FEIAP Engineer of the Year Award 2008

CITATION FOR ENGR. DR. MOH ZA CHIEH

Dr. Moh received his B.S.C.E. from the National Taiwan University, M.S. degree from the Iowa State University and D.Sc. from MIT. Upon graduation, Dr. Moh taught at Yale University in the USA and later was Professor, Vice President & Provost at the Asian Institute of Technology in Thailand. During his tenure in AIT, he was responsible for the development of geotechnical engineering at the university and in the region and the overall academic development of the Institute. Today, AIT graduates are in most of the FEIAP member countries. Many of them are working in important positions, including academic institutions, governmental agencies and industries.

In 1975, he left the academic world and co-founded with his brother MAA Group Consulting Engineers, in Taiwan and Singapore. Today, MAA is internationally recognized and has grown into a reputable multidisciplinary engineering consulting firm in the East and Southeast Asian region. MAA currently has more than 700 people with companies in Taipei, Hong Kong, Beijing, Shanghai, Bangkok, Penang and Singapore. Under Dr. Moh's leadership, MAA has made significant contributions to the technological development in the region, including the ground improvement of the Survarnabhumi Airport in Thailand, civil works of Taiwan High Speed Rail (the largest BOT project in the world), Geotechnical Engineering Specialty Consultant for the Taipei Rapid Transit Systems; numerous tall buildings and deep excavations in Taiwan, Hong Kong, Singapore, Malaysia, Thailand and Indonesia. MAA staffs have published more than 450 technical papers worldwide to promote the combination of research and practice. Dr. Moh himself has authored or co-authored over 140 papers.

Being a Fellow in many learned societies, including ASCE, ICE, HKIE, IES and IEM, Dr. Moh is very active in professional institutions. He is the founding President of the Southeast Asian Geotechnical Society (1967-72), which helped the development of that important but not well recognized field in civil engineering in many developing countries in the region. He was also Vice President for Asia of the International Society for Soil Mechanics & Geotechnical Engineering (1973-77), President of the Chinese Union of Professional Civil Engineers Associations (1992-95), and served on many Board of Directors. He is the first and current Chairman of the Chinese Taipei APEC Engineer Monitoring Committee, and Chairman of the Chinese Taipei EMF Engineer Monitoring Committee (Provincial).

Dr. Moh has received many honors including Honorary Doctor of Technology from the Asian Institute of Technology in 1999, Honorary Member of the Road Engineering Association of Asia and Australasia in 2000, Honorary Member of the Japanese Geotechnical Society in 2003 and listed in many international directory, including Marquis Who's Who in the world, Who's Who in Science and Engineering etc.

FEIAP Engineer of the Year Award

Nomination Form 2008

Nominee: Dr. Za-Chieh Moh

Nominated by: FEIAP Member Economy Chinese Taipei

SECTION 1
PERSONAL

1. Nominee's Full Name: Za-Chieh MOH
2. Office Address: Moh and Associates, Inc.
   11th Floor, No. 3, Tunhwa South Road, Section 1,
   Taipei 10557, Taiwan, R.O.C.
   Tel: (886-2) 2579-8731
   Fax: (886-2) 2579-8635
3. Email Address: zachieh.moh@maaconsultants.com
4. Date of Birth: 19 August 1931
5. Place of Birth: Shanghai, China
6. Sex: Male
7. Nationality: Taiwan, R.O.C.
8. Marital Status: Married
9. Passport No: 133400067

SECTION 2
EDUCATION & TRAINING

10. Highest Engineering Educational Attainment:
    Doctor of Science, Civil Engineering

11. University / college:
    Massachusetts Institute of Technology, U.S.A., 1961

12. Published works / technical papers given:
    Over 140 technical papers have been published. Complete list is shown in Appendix A.
    Ten papers are selected with titles shown below. Copies of the complete papers are included in Appendix B.

Moh, Z.C. and Hwang, R.N. (1997), Geotechnical Problems Related to Design and Construction of the Taipei Rapid Transit Systems, *Proc. Chin Fung Kee Lectures,* Institution of Engineers Malaysia, Kuala Lumpur.

Moh, Z.C., Hsiung, K.I., Huang, P.C. and Hwang, R.N (1999), Underpass Beneath Taipei International Airport, *Proc. Conference in New Frontiers and Challenges,* Bangkok.

Moh, Z.C. (2001), Geotechnical Engineering in Infrastructure Development, *2001 Milton E. Bender Lecture, 28 June, Geotechnical Engineering Journal*, Southeast Asian Geotechnical Society, Vol.32, No.2

Moh, Z.C., Hwang, R.N., Ueng, T.S. and Lin, M.L. (2002), 1999 Chi Chi Earthquake of Taiwan, *Invited Lecture, Proc. 17th Australasian Conference on the Mechanics of Structures and Materials*, Gold Coast Queensland, Australia, pp.3-10.

Moh, Z.C. and Lin, P.C. (2003), From Cobra Swamp to International Airport：Ground Improvement at Suvarnabhumi International Airport, Thailand, *Ground Improvement Journal*, Vol. 7, No.2, pp.87-102

Moh, Z.C. (2003), Geotechnical Issues in Design and Construction of Viaducts of the Taiwan High Speed Rail, Keynote Lecture, *Proc. 12th Asian Regional Conference on Soil Mechanics and Geotechnical Engineering*, Singapore, Vol. 2, pp. 1097-1107

Moh, Z.C. (2004), Site Investigation and Geotechnical Failures, Keynote Lecture, *Proc., International Conference on Structural and Foundation Failures,* Singapore, IES and IStructE Singapore, pp. 58-71

Moh, Z.C. and Hwang, R.N. (2004), Risk Assessment for Subway Constructions, *Proc. Project Management Conference*, Hong Kong, China.

Moh, Z.C. and Yao, D.T.C. (2005), Natural Disasters in Taiwan, Keynote Lecture, *Proc. International Conference on Geotechnical Engineering for Disaster mitigation and Rehabilitation*, Singapore, pp.23-45.

Moh, Z.C. and Hwang, R.N. (2007), Lessons Learned from Recent MRT Construction Failures in Asia Pacific, Opening Keynote Address, *Proc. 16th Southeast Asian Geotechnical Conference*, Kuala Lumpur Vol.1, pp.3-20.

SECTION 3
AFFILIATIONS

13. Professional Organisations:

Name of Affiliations	Positions Held	Date (Year)
Southeast Asian Geotechnical Society	President Council Member	1967-1972 1973-present
International Society for Soil Mechanics and Foundation Engineering • Technical Committee on Lateritic Soils • Technical Committee on Geotechnical Engineering and Environmental Control • Committee on Professional Practice	Vice President Board Member Chairman Chairman Member	1973-1977 1989-1993 1967-1969 1973-1977 1997-2001
Taipei Civil Engineers Association	President	1980-1986
Chinese Union of Professional Civil Engineers Associations	President	1992-1995
Taipei Federation of Engineering Consultants	President Executive Supervisor	2001-2003 2003-2005
Chinese Association of Engineering Consultants (ROC)	President Vice President	1995-1997 1991-1995, 1997-present
Chinese Institute of Engineers (ROC) • Professional Activities Committee	Director Chairman	1983, 1985-1988 1977-1980

Name of Affiliations	Positions Held	Date (Year)
Chinese Institute of Civil and Hydraulic Engineering (ROC) • Geotechnical Engineering Committee	Vice President Chairman	1981-1983, 1997-1999, 2003-present 1978-1985
Chinese Taipei APEC Engineer Monitoring Committee	Chairman	2005-present
Arbitration Association of ROC	Board Supervisor Board Executive Supervisor	1998-2004 2004-present
Institute of Engineering Education Taiwan	Member of Accreditation Council	2005-present

14. Other Organisations:

Name of Affiliations	Positions Held	Date (Year)
China Road Federation (ROC)	Director Vice President	1977-1985, 1991- 1993, 1995-1997, 1997-1999, 1999-2001; 2001-2003 1993-1995, 1997-1999, 2003-present
The Road Engineering Association of Asia and Australasia	Board Director & Hon. Treasurer-General	1987- 2000
School of Civil & Structural Engineering, Nanyang Technological University	Member of the Advisory Committee	1988-1992
NTU-PWD Geotechnical Research Center, Nanyang Technological University	Chairman of Advisory Committee	1996-2003
Taipei Municipal Government	Member of Expert Advisory Committee	2003-2005
Public Construction Commission, ROC	Member of Technical Committee	2003-present
Journal of Geotechnical Engineering, China	Member of Editorial Board	1995-2003
Journal of Building Structures, China	Member of Editorial Board	2003-2006

SECTION 4
JOB HISTORY

15. Present position and brief history of job:
    (a) Present Position
    Chairman of the Board of Directors
    (b) Date Commenced
    January 2008
    (c) Name and address of company
    Moh and Associates, Inc.
    11th Floor, No. 3, Tunhwa South Road, Section 1,
    Taipei 10557, Taiwan, R.O.C.
    Tel: (886-2) 2579-8731
    Fax: (886-2) 2579-8635
    (d) Brief history and nature of job:
    Dr. Za-Chieh Moh is a co-founder of the MAA Group consulting Engineers. The company was founded in 1975 with operations in Taiwan and Singapore. Today, MAA has grown into a leading engineering and consulting service provider in the East and Southeast Asian region focused in the areas of infrastructure, land resources, environment, buildings and information technology. To meet the global needs of both public and private clients, MAA has a full range of engineering capabilities to provide clients with integrated solutions-ranging from conceptual planning, general consultancy, engineering design to project management. Today, MAA has more than 700 people with companies in Taipei, Hong Kong, Beijing, Shanghai, Bangkok, Penang and Singapore (www.maaconsultants.com).

    As a co-founder of the company, Dr. Za-Chieh Moh served as the President of the companies until January 2008 when he became the Chairman of the Board of Directors. Besides being the CEO of the company, Dr. Moh was also involved in technical matters of many important projects. Major works undertaken include planning, execution and consultation on soil investigation work, study and evaluation of soil behavior, performance study of earth structures, soil stabilization and ground improvement for foundations, highways, airfields and earth dams. Extensive research and practical experience on problems related to soft clays and lateritic soils. Served as Project Director or Advisor for many major projects, including: performance evaluation of test embankment at AIT new campus, Bangkok; performance study of test embankment for proposed new international airport in Bangkok; geotechnical study for reclamation of an abandoned river channel for development in Taipei; instrumentations for numerous deep excavations; geotechnical studies for the Mass Rapid Transit Systems Taipei; Railway Underground Project, geotechnical consultancy for the Second Bangkok

International Airport; Geotechnical Engineering Specialty Consultant for the Taipei Rapid Transit Systems; geotechnical design for Singapore MRT contract 405; numerous tall buildings and deep excavations in Taiwan, Hong Kong, Singapore, Malaysia, Thailand and Indonesia. Served as a member of the Review Committee for the Site Selection Study for the fourth nuclear power plant in Taiwan.

16. Previous positions held
    - Professor of Geotechnical Engineering; Vice President and Provost, Asian Institute of Technology, 1974-1976
    - Professor and Chairman, Division of Geotechnical Engineering, Asian Institute of Technology, Bangkok, 1967-1974
    - Associate Professor of Civil Engineering, SEATO Graduate School of Engineering, Bangkok, 1965-1967
    - Assistant Professor of Civil Engineering, Yale University, 1961-1965
    - Chief Engineer, Woodward-Clyde-Sherard & Associates, Consulting Civil Engineers, Omaha, Nebraska, 1959-1961

[redacted]

MAA GROUP

Integrated Solutions For Global Impact

亞新工程顧問股份有限公司
MOH AND ASSOCIATES, INC.

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Sec. 1,Taipei 10557, Taiwan, ROC
E-MAIL:maagroup@maaconsultants.com
TEL: (886-2) 2578-5858
FAX: (886-2) 2570-5566
www.maaconsultants.com

Your Reference:

Our Reference: HCH08092901

Contact:

Direct Line:

E-mail:

To Whom It May Concern:

This is to certify that Dr. Za-Chieh Moh is a co-founder of the MAA Group Consulting Engineers and served in the following positions of all MAA (Moh and Associates) companies:

President 1982-2007
Executive Vice President 1976-1982

He is the Chairman of the Board of Directors of the MAA Group companies since January 2008.

MAA Group Consulting Engineers

Za-Lee Moh, Ph.D., P.Eng.
Chairman Emeritus

SECTION 5
CAREER HISTORY

17. Provide information which would help the judges understand the challenges, problems and scope of the nominee's area of activity.

    (a) In the field of specialization of geotechnical engineering, the nominee has played important role of educating, promoting and developing the practice in the developing countries in Asia-Pacific. In the sixties, geotechnical engineering was relatively unknown and undeveloped in the FEIAP region. Through educational program and promoting rational-scientific approach, geotechnical engineering played a vital part in the development of many countries, including underground construction, MRT facilities, land development etc.

    (b) In engineering practice, in the 60s' and 70s' majority of important works were carried out by consulting and construction companies from western developed countries. Alternatively, works were handled by governments or government-related firms. Promotion and establishing reputable private services were big challenges.

18. Describe the nominee's exceptional achievements or contribution for the promotion, advancement and development of engineering professions in the FEIAP region.

    Over the fifty years career history, the nominee has demonstrated to be a true "Engineer".

    He was, and still is, active in academic, research and practice. In addition to his primary occupation, he has been involved and made significant contributions to the development of engineering practice and advancement of academic activities. Some of his achievements and contributions for the promotion, advancement and development of engineering professions in the FEIAP region are summarized below:

    (a) As Professor at the Asian Institute of Technology, he was primarily responsible for the development of the discipline in geotechnical engineering at AIT and in the region. Through his effort, the Southeast Asian Geotechnical Society was established which helped the development of that important but not well recognized field in civil engineering in many developing countries in the region. The Society now is one of the most active members Society of the International Society for Soil Mechanics and Geotechnical Engineering which has member Societies in more than 70 countries around the world.

    (b) As Vice President & Provost at AIT, the nominee was responsible for the overall academic development of the Institute during the critical development stage of the Institute. Today, AIT graduates are in most of the FEIAP member countries. Many of them are working in important positions, including academic institutions, governmental agencies and industries.

(c) As the co-founder of the MAA Group Consulting Engineers, the nominee has established reputable engineering practice and gained international recognition. MAA is one of few engineering consultants established by Asian and working on a regional basis to serve the region.

(d) Under the nominee's direction, MAA has made significant contributions to the technological development in the region. MAA's staff often combine research with practice. More than 450 technical papers have been published by the MAA staff. The nominee has authored or co-authored over 140 papers.

(e) The nominee is active in international engineering activities including playing active part in FIDIC, REAAA etc. He is the first and current Chairman of the Chinese Taipei APEC Engineer Monitoring Committee, and Chairman of the Chinese Taipei EMF Engineer Monitoring Committee (Provincial).

19. Summarise the nominee's most outstanding humanitarian effort and have made a substantial contribution in the field of engineering benefiting society as a whole.

The Nominee's most outstanding humanitarian contributions in the field of engineering benefiting society as a whole are:

(a) Education
(b) Continuing education
(c) Internationalization of engineering education and practice

20. Please list and supply evidence of any commendations, statements from authorities, honours and awards received which attest to and substantiate the nominee's achievements or contributions.

Honours and Awards

(a) Honorary Doctor of Technology, Asian Institute of Technology, 1999
(b) Honorary Member, The Road Engineering Association of Asia and Australasia, 2000
(c) Honorary Member, Japanese Geotechnical Society, 2003
(d) Distinguished Alumni Award, National Taiwan University, Department of Civil Engineering, 1994
(e) Gold Medal Award for Academic Achievement, Chinese Institute of Civil and Hydraulic Engineering, 1983
(f) Man of the Year Award, China Road Federation, 1983
(g) Member of Sigma Xi, Honorary Research Society, USA
(h) Listed in many international and national bibliographies, including Who's Who in the World (1986, 2009), Who's Who of the Republic of China

(1982), Who's Who in Hong Kong (1997), Who's Who in Engineering Singapore (2006), Who's Who in Engineering U.S.A. (1992), International Directory of Distinguished Leadership (1986), Who's Who in the Asia Pacific Rim (1999), International Who's Who of Professionals (2000), Asia Men & Women of Achievement (2003), Who's Who in Science and Engineering (2005), etc.

21. Summarise the reason why you (the nominator) believe that your nominee should be selected as FEIAP Engineer of the Year.

    The nominee is a true "Engineer". He is an academician as well as a practicing engineer. He has made significant contributions in both academic development and engineering practice, on regional basis. He is a believer and doer in "Engineering Globalization".

    The nominee has excellent character and is well respected by the engineering community, both domestically and internationally.

    A complete curriculum vitae is included in Appendix C.

The Board of Trustees of the

Asian Institute of Technology

Has Conferred Upon

Za-Chieh Moh

The Honorary Degree of

Doctor of Technology

Given this Twenty-second day of April 1999

President of the Institute

Chairperson of the Board

CITATION TO BE DELIVERED ON THE OCCASION OF THE AWARD TO DR ZA-CHIEH MOH OF THE ASIAN INSTITUTE OF TECHNOLOGY HONORARY DEGREE OF DOCTOR OF TECHNOLOGY THURSDAY 22 APRIL 1999

Mr President, I am privileged to present Dr Za-Chieh Moh, President of Moh and Associate Group Consulting Engineers, for the conferment of the Honorary Degree of Doctor of Technology of the Asian Institute of Technology.

The award is made on the unanimous recommendation, in June 1998, of the Board of Trustees of the Institute. In endorsing the award, the Trustees take account of Dr Za-Chieh Moh's outstanding contribution, through his academic, professional and managerial expertise in soil mechanics and foundation engineering, to the development of the Region.

Dr. Za-Chieh Moh, born in 1931, received his BS in Civil Engineering from the National Taiwan University in 1953, his MS in Civil Engineering from Iowa State University, USA, in 1955, and his Doctorate of Science from the Massachusetts Institute of Technology in 1961. After work in the United States as a consulting engineer and as an academic, Dr. Za-Chieh Moh joined the Asian Institute of Technology in 1965, where he was entrusted with the crucial responsibility of establishing a new field of study in Soil Mechanics and Foundation Engineering. Dr Za-Chieh Moh was thus a pioneer figure in AIT's lasting world renown in geotechnical engineering, and its key role in the training of geotechnical engineers to design and implement development projects throughout the Region.

Dr Za-Chieh Moh's research focus has always been on engineering problems, including the local behaviour of soils, in particular soft clays and lateritic soils; in-situ and large-scale testing; the performance of earth structures, and the stabilization of soil for foundations, highways, airfields and earth dams.

Dr Za-Chieh Moh founded the Southeast Asian Geotechnical Society (SEAGS) in 1967, serving as its President until 1973, and the Asian Information Center for Geotechnical Engineering, both at AIT. Dr Moh served, from 1973 to 1977, as Vice-President for Asia of the International Society for Soil Mechanics and Foundation Engineering, for which he has led numerous technical committees, internationally and nationally.

At the Asian Institute of Technology, Dr Za-Chieh Moh's managerial qualities, too, were recognised in his appointment as Vice-President and Provost, a position in which he served with distinction from 1974 to 1976, showing, as always, his typical intelligence, enthusiasm, and capacity for hard work.

Since leaving the Institute for consulting practice 23 years ago, Dr Moh has set up and led, in the Moh and Associate Group of Consulting Engineers, a company of prestige and influence in major regional projects. These include the Kaohsiung Cross Harbour Tunnel; river channel reclamation, mass rapid transit systems and many other projects of significance.

Dr Za-Chieh Moh has continued always to combine his professional and academic interests and expertise, as indicated by the level and internationality of his professional registrations, affiliations and activities. He is listed in the International Scholars Directory, the Directory of Selected Scholars and Researchers in Southeast Asia, *American Men and Women of Science*, and *5,000 Personalities of the World*. He was recently named among the *Men of the Year* by the China Road Federation, and received the Gold Medal Award for Academic Achievement from the Chinese Institute of Civil and Hydraulic Engineering.

Dr Za-Chieh Moh is author or co-author of over 100 technical papers and reports on soil properties, soil behavior, engineering performance and soil stabilisation. He has also edited ten regional and international conference proceedings, and many seminar and symposia proceedings.

Mr President, I have the honor, on behalf of the Board of Trustees and the whole AIT community of faculty, staff, students and alumni, to present Dr Za-Chieh Moh for the conferment of the Honorary Degree of Doctor of Technology, accompanied by Professor Tawatchai Tingsangchali, of the Institute's School of Civil Engineering.

ML Birabhongse Kasemsri
Chair, Board of Trustees
Asian Institute of Technology

22 April 1999

The Road Engineering Association of Asia & Australasia

**Be it known by those present that
Honorary Membership of the Association
has been awarded to**

Dr. Za-Chieh Moh

on the occasion of the 10th Conference
of the Association in Tokyo
4 - 9 September 2000

in recognition of

his invaluable contribution to the science and practice of road engineering and his outstanding honorary services to the Association.
During his distinguished career spanning over some 46 years as a geo-technic expert, he has initiated extensive activities to improve and elevate the technical knowledge in this field throughout the region.
His involvement with REAAA dates back to as early as the seventies when he was the Vice President of the Organising Committee of the First REAAA Conference.
His hard work and dedication was pivotal to the success of the Third and the Eighth Conference in Taiwan in 1981 and 1995 respectively.
He was elected to the Council of REAAA in 1978 and was entrusted the position of Honorary Treasurer-General for the last fourteen years, making him one of the longest serving Council Members in the Governing Council of REAAA.
Through his relentless efforts and unfailing support, he has contributed greatly to the stature of REAAA internationally.

Dated this 4th day of September 2000

By Order of the Governing Council

Sadamu Mino

President
Sadamu Mino, Dr. Eng.

R. J. Dunlop

Immediate Past President
Robin Dunlop, Dr. Eng.

Japanese Geotechnical Society

第一一四号

名誉会員推挙状

莫　若　楫　殿

あなたは多年にわたり地盤工学ならびに本会の発展に貢献しその功績がとくに顕著と認められますので名誉会員に推挙いたします

平成十五年　五月二十九日

社団法人　地盤工学会
会長　足立紀尚

(72)土水(5)發字第184號

中國土木水利工程學會工程獎章證書

本會會員莫若楫先生，浙江省吳興縣人，國立臺灣大學土木工程學士，美國麻省理工學院博士。歷任亞洲理工學院教授、系主任、副校長兼教務長。曾於亞洲理工學院創設大地工程學系，作育人才。嗣回國創辦亞新工程顧問公司，自任總經理，為國內工程界貢獻所學。經發起組織東南亞土壤工程學會，膺選會長；復當選國際土壤及基礎工程學會副會長；本會理事及常務理事。為本會主持大地工程研究會，舉辦國際會議、研討會及講習會。並多次代表本會及有關學會組團出席國際會議，發表論文及研究報告五十餘篇，於學術研究與交流，成就卓越。爰依據本會工程獎章頒授辦法第七條第一款之規定，頒贈金質獎章一座，以資表揚。

此證。

理事長 楊學涑

中華民國　　月二十日

China Road Federation (ROC)

中華民國道路協會

茲證明莫若楫君為本會七十二年度協會獎章得獎人。

中華民國道路協會

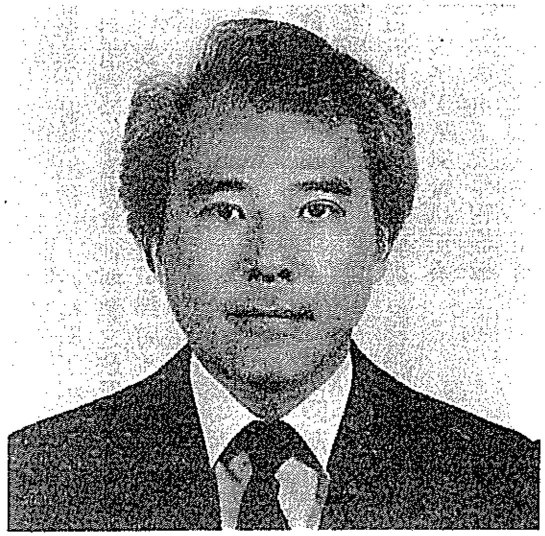
理事長陳世圯

西元二[illegible]年四月八日

會址：104臺北市長安東路二段一八四號九樓
電話：二七四〇—八二八六
傳真：二七四〇—八二九〇

本年度獎章得獎人簡介

得獎人莫若楫先生



會員號碼：一五一九
年齡：五十二歲
學歷：美國麻省理工學院博士
現職：亞新工程顧問公司總經理

莫若楫博士，浙江吳興人，民國廿年生。四十二年畢業於國立台灣大學土木工程系，旋即赴美深造，初入愛奧華州立大學得碩士學位後，轉讀麻省理工學院專攻大地工程，獲博士學位。畢業後即任職于美國著名之大地工程顧問公司及執教於耶魯大學。於民國五十四年應美國政府之聘，至泰國曼谷之亞洲理工學院任教。

在亞洲理工學院任職十一年間，歷任教授、系主任及副校長兼教務長。除掌理校務外，並創辦大地工程學系，充實大地工程研究設備，大力培植青年工程師，研習高深之大地工程科學，聞名于東南亞各國。此外復設立東南亞土壤工程學會及國際土壤與基礎工程學會，均獲選爲會長及副會長，爲我國人士，在國際大地工程界，獲最高之地位。

莫君爲有效提高我國大地工程學上之技術發展，於六十三年辭去教職，回國創辦「亞新工程顧問有限公司」，直接推動大地工程，實際應用於土木及道路工程方面。

回國後即加入中華民國道路協會爲會員，連任數屆理事會理事。曾於六十八年爲道路協會極力爭取主辦亞澳道路工程協會(REAAA)第三屆(民國七十年)年會，在我國召開。莫君兼任籌備會秘書長、策劃籌備工作，使會議極爲成功，也使台灣地區的道路建設，獲得了國際上高度的評價。

此外莫君亦爲「中國工程師學會」及「中國土木工程學會」籌辦了四次有關大地工程之研討會及講習會，參加講習人員均達三、四百人，使我國土木工程界，加深大地工程技術之認識而提高了設計與施工水準。

民國六十九年在台北召開之第六屆東南亞大地工程會議，亦由莫君負責籌劃，參加者，多爲國際大地工程專家，無形中使我國土木工程界，在國際之地位受到重視。

莫君主持之「亞新工程顧問有限公司」，數年來在國內、外聲譽日隆。曾受委託爲大規模土木或建築工程擔任大地工程顧問。如香港新機場規劃，泰國國際機場之大地工程設計、馬來西亞東西公路邊坡穩定研究、印尼高速公路沉陷與基礎改良研究、及國內重大工程中，有關大地工程之規劃等，均著有績效，極受主辦單位之信賴。

莫君尚在公餘，不斷研究高深學識，陸續提出大地工程方面論文五十一篇，並數次主編國際會議論文專集，均受工程界之矚目與稱道。

SECTION 6
NOMINATION

Nominated by FEIAP Member Economy:

Organisation: Chinese Institute of Engineers, Chinese Taipei
Address: 3rd Floor, No. 1, Jen-Ai Road, Section 2, Taipei 10055, Taiwan
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APPENDIX A

List of Publications

Appendix A

A. Papers

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10. Tenth Southeast Asian Geotechnical Conference, April 1990, Taipei, Southeast Asian Geotechnical Society and Chinese Institute of Civil and Hydraulic Engineering.

APPENDIX B

Copy of Ten Selected Papers

GEOTECHNICAL PROBLEMS RELATED TO DESIGN AND CONSTRUCTION OF THE TAIPEI TRANSIT SYSTEMS

by

Z. C. Moh and R. N. Hwang

Reprinted from
Proceedings of Keynote Speech,
Professor Chin Fung Kee Memorial Lecture,
September 6, 1997, Kuala Lumpur, Malaysia

Za-Chieh Moh, President, Moh and Associates, Inc., Taipei, Taiwan
Richard N. Hwang, Manager, Rapid Transit Projects, Moh and Associates, Inc., Taipei, Taiwan

Geotechnical problems related to design and construction of the Taipei Transit Systems

Professor Chin Fung Kee Memorial Lecture
6 September, 1997, Kuala Lumpur, Malaysia

SYNOPSIS The Initial Network of the Taipei Rapid Transit Systems comprises six lines with a total of 88 km of track and 77 stations. About half of the stations and tracks are underground. Except a short section of one of the lines, the majority of the Initial Network is located in soft ground. At the present, two lines in the Network have been completed and open to operation. The remaining four lines are anticipated for revenue operation in the next two to four years. For the underground construction, about 45 km of diaphragm walls were constructed in the cut-and-cover excavations whilst bored tunnels have a total route length of 22 km. This paper describes some of the most significant geotechnical concerns associated with design and construction of the Initial Network. Discussions are made on the soil characterization, effects of groundwater movements, ground subsidence, earthpressures on walls and ground movements during construction.

1 INTRODUCTION

Geotechnical engineering plays a vital role in constructions for rapid transit systems in cities, in which usually the ground is soft and difficult to deal with, and the Taipei Rapid Transit Systems (TRTS) is certainly not an exception. The challenge in solving various types of problems enables geotechnical engineers to accumulate experience and sharpen their skills, also offers geotechnical engineers opportunities to contribute their knowledge and wisdom.

This paper illustrates the importance of geotechnical engineering in construction of rapid transit systems by using TRTS as an example. It covers the three major elements in underground constructions, i.e., soil investigation, cut-and-cover construction, and tunnelling.

2 TAIPEI RAPID TRANSIT SYSTEMS

A system map of TRTS is depicted in Fig. 1. The Initial Network of the Systems comprises six lines, namely the Mucha, Tamshui, Hsintien, Nankang, Panchiao, and the Chungho Lines. Because this is the first rapid transit system constructed in Taiwan, the Department of Rapid Transit Systems (DORTS) of the Taipei Municipal Government engaged a Geotechnical Engineering Specialty Consultant (GESC) to assist in the design review and construction supervision right at the beginning of the project. This has been proved very fruitful as the design was optimized and many potential problems avoided. At the peak of construction, a total of 42 field stations were setup and managed by the GESC to assist the field staff of the DORTS in solving on-site geotechnical problems. This also enabled high-quality geological and instrument data to

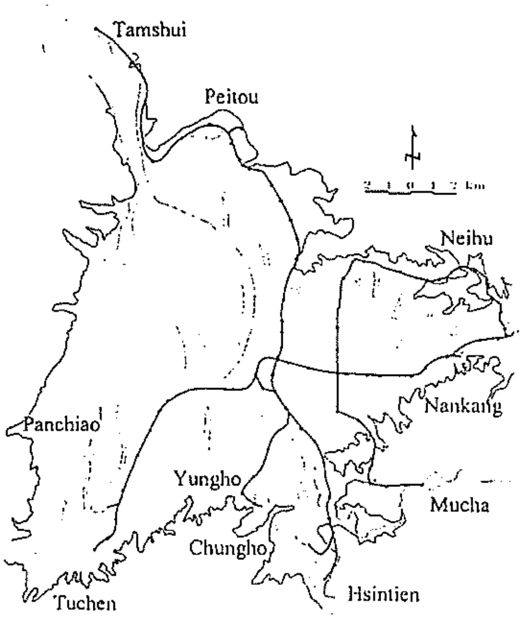

Fig. 1 Initial Network of Taipei Rapid Transit Systems

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be obtained to facilitate back-analysis for verifying the designs and the design assumptions (Moh and Hwang, 1996). A Data Center was established at the headquarters of GESC to process the tremendous amount of field data in a systematic manner.

3 GROUND CHARACTERIZATION

Geotechnical engineers shall always have an appreciation of local geology and geological map shall always be the first piece of information to study whenever a project starts. Previously, the geological mapping for the Taipei Basin was limited to the east of the Tamshui and Hsintien Rivers. A recent study, refer to Fig. 2, extended the zonation westward beyond these rivers to cover the entire basin (Lee, 1996). The number of geological zones increased from 7 to 22. For the benefit of the readers, an east-west and a north-south soil profiles across the Basin are presented in Fig. 3. As can be noted that at the surface is a thick layer of Sungshan Formation. Toward the east, silty clay dominates while in the central city area, where the Taipei Main Station is located, the six-sublayer sequence is evident. Toward the west, the stratigraphy becomes rather more complex with silty sand and silty clay seams interbedded in these sublayers. The Sungshan Formation is underlain by the so-called Chingmei Gravels which contains gravels of various sizes and is extremely permeable. This gravelly layer is practically a reservoir and has been responsible for several major failures during the underground construction of TRTS.

3.1 Soil Investigations

In modern cities, in which rapid transit systems are to be constructed, usually numerous boreholes have already been sunk, for example, for the construction of foundations

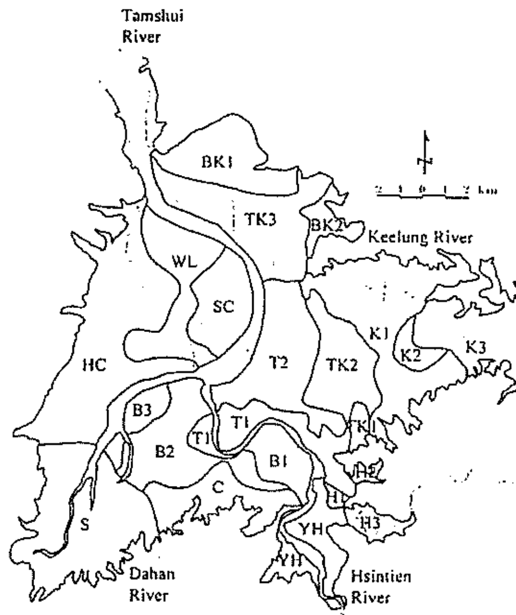

Fig. 2 Geological zonation of the Taipei Basin

and basements of highrise buildings prior to the implementation of the programs. It is also quite common nowadays for city governments to compile borehole data into databases and these databases can be utilized in the preliminary assessment of construction methods. However, these boreholes may not be sufficient in either quantity or quality for the design and construction of rapid transit systems. First of all, excavations for rapid transit systems are usually far deeper than basements. Secondly, the previous investigations generally lacked a unified quality control and may lead designers astray. Once a route is decided, it is a normal practice for the authority in-charge of the program to engage a qualified geotechnical consultant in

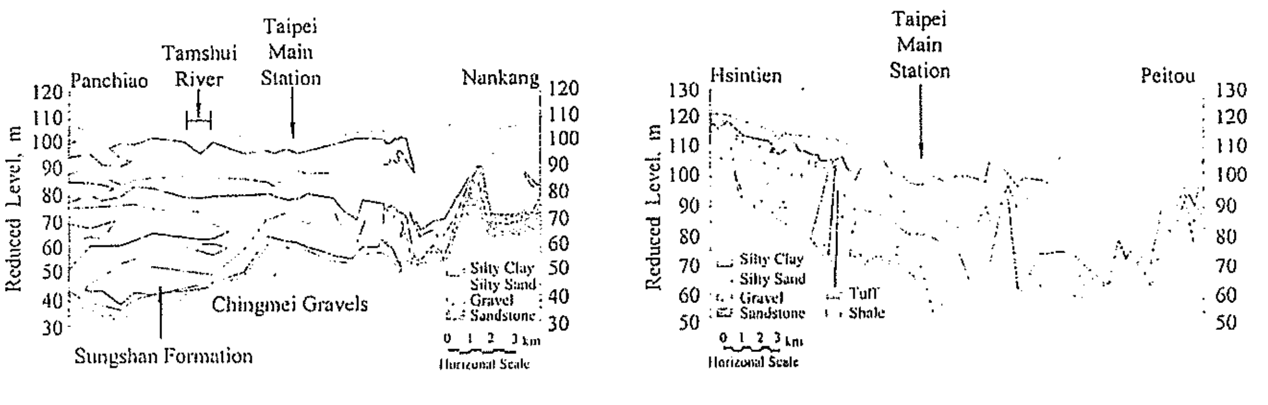

Fig. 3 Geological profiles of the Taipei Basin

the planning stage to compile all the available information to a consistent format and to perform additional investigations for calibrating such information on a unified basis. The designers awarded the job have to perform their own investigation to satisfy themselves subsequently and contractors will have to do the same.

There are no rules regarding the quantities of boreholes and tests to be performed in each phase of the investigation. For a system with stations and tunnels buried in uniform ground of the same geological formation, say, six to eight boreholes are usually sufficient for a station as far as structural design is concerned. However, additional boreholes may be required to optimize the design of retaining walls or pile foundations if the top of the bearing stratum is erratic and/or the soil stratigraphy is complex. Additional boreholes are also required if blow-in and piping are potential threats and the thickness of the impervious clay blanket is critical to the safety of construction. For shield tunneling, boreholes at 50m intervals are sufficient for the design purpose but additional boreholes are required locally to ascertain mix-face conditions.

For routes running through varying geological formations, investigations have to be more detailed and sophisticated in-situ and laboratory tests are sometimes required to enable tenderers to judge the situation with a better accuracy and to reduce potential claims from contractors. The spending invested in soil investigations will be well paid off as potential problems are eliminated beforehand. Unfortunately, such a viewpoint is seldom shared by clients till it is too late.

It is a common clause in all the tender documents that tenderers of a construction contract shall satisfy themselves by performing additional investigations before submitting their tenders. However, very few tenderers would do so not only because of the unwillingness in spending the money but also because of the short time available for preparing the tenders.

During constructions, boreholes are sunk and tests are performed by contractors for revealing ground conditions with a better accuracy. Boreholes are also sunk for other purposes such as installing instruments, confirming the results of ground treatment, and solving specific problems, etc. While doing so, in-situ tests are usually performed in boreholes for revealing ground conditions. The investigations performed by contractors are usually many times greater in quantities but much less in sophistication than the investigations carried out by designers.

Soil stratigraphy is conventionally determined by boring and standard penetration tests. However, cone penetration test (CPT) is gaining its popularity as a major tool for determining soil stratigraphy in soft ground primarily because of its consistency in results and also because of the fact that a continuous profile can be obtained. Modern CPT apparatus allow the measurement of porewater pressures induced at the tip as the cone advances. This enables fine seams to be identified along the depth. Some cones can even measure shear wave velocities in soils but such an application is limited to specific purpose, for example, evaluation of liquefaction potential.

A typical CPT profile obtained in the central city area of Taipei is shown in Fig. 4. As can be noted that the six-layer subdivision is evident. Although CPT is considered as an excellent tool in characterizing soil stratigraphy. It however should be noted that interpretation of the results requires local knowledge. For example, it has been found that the subsoils in the Sungshan Formation in the Taipei Basin cannot be classified by using any of the charts available in hand and new charts have to be established (Wong, Chu and Yang, 1993).

One phenomenon worthy of mentioning is that negative pore pressures were measured throughout the entire depths of Sublayers 3 and 5 as the cone advanced. The implication of this finding deserves further studies. The phenomenon would have indicated a dilative nature of the silty sand in these two sublayers upon disturbance. However, the data available are insufficient to sustaintiate this hypotheses.

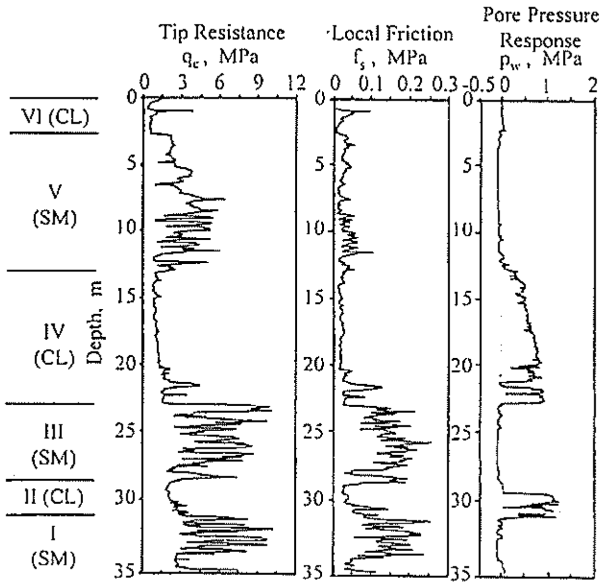

Fig. 4 CPT profile in Central Taipei

As can be noted from Fig. 4, the soil stratigraphy is more clearly identifiable in the porewater pressure profile than the two other profiles. However, it should be noted that porewater pressures induced are dependent on the type of the cone used and, more importantly, are affected by the workmanship. Figure 5 shows a comparison of the results obtained at two neighboring locations. At CPT-39, the piezometer tip was simply submerged in water for 24 hours before the commencement of the test. The pore pressure response was poor and the boundaries between consecutive layers could not be identified. The test was repeated at CPT-39A, which was 2m away from the location of CPT-39, with the piezometer tip submerged in a water-glycerin mix and boiled for 10 minutes to drive air bulbs out. The results were much improved as indicated in the figure.

Theoretically, it is possible to determine the consolidation characteristics of soils by observing the dissipation of porewater pressures induced at the tip. A considerable quantity of dissipation tests have been performed in the central city area of Taipei and typical results are shown in Fig. 6. In silty clays, i.e., in Layers II, IV and VI, positive excess porewater pressures were obtained during the advancement of the tip and these excess pore pressures dissipated with time. In silty sands, i.e., in Layers I, III and V, the porewater pressures measured were smaller than the atmospheric pressure during the advancement of the tip and these porewater pressures increased with time till they equalized with the background pressures. The t_{50} values, which are the times for 50% of the excess pore pressures to dissipate, obtained for clays in Contracts CN251, CN253A and CN253B of the Nankang

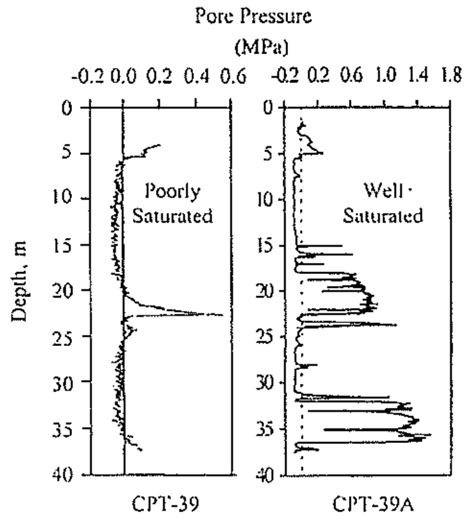

Fig. 5 Results of piezocone tests as affected by saturation of piezometer

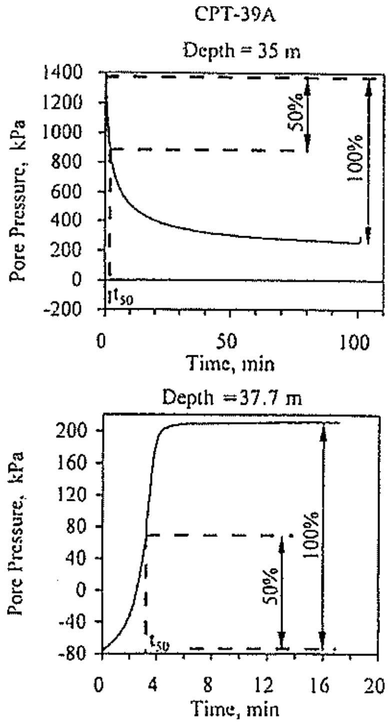

Fig. 6 Dissipation and recovery of porewater pressure

Line are plotted in Fig. 7(a). The times for half of the deficient pore pressures to recover in silty sand are plotted in Fig. 7(b). It is conceivable that the speed of dissipation of excess pore pressures in the case of clay and the speed of recovery of deficient pore pressures in the case of silty sand are somehow related to the consolidation characteristics of the soils in which the tip was buried. Mathematical formulations have been developed for interpreting the results of dissipation tests for partially saturated soils (Chin, et al., 1993). However, as a cone penetrates into the ground, the stress field in the surrounding soils will not only depend on the consolidation characteristics of soils, but will also depend on the geometry of the cone and the position of piezometer. The mechanism involved is far too difficult to be simulated by any mathematical model which inevitably involves assumptions and some of these assumptions may not be realistic at all.

It is envisaged that, as more studies are conducted, consolidation properties of soils can be interpreted from dissipation tests with confidence. If so, dissipation tests are superior to other types of consolidation tests and permeability tests for its simplicity in procedure and the repeatability in results. Furthermore, tests can be conducted at very close intervals for obtaining a nearly continuous profile of whatever the property is along the depth.

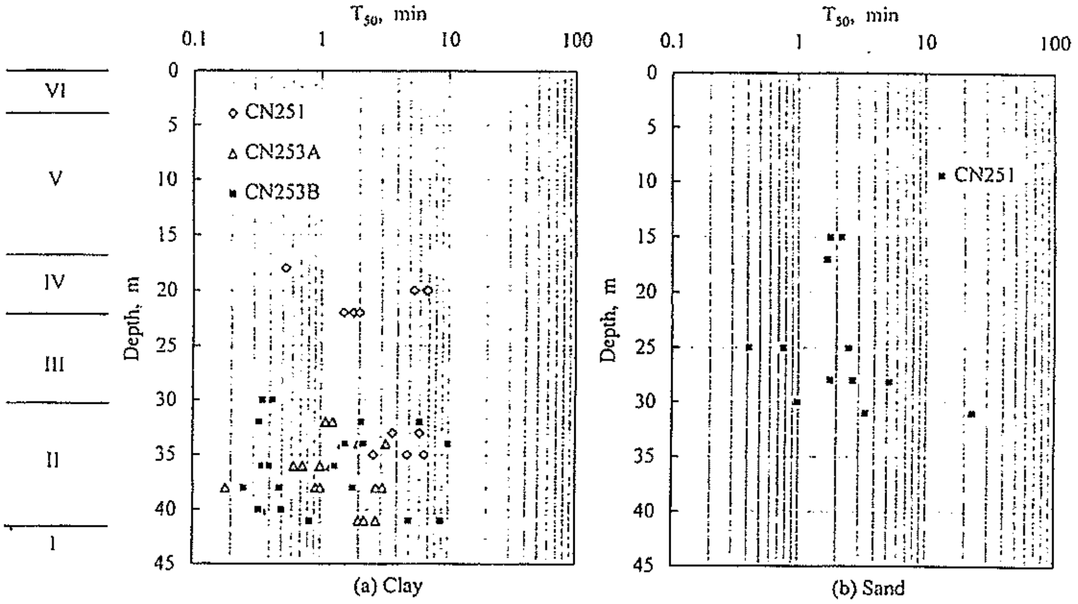

Fig. 7 Time required for porewater pressure to dissipate or to recover

3.2 Groundwater Conditions

Without any doubts, groundwater conditions have dominating effects on the designs and constructions of temporary and permanent underground structures. Experience does indicate that the majority of failures occurred during underground construction were directly related to water problems and some of them were disastrous and even fatal. To provide background information to enable designers to arrive at optimum designs, groundwater movements have to be monitored for a sufficient length of time before design starts. This is particularly true in cities where groundwater is known to experience large drawdowns, for examples, Taipei and Bangkok. In the Taipei Basin, historical records on groundwater movements are available for the last 50 years or so. As can be noted from Fig. 8 (Wu, 1967; 1968), the piezometric level in the Chingmei Gravels was once lowered by as much as 40m due to the excessive pumping. The use of groundwater as water supply was banned in late 60's and the piezometric level in the Chingmei formation gradually recovered subsequently.

Based on the records available prior to 1990, it was envisaged that, in the central city area, the piezometric level in the Chingmei Formation would have risen to a level of RL 97m (mean sea level: RL 100m) this year, i.e., the year of 1997. However, as the construction for TRTS started in 1990, the recovery of the piezometric level did slow down because pumping was carried out at many sites as a measure against blow-in. Large scale pumping, exceeding 2,000 cubic meters per hour in rate, was carried out for three deep excavations (Hwang, et al., 1996; Moh, Chuay and Hwang, 1996) and, as can be noted from Fig. 8, the current piezometric level is 4m below what was predicted. As nearly all the underground works will be completed this year, whether the recovery will return to its previous track remains to be observed. It shall be noted that before the turn of this century, the water in the Chingmei Gravels was in an artesian condition with the piezometric level above the ground level (Wu, 1967; 1968). Therefore, it is

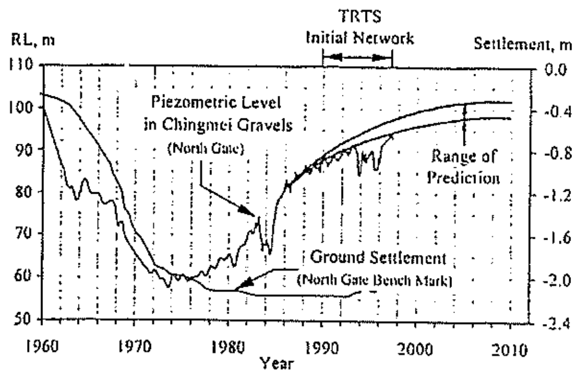

Fig. 8 Piezometric level in the Chingmei Gravels and historical ground settlement in Central Taipei

conceivable that the piezometric level will keep on rising for the many years to come.

The lowering of piezometric levels has been found to be beneficial to underground constructions. In the first place, it reduces active pressures and increases passive resistance on retaining systems. Secondly, the ground in the central city area had settled by 2.2m prior to the implementation of the Initial Network and the preconsolidation effects greatly reduced consolidation settlements of soft ground during the construction and, hence, reduced damaging potential to buildings nearby. Since the Chingmei Gravels is relatively incompressible, settlement mainly came from consolidation of soft clays in the Sungshan Formation. To obtain data on the groundwater movements in the Sungshan Formation along the routes of all the six lines in the Initial Network, GESC carried out monthly monitoring of piezometric heads at 798 locations, including 287 observation wells and 511 piezometers, since October 1987 till May 1993 when the duty was assumed by the DORTS. It was found that, as depicted in Fig. 9, over the entire basin, Sublayers III and V are indeed separated by two aquitards, i.e., Sublayers II and IV, and become separate aquifers. However, the piezometric levels in all the sublayers in the Sungshan Formation did move in phase with the piezometric level in the Chingmei Gravels, which implies moderate leakage in these two aquitards. With time, it is anticipated that the piezometric levels in the Sungshan Formation will eventually be equalized with the piezometric level in the Chingmei Gravels.

3.3 Local Geological Features

Every city has its own unique geological features which deserve special considerations. For examples, cavities in the limestone formation are common in Malaysia and have caused difficulties in pile foundations and large boulders are obstacles for underground works in Singapore and Hong Kong.

In the Taipei Basin, methane and drift woods are the two unique geological features. Methane was encountered in several boreholes during investigation. Its presence is related to the capture of gas in dooms encapped by impermeable clays (Lin, Chang and Chu, 1997). In one case, the eruption sent a jet of methane-water mix to a maximum height of about 10m into the air and continued for more than 3 days. Contractors have been warned of the possibility of encountering methane in tunnel drives and possible consequences. As a precaution, specifications require the concentration of methane be continuously monitored during tunnelling and shield machines be

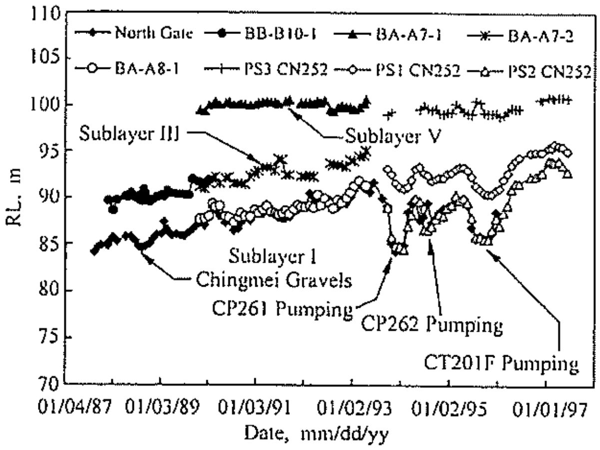

Fig. 9 Piezometric pressures in the Sungshan Formation

equipped with detective devices and alarm systems. The power supply shall be automatically cut off when the concentration of methane reaches 1.25% and resume only after the concentration of methane drops to 1%. The capacity of ventilation in tunnels was increased from 780 cubic meters per minutes to twice as much. Although methane was encountered at locations all over the Taipei Basin during investigation, the problems, to the authors' knowledge, were limited to the Chungho Line during TRTS construction.

Drift woods were recovered in numerous excavations and were responsible for a few accidents encountered during tunnelling (Lin, Chang and Chu, 1997; Ju, Kung and Duann, 1997). Chunks of 1m or so in diameter were frequently encountered and they may be as long as 5m. A piece of wood found at a depth of 9m in Observation Well 2 (W-1797) in Panchiao dated back to 6,760 years and that found at a depth of 23m dated back to 7,950 years (Liew, 1994). The presence of drift woods was well recognized and has been well reported, however, it is still very difficult to predict the exact locations and depths of drift woods beforehand. For cut-and-cover sections along the routes, problems were usually localized and were relatively easy to solve. The problems were much more serious for tunnelling sections. All the shield machines adopted in TRTS have sufficient strength and power to cut through the woods as long as they are not too large in size. However, there was one occasion in which the presence of drift wood caused the ground to collapse as the movement of the shield was hampered and soft clay kept on moving into the earth chamber. Finally, a sinkhole of 5m in diameter occurred right above the head of the shield machine. Workers entered the earth chamber and recovered two pieces of drift wood which were 500mm and 400mm in length.

A gravelly layer exists at the tunnelling depth along the Hsintien Line, refer to Figs. 1 and 2 for location and profile. The shield machines used were capable of handling cobbles of 200mm or less in size. There were no reported cases which required the removal of cobbles inside the earth chambers. The presence of this gravelly layer, however, did lead to leakage of compressed air in the NATM tunnel for Contract CH221. Although it was possible to maintain the pressure in the tunnel by increasing air supply, the air did travel to a far distance causing damages to underground facilities which would not have been damaged otherwise. Therefore, it is important to investigate ground conditions more thoroughly if compressed air is to be used.

4 CUT-AND-COVER CONSTRUCTIONS

For cut-and-cover constructions, the major fields of interest are: a) method of construction, b) design of retaining system, c) groundwater control and d) ground movements. In TRTS, the deepest excavation was 36.6m and was carried out for the construction of Ventilation Shaft A in Contract CP262 of the Panchiao Line. Although this depth of excavation is not surprisingly large in today's standard, the presence of the Chingmei Gravels was a potential threat to the safety during construction and called for precautionary measures. A similar situation was faced in some other contracts and different solutions were worked out.

4.1 Methods of Excavation

Basically, deep excavations in soft ground can be carried out by using: a) the bottom-up method, b) the top-down method, and c) the semi-top down method. As always, time and money are the two elements to be considered in the evaluation. As far as the construction period is concerned, the top-down method has the advantage that super-structure can be constructed while excavation is being carried out, and is hence favored for the construction of, for example, highrise buildings with deep basements. For rapid transit systems, however, it is usually necessary to complete the base slabs for the rails to be laid as soon as possible. Therefore, the bottom-up method is often the choice. On the economy side, it may be argued that the use of floor slabs as struts in the case of top-down construction leads to savings. However, on the other hand, earthworks are more efficient in the case of the bottom-up construction. Besides, to support the upper floors while excavation is proceeding in top-down construction does require additional efforts.

From a geotechnical point of view, studies indicate that the use of top-down method may lead to larger lateral deflections of the walls in comparison with the bottom-up method, hence, larger ground settlements behind the walls and greater damaging potential to adjacent buildings. The reasons are:

(i) The spans between floors in top-down constructions are usually larger than the spans between struts in bottom-up constructions, thus, it will take longer time to excavate for each level in the former case than the latter.

(ii) The walls will also have less number of levels of lateral support in the former case than the latter.

(iii) Casting of floors is more time consuming than strutting. More wall deflection will be induced as soils creep.

(iv) Struts can be preloaded to minimize lateral deflections of walls but not the floors.

It is possible to construct the first floor first using the top-down method and proceed with the remaining excavation using the bottom-up method. This becomes the so-called semi-top-down method. Of the 34 underground stations in TRTS, 23 were constructed by using the bottom-up method, 10 by using the semi-top down method but only 1 by using the top-down method. It is clear that the top-down method was not favored in construction of the TRTS systems.

4.2 Retaining Systems

Thick diaphragm walls are usually chosen for maintaining the stability of the side walls of deep excavations in soft ground. However, other types of retaining structures such as contiguous bored piles, soil-mix wall, and heavy sheet piles are also frequently adopted. Because of the poor ground conditions and the great depths of excavations, diaphragm walls were exclusively used for station excavations for TRTS. For shallower excavations, for example, at entrances, contiguous bored piles and sheet piles were sometimes used.

There are basically three types of diaphragm wall: a) single wall, b) double wall, and c) composite wall. In the old days, the single wall system was quite common because the diaphragm wall served as the structural wall as well. There is a growing concern on the poor water-proofing of diaphragm walls, and as a result, single wall system is gradually phased out and the double-wall system is more popular nowadays. In a few stations in TRTS, composite wall system, in which the permanent wall is structurally connected to the diaphragm wall by dowels, is adopted. This reduces the thickness of the permanent wall and saves some space because the diaphragm wall and the permanent wall form a single structural element. In hindsight, composite walls appear to be a nuisance. First of all, the provision of dowels makes the tremieing of concrete difficult. Secondly, these dowels have to be manually exposed and bent for the permanent wall to be cast. Unless space is crucial, the use of composite walls shall be discouraged.

Strutting is a common practice. However, ground anchors are sometimes used if an adequate hard pan is available at shallow depths. At launching and arrival shafts where struts may become obstacles to shield machines, anchoring is frequently the choice. It should be noted that in quite a few cases, ground anchoring led to serious inflow of water through holes made on walls and resulted in excessive ground settlements and damages to adjacent structures. Therefore, so long as possible, ground anchoring shall be avoided if the ground consists of sandy soils. Secondly, the consequences of failure, if so happens, will be more serious for anchored walls. Failure of an anchor may soon progress to the rest of anchors as soils are disturbed and lose their strengths. On the other hand, failure of a strut usually is localized because the capacities of struts do not rely on soil strength. It will also be much easier to remedy a failed strut than a failed anchor because the failed anchor might be several meters above the bottom of excavation and might not be accessible by machines. For all these reasons, preference shall always be given to strutting, rather than anchoring.

For the structural design of retaining systems, the use of Peck's apparent earthpressure diagram is till a popular procedure. As pointed out by Prof. Peck that (Peck, 1974)

> "It has been found that, even in a single open cut in which the work has been executed in expert fashion, the loads in equally spaced struts at a given level vary over a wide range and, correspondingly, the pressure diagrams for struts in various vertical profiles differ from each other. Since it is not possible to predict which of apparently identically situated struts will experience the greatest loads, conservative use of the empirically derived pressure diagrams for design requires that each strut be proportioned as if it would be subjected to the maximum load indicated by any of the pressure diagrams. Hence, for design of struts, it is appropriate, to use a pressure envelope that encloses all the pressure diagrams derived from observations"

Peck's diagram was first published in the year of 1943 (Peck, 1943). Fifty years since then, this situation remains unchanged. Although computer analyses are claimed to be able to handle complex ground conditions, variable construction procedures and different structural configurations, they are unable to produce the ranges of strut loads observed in field. Strut loads appear to be affected by minor construction details and environment factors which are far beyond the designers control.

It has to be admitted that, however, computer analysis does represent state-of-the-art technology and, as time goes by, will eventually become a reliable design tool. The increasing popularity in the use of computer analyses for analyzing retaining structures, however, leads to the worry that inexperienced engineers may blindly rely on the results of computer analyses for designing structural elements. An even more serious worry is that some structural engineers may perform computer analyses using soil parameters from textbooks and commercial software packages, of which the use is subject to limitations, and design the walls without consulting to a geotechnical engineer.

With proper judgment, Peck's earthpressure diagram appears to work well for various types of soils in Singapore (Hwang, Quah and Buttling, 1987; Hulme, Potter and Shirlaw, 1989). Studies on the TRTS are undergoing to see if Peck's diagram is applicable to the Sungshan Formation as well. Early findings are inconclusive for the reasons that the retaining systems were designed on the principle of limiting wall deflections so the ground settlements behind the wall would not be detrimental to structures adjacent to the excavation. Furthermore, all the struts were pre-loaded to at least 50% of their design loads. In other words, the designs are "stiff", as opposed to the "soft" design adopted in the case histories considered in developing the Peck' diagram. In such a case, soils would behave quite differently from what was assumed and it is doubtful that the diagram will be suitable without modifications.

It is a well known fact that earthpressure on a wall is a function of lateral deformation of the wall. Figure 10 shows a typical plot frequently appearing in textbooks (Terzaghi, 1954). As can be noted that earthpressure on the wall also depends on the soil properties and direction of wall movement. Figure 11 shows a case history reported by Moh and Hwang (1993). The site, i.e., CPH Building, is located in the central city area of Taipei. Excavation was carried out to a depth of 17.4m using the top-down construction method and a maximum lateral deflection of 110mm was observed. Presented in Fig. 11(c) are the ratios of effective lateral earthpressures to the effective overburden pressures, denoted as Ra values, interpreted from the readings obtained by four earthpressure cells mounted on the active side of the diaphragm wall. As can be noted that the initial conditions for all the earthpressure

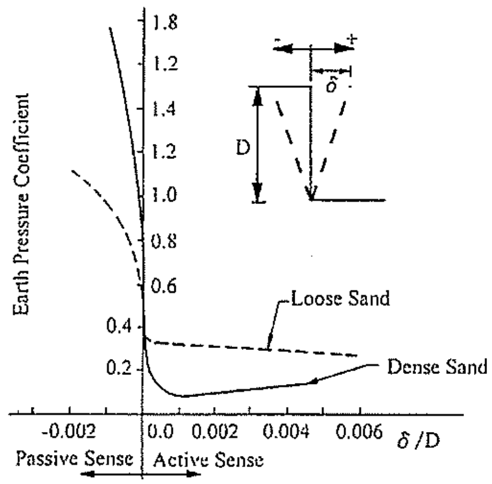

Fig. 10 Earthpressures as affected by wall movement (after Terzaghi, 1954)

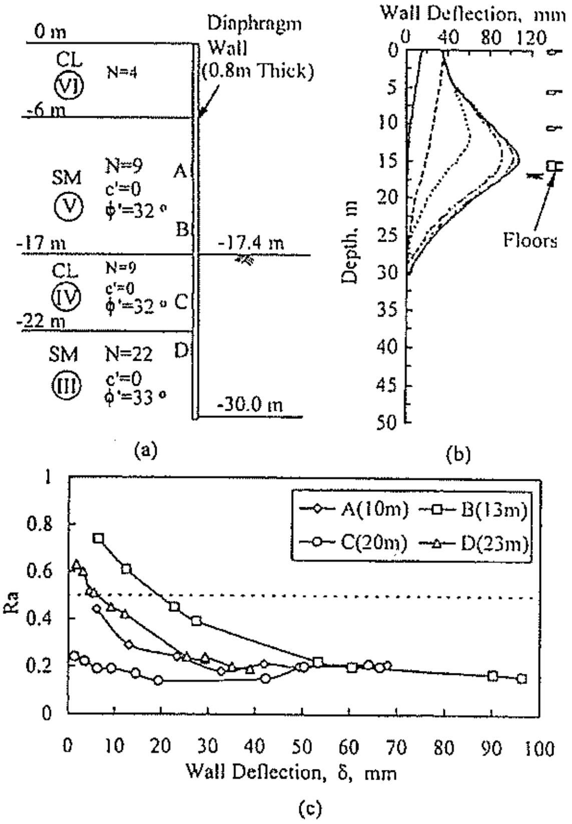

Fig. 11 Lateral earthpressure ratios at CPH building

cells were quite different with Ra values varying from 0.2 to 0.7. During the installation of these earthpressure cells, it was necessary to extrude the two loading plates of each cell by jacking laterally to make a good contact with the soils at the interfaces with the diaphragm wall on both sides. The initial horizontal earthpressures recorded by the cell were thus a result of jacking and do not necessarily represent the in-situ pressures.

The true in-situ horizontal earthpressures at the soil-wall contacts are difficult to ascertain even for the simplest type of soil, i.e., normally consolidated clay, because the ground has been disturbed in the process of installing the diaphragm wall. For all practical purposes, an initial Ra value of 0.5 can be assumed. Three of the four cells recorded pressures with initial Ra values greater than 0.5, presumably, because of over-jacking. The initial lateral pressures sensed by these cells, i.e., A, B and D, were in the passive state and corresponded to certain outward movements of the jacking plates. It is reasonable to correct the readings by shifting the three curves laterally to yield a Ra value of 0.5 at zero displacement, i.e., δ=0. The results are shown in Fig. 12 and, as can be noted that, after adjustments, the three curves are surprisingly consistent. The fourth curve with an initial Ra value of 0.2 is difficult to be corrected because the lateral pressure was too low to start with. The initial lateral pressure on this cell was already near its lower-bound, obviously, because the jacking plate did not have a good contact with the soil.

Based on Fig. 12, it is concluded that a displacement of 20mm is sufficient for the lateral earthpressure ratios to reach their lower-bound values. Ideally, earthpressure cells shall be loaded to yield a lateral pressure ratio of 0.5, as assumed, during installation. However, this cannot be guaranteed as soils tend to creep with time. It is suggested to start with a pressure ratio of 0.6 to allow for creep. In such a case, according to Fig. 12, an extra amount of wall

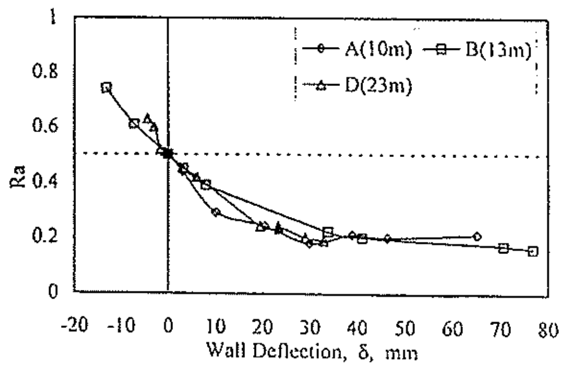

Fig. 12 Lateral earthpressure ratios after adjustment- CPH Building

deflection of 10mm is required for the active state to fully develop.

In the TRTS constructions, a considerable quantity of earthpressure cells have been installed on diaphragm walls. Some of the results are shown in Fig. 13. Because all the deep cuts were retained by thick diaphragm walls and all the struts were preloaded, wall deflections were generally small and only in few cases the lateral pressure ratios reached their lower-bound values.

It is a fact that the magnitudes of lateral pressures on walls are a function of wall friction. The angle of wall friction is often assumed to be a material property but this is incorrect. It depends on the directions of soil and wall movements, the amounts of movements and the properties of soils. Moreover it may vary along the wall. Hence, it is a response and not a property. Schofield (1961) reported observations of passive pressure on a model wall rotating into sand and found a substantial variation of wall friction during a test. A comprehensive series of large passive model tests have been described by Rowe and Peaker (1965) who explored in details the mobilization of wall friction as a function of relative density, wall movement and mobilization of shearing resistance. It is of interest to note that maximum mobilization of wall friction is not necessarily coincident with maximum shearing resistance. The practical significance of knowing more about the movements of a retaining wall and their influence on wall friction is illustrated by the failure of a high crib wall discussed by Tschebotarioff (1965). In this case the wall settled substantially causing the direction of the wall friction to reverse its usual attitude. This effect was sufficient to reduce the factor of safety against sliding along its base from 2.82 to an unsafe value of 0.95 and hence account for the distress which was apparent in the wall.

As indicated in Fig. 11 that in all the four cases shown the lateral earthpressure ratios dropped to a value of 0.2 or so. This value is considerably low for the type of soil involved. The data are compared with the theoretical values proposed by Caquot and Kerisel (1948) in Fig. 14. As can noted that the four points (solid triangles) are below the line corresponding to $\varphi'=\phi'$, in which φ' = angle of wall friction and ϕ' = angle of shearing resistance of soil. The TRTS data, however, scatter over a wide range. There are cases, similar to the case of CPH Building, in which the data points fall below the line corresponding to $\varphi'=\phi'$, but there are also many cases in which the data points fall above the line corresponding to of $\varphi'=0$, implying negative friction angles on the active side of the walls.

The implication of Fig. 14 is unclear at this moment. The scatter of data points could have something to do with the vertical movement of the wall. As shown in Fig. 15, as excavation proceeds, the soils on the active side of a wall tend to drag the wall down. The downdrag and the self-weight of the wall are resisted by the frictional force on the passive side of the wall and the reaction at the toe. It is a fact that sludge is frequently present at the bottom of trench and the resistance at the toe of diaphragm walls is usually minimal, if any. As the excavation goes deeper and deeper, the frictional resistance to wall settlement on the passive side becomes less and less. It will not be a surprise if

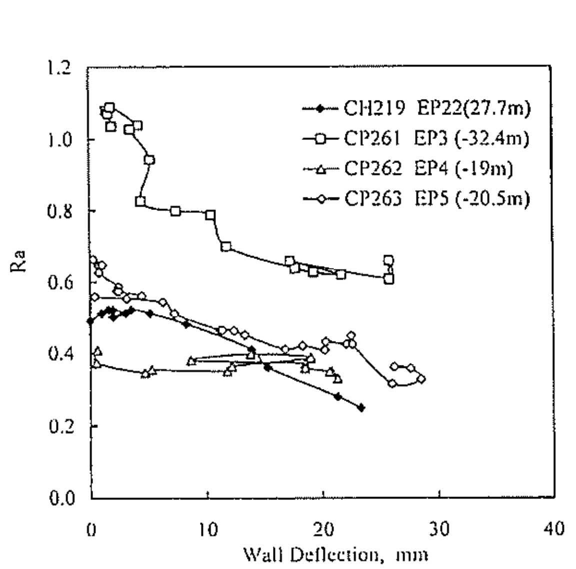

Fig. 13 Earthpressure measurements for TRTS

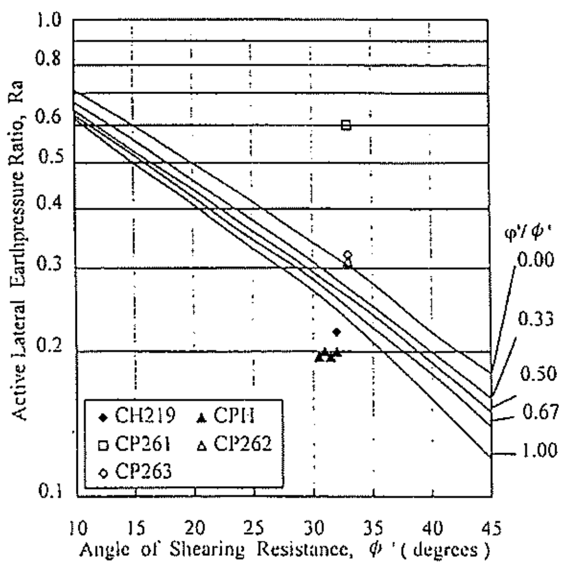

Fig. 14 Lateral earthpressure ratios for Taipei Silts

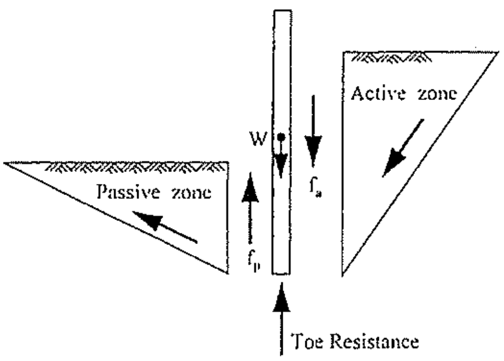

Fig. 15 Force equilibrium on diaphragm wall

some of the walls did settle. As a wall settles, the friction angle on the active side of the wall may drop. If the settlement of the wall is sufficiently large, the friction on the active side may even reverse its direction. This could explain the fact that some of data points in Fig. 14 correspond to very low, even negative, friction angles.

Nearly all the formulas imply a triangular distribution of earthpressures. However, this is far from being true in reality even for the most ideal situation of rigid walls rotating about their toes. Preloading of struts does add to the complexity of the problem. The way the lateral earthpressure ratios varied with wall movement in Fig. 13 is certainly different from what is shown in Fig. 11. Preloading created loops in the Ra-δ curves. Because of the fact that measurements were taken only weekly or even less frequently, what happened in-between readings could have been missed. Therefore, the curves are not as ideal as expected. It is reasoned that more wall deflections are required for the Ra values to drop to their lower bounds. This could be another reason that the Ra values for TRTS excavations are on the high side in Fig. 14.

The phenomenon that some of the data points fall below the line corresponding to $\varphi'=\phi'$ is rather difficult to explain. It could have something to do with the fact that the soils in the Sungshan Formation all have very high fine contents and possess apparent cohesion. The soil parameters obtained in drained tests may not be representative of the soil behavior during excavation. In any case, there are sufficient number of data points to indicate that if conditions are right, the full soil friction can develop on the wall, i.e., $\varphi'=\phi'$. This is of course limited to the cases of soft to medium stiff ground. It has been argued that the presence of bentonite cake at the soil/wall interface tends to reduce wall friction and a range of wall friction angle of $\varphi'=\phi'/3$ to $\varphi'=\phi'/2$ has been proposed by various researchers. The authors are of the opinion that the desiccation as concrete hardens is able to reduce the water content in the bentonite cake to the extent that the strength of the cake equals to the soil strength.

The above discussion does not necessary lead to the conclusion that $\varphi'=\phi'$ shall be assumed in design. As indicated in Fig. 14 that the wall friction varied from one case to another and it is conceivable that it may vary from one depth to another in any particular case. The amount of wall settlement is unpredictable, therefore, a relatively conservation attitude may be justifiable.

It must be pointed out that, although the above discussion serves the purpose of illustrating how complex the problem could be, earthpressure readings must be interpreted with care because measurement of earthpressure is an extremely difficult task, particularly, on diaphragm walls. Apart from the uncertainties associated with the installation, it may also be questionable that the readings obtained by pressure cells indeed represent the pressures on the wall because of the arching effects. In fact, in TRTS constructions, many earthpressure cells failed to give reasonable response even during installation and many became malfunctioned shortly afterward. Those readings which appear reasonable must be examined with care and frequently require corrections.

4.3 Groundwater Control

As mentioned previously that water was responsible for a great majority of failures occurring during excavations. As illustrated in Fig. 16, in the central city areas of Taipei, the ground surface is roughly at an elevation of RL 102.6m (mean sea level = RL100m) while the top of the Chingmei Gravels is at RL 53m or so. The piezometric level in the Chingmei Gravels was at RL 88m, for example, in 1990 when the construction for TRTS started and excavation was able to be carried out to a depth of 26.6m safely if a factor of safety of 1.25 is applied against blow-in. As indicated in Fig. 8 that the piezometric level may rise to RL 98m in 1999 when the construction of the Hsinchuan Line is scheduled to start. By that time, excavation can only proceed to a depth of 20m, instead of 26.6m, without additional measures for controlling groundwater. It should be noted that it has been assumed in the above discussion that a clay blanket is present right at the contact between the Sungshan Formation and the Chingmei Gravels. Such a blanket may be at a higher level at places and the safe excavation depth will be less in such cases.

There are several ways to increase the factor of safety if blow-in is indeed a concern. In constructing the

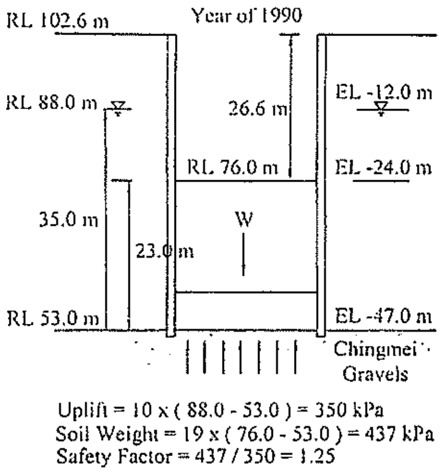

Fig. 16 Analysis for blow-in

ventilation shaft of Contract CH221 of the Hsintien Line, as depicted in Fig. 17, excavation was carried out to a depth of 35m below surface. The shaft is 23.6m in its inner diameter and is formed by 16 diaphragm wall panels of 1.2m in thickness. To prevent blow-in from happening, diaphragm walls were extended by 31m into the Chingmei Gravels, giving a total length of 65m, and a grout seal was formed at the toe level to increase the length of the soil plug. Because of the extreme difficulty in penetrating into the Chingmei Gravels, it took half a year to complete these diaphragm wall panels.

The effectiveness of this grout seal in cutting off seepage flow was confirmed by pumping tests carried out in the soil plug. A 30 Hp submersible pump was installed at RL 60m. Pumping continued for 19 hours and a quantity of 204 cubic meters of water was pumped out. The water level in the soil plug dropped from RL 92m to RL 67m. It rose to RL 72.5m in 6 hours after the pump was turned off. Subsequently, it rose by 1.14m in 32 hours. Based on these data, the permeability of the grout seal was estimated to be roughly 4 x 10^{-7} m/sec.

The wall was supported by two ring beams without internal bracing. Lateral deflections of the wall were within 10mm, which were very low in comparison with the deflections observed for box-shape braced excavations with similar depths, indicating that circular shafts outperformed rectangular ones as far as wall deflection is concerned.

In constructing Ventilation Shaft A (Contract CP262) and Shaft B (Contract CP261) in the Panchiao Line, refer to Figs. 18 and 19, respectively, excavation was carried out to depths of 36.6m and 34m. Pumping was carried out to lower the piezometric levels in the Chingmei Gravels by 10.7m and 9.5m in the two cases and the rates of discharges reached maxima of 4,170 and 3,600 cmh (cubic meters per hour), respectively (Hwang, et al., 1996). Similarly, in constructing the Taipei Main Station, refer to Fig. 20, excavation was carried out to a depth of 28.9m and the piezometric level in the Chingmei Formation was lowered by 5.2m by pumping at a maximum rate of 2,450 cmh.

During the peak of construction, the zone of influence with a drawdown of 2m or greater in the Chingmei Gravels stretched to a radius of 10 km. In carrying out pumping at such a scale, it is important to ensure that ground settlements are limited to within tolerances so the adjacent

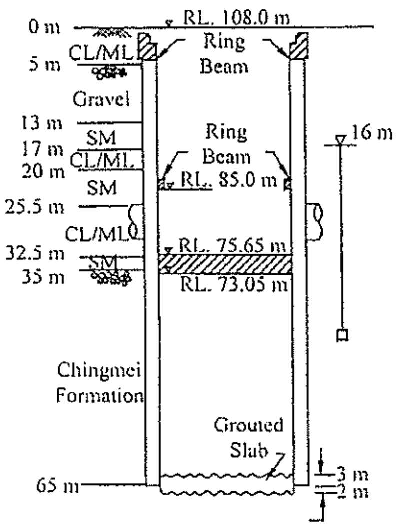

Fig. 17 Grout plug at Ventilation Shaft in Contract CH221

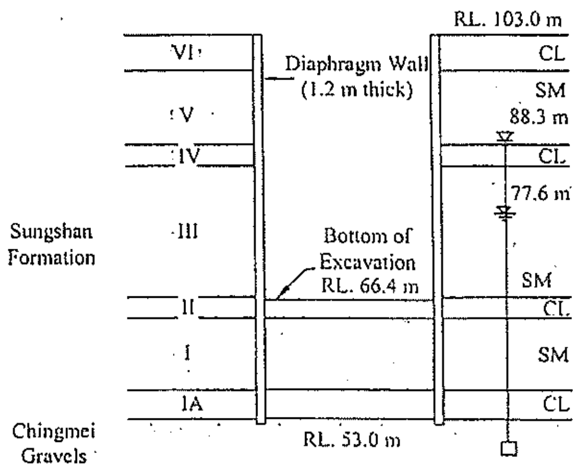

Fig. 18 Pumping at Ventilation Shaft A in Contract CP262

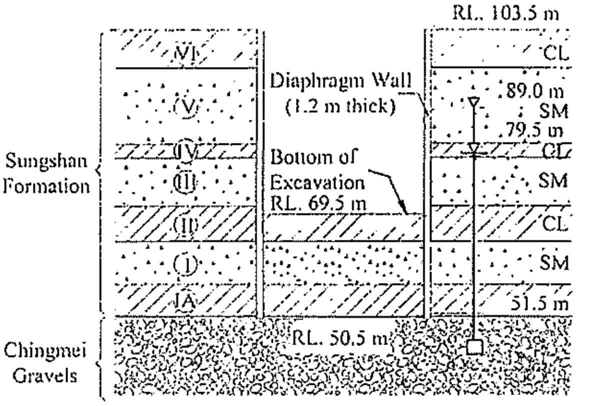

Fig. 19 Pumping at Ventilation Shaft B in Contract CP261

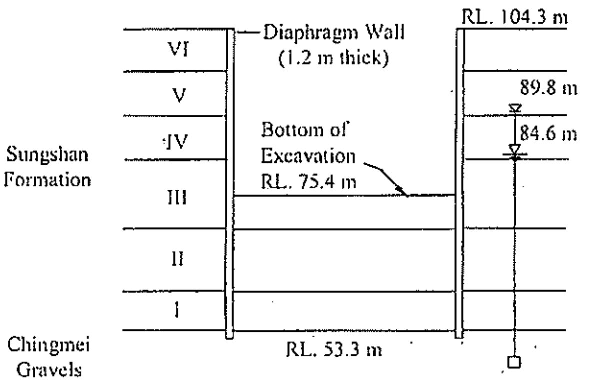

Fig. 20 Pumping at Taipei Main Station in Contract CT201F

buildings will not be damaged. Fortunately, the piezometric levels in Sublayers III and V in the Sungshan Formation, refer to Fig. 9, were not much affected and ground settlements induced as a result of pumping were, in general, well within 10mm.

It may be of interest to mention that in Contract 310 of the Singapore Mass Rapid Transit System, an innovative scheme was adopted to deal with the problem of blow-in in a different way. As illustrated in Fig. 21, excavation was first carried out to a depth of 7m with two levels of struts installed (Hulme, Potter and Shirlaw, 1987; Clark and Prebaharan, 1987). The cofferdam was then flooded with a water level in the cofferdam 2m higher than the ground and the remaining excavation was carried out underwater by dredging. When the final excavation level was reached, bored piles (drilled shafts), 1m in diameter, were installed to act as the station foundation. A 1.5m thick concrete slab was then poured using the tremie method before the cofferdam was dewatered. This unreinforced slab acted as bottom strut and were anchored down by these bored piles. They resisted base heave pressures once the cofferdam was dewatered. The cofferdam was dewatered and the station box constructed. In fact, the entire offshore section of tunnel box between Marina Bay Station and Raffles Place Station was completed in a similar manner.

This scheme was adopted because of the presence of very poor soils to a depth of 40m or so. The site was located in a piece of land reclaimed not too many years before the commencement of construction and ground settlements were continuing when the construction started. The deep excavation, 18m, would require very thick retaining walls and very many levels of struts. Although it was not meant for solving groundwater problem, this method can indeed serve the purpose.

4.4 Lateral Ground Movements

In addition to the safety of retaining structures, a major concern in deep excavations is ground movement outside the excavation. Excessive ground movements may become detrimental to adjacent buildings. One of the major contributors to ground settlement is the lateral deflection of retaining walls. For deep excavations, the lateral deflections of walls are routinely monitored by using inclinometers. Inclinometers are amazingly accurate and can be considered as one of the most reliable types of instruments for geotechnical engineering. However, this does not mean that inclinometers always faithfully report wall deflections. In quite a few cases, inclinometers were not anchored in a stable stratum and toe movements were

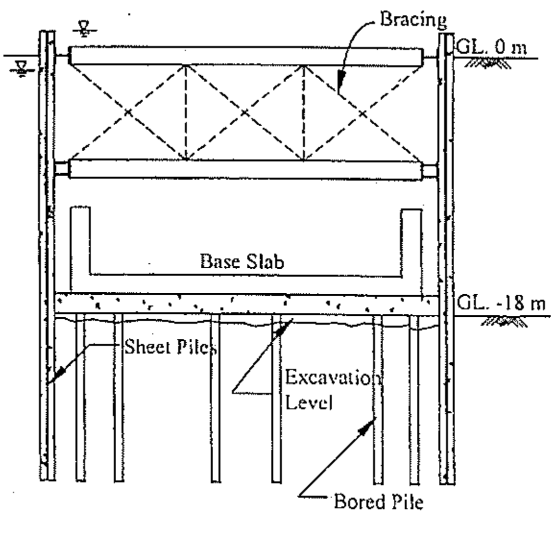

Fig. 21 Flooding at Marina Bay in Contract 310 of SMRT

large. This is particularly true for inclinometers which are cast in diaphragm walls. In such cases, usually, inclinometers are only installed to the toe levels of the walls. The toe of an inclinometer is normally assumed to be fixed and the movements at all other depths are computed in relation to the toe. Although sometimes specifications do require the movement at the top be measured for calibration. In reality, this is difficult to be done. Figure 22 shows the readings obtained for two inclinometers, SID4 (at Sun Yat Sen Memorial Hall Station) and SID6 (at Subway to Chungshan Science Park) of Contract CN256 of the Nankang Line, installed in diaphragm walls, both to the toe levels. Because of the different excavation depths in the two cases, 16.2m and 11.1m, respectively, the diaphragm walls were different in thickness, i.e., 1,000mm and 800mm, and in length, i.e., 53m and 26m. The two sites were both located in the K1 Zone, refer to Fig. 2, and the ground conditions at the two places are apparently similar. When excavation reached a depth of 11.1m, Inclinometer SID4 showed a maximum deflection of 45mm while Inclinometer SID6 showed a maximum deflection of only 20mm. At a depth of 26m, which corresponds to the toe level of Inclinometer SID6, a movement of 30mm was observed by Inclinometer SID4. It is thus suspected that the toe of SID6 would have moved by a similar amount. After correcting the readings of Inclinometer SID6 for this anticipated toe movement, the two sets of readings were very close. The slightly smaller deflection for Inclinometer SID4 could be due to the thicker wall and deeper penetration of the wall.

Whenever a large outward movement is observed, as is the case for the top of Inclinometer SID6 in Fig. 22, an immediate question to ask is "Is the toe moving?". This however does not preclude the possibility of having outward wall movement. Outward movement, if any, could be due to (a) different ground conditions between the two sides, (b) unbalanced excavation, (c) excessive preload of struts, and (d) bulging in of the lower portion of the wall. Figure 23 shows a case in which the site is underlain by an inclined hard pan with a differential depth of 20m between the two sides. Excavation was carried out to a depth of 23.3m and six levels of struts were used. When the first level struts were installed, the deflections of the two inclinometers installed in diaphragm walls on the two sides were practically the same with maximum deflections of about 40mm at the top. However, at the final stage, Inclinometer SI5, refer to Fig. 23(a) on the side with a deeper depth to the bedrock bulged in to give a maximum deflection of 200mm while Inclinometer SI3, refer to Fig. 23(c) on the other side with a shallower depth to the bedrock was pushed outward with a net outward movement of a few millimeters at the top. This was obviously a result

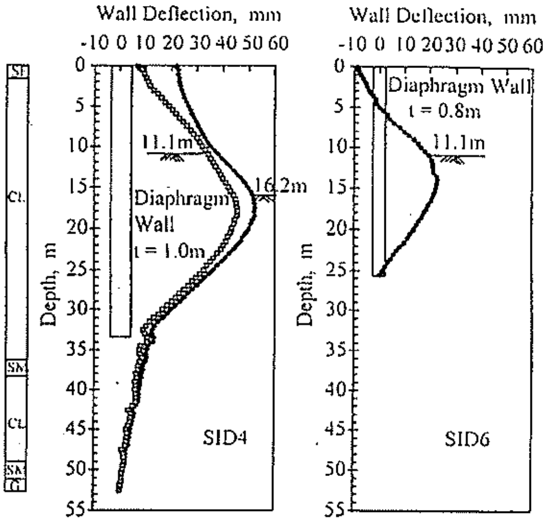

Fig. 22 Effects of toe stability on inclinometer readings

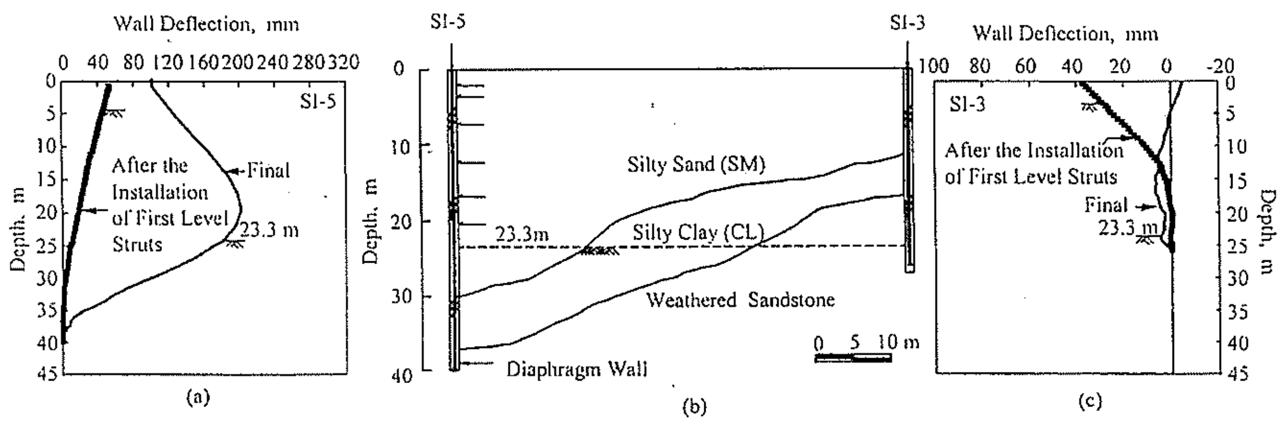

Fig. 23 Effects of geology on wall movement

of imbalance of pressure on the two sides.

To assist the interpretation of inclinometer readings, it is suggested to install inclinometers in pairs, one at each end of the same strut so the wall deflections can be double-checked. It is not difficult to show that once a strut is preloaded to, say, 50% of the design load, the subsequent shortening shall be very minimum. Therefore, the two ends of a strut shall move by the same amount subsequently but in the opposite directions.

Because of the stringent limitation on ground settlements for the purpose of limiting damaging potential to adjacent buildings, the diaphragm walls used in the TRTS are, in general, thicker than the walls used previously in Taipei by 200mm or so. This, together with the heavy preloading and strict supervision, reduced the lateral movements of retaining walls in TRTS to only 30% to 50% of what were reported in the old days. Because damages to a building may result in disputes, court injunction, and many undesirable consequences, in comparison, the extra 200mm thickness of diaphragm wall is indeed cost-effective.

4.5 Ground Settlements

Ground settlements induced during deep excavations have been extensively studied by many researchers, and for this reason, it is not intended to discuss ground settlements in detail herein. Recent studies indicate that the geometry of the pit has profound influence on the pattern of ground settlements. Figures 24 and 25 show the ground settlements as a function of distance to the corner of excavation, the so-called corner effects (Wong and Patron, 1993). In the former figure, settlements were normalized to the maximum settlements in individual sites and the distances were normalized in respect to the lengths or widths of individual excavations. In the latter figure, the distances were normalized in respect to the excavation depths. Both figures indicate that ground settlements at corners are about 20% of the maxima and as the location moves away from the corner and towards the mid-span of excavation, ground settlement increases. This finding is in line with those obtained by others (Wong, 1987; Ou and Chiou, 1993).

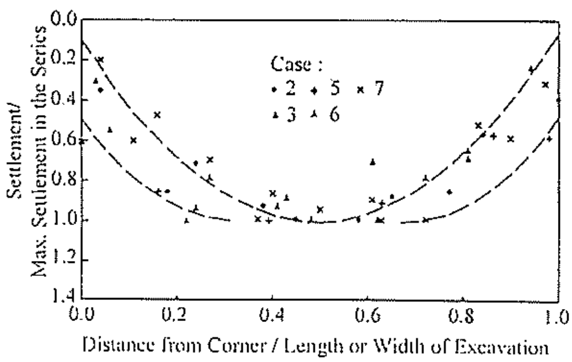

Fig. 24 Corner effects on ground settlement

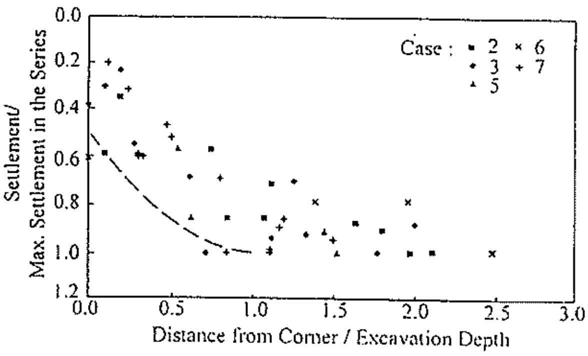

Fig. 25 Corner effects for long excavation

It should be noted that presumably the data presented in these two figures include only settlements induced by excavation and excluded settlements induced by diaphragm wall installation. Figure 26 shows a case in which the settlements at the corner of a building induced as a result of installation of diaphragm wall panels for a subway station and the entrance accumulated to 30mm. The influence of each panel was found to stretch to a distance of 10m and there were a total of nine panels within this distance. In a few incidents, digging trenches, usually 2m or so in depth, in very poor ground, or in poor backfill, for constructing guidewalls led to settlements of 30mm or so. It is therefore suggested that monitoring of building settlements shall start before the installation of diaphragm walls.

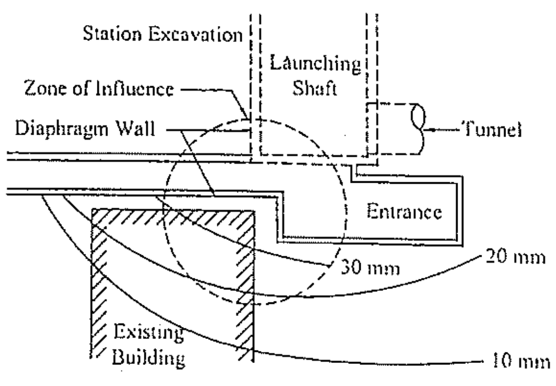

Fig. 26 Settlement due to diaphragm walling

5 TUNNELLING

Tunnels linking subway stations are usually constructed using: a) cut-and-cover method, b) NATM tunnelling method and c) shield tunnelling method. The choices are governed by tunnel configuration, ground conditions, traffic conditions, etc. The cut-and-cover constructions for tunnels are not different from the cut-and-cover constructions for stations and thus will not be further discussed.

5.1 NATM Tunnelling

In this part of the world, the so-called "New Austrian Tunnelling Method" (NATM) appears to have deviated from its original context of being merely a concept in the spirit of the observational method for tunnelling and have been extended to mean all types of tunnelling without using a shield. Such a deviation does solve the dilemma of lacking a proper name for tunnelling in soft ground other than shield tunnelling.

The New Austrian Tunnelling Method was used in the TRTS to drive two adjoining sections, one of 225m in length and the other of 487m in length in the Mucha Line. The two sections are separated by a short cut-and-cover section of only 32m in length. The twin tunnels have a horse-shoe shape with heights varying from 6.09m to 7.14m and a base varying from 9.17m to 9.48m in width.

The twin tunnels run through highly fractured shale and were lined with shotcrete of 100mm in thickness and wire mesh. Steel ribs were installed only as necessary. Rock bolts, 29mm in diameter and 4m in length, were installed at 1m intervals along the longitudinal direction and at 2m intervals along the transverse direction. Secondary lining was made of 300mm in-situ concrete. Excavation was carried out without major problems except that the roof caved in accidentally when an abandoned passageway was encountered above the crown. This passageway was once used for coal mining and was abandoned years before the TRTS construction (Guo, Yeh and Cheng, 1992). As a result, a volume of 50 cubic meters of debris fell into the tunnel.

The convergences of sections were closely monitored and they varied between 10mm to 20mm except that a maximum of 40mm was observed in one of the sections. Settlements of the crown were generally less than 40mm while a maximum of 63mm was observed in one of the sections.

A section the Hsintien Line was also driven using the NATM tunnelling method (Huang, 1997; Yang, et al., 1997). The section is 222m in length and is too short for shield tunnelling. Figure 27 shows a cross section of the tunnels. The tunnels were bored through Sublayer 5 of the Sungshan Formation with the crowns at depths of 8m to 11m below surface.

Excavation was carried out in two headings in each tunnel drive. The upper heading was kept at a distance of 2m to 4m ahead of the lower heading. Lattice girders were installed at 1m intervals and the tunnels were protected by shotcrete, 250mm in thickness, and wire mesh as primary lining. For maintaining the stability of the headings, steel lagging sheets, 6mm in thickness, 200mm to 300mm in width and 2m in length, were closely spaced to make a canopy. The tunnels were finally lined by 350mm reinforced concrete as permanent lining.

The soft ground called for the use of compressed air to a maximum of 1.35 bar. Construction was carried out in such a way that the two tunnels were inter-connected, as shown in Fig. 28, by a cross drift so that both tunnels were able to be pressurized by using a single set of compressed air facility. Also shown in the figure is the sequence of excavation. Excavation was carried out in five stages. Stage 1 excavation was carried out in free air for providing a space to house the compressed air plant. The rest of excavation was carried out in compressed air. Air pressure was not released till both tunnels were fully excavated and primary lining was completed.

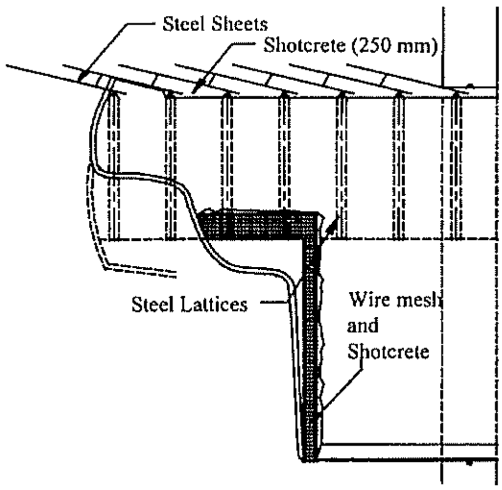

Fig. 27 Profile for NATM tunnels in Contract CH221

The consumption of compressed air was about 110 cubic meters per minute, refer to Fig. 29, when tunnelling was carried out in the Up-Track tunnel in the Stages 2 and 3 excavation before a layer of gravel was first encountered at the face at the halfway of the drive. It increased to 270 cubic meters per minute by the time the heading reached the end of drive. It was maintained at 170 to 190 cubic meters per minutes during the Stage 4 excavation for the Down-Track tunnel. Again, as the gravel layer was encountered, the air consumption increased to a maximum of 280 cubic meters per minute and the four compressors, with a power of 340 kilo-watts each, was fully loaded. As the tunnels were fully lined, the air consumption dropped to 140 cubic meters per minute.

Tunnelling was completed not without problems. Pressurized air traveled to a distance of as much as a couple of hundred meters and escaped to the ground through fissures and/or poorly backfilled utility trenches. An emergency situation was encountered on 29 March, 1994 when air escaped through the fissure left in place after sheet piles were withdrawn and the cracks in the base slab of a pedestrian underpass, refer to Fig. 30, and carried much water and solids into the underpass. As a result, a sinkhole of 70 cubic meters in volume was created at ground surface. Five days later, a minor explosion, presumably, due to the ignition of gas leaking from a gas line, shook nearby houses and broke window glass. The gas could have accumulated in a covered box culvert to a sufficient concentration for ignition. In fact, the explosion occurred at a location quite far away from the tunnel alignment and the steel covers of a couple manholes were blown off as a result of explosion. Although damages were minimal, it did cause panic of local residents. As a precaution, the pressure of compressed air was lowered from 1.2 bar to 0.4 bar and maintained at that level for about half a month. The excavation was suspended and the method of construction was carefully examined. It was resumed 4 months later after the situation was judged to be stable and the safety of the works was assured.

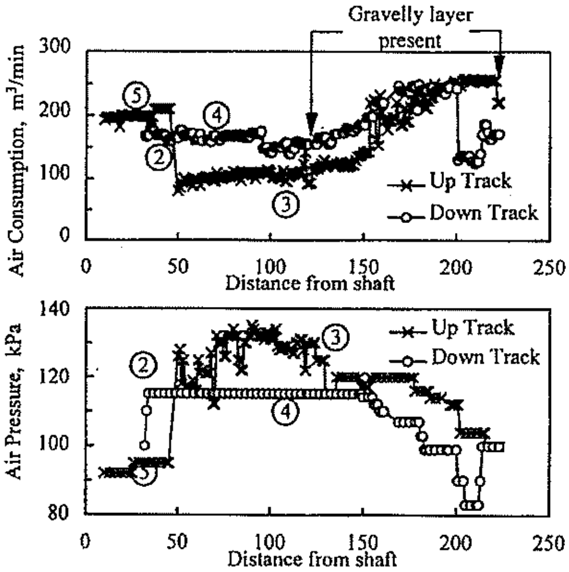

Fig. 29 Pressure and air consumption for NATM tunnelling

The most scary incident occurred on 22 March, 1995 when a transformer malfunctioned and disrupted the electrical supply. Air pressure dropped to 0.2 bar in 12 hours before the transformer was replaced and the electrical supply was back. At that time, refer to Fig. 28, Stage 3 excavations had already been completed and the heading was at Ch 0+180 of the Down-Track tunnel. In other words, there was a length of 352m of tunnel already been driven. Although the primary lining was designed to hold the tunnels even without compressed air, the face did rely on compressed air to stand up. A 2000 mm diameter water main runs across the two tunnels and supplies water to the entire Taipei City. Should it rupture, the consequence would be disastrous. Fortunately, the face had been stabilized by grouting for reducing the loss of compressed air. The contractor was able to replace the transformer promptly and

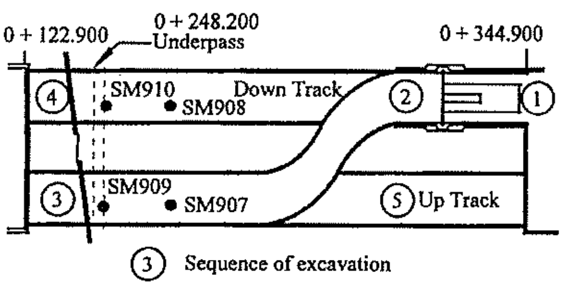

Fig. 28 Plan of NATM tunnels in Contract CH221

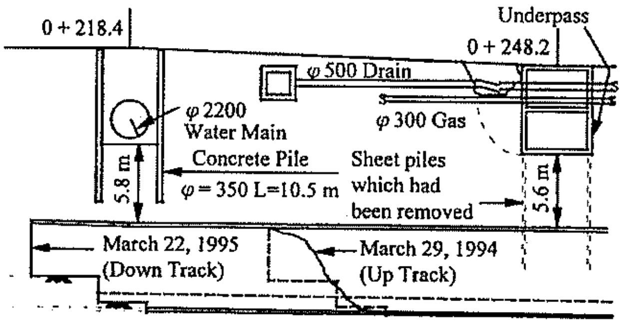

Fig. 30 Incidents occurred along the route of NATM tunnels

the crisis was resolved without even minor damages.

Shown in Fig. 31(b) are the readings of air pressuremeter AP-908, which was buried at a depth of 14m. Figure 31(c) shows the variations of water levels in two observation wells, i.e., OW-905 and OW-906, of which the tips were buried in Sublayer V of the Sungshan Formation. In general, the water levels moved in phase with the air pressures. The compressed air was totally released on July 25, 1995. The Up-Track tunnel was lined in the period of February to October, 1995 and the Down-Track tunnel was lined in the period of July, 1995 to February, 1996. As can be noted that as the tunnels were lined, the water levels in the Sungshan Formation gradually returned to their natural level of RL 102m.

Figure 31(d) shows the time histories of ground settlements. Settlement Markers SM-909 and SM-910 were located on top of the pedestrian underpass and showed much smaller settlements in comparison with others. The final ground settlements were between 120mm to 170mm which are about 2 to 3 times of the settlements induced by shield tunnelling without compressed air. Whether the ground settlements would have been the same had the two incidents not occurred is arguable. The two incidents did result in two unexpected cycles of compression-and-decompression. However, at the times when these incidents occurred, the excavated tunnel drives had already been supported by steel lattices and lined by using shotcrete and wire mesh. This situation was not any different from the situation when the compressed air was totally released at the end. It is unlikely that the final settlement would have been much less than what was observed.

A few workers who worked in the tunnels for a considerable duration, suffered from diver's disease (aeroembolism) due to improper decompression and had to be treated. This has raised serious concern by the Labors' Commission and the City Council of Taipei. The use of compressed air in another contract which was still ongoing when the problem surfaced was banned. Therefore, it is doubtful that compressed air tunnelling would gain a broad acceptance in Taiwan in the future.

Despite the fact that NATM tunnelling in soft ground is popular in Europe, this is the first time it was carried out in Taiwan. The method cast serious doubt when it was first proposed. Notwithstanding all the problems, purely from a technical point of view, the method was proved to be a success. The twin tunnels were driven with less disruption to traffic and less construction cost in comparison with the cut-and-cover method. However, it has to be admitted that the operation is highly risky and any major accident would put the designer in an extremely difficult position defending himself.

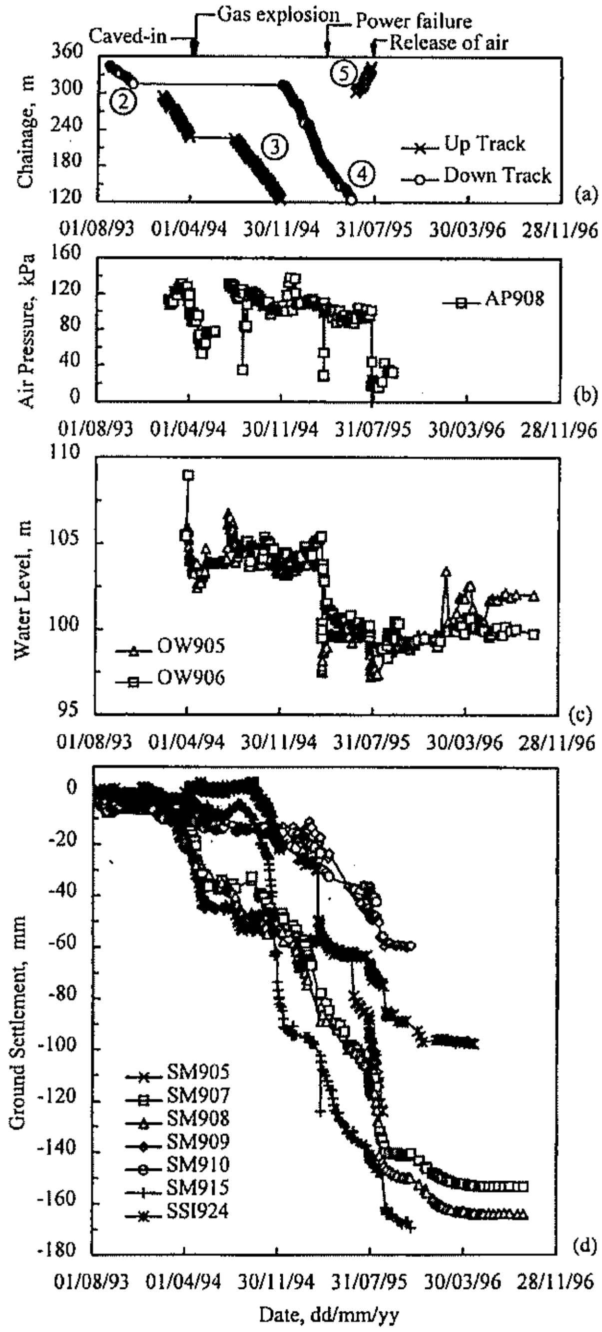

Fig. 31 Progress of NATM tunnelling and instrument readings

5.2 Shield Tunnelling

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Geotechnical problems related to design and construction of the Taipei Transit Systems

Up to this year, there are a total of 58 drives in the Initial Network of TRTS excavated by shield tunnelling and 30 shield machines have been adopted. Except the two slurry shield machines, which were used to drive the four tunnels in the Contract CH221 of the Hsintien Line, all other contractors used earthpressure balancing shield machines for the job. Notwithstanding, all the earthpressure balancing shield machines were equipped with facility of injecting slurry or chemicals to deal with difficult ground.

Apart from the problems with methane and drift woods, the ground conditions in the Taipei Basin are ideal for shield tunnelling. The silty sand and silty clay in the Sungshan Formation can be handled by either earthpressure balancing shields or slurry shields with ease. The progress was in general satisfactory and a daily production rates of 10 to 20 rings, 1m in length each, were quite normal. A maximum daily rate of 47 rings was achieved in one of the tunnel drives in the Chungho Line. This impressive rate was achieved without any particular measures taken other than the incentive given to the crew.

There were a few incidents which led to serious consequences during tunnelling (Lin, Ju and Hwang, 1997; Moh, Ju and Hwang, 1997). Nearly all of them occurred either upon launching or upon arrival of shield machines and groundwater was a major source of problem. As openings are made on diaphragm walls, refer to Fig. 32, to prepare for launching or arrival of shields, water tends to leak into the shafts from gaps behind the retaining walls. The Taipei Basin was once a giant lake and was infilled by sediments not long ago. Sublayers 3 and 5 in the Sungshan Formation are very permeable and the Chingmei Formation underlying the Basin is an ideal aquifer with ample water reserve. If an opening is made at a location very close to the Chingmei Gravels, chances are, ingress of water may soon become uncontrollable.

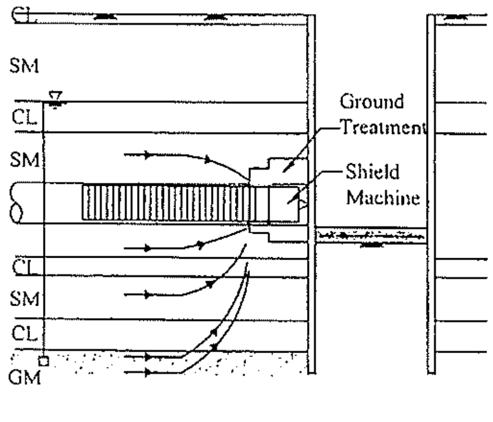

Fig. 32 Ground treatment and potential problem at portal

To prepare for launching or arrival of shield machines, it is a common practice to form protective annular shelters for the shield machines to temporarily stay by treating the ground with high pressure jet grouting. The quality of ground treatment is certainly of great importance and has to be confirmed by coring and tests. Strength and permeability of the treated ground are the two parameters frequently specified. However, it has been found that check borings did not reveal all the potential problems. Particularly, the results of unconfined compression tests tend to lead to misjudgment because only good specimens are tested. Furthermore, in most of the incidents, strength of the treated ground is irrelevant. In fact, too-high strength of the treated ground was responsible for a few incidents in which the advancement of shields was hampered and sinkholes were formed in front of the faces as soils kept on moving into the earth chambers. One way to prevent such a problem from occurring is to add a 2m to 3m extension at the end of the block with weaker strength.

It is the permeability of the treated ground that matters. The conventional type permeability tests are not useful at all. Even Lugeon tests do not necessarily provide meaningful results. In the authors' opinion, the integrity is a better indication of the quality of treated ground and the integrity of treated ground is better represented by core recovery ratio and rock quality designation (RQD). Unfortunately, criteria have not been established using these two indices. In the lack of precedents, the authors wish to suggest a core recovery ratio of 90% and a RQD of 60% as the minimum requirements for ground treated using jet grouting. This is subject to discussion and suggestions are certainly welcomed.

For reducing the risk associated with seepage flow, it is more effective to increase the length of water path than to reduce the permeability of the treated ground. It is thus very necessary to have extra length and extra thickness of the ground treatment whenever the tunnel-station connection is very close to a water-bearing stratum. As a general rule, never rely on a single row of jet columns, no matter how large the diameter is, to serve as a curtain for cutting off seepage flow. The verticality of drilling is difficult to ascertain and it could well be gaps between columns. A minimum of two rows is absolutely necessary and, at locations where consequence of seepage could be serious, three rows are suggested.

As an excavation proceeds, the wall at the tunnel-station connection tends to move inward. Because the block of treated ground is relatively rigid, gap may occur behind the wall to become a water path. It is therefore necessary to drill a few holes on the wall to see if water does flow. It will be a good idea to apply sealing grouting anyway. The cost is really minimum and many problems can be avoided beforehand.

5.3 Ground Settlement over Tunnels

Settlements over tunnels are usually analyzed by using the Peck's method by assuming that settlement troughs are in a shape of normal distribution (error function), refer to Fig. 33 (Peck, 1969):

$$\delta = \frac{vA}{2.5i}\exp\left(\frac{-x^2}{2i^2}\right) \qquad (1)$$

$$v = \frac{2.5i\delta_{max}}{A} \qquad (2)$$

where

δ	= ground settlement
δ_{max}	= maximum ground settlement
x	= distance to tunnel center
i	= width parameter
v	= ground loss
A	= sectional area of the tunnel

Performance of tunneling is usually evaluated in terms of ground loss which is the ratio of the area of settlement trough to the sectional area of tunnel. For soft ground, long-term consolidation settlements could be significant. Therefore, it is important to differentiate immediate settlement from long-term settlement because they have different mechanisms and are governed by different factors. Furthermore, since long-term settlement may drag on for months, settlement troughs in different cases cannot be compared fairly unless a common criterion is adopted in determining the portion of settlements to be included in the evaluation. Figure 34 shows an idealized time history of settlement in a semi-log scale with the elapsed time after passing of the shield in the abscissa. For simplicity, the straight portion of the plot toward the end can be considered as long-term settlement which may be attributed primarily to consolidation and the preceding settlement can be considered as immediate settlement which is primarily a result of the closure of tail void. Studies showed that the transition occurred in four to ten days after the passing of the shield machine. For all practical purposes, it is appropriate to take the settlement in 10 days as immediate settlement and denoted as δ_{10} (Hwang, Moh and Chen, 1996; Hwang, Sun and Ju, 1996). Consolidation settlement is represented by the slope of the curve toward the end, and for all practical purposes, can be taken as the settlement occurring between the 10th day and the 100th day after the passing of the shield.

Ground loss can be computed by using Eq. (1) based on settlement readings. However, settlement readings do not necessarily fall on a nice curve as expected and it is frequently necessary to find the best-fit curves by using a trial-and-error process. This may serve practical purposes but the results may be subjective. Furthermore, the process does take time if the quantity of data is large. A more rational and less time-consuming method is thus desired. First, Eq. (1) is transformed to a linear equation,

$$t = ms + b \qquad (3)$$

by letting

$$t = \ln\delta \qquad (4)$$

$$s = X^2 \qquad (5)$$

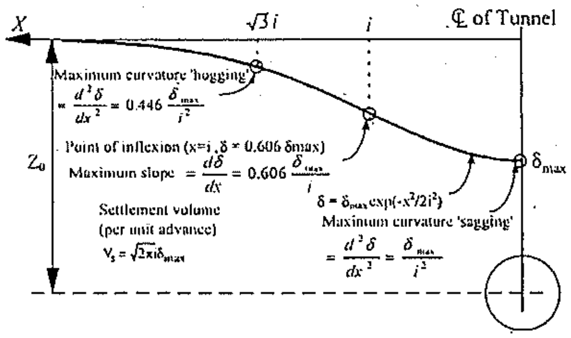

Fig. 33 Idealized settlement troughs above tunnels

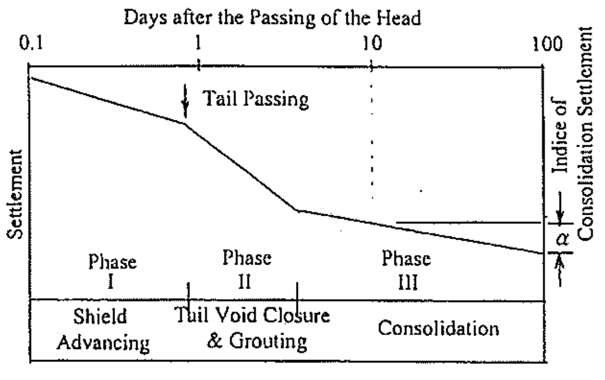

Fig. 34 Idealized time history of settlements above tunnels

The two parameters, m and b, which are the slope and the zero-intercept of the line, are readily obtainable from regression analyses using spreadsheet computer software, such as EXCEL or Lotus 123. The width parameter, i, and maximum settlement, δ_{max}, can be computed as follows:

$$i = \sqrt{\frac{1}{m}} \tag{6}$$

$$\delta_{max} = \exp(b) \tag{7}$$

It has been noted that most of the settlement troughs are not symmetrical to the center of the tunnel as the theory assumes. The deviations may be due to spatial variation in soil stratum, imbalance of earthpressure in front of face, back grouting, and secondary injection, etc. It is also logical to expect such deviations in sections involving horizontal curves because of the use of copy cutters. In any case, such eccentricity shall be adjusted for the regression analyses to be correct. First, let

$$X = x + \varepsilon_0 \tag{8}$$

where

ε_0 = deviation of the center of trough from the tunnel center

and choose the ε_0 value which gives the greatest "correlation coefficient" by the trial-and-error process.

Figure 35 shows the settlement trough obtained at Ring No. 224 of the Down-Track tunnel, driven from NTU Hospital Station toward CKS Memorial Hall Station, of Contract CH218 of the Hsintien Line. At this section, a deviation of ε_0 =2.15m gave the best fit of data. Part of this deviation was due to the use of copy cutter for making the turn. The radius of gyration in this section is 380m and a copy cutting of 40mm to 60mm was used. For an outer diameter of 6,050mm for the shield and 5,900mm for the segment, a theoretical eccentricity of 0.5m can be expected from over-cutting alone. The remaining eccentricity could be due to different ground conditions on the two sides of the tunnel alignment and other factors which are yet to be quantified.

Ground loss, v, and width parameter, i, are the two parameters representing a settlement trough. They are needed for estimating ground settlements in the design. These two parameters can be correlated with ground conditions and other factors such as tunnel depth, method of tunnelling, etc. Empirical relationships in various types of soils have been proposed by many researchers. However, their validity to local soils has to be examined.

Figures 36 shows the soil profiles for Contracts CH218 and CH223 of the Hsintien Line and CP261 of the Panchiao Line. These three contracts are located in Zones T2, T1 and B2, refer to Fig. 2 for geological map, respectively. The trough width parameters of these three contracts are compared with the charts proposed by Peck (1969), Clough & Schmidt (1981) and O'Reilly & New (1982) in Fig. 37. To be honest, although all the data points fall in the ranges proposed by these researchers, it would have been difficult to predict what has been observed based on the simple soil classifications given. This is quite understandable because,

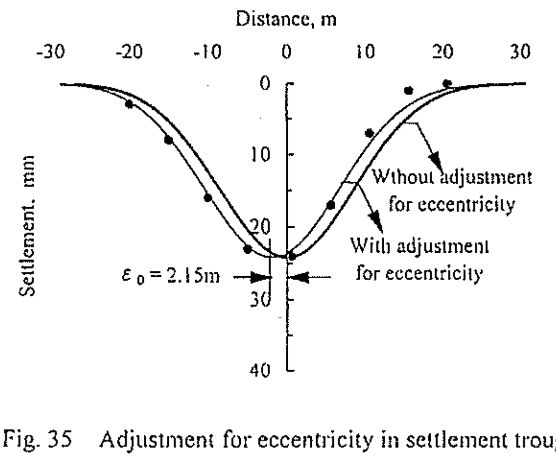

Fig. 35 Adjustment for eccentricity in settlement troughs

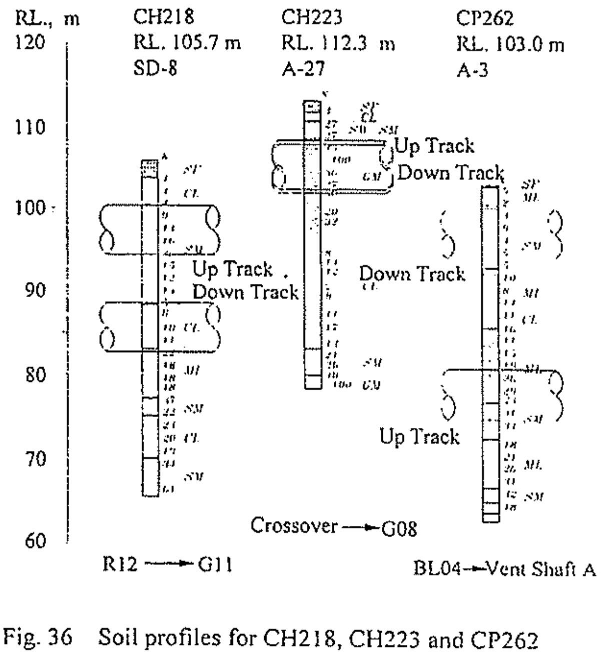

Fig. 36 Soil profiles for CH218, CH223 and CP262

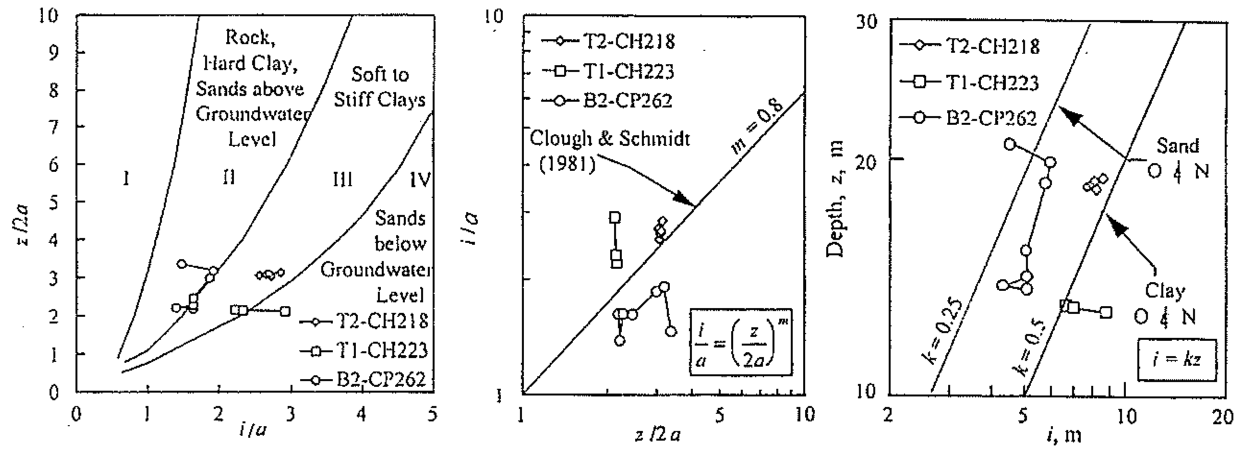

Fig. 37 Estimations of width parameters using Peck, Clough & Schmidt and O'Reilly & New's relationships

apart from the obviously different geological origins, very few of the case histories included in these previous studies involved the use of earthpressure balancing shields. Furthermore, considerable amount of consolidation settlement could have been included in the previous studies while only immediate settlements occurred in 10 days are included in this study.

Ground loss in TRTS was studied by quite a few researchers. Table 1 is a summary of the results. It should be noted that in many of these studies the width parameters were estimated by using the relationship proposed by Clough & Schmidt and the results, strictly speaking, may not necessarily be accurate.

Table 1 Ground Loss and Indices of Consolidation Settlement

Contract	v (%)	α (mm)	Reference
CC275	0.5-1.9	1-4	Lin, Chang and Chu (1997)
CC276	0.7-2.3	6-18	Lin, Chang and Chu (1997)
CC277	sand: 0.3-1	3-16	Lin, Chang and Chu (1997)
	clay: 1-1.6	3-10	Lin, Chang and Chu (1997)
	1.4-2.6	9-15	Hu and Hsieh (1997)
CH218	1.3-2.0	4-6	Yang, Wang and Fan (1995)
	1.7-1.9		Hwang, et al. (1997)
	1.1-1.6		Hwang, Sun & Ju (1996)
CH223	0.6-1.1	1-3	Yang, Wang and Fan (1995)
	0.9-1.0		Hwang, et al. (1997)
CN256	0.2-1.5	5-14	Hwang, Sun & Ju (1996)
CN258	0.5-2.0	4-14	Wu, Zhuang and Wang (1997)
CP262	0.5-1.6		Hwang, et al. (1997)
	0.9-2.0		Hwang, Sun & Ju (1996)

5.4 Ground Heave

Ground heaves were observed at quite a few locations during tunnelling. In the past, heaves were generally attributed to the excess pressure on the face (Yi, Rowe and Lee, 1993). Recent evidences indicate that grouting for filling tail voids might have played a by far more important role than face pressures in inducing ground heaves in tunnelling using earthpressure balancing shields. To start with, face pressures (rather, the pressures on bulkheads) are usually kept at, or attempted to be kept at, levels corresponding to at-rest lateral earthpressures and a review of literatures confirms this practice. In such cases, it is highly unlikely for face pressures of such magnitudes to induce ground heaves of meaningful amounts. On the other hand, line pressures for back grouting usually vary from 2 bars to 4 bars regardless of the overburden pressures. It is quite conceivable that, as illustrated in Fig. 38, for shallow tunnels, ground heaves could well be a result of grouting.

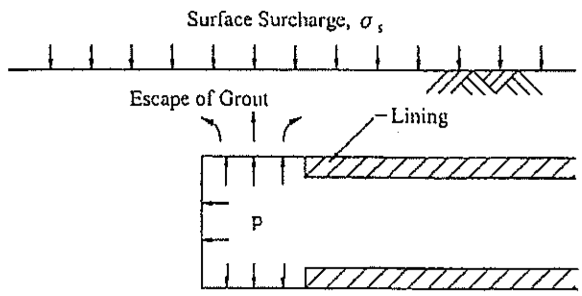

Fig. 38 Effects of back grouting on ground heave

Figure 39 shows the soil profile at Section T3 of Contract CN256 of the Nankang Line. The twin tunnels, with horizontal alignments, are embedded in the Sungshan Formation at a depth of 12.5m and are surrounded by highly silty clays and clayey silts with a representative water content of 32%, a unit weight of 19 kN/m^3, and an OCR value of 1.3. The two tunnels were driven by using a spoke type earthpressure balancing shield with a diameter of 6,050mm and a length of 7,180mm. The outer diameter of the segmental linings is 5,900mm.

Three settlement markers, i.e., SM51, SM52 and SM53, were available at a section corresponding to Ring No. 98 and the readings are shown in Fig. 40. Also shown in the figure is the progress of tunnelling so the ground movements can be correlated with the tunnelling activities. Ground was quite stable during the passing of the head of the shield in the Down-Track tunnel. Field records indicate that in the period in which the face passed through the space occupied by Ring Nos. 94 to 99, the total thrust varied from 810 to 900 tonnes and the average quantity of slurry injected into the earth chamber was 1,718 liters per ring and the average quantity of sludge removed was 28 m^3 per ring.

Earthpressures on the bulkhead were monitored by 4 pressure cells and the readings varied from 170 kPa to 190 kPa during shoving while the overburden pressure at the springline was about 238 kPa for a unit weight of 19 kN/m^3 and a depth of 12.5m. Ignoring the excess pore pressures

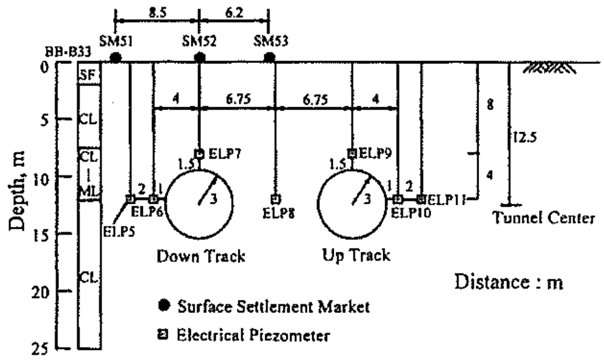

Fig. 39 Profile for Section 256T3 and instrument layout

induced, the coefficient of lateral earthpressure would be between 0.47 and 0.63 for a hydrostatic pressure of 110 kPa at the springline corresponding to a groundwater table of 1.5m below the surface.

When the tail of the shield passed Ring 98, heaves of 3mm to 5mm were observed, presumably, as a result of back grouting. Grouting was automatic and synchronized with shield advancement with line pressures varying from 250 kPa to 400 kPa. A pressure of 400 kPa, if indeed developed in the grout, was more than sufficient to cause hydraulic fracturing at the crown for an overburden pressure of 180 kPa and a laboratory undrained shear strength, s_u, of 40 kPa. The average take of the grout for Rings 92 to 99

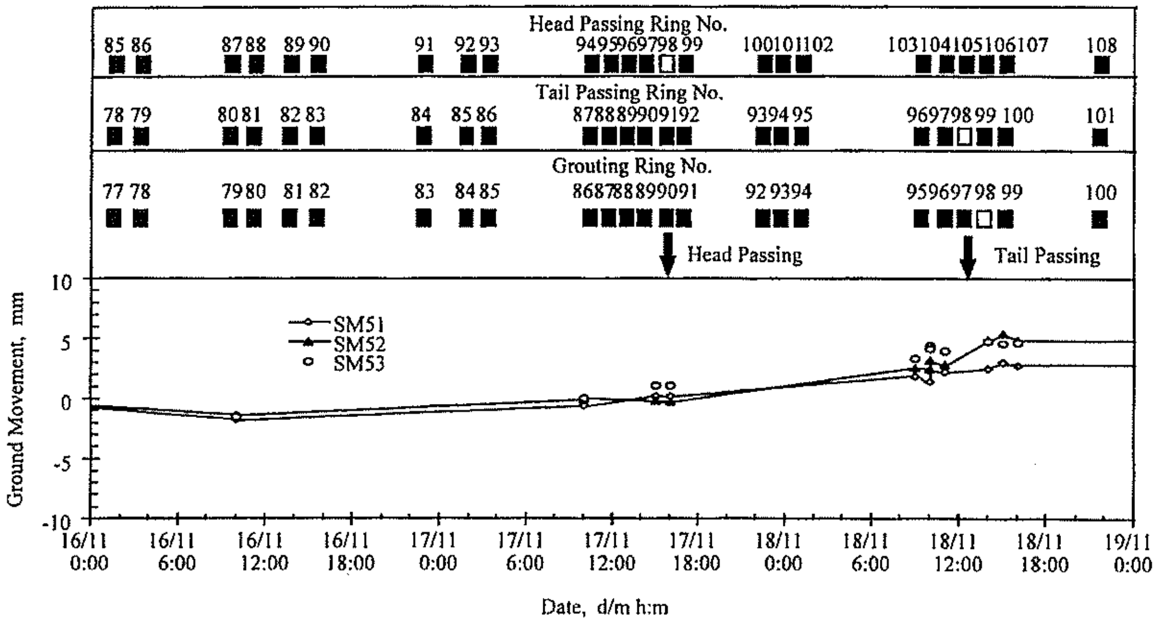

Fig. 40 Ground heave at Section 256T3

was 1,810 litres while the theoretical volume of tail void was 1,598 liters per ring of 1m in length (assuming a 10mm over-cut). It is thus conceivable that some of the grout did penetrate into the ground and cause the ground to heave up. In fact, grout escaped through fissures at Section T4, refer to Fig. 41, at a distance of about 600m to the east of Section T3 during grouting carried out in the Up-Track tunnel.

The overload factor at the tunnel crown is of an order of 4.5 for an overburden pressure of 180 kPa and a s_u value of 40 kPa, therefore the ground was expected to close in at the tail as the shield advanced leading to ground settlement. This did not happen and, on the contrary, heaves were observed. It is thus postulated that the grout pressurized the tail void all the time and prevented the closure of tail void. Back grouting synchronizing with shield advancement and carried out at deliberately high pressures essentially functions as compensation grouting. The long-term settlements can be read from Fig. 42. Notwithstanding the high grouting pressures, the subsequent consolidation settlements were pretty small. Indices of consolidation settlement, refer to Fig. 34, were only 3mm to 5mm which, as indicated in Table 1, are on the lower side of the values given therein.

6. CONCLUSIONS

The above discussions lead to the following conclusions:

(i) Groundwater plays a dominating role in underground constructions and groundwater conditions have to be fully explored before design starts. Long-term monitoring of groundwater movements is necessary in areas experiencing groundwater drawdown.

(ii) Geological features which are unique to an area have to be clearly described in tender documents as international designers and contractors may not have sufficient local

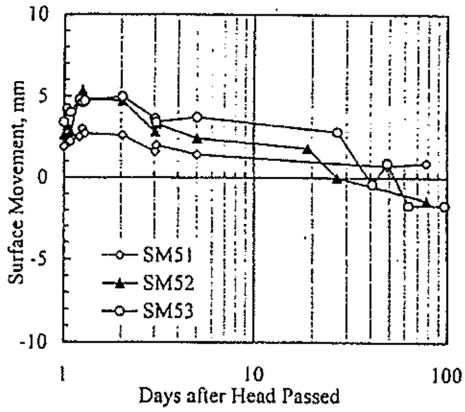

Fig. 42 Ground movements at Section 256T3

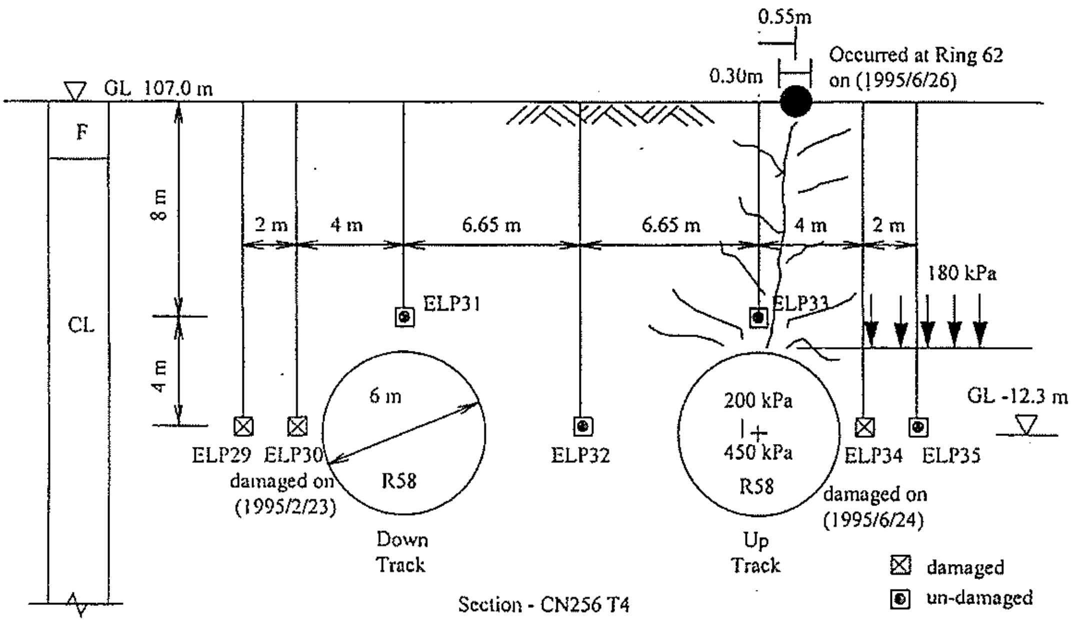

Fig. 41 Escape of grout at Section 256T4

knowledge and may misjudge ground conditions.

(iii) Earthpressures on retaining walls of braced excavations are dependent on wall movements and are difficult to be ascertained.

(iv) For deep excavations, blow-in may become a major problem if there exists a water bearing aquifer under the bottom of excavation. . Three measures are presented herein, i.e., lengthening soil plug, lowering water head and flooding the cofferdam.

(v) Both slurry shields and earthpressure balancing shields are suitable for tunnelling in soft ground. Major problems in tunnelling are associated with making openings on diaphragm walls to prepare for launching and/or arrival of shields. Cautions must be exercised if such openings are close to water-bearing strata.

(vi) Peck's approach of approximating settlement troughs by error functions is suitable for analyzing settlement troughs over tunnels. However, local experience is needed for correlating ground loss and width parameter with ground conditions.

(vii) Ground heave over tunnels may be a result of grouting for filling the tail void. It is most likely to occur to shallow tunnel drives overlain by soft clay.

In summary, quality of soil investigation, instrumentation and ground treatment is of primary importance and experienced geotechnical engineers must be engaged in supervision of the field works and interpretation of the results.

ACKNOWLEDGEMENTS

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UNDERPASS BENEATH TAIPEI INTERNATIONAL AIRPORT

by
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UNDERPASS BENEATH TAIPEI INTERNATIONAL AIRPORT

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ABSTRACT

A 4-lane highway is to pass underneath a taxiway and a runway of the Taipei International Airport located to the north of the city center of Taipei. The requirement that air traffic shall in no case be interrupted during the construction necessitates the adoption of special construction techniques. Working shafts are constructed and tunneling is carried out between these shafts. The tunnel is sheltered by interlocking steel pipes installed by using the pipe-jacking technique. Under the taxiway and the runway, the so-called Endless Self Advancing Method (ESA Method) is adopted to ensure that the settlements are within tolerances. Furthermore, a comprehensive instrumentation program is implemented for monitoring the performance of the temporary works and the ground response to various construction activities.

INTRODUCTION

As depicted in Fig. 1, to ease the traffic congestion in the northern part of the Taipei City, an underpass is currently being constructed to extend Fuhsing N. Road northward to connect to Tachieh Bridge which crosses the Keelung River. A major portion of this extension is underneath the field of the Taipei International Airport which is a busy airport serving both civilian and military air traffic with more than 300 commercial flights per day. Figure 2 is a layout showing the configurations of the underpass, the taxiway and the runway. As shown in Fig. 3, the underpass has to dive to a depth of 21.37m (road level) at its south end because of the provision of a tunnel box for the Taipei Rapid Transit Systems (TRTS) on the top and also because of the presence of a drainage box culvert. At its northern end, it is to meet the existing Bingjiang Street and therefore has to pass underneath the runway with a very thin cover of only 5.6m in thickness above its roof.

As shown in Fig. 4, the underpass has 2 lanes in each direction and the twin-cell box is 22.20m in width and 7.80m in height. The requirement that the air traffic shall be maintained all the time eliminates the possibility of using the cut-and-cover construction method in the sections where the taxiway and the runway are present. In fact, because construction activities above the surface are limited to the period between 11 pm to 5 pm within the entire boundary of the airport, the cut-and-cover method is used only for constructing five working shafts which are absolutely necessary for the work to be done. Elsewhere, an innovative method is required for work to be carried out with minimum interruptions.

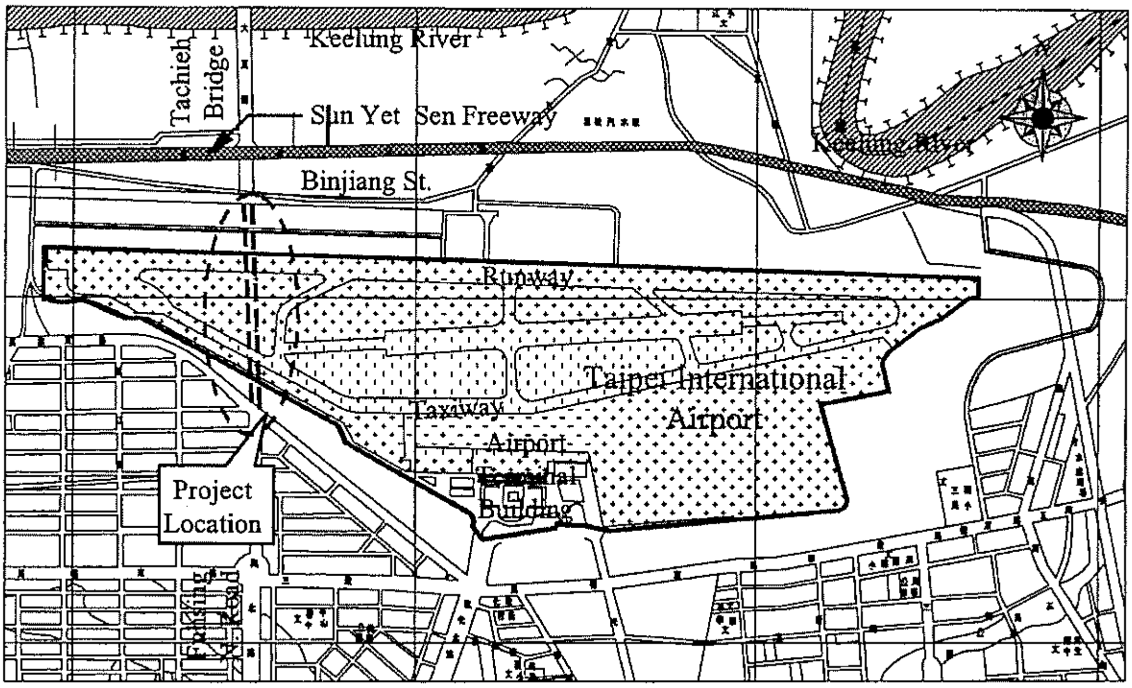


Fig. 1 General Layout

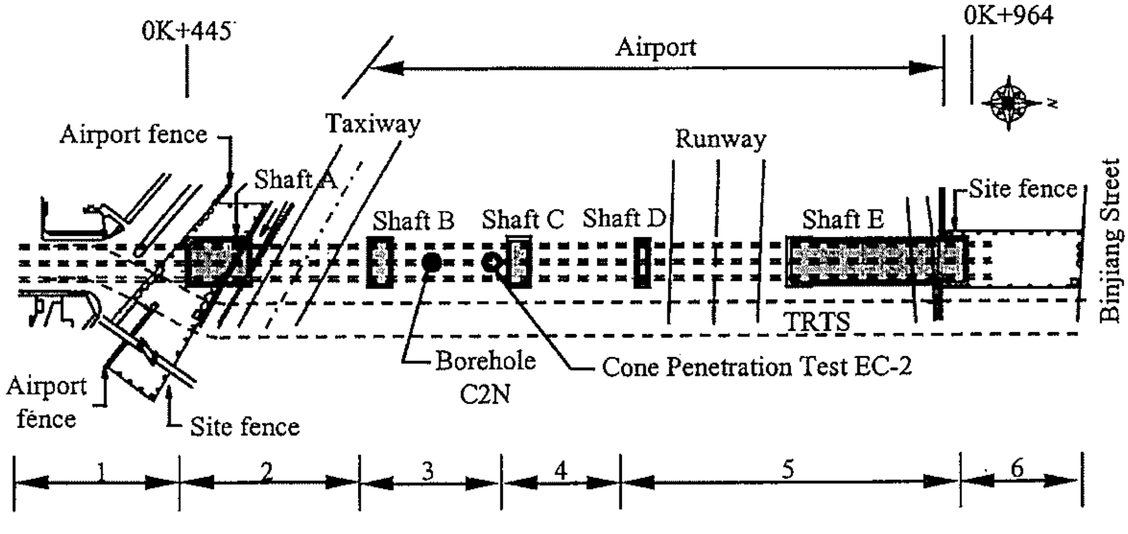


Fig. 2 Site Plan

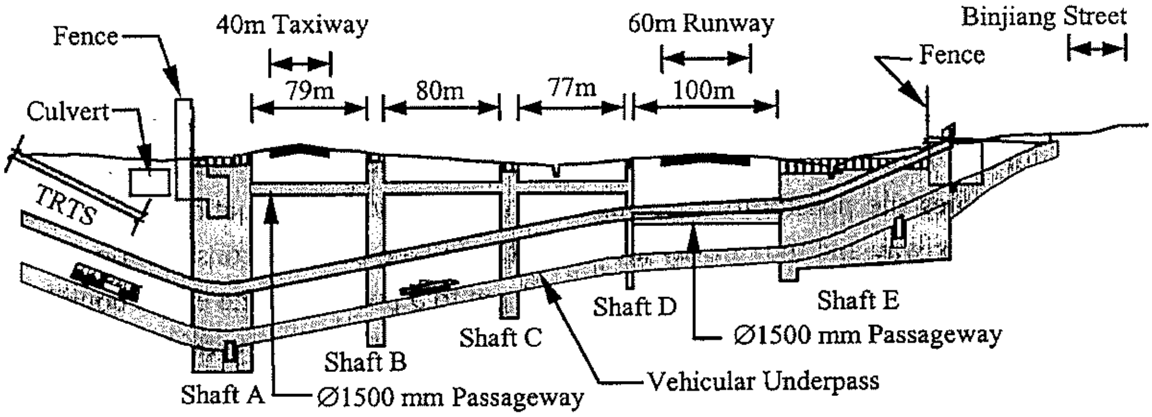


Fig. 3 Longitudinal Section

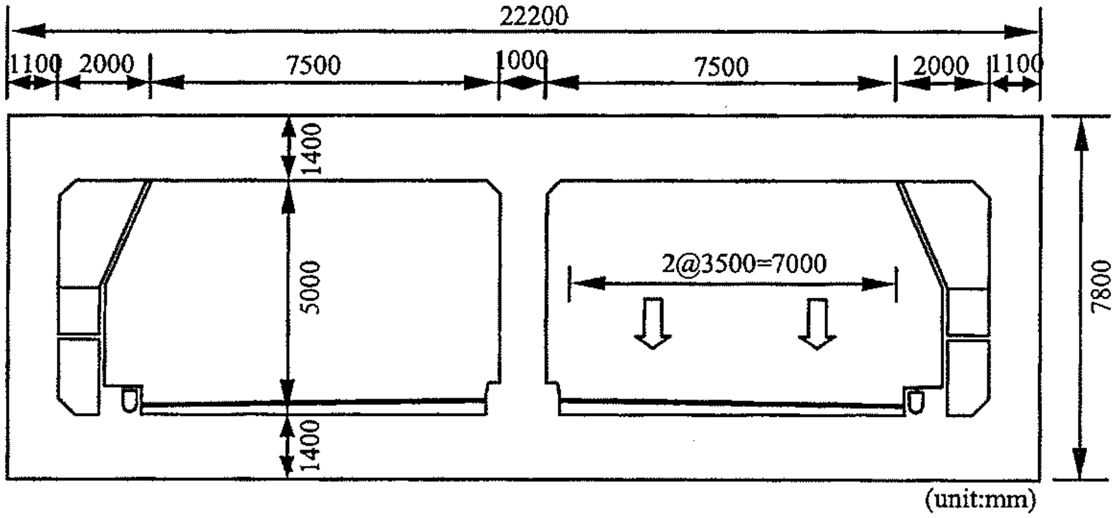


Fig. 4 Typical Tunnel Section

If circular tunnels were to be adopted, they both would have to be 11m in diameter. Tunnels of this size are impossible to be constructed at this site in consideration of the shallow depth of embedment and the poor ground conditions. Even if they were possible, the ground settlement would have been enormous and would endanger the safety of the flights. One of the critical criteria imposed is that settlements of the runway must be maintained within 25mm. If settlements exceed this tolerance, the runway must be resurfaced.

After a thorough study and extensive evaluation of all the options, it was decided that the sections of the underpass within the boundary of the airport be constructed by tunneling in a protective shelter formed by interlocked pipes. Furthermore, for sections directly underneath the taxiway and the runway, the so-called Endless Self Advancing (ESA) method will be adopted (Hsiung, 1997; Continental Engineering/Tekken JV, 1997, 1998) to construct the tunnel box.

The owner of the project is the Division of New Construction Projects, Bureau of Public Works of the Taipei Municipal Government. Moh and Associates, Inc. is the consulting engineer responsible for the design and construction supervision.

GROUND CONDITIONS

A total of 18 boreholes, of which 5 are located within the airport, were sunk to a maximum depth of 75m. As depicted in Figs. 5 and 6, at the surface is a thick layer of silty clay (CL) with a sandy silt (ML) layer present at depths varying from 3 to 10m. The blow counts obtained from standard penetration tests vary from 2 to 5 within a depth of 25m or so. As shown in Table 1, the natural water content of the clay is very close to the liquid limit indicating how soft the clay is. The undrained shearing strengths of this clay layer obtained from triaxial compression tests increase linearly with depth with an average of 100 kPa at a depth of 40m. The sensitivity of the clay varies from 4 to 10. Table 1 also lists the design parameters adopted.

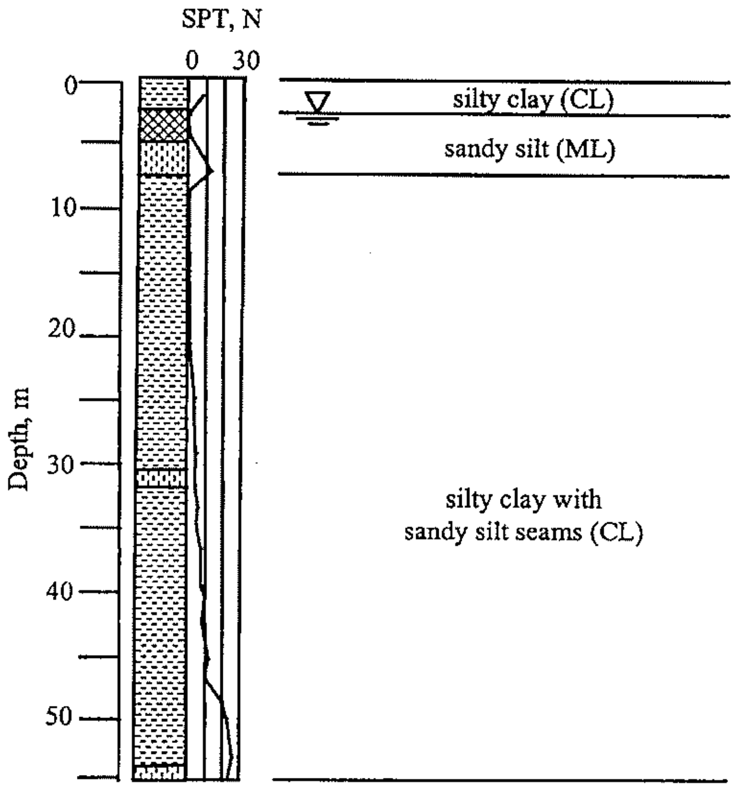


Fig. 5 Log of Borehole C2N and Results of SPT

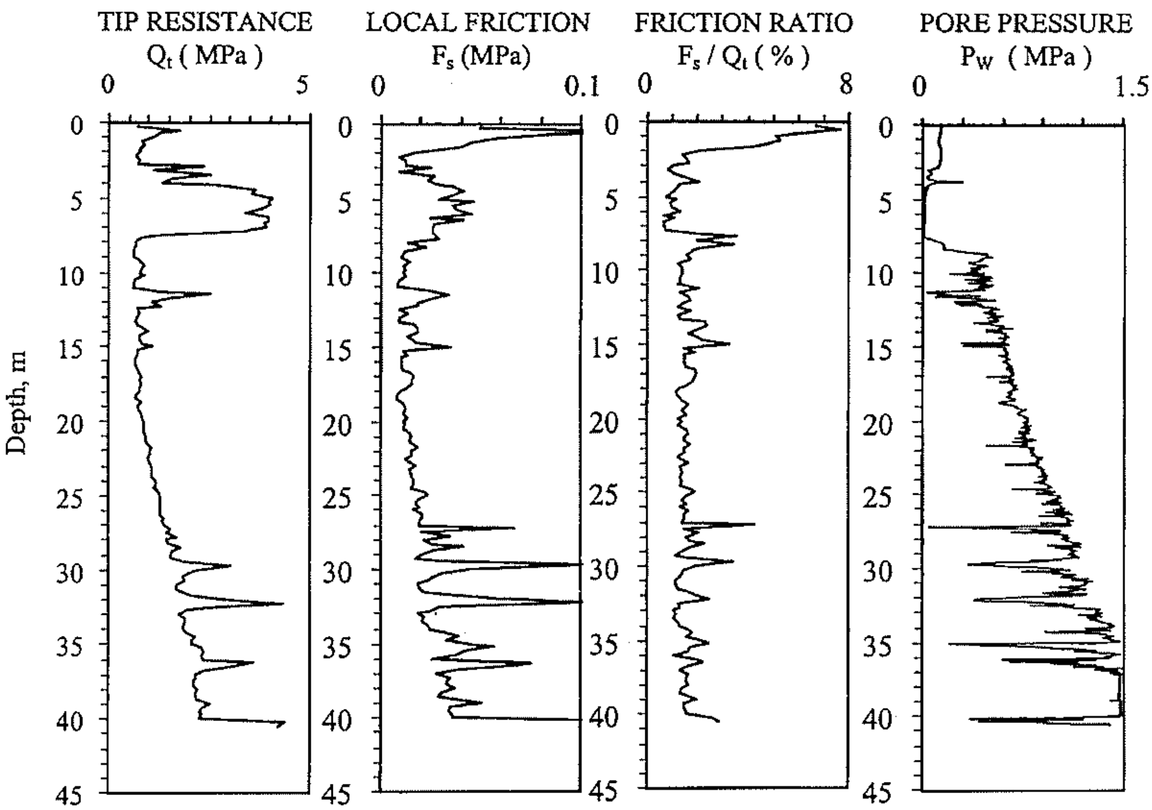


Fig. 6 Results of Piezocone Test EC-2

Table 1 Design Parameters

0k+450 to 0k+740

Depth (m)	Soil Type	N (blows)	γ_t (kN/m^3)	w_n (%)	w_l (%)	I_p (%)	c' (kPa)	ϕ' (deg)	s_u (kPa)	E (MPa)
0~3	CL	4	18.2	28	28	8	0	33.5	26	26
3~8	SM	7	19.2	28			0	32		28
8~24	CL	4	18.6	34	34	10	0	32.5	38	38
24~41	CL	8	19.0	29	29	9	0	33	70	70
41~56	CL	17	20.0	23	30	10	0	34	128	128
56~60	CL	>50	19.0	25			0	34		200

0k+740 to 1k+200

Depth (m)	Soil Type	N (blows)	γ_t (kN/m^3)	w_n (%)	w_l (%)	I_p (%)	c' (kPa)	ϕ' (deg)	s_u (kPa)	E (MPa)
0~9	CL	4	18.5	31	32	12	0	31.5	28	28
9~13	ML	2	18.9	31			0	32		8
13~22	CL	3	18.5	34	35	12	0	33	39	39
22~32	CL	6	19.0	32	33	12	0	33	60	60
32~34	ML	11	19.2	24			0	33		44
34~40	CL	13	19.6	26	26	8	0	33.5	87	87

Piezocone penetration tests were carried out at 5 locations within the airport to a maximum depth of 40m and 9 dissipation tests were performed. The results of one of the piezocone tests are given in Fig. 6 and, as can be noted, thin seams of sandy soil are clearly identifiable. This piezocone test was carried out at a location of 50m north to the borehole of which the log is shown in Fig. 5. The two sets of results obtained are quite compatible.

Because the site is very close to the Keelung River and is protected from flooding by a levee located only a couple hundred meters away, the groundwater table at the site is high and is affected by the water level in the river. Normally, the groundwater table is at depth of a couple meters, or even less, below the ground surface. However, whenever the river was flooded in a typhoon season, the groundwater table rose to a level very near the surface. Therefore, the groundwater table is assumed to be at the surface in the design.

METHODS OF CONSTRUCTION

As depicted in Fig. 2, five working shafts are sunk and the underpass is to be constructed in 6 sections. For Sections 1 and 6, spaces are available for these two sections to be constructed by using the cut-and-cover method. Other sections have to be constructed by tunneling. As discussed above, the ground conditions are very poor and tunneling could not be carried out without auxiliary measures. Various options have been evaluated and the pipe-roof technique was opted to be the most favorable method in consideration of the safety of the tunneling operation. Furthermore, instead of just providing a roof, excavation is to be carried out in a protective shelter enclosed by interlocked steel pipes in Sections 2 to 5.

The five shafts, in the sequence of A to E, are 40m, 12m, 12m, 5m and 116m in length, respectively. Shaft D is used only as an arrival shaft and is therefore shorter than others. Shaft A has the greatest depth of 26.5m. Within the boundary of the airport, work is allowed to be carried out only between 11pm to 5am, diaphragm wall panels cannot be completed in such a short time. Secant piles of 1,500mm in diameter are used as retaining walls. Similarly, Shafts B and C are retained by secant piles. Shafts D and E which are relatively shallow in depth are retained by soil-mixing-wall (SMW) of 600mm in thickness. Because each pile cannot be completed in one go, these SMW piles were fully cased to prevent the bored holes from collapsing during the intermission. Casings were then withdrawn during concreting.

Excavation is first carried out inside each shaft to the bottom level of the roof pipes and the roof pipes are installed by using the pipe jacking technique. Subsequently, excavation is carried out in three stages and in each stage, three pipes on each side are installed. Finally, excavation is completed and the bottom pipes are installed. In addition to the pipes which form the shelter, there are 11 pipes to be installed in Sections 3 and 4 to serve as intermediate beams during tunneling as shown in Fig. 7, and 12 pipes to be installed in Sections 2 and 5 to serve as cable ducts, of which the locations are indicated in Fig. 8. The latter are to be used to house cables which are to be used to jack segments to their positions as shown in Fig. 9. All these additional pipes are of the same size as those forming the shelter and they are installed as excavation reaches appropriate depths.

Once all the shafts are completed, they will be covered by steel decks so works can proceed underground during the day time. The shafts at the two ends will be the only accesses for workers and materials. It will then no longer be necessary for workers and machines to enter the airport except for taking instrument readings.

Sections 3 and 4 are in an open field and settlement is of less concern. Excavation is therefore carried out in a conventional way in which two headings are made and the pipe-shelter is propped by steel members as depicted in Fig. 7. The loads on the roof pipes are temporarily transferred to pre-installed intermediate beams resting on the soil bench. These intermediate beams are to be removed as the bench is excavated and the loads are transferred to the bottom pipes. The tunnel box is to be cast-in-situ after the completion of mining.

For Sections 2 and 5, settlement of the taxiway and the runway is of primary concern. It is envisaged that settlements would exceed the tolerance if the above-mentioned approach is adopted without additional measures. Therefore, these two sections are to be completed by using the so-called endless-self-advancing method (ESA method) in which, as illustrated in Fig. 9, the tunnel boxes are advanced segment by segment in a manner resembling the movement of centipedes. This method is patterned by Uemura Engineering Co., Ltd of Japan.

For these tunnel boxes to be properly positioned, refer to Fig. 8, five guide tunnels are to be manually dug and guide rails are to be installed. These guide tunnels are in two types: Type A at the two lower corners and Type B at the intermediate spans. They are in a shape of shield and are supported on ribs formed by steel tubes. Wood planks are used as laggings behind these ribs. Type A tunnels are roughly 2,360mm in height and Type B tunnels are roughly 2,100mm in height. Three guide rails are provided in each tunnel. These guide tunnels serve as pilot tunnels for the purpose of unveiling ground conditions along the path and also serve as drains.

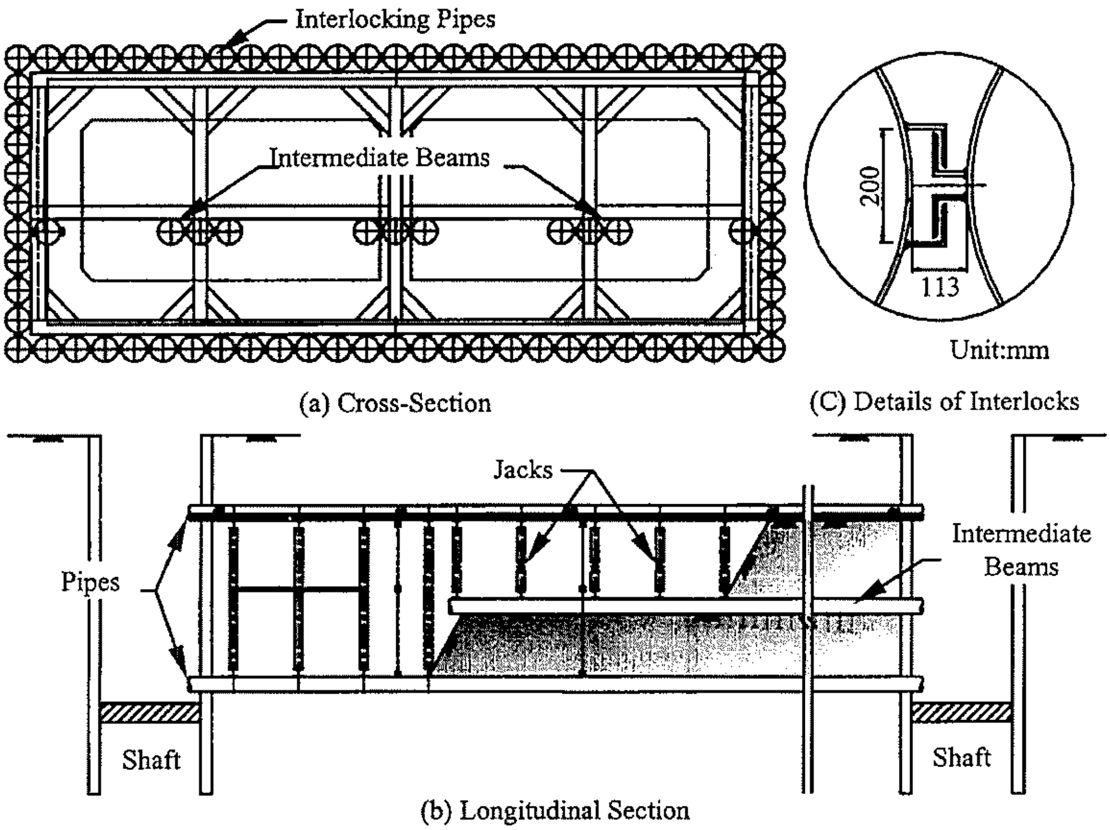


Fig. 7 Tunneling in Sections 3 and 4

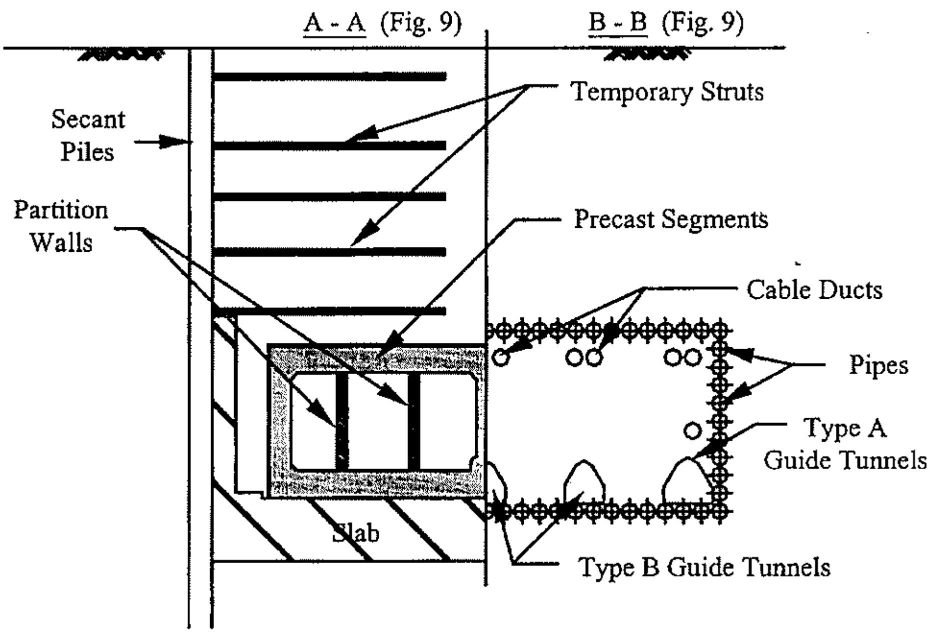
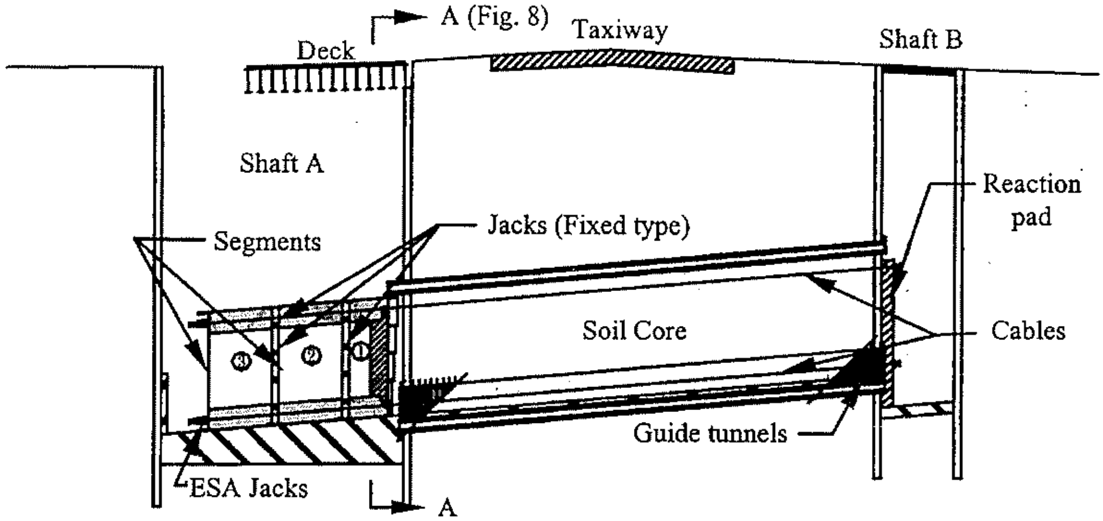
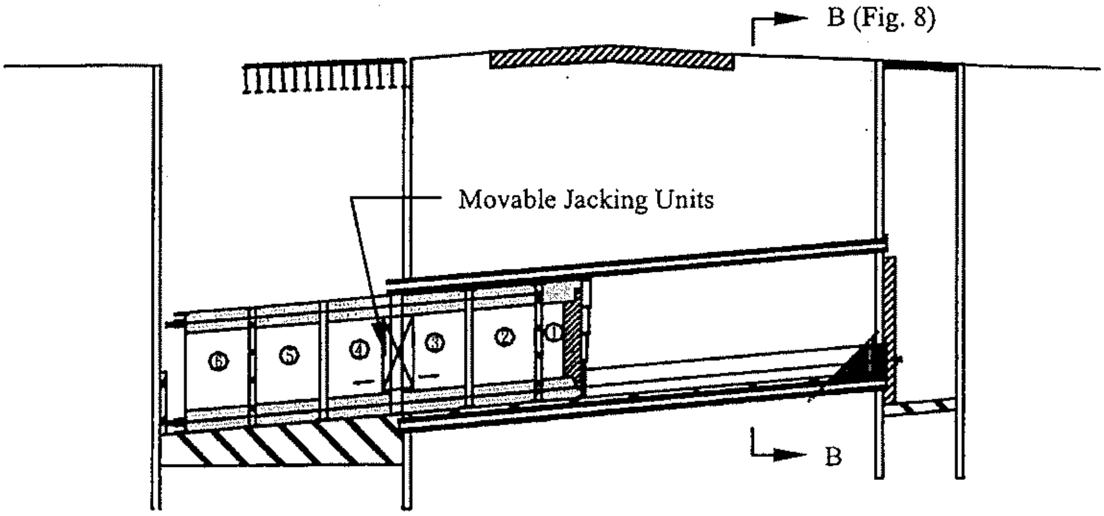


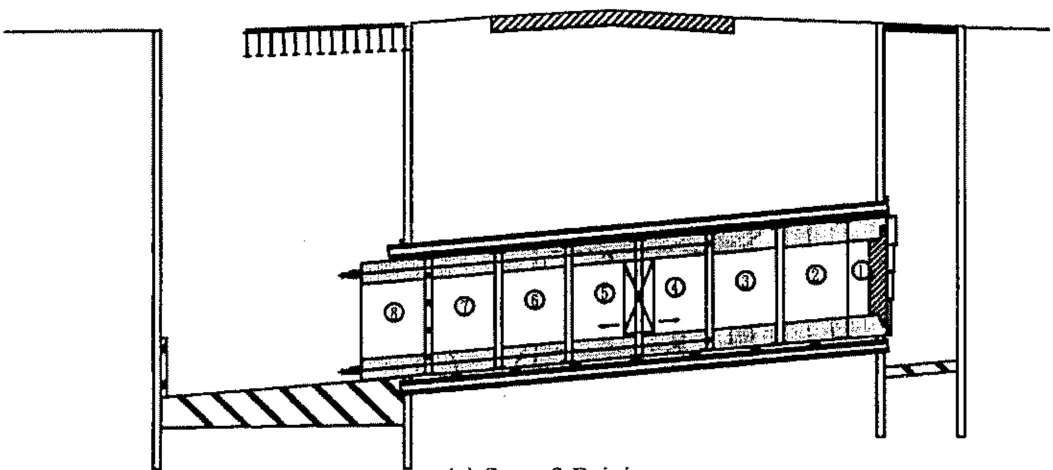
Fig. 8 Cross-section of ESA Layout



(a) Stage 1 Driving



(b) Stage 2 Driving



(c) Stage 3 Driving

Fig. 9 Tunneling in Section 2

Once the guide tunnels are completed, the soil core inside the pipe-shelter is excavated and the tunnel box is jacked into the shelter in full-section segments of 10.5m each, except the first segments which are 6.5m in length. At each time, only one segment is moving and the remaining segments provide reactions required. The tunnel box under the taxiway, i.e., Section 2, is divided into 8 segments and is driven in 3 stages as illustrated in Fig. 9. In front of this segment is a cutting edge made of steel plates. This cutting edge is partitioned into 24 chambers arranged in 4 rows. Excavation is carried out chamber by chamber and the face is braced by breast jacks, 4 in each chamber.

The basic idea of the ESA method is that each time a segment is being driven, the majority of the reaction comes from the frictional resistance acting on the rest of segments. For example, when Segment 3 is being driven by jacking against Segment 4, refer to Fig. 9(b), part of the jacking force is taken by the frictional resistance acting on Segments 4, 5 and 6, and the remaining force goes to the end of Segment 6 and is taken by cables. The force taken by the cables which run through the guide tunnels is transmitted to the reaction pad in Shaft B. The force taken by the cables which run through cable ducts is transmitted to Segment 2 and is resisted by the frictional resistance acting on Segment 2. This way, the size and the rigidity of the reaction pad required are much reduced. The tunnel boxes are surrounded by soil on their sides and on their roofs all the time. During jacking, lubricant will be injected from grouting holes on the segments to reduce frictional resistance. In theory, the forces taken by the cables are not affected by the length of the tunnel box and the tunnel box can be as long as desired. In addition to the jacks which are fixed to segments at the joints, four movable jacking units are used to provide extra driving forces and each unit is equipped with 15 jacks with a capacity of 150 tons per jack. Shaft E is long enough for all the 10 segments to be cast at one time and jacked consecutively toward Shaft D.

DESIGN CONSIDERATIONS

Ground settlement is the primary concern in the design. Settlements may occur as a result of excavation or as a result of lowering of groundwater table. It is therefore important for the retaining systems for shafts to have sufficient rigidity and watertightness. It is even more important for the pipe-shelter to satisfy these requirements. As an auxiliary measure, extensive ground improvement work was carried out to strengthen subsoils and to cut off seepage paths.

The shafts are rectangular in shape and the design of shafts is conventional, except that it is necessary to include the loading from airplanes into consideration. It has been found that lateral pressure on walls from airplanes is rather small in comparison with that from the surcharge due to construction activities. The surcharge was increased by 5 kPa to simulate the loading from airplanes wherever necessary.

Since this is the first time for the ESA method to be used in Taiwan, there are a lot to be learned. Furthermore, the ground conditions encountered at this site and the fact that the tunnel is to be driven underneath a busy runway are unprecedented. Therefore, a comprehensive instrumentation program is desirable for maintaining the safety of construction, enhancing construction procedures and documenting experience.

PIPE JACKING

As of August 1999, roof pipes in all the sections have been completed and jacking is continued for installing the rest of pipes. The pipe-shelter running from Shaft E to Shaft D will be completed in October and advancement of the tunnel box is to be commenced in August, 2000. Jacking is being carried out from Shafts A to B, B to C, C to D and E to D. These pipes are 812.8mm in their outer diameter and are interlocked, as illustrated in Fig. 7, for better watertightness and better alignment of pipes. At the location of the runway, the top of the pipes is only about 4.5m below the runway. As depicted in Figs. 5 and 6, there exists a sandy silt layer at depths varying from 3m to 10m, seepage could become a problem if pipes were not interlocked. These interlocks are filled with styrofoam so they will not become water paths. The openings made on the retaining walls for launching pipes and for receiving pipes are sealed by sealant and rubber gaskets.

In Sections 2 and 3, two units of earthpressure balancing type Iron-Mole manufactured by Komatsu Ltd. are used. The bearing of the cutter head can be adjusted by 3 degrees in any direction for correcting the alignment of pipes. In Sections 4 and 5, four units of slurry type Uncle-Moles manufactured by Iseki Poly-Tech, Inc. are used. The bearing of the cutter head can be adjusted by 1.7 degree in the vertical direction and 1.2 degree in the horizontal direction. For both types of machines, the cutter heads are 830mm in diameter giving a theoretical clearance of 9mm between the ground and the pipes. During jacking, lubricating fluid is injected to fill up this void for reducing frictional resistance acting on pipes. Four jacks, each with a capacity of 150 tons, are employed and for most of the pipes which have already been completed the jacking forces were generally below 200 tons. However, a peak force of 280 tons was occasionally required.

Pipes are kept open so compensation grouting can be carried out from the grouting holes in the pipes to rectify ground settlement should it exceed tolerance and will be filled with concrete upon the completion of tunneling. In some of the pipes, instruments are installed for monitoring the deformations of these pipes and the stresses induced.

Large pieces of timber were encountered at depth of roughly 6m during jacking of roof pipe No. CU17 between Shafts C and D and jacking force exceeded 800 tons. A temporary shaft was sunk for the purpose of removing these obstacles. It was retained by sheet piles as illustrated in Fig. 10. The sheet piles on the western side of the shaft were obstructed by roof pipes which had already been installed and ground improvement was carried out at their toes to prevent water from seeping into the shaft. Several pieces of timber were discovered and removed. Two of them were about a half meter in length and 200mm in diameter. Pipe No. CU17 was extended all the way to Shaft D and the shaft was backfilled after the completion of the remedial work.

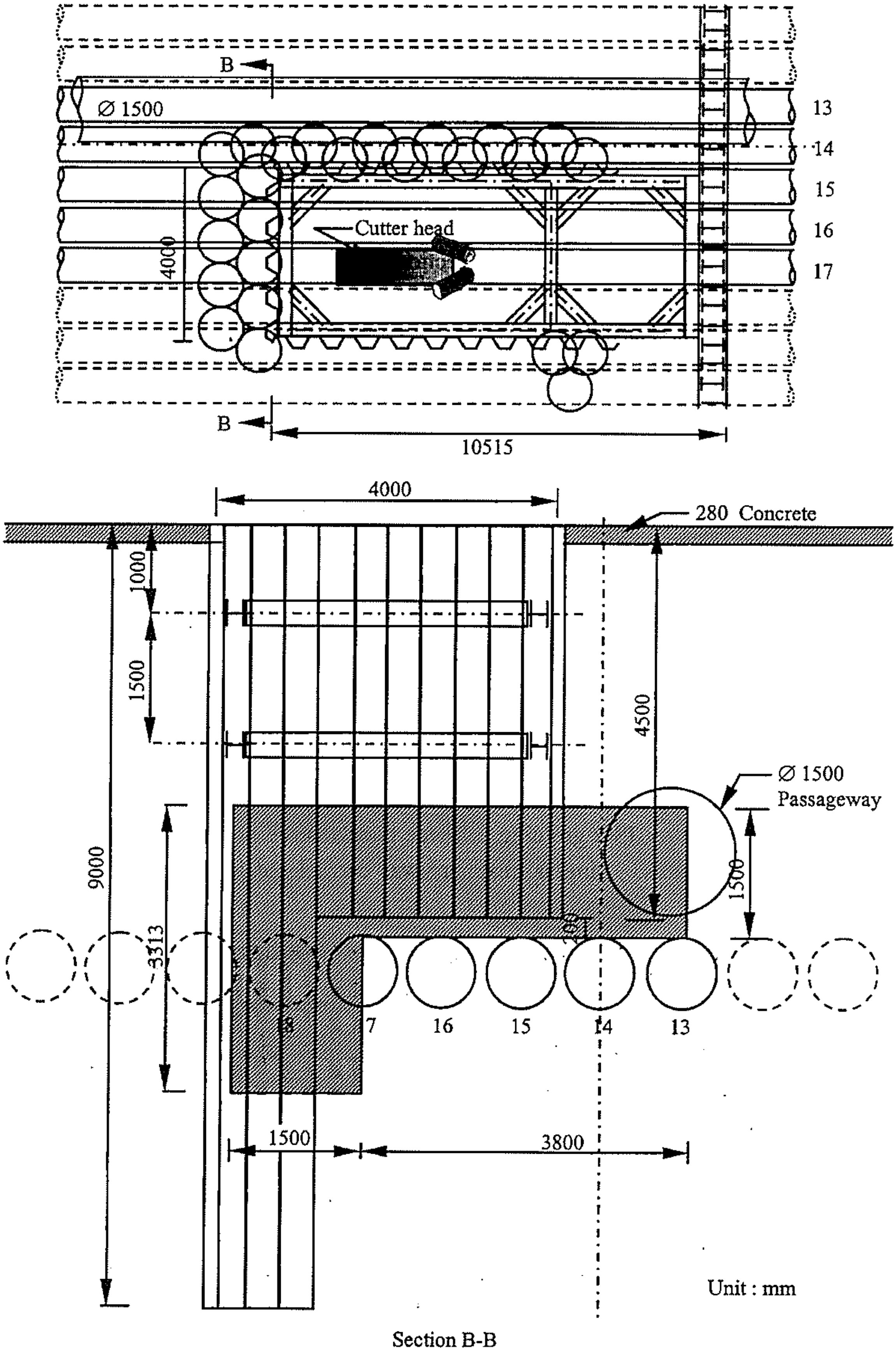


Fig. 10 Temporary Shaft for Removing Timber

MATERIAL PASSAGEWAY

As depicted in Fig. 3, a passageway of 1,500mm in diameter is installed for transporting material between shafts. This passageway allows works to be carried out underground in the day time and is a key element in speeding up the work. For more than two-third of the construction period, it serves as a walkway for workers and is the only access/exit for materials and light equipment/machines required for works to be carried out in Shafts B, C and D.

This passageway is installed between shafts by using the pipe jacking technique in a similar manner as described above. The section of the passageway between Shafts D and E has to be lowered by 5m or so to stay away from the pipe-roof. Even so, it still conflicts with the tunnel box and has to be removed during tunneling.

GROUND IMPROVEMENT

As mentioned above, the ground is very soft and it is therefore necessary to perform ground improvement to ensure the safety of construction and to limit ground movements. Figure 11 shows, for example, the improvement works carried out at Shaft E. Grouting was carried out by using the JMM (Jumbo Mini Max) method at the south end to solidify the subsoil all the way from surface to a depth of 6.7m below the final depth of excavation. This will help reducing ground movements, and hence, settlement of the runway. Outside the south end wall, treatment was carried out by using the Jumbo Special Grout (JSG) method to prepare for launching of pipes and tunnel segments. In the middle section of the shaft, JSG grouting was carried out to form 4 panels, running in the longitudinal direction. Across the shaft, there are 21 panels formed by using the JMM method and one panel, i.e., the very southern one, formed by JSG grouting. These panels serve as bracings for the purpose of limiting wall movements. Similar treatment was carried out in the northern end with one

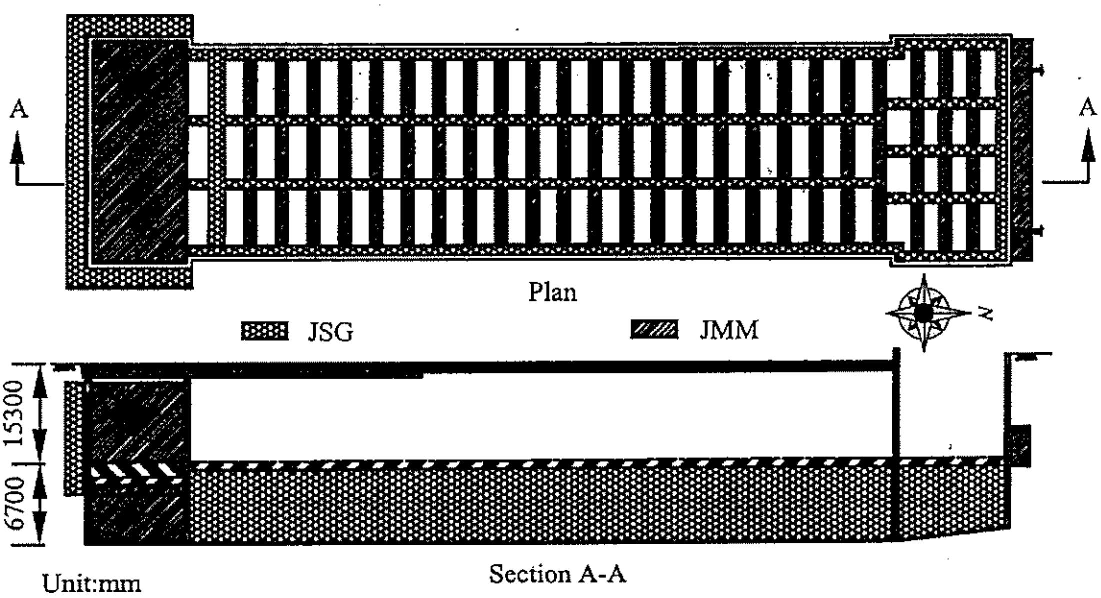


Fig. 11 Ground Improvement at Shaft E

row of JSG grouting along the perimeter, 3 panels in the longitudinal direction by JSG grouting and 3 panels across the shaft using the JMM method. Outside the northern wall, a block of subsoil was treated by using the JMM method to back the reaction pad which is to be used to drive tunnel segments.

In the JMM method, a blade of 800mm in width is rotated to disturb subsoil and grout is injected from nozzles on the edges of the blade at pressures of 180 bars to 300 bars to form columns of 2m in diameter. Columns have an overlap of 300mm to ensure continuity. In comparison with high pressure jet grouting, the JMM method offers better quality of the products. However, alongside the perimeter and at places where the operation is obstructed by struts and pin posts, high pressure jet grouting, such as JSG method, will be easier to carry out. This is the reason that both types of grouting are adopted herein.

Similarly, ground treatment was also carried out at other shafts but to a much less extent. Considerable treatment is to be carried out inside the pipe-shelter, but the program is yet to be finalized after the ground conditions are better confirmed during the installation of pipes. In principle, the soil core is to be solidified by cement-bentonite-water mix to improve the stability of the face and to reduce water flow, if any. The soil surrounding the guide tunnels, refer to Fig. 12, will be treated by using the double-packer grouting method to yield a minimum unconfined compressive strength of treated ground of 160 kPa while the unipack method will be used elsewhere to yield a minimum unconfined compressive strength of 80 kPa.

INSTRUMENTATION

Because of the utmost importance of the safety of air traffic, a comprehensive instrumentation program has been implemented at the site. In addition to temporary works, some navigation facilities, such as radar towers, are also being monitored to ensure that their movements will not affect their performance. The most important item to be monitored

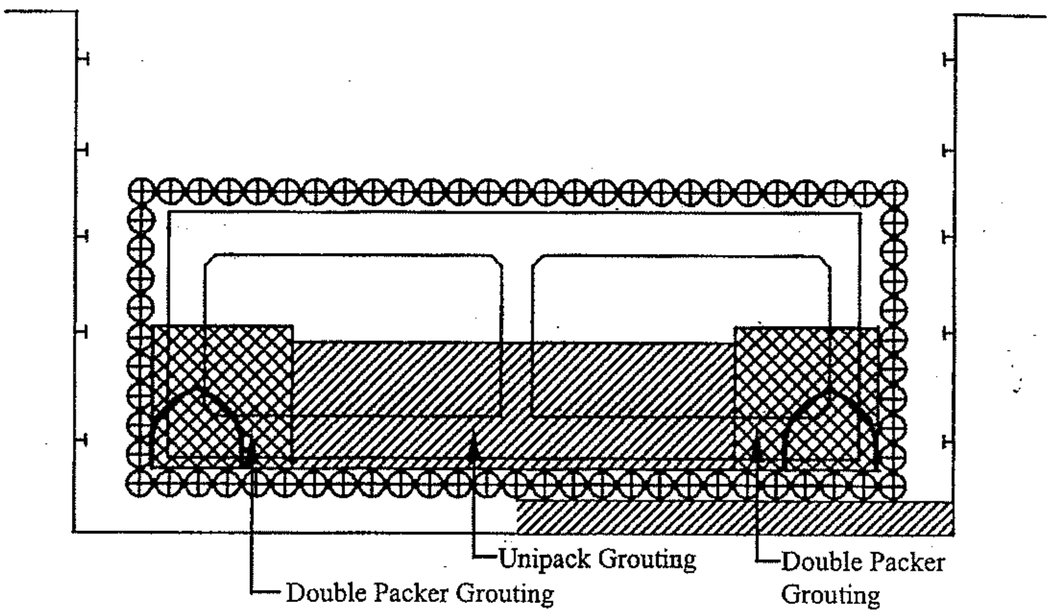


Fig. 12 Ground Improvement for ESA Work

among all is certainly the settlements of the runway and the taxiway. Settlement points are arranged in a 7 by 23 grid in the primary area above the tunnel on the pavement of the taxiway and in a 13 by 15 grid above the runway. They are spaced at 5m intervals. Intermediate points will be added as needs arise. Since survey is not allowed to be carried out on either the taxiway or the runway during the period in which the airport is open to services, inclinometer casings are installed in two of the roof-pipes under the taxiway and two of the roof-pipes under the runway and deflections of these pipes are monitored by using horizontal inclinometer probes. Deflections of pipes are measured at 1m intervals every morning and every night during ESA works.

What is worthy of mentioning is the automatic alert system adopted at the site. There are more than a thousand pieces of instruments which can be read automatically at any desirable frequencies. As illustrated in Fig. 13, a total of 14 data loggers are located in shafts and data are transmitted to a control room located at the site office where data are processed and analyzed. In addition, there are more than 500 settlement points and many other instruments of which readings must be taken manually and keyed in to the databases. For each instrument, two trigger values are defined and the current reading is compared with these trigger values. Once the first-level trigger value is exceeded, the contractor is alerted of potential risk and is urged to prepare contingency measures. Once the second-level trigger value is exceeded, contingency measures shall be taken unless it is proved that the situation posts no immediate danger.

The status of instruments is displayed on three panels, one at the site office, one at the office of the project owner, i.e., the Division of New Construction Projects of Bureau of Public Works of the Taipei Municipal Government and the third at the office of the Airport Authority. A green light will indicate that the current reading of the corresponding instrument is within the first-level trigger value and a yellow light will indicate that the reading has exceeded the first-level trigger value but is still within the second-level trigger value. A red light will indicate that the second-level trigger value has been exceeded.

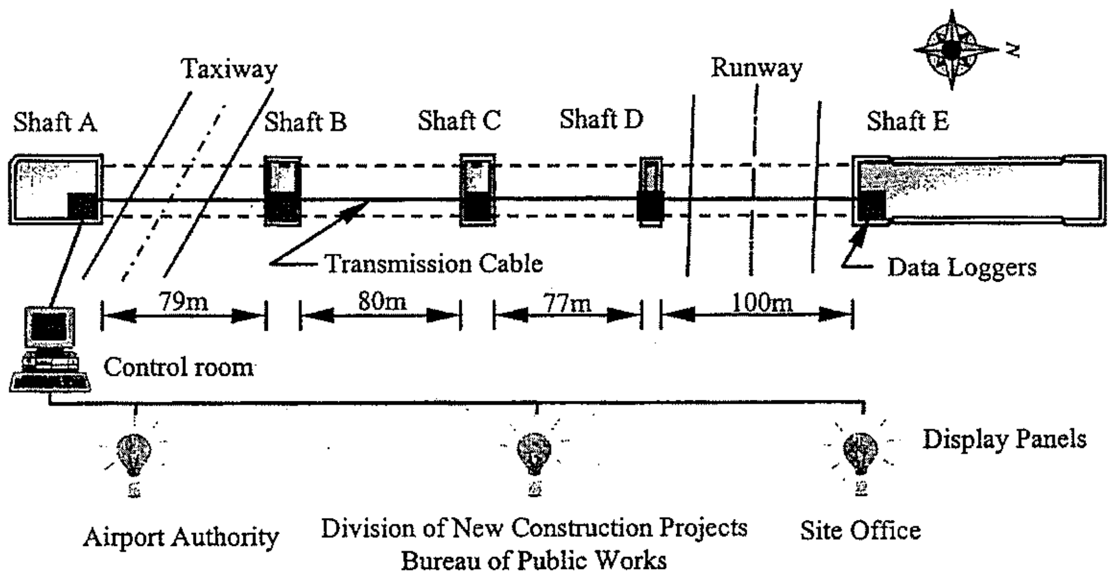


Fig. 13 Transmission of Instrument Readings

Upon the completion of all the pipes running from Shaft E to Shaft D, including the passageway, the settlement of the runway was within 10mm as illustrated in Fig. 14. This magnitude of settlement is remarkably small in consideration of the poor ground conditions encountered and was achieved by back grouting and compensation grouting carried out from the passageway and the roof pipes.

SUMMARY

Although the ESA method has been adopted in quite many occasions in Japan and, presumably, elsewhere, the situation faced at this particular site is believed to be unprecedented in consideration of the extremely poor ground conditions and the risk involved in underpassing the runway of a very busy airport with a thin cover. Considerable thoughts have been given on how constructions can be carried out safely without affecting air traffic and how ground settlements can be minimized. It is believed that the methods of construction chosen are among the best for the purposes.

ACKNOWLEDGEMENTS

The authors are grateful to the Department of New Constructions of Taipei Municipal Government for the permission of publishing the information contained herein. They are indebted to the main contractor, Continental Engineering and Tekken Joint Venture, and its

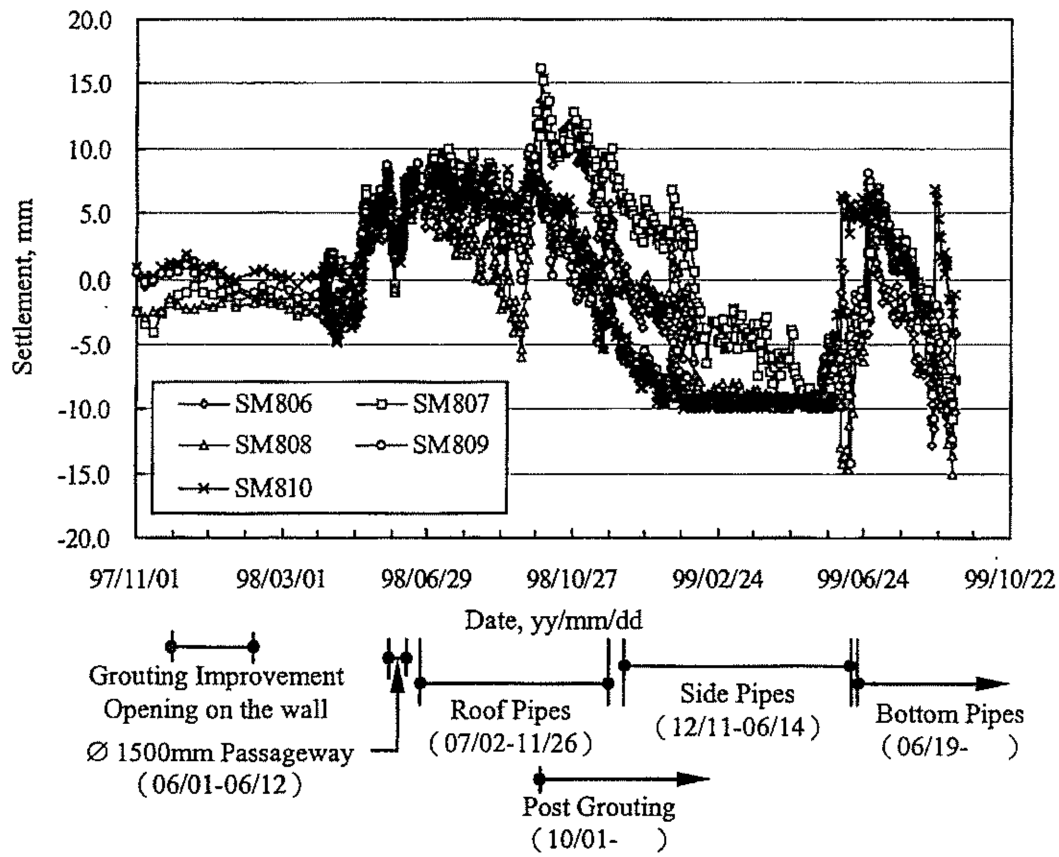


Fig. 14 Settlements of the Runway during Pipe Jacking

specialist subcontractors Komatsu Ltd and Iseke Poly-Tech, Inc of the project for providing the detailed working procedures. Many of the figures were either reproduced or modified from the figures presented in their method statements. Appreciation is due to Uemura Engineering Co., Ltd for the many motivative suggestions made in the design.

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2001 MILTON E. BENDER LECTURE

GEOTECHNICAL ENGINEERING IN INFRASTRUCTURE DEVELOPMENT

Z.C. Moh[1]

ABSTRACT: Four major infrastructure projects in Taipei and Bangkok were discussed to illustrate the importance of geotechnical engineering in any infrastructure development in terms of safety and economy. The four projects are the Taipei Rapid Transit Systems (TRTS), the Taipei Airport Underpass, the Bangkok-Chonburi New Highway (BCNH) and Ground improvement of the Second Bangkok International Airport (SBIA). They were selected not only because of the mega size in terms of construction cost but also because of the complexity or uniqueness of the project. Emphasis has been placed on the importance of adequate and reliable subsurface information, appropriate selection of analysis principles and construction methodology/details, ability to copy with variation in ground conditions and timely interpretation of field performance data.

INTRODUCTION

This lecture is the eighth in the series of lectures organized by the Asian Institute of Technology started in 1993 to honor the Founding President of the Institute—Professor Dr. Milton E. Bender. This is also the first memory lecture since the demise of Dr. Bender last year. I felt greatly honored to be invited today to deliver this lecture, not only this is the first memory lecture but also being the first former faculty member of AIT who has worked under Dr. Bender to be the Bender Lecturer.

I had a profound memory of Dr. Bender as a boss, a colleague and a friend. It was Dean Bender who appointed me to the faculty of the SEATO Graduate School of Engineering in 1965 and pursued me to remain at AIT for 11 years. In the following years, I had the opportunity to work closely with him in developing a new field of study—soil engineering. I also had many opportunities to listen to his many up and downs relating to his historical accomplishment in transforming the SEATO Graduate School to an independent and truly international Asian Institute of Technology, and later worked with him on the development of the new campus at Pathumthani.

At AIT, I started the first graduate program in Geotechnical Engineering, probably one of the first in the region, the first regional information center, organized the first regional conference and then founded the first regional professional society, (i.e. the Southeast Asian Society of Soil Engineering). Without the vision and unfailing support of Dr. Bender, these activities were not possible. Dr. Bender was an engineer, academic, diplomat and promoter. He was a man with principle and determination. He had no hesitation to make decisions but he always was willing to listen and to adopt good ideas.

Recognizing infrastructure development is the key to economic development of developing countries, D. Bender has placed major emphasis on the development of academic programs that uniquely fit the needs of the region. In the early days, most of the major fields of study are related to civil engineering, i.e. the base of physical infrastructure development. Even today, those basic fields of study are still playing major roles in AIT's academic program. For this reason, I have chosen the topic "Geotechnical Engineering in Infrastructure Development" for today's lecture.

For any physical infrastructure development, whether they are founded on ground, built in the ground or utilize earth material as construction material, the role of geotechnical engineering in design and construction cannot be over-emphasized. However, geotechnical engineering, not like other branches of civil engineering, up to the present, still has not been developed into an exact science due to the complex nature of the earth. Sound geotechnical practice depends on integration of theory, reliable data, and experience. Observational method by utilizing field monitoring data during construction for design revisions is the best approach to achieve economy and safety in major infrastructure development. This lecture presents case studies of four major infrastructure projects to illustrate the complexity of geotechnical engineering.

SOFT GROUND CONSTRUCTION IN TAIPEI BASIN

Projects

(A) The Initial Network of the Taipei Rapid Transit Systems

The Initial Network of the Taipei Rapid Transit Systems (TRTS) comprises of six lines with a total of 88km of track and 77 stations. About half of the stations and tracks are underground. Except a short section of one of the lines, the majority of the Initial Network is located in soft ground. A system map is depicted in Fig. 1. Of the six lines only the Mucha Line is of medium capacity. The other five lines are all of heavy capacity. Planning and design of the Initial Network started in 1987. The first line, i.e., the Mucha Line was completed and open to operation in March, 1996. At present, except for a short section of the Panchiao Line, all six lines of the Initial Network are open to revenue services with average daily traffic of 900,000 passenger-trips per day and holiday traffic exceeding 1.2 million per day. The total construction cost of the six lines, including the extensions to Neihu and Tucheng, is about NT$440 billion (about 13 billion US dollars).

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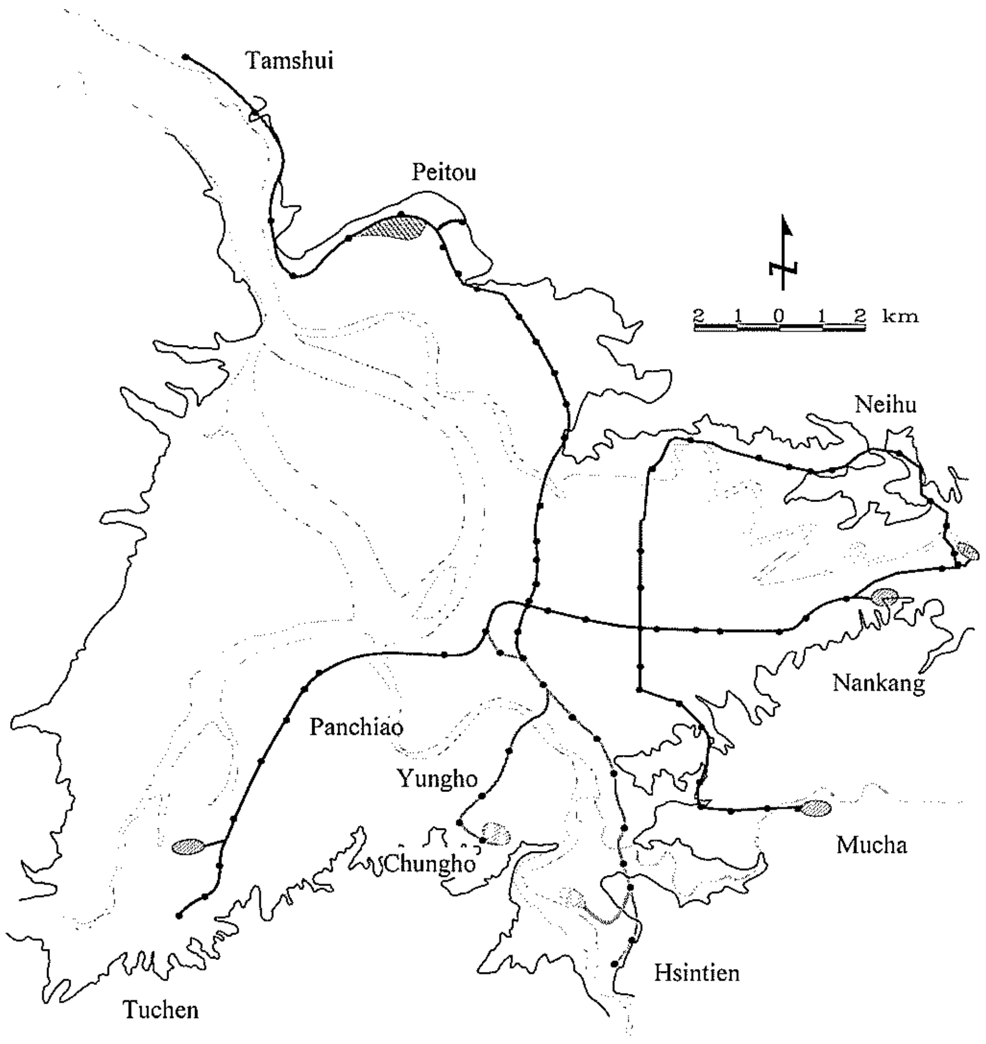


Fig. 1 Initial Network of Taipei Rapid Transit Systems

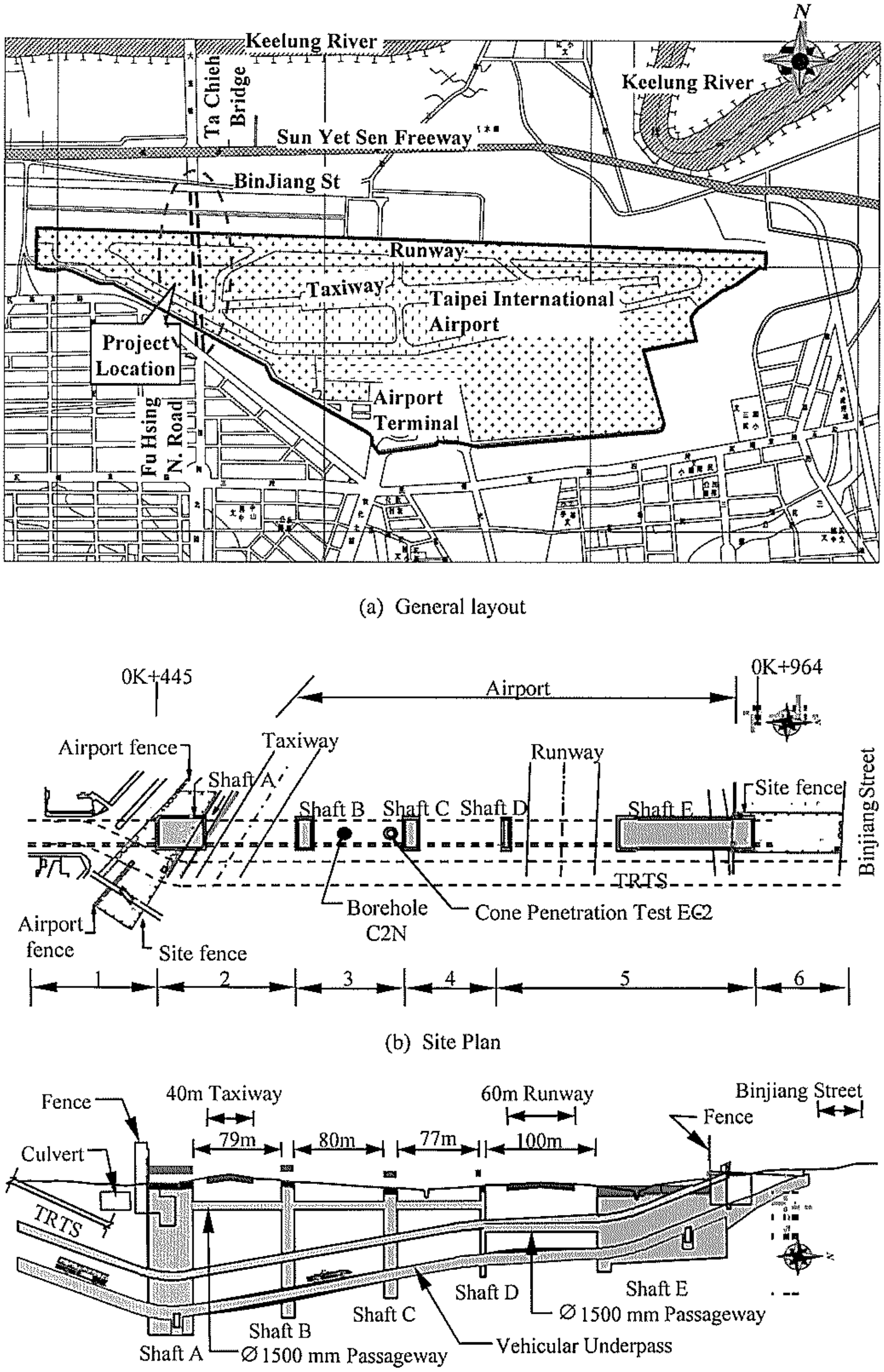
For the underground construction, about 45 km of diaphragm walls, with thickness varying from 800 mm to 120 cm, were constructed in cut-and-cover excavations whilst bored tunnels have a total route length of 22km.

For the initial network of the TRTS, besides being detailed design consultant for one of the design package, Moh and Associates, Inc. was the overall Geotechnical Engineering Specialty Consultant for the entire project from planning to construction.

(B) Taipei Airport Underpass

To ease the local traffic congestion in the northern part of Taipei City, an underpass is currently being constructed to extend Fuhsing North Road northward to connect to Tachieh Bridge which crosses the Keelung River. A major portion of this extension is underneath the field of the Taipei International Airport which is a busy airport serving both civilian and military air traffic with more than 300 commercial flights per day. As shown in Fig. 2(c), the underpass has to dive to a depth of 21.37m (road level) at its south end because of the provision of a tunnel box for the TRTS on the top and also because of the presence of a drainage box culvert. At its northern end the underpass is to meet the existing Bingjiang Street and therefore has to pass underneath the runway with a very thin cover of less than 5m in thickness above its roof. As depicted in Fig. 3, the underpass has 2 lanes in each direction and the twin-cell box is 22.20m in width and 7.80m in height.

Since the Taipei International Airport is the only airport in the vicinity of Taipei metropolis, the air traffic must be maintained all the time and there are several constrains which must be considered in the design and construction of the underpass. Major constrains include: (1) construction activities above the ground surface are limited to the period between 11pm and 5am within the entire boundary of the airport; and (2) settlement of the runway during construction must be maintained within 25mm. To assure safety of the airport operation, a comprehensive instrumentation program has been implemented. There are more than one thousand pieces of instruments, including settlement points, horizontal inclinometers, installed at the site. Majority of these instruments can be read automatically at any desirable frequencies. Data are transmitted from data loggers to the central control room at the site office through cables for immediate processing and



(a) General layout

(b) Site Plan

(c) Longitudinal section

Fig. 2 Underpass beneath Taipei International Airport

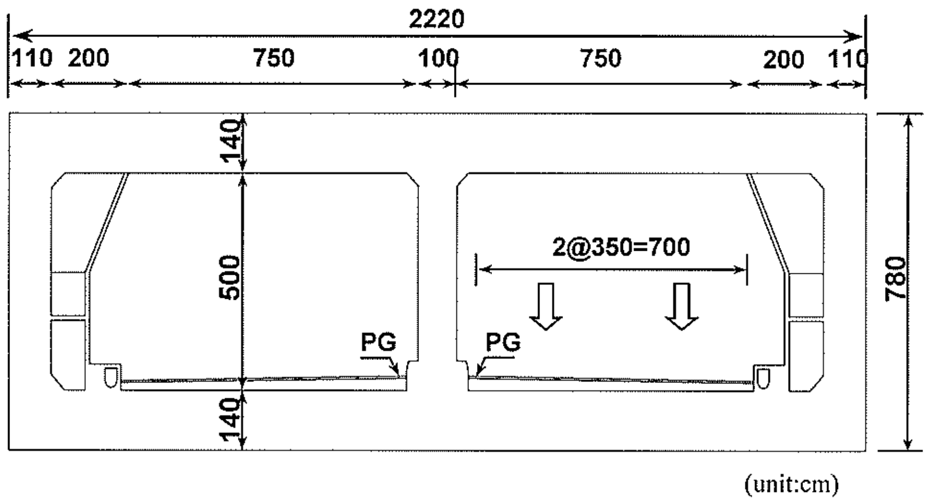


Fig. 3 Typical tunnel section of the airport underpass

analyses. An automatic alert system is adopted for prompt actions to ensure operational safety of the airport.

For this project, Moh and Associates, Inc. is the consultant responsible for the design and construction supervision.

Difficult Geological Features

Taipei Basin was formed about 10,000 years ago as a result of tectonic movement. It was a lake until a few thousands of years ago and particles brought from surrounding hills by rivers accumulated at the bottom to form thick alluvial strata. As the sea retreated, the lake was drained and the bottom of the lake was exposed. Figure 4 shows typical profiles of subsoils in the central city area of Taipei. As can be noted that there exists a thick layer of young sediments, i.e., the so-called Sungshan Formation, underlain by the Chingmei Gravels. The Sungshan Formation contains interbedded soft silty clay and loose silty sand layers, both of which are fairly compressible. Therefore, the city has experienced large ground subsidence as the basin was inhabited and excessive withdrawal of groundwater for domestic and industrial uses led to considerable drop in piezometric levels in the Chingmei Gravels and the subsoils in the Sungshan Formation.

The results of a cone penetration test carried out in the central city area are shown in Fig. 5. As can be noted that the six sublayers in the Sungshan Formation are clearly identifiable. The weak strengths together with the high compressibility of the surface layers necessitate thick retaining walls and deep penetrations of walls in deep excavations to limit wall deflections and reduce seepage flows. In addition, there are numerous drift woods buried in the Sungshan Formation and obstacles were frequently encountered during tunneling and/or pipe jacking. Accidents indeed occurred during the construction of the rapid transit systems and some of these accidents caused considerable financial loss and delays of the project.

The presence of methane in the Sungshan Formation is another unique feature which calls for special attention in underground works. Methane is potentially dangerous as quite many fatal accidents have occurred in the past as workers entered manholes, storage tanks, etc for maintenance. These accidents are not what the author intends to discuss herein because the methane encountered in these cases was not released from soils but from organic wastes. What of interest is the methane entrapped in soils because it might seep into confined underground spaces and accumulate to a concentration high enough to become harmful to people. It might even explode if its concentration reaches the critical concentration for ignition.

The Chingmei Gravels underlying the Sungshan Formation, refer to Fig. 4, is equally problematic. It is extremely permeable and is very rich in water reserve. Although its presence has been carefully considered in the design, accidents still occurred during the construction of the rapid transit system. In fact, the Chingmei Gravels was responsible for all the disastrous accidents which occurred during the construction of the said system. Problems usually started with small leakages of water either on retaining walls or at the bottom of excavations and the situations soon became uncontrollable as the water paths connected to the Chingmei Gravels. The lessons learned are particularly valuable in the sense that it is anticipated that the tunnels in the future lines of the rapid transit systems are inevitably deeper in depth than before. The situation is aggravated by the fact that, as to be discussed in a later section, the piezometric levels in the Chingmei Gravels are rising with time.

Problems with Soft Ground

The soft clays in the Sungshan Formation are highly compressible as evidenced by the large ground subsidence experienced in the past. The ground subsidence in the Taipei Basin is comparable in magnitude with, if not more than, the subsidence experienced in many other cities located on recent alluvium. Figure 6 shows the ground subsidence recorded in the past in the central city area, and as can be noted, a settlement of 2.2 m ccurred in a 20-year period between 1960 and 1980. This was a result of lowering of piezometric levels in the Chingmei Gravels due to excessive pumping of groundwater in the formation. Fortunately, in the early days, buildings were mostly 6 stories or lower till the late 80's and very few buildings were founded on piles. Therefore, to the author's

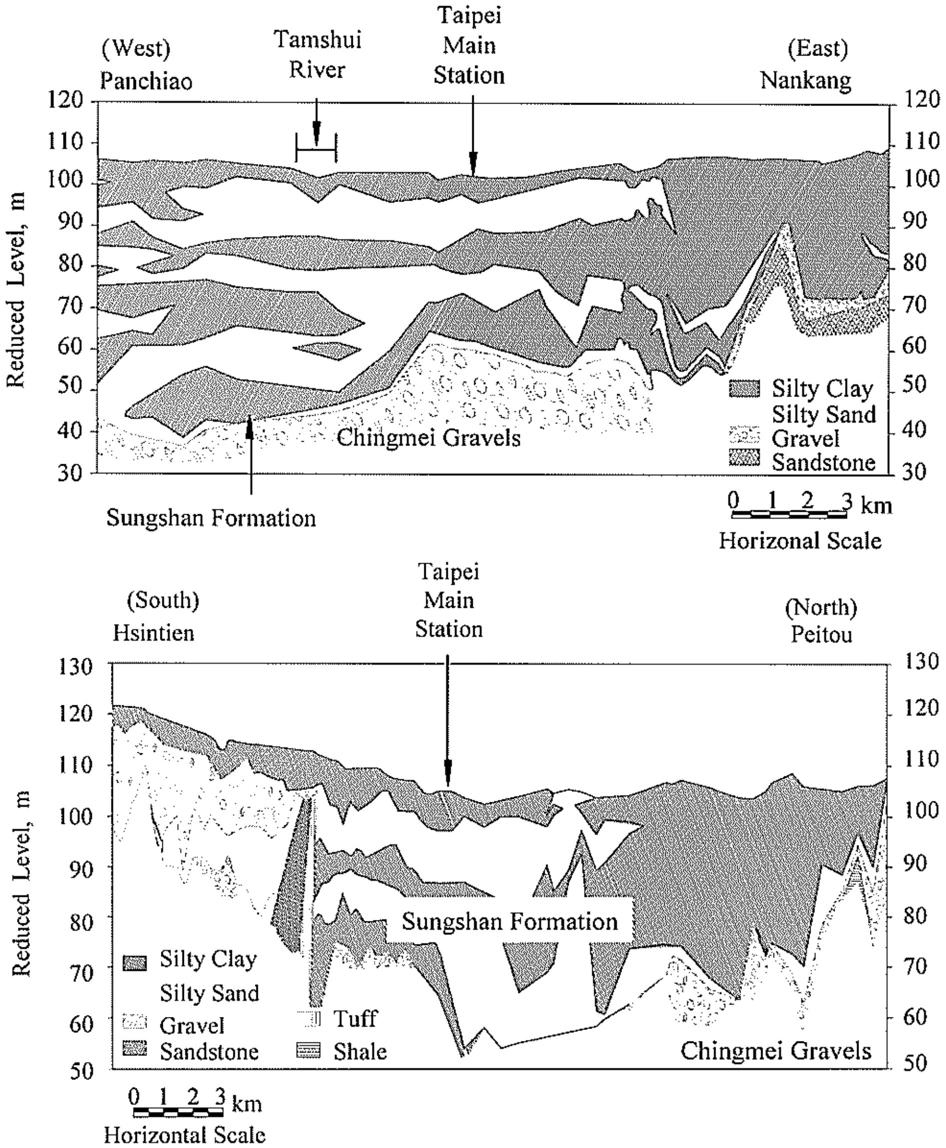


Fig. 4 Geological profiles of the taipei Basin

knowledge, there are no reported cases of failed piled foundations associated with ground subsidence. It was reported that groundwater in the city was in an artesian condition at the beginning of the 20th century. This means, refer to Fig. 6, with a current ground level at RL 102m (mean sea level = RL 100m) or so, the piezometric level in the Chingmei Formation once dropped by more than 50m. After the completion of the Fei Tsui Reservoir in the late 80's which has since supplied sufficient water to the entire Taipei City and part of Taipei County for domestic and industrial uses, deep well pumping in the city area was strictly regulated. The piezometric levels in the Chingmei Gravels, and those in the Sungshan Formation as well, have gradually recovered as depicted in Fig. 7 and ground subsidence has been small in recent years as depicted in Fig. 6.

The high compressibility of the soft clays in the Sungshan Formation, however, necessitates deeper penetrations of retaining walls and better water tightness of the walls for avoiding large drawdown of groundwater table due to seepage which might lead to intolerable ground settlements in the vicinity of excavations. Furthermore, the weak strengths of the subsoils call for thick walls and heavy strutting for limiting wall movements which will inevitably cause ground to settle. In comparison with the practice of the old days, design concept has advanced with due considerations given to ground movements and workmanship has been much improved in the construction of the rapid transit systems. As a result, drawdown of groundwater was generally small and lateral movements of diaphragm walls were less than one third of what was experienced before. Ground settlements in the vicinity of excavations were generally within tolerance and few buildings, if any, suffered from structural damages except in a few unexpected accidents.

The same can be said on ground settlements over tunnels. The construction of the trunk lines of the sewerage systems along Mingtsu Road in 1977 was the first application of shield machines in Taiwan and the tunneling operation had to be suspended shortly after launching as a result of excessive inflow of water into the tunnels and loss of compressed air. The tunneling technique has evolved

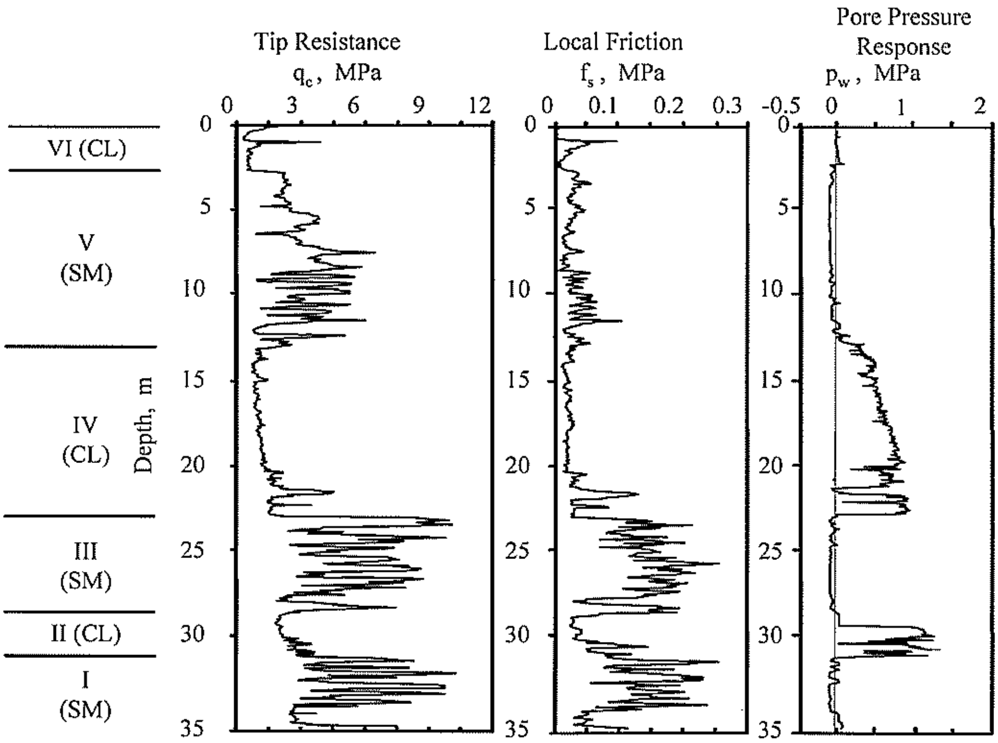


Fig. 5 Typical CPT profile in Central Taipei

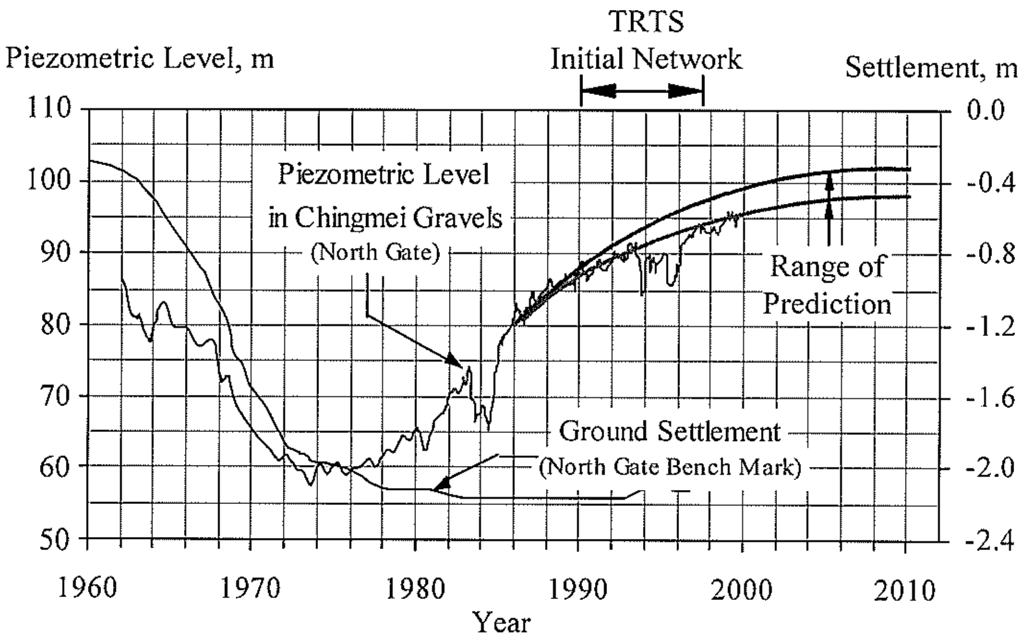


Fig. 6 Piezometric level in the Chingmei Gravels and historical ground settlement in Central Taipei

ever since and the use of earthpressure balancing type and slurry type shield machines in the construction of the rapid transit systems has drastically reduced ground settlements to less than a half of what was before.

A rather unique situation was faced in constructing the underpass beneath the Taipei International Airport (Moh et al, 1999). This underpass is a 4-lane highway tunnel and, as depicted in Fig. 2, is to go underneath a taxiway of 40m and also a runway of 60m in width. The top of the underpass is only 5.6m below the surface of the runway. The service of both the taxiway and the runway shall in no case be interrupted except in the period between 11pm and 6am. This makes the project technically challenging and difficult to manage. As depicted in Fig. 8, because of the poor nature of the ground, interlocked steel pipes of 812.8 mm in diameter were installed by jacking to provide an enclosed protective shelter for excavation to be carried out

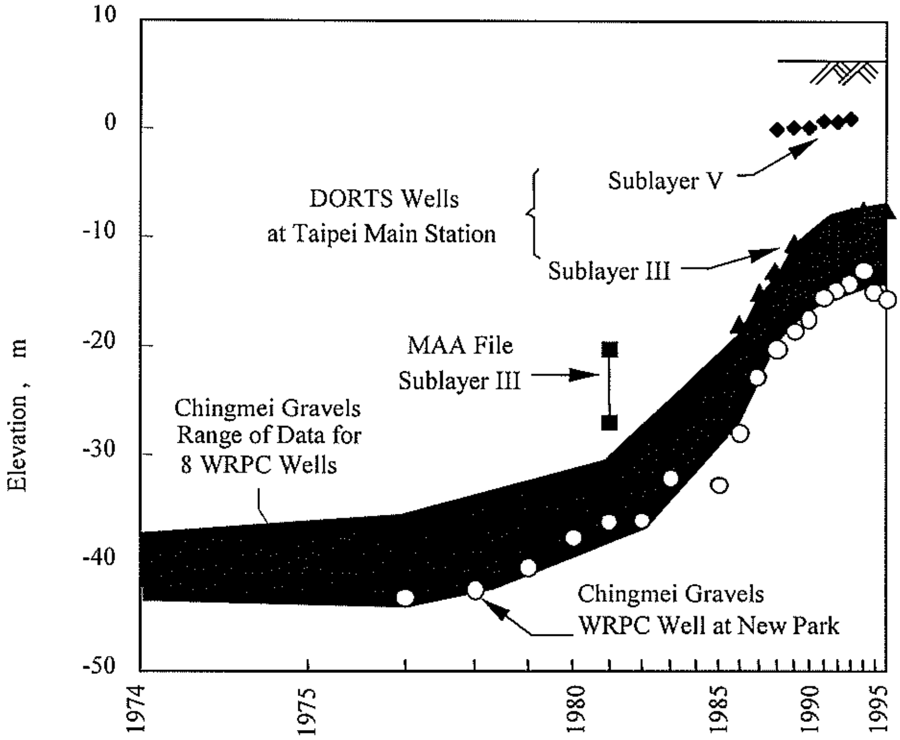


Fig. 7 Piezometric levels in the Sungshan Formation

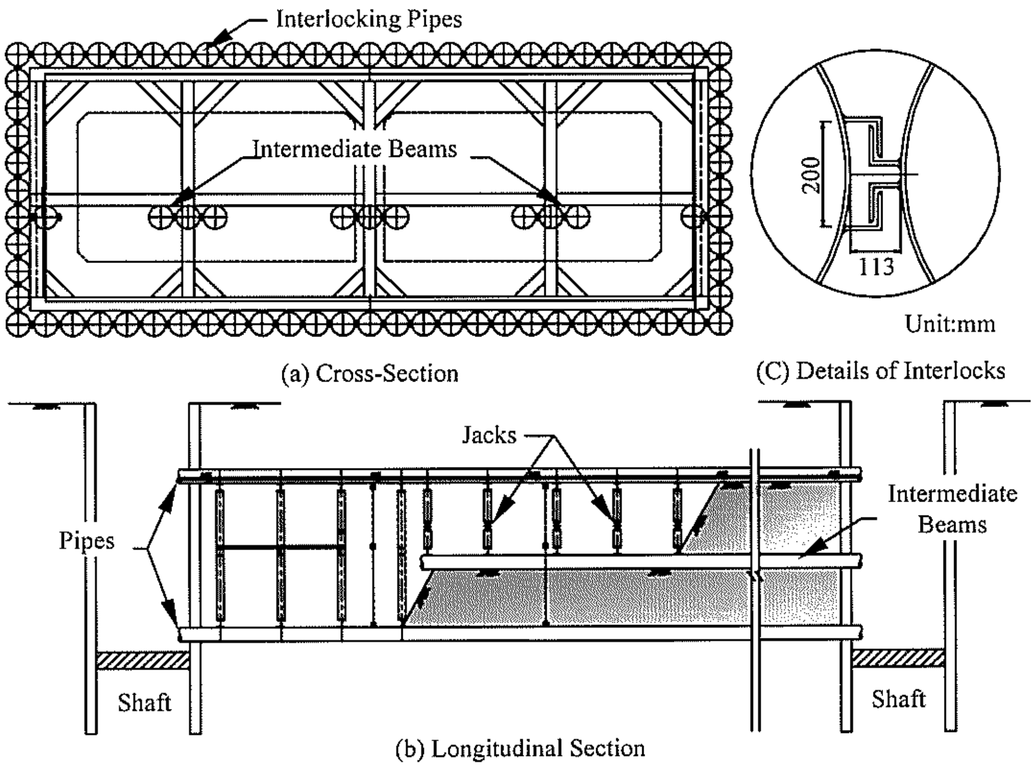


Fig. 8 Tunneling in sections 3 and 4 of the Underpass

inside safely. This, however, still does not guarantee that the stringent settlement criteria can be made. To further reduce ground settlements, as depicted in Fig. 9, the precast tunnel boxes are to be jacked into the shelters under the taxiway and the runway by using the so-called endless-self -advancing method (ESA) in full-section segments of 10.5 m each, except the head segments which are only 6.5m, in length.

Five working shafts, refer to Fig. 2, have been sunk and are inter-connected by a 1,500mm passageway for transporting materials and crew so work can be carried out underground without surface activities. The section of the passageway under the runway is in the way of the tunnel box and has to be removed, piece by piece, while the box is being jacked into the shelter. There will be five guide tunnels, as depicted in Fig. 10, in which rails will be laid for tunnel segments to rest. In addition, there are 12 cable ducts. The basic concept of the ESA method is that each time a segment is being driven, either by jacking or pulling, the majority of the reaction comes from the frictional resistance acting on the rest of segments. For example, refer to Fig. 9b, when Segment 3 is being driven by jacking against Segment 4, part of the jacking force is taken by the frictional resistance acting on segments 4, 5 and 6, part of the jacking force goes to the end of Segment 6 and is taken by cables. The force taken by the cables which run through guide tunnels, refer to Fig. 10, is transmitted to the reaction pad in the arrival shaft. The force taken by cables which run through cable ducts is transmitted to Segment 2 and is resisted by the frictional force acting on Segment 2. This way, the size and the rigidity of the reaction pad can be much reduced. During jacking, lubricant will be injected from grouting holes on the segments to reduce frictional resistance between the tunnel box and sheltering pipes. In theory, the forces taken by the cables and the reaction pad are not affected by the length of the tunnel box and the tunnel box can be as long as desired.

At the time this paper was prepared (April, 2001), all the sheltering pipes have been installed and precast tunnel segments, refer to Fig. 9, have been erected in Shaft E to prepare for launching. Because of the poor nature of the soils, as depicted in Fig. 11, extensive ground improvement has been carried out for increasing face stability and reducing water seepage. In principle, the soil core was solidified by cement-bentonite-water mixture. The soil surrounding the guide tunnels at the two corners was treated by using the double-packer grouting method to yield a minimum unconfined compressive strength of 160 kPa while the unipack method was used elsewhere to yield a minimum unconfined compressive strength of 80 kPa.

Problems with Drift Woods

Jacking of sheltering pipes at the airport underpass was completed not without problems. Pieces of drift woods were encountered at a depth of roughly 6m during jacking of one of the pipes between Shafts C and D and a temporary shaft had to be sunk from surface for them to be removed. Two of the pieces removed measured about a half meter in length and 200mm in diameter.

Chunks of drift woods of 1m or so in diameters were recovered in numerous excavations in the past and they were as long as 5m in length and as thick as 1 m in diameter. Most of the woods recovered appear to be reasonably fresh. For academic interest, a piece of wood found at a depth of 9m in Observation Well 2 in Panchiao was sent to laboratory for dating and the results indicated an age of 6,760 years and another piece found at a depth of 23m dated back to 7,950 years (Liew, 1994). Although drift woods were frequently encountered, to the author's knowledge, few problems occurred prior to the construction of the rapid transit system.

The problem with drift woods was, however, serious during shield tunneling in the constructions of the rapid transit systems. Although the presence of drift woods was recognized in the design stage and was well reported, however, it is still very difficult to predict the locations and depths of drift woods with any degree of accuracy. To prepare for the problem, all the shield machines adopted in the construction of the rapid transit system have sufficient strength and power to cut through the woods as long as they are not too large in size. Pieces of drift woods were indeed encountered at many locations along the Panchiao Line and the Chungho Line and in most of the cases the shield machines were able to advance with efforts . However, there was one occasion in which the presence of drift wood caused the ground to collapse as the shield machine was slowed down while mud kept on running into the earth chamber. Finally, a sinkhole of 5m in diameter occurred right above the head of the shield machine. As depicted in Fig. 12 (Lin et al., 1997a), ground treatment was carried out in front of the machine to enable workers to go out of the machine to remove the obstacles and to repair damaged cutting blades. Two pieces of drift wood, 500mm and 400 mm each in length, were recovered. To avoid road surface from collapsing should drift woods be encountered again, the remaining section of tunnel was treated as illustrated in the figure.

The problem with drift woods was the most serious in the Panchiao Line and several disastrous accidents occurred during tunneling. Although shield machines were able to chop drift woods into pieces, the movements of the woods as they were stirred by the cutting blades destroyed the integrity of ground treatment in front of working shafts and created fissures leading to leakage of water into these shafts. Figure 13 shows one of such cases in which groundwater spurted into the arrival shaft when an opening was made on the diaphragm wall for receiving the shield machine (Lin et al., 1997b). The flow soon became uncontrollable as the water path connected to the Chingmei Gravels and a sinkhole of roughly 4,000 m^3 in volume was created. The shaft had to be recharged to stop water from running into the shaft. Both tunnels were flooded and one of them had to be abandoned as a result. Probing from surface indicated that 34 rings in the down-track tunnel and 39 rings in the up-track tunnel were damaged and had to be replaced. The opening was later sealed by a gravity retaining wall formed in water and by ground freezing to solidify the surrounding soils (Ju, et al. 1998). When the opening was re-opened after damaged rings were replaced, a piece of wood was found at the location where water spurted. Right next to this piece of wood was a PVC pipe, which is believed to be an abandoned pumping well, extending all the way down to the Chingmei Formation. It is hypothesized that the ground treatment, which was carried out by using the CJG method of grouting, was disturbed as the wood was stirred by the shield machine and groundwater was able to find its way into the shaft. The PVC pipe was chopped off and the lower portion of

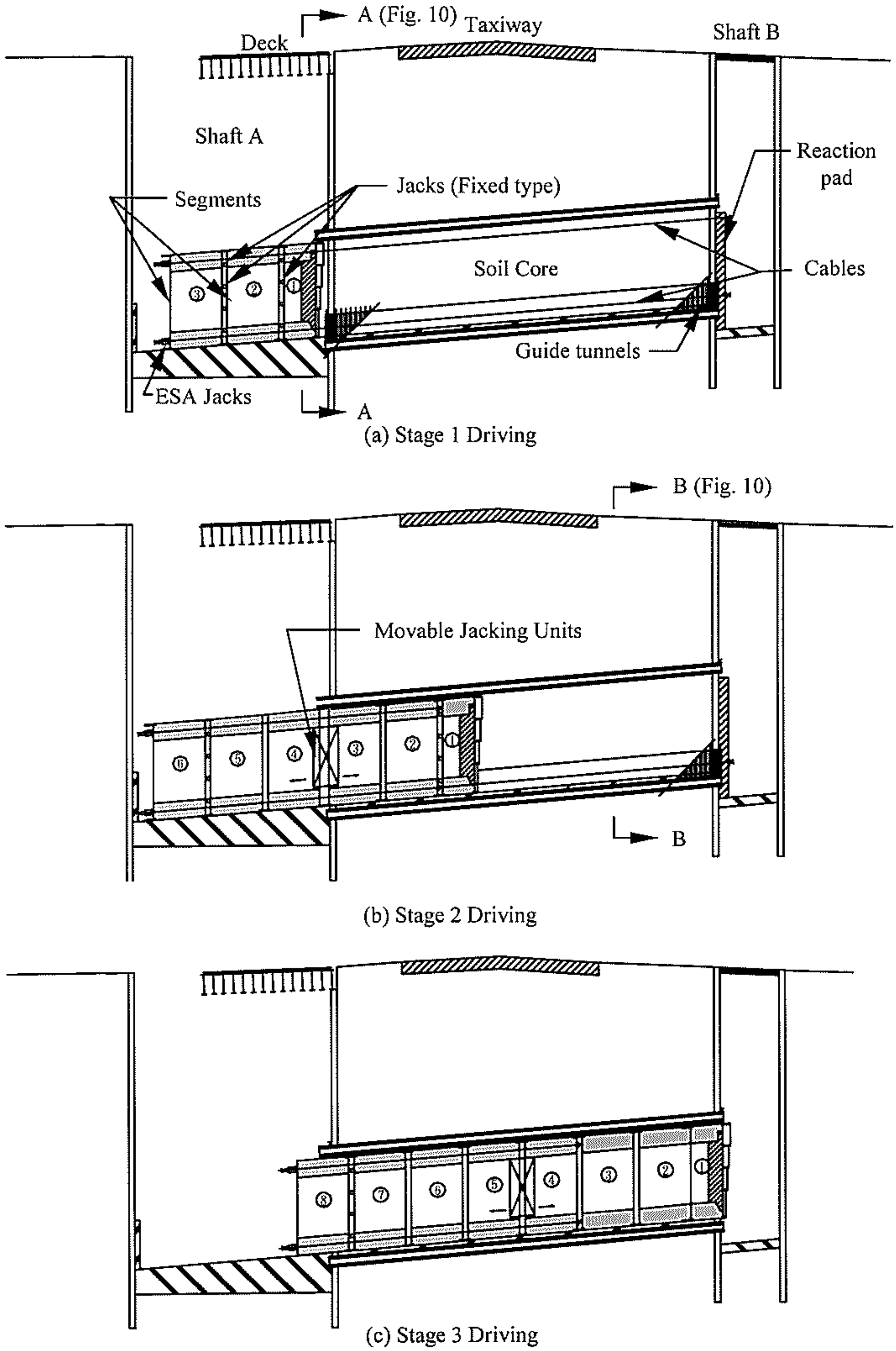


Fig. 9 ESA Tunneling in Section 2 of Underpass

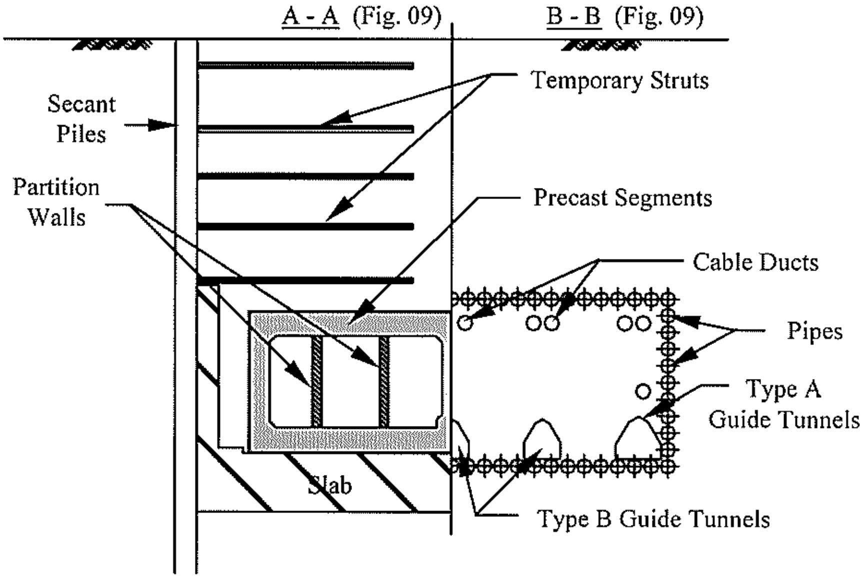


Fig. 10 Cross-section of ESA layout of the Underpass

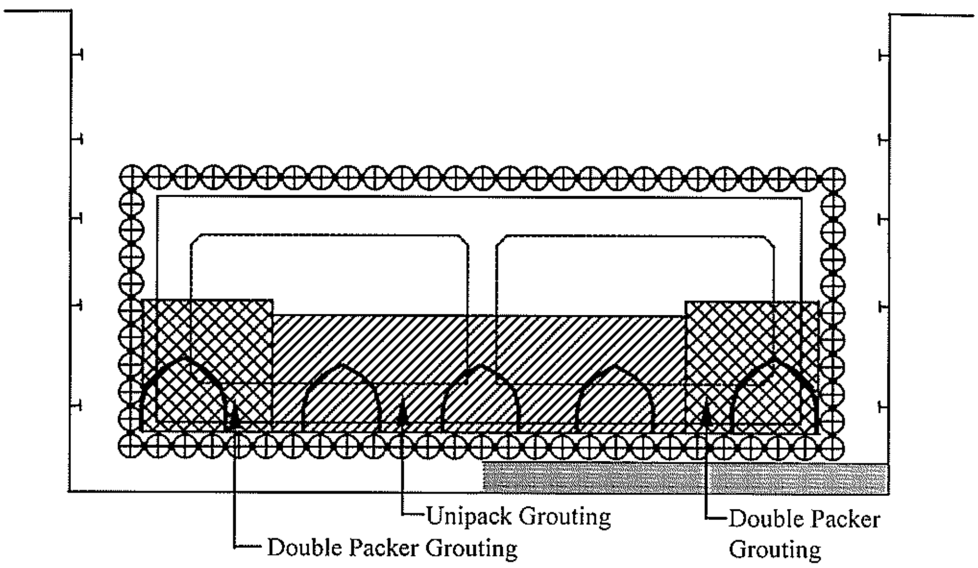


Fig. 11 Ground improvement for ESA work

the pipe became a water path connecting to the Chingmei Gravels which practically became a reservoir with a huge water supply.

Problems with the Chingmei Gravels

The problems with the underlying Chingmei Gravels were many and the above-mentioned case is definitely not an isolated one. This gravelly layer is extremely permeable and very rich in water. The fact that it was the sole water supply for the entire Taipei City prior to the 70's tells how ample the water reserve is in this layer. Experience indicates that once water path connects to the Chingmei Gravels, it will be extremely difficult to stop water from running into excavations. Recharging appears to be the only option although such a decision is painful to make because the consequences are usually grievous. Another accident as disastrous as the one mentioned above happened in the Hsintein Line when a tunnel portal on the diaphragm wall of a working shaft was enlarged to enable the expansion joint to be installed between the tunnel and the shaft (Hwang, et al. 1998). As shown in Fig. 14, although the surrounding ground had been treated before, water was able to find its way into the shaft. Because the

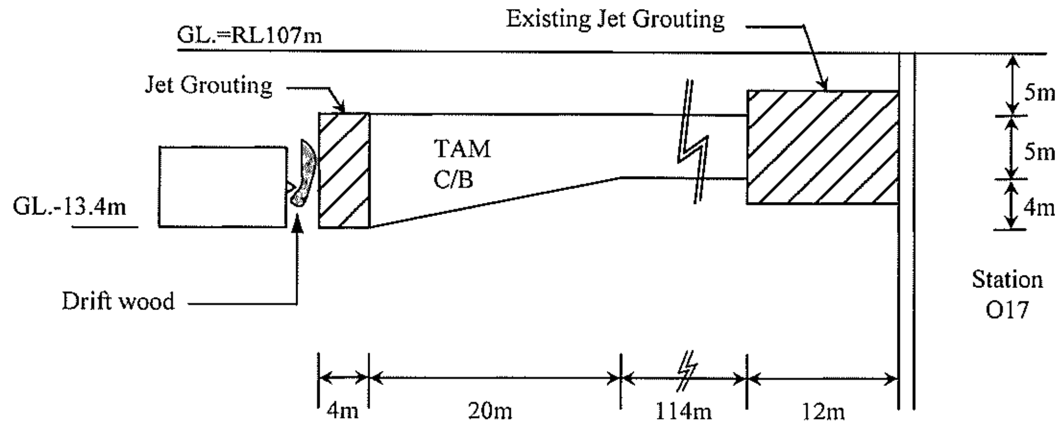


Fig. 12 Ground treatment at the up-track tunnel of Contract CC277 of TRTS

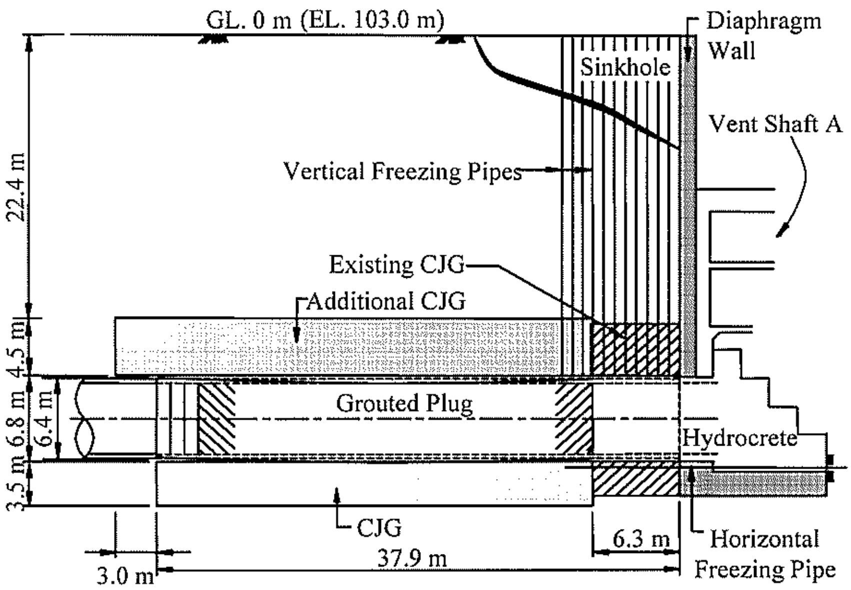


Fig. 13 Settlement profile at the arrival shaft in Contract CP262 of the TRTS

portal is so close to the Chingmei Gravels, flow soon became uncontrollable. The shaft also had to be recharged to balance the groundwater pressure. A section of the up-track tunnel was damaged and 23 segments had to be replaced. The down-track tunnel, fortunately, was not affected. To enable the remedial measures to be taken, the portal was sealed by ground freezing to form a patching pad. A steel diaphragm was installed at the other end of the damaged section of the tunnel and the damaged segments were replaced in compressed air.

The problems with the Chingmei Gravels were not limited to tunneling. Blow-in and piping are serious concerns in deep excavations wherever the Chingmei Gravels exists and provisions had to be made for excavations to be carried out safely. For examples, excavations for constructing the three ventilation shafts of TRTS were carried out to depths exceeding 30m and could not be carried out safely unless measures were taken to resist the groundwater pressures. For Ventilation Shaft A in the Panchiao Line shown in Fig. 13, continuous pumping had to be carried out to reduce the piezometric level in the Chingmei Gravels, as shown in Fig. 15, from RL. 88.3m to 77.6m to give a factor of safety of 1.25 against blow-in. The rate of pumping was as much as 4170 m^3/hr. For constructing Ventilation Shaft B in the same line, refer to Fig. 16, pumping was carried out at a rate of 3600 cmh to lower the piezometric level in the Chingmei Gravels from RL. 89m to RL. 79.5m. As shown in Fig. 17, groundwater drawdown was significant at distances exceeding 5 km or so in these two cases.

In each of the two ventilation shafts in the Panchiao Line, refer to Figs. 15 and 16, there were a soil plug of significant thickness below the bottom of excavation and above the Chingmei Gravels. For constructing the ventilation shaft in the Hsientein Line shown in Fig. 14, the situation was more critical because the excavation was carried out all the way down to the top of the Chingmei Gravels. Instead of dewatering, the contractor opted to extend the diaphragm wall to a depth of 30m into the Chingmei Gravels and seal off the bottom by grouting to form an artificial soil plug. Installation of diaphragm wall in this gravelly layer to such a depth was, however, a difficult task. One of the panels took 33 days to dig and it took half of a year to complete the entire installation. The grouted pad at the bottom, however, did serve the purpose of sealing off the seepage water and the pit was dry all the time during the excavation.

In all the three cases depicted in Figs. 14, 15 and 16, excavations, per se, were completed without accidents. An unexpected event, however, refer to Fig. 18, did occur at one station during excavation where drilling for replacing a malfunctioned piezometer penetrated through the clay blanket covering the Chingmei Gravels and caused water in the Chingmei Gravels to rush into the excavation (Moh et al., 1997). At the time when this happened, excavation had reached its final level and the bottom of the excavation had been paved by using lean concrete. Similar to the two cases mentioned above, the pit had to be flooded to balance the water pressure. The water path was sealed by grouting for the rehabilitation work to be carried out.

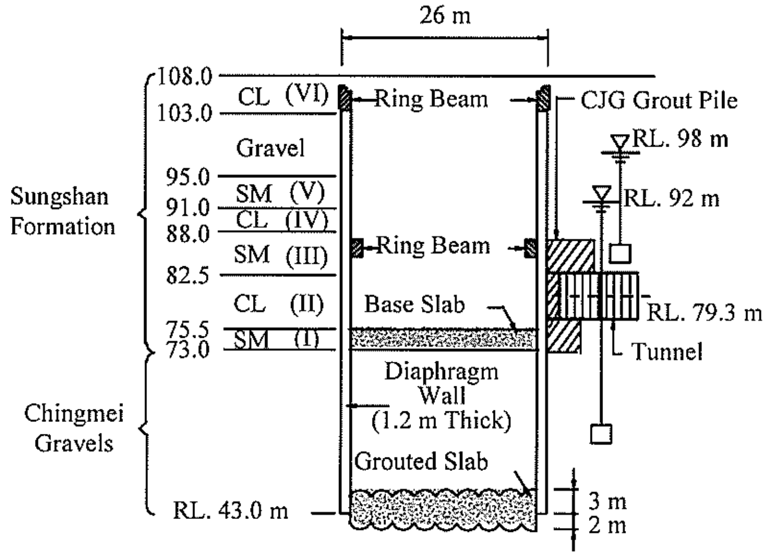


Fig. 14 Ground conditions and configuration of the Contract CH221 ventilation shaft, TRTS

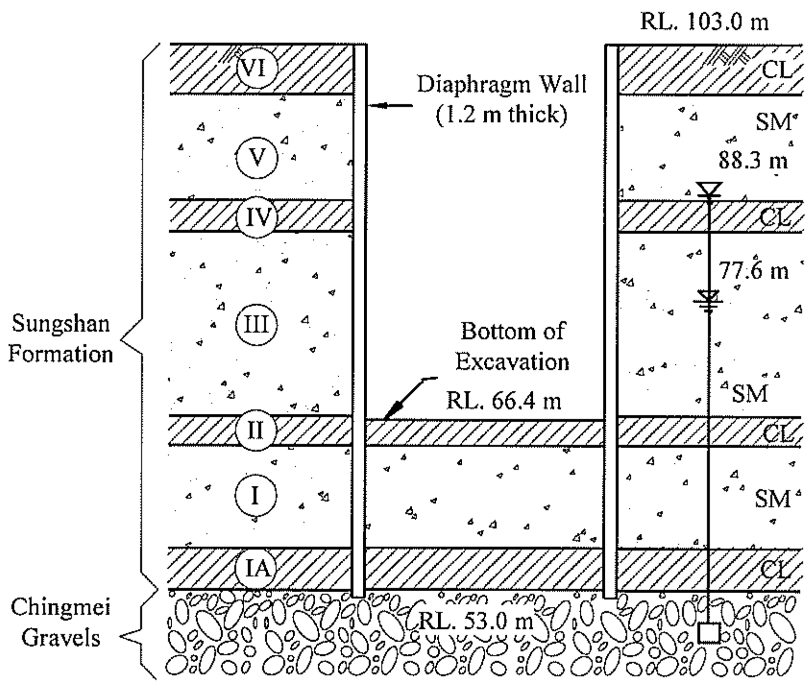


Fig. 15 Pumping at Ventilation Shaft A in Contract CP262, TRTS

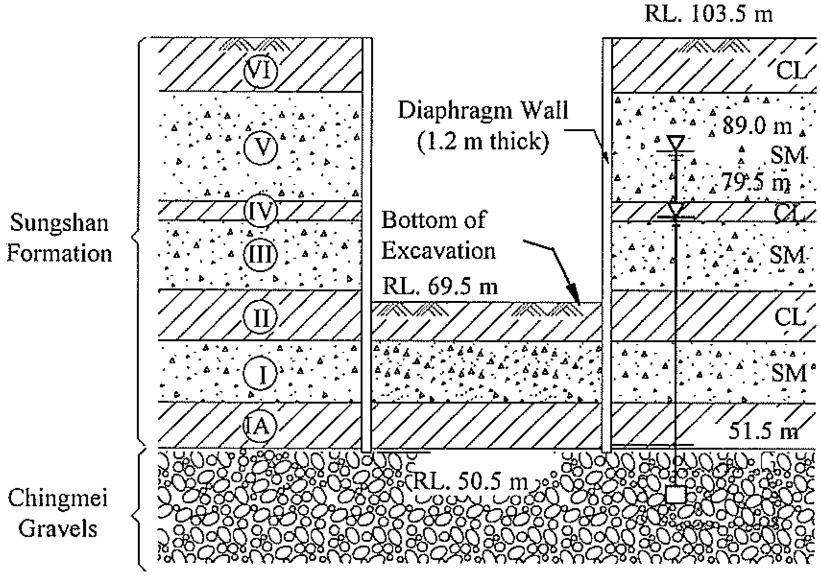


Fig. 16 Pumping at Ventilation Shaft B in Contract CP261, TRTS

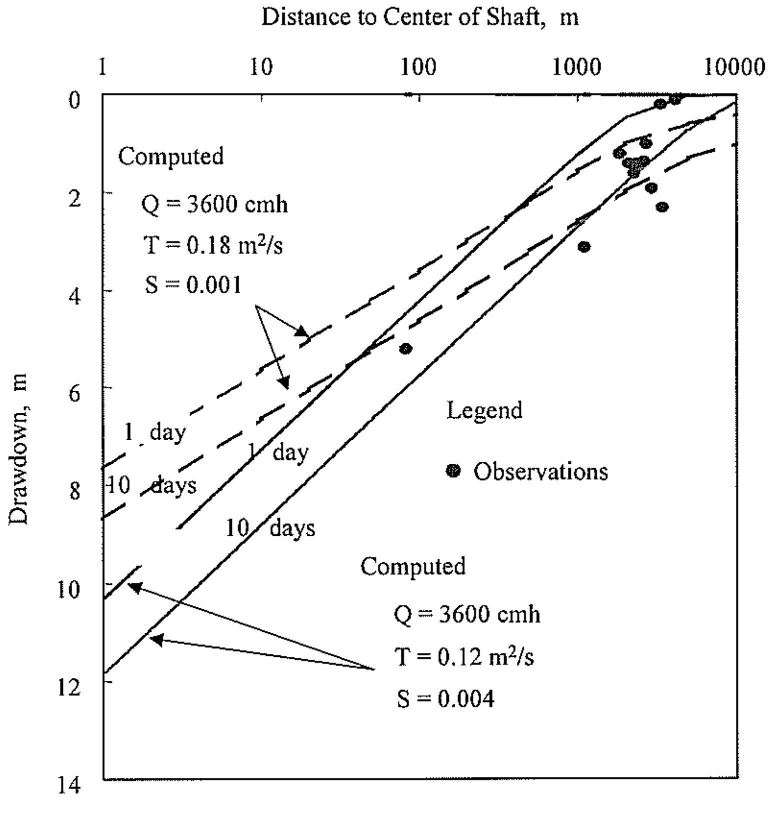


Fig. 17 Influence of dewatering in the Chingmei Gravel

Problems with Methane

Methane was encountered in several boreholes during site investigation. Its presence is related to the capture of gas in domes encapped by impermeable clays (Lin et al., 1997a). The depths at which methane was encountered vary from 20 m to 33 m and correspond to Sublayer 1 of the Sungshan Formation. In one case, eruption sent a jet of methane-water mix to a maximum height of about 10 m into the air and continued for more than 3 days. A piezometer which was installed previously at a depth of 34.75m in the hole was blown out. It took 2 months for the pressure to totally release. Measurements were taken to determine the concentration of methane and the results indicated concentrations up to 5%. Laboratory tests indicated a chemical composition of CH:24 ppm, O:0.3 ppm, CO:18 ppm and H_2S:0 ppm.

Methane is usually dissolved in groundwater and might be carried by seepage water into tunnels wherever leakage exists. Therefore, as long as the tunnel is watertight and provided there is some circulation of air, the danger for methane to accumulate to a harmful concentration is practically small. Methane is potentially explosive when its concentration is in the range of 5% to 15% and the results could be fatal. In the construction of the Taipei rapid transit system, contractors had been warned of the possibility of encountering methane in tunnel drives and possible consequences before they tendered. As a precaution, specifications required the concentration of methane be continuously monitored during tunneling and shield machines be equipped with detective devices and alarm systems. The capacity of ventilation in tunnels was increased from 780 m^3/min to twice as much to reduce the concentration of methane, if any. The power supply would be automatically cut off whenever the concentration of methane reached 1.25% and resume only after the concentration dropped to 1% or below.

Although methane was encountered at locations scattering all over the entire Taipei Basin during investigation, the problems, to the author's knowledge, were limited to the Chungho Line during the construction of the rapid transit system. To play safe, whenever methane was found in a drilled hole, relief wells were sunk in the neighborhood of the hole to release methane. Tens of such relief wells were sunk for the purpose. During the drive of down-track tunnel of Contact CC 275 of the Chungho Line, methane was indeed encountered at Rings No. 127, 150, 176, 187 with concentrations varying from 17% to 57%. However, because of all the precautionary measures, its presence did not cause any consequences.

SOFT GROUND CONSTRUCTION IN BANGKOK REGION

Projects

(A) Bangkok-Chonburi New Highway

Traffic congestion is always a major problem in Bangkok and its outskirts. During the recent years, the industrial activities and associated urbanization in the east and southeast of Bangkok have been growing up rapidly. Two ports at Laem Chag and and Map Ta Pud and the future Second International Airport at Nong Ngu Hao are also expected to increase the traffic volume. In order to increase the capacity and carriageways in the regional network, the Department of Highway (DOH) decided to construct its first motorway project – the Bangkok-Chonburi New Highway (BCNH) with fully-controlled access facilities including grade separated interchanges and flyovers. Table 1 summarizes the design criteria of the BCNH.

Table 1 Design Criteria of BCNH

Project Length	81.7 km
Design Standard	Divided 4 lanes
Design Speed Main Road Frontage Road Ramp	 110 km/hr 80 km/hr 40 km/hr
Design Service Life	15 years
Design Elevation	30 cm min above high water level during service life
Traffic Expectation Year 2000 Year 2008	 16,500~39,700 vehicles/day 25,800~61,300 vehicles/day

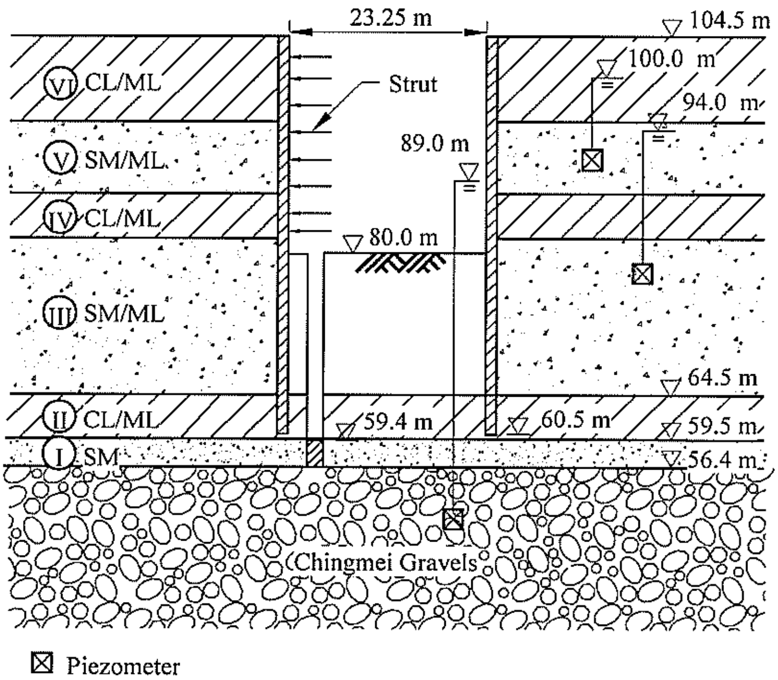


Fig. 18 Soil profile and configuration of excavation

As shown in Fig. 19, the BCNH begins at Sri Nakarin Road connecting the Second Stage Expressway and extends eastward to join Route No. 36, the New Chonburi-Pataya Highway. The total length of the BCNH is 82 km including eight interchanges. The total construction cost of the BCNH was 12,750 million Bahts (about 500 million US dollars), of which 50% was OECF loans from Japan. The construction of the BCNH was commenced in June 1994 and the highway was open to traffic in December 1997.

Almost all of this project route, except about 15 km at the end, is traversing on the flat central plain of Thailand where the underlying soils consist mainly of soft marine clay up to depths of 15 to 20m. Figure 20 shows the soil profile along the project route with related design profile grade. Based on the subsoil properties, the whole project route has been divided into five zones, i.e., Zone 1 to Zone 5. The basic properties of the very soft to soft clay in each zone are summarized in Table 2.

Detailed design of the highway was carried out by a consortium led by Thai Engineering Consultants Co. Moh and Associates was responsible for all the geotechnical related work during construction.

(B) Second Bangkok International Airport

Development of the Second Bangkok International Airport (SBIA) in Thailand has been under planning for more than 20 years in order to meet the foreseeing air traffic growth in the region. This growth is a result of Thailand's dynamic economical and tourism development and will accelerate the SBIA to be the region's key central aviation hub. Master plan and engineering studies for the new airport were carried out in 1984 and the government finally approved the SBIA project in May 1991. The New Bangkok international Airport Company Limited (NBIA), a state-enterprise under the Ministry of Transportation and Communications, was formed in February, 1996 to implement the SBIA project, which is scheduled to open for operation in 2004 to serve 30 million passengers and 1.46 million tons of cargo each year during the first phase. As a final goal, the SBIA will accommodate up to 100 million passengers and 4.6 million tons of cargo annually. The total construction cost is estimated to be 120 billion Bahts (about 3 billion US dollars) of which over 60% will be used for engineering.

The project site is located at Nong Ngu Hao in Samut Prakan Province, about 30 km east of central Bangkok (refer to Fig. 21). The site is located in a low-lying flood-prone area and covers an area of, approximately, 3,200 hectares with an average elevation of less than one meter above the sea level.

The subsoil conditions are relatively uniform over the entire airport site. The soil profile, as illustrated in Fig. 22, is typical of the deltaic deposit in the central plain of Thailand. The well known Bangkok Clay at the SBIA site consists of relatively distinct layers of weathered crust, up to 1.m thick, followed by 1.5 to 11 m of very soft to soft clay. The soft soil is underlain by a few meters of medium clay and then stiff clay. Below a depth of about 25m, the site is underlain by a dense sand stratum. The layer of very soft to soft clay possesses very high water content, usually over 100%, low shear strength and high compressibility. It is this layer of soil which dictates the design of the airport facilities.

Detailed design of the airside pavement for the SBIA was carried out by the Airport Design Group consisting of DMJM International, Scott-Wilson-Kirkpatrick, Norconsult International, SPAN and SEATEC. Construction supervision of the ground improvement work for the airside pavement is being carried out by a consortium of which MAA Consultants, Co. is a member and is responsible for all geotechnical related work.

Ground Improvement Scheme – Design and Construction

As described in the previous section, the subsoils at the two project sites are typical of the Bangkok Clay deltaic deposit in the central plain of Thailand. The underlying layer of very soft to soft clay with high natural water content, low strength, and high compressibility is the source of many failures or severe damages, including stability problem and large settlement. Undulating pavement surfaces and unstable embankment slopes are common scenes at many highways, particularly the existing Highways No. 3 and No. 34. Extensive research studies have been carried out in the last 20 years to improve the soft Bangkok Clay for large scale construction. Many different methods and schemes have been evaluated either to increase the strength or to accelerate consolidation or both, including preloading with surcharge or vacuum, vertical prefabricated drains (PVD), sand drains, large diameter non-displacement sand drains, cement/lime columns, etc. (e.g., AIT, 1974; Moh and Woo, 1987; Bergado, et al., 1998). Based on these study reports, the use of preloading with PVD appeared to be the most suitable method of ground improvement for the two projects since compression of the Bangkok Clay is dominated by primary consolidation and there were sufficient time available for the compression to take place prior to the target completion date of the projects.

According to available records, sand drains and PVD were used only in small scale projects or on a trial basis in the Bangkok area. The Bangkok-Chonburi New Highway was the first large scale highway project that used PVD to accelerate the consolidation of Bangkok Clay. A total of 23 million linear meters of PVD was installed in the BCNH. Ground improvement for the Airside Pavement, one of the initial projects for the SBIA, was commenced in November, 1997 in order to prepare the site for the construction of permanent structure in future. Besides site clearing and leveling for about 53% (1,699 hectares) of all the airport sites, ground improvement by using PVD and surcharge load is applied to the Airside Pavements including West Runway, Taxiways, Apron, part of East Runway and two access roads to accelerate consolidation settlement before the construction of pavement or the facilities and to minimize maintenance cost after the operation (Fig. 23). The total improvement area is about 10% (308 hectares) of the total airport site. The contract amount for the first phase ground improvement project is 8.24 billion Bahts with 32 million linear meters of PVD installed. The project will be completed in April 2002. Further ground improvement work on the remaining East Runway, Cargo

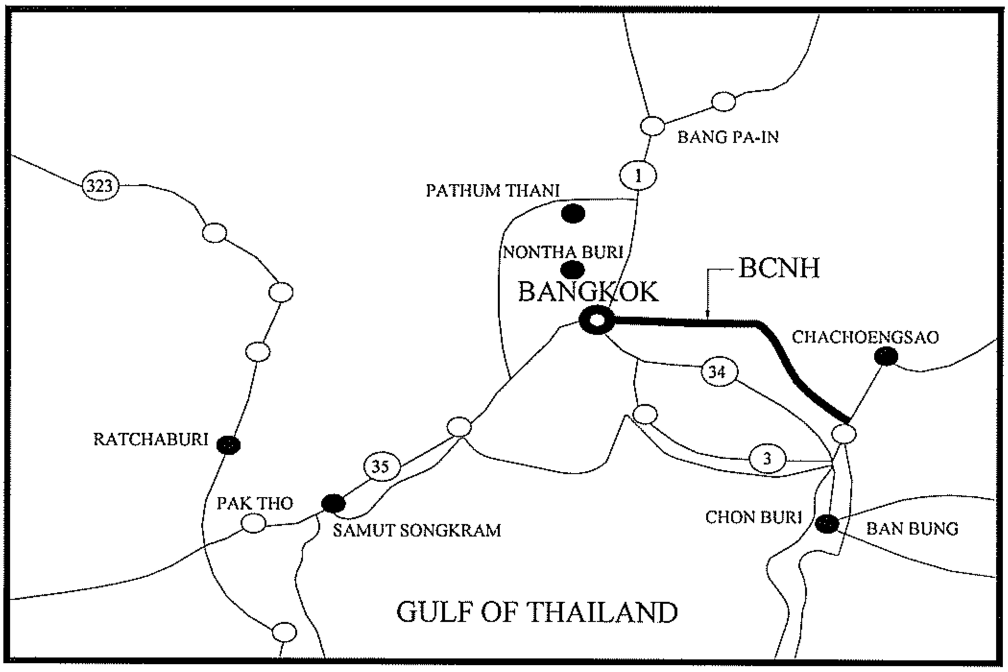


Fig. 19 Location Map of Bangkok-Chonburi New Highway

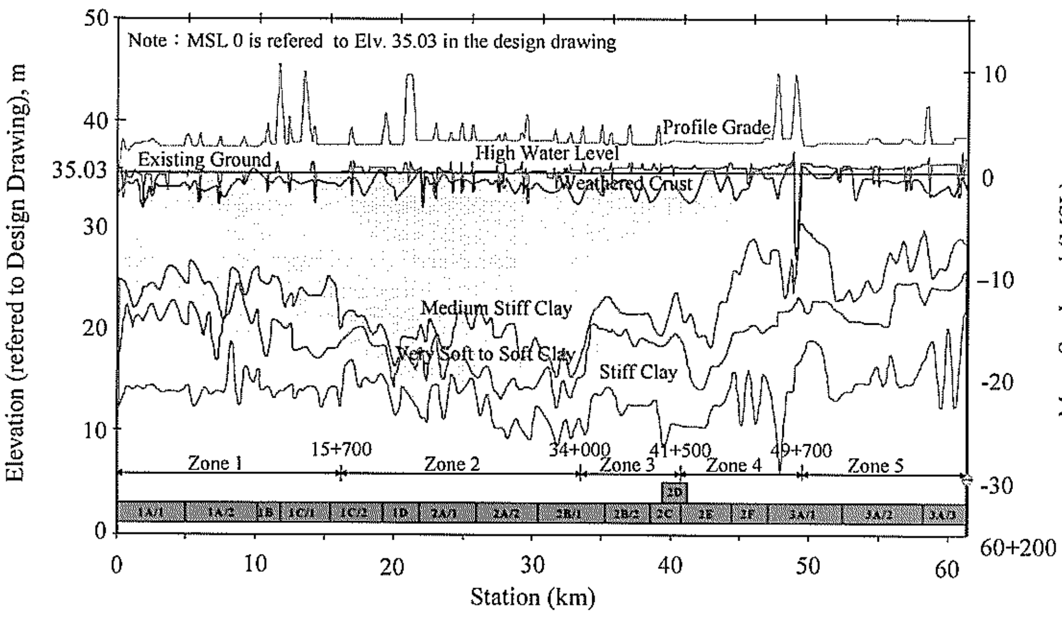


Fig. 20 The soil profile within PVDs' installation area along the BCNH

Table 2 Soil Properties of Soft Clay along BCNH

Section (km)	w (%)	γ_t (t/m^3)	w_L (%)	Ip (%)
Zone 1 (0+000~15+700)	60~120	1.4~1.5	70~140	30~40
Zone 2 (15+700~34+700)	70~160	1.4~1.5	90~140	30~60
Zone 3 (34+000~41+500)	70~120	1.4~1.5	70~120	40~60
Zone 4 (41+500~49+700)	70~90	1.4~1.5	70~90	30~40
Zone 5 (49+700~60+200)	70~120	1.4~1.5	70~100	30~50

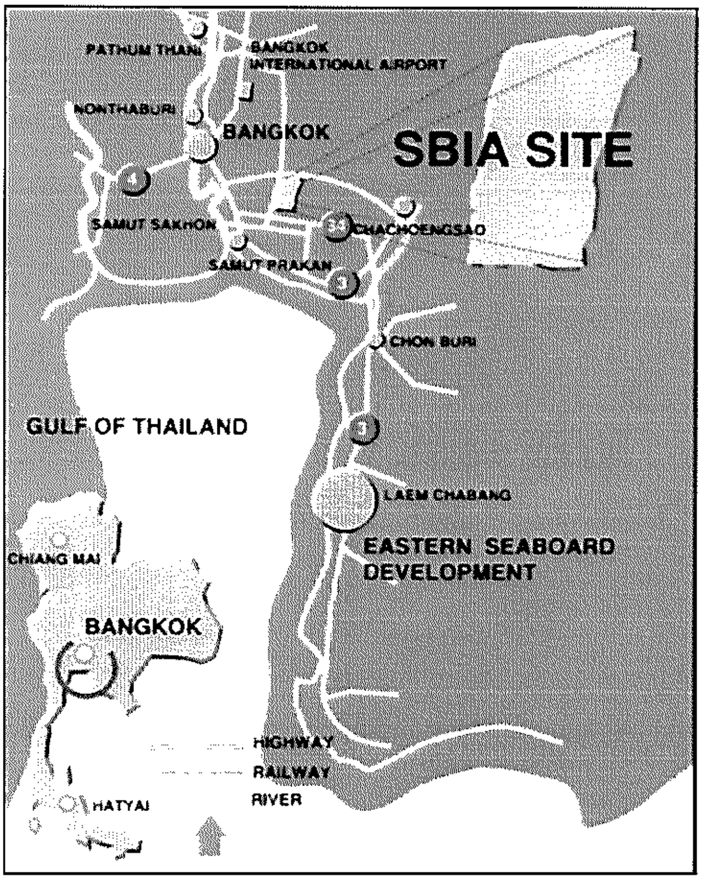


Fig. 21 Location Map of the SBIA Site

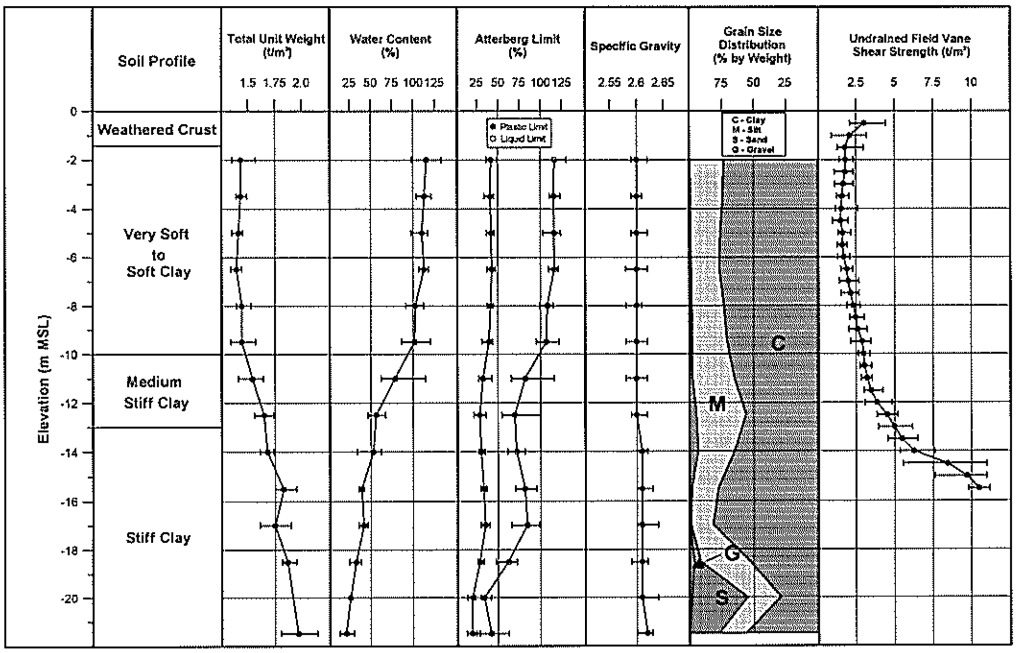


Fig. 22 Typical Soil Profile at SBIA site

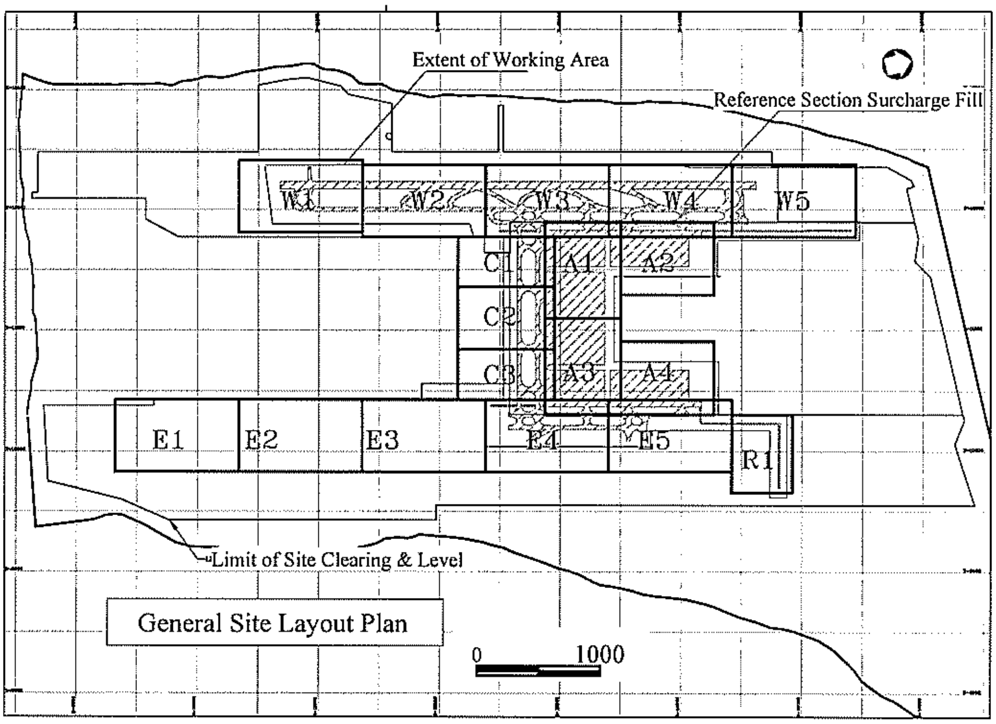


Fig. 23 Layout plan of ground improvement work for Airside Pavements, SBIA

Aprons and Landside Road System will be implemented in 2001 at an estimated cost of about 5 billion Bahts under a separate contract and will not be discussed in this paper.

Due to cost constrain and different design philosophy of the design consultants, details of the design for the two projects were somewhat different. Figure 24 shows typical cross-section of the highway embankment and the airfield pavement. Comparisons of the design parameters for the two projects are summarized in Table 3. It should be noted that in both cases, the ground improvement target has been set at 80% of estimated primary consolidation settlement.

Instrumentations including piezometers, surface settlement plates, deep settlement gauges, inclinometers and lateral movement stakes were installed at controlled sections. For the SBIA, a 500m long "Reference Section" with full instrumentations was constructed prior to the full scale ground improvement work for the purposes of: (1) to confirm the design assumptions and criteria for acceptance of improved ground, (2) to check the contractor's working methods, and (3) to check the installation procedures and suitability of the instruments used.

The following sections give a brief summary of the extent of the ground improvement work in the two projects. Special discussions regarding problems encountered during construction and counter-measures undertaken are presented. More detailed discussion of the two projects were reported by Moh et al. (1998), Ruenkrairergsa et al. (2001) and Lin et al. (2000)

Performance of Ground Improvement

(A) Settlement

For the BCNH, settlement values along the entire route corresponded well with the thickness of the soft clay with the highest value occurring at sections of Zone 2 as depicted in Fig. 25. More than 250 cm of settlement was recorded at Stations 2A/1, 2A/2 and 2B/1. Figure 26 shows typical settlement profiles and lateral movement of the foundation soils. The observed vertical and lateral movement with time are illustrated in Fig. 27.

Since the field measured settlement values appeared to be much higher than the value originally predicted by the designer, Asaoka's method (Asaokia, 1978) was used to predict the ultimate consolidation settlement by use of field monitored data in order to determine the 80% degree of consolidation criterion for removing surcharge load. The results are shown in Table 4. Post construction survey one year after the completion of construction at Section 2A/1 between Sta 24+800 and Sta 25+400 indicated additional settlement of 3.6 to 5.4cm (JICA, 2000). Adding these values to the observed settlement at the end of construction, 275.4 cm (average), it is still less than the total settlement value estimated by Asaoka's method. Judging from the settlement-time curve, the highway appears to be continuing to settle, but at a slower rate.

For the SBIA project, comparison of settlement among field performance of the reference section, design estimation and data from AIT test embankment (AIT, 1995) are shown in Fig. 28. In Fig. 29, typical settlement and lateral movement profiles are shown. The field measured settlement, both the amount and trend of variation with time match well with that predicted. Comparing with the data in the highway project, the low predicated settlement values in the latter case could partly be contributed to the larger lateral movement of the subsoil, with maximum in the order of 500mm as comparing to 120mm in the SBIA case.

The effectiveness of PVD in accelerating consolidation of the Bangkok subsoils is well illustrated in Fig. 30. There was a difference of 50cm in settlement between PVD treated area and area without PVD one month after the second stage loading.

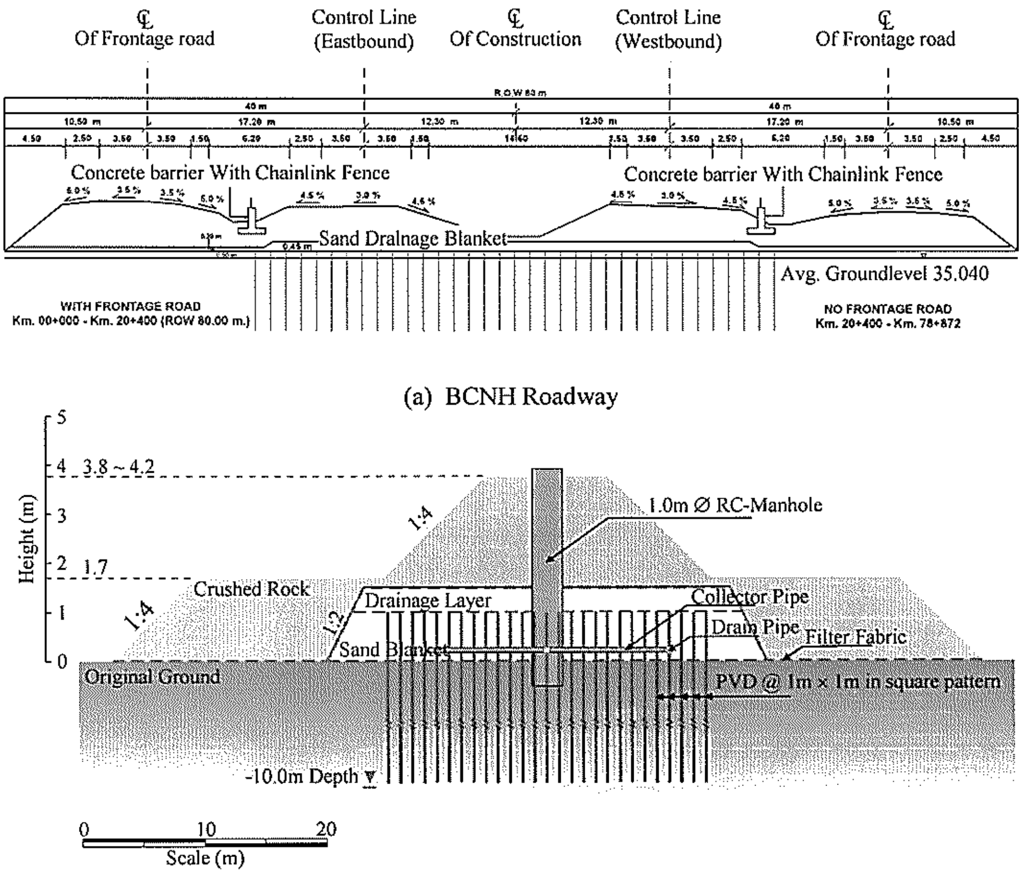
(B) Stability

Stability of embankment during the ground improvement period was one of the major considerations in controlling the rate of surcharge loading as well as removal of surcharge in both projects. Instrumentations have played an important role in the safety control. Besides a regular program, monitoring frequency of the instruments were adjusted according to the construction activity, to the rate of which the readings are changing, and to the requirements of data interpretation.

In the design, the calculated total settlement consisted of consolidation settlement only, without considering the effect of shear strains caused by lateral displacement of the soft subsoils. Ladd (1991) has reported that the ratio of maximum lateral movement to maximum settlement for embankment fill on normally consolidated clay could be as high as 0.9±0.2 in the initial stage of loading and drops to 0.16±0.07 when consolidation settlement predominates. A value of 0.33 was adopted as the criteria for stability control in the two projects. Special attention was however given to the control of construction rate (i.e., loading by fill) when the ratio reached 0.25. For the BCNH, the amount of lateral movement of the soft subsoil was quite large and the value is closely related to the thickness of the soft clay, as shown in Table 5.

(C) Changes in Soil Properties

In general, consolidation of soil should cause change in soil properties, including decrease in water content, void ratio and permeability and increase in dry unit weight and shear strength. Significant improvement in the properties of the subsoils were achieved in both projects due to the preconsolidation by using ground improvement. Figures 31 and 32 show comparison of soil properties before and after ground improvement work. The figures clearly indicate that in both projects there were significant decrease in the water content and increase in undrained shear strength. These changes of properties became very small beyond the depth of the PVD. Figure 31 further shows that the effect of PVD was most obvious during the first year, maybe within even shorter period, after the ground improvement work was started



(a) BCNH Roadway

(b) SBIA Airside Pavements

Fig. 24 Typical cross-sections of ground improvement

Table 3 Comparison of Ground Improvement Work between BCNH and SBIA

	BCNH	SBIA
Average Thickness of Soft Clay	8~12m	9~11m
Embankment Fill Thickness	2.55~3.00m	3.80~4.20m
Thickness of Drainage Sand Layer	40 cm (PVD portion) 20 cm (berm)	1.5m
Filter Fabric	none	2 layers
PVD Length	6m ~ 12m	10m
Preloading Material	sand	crushed rock
Surcharge Load	50~60 kPa	75 kPa & 85 kPa
Stage Loading	3 stages	2 stages
Required Waiting Period after Final Stage Loading	9 months	6 & 11 months
Required Settlement Rate before Surcharge Removing	≦2 cm/month	≦4% or 2% of settlement ratio
Required Percentage of Primary Consolidation	80%	80%

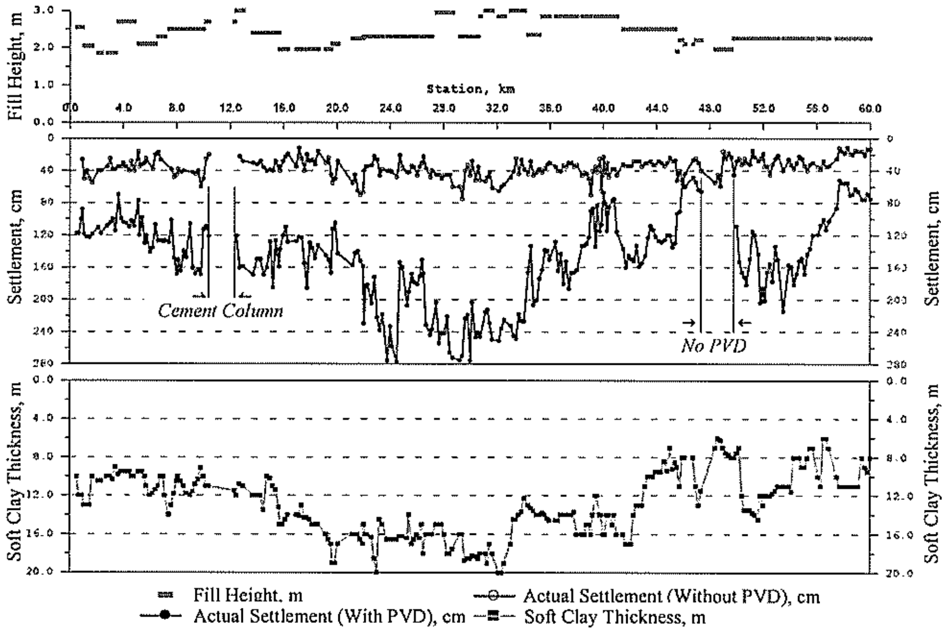


Fig. 25 Settlement profile (Up to Sept, '97) with fill height and soft clay thickness

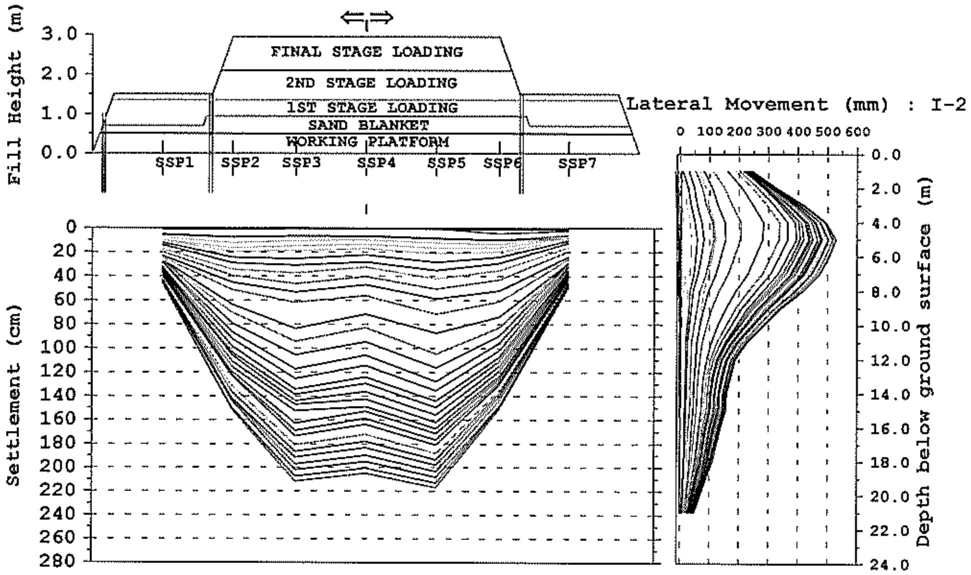


Fig. 26 Vertical and horizontal movements at Section 2A/2 (Sta. 28+200), BCHN

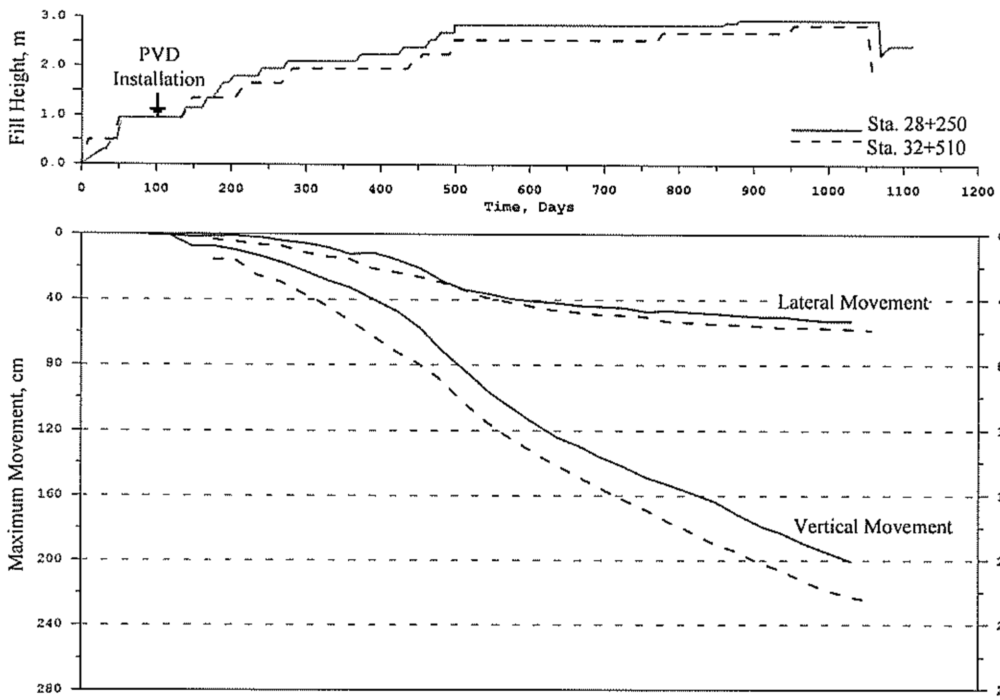


Fig. 27 Observed vertical and lateral movements vs. time with fill height at sta.28+250 and 32+510, BCNH

Table 4 Comparison of Field Measured Settlements with Estimated Values

Section	Station	Waiting Period After Final Stag		Design Settlement, cm	Measured Maximum Settlement, cm	Asaoka Estimated Total Settlement, cm
		Planned Month	Actual Month			
1-A/1	0+000~5+100	9~12	under construction	124~150	122.50	160.0
1-A/2	5+100~10+100		12~14		168.00	178.0
1-B	10+100~12+400		under construction	140~160	106.70	130.0
1-C/1	12+400~15+700		10~13	152~185	185.00	190.0
1-C/2	15+700~19+600		12~14		173.70	210.0
1-D	19+600~22+000		9~11	152~169	169.20	190.0
2-A/1	22+000~26+400		14~15	170~183	275.40	285.0
2-A/2	26+400~30+700		15~17		272.70	290.0
2-B/1	30+700~35+200		15~17	155~165	260.00	300.0
2-B/2	35+200~38+600		12~15		186.10	230.0
2-C	38+600~41+500		9~13	100~117	134.20	160.0
2-D	0+000~1+000		12~13	118~126	166.10	170.0
2-E	41+500~15+450 (main road)		9~13	95~135	160.20	165.0
	1+000~2+880 (access road)				182.30	195.0
2-F	45+450~47+675		10~11	96~106	106.70	110.0
3-A/1	47+675~52+000		12~13	114~214	181.50	210.0
3-A/2	52+000~57+600		10~11		201.50	215.0
3-A/3	57+600~64+500		9		76.30	80.0

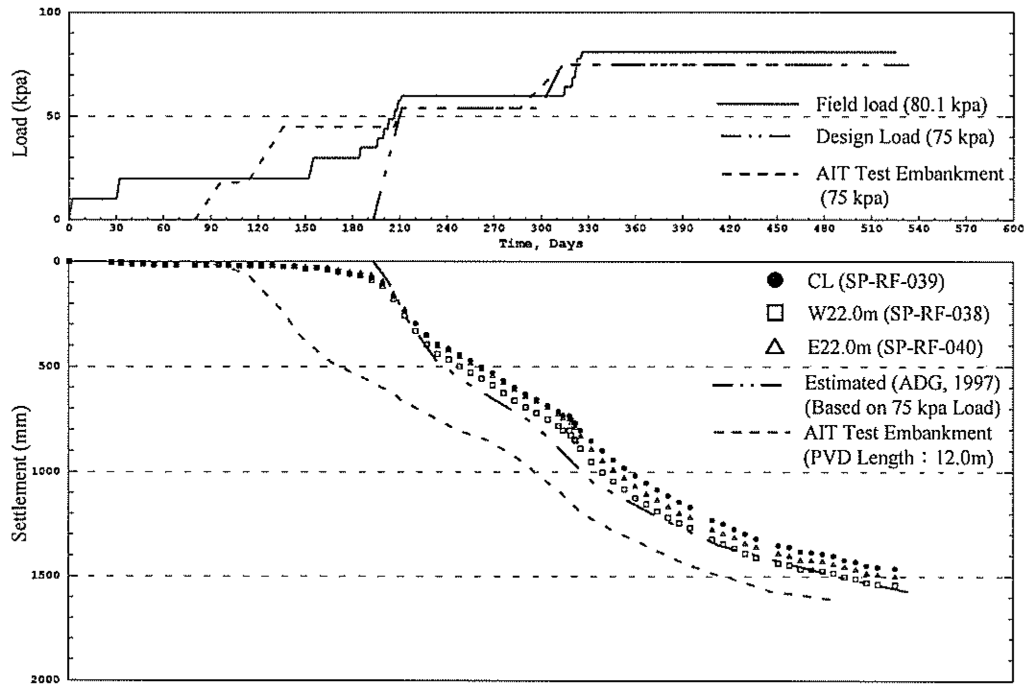


Fig. 28 Settlement comparison among the SBIA Reference Section, design prediction and AIT test embankment

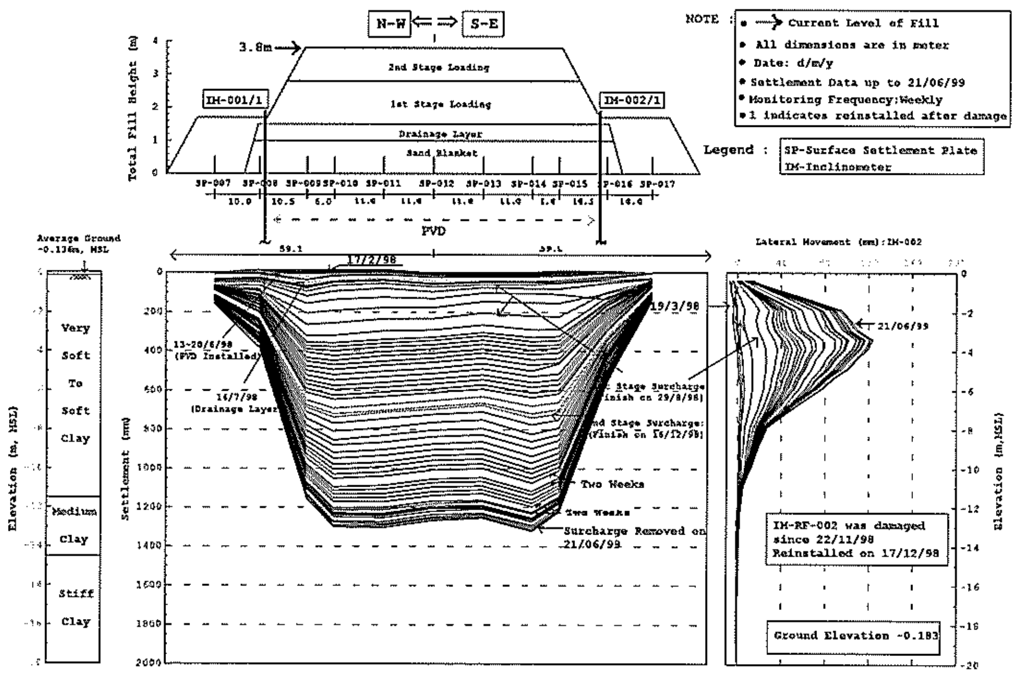


Fig. 29 Profile of settlement and lateral movement at SBIA Reference Section

Table 5 Summary of Observed Maximum Settlement and Maximum Lateral Movement in BCNH

Zone	Thickness of Soft Clay, m	Actual Preloading, m	Design Settlement cm	Observed max. Settlement cm	Observed max. Lateral movement cm
1	4.3~12.5	2.70~3.00	124~185	122~185	9.5~22.7
2	11.7~18.0	2.85~3.00	152~185	169~275	15.1~58.6
3	10.9~14.8	2.85	100~165	134~186	33.3~34.2
4	7.0~16.4	2.55~2.85	95~135	107~182	13.1~34.4
5	5.5~10.2	2.55~2.70	104~214	76~181	13.5~27.8

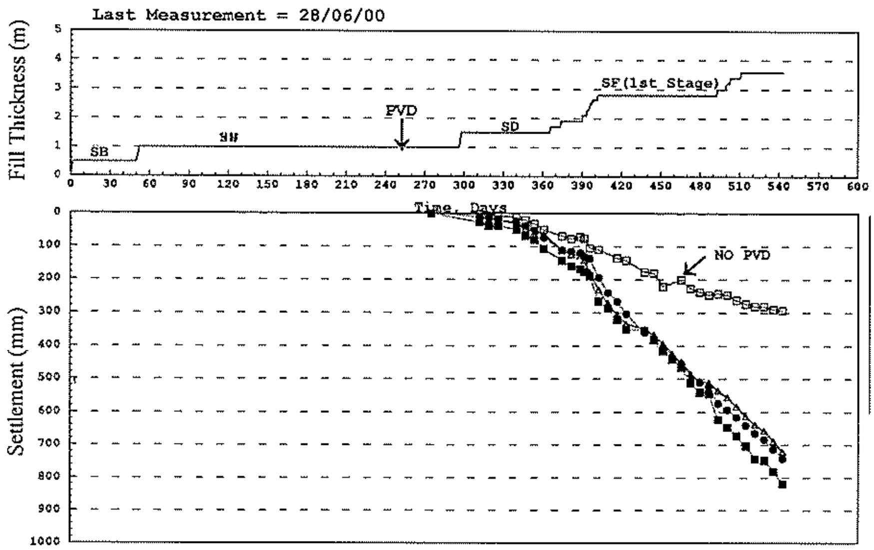


Fig. 30 The comparison of settlements in areas with PVD and without PVD

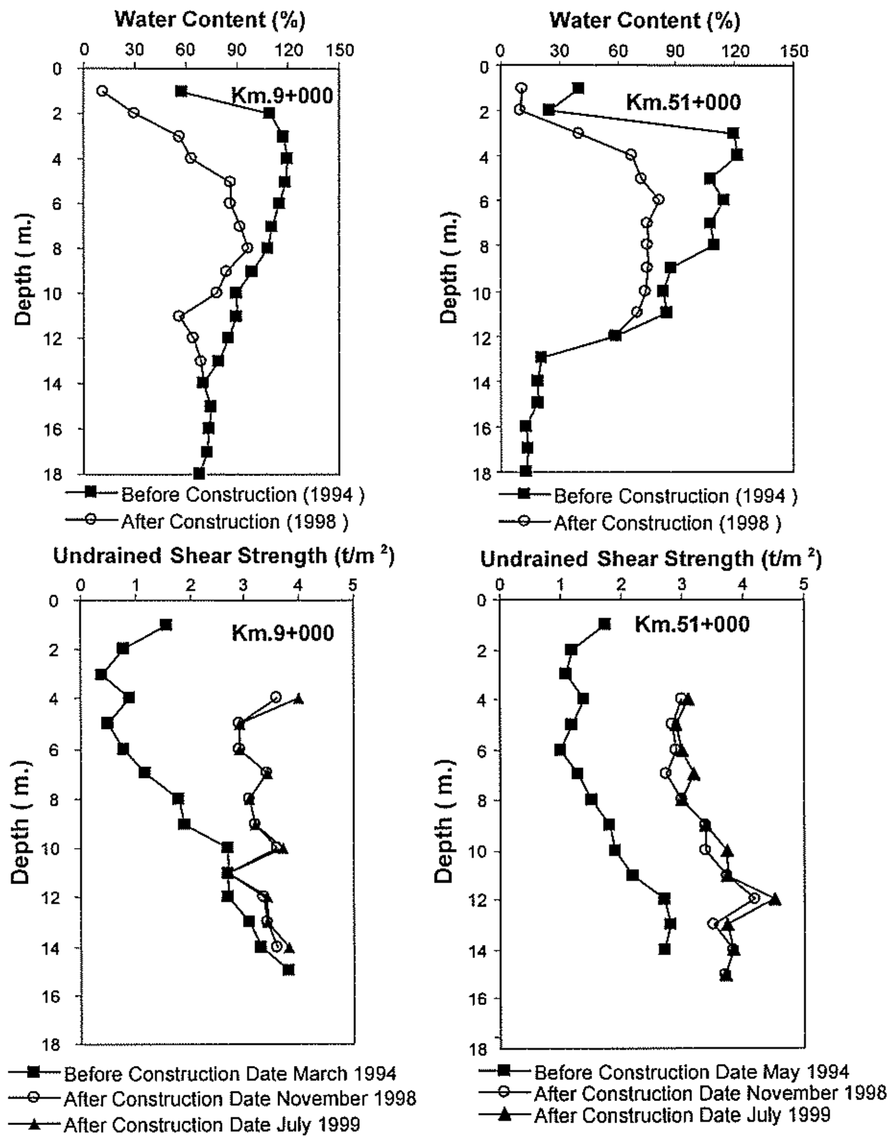


Fig. 31 Soil Properties before and after ground improvement in BCNH

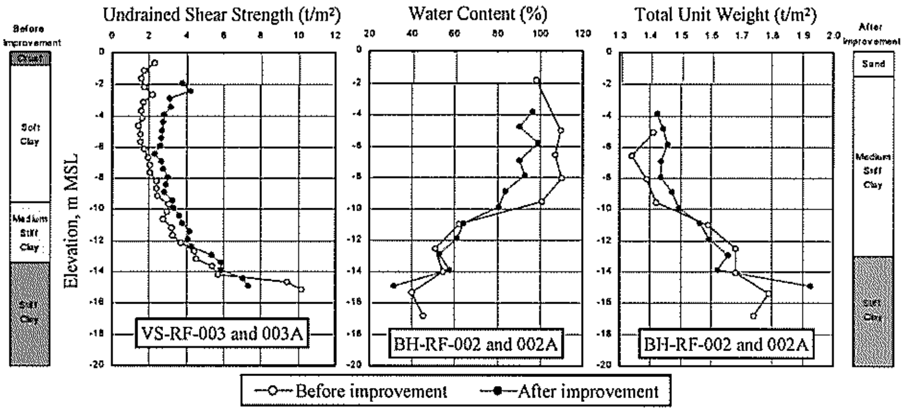


Fig. 32 Soil properties before and after ground improvement at SBIA

Problems Encountered and Countermeasures

(A) Inadequate Under-drainage

In the original design of the ground improvement work for the BCNH, a total of 45 cm thick sand drainage layer was placed above a 50cm thick sand-filled working platform over the original ground. There was no other under-drainage facilities. During the initial review of the design prior to undertaking the construction work, the supervising consultant has noticed this problem since the amount of settlement under the embankment load would be very large, the sand drainage layer together with the working platform would sink into the ground and shear off at junctions between the main embankment and the berm where no PVD were installed. This phenomena would impede proper flow of water discharged from the PVD to the sand drainage layer. High excess porewater pressure and slow rate of dissipation were indeed observed after the second stage loading. Additional drainage trenches traversing the side slope of the road embankment and pumping wells were installed to accelerate the groundwater dissipation, Fig. 33 (Moh, et al., 1998). This problem did not occur at the SBIA site because the designer had incorporated adequate underdrainage facilities including filter fabtric, sub-drainage pipes, collector pipes and manholes in addition to the sand drainage layer (Lin et al., 2000)

(B) Cracks in Embankment

Due to consideration of cost, PVD were installed only under the main embankments in both projects. Large differential settlements occurred between the embankment section with PVD and the berms where no PVD were installed. This led to development of cracks along the boundaries of the two sections and sometimes even along centerline of the main embankment. In order to reduce potential of stability failure, actions were taken during the preloading period including reduction of preloading height, extending counterweight berms and improving under-drainage for relieving the excess porewater pressure. In fact, all these measures either prolonged the waiting period or directly increased the construction cost. It is advisable to install PVDs under the berms as well but the depth of installation could gradually decrease from the main embankment towards the toe of the berm.

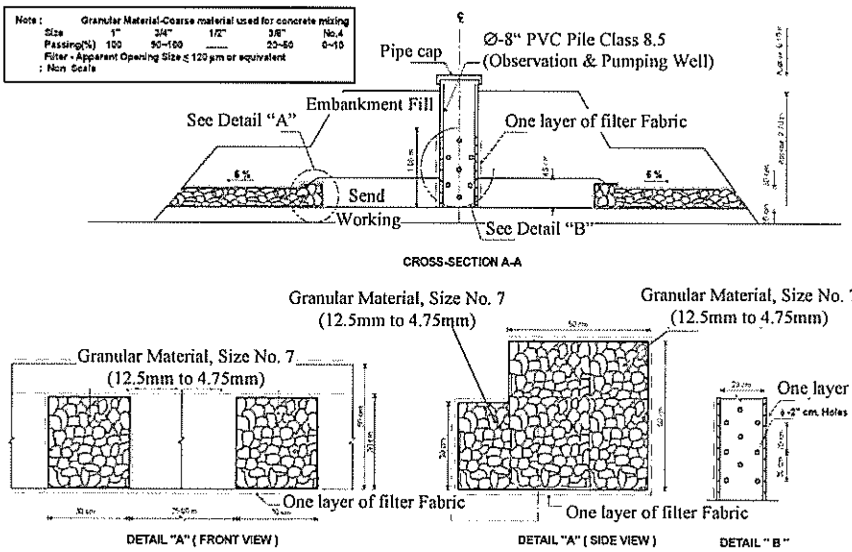


Fig. 33 Typical Cross – Section of additional trenches and pumping wells in BCNH

(C) Embankment Erosion during Raining Season

In Thailand, heavy rains often occurred during raining seasons. When sand or clay were used as the embankment fill, special protections of the side slopes of the embankments against erosion are needed. Top soils are commonly used for this purpose. However, precautions must be exercised in placement of the top soil on the side slope so that it would not block the drainage outlet for the dissipated water to flow out. Blocking of water outlets actually accounted for several cases of failure or unsuccessful of ground improvement with PVD.

(D) Problems with Right of Way

In the BCNH project many road embankments were bounded by fishponds and canals with limited right-of-way resulting in inadequate counter-weight berms. Local slope failures occurred at a number of places. To counter this problem, special attention was given to the rate of fill by observing the ratio of lateral movements to vertical settlement. At a few locations, other stabilizing measures such as cement/lime columns were installed at the toe to enhance the stability of the embankment.

CONCLUSIONS

Geotechnical engineering plays a critically important role in any infrastructure development, in terms of safety and economy. Due to the vast complexity of materials and techniques involved, geotechnical engineering up to the present time, is still not an exact science. It is a combination of engineering principles with observed experience and sound engineering judgment. The four case records reported herein clearly indicate several important factors in carrying out a geotechnical design in addition to the normal considerations in an engineering design. They include adequate and reliable subsurface information, appropriate selection of analysis principles and construction methodologies/details, ability to cope with variations in ground conditions, and timely interpretation of field performance data.

A good analogy is comparing geotechnical engineering to the medical practice, as shown in Table 6. It is appropriate to say that a geotechnical engineer is not just to compute numbers accurately but to make judgment soundly.

ACKNOWLEDGEMENTS

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Table 6 Comparison between Geotechnical Engineer and Medical Practitioner

Geotechnical Engineer	Medical Practitioner
Understand project	Understand illness
Reconnaissance and data collection	Medical history and background
Site investigation & testing	Physical examination and tests
Soil, rock mechanics, engineering geology	Physiology
Experience	Experience
Simplifications, analyses, models	Diagnosis
Recommendation and design	Prescription
Monitoring, comparing and modifying	Observation and further examination

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1999 Chi Chi Earthquake of Taiwan

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ABSTRACT: A devastating earthquake, namely, Chi Chi Earthquake (or Ji Ji earthquake in some literatures), with a magnitude of Ms = 7.6 (Richter Scale by USGS) or M_L =7.3 by Central Weather Bureau of Taiwan, struck central Taiwan at 01:47 on 21 September, 1999 (17:47, 20 September, UCT). Death toll reached 2,434 and there were 54 persons missing, 723 persons seriously injured and 11,306 minorly injured. In addition, there were 51,925 buildings collapsed and 54,402 buildings seriously damaged. Presented herein are structural and geotechnical issues of the earthquake, including damages to bridges and buildings, landslides and liquefaction problems, etc. Also discussed are the recovery and reconstructions subsequent to the earthquake and the improvement made to the mitigation programs to prepare for future earthquakes.

1 INTRODUCTION

The Chi Chi Earthquake (M_L = 7.3 by Central Weather Bureau and Ms =7.6 by USGS) was the strongest earthquake with epicenter on the Taiwan Island in the 20th century. It is unique for its rupture length of 105 km and upheave of, upto, 9.8m. Strong shaking lasted for more than 40 seconds with a peak horizontal acceleration of 989 gals (east-west direction of Station TCU084) and a peak vertical acceleration of 716 gals (Station CHY080). Because of amplification effects, peak accelerations generally exceeded 100 gals in the Taipei Basin, which is more than 100 km from the epicenter.

In addition to the loss of lives, damage to buildings, infrastructures, and lifelines, etc., was wide spread. The earthquake and the damage made were extensively studied by professionals and researchers from Taiwan, Japan and the US, and a large body of data were collected, making it the best documented earthquake in the history. A series of reconnaissance reports were published by the National Center for Research on Earthquake Engineering (NCREE) of Taiwan and Multidisciplinary Center for Earthquake Engineering Research (MCEER) of the USA. A commemorate symposium was held in September, 2000 with 140 papers presented in four volumes, one each on science, geotechnical, structural and human aspects of the earthquake. Presented herein is a brief summary of the information given in these reports in an attempt to give an overview of the event and its consequences. This paper is not research-oriented and was prepared as a compilation of available information which might be of interest to structural and geotechnical engineers in a country with minor seismic activities.

2 THE EARTHQUAKE

As depicted in Fig.1, Taiwan Island is located at a complex juncture between the Eurasian Plate and the Philippine Sea Plate. Therefore, seismicity is extremely active on the island and major earthquakes were quite common.

The Chi Chi earthquake occurred at 1:47 of 21 September, 1999 (20 September, 17:47 UTC).

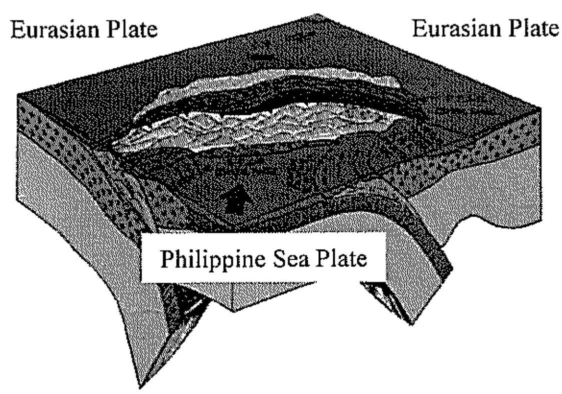

Figure 1. Plate boundary model of Taiwan

Figure 2 shows the locations of the epicenter and the contours of the levels of the ground shaking (Central Weather Bureau). The Taiwan Central Weather Bureau's Seismic Network (CWBSN) reported the earthquake information rapidly within minutes. The earthquake is believed to be associated with the Chelungpu Fault and the Shuangtung Faults, refer to Fig. 3 for geological section at the epicenter. These two faults are 10 km apart and subparallel to each other. They dip towards the east at high angles and are reverse faults with a significant left-lateral strike-slip component. The hypocenter at Chi Chi lies very close to the Shuangtung Fault and is located at a depth of 8 km, near the intersection of the two faults.

The number of aftershocks reached 10,252 as of 10 October, 1999 with six greater than magnitude 6.5. Even one year later, a few earthquakes of moderate magnitudes were still believed to be the aftershocks of the Chi Chi earthquake.

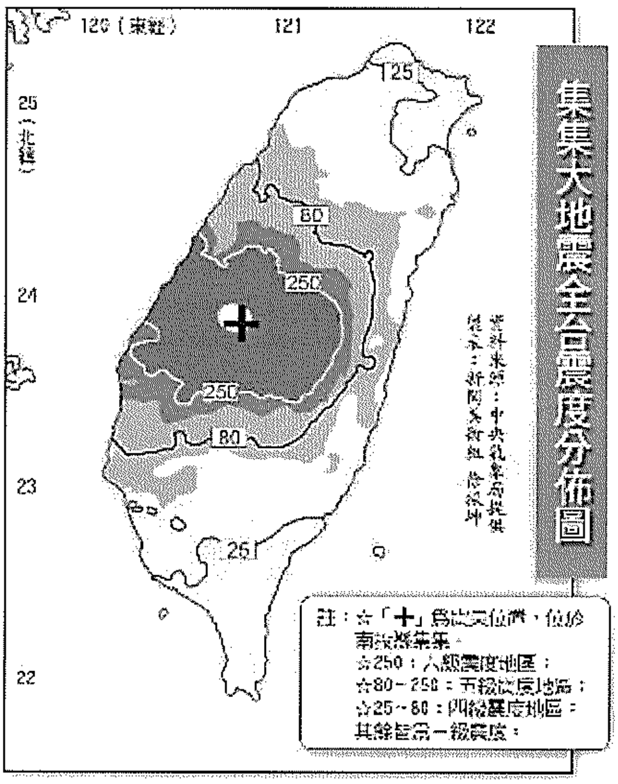

Figure 2. Epicenter and Contour of Levels of Shaking

As a direct result of this earthquake, 2,434 lives were lost, 12,029 people were injured, 51,925 buildings collapsed and 54,402 buildings suffered damages. This was Taiwan's worst disaster since the Hsinchu -Taichung Earthquake of April 1935, which measured M_L= 7.1 and took 3,325 lives (Central Weather Bureau). As of February 2000, the financial loss was estimated to be of an order of NT$362 billions (US$11.4 billions) by the Directorate General of Budget Accounting and Statistics of Executive Yuen.

3 RESPONSE TO EMERGENCY

Immediately following the earthquake, the government quickly established the Council for Disaster Response (CDR), convened by the Vice President Lien. Upon inspecting the disaster area, the Council recommended to the President to declare an emergency decree based on the Constitution. The central island was officially declared as disaster districts by the President in the evening of 25 September and the emergency decree was issued to empower the government to override some laws and statutes so resources could be quickly mobilized to meet urgent needs. The decree covered the city of Taichung and the five counties suffered the greatest damage and remained effective for six months from the date of issuance.

On 28 September, 1999 (one week after the disaster), the Executive Yuan announced the formal establishment of a restoration commission, namely, the 921 Earthquake Reconstruction Committee. Under the leadership of the Premier and the Vice Premier, thirteen senior government officers were appointed to head thirteen different task forces. Each task force was charged to handle problems within a particular subject area such as finance, education, environmental preservation, etc. Some task forces

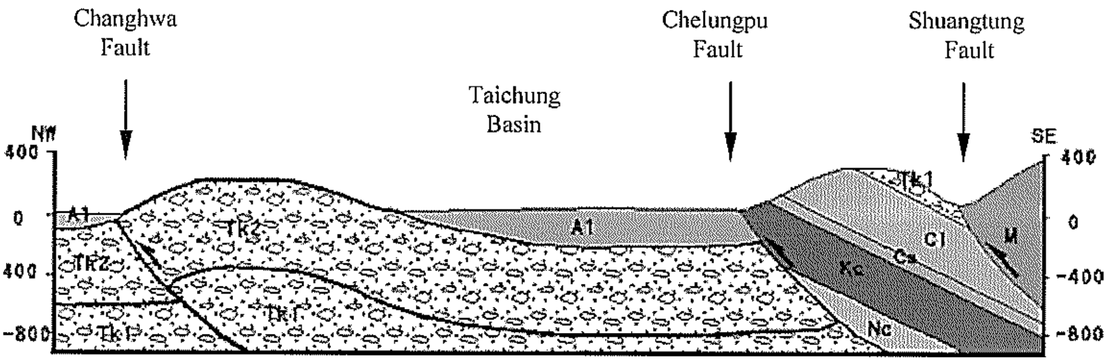

Figure 3. Geological Section at Epicenter

included, as their members, officials from the national and local government agencies working together to develop measures concerning restoration and reconstruction.

The public showed tremendous spirit as they worked together to save lives and help each other to go over the tragedy. Tens of medical teams were dispatched to the disaster areas by hospitals all over the island. Numerous charity groups and individuals offered their helping hands and massive supplies were rushed to the disaster districts by volunteers using their own vehicles. International assistance was particular touching. Japanese rescue team arrived at 5pm of the same day followed by 14 teams from other countries, totaling to 530 members and 38 dogs. Their efforts certainly won the applause from the local citizens.

The Ministry of Defense played a major role in the immediate emergency response and provided the major resources till local governments were able to regroup their forces. The emergency relief functions conducted by the military forces included shower stations, shelter centers, provision of security in the hard-hit areas, deployment of mobile medical teams, assemblage of temporary housing units, etc. Military forces were also deployed in demolition of severely damaged buildings and the emergency repair of bridges and roads to speed up reconstruction and recovery,

The civil engineering society did not overlook its responsibility to the community. NCREE was able to form several reconnaissance teams to collect first hand information and numerous researchers from local and international universities joined the activities voluntarily. Many of these teams arrived the scenes within hours after the earthquake. Several professional bodies were able to deploy practicing engineers from consulting firms to assist local governments to assess the conditions of buildings. Their judgments formed the basis for the demolition of dangerous structures and their observations become valuable sources of information for subsequent studies.

Within days after the earthquake, donations from all sources accumulated to NT$30 billions (US$ 1 billion). This fund was mainly used in humanity aids and has helped numerous homeless people. Part of the fund was used in construction of temporary housings and schools. This enables the disaster areas to recover in a rather short time.

4 CONSEQUENCES

Most of structures along the fault were non-engineered old structures constructed prior to the implementation of modern design codes. Even modern structures near the rupture were unable to resist the large ground movements and the intensity of shaking exceeded what was specified in the design code. This resulted in a pretty high damage rate of structures.

4.1 *Damage to Bridges*

There are approximately 1,000 bridges in the areas with strong shaking and 20% of them suffered damages to certain degrees. Twenty bridges were seriously damaged and required extensive repairs or had to be demolished and rebuilt. Many of them are located on Route 3 which is a major north-south highway running the length of Taiwan from Taipei in the north to Pingtung in the south. There are approximately 65 bridges on this route as it passes through Taichung and Nantou counties. Five of these bridges suffered collapsed spans (Photo 1) or cracked piers (Photo 2). Another five bridges on county and city highways experienced similar distress, including one cable-stayed bridge under construction. All these ten bridges are within 10km of the fault and most are within 5 km. Seven are located directly on the causative fault or on one of its branches. All are considered to be in the near

Photo 1. Bridge with Falling Spans

Photo 2. Bridge with Pier Damaged

field and thus subjected to intense ground motions both horizontally and vertically. The average fault dislocations were approximately 1.5m horizontally and 3m vertically.

4.2 *Damage to Buildings*

A large percentage of buildings that collapsed were non-engineered one-to-three story reinforced concrete frame structures constructed with brick infill partitions and exterior walls. Many of them had pedestrian corridors and open front at the ground floor (Photo 3). More than two dozen modern high-rise apartment buildings overturned or collapsed (Photo 4) because of inadequate design. Some of the buildings collapsed because of the so-called "short-column effects" as the spaces between columns were partly filled by partition walls (Photo 5). These partition walls trap the columns preventing the development of their normal flexural behavior over their height and allowing them to only deform over their free-height, i.e., the length of the column not surrounded by partitions. As a result, although the shorter length of the trapped column would make it possible to resist higher lateral forces before the flexural strength of the column is reached, the shear strength of a short column is often first reached and typical non-ductile shear failures ensue.

Photo 3. Building with Weak Ground Floor

Photo 4. Poorly Designed Modern Highrise Buildings

One of the significant features of the Chi-Chi earthquake was its large lateral and vertical displacements at surface. Buildings constructed across faults typically cannot survive the relative displacements that occur at their foundation due to fault movements. This type of failure was commonly seen along the fault (Photo 6).

Photo 5. Building Damaged Due to "Short-Column" Effects

Photo 6. Building Damaged by Ground Movements

4.3 *Damage to Schools*

Roughly, a total of 786 schools were damaged. Most of them were damaged because of the so called "short-column" effects described in Section 4.2 as the lower part of the space between columns were partially infilled by partition walls leaving the upper part open for windows (Photo 5). This type of damage in these reinforced concrete structures was rather common in the direction parallel to the exterior corridor outside the classrooms. The severity of damage to school buildings exceeded that of other structures due to primarily to the similarity of their design and construction. The eccentricity of most school buildings associated with cantilevered corridors at upper floors made the situation worse.

4.4 *Damage to Hydraulic Structures*

Damages to hydraulic structures were rather light. Of special interest is the Shikung Dam which is a concrete dam and was seriously damaged by ground movements (Photo 7). The fault directly ran through the foundation of the dam. The eastern portion of the dam was jacked up by 9.8m and the western portion of the dam was jacked up by only 2.1m, resulting in a differential movement of 7.7m.

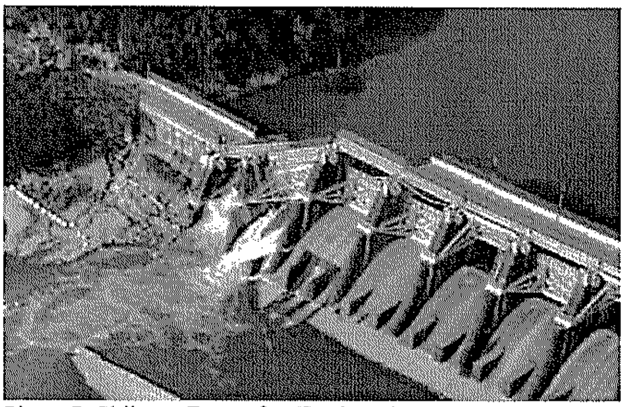
Photo 7. Shikung Dam after Earthquake

4.5 *Damage to Critical Facilities*

Damage to critical facilities and lifelines, including hospitals, police and fire stations, power houses and transmission, etc., was widespread in central island. Nonstructural damage was found to be a major factor adversely affecting the functionality of major hospitals. The ones which were seriously damaged included Chritian Hospital at Puli, Veterans Hospital at Puli, and Shiu-Tuan Hospital at Tsushan. It was fortunate that fire broke out at only a few places and the consequence was relatively minor in comparison with those observed in some of the previous earthquakes in which fire was the major cause of casualty and financial losses.

4.6 *Damage to Power System*

The earthquake severely impacted the power system. With more population and less electricity generated, the northern half of the island relies on the southern half of the island for power supply. A significant number of transmission towers were seriously damaged causing failure in transmission of electricity from the south toward the north. Worst of all, Chung Liaw switchyards which is right at the junction of two trunk transmission lines was seriously damaged and, as a result, the northern island suffered a power outage for more than a week. The rationing of electric power to industries was finally lifted on 5 October and the rationing to residential users was lifted on 10 October.

The power failure severely impacted the industrial output as many factories were shut down because of outage of power. For example, Hsinchu Industrial Park which situates about 110 km from the epicenter and houses approximately 239 high-technology firms was seriously affected. In order to minimize the impact to economy, priority was given and power to this park was restored to full capacity on 25 September, four days after the earthquake. Even so, loss was estimated to be around US$400 million, most of which incurred at the semiconductor and silicon wafer production facilities.

5 GEOTECHNICAL ISSUES

Geotechnical hazards as a result of the earthquake included landslides, soil liquefaction, and foundation failures and were rather widespread.

5.1 *Liquefaction*

Soil liquefaction was the most dramatic in Yuan Lin Town in Changhwa County with an area of near 60 square kilometers seriously affected. More than two hundreds of dwellings were either destroyed or were damaged beyond repair. Most of the dwellings destroyed were poor single-story houses made of bricks or adobes. Modern buildings performed rather well and suffered only tilting with little structural damage (Photo 8). Sand boils were found only at locations at which the clay cover is either too thin or totally missing. At locations where the clay cover is thick, no surface features could be related to liquefaction except that ground subsidence led to settlements, up to a meter or so, of a few buildings.

At a nearby ground motion monitoring station (Station TCU110), a peak acceleration of 187 gals was registered. An extensive investigation program was carried out to unveil the ground conditions for the purpose of studying the mechanism of liquefaction (Ueng, Lin and Chen, 2001; Lin, Ueng, Chen and Huang, 2000). It included borings and penetration tests at 95 locations. Based on the studies, it was found that Seed's evaluation method as well as the associated liquefaction evaluation curve fitted the field data quite well.

In Nantou, a section of levee along the Maolo River was seriously damaged as a result of liquefaction. The monitoring station (TCU076) at a distance of 15km away recorded a peak acceleration of 420 gals and strong motion lasted for 41 seconds (Lin, Lai, Lin and Hseih, 2000). Liquefaction was found to be limited within depths of 4m to 8m. There was no indication of

Photo 8. Foundation Failure Due to Liquefaction

liquefaction in the underlying gravel layer. Post-liquefaction settlement ranged from 5cm to 26cm. According to the Seed Simplified Procedure, the critical peak acceleration that would lead to liquefaction would be about 0.29g.

In Wufeng which is within 1 km from the Chelungpu fault., liquefaction resulted in lateral spreading at four locations along the Koniaokeng Creek (Chu, Hsu, Lay and Chang, 2000) and damaged a few low-rise structures (Photo 9). A nearby monitoring station (TCU065) measured peak acceleration of 774 gals in the east-west direction, 563 gal in the north-south direction and 257 gals in the vertical direction.

Liquefaction at Taichung Port, which is located about 55 km northwest of the epicenter, damaged 4 of its 45 docking wharves. The port was built on a

Photo 9. Building Damaged by Lateral Spreading

reclaimed land in four stages. The hydraulic fill behind Wharves 1 to 4 are retained by caissons which sit on a thin layer of cobbles and boulders. During the earthquake, the loosely dumped sands behind the caissons were liquefied and sand boils occurred all over the places (Chen and Hwang, 2000). Sands erupted from the ground can be found as far as 150 meters from the waterfront. Due to liquefaction, the caissons moved seaward by 1 meter on an average and the backfill behind these caissons settled by about 70cm. Due to the outward movements of the caissons, gaps were created at the interlocks between caissons and permitted the materials behind the caissons to be washed away by tides resulting in cavities of, up to, 30m in diameter and 4m in depth (Photo 10).

The nearest seismology station located at a distance of 4.7 km southeast of the Port registered a peak acceleration of 165 gals in the east-west direction and 152 gals in the north-south direction. The vertical component of the ground motion was small. Analysis indicates that this magnitude of ground motion would not be able to trigger the sliding of the caissons and the movements of the caissons must be the result of liquefaction of the backfill (Chen and Hwang, 2000). It is worthy of mentioning that the rest of wharves were retained by sheet piles and suffered no damage.

Photo 10. Damage to Taichung Port Due to Liquefaction

5.2 *Landslides*

There were thousands of landslides in the mountainous terrain within and adjacent to the epicentral area. A total of 436 scores of slope failure were investigated and documented in the reconnaissance report coordinated by NCREE. The Bureau of Water and Soil Conservation reported more than 2,300 items of variation based on the SPOT satellite photos taken before and after the earthquake (Ueng, Lin and Chen, 2001).

Nearly all of the slope failures are located to the right of Chelungpu fault, i.e., on the hanging wall, and most slides were relatively shallow slips in residual soils, typically involving depths of 1 to 5m. A massive slope failure involving 120 million cubic meter of debris occurred at Tsao-Ling. Twenty percents (about 25-million cubic meters) of the sliding mass dropped into the valley of the Ching-Shui River and blocked the flow causing flooding of the upstream valleys (Lin, Wang and Chen, 2000; Hung, 2000). Most of the sliding mass (of about 100-million cubic meters), and 39 people who lived behind the crest of the dip slope, flew over the Ching-Shui River, and landed on the other side of the river. Thirty two people were killed and 7 survived after the sliding-flying-landing process. Air-blast or release of compressed air cushion under the sliding mass is believed to be responsible for this abnormal phenomenon (Hung, 2000).

The debris blocked the river and formed a reservoir behind. As the water in the reservoir rises, there is the potential of overtopping the landslide dam and causing catastrophic flooding downstream. To ease the crisis, the plugged section of the Ching-Shui River channel was modified to allow for smooth and safe passage of the overflow. Check dams were also constructed in the downstream sections of Ching-Shui River for safeguarding the people and the lands from being smashed by possible debris flow due to sudden break of the landslide dam.

6 RECOVERY

Soon after the situation was under control, the government turned its attention to the reconstruction in the disaster areas. The recovery proceeded at a rather quick pace. Electric power was back within 2 weeks on the entire island and within 3 months all the damaged bridges were either repaired or replaced by new ones of which some are temporary structures.

In areas which experienced liquefaction, underpinning and grouting were carried out to remedy tilted buildings. Low-rise buildings, up to 4 stories, were successfully made right by underpinning and mid-rise buildings, up to 7 stories, were successfully made right by grouting.

The Council for Economic Planning and Development (CEPD) prepared a five-year reconstruction program, namely, the Post-Earthquake Reconstruction Plan (PERP), which received the Executive Yuan's approval on 9 November, 1999. A governmental agency, namely, "921 Earthquake Post-Disaster Recovery Commission", was formed on 1 June, 2000 to undertake the responsibility of reconstruction and a budget of 200 billion NT dollars (7 billion US dollars) was allocated for the three years to come. The use of the fund is supervised by central government and by civilian organizations as well to ensure that the fund is used to the maximum benefits of people.

7 MITIGATION PROGRAM

The Chi Chi earthquake certainly increases the awareness of people of potential threat of earthquake and provides an opportunity for improving the preparedness for earthquakes in future. The government has allocated a generous budget to support researches related to earthquakes and earthquake engineering. The following are a few programs which have already been implemented or being implemented:

7.1 *Instrumentation and Monitoring of Ground Motions*

Under the Taiwan Strong Motion Instrumentation Program, there are more than 650 strong motion observation stations distributed and maintained by the Central Weather Bureau, Ministry of Transportation and Communications. About 70 percent of these observation stations were triggered by this earthquake. With 73 real-time monitoring stations spread over the island and directly connected to the Central Weather Bureau, the epicenter and the magnitude of the Chi Chi earthquake were determined in, as short as, 102 seconds, which is unprecedented in history. The Bureau is making further efforts to shorten this time to only 20 seconds (CWB, 2000). Any time shortened is significant in reducing damages and saving lives because warning can be issued in time for emergency prevention mechanism to be activated. For example, trains of rapid transit systems and high speed rails can substantially reduce their speeds if it is predicted that the shaking will exceed the safe level.

7.2 *Enhancement of Geological Information System*

Web geographic information system (WGIS) was extensively used in the investigations following the earthquake. When performing the reconnaissance, a standardized inventory format together with the input system was established by NCREE so that the information collected from field could be input to the database directly. The database was connected to the geographic information system for data management and further analysis, and development of thematic maps could be done accordingly. This enabled information from various sources to be compiled in the shortest time possible. Furthermore, because of its ability to display maps and graphics, GIS also enabled users of all ranks to comprehend information much more easily. The GIS systems were combined with satellite images and aerial photos to identify areas with large damage. The results were useful in the rescue operations and reconstruction.

Realizing the importance and the potential applications of GIS, efforts are being made by the government to centralize the information managed by various agencies so updated information can be readily available to all levels of government and various organizations. An ambitious program is to be implemented to prepare a GIS version of geological hazard maps for the entire island so potential risks can be assessed.

7.3 *Revision of Seismic Design Code*

Another major development resulted from the earthquake is the revision of criteria for seismic design of structures, particularly in areas in which ground shaking certainly exceeded what was specified. The entire island is now divided into three seismic zones with design peak acceleration upgraded.

7.4 *Emergency Response Programs*

Many lessons have been learned from this earthquake and the island is doubtlessly better prepared for the future. The government is now pushing for the expansion and enhancement of the hazard prevention system, which is not limited to earthquakes but also includes other types of natural hazards such as fire, flood, and debris flow. To enable studies to be carried out in advance and actions to be taken promptly in case of emergencies, databases are established to compile relevant information and to assess risks involved in various types of disasters. The most comprehensive program is the so called Haz-Taiwan which is an application software for simulating the scenarios of earthquakes. It was derived from HAZUS97 (Risk Management Solutions, 1997) which was developed and tested in the United States for the Federal Emergency Management Agency (FEMA) by the National Institute of Building Science (NIBS). The basic information to the program includes ground motions, soil map, liquefaction susceptibility map, landslide susceptibility may, water depth, etc. To assess the loss to incur, information such as distribution of population and buildings, locations of critical facilities including police stations, schools, hospitals and clinics, lifelines, is required. Attempts are being made to incorporate the information collected in the Chi Chi Earthquake to refine the analytical model adopted in the program.

8 ACKNOWLEDGEMENTS

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From cobra swamp to international airport: ground improvement at Suvarnabhumi International Airport, Thailand

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The construction of Suvarnabhumi International Airport has been planned since 1960 to accommodate the rapid growth of air traffic in this region. The 41-year project was finally approved for construction by the Thai government in 1991, and the airport is scheduled to open in December 2004 with capacity to deal with 30 million passengers and 1·46 million tonnes of cargo per year. Owing to the high compressibility and low strength of the underlying soft marine clay, ground improvement is necessary to reduce post-construction settlement prior to construction of the permanent airport facilities. Two ground improvement projects, for the airside pavements and the landside road system, are currently being implemented at the new airport site. This paper discusses the design concept, construction method, and up-to-date ground improvement performance of both projects.

La construction de l'aéroport international de Suvarnabhumi a été projetée en 1960 afin de faire face à une croissance rapide du trafic aérien dans cette région. Ce projet vieux de 41 ans a finalement été ratifié par le gouvernement thaïlandais en 1991 et l'aéroport devrait ouvrir en décembre 2004; il fera transiter 30 millions de passagers et 1,46 tonnes de fret par an. En raison de la forte compressibilité et basse résistance de l'argile marine tendre sous-jacente, il est nécessaire d'améliorer les sols afin de réduire les tassements post-construction, avant de bâtir les installations aéroportuaires permanentes. Deux projets d'amélioration du sol, pour les trottoirs côté air et le système routier côté terre, sont actuellement mis en uvre sur le site du nouvel aéroport. Cet exposé examine les principes de conception, la méthode de construction et la performance d'amélioration du sol à ce jour pour les deux projets.

Introduction

The construction of Suvarnabhumi International Airport (or the Second Bangkok International Airport) has been planned since 1960 to accommodate the rapid growth of air traffic in this region. Suvarnabhumi International Airport (SIA) will not simply provide additional airport capacity to supplement the existing Bangkok International Airport at Don Muang, but will also develop Bangkok into an international aviation hub in South East Asia. Since the initial planning in 1961 the new airport project has passed 16 prime ministers and 30 cabinets, and was finally approved for construction by the Thai government in May 1991. The first phase of SIA is scheduled to open in December 2004, with capacity to deal with 30 million passengers and 1·46 million tonnes of cargo per year. In the future, the new airport will be able to serve 100 million passengers and 6·40 million tonnes of cargo annually. The New Bangkok International Airport Company Limited (NBIA), a state enterprise under the Ministry of Transportation and Communications, was formed in February 1996 to implement the construction of the SIA. The total construction cost is estimated to be more than 120 billion Bt (Thai baht), of which over 60% will be engineering costs. In December 2001 the construction of the passenger terminal building, a major milestone, was finally launched.

The SIA is located at Nong Ngu Hao (which means 'cobra swamp' in Thai), about 30 km to the east of Bangkok metropolis, as shown in Fig. 1. The SIA site is 8 km long and 4 km wide with a total area of approximately 3200 ha. The new airport site is situated on swampy land in a flat marine deltaic deposit, and most of the area was covered either by ponds of shrimp farms or by agricultural usages, with several crossing canals. Owing to the underlying high compressibility and low strength of the soft marine clay, ground improvement is necessary before construction of the permanent airport facilities, in order to reduce the maintenance costs.

Previous engineering studies at airport site

Previous major engineering studies with soil investigation at the SIA site can be divided into five phases, as follows.

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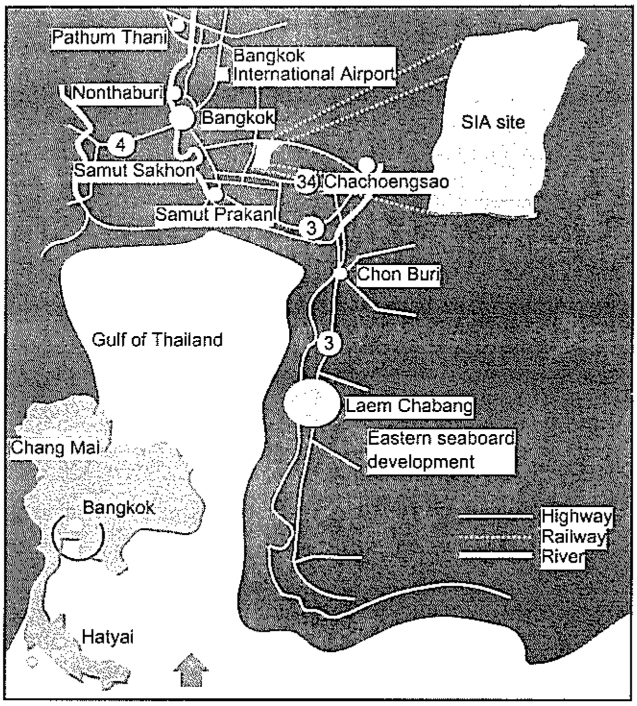

Fig. 1. Location map of Suvarnabhumi International Airport (SIA)

Phase I: Performance study of test sections by Northrop/Asian Institute of Technology (AIT), 1972–1974 (STS/NGI, 1992)

A total of 28 soil borings and 64 vane shear borings were taken on site during this study. The test embankments included two test fills with one embankment constructed to failure (up to 3·4 m or 61 kPa). No ground treatment was applied under the embankment.

Phase II: Master plan study, design and construction phasing by Netherlands Airport Consultants (NACO)/Moh and Associates Inc. (MAA), 1983–1984 (Moh and Woo, 1987)

A total of 11 boreholes, 40 electric cone penetration tests and 40 pore pressure probe tests were carried out during the phase II study. The field-testing programme also included three test embankments, one with embankment surcharge fill, one using vacuum loading, and one using pumping techniques for groundwater lowering. Non-displacement sand drains (0·26 m in diameter) were installed 14·5 m deep at 2 m spacings in a triangular pattern. Hydraulic connection induced by sand drains to the underneath sand layer was first observed during this study.

Phase III: Independent soil engineering study by STS Engineering Consultant Co. Ltd. (STS)/ Norwegian Geotechnical Institute (NGI), 1992 (STS/NGI, 1992)

A total of 51 boreholes, 100 vane shear borings and over 80 open tube (standpipe) piezometers were carried out in phase III. Several ground improvement alternatives were studied, including preloading with vertical drains, deep soil improvement, piling support with a free-spanning plate, relief piles with caps and soil reinforcement, and lightweight fills. Preloading with vertical drains was finally recommended, based on a comparison of cost, schedule and technical limitations.

Phase IV: Full-scale PVD test embankments by AIT, 1993–1995 (Asian Institute of Technology, 1995)

Three boreholes and six vane shear borings were carried out on site before construction of the test embankment. Three 4·2 m high (75 kPa) test embankments with prefabricated vertical drains (PVDs) spacings of 1·0 m, 1·2 m and 1·5 m in a square pattern to 12 m deep were constructed to evaluate the technique of ground improvement using PVDs. It has been concluded that PVDs provide a suitable technique for accelerating consolidation of the SIA soft clay under a careful design.

The above engineering studies were carried out mainly for feasibility study purposes. Accompanying the ground improvement design contracts with the NBIA, two independent soil investigations were also carried out to confirm the soil data obtained from previous studies.

Airside pavement design (ground improvement) by ADG, 1995–1996 (Airside Design Group, 1995)

Owing to hostility from local villagers and flooding at the site, only 50% of the planned soil investigation programme, including ten shallow boreholes (20 m), five deep boreholes (40 m), 11 piezocone tests and 11 vane shear tests, was carried out during the ground improvement design of the airside pavement.

Landside road system design (ground improvement) by MAA, 1996 (Moh & Associates, 1996)

A total of 31 boreholes, six vane shear tests and 37 cone penetration tests were performed at site by the designer. Some deep boreholes up to 27 m were made, mainly for pile design purposes.

As summarised above, a total of 144 boreholes, 236 vane shear tests and 97 cone penetration tests have been carried out at the SIA site since the 1970s, at the locations shown in Fig. 2.

Project outline

Two ground improvement projects, Ground Improvement for Airside Pavements (GIAP) and Ground Improvement for Landside Road System (GILRS), are currently being implemented at the SIA site. Preloadings with the installation of prefabricated vertical drains were adopted by the NBIA for both projects to accelerate the consolidation process and reduce post-construction settlement. The design, construction and financial source of the two projects are different. MAA is responsible for the design of GILRS and the construction supervision of both projects. Major project data of GIAP and GILRS are summarised in Table 1.

The GIAP includes west and east (partial) runways, apron, taxiways and two emergency access roads with a total area of 308 ha. The construction of a reference section (a trial section located at the west taxiway) was required in the initial stages in order to confirm the design assumptions and the contractor's working method (Lin *et al.*, 2000). Total construction progress in financial terms for GIAP was 98·69% at the end of 2001, and the remaining areas—including the south end of the west runway, the east taxiway and the emergency roads—are all in their final waiting period.

The GILRS covers 23 internal access roads and the remote parking lot, with a total improved area of 132 ha. There are three types of preloading embankment in the GILRS, with heights of 2·2 m, 3·5 m and 4·5 m. The total lengths of the

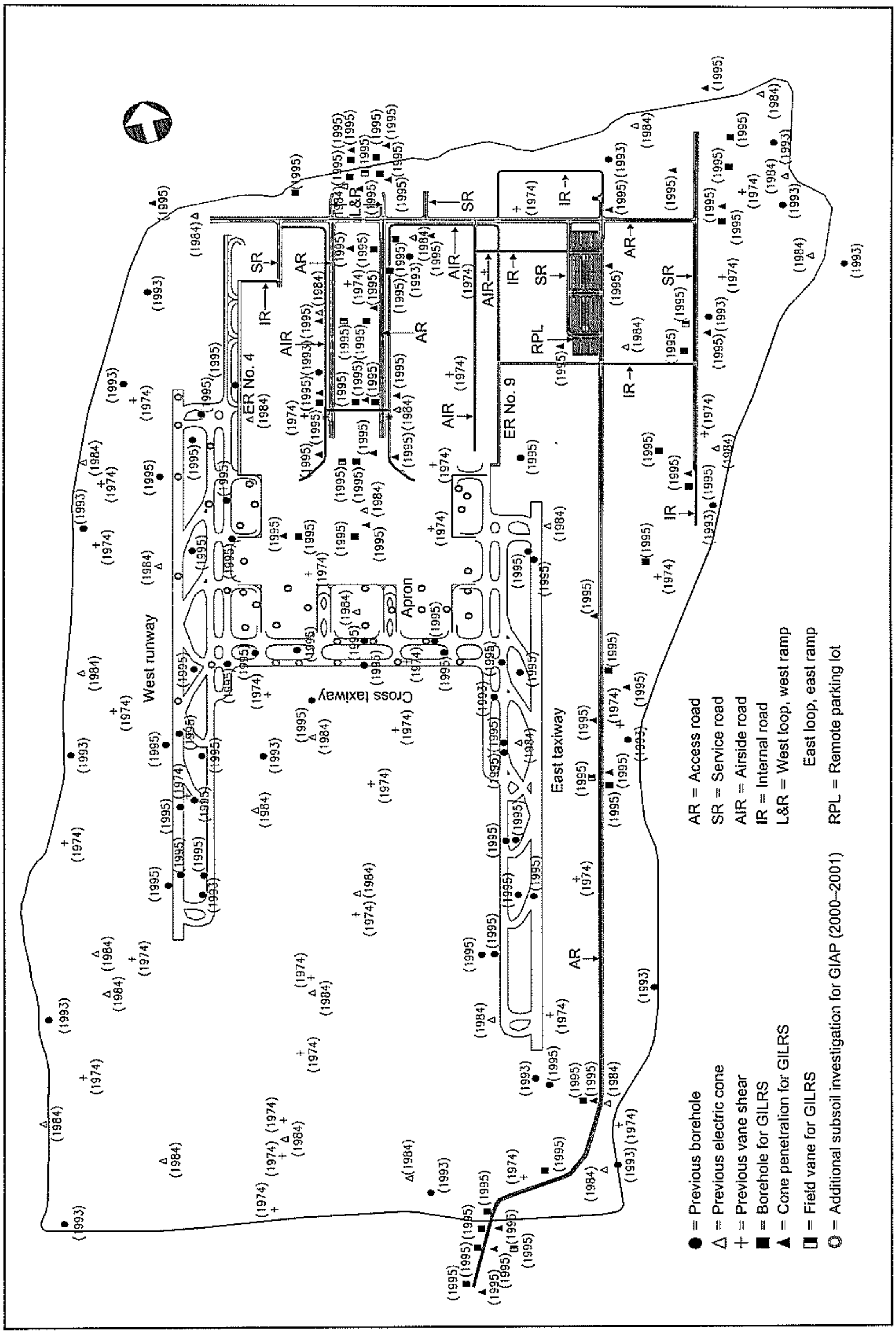

Fig. 2. Locations of soil investigation at SIA site

Table 1. Project data

	GIAP	GILRS
Total construction cost: Thai baht	8 419 205 000	1 767 488 000
Financial source	Government budget	OECF loan (Japan)
Construction period	01/11/97–30/04/02 (54 months)	01/12/00–19/04/03 (29 months)
Design	Airside Design Group	Moh & Associates
Construction supervision	TEC/MAA/SIGEC/UIC/MTL	TEC/MAA/NK
Ground improvement area: ha	308	132
PVDs: m	33 580 000	10 889 600
Sand blanket: m^3	4 550 000	899 600
Preloading material: m^3	2 890 000 (crushed rock)	1 722 800 (sand)

type I, II and III embankments are 13 203·28 m (44·1%), 15 708·4 m (52·4%) and 1040 m (3·5%), respectively. Except for the remote parking area, with type II embankment only, type I and II preloading embankments appear in every 200 m alternately according to the subgrade profile in the GILRS. The varied road profile along the landside road system is intended mainly to provide a horizontal drainage function according to the design. Although construction progress (in financial terms) is currently over 90% in the GILRS, only less than 10% of the preloading embankments have satisfied the surcharge-removing criteria. The remaining areas are either under a waiting period or waiting for the surcharge fill to be removed from the completed section under the cycle loading plan.

Subsoil conditions

The subsoil conditions at the SIA site are relatively uniform, consisting of weathered crust, very soft to soft clay (Bangkok clay), medium stiff clay and stiff clay within the depth of 20 m. Underlying the stiff clay, the first dense sand layer is found after 25 m depth. The changes of physical properties with depth are associated with the increasing silt or fine sand content and decreasing clay fractions. The major concern for the airport construction is the 8–10 m thick of very soft Bangkok clay, which usually has over 100% natural water content with very low bearing strength. The general soil properties, including total unit weight, natural water content, Atterberg limits, specific gravity, grain size distribu-

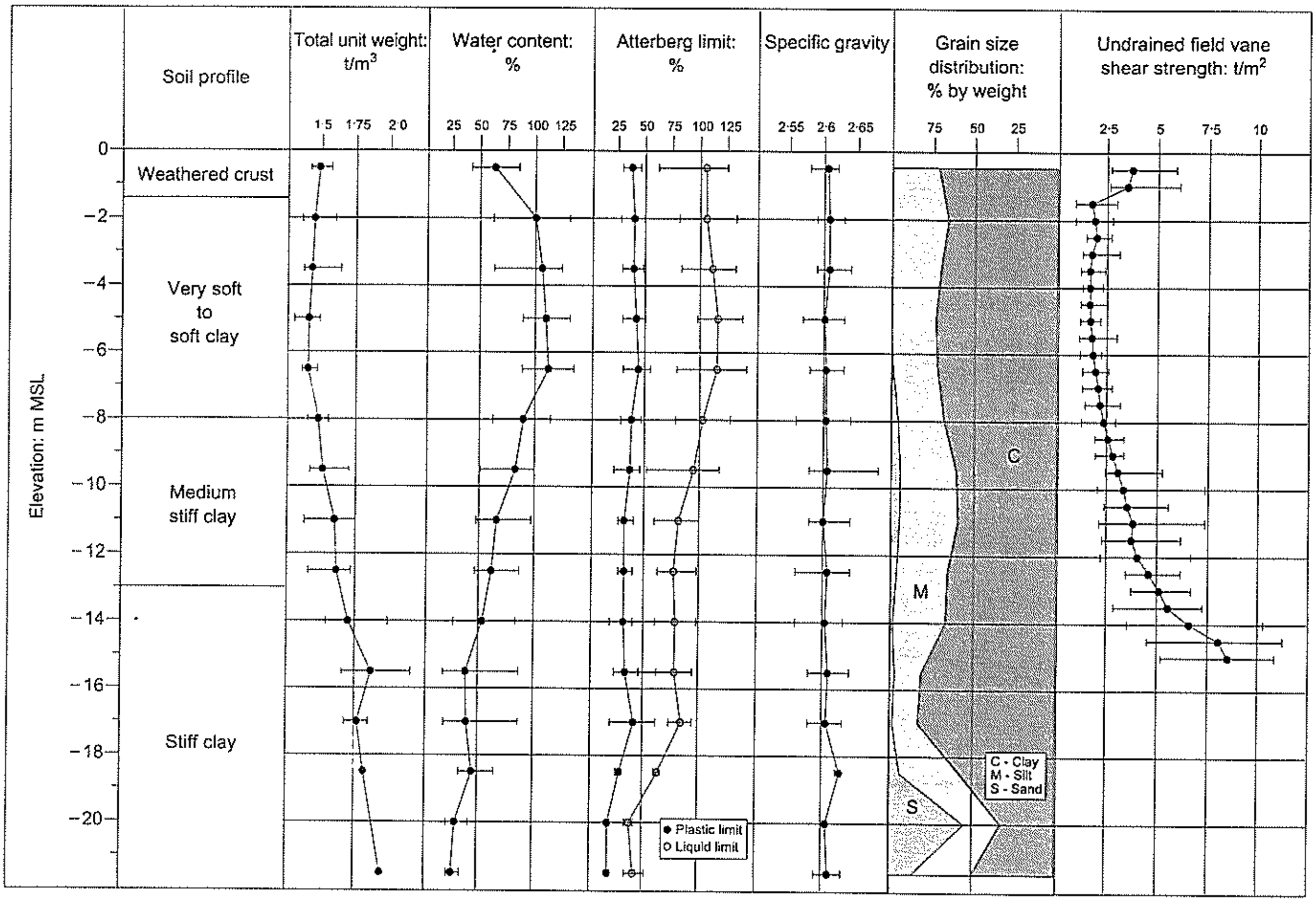

Fig. 3. General soil properties at SIA site

tion, undrained field vane shear strength and consolidation parameters, are summarised in Fig. 3.

Subsidence and groundwater conditions

Deep well pumping has been a common practice for the shrimp farms and agricultural land in the greater Bangkok area for many years. Serious ground subsidence due to the exploitation of groundwater has been observed for about 30 years, and has caused flooding in Bangkok city during the annual rainy season. Most of the subsidence was expected to take place in layers deeper than 30 m. Earlier studies indicated that the subsidence rate at the Nong Ngu Hao area was estimated to be about 30–50 mm per year. To reduce the subsidence rate, in 1983 the government took remedial measures to control groundwater pumping. Based on the monitoring stations around the SIA site, as shown in Fig. 4, a total of 600 mm of subsidence has occurred at station 29 during the past 20 years. The average ground elevation at the SIA site changed from about Elev. +0·5 during the earlier study period to mean sea level (MSL) used in GIAP and GILRS. Unfortunately, there are no data from 1996 to 2000 at station 20, and the survey data in 2001 showed contrasting results at two stations. Further study to establish reliable data on subsidence surrounding the SIA site is essential.

The phenomenon of under-hydrostatic water pressure within the depth of 10–20 m (soft to stiff clay) was first observed in 1973 and further confirmed during the phase II study in 1984. This was most probably due to a decrease of piezometric head in the sand layer caused by deep well pumping. Fig. 5 summarises the recorded dummy readings of water pressure since 1973. The water pressure data below 20 m were obtained from open-tube piezometers. Based on the dummy piezometer data in GIAP, underpressure became

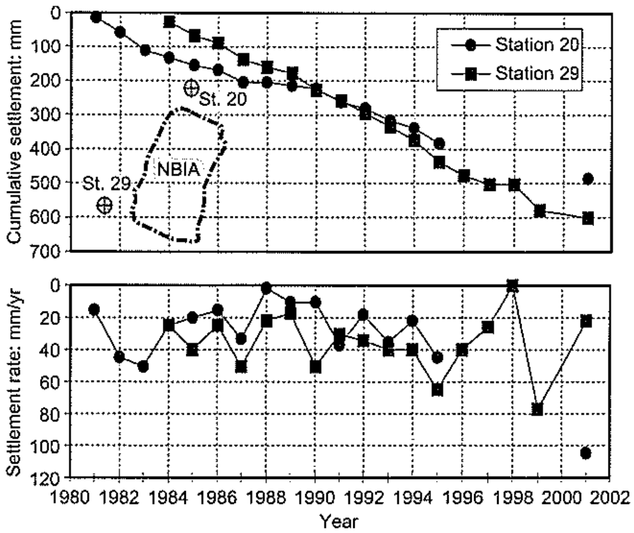

Fig. 4. Ground subsidence at SIA site. Source: Royal Thai Survey Department

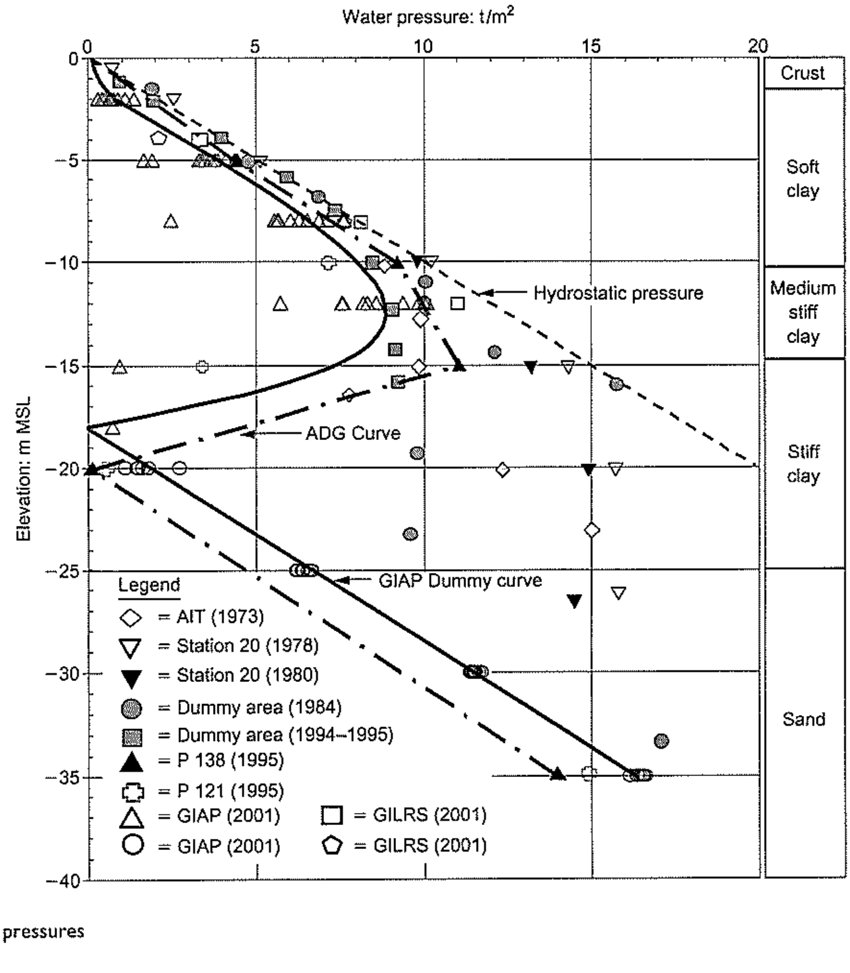

Fig. 5. Dummy pore water pressures

more significant owing to the increase of deep well pumping recently by comparing the average pore pressure data from 1973 to 2001. Owing to the installed PVDs in the dummy area in the GILRS, the water pressure tends to be close to the hydrostatic up to the depth of the PVD installation, as observed from Fig. 5. A lower pore pressure below 10 m depth within the clay layer was also observed when compared with the previous data. Zero pore pressure was first observed at 20 m depth during GIAP design in 1995, and it was found to be at about 18 m during GIAP construction. From a depth of 18 m to the maximum 35 m depth of the open tube piezometer installation, the water pressure varied linearly with depth.

Ground improvement scheme

Ground improvement with preloading is used to preconsolidate the underlying soft soil in the relevant area by applying a temporary surcharge load, which should be greater than the permanent load in order to reduce post-construction settlement. The ground improvement sequence in the GIAP and GILRS, as shown in Fig. 6, includes construction of a sand blanket, drainage facilities, instrumentation, PVDs and preloading. The concept of cycle loadings by using removed surcharge material was planned in both projects to reduce the construction cost. The GIAP deals mainly with large areas such as the apron and runways. All surcharge fills have to be removed to MSL after ground improvement. The GILRS is for the road embankments only (except for the remote parking area). Surcharge fill will be part of the road structure (removed to subgrade level). The major design consideration for the GILRS is that the maximum monthly settlement of the road embankment prior to pavement construction should not be more than 5 mm, whereas for the GIAP the objective of ground improvement was to achieve a minimum 80% primary consolidation of the underlying soft soil. A comparison of ground improvement design in the two projects is summarised in Table 2.

In order to estimate the degree of primary consolidation, a graphical method developed by Asaoka in 1978 was adopted to predict the ultimate primary consolidation settlement from field data. The length of PVDs in both projects was limited to 10 m in order to reduce the risk of hydraulic contact with permeable layers with low pore water pressures, based on previous study at the SIA site. Most of the field embankment thicknesses in GIAP were determined by the surcharge load (75 kPa and 85 kPa) during construction instead of the design thickness, in order to reduce the construction cost. The actual fill thickness was about 10 cm less than the design thickness.

In addition to the 6 months final waiting period (or 11 months for aircraft stands in the GIAP), other surcharge-removing criteria in the two projects were also different. A minimum 80% primary consolidation and maximum 4% (under 75 kPa) and 2% (under 85 kPa) settlement ratio, which is the ratio of the last month's settlement to the accumulated settlement, are required in the GIAP. The GILRS specifies a maximum settlement rate of 30 mm per month prior to surcharge removal.

Instrumentation

Instrumentation plays an important role in ground improvement projects. Proper and prompt monitoring data can provide effective safety control during embankment construction. The types of instrument installed in the two projects are similar: they consist of surface settlement plates, deep settlement gauges, pneumatic (GILRS) or electric (GIAP) piezometers, inclinometers and observation/pumping wells. Surface settlement plates were placed after PVD installation in the GIAP. Surface settlement monuments to monitor fill settlement and two 150 m deep permanent benchmarks were also installed in the GIAP. There were some dummy instruments installed outside the preloading embankment area in both projects. For the GIAP, dummy instruments include deep settlement gauges and electric/AIT-type (open tube) piezometers. Two sets of dummy instruments including surface settlement plates and piezometers were planned together with PVD installation in the GILRS. Typical cross-sections and the total quantity of instrumentation in the two projects are shown in Fig. 7 and Table 3 respectively.

The required monitoring frequencies in the two projects are slightly different. In general, monitoring is required before and after each stage of fill work, followed by monitoring once a week in the GIAP. Three types of monitoring frequency, varying from once a week to once every two weeks, were specified in the GILRS.

Monitoring data

The monitoring data can be divided into two parts: vertical/lateral movement obtained from the surface settlement plates, deep settlement gauges and inclinometers, and pore water pressure obtained from the piezometers. Owing to space limitations, the data presented here focus mainly on the apron and west runway of the GIAP, and the south access road (SAR), the major and first constructed section in the GILRS.

Vertical/lateral movement

Under similar subsoil conditions, variations of vertical and lateral movement were obtained in accordance with the time and height of surcharge fill placement. A marked increase in settlement was observed after the first or second stage of surcharge fill loading was placed in both projects. Settlement contours in the apron area of the GIAP are presented in Fig. 8. Uniform settlement has been observed throughout the apron area, with average settlements of 1400 mm and 1700 mm under 75 kPa and 85 kPa loading respectively. Settlements at the west runway and cross taxiway varied from 1100 mm to 1500 mm. A small loop area in the middle of the west runway had only half of the settlement of other areas under the same fill height, as shown in Fig. 9. It was later found that PVDs were not needed in this area. However, the settlement results have shown the effectiveness of PVD installation. Fig. 10 shows typical settlement profiles at various depths as monitored by deep settlement gauges and surface settlement plates. In general, the soft clay layer experienced the largest proportion of settlement, as expected. Below the depth of 16 m, which the stiff clay layer underlies, the amount of settlement was minimal. Similarly, the top hard, weathered crust underwent only a small amount of settlement, as shown by the nearly equal value of settlement measured by settlement gauges at 2 m depth and those at the ground surface. In general, the maximum lateral movement at most of the inclinometer locations occurred at an elevation of 3·0–6·0m below MSL, and less lateral movement was observed at the

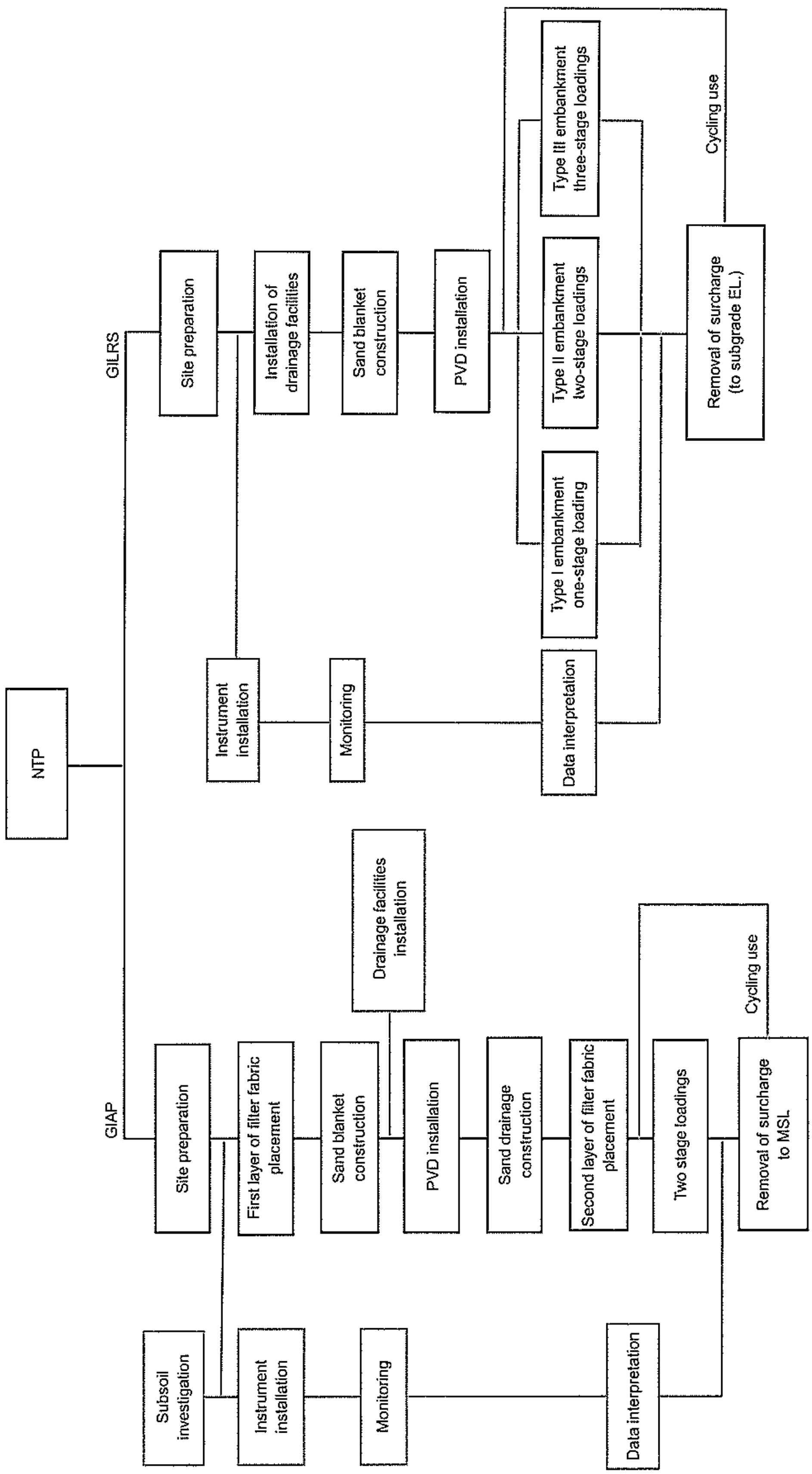

Fig. 6. Ground improvement sequence

Table 2. Comparison of ground improvement design for GIAP and GILRS

Project item	GIAP	GILRS		
		Type I	Type II	Type III
Design criterion	Minimum 80% of primary consolidation should be completed	Rate of consolidation settlement of subsoil should be less than 0·5 cm/month before pavement construction		
Sand blanket	150 cm	50 cm	80 cm	130 cm
PVD	10 m deep with 1·0 m spacing in square pattern	10 m deep with 1·0 m spacing in triangular pattern		
Filter fabric	Below and above sand blanket	None		
Preloading material	Crushed rock	Sand		
Stage loading	Two†	One	Two†	Three†
Embankment thickness	3·8 m and 4·2 m	2·2 m	3·5 m	4·5 m
Berm	15 m wide and 1·7 m high with 1 : 4 side slope	No berm, with 1 : 3 side slope		
Design load	75 kPa and 85 kPa	41·8 kPa*	66·5 kPa*	85·5 kPa*
Removal criteria	Minimum 6 (or 11) months waiting period, minimum 80% consolidation and maximum 2–4% settlement ratio	Minimum 6 months waiting period with maximum 3 cm monthly settlement rate		

* Embankment unit weight is assumed to be 19 kN/m^3.
† Three months waiting period is required before next stage of loading construction.

apron than in other areas. Fig. 11 illustrates a typical cross-section of settlement and lateral movement profile at the west runway. It is interesting to note that most of the maximum settlement did not occur at the centre of the embankment, which may result from contributions of lateral movement and a short drainage path to the side settlement plates. The ratio of lateral movement to vertical settlement was used as the criterion for safety control in both projects. Special attention was given to the control of rate of construction at the site if the ratio exceeded 0·25. Fig. 12 summarises values of the ratio of lateral to vertical movement at the apron and cross taxiway during construction. The narrow road configuration generally had a higher ratio. The ratio of 0·25 appeared to be a reasonable criterion to be used in view of the performance results.

Based on settlement data along the SAR of the GILRS, an average of 85 cm and 130 cm settlement was recorded for the type I and type II embankments after waiting periods of 6·5 and 3 months respectively. Deep settlement data from the sensor-ring type of deep settlement gauge, however, showed much smaller settlement within the soft clay layer than the data obtained from the GIAP. This might have been caused by blocking of the sensor rings by the backfill grout. Owing to its road embankment configuration, lateral movement for the SAR is higher than for the GIAP. Ratios of lateral to vertical movement at embankment types I and II are also plotted in Fig. 12.

Pore water pressure

Excess pore water pressures were calculated based on the difference between piezometer readings under surcharge loading and dummy readings or the dummy curve as shown in Fig. 5. The elevation of dummy readings was corrected for settlement of the piezometers. In general, excess pore water pressure increased during fill construction and gradually decreased during the waiting period. Fluctuating data were often observed, especially in the rainy season. The measured dissipation of excess pore pressure during first-stage loading, which was below the preconsolidation stress, was rapid, and the dissipation rate then decreased with increasing effective stress. Dissipation of excess pore water pressure at 12·0 m depth was slower than at other depths, which may due to the fact that the piezometers were installed below the depth of the PVDs. The excess pore water pressure at depths of 5·0 m and 8·0 m, in very soft to soft clay, were higher than that at 2·0 m, as expected. A typical excess pore pressure dissipation curve with time and the fill status at the apron are shown in Fig. 13.

Evaluation of ground improvement performance

The ground improvement performance was evaluated by comparing the settlement data both with the design estimate and with changes in soil properties obtained from additional soil investigation results after ground improvement.

Based on the design perspective

Before comparing the design settlement with the field data, the following factors had to be taken into consideration:

(*a*) The actual waiting period was about 1–2 months longer than the minimum requirement of 6 months in both projects, mainly because of the settlement rate/ratio criteria.
(*b*) The field settlement consists not only of consolidation settlement but also of immediate settlement, subsidence and contributions from lateral movement.
(*c*) Initial settlement under the first metre of sand blanket in the GIAP was ignored since all settlement plates (except the reference section) were installed after the PVD installation.

The design settlement at the end of the final 6-month waiting period was about 1500 mm, 1700 mm, 840 mm,

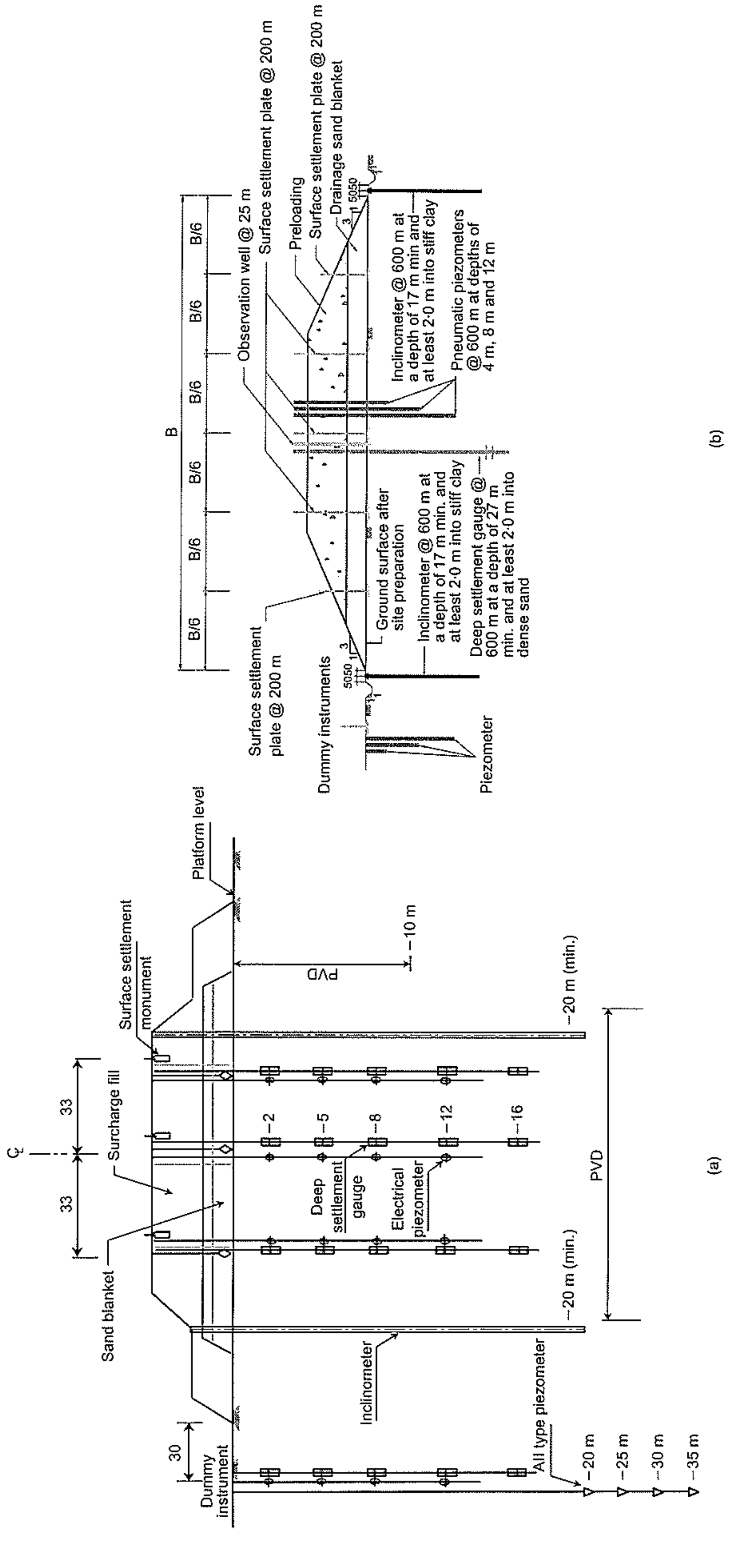

Fig. 7. Typical cross-section of instrumentation: (a) GIAP; (b) GILRS

Table 3. Total quantity of instrumentation

Project item	GIAP		GILRS	
	Embankment	Dummy	Embankment	Dummy
Surface settlement plate	1724	–	730	4
Surface settlement monument	553	–	–	–
Permanent benchmark	–	2	–	–
Inclinometer	56	–	88	–
Deep settlement gauge	111*	11*	53†	–
Pneumatic piezometer	–	–	159	6
Electric piezometer	444	46	–	–
Air-type piezometer	–	40	–	–
Observation well	1722	–	1236‡	–

* One set includes five individual deep settlement gauges at 2 m, 5 m, 8 m, 12 m and 16 m.
† A deep settlement gauge includes sensor rings at every 2 m to a minimum depth of 27 m.
‡ Also used as pumping wells.

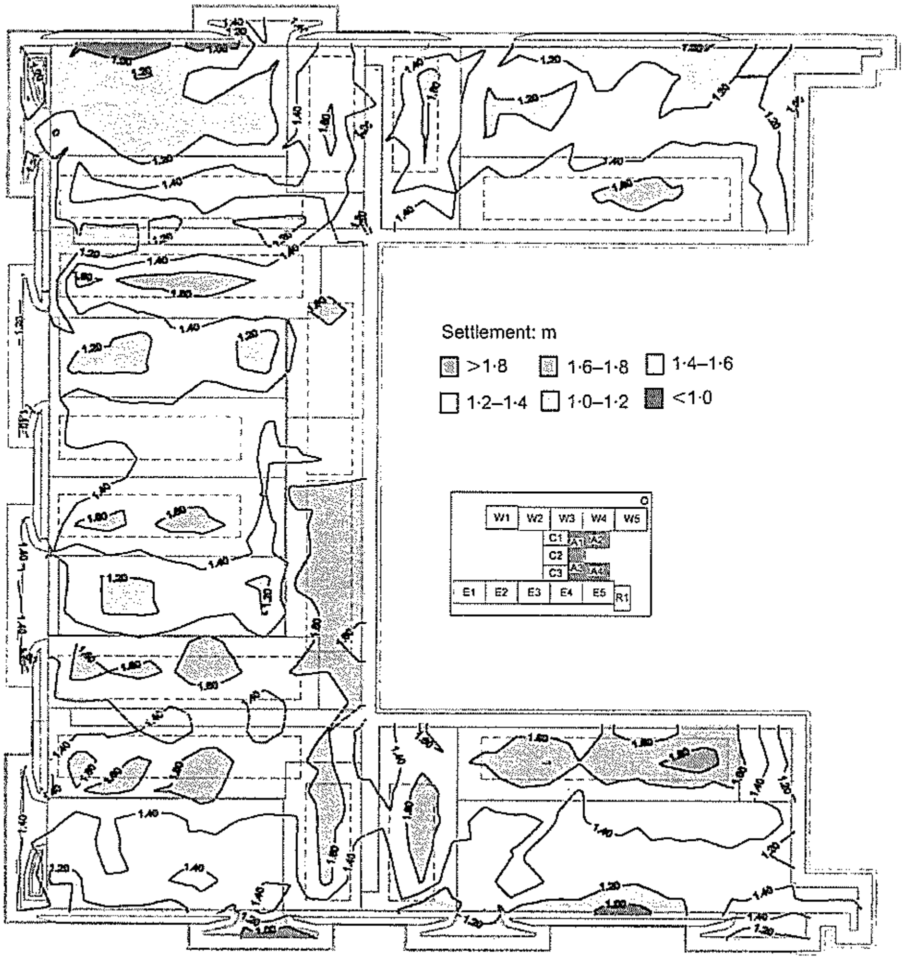

Fig. 8. Settlement contours in the apron area

1400 mm and 1900 mm under 75 kPa and 85 kPa in the GIAP and for type I, II and III embankments in the GILRS respectively. The overall field settlement performance (neglecting the initial settlement from the 1 m thick sand blanket) in the GIAP was about 100–200 mm less than the design expectations, as shown in Fig. 14. Settlement from the AIT test embankment (PVDs at 1 m spacing with 12 m length) is also included. In the GILRS, settlement of the type I embankment of the SAR at the end of the final waiting period is similar to or a little over the design value. After the

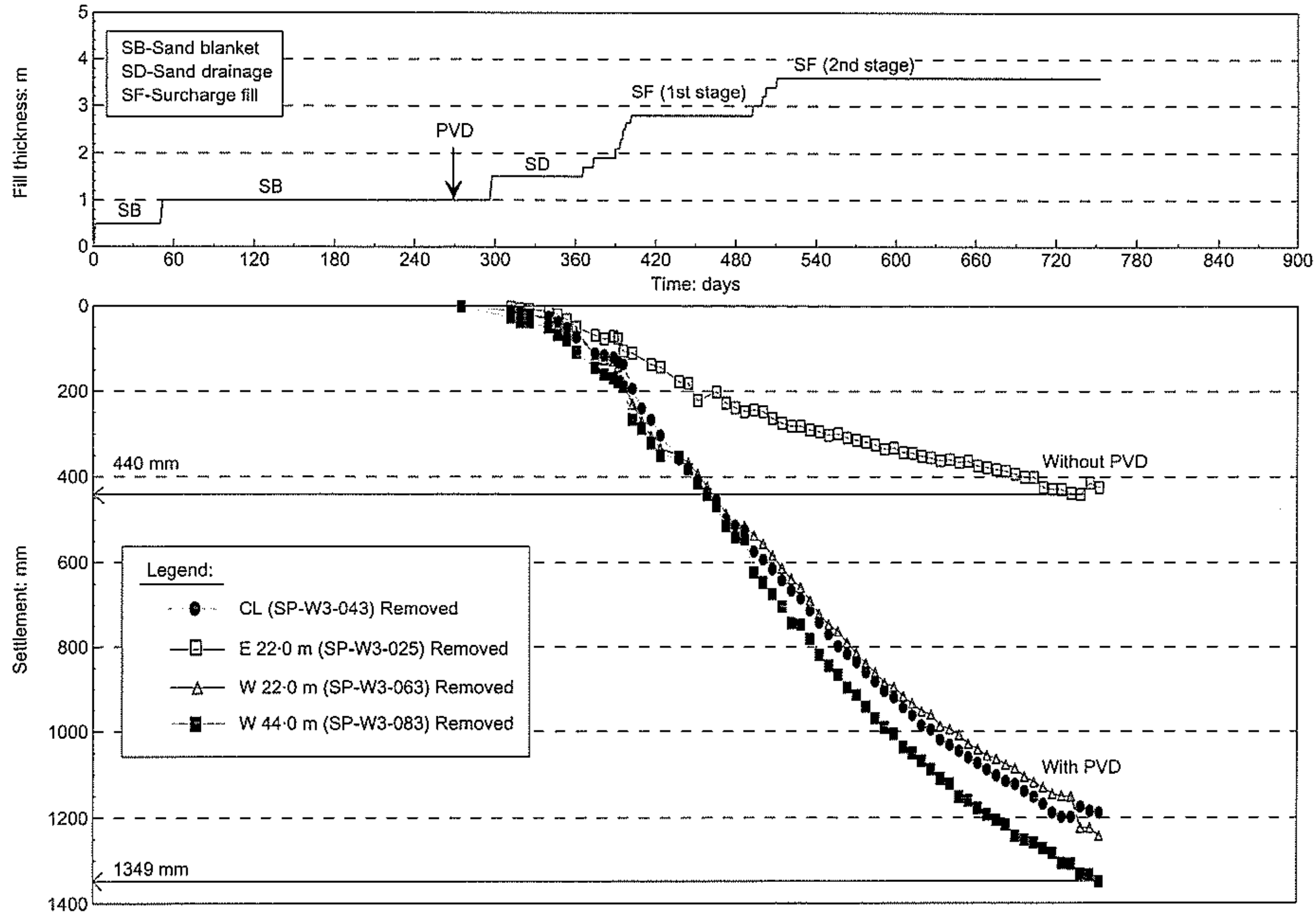

Fig. 9. Comparison of settlement between PVD and non-PVD areas

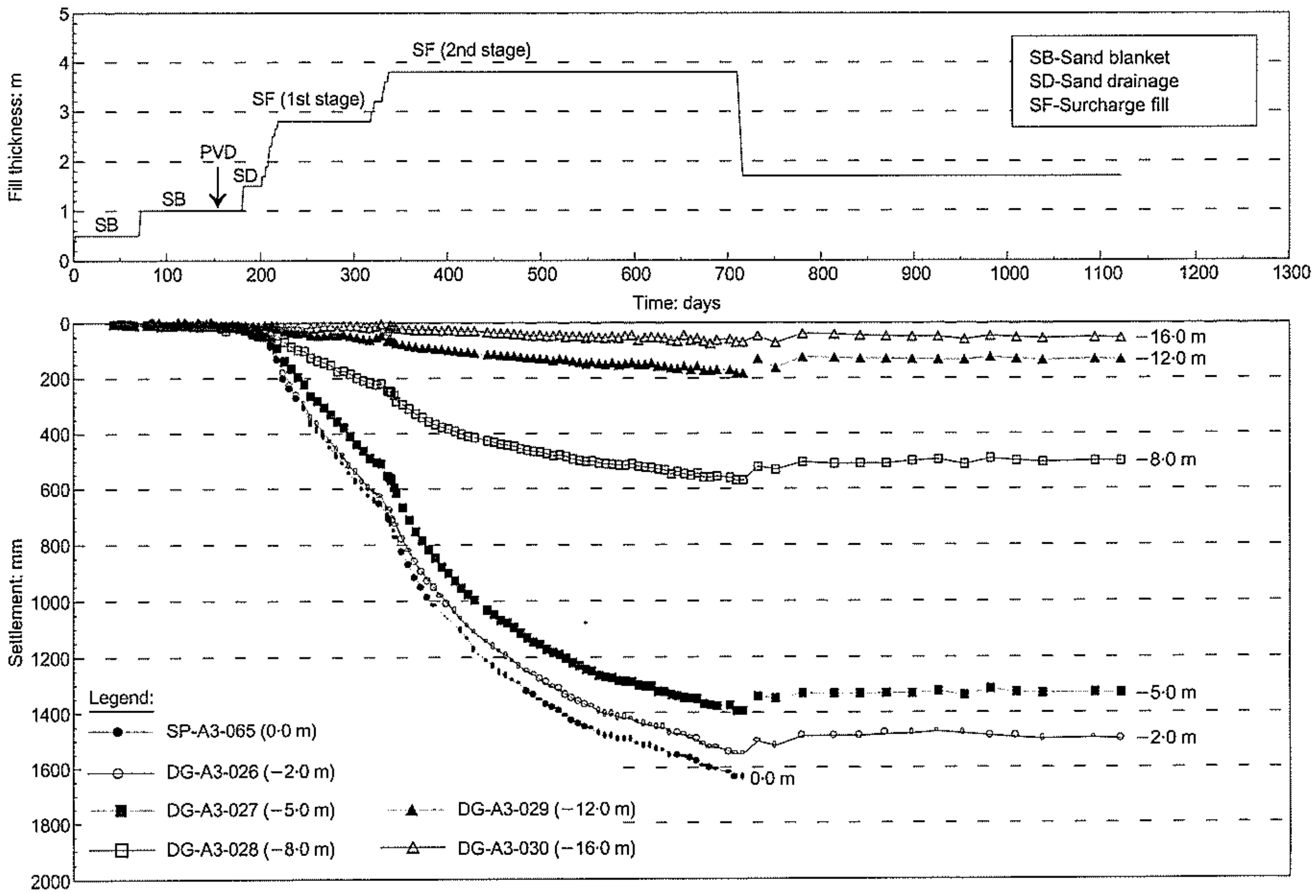

Fig. 10. Deep settlement profile in the apron area

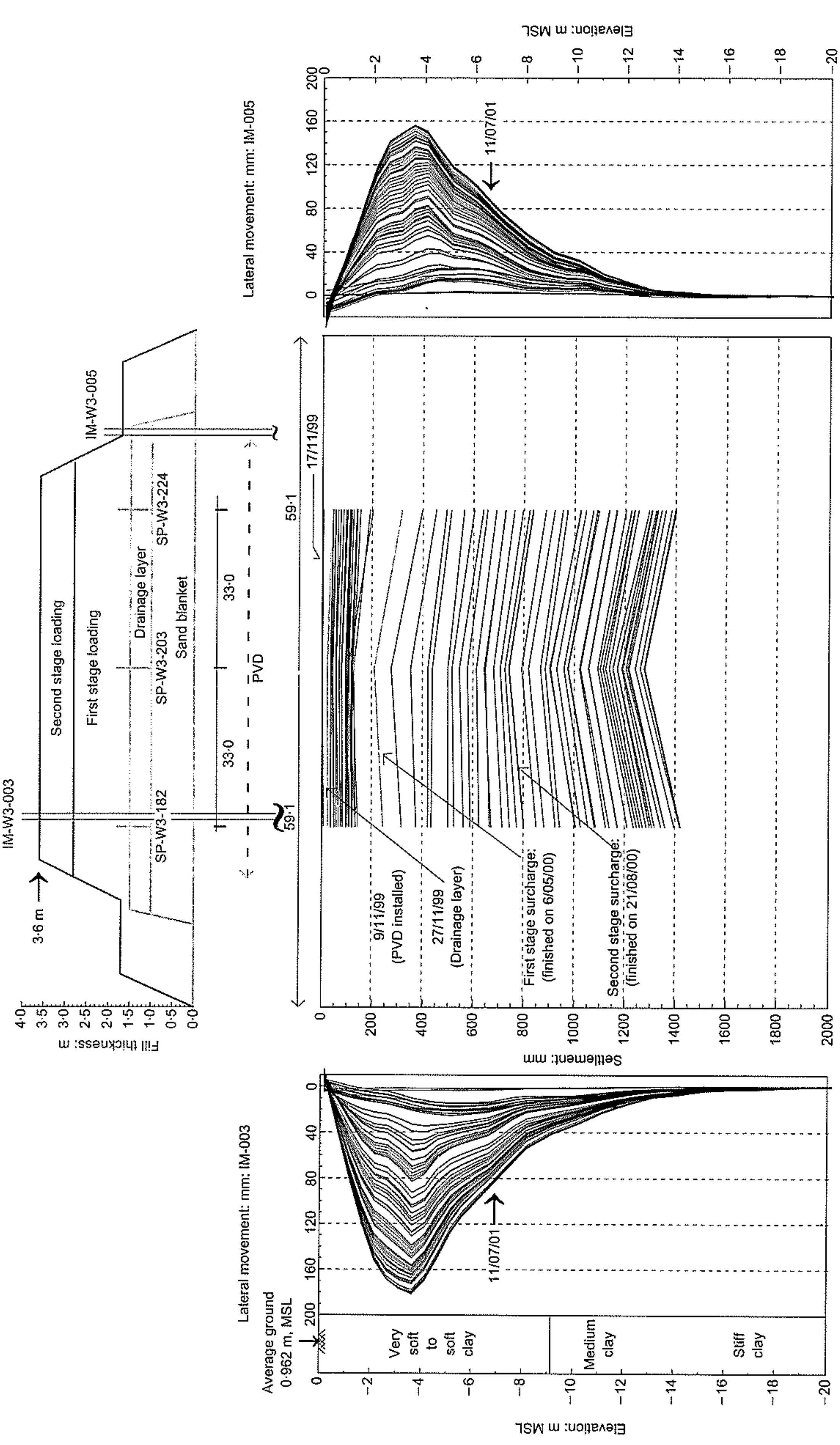

Fig. 11. Observed settlement and lateral movement profile at west runway

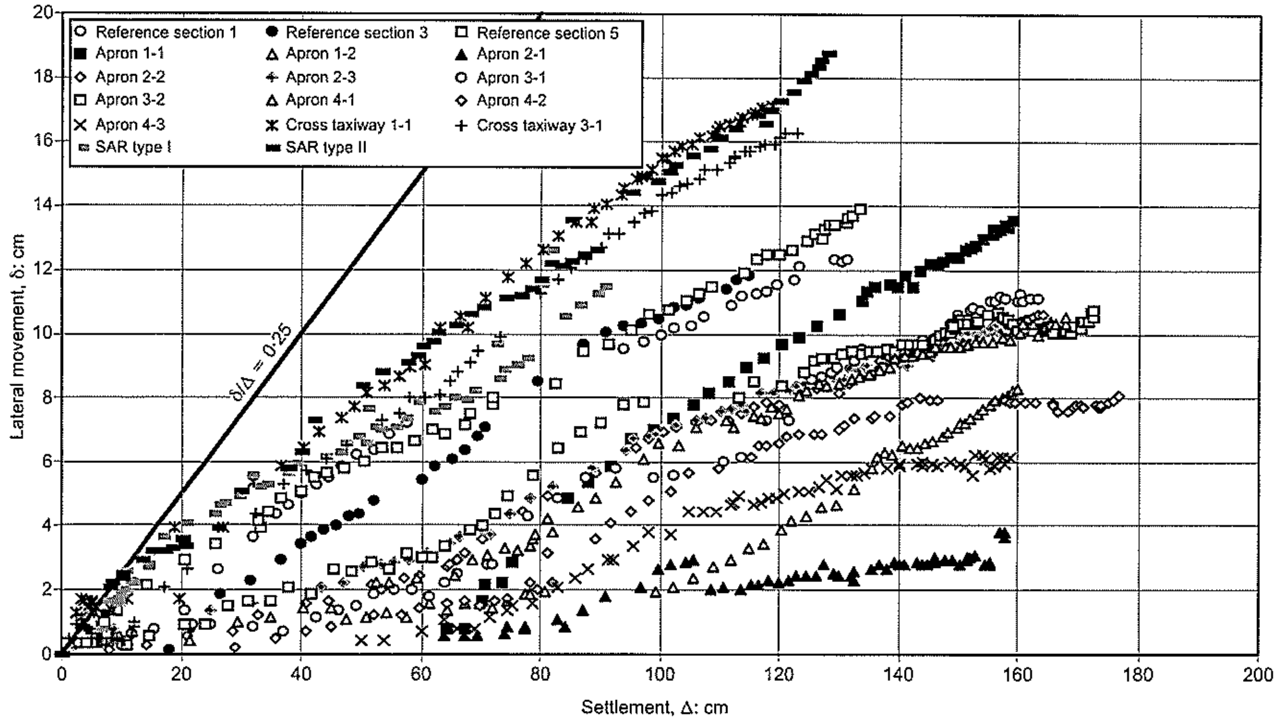

Fig. 12. Ratios of lateral to vertical movement at apron, cross taxiway (GIAP) and SAR (GILRS)

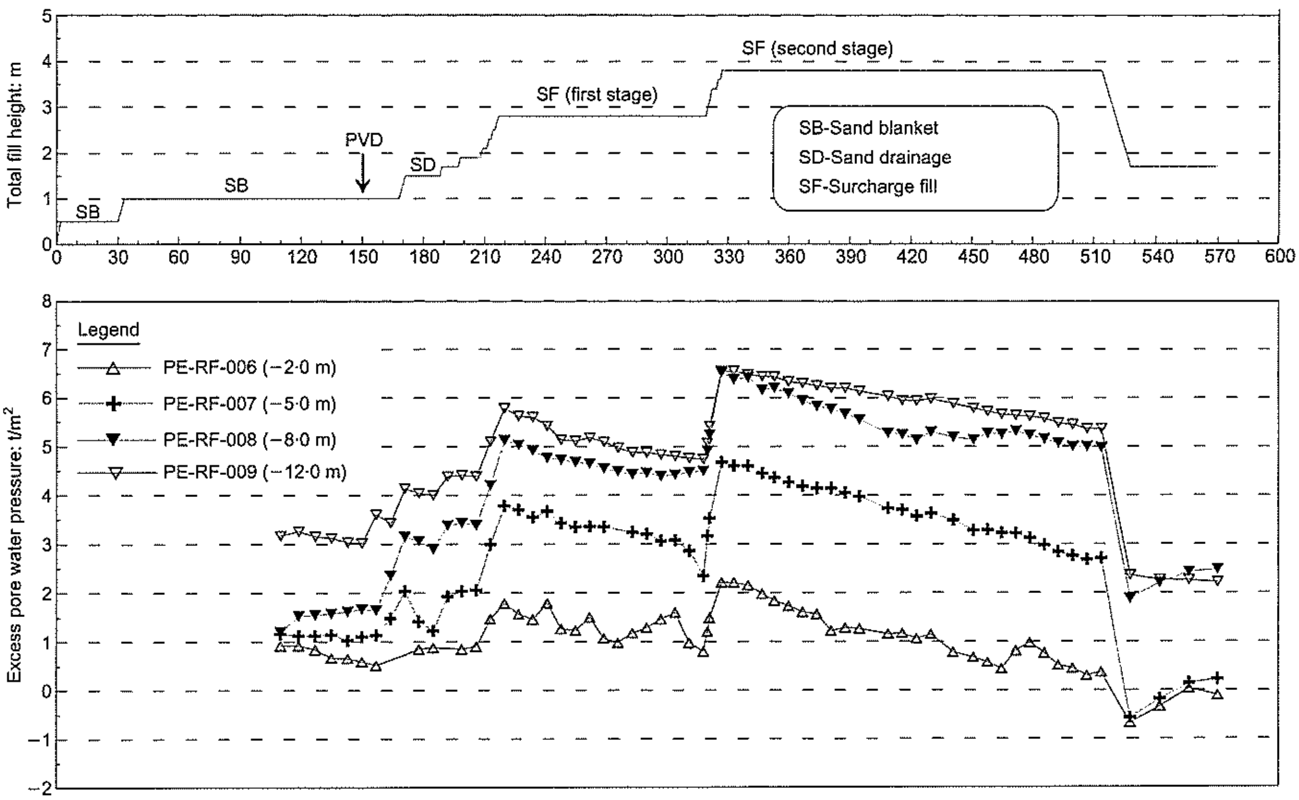

Fig. 13. Observed excess pore pressure distribution with time and fill status at the reference section

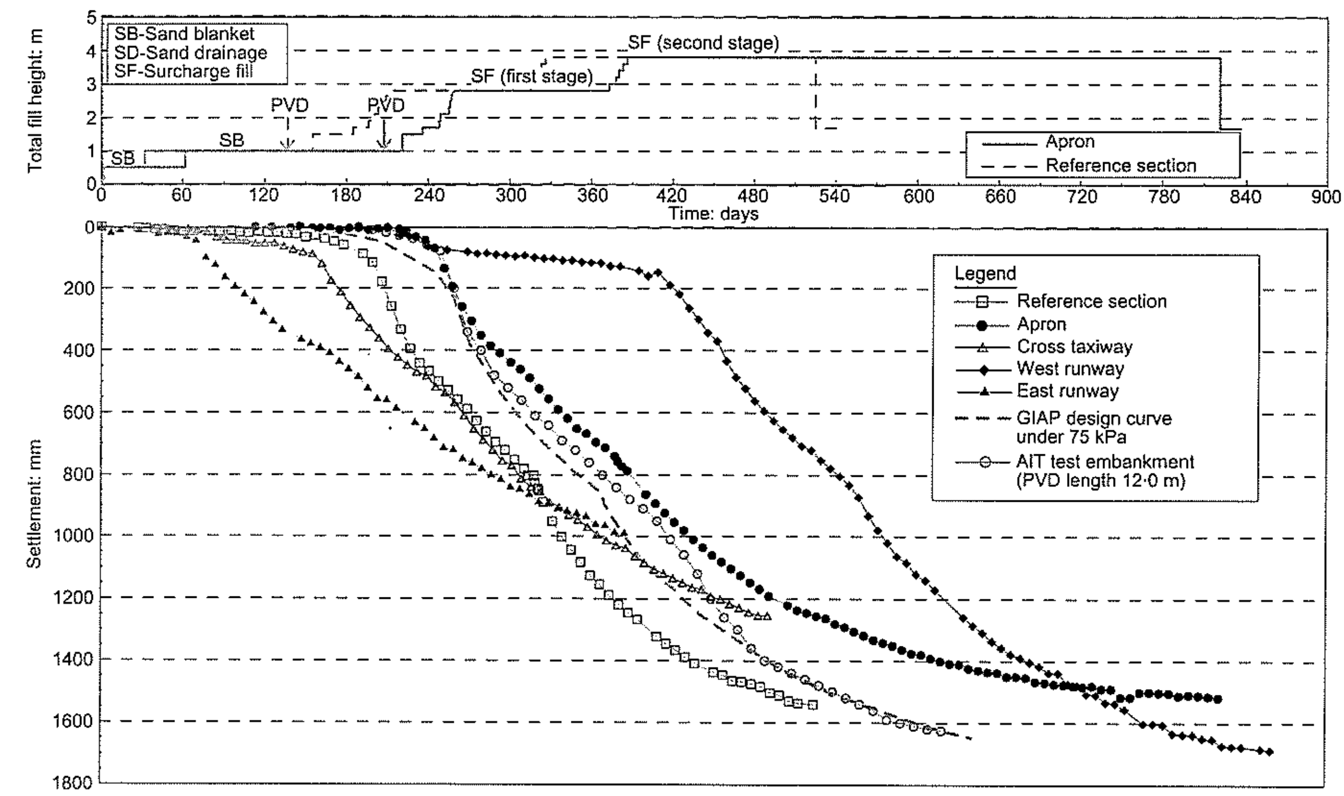

Fig. 14. Comparison of field settlements with the design and AIT test embankment

3-month waiting period, settlement of the type II embankment was also close to the design estimate under the same period. However, more data are necessary in the GILRS for reaching final conclusions.

Based on changes in soil properties

A decrease in natural water content and an increase in total unit weight and shear strength are expected for the soft clay after ground improvement. Additional subsoil investigation including undisturbed sampling, filed vane shear test and piezocone penetration test has been carried out in the apron area and also in part of the west runway of the GIAP, at the locations shown in Fig. 2. More soil investigation is expected to be done in both projects in the future. A comparison of soil properties before and after ground improvement is shown in Fig. 15. Water content within the soft clay zone reduced by 20–25%, whereas the total unit weight and undrained field vane shear strength increased by up to 10% and 100% respectively. The upper stratum of very soft to soft clay has been improved to medium stiff clay, according to the soil properties.

Conclusions

Some conclusions can be drawn based on the current performance of the GIAP and GILRS, as follows:

(*a*) The ground subsidence rate decreased from 1995 to 1998 and increased in 1999 and 2001 based on survey data near the site. Closely monitored surveying and strict control of deep well pumping should be continued.

(*b*) It is further confirmed that the piezometric pressures in the upper 18 m of the subsoils at the SIA site exist in an under-hydrostatic condition. Further drawdown was also observed below the depth of the PVD installation.

(*c*) An effective drainage system and proper monitoring are the key for success of ground improvement by preloading with PVD installation.

(*d*) The criterion of 0·25 for the ratio of lateral to vertical movement appears to be reasonable for embankment safety control.

(*e*) Settlement data are more reliable than pore pressure values in both projects. In general, field settlements in both projects are close to the design estimates.

(*f*) The overall performance of the ground improvement work in the GIAP is satisfactory, based on changes in the soil properties after improvement. More data are necessary in the GILRS for final conclusions.

Acknowledgements

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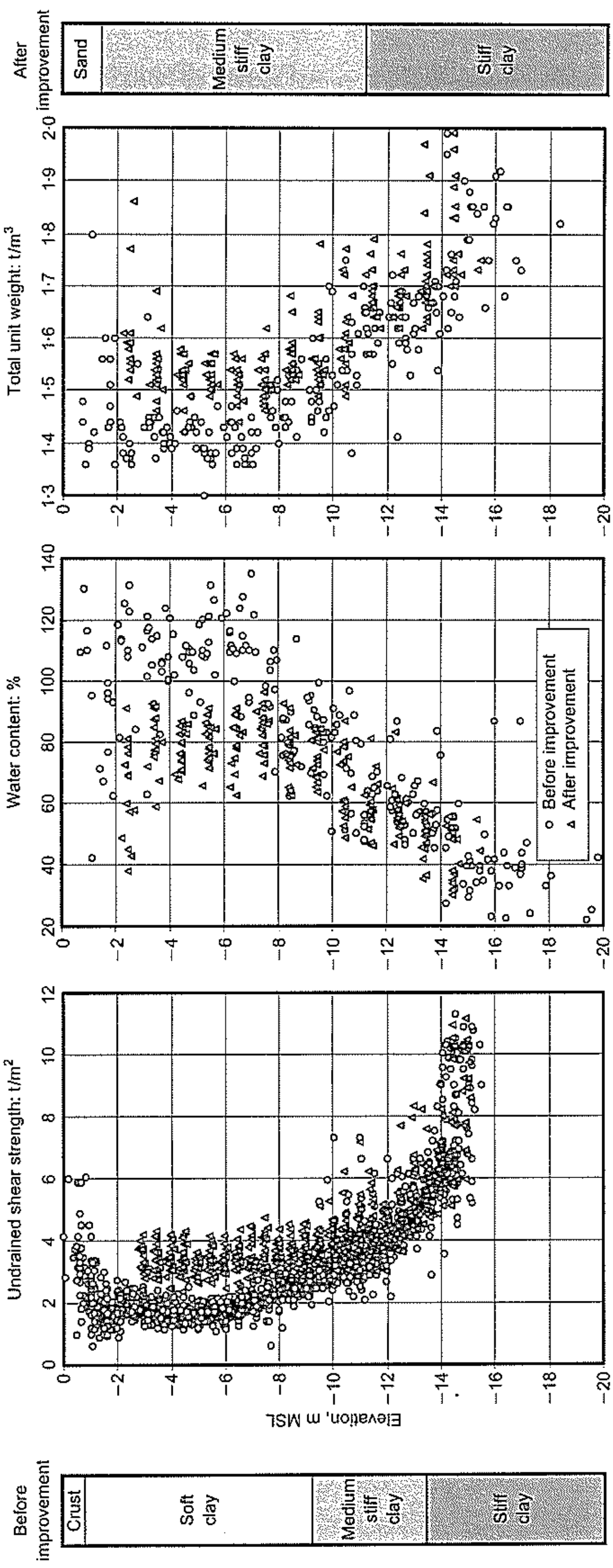

Fig. 15. Comparison of soil properties before and after ground improvement

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Geotechnical Issues in Design and Construction of Viaducts of the Taiwan High Speed Rail

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Abstract: With a total length of 345 km and an estimated construction cost of US$15 billions, the Taiwan High Speed Rail Project is considered to be one of the largest BOT (Build-Operate-Transfer) projects in the world. The route of the Rail runs through the populated west coast of Taiwan Island and links the City of Taipei at its northern end and City of Kaohsiung at its southern end. The geological conditions along the route vary tremendously, from rocky mountains in the north to thick sedimentary deposits in the south and, as a result, the northern section is constructed mostly by cut-and-fill, with bridges and tunnels, while the southern section is mostly elevated. Discussed herein are some major geotechnical issues in the design and construction of viaducts in the southern section with considerations primarily given to pile load tests, negative skin friction on piles, land subsidence, scouring and protection of riverbed, etc. Furthermore, as the route crosses three active faults, soil liquefaction potential and characteristics of ground motions have significant influences on the design of piles and are also discussed.

1. INTRODUCTION

With a total length of 345 km and an estimated cost of construction of US$15 billions, the Taiwan High Speed Rail Project (the THSR Project, hereinafter) is considered to be one of the largest BOT (Build-Operate-Transfer) projects in the world. The route of the High Speed Rail (the Rail, hereinafter) runs through the highly populated western coastal plains of the island and, as depicted in Fig. 1, links many major cities. With a design maximum speed of 350 km/hr, passengers will be able to make the trip from Taipei to Kaohsiung in 90 minutes instead of four and half hours by normal trains or cars. With an anticipated ridership of 300,000 per day, the Rail will certainly become a major transportation artery of the island. Once the Rail is open for revenue services, it is expected that traffic congestion will be a distant memory. The Rail is also expected to boost the national economy because of the increase in productivity as a result of better efficiency in transportation and redistribution of population and resources.

Taiwan is located at the Circum-Pacific earthquake belt. In fact, the island was created by the subduction of the Philippine Sea Plate underneath the Eurasian Plate. An annual movement of 7cm has been observed and, as a result, a complex geological structure is formed and geological conditions vary drastically from one place to another, as shown in Fig. 1. The foundations supporting the super-structures of the Rail also vary from place to place along the route. In the northern section, which accounts for one-fourth of the route, the terrain is mountainous and the Rail line is mostly constructed by cut-and-fill with bridges across rivers and tunnels through mountains. The rest of the route runs through the coastal plain and is mostly elevated with viaducts founded on piles.

Because of the differences in ground conditions and different methods of construction, geotechnical issues to be considered are also different from section to section. For the northern section, the major considerations are slope stability and prevention of geo-hazards, and for the rest of the route, economical design of piles is of primary importance. Discussed in this paper are the various geotechnical issues related to the design of the viaduct foundations, including pile loading tests, effects of liquefaction, ground subsidence, characteristics of ground motions and scouring of river bed, etc., on piles.

2. DESIGN REQUIREMENTS

Because of the high speed of trains, design criteria for the Rail are more stringent than those adopted for other structures. It is stipulated in design specifications that settlements of piles due to all the superimposed dead loads, including trackwork, shall be accounted for in design to ensure that the vertical alignment of the rails meets the criteria. To ensure that trains can run at their maximum speed of 350 km/hour, changes in the vertical alignments of rails as a result of differential settlements between neighboring piers after the completion of construction shall not exceed 1:1000 for simply supported spans and 1:1500 for continuous spans.

As mentioned above, the Taiwan Island is located on the Circum-Pacific Earthquake Belt, it experiences several major earthquakes annually. The devastating Chi Chi Earthquake of 1999 took more than 2,000 lives. Therefore seismic loads are major factors to be considered in designing infrastructures. For the THSR Project, two levels of ground shaking are considered:

(a). Type I Earthquake - damages are repairable

Type I Earthquake corresponds to earthquakes with a return period of 950 years, which has a 10% probability of exceedance in 100 years. In Type I earthquakes, structures are allowed to yield but damages to structures, if any, shall be repairable. The design peak ground accelerations (PGA) in the horizontal direction vary from 0.22g to 0.40g along the THSR route for Type I Earthquakes and the peak ground accelerations in the vertical direction are two-third of those in the horizontal direction.

(b). Type II Earthquake – trains can run at maximum speed and structures do not yield

Structures are not allowed to yield in earthquakes of this level and the operation of the system subsequent to earthquakes is expected to be unaffected. In order to make sure

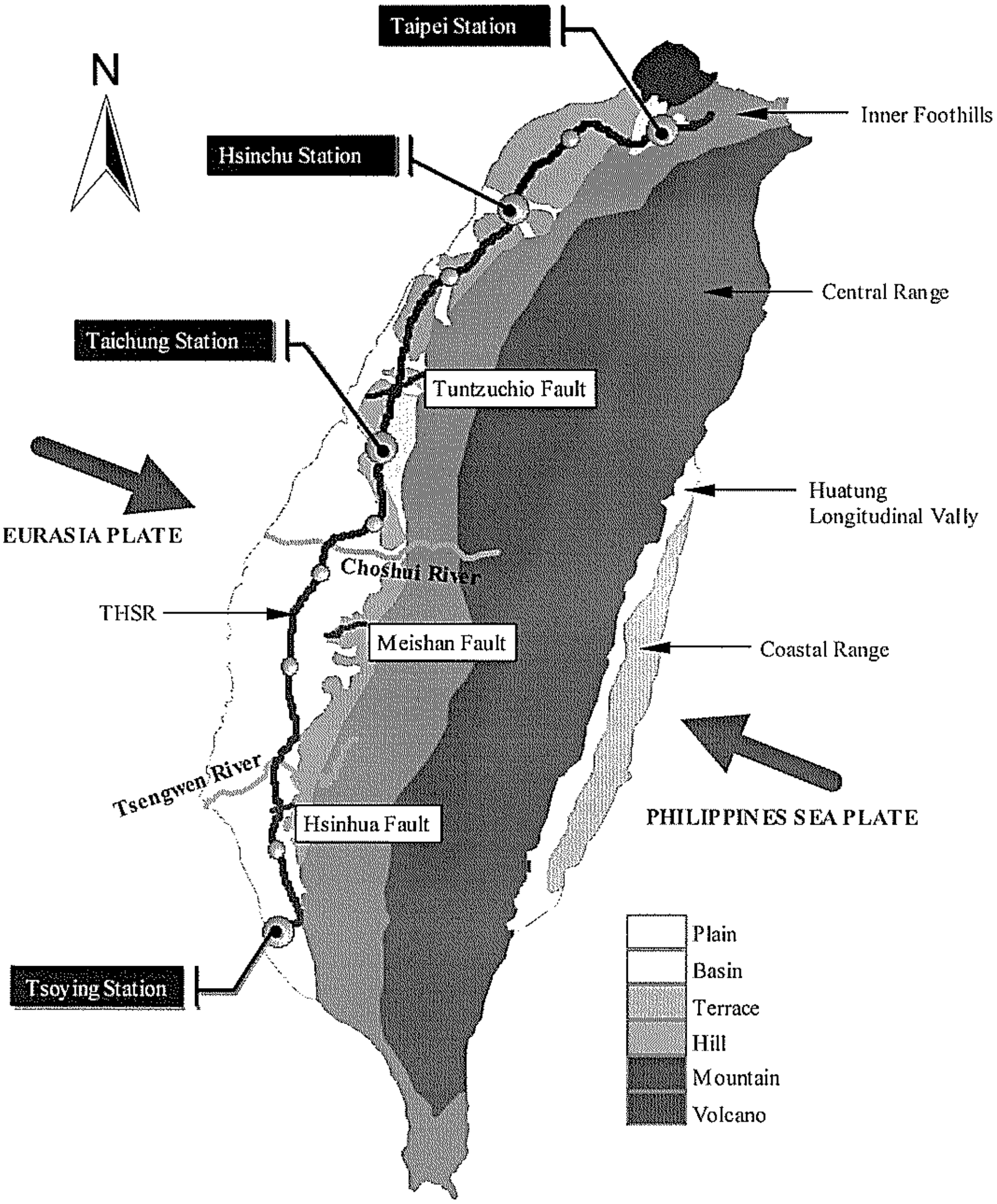


Fig. 1. General geology of Taiwan Island and route of the High Speed Rail.

that trains still can run at their full speed of 350 km/hr, it is mandatory to inspect the deformations of tracks after earthquakes and make sure they are within allowable ranges. The design PGA in the horizontal direction for Type II earthquakes are one-third of those specified for the Type I earthquakes and the design ground acceleration in the vertical direction are two-thirds of those in the horizontal direction. Under Type II earthquakes, maximum lateral displacement shall not exceed: 50mm tilt from vertical axis at top of caissons, 50mm relative displacement between pile head and pile toe, and 50mm relative displacement between barrette head and barrette toe.

In the Chi Chi Earthquake of 1999, many bridges collapsed with their decks falling to the ground. Learning from this lesson, design codes have been revised with considerable improvements on structural details. Shear keys, longitudinal restrainers and transverse girder stoppers are provided to prevent bridge decks from falling. Furthermore, emphases have been given to ground motions near faults.

3. PILE FOUNDATIONS

Shallow foundations are usually more economical to use and are used where there is a suitable bearing stratum at shallow depths and settlements are expected to be within tolerance. Deep foundations are used when shallow foundations are unsuitable or uneconomical. However, for the viaduct section of the THSR, with a total length of 244km, piles are exclusively used to support the superstructures because of (i) presence of thick alluvium soil deposit, (ii), large earthquake forces, and (iii) limited right of way (generally only 18m).

3.1 Design of Pile Foundations

Bearing capacities of piles are doubtlessly the primary consideration for pile design. According to the design specifications, to ensure piles to have sufficient end bearing, piles are required to enter the bearing soil formations for at least 2.5 m. In rock formations, the minimum penetrations of piles required vary from 0.5 m for rocks with compressive strengths of 5 MN/sq m to 2.5 m for rocks with compressive strengths of 0.5 MN/sq m. If there is a compressive stratum underlying the bearing soil formation, the thickness of this bearing soil formation below the toe of the pile shall be adequate to resist punching and settlement of the pile is required to be within the specified limits. Notwithstanding, in no case shall this thickness be less than three times of the diameter of the pile.

Since both physical and engineering properties of the subsoils have profound effects on the behavior of pile foundation and the THSR route passes through variable subterrain conditions, it is important to understand the effects of soil parameter on pile foundation in order to arrive at economic designs within relatively short period of time. Parametric studies of sensitivity of pile design to soil parameters were carried out. Figure 2 illustrates one case of such study. A group of 4 piles with a same diameter of 2 m and same length of 59 m was subjected to a lateral load of 1,200 tons which corresponds to the ultimate load case incorporated with the Type I earthquake load. Piles are spaced at a distance of 8.5m center to center and a factor of 0.78 is adopted for group effects. The connections between the piles and the pile cap are assumed to be rigid and rotation is not allowed.

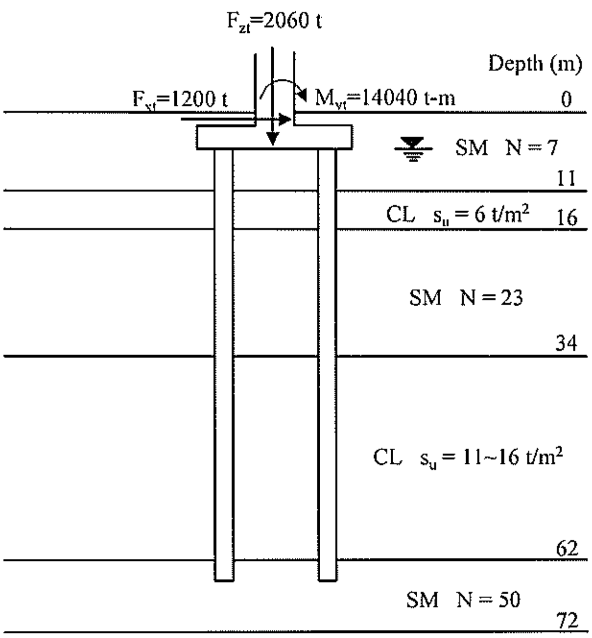


Fig. 2. Case for a sensitivity study of soil parameters on design of pile foundation.

The ground consists of alternating layers of sand and clay as shown in the figure. If the soil strength and stiffness, represented by N values of Standard Penetration Tests, for the top 10 m below ground surface decrease to one-third of their original values, then it is necessary to increase the pile length by about 4m and to increase the amount of steel reinforcement in the pile head by 50 per cent in order to satisfy the design requirements. On the other hand, if the soil strength remains unchanged but the horizontal soil stiffnesses decreases to one-third of their original values along the full length of piles, the lengths of piles have to be increased approximately by about 2m and the steel reinforcement in the upper portion of the piles has to be increased by 40 per cent. The results are summarized in Table 1.

Table 1. Pile design as affected by changes in soil parameters.

Change in soil parameters	Pile Length	Reinforcement
Decrease N and kh in top 10m to 1/3	+ 4m	+ 50%
N unchanged, kh for the full length reduces to 1/3	+ 2m	+ 40%

Response of a full structure-foundation-soil interaction system is usually analyzed by replacing soils with equivalent springs, including horizontal spring (k_h), vertical spring (k_v), rotational spring (k_r) and torsional spring (k_t). The same pile group is used as an example to demonstrate the sensitivity of design of superstructure to soil parameters. For bridge piers with 8m, 12m and 16m in heights, the amount of steel reinforcements required is 2.1, 2.4 and 2.7 per cent, respectively. Decreasing the stiffness of horizontal springs by 4 times will reduce the amount of steel reinforcement required in the bridge pier by 21, 3 and 0 per cent respectively. Decreasing both the stiffness of vertical and rotational springs by 4 times will decrease the amount of steel reinforcement required by 17, 14 and 3%. The results are summarized in Table 2.

Table 2. Reinforcement required in bridge piers as affected by changes in equivalent soil springs.

Change in soil parameters	Reinforcement in piers		
	Pier ht. 8m	Pier ht. 12m	Pier ht. 16m
kh decrease by 4 times	− 21%	− 3%	− 0%
kv and kr decrease by 4 times	− 17%	− 14%	− 3%

An analysis is also performed for the case of 8m high bridge pier and it is found that the results are not sensitive to changes in torsional spring (k_t).

The above analysis indicates that decreasing soil stiffness will increase the reinforcement required in piles but will decrease the reinforcement required in piers. The use of conservative soil properties is not necessarily always conservative. Therefore, the soil parameters adopted in the design should be obtained from representative soil strata and reliable in situ and laboratory test results. Local experience and back analyses from full scale loading tests and monitoring results often play important roles in the foundation design.

3.2 Construction of Piles

For a 155 km section in the mid to south section of the route, more than 20,000 piles have been installed for supporting the viaducts, mostly 1.5m to 2.5m in diameter and 35m to 72m in length. Piles with 1.8m and 2.0m in diameter and 50m to 60m in length are most common. Because the variations in ground conditions along the THSR route, different construction methods are adopted for the installation of piles. Due to restriction on noise and vibration, pile driving is banned and all the piles are cast-in-place piles. In the north segment of the viaduct section, the route mostly runs along foothills and the ground consists of mostly terrace deposits with a layer of gravel at surface underlain by layers of sandstone or mudstone. Bored holes were protected by casing and dug by hammer grab (within gravel layer) or drilling bucket (within rock). In some cases, the holes were fully cased and in others holes were cased only to the top of bearing stratum.

Toward the south, the major soil compositions are interbedded sand and clay. Piles are generally installed by using the reverse-circulation piling method for the reason that the method is popular in Taiwan and a large number of piling machines are available. In the Tainan and Kaohsiung areas, mudstone layer and sandstone layers exist at the toe levels of piles. Although all the machines were capable of penetrating into mudstone and sandstone, the progress of drilling was rather slow.

In some local areas, limestone was encountered and escape of drilling mud was a problem. Grouting had to be carried out to seal off water paths and sealant (e.g. wood scraps) was added in the drilling mud to reduce leakage. For limestone layer over 5m in thickness, steel casing with shoe was used for the full thickness of the limestone layer to contain drilling mud and improve the efficiency of construction.

At the peak of piling, there were more than 100 sets of piling machines working along the route at the same time. In addition, because the schedule is extremely tight, in most of cases, design had to go parallel with construction and confusion might occur as a result. This led to the concern on the quality of works. To ensure the quality of works, first of all, as a rule, a check boring had to be made at the location of each pier to reveal the local ground conditions and to determine the required length of piles to be installed. This is particularly important for piles of which significant portion of their capacities come from end bearing. Furthermore, it is required by the specifications that Designers and Contractor's Independent Checking Engineer (CICE) must certify that the geotechnical conditions actually encountered are consistent with those assumed in the designs and the Designers must modify their designs if not. Any modifications in design should be certified in writing by the CICE in accordance with the Quality Plan.

3.3 Integrity Tests of Piles

Honeycombing, segregation and necking are common problems with bored piles and presence of soil and cracks in pile shafts may also occur occasionally. For the THSR project, as a rule, over 20 per cent of the total number of piles were subjected to non-destructive integrity tests. In addition, piles suspected to be defective as indicated by piling records were also subjected to tests. If test results indicated that the pile might be defective, the test results should be provided to the Designer to evaluate the influence of these defects and take necessary remedial measures. Sometimes, continuous coring was performed to confirm the quality of the piles in doubt.

3.4 Pile Load Tests

3.4.1 Tests for bearing capacities

It is stipulated in the specifications that an appropriate number of advance piles have to be tested to failure to verify the design assumptions and to confirm the ultimate bearing capacities of piles. The locations and lengths of test piles were determined in considerations of pile type, pile capacity, and soil conditions. Generally, for each geotechnical unit with similar ground conditions, one out of every 1,000 piles was tested. Conventional loading tests were normally carried out to loads of, up to, 4,400 tons. Test piles were instrumented with rebar stress transducers mounted at different depths along their shafts for measuring the strains induced together with displacement transducers at top for measuring total settlements. With sufficient instrumentation, it was possible to compute distribution of skin friction and end bearing of the piles from the test results.

Osterberg Cells (O-Cell) were used in a few occasions as an alternative to kentledges as loading devices. Other methods, such as Dynamic Pile Load Test Method and Statnamic Load Method were acceptable, however, their validity had to be verified by performing conventional loading tests on piles with similar ground conditions. In tests using Osterberg cells, as shown in Fig. 3, piles were cast in three sections with two cells installed between neighboring sections for jacking against each other. The bottom sections were very short so the frictional resistance could be to

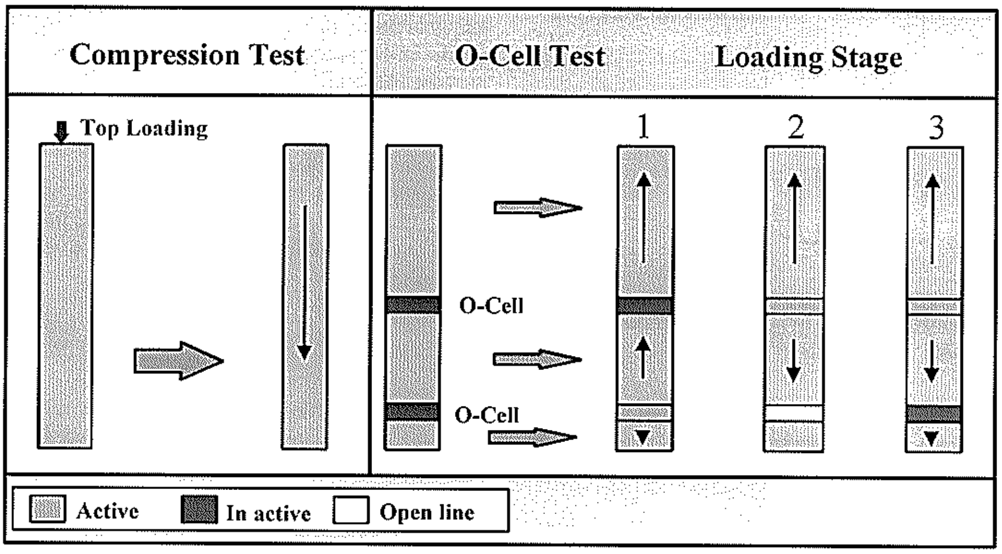


Fig. 3. Pile load tests using Osterberg Cells.

tally ignored. In the first stage of test, the upper cell were locked so the upper and the middle sections moved together. The lower cell was pressurized and the end bearing of pile could be determined with the friction on the upper and the middle section of pile as reaction. In the second stage of test, the lower cell was released so there would be no reaction from the bottom section of pile simulating soft-toe. The upper cell was pressurized and the upper and the middle sections of piles moved in opposite directions and the frictional resistance on the two sections could be determined separately. In the third stage of test, the lower cell was locked so end bearing came into play and allowed the frictional resistance on the upper section of pile to fully develop.

The use of Osterberg cells has the great advantage that the ultimate end bearing capacity of full-scale piles can be measured. Cells installed at the bottom of test piles were able to mobilize end bearing of a pile with the frictional resistance on the full length of piles as reaction. The position of the cells were determined in consideration of ground conditions so that cells would be capable of mobilizing the full skin frictions for the upper and lower sections of the test piles. Skin frictions of piles were grossly underestimated when test programs were prepared and in many cases the cells were incapable of mobilizing the skin friction on piles to the full extent. The frictional resistance on piles was re-estimated and the positions of cells were adjusted in later stages based on the results obtained previously.

It has been a well known fact that workmanship has profound influences on bearing capacities of piles. Relaxation of geo-stress tends to reduce the strength of soil with time, it is thus desirable to cast piles as soon as possible. Furthermore, water tends to soften soils and usually bentonite is used for stabilizing the side wall. It has been found that better skin friction can be obtained if polymer is used to stabilize the holes. The use of polymer is getting more and more popular. Test results indicated that a 50 per cent increase in skin friction of sandy soils could be achieved if polymer is used and concrete is promptly cast through reduction in caking effect.

The end bearing capacities of piles are greatly affected by the construction method and quality of construction as well. Soft toe is a common problem for cast-in-situ piles. The problem of soft toe caused by sedimentation of soil at the bottom of excavated holes can be solved via injecting cement grout at high pressure in stages to strengthen the weak zone surrounding the pile toe. With pressure grouting at toe, it has been found that the end bearing capacities of piles are much improved and settlements of piles are much reduced as shown in Fig. 4. The effects of pressure grouting for piles embedded in clays are, however, less certain.

For ensuring the quality of construction and for confirming that the settlements of working piles are within required design limits, an average of 0.5 per cent of working piles were tested. Conventional static tests were carried out to loads of, up to, 2,200 tons. However, some of the contractors opted dynamic load tests or Statnamic load tests instead of conventional loading tests and performed comparative studies beforehand to establish correlations between the results obtained by using different methods.

3.4.2 Pile resistance in tension

To determine the tension resistance of piles, conventional tension tests were carried out individually on piles located neighboring those for compression tests. The results indicate that the ratio of ultimate tension capacities to compression capacities without end bearing ranging from 0.62 to 1.05 (62 to 105 per cent), with a mean of 81 per cent and a standard deviation of 16 per cent.

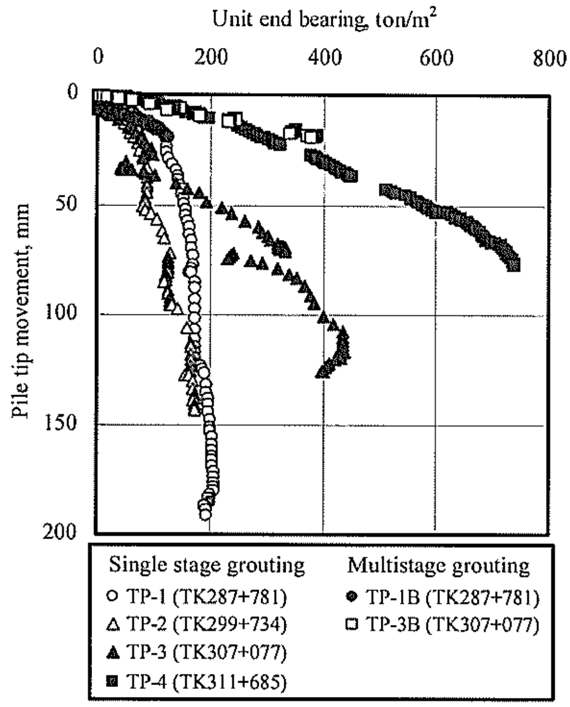


Fig. 4. Effects of the grouting on end bearing of capacity of piles.

These ratios are much greater than the limit of 40 per cent stipulated in the design specifications. It may be worth the effort to reconsider this limit if tension capacity is a governing factor in design of piles in the case due to seismic loads. If this limit of 40 per cent were raised to 50 or 60 per cent, considerable savings could be achieved. In such cases, the governing factor will then be compression, instead of tension, capacity of the piles. For river crossing bridges, the lengths of piles are generally dictated by the compression capacity due to existence of relatively thick overburden and consideration for scouring.

As mentioned in above paragraphs, Osterberg Cells were used in several cases for pile loading tests. In those tests, the upper sections of the test piles were subjected to upward loads at their bottom and practically behaved as tension piles. Therefore the skin friction mobilized tend to be lower than those obtained in the conventional loading tests. The ratios of ultimate tension capacities of piles from conventional tension tests to the compression capacities in tests using Osterberg Cells range from 1.0 to 1.14 (100 to 114 per cent).

3.4.3 Lateral pile load tests

Lateral pile load tests were conducted in ground with different soil conditions for determining horizontal modulus of subsoil reaction and to verify the design parameters. Horizontal displacements and lateral loads were measured by using displacement transducers and load cells, respectively. An inclinometer was also installed in each test pile to determine the profile of horizontal displacements along the pile during the test. Test results revealed that the responses of test piles are quite similar to those predicted by the computer programs LPILE as shown in Fig. 5. It is therefore concluded that the input soil parameters are reasonable and the program LPILE is a useful tool for predicting response of piles.

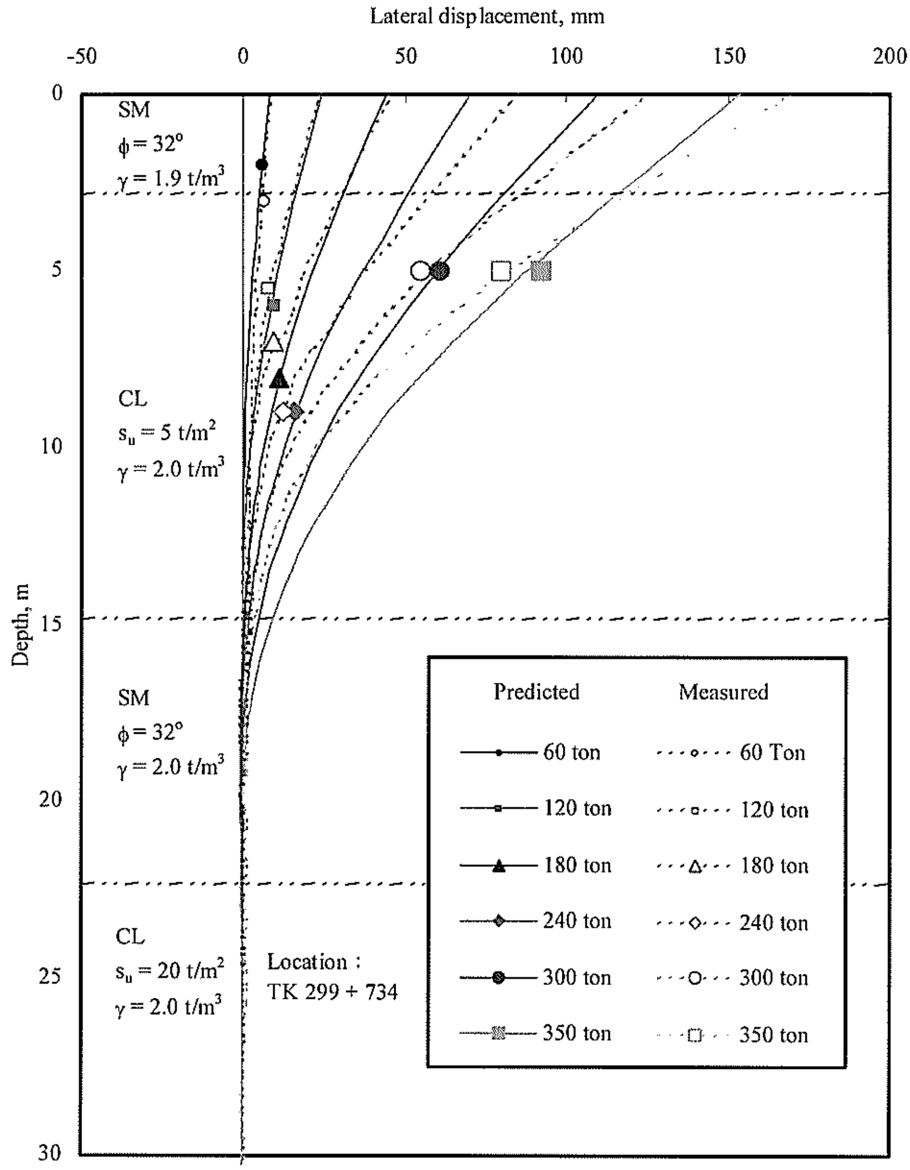


Fig. 5. Comparison of lateral pile load test results with predicted values from LPILE program.

4. SOIL LIQUEFACTION

During strong earthquakes, pore water pressure in saturated soils may increase due to the application of cyclic shear stresses induced by ground motions and cause reduction of effective stresses leading to partial or total loss of shear strength of soils. The soil deformation accompanied may be limited (cyclic mobility) or unlimited (liquefaction). According to the records of the Central Weather Bureau in Taiwan, there were at least 5 earthquakes in which soil liquefaction was observed in the southwestern part of Taiwan in the last century, therefore, liquefaction is a real threat. As the consequences of liquefaction, loss of foundation support, flow slides, slumping, lateral spreading, ground subsidence, etc., may occur. For the THSR project, contractors are specifically required to pay attention to liquefaction potential of soils and take it into consideration in foundation design. If liquefaction is indeed a potential threat, either the liquefiable soils be removed or piles be used to transfer loads from structures to a stable stratum.

4.1 Evaluation of Liquefaction Potential

Basically, liquefaction potential is expressed in terms of the ratio of soil resistance to the shearing stress induced. The average earthquake-induced shearing stress is usually estimated from the expected peak ground acceleration at the site using empirical correlations with earthquake magnitudes. Regarding soil resistance to cyclic loadings, it is normally determined either by laboratory tests, such as cyclic triaxial or simple shear tests (the so called analytical method of evaluation) or by in-situ tests such as standard penetration tests and cone penetration tests (the empirical method of evaluation).

The commonly used simplified methods for liquefaction evaluation in Taiwan are those proposed by Seed et al (1985), Tokimatsu & Yoshimi (1983), Japan Road Association (1996), and National Center for Earthquake Engineering Research, USA (Youd & et al, 2001). According to a study conducted by the National Center for Research on Earthquake Engineering, Taiwan for Chi-Chi Earthquake in 1999, the results obtained by using the method proposed by the National Center for Earthquake Engineering Research, USA in 1998 (Youd et al, 2001) gives better correlation with the field observations in comparison with others. However, for the THSR project, the method proposed by the Japan Road Association (1996) is adopted. This method has incorporated the experience from the Hyogoken-Nanbu Earthquake in 1995 and has the merits that reduction in soil strength as liquefaction develops is accounted for.

It is a well known fact that characteristics of ground motions at a site are affected by the proximity of the site to faults. Furthermore, the characteristics of faults are important to be considered in estimating ground motions. In the early days, studies were mostly based on data obtained in California where San Andreas Fault was responsible for several major jolts. Therefore, expressions for attenuation of ground motions established previously are more applicable to motions due to rupture of strike slip faults. It is only in recent years that ground motions generated by rupture of thrust faults drew attention from researchers. In earthquakes due to rupture of thrust fault, such as the Chi Chi Earthquake of 1999, ground motions on the two sides of the faults are drastically different. The THSR route crosses 3 major faults and ground motions near these faults could be significantly different from those far away from the faults.

During the design, the National Center for Research on Earthquake Engineering was engaged by the Designer to study ground motions for the section of the Rail route near the Hsinhua Fault. (Moh and Associates, 2001). Figure 6 shows the Boore-Joyner-Funel near field attenuation of ground motion with distance from the fault, as well as the Campbell's relationship using Taiwan's local strong motion data. Based on the results of this study, it is recommended that when the distance is less than 2.5km from the fault line, the near source effect should be considered. In other words, over a distance of 5km, the viaduct design shall be based on the near-field seismic hazard. However, the THSR specifications also refer to the use of the method proposed by the Japan Road Association (1996). The JRA method takes into account of the effect of in-situ stress which has unique properties of higher seismic intensity and lower number of cycles due to near source. Therefore, by using the JRA method, although the PGA is increased from 0.34 to 0.50g, liquefaction potential at the site in fact becomes even slightly lower.

It should be noted that notwithstanding the fact that soil liquefaction potential does not increase at locations near faults, the seismic loads on structures do become larger because of increased PGA. Taking the case of the Hsinhua Fault for example, for a bridge pier foundation consisting of 4 numbers of piles of 2.0m diameter, the designed pile length increases from 59m to 61m and the amount of reinforcing steel increases by 8 per cent when the near fault effect is considered.

4.2 Soil Properties Affected by Soil Liquefaction

Where potential for liquefaction exists under the excitation associated with either the Type I or Type II earthquakes, the liquefiable soils shall be removed, or soil improvement techniques be used, or deep foundations such as piles or caissons be used. The design should consider reduced soil strengths depending on the value of liquefaction resistance factor F_L and the depth of soil susceptible to liquefaction.

For piles, reduction in soil strengths will lead to decrease in bearing capacities. In the mean time, the reduced soil stiffness will induce larger lateral movements and larger bending moments when piles are subjected to lateral force due to earthquakes. Consequently, the pile length, reinforcement, and even the pile diameter have to be increased. This has significant effect on the pile design. The example of soil parametric study discussed in section 3.1 clearly illustrates such effects.

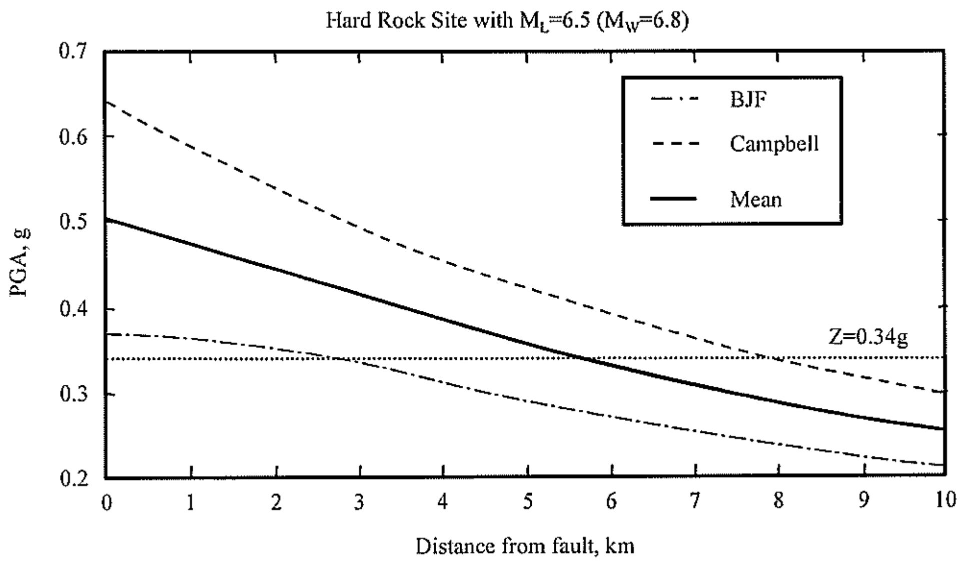


Fig. 6. Near-field ground motion attenuation with distance from fault.

4.3 Negative Skin Friction Caused by Soil Liquefaction

The design of piles needs to take into consideration the effects of negative skin friction resulting from dewatering or liquefaction of surrounding soils. Analysis should be carried out to compute ground subsidence caused by dissipation of excess pore pressure within liquefied soil layers after an earthquake and the resultant negative skin friction on piles. If negative skin friction were developed, it is considered as an addition to the working load as stipulated in the THSR design specifications. However, the Chinese Building Codes issued by the Ministry of Interior(1998a, b) stipulate that negative skin friction due to liquefaction is considered as short-term forces and is not necessarily to be combined together with other short term forces, such as wind loads, impacts, and traffic loads, at the same time. Therefore, checks are undertaken separately. In general, it is found that for the THSR Project, negative skin friction does not govern the pile design.

4.4 Flow Slide

According to the THSR design specifications, it is necessary to study the potential influence of flow slide caused by soil liquefaction. Based on Seismic Design Specification for Railway Bridges issued by the Ministry of Transportation and Communications (1999), and codes by Japan Road Association (1996), flow slide should be considered if one of the following situations is encountered:

(a). The differential height between the bottom of the sea (river in the case for the Rail) and seashore (or riverbank) revetment exceeds 5m.

(b). The thickness of liquefiable soil within 100 m from seashore (riverbank in the case for the Rail) exceeds 5 m and the liquefaction soil is continuous.

Because most rivers along the route have more than 5m of height difference between the river bottom and adjacent revetment, effects of flow slide due to soil liquefaction have to be considered in design of the bridge foundations.

The dynamic behavior of pile foundations in liquefiable zone is very complicated because liquefaction advances with time during earthquakes. In the beginning of the earthquake, both reactions and displacements of the pile foundation are mainly controlled or forced by the inertia force transferred from the superstructure. Once after the peak of earthquake, the soil may have already been liquefied and even the phenomenon of flow slide of stratification may start to occur. During this phase, the inertial force transferred from the superstructure is possibly decreased due to the shock absorption effect of soil liquefaction. Displacement of the foundation caused by cyclic seismic loading will obviously increase due to weakening soil condition. After the earthquake stops, the loads on the pile foundations are mainly those due to permanent displacement caused by the flow slide.

From the above recounted interactions between soil and structure under liquefaction of stratification caused by seismic action, it is understandable that different time frame exists between the development of flow slide and occurrence of the peak of earthquake. Therefore, the effect of flow slide does not have to be included or considered as a case in the earthquake forces in order to prevent being over-conservative in design.

According to both seismic design specifications mentioned above, the forces acting on piles and the resultant displacements should be checked for the following three conditions:

(a). no liquefaction or flow slide

(b). only liquefaction occurs

(c). only flow slide occurs

After investigating designs in several cases in the Rail Project, it is found that the forces produced by flow slide are relatively small as compared to the seismic forces. However, when the range and depth of liquefaction are great and the liquefaction is continuous from shallow stratification, the amount of steel reinforcement in the pile head may need to be increased.

5. GROUND SUBSIDENCE

5.1 Ground Subsidence Caused by Dewatering

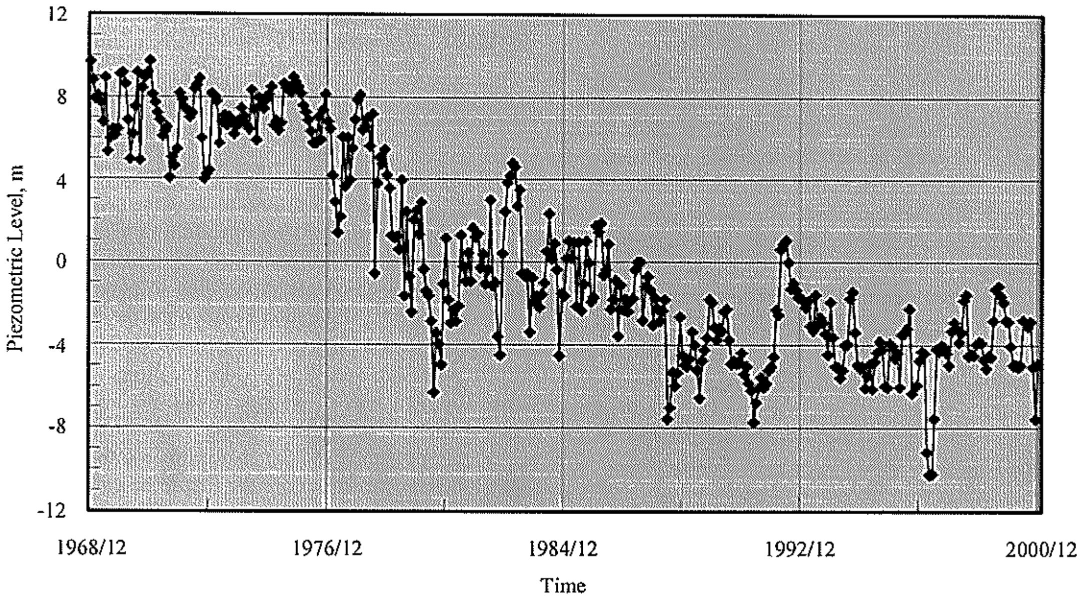
According to a report published by the Water Resources Planning Commission (WRPC) of Taiwan, there were once 200,000 pumping wells in operation on the island, drawing 7.3 billion tons of water from the ground annually, of which 44 per cent was utilized for irrigation, 33 per cent for fish farming, and the remaining 23 per cent for industrial and domestic use. A total of 1,170 sq. km of land has experienced ground subsidences varying from 0.5m to 2.5m. Although dewatering of groundwater by means of pumping wells obviously leads to serious land subsidence along the southwestern shoreline of Taiwan, the influence of land subsidence on the THSR project may be small since the route is located at least 10 km away from the seashore. Notwithstanding, further investigation and exploration are required due to the great significance of the THSR project and the importance of the structures.

Along the THSR route, ground subsidence is the most serious in Yunlin area which is located to the south of the Choshui Creek because of excessive pumping for irrigation and for fish farming. There are no reservoirs in the area, groundwater is the sole source of water supply. The area is generally underlain by thick layers of clay, consolidation settlements become eminent. Figure 7 (Water Conservancy Agency, 2001) shows groundwater levels along the THSR route at Kecuo in the Yunlin area. It can be obviously noted that the underground water level in the area has dropped by 15 m since 1968. Figure 8 (Water Resources Bureau, 2000) shows the accumulative ground subsidence in the two years period of October 1990 to September 2001 in the Yunlin area along the THSR route and the drawdowns of groundwater level in the period from 1968 to 2000. It can be noted that the total land subsidence in the 11- year period vary from 20 cm to 100 cm.

From the data mentioned above, the average subsidence rate is more than 2cm/yr in the Yunlin area in the past ten years. Based on this fact, the influence of land subsidence on pile foundation was studied for estimating the influence of negative skin friction on piles. Furthermore, differential settlements of the bridge piers were checked in order to make sure that design specifications are met. A special study was carried out to evaluate the long term effect of land subsidence on structures along the THSR in the section south of the Chosui River (Moh and Associates, 2002).

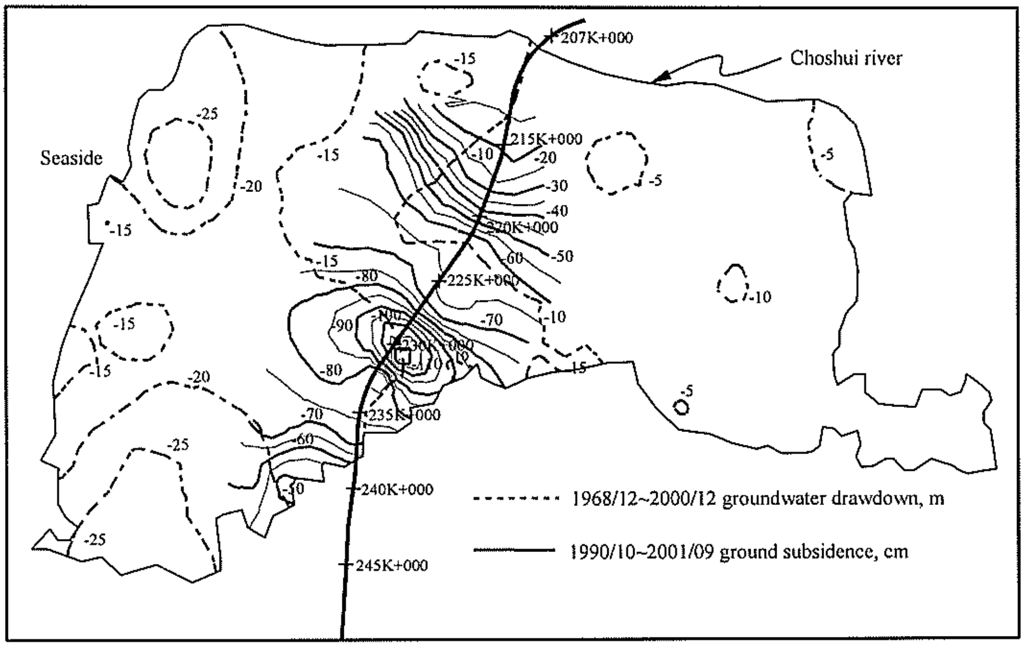
5.2 Effect of Ground Subsidence on THSR Structures

Assessment of potential effects of land subsidence on the safety of THSR structures includes:(a) bridge foundations, (b) vertical alignment and (c) negative skin friction on foundation piles. According to the design specifications, allowable angular distortion between any two points along the THSR should not exceed 1/1,000. Based on measurements carried out by the THSR Corp. in 1998-2000, the estimated angular distortions, except at locations near St. TK245+000, were all below the allowable value.



(Data from Water Converancy Agency, MOEA (2001))

Fig. 7. Monitoring results of observation wells along the THSR route at Kecuo.



(Data form Wtaer Resources Bureau, MOEA (2000) and THSR Corporation (2001))

Fig. 8. Observed ground subsidence and drawdown of groundwater level along the THSR route in Yunlin area.

The THSR design specifications also stipulate that "...Vertical Curves need not be provided in the structure or embankment if the difference in gradient is less than 1.0 per cent in full-speed sections, or if the calculated vertical curves mid ordinate is less than 10mm in lower-speed sections...". Since the calculated angular distortions along the THSR route at the present are all less than the specified limit, no adjustment of the vertical alignment appears to be needed.

According to the design standards for Japanese National Railways, design of foundation piles should consider effect of negative skin friction when the rate of land subsidence exceeds 2cm/year. When the subsidence rate is more than 4cm/year, 100 per cent negative skin friction should be considered.

For the section of the THSR considered, between TK218+000 and TK+237+000, the average rate of land subsidence is between 0.5 and 16.0 cm/year, development of negative skin friction is a potential problem. However, it is important to identify the source of land subsidence caused by withdrawal of groundwater. If the land subsidence as measured by the settlement of ground surface, is caused by consolidation of deep soil strata, negative skin friction may not need to be considered. Results of analysis indicate that about 30 per cent of the consolidation settlement occurred within the depth of most of the piles and it is less than 2cm/year. It is therefore concluded that influence of negative skin friction due to ground subsidence on bearing capacity of pile foundation need not be considered.

Analysis has also been conducted to determine whether the differential settlement are within tolerable allowances. Based on the observational data and results of analyses, it has been found that if excessive pumping is continuing for a long period to come, the safety of THSR operation may be endangered. Pumping thus has to be strictly regulated and monitoring of groundwater table is crucial for ensuring that the situation will not deteriorate beyond control.

6. FOUNDATIONS IN RIVERS

The Rail crosses several major rivers along its route and design specifications require that the safety of foundation shall be checked against scoring of riverbed. These rivers usually are dry but are quickly flooded in heavy rains. The annual rainfall on the island is more than 2000mm and a significant portion of this amount is caused by a number of typhoons in summers. Downpours frequently resulted in landslides and debris flow in mountains and some of them were fatal with heavy casualties. These rivers are very short with large gradients. As the water rushes toward the sea through plains, it scours riverbed and bridge foundations and quite a few bridges collapsed in the past as a result.

Scouring is a dynamic phenomenon and may occur as a gradual event with predictable amount annually or as a rapid event with sudden formation of large cavities under bridge piers within hours. Furthermore, how the materials will shift from place to place will depend on the configuration of bridge foundations and the terrain of the ground and is rather difficult to predict. Take the Tsengwen Creek as an example, a long-term scouring depth of 0.6m is expected and depth of local scouring is expected to be 5.41m. Furthermore, a depth of 2m is expected for temporary shifting of the river bed.

According to specifications issued by the Water Conservancy Agency of the Ministry of Economic Affairs (2001a) and the design specifications for the THSR project, structures should be founded at depth below the lowest point along the cross-section of the river and below the depth of long-term scouring at the location of the foundation, whichever is lower. This requirement is reasonable in normal cases but appears to be much too conservative if it is combined with requirements for other temporary conditions, such as earthquakes. It is also stipulated in the design specification for the project that the depth of scouring to be considered is only half of its value during earthquakes. Similarly, during earthquakes, a return period of 1 year of flow is adopted in determining the hydrodynamic loads on foundations of bridge piers, instead of a return period of 200 years for major rivers and 100 years for minor rivers for cases without earthquakes.

The structural design of substructure systems shall take both the existing ground elevation and the planned riverbed elevation after scouring into consideration. When pier foundations are constructed in water, it is only necessary to bury the foundations below the riverbed by 0.6m if protective measures are taken against scouring. However, such protective measures must not raise the water to a level which would cause concerns on safety of flood control works or the ecological environment of the river.

7. CONCLUDING REMARKS

The Taiwan High Speed Rail Project is a mega size infrastructure development in terms of investment, construction size and complexity. In order to meet the operation time target, the Project is being carried out on fast track turn-key basis. In other words, design and construction are proceeding in parallel. This requires extremely close coordination between the designers and constructors. Due to the high operation speed, safety becomes the top concern. Design Specifications for the project are therefore much more stringent than conventional highway and railway development.

In this project, just like many other infrastructure projects, foundations of structures are the first to be constructed which often dictates the entire time schedule of the construction. Under the extremely tight schedule of the THSR, and the variable complex geological conditions along the route, the tasks for producing an economical, safe and timely design are great challenges to the geotechnical engineers. In this paper, some of the major geotechnical issues in the design and construction of viaducts of the project are described and discussed, including soil characterization, foundation type, soil liquefaction, ground subsidence, and negative skin friction, near fault effect and scouring of river crossing bridge foundation.

For ensuring safety and economy of the project, pile load tests are essential for verification of the design assumptions and selection of parameters in the design. In the THSR Project, the specifications require check borings to be carried out at every pier location. Furthermore, it requires the verification of soil bearing stratum in comparing with the design condition by the Designer as well as Independent Checking Engineer. All these procedures ensure the construction quality of this BOT project.

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SITE INVESTIGATION AND GEOTECHNICAL FAILURES

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Abstract

Site investigation is normally required and carried out prior to the commencement of design of a construction project. Due to lack of or inadequacy of guide/code requirement regarding the extent as well as quality of site investigation work, geotechnical failures often occurred. These failures sometime led to catastrophic disaster and imposed serious threat to public safety. This paper gives a brief review of the code requirements and current practice in several countries in the region, including Singapore, Malaysia, Hong Kong, Taiwan and China. Three case histories are presented to illustrate the problems of poor ground investigation, inadequate knowledge of site condition, and importance of geotechnical supervision.

Keywords: Site Investigation, Geotechnical Failure, Quality Control, Code.

Introduction

For any construction project, a site investigation is normally carried out. Site investigation has been defined as investigation of the physical characteristics of the site and includes documentary studies, site surveys and ground investigation. The last item refers to the actual surface or subsurface investigation, including on site and laboratory tests. In broad sense, site investigation should also include study of the site history and environment, interpretation and analyses of all available data, and making recommendations on the favorable/unfavorable locations, economic and safe design, and prediction of potential risks. A proper site investigation work is carried out by the combined effort of grotechnical engineer and ground investigation contractor.

In any site investigation work, the questions which should be resolved in determining the investigation program are: (1) what type of investigation is needed, (2) why they are needed, (3) where the actual field work should be performed, and (4) how the work is to be done. Another question which one should always ask oneself is whether the investigation is sufficient or too much. From this program, the stages of investigation, the type of field work, the number and location of boreholes, type and number of sampling, type and number of testing, etc., are determined.

The ground investigation contractor is responsible for providing reliable factual data. The geotechnical consultant should be responsible for the planning & execution of the investigation program, interpretation and analyses of results, and making appropriate design recommendations, to avoid over design as well as unsafe design. In order to ensure the quality and reliability of the investigation results, full time supervision on site by qualified geotechnical engineer is a must for any ground investigation work.

Proper expertise for geotechnical engineering solutions and recommendations should be referred to by engineers in other disciplines, and not rely on the opinions of the ground investigation contractors alone. The division between the investigation contractor and geotechnical consultant, in terms of professional judgment and responsibility must be clearly defined.

In the following sections of this paper, a brief review is made on the code/regulations and current practice of site investigation in several countries in the region, including Singapore, Malaysia, Hong Kong, Taiwan and China (PRC). Three case histories are presented to illustrate geotechnical failures attributable to poor ground investigation, inadequate knowledge of site condition, and lack of geotechnical supervision.

Codes and Regulations

In this section, a brief review is made regarding codes and regulations on site investigation currently in practice in several countries in the region, including Malaysia, Singapore, Hong Kong, Taiwan and China (Peoples' Republic of China). A summary of the requirements is shown in Table 1.

Table 1. Summary on Code/Regulation Requirement for Site Investigation

Items / Country	Singapore	Malaysia	Hong Kong	Taiwan	China
Site Investigation Code/Regulations	No	No	Yes	Yes	Yes
SI Contractor Special Registration	Yes	No	Yes	No	Yes
SI Contractor Personnel Requirement	No	No	Yes	No	Yes
Site Supervision by Geotechnical Consultant	No	No	Yes	No*	Yes
Professional Geotechnical Engineer	No	No	Yes	Yes	Yes
SI Report Endorsed by P.E.	Yes	Yes	Yes	Yes	Yes

*Not required, but often practiced by major geotechnical consulting firms.

Singapore

There is no building codes or regulations regarding site investigations in Singapore. Clause 31 of Building Control Regulations says: "Where foundations or related earthworks are proposed on any premises, an investigation of the site shall be undertaken by the qualified person". This is in fact not followed by many PEs in the local practice as the works are usually handled by the drilling contractors. This clause is currently being discussed between the Association of Consulting Engineers Singapore (ACES) and the Building and Construction Authority (BCA) as many PEs feel not comfortable for being held responsible for the accuracy/validity of the SI report prepared by others under this clause. Recently, Advisory Notes have been issued by the BCA to PEs as guidelines for SI works. Advisory Note 1/03 gives advices on SI work for building structure of 10-storey or more. They include: (i) the number of boreholes shall be the greater of one hole per 300 sq.m or one hole at 10 to 30 m c to c, and not less than 3 number in a site; (ii) the depth of borehole should be minimum of 5 m into N>100 layer or 3 D beyond the design founding level of piles.

In Singapore, geotechnical engineering is not a discipline for PE license. There is neither any special recognition of qualified geotechnical firm by the Professional Engineers' Board (PEB) or any other relevant government authorities. On the other hand, drilling contractors are required to register with the Construction Industry Development Board (CIDB), with little or no control on the professional level knowledge.

Current practice in selecting SI organization can be classified into Public Projects and Private Projects. For Public Projects, SI works are usually carried out by invitation or open tender. Only CIDB registered contractors are qualified for submission and award of contract is generally based on price. A number of "qualified" and experienced geotechnical consulting firms are unfortunately being excluded unless they are willing to be treated or considered as a "contractor". The practice of award purely based on low price is definitely a problem since this system would exclude quality work.

For Private Projects, selection of SI works firm depends very much on the understanding of problems of the Architect/Engineers. Generally, three types of SI invitations are in practice: (i) drilling contractors only, generally adopted by small Architects/Consulting Engineers, (ii) drilling contractors and geotechnical consultants, and (iii) geotechnical consultants only. Usually (iii) is preferred by large architects/consulting firms. When invitation is issued to both drilling contractors and geotechnical consultants, cost comparison becomes an improper factor because these two categories of SI firms are not on the same basis, just like comparing pharmacy with medical practitioner, or book keeping company with accounting/audit firm (CPA). In many cases, the owners' (or their representatives') influence to the selection is significant where architects/engineers' ability to convince the owner regarding quality work becomes important.

Malaysia

There is no specific codes or regulations in Malaysia regarding SI requirements. Although many of the principles for carrying out proper SI works have been discussed in details but very few SI work really followed the recommendations (IEM, 1997). The current practice in Malaysia in selecting SI contractors depends a great deal on the consulting engineer (i.e. the architect/structural consultant). Reputable consulting engineer will normally select SI contractor on the basis of qualification, merits and reliability but cost often plays an important factor. Majority of the consultants, however, select SI contractor purely on the basis of cost. In addition, there is one practice in

Malaysia which is of great concern is that the architect/structural consultant would leave it to SI contractor to propose the SI program, leading to many occasions with insufficient information or even wrong information. In most of the cases, there is no supervision, or best by technicians, of the site work.

After the Highland Incident in Kuala Lumpur in 1993, the government initiated a second opinion or independent checking system on geotechnical reports submitted by consultants. However, there is no strict requirements or regulations on this, but left to the discretion of each local council. Some states, for example Penang, has an Ad Hoc Committee on Hill Land Development vested with authorities of approving the geotechnical study report and the second opinion report. Recently, second opinion report will also be required in some states on geotechnical study report for buildings with basement excavation.

Hong Kong

Hong Kong is probably one of the few countries in the world who has extensive and detailed regulations and code requirements for the practice of geotechnical engineering including SI work. Hong Kong Building (Construction) Regulation (Hong Kong Government, 1990) specifies, "where foundations are proposed to be constructed, an investigation of the site shall be undertaken to establish, to the satisfaction of the Building Authority, the type and character of the ground on which the foundations are to be placed. Site investigation shall be carried out in such a manner and to such recognized standards as to provide adequate geotechnical and other relevant data for design and construction of building/foundation works".

PNAP 132 (Hong Kong Government, 2000) further stipulates that "site investigation" is defined as "investigation of the physical characteristics of the site and includes documentary studies, site surveys and ground investigation". Ground investigation refers to the actual surface and subsurface investigation and are required to be carried out by registered specialist contractor under ground investigation field works category [RSC (GIFW)]. The RSC (GIFW) organization shall have the following personnel with academic qualifications and work experience specified (PNAP 244, Hong Kong Government, 2000):

(i) Authorized Signatory to act for the contractor.
(ii) Technical Director if the contractor is a corporation.
(iii) Geologist or geotechnical engineer to carry out logging and preparation of borehole logs.
(iv) Technical competent person to supervise on site the different stages of GIFW.
(v) Geotechnical field technician to provide full time on-site supervision.

For private projects, selection of SI organization is usually based on cost only with little reference to company reputation. For government projects, technical qualification, capability, resources available and past performance are also taken into consideration in addition to cost. In Hong Kong, the ground investigation contractor usually only prepares factual report. Geotechnical consultants are generally engaged by Authorized Person

(Architect or Engineer) to provide adequate site supervision and to prepare geotechnical report. PNAP 132 has detailed regulations on the level of supervisions to be provided.

Geoguide 2 published by the Geotechnical Engineering Office (Hong Kong Government, 1987) present recommended standards of good practice for site investigation in Hong Kong. Site investigation carried out with the recommendations of Geoguide 2 is deemed to meet the minimum acceptable standards. The current ground investigation requirements/practice in Hong Kong are summarized in Table 2.

Table 2. Current GI Requirements/Practice in Hong Kong

Foundation Type	Borehole Spacing	Borehole Depth
Percussion Pile (a) H-Pile (b) Concrete Pile	 15 m – 20 m 15 m – 20 m	In-situ weathered soil with $N \geq 200$ $N \geq 80$
Mini-Pile, H-Pile Socketting into Rock	10 m	At least 5 m below rock head of specified grade or designed length of the rock socket of the nearest pile, whichever is the deeper[(a)]
Bored Piles Founded on Rock	One borehole at each Pile location	At least 5 m below rock head of specified grade[(b)]

Notes:

(a) For area underlain by marble, ground investigation is recommended to be carried out in stages. It must be adequate to ascertain whether marble with cavities exist beneath the site.

(b) If marble is encountered, a minimum penetration of 20 m of the drillhole into sound marble rock is recommended. Where cavities are encountered in the hole being drilled or in adequate drillholes, increased penetration is necessary.

Taiwan

In Taiwan, the Building Design Code (Ministry of Interior, 2001) revised in year 2001 has relatively simple requirements on the extent of ground investigation. The code says: "the ground investigation program should be planned in such away by considering the stages of construction plan, complexity of the site, type of building construction including it's size and importance". In principle, at least one borehole shall be drilled for every 600 sq. m of site area or every 300 sq. m of building area. For every site, the minimum number of boreholes should be two. For development on large sites, when the site area is large than 6,000 sq. m or building area in excess of 3,000 sq. m, the number of boreholes to be drilled may be adjusted according to geological condition of the site and the structural requirements of the building.

The guidelines are provided for depth of boreholes as summarized in Table 3.

Table 3. Current Requirements for Boring Depth for Building Construction in Taiwan

Foundation Type	Minimum Borehole Depth, m
Shallow Foundation	bearing layer or 4 times footing width below foundation level
Pile	bearing layer or 4 times pile diameter below pile tip
Caisson	bearing layer or 3 times caisson diameter or width below bottom of caisson
Floating Foundation	depth where the vertical load increase <10% effective vertical overburden
Deep Excavation	1.5 to 2.5 times depth of excavation or bearing stratum or impermeable layer

For highway bridge design, the code (Ministry of Transportation and Communications, 2001) specifies the minimum number of boreholes as well as depths for ground investigation as summarized in Table 4.

Table 4. Ground Investigation Requirements for Highway Bridge in Taiwan

(A) Borehole numbers

	Minimum Number of Boreholes
Each bridge	2
Each structure element of substructure	1 2(when width >30m)
Simple and uniform geological condition	1 every 100 m

(B) Borehole depths

Foundation type	Minimum Borehole Depth
Spread Foundation	$2 \times B$ when $L \leq 2B$ $4 \times B$ when $L > 5B$
Deep Foundation	Estimated bottom depth of foundation or 3m into rock

Geotechnical site investigations in Taiwan are usually carried out by geotechnical consultants. The executions of ground investigation are performed by ground investigation contractors. Geotechnical reports are normally required to be endorsed by Registered Professional Geotechnical Engineer or Civil Engineer. For public infrastructure works, geotechnical investigation is often a part of the overall study or design. Selection of the consultants is normally made on the basis of two envelopes system, i.e. work proposal & qualifications in one envelope and cost proposal in a separate envelope. In majority of cases, selection is made on the basis of first envelope and price negotiation is made with the first ranked party. Recently, a number of public projects adopt the policy of pronouncing the price before the selection process.

For private projects of significant size, geotechnical consultants are often engaged by the Architect/Engineer. Cost element usually becomes the decisive factor.

China (The Peoples' Republic of China)

After eight years' intensive study, evaluation and discussion among geotechnical specialists, construction industries and government officers, the government of the PRC in March 1994 issued a very comprehensive Code for Investigation of Geotechnical Engineering as national standards (Ministry of Construction, 1994). The code contains 13 chapters and 17 appendices. It not only sets requirements for site investigation, but also gives specifications and procedures for analyses, calculations and verifications. In view of the vast geographic territory, complex geological conditions and variable nature of the construction projects which need site investigation, the code divides site investigation work into three levels on the basis of safety, site condition, and complexity of geotechnical condition. The extents of investigation required for each level are different.

The code also gives detailed requirements for site investigation in different types of soil/rock formations and for different categories of construction. Parts of the code were further revised in 2001. The codes in fact are so detail and rigid which leave little room for a geotechnical engineer to exercise his knowledge and judgment.

Up to the present, all the site investigation works are carried out by state-owned Survey and Investigation Design Institutes. There are three different levels of Institutes qualified on the basis of personnel qualifications, experience and facilities. Possibly due to the rigid code requirements, many site investigation reports, except for large infrastructure projects, only contain description of the ground investigation. Design recommendations are often made by just quoting the code without actual analyses.

Under the PRC's planning economy system, site investigation works normally were assigned to the investigation institute under standard fees (Ministry of Construction, 2002). With the promotion of market economy, the investigation and design institutes are encouraged to be commercialized and privatized. Competition by open tender based on price is gradually taking place. However, the standard fee structure is still in effect, and theoretically it should be enforced. During this transition period, the market is somewhat confusing and quality of work will undoubtedly be affected.

Cases of Geotechnical Failure

<u>*Case One*</u>

The site

On 18 August 1997, a major landslip occurred in the Lincoln Community during Typhoon Winnie where 28 lives were lost. The community was a relatively high density residential development on slopeland in Taipei Country. According to report prepared after the disaster (Chen et al, 1998), the topography of the site before the landslip has relatively steep slopes in the northwest and east part of the site at slope angle of about 30°. The southwest and southern parts of the site were at lower elevation. Maximum difference in elevation was about 75m. During the site inspection after the landslip, it was observed that the direction of the upper sliding surface was N73°~84°E with dip angle of 28°~29°S. The direction of the lower sliding surface was N79°~81°E with dip angle of 29°~30°S. The inclination of the slopes was about the same as the dip angle of the rock formation.

From the outcrops exposed after the landslip, the geological structure at the site is primarily composed of alternative stratum of sandstone and shale. The sandstone in the upper strata was weathered with well developed joints. The sliding surfaces were relatively smooth shale.

The sliding area was rectangular in shape, about 140m in the E-W direction and 50m in the S-N direction. Figure 1 shows the extent of the landslip, displacement and distribution of tension cracks.

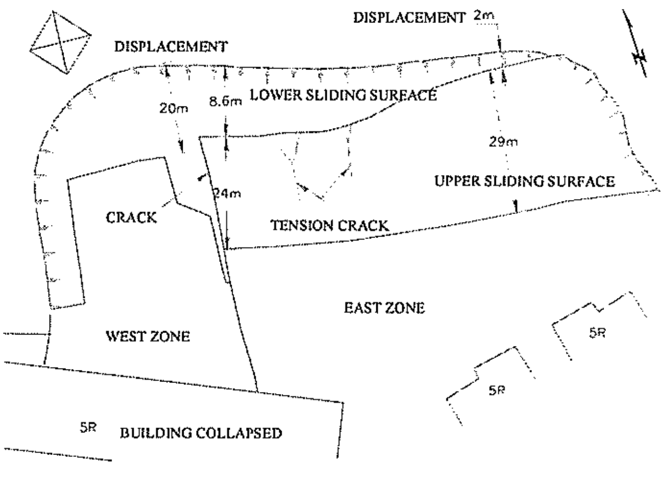

Fig.1 Extent of Landslide and Displacement, Case One

The cause

Extensive investigation and evaluation were carried out after the incident, including testing of concrete cores taken from the R.C. retaining wall, additional soil boring and testing, review of the design and construction records. The conclusions included:

(1) Inaccurate site investigation report—The site investigation report did not indicate the presence of shale.

(2) Improper selection of geotechnical design parameters—The design did not give due considerations of effect of groundwater.

(3) Poor construction quality—Construction of the retaining structure was not properly supervised by qualified person for quality as well as quantity control.

<u>*Case Two*</u>

The site

The case is the construction of one of the underground stations of the Taipei Mass Rapid Transit Systems. Excessive settlements and tilting of adjacent structures occurred during construction of the guide trench of the diaphragm wall for one of the station exit. As shown in Figs. 2&3, there are two 3-storey building, i.e. SR-096 and SR-097, located immediately adjacent to the station Exit B. These two buildings were founded on R.C. spread footings. In view of the fragile nature of the structure and foundation, micropiles were installed to reinforce the building support and pressure grouting were made on both sides of the guide trench to a depth of about G.L.-12m prior to the excavation work.

The site area of the station is 3,750 sq.m. A total of 7 boreholes were drilled at the site during the design stage according to the Code for Building Foundations, i.e. one borehole for every 600 sq.m. Additional boreholes were also drilled during construction. In both cases, the data were quite consistent, the subsoil consists primarily of sandy soil with some gravel from ground surface to a depth of about 2.5m and followed by clay (CL) layer. Nothing unusual was revealed by the boring data.

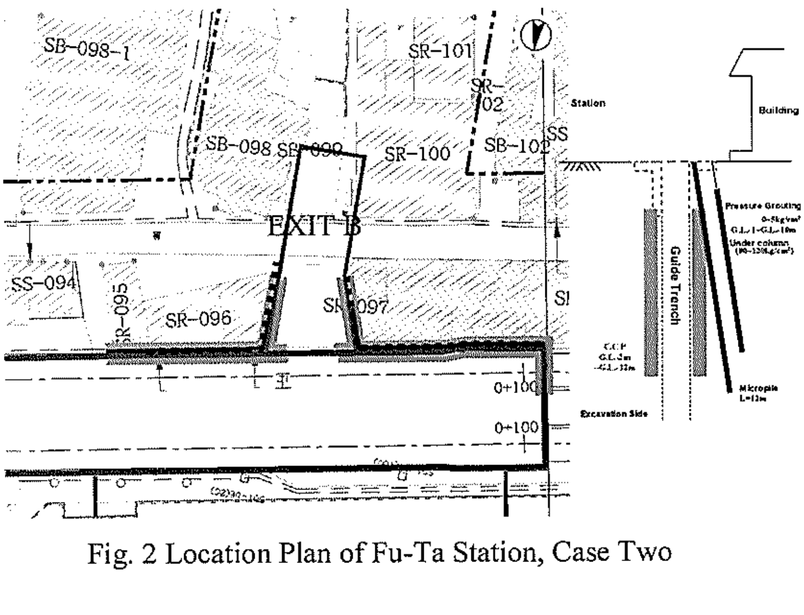

Fig. 2 Location Plan of Fu-Ta Station, Case Two

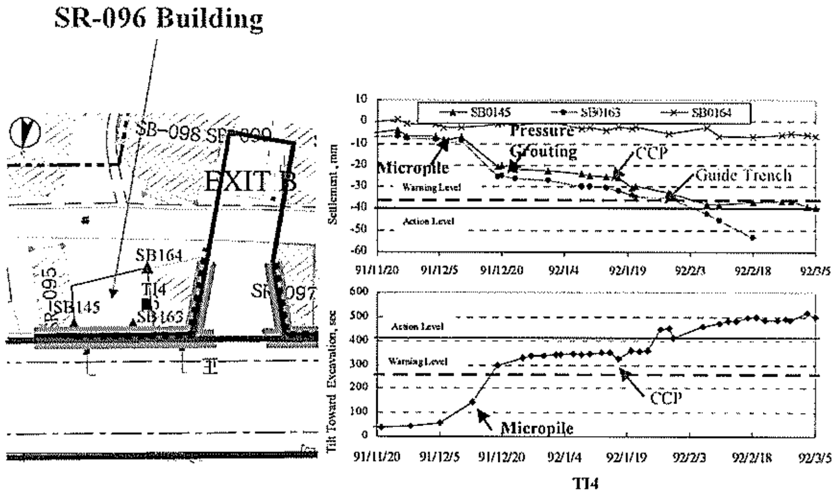

Fig. 3 Instrumentation data for Building SR-096, Case Two

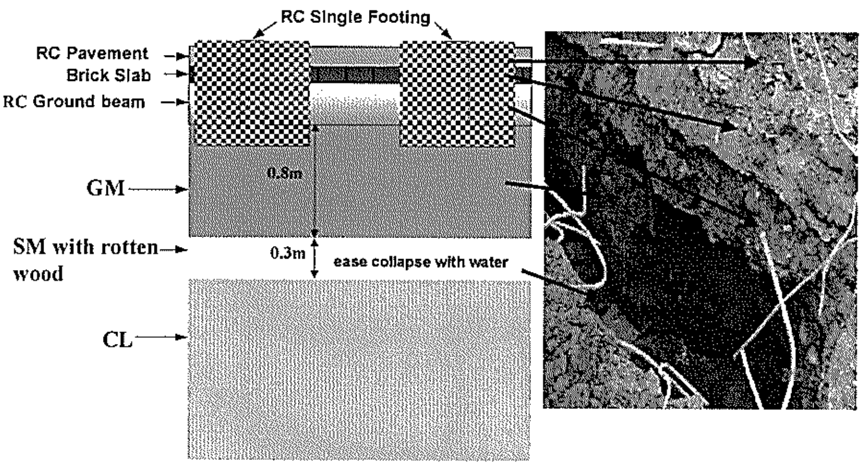

Fig. 4 Subsurface Condition Below Building SR-097, Case Two

The cause

During test excavation of the area after demolishing of the tilted buildings, it was found that there was a thin layer (about 30cm) of sandy soil containing decayed woods within the upper sandy soil stratum as shown in Fig. 4. This compressible soil was not discovered during the site investigation even if the number of boreholes drilled was in accordance to the code requirement. However, if information were gathered from neighboring occupants during the SI work, continuous sampling of the upper subsoil payer would certainly reveal the existence of such unfavorable subsoil condition.

<u>Case Three</u>

The site

Described below is a case history in which the replacement of a faulty piezometer without supervision by qualified geotechnical engineer led to ingress of water at the bottom of a deep excavation.

The excavation of interest was for construction of an underground station of the Taipei Rapid Transit Systems. It is located in central Taipei City. The subsoil distribution is typical of that in the Taipei Basin (Moh and Ou, 1979), with a thick layer of young sediments i.e. the so-called Sungshan Formation from the ground surface to a depth of about 48. The Sungshan Formation comprises 6 alternative layers of silty sands (SM/ML) and silty clays (CL/ML). Underneath the Sungshan Formation is a highly permeable gravel layer, i.e. the so-called Chingmei Gravels. This gravel layer was in artesian condition decades ago and the piezometric level in this layer had dropped to as low as RL 60m as result of excessive pumping before the 70's. The clayey sublayers in the Sungshan Formation are relatively impermeable and separate the entire subsurface soil stratum into three aquifers with different piezometric levels. The piezometric levels in the Sungshan Formation responded to the lowering and rising of the pressure heads in the Chingmei Gravels, there obviously are flows across neighboring aquifers. A typical subsoil profile is shown in Fig. 5.

The events

As shown in Figs. 5 and 6, the excavation was 23m in width and was carried out to a depth of 24.5m by using the bottom-up method. The pit was retained by diaphragm walls, 1.2m in thickness and 44m in length, and braced by 8 levels of temporary struts. The diaphragm walls toed in Sublayer II to provide a seepage cutoff. Sublayer II, which is an impervious layer consisting mainly of silty clay, essentially served as a seal at the bottom of soil plug which was enclosed by diaphragm walls on its four sides. With a length of soil plug of 20.5m, a factor of safety of 1.3 was obtained against blow-in for a piezometric head of 29.5m at the bottom of the plug. At the time when the incident occurred, the bottom of excavation had been reached and, except at the southern end where the incident occurred, base slab had already been cast.

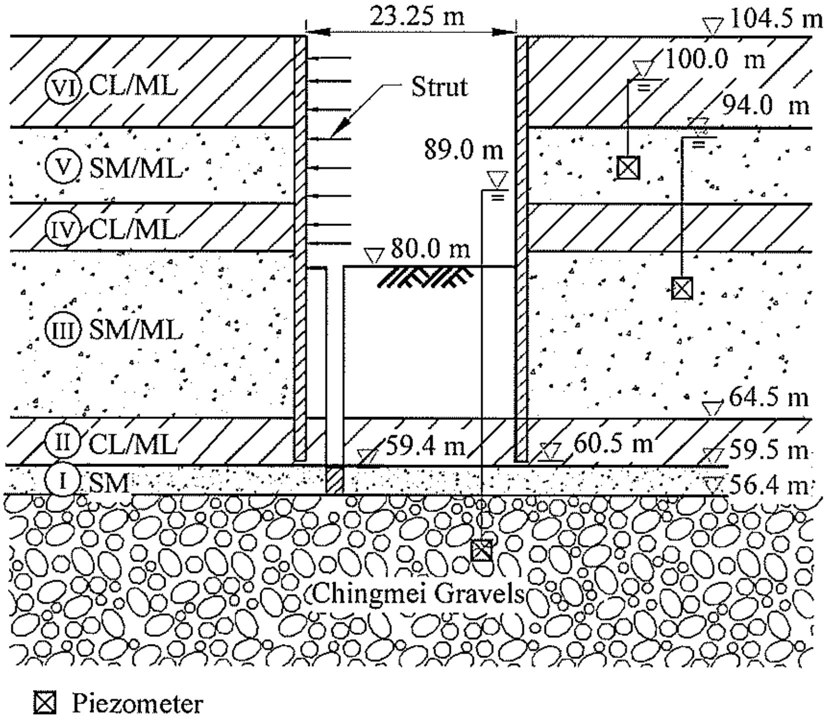

Fig. 5 Soil profile and configuration of excavation

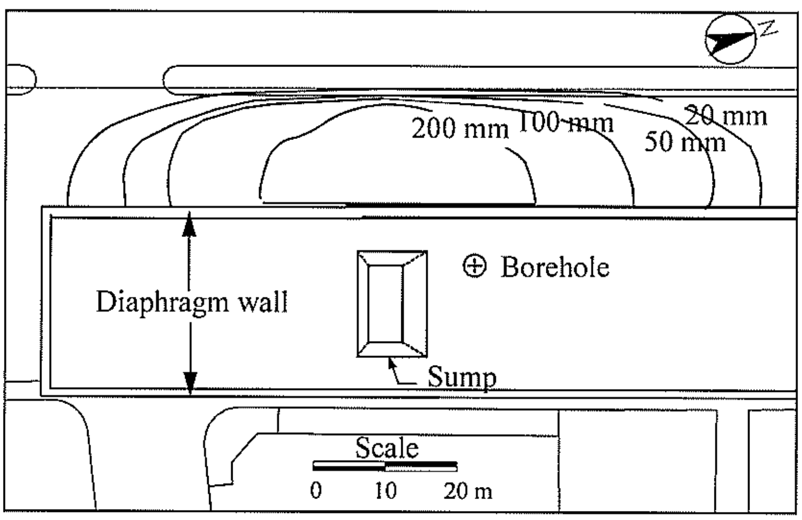

Fig. 6 Borehole, sump and settlement contour

The excavation was well instrumented with settlement markers, inclinometers, load cells, etc. Because blow-in was one of the major concerns, the piezometric levels in the Sungshan Formation and the Chingmei Gravels were closely monitored. One of the piezometers became faulty and the contractor attempted to replace it by a new one. At that time, the excavation at the southern end had already been completed and the bottom of excavation was protected by a layer of plain concrete. Drilling was carried out from the bottom of excavation at the location shown in Fig. 2. As drilling reached RL 59.4m, refer to Fig. 1, water started to overflow from the borehole.

Although various means have been tried, including extension of drilling casing, placing of sand bags on top of the borehole, grouting etc., the flow soon became uncontrollable and the pit had to be flooded to prevent the situation from deteriorating. As much as 70,000 tons of water was recharged to balance the hydrostatic pressure from the groundwater. It took 6 months to mend the damaged clay blanket under the bottom of excavation before the pit was drained and the works resumed. A total of about 3,000 cu m of LW (Labile Wasserglas) grout consisting of cement, sodium silicate and water and about 2,400 cu m of cement-bentonite grout was injected into the ground to fill up cavities made by the seepage flow. Figure 6 shows the settlement contour of the surrounding area. Effect of the incident extended to a distance more than 60m from the location of the borehole. Maximum ground settlement exceeded 250mm.

Conclusions

1. The code requirements for site investigation, if any, usually stipulate the minimum amount of site work required.
2. Site investigation is a specialized operation, requires specialized organizations and specialized personnel.
3. Site investigation is the combined product from ground investigation contractor and geotechnical consultant. The contractor is responsible for obtaining reliable data. The geotechnical consultant is responsible for planning and execution of the site investigation work, interpretation and analyses of data, recommendations of design and assumed professional responsibility.
4. The extent and cost of site investigation should be such that the risk is at an established acceptable level to the designer and also comply to the accepted code of practice.
5. The practice of recommending lowest tender as the main criteria for site investigation should not be preferred but be discouraged. Selection should be made on the basis of the geotechnical consultant's competency and investigation contractor's ability to provide reliable factual data.

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地鐵工程之風險評估

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摘要

風險管理在土木工程界尚在萌芽階段，有賴學術界及工程界共同推動。本文以地鐵工程為例介紹風險評估之概念及作法。由於統計資料欠缺，目前無法進行量化分析，僅能列舉地鐵工程的工程風險以及風險因子，以為日後發展風險管理體系之參考。

1. 序言

「風險管理」是一門新興學門，已廣泛應用在許多行業以規避可預期的風險。在土木工程界，風險管理已受到學術界重視，假以時日必能開花結果，成為一個實用工具，但目前仍受限於資料不足，風險評估模式迄未建立，不但妨礙工程保險制度的健全發展，也使得營造業面臨無人承保的困境。

由於可利用土地有限，而人口的膨脹壓力迄未中止，地下空間的開發與利用是時勢所趨，必成風潮。由於其中不確定因素太多，地下工程本來就是高風險施工項目，而無論是就規模或者就困難度而論，地鐵工程又是地下工程中之最，所以地鐵工程的風險管理尤為重要。在人口密集、交通擁擠、管線密布的市區進行地鐵工程，難度頗高。如果再加上地質軟弱的話，意外事故在所難免。世界各地的地鐵工程鮮有不出事者，一旦出事，生命財產之損失頗鉅，而因工程延宕所衍生的社會成本更是難以估計，所以及早建立風險管理之制度及發展合理的保險機制實屬當務之急。

2. 地鐵工程之風險管理

如圖1所示，「風險管理」的手段因行業之不同而不同，但大致可以劃分為「風險辨

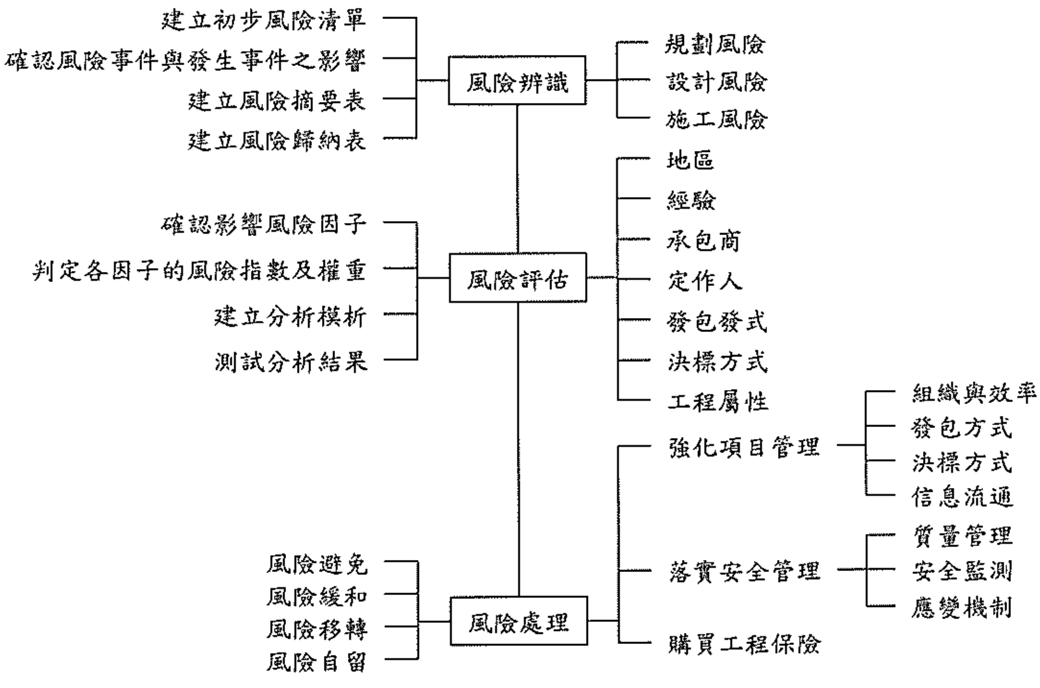

圖1 地鐵工程風險管理體系

識」、「風險評估」以及「風險處置」三個階段，循序漸進。考慮到造價、交通及土地徵收等因素，地鐵工程有地面、高架與地下路段之分。在市中心區，一般都是地下路段，在市中心區的外圍，一般都是高架路段，在偏遠地區，一般都是地面路段。在非常早期的地鐵系統，如紐約及芝加哥，在市中心尚有高架路段，現代的地鐵系統在市中心區採高架路段者已絕無僅有。除了妨礙市容之外，交通狀況亦不容許。

地面段及高架段的結構型式較為單純，即使是車站也是低矮建築，風險有限。地下段的結構則遠為複雜，除車站外，有隧道、通風豎井、轉轍段等等。這些結構一般位於地下15公尺至35公尺之間，甚至更深。地鐵工程之地面段、高架段與地下段之風險各有不同，而以地下段之風險為最高、高架段次之、地面段最低。而在地下段中，又因構築方式不同而風險不同，以盾構施工的風險最低，明挖施工次之，暗挖施工最高。

除工程本體的風險之外，鄰近地鐵工區的結構體的安全也十分重要，維生管線之安全更是不可忽視。在地下段施工中，有相當多的災變是因為地盤變位導致自來水水管破裂，在高壓之下，形成水柱沖刷土體，最終造成地陷。也有因瓦斯管破裂而造成氣爆或火災之情事。維生管線如受到破壞，除會危及工程主體之外，更會造成社會大眾的不便及不安，即使是停話、停電也會廣受責難，所以必須竭盡可能維持維生管之正常運作。

2.1 風險辨識

從工程專業生命週期的觀點來看，可將工程風險的發生依時間順序劃分為「開工前」、「施工中」以及「完工使用」等三個期間。如表1所示（郭斯傑、邱必洙，1999），各階段均面臨不同性質的風險。雖然該表並不能涵蓋所有的風險，而且也過於籠統，但已足以顯示工程風險之複雜性。

表1 工程風險之種類及各類風險之責任分擔 (郭斯傑、邱必洙，1999)

<table>
<tr><th colspan="2">工程進度</th><th>風險種類</th><th>風險承擔者</th></tr>
<tr><td rowspan="2">開工前</td><td>規劃階段</td><td>專業顧問的選擇不當；定作人對工程顧問為不當之指示；工址的選擇不良；地質的調查不足；測量與勘查不足；財務計畫不當；其他（政治因素、經濟因素、戰爭、核子）</td><td>業主風險</td></tr>
<tr><td>設計階段</td><td>疏忽或殆於注意；設計錯誤；施工規範遺漏或疏失；採用不適的施工方式，如：施工機具、工法選擇、未成熟或未經試驗之技術；承包商、材料供應商的選擇。</td><td>設計者風險</td></tr>
<tr><td rowspan="3">施工期間</td><td>施工處所及環境影響因素</td><td>自然環境：雨量、洪水、風速、颱風、山崩、地震、地下水、地形、地質。
人為環境：政治環境、風俗習慣、公共設施</td><td rowspan="3">施工廠商風險</td></tr>
<tr><td>工程技術</td><td>工期的延誤；新工法的使用；倒塌；材料瑕疵；動力設備或施工機具設備故障；土地下陷；地震。</td></tr>
<tr><td>人為因素</td><td>疏忽；欺騙或不忠實行為；施工計畫錯誤；工地管理不當；碰撞；火；竊盜；罷工、暴動、民眾騷擾。</td></tr>
<tr><td colspan="2">使用期間</td><td>安全性；耐久性；火災及各項災害防制</td><td>業主風險</td></tr>
</table>

在地鐵工程中，經常發生而可能索賠的破壞型式如表2所示。如前所述，地鐵工程施工中，絕大部分的工程事故是發生在地下段，而在地下段，除了少數結構物（如出入口、通風豎井）必須突出地面外，地鐵工程全在地下進行，所以其工程風險與地質狀況密切相關。而絕大多數的地質災害又與地下水相關，所以如何控制地下水使其不致造成災害便成降低地下工程工程風險的關鍵。

2.2 風險評估

影響工程風險的因子也就不勝枚舉，而且因工程性質不同而不同。不但如此，風險評估的方法及步驟也因其分析目的之不同而不同。業主、設計者、施工者、保險業者的目的不盡相同，所應納入評估的因子也就不同。舉例來說，保險業者必須將所有因子納入其評估模式作通盤考慮以訂定保費，但施工廠商則只要考慮工程屬性即可。

表3說明影響工程風險之主要因子及考慮因素，各因子間的關係有些是獨立的，但有些並非獨立，而是互相依賴的，這更大大地增加了評估的複雜性。在無任何評估模式可供參考的情況下，下式不失為一個可以考慮的建議：

$$I_R = C_1 C_2 C_3 C_4 C_5 C_6 C_7 \quad (1)$$

其中I_R（risk index）是工程綜合風險指數，C_1至C_7是影響工程風險主要因子的風險指數。至於與工程屬性相關的風險指數，C_7，可以下式表示之：

$$C_7 = \frac{R_1 L_1 + R_2 L_2 + R_c L_c + R_m L_m + R_s L_s}{L_1 + L_2 + (L_c + L_m + L_s)} \quad (2)$$

式(2)中之R_1及L_1分別表示地面段的風險指數及長度，R_2及L_2分別表示高架段的風險指數及長度，地下段又分明挖、暗挖及盾構三類工法，分別以c（cut），m（mining）及s（shield）代表之，其風險大不相同，分開估計較為合理。

表2 地鐵工程工程風險

	破壞部位	破壞型式
明挖工法	擋土系統	擋土牆漏水/破壞
		支撐系統破壞
	開挖面下方土體破壞	隆起
		上舉
		管湧
		砂湧
盾構工法	鏡面	湧水以致地陷
	上方地表	沉降而損及鄰近結構物或管線
	盾首	遇障礙物以致無法推進
暗挖工法	隧道上方	沉降/崩塌/湧水
	隧道四周	崩塌/湧水
	管幕	受損/破壞
鄰近結構體	基礎/結構	受地表沉降、坍塌之影響以致受損/破壞
	地下室	受注漿之影響以致受損/破壞
維生管線	自來水幹管、供電電纜、通信電纜、瓦斯管等	受地表沉降、坍塌之影響以致受損/破壞

表 3 風險因子及考慮因素

因子	定義	考慮因素
C_1	「地區」風險指數	生活水平、工業水平不同、地鐵的單位造價因之不同
C_2	「經驗」風險指數	第一條地鐵路線風險高，隨著經驗的累積而降低
C_3	「承包商」風險指數	承包商的技術水平，管理能力
C_4	「定作人」風險指數	主持地鐵工程的業主的組織、效率、決策之明快，是否有安全管理機制
C_5	「發包方式」風險指數	傳統式(設計與施工分別發包)風險最低，統包風險最高
C_6	「決標方式」風險指數	最低價格標風險高，最有利標風險低
C_7	「工程屬性」風險指數	地質、施工方法、.等等

對保險公司而言，風險評估的最終目的是希望能算出合理保費，所以要根據過去的理賠經驗測試上式的合理性以及實用性。也就是說要建立理賠比 R_L（Loss Ratio）與工程綜合風險指數，I_R，的相關性。這兩者間的關係可以下式表示之：

$$R_L = f(I_R) \qquad (3)$$

其中

$$R_L = \frac{理賠金額}{土建費用} \qquad (4)$$

理論上，兩者的關係非常複雜，不是一個簡單函數所能表示，但從實務的觀點來看，最簡單的線性函數也許就可以滿足需求，也就是說可以假設兩者成正比：

$$R_L = k \bullet I_R \qquad (5)$$

其關係可以用十分簡單的迴歸分析求得。至於是用土建費用而不以保險金額計算理賠比，是因為理賠的事故以土建部份為最多，理賠金額最大。所以原則上應以土建費用為計算基礎，排除機電及系統之工程費。

影響工程風險之因子林林總總，不限於表 3 所列，但該表所列之因子幾可涵蓋所有主因子。茲將該等因子及其考量因素說明如下：

地區（C_1）

不同國家的生活水平、工業水平不同，即使是同一國家，不同地區的物價指數也可能有相當大的差距，所以不同地區的地鐵造價亦不同，而保費及理賠比自亦不同。不同國家工程界的生態亦不一樣。例如，日本的地鐵常以統包方式由承包商負責設計及施工，而歐美各國地鐵的設計及施工常分由設計者及承包商負責。

各國的民族性與對保險的認知亦有所不同。東方人習慣於息事寧人，有些小事故不願聲張，而西方人則習於事事據理力爭，即使是些微小事也會索賠，所以理賠的頻率會高。

經驗（C_2）

每個國家或每一個都市，在興建第一條地鐵路線時，由於對工程的不瞭解以及專業人員的不足，風險會較高。隨著時日的增加，經驗的累積會使得風險降低。

承包商（C_3）

承包商的組織、管理能力、專業能力、財務能力、甚至敬業精神都會影響工程風

險。大型跨國公司人力資源充沛、人員調度較富彈性，員工的敬業精神也較高，質量管控佳，應變能力強，工程風險自然小。反之，小公司人力資源有限，資金調度不，應變能力不足，風險就會高。

承包商是否有項目管理的能力以及安全管理體系也是影響風險的要素。承包商的項目管理著重於組織與效率，包括 ISO, SOP, QA/QC 等行政程序都是健全組織與促進效率的手段。承包商的安全管理體系包括質量管理、監測計畫、應變計畫等等。

定作人（C_4）

主持地鐵工程的業主通常同時負責興建數條路線，其本身組織即已十分龐大，又要掌控為數眾多的設計者與承包商，所以其管理能力至為重要。如果業主的組織鬆散、無法有效地掌控各有關單位，風險就高。反之，如果組織嚴謹、能確實掌控所有單位，風險即可降低。業主的項目管理與承包商的項目管理目標一致，其重點都是要求組織與效率，但業主的項目管理範疇除了組織與效率外，尚包括發包方式及決標方式。因為這二者都受業主主控，所以在風險評估作業中，是獨立因子，但在風險處理時，卻納在業主的項目管理項下。

業主的安全管理理念同樣重要，其重要性甚至超過承包商。因為只有業主具有安全管理的理念才會要求承包商建立安全管理體系。業主的安全管理與承包商的安全管理目標雖是一致，但業主的安全管理偏重設計之檢核及監理，而承包商的安全管理偏重施工時的質量管理、安全監測與應變機制。

發包方式（C_5）

傳統的發包方式是設計與施工分別發包，一般而言，在這模式之下，設計會較為嚴謹及保守，所以風險會低些，但其缺點是設計理念在施工過程中常常無法貫徹，而且可能因設計保守以致建設經費增加。如果用統包的方式，將設計與施工發包給同一家承包商的話，設計可以優化、工期可以縮短、建設經費可以減少，但其缺點是廠商常會因成本的考量，忽略安全，所以風險會增高。

決標方式（C_6）

如果是採最低標，由出價最低者得標的話，廠商的資質不會太好，而且因價格低，工程質量也堪慮，風險會高。反之，如採最有利標決標，由資質最佳的廠商得標的話，風險會低些。

工程屬性（C_7）

這項目包括許多風險因子，有些因子可以具體呈現，例如地質、施工方法、施工地點等，有些因子只能主觀地判斷，如工地管理、施工規範、特約條款等。大體而言，地面段、高架段及地下段的風險有顯著的差異，所以應將這三者分開考慮，而以其長度作為權重，但其個別的風險指數又會受其他因素的影響。無論是地面段、高架段或地下段，地質都是影響風險的因素，所以掌握地質風險是掌握地鐵風險的關鍵。

由以上論述可見，影響風險的因子是相當地多，而針對每一因子，要考慮的因素更多。如果要用純統計的方法求得各個因子的風險指數，必須要有上萬筆資料，才能得到足以信賴的結果。以台北捷運(地鐵)為例，營造綜合險是由捷運(地鐵)工程局統一購買，所以一條線是一個保單，甚至兩、三條線合在同一保單。每個保單只能計算出一個綜合風險指數，I_R，所以說資料筆數十分有限。即使彙整全世界各地所能收集到的理賠資料也不足以進行嚴謹的統計分析，何況並無可能收集到所有的理賠資料，所以所有的風險指數必須以經驗法則，由資深有豐富地鐵經驗的工程師及

核保人主觀判斷，並參考過去經驗共同訂定之。

在初步判定各因子的風險指數後，再以實際已完工的地鐵路線的理賠經驗測試這些指數的可靠度及可用性。如果這些指數合理的話，以式5求得R_L與I_R的相關性（以相關係數表示之）即高，否則即低。在這過程中可以調整各因子的風險指數以得到最佳的相關係數。

3.結語

基本上，保險是一個風險分攤的機制，要是沒有這機制，許多工程會在施工過程中可能會因承包商財務不堪負荷而停頓。但是因為工程風險過高，目前不但保險業者對工程保險卻步，再保公司或者拒保，或者要求天價，造成許多重大工程無法投保之困境。要走出這困境，首重加強安全管理以降低風險，其次是要分析過去理賠資料以訂定合理保費。保費必須合理，保險業者才會有承保的意願，而合理的保費更是業主權益的保障。

保費的訂定涉及保險及工程兩專業，必須由有經驗的核保人員及工程人員協力合作，才能達到公平合理的目標。而安全管理更是達到各方都贏的重要手段。安全管理不但能減少所有各方的財務損失，而且使得工程進行順暢，使整個社會皆蒙其利。

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NATURAL DISASTERS IN TAIWAN

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Taiwan is an island that lies on the western edge of the Pacific-rim earthquake belt, which is an extremely active tectonic region with seismic activities among the highest in the world. The boundary of the Eurasian and Philippine Sea Plates is near the island of Taiwan. Earthquake occurs frequently, and a recent catastrophic event, the Chi-Chi earthquake, is known to the world. Taiwan also is located in the subtropical climate region with high average temperature and precipitation. May and August are periods influenced by continuous seasonal rains that occasionally come with an extraordinary intensity. From summer to fall, Taiwan is affected by typhoons. Some extreme events would accumulate significant rainfalls within a relatively short period of time and result in disastrous consequences such as floods. Given the topography of Taiwan, which is covered by slopelands and high mountains over two third of the island, landslides and debris flows often were triggered after heavy rainfalls. The remaining quarter of the land is plains, but they have been densely inhabited and developed. Due to excess extraction of groundwater for fish farming, significant portion of the plains have suffered severe ground subsidence over the past decades. As a result, Taiwan has to cope with various hazardous situations caused by the Mother Nature. Most significant natural disasters in this island include earthquakes, floods, landslides, debris flows, and land subsidence. The government and people in Taiwan have long being aware of the disastrous consequence of these natural hazards, and continuous efforts have been placed on prevention and mitigation of these natural disasters. This paper discusses these natural disasters that confront the people in Taiwan. Studies and investigations carried out to understand the characteristics of these natural hazards are discussed, and development of programs and approaches in preventing and mitigating natural disasters in Taiwan are also addressed.

INTRODUCTION

Since the end of the 20th century, natural disasters started to take heavier tolls than ever on human lives and properties around the world owing to the unprecedented global climate variation and accelerating urbanization in mankind history. Among all kinds of natural disasters in the world, the earthquakes, followed by floods and fierce winds (typhoons, hurricanes, tornados, etc.), are the most catastrophic and have taken the most monetary loss in urban areas. The damages caused by earthquakes, floods, and fierce winds have totaled almost 90% of the monetary loss from all the natural disasters worldwide in the past decade (Guy Carpenter & Company, 2000). In addition, the death tolls of earthquakes ranked the highest and were the major part of human live loss in natural disasters in the world history. Due to the continuous increase of live and monetary losses from natural disasters, prevention and mitigation of natural disasters became an urgent issue for governments and people in areas with high natural hazards potentials.

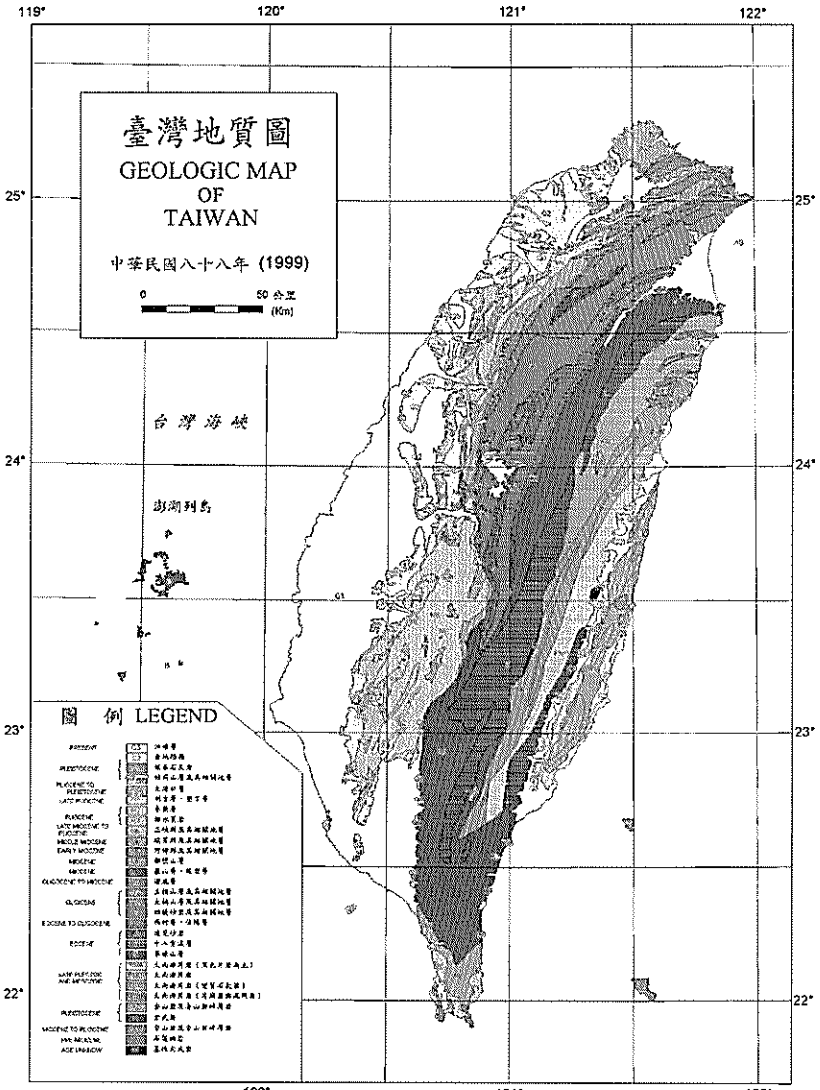


Figure 1. Geological map of Taiwan.

Taiwan is located along the west rim of Pacific Ocean with Tropic of Cancer across the middle. It is at all times under the threats of earthquakes, typhoons, and torrential rains. In 1999, the Chi-Chi earthquake declared a death toll of 2,434, 12,029 wounded, and a total loss of about 11.4 billion US dollars. In 2001, the Toraji typhoon claimed 214 lives including the missing. Later in September of the same year, the flood caused by the extremely heavy precipitation (425mm/day, record high in Taipei City) brought by the Nari typhoon cast a severe hit on northern Taiwan with a monetary loss estimated to 5.7 billion US dollars (Chang, 2002). The monetary loss in the event of Chi-Chi earthquake alone was approximately 3% of the Taiwan GDP in 1999 and human life loss was approximately one-ten thousandth of the total Taiwan population.

Along with the earthquakes and typhoons, landslides and debris flows in Taiwan seem to have higher occurrence rates after the Chi-Chi earthquake (Chin *et al.*, 2005). The loss of lives and properties from debris flows was the highest in 2001 with a death toll of 129. Damages from landslides and debris flows on landscapes, water resource systems, transportation and lifeline systems are much more often reported during heavy precipitation.

In light of the significant increase in damages caused by natural disasters island-wide and around the world, many studies initiated by government and private sectors have been carried out in various aspects for the prevention and mitigation of natural disasters in Taiwan. Some resolutions have been implemented. As the effort for prevention and mitigation of natural disasters continues, it is of supreme importance to understand the characteristics of these natural hazards and their interaction with local environment. The dynamic interaction between unpredictable natural events of earthquakes, typhoons, precipitation and changing local environment by continuous urbanization as well as civilization has significant impact on the outcomes of natural hazards.

NATURAL ENVIRONMENT OF TAIWAN

Tectonic formations, geology, and topography

The island of Taiwan is located approximately 150 km off the southeast coast of China mainland on the convergent boundary of the Eurasian and the Philippine Sea tectonic Plates. It is spindle-shaped, with the longitudinal axis extending roughly north-south for a length of 385 km and an east-west width of 143 km (Moh and Ou, 1979). The island is composed of geosynclinal deposition of Tertiary sediments of the most recent era (the Cenozoic) to a thickness of more than 10,000 meters on a metamorphic base formation (Figure 1). As the result of northwestward movement of the Philippine Sea Plate, the tectonic interactions between the Philippine Sea Plate and the Eurasian Plate are extremely complex in the vicinity of Taiwan. There has been apparent northward subduction of the Philippine Sea Plate beneath the Eurasian Plate and an eastward subduction of the Eurasian Plate underneath the Philippine Sea Plate (Figure 2). This active tectonic interaction and collision have produced the Central Mountain Range in Taiwan (Penglai Orogeny) and the prevailing structural pattern of rock formations on the island aligning in long narrow belts of elongated arcs with their convexities facing the west. All the major geological structures, including the faults and fold axes, align fairly well with this structure throughout the whole island and roughly parallel to the longitudinal axis of the island. The Penglai Orogeny has been continuously elevating the surface of Taiwan. In term of geological time, Taiwan is still a young and developing land (Lin and Chou, 1978).

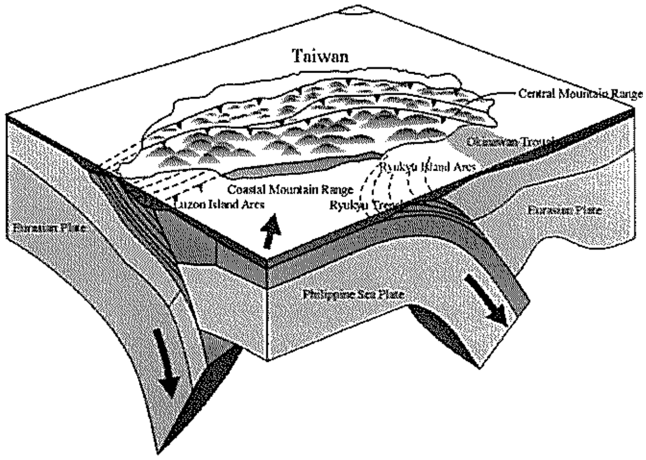


Figure 2. Tectonic activity in vicinity of Taiwan.

The topography and geological condition of Taiwan are closely related to the above tectonic activity. The rock formations are often fractured. In the island of approximately 35,960 sq. km, there are more than 200 peaks of over 3000 m and occupy approximately 1.3% of the island surface (Table 1). The plains with elevations below 100 m take about 31.3% of the island surface and lie mostly on the western side of island, which leaves approximately two-third of the island covered by slopeland merging from mountains to plains (Figure 3). Considering the dimensions of Taiwan, the heights of mountains, and the area of slopeland, the slopes in slopeland area are relatively steep and pose high threats to development in these areas.

Table 1. Topographical distribution of Taiwan (from Lin and Chou, 1978).

Elevations (m) (From/To)	0/100	100/500	500/1000	1000/2000	2000/3000	3000+
Area (%)	31.3	23.5	13.7	19.7	10.5	1.3

Central Mountain Range generally serves as the water divide of river system on the island of Taiwan. Conforming to the topography, the rivers are generally short with steep slopes and possess high scouring capacity in the upstream due to high gradients (Figure 4). Moreover, the flow rates are highly seasonal and fluctuated with the precipitation.

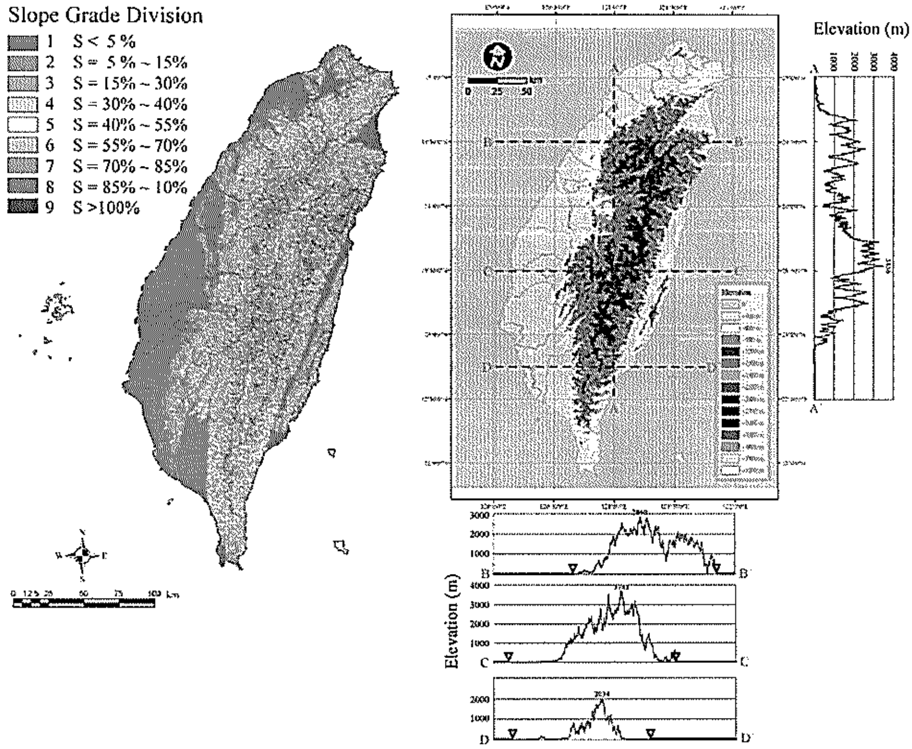


Figure 3. Topography of Taiwan.

River Name	Elevation (m) (origin-end)	River Length (km)	Average Slope (%)
Danshuei River	3529-0	158.7	0.82
Fongshan River	1320-0	45.5	0.44
Toucian River	1913-0	63.0	0.53
Jhonggang River	2616-0	54.0	0.67
Houlong River	2616-0	58.3	0.63
Da-an River	3488-0	95.8	1.33
Dajia River	3740-0	124.2	1.67
Wu River	2596-0	119.1	1.09
Jhuoshuei River	3220-0	186.6	0.53
Beigang River	516-0	82.0	1.69
Puzih River	1421-0	75.9	1.89
Bajhang River	1940-0	80.9	2.38
Jishuei River	550-0	65.0	0.85
Zengwun River	2609-0	138.5	0.50
Yanshuei River	350-0	41.3	0.34
Eren River	460-0	61.2	0.13
Agongdian River	300-0	38.0	0.14
Gaoping River	3330-0	171.0	0.67
Donggang River	1166-0	44.0	0.20
Sihchong River	1062-0	31.9	1.69
Lanyang River	3536-0	73.0	1.82
Heping River	3536-0	48.2	2.70
Hualien River	1200-0	57.3	0.35
Sioguluan River	2360-0	81.2	2.94
Beinan River	3293-0	84.4	0.61

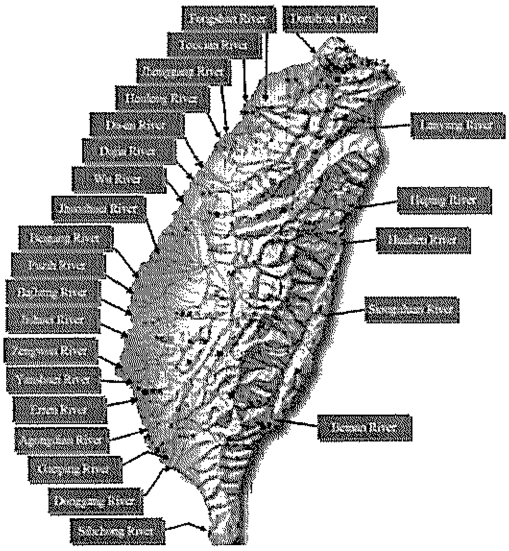


Figure 4. Rivers of Taiwan.

As the result of tectonic interaction and collision of the Eurasian and Philippine Sea Plates, the seismicity in Taiwan is among the highest in world (Figure 5). More than 200 earthquakes with tremors sensible to human (usually with Level 2 intensity and above) each year in Taiwan were recorded in the past century. In the last 15 years, more than 30 earthquakes of Ritcher's scale of 5.0 and above were recorded (Figure 6).

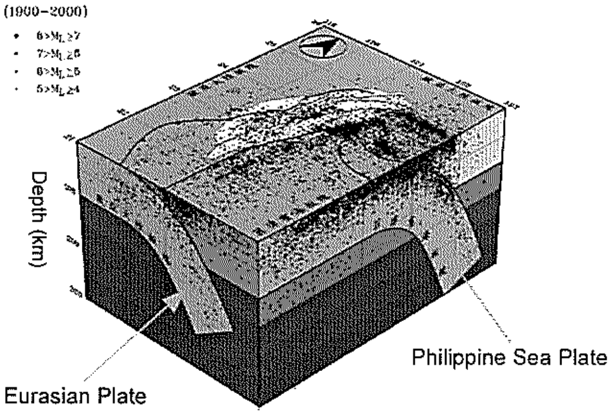


Figure 5. Earthquake hypocenter distribution in vicinity of Taiwan (after Lee, 1991).

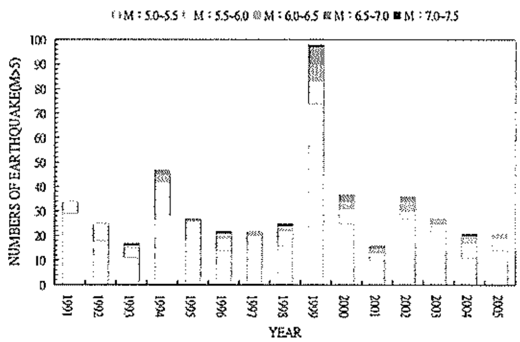


Figure 6. Number of earthquakes with magnitudes ≥ 5.0 in vicinity of Taiwan.

Climatic features

Located in western Pacific Ocean with Tropic of Cancer across the middle, Taiwan is warm and humid. The average annual precipitation in Taiwan over the past 30 years is around 2,400mm, which is almost 3 times of the world average annual precipitation, 800 mm. Approximately 80% of the precipitation is cumulated from May to October (Figure 7). With the topographical and geological conditions of Taiwan, approximately 71% of the precipitation becomes surface runoff, 24% evaporation, and 5% infiltration. Only about 15% of the precipitation can be retained in surface water bodies. The amount of fresh water stored in the island is around 20% of annual precipitation including 5% of infiltration into groundwater.

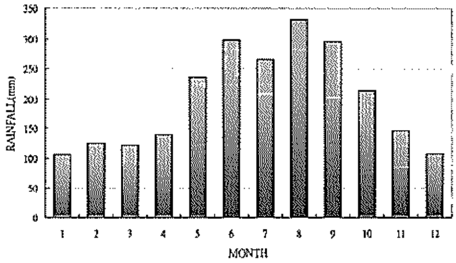
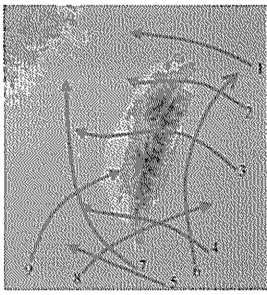


Figure 7. Average monthly precipitation in Taiwan.



Tracks	Visits	%
1	50	13.4
2	46	12.3
3	40	10.7
4	35	9.3
5	76	20.3
6	53	14.2
7	30	8
8	16	4.3
9	28	7.5
Total	374	100
From 1897 to 1996		

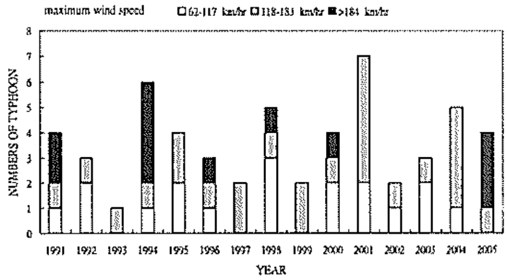


Figure 8. Major tracks and statistics of typhoons visiting Taiwan.

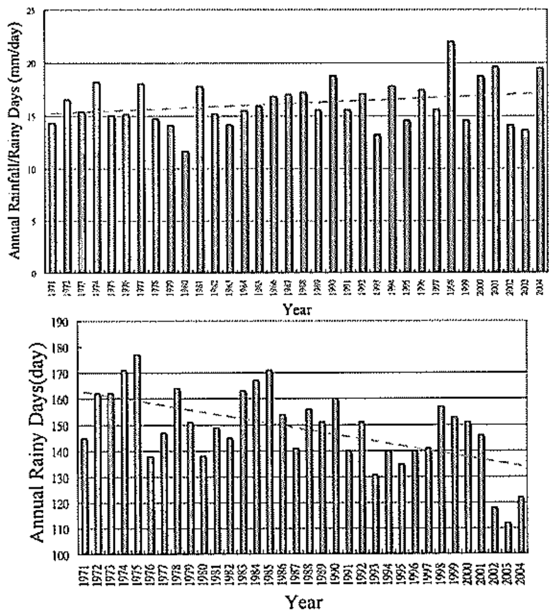


Figure 9. Precipitation pattern change in Taiwan.

In the past, Taiwan was subjected to the attack of typhoons mostly from May to October at an average rate of 3.7 times a year with dominant routes in northwestward direction (Figure 8). Heavy precipitation is usually associated with typhoons visiting Taiwan and constitutes one of many major parts of precipitation in addition to monsoon in May and June.

As a result of global climate variation, Taiwan has also been experiencing a gradual climate change. The number of rainy days is decreasing with higher rainfall intensity (Figure 9) even though the annual precipitation remains fairly constant in the past few years.

NATURAL DISASTERS IN TAIWAN

Earthquakes

The tectonic setting and dynamics of the Eurasian and Philippine Sea Plates are the major triggering mechanism of seismic activities in vicinity of Taiwan. Although occurring often, most of the seismic activities caused very minimal damage on the island. However, the catastrophic ones from time to time rock the island and induce significant loss of lives and properties.

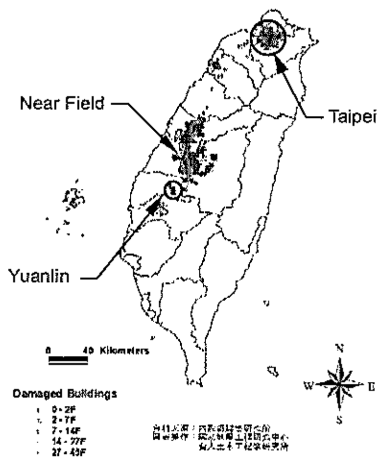


Figure 10. Most damaged areas in the Chi Chi earthquake.

In the Chi-Chi earthquake, the total loss in Taiwan was roughly 11.4 billion US dollars including an asset loss of more than 8 billion dollars and indirect loss of approximately 3 billion dollars. Three areas, respectively the vicinity of Chelungpu fault, the densely populated Taipei City and County area, and Yuanlin area, were identified later in study with major building and infrastructure damages (Figure 10) in the earthquake. In the area within a 6 km distance from Chelungpu and Shuangtung Faults, the damages were caused primarily by high seismic forces (a recorded horizontal peak acceleration of 989 gals in east-west direction at Station TCU084, and a vertical peak acceleration of 716 gals at Station CHY080) and large relative displacement at surface (average fault dislocations of 1.5 m horizontally and 3 m vertically). Most of the structures near the fault were severely damaged even for buildings designed in compliance with modern design codes. The bridges in central Taiwan suffered severe damages from collapsed spans, cracked piers, and distressed components (Figure 11). The damages on dams, lifelines and critical facilities, and power systems were also reported. In Hsinchu Industrial Park along, an estimated loss of 400 million US dollars was reported and most of which was incurred in semiconductor and silicon wafer production facilities.

In Yuanlin Town and Changhwa County, soil liquefaction seriously affected an area of approximately 60 sq. km in the Chi Chi earthquake (Moh *et al.*, 2002). Sand boils were found at locations at which the clay covers are either too thin or totally missing. At some locations where the clay covers are thick, the settlements of buildings up to 1 m or so as a result of ground subsidence were observed. The buildings in these areas suffered severe damages from soil liquefaction. Soil liquefaction also caused damages on levees and resulted in lateral spreading at several locations along Koniaokeng Creek near Chelungpu Fault. Moreover, over the reclaimed land of Taichung Port liquefaction, damaged 4 of its 45 docking wharves (Figure12).

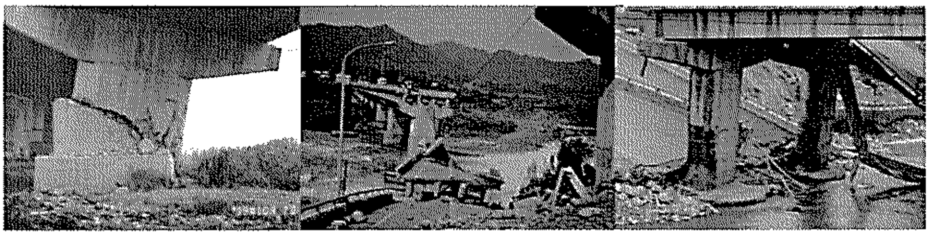


Figure 11. Bridge damages in the Chi Chi earthquake.

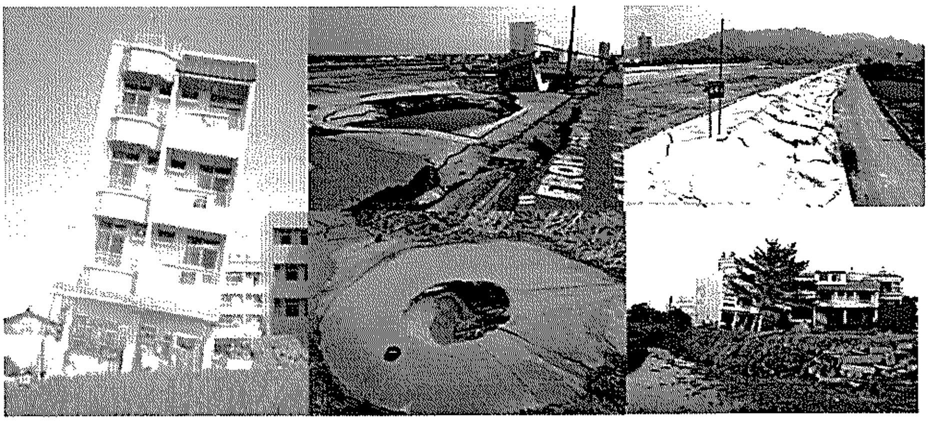


Figure 12. Liquefaction induced damages in the Chi Chi earthquake.

Located approximately 150 km north of the epicenter, Taipei area suffered damages more than anticipated in the Chi Chi earthquake. The result of study indicates that the geological condition with thick soft and poorly cemented young sediment deposits could have adverse site response effect in bowl shape base formation of Taipei basin and be rationally argued to contribute to this consequence (Tsai, 1987). The adverse site response effects in Taipei basin include amplification of earthquake magnitudes and extended period of maximum design spectral response acceleration (Chen and Chen, 1995). In 1986, two earthquakes with magnitudes of 6.2 and 6.8 and epicenters approximately 15 and 10 km east of Taiwan respectively have caused damages on buildings in Taipei predominantly of 10 to 16 stories (estimated predominant structural periods between 1.0 and 1.6 sec). In light of the above findings, the seismic design code revised in 1989 included the site response effect and extended the predominant period with revised design response spectra for Taipei area. In July 2005 the design response spectrum was further modified with higher design seismic force (Figure 13).

Landslides were also reported during the Chi Chi earthquake. Almost all of the slope failures occurred east of Chelungpu Fault on the hanging wall with shallow slips in residual soils of depths between 1 m to 5 m. At Tsao Lin, the debris from a massive landslide slammed into the valley of Ching Shui River, blocked the river, and formed a reservoir behind (Moh *et al.*, 2002).

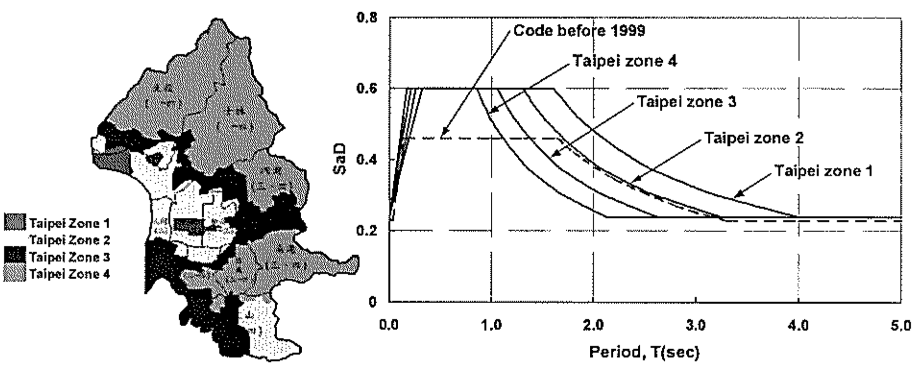


Figure 13. Design response spectrum for Taipei area.

Landslides and debris flows

Landslides and debris flows are two major natural disasters that threaten the people and environment in Taiwan, and the later has caused greater disasters in this island over the past few years. The geological settings, topographical features, and climatic conditions of Taiwan provide the inherent factors for potential cause of these disasters. In addition, the population congregation, excessive land development, and rapid economy growth over past decades contributed more potential adverse effects to the environmental condition. The precipitation statistics of Taiwan indicate the tendency of decreasing total raining days per year but increasing precipitation intensity per rainy day. The change in raining pattern suggests that the intensity of a raining event is becoming higher, which can be easily related to the increased possibility of landslide and debris flow hazards.

Table 2. Losses from recent debris flow disasters in Taiwan (from Chin *et al.*, 2005)

Time	Typhoon Event	Location (Township/County)	People Missing and Dead	Property Loss
6/23/1990	OFELIA	Sioulin / Hualien	35	35 houses damaged
7/31/1996	HERB	Sinyi / Nantou	2	21 houses and roadways damaged
7/31/1996	HERB	Shueili / Nantou	8	17 houses damaged
10/29/2000	XANGSANE	Rueifang / Taipei	8	20+ houses, roadways, and bridges damaged
7/28/2001	TORAJI	Fonglin / Hualien	6	3 houses and a wastewater treatment plant damaged
7/28/2001	TORAJI	Guangfu / Hualien	43	150+ houses damaged
7/28/2001	TORAJI	Sinyi / Nantou	16	67 houses, roadways, and bridges damaged
7/28/2001	TORAJI	Shueili / Nantou	1	21 houses and roadways damaged
7/28/2001	TORAJI	Shueili / Nantou	17	55 houses damaged and all public infrastructures fully destroyed
7/28/2001	TORAJI	Sinyi / Nantou	46	20+ houses, roadways, and bridges damaged
6/30/2004	MINDULLE	Heping / Taichung	3	30 houses and roadways damaged
6/30/2004	MINDULLE	Heping / Taichung	1	40 houses damaged
6/30/2004	MINDULLE	Dongshih / Taichung	1	Roadways damaged
6/30/2004	MINDULLE	Puli / Nantou	2	17 houses damaged
6/30/2004	MINDULLE	Hezuo / Nantou	1	6 houses damaged
8/23/2004	AERE	Wufong / Hsinchu	28	20+ houses damaged
9/11/2004	AERE	Jianshih / Hsinchu	4	1 house damaged

In 1999, the Chi-Chi earthquake triggered over 20,000 landslides island-wide (Chin *et. al.*, 2005). The deposited landslide masses of critically stable conditions became the source materials for debris flows during or after intensive precipitation. As a result, approximately 250 hazardous debris flow events have been reported during or after the earthquake. This hazardous threat of landslide and debris flow is aggravated with a higher occurrence rate and greater intensity after the Chi-Chi earthquake, and debris flows were easily triggered than ever during an event of heavy rain or typhoon (Table 2).

Typhoons and floods

Taiwan is located in the area with primary passages of typhoons originated in northwest Pacific Ocean. The average number of visitation of typhoons to the island is approximately 3.7 per year, which is roughly 15% of the typhoons originated in the region of northwest Pacific Ocean and South China Sea. When visiting Taiwan (Figure 8), the typhoons with tracks through either northern or southern Taiwan (Tracks 1, 2, 4 and 9) tend to pose higher threats to the island.

The damages by typhoons in Taiwan are primarily incurred by the strong winds, storm surges in the coastal area, and heavy precipitation. In addition to strong winds, the foehn and salty winds associated with typhoons have caused significant damages on agricultural produces and plants. The ground subsidence induced by excessive groundwater extraction has worsened the harm of storm surges to coastal areas, and the drainage of areas with seawater encroachment takes longer to complete. The precipitation further can easily trigger the landslides and debris flows. Moreover, the concentrated precipitation can cause extremely high flow rates in water channels and serious scouring problems on riverbeds and bridge foundations.

Nevertheless, the floods generated by heavy precipitation associated with typhoons have been the predominant cause of damage in Taiwan. In 2000 and 2001, the Xangsane, Troaji, and Nari typhoons caused severe damages on agriculture and transportation systems, and triggered landslides and debris flows island-wide. The flood in Taipei caused by the Nari typhoon severely damaged the MRT system at Taipei Station and it took several months for restoration to complete.

Ground subsidence

Ground subsidence can be triggered by many factors such as seismic activities, volcano eruptions, dissolution of limestone, etc. and exploitation of underground resources (groundwater, oil, coal, etc.) as well as underground space excavation. In Taiwan, the primary reason for regional ground subsidence is excessive exploitation of groundwater. Until now, more than 11% of the land in Taiwan has subsided, which was increased from 7.4% in 2001 and mostly along west coast of the island (Figure 14).

In 1960s, the ground subsidence in Taipei City was recognized as a result of excessive usage of groundwater for industry and domestic consumption. A surface settlement of more than 2 m was recorded near the center of city (Figure 15). Regulatory restriction on groundwater pumping activity was later reinforced within the city limit and had successfully stopped the subsidence in 1980s. With the gradually elevated groundwater levels, rebound of ground surface was observed from the elastic portion of soils due to decreased effective overburden stresses (Figure 15).

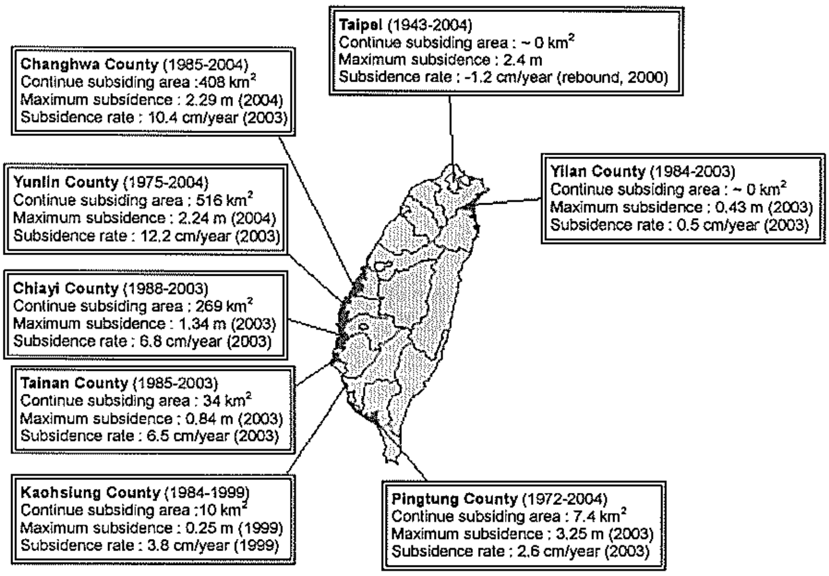


Figure 14. Ground subsidence in Taiwan.

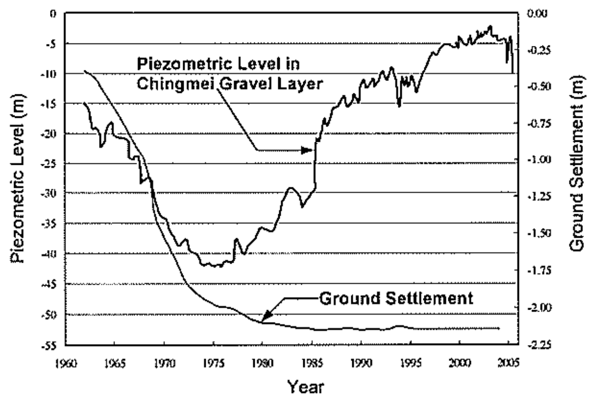


Figure 15. Ground subsidence in Taipei City.

However, continuous subsidence with extending territory still occurs along the southwest coast of Taiwan, where the land has long been used for agricultural purposes and fishery farms. Due to limited precipitation, relatively level topography, predominately granular alluvial formation, and unstable flow rates of rivers in the western plains of Taiwan, the surface water resources are rare and insufficient to provide steady supply of fresh water for consumption. Groundwater became the only economical and steady water supply in this area. Extensive pumping of groundwater for usage in fishery farm and agricultural purposes further aggravates ground subsidence and causes the intrusion of salt water into the groundwater systems.

Ground subsidence over the western plains of the island leads to irreversible consequences of infrastructure settlements and regional drainage difficulties in addition to the loss of land. Floods in these areas have occurred more often and the retention times become longer than ever.

Water deficiency

The average annual precipitation of Taiwan is roughly 2,400 mm/year, approximately three times of the world average. However, with concentrated precipitation over the period from May to October and with regional discrepancy, topography of steep slopes, and rivers of limited lengths, only one-sixth of the precipitation was able to be retained as surface water for consumption in addition to 5% of infiltration into groundwater. The surface water features and geological as well as topographical conditions in the island provide very limited locations for construction of water storage facilities. Retention of fresh water has long been a challenging task in Taiwan. In the densely populated western plains of Taiwan, sufficient and steady supply of fresh water for agricultural, industrial, and domestic consumptions has been sustained by continuous exploitation of groundwater, which is the main cause of ground subsidence in this area.

After the Chi-Chi earthquake, significantly increased events of landslides and debris flows, mostly in areas of watersheds, have stripped, to certain degrees, the slopeland and damaged hydraulic facilities. Surface erosion during heavy precipitation is aggravated on non-vegetated and steep slopes, which leads to large quantities of sediments in reservoirs, and generates high turbidity in surface water and causes much difficulty in water treatment processes.

In the past, the precipitation incurred from visitation of typhoons has been a major replenishing source of water storage in the island. However, in recent years, the high turbidity of surface water after typhoons significantly reduced the treatment capacity of water treatment plants and created water supply shortage in certain areas.

NATURAL DISASTERS MITIGATION

Disaster warning and monitoring systems

The issues regarding natural hazards started to receive more attention in Taiwan since the disastrous events of the Chi Chi earthquake in 1999 and the floods in 2000 and 2001. The issues regarding natural hazard forecast, prevention, and mitigation have been intensively discussed and studied by government agencies and private sectors. The government has taken initiatives to integrate loosely kept information in various different government agencies and to enhance capabilities in tasks of hazard warning and monitoring.

The Central Weather Bureau (CWB), Ministry of Transportation and Communications, is the primary government agency responsible for collecting and interpretation of climatic and seismic information in Taiwan. As part of the island-wide climatic monitoring system, it implements a total of 362 automated rain gauges island-wide (Figure 16) to collect real-time precipitation data, which is now incorporated with geographical information system (GIS) to provide early warnings on floods and debris flows. The CWB also operates one of the most advanced systems in the world on seismic monitoring in Taiwan area. The system includes 75 short period seismometers, 680 strong motion seismometers, 12 broadband seismometer stations, and 68 strong motion seismometers in major cities on buildings and bridges (Figure 17). With this system, the CWB made crucial information of the Chi Chi earthquake available within 102 seconds from the moment when earthquake occurred. The information included location of hypocenter, magnitude, and island-wide intensities.

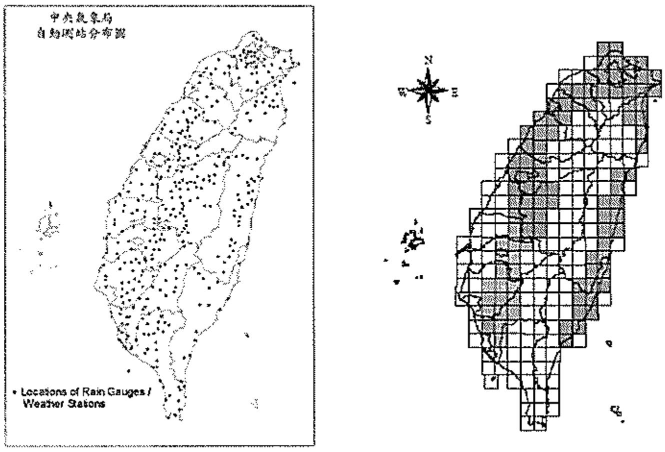


Figure 16. Locations of the CWB automated rain gauges and areas under natural hazard study.

Also in the Chi Chi earthquake period, the groundwater monitoring system (Figure 18) implemented and maintained by the Water Resources Agency (WRA), Ministry of Economic Affairs, supplemented invaluable information on groundwater variation in elevations and quality before, during and after earthquakes. The system, consisting of 492 monitoring wells mostly in the western plains of Taiwan, is set up to provide continuous monitoring data on groundwater levels and quality, and additional reference for ground subsidence in these areas.

Risk management and mitigation strategy formulation

To enhance the disaster response capability as part of the disaster mitigation resolution, the government enacted a Disaster Prevention and Response Act (DPRA) in 2000 (Bylaws in 2001) soon after the Chi Chi earthquake. The National Disasters Prevention and Protection Commission of the Executive Yuan was also established to cope with the natural disaster related issues in the central government. DPRA is basically structured on four stages of disaster management, i.e., mitigation, preparedness, response, and recovery. With respect to different types of natural disasters, different government agencies are in charge of the matters regarding various stages of disaster management according to DPRA (Table 3).

Table 3. Administrative agencies for natural disasters in Taiwan (DPRA).

Agency	Types of Natural Disasters
Ministry of the Interior	earthquakes and strong winds
Ministry of Economic Affairs	floods and droughts
Council of Agriculture, Executive Yuan	landslides and debris flows

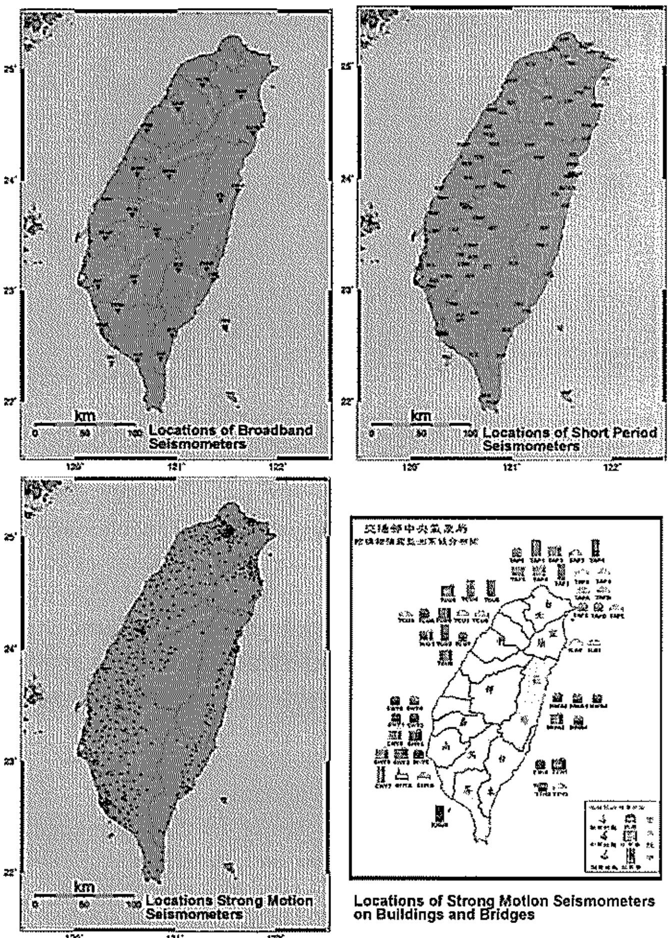


Figure 17. Locations of the CWB seismometers.

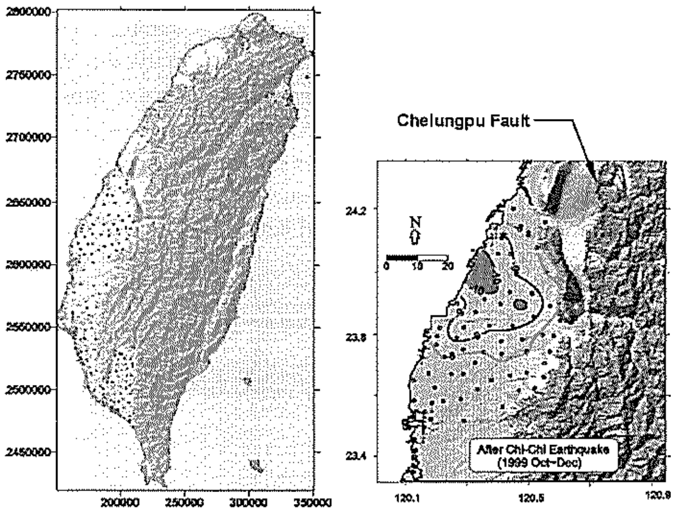


Figure 18. Distribution of the WRA groundwater monitoring wells.

The Taipei City is the most populated area in Taiwan and had suffered serious damages in various events of natural disasters. In response to the urgent need for disaster mitigation in the city, the Taipei Disaster Prevention and Rescue Committee was established under the city government to develop and promote hazard mitigation work on earthquakes, typhoons and inundations, and landslides as well as debris flows (Figure 19).

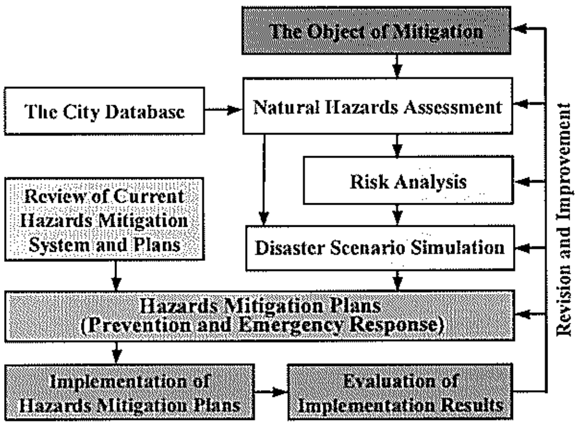


Figure 19. The conceptual framework of mitigation program of Taipei City.

The earthquake risk assessment model using HAZ-Taiwan and TELES (Taiwan Earthquake Loss Estimation System) based upon the framework of HAZUS (developed by the Federal Emergency Management Agency, FEMA, U.S.A.) is adopted by the Taipei City government to provide necessary information in forming the earthquake mitigation plans. Integrated with geological GIS information, demographic data, structure and building information, lifelines and critical facilities in the city, the assessment model can provide estimation on direct physical damages due to earthquakes and associated social losses (Figure 20). The city emergency preparedness and response plans are then developed in combination with transportation networks. With the city geotechnical boring data and geological GIS information, liquefaction potential maps can also be developed for reference of earthquake hazard and land use as part of mitigation plans in the city.

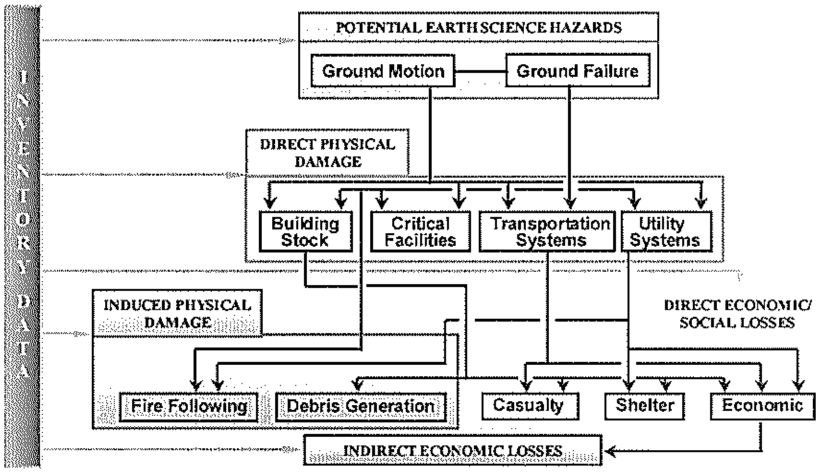


Figure 20. Structure of HAZ-Taiwan/TELES.

The Taipei City is located in the center of Taipei basin and surrounded by Tanshuei River, Keelung River and Hsintein Creek. The city is protected by levees and pumping facilities along these rivers against floods during heavy precipitations. The flood mitigation plan of Taipei City is primarily constituted on the basis of the inundation potential maps. These maps are constructed in consideration of precipitation with various intensity and elapsed times, land development conditions, topography, drainage capacity from the sewer system and pumping facilities. Different inundation hazard scenarios are formulated with different return periods of rainfall intensity and elapsed times and presumed drainage capacity with fully operational or partly failed pumping facilities. The inundation potential maps with inundated areas and depths of flood can be generated for mitigation plans and decision making process of evacuation (Figure 21). With the transportation GIS information, the routes of evacuation and shelter information can also be supplemented.

To form the landslide/debris flow mitigation plan, the Taipei City government built a comprehensive geological and natural environment database for the slopes in the area. The database includes 49 areas of potential debris flow rivers, situations of the existing slope management by the administration, documentation of historical landslides, the relationship between precipitation and landslide potential, and geological, hydrological, as well as geomorphic information of potential landslide areas. In addition to the rain gauges installed by the CWB, many other rain gauges were established by the Taipei City government to collect real-time precipitation data as a crucial index for issuing early warning of debris flows. In addition to the warning issued during precipitation in areas with high risk of landslides and debris flows, long-term resolutions of slope management and relocation of villages along the potential paths of debris flows were implemented as part of the city mitigation plans.

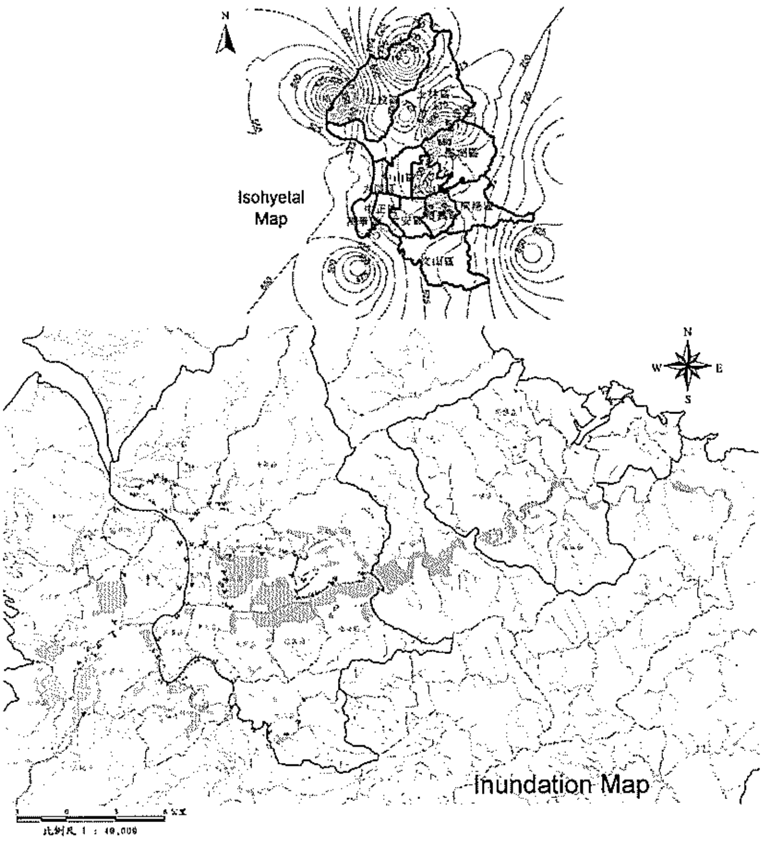


Figure 21. Isohyetal map and inundation map of the Nari typhoon.

Latest technical development

The CWB is conducting the Climate Variation and Severe Weather Monitoring and Forecasting Development Project, which is a multi-year project to establish monitoring and warning capabilities for climatic disaster prevention, to develop techniques of short-term climate analysis and prediction, and to strengthen the capabilities of very short-term severe weather analysis and forecasting. The capability of the CWB in typhoon monitoring and prediction, precipitation monitoring and forecasting, and marine climate forecast can be significantly improved. With the Third-Phase Strong-Motion Observing Project to develop an earthquake real-time warning system, the CWB will be able to reduce the earthquake rapid reporting time from the currently 60 seconds to 30 seconds by 2009 and even provide an early warning of major earthquakes in matter of seconds to organizations and facilities in pre-established system.

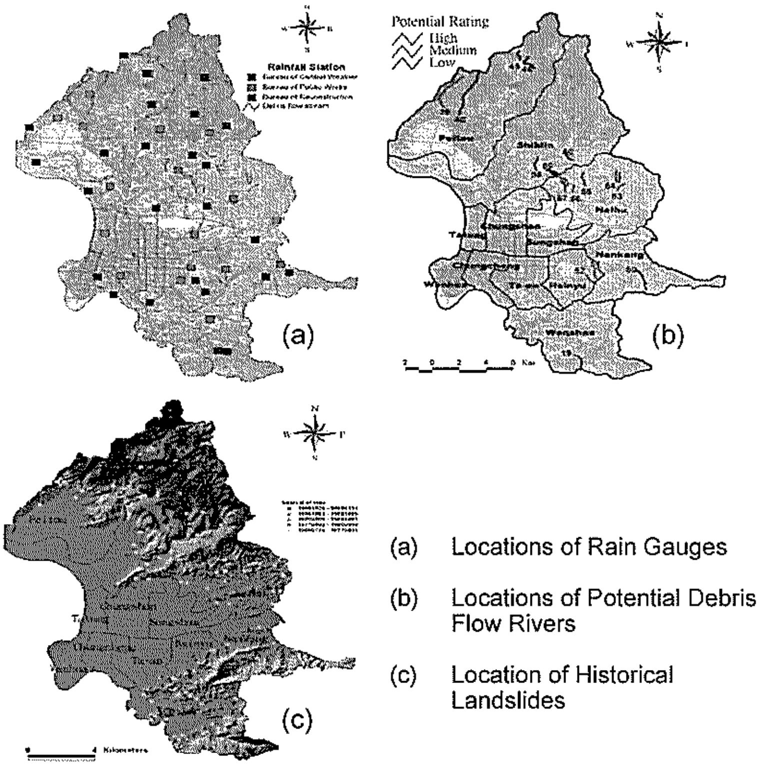
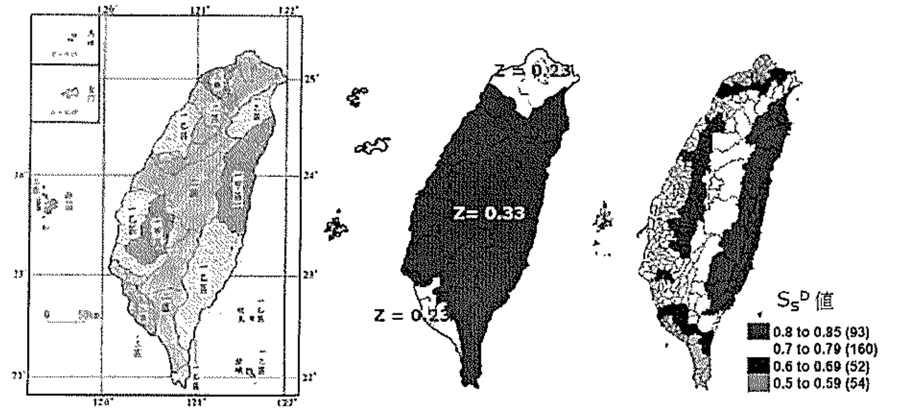


Figure 22. The city database of Taipei.

After the Chi Chi earthquake, the building seismic design codes in Taiwan were revised immediately in 2000 according to recorded seismometer data and field observations. Two seismic zones were adopted for seismic design of buildings and infrastructure in Taiwan (Figure 23). However, based upon further analysis of seismic data and new findings in earthquake, the latest revision of building seismic design codes was issued in July 2005. The major changes include larger seismic design loads in most areas, additional requirement on structure ductility under maximum considered earthquake with return period of 2,500 years, supplement of near fault effects and special consideration on site response characteristics of soft deposits, addition of provisions on seismic retrofit, isolation, and damper design guidance.



Before Chi Chi Earthquake **After Chi Chi Earthquake** **Starting from July 1, 2005**

Figure 23. Seismic zoning change in design codes of Taiwan.

Modeling techniques of debris flows is also advanced in predicting the flow paths. A numerical solution using rheological model with two-layer system consisting of a plug region and the bottom boundary layer has provided better simulation on debris flow propagation (Liu and Huang, 2005). This advancement enables a physics based model to estimate the extent of influence of a given debris flow event. The result can be implemented to provide useful information for slopeland management and necessary information for urgent evacuation in debris flow areas.

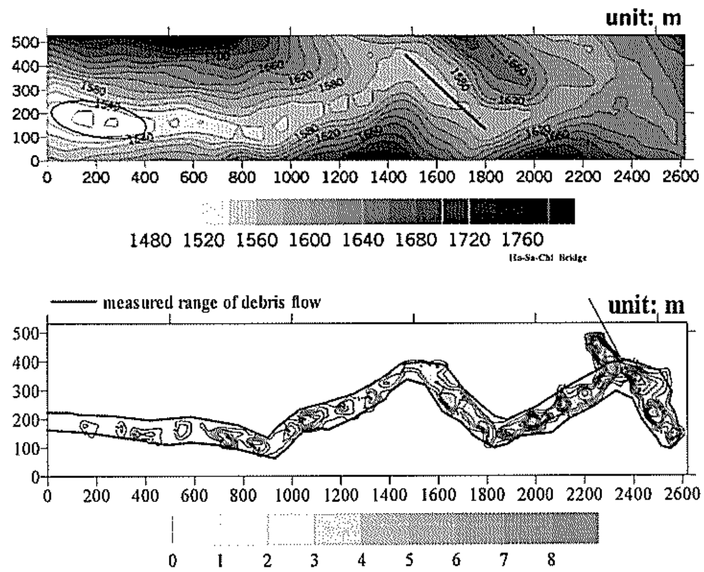


Figure 24. Result of numerical modeling of debris flow path (from Chin *et al.*, 2005).

CLOSING REMARKS

Typhoons, earthquakes, and precipitations are natural events occurring in Taiwan all the time. It is, however, the interaction between those natural events and the natural as well as man-made environments that determines the outcomes and consequences of these events. Even though the occurrences of these natural events are inevitable, the extent of preparedness beforehand and action taken afterwards certainly decide the cost incurred at each event. Taiwan has encountered many different types of natural hazards. The unique natural environment brings out the complexity of natural hazards. Systematic resolutions in many aspects to mitigate the disasters caused by natural events are needed. Even though the natural hazards may evolve with changing environments, knowledge of fundamental issues can always be exercised to provide valuable insights to the trends and resolutions to these disastrous events.

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Lessons Learned from Recent MRT Construction Failures in Asia Pacific

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Abstract: Modern urban mass rapid transit systems (MRT) are frequently constructed underground. Since constructions are carried out mostly at depths of 15 to 30 m below ground surface, the risks are very high. This paper compiles major failures in underground MRT constructions since the year 2001 and classifies them in accordance with their causes of failure and consequences, with the ultimate goal of developing a system for managing risks involved in underground, particularly MRT, constructions. Basically, the classification system recommended by International Tunnelling Association is adopted for the purpose. Five major failures with disastrous consequences are discussed with their causes identified and consequences given. In consideration of the high fatality and huge economic losses involved in these failures, there is an urgent need for the construction industry to establish risk management policies, especially for underground works. Accordingly, the general principles of risk management are discussed.

1 INTRODUCTION

As metropolitan populations grow, rapid transit systems become a necessity for solving traffic problems and their benefits are widely appreciated. In this region, Japan has the longest history of MRT construction with its first route open for service in 1927, followed by China (Beijing) in the 60's, North Korea (Pyongyang), Korea (Seoul), Hong Kong in the 70's, Singapore in the 80's, R. of China (Taipei) in the 90's, and Thailand joined the bandwagon in 2004. Most of the cities are still extending their existing lines and expanding their existing networks, and many new systems are to be constructed in the coming years. For example, there are MRT systems already in operation in 6 cities in China and new MRT systems have been approved to be constructed in 5 more cities by the Chinese government. In addition, 11 cities are applying or going to apply for approvals.

Most of these major Asian cities, with thick young sediments deposited at the surface, have ground conditions which are hostile to the underground works necessary for rapid transit systems, so failures frequently occurred during construction. This paper attempts to summarize major events, as listed in Table 1, which have occurred since the turn of the century, i.e., the year 2001 and onward. Information was collected mainly from 3 sources: (1) the authors' personal involvement, (2) available literature, (3) web pages. The list is by no means exhaustive as numerous events are unreported. Although it is intended to cover the entire Asia Pacific region, the information collected is rather limited to that related to MRT constructions in Guangzou, Beijing, Shenzen, Taipei, Kaohsiung, and Singapore. The authors will continue their efforts of collecting information from other cities and obtaining more details on individual events. The eventual goal of this exercise is to establish a database for risk management of MRT constructions.

2 CLASSIFICATION OF EVENTS

To facilitate statistical analyses for the purpose of establishing parameters for risk management, events are categorized according to the causes of failure and the consequences as a result of failure.

2.1 *Classification of Causes of Failures*

As listed in Table 2, there are 22 failure modes which are commonly associated with underground works, including open cut, cut-and-cover construction, shield tunnelling, mining (i.e., tunnelling by using the NATM method), and ground treatment. Failures are also frequently associated with rupture of underground utilities so this has also been included in the list. Codes are assigned to these failure modes so they can be defined consistently in the future to make statistical analyses easier.

2.2 *Classification of Consequences*

The guidelines, entitled "Guidelines for tunnelling risk management: International Tunnelling Association, Working Group No. 2" (hereinafter called the Guidelines), proposed by the International Tunnelling Association (ITA, 2004) for classifying the consequences of potential hazards in tunnelling works provide an excellent framework for classifying the consequences of failures and the principles given therein are applicable for other types of underground constructions. The items proposed by ITA for classifying consequences are:

- Injury to workers or emergency crew
- Injury to third parties
- Damages to third party property
- Delay (to project schedule)
- Economic loss to owner
- Harm to the environment
- Loss of goodwill

Table 1 Significant events in MRT construction since year 2001

Event	MRT System	MRT Line	Causes		Consequences					
			Code (Table 2)	Description	Description	Classification				
						Injury (Table 3)	Economic Loss to Third Parties (Table 5)	Total Economic Loss (Table 4)	Delay to Critical Path (Table 8)	Disruption to Traffic (Table 9)
1-02-14	Guangzou	No. 2	OT	Collapse of hand-dug caisson (depth = 17m), could be due to rupture of a watermain	1 worker died	serious	insignificant	insignificant	insignificant	insignificant
1-05-25	Shenzhen	No. 1	OC-1	Collapse of trench (width = 1.7m, depth = 2m)	1 worker died and 2 were injured	serious	insignificant	insignificant	insignificant	insignificant
1-08-20	Shanghai	No. 4	OC-1	Earth slip	4 workers died	severe	insignificant	insignificant	insignificant	insignificant
2-02-04	Beijing	No. 5	ST-3	Settlement/sinkhole over shield tunnel	3 residential houses collapsed	none	serious	serious	insignificant	insignificant
2-09-03	Nanjing		MG-1	Settlement/sinkhole over mined tunnel	sinkhole diameter = 4m, depth = 2.4m, rupture of watermain and sewer	none	considerable	considerable	insignificant	considerable
3-02-01	Taipei	Banqiao	ST-1	Leakage at tunnel eye	more than 100 residences were damaged	none	severe	severe	severe	considerable
3-07-01	Shanghai	No. 4	MG-6	Failure of crosspassage	refer to Section 3.1	none	disastrous	disastrous	disastrous	considerable
3-07-01	Nanjing		ST-2	EPB Shield machine encountered piles	Sinkhole of unknown size	none	considerable	considerable	insignificant	insignificant
4-03-17	Guangzou	No. 3	CC-1	Failure of retaining wall	1 worker died	serious	insignificant	considerable	insignificant	insignificant
4-04-01	Guangzou	No. 3	CC-1	Failure of diaphragm wall	area affected = 1000 m^2, 1 building (3-story) collapsed	none	serious	considerable	insignificant	insignificant

Table 1 Significant events in MRT construction since year 2001 (continued)

Event	MRT System	MRT Line	Causes		Consequences					
			Code (Table 2)	Description	Description	Classification				
						Injury (Table 3)	Economic Loss to Third Parties (Table 5)	Total Economic Loss (Table 4)	Delay to Critical Path (Table 8)	Disruption to Traffic (Table 9)
4-04-20	Singapore	Circle	CC-1	Failure of strutting system	refer to Section 3.2	severe	disastrous	disastrous	disastrous	disastrous
4-05-29	Kaohsiung	Orange	ST-1	Leakage at tunnel eye	refer to Section 3.3	none	disastrous	severe	severe	considerable
4-08-09	Kaohsiung	Orange	CC-3	Leakage of diaphragm wall	refer to Section 3.4	none	disastrous	severe	disastrous	considerable
4-09-25	Guangzou	No. 2	UT-1	Rupture of watermain led to collapse of retaining wall	area affected = 100m^2, 3 persons fell into the hole, one sustained minor injuries	insig.	considerable	insignificant	insignificant	insignificant
4-11-02	Taipei	Luzhou	UT-1	Rupture of watermain	sinkhole, diameter = 3m, depth = 4m	none	considerable	insignificant	insignificant	insignificant
4-12-05	Taipei	Hsinzhung	ST-1	Leakage at tunnel eye	Sinkhole of unknown size	none	considerable	considerable	insignificant	insignificant
5-01-24	Guangzou	No. 3	ST-3	Settlement/sinkhole over shield tunnel	sinkhole - 20 houses settled	none	considerable	considerable	insignificant	insignificant
5-07-07	Kaohsiung	Orange	CC-3	Leakage of diaphragm wall	refer to Section 3.4	none	considerable	considerable	insignificant	insignificant
5-11-30	Beijing	No. 10	CC-1	Failure of retaining wall	sinkhole 500m^2, 16m deep, rupture of utilities, estimated loss = US$ 0.4 millions	none	serious	serious	insignificant	serious
5-12-04	Kaohsiung	Orange	MG-6	Failure of crosspassage	refer to Section 3.5	none	severe	disastrous	disastrous	disastrous
6-01-03	Beijing	No. 10	UT-1	Rupture of sewer	sinkhole 350m^2, depth 12m	none	serious	insignificant	insignificant	considerable
6-06-27	Beijing	No. 10	CC-1	Failure of retaining wall at work shaft	Sinkhole 30m^2, depth 3m, 2 workers died	severe	insignificant	insignificant	insignificant	insignificant
6-08-02	Guangzou	No. 3	ST-3	Settlement/sinkhole over shield tunnel	1 worker died and 2 were injured	serious	insignificant	insignificant	insignificant	insignificant

Table 2 Classification of Failure for Underground Works

Type of Work	Code	Failure Mode (see Note)
Open Cut	OC-1	Earth slip
	OC-2	Failure of base
	OC-3	Groundwater problems
Cut-and-Cover	CC-1	Structural failure of retaining system, Including failure of struts and walls
	CC-2	Failure of ground anchor
	CC-3	Leakage of wall
	CC-4	Failure of base soil
	CC-5	Groundwater problems, piping, uplifting
	CC-6	Settlement/sinkhole behind the wall
Shield Tunnelling	ST-1	Leakage at tunnel eye
	ST-2	Obstruction
	ST-3	Settlement/sinkhole over tunnel
	ST-4	Collapse of lining
Mining	MG-1	Settlement/sinkhole over tunnel
	MG-2	Earth slip at face
	MG-3	Failure of temporary support
	MG-4	Failure of permanent lining
	MG-5	Leakage of lining
	MG-6	Failure of crosspassage
Ground Treatment	GT-1	Ground heave/settlement
	GT-2	Damage to structure
Utility Problem	UT-1	Rupture of utilities
Others	OT	

The last two items are not considered herein because of the lack of information. In addition to the remaining 5 items, the authors are of the opinion that disruption to traffic may lead to great social loss and is an important item to be considered in classifying consequences.

It should be noted that the Guidelines are applicable to underground construction projects with a project value of approximately 1 billion Euro and duration of approximately 5 to 7 years. The construction of MRT lines, in general meet these two criteria, however, individual construction contracts are usually shorter in duration and lower in value.

(a) Injury to workers, emergency crew and third parties

ITA differentiates injuries to workers and emergency crews from injuries to third parties, as shown in Table 3, with much less tolerance for the latter in comparison with the tolerance for the former. The argument is that the third party has no benefit from the construction work and should not be subjected to a higher risk than if the construction work not being carried out.

Table 3 Classification of Injury (ITA, 2004)

	Injury to Workers & Emergency Crew	Injury to third Parties
Disastrous	F>10	F>1, SI>10
Severe	1<F≤10, SI>10	1F, 1<SI≤10
Serious	1F, 1<SI≤10	1SI, 1<MI≤10
Considerable	1SI, 1<MI≤10	1MI
Insignificant	1MI	

Note: F = fatality, SI = seriously injured, MJ = minor injured

Table 4 Classification of Economic Loss (ITA, 2004)

	Economic Loss to Third Party	Economic Loss to Owner
	million Euros	million Euros
Disastrous	>3	>30
Severe	0.3 ~ 3	3 ~ 30
Serious	0.03 ~ 0.3	0.3 ~ 3
Considerable	0.003 ~ 0.03	0.03 ~ 0.3
Insignificant	<0.003	<0.03

(b) Economic losses to third party properties

For the same reason, the tolerance for economic loss to third parties is much less than that to the owner. As shown in Table 4, the former is one-tenth of the latter.

Economic losses (damages) to third parties include damages to adjacent buildings, infrastructures, utilities, etc. It is nearly impossible to know the damage caused by any event without being involved. It is even more difficult to estimate the value of the damage because buildings of the same type of structure and of the same quality, for example, will have very different values depending on where they are. Therefore, unless the actual costs have been estimated, damages to third parties are proposed to be classified by the nature of the damages and the importance of the object which was damaged. Accordingly, Table 5 is recommended to replace Table 4.

In Table 5, no attempt has been made to differentiate buildings by type, floor area, age, importance, etc. However, it is necessary to differentiate lifelines and public facilities by their importance and Tables 6 and 7 are proposed.

Strictly speaking, damage to lifelines as defined in Table 5 is not limited to the direct costs for repairing or replacing the damaged lifelines, but also includes the social costs to the public due to the loss of services or the inconvenience caused.

It is rather difficult to classify public facilities in a similar manner because there are too many types of facilities with different sizes and functions, and classification of damage to facilities can only be based on intuitive judgment using Tables 5 to 7 as guidelines.

Table 5 Classification of Damage or Economic Loss to Third parties (this study)

	Buildings	Infrastructure
Disastrous	Collapse of a 10-story or taller building Collapse of 2 or more 5~10 story buildings	Permanent damage to vital lifelines or public facilities
Severe	Collapse of a 5~10 story building Collapse of 2 or more 2~5 story buildings Heavy damage to many 5-story or taller buildings	Major damage to vital lifelines or public facilities Permanent damage to major lifelines or public facilities
Serious	Collapse of a 2~5 story building Collapse of 2 or more single houses (bungalows) Collapse of a luxury residence Heavy damage to many 2~5 story buildings	Minor damage to vital lifelines or public facilities Major damage to major lifelines or public facilities
Considerable	Collapse of a single house (bungalow) Heavy damage to many single houses (bungalows), shops	Minor damage to major lifelines or public facilities Major damage to minor lifelines or public facilities
Insignificant	Minor damage to single houses (bungalows), shops Collapse of temporary structures	Minor damage to minor lifelines or public facilities

Note: refer to Tables 6 and 7 for definitions of lifelines and damage

Economic loss is not limited to damage to buildings, lifelines and public facilities. Frequently loss of ground led to sinkholes on public roads, and underground utilities may be damaged as a result. If a failure involved collapse of a section of road outside the site, the damage has been classified as "considerable" unless information is available to indicate more severe consequences. On the other hand, if a failure is totally confined in the boundary of the site, the loss is totally borne by the contractor and the loss to third parities will be classified as "insignificant".

(c) Total Economic Losses

It is recommended in the Guidelines that if it cannot readily be established whether additional costs are to be covered by the owner or by other parties, it should be assumed that the loss is defrayed by the owner. With limited information available, it is not possible to differentiate losses to owners and losses to contractors, therefore, total losses, no matter who is to bear them, are used in the classification.

Table 6 Classification of Lifelines (this study)

	Definition
Vital	more than 10,000 people will be affected by failure
Major	1,000~10,000 people will be affected by failure
Minor	less than 1,000 people will be affected by failure

Table 7 Classification of Damage to Lifelines (this study)

	Definition
Permanent damage	not repairable - replacement is required
Major damage	requiring more than 1 week to repair
Minor damage	requiring less than 1 week to repair

Table 8 Classification of Delays (ITA, 2004)

	Alternative 1 months	Alternative 2 months
Disastrous	>10	>24
Severe	1 ~ 10	6 ~ 24
Serious	0.1 ~ 1	2 ~ 6
Considerable	0.01 ~ 0.1	0.5 ~ 2
Insignificant	<0.01	<0.5

Note: Alternative 2 is adopted herein

(d) Delays

Delays can refer to delays to the critical path or delays of specific activity regardless of whether the activity is on the critical path. The two alternatives proposed in the Guidelines are given in Table 8. Alternative 1, with intervals in a factor of 10, is proposed therein in order to achieve only one risk matrix to cover all consequences, but the ranges proposed for "insignificant" and "considerable" are rather ambiguous. Alternative 2 is more meaningful and is therefore adopted herein. The critical path refers to the critical path for the pertinent construction contract, not the entire MRT line or the MRT network.

(e) Disruption to Traffic

Disruption to traffic may or may not involve heavy economic loss but will certainly cause inconvenience to road users. Closure of a major road, freeway, or MRT line may even cause political chaos. In the lack of precedents, the authors propose to classify disruptions to traffic in accordance with Table 9.

2.3 *Classification of Events*

The authors have collected information on 43 events, of which 23 events have at least one consequence classified as "considerable" or severer. These events are identified by the year-month-day of their occurrences and are listed chronologically in Table 1. The remaining 20 events were minor events with insignificant consequences.

Table 9 Classification of Disruption to Traffic (this study)

	Closure of a Major Road	Closure of a Freeway/MRT Line
Disastrous	> 3 months	> 1 month
Severe	1 ~ 3 months	1 week ~ 1 month
Serious	1 week ~ 1 month	1 day ~ 1 week
Considerable	1 day ~ 1 week	< 1 day
Insignificant	< 1 day	

Note: one level lower if a road is only partially closed

Table 10 Summary of Events in the Period 2001 to 2006

Code	Disastrous	Severe	Serious	Considerable	Insignificant
OC-1		1	1		1
CC-1	1	1	3		1
CC-3	1			1	1
CC-5					3
CC-6					3
ST-1	1	1		1	
ST-2				1	
ST-3			2	1	4
MG-1				1	
MG-3					
MG-6	2				1
GT-1					1
UT-1			1	2	4
OT			1		1
Total	5	3	8	7	20

Events can be classified by the highest level in the classification of consequences. An alternative which is frequently adopted is to assign a weight to each class, for example, 4 for disastrous, 3 for severe, 2 for serious, 1 for considerable and 0 for insignificant, and to classify events by summing up all the weights. Such a weighting system is inevitably arbitrary and is highly arguable. To avoid confusion, the former approach is adopted herein even though there is more than one consequence with the same level.

As information is rather limited, classification of both causes and consequences requires much guesswork and judgment based on vague, sometimes even controversial, reports in news. In many cases, it is uncertain whether failures of retaining systems (ST-3) were due to rupture of utilities (water main or sewer) or the rupture of utilities was a result of ground movements behind retaining walls. Similarly, settlements/sinkholes over tunnels could be a result of rupture of utilities, but on the other hand, rupture of utilities must be due to excessive ground settlements over tunnels. In most of cases, failures were due to multiple reasons. Therefore, much guesswork is required in determining the cause of each event. The same can be said regarding consequences. The authors have had to make their best judgments to classify consequences based on indirect evidence.

The events listed in Table 1 have been classified accordingly and a summary is given in Table 10. As can be noted, of the 23 significant events listed, 5 are disastrous, 3 are severe, 8 are serious and 7 are considerable. Although not specifically listed in Table 1, the death toll of 14 is considered to be alarming bearing in mind the fact that non-geotechnical events are not accounted for.

3 MAJOR FAILURES IN RECENT YEARS

The five events with disastrous consequences are discussed as follows:

3.1 *Event 3-07-01: Shanghai Metro (MG-06: Failure of Crosspassage)*

The collapse of a crosspassage at the west bank of the Huangpu River led to the collapse of an 8-story building and 2 annexes attached to this building. A 20-story building and several others also suffered from serious tilting. A section of levee, 30m in length, settled by a few meters and eventually collapsed. Water rushed in from the river through the opening and caused flooding of streets in a large area. However, there are no reports on the traffic disruption or damage to public facilities.

The crosspassage, 7.8m in length, was constructed at a depth of about 30m below surface by mining. The surrounding soils were solidified by using the ground freezing technique. At the time of collapse, excavation was almost completed with less than a meter to go. According to information available on the web, the collapse was caused by thawing of the frozen soil as a result of power breakdown.

Other sections of Line 4 were opened for revenue service at the end of 2005 as scheduled. The section influenced, from Da Mu Qiao Road Station to Lan Cun Road Station, is unlikely to be ready by the end of 2007, therefore, the delay is estimated to exceed 2 years.

The loss was estimated to be US$80m (Wannick, 2006) but details are unavailable. Presumably it includes economic loss to the owner, the contractor and the third parties. Most of the loss is believed to be covered by insurance. It has been reported that insurance premiums for MRT works in China (for example, new lines in Guangzhou Metro and Beijing Subway) have doubled or even tripled subsequent to this event.

3.2 *Event 4-04-20: Singapore MRT (CC-1: Failure of Retaining System)*

The collapse of a section of cut-and-cover tunnel led to the death of 3 workers from the contractor and 1 supervisor from the Land Transport Authority (LTA). A 150-m section of Nicoll Highway, which is one of the arteries of the southeastern Singapore Island, was seriously damaged (COI, 2005). The highway, refer to Fig. 1, was closed for 7½ months and was rebuilt at a cost of S$3 millions (US$2 million). Several buildings nearby the site were affected by settlement but none of them collapsed nor was seriously damaged.

The site is located in a piece of land reclaimed in the 80's. As shown in Fig. 2, the subsoils at this site contain two thick layers of marine deposits (the upper marine clay and the lower marine clay) and are underlain by the Old Alluvium which is a competent base stratum. Fig. 3 is a plot of the results of a cone penetration test carried out at the site. The excavation was supposed to be carried out to a depth of 33.5m and diaphragm walls with a thickness of 800mm (locally, 1000mm) were used. The collapse occurred on 20 April 2004 while the 10th dig was completed and excavation reached the depth of 30.5m on 16 April.

Fig. 4 shows the wall deflection paths, which are the plots of maximum wall deflections versus depth of excavation in a log-log scale, for the two inclinometers installed on the two sides of the excavation (Hwang, Moh and Wong, 2007). Wall deflections on the two sides were about the same till 9 March, 2004, when excavation reached a depth of 25m, and deflections of 198mm and 215mm were recorded by Inclinometers I65 and I104, respectively. Subsequently, there was a period in which I104 was not read because it was damaged. When monitoring resumed on 26 March, the deflection of the southern wall was found to have increased by 67mm to 282mm while the readings for Inclinometer I65 on the north were fairly steady in this period. The readings for Inclinometer I104 kept on increasing while those for I65 remained to be steady subsequently, presumably, because of the asymmetry of ground conditions. In fact, I65 appeared to move outward by 27mm, from 202mm on 26 March to 175mm on 20 April, as depicted in Figs. 2 and 4. On the other hand, Inclinometer I104 moved inward by 90mm to 441mm in the 3-day period from 17 April to 20 April.

Failure started as the waling on the northern wall buckled at 9am on 20 April and by 3pm all the struts for a 100m section totally failed. As Nicoll Highway sank, gas, water and electricity cables ruptured, causing power to go out for about 15,000 people and 700 businesses in the Marina and Suntec City area. Tremors were felt at Golden Mile Complex. Tenants and residents in the building were also evacuated. One of the spans of Merdeka Bridge was demolished as a precaution and was rebuilt shortly afterward.

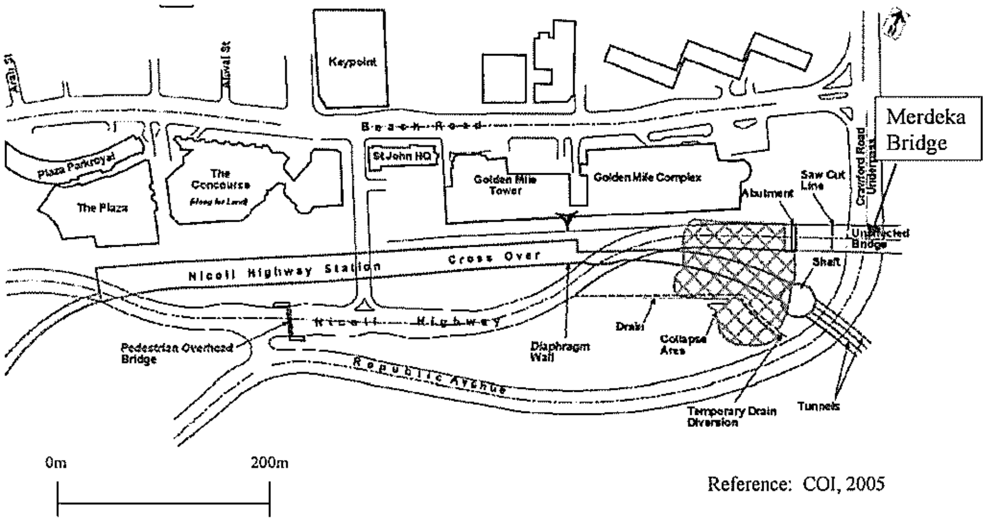

Fig. 1 Collapse of Nicoll Highway due to construction of the Circle Line, Singapore MRT

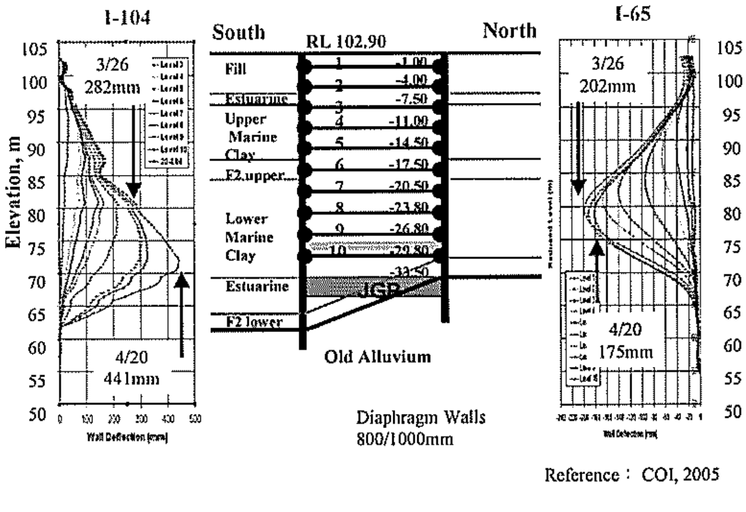

Fig. 2 Ground conditions and retaining system at the MRT site next to Nicoll Highway, Singapore

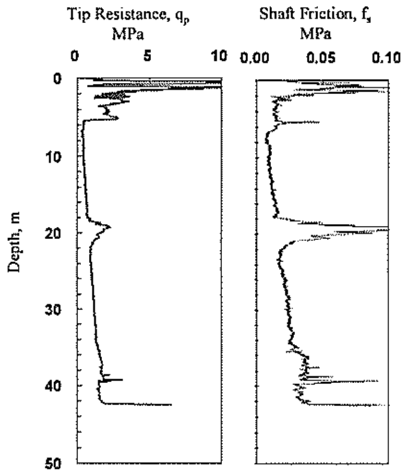

Fig. 3 Results of a cone penetration test next to Nicoll Highway, Singapore

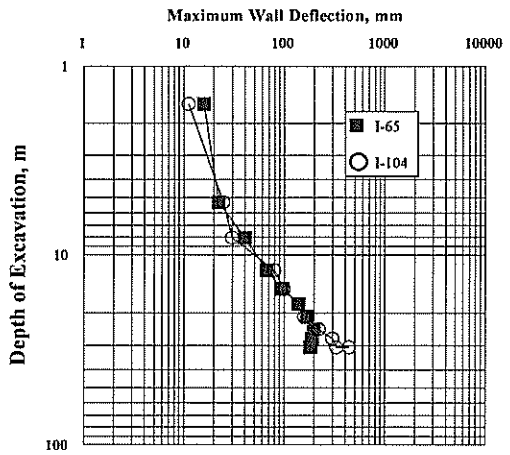

Fig.4 Deflection paths for diaphragm walls at MRT site next to Nicoll Highway, Singapore

As a result of this accident, the completion of the construction contract was postponed by at least a couple of years from the original schedule. Nicoll Highway Station has been shifted about 100 m southward from the accident site to Republic Avenue. The original station was meant to be the southern terminus of the future Bukit Timah Line but this has now been shifted to Promenada Station.

After the incident, the Singapore Government formed an independent Committee of Inquiry (COI), headed by a Senior District Judge, to look into the incident. After thorough investigations, in which 173 witnesses were interviewed and 20 experts offered their professional opinions, an Interim Report was released on 13 September 2004 and a very comprehensive Final Report was made available to the public on 13 May 2005 (COI 2005). Ministry of Manpower (MOM) also made a press release upon the publication of the Final Report and the key points made in the COI's report were quoted therein (MOM 2005a, MOM 2005b).

The Committee identified critical design and construction errors, particularly the design of stiffeners on the walings at the connections between the diaphragm walls and the struts, that led to the failure of the earth retaining system. The Committee also found deficiencies in the project management that perpetuated and aggravated the design errors, including inadequate instrumentation and monitoring of works, improper management of instrumentation data, and lack of competency of persons carrying out specialized work.

3.3 *Event 4-05-29: Kaohsiung MRT (ST-1: Leakage at Tunnel Eye)*

As the shield machine in the Up-track tunnel arrived at O2 Station and a portion of diaphragm wall was knocked out to make a portal at the tunnel eye, groundwater spurted into the pit (Lee *et al.*, 2005). The site is very close to the seashore as shown in Fig. 5 and was a salt pan decades ago. It is located in a delta formed by the estuary of the Ai River. Groundwater table is very close to the surface and is influenced by the fluctuations of tides. The ground conditions at the site are shown in Fig. 6 and the N-values obtained in SPT tests at a nearby borehole are shown in Fig. 7. The properties of subsoils below the invert of the tunnel are given in Table 11. Although the SPT blow counts are in general greater than 15, subsoils consist predominantly of silts with natural water contents very close to their liquid limits. Such soils may easily be softened once disturbed or even liquefied when subjected to large hydraulic gradients.

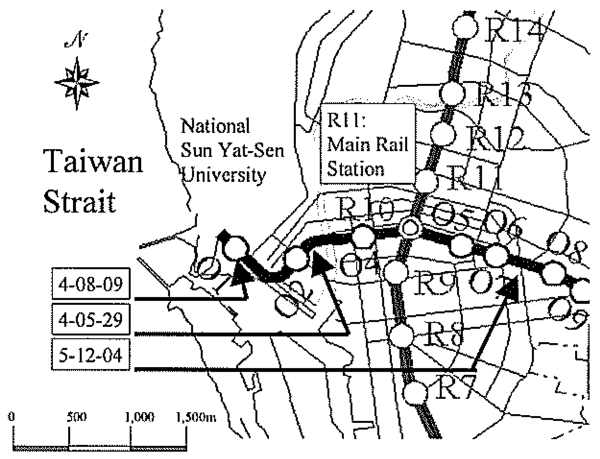

Fig. 5 Western segment of the Orange Line, Kaohsiung MRT

Leakage started at the invert of the tunnel eye at 20:30 on 29 May 2004 and ground started to settle at 22:30 as the flow increased and sands were washed out. A sinkhole was finally formed and measurable ground settlements reached a distance of 50m. Seven 3-storey buildings in the vicinity were seriously damaged as the foundations settled by more than a meter and about 40 families were evacuated. The buildings were demolished shortly afterwards and had already been rebuilt by the time this paper was prepared.

The tunnel was driven using an earthpressure balance shield machine with a diameter of 6.3m. The axis of the tunnel is located at a depth of, roughly, 17m below surface. At the time the accident occurred, the portal had been completed and the cutter of the shield machine had intruded into the portal by 700mm.

Table 11 Properties of Subsoils Below the Tunnel at the Eastern End of O2 Station, Kaohsiung MRT (after Lee *et al.* 2005)

SPT Sample	Depth m	Soil Type	Sand %	Silt %	Clay %	water content %	Liquid Limit %	Plasticity Index
S-14	20.55	ML	19	76	5	22.4	27	4.1
S-15	22.05	SM - ML	48	52	0	25.6	21.4	0.8
S-16	23.55	ML	15	82	3	24.4	27.5	1.2
S-17	25.05	ML	4	86	10	19.5	35.2	6.4
S-18	26.55	ML	20	74	6	21.7	26.5	3.3
S-19	28.08	CL	6	56	38	42.9	43.9	20
S-20	30.00	ML	15	84	1	25.1	26.1	1.2

Table 12 Chloride Contents at O1, O2 and O3 Stations of the Orange Line, Kaohsiung MRT (after Lee *et al.* 2005)

	Chloride Concentration (ppm)
Groundwater	10,442 12,546 13,796 11,197 *17,000 15,300
Soils	*3,000 *8,500

Notes: * by Mass Rapid Transit Bureau, Kaohsiung

As shown in Figs. 6 and 8, the ground behind the diaphragm wall was treated by installing JSG piles, with a spacing of 1.6m center to center, in November 2003 as a precautionary measure. Treatment was carried out for the full face in the 4.71m section immediately next to the diaphragm wall, but was carried out partially, leaving a soil core in the middle, in the remaining section away from the wall. As a normal practice, six water tests were carried out to determine the quality of the treated ground. Minor leakages occurred and ARON and LW grout was injected to seal off water paths. The attempt obviously failed to achieve the purpose.

Failures of trenches occurred frequently in this area during the installation of diaphragm walls. Therefore, one row of CCP piles of 350mm in diameter was sunk in November of 2002 to prevent the trenches from collapsing prior to the installation of diaphragm wall panels. Subsoils at the site have high chloride contents as depicted in Table 12. According to CNS13961 for concrete mixing, the chloride contents of water are limited to 250ppm while, as depicted in Table 12, the chloride contents in the groundwater exceed 40 times this limit. Since the CCP piles were intended to serve only for temporary purpose, Type I Portland cement was used. Therefore, it is suspected that, as a result of chloride attack, the CCP piles had already deteriorated by the time JSG was performed a year later and could be cut into fragments by JSG. The presence of these fragments might have led to poor quality of the treated soils at the interface between CCP piles and JSG piles and water paths might have been formed as a result, allowing groundwater to seep into the pit. It is also possible that the quality of the ground treatment was poor at the bottom as the soils were much disturbed at the tip during JSG operation and they were not well mixed with cement (Lee *et al.* 2005) as the grouting pipe was raised. In case this was indeed the cause of failure, it will be a good idea in the future to sink grout pipes to, and to start the injection of grout at, different depths to avoid forming a continuous blanket with high permeability at the bottom.

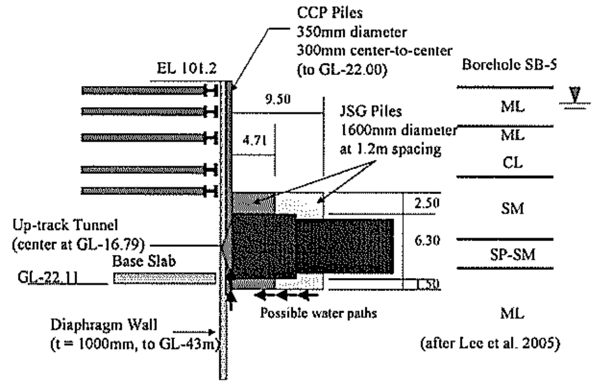

Fig. 6 Collapse of tunnel portal, O2 Station, Kaohsiung MRT

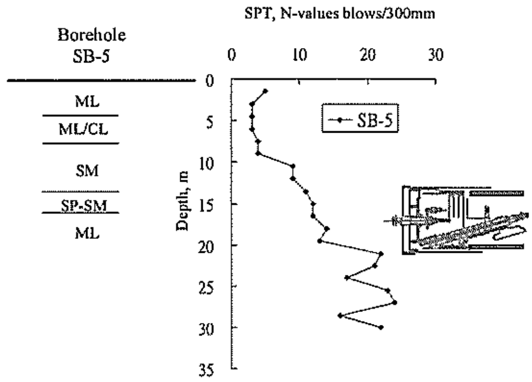

Fig. 7 N-values obtained in Borehole SB-5 next to O2 Station, Kaohsiung MRT

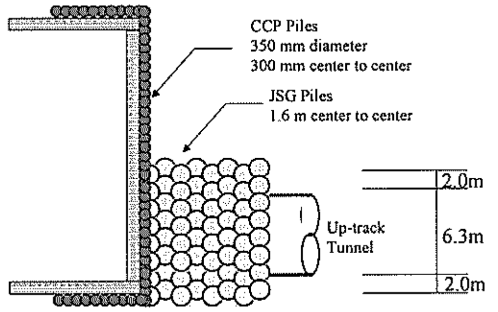

Fig. 8 Ground treatment at tunnel eye portal, O2 Station, Kaohsiung MRT

The attempt to recover the shield machine was abandoned and the shell of the shield was left in the up-track tunnel. At the time the incident occurred, the shield machine in the down-track tunnel was still far away from the station. To prepare for its arrival, a steel chamber was installed at the portal so compressed air could be applied to stop water flow should a similar incident occur again. Since the ground was much disturbed, additional grouting was carried out to solidify the ground. However, the ground was then too hard for the shield to go through because of over-treatment, therefore the shell was also abandoned in the down-track tunnel. A new shield machine had to be purchased for other sections of tunnels to be driven.

One of the two lanes of the street next to the site was closed for a few months. Because the local traffic is not heavy, the disruption to traffic was not serious. The completion of the construction contract was delayed by 8 months.

3.4 *Event 4-08-09: Kaohsiung MRT (CC-3: Leakage of Diaphragm Wall)*

Water spurted from the bottom of the excavation in front of Panel S60M, refer to Fig. 9, on the southern wall at 13:20 on 9 August 2004 as excavation reached a depth of 15m at O1 Station which is located near the seashore as shown in Fig. 5. A sinkhole of 3m maximum depth was formed behind the diaphragm wall and the area affected was around 500m^2. Four 3-story buildings collapsed in less than an hour and were demolished overnight. Several low-rise shops were severely damaged and were demolished sometime later.

The site is located right at the mouth of the Ai River and the ground conditions are extremely poor. Fig. 10 shows the soil profile obtained at Borehole WB-11. The pit was retained by diaphragm walls, 800mm in thickness and 39m in length. As mentioned in Section 3.3, subsoils on the west coast of Kaohsiung City can easily be softened once disturbed, or liquefied when subjected to large hydraulic gradients and, therefore, failures of trenches during installation of diaphragm walls were quite common. In many cases, mini-piles and/or micro-piles were installed to prevent trench collapse. Even so, necking frequently occurred and reduced the sectional areas of diaphragm walls. Furthermore, groundwater has extremely high chloride contents and the quality of diaphragm walls deteriorates quickly as a result of chloride attack. Ground treatment frequently had to be applied behind defective diaphragm walls to stop ingress of groundwater into pits.

Subsoils at this site, being closer to the sea, are even worse than those at O2 Station so the incident was not a surprise. One row of CCP piles was used along the perimeter of the area to be excavated for maintaining the stability of trenches prior to the installation of the diaphragm walls. After the incident, coring was carried out at the end of September 2004 and it was found that Panel S59F was defective with pockets of rock fragments and soils at depths ranging from 15.95m to 16.55m below surface. One row of 11 bored piles was installed behind Panels S58M to S60M. Pumping tests were performed in November 2004 to see if there were other defective panels. A total of 3,285 cubic meters of water was drawn from 6 wells and water levels inside the excavation closely monitored at 60 wells. The groundwater table inside the excavation dropped; on an average of 2.2m as a result of pumping. The recovery of water levels was monitored for more than 10 days, however, the desired purpose was not achieved as the locations of leakages could not be identified (Ho *et al.*, 2007).

To be on the safe side, one row of JSG piles was added along the entire perimeter of the station. Furthermore, the joints between JSG piles were treated by using CCP piles. Pumping tests were again performed subsequently to confirm the effectiveness of these measures. The results were not satisfactory as the rate of recovery of groundwater inside the excavation was only slightly smaller than that obtained previously (Ho *et al.*, 2007). It was decided to add more JSG piles at the back of Panel S58M. Three new piles were installed without problem. As No. 4 pile, refer to Fig. 9, was installed on 7 July 2005, groundwater brought a large quantity of soil into the pit. A nearby hospital was endangered and the patients in the hospital were urgently evacuated as a precautionary measure. It however survived with only minor damage. The sinkhole was about 1m in depth and settlement spread over an area of about 1,000m^2.

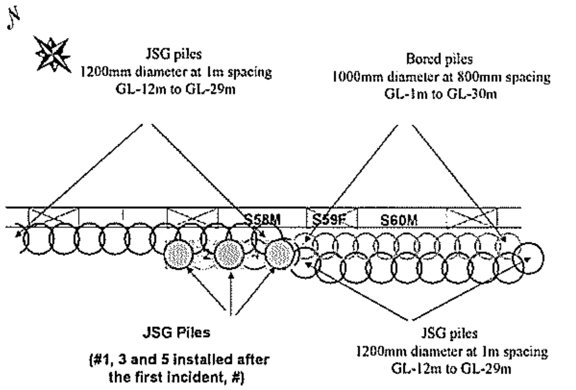

Fig. 9 Ground treatment prior to July 7, 2005, O1 Station, Kaohsiung MRT

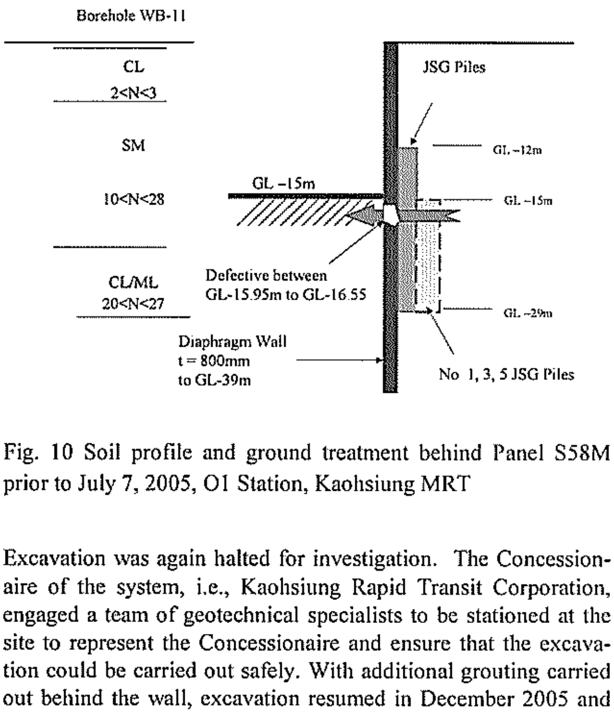

Fig. 10 Soil profile and ground treatment behind Panel S58M prior to July 7, 2005, O1 Station, Kaohsiung MRT

Excavation was again halted for investigation. The Concessionaire of the system, i.e., Kaohsiung Rapid Transit Corporation, engaged a team of geotechnical specialists to be stationed at the site to represent the Concessionaire and ensure that the excavation could be carried out safely. With additional grouting carried out behind the wall, excavation resumed in December 2005 and took 6 months to reach the bottom.

3.5 *Event 5-12-04: Kaohsiung MRT (MG-6: Failure of Crosspassage)*

As excavation was carried out at the bottom of a crosspassage, refer to Fig. 5 for the location of the site, for constructing a sump, water started to spurt out at 5pm on 4 December 2005 and the flow soon became uncontrollable because of the huge water pressure. The center of the crosspassage is located at a depth of about 26m below the surface and excavation was supposed to go down to a depth of, roughly, 33m for constructing the sump as depicted in Fig. 11. Excavation for the sump, with an outer diameter of 3.9m, was carried out in 7 stages, each 0.5m in thickness, as shown in Fig. 12. The sidewall of the excavation was protected by shotcrete as temporary lining. At the time the incident occurred, excavation had reached the last stage and half of the area had already been completed.

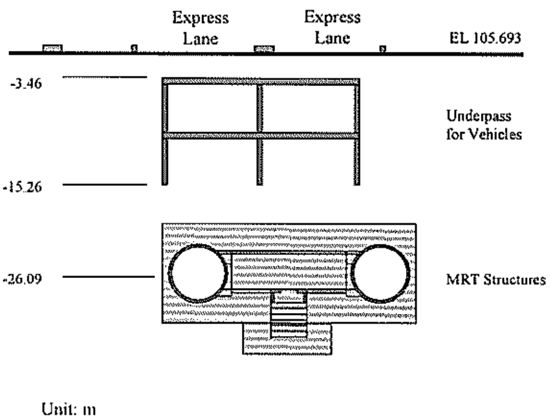

Fig. 11 Section at the location of the crosspassage and the sump

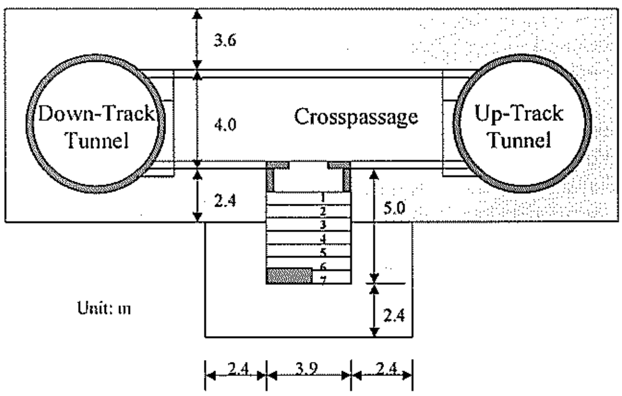

Fig. 12 SJM treatment for excavation for the crosspassage and the sump

The soils surrounding the crosspassage and the sump had been treatment by using the SJM (Superjet Midi) grouting method prior to the commencement of excavation as depicted in Fig. 12. The quality of ground treatment might have been affected by the fact that grouting had to be carried out mostly in inclined holes because of the presence of an underpass running in parallel with the tunnels. Water tests were carried out to confirm the effectiveness of the treatment. Additional grouting was carried out to seal off water paths when seepage was noticed. This however did not prevent the failure from happening.

The situation was aggravated by the rupture of 300mm and 600mm water pipes. The tunnels, together with the underpass above the tunnels, about 100m in length were totally damaged as a result. The tunnels and the underpass will be rebuilt by using the cut-and-cover method of construction with diaphragm walls to depths exceeding 60m. Ground freezing technique will be applied at the two ends of the damaged section to seal off the gaps between the diaphragm walls and the tunnels and to solidify the sludge in the tunnels to become plugs to enable excavation to be carried out. Finally, these plugs will be removed and damaged segments replaced.

Ground freezing work started on 27 June 2006 and the rehabilitation of the tunnels is expected to be completed by mid-2007. The economic loss is estimated to exceed US$50 million which is considered to be the biggest loss in the Kaohsiung MRT construction. Other than the underpass, damage to third parties was small because the sinkhole is located in a park and there were no buildings nearby. Although the underpass will be closed till the end of 2007, fortunately, space is available to divert some of the traffic at surface to lessen the impact.

4 LESSONS LEARNED

Lessons learned from failures contribute to the advancement of technology and reduce fatalities and economic losses in future construction. The recent failures quoted above indicate that:

4.1 *Water Is a Prime Cause of Failures*

Water is a prime cause of major failures in underground construction. For example, four of the five disastrous events, discussed in Section 3, were associated with ingress of groundwater. Therefore, works must be carried out with great care whenever and wherever excavation is carried out in water bearing strata. This is particularly true if openings are to be made on underground structures at great depths, for examples, for making tunnel portals through diaphragm walls.

In the first stage construction of the Taipei Rapid Transit Systems (TRTS) carried out in the 90's, all the top four failures with disastrous or severe consequences were also caused by ingress of water and two of them occurred at tunnel portals. There exist the Chingmei Gravels, which is practically an underground reservoir with extremely high permeability, at depths varying from 35m to 60m below ground surface in the Taipei Basin, and it will be very difficult to stop the flow once water finds its way to seep into any openings. Learning from failures in these 2 cases and near-failures in several other cases, bulkheads were provided in a few contracts in the subsequent constructions to enable the portals to be sealed off once the flow became uncontrollable. In some cases, ground freezing, in lieu of or in addition to grouting, was adopted to solidify surrounding soils and to seal off any fissures or cracks. Ground freezing was adopted more frequently in the second stage TRTS construction because excavations go deeper and also because the technique has gained popular confidence.

Construction of crosspassages in sandy strata is also a very dangerous operation, particularly under rivers. It is very popular to adopt ground freezing to solidify surrounding soils for excavation to be carried out safely. An alternative is to use double-O-tube (DOT) shield machines which, to the authors' knowledge, have been used successfully in 13 projects in Japan and 5 projects in Shanghai. The Department of Rapid Transit Systems of Taipei City Government is also evaluating the feasibility of adopting DOT machines in some of the sections in the future extensions of TRTS.

Defective retaining walls may also lead to severe consequences as groundwater may seep its way into the pit/excavation, leaving large cavities behind the walls. Leakage in diaphragm walls has frequently occurred in the coastal area in the Kaohsiung City in the past because the subsoils consist predominately of silty sands and sandy silts with water contents very close to their liquid limits, making it very difficult to maintain the stability of trenches and resulting in necking of walls. In many cases, micropiles or minipiles were installed to stabilize trenches, but even this did not fully solve the problem. Grouting was frequently carried out at the back of the walls which were suspected to be defective, but failures still occurred. The results of on-going research indicate that the silty sands in the Kaohsiung area are unique with strange behavior once disturbed. This, however, is beyond the scope of this paper.

Failures resulting in sinkholes also frequently occurred as soils were washed away when utilities (watermains, sewers, drains, etc) ruptured as a result of large ground deformations. Such incidents should not have happened because movements of ground and utilities should be closely monitored and precautionary actions should be taken.

Another factor supporting the view that water is a prime cause of major failures is the lack of them during construction of the MRTA Chaloem Ratchamongkhon Line in Bangkok from 1997 to 2004. The soil profile in Bangkok means that at typical excavation levels of 15 to 30 m below ground surface, the strata are firm to stiff clays, and dense silty sands in which the ground water level is already drawn down by about 20 m. Thus the very high water pressures which existed and caused problems in Taipei and Kaohsiung are not present in Bangkok.

4.2 *Design Codes on Ground Treatment Require Revision*

Ground treatment is routinely applied to solidify the surrounding soils and to seal off any fissures or cracks, when openings are to be made in underground structures. In four of the five cases quoted above, failures could have been avoided if ground treatment had served its purpose. Even for the case of the collapse of the Nicoll Highway in Singapore, refer to Fig. 2, a grouted slab was placed below the 9th level of struts and served as a temporary support and another grouted slab was placed below the bottom level of excavation. Although it was concluded that the poor quality of these slabs was not directly responsible for the failure, it is arguable whether failure would still have happened if they had done their duty.

To the authors' knowledge, ground treatment in this region is either designed in accordance with Japanese codes or designed to codes which were developed mainly based on Japanese codes/experience. This has been proven to be inadequate as the quality of the treatment is highly dependent on the equipment used and workmanship. The equipment used in this region may not be as capable as those used in Japan in cutting soils and forming piles and the workmanship may not be compatible with that in Japan. One of the factors attributing to the differences between this region and Japan is the prices awarded to the specialist contractors. The price of grouting in this region is far below that in Japan and it is thus unreasonable to expect the workmanship to be the same as that in Japan. It is therefore important to develop local codes based on local experience to account for these differences. Furthermore, most, if not all, of the codes of practice on grouting fail to deal with the situations with steep hydraulic gradients. Therefore, they may be inadequate for excavations with depths of, say, 15m or greater, or making openings in underground structures at similar depths.

As shown in Fig. 6, the treatment was thicker above the crown of the shield than that below the invert. For shield tunneling, close-in of surrounding soils toward the shield machine is only a minor concern. Therefore there is no good reason for the treatment to be thicker above the crown. On the contrary, the treatment below the invert should have been thicker because the water pressure was greater. For the same reason, it is reasonable to expect the treatment below the bottom of the sump shown in Fig. 12 to be thicker than that below the bottom of the crosspassage.

Treated ground tends to be brittle, and cracks may develop as geo-stresses are changed as a result of excavation. These cracks may be undetected during water tests. For tunnelling using the mining technique, it may be a good idea to treat the ground to form circular (or annular), instead of rectangular, sections. The treated ground will then behave as a compression ring and cracks, if any, tend to close up as the core is excavated while tension cracks are likely to develop at the mid-spans of the four sides if rectangular sections are used.

If the treatment is to serve a long-term purpose, say, more than 6 months, the effects of chemical content in groundwater on the quality of ground treated by grouting should not be overlooked and the cement used should be sufficiently resistant to the chemical attacks.

As excavations go deeper and deeper, revision of codes is urgently needed with water problems properly addressed. Before such a time, to compensate for the deficiency in quality, it is suggested extra thickness of treatment be provided. This will effectively increase the lengths of water paths, if any, and reduce the potential of piping.

4.3 *Quality Control of Ground Treatment Is Vital*

It is difficult for supervising engineers, even experienced geotechnical engineers, at sites to be sufficiently knowledgeable to perform their duties of ensuring the quality of the works because even minor details in handling the machines will affect the quality of the product and only skillful technicians know the techniques. Commercial pressures mean that it is natural for the specialist subcontractor to save time and material by speeding up the operation and/or adding more water to the mix, and the effects on strength are very difficult to monitor.

For treatment to be carried at great depths, the tips of grout pipes (or freezing pipes) could easily be off their intended locations by a few hundreds of millimeters as the alignments of pipes deviate from the vertical. It is even more difficult to ensure the alignment for inclined holes which are necessary because of site constraints or because of the presence of obstacles. The case shown in Fig. 11 is an example in which grouting had to be carried out mostly in inclined holes due to the presence of vehicular underpass structures. In cases like that, grout piles may not overlap each other as they are shown on design drawings and the gaps between piles are likely to become seepage paths for water. Therefore it is necessary to be extraordinarily cautious whenever and wherever risks are evident. In critical cases, inclinations of grout pipes must be measured and the tips of grout pipes located to see if grout piles indeed interlock as desired, and at least 2 rows of grout pipes are necessary to ensure that gaps, if any, between the grout piles will not be continuous.

Check boring is normally specified for confirming the quality of treatment but, more often than not, the importance of strength of treated ground was over-emphasized while uniformity was overlooked. Usually, the best parts of the cores are tested for determining the strengths of treated ground but, in fact, it is the poorest parts that lead to failures. Therefore, test results are not representative of the entire body of treated ground nor are they meaningful for practical purposes. Instead of strength, the uniformity of the treatment should be the key in judging the quality of treated ground. Although the use of core recovery ratio and rock quality designation as indices for the quality of treated ground is challenged by many engineers, it remains the most practical approach until better ways have been found.

4.4 *Computer Analyses Shall Not Be Trusted Blindly*

The advancement in technology for the design and installation of retaining systems has indeed reduced failures drastically in recent years. For example, the adoption of performance-based design, instead of capacity-based design has reduced wall deflections, and hence settlements behind walls, by a factor of 3 as proved by comparing the inclinometer readings obtained during the construction of the Taipei Rapid Transit Systems in the 90's with those obtained previously. This becomes possible because of the availability of advanced computer codes which enable complicated geometry and nonlinear soil behavior during excavation to be modeled. Yet, a disastrous failure occurred (refer to Section 3.2) in the construction of the Singapore MRT System as a result, mainly, of inadequate design of the retaining system based on erroneous computer analyses due to the adoption of a wrong soil model. Failure did not occur without warning as wall deflections exceeded their design values in the very early stages of excavation. Back analyses were carried out, but with improper procedures, to predict what would happen in the subsequent stages and tolerances were relaxed accordingly as the results appeared to indicate that the retaining structures would still be safe.

As engineers rely on computers more and more for their daily works, they have lost engineering sense and can no longer judge the reasonableness of the results obtained because they no longer have to go through the procedures step-by-step and understand the mechanism involved. In fact, most failures could be attributed to improper use of computer codes, rather than improper codes. While the reliance on computer codes is an irreversible trend, the profession is urged not to ignore the fundamentals of geotechnical engineering and not to abandon empirical approaches totally. The results of computer analyses should always be checked against experience until the day computer codes are confirmed to be truly reliable and are easy to be used by all the engineers without misconception.

4.5 *Most Failures Could Have Been Avoided*

With modern technology, poor ground conditions should not be an excusable reason for failures to occur. Most failures could have been avoided if potential risks were managed properly. As mentioned in Section 3.2, a Committee of Inquiry was formed by the Singapore Government to investigate the cause of the collapse of the Nicoll Highway. After interviewing 173 witnesses and 20 experts, the Committee concluded that "The collapse did not develop suddenly. A chain of events preceded the collapse" and "The Nicoll Highway collapse could have been prevented".

There is a growing awareness of the importance of risk management. However, although various measures have been adopted in nearly all the major construction projects for reducing potential risk and for ensuring construction safety, it is desirable to establish a system so that risks can be managed properly and the efforts spent can be optimized.

5 RISK MANAGEMENT

Since the investigation conducted by COI was carried out to an unprecedented depth and the Singapore Government has taken many positive measures to prevent similar events from happening, the case can be considered as a milestone for underground construction and the measures taken set new state-of-the-art standards for managing risks involved in underground constructions.

The COI made specific recommendations on the following issues (COI, 2005):

(1) Effective Risk Management
(2) Managing Uncertainties and Quality
(3) Management and Monitoring of Geotechnical Instrumentation and Data
(4) Robustness of Design
(5) Design Review and Independent Check
(6) Numerical Modeling in Geotechnical Design
(7) Jet Grout Piling (JGP)
(8) Codes and Specifications
(9) Stop Work
(10) Emergency Preparedness
(11) Competence of Professionals, Contractors and Sub-contractors
(12) Contract and Tender Evaluation System
(13) Safety Culture
(14) Chain of Command
(15) Independence of QP
(16) Building Control Functions

These issues are essentially the major elements in risk management programs and the recommendations given in the report can readily be adopted to form the framework of risk management programs.

In response to COI's recommendations, a Joint MND-MOM Review Committee was convened to examine safety standards in the construction industry. In respect to geotechnical engineering, the Singapore Government, inter alia, has adopted the following measures (MND, 2005):

<u>Reform of Occupational Safety and Health (OSH) Framework</u>

- Contractors will be required to have a comprehensive safety and health plan for every worksite, which includes a structured risk assessment, names of persons responsible for safety for each aspect of work, and contingency planning. The risk assessment must also address the potential risk/impact of the work to members of public. There must be emphasis on translation of the plan into ground action.
- Developers as a key stakeholder will in future also be required to ensure that the designers they appoint also assess and address any major design risks.
- Contractors' safety and health plans must consider the flow of safety information among parties. Their risk as-

sessments must clearly set out the triggers for stopping work.

By MND-MOM Review Committee

- Requirement for OSH management certification (OHSAS18001 standards) will be extended to all contractors in BCA's Contractors Registry System.
- Tougher penalties for professionals (PEs and architects) who do not perform their duties with due diligence. The maximum period of suspension for moderate contraventions should be raised from 1 to 2 years.
- Licensing will be introduced for specialist contractors engaged in critical geotechnical work such as soil investigation and instrumentation.
- PE(Geotec) will be responsible for instrumentation and monitoring for TERS in deep excavations, including data interpretation and specification of review trigger levels.
- AC(Geotec) will have to conduct field reviews and site inspections, including review of data interpretation and trigger levels.
- It will be mandatory for PE(Geotec) and contractor's Technical Controller to stop works when the relevant trigger levels are exceeded.
- Construction of all temporary structures will have to be supervised and certified by a PE before being subjected to its intended loading. The PE for the permanent works should be consulted where appropriate.
- AC(Geotec) will be required to undertake independent review of design.
- AC(Geotec) has to meet prescribed requirements in terms of qualifications and experience in geotechnical works.
- AC(Geotec) has to be appointed independently by the owner and not the contractor, as with the AC in permanent works.

By Land Transport Authority (LTA)

- LTA has set up comprehensive risk registers for each site. The risk register identifies the hazards/risks involved in each construction activity with associated risk index and describes the risk mitigation measures to be undertaken to bring the residual risks to an acceptable level.
- On each contract/project there will be weekly design review and instrumentation and monitoring meetings.
- Design and quality shortcomings as well as instrumentation breaches of alert and work suspension levels are to be immediately reported to higher management.
- For new projects, LTA will be engaging specialist instrumentation contractors to carry out the instrumentation and monitoring works.
- For ongoing projects, where there are existing contracts, LTA has instituted quality control of the instrumentation contractors.
- LTA has engaged external independent consultants / QP (Supervision) to check the design of temporary works prepared by the contractor.
- To ensure the independence of the PE (Temporary Works), LTA will require that the PE (Temporary Works) engaged by the contractor shall not be an employee of the contractor or have economic interest in the firm.

Note: The abbreviations quoted above are:

BCA = Building and Construction Authority of Ministry of National Development
PE = Professional Engineer
AC = Accredited Checker
TERS = Temporary Earth Retaining Structures
QP = Qualified Person = Person-in-Charge

Safety related instrumentation works are now directly tendered out to specialist contractors by LTA, and these specialist contractors will report to LTA, instead of contractors. Late in 2004, LTA also engaged an independent party to carry out the geotechnical services, refer to Section 8.2, for the Circle Line and such a practice is expected to continue in the future. The Government has also legislated a new category in professional registration for geotechnical engineers and requires underground works to be certified by Professional Geotechnical Engineer. It is also compulsory that designs of underground works be checked by accredited checkers.

6 FUNDAMENTAL ISSUES

The collapse of Nicoll Highway mentioned above is by no means an isolated case. As listed in Table 10, there have been 5 disastrous events, 3 severe events and 8 serious events associated with the constructions of rapid transit systems in this part of the world since year 2001, resulting in deaths, serious delays and heavy economic losses.

It should be noted that consequential loss could be many times the direct loss to a project, if failure extends beyond the site boundaries. Furthermore, the frequent occurrence of accidents reduces the confidence of the public in construction safety, and property owners become more and more resistant to construction carried out near their properties. This not only hampers the progress of projects but also indirectly jacks up construction costs.

While there are numerous factors to be considered in risk assessments, there are several fundamental issues which are more influential than others:

6.1 *Quality Shall Be Emphasized Over Prices*

The fundamental elements for poor quality of works are budget constraints, construction period and mentality of all the parties involved. For example, the policy of awarding projects to bidders with lowest prices will certainly reduce the quality of works. Because of the competition, whoever wins the contract will inevitably takes more risks than he should and cut corners to remain profitable. This issue is addressed in the COI Report that "A strict weightage system should form part of the contract and tender evaluation system. The weightage system should include non-technical and non-commercial attributes such as safety records and culture of the bidder, and its core or corporate competency. Such a weightage system should apply even if the tenderer is a joint venture or a consortium" (COI, 2005). MND-MOM JRC responded to this directive by stating that "Use of current price and quality (including safety) attributes in tender evaluations will be formalized for public sector projects through the Price-Quality Method (PQM). This is recommended as the preferred method of procurement for public works." (MOM, 2005b, MND, 2005). LTA also states that "For all its major projects, LTA adopts a prequalification process to evaluate, assess and pre-qualify potential bidders based on their technical competencies shown in past projects and their safety performance track record. A more structured safety dimension will be added for future tender evaluation in line with the proposed Price Quality Method (PQM)." However, in practice, prequalification does not give good contractors any margin on bidding, and at the end price still plays a dominating role in tender evaluation and unhealthy competition still continues. It is therefore suggested that preferential status be given to contractors with better safety records and a certain margin be allowed on their bidding prices. This also complies with the FIDIC's recommendation of "Selection on Quality". Selection of designers is even more critical than selection of contractors and shall be based on quality rather than price because design fees are only a few percents of construction costs while a poor design, either over-design or under-design, might lead to economic losses which could be many times the design fees.

Mode of tendering is another factor to be considered in evaluation of potential of risks. It has become more and more popular for projects to be tendered on a turnkey basis. Turnkey projects (design-and-construct), in contrast to Engineer's-design projects, do have the benefits of speediness and cost-saving, but often at the expenses of quality of works. To avoid any deficiency in quality, stringent specifications and tight supervision are necessary. This may or may not work because specifications sometimes are difficult to enforce at sites. Engineer's-design projects, on the other hand, have the drawback that there is a discontinuity in concept between the designer and the contractor and, as a result, some of the safety measures adopted in the design may not be appreciated by the contractor. This situation becomes more serious if the designer is not playing a major role in supervision during construction. As pointed out by COI that "problems in the inter- and intra-party chain of command and communication between the project owner, contractor and subcontractors; and lack of clarity in the reporting structure for decision-making among the different parties involved in the project" were partly responsible for the collapse of Nicoll Highway. For a MRT line, there are several designers and many more contractors and all of them have different design concepts and diverse approaches. Miscommunication and breakdown of chain of command are often the causes of failures. It will thus be useful to have someone with the specific duty of performing system-wide risk assessment and standardizing safety-related operation procedures.

6.2 *Responsibility Shall Be Clearly Defined*

It is also important for everyone involved in a project to realize the responsibility he is taking and the legal consequences he will face if he fails to perform his duties. In the case of Nicoll Highway, COI attributed "Lack of safety culture" to be a primary factor responsible for the collapse, and recommended criminal charges against 4 persons, 3 from the contractor and 1 from LTA. Engineers are now more alert and cautious about safety and the safety culture will certainly be improved. However, on the other hand, over-caution has the side effect that engineers either look for ways to disclaim their responsibility or become reluctant to give their approvals making it difficult to proceed with the construction. It is therefore important to clearly define everyone's duties and the associated responsibilities and avoid ambiguities in job descriptions. It is also essential to ensure that senior positions, where responsibility needs to be taken, are filled with people who have enough relevant experience to take their decisions with confidence.

6.3 *Information Shall Be Shared Promptly*

Information sharing is a vital element in project management and a hybrid computer system incorporating MIS (management information system) and GIS (geographical information system) functions will enable engineers to visualize what is going on at sites and, at the same time, to obtain all the information they need at their finger tips. The system shall have the capability of checking instrument readings for accuracy and reliability. Information, such as borehole logs, buildings and structures, instrument locations and readings, utilities, etc., can be linked to warnings as soon as abnormal instrument readings are detected. This will enable all the parties to be in a position to take necessary actions promptly.

6.4 *Insurers Shall Participate In the Works*

Insurance is an important element in risk management, as a lifeboat whenever disastrous events occur. However, because of the frequent occurrence of failures, insurers suffered great losses and have become reluctant to cover underground construction. Insurance premiums have increased drastically in the past 10 years and indirectly jacked up construction costs. Many projects even face the difficulty of obtaining insurance coverage.

In most projects, insurers are not involved in the works till after a failure has occurred. Once a failure occurs, the insurer is in a disadvantaged position because of lack of information. It will be more effective for an insurer to alert the contractor or the project owner to pay attention to signs of dangers and to take necessary actions because insurer may refuse to pay for damages due to negligence or nonperformance. Therefore, it is recommended that insurers have their representatives at sites and obtain safety related information, for example, progress of excavation and instrument readings. It is to everyone's benefits to reduce the occurrence of failures so insurance premiums can be reduced. However, insurers and their representatives shall not be empowered to interfere with the works.

7 PROTECTING OTHERS AND BEING PROTECTED

It is usually mandatory for contractors to protect adjacent buildings and properties during construction of rapid transit systems and procedures have been more or less standardized. Firstly, designers have to identify buildings and properties which are likely to be affected by the construction and determine their tolerances to ground movements. Contractors have to propose measures to protect those buildings and properties which are likely to suffer damage due to construction. During construction, the response of these buildings and properties are constantly monitored to see if it will be safe to continue the construction. Mitigation measures are taken to safeguard endangered buildings and properties if necessary.

What is less clear is the protection of MRT structures and facilities if someone is carrying out construction nearby. Because a large number of lives are at stake, it is just as important to protect MRT structures against adjacent construction as to protect adjacent buildings and properties against MRT construction. Even without fatalities, disruption of services will cause inconvenience to numerous commuters and serious traffic congestion. Severe damage to MRT structures and facilities may result in long closure of the system and such an event may even lead to political chaos.

Many cities already have in place laws regulating construction activities near MRT structures but in many other cities such legislation is long overdue. A recent failure (21 July 2005) in Guangzhou, which took 3 lives and resulted in serious damage to several adjacent buildings, of which one had to be demolished, is worth mentioning. Since the excavation is very close to No. 2 Line of the Guangzhou Metro, as a precaution, a section of the route (4 stations) was closed for an hour and 18 minutes for inspection. Fortunately MRT structures and facilities appeared to be undamaged and services were soon resumed. What is astonishing is the fact that the 20m excavation had been carried out for more than 2 years and was carried out not only without supervision but also without being approved by the building control authority. It is to the authors' knowledge that there are laws governing construction activities near the Metro lines in Guangzhou, however, they were obviously disregarded. Fourteen government officials were punished for not performing their duties and seven persons who were in charge of the project have been prosecuted.

After the completion of a section of tunnels in the Banqiao Line of TRTS in November 1995, excavation was commenced in August 1996 to construct a highrise building with a 5-level basement in a close proximity to MRT tunnels (Chang *et al.*, 2001). Cracks in segments were observed in the up-track tunnel in July 1998 while excavation had reached its final depth of 21.1m two months earlier. Inclinometer readings indicated that the tunnel had moved horizontally 44mm by then, and the movement reached 54mm in November. Configuration survey indicated that the tunnel lining was shortened by 45mm in the vertical direction and elongated by 26mm in the horizontal direction due to squashing as a result of relaxation of geo-stress as the diaphragm wall moved toward the excavation and away from the tunnel. In the end, steel segments had to be installed as secondary lining to support the 51 rings of concrete segments which were damaged. Fortunately the line was not in service; otherwise, the repair work would have been difficult to carry out. Although the excavation was approved by the Department of Rapid Transit Systems (DORTS) and sufficient instruments were available to monitor the response of the ground and the tunnel, instrument readings were not available to DORTS till damages to the tunnel were observed. The quality of the readings was so poor that considerable effort had to be spent to make sense out of them. It was later realized that there were already signs of distress in the lining in February 1998 when excavation was at a depth of 14m and damage to the lining could have been prevented if mitigation measures were carried out soon enough.

Approval from the MRT authority is necessary for the adjacent construction to proceed on the condition that it will be carried out with care and with necessary precautionary measures. It does not relieve the developer the responsibility of ensuring the safety of MRT structures. On an earlier occasion during the construction of the Tuen Mun LRT in Hong Kong, a structural engineer for a private high rise development with deep basement adjacent to the line was convinced that his excavation would not disturb the rail line because he had a sheet pile and strutting drawing approved under the Buildings Ordinance. This is obviously a misconception.

These cases demonstrate the fact that not only are laws required to protect MRT structures and facilities, they have to be understood and enforced. It is also important for MRT authorities to monitor construction activities carried out near their routes and to pay great attention to what happens to their property.

8 SPECIAL GEOTECHNICAL SERVICES

It is beyond any doubt that geotechnical engineering plays a very dominant role in underground constructions and geotechnical engineers shall be engaged in all stages of construction. It is important to identify potential risks beforehand and constantly monitor the performance of temporary and permanent works during the course of construction. As construction goes deeper and deeper, failures have occurred more and more frequently. Many failures were disastrous and took many lives. It is therefore advisable for owners of large projects to engage independent geotechnical teams to safeguard their interests. This is particularly true for construction of underground rapid transit systems in urban areas which involve construction of numerous deep excavations and long tunnels in poor ground. The scale of projects and length of construction periods make it justifiable to engage a team of specialists. On behalf of the owner, an independent geotechnical team shall participate in, or sometimes lead, the following tasks:

(1) Interpreting and characterizing ground conditions

(2) Preparing design manuals, specifications and tender documents

(3) Reviewing designs of temporary works and instrumentation programs

(4) Identifying potential risks and reviewing of contractors' contingency plans

(5) Assuring the quality of instrument installation works and monitoring

(6) Inspecting sites and monitoring site activities

(7) Examining instrument readings and managing databases

(8) Issuing warnings if signs of dangers are detected

(9) Reviewing contractors' mitigation plans if adverse conditions are encountered

(10) Ensuring that building protection measures are carried out effectively

(11) Evaluating contractors' claims on adverse ground conditions

Characterization of ground conditions is fundamental for underground works. Ground conditions are easily misinterpreted if investigation is not carried out properly. The same can be said for instrumentation works. Instruments can easily be misread and experience is the key to avoid such problems. Furthermore, it is becoming impractical to deal with the large quantities of instruments installed and the instrument readings collected, and powerful software packages are necessary for data management and identification of potential risks based on these readings.

8.1 *Taipei Experience*

Because the Initial Network of TRTS is the first rapid transit system constructed in Taiwan, the Taipei City Government foresaw the difficult situations to be encountered and engaged a Geotechnical Engineering Specialty Consultant (GESC), even before the inauguration of the Department of Rapid Transit Systems (DORTS). A team of specialists was formed to serve the Department in system planning, design review and construction supervision. This has been proved very fruitful as the design was optimized and many potential problems avoided.

At the peak of construction, a total of 18 field stations were setup and managed by the GESC to assist the field staff of the DORTS in solving on-site geotechnical problems. This also enabled high-quality instrument readings to be obtained to facilitate back-analysis for verifying the designs and the design assumptions (Moh and Hwang, 1996). A Data Center was established at the headquarters of GESC to process the tremendous amount of field data in a systematic manner. The database has become a major resource of numerous research studies and has contributed tremendously to the advancement of technology. More than 200 technical papers have been published on various research subjects and engineering applications based on the data obtained and provided valuable references to the subsequent constructions.

8.2 *Singapore Experience*

Subsequent to the collapse of Nicoll Highway, LTA engaged teams of engineers to provide geotechnical services to new construction contracts, for example, Contracts C854, C855 and C856 of the Circle Line. It is now a requirement that designs of underground works have to be checked by independent licensed geotechnical engineers. In addition, construction supervision which was previously conducted by LTA is now tendered out to private consulting firms and the Qualified Person (QP) who are in charge of the supervision will be appointed by the firms awarded. Furthermore, to make up the deficiency in technical capability, LTA has engaged consulting firms to supply qualified geotechnical engineers to be seconded to LTA. It is also a LTA's policy for safety-related instrumentation works to be tendered out to independent specialist subcontractors by LTA directly. It is a clear direction of Building Control Authority of Ministry of National Development to accredit specialist subcontractors for soil investigation and steel works, but the plan is still under study.

8.3 *Kaohsiung Experience*

Although specialist geotechnical engineers, per se, were not engaged, all the designs were checked by Independent Check Engineer (ICE). Learning from the many incidents, the Kaohsiung Rapid Transit Corporation, the Concessionaire of the BOT (build-operate-transfer) project, engaged a team of geotechnical specialists in December 2005 to be stationed at the site to provide geotechnical services to excavations at O1 and another team at O5/R10 Stations to safeguard her interests.

9 CONCLUDING REMARKS

Based on information available, there were 43 events of geotechnical nature associated with MRT constructions in this region since year 2001. The economic losses are estimated to exceed quarter a billion US dollars. The death toll of 14 is rather alarming in consideration of the fact that non-geotechnical events are not accounted for and also the fact that many failures were not reported.

Many failures could have been avoided if risk management programs were implemented. As most of the existing MRT systems are either extending the existing lines or expanding their networks by adding new lines and, furthermore, constructions of many new systems are to be commenced in the near future, risk management for underground works deserves urgent attention and extensive studies.

As an attempt to establish risk management procedures for underground works; this paper classifies these 43 events, basically, based on the Guidelines proposed by ITA. The appropriateness of the classification system adopted is subjected to further studies as more information becomes available. The database is to be enriched and cases compiled are to be systematically analyzed with the aim of identifying potential risks involved in underground construction and quantifying their occurrence and consequences.

Although it has been known for long, groundwater is the major cause of failures and is the cause of major failures discussed herein. Ground treatment has been adopted in all the 5 disastrous events (jet grouting in 4 and ground freezing in one) discussed herein to deal with problem but it obviously did not achieve its purpose. The design codes currently adopted in this region do not address to situations involving steep hydraulic gradients and, therefore, have to be carefully examined and revised based on local experience and local practice. Furthermore, the quality control of ground treatment works is vital and efforts are needed to enhance codes of practice and specifications for field works.

Risk management requires the collaboration of the project owner, designer, site supervisor and contractor. For large project, it is advisable for the project owner to engage specialist geotechnical service to provide independent checking of the designs and to assist in supervision of field works. It is also advisable for the insurer to engage specialist geotechnical engineers to safeguard his interest.

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APPENDIX C

Curriculum Vitae of Nominee

ZA-CHIEH MOH

MAA

NAME	:	Za-Chieh MOH
PRESENT POSITION	:	Chairman
DATE OF BIRTH	:	1931/08/19
NATIONALITY	:	United States of America, Republic of China
LANGUAGE SKILL	:	Chinese (mother tongue), English
TOTAL YEARS OF EXPERIENCE	:	
YEARS WITH MAA	:	32
EXPERTISE	:	

EDUCATION

- D. Tech. (Honorary), Asian Institute of Technology, Bangkok, 1999
- Sc.D., Massachusetts Institute of Technology, U. S. A., 1961
- M.S.C.E., Iowa State University, U. S. A., 1955
- B.S.C.E., National Taiwan University, R. O. C., 1953

PROFESSIONAL REGISTRATION

- Registered Professional Engineer (Civil), C'wealth of Massachusetts, U. S. A.
- Registered Professional Engineer (Civil), Singapore
- Registered Specialist Professional Engineer (Geotechnical), Singapore
- Registered Professional Engineer (Civil), R. O. C.
- Registered Professional Engineer (Civil, Geotechnical), H.K.
- Chartered Engineer (Civil), U.K.
- APEC Engineer (Civil, Geotechnical)
- EMF Engineer (Civil, Geotechnical)

PROFESSIONAL AFFILIATION

- Fellow, The Institution of Civil Engineers
- Fellow, American Society of Civil Engineers
- Fellow, Hong Kong Institution of Engineers
- Fellow, Institution of Engineers, Singapore
- Fellow, Institution of Engineers, Malaysia
- Fellow, Geological Society of London
- Member, Association of Consulting Engineers of Hong Kong
- Member, Association of Consulting Engineers of Singapore
- Member, Chinese Institute of Engineers, R. O. C.
- Fellow, Chinese Institute of Civil and Hydraulic Engineering, R.O.C.
- Member, China Road Federation
- Member, Southeast Asian Geotechnical Society
- Member, British Geotechnical Society
- Member, British Tunneling Society
- Member, International Society for Soil Mechanics and Geotechnical Engineering
- Member, International Association for Engineering Geology
- Member, International Society for Rock Mechanics
- Honorary Member, The Road Engineering Association of Asia and Australasia
- Honorary Member, Japanese Geotechnical Society

PRROFESSIONAL SOCIETY ACTIVITIES

- President, Southeast Asian Geotechnical Society, 1967-1972; Executive Committee Member, 1973-present
- Vice President, International Society for Soil Mechanics and Foundation Engineering, 1973-1977
- Research Consultant, National Science Council of the R.O.C., 1974-1985
- Chairman, Organizing Committee, First Southeast Asian Conference on Soil Engineering, Bangkok, 1967
- Organizer, Specialty Session on Engineering Properties of Lateritic Soils, 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico City, 1969

- Chairman, Organizing Committee, Fourth Asian Regional Conference on Soil Mechanics and Foundation Engineering, Bangkok, 1971
- Vice Chairman, Organizing Committee, First Road Engineering Conference in Asia and Australasia, Bangkok, 1976
- Member, Advisory Committee, 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, 1979
- Organizer, Specialty Session on Geotechnical Engineering and Environmental Control, 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, 1977
- Chairman of Conference Committee, 6th Southeast Asian Conference on Soil Engineering, Taipei, 1980
- Secretary-General of Organizing Committee, 3rd Conference of the Road Engineering Association of Asia & Australasia, Taipei, 1981
- Special Consultant to the Minister, Ministry of Communications, R.O.C., 1981-1990
- President, Taipei Civil Engineers Association, 1980-1986; Honorary President, 1995-1998
- Member, Subcommittee on Site Investigation, and Advisor, Subcommittee on Soil Sampling, International Society for Soil Mechanics & Foundation Engineering, 1977-1985
- Chairman and member, Technical Committee on Environmental Control, and Member, Committee on Professional Practice, International Society for Soil Mechanics and Foundation Engineering, 1986-1997, 1997-2001
- Chairman, Papers Committee and Member of Organizing Committee, Tenth Southeast Asian Geotechnical Conference, Taipei, 1990
- Board Member, International Society for Soil Mechanics and Foundation Engineering, 1989-1993
- Chairman of Organizing Committee, 13th Southeast Asian Geotechnical Conference, Taipei, 1998
- Session Chairman, General Reporter of many international and regional conferences and seminars
- Hon. Treasurer-General, The Road Engineering Association of Asia and Australasia, 1987- 2000
- Director, Chinese Institute of Engineers, 1983, and 1985- 1988; Chairman of Professional Activities, 1977-1980
- Director, Chinese Institute of Civil and Hydraulic Engineering, 1978-1981, 1995-1997, 2001-2003, 2005-2007, 2007-2009; Vice-President, 1981-1983, 1997-1999, 2003-2005; Chairman of Geotechnical Engineering Committee, 1978- 1985; Vice-Chairman, CICHE-ASCE Liaison Committee, 1982-1984; Member of International Relations Committee, Professional Practice Committee, Professional Activity Committee, Fellow Nominating Committee.
- Director, China Road Federation, 1977-1985, 1991- 1993, 1995-1997, 1999-2001; 1997-1999, 1999-2001, 2001-2003; Supervisor, 1985-1991; Vice President, 1993-1995, 1997-1999, 2003-2005, 2005-2007, 2007-2009; Chairman of Scholarship Committee, 1983-1985; Chairman of International Relations, 1978- 1983; Vice Chairman of Awards Committee, 2005-2007; Chairman of Awards Committee, 2007-2009
- Director, Chinese Institute of Engineering Environments, 1976-1986, 1991-1995
- Vice President, Chinese Association of Engineering Consultants, 1991-1995, 1997-1999, 1999-2001, 2001-2003, 2003-2006, 2006-2009; President, 1995-1997
- President, Chinese Union of Professional Civil Engineers Associations, 1992-1995; Honorary President, 1995-1998
- Director, MIT Enterprise Forum of Taiwan, 1995-1997, 1997-1999, Supervisor, 1999-2001, 2001-2003
- Supervisor, the Arbitration Association of R.O.C., 1998-2001, 2001-2003, 2003-2004, Executive Supervisor, 2004-2005, 2005-2007
- Director, Civil Engineering Technology Research and Development Foundation, Taipei, 1993-2005; Chairman 2005-present.
- President, Taipei Federation of Engineering Consultants, 2001-2003, Executive Supervisor, 2003-2005

OTHER PUBLIC SERVICES

- Member of Review Committee on Civil Engineering Departments in Universities and Colleges in the R.O.C. for the Ministry of Education, 1986
- Expert Witness for the Attorney General's Office, Republic of Singapore on the Collapse of Hotel New World Building, 1987
- Member of the Advisory Committee for the School of Civil & Structural Engineering, Nanyang Technological Institute, Singapore, 1988-1992
- Chairman of Review Committee on Civil Engineering and Related Disciplines in Universities and Colleges in the R.O.C. for the Ministry of Education, 1989

- Chairman of Advisory Committee for the NTU-PWD Geotechnical Research Center, Nanyang Technological University, Singapore, 1996-1997, 1997-1999, 1999-2001, 2001-2003
- Member of Expert Advisory Committee for Disaster Prevention, Taipei Municipal Government, 2003-2005.
- Chairman of Review Committee on Civil Engineering Department of the National Taiwan University, May 2004.
- Member of Technical Committee, Public Construction Commission, Executive Yuan, ROC, 2003-2005
- Member of Editorial Committee, Chinese Journal of Geotechnical Engineering, 1995-2003
- Member of Editorial Committee, Chinese Journal of Building Structures, 2002-present
- Member of Accreditation Action Committee of the Institute for Engineering Education Taiwan, 2004-present
- Resource Person, Singapore MND-MOM Construction Safety Review Committee
- Member of Accreditation Council, Institute for Engineering Education, Taiwan, 2005
- Chairman, Chinese Taipei APEC Engineer Monitoring Committee, 2005-2009

EMPLOYMENT RECORD

- Principal, MAA Group Consulting Engineers, 1976-present
- Chairman of Board, MAA Group Consulting Engineers, 2008-present
- Professor of Geotechnical Engineering; Vice President and Provost, Asian Institute of Technology, 1974-1976
- Professor and Chairman, Division of Geotechnical Engineering, Asian Institute of Technology, Bangkok, 1967-1974
- Associate Professor of Civil Engineering, SEATO Graduate School of Engineering, Bangkok, 1965-1967
- Soil Engineer, Tippetts-Abbet-McCarthy-Stratton, Consulting Engineers, New York, 1963
- Assistant Professor of Civil Engineering, Yale University, 1961-1965
- Chief Engineer, Woodward-Clyde-Sherard & Associates, Consulting Civil Engineers, Omaha, Nebraska, 1959-1961
- Research Assistant, Massachusetts Institute of Technology, 1955-1959
- Research Assistant, Iowa State University, 1954- 1955

HONORS

- Member of Sigma Xi, Phi Kappa Phi. Listed in International Scholars Directory, American Men and Women of Science, Directory of Selected Scholars and Researchers in Southeast Asia, Who's Who of the Republic of China, 1st Ed. (1982), 5000 Personalities of the World, 1st Ed. (1985), Biographee of the Year Award (1986), International Book of Honor, 2nd Ed. (1986), International Directory of Distinguished Leadership, 1st Ed. (1986), Marquis Who's Who in the World, 8th Ed. (1986), Who's Who in Hong Kong (1997), Who's Who in Engineering U.S.A. (1992), Baron's Who's Who in the Asia Pacific Rim (1999), International Who's Who of Professionals (2000), Asia/Pacific Who's who (2002), Asia Men & Women of Achievement (2003), 21st Century Award for Achievement, IBC (2003), Marquis Who's Who in Science and Engineering (8th Ed.) 2004 and (10th Ed.) 2008, Who's who in Asia (2008)
- "Man of the Year" Award by the China Road Federation, 1983
- "Gold Medal Award for Academic Achievement," Chinese Institute of Civil and Hydraulic Engineering, 1983
- Distinguished Aluminus Award, Department of Civil Engineering, National Taiwan University, 1994
- Honorary Doctor of Technology, Asian Institute of Technology, Bangkok, 1999
- Honorary Member, The Road Engineering Association of Asia and Australasia, 2000
- Honorary Member, Japanese Geotechnical Society, 2003

SUMMARY OF EXPERIENCE

Major works undertaken include planning, execution and consultation on soil investigation work, study and evaluation of soil behavior, performance study of earth structures, soil stabilization and ground improvement for foundations, highways, airfields and earth dams. Extensive research and practical experience on problems related to soft clays and lateritic soils. Served as Project Director or Advisor for many major projects, including: performance evaluation of test embankment at AIT new campus, Bangkok; performance study of test embankment for proposed new international airport in Bangkok; geotechnical study for reclamation of an abandoned river channel for development in Hong Kong; instrumentations for numerous deep excavations; geotechnical studies for the Mass Rapid Transit Systems Taipei; Railway Underground Project, geotechnical consultancy for the Second Bangkok International Airport; Geotechnical Engineering Specialty Consultant for the Taipei Rapid Transit Systems; geotechnical design for Singapore MRT contract 405; numerous tall buildings and deep excavations in Taiwan, Hong Kong, Singapore, Malaysia, Thailand and Indonesia. Served as a member of the Review Committee for the Site Selection Study for the fourth nuclear power plant in Taiwan.

PUBLICATION

Author or co-author of over 140 technical papers and reports on soil properties, soil behavior, engineering performance and soil improvement. Editor of ten regional and international conference Proceedings, many seminar and symposia Proceedings.