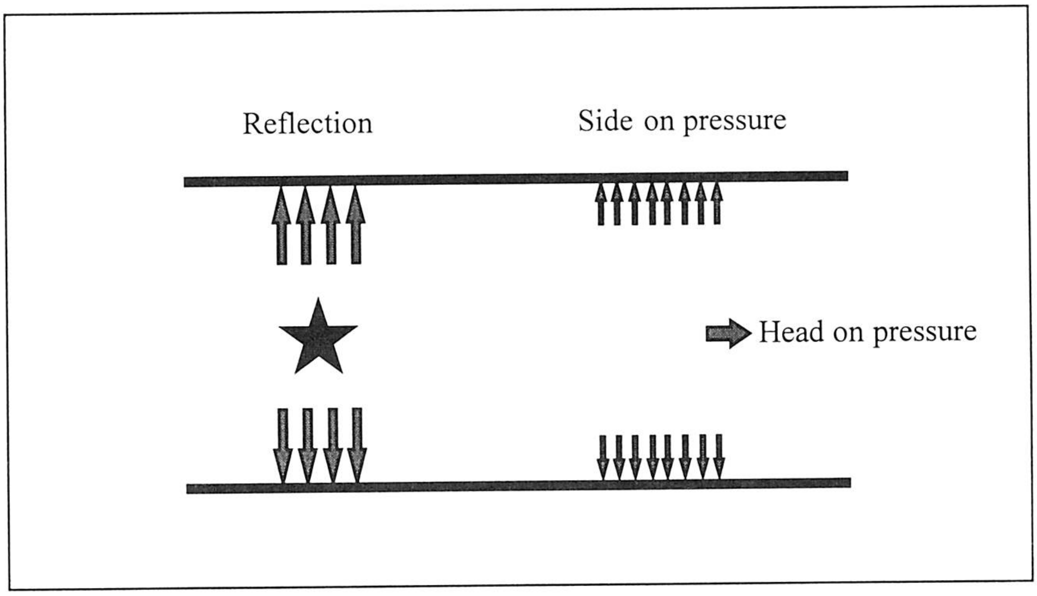

Fig. 5 The pressure after the explosion in a tunnel

Pressure near the place of the explosion is named reflection.

Pressure at a greater distance from the place of the explosion is named:

- head on pressure which acts in the direction of the tunnel axis,

- side on pressure which affects all sides of the tunnel.

The values of the pressure above atmospheric pressure after the explosion of an explosive charge of 30 and 300 kg TNT are presented in the following table:

Explosive charge [kg]		Distance [m]	Pressure [kPa]
30	Reflection	2	21 000
	Reflection	4	3 000
	Head on	30	100
	Side on	30	70
300	Reflection	2	120 000
	Reflection	4	34 000
	Head on	30	600
	Side on	30	300

Table 3 The pressure after the explosion in a tunnel

Assumptions for the calculation were: cross-section of the tunnel is 75 m^2, explosion occurs 1 m above the floorlevel and the explosive substance is TNT.

The conclusions were that:

1. Reflection depends on the distance between the explosive charge and the wall.
   Reflection caused by the explosion of 300 kg of TNT at a distance of 4 m from the wall is of the same magnitude as that caused by the explosion of 30 kg of TNT at a distance of 2 m from the wall.

2. The pressure decreases rapidly with the increased distance from the place of the explosion.
   Reflection affects the surface which is limited by the angle of 40 degrees to the line perpendicular to the wall.

Consequences expressed in terms of cost contain costs for the repairs of the damage to the structure and the equipment and costs for the interruption of traffic.

The consequence of an explosion have been studied for two different alternatives of the tunnel structures.

Alternative 1 - "normal resistance" - the concrete and rock structure was designed for explosion of 30 kg of TNT.

Alternative 2 - increased resistance - the concrete and rock structure was designed for an explosion of 300 kg of TNT. The increased resistance of the concrete structure was obtained by increased amount of reinforcement. Increased thickness of roofs and walls would reduce the flexibility of the structure and make the response to dynamic loads worse. Increased resistance of the rock structure was obtained by increased amount of the rock bolts. These measures have to be applied to all the surfaces of roofs and walls in the whole length of the tunnel.

Alternative 2 gives two effects:

1) The range of damage and cost of repairs will be decreased.

2) The construction cost will be increased.

Quantification of the consequences to the tunnel structures was based on the experience of the response of similar concrete or rock structures to the pressure caused by an explosion.

Three different ranges of consequences was defined and expressed in terms of cost.

Name	Range of damage	Interruption of traffic
Consequence 1	None or slightly damage to one tunnel tube.	24 hours
Consequence 2	Medium damage to one tunnel tube.	1 - 20 weeks
Consequence 3	Large damage to both tunnel tubes.	1 year

Table 4. Quantification of the consequences

The calculation of the consequences for different alternatives of the structure was based on the valuation of the costs for the repairs of the damage to the structure and costs for the interruption of traffic. The costs connected with repairing equipment was assumed to be the same for both alternatives of the structure.

Alternative 1, with normal resistance:

- loads of group 1 (0-30 kg) will cause the consequence 1,
- loads of group 2 and 3 (30-300 kg and 300-1000 kg) will cause the consequence 2,
- loads of group 4 (1000-15 000 kg) will cause the consequence 3.

Alternative 2, with increased resistance:

- loads of group 1 and 2 will cause the consequence 1,
- loads of group 3 will cause the consequence 2,
- loads of group 4 will cause the consequence 3.

6. Risk

The yearly risk could then be estimated as a yearly probability of an explosion in a tunnel times the cost of consequence for this explosion for all groups of loads and both classes of goods. The same calculation was done for both design alternatives of the tunnel structure.

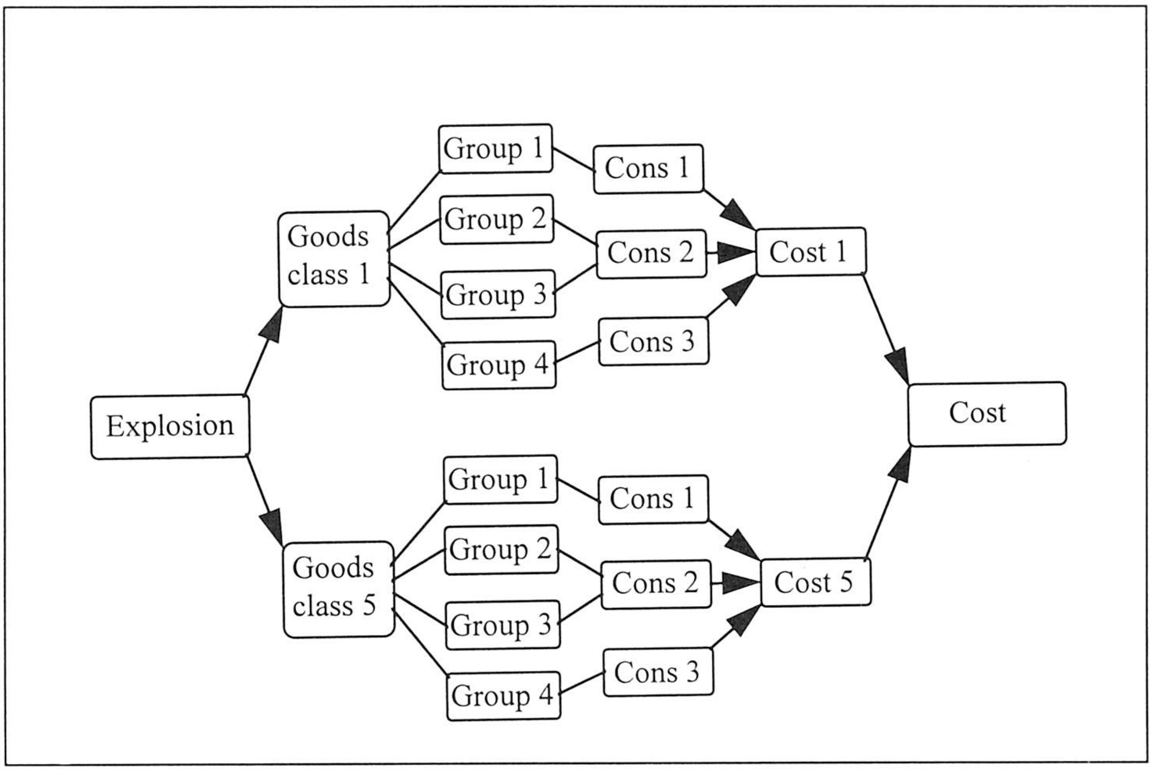

Fig. 6 Calculation of risk for Alternative 1.

Finally, the yearly savings in risk were calculated as the difference between the yearly cost of risk for Alternatives 1 and 2 and compared with yearly costs for increase of the resistance of the tunnel structure. The pay-back time for the tunnel was assumed to be 30 years.

The result of this comparison was that yearly cost for increase of the resistance was 10 thousand times higher than the savings in risk.

The conclusion was that it is not economically worthwhile to increase the resistance of the tunnel structure above the value determined for the explosion of 30 kg of TNT.

7. Design load

An explosion causes an impulse which can be expressed as the force times the duration of the impulse or, which is equivalent, as the mass times velocity. Calculations based on the static load often give the incorrect picture of the forces caused by an explosion. To design the tunnels for the impulse load and not for the static load is more equivalent to the structure´s real behaviour. The mass has much a bigger influence on the value of the bending moment and internal forces when the calculations take into account the dynamic response of the structure.

A simplified method has been developed to calculate the bending moments and the deformations of the concrete tunnels exposed to the impulse load caused by an explosion, and determined by the risk analysis.

The basis for the calculation was a equalisation between, on the one hand, the energy of the explosion, and on the other, the internal work in the structure and the work necessary for lifting the mass of the earth which loads the structure.

The assumptions for the calculation were:

- the duration of the impulse is short in comparison with the natural frequency of the structure.
- pressure is expressed as the density of the impulse or force times the time of the duration of the impulse per m^2, [kPa][s],
- plastic deformations are allowed.

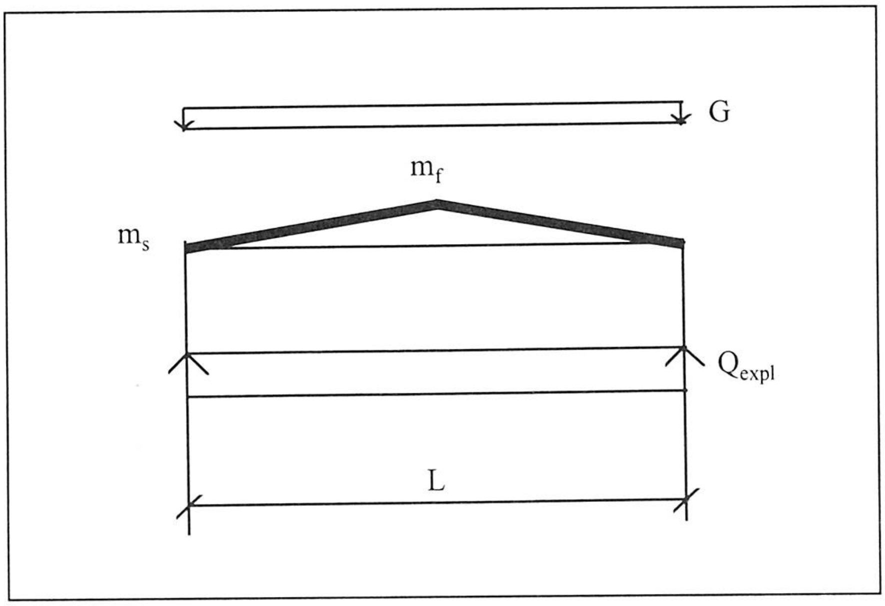

Fig. 7 The plastic deformation caused by an explosion

The deformation was limited to 1/100 of the span of the structure. A distinction was made between the dynamic response of a roof and a wall, depending on the benefit of the mass of the earth. The dissipation of the energy caused by friction in the earth was estimated.

The design load for the tunnel structure was defined as the impulse load with the same duration, density of impact and pressure, with the triangular distribution as the load caused by an explosion of 50 kg of TNT. The increased value of the explosive charge was chosen with consideration to uncertainties about the distance between the place of the explosion and the tunnel structure.

Finally, the Swedish National Road Administration has accepted the suggested method for the calculation.

8. References

[1] TUNNEL 95

[2] ADR, ADR-S, Inrikes transport av farligt gods på väg och i terräng, Statens Räddningsverks författningssamling SRVFS 1994:5.

[3] Brockhoff H, Extraction and classification of accident data on road transport of dangerous goods.

[4] Health & Safety Commission, Major hazard aspects of transport of dangerous substances

[5] Nilsson G, Vägtransporter med farligt gods - Farligt gods i vägtrafikolyckor, VTI rapport nr 387:3, 1994

[6] Ringengemensam Byggnadsteknisk Beskrivning (RiBB)

Security aspects of the new railway link with Brussels Airport

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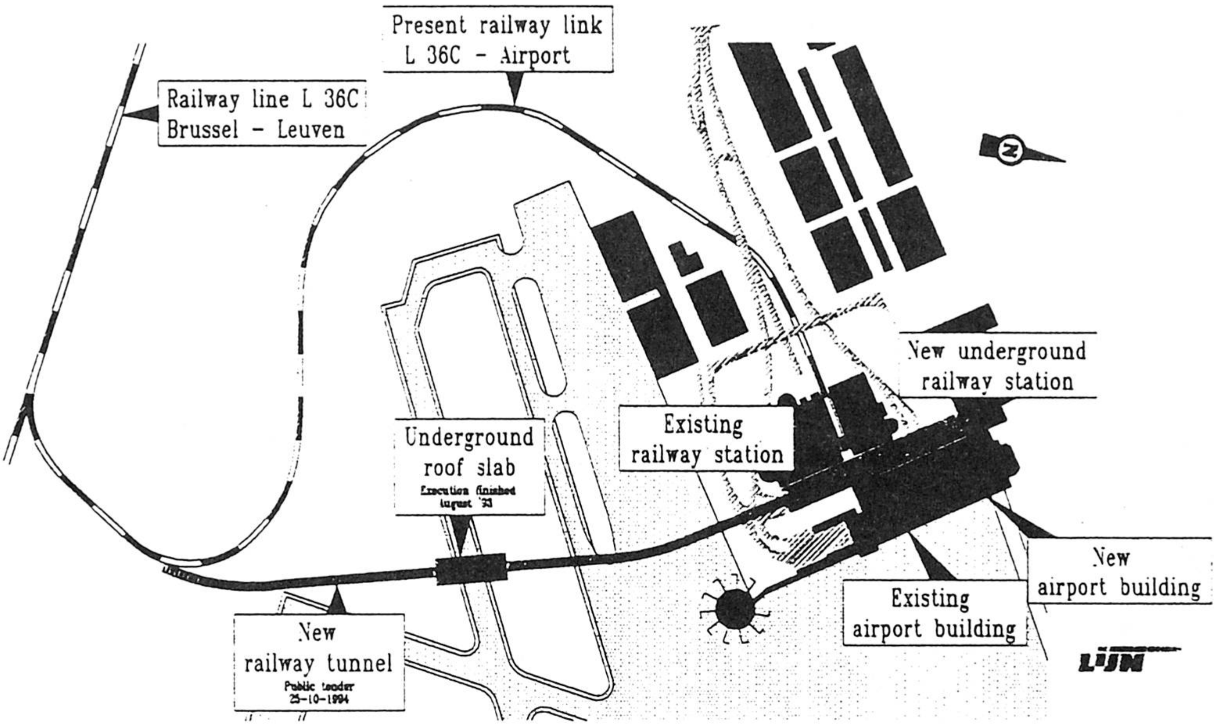
Etienne Hemerijckx is civil engineer (Leuven University 1973). After a successfull career of 17 years, he was in 1990 promoted to Head of Department of the Central Study Office of De Lijn, where he is charged with the follow-up of the general management of studies and execution of works for the advancement of the public transport in Flanders.

A recent railway project, managed by the "Centrale Studiedienst of De Lijn" advances the railway link of the NMBS (National Railway Company of Belgium) between the important Brussels - Leuven line and Brussels Airport. The project comprises the construction of a new railway station and a 1 km long tunnel under the airport installation, coupled with the architectural finishing, the electro-mechanical equipment, the track installation and the connection with the existing railway line. The execution of important works at and under the airport implies the observance of some security measures, providing a minimal hindrance of the airport services.

1. Project management Centrale Studiedienst De Lijn

The Centrale Studiedienst De Lijn provides for the project management of the construction of a railway station, the tunnelling works and the track installation.
Master builder is the Belgian Railway Company.
The Centrale Studiedienst De Lijn has a large experience with study and supervision on the main structure, the architectural finishing and the electro-mechanical equipping of underground infrastructures, especially for the Antwerp premetro.
In the past, many impressive underground techniques were studied and executed, such as a.o. the shield method, the pipe-jacking method, the freezing method, injection.



2. Execution methods

2.1. Railway station

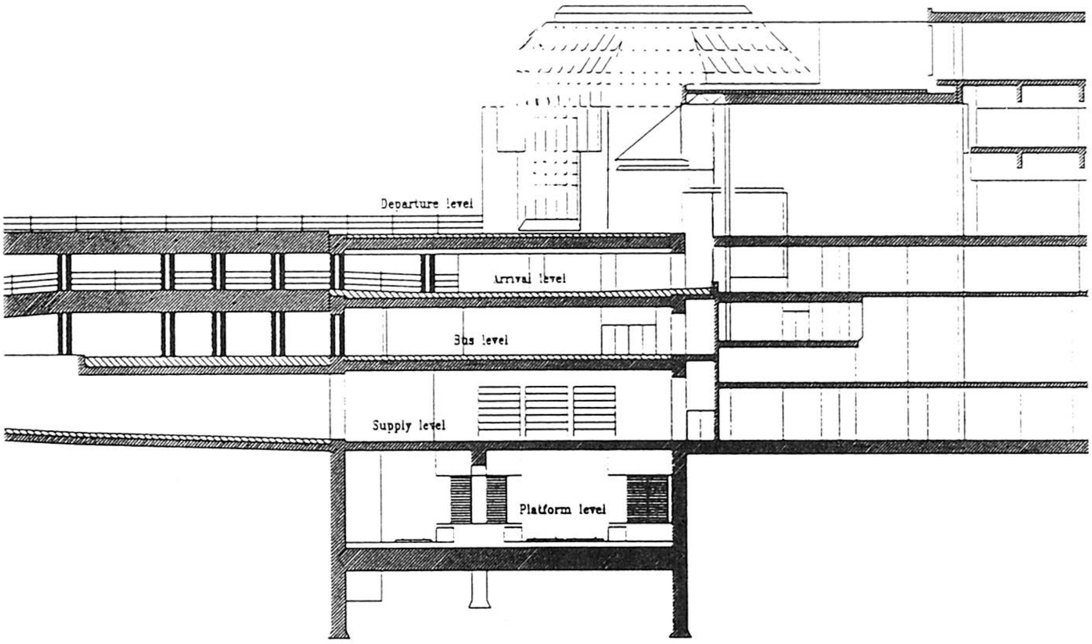
2.1.1. General concept

The station has 3 underground levels and has 2 flights of stairs, the one at the entrance and exit of the new airport building and the other at the entrance and exit of the old airport building.

- <u>The lowest level</u> has 3 tracks and platforms with a length of about 385 m and a width of about 4 m.

- <u>The level on top of the lowest one</u> (called supply level) stands for the connection between the platforms and the airport building and creates room for technical equipment that is necessary for the railway station operations.

- <u>The 3rd underground level</u>, that is situated straight under the street level (called bus level) provides for the train passengers flow from the platform to the airport and to the railway company ticket offices.
Moreover, it is possible to move in with technical and administration services, which are involved with the station services such as luggage treatment, telecommunication, sanitary, dressing room etc.

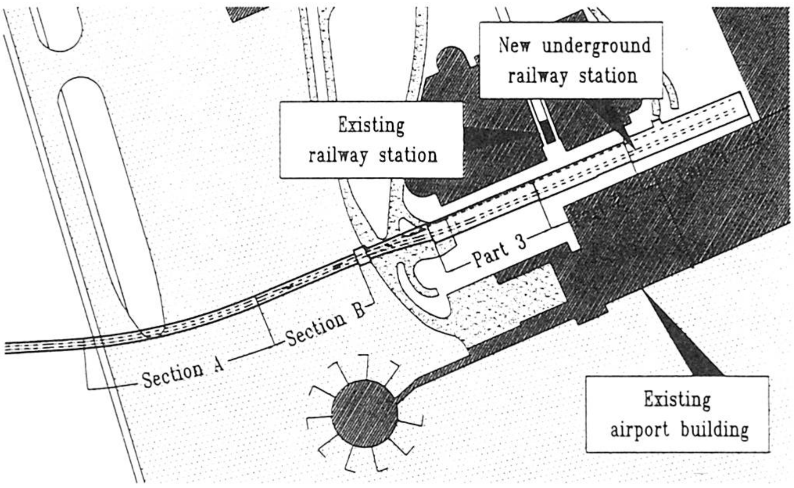


One flight of stairs is situated in the immediate neighbourhood of the new airport building, where all the passengers will have to leave the airport and where the international passengers will have to pass the customs.
The other flight of stairs is situated at the present railway station which is already linked up at the moment with the present airport building.
The new station maintains this connection.

After the new airport building is operative, the older one will be entirely adapted and renovated. Both flights of stairs are connected with each other as well through the platform level, the bus level b.m.o. a corridor neighbouring the above mentioned technical and administration rooms, as via the arrival hall of the airport building.

2.1.2. Execution methods

The execution of the railway station can be splitted up into 3 parts.

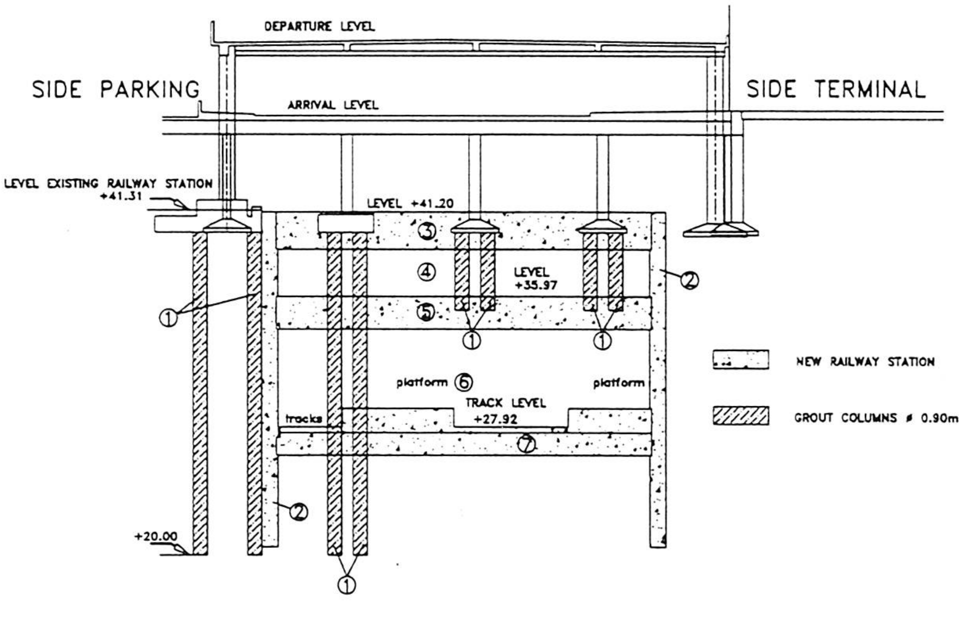


Part 1

This 145 m long section comprises the flight of stairs at the new airport building with which it was simultaneously executed. Because of its implantation, it was possible to construct it according to a classical method. The exterior walls were executed as slurry walls and the roof slab was concreted at the bare ground. The further excavation under the roof slab was executed from top to bottom, respectively concreting the intermediate and bottom slab.

Part 2

This 110 m long section is situated at the existing railway station, heading the old airport building. It comprises the construction of a flight of stairs situated at the entrance and exit of the old airport building.
The difficult situation, between the parking building and the airport complex on the one hand and the presence of the esplanade (departure level), which has restrictions in height during the execution, asked for a lot of consideration when determining the execution method.
A first thought was to demolish the entire part of the existing station that should be crossed. Because of the presence of a lot of public utility pipes, which were a must for the airport operations, this solution was too expensive and technically not favourable.
Consequently, the study was adapted in such a way that the pipes, the roof and the columns of the existing railway station could be maintained. The columns and the roof are caught up in the new intermediate slab, but are to be supported in a provisional phase with help of V.H.P.-grout piles.

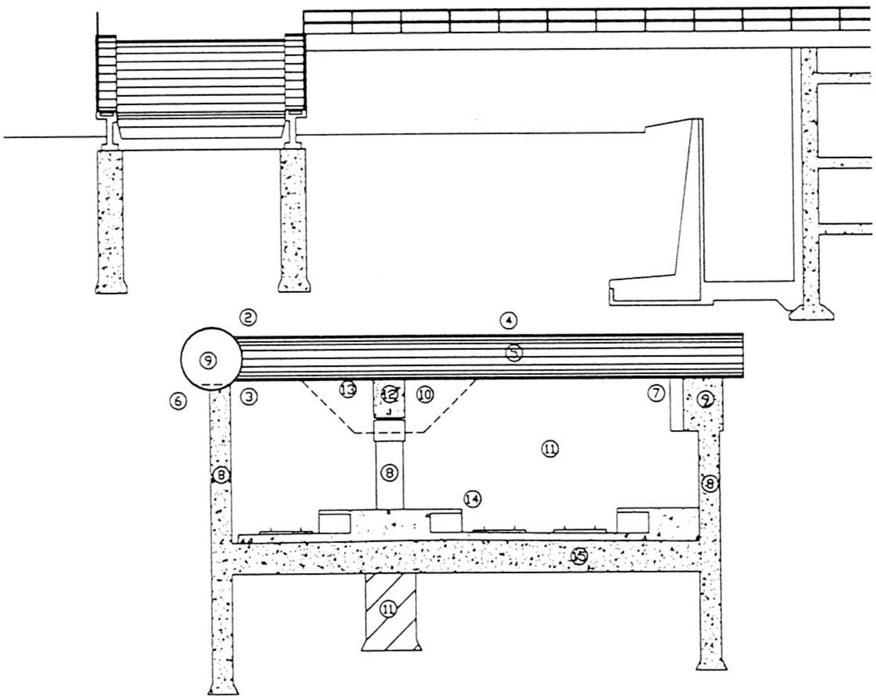


1. Execution of V.H.P.-columns after the boring of pits throughout the existing foundation soles.
2. Execution of the walls in sheeted trench.
3. Execution up to bottom side intermediate slab bus level and concreting of the slab and the anchoring of the foundation soles.
4. Excavation under the slab at bus level and the demolition of the grout piles.
5. Reinforcing and concreting of the intermediate slab at the supply level.
6. Excavation of the bottom slab.
7. Execution of bottom slab and platforms.

Part 3

This 130 m long part is constructed simultaneously with the tunnel under the airport and is used as track level. In order to minimise the disturbance at the airport, the following execution method was chosen:

From a work site situated at the border line land-airside, a longitudinal tube with a large diameter (Ø 3 m) is jacked in the direction of the second part of the station. This tube is used as working gallery from where transversal tubes with a smaller diameter (Ø 2,1 m) are jacked and forming the roof slab once the tubes are reinforced and concreted.
Out of the working gallery (tube Ø 3 m), also the side wall is executed in sheeted trenches.
The other side wall and the intermediate column is excavated in sheeted trenches out from the galleries under the roof slab.
After the roof, the walls and the mid columns are realised, the bottom slab and the platforms can be concreted.



Working method 1 longitudinal and transverse tubes

1. *Realisation of the construction shaft at the border line landside-airside*
2. *Jacking of the longitudinal metal tube Ø 2960*
3. *Realisation of the working slab in the tube*
4. *Jacking of transverse concrete tubes Ø 1900-2100*
5. *Reinforcing and concreting of transverse tubes*
6. *Demolition of a part of the longitudinal tube*
7. *Construction of a gallery under the transversal tubes*
8. *Construction and concreting of the sheeted trenches*
9. *Reinforcing and concreting of the longitudinal tube and construction of sheeted trenches*
10. *Restricted excavation under the transversal tubes*
11. *Construction of sheeted pits and columns*
12. *Construction of longitudinal beam under the transversal tubes*
13. *Installation of flat jacks*
14. *Excavation up to bottom side of the bottom slab*
15. *Execution of the working floor and bottom slab in parts*
16. *Final joint between the tubes*

2.2. Railway tunnel

2.2.1. General problematic of the works under the airport

The execution of important works at and under the airport implies undoubtedly the taking of some security measures.

The study has to take into consideration in the first place a minimal hindrance of the airport services. On the other hand, construction costs may not be too high because of this particular reason. A solution which is as technical as economical justified, but also acceptable for the airport services should be found.

For these reasons a part of the roof slab of the tunnel at the height of the runway 07.R was preliminary executed. Initially was chosen for the entire underground pipe jacking method. Further studies and discussions with the airport management lead to a more economical but technical solution. The runway 07.R should be shortened during one month and the roof slab (over a length of 157 m) at this runway could be executed with the cut-and-cover-method.

As August is the month with the less nebulosity, the airport management admitted the interruption only in this month. The runway 07.R is the only one which can be used to land in case of mist as it is equipped with an instrument landing system (ILS).

The construction methods were strongly influenced by the security directions of the airport.
- The necessary safety distances versus the air traffic (safety distances which are on their turn dependent of the weather circumstances and visibility limits) should be taken into account.
- Machines and tools can only be used respecting the valid height restrictions.
- Machines, portable phones etc. may not disturb radar and control tower.
- The necessary measures should be taken to protect the workers and the materials against the blast-effect, this is the enormous air displacement caused by the take-off of the planes.

Next to these technical elements a number of administration directions have to be taken into consideration.
- All the persons and vehicles passing the airside area have to possess a valid entrance identification.
- Each passage landside-airside is checked by watchmen in guardhouses.
- All vehicles entering the airport have to obey the own specific traffic reglementations and have to be ensured with a minimum amount of 100 million B.F. for claims as a result of fire or explosions.
- For the execution of the works, an insurance covering all worksite risks up to an amount of 20 milliard B.F. for damage at planes and passengers needs to be settled.

Another aspect of the problematic is comprised in the administrative treatment of the files. Special attention has to be paid to a good timing with reference to approbations and starting dates according to the admitted working periods defined by the airport management.

2.2.2. *Execution methods*

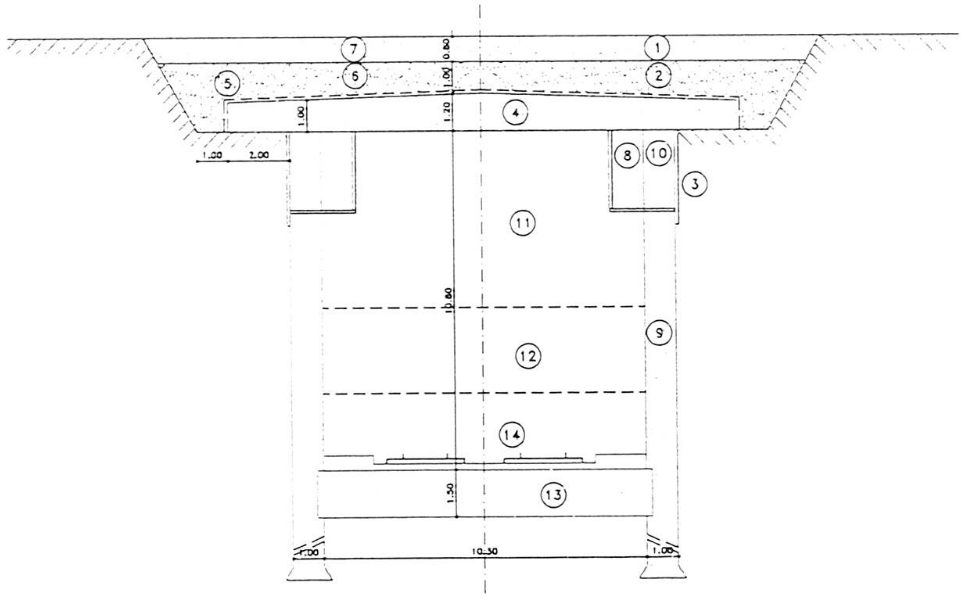
Roofslab 07.R

Based on the above mentioned problematic the following execution methods were chosen as possible solutions. If possible, a classic construction of the tunnel with the cut-and-cover-method was restrained.

In the area of the roof slab that is already executed, two longitudinal galleries are excavated from where the sheeted trenches are executed. After reinforcing and concreting of the sheeted trenches, they are concreted up to the bottom side of the roof slab.

Sequently the excavation can start from top to bottom.

The bottom slab can be reinforced and concreted.



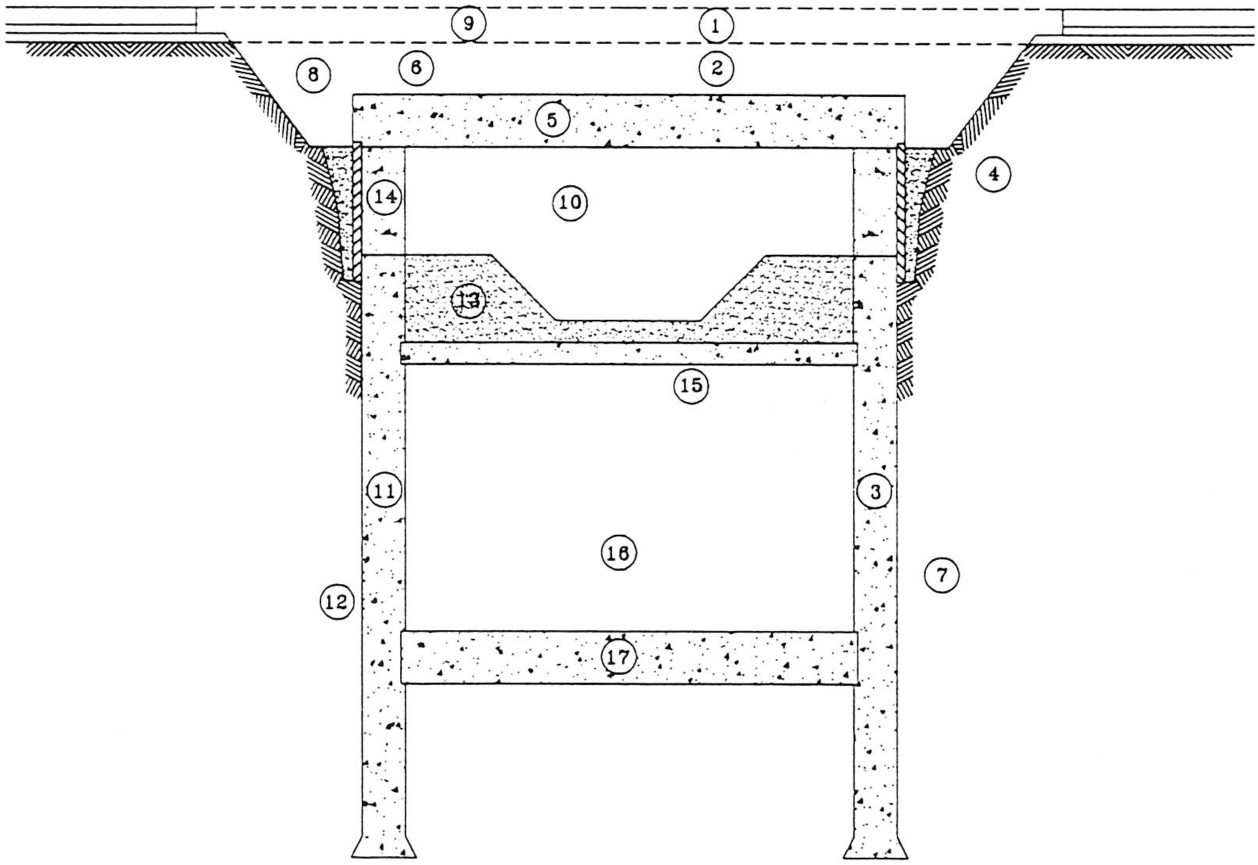
1. *Demolition hardening runway 07.R*	10. *Sheeted wall*
2. *Slope excavations up to bottomside roof slab*	11. *Excavation under the roof slab and demolition of the sheet pilling of the gallery*
3. *Drive in of sheet piles to 0,5 m under the gallery*	12. *Excavation in restricted length up to bottom side bottom slab*
4. *Working floor, reinforcement and concreting of the roof slab*	13. *Working floor, reinforcement and concreting of bottom slab*
5. *Watertight coat and protective concrete*	14. *Foot paths, track installation and electro-mechanical equipment*
6. *Refilling with soil*	
7. *Repair of the runway 07.R*	
8. *Execution of galleries under the roof slab*	
9. *Execution, reinforced concrete-sheeted trenches*	

Tunnel at the standing places - Section A and B

As the standing places were out of order during 3 months (August, September and October 1995) the most economical solution was obvious.

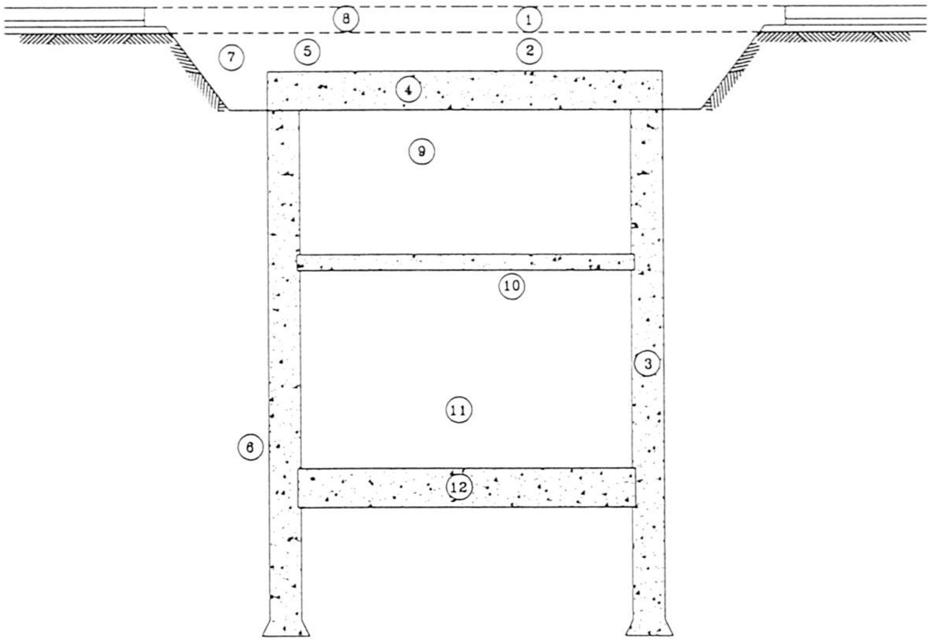
This solution can be compared with the one that was applied for the execution of the roof slab under the runway 07.R in August '93 (although the longer distance ± 330 m and the necessity to maintain the air traffic in several phases). Because of the gained experience with the roof slab in 1993, this method was still refined.

Section A



1. Demolition hardening platform
2. Excavation up to bottom side of roof slab
3. Construction of primary sheeted trenches (1 to 2 = 50 %)
4. Concrete slab, to be placed at 20 cm behind the primary sheeted trench and filling up with sand cement
5. Working floor, reinforcing and concreting of roof slab
6. Watertight coat and protective concrete
7. Injection of primary sheeted trenches
8. Refilling with sand cement
9. Repair of platform and ground surface
10. Excavation of dumpling 1st phase
11. Secondary sheeted trenches out from dumpling 1st phase (1 to 2 = 50 %)
12. Injection secondary sheeted trenches
13. Excavation of dumpling 2nd phase up to the intermediate slab
14. Connection secondary sheeted wall - roof slab
15. Reinforcing and concreting of the intermediate slab
16. Excavation of dumpling up to bottom slab
17. Working floor, reinforcing and concreting of the bottom slab

Section B



1. Demolition hardening platform *2. Excavation up to bottom side roof slab* *3. Construction of sheeted trenches* *4. Working floor, reinforcing and concreting of roof slab* *5. Watertight coat and protective concrete* *6. Injection sheeted trenches* *7. Refilling with sand cement*	*8. Repair of platform and ground surface* *9. Excavation of dumpling 1st phase up to the intermediate slab* *10. Reinforcing and concreting of intermediate slab* *11. Excavation up to bottom side of bottom slab* *12. Working floor, reinforcing and concreting of bottom slab*

In 3 months time the hardening of the platform needs to be demolished and excavated to the bottom side of the roof slab; also the walls need to be executed in sheeted trenches (in some areas 100 % according to section B, in other areas 1 to 2, according to section A), the roof slab reinforced and concreted, filled up and the concrete hardening repaired in its original condition. The further part of the tunnel is executed entirely with underground construction methods.

3. Present situation

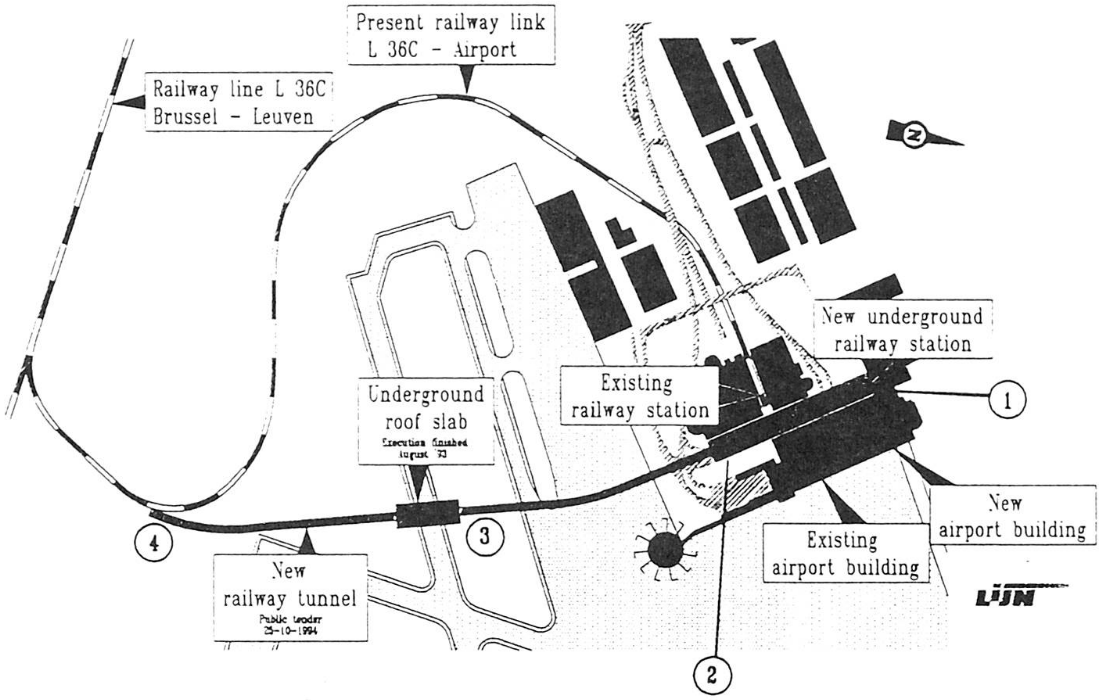
3.1. Main structure

Parts 1 and 2 of the station are finished.
Part 3 is a part which is constructed together with the tunnel under the airport. Works started on 03.04.1995 with finishing date at the end of 1997.

3.2. Architectural finishing and electro-mechanical equipment

Works are in execution and will be finished mid 1998.

3.2.1. Security measures



A lot of attention is paid to the security of the passengers. In case of fire or emergency situations, four emergency exits can be used, as well in the northern and the southern part of the station as in the mid part and at the southern end of the tunnel.

The emergency exit Nr. 1 is situated at the northern side of the station and runs from the platform level into the open air. The northern side of the station is situated at the Diamant area of the new airport building.

The emergency exit Nr. 2 is situated at the southern end of the station. From platform level, it runs into the English basement of the old airport building. From this basement it is possible to reach the arrival hall, which is situated in the immediate neighbourhood of the local fire services.

The emergency exit Nr. 3 is about situated halfway the 1 km long tunnel under the airside lane of the airport. As the airport management did not allow the construction of a direct emergency exit, running into the open air at airside, an emergency stair runs from the track level into an underground escape route that is situated on top of the tunnel. The escape route runs into the open air in the airside area at the arrival level.

The emergency exit Nr. 4 is located in the southern end of the tunnel. The tunnel is provided with a platform for fire extinction and a special approach for the fire engines.

Both the tunnel and the station have sufficient fire extinguish devices, such as fire detectors, ventilators to suck off smoke, heat and gas, sprinklers, smoke screens and curtains to keep the smoke in a clearly defined area, fire hydrants, fire boxes, movable platforms for evacuations by track etc.

The ceiling of the station is sprayed with a product that increases the insulation and the fire-resistance. The product is on a rockwool base and does not contain asbestos particles, cellulosis fibres, cement or any solvable silicate. It is sprayed with a layer thickness of 40 mm and fireproof. A fire resistance Rf of 2 hours of the lining is guaranteed. The product also reduces the noise level in the tunnel.

3.3. Track installation and connection with the existing railway line Brussel - Leuven

As the track laying is concerned, both station and tunnel are provided with reinforced mats. Track laying occurs on concrete cross beams which are captured in neoprene shoes, functioning as anti-vibration elements. The whole study of the anti-vibration is figured out by the D2S company - Leuven - Belgium.
Once the track laying is finished, a slab is concreted under the cross beams, hanged on metal profiles. The assembly can be manipulated in heigth and width in order to determine the right alignment.
Track laying in the open air occurs according to the traditional ballasted track laying system. The target date for operation is mid 1998.

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Cut-and-cover Tunnel in a Karst Environment

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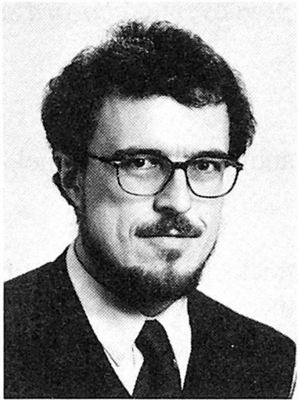

Thomas Vogel, born in 1955, graduated from the Swiss Federal Institute of Technology ETH Zurich in 1980. He gained eleven years of experience in bridge and structural design in consulting firms in Chur and Zurich. In 1992 he joined ETH as a professor of Structural Engineering.

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Summary

A tunnel of the Swiss railway network traverses in its cut-and-cover section over a Triassic limestone stratum which exhibits the karst phenomena that causes dolines and subsidences. The tunnel is to bridge the dolines as a stiff tube and deform with the long term subsidences. The paper describes the hazard scenarios that were considered, the derived design concepts, the chosen solution, the calculations made to ensure structural capacity and serviceability and how these considerations influenced the final design.

1 The project

The Adler Tunnel is a paramount part of the Swiss railway project "Railway 2000" to provide travel times under 60 minutes between the major towns of Switzerland. The two-track tunnel constitutes a more direct route than the existing railway line that leads from Basle southbound to the central parts of Switzerland and through the Alps. The tunnel consists of a drilled core section of 4.3 km in length and cut-and-cover sections at each end. A general overview of the project with contributions covering design and construction is given in ref. [1].

The western cut-and-cover section which is 750 m in length traverses a gravel bed that was deposited by the Rhine river following the last glacial period. Beneath the gravel lies a Triassic limestone stratum, known as Muschelkalk (shell limestone), which exhibits the karst phenomena. This causes dolines (sink holes) and extensive subsidences to occur. Within the last 40 years about 20 new dolines with diameters between 5 and 10 m have arisen.

According to estimations summarised in a geotechnical report dolines up to 22 m in diameter and 5 m in depth can form within a few hours and subsidences with a diameter of 100 m can develop with a settlement speed of 10 mm per year.

2 Conceptual considerations

According to the Swiss Actions Code [2] the intended use of a structure and its performance requirements are defined by the client and the designer in the *utilisation plan*. On this base the design engineer identifies and evaluates critical situations both during construction and throughout the projected service life. In the *security plan* eventual *hazard scenarios* are listed with the measures specified to ensure safety.

2.1 The utilisation plan

The Swiss Federal Railways as client and operating authority of the tunnel specified the following service requirements:

- a planned service life of at least 150 years
- partial replacements requiring lengthy closure of tracks not before the first 100 years
- speed limits of 160 km/h at present, 200 km/h in the future
- sophisticated restrictions of availability for maintenance taking into account time of day (day or night), number of tracks (single track or total closure) and intervals (daily, weekly, monthly, yearly closure)
- acceptance of local moisture, no acceptance of dripping water that could form ice
- minimum radius of curved deformation due to subsidences: 5000 m
- maximum subsidence: 250 mm
- keeping within the indicative values of deflections for railway bridges of [2] for the local spanning of dolines.

The slowly occurring subsidences can be taken into account by designing the cross-section with the necessary additional space which allows a realignment of the tracks. Suddenly occurring dolines, however, should be bridged by the tunnel acting as a stiff tube.

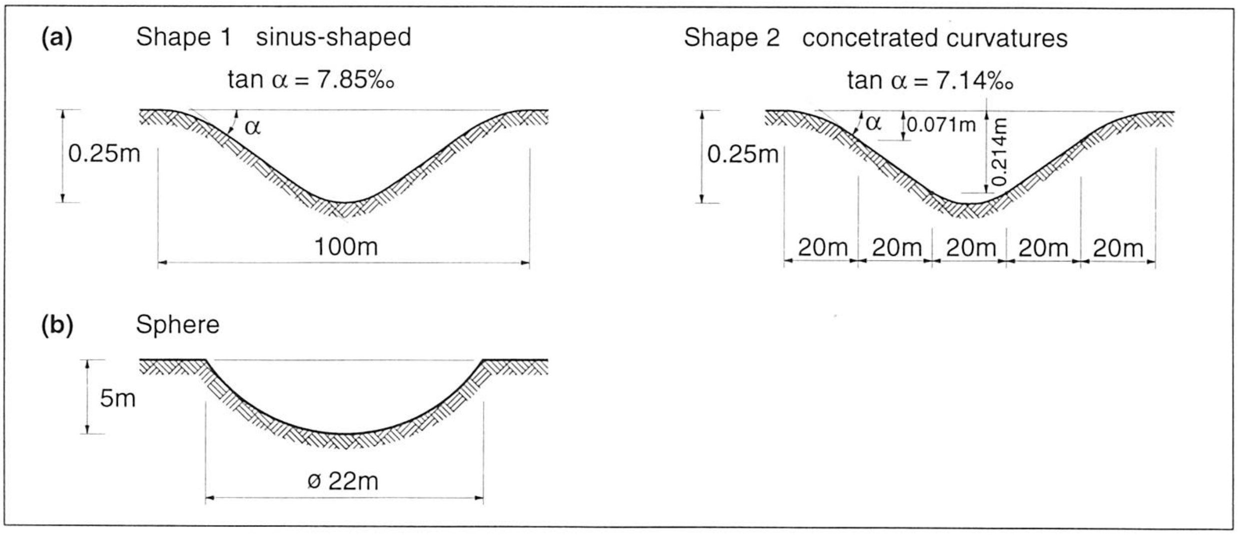

Fig. 1 Standard subsidences shape 1 and 2 (a) and standard doline (b)

2.2 Hazard scenarios and safety plan

In addition to the usual loads on a cut-and-cover railway tunnel the following situations were taken into account:

1. A slowly developing extensive subsidence with an average speed of 10 mm per year and a maximum of 200 mm per year. These are caused by the collapse of caverns in great depth as a consequence of a dissolving of the limestone or a changed stress state.
2. A superficial collapse of a karst cavern due to the same reasons resulting in a funnel that

reaches the surface or the tunnel level.

In both cases a reliable detection and subsequent refill are not possible within reasonable economical limits. The above cases were therefore defined numerically in order to incorporate them as hazard scenarios into the structural analysis. Figure 1 shows the two types of standardised subsidences (sinus shaped and with concentrated curvature) and the standardised spherical doline. The doline may occur at or eccentric to the tunnel axis within hours. Thus a very high accuracy in the design process is not justified as these assumptions and their numerical values rely mainly on sound judgement and experience.

3 Structural analysis

3.1 The cross-section

To bridge local dolines at any position leads to a monolithic tube over the whole endangered length. Any joints or hinges would reduce the bridging effect. The adjacent drilled section and the large overburden of up to 5 meters led to a circular cross-section with an external diameter of about 12.5 m. The crown vault has a depth of 400 mm and the invert vault a depth of 600 mm. An additional vault of 280 mm depth is located over the invert and forms the service channel. The remaining invert area is filled with gravel (fig. 2(a)).

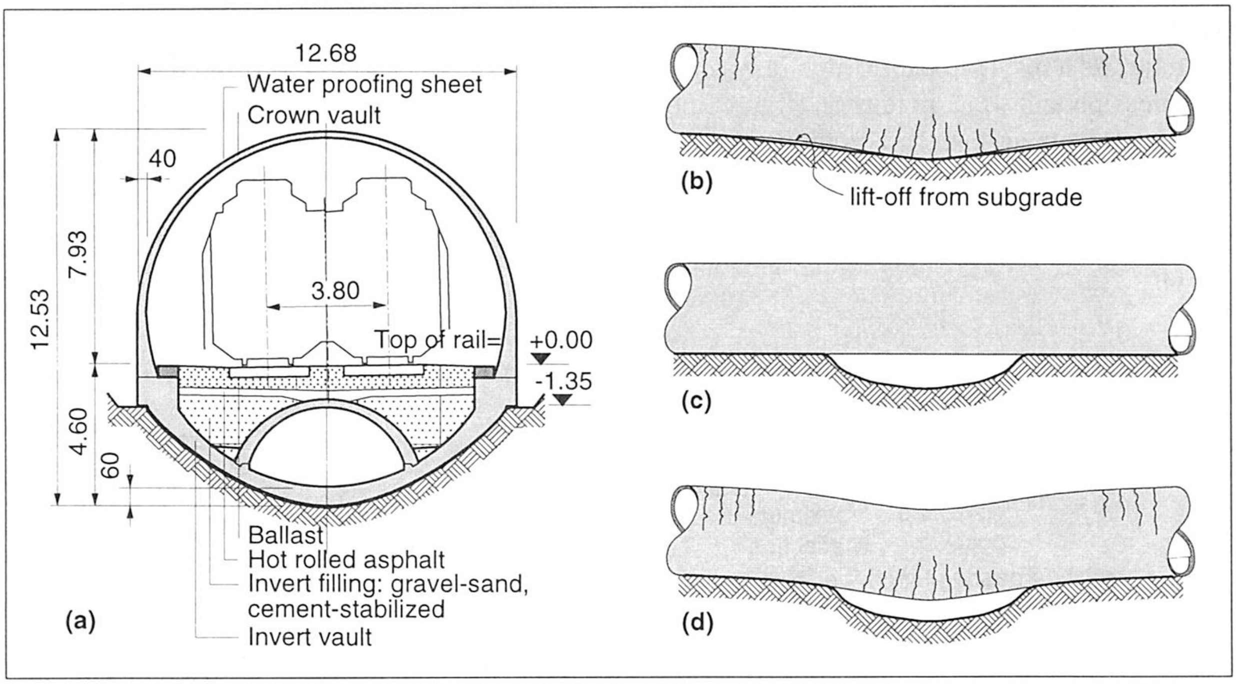

Fig. 2 Typical cross-section of the tunnel (a), fundamental cases of structural performance: extensive long term subsidence (b), doline (c) and subsidence with doline (d)

3.2 Structural performance and the statical model

The model used in the structural analysis is an elastically supported beam with self-weight, imposed and traffic loads. Since the tube is unable to span a subsidence of 100 m, it is forced to deform with the settlement. If the tube was fixed to the ground, the shape of the subsidence would equal the deflection curve of the tube and the second derivation would equal the curvature. The elastic support allows a certain balance of deflections due to the bending stiffness of the tube. As the load is limited a local lift of the tunnel from the ground has to be taken into account. Nevertheless the situation can be treated as a serviceability problem; the deformations result in cracks and at most in the plastic hinges, but failure does not occur (fig. 2(b)).

If a doline develops, however, the tube is to span the funnel and act as a tunnel bridge. That means a safety problem arises and the ultimate limit state has to be considered. A doline eccentric to the tunnel axis leads to asymmetric earth pressure which can be taken up by a tube of relatively large thickness by transversal bending (fig. 2(c)).

It is very likely that a collapse of a deep cavern and the subsequent subsidence will lead to the failure of a cavern at a minor depth causing a doline to form simultaneously. The relevant and most plausible case to be considered is the superposition of a standard subsidence and a standard doline.

The important factor at the ultimate limit state is that the available ductility is reduced at the relevant cross-section as it is already drawn upon to deform with the subsidence. Therefore the behaviour of the tube spanning the doline is less ductile or more brittle (fig. 2(d)).

3.3 Implementation of the statical system

The preceding description shows that linear-elastic models are unsuitable to the task. Local problems such as transversal bending due to asymmetric earth pressure and concentrated reactions at the edge of the doline, however, can be separated from the global bearing function of the tube. As a result a beam structure was modelled, supported by springs that are stiff in compression and weak in tension (figure 3(a)). The spring stiffness in compression can be calculated from an assumed modulus of subgrade reaction k and the affiliated reaction area (figure 3(b)).

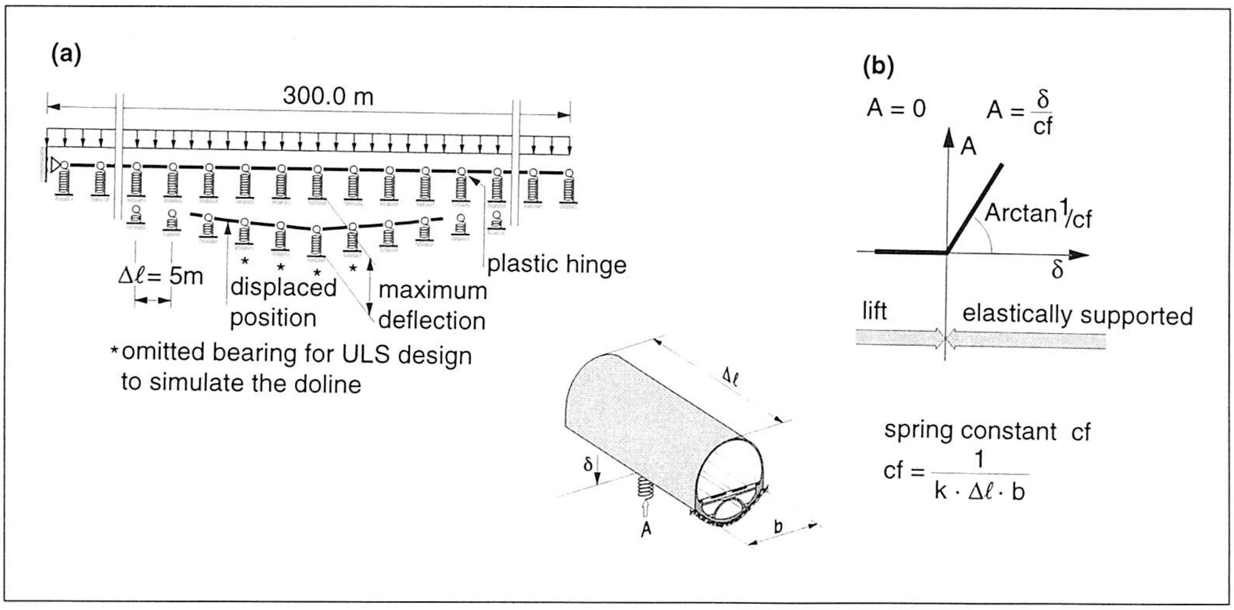

Fig. 3 Modelling as a series of bars (a) and characteristics of springs (b)

A rough approximation shows that the cross-section is under-reinforced even with two layers of Ø 30 mm @ 150 mm in the upper and the lower vaults respectively. Thus for pure bending and limited steel strains the concrete does not reach its ultimate compressive strain although the moment-curvature diagram, calculated with the material properties given in [3] and a failure steel strain of 10‰, shows a typically ductile behaviour (figure 4(a)).

4 Numerical calculations

The calculations were made with the program STATIK-N [4], a postprocessor to STATIK-2, a program for frame structures widely used in Switzerland. To meet the requirements of STATIK-N the following simplifications were made:

- The elastic domain of the moment-curvature diagram up to the yielding moment M_y (containing the uncracked and the cracked stage) is linearized and the gained bending stiffness attributed to the cross-section. By this simplification creep is neglected i.e. the difference between the long-time and short-time value of Young's modulus is not taken into consideration.
- The plastic domain of the moment-curvature diagram between the yielding moment M_y and the bending resistance M_R is concentrated at the nodes as yielding with hardening (fig. 4 (a)).

For the serviceability limit state evidence is required on the crack width resulting from the calculated curvatures . This is done using equation (1), where w_m denotes the average crack width, ε_{sb} and ε_{sc} the average strains calculated from the bending moment and the affiliated curvatures as well as the shrinkage (assumed to be 0.2‰) respectively. s_{rm} denotes the average crack distance (assumed to be the spacing of the lateral reinforcement @=150 mm). The factor 0.9 accounts for the tension stiffening, i. e. the participation of the concrete between the cracks.

$$w_m = 0.9(\varepsilon_{sb} + \varepsilon_{sc})\, s_{rm} \tag{1}$$

4.1 Executed calculation runs

The calculation procedure went as follows:

- Serviceability limit state check with both forms of the standard subsidence; with loads, but without load factors. The settlement was increased step by step up to the maximum value of 250 mm. The formation of plastic hinges and of local gaps between tunnel and subgrade could be observed.
- Ultimate strength limit state check with a doline; introduced by omitting 4 springs and loads increased by applying the relevant load factors. Subsequently the formation of the standard subsidence was imposed and increased up to the same maximum value as above. To meet the design level the moments derived from the moment-curvature diagram were reduced by dividing by the resistance factor $\gamma_R = 1.2$.

The first calculation run was executed with two moduli of subgrade reaction ($k = 10^4$ kN/m^3 and $k = 10^5$ kN/m^3) to evaluate their influence. The amount of reinforcement corresponded to two layers of Ø 30 mm @ 150 mm in both the crown and invert vaults and Ø 18 mm @ 150 mm in the lateral regions. For the second calculation run all rebars were reduced to Ø 18 mm and k fixed at 5 x 10^4 kN/m^3. This run served to establish an intervention point for the deflection that leads to average crack widths of 0.4 mm. A crack width of 0.4 mm was regarded as the threshold value for corrosion to begin. The third calculation run was identical to the second but with a main reinforcement of Ø 26 mm.

5 Results

In the serviceability limit state calculation the modulus of the subgrade reaction governs the appearance of plastic hinges. The stiffer the subgrade the sooner the hinges appear. The length of the plastic zones depends mainly on the presumed shape of the subsidence and amounts to between 10 and 20 m. Adequate flexural strength to span the standard doline alone is guaranteed in any case. When the standard doline and standard subsidence are applied combined the plastic hinges appear at an earlier stage. A reduced bending capacity M_R/γ_R is reached when $k = 10^5$ kN/m^3 at a maximum settlement of about 190 mm. As expected the intervention point is only minorly dependent on the reinforcement content. Nominal crack widths of 0.4 mm occurred with rebars of Ø 18 mm at a settlement of about 120 mm, with Ø 26 mm at 130 to 140 mm.

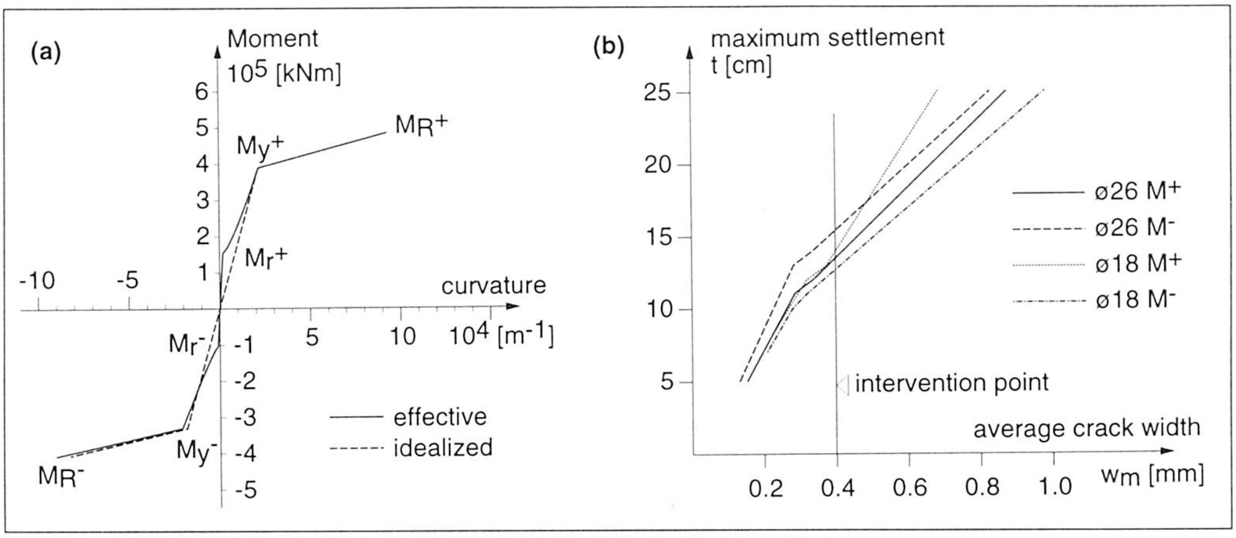

Fig. 4 Moment-curvature diagram for rebars Ø 26 mm (a) and settlement - crack width diagram for standard subsidence shape 1 (b)

Figure 4 (b) shows the relationship between crack width and maximum settlement for different reinforcement contents and both positive and negative bending. The crack widths were calculated at the cracking moments both before and after the yielding moment was reached and for the maximum deflection of 250 mm. All results were arrived at quite simply as described above. The figure shows that the yielding point is not a marked event as the deflections which govern the process are forced. This explains why the amount of reinforcement alone does not greatly influence the crack width.

5.1 Influence on the final design

The amount of reinforcement was governed by the ultimate strength limit state design with the load case of doline and subsidence combined at the same location. The Swiss Federal Railways specified an adequate safety even at a late intervention point concerning cracks and deflections. Reinforcement bars of Ø 26 mm @ 150 mm were finally chosen for both the crown and invert vaults. The ductile behaviour of the tunnel tube and its rotational capacity is herewith guaranteed.

6 References

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Seismic Design for Large-Scale Cut-and-Cover Tunnels

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Summary

The cut-and-cover tunnel described in this paper pertains to a large-scale underground station with a length of 400 m, a cross section of 25 m height, and 25 m width in soft ground. . One part of the top slab of the station will be required to serve as the foundation for an elevated viaduct structure, and the entire structure will be a very complicated to be built in a seismically high risk area, and in view of the fact that the structure will be an important train station, there will be many passengers, and seismic safety will be an important design task.

It has been the practice to consider underground structures as highly safe against earthquakes,. but in 1995 in the South Hyogo Prefecture Earthquake struck for the first time in the world. The cause for this disaster was due to the fact that the underground structure was located where relative displacement occurred in the soils and shear deformation took place during the earthquake. The capability of the reinforced concrete center column to resist the shear forces was exceeded.

In view of the calamity experienced by this catastrophe, in this report seismic safety could be satisfied to secure ductility for large-scale earthquake loads. The important factors in the seismic design are

1) to understand the non-linear seismic behavior of structures with high-degree redundancy supported by the ground, and
2) to secure the seismic safety as a whole structure.

1. The Earthquake Damage Experienced by the South Hyogo Prefecture (Kobe) Earthquake and The Causes of Why It Happened.

1.1 The Damage Experienced by the South Hyogo Prefecture (Hyogo) Earthquake.

On 17 January 1995, Hyogo Prefecture was struck by an earthquake with a magnitude of 7.2 in the "South Hyogo Prefecture Earthquake". The earthquake occurred near a densely populated area and more than 5,500 people perished with a high loss of property. All types of structures were damaged including high-speed railways and roadways, harbor facilities, water supply, sanitary collection and treatment facilities, gas, and electric power and distribution systems.

Of the civil structures that were damaged, this was the first time in the world that a subway system was subjected to large damage on a major scale. Especially, the Daikai Subway Station was almost completely demolished . Fig.-1.1 gives a cross section view of the station and describes the damages incurred[1)]. The ground in the vicinity of the station had a shear wave velocity Vs of more than 200 m/s which is fairly good, the ground higher than the tunnel was Vs = 140 m/s. From the damage sustained by the station, it can be seen that the reinforced concrete central columns supporting the station upper slabs failed, and the upper slabs collapsed by breaking in two.

Although they did not collapse in a similar manner to the Daikai Station, there were ruptures due to shear failures in the reinforced concrete columns at Kamisawa and Sanno-Miya Stations, and the top and intermediate slabs gave way.

It has been the rule that underground structures are almost never damaged in earthquakes, and the few damages that have occurred have been very minor. There has been a strong belief that "underground structures are very stable since they will move in the same manner as the surrounding ground". In fact, the underground subway stations that sustained earthquake damage were not designed for earthquakes of the South Hyogo Prefecture Earthquake Class. With the experience gained by this earthquake, it has become necessary to reconsider the design methods for the design of underground structures.

1.2 The Causes of the Earthquake Damage to the Excavated Tunnels[2) ~ 5)]

The damages sustained by the Daikai Subway Station and the excavated tunnels are now being investigated by the various authorities to determine why the damages were so widespread. From their findings, the following points are some of the reasons for the damages to the excavated tunnels.

1) At the time of the earthquake, there was shear deformation within the layers of the ground, and there was shear deformation caused transverse to the tunnels. As a result, there was relative differential deformation caused between the upper and lower slabs.
2) There was average axial stress in excess of 80 MPa in the reinforced concrete center columns which was greater than in the side walls, and the center columns had less ductility where shear collapse set in ahead of bending collapse.
3) Due to the relative displacement between the top and bottom slabs, the central column support failed as it could not move in consonance with the side wall.

From the above, in order to maintain the earthquake strength of the excavated tunnels, deformation of ground-structure system considering its non-linear characteristics is necessary to be found out and sufficient ductility shall be ensured. Especially, it has become necessary to ensure that the center column does not collapse by shear force.

2. The Structure to be Targeted.

2.1 A Description of the Yokohama MM21 Line Underground Station.

The objective of this report is the large-scale subway station in the excavated tunnel as shown in Fig.-2.1. The main features that will require earthquake design are as follows:

1) The structure is located in an area with high seismic risk.
2) The station has heavy passenger traffic, and be a large-scale underground structure of high importance.
3) The station is located some -35 m underground in a soft clayey ground.
4) One part of the upper slab of the station will serve as the foundation for an elevated viaduct structure and the subway station will have a pile foundation.

The center column is a steel pipe column filled with concrete and all other member are made of reinforced concrete. The reasons for using concrete filled steel pipe is to ensure sufficient ductility of the center column, and not to set in ahead of shear collapse.

2.2 Description of the Earth Foundations.

Fig.-2.1 gives a description of the foundation soils for the seismic analysis. The depth of ground layer is approximately 30 m from the bed rock for the seismic design, and the soils are soft clay with a $V_s = 120 \sim 150$ m/s.

3. Seismic Analysis Conditions and the Methods Used.

3.1 The Purpose for Seismic Analysis

The following three types of seismic analysis was performed for this study:

1) Seismic deformation method by the uniform loss of stiffness model[6).
2) Dynamic analysis by the uniform loss of stiffness model.
3) Non-linear seismic deformation method.

In comparing the “Dynamic analysis by the uniform loss of stiffness model” with the “Seismic deformation method by the uniform loss of stiffness model”, the static model of the seismic deformation method can be evaluated.

From these results, the seismic characteristics of structures can be evaluated by the “non-linear seismic deformation method”.

3.2 Seismic Deformation Method by the Uniform Loss of Stiffness Model.

Fig.-3.1 is the concept of the Seismic Deformation Method, and Fig.-3.2 gives the flow diagram of the Seismic Deformation Method. The analysis model is a framework model supported by the ground spring, and the bending stiffness are assumed at $1/5^{th}$ of their total cross sectional state considering the loss of stiffness caused by earthquakes.

3.3 Dynamic Analysis by the Uniform Loss of Stiffness .

The analysis model is a 2-dimensional FEM “FLUSH” of the ground~structure interaction system. The structure (excavated tunnel, viaduct, pile foundation) is a space frame model, and the ground foundation is displayed as a 2-dimensional solid model. In addition, the bending stiffness of the components are similar to the seismic deformation method. For the ground ,the loss of stiffness at earthquake time is considered.

3.4 Non-Linear Seismic Deformation Method.

The non-linear seismic deformation method is basically similar to the uniform loss of stiffness model. The different points are that the stiffness of the structure is not reduced uniformly, but the components have been given non-linear characteristics. For this reason, the components have been given an initial stress analysis under permanent load prior to applying earthquake loads.

4. Input Motions.[7)]

For the earthquake motion inputs, Fig.-4.1 gives the velocity response spectrum, and Fig.-4.2 gives the acceleration corresponding to the spectrum. The velocity response spectrum in Fig.-4.1 is based on the ground motion observed of the South Hyogo Prefecture Earthquake .

5. Seismic Deformation Method by the Uniform Loss of Stiffness Model.

5.1 Calculation of the Ground Response

Fig.-5.1 gives the ground response to be used for the seismic deformation method. This is calculated according to the One Dimensional Dynamic Analysis "SHAKE".

5.2 Establishing of the Analysis Model.

Fig.-5.2 gives the analysis model of the seismic deformation method . The analysis model is a two dimensional framework supported by the ground spring, and the bending stiffness are assumed at $1/5^{th}$ of their total cross sectional state considering the loss of stiffness caused by earthquakes.

The various loads of the seismic deformation method is based on the ground earthquake response from Fig.-5.1, and the ground displacement loads are obtained from the ground displacement and the ground spring, and the peripheral shear force is obtained from the ground shear force, and the inertia force was obtained from the ground acceleration

5.3 Results of the Analysis.

Fig.-5.3 gives a display of the displacement, bending moment, and the shear forces obtained from seismic deformation method by the uniform loss of stiffness .

6. Dynamic Analysis by the Uniform Loss of Stiffness Model.

6.1 Establishment of the Analytical Model.

Fig.-6.1 gives the Dynamic Analysis Model by the Uniform Loss of Stiffness Model. The input acceleration from Fig.-4.2 has been applied to the bed rock.

6.2 Results of the Analysis.

Fig.-6.2 gives a display of the deformation and maximum acceleration obtained from dynamic analysis.

6.3 Comparison of the Seismic Deformation Method and the Dynamic Analysis Method.

Fig,-6.3 gives a comparison of the bending moment of analysis by the Seismic Deformation Method and the Dynamic Analysis. This shows that they are generally compatible, and the static analysis of the Seismic Deformation Method indicates that the interaction response (ground~structure, viaduct~subway tunnel~pile foundation) are generally displayed.

7. Non-Linear Seismic Deformation Method.

7.1 Establishment of the Analytical Model.

The analysis model in Fig.-5.2 of the Seismic Deformation Method by the Uniform Loss of Stiffness Model basically agree with the Non-Linear Seismic Deformation Method. The different points are that the stiffness of the structure is not reduced uniformly, but the components have been given non-linear characteristics.

Fig.-7.1 gives the non-linear characteristics which have been obtained from References 8) and 9). The characteristics are M ~ Φ Type (bending moment ~ curvature).

7.2 Results of the Analysis.

Fig.-7.2 gives a display of the displacement, bending moment, and the shear forces obtained by the non-linear seismic deformation method

7.3 Evaluation of the Seismic Safety of Structures.

The shear reinforcing steel in each member have been placed so that bending collapse will not set in ahead of shear collapse. After which, the deformation capability of side walls and center columns at each floor are evaluated by comparing the relative displacement of side walls and center columns at each floor and allowable displacement generated. In this case, the allowable displacement was determined based on reference 10). The result is given in Table.-7.1. show that the side wall and center column at each floor ensure sufficient ductility.

8. Conclusion.

As a result of the experience gained by the South Hyogo Prefecture Earthquake, and the results of analysis performed for large-scale seismic safety of structures, the following facts have been disclosed:

1) A comparison of the results of the Seismic Deformation Method and the Dynamic Analysis show that both methods are relatively compatible, and that the Seismic Deformation Method can almost fully display dynamic interaction (ground~structures, elevated bridge structure~excavated tunnel~pile foundation).
2) According to the non-linear seismic deformation method, the seismic design for large-scale earthquake loads are to secure the seismic safety as a whole structure.

References:

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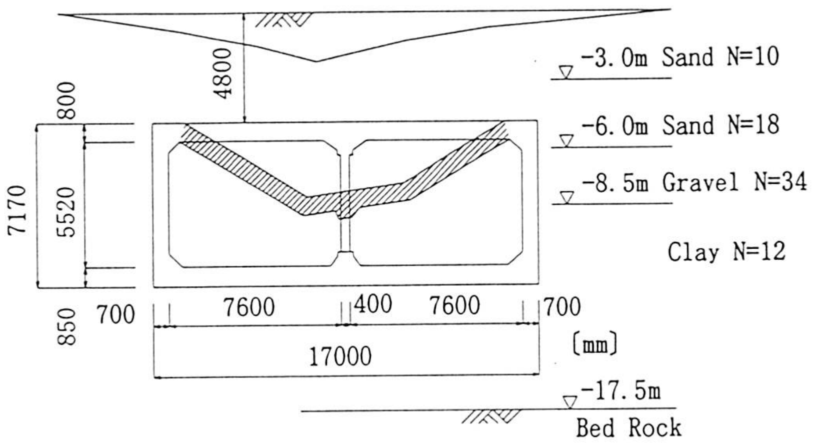


Fig.-1.1 Damages at Daikai Station

Depth (m)	Soil Type	γ t (tf/m³)	V s (m/s)	N-Value 10 30 50
	A sl	1.70	140	
5	A c2-1	1.60	120	
10	A c2-2	1.60	120	
15	A c2-3	1.60	120	
20, 25	A c2-4	1.60	120	
30	A c3	1.60	150	
35, 40	K hc	2.00	420	

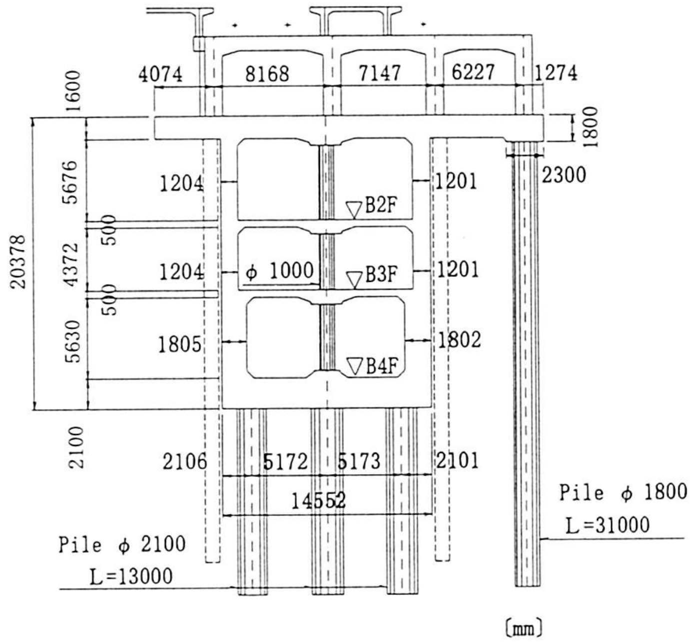


Fig.-2.1 Cross Section View of Yokohama MM21 Line Underground Station

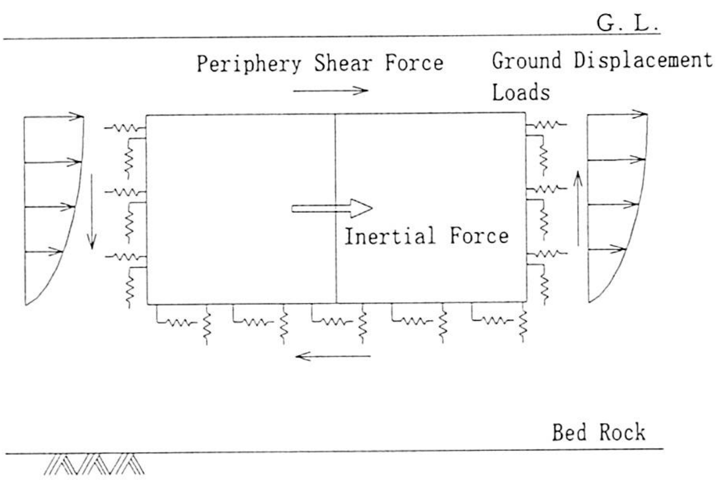


Fig.-3.1 The Concept of Seismic Deformation Method

START
↓
Establish Ground Response from One Dimmentional Dynamic Analysis (Displacement, Acceleration, Shear Forces, Eqivalent Stiffness)
↓
Establish Ground Spring
↓
Calculate Seismic Load
↓
Analysis Framework Supported by Ground Spring
↓
END

Fig.-3.2 Flow Diagram of Seismic Deformation Method

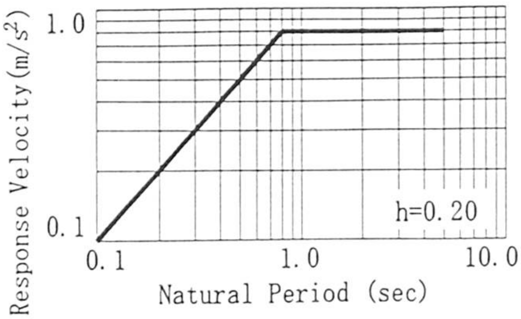


Fig.-4.1 Response Velocity Spectrum

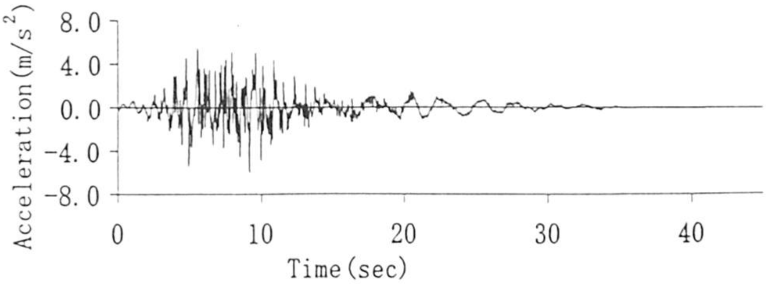
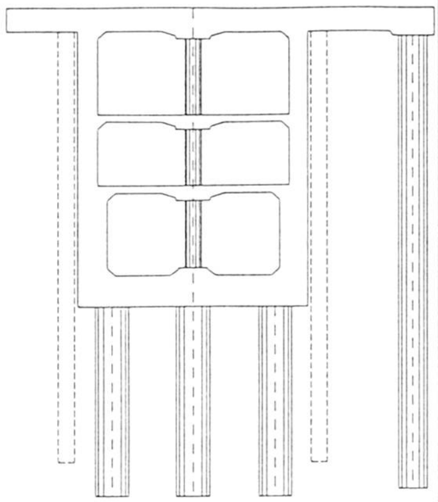


Fig.-4.2 Input Motion


Cross Section of Structure

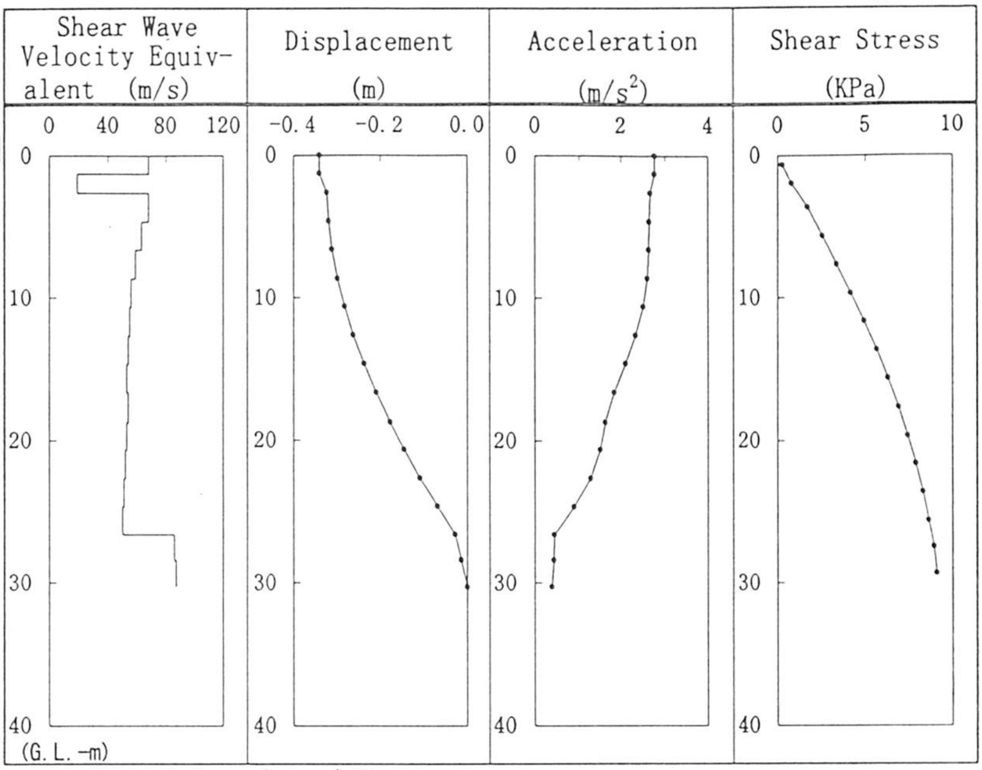


Fig.-5.1 Ground Response

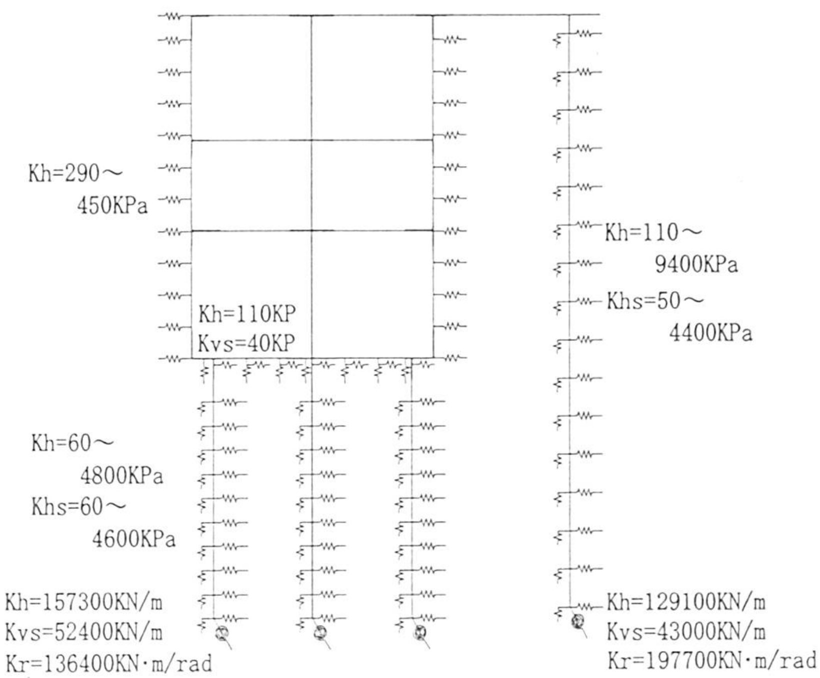
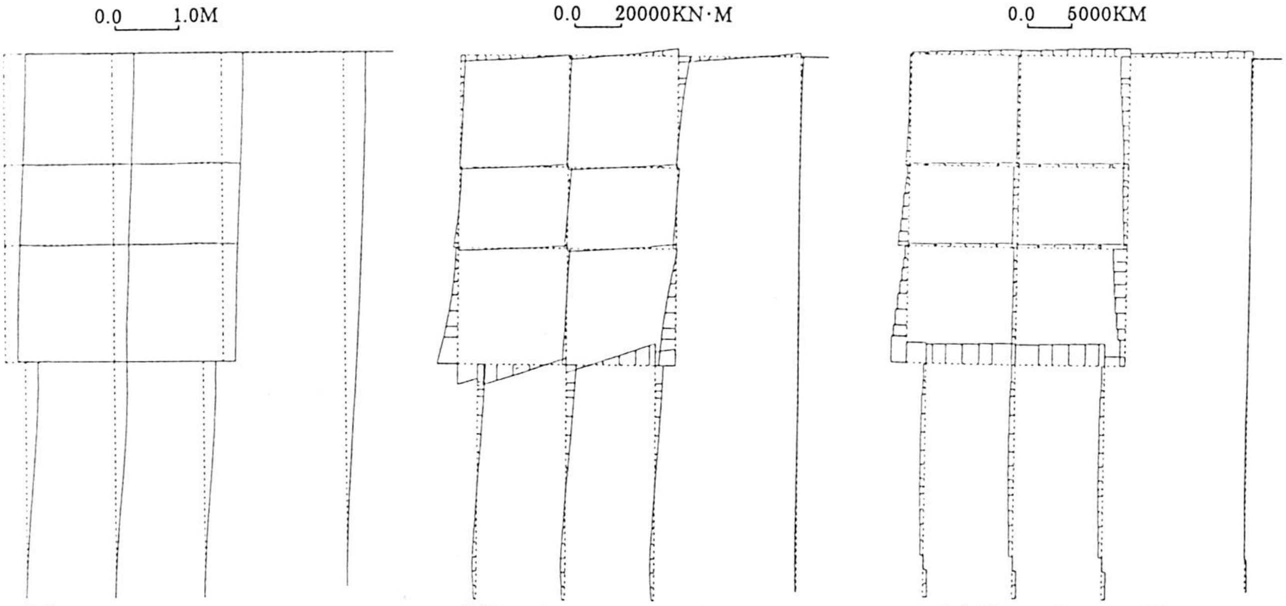


Fig.-5.2 Model of Seismic Deformation Method



(a)Deformation Diagram (b)Bending Moment Diagram (c)Shear Force Diagram

Fig.-5.3 Result of Seismic Deformation Method

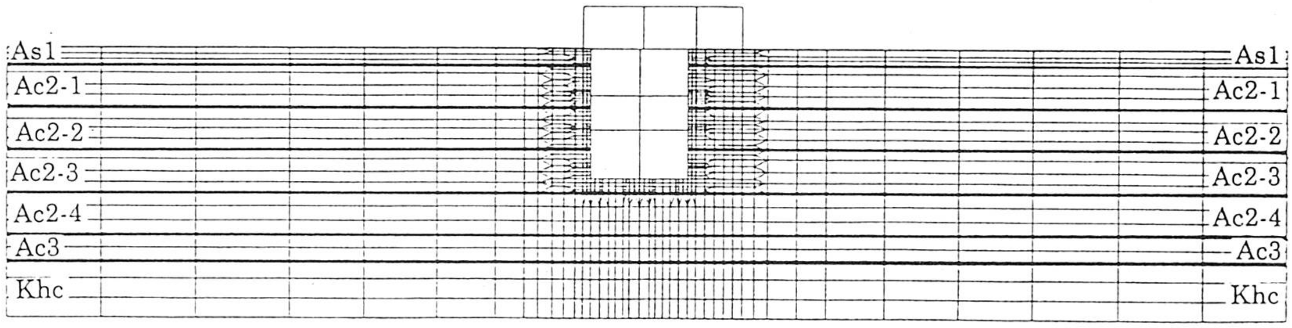
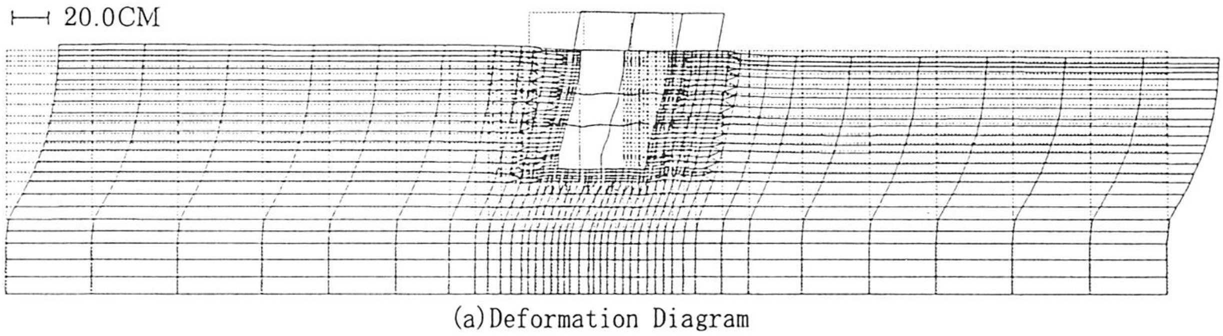
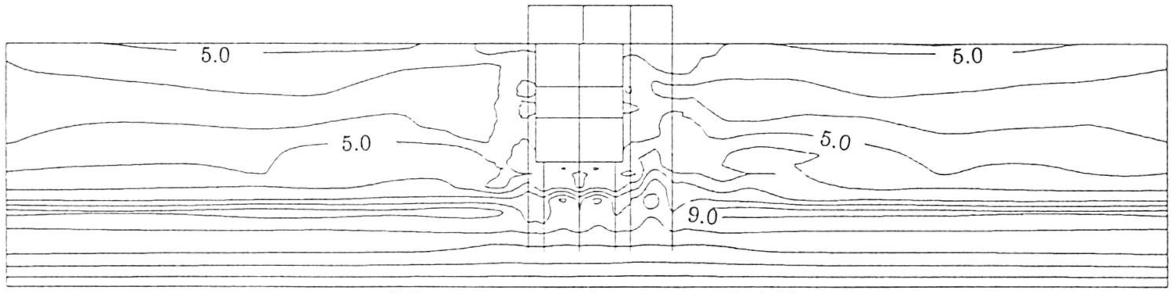


Fig.-6.1 Dynamic Analysis Model



(a)Deformation Diagram



(b)Maximum Acceleration [m/s²]

Fig.-6.2 Result of Dynamic Analysis

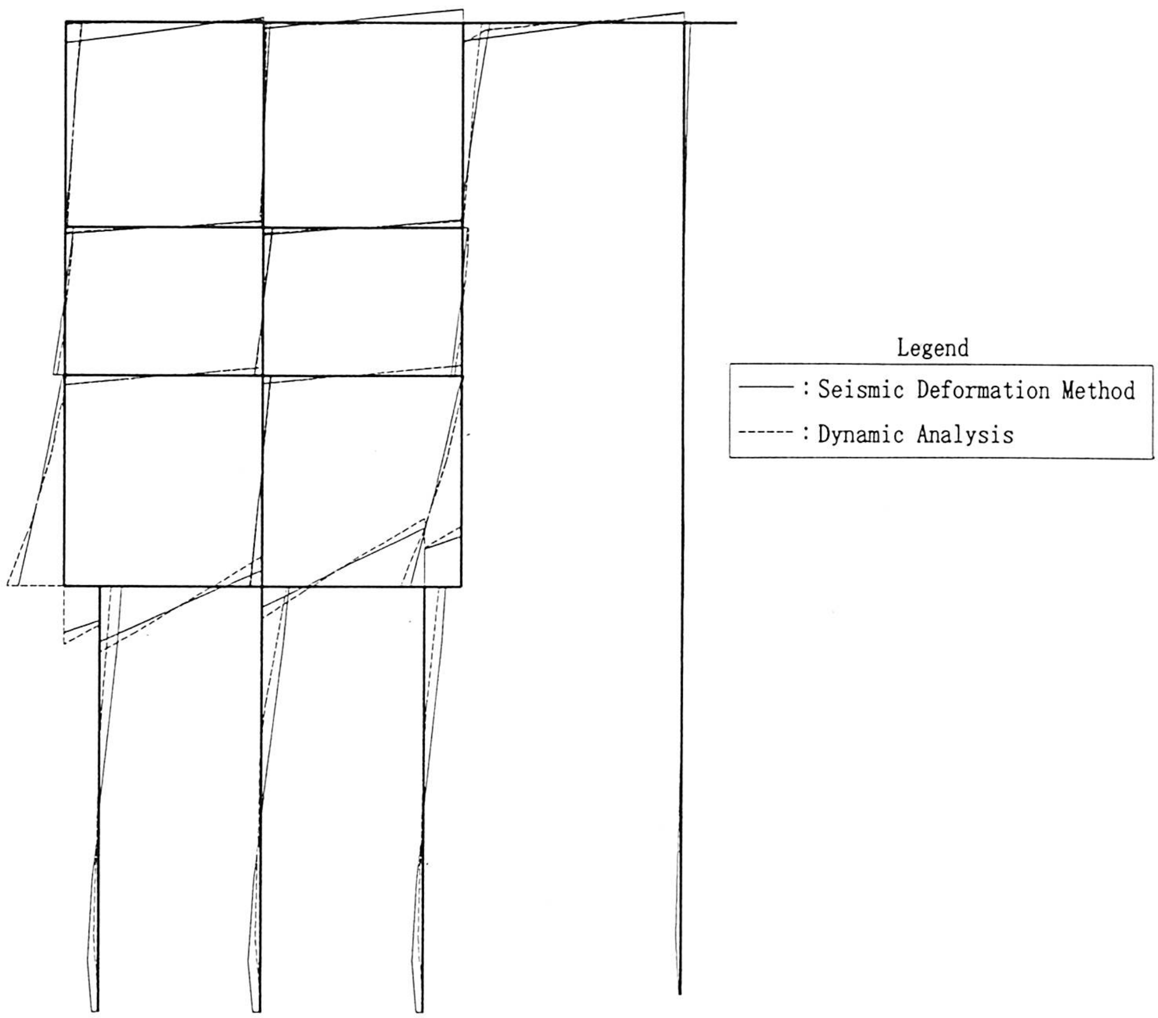


Fig.-6.3 Comparison of Bending Moment

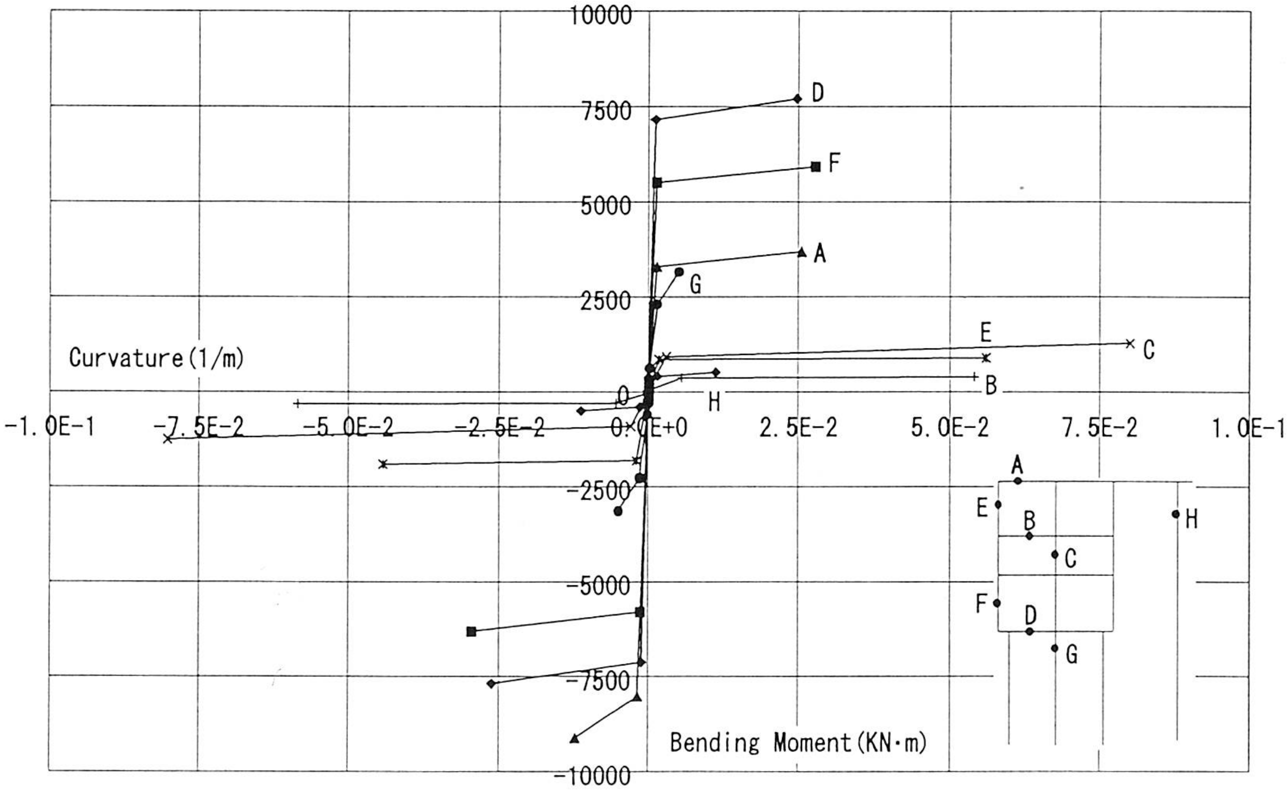
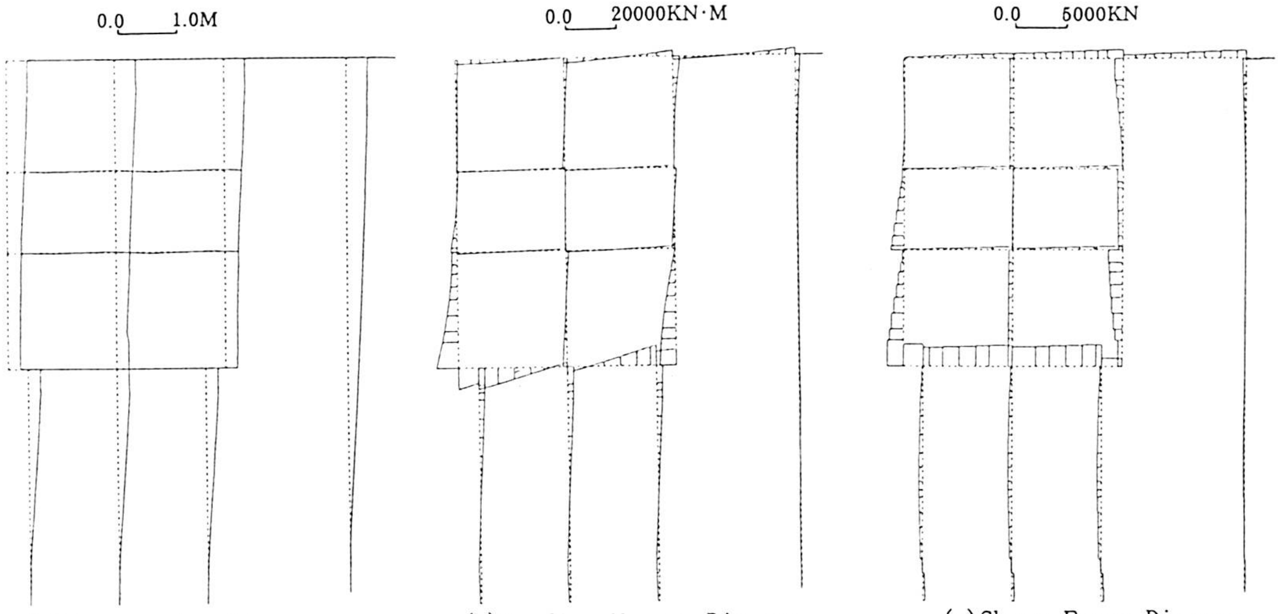


Fig.-7.1 The Characteristics for Bending Moment～Curvature



(a)Deformation Diagram (b)Bending Moment Diagram (c)Shear Force Diagram

Fig.-7.2 Result of Non-linear Seismic Deformation Method

Table-7.1 Results of Ductility Check

Location		Relative Displacement δud (cm)	Allowable Ductility μo	Yield Displacement δy (cm)	Allowable Amount of Displacement (μo·δy) (cm)
Left Wall	B 2 Fl.	0.7	10.0	1.6	16.0
	B 3 Fl.	2.0	4.3	0.3	2.2
	B 4 Fl.	0.9	6.8	1.0	6.4
Right Wall	B 2 Fl.	0.7	4.7	1.8	8.3
	B 3 Fl.	2.0	4.5	0.2	2.0
	B 4 Fl.	0.9	6.7	0.9	6.2

Location		Relative Displacement δud (cm)	Allowable Amount of Displacement (δu) (cm)
Center Column	B 2 Fl.	0.7	28.4
	B 3 Fl.	2.0	8.0
	B 4 Fl.	0.9	22.8

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Environmental Considerations for Highway Tunnels

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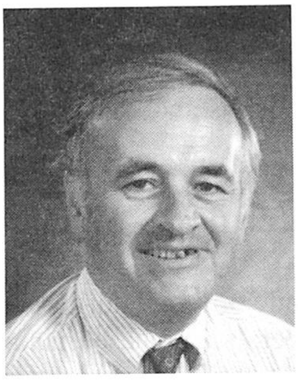
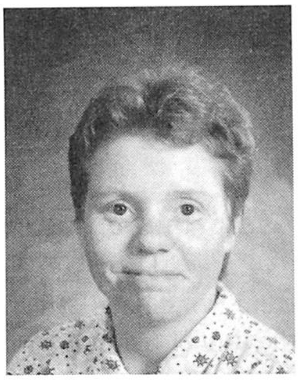
Dr Robin Andrews, a mathematician, joined TRL in 1970. He has researched into concrete buckling, earthworks stability and predicting ground movements due to tunnelling. In addition to environmental issues, his interests are tunnel operation and safety, including ventilation and fires in tunnels.



Jane CLOKE
Environmental Researcher, Transport Research Laboratory, Crowthorne, Berkshire RG45 6AU, United Kingdom.

Dr Jane Cloke, an environmental scientist, joined TRL in 1995. She has been involved in research on the impacts of road transport on the environment, particularly on airborne pollution. Also she has experience in the production of Environmental Statements in support of planning applications.



Summary

Road tunnels may be constructed in order to control environmental problems that an open road would otherwise produce. Nevertheless, there are many environmental effects generated by the construction, use and decommissioning of highway tunnels, and not all of these are beneficial. Major impacts are identified and discussed briefly. Where appropriate, predictive methods are outlined.

1. Introduction

Tunnels are increasingly perceived as a means of reducing adverse environmental impacts of roads, in both urban and rural situations. Although tunnels do have major beneficial effects when compared to open roads, there are also environmental problems specific to tunnels. Possible environmental consequences of constructing, operating and, ultimately, decommissioning tunnels are discussed.

One major problem is that almost anything can be considered as "environment", and it is difficult to know where to "draw the line". Objectors to road schemes are becoming ever more diligent, so that it is best to consider more environmental impacts rather than less in an Environmental Statement (ES) in order to anticipate as many objections as possible. In any case, it is good practice to identify all potentially significant environmental problems. The ES for any tunnel scheme will need to address the local situation. Thus, environmental considerations will be significantly different for an urban tunnel than for a tunnel in a rural location. **It must be emphasised that tunnels may be built solely for environmental reasons, so by no means all impacts are adverse.**

The purpose of this paper is to draw attention to possible environmental issues surrounding the construction, operation and decommissioning of highway tunnels. It is not intended to provide a complete list of all possible environmental impacts due to tunnels, but rather to provide a starting point for environmental assessment exercises. Some environmental effects have been the subject of recent research, and are well understood, in particular tunnel construction induced ground movements and vibration and the consequent effects on nearby buildings. There is some understanding of noise in the vicinity of the portals of operational tunnels, but the prediction methods require further development and validation. Much the same is true of the dispersion of airborne pollution. Assessment of the effects on the general community, on natural habitats and the flora and fauna, are less amenable to general civil engineering expertise, and will require specialised knowledge. It is suggested that for major tunnel projects, consideration should be given to appointing an independent environmental assessor who will continuously monitor all environmental effects during construction.

A fuller version of this paper may be found in Andrews and Cloke (1998).

2. Environmental considerations during the life of a tunnel

The environmental effects of a tunnel are significantly different during the major parts of the life of the tunnel, namely the construction phase, the operational phase (including maintenance), and the final decommissioning, or abandonment, of the tunnel, but some effects will be common to more than one era.

2.1 Construction

Many tunnel construction activities are common to any major surface construction; for example plant operation and heavy goods vehicle movements. This paper identifies activities peculiar to tunnel construction. Construction techniques for cut and cover tunnels are not dissimilar to those for retained cuttings, but nevertheless, cut and cover tunnels do have environmental impacts distinct from bored tunnels.

The following impacts might arise during tunnel construction.

a) Vibrations from subterranean construction activities, including tunnel boring machine or roadheader operations, installation of dowels and rock bolts, and blasting may cause nuisance or damage to nearby structures, including buildings and other infrastructure. In cases where vibration is thought likely to be a problem, careful selection of construction methods and the plant used might be sufficient to reduce the effects to acceptable levels. Where use of explosives is likely, trial measurements of site specific vibration transmission characteristics will provide data which can be used to determine explosion sequences and weights so that vibration levels do not exceed required limits.

b) Work inside tunnels normally requires forced ventilation and airborne pollution generated by plant and other operations may require careful treatment, as it will be released only at particular locations on the construction site. In exceptional circumstances, contaminated air will require cleaning before release. Some construction techniques release large quantities of dust or similar contaminants, notably sprayed concrete which has rebounded during New Austrian Tunnelling Method (NATM) construction.

c) Ground settlement may cause damage to nearby structures, buildings and services if not controlled. In the worst cases buildings may require demolition. In less extreme cases, they may need to be temporarily evacuated in order to effect repairs or to remove occupants from potential danger, and this could result in considerable costs, particularly if they are business premises. There exist well established, empirically based methods for predicting settlement induced by tunnel construction, especially in soft ground. Methods appropriate to individual structures, which may include nearby buildings and buried services, can then be used to predict damage levels. Ground movement problems can be reduced, or eliminated totally, by, for example, careful routing of the tunnels (including lowering them), employing construction techniques which minimise ground loss, or by the use of compensation grouting.

d) Lowering of groundwater levels may occur either because tunnel excavation opens up new drainage routes or because of dewatering activities during tunnel construction. This could have an indirect impact on surface water features and groundwater abstractions. Ecological effects could be permanent, particularly if the groundwater level remains low for a long period of time.

e) Contamination of surface waters and aquifers might occur due to the spillage of construction materials or wastes either within or outside the tunnel. This could have an indirect impact on aquatic ecology and water abstractions. A particular concern arises from the use of shotcrete in NATM tunnels because some concrete accelerators are very alkaline.

f) Compressed air working may force in-tunnel contaminants into the surrounding ground and aquifers. A particular concern occurs where a bentonite shield method is employed for spoil removal purposes. There is opportunity for contamination from both above ground and below ground activities. Air leaking from the tunnel may also disturb the bed of any overlying water feature, potentially leading to contamination of the aquatic environment. There have been unattributable reports of linoleum floor covering being disturbed in an old dwelling above a compressed air working.

g) Health and safety impacts of working in and below, potentially contaminated land must be considered. National and local legislation will determine precise details which must be observed.

h) There will be legal implications for the re-use of clean excavated material and the disposal requirements for potentially contaminated spoil. Again, national and local legislation must be observed.

i) The effects of temporary loss of land and habitat to provide work sites and access, including cut and cover construction, and immersed tube fabrication yards will require assessment. On completion of construction, there may be opportunities to return work sites to their original condition, or to develop or otherwise improve them. A good example is the immersed tube fabrication yard at Conwy in North Wales, which was used as the basis for an attractive new estuarine marina.

j) A beneficial impact of the scheme may be the (temporary) exposure of the geological resource of the area providing an opportunity for study or exploitation.

k) Abnormal occurrences, such as fire or explosion during construction could affect areas outside the tunnel. For example, toxic smoke could be released at portals and invade the neighbourhood. A major collapse during construction (eg Heathrow, Münich) could expose nearby environmental entities to risk.

2.2 Operation

During the operation of a road tunnel the following impacts may become environmentally significant.

a) Elevated air pollutant concentrations will be found within the tunnel and near the portals due to vehicle emissions within the tunnel. Acceptable in-tunnel levels will generally be governed by short term exposure limits, determined by consideration of occupational health standards (and should include the effects on vulnerable members of the community), or other more subjective criteria. Ventilation is the main tool for controlling these levels. Outside the tunnel, limits set by national or local air quality standards regulations will apply. Various methods are available for predicting dispersion of vitiated air from tunnels, including simple Gaussian models, computational fluid dynamics models and wind tunnel scale models. All have some limitations. Recirculation between adjacent tunnel bores, or ventilation stacks and tunnel portals should be avoided, for this will raise pollution concentrations both within the tunnel and at the final exhaust point. In addition, there is a trend towards limits being placed on pollutant emissions from portals and exhaust stacks.

b) Noise and vibration will be generated by vehicle movements within the tunnel and at the portals. There is anecdotal evidence to suggest that noise and vibration from tunnel fan operation may also give rise to nuisance above shallow tunnels. The sudden increase in noise levels as vehicles emerge from the tunnel may heighten noise nuisance (impulsivity).

c) An assessment of the visual impact of above ground features such as portals, ventilation shafts/towers, and control buildings must be made.

d) Long-term settlement may continue at a reduced rate through the operation stage with impacts on nearby buildings and structures.

e) Groundwater flows may be disrupted or enhanced, which could have an indirect impact on water abstractions, surface water features and aquatic/wetland ecology. There exist commonly used methods for predicting underground water movements.

f) Contaminated water collected in tunnel sumps may contain pollutants and require treatment before release.

g) A road in tunnel may require less landtake, thus reducing the visual impact of a road scheme and reducing the loss of habitat and other amenities.

h) Existing community severance may be reduced or removed by routing a busy urban road in a tunnel.

i) Abnormal occurrences, such as fire, explosion or hazardous spill could affect areas outside the tunnel. For example, an accidental toxic gas release within the tunnel could lead to a toxic cloud emerging from a portal and invading the neighbourhood. On the other hand, accident rates are generally lower in highway tunnels than on the open road.

j) Maintenance, repair and remedial activities following commissioning of a tunnel may have environmental consequences similar to some of those associated with the construction phase, in particular noise from maintenance plant operating during a night time closure could disturb the sleep of nearby inhabitants. It is not unknown for a collapse to occur during major maintenance activities.

k) Improved traffic flows in the road network arising from the use of tunnels, including small car-only tunnels at busy junctions, which is effectively grade separation, could lead to air quality improvements around the connecting road system.

2.3 Decommissioning

The environmental consequences of any activities required to close a tunnel will need to be assessed. Permanent tunnel closure is a rare event and there is little or no information available. The environmental assessment will necessarily be made on an ad hoc basis.

Once out of use, a tunnel may still have an environmental impact. If it collapses, then there could be damage to overlying structures. Water seeping into the tunnel will need to be removed, possibly by continuous pumping, to prevent flooding or contamination of groundwater. Parts of the tunnel could be invaded by vermin. On the other hand, it could be colonised by protected species, such as bats. If the tunnel is not secured, then rubbish might be dumped in or close to the tunnel giving rise to health and fire hazards. Thus a decommissioned tunnel will still require occasional inspection, and possibly ongoing maintenance for environmental reasons.

Alternatively, consideration may be given to backfilling the tunnel, a technique which has been adopted for disused mine workings. If the backfill material is inert, then the environmental impacts will be similar to those encountered during construction. However waste that is putrescible or hazardous will require special precautions and more detailed assessment.

3. Palliative measures

Good construction site practice, tunnel design and tunnel operation may be able to alleviate some of the environmental disbenefits. Reinstatement may be viable in some cases (eg Bell Common Tunnel), and re-development of the land above cut and cover tunnels in others (eg Hatfield Tunnel, Holmesdale Tunnel and Blackwall Tunnel at Greenwich in East London, above which the proposed Millennium Experience Dome will be constructed).

4. Discussion

a) It must be emphasised that locating roads in tunnels provides significant major environmental benefits, and these must be balanced against any adverse effects. Benefits include removing noise and airborne pollution from the areas through which the road passes, reducing severance, releasing land which would otherwise be lost to the road for development, amenity or natural habitats, improving traffic flows through the connecting highway network (with benefits of reduced pollution), reduction or elimination of visual impact, etc.

b) The scope of any ES will reflect the particular location of the tunnel. It should include as much as is reasonably possible, because more effects are becoming matters of legitimate environmental concern.

c) Some effects, such as ground settlement and the resulting damage to structures, are predictable with confidence. Others, such as construction vibration may be predicted, provided that preliminary measurements are taken on site. In general these effects can be reduced, or even completely eliminated, by good engineering practice, including, for example, compensation grouting, careful design and routing of the tunnel, and working only at acceptable times of the day.

d) Operational effects, particularly noise and airborne pollution, are in principle predictable. However, existing methods require some enhancement and verification before they can be applied to highway tunnels and their neighbourhoods satisfactorily. Some airborne pollution dispersion models are expensive to set up and use, especially wind tunnel scale models. Also there are some conceptual difficulties in extending their results to a full annual weather regime and to diurnal and seasonal traffic variations.

e) Drainage effects are essentially engineering considerations. Where consequences are expected to be environmentally sensitive, then a high quality site investigation will be helpful. Drainage effects may include water table draw down, which might be a temporary effect, and interference with underground water movement.

f) In areas where the ecology is likely to be an issue, a careful advance survey will be necessary and this could result in changes to the tunnel design and route, and to construction methods. Access to specialised knowledge will be essential, and may be an on-going requirement after the tunnel opens.

g) Evaluation of heritage and cultural impacts generally needs specialised expertise, which might be available from within the architectural community.

h) For major projects, consideration may be given to appointing an environmental arbitrator who is independent of the main interested parties (including the scheme promoters, designers, contractors, operators and environmental pressure groups), who is able to function on a full time basis, and who is answerable to all parties having a legitimate interest. Such an approach was successfully employed during the planning and construction of the Sydney Harbour Tunnel in Australia.

5. Acknowledgements

The research described in this paper was funded by the Civil Engineering Division of the Highways Agency of the United Kingdom Department of the Environment, Transport and the Regions (DETR). The views expressed are not necessarily those of the DETR.

6. Reference

Andrews RD & Cloke J (1998) *Environmental considerations for highway tunnels*. Transport Research Laboratory Report, to be published.

VENTILATION COMBINED WITH AIR CLEANING TECHNOLOGY FOR PARTICLES AND NO_X FOR ROAD TUNNELS

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Summary

In Norway, we have extensive experience with longitudinal ventilation of road tunnels. Long tunnels and tunnels with high traffic volume in densely populated areas, however, require new solutions.

On the national highways in Norway, approx. 700 tunnels are under traffic, amounting to a total distance of 575 km. The longest one is the Gudvanga Tunnel, situated in Sogn og Fjordane in western Norway, with a length of 11.4 km. Another tunnel of 24 km is now under construction in the same area.

1. General Information on Norwegian Tunnel Ventilation

The majority of the tunnels have the most simple form of longitudinal ventilation, namely portal to portal ventilation. Lately, however, environmental considerations have caused the construction of tunnels of a greater length, as well as many new tunnels in urban areas with a high traffic volum. If necessary, a tunnel is divided into ventilation sections by use of shafts or side adits. This makes it possible to renew the air inside the tunnel. New technology also makes it possible to clean the polluted air in sections along the length of the tunnel.

In 1989, a research programme was started in Norway to determine the possiblilty of cleaning polluted tunnel air. As a result of this, the technology for extracting particle pollution with a high extraction rate is now used. In Norway we have installed full scale equipment for particle cleaning in 6 tunnels from 1990 to 1996. All these tunnels have solutions based on the use of electrostatic filters. The main purpose with some of these installations was to reduce the emission from the tunnel portals, whereas with others the main purpose was to improve visibility inside the tunnels.

A pilot system removing NO_2 gas has been installed in the Oslo tunnel since 1992. So far, after running continuously for approximately three years the cleaning system for removing NO_2 gas seems to be very promising, with a high efficiency. We are now extending the technology also to remove NO-gas.

In the 24 km long road tunnel we will install jet fans for longitudinal ventilation without shafts. In this case, we will use new technology for cleaning polluted air in a cleaning circuit inside the tunnel, for both particles and No_x-gas.

2. Experience from use of Particle Cleaning Systems

So far, the Public Roads Administration has installed equipment for particle cleaning in five tunnels, from 1990 to 1996. All tunnels have solutions based on the use of electrostatic filters combined with mechanical filters.

At present, the following tunnels are equipped with particle cleaning plants:

- The Oslo Tunnel (installed in connection with the ventilation tower)
- The Granfoss Tunnel (bypass)
- The Ekeberg Tunnel (bypass) northbound
- The Ekeberg Tunnel (bypass) southbound
- The Hell Tunnel (installation in the ceiling)

The purpose behind installing particle cleaning systems in these tunnels varies slightly:

In the Oslo Tunnel the purpose was to reduce the particle pollution on the environment, in the tunnel's neighbourhood.

In the Granfoss Tunnel and the Ekeberg Tunnels the purpose was to reduce the emission from the tunnels as well as to improve visibility inside the tunnels.

In the Hell Tunnel, which is the only one of these tunnels with two way traffic, the purpose was to improve visibility in order to obtain better driving conditions and traffic safety.

I will present a more detailed description of the tunnels in question and describe the solutions which have been selected for particle cleaning.

2.1 Gas Cleaning

Research was started in 1992 to determine the possiblility of cleaning NO_x. A pilot plant was installed in the Oslo tunnel.

For this purpose we use a special type of activated carbon catalyst and the NO_x gas is injected with ozon and ammonia to convert the NO part of NO_x into NO_2.

The cleaning process is a combination of a catalytic reduction process and absorbtion process.

So far, results from measurements show a reduction of the NO_x concentration after cleaning of approximately 70%. After the cleaning process we cannot measure concentration of NO_2 at all. We are still working on this process, but we assume that research will be finished in 1997, and we are very optimistic regarding this solution.

2.2 The Oslo Tunnel

The Oslo Tunnel is a highway tunnel under the centre of Oslo, consisting of two tubes with six traffic lanes.

The length of each tube is approximately 1800 m. The annual average daily traffic today is approximately 70000 - 80000 vehicles/day. Both tunnels have a longitudinal ventilation system, with a vertical shaft at the end of each tube to reduce emission of polluted air from the tunnel portals.

The ventilation capacity is about 1000 m^3/s in each tube.

In connection with one of the ventilation towers, electrostatic filters have been installed to extract particles before the air is emitted through the ventilation tower. The principle is illustrated in Fig.1.

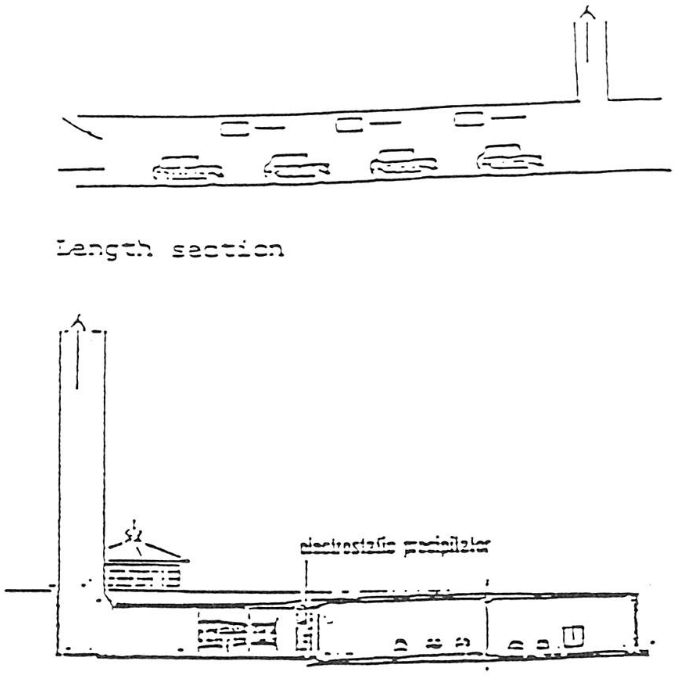

Fig. 1 Principle for installation of electrostatic filters in the Oslo Tunnel

The capacity of the air cleaning system is 600 m^3 air/s.

In order to improve the air quality in the city, polluted air from one tube is cleaned before entering the tower. Even with ventilation towers for discharge of polluted air, the spread of particles is a problem in the outside area around the towers.

Since the opening of the tunnels, the pollution has been monitored into and out of the tunnel. We have also monitored pollution intensity at different points in the city. These measurements have given information on the concentration of particles before and after the tunnels were opened. Experiences so far veryfy that the cleaning system has a positive effect on the area around the ventilation tower. The test in the Oslo tunnel shows that it is very effective to extract particles before emission through the ventilation tower. This in turn implies a clear positive effect on the environment surrounding the emission point.

2.3 The Granfoss Tunnel and the Ekeberg Tunnels

In these tunnels a bypass inside the tunnels has been selected for extraction of particles instead of a ventilation tower. This solution, where the air is cleaned inside the bypass tunnel, has considerable advantages compared to the construction of a ventilation tower. The project planning procedure is very much simplified without the need to implement a ventilation tower for emission of polluted air.

The Granfoss Tunnel is approximately 1000 m long, with an AADT of 15000 - 20000 vehicles/day.

The Ekeberg Tunnel is approximately 1500 m long with an AADT of 50000 vehicles/day.

The Granfoss Tunnel and the Ekeberg Tunnels each have two tubes with one way traffic.

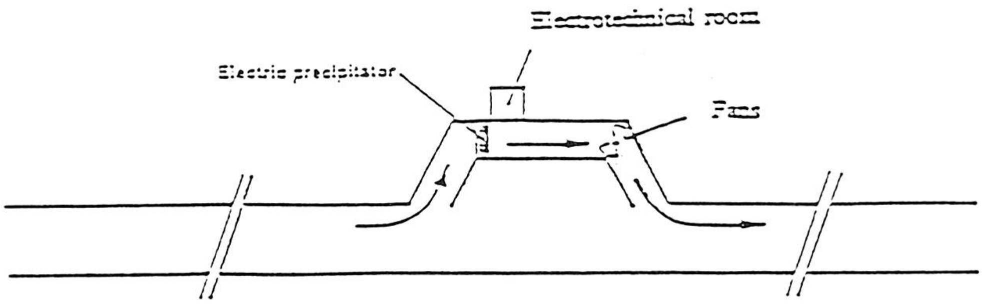

Fig. 2 The principle for a bypass solution selected in the Granfoss Tunnel and the Ekeberg Tunnels

The main purpose behind selecting this type of solution in the Granfoss Tunnel as well as in the Ekeberg Tunnels, was to achieve better visibility in the tunnels, and to reduce the pollution and emission of particles to the area surrounding the tunnel portals. In all three tunnels, the length of the bypass behind the electrostatic filter is designed to have sufficient space for the installation of a plant for clearing of nitrous gases.

2.4 The Hell Tunnel

Current research on different types of electrostatic filters shows that certain filters have a high extraction rate with an air velocity as high as 7 m/s. By increasing the air velocity through the filter, without renunciation of cleaning effectiveness, the necessary filter area to clean the same volume of polluted air can be reduced. This will reduce the construction and installation costs. Increased allowable air velocity will in some cases also make it acceptable to install the cleaning units in the tunnel ceiling, provided that the air volume to be cleaned is not to large.

The Hell Tunnel is 3880 m long, has two way traffic in one tube, and a traffic volume of approximately 10000 vehicles/day.

In this tunnel electrostatic filters were installed in the tunnel ceiling at points along the tunnel as shown in Fig. 3.

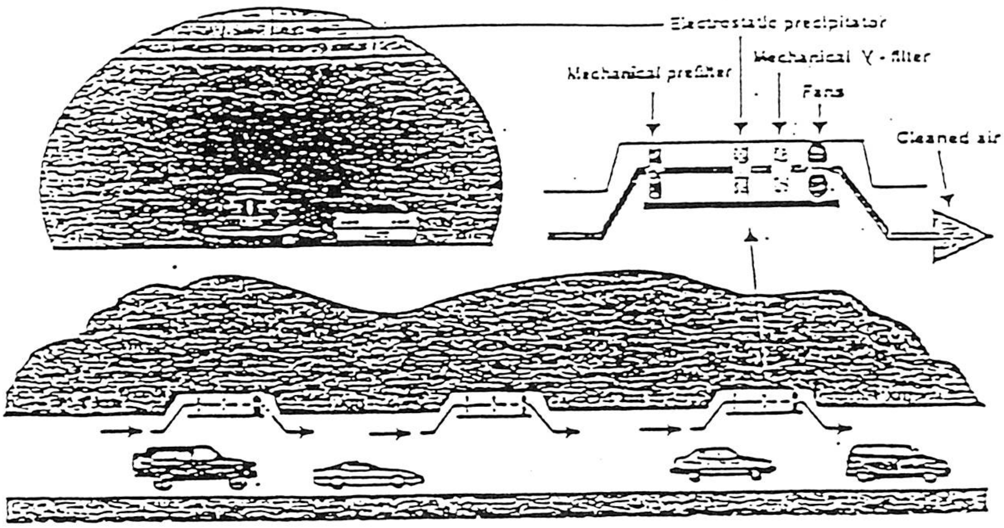

Fig. 3 Principle for installing electrostatic filters in the Hell Tunnel

Cleaning equipment is installed at three points along the tunnel. The distance between the stations is approximately 1000 m. The capacity is 100 m^3 air/s at each cleaning station. The air velocity through the electrostatic filter station is 7 m/s.

This solution has been selected with the purpose of improving the visibility inside the tunnel, but the solution will also reduce the emissions to the area surrounding the tunnel portal.

By an efficiency of 90% of the particles passing through the electrostatic filter we expect a practical extraction rate of 70% of particles in the tunnel following the cleaning station.

Measurements to decide the extraction rate by this solution will be made and completed during the winter/spring 1997.

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Managing Impacts Of Ground Movement In Urban Tunneling

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Summary

This paper reviews technical and management issues associated with construction- induced ground movement during tunnel construction in urban areas.

1.0 Introduction

This paper discusses effects caused by excavation of the Massachusetts Highway Department Central Artery tunnels in downtown Boston. Designers and contractors must tread a fine line between satisfying owners of adjacent properties and various organizations, who wish construction to proceed without any effects on the surroundings, and the construction realities of the project, in which it is impossible to build without any disturbance at all. The discussion includes:

- Methods used to predict excavation soil movement and effects on nearby structures.
- Construction methods for mitigation of soil movement.
- Issues related to management of excavation soil movement and its effects.

2.0 Causes Of Ground Movement During Excavation

Sources of ground movements beyond the limits of an excavation can be placed in two general categories:

(1.) Deformations of the excavation support system components.
(2.) Ground deformations as influenced by the response of surrounding soils and groundwater to excavation activities.

For simplicity, these categories are referred to as internal and external deformations, respectively.

Internal sources of deformation largely concern the structural response of the excavation wall system. During excavation, the wall deforms in bending between bracing points and as a cantilever above the top bracing point. The tiebacks extend in tension, or the struts deform in compression. External sources of deformation are related to the behavior of the soil mass as a whole. External sources of deformation include the overall global behavior of the soil mass and effects of consolidation.

2.1 Internal Sources of Deformation

2.1.1 Lateral Pressures.

Numerous methods have been developed for prediction of earth pressures. However, no single method can be considered precise due to the non-uniformity of the soil mass (actual versus modeled conditions) and the fact that actual earth pressures cannot be measured with adequate precision during construction. Also, the geologic conditions relating to seepage of groundwater, and the resulting magnitude of hydrostatic pressure, present similar challenges in modeling of the soil/structure behavior.

The magnitude of earth pressures is generally inversely proportional to the strength of the soil, particularly when Coulomb and Rankine parameters are utilized. The stress history also influences the magnitude of lateral earth pressure coefficients, particularly in clays.

The magnitude of earth pressure is further proportional to the amount of yielding of the wall support system. For a support system which is theoretically non-yielding, the at-rest, or K_o, parameter applies for calculation of lateral pressure. However, as the support system yields, the coefficient of lateral earth pressure approaches the active, or K_a condition. It is generally unrealistic to design a temporary support system which is non-yielding. Design for the at-rest condition is therefore most frequently applied to the design of permanent structures. However, when in fact a relatively rigid temporary support system is desired, it may become necessary to apply an earth pressure coefficient which falls between the at rest and active condition. The determination of the magnitude of such a coefficient generally requires considerable judgement.

In cases where impervious walls are installed, the hydrostatic pressures can be greater than those induced by earth pressures. The distribution of hydrostatic pressures along the length of the walls thus has considerable influence on the total forces acting on the support system. The shape of the hydrostatic pressure diagram is governed by the permeability of the support wall and seepage conditions which are mobilized as a result of dewatering within the excavation. The combination of an impermeable wall and the assumption of no seepage into the excavation represents the case of maximum hydrostatic pressures acting on the wall. The associated pressure increases linearly with depth, from the design groundwater level to the bottom of the wall. In the case where seepage is assumed, the magnitude of pressure distribution, particularly at the lower reaches of the excavation, can decrease considerably from the no seepage case. A seepage analysis can be preformed to establish the steady state pressure distribution along the length of the wall.

When lateral loads are imposed on a support system, elastic deformations of structural elements are induced. The walls move inward, resulting in a field of horizontal and vertical soil movements induced in the soil mass beyond the excavation. The initial stage of most cut-and-cover excavation usually results in a cantilever condition of the wall. Such a condition is present both prior to and after installation of the first level of bracing. In the initial state, the soil at the interim subgrade represents the cantilever reaction point. The effective length of the cantilever exceeds the depth of the excavation since the point fixity cannot occur at the excavated surface. The depth of fixity is governed by soil strength and wall stiffness. Once the first bracing member is installed, the cantilever length is effectively decreased. However, the settlement outside the excavation which occurred in the prior stage is not recovered. This is to say that during the staged excavation, the magnitudes of soil settlement are cumulative.

At subsequent stages of excavation the wall acts as a continuous beam, with bracing members behaving as reaction points. Wall deformations now become a factor, in part, of span length between the brace points. As with the cantilever condition, the condition which influences the magnitude of deformation is that which exists immediately prior to bracing installation, wherein the effective span length is greater than the vertical spacing of bracing levels. In effect, the soil deforms below the excavation line before the next brace level can be installed. This phenomenon, where the soil at subgrade deforms with the imposition of wall loads, is termed the "bulge effect".

In cut-and-cover tunneling through compressible soils, the "bulge effect" commonly represents the largest single source of deformations and is the most difficult to control. When the subgrade soil immediately below the excavation acts as a reaction for wall loads, this mass of soil is placed in compression. The wall pushes against it in a passive mode of soil loading. As a result, the soil mass deforms and inward movement of the wall results. The resulting settlements are generally non-recoverable and accumulate as excavation progresses. The condition becomes more problematic when compressible soils extend to depths beyond that of the final excavation. Measures designed to lessen these deformations may have limitations on the depths at which they remain technically and economically feasible to implement.

2.2 External Sources of Deformation

2.2.1 Consolidation Due to Seepage and Dewatering

For excavation through compressible soils, consolidation settlement can be caused by changes in piezometric head. The magnitude of consolidation settlement is primarily a function of the thickness, consolidation or recompression ratios, and stress history of the compressible strata; the magnitude and duration of groundwater drawdown, and the permeability of the drained aquifers.

Dewatering is almost unavoidable when constructing a cut-and-cover tunnel. Dewatering can be done by the installation of pumping wells outside the excavation, sumps and pumps within the excavation, or both. However, in cases where it is necessary to maintain groundwater levels beyond the excavation, external pumping wells cannot be installed.

Pressure relief, a form of dewatering, is required when hydrostatic uplift of the invert is to be prevented. Such a condition can occur when a relatively impervious soil within the limits of excavation is underlain by relatively pervious stratum. If the weight of remaining impervious layer is less than the hydrostatic force acting on the bottom of the stratum, uplift can occur. To avoid this situation, the underlying pervious layer should be penetrated by a series of pressure relief wells designed to effect depressurization. However, if the walls of the excavation do not extend into an impervious stratum below the pervious layer, the depressurization will extend beyond the limits of the excavation, thus resulting in the potential for consolidation settlement.

3.0 Methods Of Predicting Deformation

The magnitude and position of the soil movement outside the excavation is important since it directly affects structures along the right-of-way. Too much deformation or a significant dropping of the water table can lead to serious damage and safety concerns for the existing structures. This is particularly important for older historic buildings.

Methods for predicting soil movement include:

3.1 Previous Experience.
Evaluation based on results from excavation of similar conditions.

3.2 Semi-Empirical Methods.
Recent studies have attempted to correlate past experience with generalized analysis of wall and bracing systems. Clough and O'Rourke (1990) prepared a study in which movement observations from several excavations were tabulated and compared against the stiffness of the wall systems and the factor of safety against basal heave. Figure 1, taken from the paper, illustrates some results from use of the method. For excavation in soft to medium clay, the figure compares normalized maximum lateral wall movement to wall system stiffness, which is defined as:

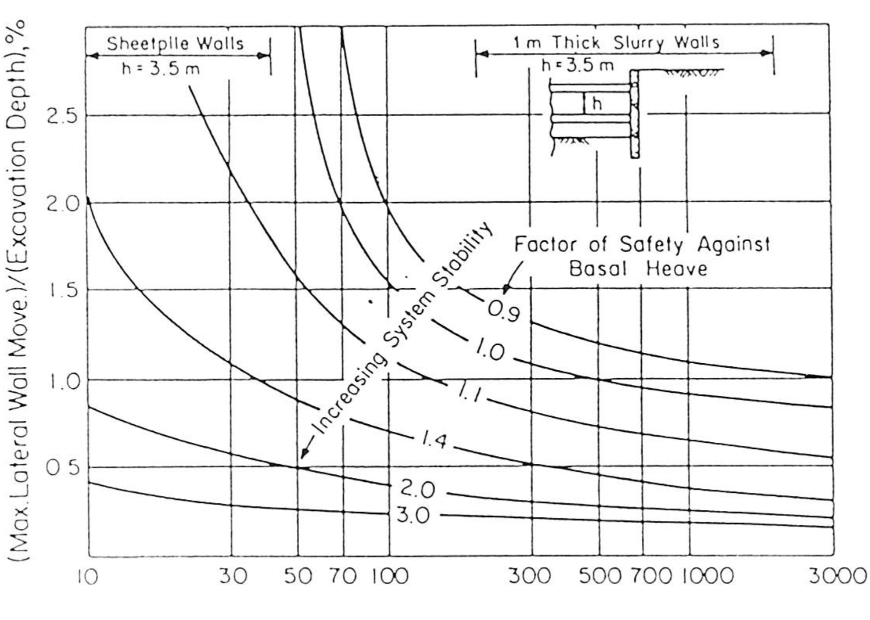
$$EI / \gamma_w h^4$$

where E is the Modulus of Elasticity of the wall, I is the moment of inertia of the wall, γ_w is the unit weight of water, and h is the average spacing between brace levels. Different curves are presented for different factors of safety against basal heave, FS, where FS is defined as:

$$N_c s / (\gamma D + p)$$

and D is the depth of excavation, γ is the density of the clay, s is the undrained shear strength of the clay at the bottom of the excavation, p is a surface surcharge, and Nc is a coefficient depending upon the dimensions of the excavation.

MOVEMENTS OF INSITU WALLS



The figure and accompanying study were prepared by correlating soil-structure interaction analyses to wall movement calculations observed from actual excavations.

3.3 Analysis for Soil-Structure Interaction.
For this analysis, the soil and structure are modeled as a mass. The mass is broken up into discrete elements and assigned elastic or inelastic properties. Using the finite element or finite difference method, stress and strain of each material in the model are related. The model predicts wall and soil movement by simulating the states of stress caused by the construction process.

Application of the program, SOILSTRUCT, serves as an example of this type of analysis. SOILSTRUCT includes available elastic models for wall and bracing elements, hyperbolic stress-strain relationships to model the behavior of the soil mass, and a capability of simulating staged construction. For this last capability, the engineer makes assumptions about the sequence of the excavation, including depth of each cut and installation of bracing. The model sequentially deactivates blocks of soil and adds bracing pre-loads. The soil mass

behind the wall reacts to the modeled construction behavior based on the constitutive material properties input into the program.

The analysis will also need to consider effects due to consolidation. Other factors to be considered include the potential for variable construction practice, such as cross lot braces not installed snugly, or preloading improperly applied.

3.4 Evaluation of existing structures.
The analysis is not complete without an evaluation of the effects of soil movement on existing structures along the right-of-way. The excavation and wall movement analysis will provide estimates for a field of soil movement behind the wall. This movement can be imposed upon models of the existing building structures to estimate effects. For example, for a particular type of building structural frame and foundation system, certain support points can be deflected based on the wall movement analysis. The resulting effects on the building structure can be quantified.

The analysis also needs to consider effects not so easily quantified, such as the condition of the existing building. A distinction can be made between "structural" damage and "architectural" damage. Structural damage due to adjacent excavation involves significant damage or failure to major structural members of the building. Architectural damage is mostly cosmetic: cracked facades, doors out of plumb, etc. Architectural damage can be more easily dealt with after construction is complete. However, for historic buildings, even the imposition of architectural damage may not be acceptable.

Boscardin and Cording (1989) prepared studies analyzing the effects of soil displacement beneath building structures. Their studies attempted to quantify various parameters associates with the soil movement, and they compared applied their method to some tunneling case histories.

4.0 Construction Methods To Reduce Soil Moment

Faced with the challenge of designing and constructing a cut-and-cover tunnel in a densely built area, the designer and contractor need to consider various methods to limit soil movement. The approaches need to consider the two major sources, wall movement and movement due to consolidation. Methods to address the problem include:

- Making the walls stiffer, deeper, or less permeable. For example, concrete slurry walls are stiffer than sheet piling or soldier piles and lagging. Soldier pile tremie concrete (SPTC) walls are stiffer than concrete slurry walls.
- Use of more or stiffer braces
- Preloading cross lot braces
- Methods to control consolidation such as curtain grouting, blanket grouting, and groundwater recharge beyond excavation support walls.
- Construction staging methods, such as limiting the longitudinal extent of excavation to take advantage of "3D" stiffness effects.
- Less conventional methods such as ground freezing
- If soil movement is still too great, underpinning existing building structures. However, underpinning must be done with great care, because the impact on the building can be greater than what would have been imposed by tunnel construction.

Geotechnical exploration and instrumentation during excavation form an important part of the overall program to manage and control construction excavation impacts. An extensive boring program provides the basis for the engineering decisions. Instrumentation for ground movement, water table impacts, and effects to existing buildings, provides checkpoints to monitor the construction and take corrective actions if needed.

Acknowledgments

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Influence of Road Enclosure Structures on Ventilation

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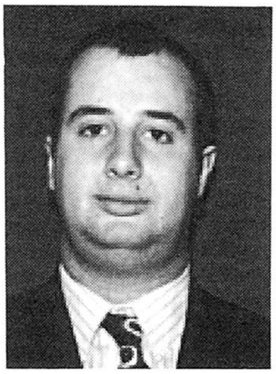

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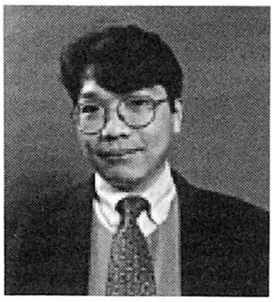

Steve Liew, born in 1957, received his BSc and PhD degrees from the University of Southampton. Steve Liew is a Chief Engineer in the Safety and Technology Division of WS Atkins. He is a Chartered Engineer and has been responsible for many tunnel ventilation and fire projects for a wide range of clients.

Summary

It is perceived that road enclosures offer a cheaper alternative to conventional tunnels as a means of environmental impact mitigation. Ventilation is required within road enclosures, for dilution of vehicle emission products and to allow control of smoke in the event of a fire incident. This paper examines the influence of road enclosure structural design on the effectiveness of the ventilation. The implications of using existing tunnel standards are also reviewed.

1. Introduction

There are already a range of measures available for mitigating the environmental impact of road schemes. Examples include the provision of environmental barriers, noise reducing surfacing, landscaping and planting. Whilst these are all effective in their own right, in areas of severe impact they may not be adequate. A road enclosure provides an alternative method of mitigation.

A road enclosure is here defined as an enclosure or covering formed over a road for the purpose of mitigating the environmental impact of the highway. The enclosure is likely to be of a lightweight form of construction, commensurate with its required acoustic performance, although it may incorporate relatively "heavy" main structural members.

A road enclosure has also been called a surface tunnel. However, the use of the word 'tunnel' is not considered appropriate, since the enclosure is likely to incorporate some or all of the following features:

- openings on the roof to allow natural ventilation.
- transparent panels to allow natural light to illuminate the road.
- access to the outside through doors in the walls.

The potential use of road enclosures does not restrict to any one road cross-section. Their application to different width roads from single carriageways to dual 3 or 4 lane motorways has been considered by designers in Europe. Varying road elevations, such as embankment, cutting, at grade, retained cutting or retained embankment, will also need to be considered in their design.

A further variable is the level of the road within the enclosure relative to the ground level outside. Where they are at the same level, the enclosure, apart from its foundations, will be fully above ground, and lightweight forms of construction can be employed. However, as the road level within the enclosure becomes lower than the adjacent ground level, the walls are likely to be of a heavier construction, since they must withstand horizontal earth pressures. Eventually, a point is reached where the walls will be fully buried and its roof will be at or just below ground level. A lightweight roof, perhaps sustaining some planting, can still be provided. This will be taken as the limiting arrangement to be considered, since any further burying of the enclosure will require a solid roof, which in effect creates a tunnel.

2. Road Enclosures in Europe

The main purpose of road enclosures in Europe is to mitigate noise pollution, but in some cases they also mitigate vehicle emissions and visual intrusion.

Road enclosures are becoming widely used in Germany where there is a very stringent daytime noise emission target. This cannot be achieved alongside motorways unless an enclosure is constructed, and as a result several are being planned. The road enclosures that have been built to date are only on more minor roads.

Examples are the Züblin type road enclosure, a partially buried concrete structure with open roof slots in Stuttgart and a proposed at grade structure with a glazed roof in Cologne, Germany. A glass roofed structure in Switzerland with cladding designed to match surrounding buildings is in operation and from France an open sided concrete framed structure has been constructed on the outskirts of Paris.

3. Ventilation

Ventilation is required for two main reasons within road enclosures;

1. dilution of pollution products from the vehicles and;
2. control of smoke in the event of a fire incident.

Enough fresh air must be supplied to the tunnel to reduce toxic exhaust products to below safe exposure limits and maintain visibility at acceptable levels. The important pollutants are CO and nitrogen oxides which are toxic, and particulates from diesel vehicles which reduce the visibility.

In a fire incident the ventilation system must be capable of controlling the smoke to allow safe evacuation of tunnel users. It should also be capable of maintaining a clear area for fire fighting operations to be undertaken.

3.1 Design Considerations

It is current tunnel ventilation practice in the UK that the longest tunnel without mechanical ventilation should be 300m and that there should be line of sight through the tunnel. The length can be extended to 400m where the traffic flow is not frequently congested. It is unclear if this maximum length is related to the requirements for pollution control or for smoke control in the event of a fire.

3.2 Pollution

The major pollutants generated by traffic in tunnels are CO, nitrogen oxides and particulates from diesel vehicles. The UK Department of Transport draft Design Guidelines for Planning, Equipping and Operating Tunnels on Motorways and Other Trunk Roads, states that the major pollutant to consider is CO. Specifically if the level of CO is below the desired limit then the levels of other pollutants will be well within safe margins. However, the visibility in the tunnel should also be considered due to the increase in the numbers of diesel vehicles in recent years.

The UK Health & Safety Executive (HSE) safety exposure limits for CO are 300 ppm for short term exposure and 50 ppm for long term exposure. The Department of Transport draft Design Guidelines for Planning, Equipping and Operating Tunnels on Motorways and Other Trunk Roads suggests that a 250 ppm limit on CO is sufficient in most British tunnels as traffic is in the tunnel for less than 2 minutes and 250 ppm is significantly less than the HSE short term exposure. However, if the traffic is likely to stop in the tunnel, and this includes in the even of an incident which stops the traffic, the limit must be reduced. The PIARC document from 1987 suggests that 100 ppm CO be used as the design limit for tunnels where the traffic may be expected to be stationary.

3.3 Smoke Control

In the event of a fire incident the smoke from the fire should be controlled to allow safety evacuation of tunnel users. In a conventional tunnel, smoke is generally forced in one direction to keep the upwind direction clear for evacuation. Provided a method of maintaining a clear escape route can be found, however, it is not essential to have the smoke forced into one direction. It is possible using fully transverse ventilation to extract the smoke at source. It may also be possible to allow smoke egress with natural ventilation openings close to the fire source utilising the natural buoyancy of the smoke.

3.4 Running and Maintenance Costs

In road enclosures, a design requirement which strongly influences the choice of ventilation system is that the running and maintenance costs of the road enclosure should be kept as low as possible. This leads towards there being no mechanical ventilation in road enclosures and therefore natural ventilation schemes are preferred.

4. Natural Ventilation

Natural ventilation relies upon the movement of air by moving traffic or on the buoyancy of pollutants and combustion products to ventilate the tunnel. To achieve sufficient natural ventilation a number of ventilation arrangements can be proposed. The key advantage of natural ventilation through louvred openings is to obviate the need for mechanical ventilation, so reducing operating and maintenance costs compared with a conventional tunnel. However, a disadvantage of louvred openings is that they allow the escape of some noise and pollution. Also, louvred openings allow the escape of smoke in the event of a fire when compared with a conventional tunnel. However, risk of flammable spills inside the enclosure could lead to explosions and fire.

4.1 Parallel Slots

Construction of slots parallel to the longitudinal axis of the tunnel, illustrated in figure 1, would allow air exchange, between the tunnel air and the open environment, along the length of the tunnel. In the moving traffic case the pressure generated by the flowing traffic would induce air exchange through the slots, thus diluting the pollutants.

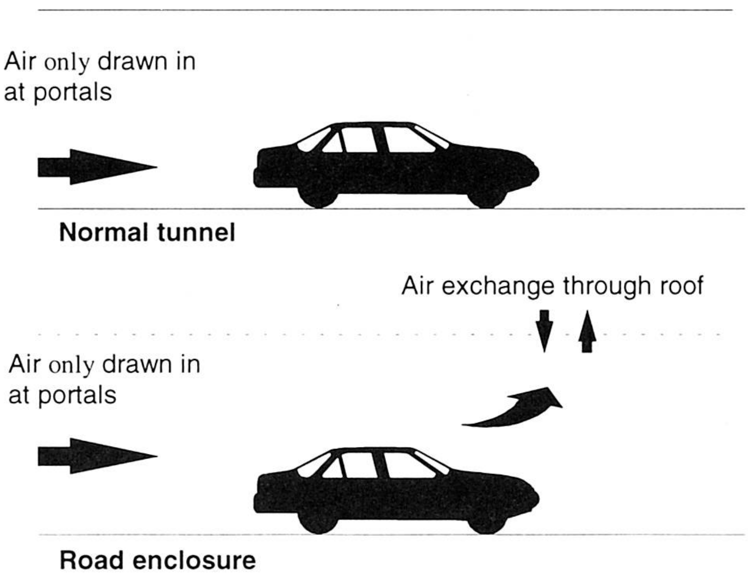

Fig 1 Air exchange through slots parallel to the longitudinal axis of the tunnel

In the event of stationary traffic the buoyancy of the pollutants would drive the pollutants through the slots. A parallel slot system has been employed in Germany. It has been demonstrated from experiments in the German tunnel, which involved stationary traffic and the portals being blocked, that the pollution levels did not become a problem, suggesting that this arrangement works sufficiently well for stationary traffic.

In the event of a fire the system would work in a similar manner as a natural smoke ventilation scheme used in large buildings. Installation of downstands (smoke curtains) in the tunnel roof would contain the smoke above a defined area, reducing the hazard range, and the natural buoyancy of the smoke would keep a clear area below for escape. The smoke would be retained within the reservoir, between the downstands, as a substantial amount would flow through the slots. The slot size should be between 3% and 15% of the roof area as used within large buildings. The slots must also be at the highest point in the surface tunnel to enable the smoke to flow out.

4.2 Perpendicular Openings

Another possible arrangement is to provide openings perpendicular to the longitudinal direction of the tunnel, illustrated in figure 2. By including contractions and expansions at the openings pressure could be induced by moving traffic which would drive flow through the openings.

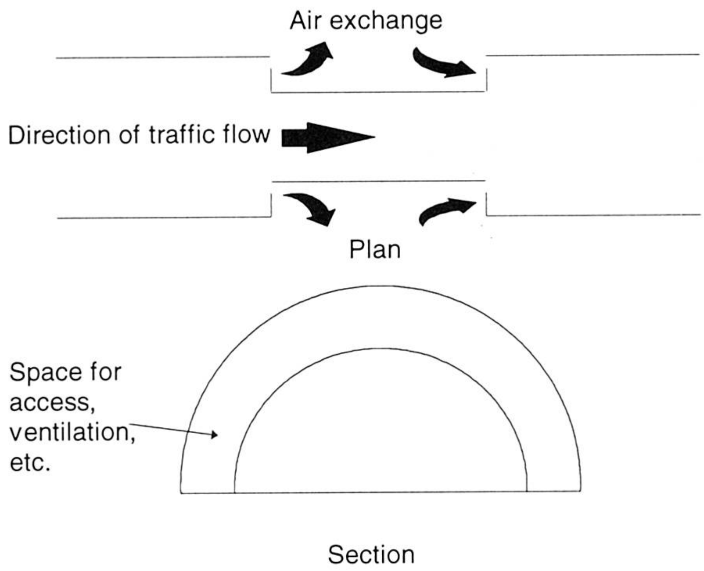

Fig 2 Air exchange through slots perpendicular to the longitudinal axis of the tunnel

In stationary traffic this scheme would not work as well as parallel slots because the openings are not arranged efficiently for the low buoyancy exhaust gases. Also, as the openings are at discrete locations the pollution must travel some distance to them which will allow it to cool and lose buoyancy. Therefore it is likely that this scheme would be suitable for use where stationary traffic would not be frequent except in an incident. Other measures could be used to ensure that in stationary traffic engines were switched off to reduce pollution levels.

In a fire incident the sections between contractions and expansions could be utilised as smoke reservoirs to collect the smoke and allow the smoke to escape through the associated openings.

4.3 Maximum Road Enclosure Length

Using the schemes outlined above it should be possible to extend the length of the road enclosure without forced ventilation from the usually accepted 300m. This extended length has been investigated using simple tunnel ventilation models. However, as the origin of the 300m maximum length is unknown the calculation of the possible extension is difficult.

Pollution levels in tunnels depend upon the traffic flow and the tunnel geometry. Calculations performed for a congested traffic flow situation in a hypothetical motorway tunnel demonstrated that with no ventilation the pollutant levels did not exceed the design criterion of 100 ppm of CO for tunnel lengths up to 900m. By allowing 5% of the roof open, as slots parallel to the longitudinal direction of the tunnel, it seemed possible that the tunnel length could be doubled. However, using the simple modelling techniques available for this study the extension in length is not certain. In particular using this simple model there is an apparent exit portal effect which increases the pollutant concentrations close to the exit portal. Further investigation of the flow regime in the exit portal area would need to be carried out for specific designs.

The finding that the pollutants did not exceed the limit for tunnel lengths up to 900m implies that a conventional tunnel, with a construction the same as that modelled, could be extended to 900m, if pollution is the only consideration. Similarly, by allowing 5% of the roof to be open in the form of a continuous slot, a road enclosure could be built to a length of 1800m, assuming the same arrangement as the hypothetical case.

The results of tunnel extension outlined above only cover the specific example modelled, i.e. a hypothetical motorway tunnel with congested traffic flow. It should be noted that other arrangements not considered may give differing results. It is clear therefore that due to the nature of road enclosure design, each road enclosure should be considered on its own merits.

5. Conclusions

The main findings of the study into the ventilation of proposed road enclosures tunnels are:

- Natural ventilation appears to be a viable option for road enclosures.

- The use of a prescriptive maximum length may not be helpful in the innovative design of the road enclosure solution. Demonstration of the ability of the ventilation system, under expected operating conditions, to maintain pollution levels below acceptable limits and control smoke in the event of a fire may be a better approach to innovative design.

- Consideration should be given to each road enclosure using advanced ventilation modelling techniques which can take account of the complex geometry including slot configuration, such as scaled model experiments or computational fluid dynamics computer modelling, to determine ventilation performance.

- In the event of a fire it is acceptable practice within large buildings to use natural ventilation for smoke control in association with smoke curtains and reservoirs. Due to the nature of road enclosures with stationary traffic it is possible to consider best practice employed in large building design as a guide to the control of smoke in road enclosures.

Hazard and Risk Analysis in the Design and Construction of Tunnels in Carbonate Rock Mass of the Adriatic Area

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Summary

An extensive data base has been formed during the design and construction of six tunnels in Croatia (Hrasten, Tuhobić, Vrata, Sljeme, Sopač and Vršek) whose total length amounts to approx. 5 km. The tunnels were excavated in rock carbonate formations formed of limestones (dating back to the Lias and Dogger) and, less often, dolomitic limestones from the Jurassic period and dolomites from the Upper Triassic period [Garašić, 1995].

Taking into account all elements specified in the paper, it is possible to pinpoint all significant factors that had to be dealt with when defining hazards and specific risks occurring during design and construction of tunnels in carbonate rock formations of the Adriatic coastal area.

1. Introduction

The paper starts with the generally accepted definition that the *natural hazard* is the probability of occurrence of a potentially harmful phenomenon (event) in a particular area and within a defined space frame, while *risk* is an expected level of loss (loss of human life, damage to materials) due to occurrence of a hazard.

An extensive data base has been formed during the design and construction of six tunnels in Croatia (Hrasten, Tuhobić, Vrata, Sljeme, Sopoč and Vršek) whose total length amounts to approx. 5 km. The tunnels were excavated in rock carbonate formations formed of limestones (dating back to the Lias and Dogger) and, less often, dolomitic limestones from the Jurassic period and dolomites from the Upper Triassic period [Garašić, 1995].

Caverns encountered in the studied tunnels are mostly located in fault zones, or next to fault paraclases, or in top parts of anticlines, or in zones characterized by frequent occurrence of bedding joints. Out of 58 caverns explored during excavation of these tunnels, 90% are of vertical type (pits) [Garašić, 1995].

A regularity in cavern occurrence was observed within relatively sound rock categories (II and III) as well as within worse rock categories (IV and V) next to fault zones filled with clayey material and some rock fragments, locally with the presence of ground water.

Investigations performed so far in these tunnels have shown that there are 58 caverns corresponding to 3 to 18 percent of the total tunnel length. The greatest cavern depth is 126 m and an average depth of other caverns ranges from 25 and 35 m, while their length varies from 15 to 25 meters [Garašić, 1995].

The preparation of final designs for these tunnels was preceded by appropriate engineering geological surveys, trial boring, geophysical and geotechnical tasting, all that with the purpose of preparing an engineering geological and geotechnical profile or model. Based on these investigations, experts proceeded to classification and categorization (K_D) of rock mass along the tunnel axis, to the level of detail required for the tunnel design preparation.

During tunnel excavation according to NATM method (tunnel driving in two stages), a detailed engineering geological mapping and a limited geotechnical testing in laboratory and in situ was conducted for the purpose of rock mass classification and categorization (K_{is}).

Based on appropriate analyses, the following relationship was established :

$$\boxed{K_{is} = a \cdot K_D^{\,b}} \qquad (1)$$

It was determined that, in addition to lithogenetic properties of rock mass, the value of K_{is} is also influenced by discontinuities (which can not easily be taken into account by any classification), rock mass fragmentation (which is very hard to define at the stage of preliminary investigations), technology used in excavation and primary support work (whose influence on classification and categorization can not readily be estimated).

This paper also analyzes the existing classifications (**RMR** and **Q**) as well as the new **JAK** classification (earlier known as "**n**" classification) which was developed during the study of carbonate rock formations in the coastal area of the Adriatic [Jašarević, Kovačević, 1996].

Taking into account all elements specified in the introductory part of the paper, it is possible to pinpoint all significant factors that had to be dealt with when defining hazards and specific risks occurring during design and construction of tunnels in carbonate rock formations of the Adriatic coastal area (Table 1).

2. Analysis of suitability and applicability of rock mass classifications

Very extensive hazard and risk investigations were undertaken in carbonate rock formations in which the studied tunnels are situated, for the purpose of designing and building two concrete arch dams each about 50 m in height [Jašarević et al, 1997]. These investigations consisted of engineering geological, geophysical and geotechnical (laboratory and in situ) surveys, and included realization of a number of boreholes and six prospection galleries each about 50 meters in length. On the basis of results obtained during these investigations, classifications were made according to "**RMR**", "**Q**" and "**JAK**" methods. The classification results are given in Figure 1.

The following relationships [Jašarević, Kovačević, 1996] were taken into account in the preceding Figure:

$$\boxed{RMR = 9 \ln Q + 44} \qquad (2)$$

$$\boxed{RMR = 110 - 20\,JAK} \qquad (3)$$

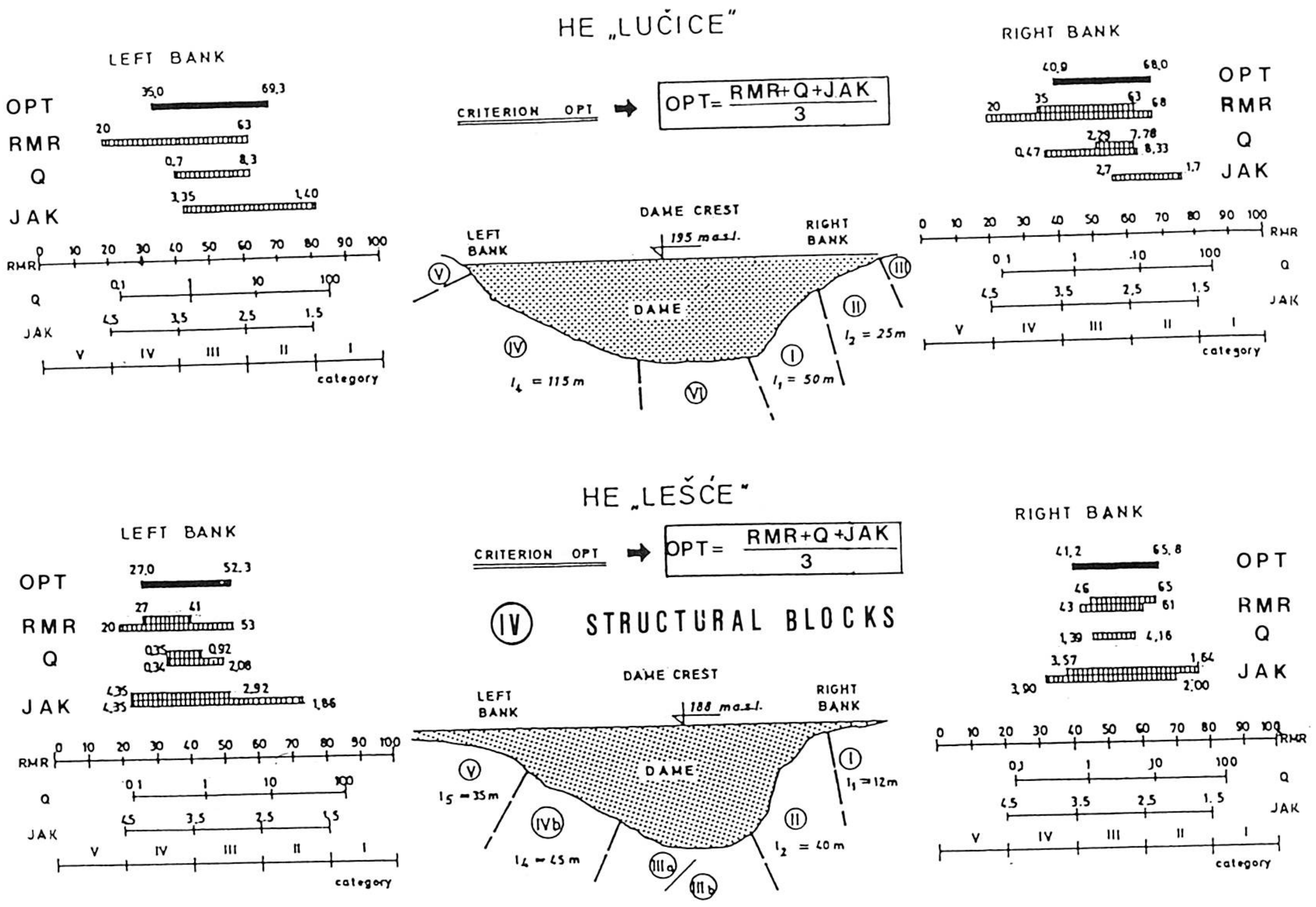


*Figure 1 Results of "**RMR**", "**Q**" and "**JAK**" classifications for hydroelectric power plants "Lučice" and "Lešće"*

The following conclusions can be made after analyzing **RMR**, **Q** and **JAK** values (which are partly presented in Figure 1) :

- maximum values obtained by **JAK** classification are up to 15% higher than the "opt" value
- minimum values obtained by **RMR** classification are up to 25% lower than the "opt" value
- classification provides a more restricted range, both in the minimum and maximum, when compared to the "opt" value.

In addition, the following was established through analyses conducted on these dams:

- in the maximum range (best categories), the (RMR+Q)/2 is lower by 8% than the (RMR+Q+JAK)/3,
- in the minimum range (worst categories), the (RMR+Q)/2 is lower (i.e. it provides a lower category) by 6% with respect to (RMR+Q+JAK)/3.

*This analysis leads to the conclusion that it would be advisable to conduct classifications according to all three procedures ("**RMR**", "**Q**" and "**JAK**") and to calculate an average value for maximum and minimum values* (marked in Figure 1 as "opt").

3. Analysis of engineering geological and geotechnical parameters indispensable in tunnel design

Many earlier studies as well as those undertaken in recent times [Einstein, 1993], [Yufin, 1993], point to the necessity of applying usual and standard procedures [ISRM, Suggested Methods, 1981] when performing engineering geological and especially geotechnical investigations.

When assessing rock mass "behavior" Einstein emphasizes the importance of knowing the position of joints and joint systems. Stochastic modeling of joints enables presentation of data about geometry of discontinuities as well as a *more reliable formulation* of engineering geological models and their insertion into reliability models. When arriving at the final conclusion, this author introduces the *risk analysis* and points to the permanent problem of measurement errors in data collection, namely :

- inadequate geological models (three-dimensional scale models based on one-dimensional and two-dimensional information)
- unsatisfactory approximation due to the lack of knowledge about mechanical effect of rock bridges (break in the continuity of joints), their deformation and fracturing
- formulation of engineering geological and geological model based on an acceptable risk

In case of underground structures, it is extremely significant - more than in any other structures - to identify and analyze risk dependent on natural conditions, i.e. on geological structure and geotechnical properties, according to the following expression :

$$\boxed{R_u = R_c = R_{GE} \cdot R_{GT}} \tag{4}$$

where

R_u - total risk dependent on natural conditions,
R_c - classification risk,
R_{GE} - geological risk (probability of occurrence of fault zones, caverns, caves, etc.)
R_{GT} - geotechnical risk (probability of occurrence of specific geotechnical properties)

As the procedures currently used for the classification and categorization of rock mass along an underground structure ("**RMR**", "**Q**" and "**JAK**") take into account engineering geological conditions and geotechnical properties, it can be stated that classification risk (R_c) is best expressed with the general equation presented under (4) above.

Based on experience gained in Germany and Austria, M. John emphasizes in his paper [M. John, 1997: Sharing of risks under changed ground conditions in design/build contracts] the significance of the following elements from the owner/contractor agreement:

1. Distribution of Excavation Classes: The geotechnical risk is borne by the owner.
2. Dimensioning of the Primary Support: Provisions are made that permit the quantitative and qualitative adjustment of the primary support (cf. Introduction, item 3).
3. On-Site Adjustments: Since the risk of changed ground conditions is borne by the owner, it is the owner who refines the design by adjusting it to the conditions actually encountered.

4. Recommendation regarding methodology for hazard and risk evaluation during tunnel construction in carbonate rock formations

The hazard and risk evaluation and categorization was performed using data base created from 1990 to 1996 during design and construction of six tunnels (total length: about 5 km) situated in carbonate rock formations of the coastal region of the Adriatic.

The hazard and risk was evaluated by appropriate use of IT (information technology). At that, individual terms were associated with quantified values based on the methodology for evaluating hazard and risk of landslides [Fell, 1993].

The study of engineering geological elements (bedding plane, discontinuities, speleological structures, occurrence of water, etc.), as well as geotechnical testing in laboratory and in situ (PLT, RQD, axial strength, geophysical surveys - SASW, weathering) served as the basis for performing the technical rock mass classification according to methods "**RMR**"and "**Q**".

Based on the analysis of the data base and engineering classifications, the following parameters were established: magnitude (M), probability of occurrence (P), hazard (H), vulnerability (V) and specific risk [R_S], as shown in Table 1.

5. Practical application of hazard and risk evaluation

As emphasized in section 2, in order to increase the level of reliability it is advisable to perform the engineering classification based on all three procedures ("**RMR**", "**Q**" and "**JAK**") and then to calculate an average value "opt" for both maximum and minimum values.

If only one engineering classification is performed according to equations (2) and (3), then the "**JAK**" value is calculated as shown in Table 1, because it also represents the magnitude M. The subsequent procedure is presented in Table 1.

The engineering classification (K_D) according to "Q" and "RMR" procedures and the K_{is} classification (rock classification during tunneling) were performed for the Vršek tunnel, as presented in Figure 2. The same figure shows estimations of hazard and specific risk along the tunnel axis.

Based on the detailed monitoring of tunneling works along the Rijeka - Karlovac highway route, it was established that the advance rate is 12 m/day depending on the rock-mass category, for round the clock work (24 hours a day) and 350 working days in a year [Brnčić, 1995], [Balen, 1995] and [Garašić, 1995]. The tunneling technology consisted in tunnel profile excavation in two stages with primary support.

Further to investigations [Jašarević, 1996] conducted in 1995 and 1996 for Rijeka - Karlovac motorway tunnels built in carbonate massif (limestones, dolomites, and dolomitic limestones) in order to determine rationality of excavation and primary support, the following correlation (Fig. 3) was established:

$$\boxed{K_{is} = a \cdot K_D^{\,b}} \tag{5}$$

where: K_D ... rock category determined on the basis of investigations for final design.
K_{is} ... rock category determined during tunneling works.

Classif. proced.	"RMR	0 20	40	60	80	100
	"Q" RMR=9lnQ + 44	0.1	0.7	6.6	54.6	503.8
	"JAK" JAK=(110-RMR)/20	4.5	3.5	2.5	1.5	
Rock Mass Category		Very poor V	Poor IV	Fair III	Good II	Very good I
Magnitude [**M**]		> 4.5	3.5 - 4.5	2.5 - 3.5	1.5 - 2.5	< 1.5
Claft Area - Fault zone [m^3]		Very large > 10000	Large 1000-10000	Medium 100-1000	Small 10-100	Very small < 10
Proba bility	Descr.	Extrem. high	High	Medium	Low	Very low
	[**P**]	8 **6.5**	5 **4**	3 **2.5**	2 **1.5**	1
	Annual prob. [$\mathbf{P_{an}}$]	**1**	**0.1**	**0.01**	**0.001**	
Hazard [**H**]= [**M**]·[**P**]		**29.25**	**14**	**6.25**	**2.25**	
		> 29..25	14 - 29.25	6.25 - 14	2.25 - 6.25	< 2..25
Vulnerability [**V**]		**0.90**	**0.50**	**0.10**	**0.05**	
Spec. risk	[$\mathbf{R_s}$]=[$\mathbf{P_{an}}$]·[**V**]	**0.10**	**0.05**	**0.001**	**0.000005**	
	Border values [$\mathbf{R_s}$]	> 10^{-1} **10^{-1}**	**$5 \cdot 10^{-2}$**	**10^{-3}**	**10^{-5}**	< $5 \cdot 10^{-5}$
	Descr.	Extrem. high	High	Medium	Low	Wery low

Table 1 Risk and hazard evaluation for underground structures built in carbonate rock formations

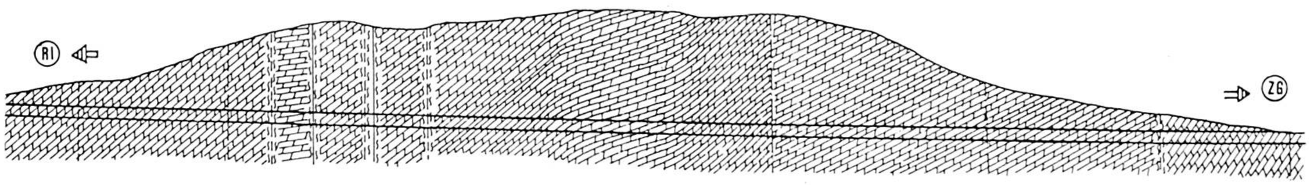
Hazard [H] is a danger affecting humans and material goods.
Risk [R] is an evaluated level of danger from a particular hazard
Vulnerability [V] is an evaluated loss of stability at the excavation contour or primary lining, and it may range from the total loss of stability (failure - cave-in) $V \geq 0.9$ to the very low vulnerability $V \leq 0{,}05$.

By analyzing this diagram (Fig. 3), we can note an increase in K_{is} category as related to K_D which is probably due to scale effect, i.e. to insufficient massif investigations at the stage prior to final design. Based on information gathered through on-site surveys presented in [Brčić, 1995], the following correlation between the advance rate (V_n) and rock-mass category (K_{is}) was established:

$$\boxed{V_n = a + b \cdot K_{is}} \qquad (6)$$

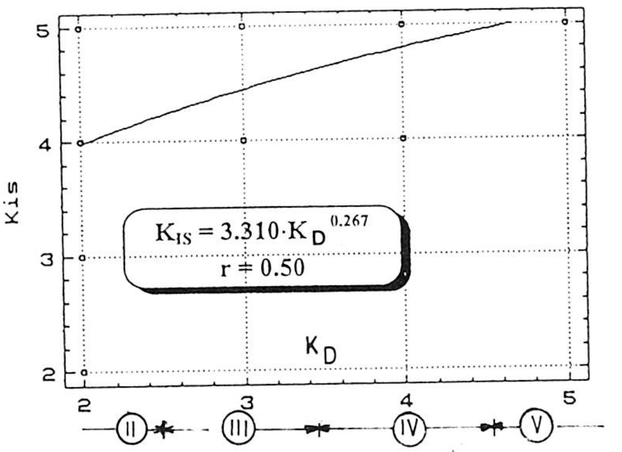
This correlation is presented in Figure 4.

TUNNEL "VRŠEK"
l = 867 m



CHAINAGE	980.00	990.00	045.00	135.00	190.00	200.00	225.00	240.00	320.00	365.00	530.00	665.00	730.00	750.00	847.97
			47+000	+100		+200			+300		+400 +500	+600	+700		+800
TERRAIN LEVEL	823.58														802.41
GRADE LEVEL	810.10														795.14
ENG. – GEOLOGICAL CLASSIFICATION [K_D]: RMR_D			23	42	54	23-8	54	23-8	44	54	78	63	23		
ROCK MASS CATEGORY [K_D]		V	IV	III	III	IV-V 60/40%	III	IV-V 60/40 %	III	III	II	II	IV	V	
Q_D		0.139	0.28	1.25	3.89	0.28 -0.02	3.89	0.28 -0.02	1.25	3.89	14.2	7.78	0.28	0.139	
CATEGORY OF SUPPOPT		31/31	31/27	23/22	22/17	31/27 35/35	22/17	31/27 35/35	23/22	22/17	14	18/13	31/27	31/31	
CATEGORY "IN SITU" [K_{IS}]		V	IV V IV	V V V	V V IV V	V	V	V	V IV V	IV V	IV II IV V IV III	IV V IV	III IV V		V V V
HAZARD (ALLORDING TO TABLE 1) [H]	$>10^{-1}$	>29	≈ 26	≈ 14	≈ 7	26-23	≈ 7	26-23	≈ 13	≈ 7	≈ 24	≈ 6	≈ 26	> 29	> 29
SPECIFIC RISK [R_S]			$\approx 10^{-1}$	$\approx 5 \cdot 10^{-2}$	$\approx 10^{-3}$	$\approx 10^{-1}$	$\approx 10^{-3}$	$\approx 10^{-1}$	$\approx 5 \cdot 10^{-2}$	$\approx 10^{-3}$	$\approx 5 \cdot 10^{-2}$	$\approx 10^{-3}$	$\approx 10^{-1}$	$>10^{-1}$	$> 10^{-1}$

Figure 2 Estimated longitudinal profile of the Vršek tunnel with separate presentation of hazard and specific risk



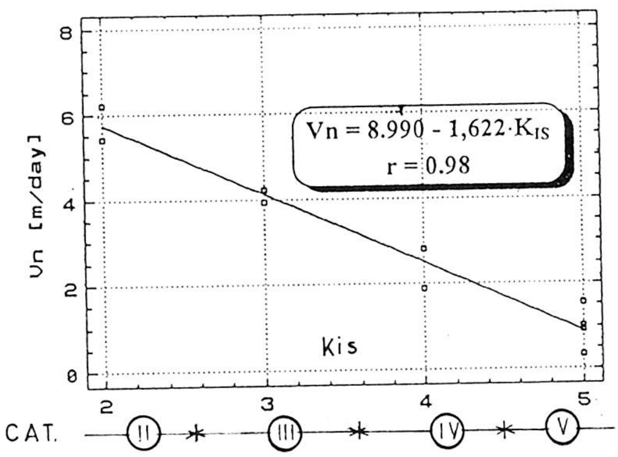


Figure 3. Correlation between the category determined in situ and the one anticipated at the design stage

Figure 4. Correlation between the advance rate (Vn) and the rock mass category (K_{IS})

6. Conclusion

The following conclusions can be derived from analyses focusing on the evaluation of hazard and risk levels during tunnel construction in carbonate rock formations :

1. The proposed methodology for the hazard and risk evaluation during design and construction of tunnels is based on the methodology used for evaluating hazards and risks concerning landslides [Fell, 1993]. The proposed methodology should therefore be checked on a number of underground structures in carbonate rock formations.

2. The proposed methodology for evaluating hazards and specific risks along the tunnel axis enables builders to keep funding allocated for tunnel construction within the planned limits, while also helping them to avoid unwanted extensions in construction time.
3. In addition, for sections presenting an increased level of specific risk, the methodology is conceived in such a way that the contractor is warned to pay a special attention during excavation and selection of an appropriate primary support.
4. Based on the analyses performed in the scope of this research, it may be concluded that the increase in hazard between neighboring categories results in an exponential (10^{-n}) increase in specific risk and hence in similar increase in the cost of construction (Fig. 4).
5. It may finally be concluded that the prognostic longitudinal profile with a rock category estimate (K_D) - which is usually regarded as a basic technical document for the procurement of tunnel excavation work - is now complemented by hazard and risk evaluation by individual sections. This enables a more precise distribution of responsibilities between the client and tunneling work contractor.

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