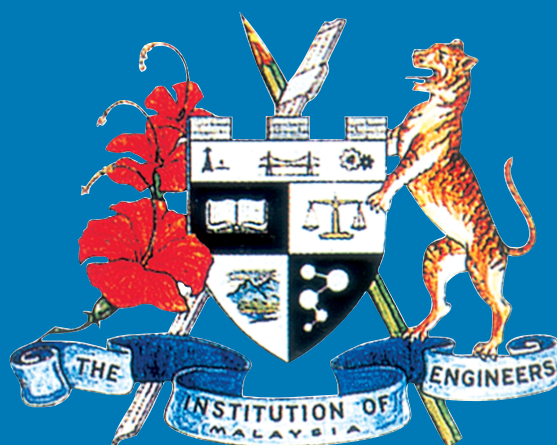


IEM JOURNAL



THE JOURNAL OF THE INSTITUTION OF ENGINEERS, MALAYSIA

KDN PP5476/10/2010 (026477)
ISSN 0126-513X

Vol. 71, No.1
March 2010



THE INSTITUTION OF ENGINEERS, MALAYSIA

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Vol. 71, No.1, March 2010
KDN PP5476/10/2010 (026477) ISSN 0126-513X

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Printed by: WISE ORGANISATION (M) SDN. BHD. (888671-H)

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THE NINETEENTH PROFESSOR CHIN FUNG KEE MEMORIAL LECTURE

**Presented at the Auditorium Tan Sri Prof. Chin Fung Kee,
Wisma IEM, Jalan Selangor, 46200 Petaling Jaya, Selangor, Malaysia
on 7 November 2009**



Ir. Dr Ooi Teik Aun

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FIEM, FMI Arb, MICE, Chartered Engineer, P Eng, ASEAN Engineer
APEC Engineer, ASEAN Chartered Professional Engineer and an International Professional Engineer**

Dr Ooi is a Consultant in Geotechnical and Civil Engineering and Project Management. He also acts as Arbitrator, Expert Witness and Adviser. Dr Ooi is a Chartered Engineer of the United Kingdom and a Registered Professional Engineer of Malaysia, an ASEAN Engineer, APEC Engineer, ASEAN Chartered Professional Engineer and an International Professional Engineer. He is a Fellow of The Institution of Engineers, Malaysia (IEM), a Fellow of the Malaysian Institute of Arbitrators and a Member of the Institution of Civil Engineers, United Kingdom. Dr Ooi graduated in Civil Engineering in 1966 from Auckland University, New Zealand and obtained his Masters degree in Civil Engineering from the same University in 1968.

He obtained his PhD from Sheffield University in 1980. He joined the Public Works Department, Malaysia (PWD) in 1968 and held the posts of Engineer, Senior Executive Engineer and Assistant Director respectively in charge of the PWD Headquarters Soils and Materials Laboratories in the Design and Research Branch before he left to join Promet Construction Sdn Bhd in 1982. Whilst in PWD he was involved in airport and port design, slope investigations, design and rectifications, Soils and Materials investigations, Highway and Building foundation designs as well as remedial works. In 1984 he joined Pilecon Engineering as an

Operations Director in charge of Design and Construction of Geotechnical and Civil Engineering Projects. In 1989 he joined Transfield Construction Group as a Director and General Manager for operation in Asia. He started his consultancy services in 2000 after retiring from Transfield. He is an active and a long serving member of IEM since 1970s. He was IEM Council Member in 1981-1984, Vice President in 1988-1990 and is a Director of IEM Training Centre since 1991. He is currently Immediate Past President of the Malaysian Institute of Arbitrators, ICE Country Representative for Malaysia since 2000, Southeast Asian Geotechnical Society President in 1993-1996, Chairman of Geotechnical Technical Division in 1991-1992. Chairman of Tunnelling and Underground Space Technical Division in 2002-2003 and 2006-2009, Chairman of the Organising Committee for the Annual Professor Chin Memorial Lecture 1995-2008 and Chairman of the Organising Committee for the 16th SEAGC held in 2007 and the 12th SEAGC held in 1996 in Kuala Lumpur respectively. He is First Chairman of the Association of the Geotechnical Societies in Southeast Asia (2008-2010). Organising Chairman, IEM Green Workshop and Exhibition in November 2009. He was a member representing IEM on the Technical Committee of MPAJ to investigate the collapse of the Highland Towers Condominium in 1993.

GEOTECHNICAL FAILURES/ISSUES, DISPUTE RESOLUTION AND MITIGATION

(Date received: 17.11.2009)

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ABSTRACT

Geotechnical failures can be caused by a number of factors. One of the most important in this regard is associated with earthworks. This lecture focuses on the crucial role of earthwork practice during construction and its subsequent maintenance. The case histories studied include the slope problems associated with the landmark Highland Towers collapse in December 1993. Whilst compaction specification may be carefully prepared by the engineer, clients often do not wish to incur the expense of supervision by the engineer, notwithstanding requirements imposed by law for both the design and supervision by the engineer before the engineer's certification. The practice of clients undertaking their own supervision work to save costs may be counterproductive, often resulting in inferior quality of the final product. This naturally arises as the quality-control of workmanship and material under a system of "self certification" is not as robust as "independent certification" by the Engineer. Serious potential problems such as obstructions to pile installation, building settlement/movement and slope failures, have been known to arise from such practices which usually have a direct impact on third parties such as end-purchasers and the occupiers of neighbouring lands. Geotechnical failures often happens in development projects where earthworks are often tip filled into neighbouring land without compaction and removal of unsuitable foundation materials thereby causing long term problems such as erosion, siltation, slope failures and/or excessive settlement resulting in embankment movements and cracks/failure in structures. The Local Authorities' role in the enforcement of good earthwork practice to successfully counter the possibility of such problems can be improved and should be more proactively addressed. This lecture examines some case histories of geotechnical failures/issues and identifies the areas of the engineer's duties, responsibilities and obligations to the third parties other than the client. The dispute resolution aspect of geotechnical issues and mitigating measures are also discussed.

Keywords: *Disputes, Earthworks, Geotechnical Failures, Mitigation, Water*

1.0 INTRODUCTION

The assessment of the risk of having a disaster and the mitigation thereof is of foremost importance in an engineering design. Failure to assess the risks appropriately could spell disaster to the completed works. What is of concern to the geotechnical engineer is the prevention and mitigation of disasters as well as rehabilitations of failures in geotechnical works. Ting *et al* (2007) pointed out the importance of "Geotechnical risk management that has become well developed (ICE 2001) and should be applied; at the least notionally, when many solutions are possible."

In Malaysia, major disasters arising out of geotechnical failures in uncontrolled earthworks are: the repeated flooding of Kuala Lumpur since 1971, the collapse of Highland Towers Condominium in 1993, the Genting Highlands access road debris flow in 1995, landslide buried the bungalow at the foothill within the vicinity of the Highland Tower site in 2002, Taman Zoo View

landslide in 2006 and the others in recent times. The landslide incidences in the Ulu Klang area are within the same vicinity and in the district of Ulu Klang mountain range of Titiwangsa.

Climate change has resulted in more incidences of flooding and landslides throughout Malaysia in recent years. Asahari (2009) reported more than 100 landslide incidences a year in Malaysia at a seminar on safe hill-site development in Kuala Lumpur. Failures of earthworks can often be traced to poor quality control of compaction of earthworks and/or the ineffective control of water / drainage. The fact that many highways have been constructed in recent time only bring about more failures because of 'limiting conditions' brought about by the steeper and higher cut slopes and higher embankments on soft ground. "Tipped-fill" remained one of the main factors that cause earthwork failure during incessant raining period of the monsoon and geotechnical failures are often traceable to poor earthwork practice and the lack of maintenance of the drainage system.

Some examples of earthwork failures/issues and court cases are cited below to illustrate the impact of poor earthwork practices and/or control of water and the duties and responsibilities of Engineers in respect of their works connected to earthworks.

Mitigation, rehabilitation and prevention of disasters are important considerations for any geotechnical design. The SMART project was born out of the disastrous flooding of Kuala Lumpur in 1971, and the subsequent flooding events in Kuala Lumpur, the date of which are summarised in Table 1, and is believed to be the first of its kind in the world where the tunnel is used as a dual purpose tunnel for both flood control and to ease the traffic congestion of the Kuala Lumpur City Centre. Figure 1 shows the alignment and cross section of the tunnel. The SMART has won the prestigious British Construction Industry International Award, BCIA Award in 2008.

The Highland Towers collapse (Figure 2) shows that it is important for the designer to consider all aspects of foreseeable future danger to the structural integrity of buildings in relation to their environment including future maintenance. The stability of slope and the structural foundation of the building are integral in the design analysis process. Engineers must put safety, health and welfare of the public above all other factors in the design consideration. A crucial issue which has surfaced from this tragedy is the need to design and implement systems to effectively drain surface and subsurface water from a project site.

Debris flow type of slope failures will increase with more development in the highlands and mitigating measures recommended must consider hydraulic factors that dominate the impact of the debris flow, whilst geotechnical factors determine the formation of the natural barrier and the materials of the debris Ooi and Ting (2005).

The tsunami that struck the Indian Ocean on 26 December 2004 has also brought about the urgent need from the geotechnical community in the region and Malaysia in particular to seriously consider and integrate mitigation features through adequate design provisions and considerations. In the authors' view, a programme of public education in awareness and training in the handling of disasters must also be implemented as has been the case in Japan for Tsunamis (Ohta, 2005) and in Hong Kong for slopes (Mak *et. al.* 2007).

Table 1: Dates of occurrences of flood events in Kuala Lumpur since 1971

1.	1971
2.	1982
3.	1986
4.	1988
5.	7 June 1993
6.	21 December 1995
7.	30 April 2000
8.	26 April 2001
9.	29 October 2001
10.	11 June 2002
11.	10 June 2003
12.	11 June 2007

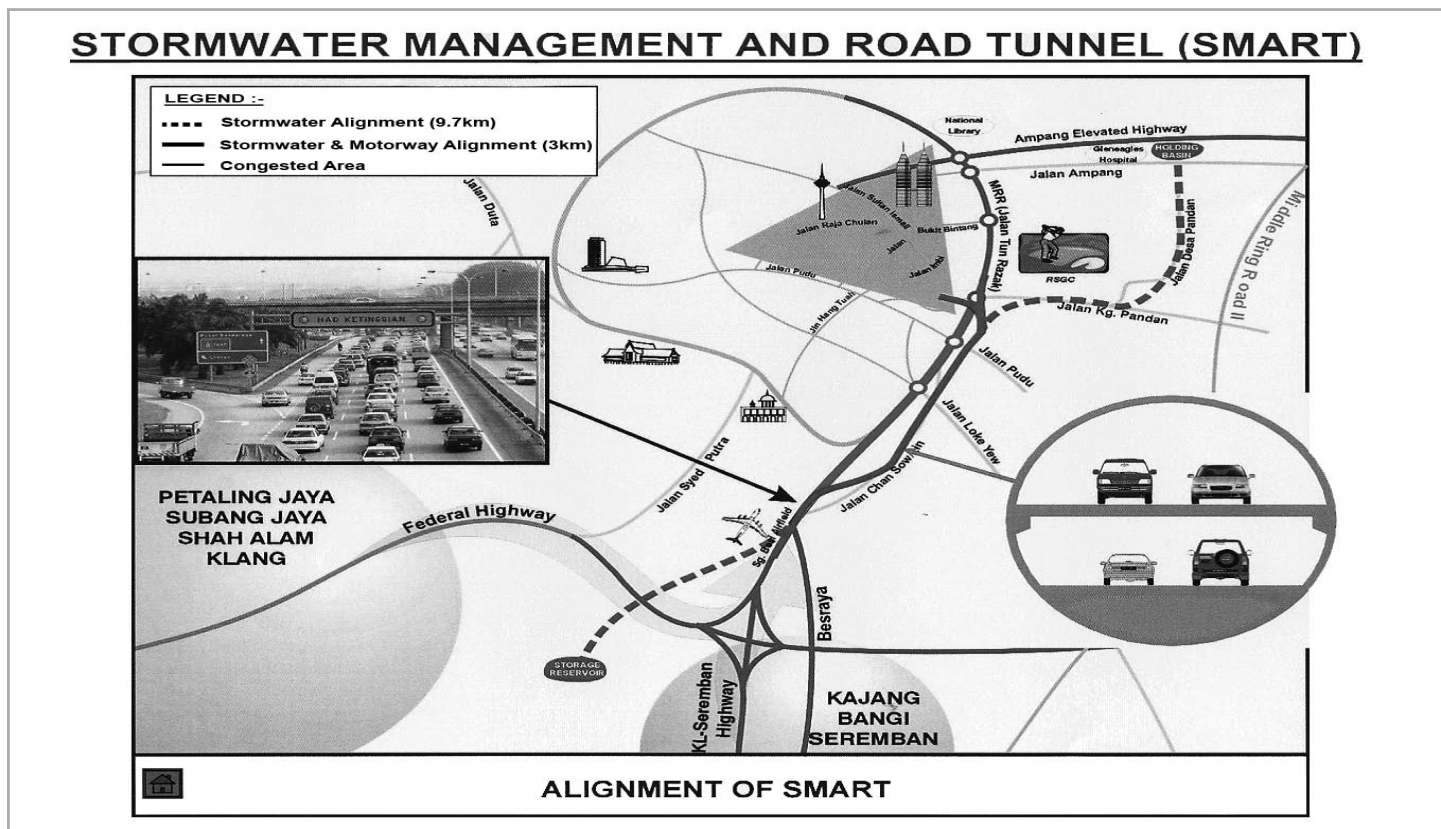


Figure 1: Schematic alignment and cross section of SMART



Figure 2: Collapse of Block 1 Highland Tower Condominium. (after MPAJ, 1994)

2.0 THE LANDSLIDE INCIDENTENCES

Ooi (2004) in his special lecture on Earthwork Practice in Malaysia discussed the effect of water in the occurrences of landslides. Table 2 shows some significant landslide event that occurred during 1961-2008. All these landslides occurred during the period of incessant rainfall.

Table 2: Some significant landslide events 1961-2008

Year	Location	Landslide Event
1961	Cameron Highlands, Perak	Landslide demolished one row of shops and 14 people were killed.
1972	Bukit Gasing, Petaling Jaya, Selangor	During the period of incessant rain, landslides at two separate locations demolished the government quarters located at the bottom of the slopes when tipped fill slope failed and flowed down the slope.
1973	Gunong Kroh, Perak	15m high rock fall killed 50 people and a row of 10 houses and shops were buried.
1976	Puchong, Kuala Lumpur	9 buried alive in landslide in tin mine in Kampung Bohol, Puchong.
1981	Puchong, Kuala Lumpur	31 buried alive in landslide in tin mine in Kampung Kandan, Puchong.
1993	Highland Towers, Ampang Jaya, Selangor	11 December 1993 Landslide caused Block 1 of the Highland Towers to collapse during period of incessant rain and rendered Block 2 and 3 unsafe and thus evacuated. Prolonged period of incessant rain in November / December 48 people were killed.
1995	Genting Highland, Selangor	20 people were killed and 15 vehicles buried at road in Genting Sempah of the Kuala Lumpur/Karak Highway during period of incessant rain.

1996	Gua Tempurung Ipoh, Perak	Debris flow with failures of soil anchors on slope Gua Tempurung North South Expressway on 5 January 1996 during period of incessant rain.
1996	Pos Dipang, Kampar, Perak	Debris flow at Pos Dipang killed 36 people in an Orang Asli Settlement and demolished the whole village.
1999	Ulu Kelang, Ampang Jaya, Selangor	Bukit Antarabangsa filled slope failure during prolonged period of incessant rain. Access road to Bukit Antarabangsa cut off. Residents of the area were evacuated. No loss of life but economic loss and anxiety suffered. Rehabilitation by installation of horizontal drainage system.
2002	Simunjan Sarawak	16 buried alive in a landslide in Raun Changkul, Simunjan, Sarawak.
2002	Ulu Kelang, Ampang Jaya, Selangor	Landslide of old tipped-fill slope buried the bungalow at the foothill and 6 people were killed.
2003	NKVE, Bukit Lanjan, Selangor	Rock slide at NKVE Bukit Lanjan during period of incessant rain caused six month closure of the Expressway in November 2003. JKR Cawangan Cerun was formed.
2004	Cameron Highland, Perak	Ringlet-Tanah Rata road widening caused 70m long wall to collapse on 24 February 2004 causing 50m stretch of main trunk road to cave-in and disrupted traffic flow. (Edition Didier Millet, 2007)
2006	Taman Zooview, Ulu Klang, Selangor	Massive landslide of an old tipped-fill slope with 15 terrace houses on top of the slope. Continuous heavy rainfall in the month of April and May 2006 before the landslide. Long houses at the bottom of the slope demolished by the landslide materials and 4 persons in the long houses were killed.
2007	Tasik Banding Grik, Perak	Tasik Banding Grik Perak newly completed Resort Hotel collapsed due to slope failure/movement. The building was not occupied hence no lives lost.
2008	Gombak, Selangor	Ulu Yam Perdana Gombak incessant rain and landslide demolished the bungalow at the foothill and killed two people alive at 5.30am on 30 th November 2008.
2008	Bukit Antarabangsa, Ulu Klang, Selangor	Massive landslide occurred on 6 December 2008 at the slope of Jalan Wangsa 9. Flow slide travelled 200m to reach the river. Continuous heavy rainfall in November / December 2008. Incessant rainfall prior to landslide incidence. 14 bungalow demolished by flow slide. 5 people died, more than 90 injured and many homes declared unsafe by the Public Works Department (JKR Cawangan Cerun).

The danger of fill slope has been reported by Hong Kong GEO (1999), Table 3 shows cases of fill slope failures in Malaysia and the bedrock geology.

Table 3: Cases of fill slope failures

Date	Location	Landslide Details	Bedrock Geology
4 January, 1971	Bukit Gasing, Petaling Jaya, Selangor	Gasing Height Development. Perimeter drains collapsed during one week of incessant rain. Tipped-fill flow slide damaged 2 government quarters. Slope was reconstructed with proper compaction and quarters rebuilt.	Sandstone / Shale

18 September, 1988	Ulu Kelang, Selangor	Tipped-fill slope failure due to water in excavation pits in neighbouring land during prolonged period of incessant rain caused slope failure and damages to Bungalow and Swimming Pool.	Sandstone / Shale
11 December, 1993	Ulu Kelang, Selangor.	Collapse of Block I of Highland Towers on 11 December 1993 during prolonged period of incessant rain High Court decided the rotational retrogressive fill slope failure was the cause of the collapse of the Block 1 of the Highland Towers and water from upslope development and its drainage system and maintenance was the major factor contributing to the slope failure.	Granite
29 June, 1995	Genting, Selangor	Debris flow, Genting Highlands on 30 June 1995 caused closure of Kuala Lumpur – Karak Highway	Mixed geology granite Sandstone / Shale
15 May, 1999	Ulu Kelang, Selangor	Bukit Antarabangsa fill slope failure during prolonged period of incessant rain.	Granite Clay < 10%
21 November, 2002	Ulu Kelang, Selangor	Landslide occurred at 4.30am during prolonged period of incessant rain. Landslide of old tipped fill slope buried the bungalow at the foothill and 6 people were killed.	Granite Clay < 10%
31 May, 2006	Taman Zooview, Ulu Kelang, Selangor	Massive landslide of an old tipped fill slope with 15 terrace houses on top of the slope. Continuous heavy rainfall in the month of April and May 2006 before the landslide. Long houses at the bottom of the slope demolished by the landslide materials and 4 people in the long houses were killed. Residents of the terrace houses on top of the slope evacuated. Local authority directed slope rehabilitation by the Developer for the bottom of the slope. Height of slope 60m, Debris flow 200m. Estimated Volume of slide material 120Km ³ (200m x 100m wide x 6m thickness)	Sandstone / Shale underlain by granite bedrock Hawthornd-en schist can be seen intruded by weathered granite and quartz veins and dykes Clay > 20%
6 December, 2008	Bukit Antarabangsa, Jln Wangsa 9 Ulu Klang, Selangor	Flow slide travelled 200m to reach the river. Continuous heavy rainfall Nov / Dec 2008. Incessant rainfall prior to landslide incidence. 14 bungalows demolished by flow slide. 5 person dead, more than 90 injured and many homes declared unsafe by Public Works Department (JKR Slope Engineering Agency). Apart from this, 14 existing hill-site housing estates in the Ulu Klang areas were also declared as being at risk of landslides by the Selangor State Government. Thickness of landslide reported is 10m.	Granite Clay content < 10%

Figures 3-5 show photos of fill slope failures.



Figure 3: Landslide at PJ Quarters 1276 (after Ooi and Tee, 2004)



Figure 5: Fill Slope Failure (after Ooi and Tee, 2004)



Figure 4: Landslide at PJ Quarters 1280 (after Ooi and Tee, 2004)



Figure 6: Reconstruction of slopes using compaction method (after Ooi and Tee, 2004)

Figure 7 shows a tipped-fill slope before failure.



Figure 7: Zooview Site Condition; Site Slope in 2004 (after Ooi, 2008)

Figure 8 shows the tipped fill slope after failure. The total height of slope is about 60m.



(a)



(b)

Figure 8: Zooview 2005 Landslide (after Ooi, 2008)

Figure 9 shows backyard before rehabilitation. Figure 10 shows the rehabilitated slope.



Figure 9: Picture showing the backyard before rehabilitation (after Ooi, 2008)



Figure 10: View of rehabilitated slope (after Ooi, 2008)

Figure 11 shows illegal tipped fill into adjacent property in one development near Kuala Lumpur. The top of slope is the boundary between the two properties. It is clear from the photo that the 20m high fill slope has suffered erosion and slope stability problem. The local council did not issue stop work order despite complaint by land owner of adjacent property. Figure 12 shows trespass of land with cut slope failure. This malpractice is also common as will be demonstrated by one court case discussed below.



Figure 11 : Illegal Tipped-Fill in Adjacent Land



Figure 12 : Trespass Cut Slope Showing Slope Failure

3.0 LEGISLATION AND GUIDELINES GOVERNING EARTHWORKS IN MALAYSIA

3.1 Penalty for failure of building or earthworks

Section 71 of the Street Drainage and Building Act 1974 (Act 133) expressly recognises the importance of design and supervision of earthworks. It reads:

“Where any building or part of a building fails, whether in the course of construction or after completion, or where there is any failure in relation to any earthworks or part of any earthworks, whether in the course of the carrying out of the earthworks or after completion thereof, and the cause of such failure is due to any one or more of the following factors:

- (a) misconstruction or lack of proper supervision during construction;*
 - (b) misdesign or miscalculation; or*
 - (c) misuse,*
- of such building or part of such building, or of such earthworks or part of such earthworks, the person responsible for—*
- (aa) such misconstruction or such lack of proper supervision;*
 - (bb) such misdesign or miscalculation; or*
 - (cc) such misuse,*
- shall be liable on conviction to a fine not exceeding five hundred thousand ringgit or to imprisonment for a term not exceeding ten years or to both.”*

In other words, in the event of any earthworks failure, whether during or after the time the work is done, if such failure was caused by misconstruction, improper supervision or misuse, there are penal consequences against the person responsible for these acts and the potential penalty is grave: a fine of up to RM500,000.00 possibly a jail term of up to 10 years and in the worse case scenario both .

Although, there do not appear to be any reported cases on actual liability under this Section 71, it is important for engineers, as the parties responsible for the design and supervision of earthworks to be mindful of its implications.

3.2 Obligations of Engineers

The importance of the role of professional engineers in earthworks cannot be underscored enough and it is worthwhile to be reminded of Section 8(1) of the Registration of Engineer's Act 1967 (Act 138) which provides:

“no person or body, other than a registered Professional Engineer who is residing in Malaysia or an Engineering consultancy practice providing professional engineering services in Malaysia, shall be entitled to submit plans, drawings, schemes, proposals, reports, designs or studies to any person or authority in Malaysia”

In so far as performance of the submission and certification obligations of professional engineers, the Board of Engineers Malaysia (BEM)'s Guidelines for Code of Professional Conduct (Circular No. 3/2005) are clear in that first:

“A Professional Engineer shall approve and sign only those documents that he has prepared or are prepared under his direct supervision.” (para 1.1)

¹Also note the presumption of ‘prima facie’ liability that rests with the submitting party of any plan drawing or calculation in respect of a failed building, notwithstanding prior approval by the local authority under Section 258(5) of the Uniform Building By-Laws 1984 (UBBL). The submitting party for any plan, drawing or calculation must be a qualified person i.e. an architect, qualified building draughtsman or engineer, and such submitting person (or any person duly authorized by him) must pursuant to Section 5 of the UBBL undertake the supervision of the erection and, setting out, where applicable, of the building.

and secondly:

“A Professional Engineer shall certify satisfactory completion of a piece of work only if he has control over the supervision of the construction or installation of that work, and only if he is satisfied that the construction or installation has fulfilled the requirements of the engineering design and specifications.”

With particular regard to earthworks, Section 70A (1) the Street, Drainage and Building Act, 1974 is clear that:

“No person shall commence or carry out or permit to be commenced or carried out any earthworks without first having submitted to the local authority plans and specifications in respect of the earthworks and obtained the approval of the local authority thereto.”

In accordance with this Section 70A(1), local councils have adopted specific By-Laws in respect of earthwork practices and for example, the Earthworks (Federal Territory of Kuala Lumpur) By-Laws 1988 specifically stipulates that the submitting engineer for earthworks is responsible for the proper execution of the works until completion and this expressly includes supervision.

Furthermore, the introduction of the new ‘self-regulation’ regime in construction works *vis-à-vis* the introduction of certificates of completion and compliance to replace the local council’s ‘certificates of fitness for occupation’, which are now to be signed off by the relevant submitting parties, goes further to underscore the importance of the engineer’s function, not just in the design but the actual supervision of earthworks.

3.3 Other Relevant Statutes

Other relevant statutes that apply to construction work and therefore should be considered by engineers in operations involving in earthworks include:

- a) Town and Country Planning Act 2001 (Act A1129);
- b) Environmental Quality Act 1974 (Act 1102)
- c) Road Transport Act 1987 (Act 333)
- d) National Land Code 1965 (Act 56)
- e) Occupational Safety and Health Act 1974 (Act 514)
- f) Factories and Machinery Act 1967 (Act 139)

3.4 Earthwork Specifications

The JKR Standard Works Specification for Earthworks is generally modelled for works related to earthwork. Care must be taken in transcribing the specifications for the work so as to avoid possible contractual disputes during construction. A common problem that often arises is when there is a mismatch of contractual provisions within the contract document, which provides a platform for divergences between the engineer and the contractor to occur during the course of construction and which inevitably leads to unnecessary delays, cost escalation and possible compromises to the integrity of the work performed.

The private sector practice as regards earthworks is generally that contracts divide the material into 5 categories namely: topsoil, unsuitable materials, suitable materials, hard materials and rocks.

Most specifications would define what unsuitable materials are. Suitable material would then be deemed to be defined as material which is not unsuitable. Consequently, a clause should be added to say “Suitable materials are materials that are not unsuitable”. This will eliminate disputes often arising in relation to what suitable or unsuitable materials are. In granular material, compaction is best carried out under water and using vibratory method of compaction. For cohesive soil, compaction is best carried out by using vibratory sheep foot or smooth wheel compactors depending on the clay content of the material. In heavy clay soil, the natural moisture content may be very high and the drying process could take a long time. The contractor is at liberty not to use the material, at his own cost, for filling. However, contractually this does not mean that the material is unsuitable for use.

In certain specific cases where ground improvement is required for elimination/control of future settlement of the ground, the material after treatment would become suitable and acceptable as it poses no further danger to the user. Vacuum consolidation is one such examples of soil treatment as it does not exert lateral load that cause instability as illustrated by the case of Ting *et al.*, (1995) to be discussed in the later section. In the light of the global “Green Revolution”, ground improvement has become a sustainable development technology.

The definition of rock from a practical aspect is always associated with blasting. The introduction in earthworks contracts of the term “hard material” causes more confusion than clarity. To distinguish the hard materials and rocks in the field, the following definitions are used: -

- a) Rock is defined as “material which would normally have to be loosened by blasting, chemical splitting or pneumatic tools”.
- b) Hard material is defined as “material which would require ripping with a single shank ripper with tractor unit of not less than 250kN in weight and 260kN in power”.

In contrast, the JKR Standard Specifications do not identify the way in which hard materials can be distinguished from rock.

Filling under water shall be carried out using rock or other granular material. However, if dewatering is carried out cohesive fill can be used.

4.0 SITE INVESTIGATION

Site investigation is an important part of earthworks and foundation works. The site investigation practice in Malaysia follows that of BS 5930 : 1999. It is essential to investigate the ground to obtain the necessary data for design as well as construction control.

Any type of ground improvements that may be required either because of stability problem or settlement criteria must be identified. Therefore the quality and reliability of site investigations is very important.

For important projects, Engineer Supervision of Site Investigation Works is essential. This is often not realized by clients who, assumedly with the intention of saving costs on site investigation expenses (which is normally less than 1% of the total project cost), choose to overlook this and the result is a compromise on the quality and reliability of the results. This in turn often leads to trouble or disputes during the construction stage of the earthwork contracts.

The Uniform Building By-Laws 1984 (revised Nov 2007) By-Laws 24 provides:

“As soon as the excavation for the foundation of a building has been completed the qualified person shall give written notice to the local authority in Form D as set out in the Second Schedule to these By-laws informing it of the fact and certifying that the nature of the soil conditions as exposed by the excavations are consistent with the design requirements and conform with these By-laws.”

This provision clearly demonstrated the importance of the engineer's supervision function.

Under BS 6031:1981 it is provided: -

“...no site investigation, however carefully done, ever examines more than a very small proportion of the ground. It is necessary to check the soil conditions revealed during progress of the excavations correspond with those forming the basis for earthworks design as interpreted from the site investigation...”

In other words, site investigation is a continuous process, from pre-design to construction stage.

The appreciation of site investigation data is very much dependent on the relevant experience of the contractor. It is important to be able to see the big picture from the site investigation results in relation to the work and the onus must be on the contractor to assess the risk associated with the work and allow for sufficient mitigation provisions if necessary.

The provision of work method statements in accordance with the work programme will help to minimise conflict between the parties. The work method statement would also assist the contractor to embark on a focused and methodical thought process as to how it will do the work, the risk involved in the work and the mitigation measures to be undertaken if a failure does occur.

5.0 UNFORESEEN GROUND CONDITIONS

Unforeseen ground conditions are described in Clause 12.2 of the FIDIC Conditions of Contract as follows:

“...physical obstructions or physical conditions, other than climatic conditions on the site, which obstructions or conditions were, in his opinion, not foreseeable by an experienced contractor.....”

The limitation is on the word “physical” on any unforeseeable obstruction or condition and the words “not foreseeable by

an experienced contractor”. This is particularly applicable to tunnelling activity.

It is important that when such event occurs, most construction contracts provide that the contractor is required to give written notice immediately to the engineer or SO who will carry out investigation to ascertain the situation.

Clause 16 of the PWD 203A on “Inspection of Site” places the burden on the contractor to carry out detailed pre-contract subsoil investigation at his own expense to adequately design and provide for the temporary works (Lim, 2004). In such a case, the contractor should price in the risk and contingencies accordingly.

6.0 COMPACTION OF EARTHWORKS

All earth filling generally shall be carried out in layers not exceeding 225mm thick loose layers. Each filling layer shall be thoroughly compacted by means of passes of a smooth wheel 6T roller or other approved compacting equipment and compacted to 95 % maximum dry density at optimum moisture content. Field trial compaction shall be carried out to determine the maximum compaction effort of the combination of construction plant.

Some major embankment failures occurred at the East-West Highway in 1980s and they were found to be either due to fill over a watercourse or fill without proper compaction. The danger of tipped-fill was well highlighted in a report by an Independent Review Panel as a result of the Sau Mau Ping disasters in Hong Kong (Geotechnical Engineering Office, 1999).

7.0 FAILURES OF EARTHWORKS

7.1 The Water Factor

The stability of a slope is often affected by the presence of water. It is evident that reports of landslides mostly occurred during periods of intense and prolonged rainfall, especially during the monsoon season. It is established that residual soil loses its suction during an extended period of intense rainfall which causes the phreatic line of the slope to migrate upwards thus inducing seepage flow in the slope. Wetting of the slope materials also causes a reduction in shear strength of the soil.

Water from run-off also causes erosion of slopes and the undermining of toe of slope. This naturally leads to slope instability. Adequate drainage and suitable turfing of slopes are essential to control the damage caused by water. In the publication by the Department of Irrigation and Drainage, Malaysia (DID, 2000) on Urban Stormwater Management Manual for Malaysia known as MASMA (Manual Saliran Mesra Alam) the need to control the discharge of run-off water to the rivers so as not to cause flooding of towns and cities located in the downstream areas was highlighted. The requirement to provide silt traps in earthwork projects has not been effective because of subsequent maintenance and enforcement problems. In Selangor, however, there have been instances of defaulting developers having to pay RM 250,000.00 fine for overflow of silt pond.

The ability to control the effect of water, both at surface and subsurface, is the key to solving the frequent occurrence of landslides on hill slopes.

In all the cases cited in Table 3, poor drainage and earthwork practice were factors that led to the collapse of the slopes.

Moh and Huang (2007) in the Opening Keynote Address at the 16th SEAGC in Kuala Lumpur also opined that “water is a prime cause of major failures in underground construction. For example, four of the five disastrous events, 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.”

Hussein and Alimat (2003) reported on slope failures that occurred in a mountainous road project, the ‘Simpang Pulai to Kampung Raja’ new road. A total of 42 slope failures were said to have occurred since construction started in 1998 and five years later the road is still not open to traffic.

The slope failures were attributed to a prolonged duration of rainfall, lack of adequate drainage, slope toe softening and tipped-fill slope in valleys as well as daylighting (exposure of unstable rock joints) and presence of weak planes in rock slopes. The authors suggest that perhaps the solution to slope failure in mountainous roads is to use viaducts, tunnels and cut and cover structures.

The roads were virtually not used long after five years of construction due to frequent landslides. It is understood that a new hill road from Gap to Fraser’s Hill for which construction started about the same time also suffered the same fate with more than 50 landslides.

Ministry of Works (MOW) through JKR organized a National Slope Seminar (MOW, 2001) in view of the erosion and slope failure problems connected with the Pos Selim – Lojing – Gua Musang Highway. It must be emphasized that analysis of slope failure needs to be thorough and rigorous. All possible modes of failure must be examined for various site conditions. Design must also allow for possible localised failure and provide

suitable mitigation to contain the risk of damage. It is obvious that analysis would be assisted by judgment as a result of past experiences and observations on the performance of the existing slopes. BS 6031 : 1981 and GEO (2000a) provides considerable guidance in handling these types of problems.

7.2 Failure of Earthworks Behind Bridge Abutments

In the early 1970s there were several bridges in the Kota Tinggi district where the 5 – 6m earth fill embankments behind the bridge abutments failed and bodily moved the bridge abutments and the piles with them during construction. At about the same time the 12m high embankment behind the new Temerloh Bridge abutment on the Temerloh side also failed during construction damaging the piles, tilting the abutment and at the same time damaging and tilting the pier immediately in front of it. The piles, abutment and pier were constructed prior to the embankment construction. The embankments were located over clay layers. All the affected abutments and pier had to be demolished and new piles, abutments and pier were constructed. Figure 13 shows the embankment failure at the Temerloh bridge abutment. Consequent to these incidences, a KPKR (Director-General of PWD) Circular was issued directing that embankments within 50m of the bridge abutment must be constructed to their full height first before piling for the bridge abutments can commence.

The implementation of this precaution and procedure appeared to put an end to the spate of embankments and abutments failures. However, in the mid-1980s, a bridge abutment in Selangor also suffered the same fate. In that case, construction of the 8m high embankment started after the piled foundation and abutment were constructed. At about 7m high, the embankment moved by about 1.12m in predominantly horizontal direction towards the river.

The failure could have been easily avoided had the embankment at the bridge abutment area be constructed first before piling and abutment construction. Any ground improvement required could also have been undertaken during the embankment construction prior to the construction of the bridge abutment. Clearly, this particular case ought never to have happened if the proper precautions and procedure had been adopted. Figure 14 shows the remedial work as reported by Chan (2000).



Cracklines in embankment.

General view of abutment and piers.

Demolition of abutment.

After demolition.

Figure 13 : Embankment Failure at Bridge Abutment, Temerloh Bridge

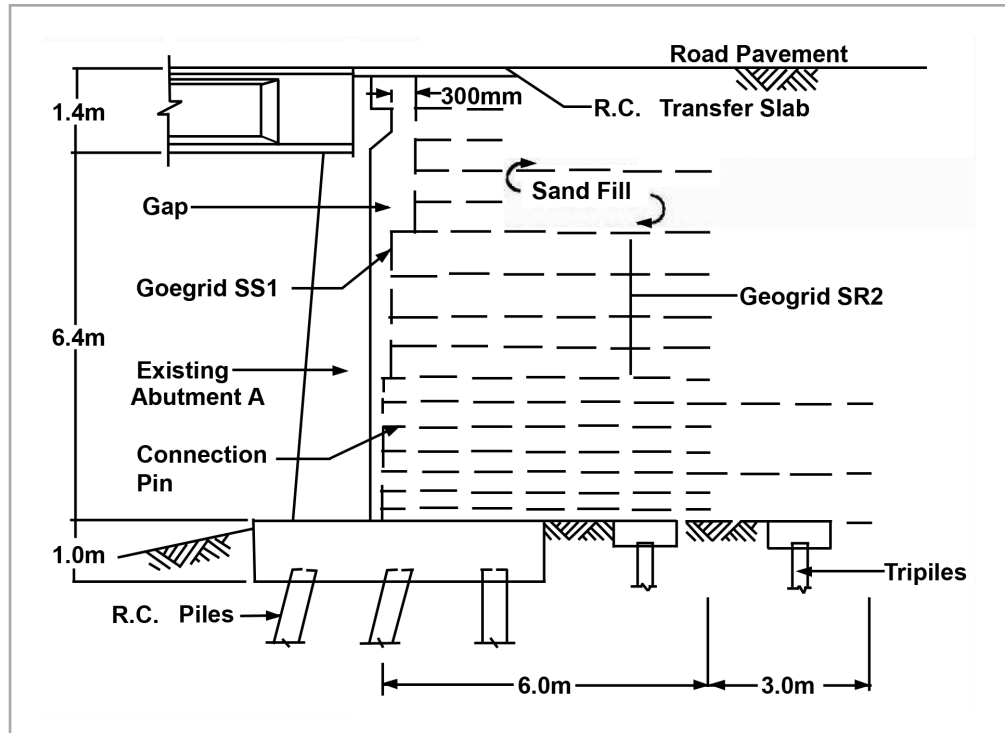


Figure 14: Shah Alam Bridge Abutment Failure, Selangor (after Chan, 2000)

Ting *et al.*, (1995) reported the use of vacuum consolidation at a bridge abutment where the strength of the soft ground had to be improved after the construction of the abutment structure. Vacuum consolidation was applied to the ground to increase the soil strength successfully without causing instability to the completed structures. The embankment was successfully constructed after vacuum consolidation, thus avoided a potential danger of an abutment failure during construction.

7.3 Other Instances of Geotechnical Works Failures

The Institution of Engineers, Malaysia held a Seminar on Failures Related to Geotechnical Works. At this seminar, Gue *et al.* (2000) reported 3 case histories of failure of houses in three separate housing schemes. Earthworks in the 3 case histories were all un-compacted and two of the case histories had boulders in the earth filled with large voids. Collapse settlement of the earth fill had caused the building to settle differentially, distorted and cracked. To begin with, the poor quality earthwork without proper compaction was bound to give problems. Had the earthworks been properly compacted, there would have been no need for piling and no failure of the building platform due to collapse settlement. Gue *et al.*, (2000) also attributed poor drainage as a contributory cause of failure of earthworks. The presence of boulders also caused obstructions to the penetration of the piles.

In a recent arbitration case in which the dispute between the claimant, a contractor and the respondent, the client arose amongst others, out of a piling contract in which the site was a former rubbish dump not identified by the client's architect and engineer during the planning and design stage. This necessitated a design and construction methodology change, thus resulting in variation claims and delay in completion of work during construction. The

claimant claimed an extension of time and cost and alleged lack of proper pre-contract planning and investigations on the part of the consultants. The learned arbitrator awarded an extension of time and also costs with interest to the claimant.

Low *et al.* (2000) reported similar use of un-compacted fill in another housing site where linked houses built on thick earth fill of 25 – 30m were experiencing distress due to settlement of fill.

There were also many unreported cases of houses on un-compacted filled grounds that were demolished during construction. It is clear that compaction of earth fill is of paramount importance in order to avoid settlement failure of houses.

The importance of compaction in dams and airfields construction has been known since the birth of soil mechanics and has been strictly enforced in the practice of dams and airfields works. The question that begs to be asked is: Why does there appear to be a total lack of such enforcement of compaction in the earthwork practice in the housing sector? A variety of reasons may be speculated but the practical solution moving forward must lie in vigilant and strict supervision by engineer and endorsement by the client.

8.0 EARTHWORKS IN EX-MINING LAND

Ex-mining land is geotechnically very complex. As a result of the mining process, the material can vary from pure sand to that of slime which can be on land or in pond. Slime is a very soft sandy silty clay material.

Ting (1992) gave a comprehensive review of the method of rehabilitation of ex-mining land in various types of ground conditions in particular the use of confinement method in the treatment of slime pond.



Figure 15: Figure 15 Ex-mining Land in 1986 Before Development

Yeow *et al.* (1993) discussed the development of ex-mining land for housing purposes at Bandar Sunway in Petaling Jaya. It was pointed out that detailed site investigations and ground settlement monitoring of earthwork are important to providing essential data for planning and design so as to achieve an economical and sound engineering solution. The old quarry site was stabilised with rock bolts and turned into a water theme park. Figures 15 and 16 show the aerial view of the site before and after development respectively. The slime materials were dried and mixed with sand and compacted in alternating layers. Raft foundation was used successfully for the 2-storey terrace houses. No materials were thrown away using the principle of sustainable development ahead of the worldwide use of the term “sustainable development” which started only in 2000s. Bandar Sunway was originally a huge 200ha of ex-mining land and disused quarry and is now a vibrant township with its Sunway Lagoon theme park being a popular recreational destination for both the locals and international tourists. The author was retained as the geotechnical specialist for the re-development of the ex-mining land in 1986.

9.0 EARTHWORKS ON SOFT GROUND

In carrying out earthworks on soft ground, it is generally accepted that ground improvement using Prefabricated Vertical Band Drain (PVD) with surcharge is the most cost effective method. Sand is usually used as the fill material. Ting *et al.*, (1987) presented some aspects of the design parameters of coastal alluvium and inland soft ground.

Where special consideration requires no external loading due to stability problem, vacuum consolidation has been used to improve the strength of the soft ground and to minimise post construction settlement before the permanent structures are built (Ting *et al.*, 1995 and Ooi and Yee, 1997). Ooi (1997) reported on the successful use of PVD and vacuum consolidation methods



Figure 16: Bandar Sunway in 2009, a Vibrant Township with Sunway Lagoon Theme Park at its Centre

for the ground improvement of the Senari Terminal of the Kuching Port, Sarawak.

In the construction of the North-South Highway, a special embankment trial known as the Muar Flat Trial was carried out to evaluate the technical and economical viability of the various ground improvement methods available at that time for the treatment of coastal clay deposits.

The allowable post construction settlement of 100mm over a two year period was imposed as an acceptance criterion. MHA (1989) held an International Symposium and invited international and local experts to present their predictions and to discuss the results of the trials.

In a country report for the 30th anniversary symposium of the SEA Geotechnical Society in Bangkok on the Soil Improvement Works in Malaysia at that time, Ooi (1997a) indicated that:

- (i) The methods that have been successfully employed for the ground improvement of alluvium are Vertical Drains (PVD) with surcharge, vacuum consolidation with or without dynamic compaction and vibro replacement using sand or stone columns. Among these methods, Vertical Drains (PVD) with surcharge is commonly used since it is the most economic solution.
- (ii) In the treatment of ex-mining land, the methods that have been successfully used are:
 - a) Vertical Drains (PVD) with surcharge for slime;
 - b) Dynamic compaction and vacuum consolidation for fill and slime;
 - c) Vibro replacement for slime using sand or stone columns;
 - d) Dynamic Replacement for sand silt clay mixtures using either sand or stone columns.
- (iii) Treatment of poor ground for test tracks and building foundations has been successfully achieved using dynamic consolidation technique.

10.0 LIABILITY ARISING FROM DISPUTES RELATING TO EARTHWORK PRACTICE

In this section, a number of common areas of liability arising from disputes relating to earthwork practice are examined with reference to five (5) reported court cases. It is common for most construction contracts to have arbitration clauses that require settlement of disputes (whether arising out of earthworks practices or otherwise) through private arbitration.

However, it cannot be emphasized enough that, particularly in the case of engineers, disputes in relation to construction failures are not restricted to contractual claims between the client and the contractor or consultant. Apart from the possible penal sanctions that may be imposed upon a guilty party by virtue of, for example, Section 71 of the Street Drainage and Building Act, 1974, third parties, such as adjacent landowners, and individuals are often affected by construction failures.

In such cases, the claims made by these third parties are tortious in nature and arbitration would not be resorted to since these third parties have no contractual relationship with the parties directly involved in the construction works. Inevitably, the engineer also faces exposure when such claims are brought, and if the failure is attributable, whether partially or wholly to a fault by the engineer in the performance of his duties, he may well be found liable to pay damages to these third parties or otherwise, even if he is not initially made a party to the action, he may in turn, find himself liable to the client by way of contribution for such damages (assuming the client is found liable).

There can be no denying that accountability for the performance of an engineer's duties is not restricted to the client. An engineer should at all times be mindful of the fact that he owes a wider duty to the public as regards works designed and supervised by him. BEM's Guidelines for Code of Professional Conduct provides:

"A Registered Engineer shall at all times hold paramount the safety, health and welfare of the public." (para 1.0)

From the perspective of non-contractual civil liability ('tort' in legal parlance) arising from earthwork practices, common exposure to third party claims of 'negligence', 'nuisance' and 'trespass' arise.

The 'wrongdoer', called the 'tortfeasor', need not have a contractual relationship with the injured party and even if there is a contractual relationship, it is possible for the 'tortfeasor' to be exposed to potential concurrent liability in both tort and contract.

10.1 Negligence

Broadly speaking, negligence is an act or omission by a person who owes a 'duty of care' to the person that is injured as a result of the consequence of that act or omission, provided there is no break in the chain of the events between the act/omission and the resulting damage (or in legal language, a 'break in the chain of causation') and provided that such damage is not

considered by the court to be too 'remote' *i.e.* unconnected with the act or omission.

A 'duty of care' is broadly considered to be owed by a person performing a certain act or in a certain position to parties that may reasonably be contemplated (by objective standards taking into account the person's function) to be injured or affected by the acts or omissions of the person in the specific capacity of his position. Commonly the duty is sufficiently wide enough to cover 'product liability' of manufacturers (as in the case of *Donoghue v. Stevenson* [1932] AC 562) to persons professing to have expertise, giving advice in the context of such expertise (as in the case of *Hedley Byrne and Co. v. Heller and Partners* [1964] AC 465).

Landowners or person occupying or having control over a site owe a duty of care to take reasonable precautions to ensure that what is done or left on the site does not injure the interest, pecuniary or otherwise of those in the surrounding areas (such as adjacent landowners) and for the occupational health and safety of workers or visitors on the site itself.

In the context of engineers, as professional persons, liability may also arise not merely from a design failure or a failure to adequately supervise works but also out of representations made in their professional capacity, whether to clients or third parties, that may induce these parties to act or restrain from acting in reliance of such representations.

It is now settled in Malaysia, by the Highland Towers case, that liability for the damage suffered by the wronged party, need not be restricted to damages for personal injury or restoration of property but may also extend to "pure economic loss", that is to say, compensation for damage for diminution in property value depending on whether the scope of the duty of care in the circumstances of the case is such as to "*embrace damage of the kind which the plaintiff claims to have sustained*" (*Majlis Perbandaran Ampang Jaya v. Steven Phoa Cheng Loon* [2006] 2 CLJ 1).

10.2 Nuisance

Nuisance may broadly be defined as the interference by a landowner or occupier with his neighbour's quiet enjoyment of the neighbour's land. In the context of construction, this may include the emission of fumes or objectionable odours from the land, excessive noise, interference with the neighbours' land (which will be touched on more specifically below) or the obstruction of free passage. It may be public, whereby the rights of the general public are affected or private, whereby a specific party (commonly the neighbouring landowner) is affected.

Although it has been considered difficult to exactly define what may constitute an actionable nuisance, the Highland Towers case has made it clear that in the case of private nuisance, this may also arise in situations where there has been an encroachment on a neighbour's land or direct physical injury to a neighbour's land resulting from the acts of the landowner or occupier on his land.

It has also been reaffirmed by the Highland Towers case that where an actionable case of nuisance has been made out, the damages remedy may extend to compensation for pure economic loss.

10.3 Trespass

Broadly, an act of trespass may be said to have been committed if a person enters, whether by himself or by placing or projecting objects, and remains upon and land possessed by another without permission or lawful justification. It has been held that even the slightest crossing of the boundary may be sufficient to constitute trespass and unlike the case of negligence and nuisance, trespass is actionable without proof of damage (*Terra Damansara Sdn Bhd v. Nandex Development Sdn Bhd* [2006] 8 CLJ 657).

In the context of construction and particularly earthworks, instances of trespass commonly occur where, contractors cut access paths through or leave earth or other materials or erect structures on adjacent land. In this regard, the engineer's duty to ensure that setting out of a building has been carried out in accordance with the approved site plan is expressly highlighted in the Uniform Building By-Laws 1984 (revised Nov 2007) By-Laws 23 which provides:

- "1. As soon as the setting out of building has been completed, the qualified person shall give written notice to the local authority in Form C as set out in the Second Schedule to these By-laws certifying either that the setting out has been carried out in accordance with the approved site plan or, if there has been any deviation from the approved site plan, that he will undertake to submit the required number of amended site plans for approval before the completion of the building.
2. In either event the qualified person shall certify that he accepts full responsibility for ensuring that all town planning and building requirements are complied with."

There are instances where structures are built on adjacent properties as in the case of *Terra Damansara* and *Yip Shou Shan* (discussed below) where landowners were held to have been guilty of permanent trespass by building structures on neighbouring land.

10.4 Consequences

The consequence of the commission of such acts may be liability in damages to compensate the wronged party and/or injunctive relief to restrain the tortfeasor from performing further damaging acts or to perform, at its own expense, the necessary works required to remedy the damage. The Highland Tower case has also made it clear that persons, even though not acting in concert, who commit a tort against another person contemporaneously causing the same or indivisible damages, will be each be liable for these damages.

10.5 Practical Examples of Earthworks Disputes – Case Studies

Dunlop (M) Industries Bhd v. Seong Fatt Sawmills Sdn Bhd

A classic example of liability of a landowner to his neighbour in negligence and nuisance arising from earthworks is the relatively

old case of *Dunlop (M) Industries Bhd v. Seong Fatt Sawmills Sdn Bhd* [1982] CLJ(Rep) 440. From the reported decision, the facts that may be ascertained are as follows.

In 1973 the defendant cut the side of the hill on its land and filled the ponds at the bottom of the hill to a level of almost 4ft above the level of the plaintiff's adjacent land and the drainage pattern of the defendant's land was altered by the replacement of natural streams running through it with boundary earth drains that joined the plaintiff's earth drain.

The plaintiff's earth drain was meant to take water from the rear of the plaintiff's land but consequently and as a result of the defendant's works had to take water from the defendant's new drain.

The evidence given by the plaintiff was that every time it rained, chunks of earth fell from the slope closing off the drain below and breaking up the plaintiff's fence. On 19 November 1976, there was a heavy rainfall and the plaintiff's land, including the factory sitting thereon was flooded. The plaintiff filed an action against the defendant for nuisance for altering the natural drainage and structure of the plaintiff's land and claimed damages for extensive damage to land, roads, buildings and goods stored.

Upon hearing the evidence, the court granted plaintiff's claim and found the defendant guilty of both negligence and nuisance. Damages and cost were awarded and the defendant was also ordered to abate nuisance. The comments by the court in that case, extracted below should serve as a strong reminder as to the importance of implementing proper earthwork practices under the supervision of a professional engineer:

"The plaintiffs had warned the defendants in May 1974 verbally as well as by letters of the dangers as there were instances of pieces of the cliff adjoining the plaintiff's land falling into the drain below. The defendants had promised to put up a retaining wall but they never did so. They had also not got the permission of the relevant authorities to build a timber shed and the deviation of the stream. According to PW3 this permission was necessary and in order to divert a natural stream a consultant engineer had to be engaged for the submission of plans and approval. The defendant had completely disregarded the consequence of their action and ought to have foreseen the damages which the rain water flowing from their land would have caused to neighbouring land because the natural flow had been diverted and the drain dug along the boundary was inadequate for the free flow of water."

It may be surmised from the facts of the case as reported that the defendant may have avoided the action if it had followed proper practice and engaged the services of an engineer to properly advise on and supervise the earthworks that were undertaken. The common factors for earthworks failures appear to be prevalent in that case in that heavy rainfall exposed an inadequate draining pattern of the filled and apparently untreated cliff that arose out of the earthworks. In this regard, the following passage quoted from *Clerk and Lindsell on Torts (13th Edition)* adopted by the court in its decision is also instructive:

“Liability in respect of water depends on whether the water is naturally on the land or whether it is artificially accumulated or interfered with in some way. The owner of land on a lower level cannot complain of water naturally flowing or percolating to his land from a higher level. Nevertheless, the higher proprietor is liable if he deliberately drains his land on to his lower neighbour’s land and this appears to be so if the water is caused to flow in a more concentrated form that it naturally would, as the result of artificial alterations in the levels and contours of the higher land.”

Dr Abdul Hamid Abdul Rashid v. Jurusan Malaysia Consultants

The High Court case of *Dr Abdul Hamid Abdul Rashid v. Jurusan Malaysia Consultants* [1999] 8 CLJ 131 affirmed by the Court of Appeal as reported in [2006] 1 CLJ 391, involved, amongst other things, a direct claim in negligence by a client against the engineer for an apparent failure to perform adequate soil testing. The facts that may be ascertained from the report of that case are summarised below.

The plaintiffs had hired the 1st defendant, an engineering firm, to construct a house on a piece of property belonging to them on Lot 3007 in Ulu Klang, Kuala Lumpur. The 5th defendant, who was the chief clerk and draftsman in the first defendant firm, attended to the plaintiffs. Plans of a house were drawn and signed by the fourth defendant who was a registered engineer and the proprietor of the first defendant. These plans were approved by the 2nd defendant, the town council.

At the recommendation of the 1st defendant, the plaintiffs entered into a written building contract with a contractor to build the bungalow. One of the terms and conditions of the building contract was that the contractor was to perform the works shown in the drawings and the specifications prepared by or under the direction of the 1st defendant.

The bungalow was duly completed and the plaintiffs and their family moved into the said building on 11 April 1985. The slope was partly cut and tipped filled without compaction to form the platform and the slope. The material of the slope was of sandstone/shale origin.

In or about the middle of 1987, the 3rd defendant, a contractor, commenced construction works on Lot 3008 which was a lot adjoining Lot 3007 to erect another residential building.

Between the night of 17 September 1988 and the early morning of 18 September 1988, the concrete deck (swimming pool) and boundary brick wall together with the side of the house facing the river at the toe of the slope at the rear portion of the plaintiffs’ bungalow collapsed due to a landslide and as a result the bungalow became uninhabitable and the plaintiffs were forced to abandon the house.

The plaintiffs thus suffered losses, damages and expenses. The plaintiffs claimed that the damage to the bungalow and Lot 3007 were due to breach of duty by all the defendants either jointly or severally.

The defendants claimed that they had fully discharged their legal and/or contractual duties by adopting the normal engineering practices based on their experience in development of building sites and housing infrastructures.

The High Court however, found in favour of the plaintiffs and ordered that the 1st, 3rd and 4th defendants to pay the sum of RM364,173.00, allowing the plaintiffs’ claims in connection with the cost of replacing a new house, furniture, fixtures, fittings, security costs and rental costs. It may be noted that whilst the claim against the 3rd defendant, the contractor, was grounded in tort (negligence and nuisance), the claim against the engineer i.e. the 1st and 4th defendants was concurrently made in contract, for breach of an implied term to exercise reasonable care and skill in the performance of duty, and in tort for negligence.

The claim against the local council, the 2nd defendant, for breach of statutory duty, was dismissed on account of immunity conferred upon it by virtue of Section 95(2) of the Street, Drainage and Building Act, 1974. The High Court also found that there was insufficient evidence to support the plaintiffs’ claim that the 5th defendant was a co-proprietor of the first defendant and thus liable as well and therefore, the claim against him was dismissed.

In the course of the judgment, the High Court found that the engineer owed a duty of care to the plaintiffs to ensure that the house was safe for habitation when he accepted his appointment as engineer to design the house and as the house was located on a steep slope and the Engineer should have exercise due care in his design to ensure the house is safe.

The High Court also had no problem implying a term into the contract between the plaintiffs and the 1st defendant that the 1st and 4th defendants would exercise reasonable skill and care. It was held:

“... the contract between the plaintiffs and the first and/or fourth defendants is one of performance of services by professionals who have described themselves as ‘consulting civil and structural engineers’. Any persons declaring themselves to be such must reasonably and equitably be expected to take reasonable care and skill in the performance of their craft. This term of the expected reasonable care and skill is so obvious in the first defendant’s appointment that the court finds it to come within the ambit of ‘it goes without saying’... For these, it qualifies to be accepted as an implied term of the contract between the plaintiffs and the first and/or fourth defendants.”

According to the reported case, shortly after the collapse, the local council instigated the appointment of an engineering consultant, to determine its cause. Apparently, in the report subsequently published, the consultant attributed the collapse to slope failure caused by lateral movement of the earth supporting the foundation of the house which was located on top of a 45° slope. It was postulated that due to infiltration of water, such as heavy rainfall which increased saturation of the soil causing a rise in water table and a reduction in soil suction resulting in a decrease in soil shear strength along the potential failure plane that led to the occurrence of landslide.

The determination of the shear strength of the soil was one of the vital factors in deciding slope stability. However, in the report of the High Court judgment, it was observed that the 4th defendant had merely relied on sight and feels to determine the strength of the soil whereas the experts conducted exhaustive tests before concluding on the strength of the soil.

The High Court also observed that the river at the toe of the slope should have worried the engineer as regards to erosion and the possible instability of the riverbank and the slope but the engineer took little consideration in assessing the stability of slope in his design in respect of heavy rainfall and the possibility of slope saturation.

The High Court held that it was essential for an engineer to determine the soil condition to a high degree of certainty. The High Court then went on to hold that a failure to do so must be accepted as a breach of the implied term of the engineer's appointment to take reasonable care as well as negligence on the part of the engineer. It was held that an engineer of such qualification and skill as the 4th defendant should have taken all the relevant matters into more serious consideration when designing and devising the plans to ensure that the house safe for habitation but instead, a casual attitude was adopted without much care and skill practised. The 1st and 4th defendants were therefore held liable for breach of contract and for negligence.

In the context of liability by the contractor for the earthworks on the adjoining land, the 3rd defendant, it was alleged that the 3rd defendant had unnecessarily allowed infiltration or seepage of water into the ground and/or allowing it to overflow onto the plaintiffs' property thereby causing saturation in the soil resulting in the landslide which brought the plaintiff's house down.

It was held that the 3rd defendant had artificially accumulated rainwater by its excavation works which constituted an alteration to the nature of the land. It was also held that the 3rd defendant had also interfered with rainwater by constructing transverse drains ending a three quarter way down the slope of the neighbouring plot of land. The High Court concluded:

"All these had affected the natural flow of the water resulting in its concentrated and increased infiltration into the land thereby causing destructive effect to the plaintiffs' property. By such deeds, the third defendant had breached their duty of care towards the plaintiffs in respect of negligence, caused nuisance to the plaintiffs, as well as being liable in part under the rule in Rylands v. Fletcher."

In apportioning liability between the engineer and the contractor, the High Court also did not accept arguments made by the 3rd defendant against the 1st and 4th defendants of contributory negligence but ordered an apportionment in the payment of damages, namely 60% be borne by the engineer (1st and 4th defendant) and 40% by the contractor.

This case underscores two important points. First the importance of safe earthwork practices, particularly adequate provision for drainage which must at all times take into account the conditions of the surrounding areas, which was also the central theme in the *Dunlop (M) Industries Bhd v. Seong Fatt Sawmills Sdn Bhd*. Secondly, an engineer should never take lightly his responsibility towards adequate design and supervision in any project under his care and this must necessarily entail performing thorough and detailed soil investigation to ensure that suitability of the project site for the erection of the structure designed.

On a side note, the rule in *Rylands v. Fletcher* referred to by the High Court is generally considered as a separate tort from negligence, in that it carries strict liability without the need for a plaintiff to prove negligence. This rule, which is derived from an old English case, *Rylands v. Fletcher* (1868) L.R. 3 H.L. 330 is stated as follows:

"... the person who for his own purpose brings onto his land and collects and keeps there anything to do with mischief if it escapes must keep it at his peril, and, if he does not do so, is prima facie answerable for all the damage which is the natural consequence of its escape."

The application of this rule as a separate tort independent from negligence was dispelled by the High Court in the Highland Towers case (which incidentally was a later decision by the same judge who gave judgment for the plaintiffs in this case) who preferred the approach of the Australian High Court in *Burnie Port Authority v. General Jones Pty Ltd* 120 ALR 42, which incorporated it into the general law on negligence.

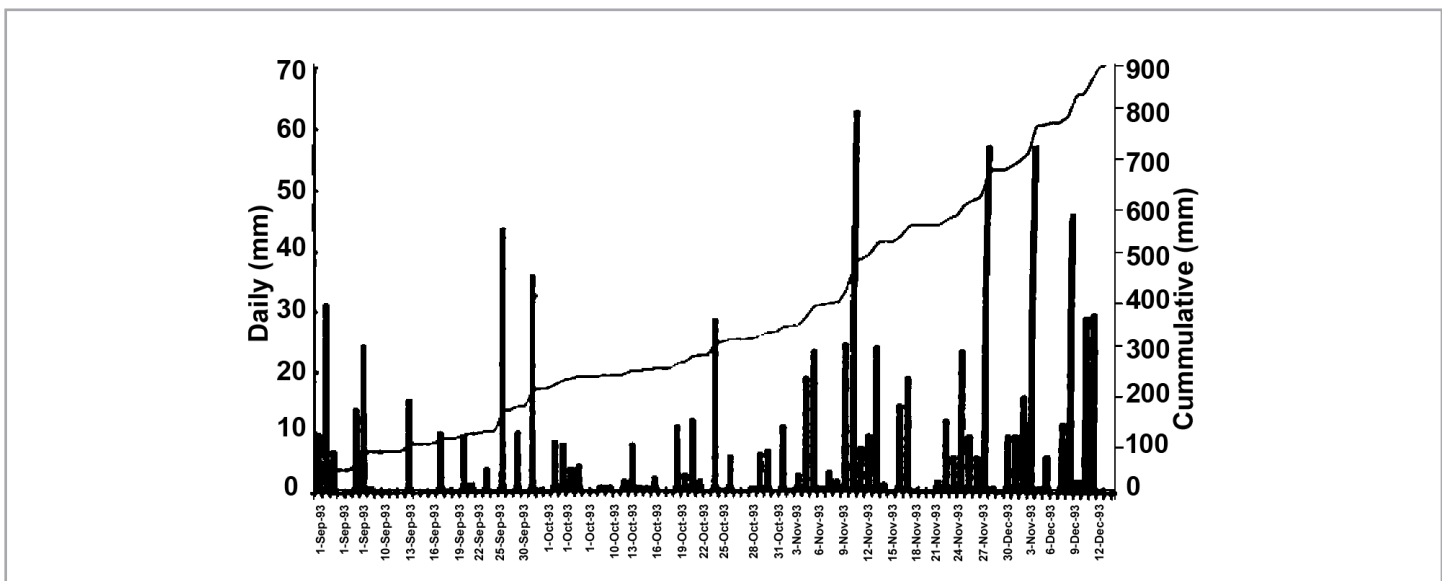


Figure 17: Daily rainfall from 1/9/93 – 12/12/93 recorded at the DID Ampang

Highland Towers

The series of reported decision by the courts in relation to the Highland Towers collapse probably remain the most often quoted example of a court case involving failed earthworks. The first of the decisions reported was the High Court decision after full trial, *Steven Phoa Cheng Loon and Ors v. Highland Properties Sdn Bhd and Ors* [2000] 4 CLJ 508.

Highland Towers comprised three 12 storey apartment blocks located in the Bukit Antarabangsa area in the district of Ulu Klang and the mountain range of Titiwangsa. The bedrock geology of the site is granite. One of the apartment blocks collapsed and the residents in the two apartments adjoining the apartment block that collapsed were evacuated. These residents then sued the developer (1st defendant), the architect (2nd defendant) and the engineer (3rd defendant) of Highland Towers, the landowners of the adjacent land (5th defendant), the landowner of the property located above the adjacent land and its management services provider (7th and 8th defendants), the contractor that carried out site clearing on that adjacent land (6th defendant) for negligence and nuisance and the local authority (4th defendant), state Government (9th defendant) and the state Director of Land and Mines (10th defendant) for negligence as well.

On 11 December 1993 at 1.30pm, during a period of 10 days of incessant rain, Block 1 of Highland Towers Condominium collapsed resulting in the loss of 48 lives and the loss of use of the remaining 2 Tower Blocks. The collapsed Block 1 of the Condominium Tower is shown in Figure 2.

Figure 17 shows the rainfall distribution from September – December 1993. On the same figure, the cumulative rainfall was also plotted. It can be seen that the cumulative rainfall on the day of the tragic event was about 900mm. The annual rainfall for 1993 was 2604mm. Thus the cumulative rainfall from September to 11 December 1993 accounts for 35% of the annual rainfall. The intensity of rainfall was severe in the month of December prior to the day when the slope and the Block 1 Tower collapsed.

The local authority (MPAJ, 1994) set up a Technical Committee of Enquiries and the findings as reported are as follows:

1. The Highland Towers Blocks was sited mainly on fill ground over granitic formation. The maximum depth from the ground surface to bedrock is about 19m. Granitic rocks found in and around the areas were not highly soluble minerals to adversely affect the stability of the foundations.
2. Soils overlying the granitic bedrock were very loose to loose silty sand and highly permeable.
3. The foundation for all the 3 Tower Blocks were supported on rail piles designed to take only vertical loads.
4. Surface drainage system provided was not in accordance to approved plan. Situation worsens when earthwork activities changed the drainage pattern on hill-slope behind the Tower Blocks and available drainage systems were not maintained.
5. Clearing of trees on upper catchments resulted in increased runoff that flowed down the terraced hill-slope immediately behind the towers.
6. Retrogressive slides progressively moved uphill starting from loss of toe mass at the back of the Tower Block 1 (see Figure 18).
7. The fallen debris accumulated behind the back terrace of Tower Block 1 caused the landslide to occur beneath the entire rail pile foundation that brought down Tower Block 1 within minutes of the landslide occurrences. (note however, Yee (2008) has analytically disproved this hypothesis).

It must be pointed out that the MPAJ 1994 report was accepted as admissible by the High Court only as to the factual data contained therein. The court rightly held that the findings and opinions expressed in the report still had to be evaluated by due process of procedural law or in other words, they could not be accepted at face value.

The High Court further held that since no member of the committee responsible for the opinions expressed in the MPAJ 1994 report were called to give evidence, the said report would only be considered on the basis of documents agreed upon by the parties.

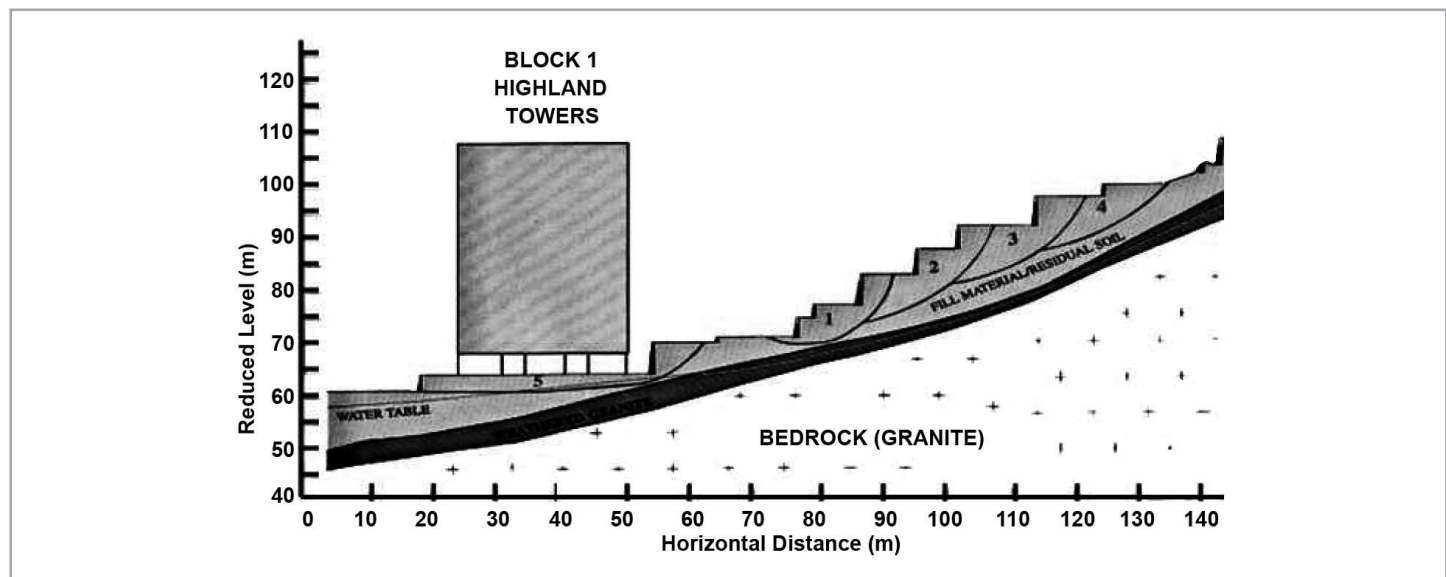


Figure 18: Retrogressive Slope Failure (after MPAJ, 1994)

Upon assessing the evidence, the High Court did go on to hold and accept that rotational retrogressive slope failure emanating from the high wall behind the second tier car park was the cause of the collapse of the Block 1 of Highland Towers.

The High Court also held that water from upslope development and its drainage system and maintenance was the major factor contributing to the slope failure that caused the high wall to fail. The following excerpts from the High Court's judgment are instructive:

"... failure of a wall as defined by Professor Simons means 'the ground (beneath it)' has failed" even though the "foundation or structure of the wall may not fail". In this case, it must be the former since this High Wall is still visible in the Mitchell pictures. With this, we must examine the soil conditions beneath this wall. We have evidence that the suspected area of failure consisted of sandy soil. Such soil material is very permeable and water will percolate into it very fast. With ten days of continuous rainfall in the area before the failure of this wall surely the ground on which it stood would be saturated with water when the draining system of the slope was either insufficient or inadequate to accommodate water... it is also established that when soil is saturated behind any retaining wall, it will create a thrust against the wall..

... But where did the water come from? From the evidence adduced it came from two sources. The first was rainfall... part of it was absorbed into the ground and percolated into the soil. The other would be runoffs and washed along the surface. With the internal drains on the Arab Malaysian Land, the water would be directed down the slope in a controlled manner. But these drains on the Arab Malaysian Land were neither sufficient nor efficient or maintained to carry the load, as designed by the drainage experts... Substantial part was earth drains and this permitted easy percolation of water into the soil to saturate it. Some were blocked or with vegetation growing over them...

... Such blockage must have caused sever overflow on the terraced slope...

The second source was water from the East Stream. As described earlier water from this stream was directed into the pipe culvert... in very poor condition, damaged in many parts with water leaking therefrom...

Not only was water not flowing smoothly along the pipe culvert, the area before the water of the East Stream enter this channel was also heavily silted... the inlet into the pipe culvert completely covered with silt. Due to this, water from the East Stream over flowed on to the slope."

Having held this, the High Court proceeded to find the architect (who at the time of his engagement was not a registered architect but a building draughtsman) and the engineer of Highland Towers both liable to the residents of the remaining Tower Blocks for negligence and nuisance, essentially for (1)

the failure to ensure adequate stability and drainage to the hill slope and (2) unreasonable usage of the land. The engineer was particularly chastised by the High Court for *"lack of consideration paid by this defendant to the hill and the slope directly behind the three apartment blocks"*.

The High Court also rejected the architect and engineer's argument that they were not responsible for design and supervision which were beyond the purview of their employment with the developer. The engineer contended that he was not involved in the design or construction of the retaining walls on the hill slope (other than the two retaining walls in front of Block 2) but the High Court had this to say in rejecting this argument:

"... this view is totally unacceptable since, and as I have stated, the paramount duty of an engineer for the Highland Towers was the safety of the buildings he was involved. This duty cannot be exempted by a mere belief of the retaining walls and terracing of the slope were designed, supervised and built by the 1st defendant, whose director was an engineer himself, or another firm of consultants, and therefore presumed safe. If this was the belief of the 3rd defendant, then it encumbered upon him to inquire and to ascertain whether: firstly, this other consultant is a qualified engineer, and secondly what he was doing would have any effect on the safety of Highland Towers."

The engineer also contended that the drainage plan which was duly approved was not fully implemented by the developer, due to various reasons including shortage of financial resources from the developer. The High Court was, however, firm that such an attitude did nothing to absolve or shift the engineer's responsibility for negligence and expressly held:

"... but to my mind, what ever the excuse may be it did not entitle and warrant the 3rd defendant to issue a notice to the authorities stating that the entire approved drainage proposal was implemented when, according to my estimates, only 10% was completed. This was a gross violation of his duty of care, which as a consultant engineer for the three apartment blocks, he owes to the plaintiffs as purchasers of Highland Towers, particularly when this approved drainage system was so fundamental to the safety of the building."

The High Court also sent out a strong rebuke to engineers that sought to shy away from their greater and paramount public responsibility to ensure safety, health and welfare of the public by making the following observation:

"I have reiterated my strong sentiments against this type of attitude of professionals whose only consideration is to guard and secure their own interest rather than their duties and obligations to those closely affected and the public on which so much faith and reliance are placed on them to carry out their professional duties."

The High Court found the developer to be liable for negligence in failing to provide adequate drainage for the discharge of 'East Stream' water and therefore, for nuisance as well.

As regards the local authority, the High Court applied the immunity provisions of Section 95(2) of the Street, Drainage and Building Act 1974 in respect of pre-collapse works but held that this immunity did not extend to post-collapse events, in respect of which the local authority was found liable for negligence and nuisance for its failure to incorporate 'East Stream' into its master drainage plan after the collapse occurred, notwithstanding assurances that it would do so.

The landowner of the adjacent land was also held to owe a duty of care to the residents and further to be in breach of the same by its failure to maintain proper drainage for the slope. This duty and breach was also held to continue post-collapse, in that it was held that the landowner ought to have taken steps to prevent water from flowing in an uncontrolled manner over its slope. This landowner was also found, as a consequence of its liability to be liable for damages caused by vandalism and theft that occurred in the remaining Tower Block post-collapse. The said landowner's site clearing contractor was, however, absolved as it was held that there was no evidence linking its work to the cause of the landslide.

The landowner of the property located above the adjacent land and its management services provider were also both found to be liable for nuisance and negligence. The High Court found that on a balance, the clearing of the land within the boundaries of this landowner's land: *"had a significant contribution to the runoff entering the drainage system, and consequently to overflow into the hillside"* and that:

"In the factual matrix of our case, the water at the Metrolux Site was naturally on the land but these defendants had artificially erected barriers on their land to redirect its natural flow path into the East Stream which consequently caused the damage suffered by the plaintiffs. Such acts of these defendants are closely and directly connected to the damage and for this, the 7th and 8th defendants must be liable to the plaintiffs."

The High Court dismissed the residents' claim for negligence and nuisance against the state Government and the state Director of Lands and Mines on preliminary legal points relating to whether they had been correctly sued (in the case of the state Government) and whether there had been a sufficiently pleaded case made out (in the case of the state Director of Lands and Mines).

In conclusion, the High Court apportioned contribution for liability in damages, which were held to be assessed separately, against those parties held so liable in the following manner: the developer – 15%, the architect – 10%, the engineer – 10%, the local authority – 15%, the adjacent landowner – 30% and the landowner of the property located above the adjacent land and its management services provider – 20%.

The High Court's decision was appealed and the Court of Appeal (reported in [2003] 1 CLJ 585 as *Arab-Malaysian*

Finance Bhd v. Steven Phoa Cheng Loon and Ors) which upheld the fact findings of the High Court. However, the Court of Appeal reversed the High Court's findings vis-à-vis the local authority by first, holding that in so far as post-collapse liability was concerned (*i.e.* the failure to act on the master drainage plan), this could not stand on the basis that such a duty to act had to be enforced by way of judicial review and not private law proceedings but then holding that the local authority could not rely on Section 95(2) of the Street, Drainage and Building Act 1974 for immunity against a negligence claim. It was on this footing that the Court of Appeal upheld the apportionment of liability prescribed by the High Court.

In so far as the measure of damages was concerned, the Court of Appeal upheld the High Court's decision that the 'pure economic loss' of diminution of property value was recoverable (on the basis that such damage was reasonably foreseeable in the circumstances of the case) but disallowed damages for vandalism and theft, which it held were too remote.

The case went further to the Federal Court (reported in [2006] 2 CLJ 1 as *Majlis Perbandaran Ampang Jaya v. Steven Phoa Cheng Loon and Ors*) which unanimously reversed the Court of Appeal's decision on the local authorities' post-collapse liability, rejecting the Court of Appeal's view of the resident's exclusive remedy being judicial review but by a majority holding that a pure economic loss remedy against the local authority should not be allowed in circumstances of the case and unanimously reversed the Court of Appeal's decision on the local authorities' pre-collapse liability relying on Section 95(2) of the Street, Drainage and Building Act 1974.

The upshot of the Highland Towers saga was that the High Court's principle findings as to liability for negligence and nuisance were upheld save as to that in relation to the local authority. One unfortunate post script to note is that as the court battle went on, the two other Towers Blocks that were declared unsafe for occupation have been left vacant and unattended even till today, some 16 years after the incident.

This case, has important implications for developers, building professionals, absentee landlords and developers of neighbouring properties in Malaysia and the findings made against the engineer that he had failed in his duty of care to the plaintiffs to design and supervise a building that was safe for occupation sound a crucial warning to all engineers, that notwithstanding client considerations or budgetary constraints, they cannot derogate from their wider responsibilities to the public at large.

Some important lessons respecting earthworks practices may be taken away from the Highland Towers collapse. In general, water has been the principal cause of many slope failures as can be seen in Table 3. The design should have taken into account of suitable surface and subsurface drainage of slopes. The use of tipped-fill on slopes and embankments should never have been allowed under any circumstances but this bad practice remains unabated.

The drainage system must be comprehensive and generous, well built and easy to maintain. Post-completion maintenance of the drainage system must necessarily be performed at all time to ensure continued safety at hill-site developments.

Adopting the recommendations in the Hong Kong Geoguide, in so far as drains on slopes are concerned, the actual capacity of a drain is only half of its design capacity and when the slope on which the drain is situated becomes overgrown, the drainage capacity is reduced further to only a quarter of its design capacity.

These general and commonsensical guidelines often appear not followed in many slope drain designs in Malaysia. In particular, the use of v-drains on slopes, which remains common in Malaysia, should be discouraged.

Slope failures are often associated with situations where water overflow the drains which may common arise when v-drains are used. Step-drains are far better alternatives for slope drainage as they are good energy dissipaters and provide easy access to slopes for inspection and maintenance.

The Institution of Engineers, Malaysia, concerned about the gravity of failure of earthworks on hill-sites with the Highland Towers incident in mind, organized a "Symposium on Hill-site Development". Ting (1995) at that symposium presented design concepts for building on hill-sites. The methods discussed in that paper are very useful and should be considered by practicing engineers.

The Institution of Engineers, Malaysia (IEM, 2000b) submitted a proposal on the classification of slopes for hill-site development to the Government. This proposal however, appears to have been since overtaken by the result of the court case and the occurrence of more recent earthwork failures that took place after the tragedy. IEM has since the more recent Bukit Antarabangsa Landslide in December 2008 reviewed its IEM, 2000b proposal.

Eu Sim Chuan v. Kris Angsana Sdn Bhd

Another case which underscores the importance of the utilising proper earthworks practices is *Eu Sin Chuan v. Kris Angsana Sdn Bhd* [2007] 7 CLJ 89. The plaintiffs in that case were husband and wife living in a property at No.290A, Lorong Palas, Off Jalan Ampang on which was constructed a double storey bungalow house that was registered in the name of the wife.

According to the reported decision, the defendant was developing the land immediately adjacent to the said property to construct two 20 storey condominium blocks and parts of the initial works carried out by the defendant were piling activities which involved excavation and removal of soil.

It would appear that during the course of this work, the plaintiffs' property began to see the development of cracks in various parts of the bungalow and the compound and on inspection by the plaintiffs' engineer, the bungalow was said to have had suffered structural damages particularly the existence of cracks on the floor area, the walls, column and beam as a result of the earthwork activities by the defendant which were said to have had caused movement and settlement of the underground soil. The plaintiffs, on the advice of their engineer, vacated the said bungalow house for fear of their safety. They then sued the defendant for negligence in carrying out the construction works adjacent to their said property.

The High Court allowed plaintiff's claim for damages amounting to a hefty sum of RM6,306,242.43. This decision was subsequently upheld by the Court of Appeal (*Kris Angsana Sdn Bhd v. Eu Sim Chuan and Anor* [2007] 4 CLJ 293).

From the reported decision of the Court of Appeal, it appeared to be admitted by the defendant's witness that the damage to the plaintiff's property was due to:

"the settlement of sub-soil strata due to the lateral movements of earth during the construction of the deep basement adjacent to the building and settlement due to the lowering of the ground water table during the construction of the deep basement"

Both the High Court and the Court of Appeal had no problem holding that the defendant did owe the plaintiffs a duty of care which had been breached causing damage to the plaintiff's property. In particular, the High Court held:

"It is a common knowledge that whenever any activities of sheet piling, excavating and removal of soil are carried out in any area it would cause movement of the water level of the land in the surrounding area. The likely consequence would be that any building constructed on the neighbouring land would develop cracks depending on the degree... ... A developer like the defendant who employs engineers for carrying out such construction works must be fully aware that the activities it carried out at the work site would likely cause damage to the plaintiffs' bungalow house and should therefore take the necessary steps to prevent damages to the plaintiffs' house."

But the defendant in the instant case chose not to take any such preventive measures before commencing the construction works..."

Both the High Court and the Court of Appeal expressly rejected a contention by the defendant based on an old English authority, *Acton v. Blundell* [1843] 152 ER 1223 that it could not be liable for extracting water from under its land even though such action may deprive his neighbour the use of that water. In this regard, the Court of Appeal held:

"With respect to the archaic view of Acton v. Blundell the realities of modern life must not be discounted. High density of population in popular residential areas in Malaysia is now the norm. Houses may have to be built very close to each other, at times on hilltops, or even hugging those slopes. To allow the incoming new house owner or contractor to take away the ground support of adjacent buildings, justifying such acts on natural user of his land, and thereafter blaming gravity and soil subsidence (or dewatering) as an operation of the laws of nature, is not in sync with reality... ... As it stands, if no reasonable steps were undertaken by the wrongdoer to ensure that no damage would befall the neighbours, and did indeed suffer them, an actionable tort of negligence may await him."

What would be pertinent to note from this case, is the extent of damages assessed and allowed by the court. The RM6,306,242.43 awarded (which was exclusive of the interest of 8% per annum starting from the time of the filing of the action that was also awarded) comprised:

1. the plaintiffs' expert and consultation fees paid to its engineers, valuers and quantity surveyors to the sum of RM43,828.85;
2. compensation for the rental incurred by the plaintiffs after they had moved out of their property which totalled RM1,230,246.58;
3. the plaintiffs' costs of employing a watchman to guard the property in their absence which was tabulated at RM88,000.00;
4. the costs of demolishing and rebuilding the bungalow tabulated at RM3,393,167.00 (assumedly this was an estimate) which the High Court held would have been cheaper than the cost of repairing the existing structure;
5. the cost of moving back into the property, which was claimed at RM50,000.00;
6. general damages of RM1,000,000.00 for mental distress and hardship to the plaintiffs, as apparently, there was evidence to show that 2nd plaintiff's health had drastically deteriorated on account of the damage to the property;
7. exemplary damages of RM500,000.00 on account of the defendant's behaviour which the High Court described as a "couldn't care less attitude".

It may be observed that the award of exemplary damages is rare, this case illustrates a court's willingness to award the same when it finds a defendant's conduct to be sufficiently 'outrageous to merit punishment'. In this case, the High Court had observed:

"The sole concern of the defendant was to quickly complete the construction and reap as much benefits with the minimum amount of costs incurred. It is a very selfish attitude most undesirable in a community we Malaysians are used to. The behaviour and attitude of developers towards their neighbours have been so degrading that appropriate authorities should take necessary steps to check their activities to ensure that their neighbours are not adversely affected"

and the Court of Appeal in upholding the award of exemplary damages against the defendant went further to say:

"For a company that was about to build a 20 storied 2 block-building, the above sum of RM500,000 was quite modest, and would make no impact on its means."

What this case reinforces is that the performance of earthworks without due regard to the safety and preservation of neighbouring property may result in real and costly consequences from a civil liability perspective, potentially exposing an errant developer to not only ordinary damages but exemplary damages as well, particularly if the persons responsible for the mishap do not take immediate proactive measures to address the same.

Whilst the engineer for the defendants' project was not made a party to the action and thus his liability not considered or discussed in the case, the hypothetical question to be posed is what would the extent of his liability, as the submitting and supervising person for the project have been, applying the approach of the courts in the Highland Towers case.

Yip Shou Shan v. Sin Heap Lee – Marubeni Sdn Bhd

The approach of the courts adopted in the *Kris Angsana Sdn Bhd* case of punishing an errant developer with exemplary damages in a civil claim by an adjacent landowner is not new or novel.

In the earlier reported case of *Yip Shou Shan v. Sin Heap Lee – Marubeni Sdn Bhd* [2002] 5 CLJ 574, a plaintiff landowner sued the defendant developer for general and exemplary damages for trespass and nuisance. The plaintiff also sought declarations and injunctive relief, primarily directed towards the defendant abating its trespass, providing the plaintiff with access to his land and the reinstatement and rehabilitation of the ground levels of the plaintiff's land.

In that case, the plaintiff was the landowner of a piece of agricultural land. The defendant was the developer of a golf course and a huge residential and commercial complex known as "Bandar Sungei Long". The plaintiff and developer's respective lands were separated by a strip of state land approximately 40 ft wide, referred to by the High Court as "the access reserve".

The trespass complaint was that the defendant had committed two separate acts constituting trespass. The first instance was that some time in 1991, the defendant excavated the access reserve (and although this does not appear clear, the High Court seems also to have concluded that the defendant also excavated the plaintiff's land) thus creating a steep slope on the plaintiff's land abutting the access reserve of about 100 ft high, 650 ft long and 49 ft deep into the plaintiff's land. The second instance was that in February 1996, the defendant proceeded to, without the plaintiff's consent construct a crib wall on the access reserve and part of the plaintiff's land.

The nuisance complaint was said to arise from the withdrawal of soil to soil support and the loss of use by the plaintiff of his land as a result to the physical damage caused to it as a result of the trespass.

It appears from the reported decision that it was common ground that the defendant constructed the crib wall to counter slope failure and apart from the issue of the defendant trespassing by virtue of its alleged unauthorised construction of the crib wall, the plaintiff's contention was that the crib wall was not sufficient for this purpose having been designed to protect 39 ft only.

The evidence led by the witnesses and subsequently accepted by the High Court was that the defendant was guilty of trespassing on the plaintiff's land with the consequence that the slope on the plaintiff's land was unstable and further failures would occur unless remedial measures were taken if the plaintiff was to retain the integrity of the use of his land.

The High Court also held, based on the evidence that the crib-wall built by the defendant was only a temporary measure that did not protect the plaintiff's land and was not sufficient to prevent future soil failures and that any future development

by the plaintiff on his land would either require very expensive foundation work or a setback very much further away from the boundary to ensure safety. The High Court also found as a fact that the plaintiff's loss of earth as a result of the excavation was 0.6294 acres and his loss of use of land resulting from the recommended setback was 1.2843 acres.

Consequently, the High Court found the defendant guilty of nuisance and that "the plaintiff had suffered actual damage in consequence of the torts committed by the defendant". It would be pertinent to note that in the course of the written judgment, the High Court made reference to the decision of the Federal Court in an East Malaysian case, *Wong See Lee and Ors v. Ting Siik Tay* [1997] CLJ 205, which although relating to the question of whether an adjacent landowner claiming nuisance and negligence against a developer for causing loss of support to her land, could have a caveatable interest in the developer's land, contained the following observation, which was reproduced by the High Court:

"In our view, the courts are entitled to take judicial notice of the fact that Malaysia records one of the highest rainfall in the world which inevitably make soil erosion and landslides a matter of foreseeable consequence wherever there is disturbance of the soil as a result of human activity. As a matter of common sense, retaining walls have become one of the bulwarks in the fight against soil erosion and in the preservation of soil to soil support enjoyed by neighbouring lands particularly where excavation activities have become necessary on one's property."

The High Court however declined to grant the declarations and injunctions initially sought by the plaintiff largely on account of such relief having become obsolete with time. What the High Court did do, was to decline an assessment of damages based on diminution of value of the plaintiff's property but instead assessed the cost of repair or reinstatement. In this regard, it was held:

"to my mind, is the true measure of damage since I am satisfied from the evidence that all along the plaintiff intended to develop the land according to the subdivision as approved. Even if the plaintiff now changes his mind and wants to sell the land it is doubtful whether the property in its existing state can be sold at all or at a good price due to the substantial costs that will have to be expended in stabilising the slope. In any event, if the plaintiff is to put the land to any use at all the construction of a retaining wall is the only solution because the danger to life and property, both to the occupants above and below is clearly foreseeable. The computation and quantification of the cost of reinstatement had been established by SP5 in his testimony and report in 1993 at RM3 million (B-71) based on plans B-21 and B-22 which had not been seriously challenged. Although the figure is only an estimate and SP5 did not have any particular type

of wall in mind, I accept that figure as reasonable for the cost of reinstatement by reason of SP5's vast experience as a professional and qualified engineer. As the figure was given some eight years ago, to my mind, it is appropriate to increase it by at least 20%, that is to say, to RM3.6 million. With that amount the plaintiff will be in a position to take positive steps to stabilise the slope in order to meet his requirements in the development of his land, which will be at his discretion and risk.

Apart from awarding RM3.6 million as compensatory damages, the High Court also awarded special damages of RM16,248 being the costs of the plaintiff's engineer and surveyor. The High Court, however, also went further to award exemplary damages using a loose formula of 25% of the compensatory damages awarded or RM900,000.00. The reasons given by the High Court justifying the order of exemplary damages included the following views:

- a) from an observation of the defendant's conduct, the trespass having first been alerted to the defendant in June 1991, was deliberate, intentional and carried out with a cynical disregard for the plaintiff's rights;
- b) that the defendant felt that the gain it would obtain a deliberate trespass would outweigh the compensation it might have to pay and its conduct was calculated to result in a profit *i.e.* that the defendant would (i) not lose any land if the slope was placed on the plaintiff's land, as sp done and (ii) save in not having to put up a retaining wall;
- c) the conduct of the defendant from 1991 to the trial of the action was reprehensible as:
 - i) the excavation on the plaintiff's land was carried out before the earthworks plan was approved and consequently illegal and unauthorised;
 - ii) despite verbal and written promises, the defendant did not approach the plaintiff to discuss resolving the problem nor were the plaintiff's consulting engineers efforts to suggest remedial works met with any response,
 - iii) despite indisputable evidence represented by the survey plans, the defendant persisted in denying encroachment and proffered unarguable defences that the earthworks were sanctioned by the appropriate authorities and/or that there were no further encroachment between the plaintiff's first survey resulting in the plaintiff's land being tied up in legal action for almost eight years and effectively frozen;
 - iv) the defendant's witnesses were, since the beginning of this action and at the beginning of this action and at the trial, less than candid, endeavouring to justify and sustain the obviously unsustainable, and
 - v) the defendant had, whilst the action was still going on, proceeded with further works at the slope by constructing the crib wall without notice to the plaintiff or his consultant resulting in permanent encroachment and affecting future remedial works.

The defendant's appeal to the Court of Appeal (reported as *Sin Heap Lee – Marubeni Sdn Bhd v. Yip Shou Shan* [2004] 4 CLJ 35) was dismissed.

This case is also instructive in highlighting another common problem arising from inadequate supervision of earthworks, namely trespass.

What may be surmised from the reported decisions is that a clear view was taken by both the High Court and the Court of Appeal that the defendant, in a bid to save its own costs and maximise its own development space, and thus its profit by taking advantage of its neighbour's vacant land.

This unfortunately appears to be a somewhat common practice and the approach taken by the courts in this case should also go towards serving as a stern warning that if civil action is taken by the neighbour, the courts would be willing to be especially harsh on the responsible parties in terms of the damages that they may award, which may include exemplary damages, which are punitive by nature, thus again, re-emphasising the need for the parties responsible for the mishap to take proactive steps to address the same.

In this case, the reported decisions suggest that the defendant had gone so far as to illegally cut into the plaintiff's land without even securing approval for its earthworks from the local authority. As was the situation, with the *Kris Angsana Sdn Bhd* case however, the defendant's engineer was not named as a party to the action, but it may be surmised that had this been the case, the engineer would have had much to answer for as well.

11.0 RECENT DEVELOPMENTS

In recent years, there has been increasing public awareness and also enforcement by the local authorities for earthworks to be carried out in accordance to environmental protection requirements and submission of Environmental Management Plan (Ooi and Othman, 2002). Erosion control has been singled out at the seminar on Best Earthworks Management Practices organized by CIDB in 2002. The needs to provide turf cover and reduce uncontrolled runoff by way of better designed siltation pond were highlighted. Health and Safety at work are important and they are monitored by DOSH.

The ICE Charter on Sustainable Development is now incorporated as an attribute in the ICE Chartered Professional Review (CPR). We need to bear in mind sustainability in whatever thing we do or design as civil engineers (Venables, 2001), *i.e.*, we must not make worse the built environment in which we are in. The wetland of Putrajaya is part of Government's effort in creating a sustainable development model. The Malaysian Government's commitment to sustainable development is demonstrated by the provision of a huge allocation of RM1.5 billion for green technology development and innovations in the 2010 budget.

Ooi and Tee (2004) have reviewed the development in the technology of slope reinforcement and rehabilitation and concluded that geogrid reinforced slope is a sustainable method of slope reinforcement and rehabilitation. Yee and Ooi (2007) reviewed the progress of sustainability of ground improvement for the last 30 years. In their recent paper to the IEM Green

Workshop Yee and Ooi (2009) have shown positively the sustainable method of ground improvement solution by a combination of dynamic replacement with vertical drains to support a high embankment on marginal ground with significant reduction in carbon footprint as compared to the conventional method of removal and replacement.

Maintenance is an important aspect of earthworks and is provided for in BS 6031. Through regular maintenance, failures can be prevented especially in the case of controlling the water factor. We need to cultivate a good maintenance culture so that we can proudly claim world class facilities that last.

12.0 MITIGATION/RISK MANAGEMENT

Accepting that not all construction mishaps may be foreseen even by parties with the best intentions, it is important for the parties involved in construction to ensure that associated risk are sufficiently managed and spread so that the added financial burden of addressing mishaps are adequately addressed by the party with the best capacity to absorb the same, thus allowing the parties to move forward to speedily address and correct such mishaps.

In this regard, the need for parties to obtain adequate insurance coverage for each project undertaken whether through "all risk policies" or "professional indemnity insurance" cannot be underscored enough and all policies taken out should be carefully scrutinised to ensure that they adequately cover the risks that may be anticipated with particular regard to the project undertaken.

It would however be pertinent to note that contractors' "all risk policies" generally exclude coverage for loss or damage suffered due to faulty design and whilst professional indemnity insurance does go some way to mitigating the financial risk of a mistake by the engineer, such coverage may often be insufficient.

Therefore, whilst greater efforts should be made to develop and implement insurance policies that may practically address the effective transfer of risk, insurance should not be seen as safety net or crutch that allows parties to be any less diligent in the performance of their functions. This is especially the case for engineers.

13.0 CONCLUSION

The Public Works Department (PWD) was the sole agency in shaping the Practice of Earthworks in Malaysia up to 1970s. The setting up of independent testing laboratories within the PWD and the project sites was fundamental in enforcing quality control of Earthworks. This was particularly important and successful in the implementation of airfield and dam construction during the period. The earthworks for Subang international airport are acknowledged to be an outstanding piece of earthwork (Skepper *et al*, 1966) that gave confidence in the development of design and construction of the Kuala Terengganu and Senai international airports in the early 1970s.

From 1980s onwards there were significant changes in the earthworks practice where marginal ground needed to be treated

before construction of houses, highway and other structures. PVD with surcharge, dynamic consolidation, vibrofloatation and vacuum consolidation were popular methods being used for ground improvements.

Steeper slopes were made possible through the use of soil reinforcement and soil nail techniques (Ooi and Tee, 2004).

Ex-mining land is complex and unique in Malaysia and indigenous rehabilitation techniques have been developed to handle such problems to make them suitable for development. Sustainable method of ground improvement has been practiced in 1980s at the re-development of the ex-mining land where Bandar Sunway is now located. Inter-layering technique of compacted dried slime and sand was used successfully in the earthwork and the raft foundation was used for the 2-storey terrace houses.

Earthworks Practice generally follows that of the British Standard, BS 6031 : 1981 which are adequate when used with JKR Standard Specifications in most circumstances. Site Investigation which is an essential prerequisite to good earthworks practice follows that of BS 5930 : 1999. A Malaysian code has also been published based on BS 5930 : 1999 with local experiences incorporated.

In most cases of earthwork on slopes, water and tipped fill has been found to be the main causes of landslides or mud avalanches. Hong Kong pays particular attention on the treatment of existing fill slope by compaction and soil nailing into fill slope as a robust solution to mitigate against slope failure.

Lack of vegetation is the main cause of erosion and silt traps are often found not functioning properly in large scale earthwork such as platform for housing development. For most cases of failure of houses, the root cause has always been due to lack of compaction, use of tipped fill and uncontrolled infiltration of water.

In the case of bridge abutment failures during construction of embankments, these can be avoided by first constructing the embankment prior to piling works for the bridge abutments. Failure to learn from the past lessons arising from the failure to adhere to these basic principles will have serious consequences and there is really no good reason as to why similar failures need ever be repeated.

Court cases had shown that engineers are duty bound to provide safe and sound design under foreseeable conditions during the entire service life of the structures. They may even be found liable for consequential economic losses as a result of failure of their designs. This is demonstrated by the case of *Highland Towers*.

The party responsible for the change of structure of the land by doing extensive earthworks thus altering the natural drainage, as in the case of *Dunlop Industries v. Seong Fatt Sawmills*, is liable for claims arising from damages as a result of consequences affecting his neighbour arising from these earthworks, albeit done on his own land. It would be pertinent to note that in recognition of the potential impact of earthworks on surrounding areas, Environmental Impact Assessment (EIA) studies are compulsory for development exceeding 50ha and it is mandatory to have EMP in place before commencement of earthworks.

In so far as engineers are concerned, the old adage of 'prevention being better than the cure' should remain the mantra. As a submitting person, the engineer should carry out sufficient due diligence as to the substructure and structure which he has undertaken to design for the client and he has a paramount duty, not just to the client but to the public to ensure that he delivers a building that is safe and fit for occupation.

The engineer is therefore obliged to examine the surrounding areas and address all loadings both dead, live and prospective, or at least other incidental loadings that may be reasonably imposed on the building during its service life and to carefully deal and provide for with the effect of surface and subsurface water on the stability of slopes in his design, bearing in mind the importance of future slope maintenance. The engineer must also ensure that all fill ground is carefully supervised and compacted to ensure that a sufficiently high degree of compaction is achieved.

It is not correct for an engineer to hide behind the limits of his contractual responsibilities to the client nor should he succumb to pressure from the client to derogate from his duties for the sake of saving the client money. Rather the engineer should, in the performance of his functions, always be mindful of his greater responsibility to ensure and uphold safety, health and public welfare. ■

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MULTI-LAYER PERCEPTRON MODEL FOR SOIL LOSS PREDICTION DUE TO FOREST LOGGING

(Date received: 4.6.2009)

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ABSTRACT

Geophysical conditions and levels of disturbances induced from forest activities have great impacts on hydrology. Clear-felling have the greatest impacts and is one of the major contributions to soil loss apart from construction of forest roads and timber harvesting. The ecological and economic forest values are largely dependent on the degree of erosion. Prediction of soil loss rate is therefore essential in order to preserve the above values. Multi-layer perception (MLP) model is proposed to predict soil loss due to forest logging in an experimental watershed comprising of three sub-catchments located in Bukit Tarek forest reserve in Malaysia. The measurement of soil loss was made in terms of sediment yield for the catchment under study. The proposed architecture uses back propagation networks with multiple hidden slabs of different activation function. The neuron architecture for each slab of the proposed models for sub-catchments 1, 2 and 3 in Bukit Tarek Watershed are 5:3:3:3:1. Five input variables namely the rainfall, length slope, soil erodibility, cropping management and conservation practice factors are used in this model. The proposed model had successfully predicted soil loss with great accuracy. This model has several advantages over other conventional methods for its simplicity and quick solution.

Keywords: Multi-Layer Perception Model, Soil Loss

1.0 INTRODUCTION

Soil loss estimate is very much required in the evaluation of different management practices, control techniques for forested catchments and for the purpose of watershed conservation. Soil loss that occur as a result of the construction of forest roads, timber harvesting, or fire would have detrimental effect on soil properties and structure. This is confirmed by [1] where soil erodibility is very much dependant on the surface cover and soil texture. A study carried out by [2] had found that the soil erodibility greater in areas between skid trails. [3] had confirmed the previous finding where soil erodibility is less affected in an undisturbed forest. [3] and [4] reported that soil loss from exposed areas or abandoned field would only decline over time if adequate foliage cover is provided. The above literature had provided useful information on the importance of the parameters namely vegetation cover, soil erodibility, rainfall and length of slope to be considered in the establishment of soil loss model. The incorporation of the above parameters are supported by the Universal Soil Loss Equation (USLE).

[5] had carried out a study to evaluate the effect of land slope and vegetal cover to erosion and runoff. However, no clear criteria were given for the 13 events used in the calibration of their model. They had indicated that the runoff volumes were better simulated than erosion losses.

The most recent erosion model Reused Universal Soil Loss Equation (RUSLE) which was developed by [6] had incorporated the watershed geomorphology, soil type, land use, distribution and derivation of rainfall in their model. The use of small grids and the modeling computation had made possible the variation of variables in a watershed be included. However, the approach may be time consuming if it involves larger watershed.

The recent development was to advance towards a much simpler approach. Many had resorted to Artificial Neural Network (ANN) for various engineering applications such as hydrological rainfall runoff modeling, stream flow forecasting, groundwater modeling, water quality, water management policy, precipitation forecasting, hydrological time series and reservoir operations [7]. ANN model to predict soil erosion is indeed an alternative to the empirical models [8]. Nevertheless, the application of ANN in erosion studies has not been fully explored.

This study aims at establishing a soil loss multi-layer perception (MLP) model based on the Universal Soil Loss Equation (USLE) as an alternative to the conventional approach. Evaluation of Universal Soil Loss Equation (USLE) was made using data collected from the Bukit Tarek Experimental Watershed in Selangor. Comparative analysis had been carried out on the predicted values using USLE equation and the proposed model.

2.0 STUDY AREA

Figure 1 shows the experimental site of Bukit Tarek watershed. The watershed drains an area of about 80 hectares into the main river, Sungai Kerling. There are three sub-catchments

within the watershed namely C1, C2 and C3. Sub-catchment C1 (33 hectares) acts as the control catchment. Sub-catchment C2 drains an area of approximately 34 hectares while the third sub-catchment C3 has an area of about 12.5 hectares which was established in October 1993. The watershed on the map is located at Latitude 3°31'30" North and Longitude 101°35'00" East. The catchment characteristics for sub-catchments C1 and C2 are shown in Table 1. The physiography of sub-catchment C3 is not available at the time of study. Weirs are located at the lowest contour level of the respective catchments of which all flows and eroded soils would be deposited. The catchment is drained by a third order stream that eventually flows into Sungai Jerneh, the tributary of the main river Sungai Kerling.

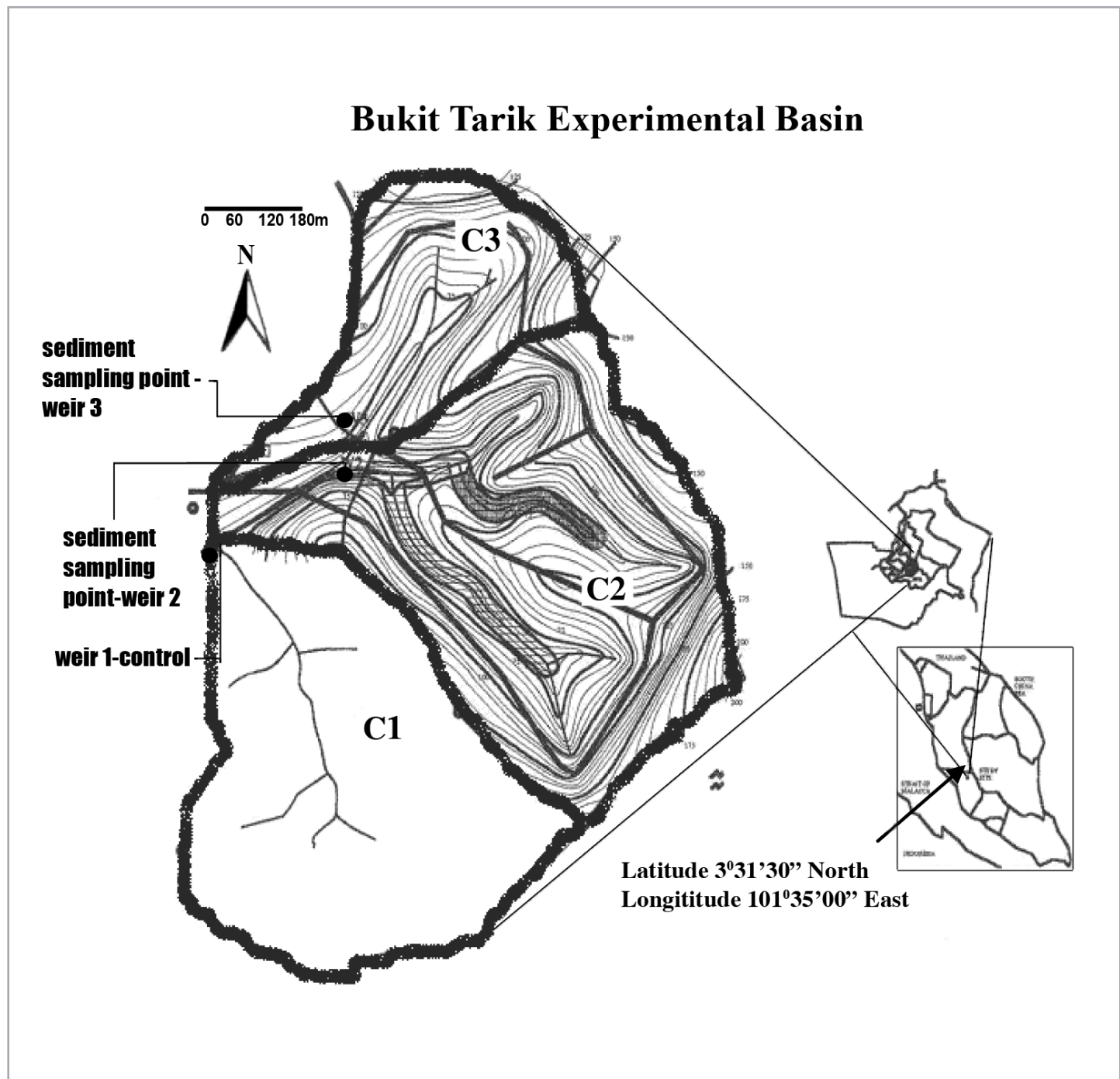


Figure 1: Bukit Tarek experimental basin [9]

Table 1: Equation and method used in determining the soil loss predictions of USLE

Characteristics	C1	C2	C3
Area (ha)	33	34	13
Elevation :			
Highest MSL	175	213	na
Lowest MSL	48	53	na
Mean slope (%)	33	45	na
Drainage network (m)	1664	1660	na
Drainage density (km ² /km)	5.1	4.9	na
Length of overland flow (m)	130	122	na

3.0 SOIL LOSS ESTIMATION USING UNIVERSAL SOIL LOSS EQUATION (USLE)

USLE was used to estimate soil loss in forest areas. The equation is as follows,

$$A = R * K * LS * C * P$$

where **A** is the soil loss in tons/ha/yr ; **R** is the rainfall erosivity factor ; **K** is the soil erodibility factor ; **LS** is the topographic factor (slope length and steepness) ; **C** is the

cropping management factor ; and **P** is the conservation practice factor.

The above equation was developed based on sediments derived from splash and sheet erosion (these are functions of soil and rainfall properties) which is specific for forest land. This equation does not consider gully or channel erosion [10]. According to [10], derivation of the USLE is specific for the forest land. Rainfall, soil erodibility, slope length, crop management and conservation practice are the four factors in this equation. Equation and method used in determining the soil loss predictors are given in Table 2.

Table 2: Catchment characteristics for sub-catchment C1 and C2 [6]

Soil loss predictors used in USLE	Equation / Method
Rainfall	R factor can be calculated using the equation proposed by Foster et al. [11] and Morgan [12] $E = 9.28 \quad P = 8838.15$ Where E is annual erosivity and P is annual rainfall $R = (E * I_{30}) / (100 * 17.02)$
Soil Erodibility	K can be calculated by using Warrington <i>et. al.</i> [13]. The data requirements for estimating K factor are <i>soil permeability, soil structure, % of organic matter, % of sand and % of silt and fine sand</i>. Soil series at Bukit Tarek Watershed is categorised as the Kuala Brang Soil Series and Bungor Soil Series, of which K factor for Kuala Brang Soil Series is 0.18 and Bungor Soil Series is 0.14. The above was taken from the soil survey result estimation for each grid in the (80m x 80m) plot was done using soil map.
Length Slope	LS factors was derived using the method proposed by Julien [14].
Cropping Management and Conservation Practice	C and P factors were calculated using the method Julien [15]. It was the modified version of the one proposed by Wischmeier and Smith [16].

The above equation was developed based on sediments derived from splash and sheet erosion (these are functions of soil and rainfall properties) which is specific for forest land. This equation does not consider gully or channel erosion [10]. According to [10], derivation of the USLE is specific for the forest land. Rainfall, soil erodibility, slope length, crop management and conservation practice are the four factors in this equation. Equation and method used in determining the soil loss predictors are given in Table 2.

In this study, a smaller grid size of 80m x 80m was used to estimate length of slope factor. Smaller grids would yield better accuracy in the estimates for rainfall, soil erodibility and vegetation covers. The factor values for every grid are used as data input for the model. Summary of the range of maximum and minimum soil loss estimates and statistical analysis for the respective catchments are given in Table 3.

Table 3: Summary of range of soil loss estimates and statistical analysis for the three catchments

	C1	C2	C1
Min (t/ha/yr)	0.3781	0.7824	8.5183
Max (t/ha/yr)	0.7194	2.1866	16.4071
Mean	0.5242	0.5242	11.9447
Median	0.5318	1.4267	12.2963
Std Deviation	0.1044	0.3508	2.4708
Variance	0.0109	0.1231	6.1048

This study had used USLE as the basis for computation of soil losses. Table 4 shows the range of values of soil loss predictors used in the analysis. The mean rainfall factors used in the analysis vary in the range from 852 to 872 mm. The mean soil erodibility factor is 0.16 for all sub-catchments and is categorised as Kuala Brang and Bungor Soil Series. Mean length of slope values are in the range of 9.0 to 15.5.

Mean of crop management and conservation practice factors are in the range of 0.001 to 0.003 and 0.12 to 0.3 respectively. The conservation practice factor depends on the type of activities in the watershed. There are substantial differences between the sub-catchments C2 and C3 as buffer zone has been established for sub-catchment C2. While in sub-catchment C3, no buffer zone is available. In assessing soil losses in forest areas, it is also necessary to account for the different forest treatment such as clear felling of trees, burning, re-planting and other activities. Due to the limitation of the current approach, the above may be difficult to incorporate.

Table 4: Range of values of soil loss predictors for sub-catchments C1, C2 and C3 used in the evaluation of USLE

Sub-Catchment	Soil loss predictors used in USLE	Values	
		Range	Mean
C1 (control catchment)	<i>R</i>	615.0 – 1170.4	852.8
	<i>K</i>	0.14 – 0.18	0.16
	<i>LS</i>	1.6 – 55.5	15.5
	<i>C</i>	0.0001 - 0.001	0.00055
	<i>P</i>	0.12	0.12
	A (tons/ha/year)	0.38-0.72	0.52
C2 (clear felling with residual trees left at the site - with buffer zone)	<i>R</i>	419.2 – 1171.7	804.3
	<i>K</i>	0.14 – 0.18	0.16
	<i>LS</i>	1.3 – 46.5	9.2
	<i>C</i>	0.003 – 0.009	0.006
	<i>P</i>	0.12	0.12
	A (tons/ha/year)	0.78 – 2.19	1.50
C3 (Clear felling but the residual trees were burnt - no buffer zone provided)	<i>R</i>	621.9 – 1197.9	872.3
	<i>K</i>	0.14 – 0.18	0.16
	<i>LS</i>	2.5 – 48.0	8.8
	<i>C</i>	0.003 – 0.009	0.006
	<i>P</i>	0.3	0.3
	A (tons/ha/year)	8.52 – 16.41	11.94

Note: *R* factor can be calculated using the equation proposed by Foster et. al. [11] and Morgan [12]. The values for the *K* factor can be calculated using Warrington et. al. [13]. *LS* factors can be derived using the method proposed by Julien [14]. Cropping management factor, *C* and conservation practice factor, *P* can be using the method proposed by Julien [15] which the methods had modified after Wischemeier and Smith [16].

4.0 PROPOSED MULTILAYER PERCEPTRON SOIL LOSS MODEL

The NeuroShell 2 developed by the Ward Systems Group Inc. is used in this study. The software is a window based system that runs under the Windows 95 operating system. The various steps involved in the development of the ANN Soil Erosion Model are discussed at length in this section. This section presents the development of the proposed model using Multilayer Perceptron Model network structure with back-propagation algorithm. Development of the proposed ANN soil loss model had undergone a series of processes such as variable selection, designation of neural network architecture, training, testing, production and validation phases.

During the preprocessing stage, the data were grouped into three distinct sets called training, testing, production or validation sets. Design and test options were chosen for the designation of neural network architecture specifically for training and testing

phases. The training set would be the largest set used by neural network to learn patterns present in the data. The testing set is used to evaluate the generalisation ability of the supposedly trained data set. Test set extraction was used to separate a test data set from the training data. A final check on the performance of the trained network was made through the production set of the trained model.

The soil loss predictors (R , K , LS , C and P) in each grid size serve as inputs in the proposed model. All data had undergone rigorous screening before they were used for model development and model testing. The robustness of the proposed model can be confirmed through the production phase. In this phase, the targets (measured soil loss value) were removed. There were only the input parameters that were being fed into the model. The performance of the trained network is measured from the discrepancy ratio. Predicted soil erosion values given by the model are solely based on the selected architecture, momentum and the learning rate parameters of the trained network. The ratio of the predicted soil loss values (calculated using the trained network) to measured soil loss values were then determined. Predictions are deemed accurate if the values lie between the discrepancy ratios of 0.5 to 2.0. About 80% and 20 % of data were used in the training and testing phases, respectively.

The selected architecture for the proposed network is error-back propagation algorithm with multiple hidden slabs of different activation function. Back propagation network was chosen because of their ability to generalise well on a wide variety of problems. According to [8], back-propagation network is a supervised type of network that uses both inputs and outputs in training the model. However, training may be slower than other paradigms (architecture) depending upon the number of pattern. Degree of accuracy will increase by creating a separate network for each output.

The hidden layers in a ward network function act as feature detectors. Selection of ward networks is based on the suitability of the input data. Different activation functions for the different

hidden layers of each slab detect different features of pattern processes through the network. The output layer will consists of different views of data that combines two feature sets that may lead to better estimates.

The detail design of neural network architectures for sub-catchments C1, C2 and C3 are illustrated in Figure 2. Number of neurons refer to the parameter predictors (R , K , LS , C and P) that are used to estimate the amount of soil loss. The neuron are then distributed to the system network for computation of soil loss in the system. In back-propagation, the network computes the mean (average) squared error between the actual and predicted values for all outputs over all patterns. The way it works is that the network first computes the squared error for each output in a pattern, totals them, and then computes the mean of the total for each pattern. The network then computes the mean of that number over all patterns in the training set. According to [17], a learning rate is used to increase the chance of avoiding the training process being trapped in a local minimum instead of global minimum.

Calibration interval refer to the specific number of events or test set patterns that are propagated through the network before the average error for the test set is computed. This is imperative in the testing phase. Calibration finds the optimum network for the data in the test set (which means that the network is able to generalise well and give good results on new data). Calibration does this by computing the mean squared error between actual and predicted for all outputs over all patterns. Calibration computes the squared error for each output in a pattern totals them and then computes the mean of that number over all patterns in the test set.

To design a neural network that gives the best estimate for soil loss when compared against the measured soil loss would involve several trial runs with different options for the momentum and learning rate parameters. The model development involved training of the input data to get the best model that can accurately predict soil loss.

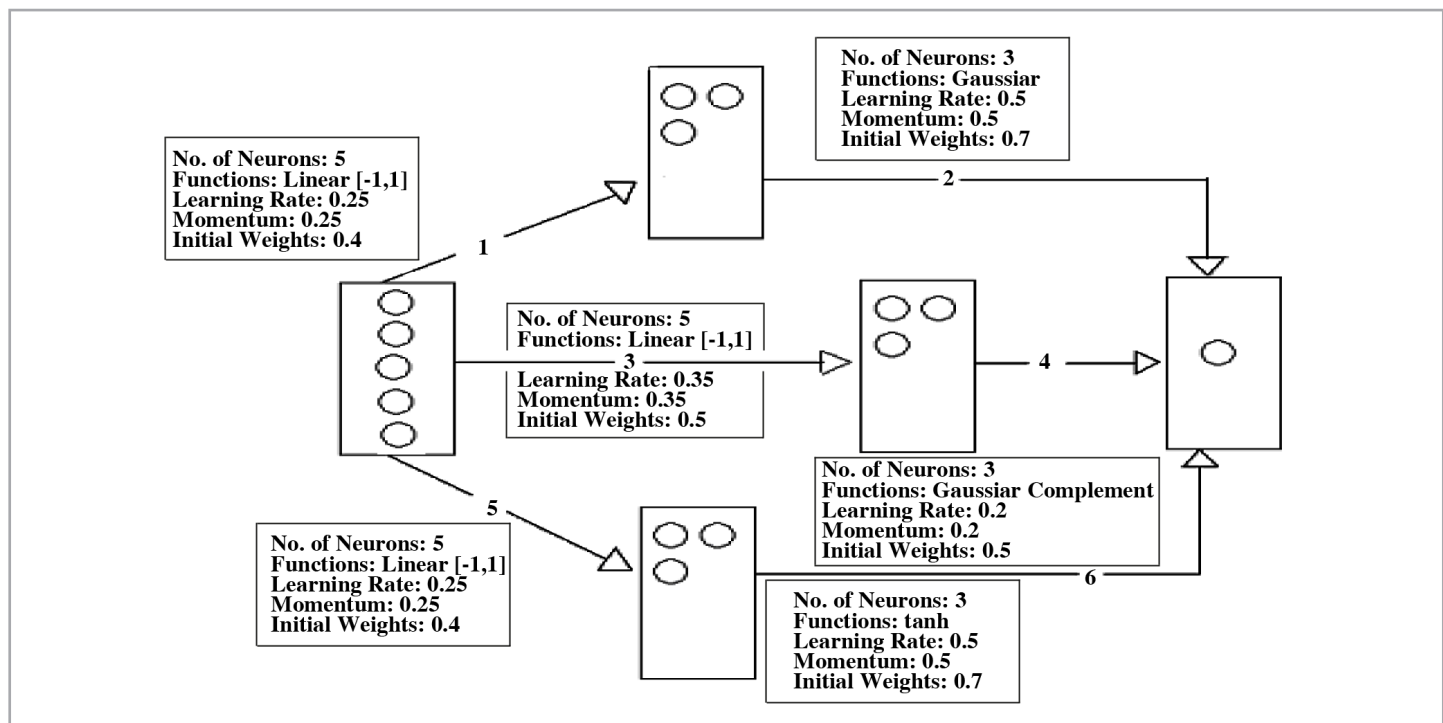


Figure 2: Neural network architecture design for sub-catchments C1, C2 and C3 of Bukit Tarek Watershed

All catchment possesses specific design for the neural network architecture. Training of the model stops once the best prediction is achieved. This is based on the training graphics and trial output processor. More often in training a network, its performance will continue to improve (measured relative to the training data) albeit at an ever decreasing pace. However, when performance is measured relative to a set of test patterns (not used for training) the performance will usually stop improving after a while, and often will start to degrade. Since the test patterns provide a more accurate assessment of the generalised performance of the network, it is best to cease training when performance is optimum relative to the testing set. The training time used for all sub-catchments is 5 minutes and the calibration interval is set at 300.

5.0 RESULTS AND ANALYSIS

Three hidden slabs with different activation function were chosen as the neural network architectural design for all sub-catchments in Bukit Tarek Watershed. The proposed architecture consists of an input layer with three hidden slabs and one output layer. The designed architecture for all sub-catchments consists of 5 neurons in the input layer, 3 neurons in each of the three hidden slabs and 1 output neuron (predicted value) in the output layer. The neural network architecture Bukit Tarek Watershed catchments can be summarised as 5:3:3:3:1. The momentum, learning rate parameters and the functions used for both architectures differ from one layer to the other. Total grid for 16 years analysis for sub-catchment C1 and C2 are 992 and 1728, respectively. The total grid for sub-catchment C3 is 1428 for 12 years grid analysis.

The performance of the model in training and testing phases are as shown in Figures 3, 4 and 5 for sub-catchments C1, C2 and C3; respectively. The soil loss ANN prediction is given in $t/ha/yr$. In the production phase only 10% of the total grid data was used to predict soil loss without the presence of the measured values. This is to test the reliability and the robustness of the model to estimate soil loss.

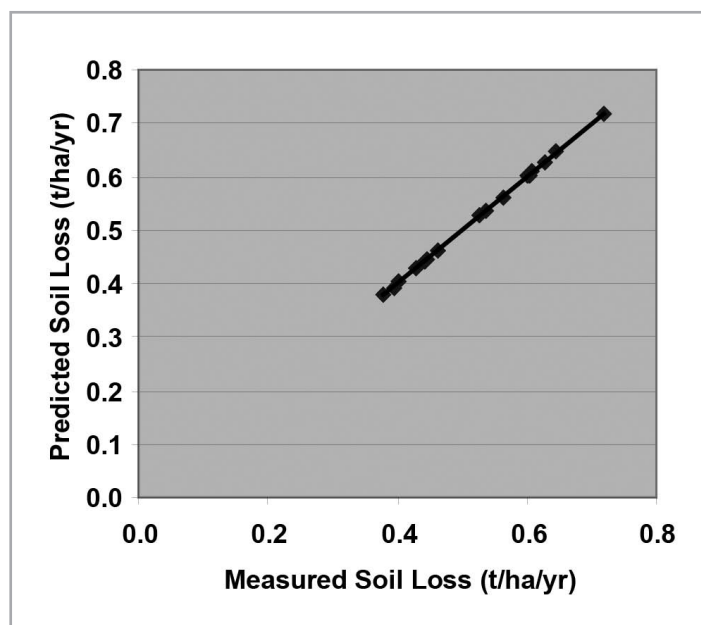


Figure 3: Predicted versus measured values for sub-catchment C1

All graphs show very good fit between the ANN predicted and measured soil loss using USLE. Results of the analysis had indicated perfect prediction of soil loss using the proposed model and the significance of the USLE parameters as soil loss predictors.

The values of soil loss and statistical interpretation outputs for sub-catchments C1, C2 and C3 are given in Tables 5, 6 and 7. From analysis and results confirmation, soil loss prediction using Artificial Neural Network Model showed very good prediction with R , K , LS , C and P as predictors. The proposed neural network architecture for Bukit Tarek Watershed catchment is 5:3:3:3:1.

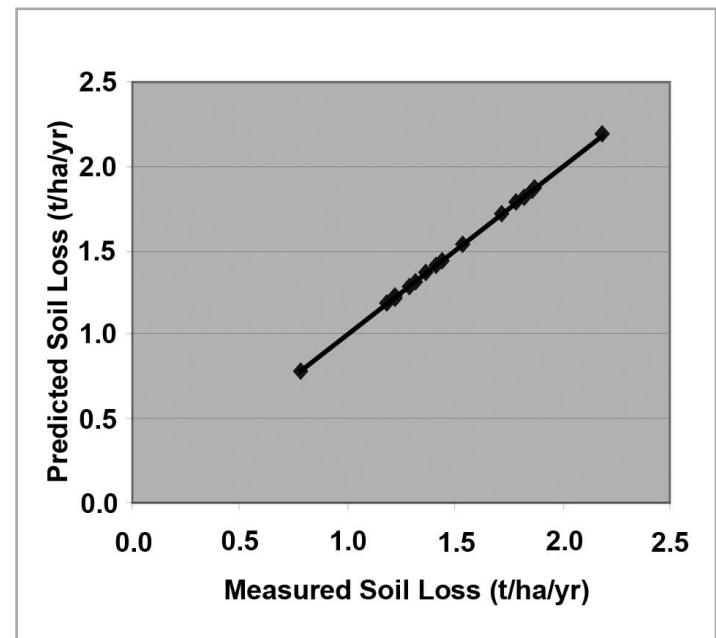


Figure 4: Predicted versus measured values for sub-catchment C2

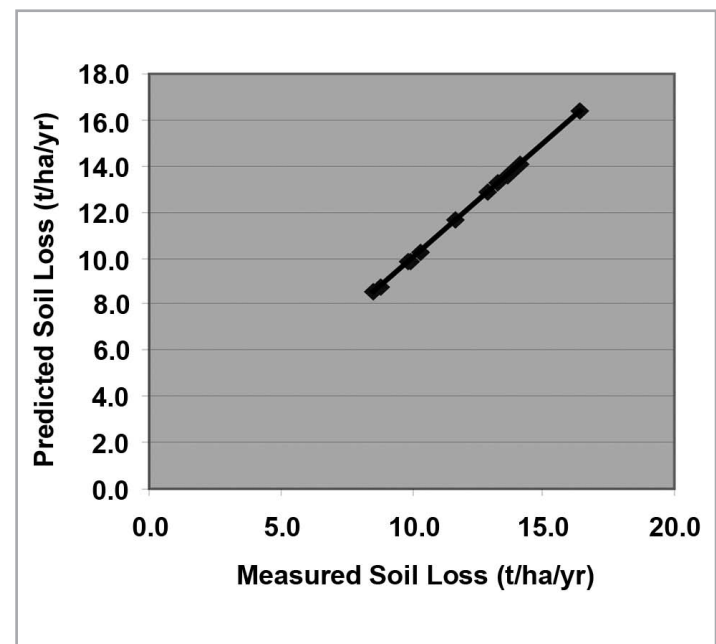


Figure 5: Predicted versus measured soil loss in sub-catchment C3

Table 5: Summary of statistical analysis for sub-catchment C1

	USLE (t/ha/yr)	Proposed model (t/ha/yr)
1989	0.4273	0.4273
1990	0.4030	0.4028
1991	0.6462	0.6460
1992	0.4626	0.4623
1993	0.6092	0.6092
1994	0.4454	0.4456
1995	0.6021	0.6015
1996	0.5625	0.5624
1997	0.3938	0.3936
1998	0.3781	0.3779
1999	0.6027	0.6020
2000	0.6272	0.6268
2001	0.4436	0.4433
2002	0.5358	0.5359
2003	0.7194	0.7190
2004	0.5279	0.5279
Mean	0.5242	0.5240
Min	0.3781	0.3779
Max	0.7194	0.7190
Median	0.5318	0.5319
Std. Deviation	0.1044	0.1043
Variance	0.0109	0.0109

Table 6: Summary of statistical analysis for sub-catchment C2

	USLE (t/ha/yr)	Proposed model (t/ha/yr)
1989	1.3178	1.3172
1990	1.2234	1.2229
1991	1.8725	1.8716
1992	1.4387	1.4380
1993	1.8204	1.8193
1994	1.2887	1.2883
1995	1.7827	1.7816
1996	1.7173	1.7166
1997	1.1872	1.1866
1998	0.7824	0.7819
1999	1.4147	1.4136
2000	1.8604	1.8592
2001	1.3667	1.3661
2002	1.5377	1.5370
2003	2.1866	2.1859
2004	1.2188	1.2182
Mean	1.5010	1.5002
Min	0.7824	0.7819
Max	2.1866	2.1859
Median	1.4267	1.4258
Std. Deviation	0.3508	0.3507
Variance	0.1231	0.1230

Figures 6, 7 and 8 show the graphs of ANN prediction versus USLE estimates in the production phases for sub-catchments C1, C2 and C3; respectively. The graphs show very good fit between the values predicted using ANN model and USLE equation. The model yields 100% accuracy in the production phase for sub-catchments C1 and C2 using a total of 160 grids. For sub-catchments C3, the model showed 95% accuracy with a total of 480 grids in the production phase. The statistical interpretation for sub-catchments C1, C2 and C3 are given in Table 8.

Table 7: Summary of statistical analysis for sub-catchment C3

	USLE (t/ha/yr)	Proposed model (t/ha/yr)
1993	13.2882	13.2485
1994	10.2964	10.2968
1995	13.9269	13.9192
1996	12.9138	12.9187
1997	8.5183	8.5398
1998	8.8154	8.7885
1999	14.1228	14.1123
2000	13.6353	13.6017
2001	9.8138	9.8250
2002	11.6787	11.6443
2003	16.4071	16.3854
2004	9.9191	9.8500
Mean	11.9447	11.9275
Min	8.5183	8.5398
Max	16.4071	16.3854
Median	12.2963	12.2815
Std. Deviation	2.4708	2.4670
Variance	6.1048	6.0863

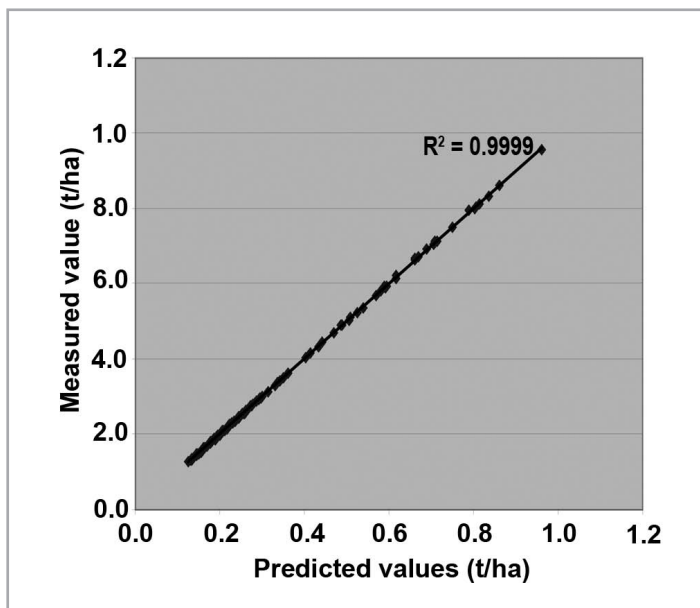


Figure 6: Graph of ANN predicted versus measured values in the production phase for sub-catchment C1

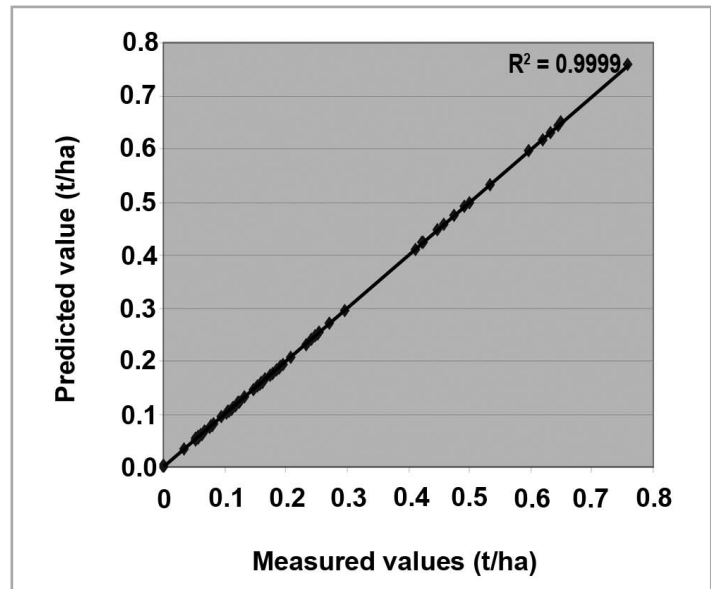


Figure 7: Graph of ANN predicted versus measured values in the production phase for sub-catchment C2

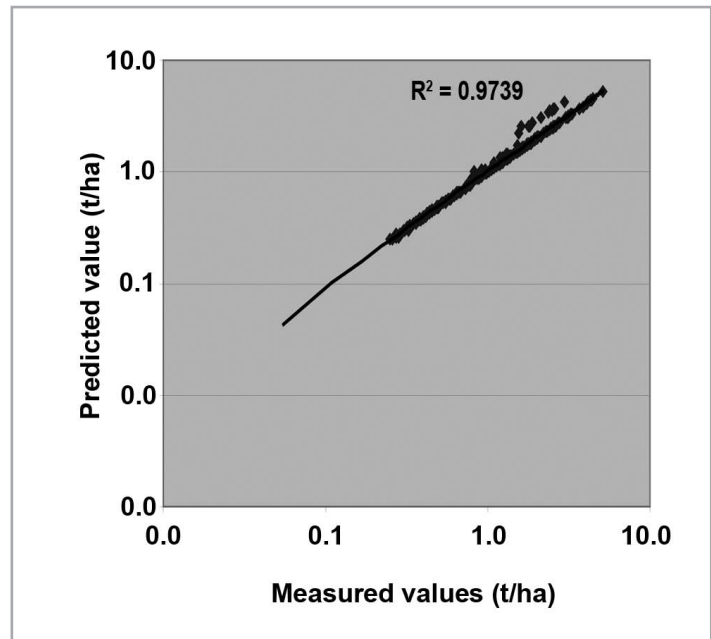


Figure 8: Graph of ANN predicted versus measured values in the production phase for sub-catchment C3

6.0 CONCLUSION

A soil loss multi layer perceptron model has been successfully developed and proposed for the experimental watershed of Bukit Tarek, Malaysia using back-propagation algorithm. The neuron architecture for each slab of the proposed model for sub-catchments 1, 2 and 3 in Bukit Tarek Watershed are 5:3:3:3:1. The derived model is applicable only for use in catchment where forest logging activity is evident.

An improved and a more reliable soil loss model of short processing time would be an advantage for the preservation of the environment. This development would be useful in strategising the appropriate conservation measure and should benefit the relevant agency in institutionalising the guidelines for soil conservation practice.

Table 8: Summary of analysis for sub-catchments C1, C2 and C3 in the production phase

	Sub-catchment C1		Sub-catchment C2		Sub-catchment C3	
	USLE (t/ha)	Proposed Model (t/ha)	USLE (t/ha)	Proposed Model (t/ha)	USLE (t/ha)	Proposed Model (t/ha)
Mean	0.2977	0.2984	0.1457	0.1457	1.0751	1.0968
Min	0.1263	0.1258	0.0000	0.0015	0.0000	0.0012
Max	0.9602	0.9578	0.7586	0.7586	5.1723	5.2017
Median	0.2255	0.2265	0.0952	0.0948	1.0640	1.0663
Std. Deviation	0.1879	0.1879	0.1536	0.1530	0.8237	0.8618
Variance	0.0353	0.0353	0.0236	0.0234	0.6784	0.7426

ACKNOWLEDGMENT

This study was funded by the Forest Research Institute of Malaysia (FRIM) under the Research Assistantship fund. The authors extend their profound gratitude to the officers and supporting staff of the Forest Hydrology Unit at FRIM who have offered valuable information and required data. The authors are deeply indebted to Prof. Pierre Y. Julien from Colorado State University for his valuable suggestions to further improve this paper. ■

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AN OVERVIEW ON CONVERGENCE ACCELERATION OF CYCLIC ADSORPTION PROCESSES

(Date received: 19.7.2007)

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ABSTRACT

Cyclic adsorption processes are inherently dynamic where the process variables are always varying with time. The cyclic processes have no steady state. Thousands of repeated cycles may be needed before cyclic steady state (CSS) is reached. In this paper, the basic concept and characteristics of cyclic adsorption processes are first introduced, using air separation by rapid pressure swing adsorption as an example. Next, different approaches to calculate and accelerate the convergence of CSS are briefly reviewed. The computational time can be reduced by having an efficient discretisation technique and accelerators to achieve the final CSS. Hybrid methods are potentially attractive.

Keywords: Adsorption, Convergence Acceleration, Cyclic Process

1.0 INTRODUCTION

The phenomenon where the gas molecules form bonds with the surface of a solid and become attached is termed as adsorption. The reverse process where the adsorbed molecules are removed from the solid surface is called desorption or regeneration. A complete operation of adsorption and desorption is called cyclic adsorption process. In a physical cyclic adsorption process, there are a few methods to regenerate the saturated bed, e.g.: Pressure Swing Adsorption (PSA) where the bed regeneration is accomplished by reducing the total pressure of the adsorber; Vacuum Swing Adsorption (VSA) where partial pressure is reduced; Temperature Swing Adsorption (TSA) where the temperature is increased. The application of these cyclic adsorption processes has been employed in many important separations such as air separation, carbon dioxide recovery from combustion process, trace volatile organic component removal, hydrogen recovery from refinery gases, air drying, separations of olefins and paraffins, and many more novel separations in the fine chemistry industry.

A unique feature of cyclic process is that it has no steady state. After some repeated cycles, cyclic steady state (CSS) is achieved. The computational load for the calculation of CSS is usually heavy. The objectives of this paper are:

- to illustrate the concept and basic characteristics of cyclic adsorption processes, and
- to provide an overview on approaches proposed in the literature for accelerating the calculation of the CSS. Possible improvements are also suggested.

1.1 Cyclic adsorption model

The transient cyclic adsorption process can be modeled by using partial differential equations (PDEs) for mass conservation in the fluid phase, ordinary differential equations (ODEs) for the sorption rate in the stationary phase, and algebraic equations for the adsorption equilibrium between phases.

Overall material and component balances for fluid phase:

$$\varepsilon_b \frac{\partial c}{\partial t} = - \frac{\partial(uc)}{\partial z} - \sum_i R_i \quad (\text{Equation 1})$$

$$\varepsilon_b \frac{\partial c_i}{\partial t} = - \frac{\partial(uc_i)}{\partial z} + \frac{\partial}{\partial z} \left(D \frac{\partial c_i}{\partial z} \right) - R_i \quad (\text{Equation 2})$$

where R_i is the uptake rate of component i by the particles per unit volume of the bed ($\text{mol m}^{-3} \text{s}^{-1}$), D is the dispersion coefficient, ε_b is the bed porosity. Depends on the adsorption kinetics of the system, the sorption rate in the stationary phase varies. The value of R_i therefore depends on mass transfer resistance of the model. The uptake rate is commonly approximated using linear driving force (LDF) model

$$R_i = \rho_b \frac{\partial \bar{q}}{\partial t} = k (q^* - \bar{q}) \quad (\text{Equation 3})$$

where ρ_b is the bed density, q^* is the adsorbed phase concentration at equilibrium, $\bar{q}$ is the average adsorbed phase concentration over an entire particle volume, and k is the LDF mass transfer coefficient.

At equilibrium, the adsorbed phase concentration is often expressed as a function of the concentration of fluid phase:

$$q^* = f(c) \quad (\text{Equation 4})$$

1.2 Definition of CSS

The nature of cyclic adsorption process has no steady state like general continuous process, it is inherently dynamic. Once the cyclic process is initiated, the process undergoes a transient stage prior to reach CSS. An adsorption cycle can be viewed as an orbit as shown in Figure 1. With initial vector that composed of process states variables in the model equations, $\bar{X}_0$, the cycle transforms the initial vector to a new vector, $\bar{X}_n$ at the end of the cycle. When the new vector completely specify the state of the bed at the beginning and end of the cycle, $\bar{X}_e$, CSS is reached and the process will repeatedly continue as a prefixed orbit (Croft and LeVan, 1994). At CSS, the process state variables at some instant within a cycle have the same value at the corresponding instant within each subsequent cycle (Choong, 2000).

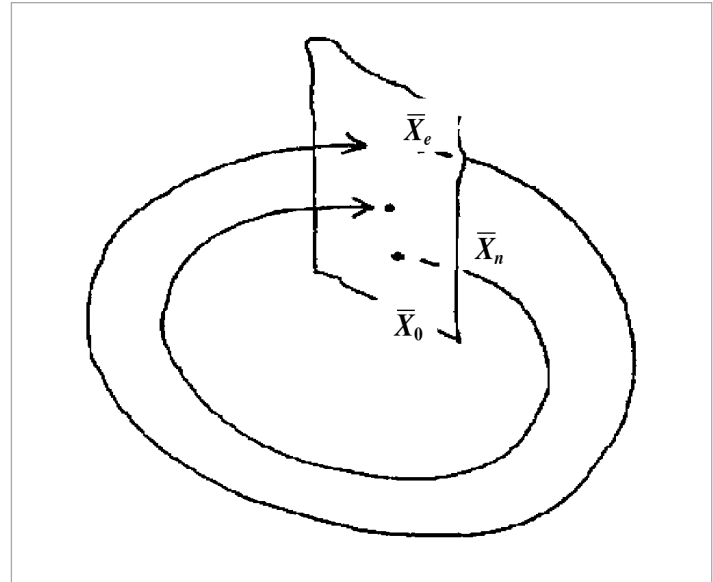


Figure 1: Depiction of an adsorption cycle

To illustrate the behavior of various process state variables at CSS, an isothermal Rapid Pressure Swing Adsorption (RPSA) model for air separation using zeolite 5A as adsorbent was used. Details of the process description can be found in Choong (2000). It is shown in Figures 2 and 3 that the process state variables, *e.g.* total pressure and oxygen mole fraction at some instant within a cycle have almost the same values at the corresponding instant within each subsequent cycle. CSS is reached and both the total pressure and oxygen mole fraction oscillate with time about a mean value as shown in Figures 4 and 5.

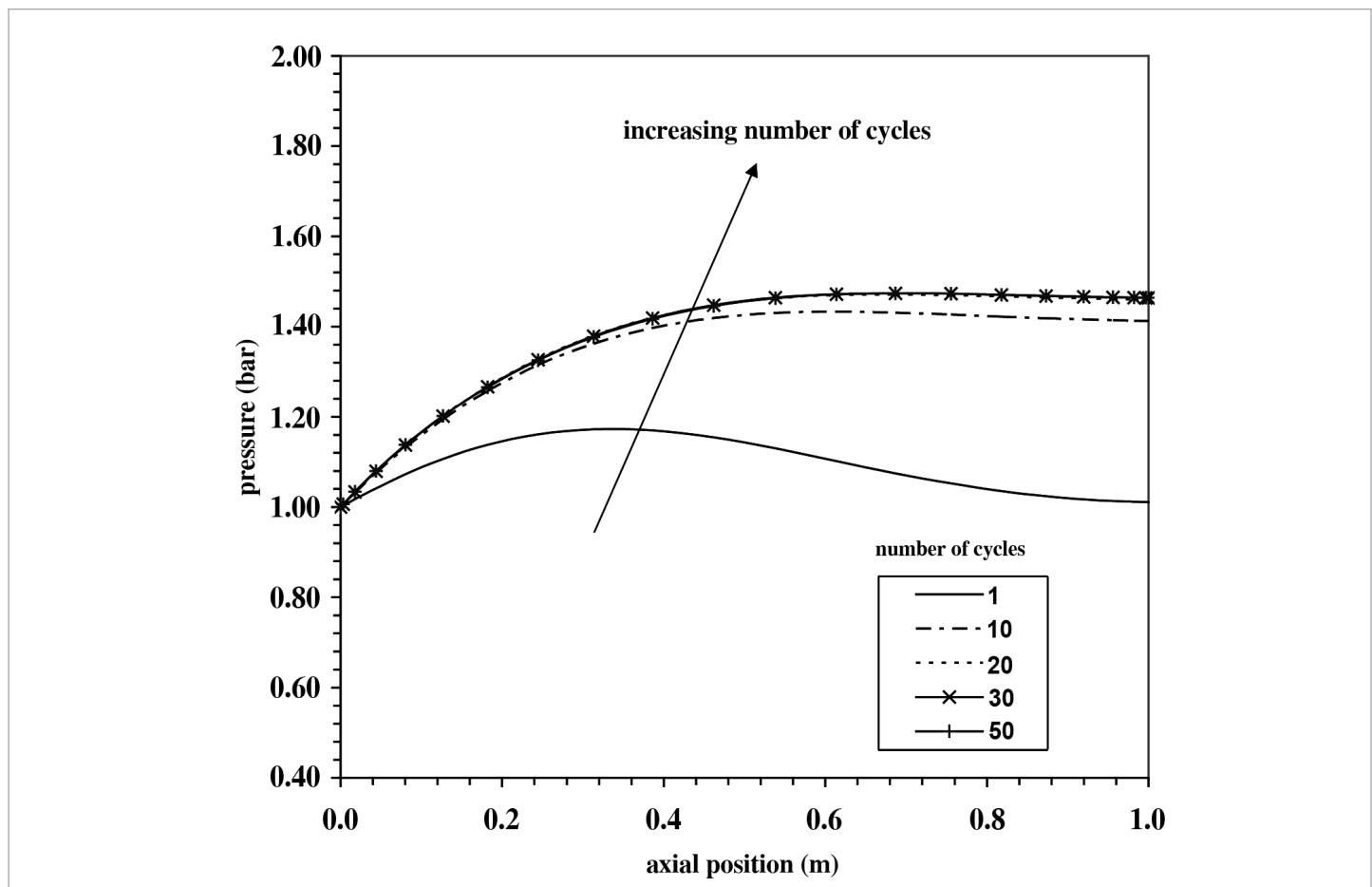


Figure 2: Total pressure at the end of the depressurisation step as a function of axial position after various numbers of cycles (Choong, 2000)

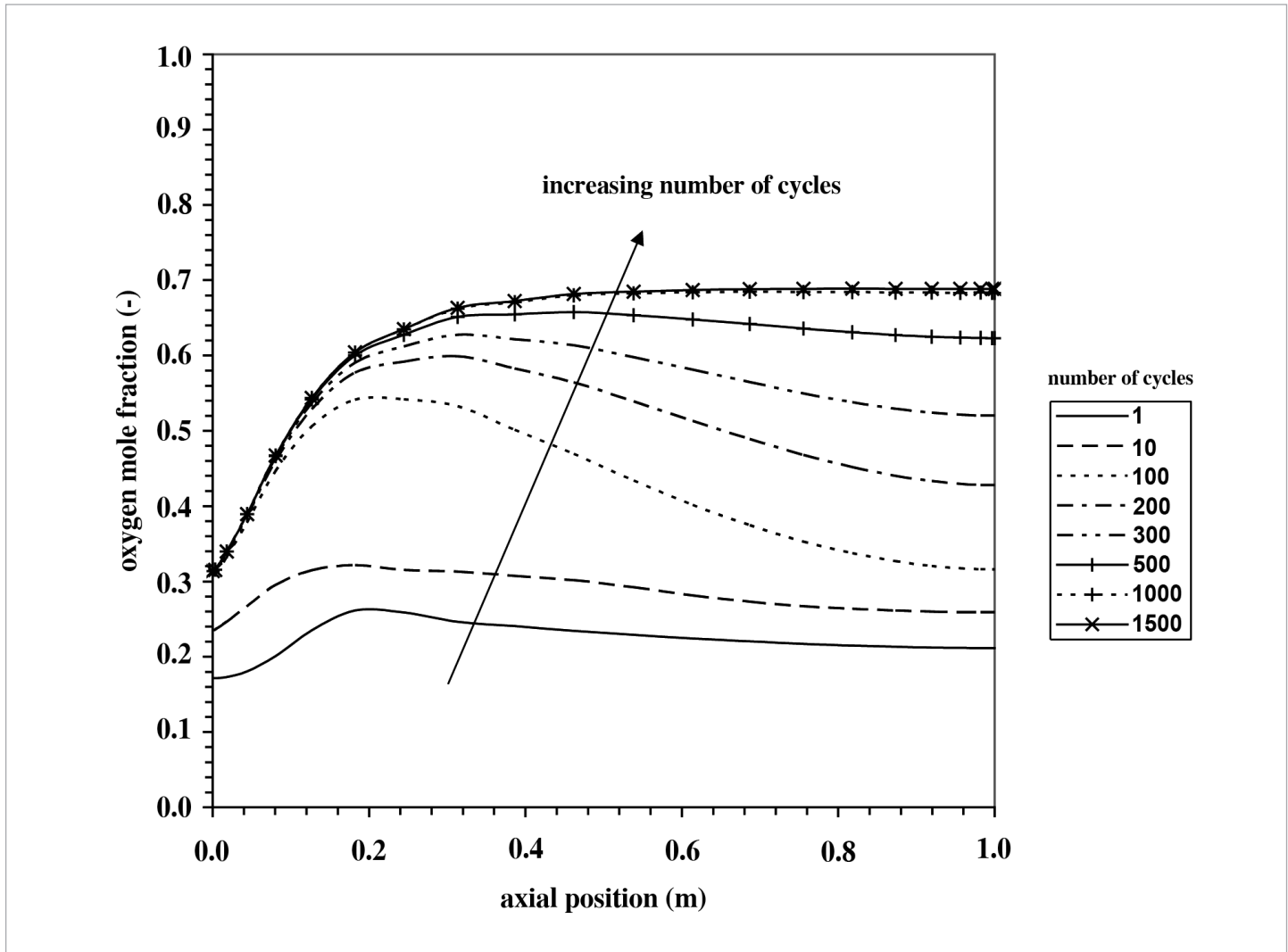


Figure 3: Oxygen mole fraction at the end of the depressurisation step as a function of axial position after various numbers of cycles (Choong, 2000)

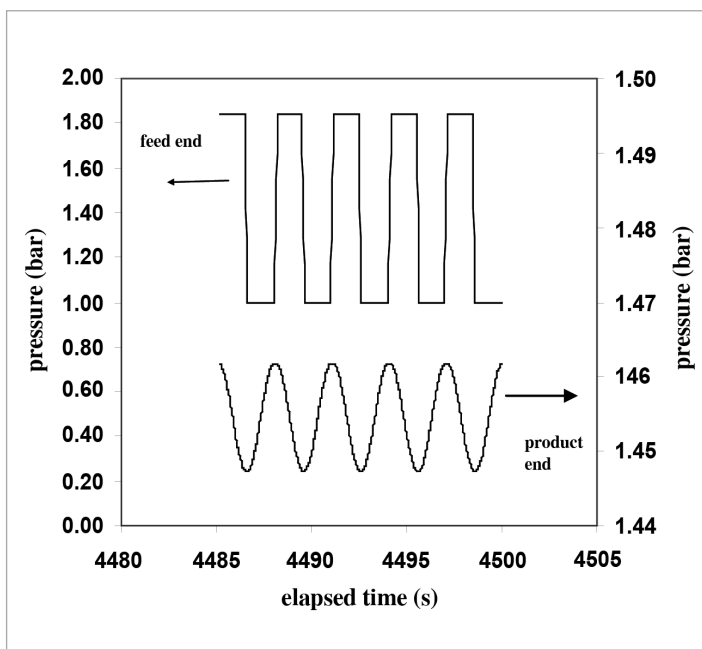


Figure 4: Total pressure at the feed end and product end of the bed as a function of time at CSS (Choong, 2000)

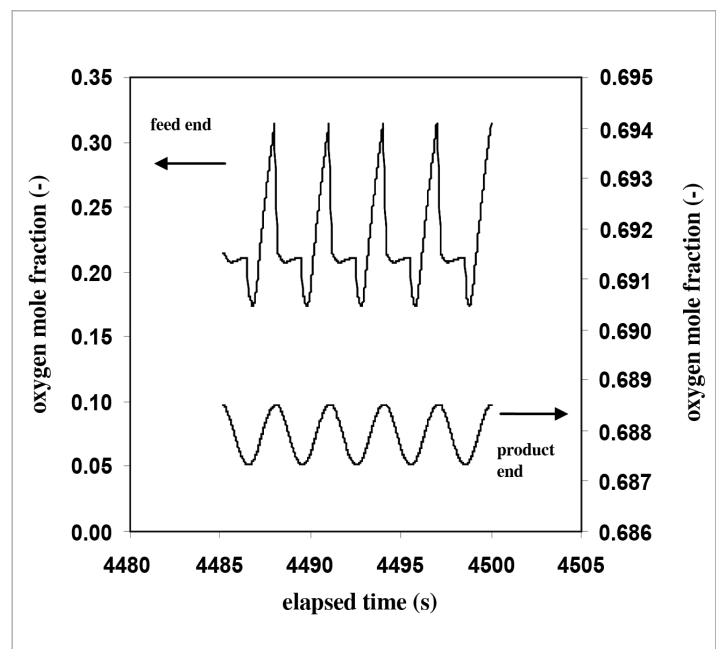


Figure 5: Oxygen mole fraction at the feed end and product end of the bed as a function of time at CSS (Choong, 2000)

2.0 APPROACHES AND CONVERGENCE ACCELERATION TO CALCULATE CSS

The most commonly approach to calculate CSS is the dynamic simulation where the simulation follows the dynamics of the real process. Mathematically, this is the Method of successive substitution (MSS). The calculation involves a series of complete cycle with the results of the previous cycles will be used as initial conditions for the next cycle. This is the preferred method if the process dynamics of the simulation are important.

Depending on the systems considered, hundreds or thousands of cycles may be required to achieve CSS using the method of MSS (Choong, 2000), and therefore this may be computationally demanding. Reduction of computational time may be achieved by (i) improving the numerical methods used to solve the model equations, *e.g.* discetisation techniques (ii) speeding up the convergence of CSS, *e.g.* accelerated MSS and direct determination. Figure 6 shows a summary on the approaches and convergence acceleration to calculate CSS.

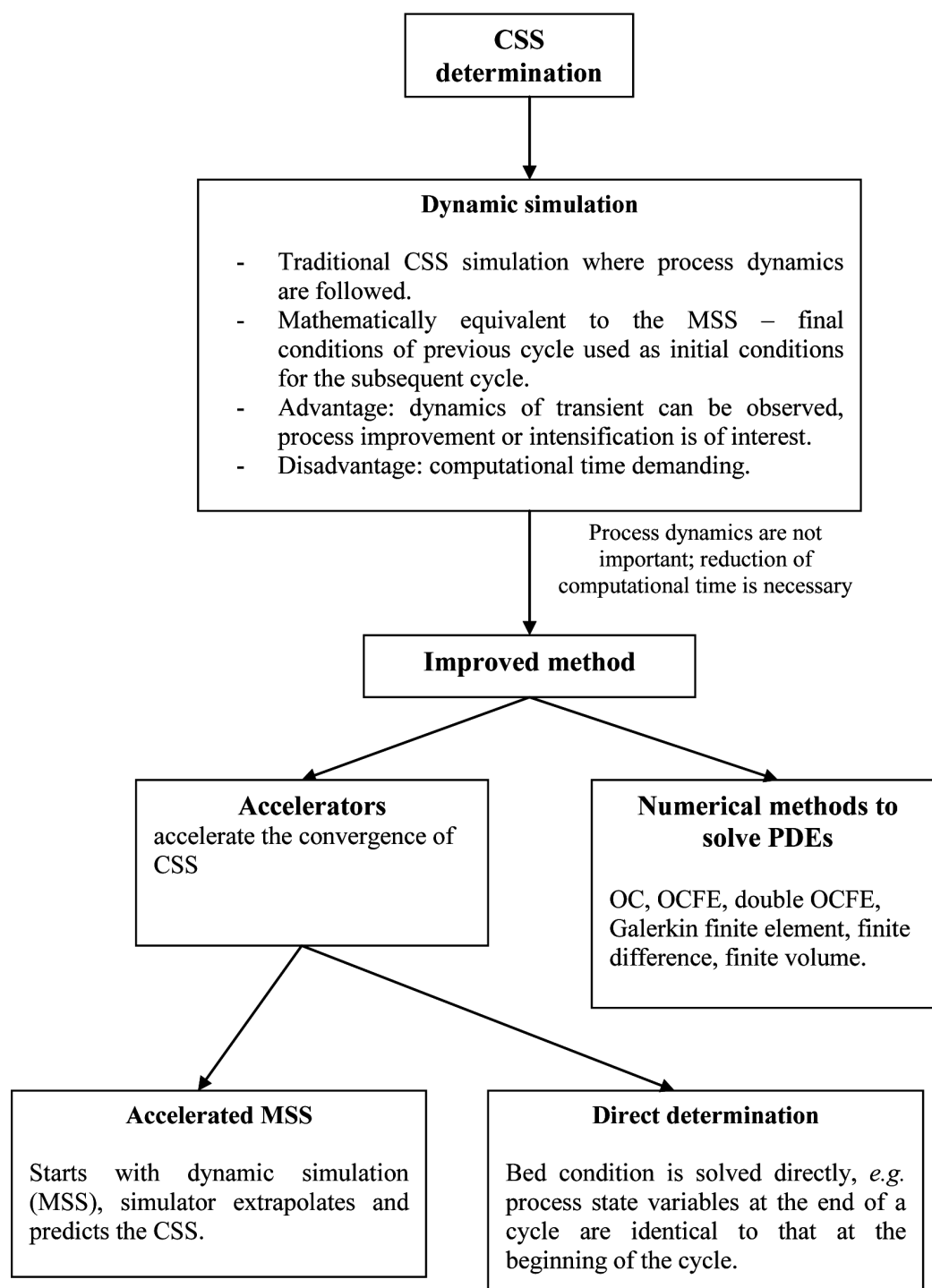


Figure 6: Summary of approaches and convergence acceleration to calculate CSS

2.1 Numerical methods to solve model equations (Discretisation techniques)

In general, there are two approaches to solve the model equations:

- (i) complete discretisation where the PDEs are discretised with respect to both time and space domain (Nilchan and Pandelides, 1998; Raghavan and Ruthven, 1985), resulting in a system of simultaneous algebraic equations;
- (ii) spatial discretisation or the method of lines where the PDEs are first spatially discretized into ODEs in the space domain. The system of ODEs is then solved by appropriate numerical integrator.

There are a few type of numerical methods used to discretize the PDEs: orthogonal collocation, OC (Hassan *et al.*, 1987; Raghavan *et al.*, 1985), orthogonal collocation on finite element, OCFE (Alpay, 1992; Da Silva *et al.*, 1999), Galerkin finite elements (Teague and Edgar, 1999), finite difference method (Ko and Moon, 2000), finite volume method (Cruz *et al.*, 2005; Webley and He, 2000). The OC method is the most commonly used spatial discretisation in the simulation of cyclic adsorption processes. It is demonstrated by Hassan *et al.* (1987) that the OC method was substantially more efficient than finite difference method for cyclic adsorption problems. In terms of computing time, conclusion was made by Alpay (1992) that OCFE was the most efficient method among OC, OCFE, double OCFE and cell-in-series for RPSA models. It is also showed by Webley and He (2000) that the finite volume method with appropriate extrapolation was flexible to accommodate boundary conditions for any type of cycle relevant operation.

2.2 Accelerators

In context of this paper, accelerators refer to the simulator that can accelerate or speed up the convergence of CSS. Accelerator is often used when the information on the transient is not important or process optimisation is of interest. Two types of accelerators are discussed in this section:

- (i) accelerated MSS and
- (ii) direct determination. Accelerated MSS starts with MSS for the first few cycles. The simulator then extrapolates and predicts the final CSS. As for direct determination, the state variables of two cycles are set to be the same between two subsequent cycles and solved by incorporating the boundary conditions at both ends of the adsorption column.

2.2.1 Accelerated MSS

Kvamsdal and Hertzberg (1997) investigated two accelerated MSS methods, namely Aitken and Muller methods. It was found that both methods reduced the number of cycles calculated to converge to CSS. However, in order to obtain the same CSS profile by the traditional MSS, they had to tune the error tolerance used in the stopping criterion of their accelerators.

Choong *et al.* (2002) recommended that the MSS can be supplemented with extrapolators that accelerate convergence to CSS. The effectiveness of this method was demonstrated using both the simulation of RPSA. Under suitable conditions, the paired extrapolators approach CSS from opposite sides, thus effectively bracket the CSS. The technique of paired extrapolators reduces the number of cycles to reach CSS by 300%. The reduction of computing time achieved by the extrapolators may not be as large

as reported by some of the accelerators in the literature. However, the improvement reported is often measured with respect to MSS without a reliable criterion for identifying CSS. Furthermore, the accelerators in the literature may not converge to the exact CSS (*e.g.* Nilchan, 1997) as the extrapolators developed by Choong *et al.* (2002) do.

Lübke *et al.* (2006) proposed a new algorithm, namely cascading algorithm to shorten the computational time of determining the CSS of simulated moving bed processes by approximating the CSS on a coarse space-time mesh first and then using the rough approximation as the initial condition on the finer mesh. Complete discretisation was adopted in this algorithm. The algorithm starts with calculating the CSS with initial condition for the first level. In case that the space grid of the next level does not coincide with the previous level, linear interpolation from the coarse to the refined grid is used. A considerable reduction in computational time was achieved.

2.2.2 Direct Determination

The direct determination method solves directly for the bed condition that at the end of a cycle that is identical to that at the beginning of the cycle, giving the desired periodicity requirement (Ding *et al.*, 2002). Croft and LeVan (1994) used Newton method to speed up the convergence rate of a PSA process. The method gave considerable reduction in number of iteration but the computational time increased correspondingly. Ding and LeVan (2001) further enhanced the direct determination of Croft and LeVan (1994) by introducing a hybrid method of Newton and Broyden to reduce the calculation of Jacobian matrix in conjunction with a novel iterative-secant approach to increase stability and to avoid the calculation of the first Jacobian matrix for the hybrid Newton–Broyden method; implemented a sensitivity interpolation technique with dynamic grid allocation; and specified a dynamic integration error tolerance. Using an isothermal trace separation PSA process as a case study, the enhanced formulation resulted in 40 times faster than the original formulation proposed in by Croft and LeVan (1994) and 100 – 1000 times faster than the successive substitution method. However, the comparative computational cost of the increasing number of nodes in the spatial discretisation as compared to the MSS was not very clear.

Kvamsdal and Hertzberg (1997) also investigated the efficiency of direct determination on calculating the convergence of CSS by using Broyden and damped Newton-based approach. Comparisons were made between the two methods, the accelerated successive method (Aitken and Muller) as well the traditional MSS. It was found both the direct determination methods took longer calculation time than the traditional MSS although the number of cycles calculated to converge to CSS has been reduced tremendously. Between the two direct determination methods, Broyden's method needs shorter calculation time as this method does not require Jacobian matrix to be calculated at every iteration as the damped Newton-based approach.

Van Nordeen *et al.* (2002) introduced a hybrid convergence method called the Newton-Picard method for cyclically operated reactors and separators. This method combines the strong points of dynamic simulation and the Newton's method. The hybrid method is based on the decomposition of the state space and is also to compute the eigenvalues that determine the stability of a periodic state. They applied the proposed method into three

non-linear systems (CO_2/N_2 PSA, H_2S /natural gas PSA and reverse flow reactor) and two linear systems (H_2O /air PSA and CO_2/He PSA). For the non-linear systems, they compared the Newton-Picard method to three existing methods: MSS, Newton's method and Broyden's method. The results were that only for the CO_2/N_2 PSA separation, the Newton-Picard converged relatively slower than MSS. The method was in most cases much more efficient than MSS or Newton's method. For the linear systems, they investigated which method was the most appropriate to compute CSS. They demonstrated that when the largest eigenvalue of the Jacobian matrix of a linear system and the dimension of the system is known, it is possible to decide beforehand whether Newton's method or MSS will be more efficient to calculate the CSS. For both linear and non-linear systems, Broyden's method was found to be most efficient in terms of number of cycles required

although it suffered from robustness problems in non-linear cases. Nevertheless, in situation where the stability of periodic states was needed, the Newton-Picard method was superior to Broyden's method because the Newton-Picard method computes approximations to the eigenvalues that determine the stability of the periodic states.

4.0 CONCLUSION

The convergence acceleration of CSS can be achieved either by using efficient PDE solvers or by devising efficient accelerators. A brief but comprehensive survey on the convergence acceleration of CSS has been presented. It appears that a hybrid method is particularly attractive. Our research group is now working on devising efficient hybrid method for the acceleration of the convergence of CSS. ■

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LOAD RESPONSE TOWARDS VOLTAGE IN TNB POWER SYSTEMS USING THE MEASUREMENT APPROACH

(Date received: 26.9.07)

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ABSTRACT

This paper reports the load-voltage dependency using field measurements for a faulted condition that caused a momentary voltage sag in a Tenaga Nasional Berhad (National Power Utility) network in the Central Region. The static load model is used in this study and its suitability is discussed in comparison to the dynamic load model. A fast method of calculating real and reactive power based on available information is devised. The effect of voltage on real and reactive power is discussed at great length. Various factors influence the load characteristic, including weather, time of day, network conditions and type of feeder load. Long-term steady state data should be made available for analysis to back up results from this study. We can conclude that the location of the feeder from point of fault also plays an important role. Certain feeders exhibited more serious conditions compared to others during the fault.

Keywords: Load Dependency, Load Modeling, Load Response, Load Sensitivity, Measurement-Based Approach, Non-Linear Least Square Regression, Voltage Dip, Voltage Sag

1.0 INTRODUCTION

The global electric power demand is rapidly increasing. The increasing disparity between demand of energy and supply leads to a number of concerns in relation to the present and future availability of energy sources in the world, the environmental costs that will be associated to this growth, and how third world countries will handle the increasing energy needs of their growing populations [1]. The power industry depends more and more on the industry growth rate and the use of the existing capacity in the most effective way. Therefore current challenges in power engineering include optimising the use of the available resources and keeping high reliability for operating conditions that will include narrow stability and security margins [1].

Changes in the power generation and transmission systems, optimising the available resources while making environmental consideration, and ensuring high reliability in the system operation, are necessary in order to match the increasing demand in the load areas. The system planning must ensure controllable generation for regulating both frequency (by controlling the output of the active power) and voltage (by controlling the output of reactive power), and must control the costs and ability to operate as spinning reserves when needed. An optimisation and coordination of the available resources, as well as the construction of new generation plants will thus be necessary.

The fact that loads are generally voltage dependent is a critical aspect for the planning and operation of the power system. The load characteristic may result in a very optimistic or pessimistic design if it is not chosen appropriately, leading

the system to voltage collapse or on the other hand to very oversized security margins.

In the early stage of electric power system development, power quality issues were not addressed as a critical issue [11]. Today, it has become important due to more sensitive loads being connected to the electrical network. It becomes a major concern in Malaysia in view of the nation's transformation towards industrialisation in year 2020 [8].

The current scenario in Malaysia is that we assume a constant power sensitivity factor. This would mean that load does not change during a voltage deviation. This is a pessimistic approach to understanding the relationship between load and voltage. However, it is the safest method as the worst-case scenario is assumed. In order to optimize load, a better dependency factor needs to be tabulated. A recent study has been done in Malaysia using the polynomial static load model via the component-based approach [12]. CIBS Billing data was used to calculate the RCI (Residential, Commercial and Industrial) Index. Then, standard IEEE sensitivity factors were used to come up with the results. In order to take the study one step further, this research is aimed at calculating specific sensitivity factors instead of using default values in the process.

Section II of the paper explains the load modeling, and different approach to monitor the load substations to determine the sensitivity. Section III then presents the data used in this modeling. Section IV explains the non-linear regression is used when a best fit needs to be computed. Section V present the results obtained and Last, Section VI summarizes the conclusion presented in the paper.

2.0 LOAD MODELING

The interest in load modeling has been continuously increasing in the last years, and power system load has become a new research area in power systems stability. We now realize that load-voltage characteristics have a significant effect on system performance, and transient stability results are known to be highly dependent upon the load characteristics assumed [2]. Several studies have shown the critical effect of load

Two main approaches to load model development have been considered by the electric utility industry [3]. They are as follows:-

a. Measurement-Based Approach

The measurement-based approach involves placing monitors at various load substations to determine the sensitivity of load active and reactive power to voltage variations to be used directly, or to identify parameters for more detailed load models [2] [4]. This approach has the advantage of direct monitoring of the true load and can produce load model parameters directly in the form needed for power flow and transient stability program input. Its disadvantages include the cost of acquiring and installing the measurement equipment and the need to monitor all system loads or to extrapolate from limited measurements.

b. Component-Based Approach

The component-based approach involves building up the load model from information on its constituent parts. Three sets of data are required; load class data, load composition data and load characteristic data. Load class data describes a category of load such as residential, commercial or industrial. For load modeling purposes, loads are classed accordingly. Each class has a similar load composition and characteristics. Load composition data describes the fractional composition of the load by load components. This term can be applied to bus load or a specific load class. Load characteristic data is a set of parameters such as power factor, variation of active and reactive power with voltage, etc. It characterizes the behavior of a specific load. The component-based approach has the advantage of not requiring system measurements and therefore being more readily put into use. Since load characteristics and load composition data should not vary widely over a particular system, they can be developed once for the entire system. Only the load class mix data needs to be prepared for each bus or area and updated for changes in the system load [2].

A combination of both the measurement-based approach and the component-based approach is best to come to a conclusion on the inherent load characteristic.

The static load model has been used widely for the past many years, even to approximate dynamic components. It is not dependent on time and therefore it describes the relationship of the active and reactive power at any time with the voltage and/or frequency at the same instant of time.

The dynamic model has lately been employed by a number of utilities, especially regions with 4 seasons that use high amounts of heating loads at certain junctures of the year [1] [4][5]. It expresses this relationship at any instant of time, as a function of the voltage and/or frequency time history, including normally the present moment.

The 3 main load models explored in this research are as follows. These are the main load models that have been developed

and used in the recent past by various researchers throughout the world.

1. Polynomial Static Load Model

This is a branch of the static model that represents the power relationship to voltage magnitude as a polynomial equation, usually in the following form [3]:-

$$P = P_0 [a_1 (V/V_0)_2 + a_2 (V/V_0) + a_3] \quad [1]$$

$$Q = Q_0 [a_4 (V/V_0)_2 + a_5 (V/V_0) + a_6] \quad [2]$$

The parameters of this model are the coefficients (a_1 to a_6). This model is referred to as the ZIP model, since it consists of the sum of constant impedance (Z), constant current (I), and constant power (P) terms. The model has been implemented in [6].

2. Exponential Static Load Model

This is another branch of the static load model that represents the power relationship to voltage as an exponential non-linear equation, usually in the following form [3]:-

$$P = P_0 (V/V_0)^{np} \quad [3]$$

$$Q = Q_0 (V/V_0)^{nq} \quad [4]$$

The parameters of this model are the exponents, np and nq . By setting these exponents to 0, 1 or 2, the load can be represented by using constant power, constant current, or constant impedance models respectively. Other exponents can be used to represent the aggregate effect of different types of load components as expressed in [3][7].

3. Exponential Dynamic Load Model

Due to the large amount of electrical heating loads in 4-season countries, and its critical effect on voltage stability a load model with exponential recovery has been proposed [5]. The model is presented, as a set of non-linear equations, where real and reactive power has a non-linear dependency on voltage.

$$P_1 = P_r + P_0 \left[\frac{V}{V_0} \right]^{a_1} \quad [5]$$

$$T_p \frac{dP_r}{dt} + P_r = P_0 \left[\frac{V}{V_0} \right]^{a_2} - P_0 \left[\frac{V}{V_0} \right]^{a_1} \quad [6]$$

Here, V_0 and P_0 are the voltage and power consumption before a voltage change. P_r is the active power recovery, P_1 is the total active power response, T_p is the active load recovery time constant, a_1 is the transient active load-voltage dependence, and a_2 is the steady state active load-voltage dependence. Similar equations are also valid for reactive power.

For this research, the exponential static load model is used. This would be a natural progression since the constant power load model is currently implemented in TNB power systems. Based on previous works, the results obtained using the static model and dynamic model does not pose much differences if the amount of heating loads are minimal and the voltage deviation of the system is not too drastic [1].

3.0 DATA

The data obtained for this study is the voltage and current readings due to a fault that occurred on Saturday, 21st January 2006 at approximately 10.28am. The double phase to ground fault occurred at a 275kV overhead line (L2), which connects KL South (KULS) and Serdang (SRDG), both of which are major substations in the Central KL region. Weather during the incident was fine.

ION 7600 Power Quality Monitoring System (PQMS)

installed at various 33kv feeders in the Central Region recorded the RMS and sinusoidal voltage and current deviations during the fault event. This is the first time PQMS data is being used to evaluate load-voltage characteristics. Previously, data was only used to analyse voltage sag index for important customers in the region.

The fault incident affected 13 feeders. They are BJLL4L5, BTGA2L5, HCOM3L5, KLJT8L5, MERU4L5, NUNI13L5, PIDH2L5, PJST6L5, PMJU7L5, PROT3L5, SHAE8L5, SRDG1L5 and TMSY2L5.

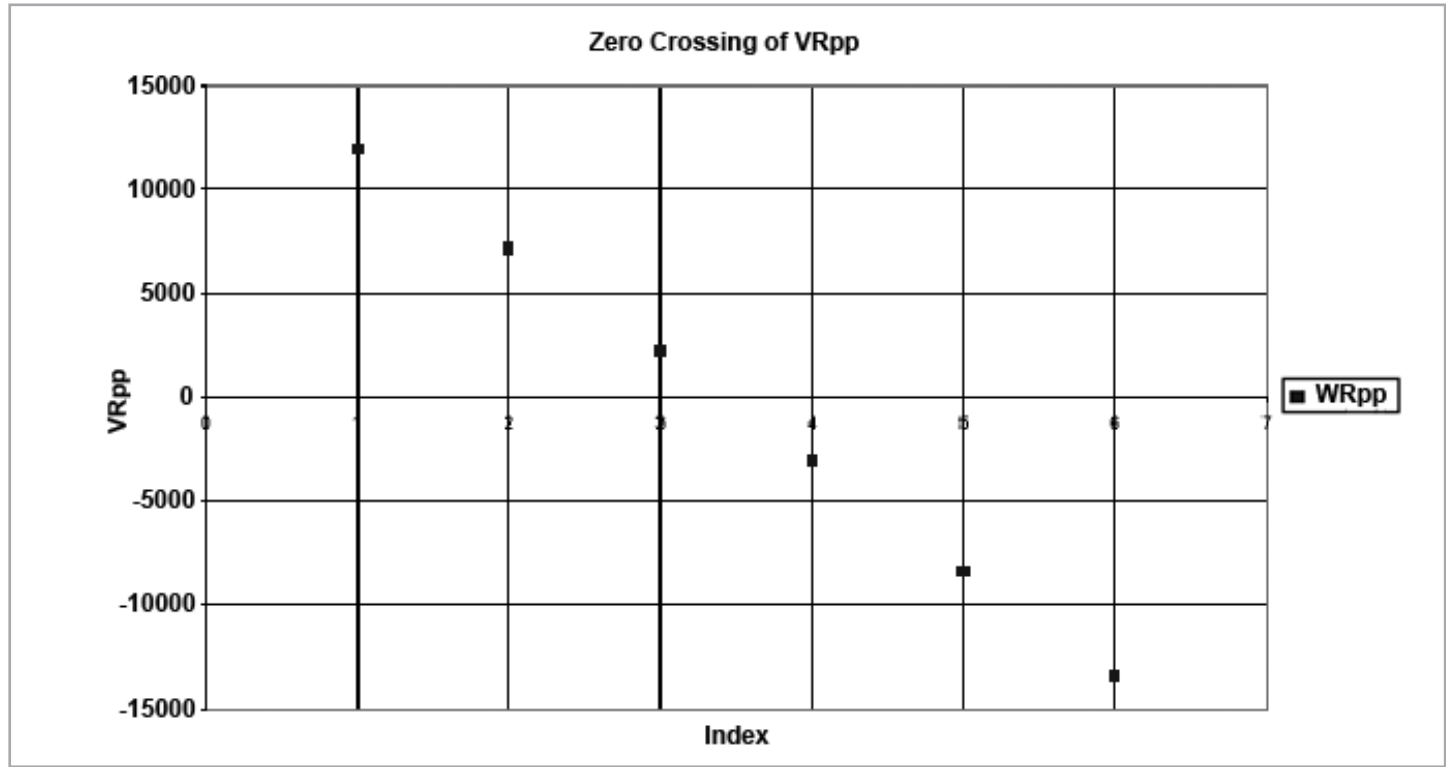


Figure 1: Linear line during zero crossing

3.1 Determination of Phase Angle

Since data is sampled at standard 32 samples per cycle, a mathematical calculation of phase angle between voltage and current is devised based on the zero-crossing technique. Microsoft Excel is used.

The 2 successive points where the data changes from positive to negative for both phase voltage and current is of interest in the tabulation. The change indicates that the zero crossing occurs in between these 2 values. The actual value needs to be calculated. It is found that points close to zero crossing of the waveform form a linear line. Figure 1 illustrates the linear line observed when 6 data points of red phase sinusoidal voltage (VRpp) is plotted during zero crossing.

With this, the value of the point that cuts through zero can be accurately calculated via mathematical formulation using the fundamentals of linear line equations. The slope of the line is a constant value, thus we can use the following equation:-

$$(V - 0) / (V - V_1) = Vz \quad [7]$$

where,

V = last positive point of VRpp

V₁ = first negative point of VRpp

Vz = VRpp data point at zero crossing

The similar principal is used for phase current. The equation is as follows:-

$$(I - 0) / (I - I_1) = Iz \quad [8]$$

where,

I = last positive point of IRpp

I₁ = first negative point of IRpp

Iz = IRpp data point at zero crossing

VRpp	IRpp	phi	
11923.01	200.9945		
7173.327	154.372		
2228.448	99.9792	1.424148607	V _{zstep}
-3025.485	35.22583	2.515151515	I _{zstep}
-8311.95	-33.15372	1.091002908	Δ
-13403.22	-97.90709	12.27378272	θ
-17925.18	-160.0703	0.21421792	rad
-21796.5	-211.355	0.97714295	cos(rad)
-24512.93	-253.8332	0.21258329	sin(rad)
-26188.33	-288.023		

Figure 2: Example of excel computation of phase angle

The difference between space units of $VRpp$ and $IRpp$ can be computed. A step approach is implemented for ease of Excel calculation. Referring to Figure 2, since voltage is leading current in this case, $VRpp$ is treated as the first unit. It can be seen that $IRpp$ is 2 steps behind $VRpp$. Thus the formula is as follows:-

$$Vzstep = Vz + 1 \quad [9]$$

$$Izstep = Iz + (\text{no. of cells away from } Vz)$$

However, in the event that current leads voltage, $IRpp$ is treated as the first unit and the following equation applies:-

$$Izstep = Iz + 1 \quad [10]$$

$$Vzstep = Vz + (\text{no. of cells away from } Iz)$$

The difference between the cells is established. In this case current is always used as the reference point no matter whether it leads or lags voltage. The equation is as follows:-

$$\Delta = Izstep - Vzstep \quad [11]$$

Now, Δ has to be changed from units to degrees. Since the sampling rate is 32 samples per cycle, the following equation is used:-

$$\theta = (\Delta / 32) * 360^\circ \quad [12]$$

Since Excel only recognises radians and not degrees in its mathematical formulation, the next step is to convert as follows:-

$$\text{rad} = \theta * (\pi / 180^\circ) \quad [13]$$

Finally, the sine and cosine of the phase angle in radians is ready to be computed. By copying all these formulas in respective columns, the varying phase angle can be tabulated easily. The formulation of Real and Reactive Power proves to be an easy task as all the relevant information is available. The following equations are used:-

$$\text{Real Power, } P = V_{rms} * I_{rms} * \cos(\text{rad}) \quad [14]$$

$$\text{Reactive Power, } Q = V_{rms} * I_{rms} * \sin(\text{rad})$$

4.0 NON-LINEAR LEAST SQUARE REGRESSION

Non-linear regression is used when a best fit needs to be computed to a set of data with an inherent non-linear equation attached to it. The main difference between linear and non-linear regression is that the solution must proceed in an iterative fashion.

Looking at the static load model, it can be linearised by taking its base-10 logarithm to give the following equation:-

$$\text{Log } (P / P_0) = np \text{ Log } (V / V_0) + c \quad [15]$$

$$\text{Log } (Q / Q_0) = nq \text{ Log } (V / V_0) + c$$

In their transformed forms, these models can use linear regression to evaluate the constant coefficients. They could then be transformed back to their original state and used for predictive purposes. This has been done in previous works [3][5]. The problem is that the transformation distorts the experimental error. Linear regression assumes that the scatter of points around the line follows a Gaussian distribution and that the standard deviation is the same at every value of V . These assumptions are rarely true after transforming data. Furthermore, some transformations alter the relationship between V and P . Thus, non-linear regression is employed.

To remove the subjectivity of the chosen method, some criterion must be devised to establish a basis for the fit.

a. Residual Analysis

The residuals from a fitted model are defined as the differences between the response data and the fit to the response data at each predictor value.

$$r_i = y_i - \hat{y}_i \quad [16]$$

Assuming the model fitted to the data is correct, the residuals approximate the random errors. Therefore, if the residuals appear to behave randomly, it suggests that the model fits the data well. However, if the residuals display a systematic pattern, it is a clear sign that the model fits the data poorly.

b. Sum of Squares Due to Error (SSE)

This statistic measures the total deviation of the response values from the fit to the response values.

$$SSE = \sum_{i=1}^n (y_i - \hat{y}_i)^2 \quad [17]$$

A value closer to 0 indicates that the model has a smaller random error component, and that the fit will be more useful for prediction.

c. R-Square

This statistic measures how successful the fit is in explaining the variation of the data. Thus, R -square is the square of the correlation between the response values and the predicted response values. R -square is defined as the ratio of the sum of squares of the regression (SSR) and the total sum of squares (SST). SSR is defined as,

$$SSR = \sum_{i=1}^n (y_i - \bar{y})^2 \quad [18]$$

SST is called the sum of squares about the mean, and is defined as,

$$SST = \sum_{i=1}^n (y_i - \bar{y})^2 \quad [19]$$

where,
 $SST = SSR + SSE$

Given these definitions, R -square is expressed as,

$$R\text{-Square} = SSR/SST = 1 - (SSE/SST) \quad [20]$$

R -square can take on any value between 0 and 1, with a value closer to 1 indicating that a greater proportion of variance is accounted for by the model.

In this study, MATLAB software is used to perform the non-linear regression analysis.

5.0 RESULTS

Results are obtained for the following points of interest:-

- Voltage Profile Analysis
- Current Profile Analysis
- $P(\text{pu})$, $Q(\text{pu})$ and $V(\text{pu})$ Vs. Time Analysis
- $P(\text{pu})$ and $Q(\text{pu})$ Vs. Voltage Analysis
- Load Change Post Fault with respect to Voltage Sag
- Load Change During Fault with respect to Voltage Sag

Voltage is seen to gradually recover at all feeders. The full recovery of voltage is delayed due to re-connection of tripped load [1]. Voltage sag behavior of each phase is also clearly explained by the PSCAD Simulation performed. Voltage sags are seen to be longer for feeders closer to the fault point.

Current profile observations indicate that red phase current increase across all feeders during the fault. This could be caused by the nature of the fault. The Y-B-N fault on the HV side caused

the yellow and blue phase current to drop to zero, leaving the red phase to supply the 132/33kV transformer for all 3 phases. This would lead to temporary increase in current on red phase during the fault. The surge in current before stabilising at all phases for each feeder indicated a large percentage of motor load [7, 8, 9]. This would also account for the rise in red phase real power during the fault.

The active and reactive power for all phases and feeders is also seen to rapidly increase for a short instance of time after fault clearing to levels even above the pre-disturbance value. Surge of power is particularly overwhelming in terms of reactive power. This could be due to the reacceleration of motors, which requires high starting currents.

Upon voltage recovery, it is seen that certain feeders observed loads that are slightly higher than its pre-disturbance values. This type of load behavior has been observed for air conditioning load in previous studies [10]. A higher reactive load post-disturbance indicates that more reactive power is required to boost the supply following the voltage sag, as observed in many feeders. Most feeders showed a slightly higher operating voltage and lower real power readings post-disturbance. This is indicative that certain loads have failed to recover, boosting the voltage automatically.

Recovery time of the voltage increases as the severity increases. Load recovery time to point of stabilisation which shows mixed results in this research, indicates that more sensitive load takes a much longer time to recover. Based on information from TNB Regional Control Centre in Table 1, it can be seen that heavy industrial loads with more motors take a much longer time to recover.

Table 1: Load information for individual feeders

Feeder	Load Information
BJLL4L5	PPU Taman Teknologi (Astro, Tmn. Teknologi, Mimos, KOMDAQ)
BTGA2L5	SSU Chung Hwa (Plastic Factory)
HCOM3L5	PPU Hicom E (Metal Stamping, Teck See Plastic, Kohno Plastic, Federal Paper)
KLJT8L5	SSU Permodalan Nasional Bhd. & PPU PNAL (Bukit Bintang Commercial Complexes)
MERU4L5	PPU Puncak Alam (Small Industrial Companies)
PIDH2L5	SSU M Food Factory
PJST6L5	PPU Dunlop (Dunlop, Bousted Switching)
PMJU7L5	SSU Texas Instruments (Electronic Manufacturing)
PROT3L5	SSU Nissan IOI (Car Manufacturing)
SHAE8L5	SSU Nippon Electric Glass Factory No.2
SRDG1L5	PPU Perindustrian Bukit Serdang

Tables 2, 3 and 4 indicate results for load-voltage dependency based on post fault load change.

Table 2 : Load change post fault with respect to voltage sag for red phase

RED PHASE		
Feeder	$dp(\text{pre-post})/dv(\text{fault})$	$dq(\text{pre-post})/dv(\text{fault})$
BJLL4L5	0.778	1.926
BTGA2L5	0.444	0.333
HCOM3L5	0.370	0.444
KLJT8L5	0.084	-0.335
MERU4L5	-0.179	-0.893
NUNI13L5	0.082	-0.164
PIDH2L5	0.105	-1.263
PJST6L5	0.433	0.767
PMJU7L5	0.393	-0.429
PROT3L5	2.630	-4.889
SHAE8L5	0.185	0.519
SRDG1L5	0.167	-0.333
TMSY2L5	0.267	-1.367

Table 3 : Load change post fault with respect to voltage sag for yellow phase

YELLOW PHASE		
Feeder	$dp(\text{pre-post})/dv(\text{fault})$	$dq(\text{pre-post})/dv(\text{fault})$
BJLL4L5	1.080	2.160
BTGA2L5	0.708	1.083
HCOM3L5	0.667	0.667
KLJT8L5	0.087	-0.174
MERU4L5	0.056	-0.500
NUNI13L5	0.435	0.174
PIDH2L5	0.433	-0.577
PJST6L5	0.846	0.962
PMJU7L5	0.760	-0.320
PROT3L5	3.217	-6.696
SHAE8L5	0.458	0.167
SRDG1L5	0.080	-0.600
TMSY2L5	0.538	-1.500

Table 4 : Load change post fault with respect to voltage sag for blue phase

BLUE PHASE		
Feeder	dp(pre-post)/dv(fault)	dq(pre-post)/dv(fault)
BJLL4L5	0.322	0.831
BTGA2L5	0.241	0.741
HCOM3L5	0.224	0.414
KLJT8L5	-0.018	-0.073
MERU4L5	-0.027	-0.109
NUNI13L5	0.109	0.273
PIDH2L5	0.067	0.044
PJST6L5	0.233	0.616
PMJU7L5	0.225	0.085
PROT3L5	1.259	-2.759
SHAE8L5	0.169	0.288
SRDG1L5	-0.026	-0.132
TMSY2L5	0.183	-0.085

The load change post fault compared to voltage sag indicates that KLJT is least dependent to voltage. This could be because the fault occurred on a Saturday morning where many of the offices were not operating. Since KLJT is mainly made up of commercial load, the effect would have been severe if the fault were to occur on a weekday during working hours. BJLL and PROT showed highest dependency since both a technology based industries which would have been operating as usual on a Saturday morning.

Figures 5, 6 and 7 indicate results from MATLAB simulation to analyse load-voltage dependency during fault.

Table 5 : Load change during fault with respect to voltage sag for red phase using MATLAB simulation

RED PHASE						
Feeder	dp/dv	R ²	SSE	dq/dv	R ²	SSE
BJLL4L5	-2.583	0.7164	0.622	9.467	0.5622	20.38
BTGA2L5	-2.026	0.7567	0.2777	7.293	0.2477	19.84
HCOM3L5	-2.186	0.7914	0.2421	8.124	0.6164	10.57
KLJT8L5	-1.142	0.8508	0.1655	2.665	0.8836	0.5799
MERU4L5	-1.729	0.7773	0.2672	16.37	0.6918	10.64
NUNI13L5	-3.048	0.8993	0.4442	7.031	0.6384	10.82
PIDH2L5	-3.997	0.8544	0.7715	13.38	0.5232	24.77
PJST6L5	-3.370	0.7903	0.8616	9.373	0.2277	64
PMJU7L5	-2.064	0.7863	0.2274	3.330	0.7709	2.753
PROT3L5	-2.548	0.7199	0.6602	4.286	0.6767	0.5003
SHAE8L5	-0.9069	0.7061	0.1221	5.375	0.6285	4.262
SRDG1L5	-1.536	0.8170	0.2812	5.077	0.7070	5.284
TMSY2L5	-1.948	0.8098	0.5228	13.39	0.3345	46.61

Table 6 : Load change during fault with respect to voltage sag for yellow phase using MATLAB simulation

YELLOW PHASE						
Feeder	dp/dv	R ²	SSE	dq/dv	R ²	SSE
BJLL4L5	4.872	0.8979	0.1687	20.22	0.6286	59.27
BTGA2L5	6.085	0.9413	0.1391	6.097	0.9448	0.1392
HCOM3L5	6.797	0.9112	0.2566	6.646	0.9201	0.247
KLJT8L5	4.879	0.8831	0.2776	4.381	0.8991	0.1247
MERU4L5	7.320	0.9560	0.179	3.002	0.3439	3.384
NUNI13L5	5.143	0.9374	0.1123	5.097	0.9374	0.1102
PIDH2L5	10.200	0.8420	1.001	8.619	0.9305	0.2455
PJST6L5	3.827	0.9017	0.108	4.484	0.9004	0.2069
PMJU7L5	5.876	0.8723	0.3231	6.125	0.8936	0.3413
PROT3L5	4.593	0.9767	0.0349	5.818	0.7209	13.39
SHAE8L5	3.609	0.9595	0.04187	3.621	0.9624	0.04688
SRDG1L5	4.408	0.8883	0.1637	4.498	0.8945	0.1723
TMSY2L5	4.785	0.9345	0.09851	4.785	0.9345	0.09726

Table 7 : Load change during fault with respect to voltage sag for blue phase using MATLAB simulation

BLUE PHASE						
Feeder	dp/dv	R ²	SSE	dq/dv	R ²	SSE
BJLL4L5	1.690	0.7229	5.677	4.295	0.4705	62.42
BTGA2L5	2.162	0.6105	7.117	5.517	0.5928	187.3
HCOM3L5	2.201	0.6307	6.131	6.508	0.5586	78.03
KLJT8L5	2.996	0.9553	0.01122	2.996	0.9553	0.01125
MERU4L5	2.128	0.8989	0.0171	13.450	0.6006	87.11
NUNI13L5	2.183	0.5992	6.805	7.850	0.4840	66.94
PIDH2L5	2.165	0.4135	14.46	10.370	0.6058	171.7
PJST6L5	1.743	0.6242	10.33	3.944	0.5007	89.07
PMJU7L5	1.338	0.6311	0.04208	4.830	0.5050	45.5
PROT3L5	1.411	0.5417	8.982	2.507	0.6681	72.98
SHAE8L5	1.402	0.8793	0.009415	7.456	0.5129	47.54
SRDG1L5	1.401	0.8373	0.01824	3.256	0.3356	37.96
TMSY2L5	1.894	0.7191	5.424	3.846	0.3611	57.91

It is seen that load change during fault compared to voltage sag also indicates the same reasoning. It is seen that industries, namely factory based loads showed higher sensitivity compared to commercial based loads.

MATLAB modeling clearly indicated that real power was more accurately computed compared to reactive power. It is also seen that the severity of the disturbances accentuates the nonlinear behavior of the load. This is noticed at the blue phase especially. Thus, accuracy of the simulation was indeed questionable for larger voltage sags. It is also seen that general linear calculations as done previously [4] shows values that are much less sensitive compared to the actual situation.

6.0 CONCLUSION

An identification procedure to estimate load dependency to voltage is investigated. The non-linear Least Squares Regression method is finally chosen and implemented using MATLAB. Since, varying phase angle values were not directly measured, a method was devised to calculate these values, which were needed to calculate real and reactive power. It is found that all feeders in question supply heavy commercial and industrial loads.

The surge in current, real and reactive power before stabilising to post-disturbance values, indicate motor and air conditioning loads are involved. Through comparative analysis and plotting of graphs, it can be seen that the larger the voltage sag for a particular phase of a feeder, the slower the recovery time. However, when comparing between feeders, the inherent nature or type of the load connected dictates how fast the recovery takes. The nature of the load at the particular time of the fault also dictates the value of the post disturbance voltage and power.

Analysis also indicates that the larger the voltage sag, the non-linear behavior of the load accentuates. This explains the MATLAB statistical analysis for blue phase reactive power results in particular. Thus, the static load model can only be used

for small voltage variations of sensitive feeder loads. A proper comparison between feeders can only be justified with better information on various aspects of the feeder including daily load demand profile, seasonal load variations and customer habits through out a long term period.

Location of the feeder from point of fault also plays an important role. Certain feeders exhibited more serious conditions compared to others during the fault. Load at KLJT is seen to have been least affected by voltage, while BJLL and PROT seem to be most affected by voltage.

For future work, steady state data should be made available to analyse the load response to voltage during small voltage variations. Data should be taken for various parts of each day, each week for a one-year period to see the effect of weather and time difference on load. High and low peak demand; determination of daily, weekly and seasonal load patterns based on weather conditions and customer habits in the area should be analysed for a long term period. This would mean more rigorous metering and data collection than presently available. Dynamic load modeling should be tested out for faults with large voltage sags. ■

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PROFILE



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EFFECT OF FREEZE-THAW ACTION ON PHYSICAL AND MECHANICAL BEHAVIOR OF MARINE CONCRETE

(Date received: 28.11.2008)

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ABSTRACT

The deterioration of structural concrete in marine environment and its progress with time is a problem of great importance. Particularly, in splash/ tidal zone, salt water spray and alternate wetting–drying cycles often lead to a build up of salt ions within the concrete pores. Moreover, concrete under alternate freeze-thaw actions suffers worstly on account of accumulated ice pressure and also due to gradual penetration of salt ions in it and the formation of expansive/ leachable compound including the rebar corrosion may lead to cracking, spalling and even the structural distress. This paper presents a part of an experimental study on the freeze–thaw effect of concrete specimens exposed to artificial seawater simulating the arctic marine environment over a period of 15 months. The test specimens made from two different grades of concrete were subjected to artificial freeze-thaw environment under different condition. The test variables include the concrete grade, seawater concentration, exposure condition, and the deteriorative effects were measured by studying the visual appearance, weight and volume change, compressive strength, permeability characteristics and XRD patterns of the deteriorated test specimens. The test results show that after 360 cycles of freezing and thawing, concrete in sea water losses about 75% of its compressive strength as compared to the strength of plain water cured concrete of similar age.

Keywords: Chloride Attack, Compressive Strength, Durability, Freeze-thaw Action, Marine Environment, Permeability, Sulfate Attack

1.0 INTRODUCTION

Concrete has been used as a basic structural material for various types of offshore/onshore structure over the several decades. Concrete structures in such locations are always required to withstand physical, chemical and mechanical action of sea water under various environmental condition throughout their life span. Moreover, the recent discovery of oil and natural gases in arctic and sub-arctic region has drawn the attention of scientists and researchers to study the performance of concrete structures in such locations. Tidal action causing alternate wetting and drying cycles, abrasion and salt water spray due to wave thrust, high ambient pressure due to large hydrostatic head, freeze-thaw (F-T) cycles due to climatic change, wide variation of ambient temperature and humidity are the major physical and mechanical loadings of marine environment. While, the chemical loadings include the gradual penetration of detrimental sea salt ions into the cement mortar matrix which, after chemical reaction, initiate the decomposition of mortar matrix and also the corrosion of the embedded rebars.

The term marine environment is generally well understood but the complexities inherent in such an environment is not clear. It is not just over the sea, but it could be deemed to be extending over the coast and the neighbourhood of tidal wave, backwater and estuaries [1]. Broadly, it covers the area where concrete becomes wet with sea water and wherever the wind will carry

salt water spray which may be up to a few miles inland [1,2].

Depending on the tidal range, sea water (SW) actions, nature, extent and mechanism of deterioration process, a structure exposed to a marine environment can be divided into four different zones (Refer Figure 1). Structural concrete at the upper part of the splash zone and at atmospheric zones are reported to experience the effect of alternative freezing and thawing. In this location, the air is heavily laden with moisture and contains substantial quantities of salt and gas [3]. Winds are more frequent with a maximum velocity as much as 250 km/hour and the ambient temperature may range from 50°C in the Persian/Arabian Gulf to -45°C in the arctic region [4].

Typically, sea water contains about 3.5% soluble salts by weight [5]. The relative ionic concentrations are 18 gpl Cl⁻, 12 gpl Na⁺, 2.6 gpl (SO₄)²⁻, 1.4 gpl Mg²⁺ and 0.5 gpl Ca²⁺. Normally, pH of sea water is about 8. However due to the presence of decaying organic matter, highly carbonated sea water show a pH value of 7 or less.

In splash zone, the building up of salt ions takes place due to wave thrust and continuous salt water spray. In cold climatic region, during freezing process, the pore water inside the concrete freezes and expand in volume creating huge pressure on the adjacent concrete. Also the penetrated sea salts in pore water reacts with the hydrated cement products and form the expansive compounds like Ettringine, Friedels salt and some

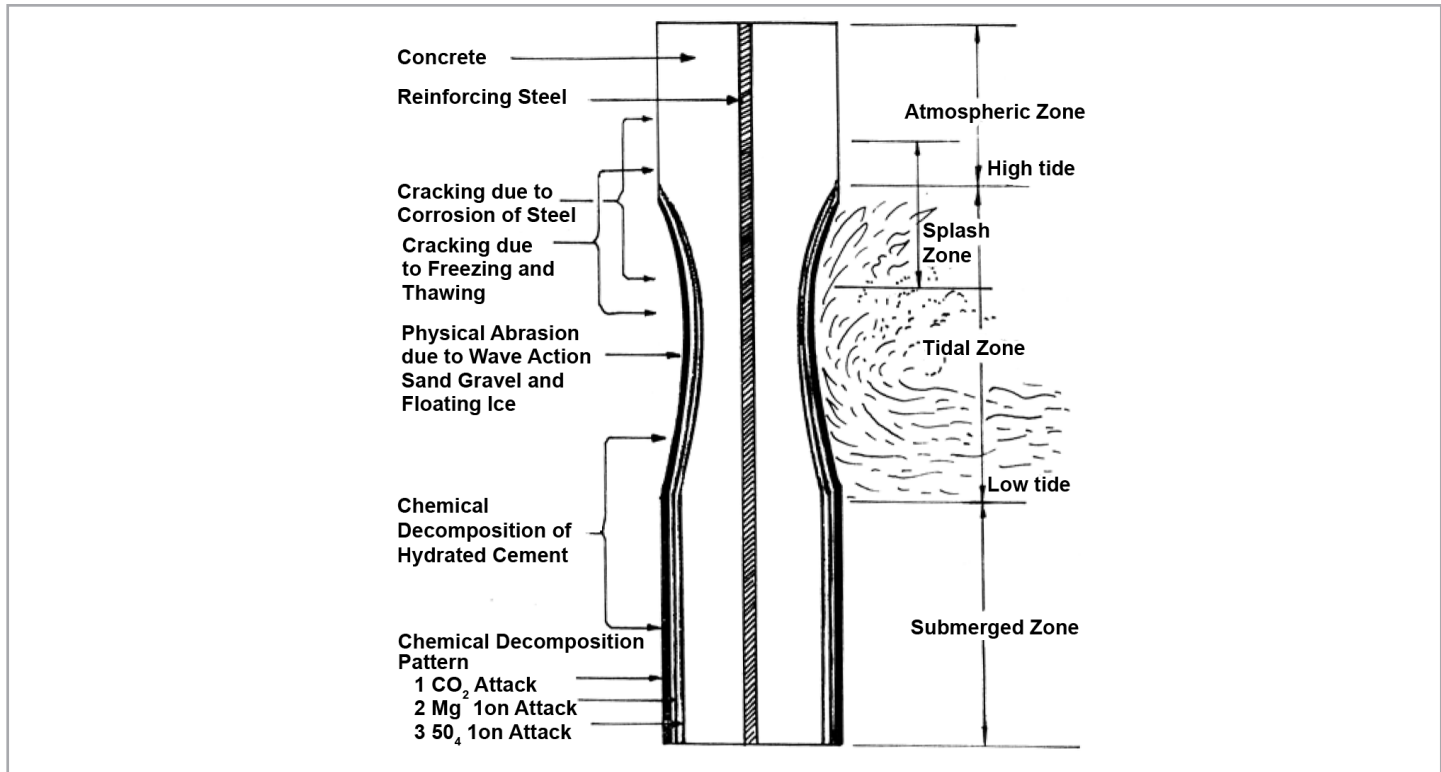


Figure 1: Diagrammatic representation of deterioration of structural concrete exposed to sea water [8]

leachable compounds. Thus in alternate freezing and thawing process, the damage in marine concrete is caused by two fold expanding activities followed by leaching of the soluble leachable compounds during thawing periods which makes the concrete more permeable thereby allowing more harmful salts ions to penetrate inside.

In arctic and sub-arctic region, the nature of deterioration of concrete structures in the sea is dependent on the condition of exposure. The part of the structure that remains fully submerged will be subjected to chemical attack mainly, whereas the section of the structure that is above the high tide level will be affected by frost action in air and by the corrosion of the reinforcement. The concrete in the tidal zone will not only be vulnerable to wetting and drying, frost action and corrosion of reinforcement but also to chemical decomposition of the cement paste [6].

2.0 FREEZE-THAW EFFECT OF MARINE CONCRETE

Freezing is a gradual process and depends on the rate of heat transfer, progressive increase in the concentration of dissolved salts in pore water and the pore size distribution. Fresh concrete contain a considerable quantity of free water and on freezing, discrete ice lenses are formed which expands about 9% in volume. The increased volume of the ice lenses disrupts the fresh concrete causing nearly permanent damage to concrete. The fully hardened concrete is also vulnerable to frost damage, particularly to the effect of alternate cycle of freezing and thawing. The severest condition for frost action arise when concrete has more than one face exposed to weather and in such a position it remains wet for a long period.

One of the theories attributes the damage directly to the empty space available being insufficient to accommodate the

additional solid produced when the free water held in concrete freezes. Second theory attributes the failure to the production of pressure due to the growth of ice lenses parallel to the surface of the concrete owing to the migration of water from capillaries like the phenomena of frost heaving in soils. Another theory explains the failure to the generation of water pressure within the capillary cavities as the ice crystals grow. In all these theories, the permeability, rate of absorption and degree of saturation of the concrete are all important factors. Freezing starts at the surface in the largest cavities and gradually extends to smaller cavities. Water contained in the gel pores are too small to get frozen till the temperature goes below -78°C [5]. In practice, no ice is formed in the gel pores. The resistance of concrete to frost action depends on the strength of the paste, water/cement ratio, type of aggregate used, age of concrete, duration and extent to which the concrete is subject to freezing action.

It has been estimated that the freezing of water in the hardened concrete may exert a pressure of about 14 MPa which is large enough to exceed its tensile strength and consequently the damage occur [5]. The extent of damage varies from surface scaling to complete disintegration as ice is formed, starting at the exposed surface to the concrete and progressing through its depth.

In a marine environment, in addition to its presence in original mix, the chloride ion penetrates into the concrete either from sea water or sea winds carrying sea salts and reacts with the hydrated cement products which produces complex compounds including Friedels salt which are leachable and expansive in nature. The chloride attacks also destroy the passivity of steel and lead to the initiation of rebar corrosion. On the other hand, the penetration of sulphate ions attack the hydrated cement matrix with the formation of gypsum and a complex compound known as calcium sulphaaluminate (ettringite). Bogue [7] is of the

opinion that the formation of gypsum hydrate causes an increase of 17.7% in volume. The action of principal sea salts namely NaCl, MgCl₂, MgSO₄ and CaSO₄ on hardened concrete results in the formation of the expansive/leachable compounds like Calcium chloroaluminate (Friedels Salts), Calcium aluminate sulfate hydrate (Ettringite), CaCl₂ *etc.* [8,9,10].

The reaction products such as Friedels salt has a property of low to medium expansion. The formation of excess calcium chloride, being leachable, results in increased permeability of structural concrete leading to materials loss and weakening. Formation of ettringite is associated with expansion and cracking. Thus, the detrimental effects associated with the chemical reactions manifest themselves as physical effects such as change in volume, weight, permeability and other properties of concrete. Bryant Mather [11] considers leaching, expansion, formation of weak compounds as the various causes of concrete deterioration when subjected to chloride or sulfate attack.

From the above discussion, it is clear that the structural concrete exposed to a marine environment with alternate freezing and thawing cycles faces twofold problems regarding deterioration. Concrete in such locations are required to withstand the stress resulting from the increased volume of frozen pore water and also from the expansive hydrated products caused by the action of sea salts. The leachable nature of some hydrated compounds lead to the formation of porous concrete which allows more objectionable salts ions to come in contact with the interior unaffected concrete and thereby enhancing the deterioration mechanism. Detwiler *et al.* [12] assessed the durability of concrete in Freezing and Thawing by examining the microstructure of the deteriorated concrete specimens with the help of scanning electron microscope and suggested to include microscopic examination of laboratory test specimens to predict the behaviour of concrete under freeze-thaw condition. Whiting *et al.* [13] studied the frost resistance of concrete subjected to a deicing agent namely NaCl at different concentration (1.5, 3 and 10%) and under various ambient condition (30%, 47%, 72% and 88% RH). Concrete specimens fully saturated in 3% NaCl solution exhibited greater expansion for successive F-T cycle. Wright *et al.* [14] carried out an investigation into methods of carrying out accelerated freezing and thawing test on concrete and showed that damage occurred more rapidly when specimens were frozen immersed in water than frozen in air although the rate of freezing was lower.

Zamam *et al.* [15] studied the deterioration of concrete due to freezing and thawing and to deicing chemical use and showed that the visual estimation could be extremely misleading in terms of strength characteristics and maximum loss in strength was recorded for saturated solution. Chandra and Xu [16] studied the influence of presaturation and F-T test condition on length changes of Portland Cement Mortar. It was seen that swelling depended upon the salt concentration gradient and was greater in dilute salt than in saturated salt solution. Maximum deterioration was observed at 3% NaCl solution. Mosongo *et al.* [6] in their Freeze-Thaw test on concrete in sea water found that surface effects probably could play an important role in the deterioration mechanism of concrete under arctic condition.

Among the several durability problems that arise in concrete exposed to a marine environment, those that have

been studied extensively are the chemical attack of sea water on cement paste, concrete and the corrosion of embedded steel. Very limited research works have been documented on the freezing and thawing in sea water. The relevant information presented in literature varies to a great extent or debatable. It is important to have a detail information regarding the concrete durability during freezing and thawing in sea water in view of the increasing use of concrete in the sub-arctic/arctic region. The freeze-thaw action on concrete structures in the splash/tidal zone has its own characteristics and is dependent on ambient air and sea water temperature. Concrete in this location can thaw at temperature as low as -1°C by the action of waves and tides and can freeze to temperature as low as -50°C when exposed to air [6]. These severe conditions, in combination with the chemical attack of the sea water on the cement constituents, are found to lead to a more pronounced deterioration in the concrete structure.

In this investigation, the freeze-thaw durability of two different grades of concrete has been studied in a laboratory simulated artificial marine environment representing the structural concrete exposed to arctic/sub-arctic region. The similar study was also conducted in fresh water environment for comparison. Physical, mechanical as well as chemical aspects of concrete deterioration in such location has been examined with view to compare the data reported by other researchers in the similar field.

3.0 EXPERIMENTAL PROGRAM

An experimental investigation together with the creation of artificial marine environment was carried out over a period of 15 months. The material used and the variables studied were as follows:

Materials Used:

Ordinary Portland Cement (OPC) conforming to IS 269-1976 from a single lot was used as binding material. The physical properties and chemical composition of the cement is given in Table-1. Coarse Aggregates were 20 mm down well graded crushed stone having fineness modulus (FM) of 6.95 while Fine Aggregates used were well graded coarse sand of FM 2.63 and both the aggregates conformed to IS 383-1970 requirements. Commercial grade salts were used for making artificial sea water which was procured in a single lot to avoid the variation in effective ion concentration.

Variables Studied:

(a) Curing Water:

Plain water (PW) as well as artificially made sea water (SW) were used for curing the specimens. SW was made by mixing tap water with exact amount and proportion of principal salts found in natural sea water. The composition of artificial sea water is given in Table-2.

(b) Exposure Condition:

The alternate Freezing-Thawing arrangement was created in a F-T chamber. In each F-T cycle, the temperature was varied from (-27°C) to (+27°C) over a total period of 30 hours (12+12

hours for freezing and thawing; 3+3 hours kept at two terminal temperatures). The test specimens were kept in three separate states namely (i) Submerged (SUB) in SW, (ii) Submerged in PW and (iii) Atmospheric (ATM)

(c) Concrete Grade:

Two different grades of concrete namely concrete A and B. The relevant information of the concrete mixes is given in Table-3.

(d) Exposure Periods:

30, 90, 180 and 360 F-T cycles.

(e) Type and Size of Test Specimens:

150 mm dia x 150 mm high cylindrical specimens.

(f) Relevant Tests:

Visual examination, weight change, volume change, compressive strength, permeability, stress-strain characteristics and X-ray diffractometry (XRD).

A total number of 84 specimens were cast from two different concrete mixes as per test requirements. Plain water (PW) was used to cast all the specimens. The specimens after 28 days PW curing at 27°C were exposed to different conditions in the F-T chamber. Separate Galvanized Iron (GI) tanks were used for PW, SW and ATM state of exposure. After a specific period of exposure, the specimens were taken out for various tests as stated above. The coefficient of permeability of the test specimens were determined according to IS: 3085-1965 based on Darcy's law whereas the compressive strength were determined as per IS: 516-1959. Various nondestructive tests including visual examination, weight and volume change, permeability tests were carried out before conducting destructive tests. Concrete chips were collected from different points of the deteriorated specimens for XRD studies. The experimental program including the various tests conducted are summarised in Table-3. The results of the investigations are presented and discussed below.

Table 1: Physical properties and Chemical Composition of Cement (OPC)

(Source: Associated Cement Company, India)

Sl. No.	Characteristics	Value obtained (experimental)	As Per IS: 269-1976
1.	Fineness (90 μ Sieve) %	9.5	<10
2.	Blaine's Sp. surface (cm ² /g)	2560	>2250
3.	Normal Consistency	24	---
4.	Soundness by Le Chatelier's Test (mm)	0.35	<5
5.	Specific gravity	3.15	3.15
6.	Setting Time		
	Initial (min.)	31	>30
	Final (min)	63	<600
7.	Compressive Strength (MPa)		
	(i) 3 days	23.70	>16.0
	(ii) 7 days	33.5	>22.0
8.	Loss on ignition	1.99	<4
9.	Insoluble matter	1.43	---
10.	SiO ₂ (silica)	17.40	---
11.	Al ₂ O ₃	11.56	---
12.	Fe ₂ O ₃	2.51	---
13.	CaO	60.05	---
14.	MgO	1.30	<5
15.	SO ₃	2.68	---

Table 2: Composition of artificial sea water

Salt Type	Chemical Formula	Amount (gm)	Remarks
Sodium Chloride	NaCl	27.20	These amounts of salts were dissolved in plain water to prepare 1000 gm of Sea water of 1N concentration
Magnesium Chloride	MgCl ₂	3.80	
Magnesium Sulfate	MgSO ₄	1.70	
Calcium Sulfate	CaSO ₄	1.20	
Potassium Sulfate	K ₂ SO ₄	0.90	
Calcium Carbonate	CaCO ₃	0.10	
Magnesium Bromide	MgBr ₂	0.10	
		Total: 35.0	

Table 3: Experimentation programme in freeze-thaw environment

Concrete	Compressive Strength (MPa)	Specimen Size (mm)	Specimen No.	Environmental Conditions				Test	
				Exposure States	Curing Solution	Temp. Condition			
						F-T cycle (± 27°C)	Const. Temp (+ 27°C)		
A B	46.0 53.0	D=150 H=150	24	SUB	SUB (1N)	30	-----	a) Visual Examination b) Weight change c) Volume change d) Permeability e) Compressive strength f) Stress-strain characteristics g) X-ray diffractometry	
						90	-----		
						180	-----		
						360	-----		
A B	46.0 53.0	D=150 H=150	24	SUB	PW	30	-----		
						90	-----		
						180	-----		
						360	-----		
A B	46.0 53.0	D=150 H=150	24	ATM	---	30	-----		
						90	-----		
						180	-----		
						360	-----		
A B	46.0 53.0	D=150 H=150	12	SUB	PW	30	38 days		
						90	112 days		
						180	225 days		
						360	450 days		
Total : 84 Nos									
Concrete A: Mix 1:1.15:2.20, w/c=0.39, 28 days cube strength=46.0 MPa Concrete B: Mix 1:1:2.10, w/c=0.36, 28 days cube strength=53.0 MPa SW: Sea Water; PW: Plain Water; 1N: Normal sea water concentration ATM: Atmospheric state; SUB: Submerged state; D=Diameter; H=Hight; F-T: Freezing and Thawing.									

4.0 RESULTS AND DISCUSSION

Concrete specimens after specific period of alternate F-T exposure in the submerged states of sea water, plain water and in atmospheric states are taken out for conducting various tests. After visual inspection, it is seen that concrete in sea water have lost their dimensional stability with substantial erosion and splitting/crumbling on the surface (Refer to Plate No.1) whereas in PW and in ATM states, concrete surface tend to become uneven. Some changes in colour from the original dark gray to lime gray in the specimens in sea water have also been observed which indicates either the salts deposition on the concrete surfaces or leaching out of portlandite, $\text{Ca}(\text{OH})_2$. Also interior surfaces have indicated a higher level of saturation with increasing number of F-T cycles both in sea and plain water environments.

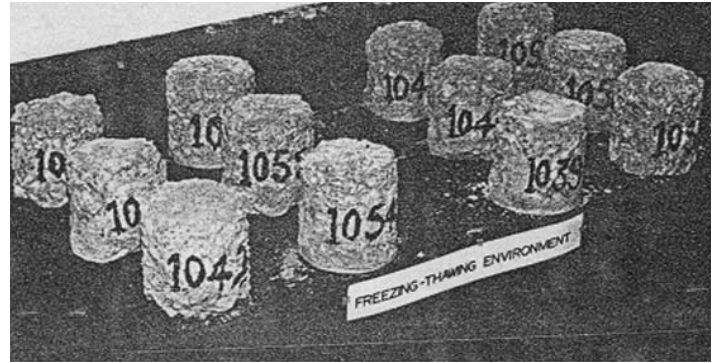


Plate no. 1: Concrete specimens after 360 cycles of freezing and thawing in seawater

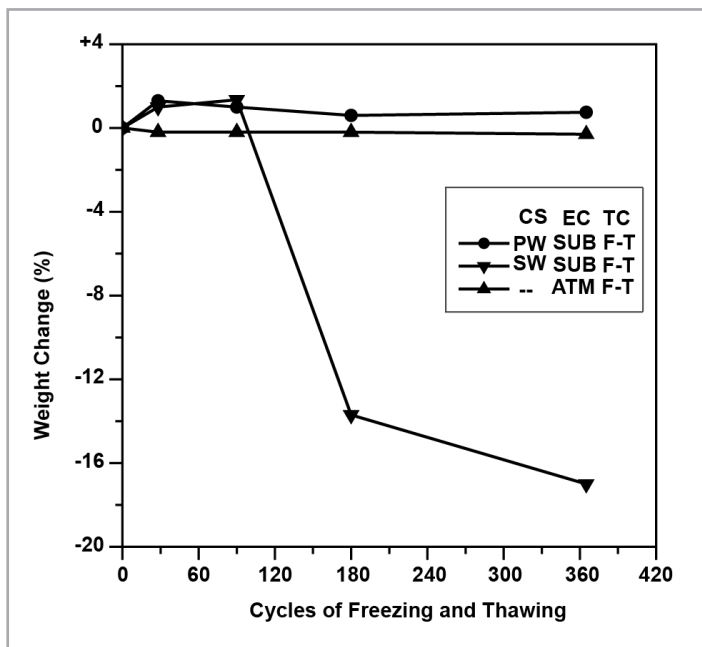


Figure 1(a): Weight change - Freeze-thaw relation for concrete-A CS-Curing Soln.; EC-Exp. Cond.; TC-Temp. Cond.

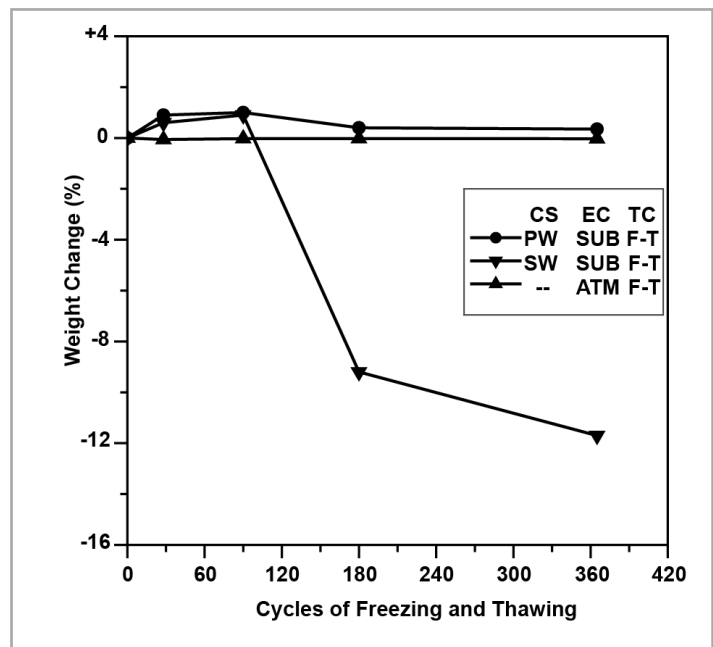


Figure 1(b): Weight change - Freeze-thaw relation for concrete-B CS-Curing Soln.; EC-Exp. Cond.; TC-Temp. Cond.

The change in the weight of the specimens of concrete A and B in different exposure conditions have been illustrated in Figures 1(a) and 1(b) respectively. A close examination reveals that initially in the first about 90 cycles, the submerged specimens exhibit a higher percentage (nearly 0.7 to 1.4 percent) of weight gain as compared to the atmospheric state specimens (nearly 0.2 to 0.3 percent). This increase in the weigh may be primarily due to the ingress of salt water or plain water into the concrete. After 90 cycles, a significant difference in the trend of the weight change of the concrete specimens exposed to sea water has been found to occur as compared to that for the specimens placed in plain water or in the atmospheric state. After 360 cycles of freeze-and-thaw, a considerable change in the weight lying between -16.5 percent and -11.5 percent is observed for the concretes A and B exposed to the sea water environment whereas for the specimens placed in plain water or in the atmospheric state, the changes in the weight are found to lie in the range of +0.3 to +1.0 percent and -0.2 to -0.1 percent respectively for both the concretes. The considerable loss of weight of the specimens exposed to

sea water is primarily due to crumbling of outer surfaces of the specimens caused by crystallisation of sea salts in the voids of concrete and their subsequent expansion during freeze-thaw cycles.

The volume changes in the specimens of concretes A and B for the above mentioned environment states are illustrated in Figures 2(a) and 2(b) respectively. It is clear from these figures that the specimens in the submerged state either in sea or plain water have exhibited a volumetric expansion of nearly 0.03 to 0.07 percent till the first 30 cycles of freezing-and-thawing, whereas a volume reduction of 0.03 to 0.07 percent has been found in the specimens placed in the atmospheric state. The increase in volume may be attributed to the ingress of sea or plain water inside the concrete mass. After 30 cycles of alternate freezing-and-thawing, the volume of the concrete specimens placed in sea water has been found to decrease due to surface erosion and splitting and after 360 cycles, this reduction is observed to lie between 1.2 and 1.7 percent for the two concretes A and B. For the specimens placed in plain water or in the atmospheric

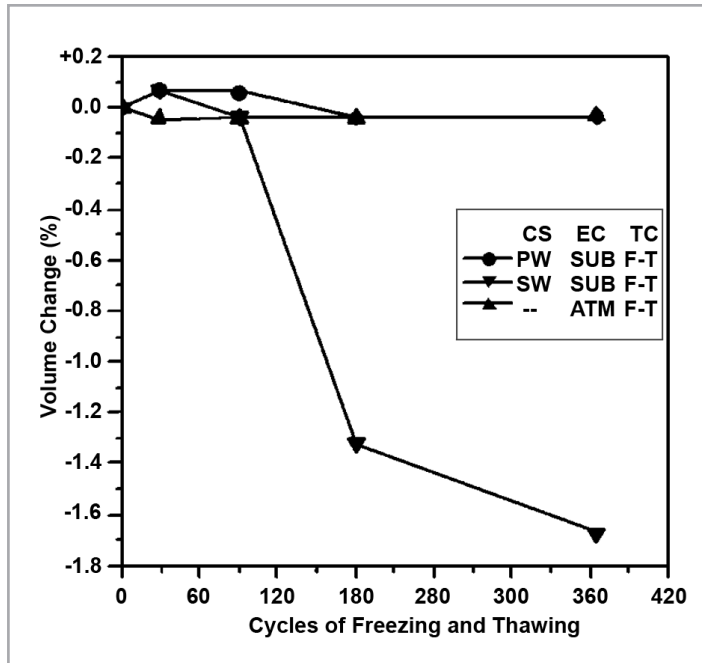


Figure 2(a): Volume change - Freeze-thaw relation for concrete-A CS-Curing Soln.; EC- Exp. Cond.; TC-Temp. Cond.

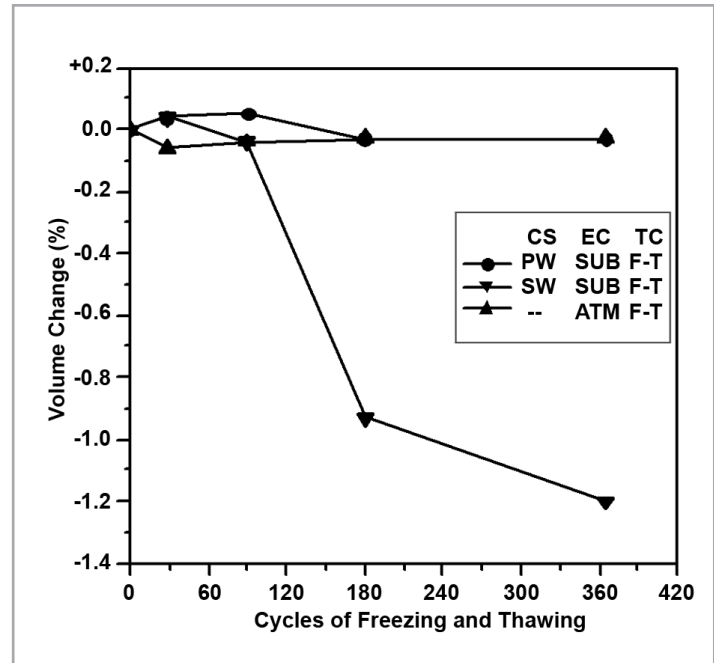


Figure 2(b): Volume change - Freeze-thaw relation for concrete-B CS-Curing Soln.; EC-Exp. Cond.; TC-Temp. Cond.

states, the corresponding reduction in the volumes is about 0.04 percent. The decrease in volume resulting from erosion/crumbling of outer surfaces of concrete may be attributed to the deposition of chemical compounds into the voids of concrete, the crystallisation as well as their expansion due to freezing of the entrapped water inside the voids.

Permeability characteristics of the concrete specimens exposed to different environmental conditions have been graphically illustrated in Figure 3(a) for concrete A and in Figure 3(b) for concrete B. For comparison, the coefficient of permeability of the specimens placed in plain water at a constant temperature of 27°C has been plotted in the same Figures. It

is seen that as the concrete exposed to sea water, it loses its mass through surface erosion and splitting, the coefficient of permeability increases significantly, whereas concrete exposed to plain water or placed in the atmospheric states show only marginal changes in their permeability. After 360 cycles, the permeability of concrete specimens exposed to sea water has been found to lie between 43 and 67×10^{-14} m/sec, whereas the corresponding value for both the concretes placed in plain water or in the atmospheric states lies in the range of 8 to 10×10^{-14} m/sec. Thus, it is seen that the permeability of concrete exposed to 1N concentration of sea water in the freezing-and-thawing environment is at about 5 to 7 times the value of the permeability coefficient

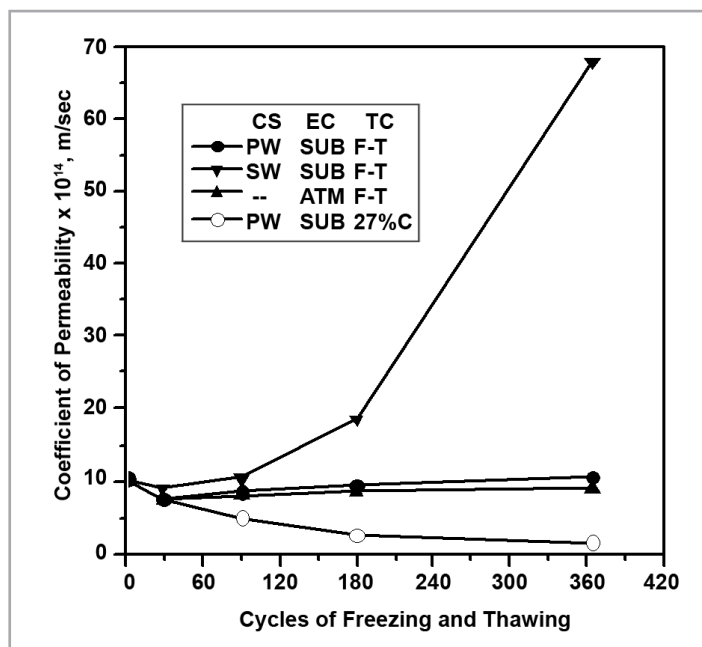


Figure 3(a): Permeability - Freeze-thaw relation for concrete-A CS-Curing Soln.; EC-Exp. Cond.; TC-Temp. Cond.

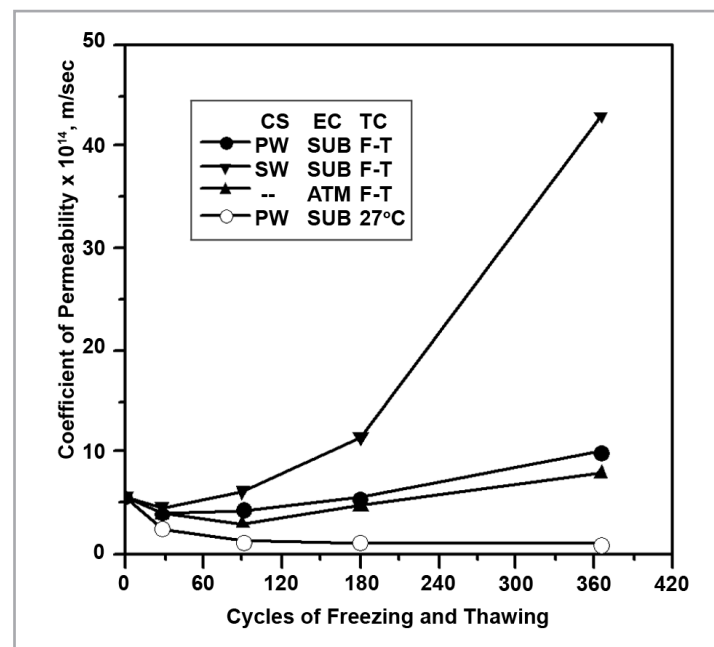


Figure 3(b): Permeability - Freeze-thaw relation for concrete-B CS-Curing Soln.; EC-Exp. Cond.; TC-Temp. Cond.

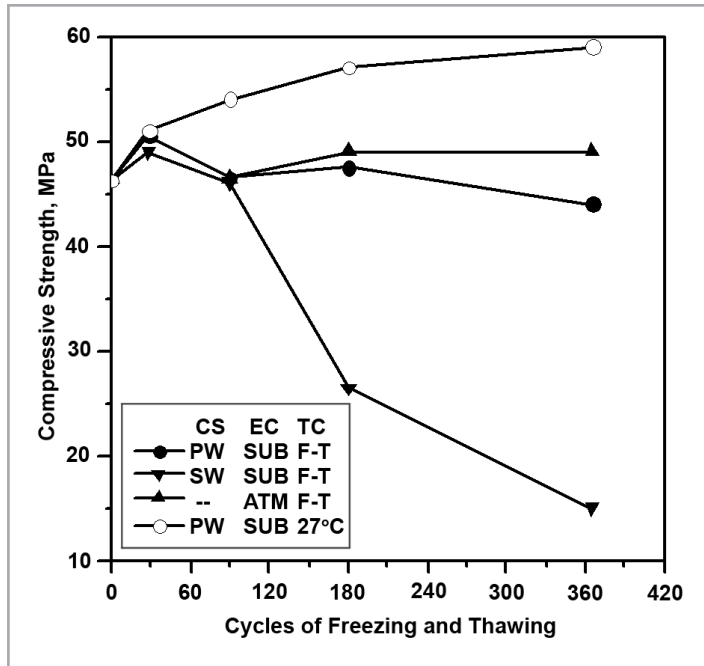


Figure 4(a): Compressive strength - Freeze-thaw relation for concrete-A CS-Curing Soln.; EC-Exp. Cond.; TC-Temp. Cond.

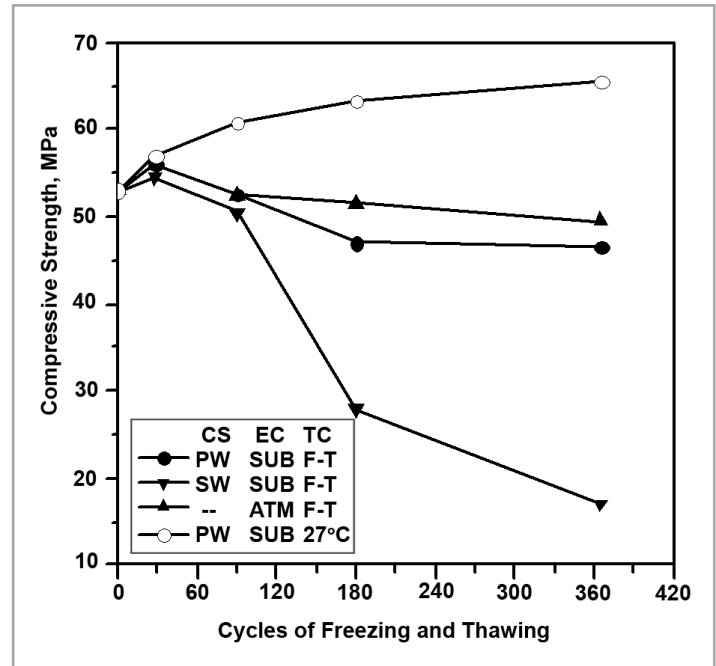


Figure 4(b): Compressive strength - Freeze-thaw relation for concrete-B CS-Curing Soln.; EC-Exp. Cond.; TC-Temp. Cond.

(8 to 10 x 10⁻¹⁴ m/sec) of the concrete placed in the other environments mentioned above. Also after 360 cycles, the permeability of the concrete specimens submerged in plain water or placed in dry (ATM) state in the freezing-and-thawing environment are around 8 times the permeability of concrete placed in plain water at a constant temperature of 27°C. The larger increase in the permeability of the concrete specimens in the freezing-and-thawing environment of sea water may be due to the formation of relatively greater amounts of total expansive/leachable compounds as compared to other environments.

Compressive strength of the test specimens of concrete A and B exposed to the different environment states are shown in Figures 4(a) and 4(b) respectively. A close examination of these curves indicates that the strength increases during the first 30 cycles of freeze-and thaw in sea water environment, whereas it decreases thereafter. This decrease in the compressive strength has been found to be significant after 180 cycles and most significant after 360 cycles. On the other hand, when submerged in plain water and kept in dry (ATM) state in similar environment, the decrease in the compressive strength is found to be only marginal. It may be noted that in comparison

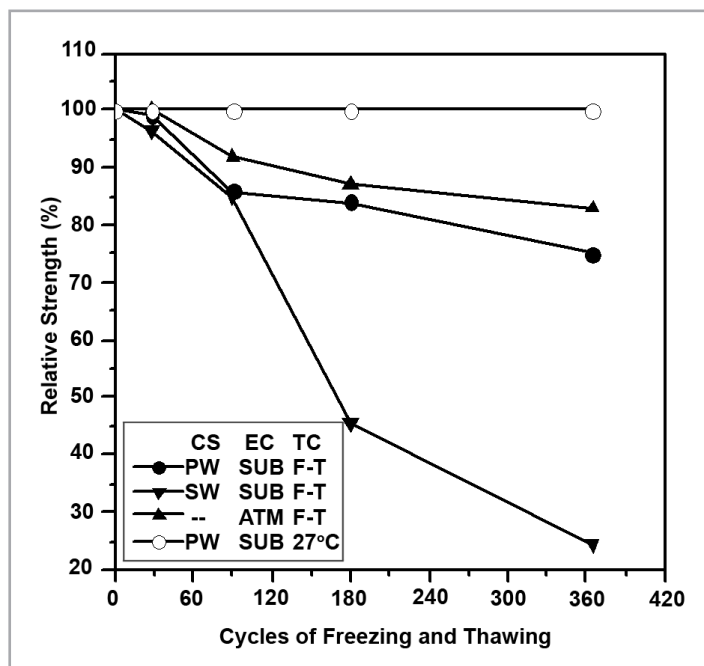


Figure 5(a): Relative strength - Freeze-thaw relation for concrete-A CS-Curing Soln.; EC-Exp. Cond.; TC-Temp. Cond.

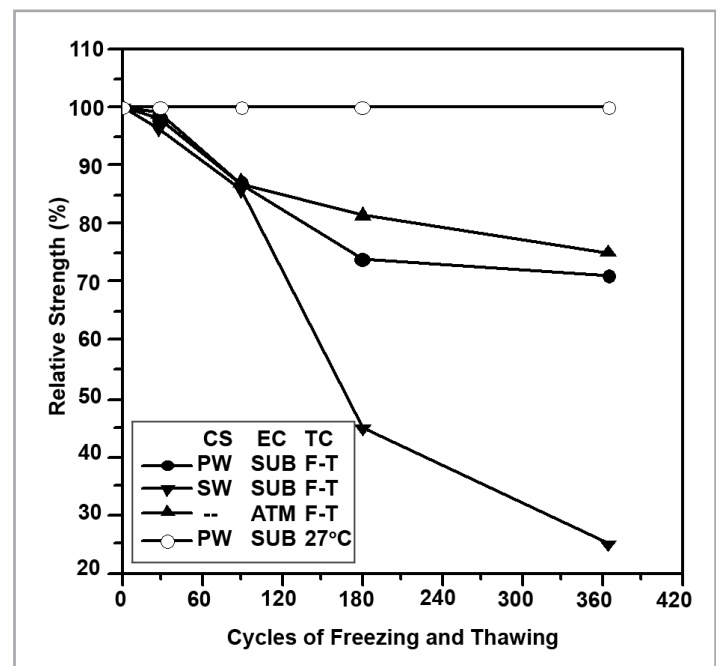


Figure 5(b): Relative strength - Freeze-thaw relation for concrete-B CS-Curing Soln.; EC-Exp. Cond.; TC-Temp. Cond.

to the 28 day compressive strength of plain water cured concrete at constant temperature of 27°C, the compressive strength of the specimens subjected to 360 cycles of freezing-and thawing in sea water, plain water and in ATM state have been found to lie respectively in the ranges of 30 to 32 percent, 89 to 95 percent and 92 to 106 percent for the two concretes. The comparison of the strengths at the same ages of the normally cured concrete specimens and the specimens subjected to freezing-and-thawing actions in different environment states are graphically presented in Figures 5(a) and 5(b) for concretes A and B respectively. It is seen that the compressive strength of the concrete specimens subjected to 360 cycles of freezing-and-thawing in sea water,

plain water and in ATM state lie in the range of 20 to 25 percent, 70 to 75 percent and 75 to 83 percent respectively for the two concretes. Thus, the losses of the compressive strength of concrete specimens after 360 cycles of freezing-and-thawing in sea water, plain water and in atmospheric state (ATM) are around 70, 8, and 5 percent respectively as compared to 28 days strength of normally cured concrete. The corresponding values are 75, 28, and 20 percent respectively as compared to 450 days (360 cycles) strength of normally cured concretes. The increase in strength after the first 30 cycles may due to the fact that the specimens do not get saturated fully by sea water during this period under normal atmospheric pressure. After 360 cycles,

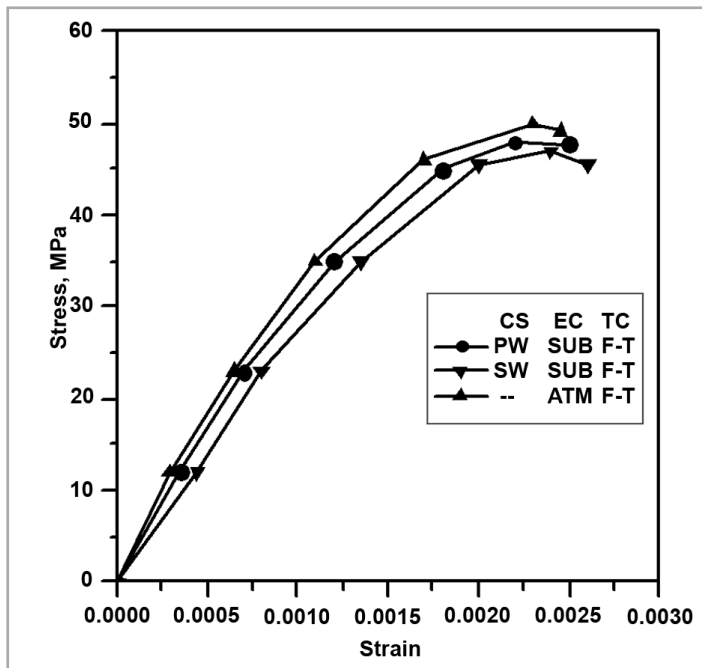


Figure 6(a): Stress-strain curves in compression for concrete-A, after 30 cycles of freezing and thawing

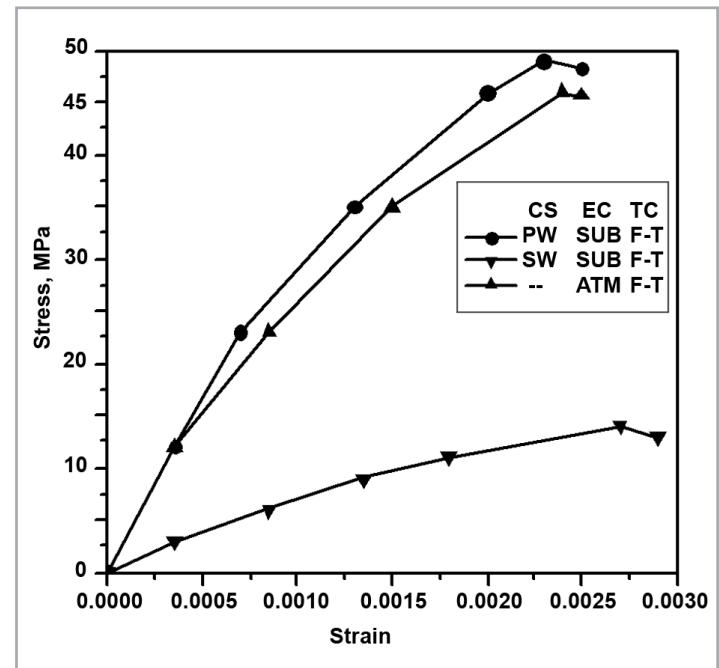


Figure 6(b): Stress-strain curves in compression for concrete-A, after 360 cycles of freezing and thawing

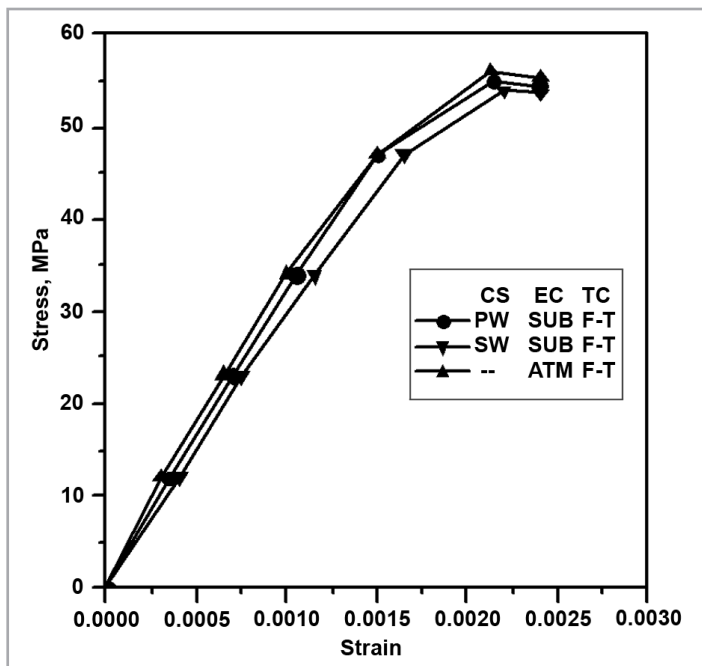


Figure 7(a): Stress-strain curves in compression for concrete-B, after 30 cycles of freezing and thawing

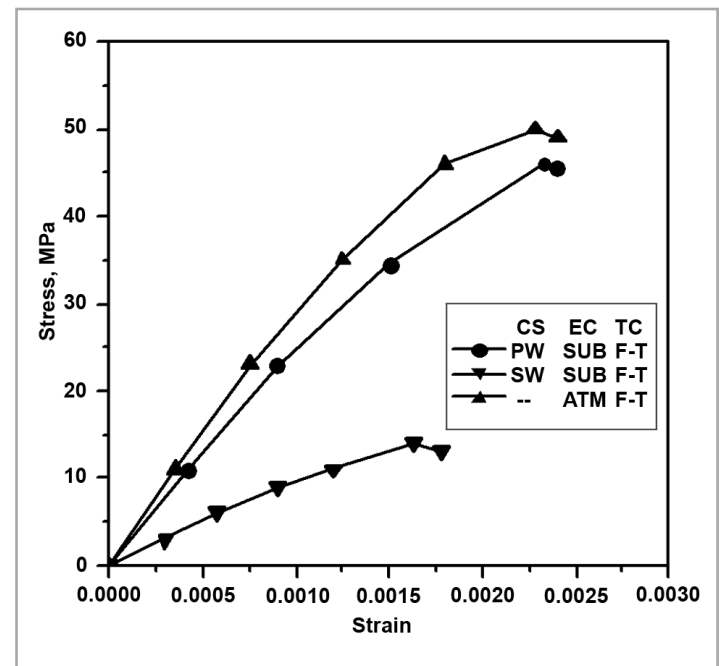


Figure 7(b): Stress-strain curves in compression for concrete-B, after 360 cycles of freezing and thawing

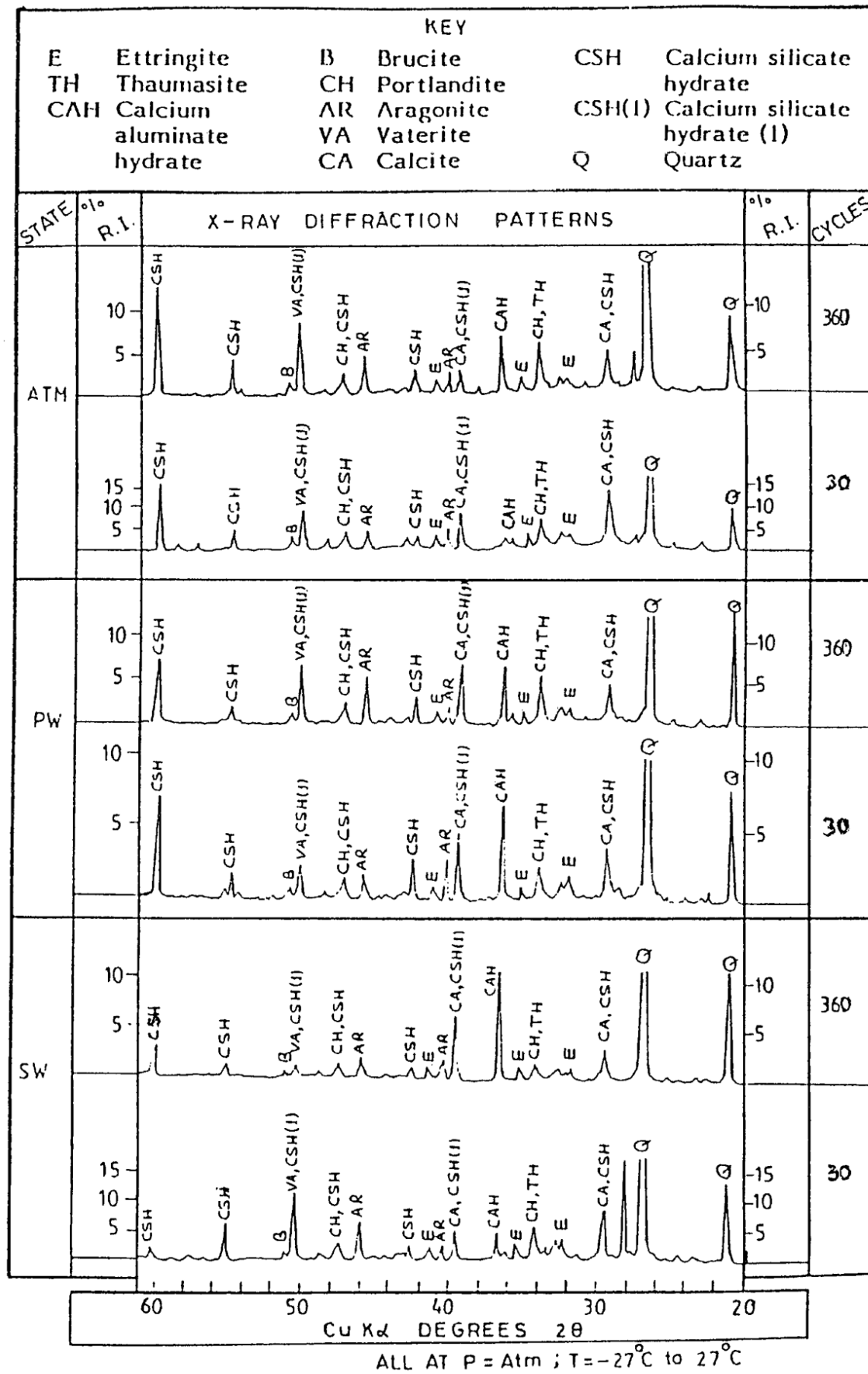


Figure 8: X-ray diffractograms of concrete specimens after exposure to SW, PW and ATM state in freeze-thaw environment

the specimens get saturated considerably by sea water and after crystallisation of the salts together with their reaction with cementitious products within the body of concrete results in a significant decrease in the compressive strength.

The characteristic stress-strain curves of concretes A and B after 30 and 360 cycles in different environment states have been drawn in Figures 6(a), 6(b), 7(a) and 7(b) respectively. It may be seen from these figures that the general shape of the stress-strain curves has not changed and except for the ultimate strength value σ_0 , it is similar to that of normal plain water cured concretes in all cases. Although, there is a substantial decrease in the ultimate strength σ_0 in the sea water environment, the strain ϵ_0 corresponding to the ultimate strength σ_0 is of the order of 0.22 percent which is similar to normal concrete.

X-ray diffractograms of the concrete specimens exposed to different environmental states after 30 and 360 cycles of freezing-and-thawing have been shown in Figure 8. The maximum intensities and the approximate percentages of the chemical compounds detected from Figure 8 have been calculated and critically studied. It is seen from these diffractograms that in all environments viz. sea water, plain water or ATM state, same cementitious compounds of the non-expansive group such as brucite (B) as well as of the expansive group such as ettringite (E), thaumasite (TH) and calcium aluminate hydrate (CAH) are formed but their percent contents are different. As the number of cycles of freezing-and-thawing increases, the percentages of non expansive compounds are found to decrease, whereas the amounts of the expansive compounds like ettringites (E), thaumasite (TH), and calcium aluminate hydrate (CAH) increase. It results in decreasing the concrete strength after the formation of microcracks due to the presence of such expansive/light density compounds.

5.0 CONCLUSION

The results of the investigation carried out on two grades of high strength concrete A and B exposed to SW, PW and ATM states over 360 nos of alternate freezing and thawing have been critically analysed and interpreted. Based on the limited number of tests and variables studied over specific periods, the following conclusion can be drawn:

- (a) Concrete exposed to Freeze-Thaw cyclic loading in sea water is much more vulnerable to deterioration including erosion, splitting and crumbling than in plain water and in atmospheric state of exposure.
- (b) Due to substantial erosion and splitting of concrete specimens in the Freeze-Thaw environment, the loss in weight is found to the extent of 11.5 to 16.5% in sea water and the corresponding loss is around 2% for the specimens placed in plain water or in the ATM state.
- (c) Concrete also shows a significant decrease in volume as much as 1.7% in sea water under freeze-thaw environment. Concrete specimens exposed to plain water and in dry (ATM) state show a decrease in volume of around 0.04 percent.
- (d) A significant change in permeability (k value) characteristics of concrete in Freeze-Thaw environment is observed particularly when it is exposed to sea water. The k value of concrete in sea water is found to be 5 to 7 times the value obtained in plain water or in ATM state exposure and nearly 8 times the permeability of similar concrete cured in plain water at 27°C.
- (e) Sea water causes the most detrimental effect on the compressive strength of concrete, the loss being in the range of 70 to 75% after 360 cycles of freezing and thawing as compared to the strength of normal plain water cured concrete at similar age. The compressive strength loss of concrete under freeze-thaw action in plain water and in the ATM state is around 8% and 3% as compared to the 28 day compressive strength of normally cured concrete.
- (f) X-ray diffraction studies confirm the formation of relatively higher percentage of expansive products in concrete exposed to sea water which are also responsible for strength deterioration and increased permeability.
- (g) The stress-strain characteristics of concrete is seen to be unaffected by alternate Freeze-Thaw action of sea water. ■

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