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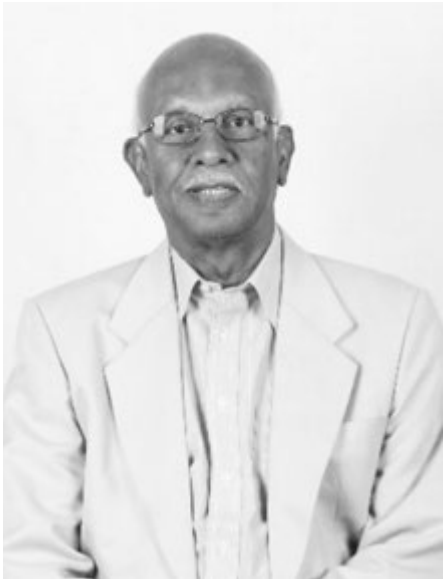
THE TWENTY FIFTH
PROFESSOR CHIN FUNG KEE MEMORIAL LECTURE

Presented at the Auditorium Tan Sri Prof. Chin Fung Kee,
Wisma IEM, Jalan Selangor, 46200 Petaling Jaya, Selangor, Malaysia
on 19th September 2015



Academician Tan Sri Dato' Ir. Shahrizaila Abdullah
BE (Mal), PEng, FIEM, FASc.

Academician Tan Sri Dato' Ir. Shahrizaila Abdullah is a Senior Fellow of the Academy of Sciences, Malaysia. He initiated and led the Sustainable Water Management Programme at the Academy from 2006 until mid-2013. His long career both in the public and private sector has been mainly in the water sector. After his graduation with a Bachelor of Engineering degree from the University of Malaya in the year 1960, he served the Government of Malaysia for over 30 years and the last position held prior to his retirement in 1995 was as Director-General, Department of Irrigation and Drainage, Ministry of Agriculture, Malaysia. Following his retirement from public service, he had an 8-year spell in the private sector, serving as Chairman and Specialist Consultant with KTA Tenaga Sdn. Bhd., a multi-disciplinary engineering consultancy firm based in Kuala Lumpur.

Internationally, he has been active with the International Commission on Irrigation and Drainage (ICID), serving as its Vice-President from 1989-90, and later as its President from 1993-1996. He served as a member on the Board of Governors of the International Water Management Institute (IWMI) for five years from 1996 to 2001. He was a member of the World Commission on Water for the 21st Century which presented its World Water Vision report at the Second World Water Forum in March 2000 at The Hague, Netherlands. He served as a member of the Southeast Asia Technical Advisory Committee of the Global Water Partnership since its establishment in 1997 until December 2002. He was responsible for initiating the formation of the Malaysian Water Partnership in November 1997. He has also served as a member of the Board of Trustees for WWF Malaysia and was its Chairman for 2 years from March 2006 to March 2008.

Ensuring A Better Water Future for Malaysia

(Date received: 02.11.16/Date accepted: 6/1/2017)

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ABSTRACT

Human overuse of water resources and diffuse contamination of freshwater are stressing the water resources in the terrestrial water cycle. As a consequence, the ecological functions of water bodies, soils and groundwater in the water cycle are hampered and being further exacerbated by threats from impending climate change. Though Malaysia is blessed with fairly abundant rainfall it still has its fair share of water woes, such as occasional droughts, flooding and pollution of its rivers and water bodies. Only recently, the country was faced with water related hazards of fairly disastrous proportions. Recurring potable water shortages that occurred in 2014 and 2015 in several states had led to water rationing. Malaysia has since the early 1990s set its vision to become a fully developed country by the year 2020 (Vision 2020). The transformation of the water sector must also evolve in tandem to meet sustainable development goals befitting a developed nation status by 2020. The National Water Resources Policy (NWRP), launched in March 2012, is based on the 3 essential principles i.e. water resources security, water resources sustainability and collaborative governance. This paper will review the various water-related issues and challenges whilst proposing the implementation of the Integrated Water Resources Management (IWRM) Road Map including the measures to be undertaken to effect the transformation of the water sector in pursuit of Vision 2020 and to achieve the post 2015 Sustainable Development Goals (SDGs). The findings and recommendations are largely based on several in-depth studies undertaken by the Academy of Sciences, Malaysia (ASM) pertaining to the water sector and the IWRM agenda.

1.0 INTRODUCTION

Human overuse of water resources and diffuse contamination of freshwater are stressing the water resources in the terrestrial water cycle. As a consequence, the ecological functions of water bodies, soils and groundwater in the water cycle are hampered and being further exacerbated by threats from impending climate change. The water crisis today is a crisis of managing water. A holistic, systemic approach relying on Integrated Water Resources Management (IWRM) must replace the current fragmented approach.

Over the last 2 decades, the UN-led Earth Summits have brought nations together to commit themselves to the Sustainable Development Agenda 21. One of the main outcomes of the United Nations Conference on Sustain-able Development (Rio+20), held in Rio de Janeiro in June 2012 and carrying the theme **"The Future We Want"**, was the agreement by Member States to launch an inclusive process to develop a set of Sustainable Development Goals (SDGs) that would address in a balanced way all three dimensions of sustainable development – economic, social and environment - and be coherent with and integrated into the UN development agenda beyond 2015.

Meanwhile, the World Water Council (WWC) led World Water Forum held triennially beginning 1997, has gathered the global water-related stakeholders and communities to chart the course for the integrated management of water resources (IWRM) for a water secure world, an essential prerequisite to achieve the Sustainable Development Agenda. The recent 7th World Water Forum held in Korea in April 2015 has accordingly carried the theme **"Water for the Future"** focusing on 3 primary objectives, namely, (a) Moving from Solutions to Implementation, (b) Bridging the Platform of Science & Technology to Water Issues, and (c) Contributing to the Sustainable Development Goals (SDGs).

Malaysia has since the early 1990s set its Vision to become a fully developed country by the year 2020. It is currently pursuing a Transformation Programme to progress from a middle income nation to become a high-income nation by 2020. The transformation of the water sector must also evolve in tandem to meet sustainable development goals befitting a developed nation status by 2020. As a signatory to most of the international agreements and conventions pertaining to water and the environment, Malaysia has formally adopted IWRM as the way forward for the sustainable management of the country's water resources.

Against the above back-drop, this paper entitled **"Ensuring a Better Water Future for Malaysia"** will review the various water-related issues and challenges and proposing the implementation of the IWRM Road Map including the measures to be undertaken to effect the transformation of the water sector in pursuit of Vision 2020 and to achieve the post-2015 SDGs.

2.0 GLOBAL WATER TODAY

The UN World Water Development Report 2015 entitled "Water for a Sustainable World" released in March 2015 on the occasion of World Water Day reported the following facts and figures on the current world water situation and its outlook for the future:

- The world's population is growing by about 80 million people per year (USCB, 2012) and is predicted to reach 9.1 billion by 2050.
- Population growth, urbanization, industrialization, and increases in production and consumption have all generated ever increasing demands for freshwater resources.

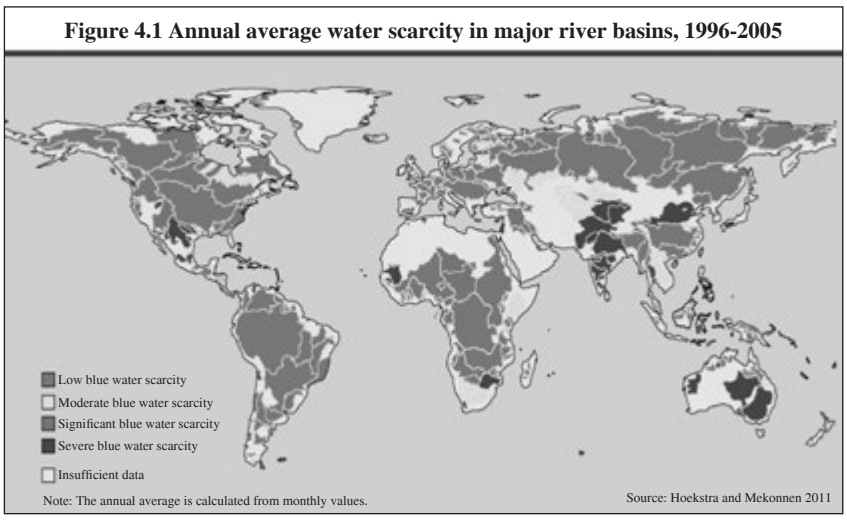

Figure 1: Annual Average Scarcity in Major River Basins.

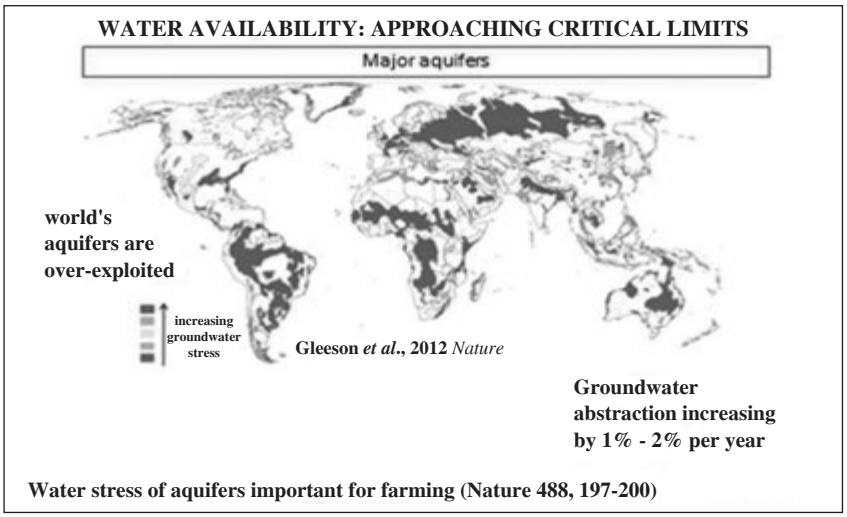

Figure 2: Aquifer Water Stress.

- By 2030, the world is projected to face a 40% global water deficit under the business-as-usual climate scenario (2030 WRG, 2009).
- Water availability faces pressures from pollution. Eutrophication of surface water and coastal zones is expected to increase almost everywhere until 2030 (UNDESA, 2012). Globally, the number of lakes with harmful algal blooms will increase by at least 20% until 2050 (Figure 1).
- Regionally, the global limit of ecological sustainability of water available for abstraction is reported to have been exceeded for about one-third of the human population and it will rise to about half by 2030 (WWAP, 2012).
- Groundwater provides drinking water to at least 50% of the global population and accounts for 43% of all of the water used for irrigation (FAO, 2010). Worldwide, 2.5 billion people depend solely on groundwater resources to satisfy their basic daily water needs (UNESCO, 2012). An estimated 20% of the world's aquifers is being over-exploited (Gleeson *et al*., 2012), leading to serious consequences such as land subsidence and saltwater intrusion (USGS, 2013) (Figure 2).
- Economic losses due to water-related hazards have risen greatly in the past decade. Since 1992, floods, droughts and storms have affected 4.2 billion people (95% of all people affected by all disasters) and caused US$1.3 trillion of damage (63% of all damage) (UNISDR, 2012).
- In most countries, funding for water infrastructure comes from government allocations, although many developing countries still depend on external assistance to fund water resources management and utilities. Over 50% of countries low on the Human Development Index have reported that financing for water resources development and management from government budgets and official development assistance has been increasing over the past 20 years (UN-Water, 2012).
- Challenges such as economic shocks, food shortages and climate change threaten to undercut economic and social progress made in recent years. The fact is there is enough water to meet the world's growing needs, but not without dramatically changing the way water is used, managed and shared. The global water crisis is one of governance, much more than of resource availability.

3.0 THE MALAYSIAN WATER SENARIO

The National Water Resources Study (NWRS 2011) commissioned by the Ministry of Natural Resources and Environment (NRE) and completed in the year 2011 provides a comprehensive assessment of the current water situation in Malaysia both at the national level and also broken down according to the various States. The Study has also undertaken projections of the state of the country's waters until the year 2050 addressing issues related to supply and demand as well as measures required to ensure the sustainable management of water resources and the protection of the environment.

In brief the Study reports that Malaysia's annual rainfall is around 973 billion cubic metres (BCM), of which 414 BCM is lost to the atmosphere as evapotranspiration, surface runoff amounts to 496 BCM and some 63 BCM contributes towards groundwater recharge. Consumptive demand was assessed to be 14.8 BCM in 2010 and predicted to rise to 17.2 BCM in 2020 and to 18.2 BCM in 2050. With total effective rainfall estimated at around 74 BCM sets the available resource some 4 times above the projected need. Hence, the adequate provision of quality water to meet the country's short, medium and long term needs is not one of water resources availability but more of sound management and good governance.

Relevant supporting tables and figures from the Study report related to population projections (Table 1), projected GDP (Table 2), annual rainfall data by States (Table 3), water demand projections (Figure 3) and comparison against surface water availability (Table 4) are shown.

Some of the major water related issues and challenges that need to be addressed nationwide, most of which have also been highlighted by the NWRS 2011, are as follows:

Table 1: Malaysia Population Projections.

STATES	POPULATION ('000)				
	2010	2020	2030	2040	2050
Perlis	246	291	319	343	361
Kedah	2,043	2,440	2,695	2,905	3,065
Pulau Pinang	1,609	1,841	1,958	2,064	2,133
Perak	2,441	2,810	3,004	3,177	3,294
Selangor and FT Kuala Lumpur	6,970	7,951	8,443	8,896	9,195
Negeri Sembilan	1,032	1,190	1,274	1,348	1,399
Melaka	785	925	1,008	1,078	1,129
Johor	3,458	4,117	4,533	4,879	5,140
Pahang	1,573	1,867	2,050	2,203	2,317
Terengganu	1,149	1,445	1,672	1,854	2,006
Kelantan	1,677	2,104	2,427	2,686	2,901
Peninsular Malaysia	**22,983**	**26,981**	**29,383**	**31,434**	**32, 940**
Sarawak	2,660	3,127	3,505	3,839	4,117
Sabah	3,267	3,874	4,400	4,719	4,958
FT Labuan	88	101	110	115	118
East Malaysia	**6,015**	**7,102**	**8,015**	**8,673**	**9,193**
Malaysia	**28,998**	**34,083**	**37,398**	**40,107**	**42,133**

Table 2: Projected GDP.

YEAR	VALUE IN RM MILLION								AAGR (%) 2010-2050	AAGR (%) 2010-2050
	2010	2020	2025	2030	2035	2040	2045	2050		
Agriculture	40	53	70	93	118	120	146	176	3.8	4.1
Mining	42	47	41	54	69	75	91	110	2.4	2.9
Manufacturing	139	245	308	410	521	662	802	971	5	4.7
Construction	17	24	34	45	57	66	80	97	4.5	4.8
Services	317	682	948	1,263	1,604	2,085	2,525	3,058	5.9	5.1
GDP Total	**555**	**1,051**	**1,400**	**1,865**	**2,369**	**3,009**	**3,644**	**4,412**	**5.3**	**4.9**

Table 3: Annual Rainfall Data by States.

STATE	AREA	Unit in Billion Cu M per year				
	(sq km)	Rainfall	Actual Evaporation	Groundwater Recharge	Surface Runoff	Effective Rainfall
Perlis	821	1.54	1.06	0.10	0.38	0.06
Kedah	9,500	21.95	13.59	1.24	7.12	1.07
P. Pinang	1,048	2.46	1.50	0.13	0.83	0.13
Perak	21,035	52.17	27.77	3.58	20.82	3.14
Selangor	8,396	18.39	10.75	1.26	6.38	0.96
Negeri Sembilan	6,686	12.24	8.09	0.87	3.28	0.64
Melaka	1,664	3.13	2.01	0.17	0.95	0.14
Johor	19,210	47.45	21.71	3.84	21.90	3.29
Pahang	36,137	89.26	45.17	4.34	39.75	6.46
Terengganu	13,035	43.15	19.16	1.96	22.03	3.31
Kelantan	15,099	39.26	19.48	2.11	17.67	2.65
Pen. Malaysia	**132,631**	**330.98**	**170.28**	**19.56**	**141.11**	**21.17**
Sabah	73,631	188.50	87.62	13.99	86.89	16.21
Sarawak	124,450	453.00	155.56	29.87	267.57	27.44
FT Labuan	91	0.28	0.13	0.01	0.14	0.03
East Malaysia	**198,172**	**641.78**	**243.31**	**43.87**	**354.60**	**53.19**
Malaysia	**330,803**	**972.78**	**413.60**	**63.45**	**495.71**	**74.35**

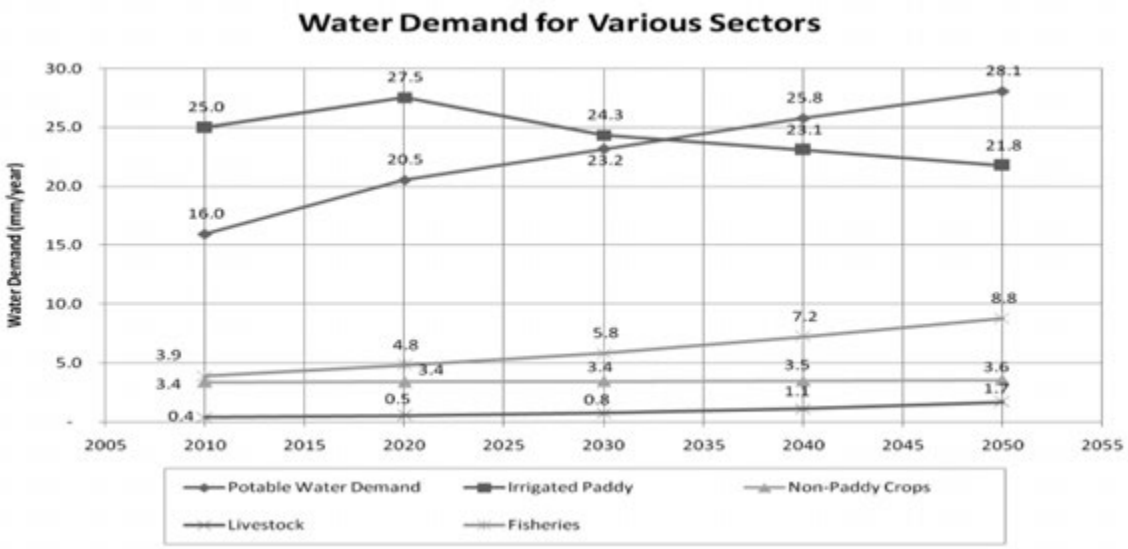

Figure 3: Consumptive Water Demand Projections (BCM/Year).

Table 4: Total Consumptive Water Demand Against Total Surface Water Availability for All Sectors.

STATES	LAND AREA (sq km)	TOTAL CONSUMPTIVE WATER DEMAND (MCM)					EFFECTIVE RAIN	EXCESS/DEFICIT (MCM) - UNREGULATED FLOWS				
		2010	2020	2030	2040	2050	(MCM/ YEAR)	2010	2020	2030	2040	2050
Perlis	821	306	299	286	284	281	60	(246)	(239)	(226)	(224)	(221)
Kedah	9,500	2,922	2,976	2,842	2,873	2,876	1,070	(1,852)	(1,906)	(1,772)	(1,803)	(1,806)
Pulau Pinang	1,048	765	829	835	874	894	130	(635)	(699)	(705)	(744)	(764)
Kelantan	15,099	1,632	1,619	1,586	1,600	1,604	2,650	1,018	1,031	1,064	1,050	1,046
Terengganu	13,035	884	975	970	999	1,026	3,310	2,426	2,335	2,340	2,311	2,284
Perak	21,035	1,949	1,923	1,798	1,801	1,811	3,140	1,191	1,217	1,342	1,339	1,329
Selangor	8,396	2,238	2,491	2,570	2,760	2,922	960	(1,278)	(1,531)	(1,670)	(1,800)	(1,962)
Pahang	36,137	726	946	897	911	959	6,460	5,739	5,514	5,563	5,549	5,501
Negeri Sembilan	6,686	340	361	358	366	374	640	300	279	282	274	266
Melaka	1,664	323	366	376	409	439	140	(183)	(226)	(336)	(269)	(299)
Johor	19,210	715	881	1,033	1,164	1,301	3,290	2,575	2,409	2,257	2,126	1,989
Pen. Malaysia	132,631	12,800	13,664	13,551	14,040	14,488	21,170	8,370	7,506	7,619	7,130	6,682
Sabah	73,631	912	1,356	1,392	1,442	1,469	16,210	15,298	14,854	14,818	14,768	14,741
Sarawak	124,450	1,054	2,162	2,125	2,175	2,247	27,440	26,386	25,278	25,375	25,265	15,193
W.P. Labuan	91	18	24	26	28	29	30	12	6	4	2	1
East Malaysia	198,172	1,985	3,541	3,542	3,645	3,745	53,190	51,205	49,649	49,648	49,545	49,445
Total Malaysia	330,803	14,785	17,205	17,093	17,685	18,233	74,350	59,565	57,145	57,257	56,665	56,117

Source: NWRS 2011

a. Regional Water Stress: Based on the current and projected consumptive water demand against total surface water availability shown in the Table 4 above, the NWRS 2011 highlighted some 'water-stressed' growth regions and states such as Perlis, Kedah, Pulau Pinang, Selangor and Melaka were reported as water deficit states (computed values shown in red in the Table 4 above). This has been borne out by recurring crises of potable water shortages that occurred recently in 2014 and 2015 in several states which had led to water rationing. The affected states were Selangor, Negeri Sembilan, Johor, Perak and Wilayah Persekutuan (Kuala Lumpur and Putrajaya).

Temporal and spatial variability of rainfall, coupled with high population densities and/or extensive agricultural activities in these regions have led to water demands exceeding the carrying capacity of the respective river basins. The situation has been further exacerbated by resultant pollution affecting the ecology and the functional capacity of the aquatic ecosystems. Measures taken in the past to augment supplies have largely been through shared river basins and inter-basin water transfers. The sharing of the Klang, Langat and Selangor rivers and supplemented further with waters transferred from Pahang (Langat 2) to service the densely populated greater Klang Valley is a case in point. Similarly, the high water demand to support the granary areas of MADA and Seberang Perai is serviced by regulated waters drawn from the Muda River catchment (falling largely in the state of Kedah) and shared among the states of Perlis, Kedah and Pulau Pinang.

b. Flooding: Located in the humid tropics, Malaysia is subject to seasonal torrential rains brought by the south-west and north-east monsoons with the year-end north-east monsoon normally being the more severe one. There are 189 river systems in Malaysia of which 85 are prone to frequent flooding. Despite the many flood mitigation measures undertaken over past years, recent trends indicate that the magnitude of flooding is on the rise and hence has become a major national issue. Following the rapid pace of Malaysia's economic growth coupled with pressures from an increasing population, development has inevitably encroached into catchment areas, river corridors and flood plains which have led to greater incidences of floods and ensuing damages. An estimated 29,720 sq.km or 9% of Malaysia is flood prone and the annual flood damages in Malaysia is approaching closer to RM2 to 3 billion in recent years.

The recent floods in December 2014 and January 2015 in Kelantan, Terengganu, Perak, Pahang, Johor, Sabah and Sarawak resulted in devastating damages forcing about 400,000 people to be evacuated. DID rainfall records showed that for the upper reaches of Sg. Kelantan, Sg. Pahang and Sg. Perak the rainfall exceeded the 100-year return period. A special Parliamentary Session was convened on 20th January 2015 to approve a special budget allocation of RM893 million for flood mitigation works, RM800 million as initial allocation to repair and reconstruct basic infrastructure like schools, hospitals, roads and bridges, RM500 million rehabilitation works and welfare programmes and RM500 million for flood relief loan Guarantee Scheme. The other contributing factors were reported to be extensive land-clearing in the highlands and encroachment into the flood corridors.

Severe land-slides and mud flows in Kampung Raja, Pekan Ringlet and Bertam Valley in the resort area of Cameron Highlands in November 2014 killed 5 people and affected 100 victims from 28 families. A similar event occurred in 2013. The main causes for these recurring flood events have been attributed to the use of rain shelters made out of plastic roofing materials for extensive vegetable farming in the highlands resulting in increased surface water runoff being directly discharged into rivers triggering flash floods. The flash floods have been further aggravated over the years by the uncontrolled opening of forest lands for illegal vegetable farming and also due to the lack of enforcement by the local authorities.

c. Pollution of Water Sources: Pollution of water bodies, be they lentic or lotic systems, has evidently been on the rise nationwide. A 2004 report entitled A Study on the Status of Eutrophication of Lakes in Malaysia, confirmed that out of the 90 lakes that were studied, 56 (62%) were in a poor condition (eutrophic), while the balance were in a mediocre to reasonably good state (mesothrophic). The study went on to conclude that eutrophication of lakes has reached levels for serious concern and restoration efforts were urgently needed for many lakes.

The Department of Environment (DOE) under the NRE Ministry is responsible for enforcing the Environmental Quality Act (EQA, 1974). The Act was enacted for the abatement and control of pollution and enhancement of the environment, which includes river water quality. According to the DOE, Malaysian rivers are degraded by both point and non point sources of pollution. The major point sources of pollution in rivers are from sewage treatment plants, agro based industries, manufacturing industries, sullage or grey water from commercial and residential premises, and pig farms. Nonpoint source (or diffuse) pollution is largely due to storm runoff after a downpour. Earthworks and land clearing activities contribute to siltation of rivers and can be both point and non-point sources of pollution. The many recent shut-down of the WTP in the Sg. Langat area were reported to be attributed to pollution caused by factories upstream.

d. Environmental Degradation: Decades of economic development comprising large-scale land development, urbanization, and industrialization coupled with efforts to meet the needs and provide opportunities for a rising population, has significantly changed both the urban and rural landscape. Such change has inevitably had its toll in the continuing degradation of both the terrestrial and aquatic ecosystems. Ecosystem services are the benefits people obtain from ecosystems (Figure 4, Millennium Ecosystem Assessment (2005). These include provisioning, regulating, and cultural services that directly affect people and supporting services needed to maintain the other services.

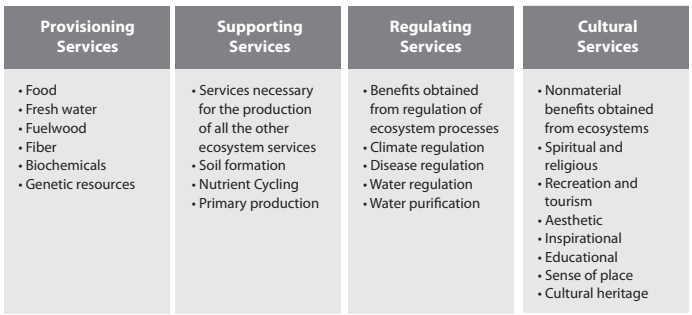

Figure 4: Ecosystem Services (Millennium Ecosystem Assessment 2005).

Based on the country's track record to-date and as reported in NWRS (2011), one can reasonably conclude that in exploiting the ecosystem provisioning service functions for multiple purpose use and development, the ecosystem regulating service function has largely been impaired resulting in depleted and degraded stream flows threatening loss of biodiversity in both the terrestrial and aquatic environment and particularly so in the more developed river basins.

e. Fragmented Management and Conflicts among Sectors: Under the Constitution, matters pertaining to natural resources such as land, minerals, forests and water fall under the jurisdiction of the states. Water becomes a federal matter only if a dispute arises as in the case of a shared river basin between two or more states. Otherwise, State Governments are responsible for water management including the gazettement of water catchments. Currently only five states have the equivalent of a state water resources council backed by appropriate legislation to oversee water resources governance in their respective states. They are Selangor (Selangor Water Management Authority); Kedah (Kedah Water Management Board); Pahang (State Water Regulatory Body); Sabah (Sabah Water Resources Council) and Sarawak (Sarawak Water Resources Council).

At the Federal level, the governance and administration of water resources involve several ministries, departments and agencies. Water resources development is sectorally based, a legacy from the past. Since the year 2004, however, the creation the NRE Ministry saw the clear separation of powers between "water as a resource" and "water for utilities". Management of water as a resource is vested with the NRE Ministry which also includes the management of water-related hazards such as floods and droughts. Water for agriculture comes under the purview of the Ministry of Agriculture, potable water supply, sewerage services, and hydropower generation falls under the Ministry of Energy, Green Technology and Water (KeTTHA). The Ministry of Health deals with water supply and sanitation in areas not covered by KeTTHA. Urban drainage and storm water retention in urban areas is under the Ministry of Housing and Local Government. Water legislations are contained within the laws that are enforced by the various water related government agencies and are focused on specific aspects of water resources that are under the jurisdiction of the respective agencies. There are gaps and overlaps. Conflicts in water resources management such as allocation of water rights, flood management, pollution control and environmental protection are resolved through inter agency coordination and consultation. In order to resolve persistent disagreements, the Federal Government established a National Water Resources Council (NWRC) in June 1998 with the foremost intention to pursue a more effective and cohesive water management by various States that includes the initiation of inter state water transfers. The NWRC as the apex advisory and coordinating body for water resources governance was entrusted to formulate, among others, a national water policy as well as establish guidelines to ensure long-term sustainable development and management of the country's water resources. The National Water Resources Policy has since been formulated and officially launched in March 2012.

f. Climate Change Impacts: Exposure of people and assets to hydro-meteorological hazards in Asia Pacific, including Malaysia, has been growing over the past few decades. Malaysia has seen rapid urbanization, economic growth, and changes in local environmental conditions whereby more assets and people are located in hazardous areas such as flood plains and coastal low-land areas. The country has of late experienced extended droughts and widespread flooding and expected to continue to be more exposed and vulnerable to such natural hazards. Climate change is anticipated to create extreme events, with some projections including an increase in the frequency of years with above normal monsoon rainfall or extremely rainfall deficient.

An increase in rainfall extremes of landfall cyclones in South and East Asia have been recently projected in the IPCC's AR5 (Fifth Assessment Report), along with enhanced monsoon precipitation and increased drought in some areas over the long term. Consequential impacts from two likely scenarios are listed in Table 5:

The impacts of climate change on the water resources characteristics cannot be avoided as it is a global phenomenon. Nevertheless, its negative impacts could be mitigated with the following general measures (NWRS 2011):-

Table 5: Climate Change and its Impact on Water Resources.

SCENARIO 1: INCREASES IN TEMPERATURE AND INCREASE IN RAINFALL	**SCENARIO 2:** INCREASES IN TEMPERATURE AND REDUCED RAINFALL
Increased inflow to water storages	Reduced inflows to water storages (dams and reservoirs)
Increased pressure on water storage infrastructure	Reduced stream flows
Increased availability of water for rain-fed agriculture	Reduced water availability for rain-fed agriculture
Increase risk of flood damage	Reduced recharge of groundwater
Possible changes to ecosytems	Threatened water supplies to cities and towns, agricultural, industrial and environmental needs
	Severe droughts

i. Construction of more storage dams to capture the higher flows. As the run of the river flows will be lower, larger release from existing dams would be necessary to enable water supply to meet demands and to maintain system reliability. For states already experiencing water stresses, inter-basin and interstate water transfer would be necessary.

ii. Efficient water supply and demand management in the areas of:
- Increasing irrigation efficiency as irrigation is currently still the largest water user and the irrigation efficiency is relatively low.
- Irrigated paddy where water savings measure in the form of farm practices and the introduction of paddy strains with lesser water demand.
- Reduce non-revenue water and other wastages.
- Potable water demand where savings in the form of lowering per capita domestic consumption and the widespread practice of water recycling and the use of alternative water resources. Prudent land-use planning for new developments in anticipation of sea level rise and raising of coastal bunds to protect existing development areas.

4.0 ENSURING A BETTER WATER FUTURE FOR MALAYSIA

a. Malaysia's Vision 2020 and the National Transfomation Programme: Since the early 1990s Malaysia had embarked on Vision 2020 to attain developed nation status by the year 2020. In renewed efforts for timely achievement of this goal and to help fast track the process, the country has, since the year 2010, launched the National Transformation Program comprising of both a Government Transformation Program (GTP) and an Economic Transformation Program (ETP) which was followed by the Political Transformation Programme (PTP), the Community Transformation Programme (CTP), Social Transformation Programme (STP), and the Fiscal Transformation Programme (FTP).

The GTP is a broad based programme of change to fundamentally transform the Government into an efficient and people-centred institution. It focuses on seven pressure points, designated as National Key Results Areas (NKRAs) to improve the socio-economic growth of the country.

The ETP was launched in September 2010 with its goal is to elevate the country to a developed nation status by 2020, targeting a gross national income (GNI) per capita of US$15,000. To achieve this, US$444 billion in investments is targeted, which will create 3.3million new jobs.

The ETP's targets for 2020 will be achieved through the implementation of 12 National Key Economic Areas (NKEAs) representing economic sectors which account for significant contributions to GNI and job creation. The ETP is also centred on raising Malaysia's competitiveness through the implementation of six Strategic Reform Initiatives (SRIs), comprising policies which aim to strengthen the country's commercial environment to ensure Malaysian companies are globally competitive.

The 12 NKEAs are as follows (Table 6). Each NKEA has Entry Point Projects (EPPs), which explore new growth areas, and business opportunities (BOs), to enable the sector to move further up the value chain.

Table 6: National Key Economic Areas (NKEAs).

1. Oil, Gas and Energy.	7. Wholesale and Retail.
2. Palm Oil and Rubber.	8. Education.
3. Financial Services.	9. Healthcare.
4. Tourism.	10. Communications Content and Infrastructure.
5. Business Services.	11. Agriculture.
6. Electronics and Electrical.	12. Greater Kuala Lumpur/ Klang Valley.

b. The Malaysian Water Vision and Framework for Action: The **Malaysian Water Vision** formulated in the year 2000 is as reproduced below:

"In support of Vision 2020 (towards achieving developed nation status), Malaysia will conserve and manage its water resources to ensure adequate and safe water for all (including the environment)".

The key objectives of the Malaysian Water Vision are:

i. **Water for people –** all communities will have access to safe, adequate and affordable water supply, hygiene and sanitation.

ii. **Water for food, agriculture and rural development –** provisions of sufficient water to ensure national food security and promote rural development.

iii. **Water for economic development –** provisions for sufficient water to spur and sustain economic growth within the context of a knowledge-based economy and e-commerce.

iv. **Water for the environment –** protection of the water environment to preserve water resources (both surface and groundwater resources) and the natural flow regimes, biodiversity and cultural heritage as well as the mitigation of water related hazards; and

v. **Water for Energy –** this has been added in to reflect the current trends on the water-energy-food nexus, looking at it in terms of both policy and process, as water is inextricably linked to agriculture, food production and where there is an urgent need for continuous improvements in water and energy efficiencies to ensure sustainable economic growth.

To complement the Vision statement, a National Framework for Action was also developed structured to achieve the key objectives of the Vision, entailing:

i. Managing water and water resources efficiently and effectively (addressing both quantity and quality aspects) as water demands increase in tandem with population growth and industrialisation;

ii. Moving forward towards IRBM and ILBM taking full cognizance of river and lake basins as geographical units with well-defined boundaries containing the sum of all hydrological processes operating within them, and transcending political and administrative constraints, making them ideal water management units to address water problems;

iii. Translating awareness to political will and capacities to create an enabling environment for the much needed institutional reforms to deal with deterioration of water quality, decrease in water availability and conflicts among users (irrigation, hydropower, industry and domestic users). There is also a need to instil awareness of the economic, social and environmental values of water among politicians, decision makers and all stakeholders; and

iv. Moving towards adequate (safe) and affordable water services (befitting a developed nation status by 2020) through the provisions of adequate infrastructure for water delivery to all sectors of the economy.

c. National Water Resources Policy (NWRP 2012): This contemporary policy was formally launched in March 2012. The Policy is based on the 3 essential principles of water resources security, water resources sustainability and collaborative governance, as elaborated briefly below:

i. **<u>Water Resource Security:</u>** Water security, similar to food and energy securities in the country, is to ensure that water is readily available to meet all demands of society and the environment. It has an intrinsic as well as a financial cost value that could be much higher than those of other economic sectors.

ii. **<u>Water Resource Sustainability:</u>** Water is a catalyst for national development and for societal and environmental well-being. It should be sustained for present and future uses. This opens up vast opportunities to develop the water industry and to explore the use of alternative water sources through science, technology and investments.

iii. Collaborative Governance: Inclusiveness and collaboration are essential elements towards ensuring the security and sustainability of water resources as well as achieving the common goals of addressing multiple resource use, governance and priorities.

d. Integrated Water Resources Management (IWRM): As highlighted in section 3 above, Malaysia is blessed with fairly abundant rainfall. The adequate provision of quality water to meet the country's short, medium and long term

needs is not one of water resources availability but more of sound management and good governance. The NWRP 2012 reaffirms Malaysia's commitment since the turn of this century to adopt internationally endorsed IWRM for the sustainable management of the country's water resources. This is a clear break away from past fragmented management practices. IWRM calls for the balanced development and management of "water as a resource" and "water for livelihood". Implementation of the IWRM agenda involves the integration of both natural and human systems (Figure 5).

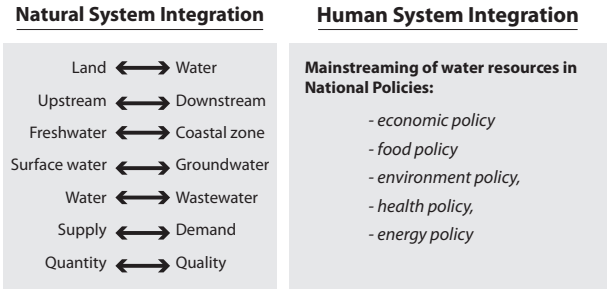

Figure 5: The IWRM Agenda Involves the Integration of Both Natural and Human Systems.

The general framework for the implementation of IWRM is based on four practical elements:

1. an enabling environment comprising policies, laws and plans;
2. an institutional framework;
3. use of management and technical instruments; and
4. investments in water infrastructure.

e. Academy of Sciences (ASM) and the National IWRM agenda: The Academy of Sciences Malaysia (ASM), an independent think-tank providing strategic advice to Government on STI matters, had taken the initiative since 2008 to focus on the water sector recognizing it as being one of the strategic sectors vital for the country's economic development. Adopting IWRM as the central thrust and noting the absence of a nation-wide IWRM Work Plan, the ASM through its dedicated Water Committee set about developing a National IWRM Strategy Plan for consideration and adoption by Government for implementation nationwide. Noting also that IWRM concept per se is rather abstract; there is a need to break it down into discrete sub-sets or sub-themes. Each sub-set or sub-theme is subject to in-depth multidisciplinary studies that include a review of the current status followed by a process of stakeholder consultations involving the public, private, and NGO sectors to formulate recommendations and strategies to chart the way forward for improved and sustainable management of the country's water resources through the infusion of IWRM principles and practices. The studies undertaken and their current status are as follows:

i. Integrated Lake Basin Management (completed)
ii. Integrated Aquifer Systems Management (completed)
iii. Water Demand Management (completed)
iv. Water Supply and Wastewater Management (completed)
v. National Agenda for Integrated Water Research (completed)
vi. Climate Change and Water (completed)
vii. NKPA on Water (completed)
viii. Integrated River Basin Management (completed)
ix. Water and Agriculture (in progress)
x. Integrated Urban Water Management (in progress)
xi. WFE Nexus (in progress)

In addition, ASM under the first phase of the Mega Science Study aimed at a longer term horizon until the year 2050 also addressed the Water Sector. The Study, completed in the year 2009, recommended the inclusion of "water for wealth creation" in addition to sustaining the resource. The water sector must also be regarded as a source for growth of the national economy by way of exploitation of the full potential of income generating value added products and services that can be derived from the sector.

f. Transformation of the Water Sector for a Better Future: The country is on the road towards Vision 2020 anchored by a National Transformation Programme. A vibrant water sector is an integral part of this mission and needs to move in tandem with a host of expectations as highlighted below:

- To ensure the pivotal role of water in economic development and as an integral part of the water-food-energy nexus;
- Implementation of IWRM across all sub-sectors and levels of hierarchy;
- A well structured and regulated water and sanitation industry providing quality and efficient services and rationalised tariff settings with provisions for targeted subsidies;
- Green growth with low water footprint and care for the environment;
- Optimum use of the full range of water resources development options used singly or conjunctively including wise waste-water reuse, treated or otherwise;
- Improved agricultural water management to ensure "more crop per drop";
- More Water Demand Management (WDM) than Supply Management in both potable and agricultural water usage;
- Integrated Urban Water Resources Management (IUWRM) to counter urbanization impacts and the "twin dilemma of cities" (provision of safe, clean water and adequate sanitation)
- Disaster ready
- Climate Change prepared
- Harnessing of Science, Engineering, Technology and Innovations developed through multi-disciplinary R&D programs;
- Achievement of Sustainable Development Goals and Solutions post-2015, and
- Concerted Government support for a vibrant water sector, wealth creation and export of services.

Hence, "business as usual" is no longer an option. The way forward would require concerted efforts for a parallel Transformation of the Water Sector through a wide array of component action plans anchored by a central IWRM Strategies Implementation Road Map. Component plans and programs would be implemented concurrently nationwide and led by the key ministries with entrusted responsibilities be it under water resource management or water utility provision and working closely with the state governments. The three (3) principles of water resource security, water resource sustainability, and collaborative governance laid

down by the NWRP 2012 will be the core rationale behind the transformation process. Under the on-going Economic Transformation Programme (ETP), water underlies all of the 12 NKEAs to varying degrees and scale. Some of the areas like agriculture and energy rely heavily on the availability and harnessing of water for growth and yet water was not explicitly recognised as an NKEA. It is vital that water be placed high on the national agenda and recognised as a National Key Priority Area (NKPA) with a slew of Entry Point Projects (EPPs) implemented to ensure timely transformation of the water sector. While it may now be longer be practicable to aim for a year 2020 transformation target, a more realistic target over three (3) Malaysia Plans by the year 2030 is considered feasible thereby coinciding with the target year set by UN to achieve the currently being finalised global Sustainable Development Goals (SDGs).

National IWRM Strategies Implementation Road Map: The ASM studies and component (sub-theme) plans referred to in section (e) above provide a sound basis for the development and implementation of a holistic and inclusive National IWRM Strategy Plan both for the short, medium and long term. Their relevance within the context of the IWRM agenda is as depicted in Table 7:

Table 7: ASM Studies and the IWRM Agenda.

No.	Component Plan	Relevance to IWRM
1.	Integrated Lake Basin Management	Water as a resource
2.	Integrated Aquifer Systems Management	Water as a resource
3.	Water Demand Management	Water as a resource and for livelihood
4.	Water Supply and Wastewater Management	Water for Livelihood
5.	National Agenda for Integrated Water Research	Water as a Resource and for Livelihood (harnessing STI)
6.	Climate Change and Water	Preparing for impending threats
7.	Integrated River Basin Management	Water as a resource
8.	Water and Agriculture	Water for Livelihood
9.	Integrated Urban Water Management	Water as a resource
10.	NKPA on Water	Investing in water infrastructure
11.	WFE Nexus	Dealing with trans-boundary issues

The component plans have been structured and formatted conforming to a common IWRM general framework (Figure 6) that includes an implementation road map where relevant. The NKPA study undertaken with the complementary objectives of (i) ensuring water security in the country and (ii) for the creation of economic opportunities, has recommended some 15 major programmes (10 under "water as a resource" and 5 under "water for livelihood") that include a total of 72 EPPs that have been identified for implementation. These component plans and programmes make up the National IWRM Strategies Implementation Road Map that would together drive the transformation of the water sector nationwide for a better future befitting a nation vying for developed status.

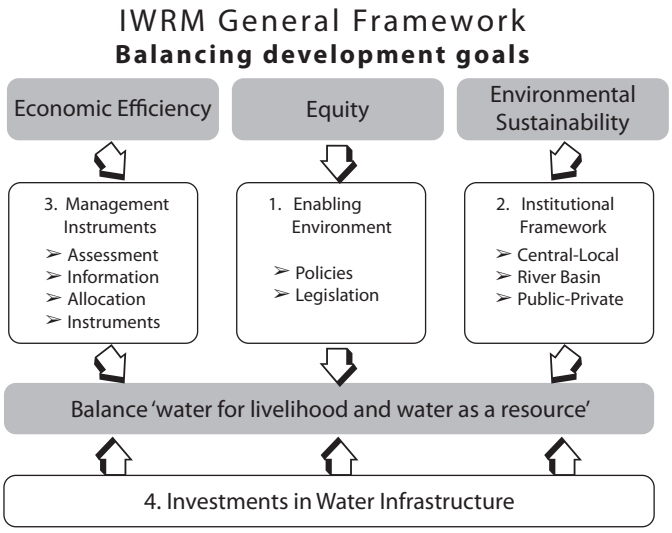

Figure 6: IWRM General Framework.

A broad summary and synthesis of the proposed National IWRM Strategies Implementation Road Map spread over a time frame until 2030 and spanning 3 Malaysia Plans, categorised under the following 4 main headers, are attached as appendices:

1. Enabling environment (Appendix 1);
2. Institutional framework (Appendix 2);
3. Management instruments (Appendix 3); and
4. Investments in water infrastructure (Appendix 4).

5.0 CONCLUDING REMARKS

In his message to the 7th World Water Forum held in Korea in April 2015, Mr. Benedito Braga, the President of the World Water Council said:

"The year 2015 is an opportunity the world cannot afford to miss: we must invent our water future together. Today, we gather in Korea at the 7th World Water Forum; in September, the United Nations General Assembly will adopt Sustainable Development Goals; and we will end the year debating a global climate agreement in Paris, in which water is to be recognised as a major factor to reach consensus. Our ability to build a water secure future will depend upon our capacity to turn future challenges into opportunities. To succeed, we need the political decision makers and the international community to come together to implement changes. There is no time to waste".

Drawing from this message, Malaysia, which is currently in the midst of implementing the National Transformation Programme on its road towards achieving Vision 2020, is well placed to meet the challenges faced by the water sector. The water sector must be accorded the high priority that it deserves on the national agenda and be included as an NKPA, if not an NKEA, together with 12 other NKEAs listed under the Economic Transformation Programme (ETP) so as to ensure that the transformation of this sector progresses in tandem. The proposed National IWRM Strategies Implementation Road Map provides the central basis and thrust that would help mobilise all water-related stakeholders to work concertedly on a common agenda to spur and accelerate the transformation process. Needless to say, it requires strong political commitment both at the Federal and State levels with timely appropriation of the necessary human and financial to ensure success.

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Modelling Infiltration in A Large Scale Paddy Field in Malaysia

(Date received: 28/07/2016/Date accepted: 04/10/2016)

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ABSTRACT

A number of models are available to measure infiltration in agricultural fields; however, their applicability is site specific. It is not easy to choose an adequate model to estimate infiltration at a particular agricultural field. In this study, ten different infiltration models have been used for the estimation of infiltration rate in data scarce Muda Irrigation Scheme, the largest paddy field in Malaysia. The model parameters are estimated from soil characteristics, secondary data and literature. The estimated infiltration rates by the models are compared with the measured infiltration rates in order to assess the suitability of the models in the paddy field of Malaysia. The results show that the infiltration rate estimated by physically based Smith-Parlange nonlinear model is very close to the observed rate. Mean monthly infiltration rate estimated by Smith-Parlange model is 0.098mm with a maximum 0.227mm during December to March and a minimum 0.005mm in May and August. All other methods are found to overestimate the infiltration rate. Though the performance of the models is assessed based on results obtained using limited data and secondary information, it is expected that the research will help select the right model to estimate infiltration, which is highly important for irrigation management especially in paddy fields.

Keywords: Infiltration Model, Infiltration Rate, Muda Irrigation Scheme, Malaysia, Paddy Field.

1.0 INTRODUCTION

Infiltration is the process of water entry from the ground surface into the soil from rainfall, snowmelt, and irrigation (Houser, 2005). Infiltration and soil water movement play a key role in surface runoff (Wood *et al.,* 1986; Winchell *et al.,* 1998), groundwater recharge (Shanafield and Cook, 2014), evapotranspiration (Cordova and Bras, 1981; Kirchner *et al.,* 2008), soil erosion (Yu *et al.,* 2003), and transport of chemicals in surface and subsurface water (Govindaraju, 1996). The ability to quantify infiltration is of great importance in watershed management (Zolfaghari *et al.,* 2012). Accurate estimation of infiltration is required to improve runoff estimation, using hydrologic models or pollution leaching or solute transport through sub-surface media (Rumynin, 2011). By understanding the variation of infiltration rates along with the variation in surface conditions, measures can be taken to increase infiltration rates, which help, for example, reduce erosion and flooding caused by overland flow (EPA, 1992).

Quantification of infiltration is also necessary to assess the availability of water for crop growth and to estimate the amount of additional water needed from irrigation (Brouwer *et al.,* 1989). If water is ponded on the surface, the infiltration occurs at the potential infiltration rate (Wilkie, 1999). Therefore, infiltration rate in paddy fields is high due to ponding of water on the surface (Wilkie, 1999).

Because of the fundamental role of infiltration in surface and subsurface hydrology, irrigation and agriculture, infiltration has received a great deal of attention from soil and water scientists, and a large number of models for its computation have been developed. Infiltration models can be classified, in general, into three groups, namely, (i) physically based, (ii) semi-empirical, and (iii) empirical. Obviously, there are a large number of infiltration models but their suitability in real life problems is not obvious. The downward flow rate of water through soil is governed by the saturated conductivity of the soil layer (Liu *et al.,* 2001; Huang *et al.,* 2012). The rate of infiltration is affected by soil characteristics (soil texture, structure and temperature), land cover (vegetation types and cover), storage capacity, rainfall intensity and transmission rate through the soil. Hence, the suitability of a model under given condition is not always evident. Some comparative studies have indicated that the Green-Ampt type models perform better than the curve-number model in predicting infiltration from catchment runoff volumes (Chahinian *et al.,* 2005; Van Mullem, 1991). Mishra *et al.,* (2003) compared the performance of fourteen physically based, semi-empirical, and empirical infiltration models, including the Green-Ampt and Horton models and concluded that the Horton model performs significantly better than the Green-Ampt model. The above studies pointed out that determining the suitability of infiltration models require testing with field data.

In the present study ten infiltration models have been used for the estimation of infiltration in the Muda Irrigation Scheme, which is considered as the largest paddy field of Malaysia. The models used in this study are due to Green and Ampt (1911), Soil Conservation Service (SCS, 1972), Horton (1940), Holtan (1961), Philip (1957), Singh and Yu (1990), Mishra and Singh (2002), Smith and Parlange (1978) Linear and Nonlinear, and Morel-Seytoux (1978). Many of these models distinguish between the actual infiltration rate, f, and the potential infiltration rate, f_p, which is equal to the infiltration rate when water is ponded at the ground surface. Thus, the aims of this study are to (i) use various infiltration models to estimate infiltration rate as a part of irrigation water management the Muda Irrigation Scheme of Malaysia and (ii) compare the performance of various infiltration models in order to propose the suited model(s) for the area. The Muda Irrigation Scheme accounts for about 40% of total rice production in the country (Tukimat *et al.,* 2012). Accurate estimation of infiltration is very essential for water resources planning and management in the scheme.

The agricultural catchments of Malaysia are poorly gauged. In most of the cases, enough information is rarely available for hydrological model calibration. This is especially true for the Muda Irrigation Scheme, where enough data were not available for model calibration. Hence, the objective of the present study was to test rather simple uncalibrated models in estimating hourly infiltration rate in the area. The model parameters were estimated from physical characteristics of soil and literature.

In the following sections of the paper, a brief description of the study area and the data used for the study are given. This follows the methodology section where all the ten models used in the study are described. The application of the models to the Muda Irrigation Scheme to estimate infiltration rate is discussed thereafter. The obtained results are then compared to identify the most suitable model(s) for measuring infiltration rate in the study area. Finally, some conclusions are drawn based on the findings of the study.

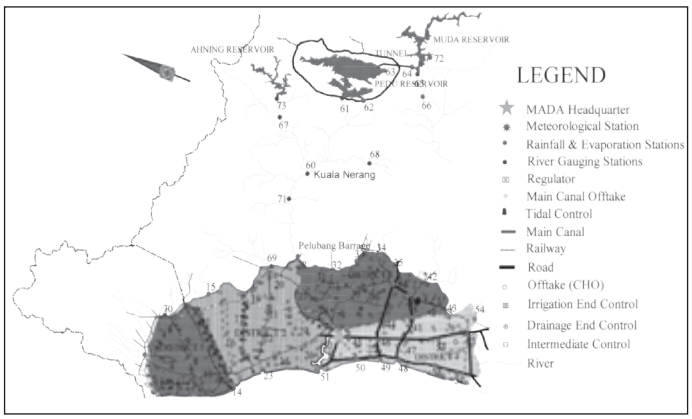

Figure 1: GIS Digitized Muda Irrigation Scheme in Kedah, Malaysia

2.0 STUDY AREA AND DATA

The Muda Irrigation Scheme covers a total gross area of 126,000 ha, out of which about 97,000 ha is under the double cultivation of paddy (MADA, 1977). It is the largest double cropping area in Malaysia. The map of the area is shown in Figure 1. The area is located at about $5^0 45'$ ~$6^0 30'$ N latitude and $100^0 10'$ ~$100^0 30'$ E longitude in the vast alluvial Kedah-Perlis Plain of about 20km wide and 65km long between the foothills of the Central Range and the Straits of Malacca. The area is generally flat with slopes varying from 1 in 5000 to 1 in 10000. The altitude varies between 4.5 m in the inland fringe and 1.5 m above mean sea level in the coastal area (MADA, 1977).

The soil in the Kedah-Perlis of Malaysia consists of heavy clay. The majority of their parent materials consist of marine sediments deposited during the rise in sea level. While the land efflorescence carried to the sea had been deposited on the sea-bed, these sediments were replenished with base and silicic acid, and are rich in mineral and chemical substances (Furukawa, 1976). The soils become very hard and compact during the non-irrigation period, and they crack markedly. When saturated with water, the cracks are filled up due to swelling and slaking phenomena and their permeability decreases rapidly.

Table 1: Step-by-Step Procedure of Calculating Infiltration Using Green-Ampt Model.

Parameter	Month											
	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
*Rainfall, P (mm month^{-1})	20.6	52.4	102.1	184.0	236.7	169.3	210.3	237.5	310.5	287.5	197.8	77.7
*Elementary area, ω (%)	9.01	34.75	49.78	84.26	100.00	77.18	97.86	100.00	91.79	97.65	98.32	43.78
**Rainfall Intensity (cm h^{-1})	2.033	2.033	2.033	2.033	2.033	2.033	2.033	2.033	2.033	2.033	2.033	2.033
Hydraulic Conductivity, K (cm h^{-1})	0.0288	0.0208	0.0161	0.0053	0.0004	0.0075	0.0010	0.0004	0.0029	0.0011	0.0009	0.0179
Porosity, η	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46
ψ (cm)	156.62	156.62	156.62	156.62	156.62	156.62	156.62	156.62	156.62	156.62	156.62	156.62
Time step, t = 1 h	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
θ_e	0.371	0.371	0.371	0.371	0.371	0.371	0.371	0.371	0.371	0.371	0.371	0.371
$\theta r = \eta - \theta_e$	0.090	0.090	0.090	0.090	0.090	0.090	0.090	0.090	0.090	0.090	0.090	0.090
S_e $(0 \le s_e \le 1)$	0.090	0.090	0.999	0.999	0.999	0.999	0.999	0.999	0.999	0.999	0.999	0.738
$\Delta\theta = (1 \le s_e)\ \theta_e$	0.3378	0.3378	0.0004	0.0004	0.0004	0.0004	0.0004	0.0004	0.0004	0.0004	0.0004	0.0973
CRC	0.71	0.71	0.71	0.71	0.71	0.711	0.71	0.71	0.71	0.71	0.71	0.71
F (t), cm	1.490	1.260	0.045	0.023	0.006	0.030	0.010	0.006	0.020	0.010	0.009	0.630
Infiltration rate, f (mm h^{-1})	7.481	6.348	0.262	0.133	0.027	0.157	0.050	0.027	0.081	0.053	0.047	3.214

**** Observed Data (Source: Muda Agricultural Development Authority, Alor Setar, Kedah, Malaysia)***
***** Rainfall Intensity (Al-Mamun and Hashim, 2004)***

The soil in the area is generally heavy marine clay type and the coefficient of permeability varies from $1x10^{-7}$ to $8x10^{-5}$cm s^{-1}. The physio-chemical properties of subsoils of the Muda plain for Chengai soil series type used in this study were: 62% clay, 27% silt, 0.3% carbon, 0.05% nitrogen, 10.7% sand, and 0.28 CEC (ratio of cation-exchange capacity of clay to percent clay) (Paramananthan, 1989). The annual rainfall in the study area is comparatively high, but not evenly distributed throughout the year because of the tropical monsoon climate.

The long-term (1971-97) mean monthly observed rainfall from 53 rainfall stations located in and around the study area were used in this study. The observed total existing water from the time period 1991-97 in each infinitesimal area for dry (February-July) and wet (August-January) crop seasons was converted to moisture content (%) and averaged. The values were then used as input to the models used in this study. The probable maximum soil moisture content in the study area is 85mm for 100% soil saturation (Kitamura, 1987).

The irrigation scheme was divided into 172 irrigation blocks. Kitamura (1990) measured seepage and percolation loss at an irrigation block named SCRBD5b (central canal right bank drain 5b) situated in the south-western part of the Muda Irrigation Scheme area. This data has been used in the present study as the observed data. Kitamura (1987; 1990) observed the average seepage and percolation loss during paddy growth stage in the area as 0.9 mm d^{-1}. MADA (1977) and Teoh and Chua (1989) measured the average seepage and percolation loss during paddy growth stage as 1 mm d^{-1}. To measure percolation, the 'quick percolation measuring apparatus' was used. The rainfall intensity and the other data used in this study are given in Table 1.

3.0 MODELING APPROACH

Ten infiltration models, namely, Green and Ampt, SCS, Horton, Holtan, Philip, Singh-Yu, Mishra-Singh, linear Smith-Parlange, nonlinear Smith-Parlange and Morel-Seytoux models were used in the present study to assess their performance in estimating the surface runoff in the Muda Irrigation Scheme. The models used in the present study can be classified into three classes: physically based model (e.g., Green and Ampt, 1911; Morel-Seytoux, 1978; Philip, 1957; Smith and Parlange, 1978), semi-empirical models (e.g., Mishra and Singh, 2002; and Singh and Yu, 1990), and empirical models (e.g., Holtan, 1961; Horton, 1940; SCS, 1972). A brief description of each model is provided below.

3.1 General Hydrologic Budget

The general hydrologic budget with all the components is shown in Eq. (1). When all other variables except the infiltration are known, then the infiltration rate can be computed using the following equation:

$$W(t+\Delta t) = W(t) + P(t,t+\Delta t) - ET_p(t,t+\Delta t) - R(t,t+\Delta t) - F(t,t+\Delta t) - S_i(t,t+\Delta t) + IR(t,t+\Delta t) \quad [1]$$

where, $W(t+\Delta t)$ is the soil moisture content at time $t+\Delta t$; $W(t)$ is the soil moisture content at time t; $P(t, t+\Delta t)$ is the aerial precipitation; ETp $(t, t+\Delta t)$ is the potential evapotranspiration; $R(t, t+\Delta t)$ is the runoff; $F(t,t+\Delta t)$ is the infiltration loss to groundwater; $S_i(t,t+\Delta t)$ is the seepage loss; and IR $(t, t+\Delta t)$ is the irrigation amount supplied between t and $t+\Delta t$, respectively.

3.2 Green-Ampt Model

The Green and Ampt model (1911) is an approximate theory-based infiltration model utilizing Darcy's law. The Green-Ampt equation for cumulative infiltration F can be obtained as

$$F(t) - \psi\Delta\theta \ln\left(1 + \frac{F(t)}{\psi\Delta\theta}\right) = Kt \quad [2]$$

where, Ψ is the Green-Ampt wetting front suction parameter, K is the hydraulic conductivity of soil, $\Delta\theta$ is the change in moisture content.

Once F is found from Eq. (2), the infiltration rate, $f = dF/dt$, can be obtained from

$$f(t) = K\left(\frac{\psi\Delta\theta}{F(t)} + 1\right) \quad [3]$$

After laboratory tests of many soil samples, Brooks and Corey (1964) concluded that ψ can be expressed as a logarithmic function of an effective saturation, S_e. If the residual moisture content of the soil after it has been thoroughly drained is denoted by θ_r, the effective saturation is the ratio of the available moisture $\theta-\theta_r$ to the maximum possible available moisture content $\eta-\theta_r$:

$$s_e = \frac{\theta-\theta_r}{\eta-\theta_r} \quad [4]$$

where, η is the total porosity and θ is moisture content. The effective saturation has the range $0 \le s_e \le 1.0$, provided that $\theta_r \le \theta \le \eta$. For the initial condition, when $\theta = \theta_i$, cross-multiplying Eq. (4) gives $\theta_i - \theta_r = S_e\theta_e$, and the change in the moisture content when the wetting front passes is $\Delta\theta = \eta - \theta_i = \eta - (S_e\theta_e + \theta_r)$; therefore

$$\Delta\theta = (1 - s_e)\theta_e \quad [5]$$

The soil porosity (total volume occupied by pores per unit volume of soil) is computed from bulk density and particle density (normally assumed to be equal to 2.65 g cm^{-3}) as follows:

$$\eta = 1 - \frac{BD}{PD} \quad [6]$$

where, BD is the soil bulk density (g cm^{-3}), and PD is the particle density (g cm^{-3}). If the cation-exchange capacity of the clay (an indicator of the shrink-swell capacity of the clay) is available, the bulk density at the water content for 33 kPa tension can be estimated as

$$BD = 1.51 + 0.0025(S) - 0.0013(S)(OM) - 0.0006(C)(OM) - 0.0048(C)(CEC) \quad [7]$$

where, S is the percent sand, C is the percent clay, OM is the percent organic matter [1.7 (percent organic carbon)], and CEC ranges from 0.1-0.9.

The ψ value can be obtained from the soil properties by the following equation (Rawls and Brakensiek, 1983):

$$\psi = \exp\left[\begin{array}{l} 6.53 - 7.326(\eta) + 0.00158(C^2) + 3.809(\eta^2) + 0.000344(S)(C) - 0.04989(S)(\eta) \\ + 0.0016(S^2)(\eta^2) + 0.0016(C^2)(\eta^2) - 0.0000136(S^2)(C) - 0.00348(C^2)(\eta) \\ - 0.000799(S^2)(\eta) \end{array}\right] \quad [8]$$

The Brooks-Corey residual water content θ_r can be estimated from

$$\begin{array}{l} \theta_r = -0.0182482 + 0.00087269(S) + 0.00513488(C) + 0.02939286(\eta) \\ - 0.00015395(C^2) - 0.0010827(S)(\eta) - 0.00018233(C^2)(\eta^2) \\ + 0.00030703(C^2)(\eta) - 0.0023584(\eta^2)(C) \end{array} \quad [9]$$

The area, bare outside canopy, was assumed to be crusted and the effective hydraulic conductivity was considered equal to the saturated hydraulic conductivity K_s times a crust factor CRC. Rawls *et al.,* (1990) developed the following relationship for the crust factor:

$$CRC = \frac{SC}{1+(\Psi_i / L)} \quad [10]$$

where, SC = correction factor for partial saturation of the soil = 0.736 + 0.0019S; ψ_i = matric potential drop at the crust-subcrust interface = 45.19 – 46.68 (SC), cm; and L = wetting front depth in cm.

The cumulative infiltration at the ponding time t_p is given by F_p, where i is the constant intensity of rainfall (cm h^{-1}); and the infiltration rate by $f = i$; substituting into Eq. (3),

$$t_p = \frac{K\Psi\Delta\theta}{i(i-K)}, \qquad i > K \quad [11]$$

The Green-Ampt equations are developed for homogeneous soils. The approach can be extended to describe infiltration into layered soils, when the hydraulic conductivity of the successive layers is known. As long as the wetting front is in the top layer, the equations remain the same. After the wetting front enters the second layer, the effective hydraulic conductivity K is set equal to the harmonic mean $K_h = \sqrt{K_1 K_2}$ for wetted depths of the first and second layers, and the capillary head is set equal to ψ of the second layer. This principle is then carried out through the third and succeeding layers.

3.3 Horton Model

A three-parameter empirical infiltration model was presented by Horton (1940) and it has been widely used in hydrologic modelling (Eq (12)). According to this approach the infiltration starts at a constant rate, f_0, and decreases exponentially with time, t. After some time when the soil saturation level reaches a certain value, the rate of infiltration will level off to the rate f_c.

$$f_t = f_c + (f_0 - f_c)e^{-kt} \quad [12]$$

where, f_t is the infiltration rate at time t; f_0 is the initial infiltration rate (also the maximum infiltration rate); f_c is the equilibrium infiltration rate after the soil has been saturated (also the minimum infiltration rate); k is the decay constant specific to the soil (T^{-1}).

Horton's equation can also be used to find the total volume of infiltration, F, after time t.

$$F_t = f_c t + \frac{(f_0 - f_c)}{k}\left(1 - e^{-kt}\right) \quad [13]$$

Alternatively, Eqs (12) and (13) can be combined to yield the following direct relationship between F and f_t.

$$F(t) = \left[\frac{f_c}{k}\ln(f_0 - f_c) + \frac{f_0}{k}\right] - \frac{f_c}{k}\ln(f_t - f_c) - \frac{f_t}{k} \quad [14]$$

3.4 SCS Model

The SCS curve-number model is the most widely used model for estimating rainfall excess. According to SCS model, the total infiltration, F, for a rainfall event, P, can be estimated as,

$$F = \frac{(P - 0.2S_p)S_p}{P + 0.8S_p}, \qquad P > 0.2S_p \quad [15]$$

where, S_p is the soil storage capacity. The infiltration rate, f, can be derived from Eq. (15) by differentiation,

$$f = \frac{dF}{dt} = \frac{S_p{}^2 i}{(P + 0.8S_p)^2} \quad [16]$$

Instead of specifying S_p directly, a curve number, CN, is usually specified where CN is related to S_p by

$$CN = \frac{1000}{10 + 0.0394S_p} \quad [17]$$

where, S_p is in mm. Clearly, in the absence of available storage (S_p = 0, impervious surface) the curve number is equal to 100, and for an infinite amount of storage the curve number is equal to zero.

3.5 Philip Model

Philip's Two-Term equation (Philips, 1957) is a truncated power series solution that could be used as an infiltration model. The equation is

$$f = \frac{1}{2}S_v t^{-1/2} + \frac{2}{3}K_s \quad [18]$$

where, f is in cm h^{-1}, t is time for ponding (h), S_v is the sorptivity (LT$^{-0.5}$), K_s is in cm h^{-1}), and S_v can be approximated using the following equation developed by Youngs (1964):

$$S_v = \sqrt{2(\eta - \theta_i)K\Psi} \quad [19]$$

The K_s can be estimated from (Brooks and Corey, 1964):

$$\frac{K(\theta)}{K_s} = \left(\frac{\theta - \theta_r}{\eta - \theta_r}\right)^n = (s_e)^n \quad [20]$$

where $n = 3 + 2 / \lambda$, and the Brooks-Corey pore-size distribution index, λ can be estimated from

$$\lambda = \exp\begin{bmatrix} -0.7842831 + 0.0177544(S) - 1.062498(\eta) - 0.00005304(S^2) \\ -0.00273493(C^2) + 1.11134946(\eta^2) - 0.03088295(S)(\eta) \\ +0.00026587(S^2)(\eta^2) - 0.00610522(C^2)(\eta^2) \\ -0.00000235(S^2)(C) + 0.00798746(C^2)(\eta) - 0.00674491(\eta^2)(C) \end{bmatrix} \quad [21]$$

3.6 Singh-Yu Model

Singh and Yu (1990) derived a model based on two postulates, namely, (i) the rate of infiltration in excess of the final infiltration rate, called excess infiltration, at any time is directly proportional to the mth power of the available storage space in the soil column at that time, and (ii) the rate of excess infiltration is inversely proportional to the nth power of the cumulative infiltration up to that time. This model is expressed mathematically as,

$$f(t) = f_c + \frac{a[S(t)]^m}{[S_0 - S(t)]^n} \quad [22]$$

where, $f(t)$ is the infiltration rate (LT^{-1}) at time t, f_c is the final infiltration rate (LT^{-1}), $S(t)$ is the available storage for water retention in the soil column at time t (L), S_0 is the potential storage space available for moisture retention in soil column at the beginning (L), and a, m, and n are the coefficient and exponents of the variables $S(t)$ and $(S_0 - S(t))$, respectively. The cumulative infiltration F is equal to $(S_0 - S(t))$.

3.7 Holtan Model

Holtan (1961) developed an empirical equation on the premise that soil moisture storage, surface-connected porosity, and the effect of root paths are the dominant factors influencing the infiltration capacity. Holtan and Lopez (1971) modified the equation as

$$f = GI\ AS_a^{1.4} + f_c \quad [23]$$

where, f is the infiltration rate (in h^{-1}); GI is the growth index of crop in percent maturity varying from 0.1 to 1.0 during the season; A is the infiltration capacity (in h^{-1}) per (in) of available storage and is an index representing surface-connected porosity and the density of plant roots which affect infiltration, considered 0.10 for fallow and row crops type land use with poor condition (Frere *et al.,* 1975); S_a is the available storage in the surface layer (in), and f_c is the constant infiltration rate (in h^{-1}) when the infiltration rate curve reaches asymptote (steady infiltration rate).

3.8 Mishra–Singh Model

By expressing the SCS model in the form of the Horton method Mishra and Singh (2002) developed an infiltration equation by assuming a linear variation of the cumulative precipitation with time or with constant rainfall intensity:

$$f = f_c + \frac{S_p k}{(1+kt)^2} \quad [24]$$

where, S_p is the potential maximum retention capacity of the catchment used in the SCS model; a general model parameter S_0, identical to the model of Singh–Yu (1990), and k is the decay coefficient identical to the decay parameter used in the Horton model. The mathematical expression of Eq. (24) is a specific form of the retention model proposed by van Genuchten (1980) relating θ with Ψ.

3.9 Smith-Parlange Linear and Nonlinear Models

Smith and Parlange (1978) derived an infiltration model expressed as:

$$f = K_s\left(\frac{C}{K_s F} + 1\right) \quad [25]$$

where, C is a parameter related to the soil sorptivity and varies linearly with the initial soil moisture. It also depends on the amount and pattern of rainfall intensity. Parameters C and K_s can either be determined graphically or by using a regression approach utilizing infiltration data. The parameter C in the Green-Ampt equation is

$$C \approx -\Psi_{avg}(\theta_s - \theta_i)K_s \quad [26]$$

where, Ψ_{avg} is average capillary tension across the wetting front. Mein and Larson (1973) proposed a relation

$$\Psi_{avg} = -\frac{1}{K_s}\int_{K_i}^{K_s} \Psi\, dK_s \quad [27]$$

Later, Smith and Parlange (1978) derived a nonlinear infiltration model expressed as:

$$f = K_s \frac{e^{(FK_s/C)}}{e^{(FK_s/C)}-1} \quad [28]$$

3.10 Morel-Seytoux Model

Morel-Seytoux (1978) model is a modification of Green and Ampt (1911) model. The model is based on the ponding time concept: runoff cannot occur as long as the soil surface retention potential is not met. Therefore, at every time stem, the model needs to determine whether the ponding time (*tp*) has been reached. For $t>t_p$ the cumulative infiltration $F(t)$ at time t, is calculated from

$$F(t) - F_p - \left[S_f + F_p\left(1-\frac{1}{\beta}\right)\right]\ln\left[\frac{S_f + F(t)}{S_f + F_p}\right] = \frac{K_s(t-t_p)}{\beta} \quad [29]$$

where, F_p [L] is the cumulative infiltration when ponding occurs; β is a new viscous correction parameter introduced by Morel-Seytoux and Khanji (1974), the value of which varies usually between 1 and 1.7 and is generally fixed at 1.3; and S_f [L] is a storage and suction factor that can be expressed as a function of the soil hydraulic properties

$$S_f = (\theta_s - \theta_i)H_c\left[1 - \frac{1}{3}\left(\frac{\theta_i - \theta_r}{\theta_s - \theta_r}\right)^6\right] \quad [30]$$

where, H_c [L] is the capillary height.

3.11 Evaluation of Models Performance

Due to the limitation of good quality field data, calibration of any or all of the above mentioned models is a challenge for many agricultural fields around the world. For the Muda irrigation scheme the limitation of the field data does not allow a comprehensive calibration of models mentioned above. Calibration increases model accuracy and reduces uncertainty of model predictions. If measured data is not available, studies on uncertainty assessment or sensitivity analysis of model parameters may not be carried out. In this regard, the IAHS initiative of Prediction in Ungauged Basins regarding modelling ungauged catchments is noteworthy. The main recommendation of leading hydrologists around the world who participated in this initiative (Sivapalan *et al.,* 2003; Sivapalan, 2006) about simulating ungauged basin was on using a model, with limited calibration and on improving the understanding of the processes with modelled results to compensate for the limitation of calibration. Furthermore, they recommended to use new data points, which may appear in future, to compare the model results and if needed to update the model parameters.

Several statistical measures, namely, correlation coefficient, coefficient of determination (R^2), variance, relative mean absolute error, Nash and Sutcliffe efficiency, etc., are available to evaluate the performance of a model. The relative mean absolute error (RMAE), which incorporates both systematic and random errors, is used in this study to assess the model performance. The relative mean absolute error (RMAE) can be expressed as

$$RMAE = \left(\frac{1}{n}\sum_{i=1}^{n}|y_i - x_i|\right)/\bar{x}$$

where, x_i = ith observation of the observed data, y_i = ith observation of the model data,

$$\bar{x} = \frac{1}{n}\sum_{i=1}^{n} x_i \text{ = mean of } x, \text{ and } n = \text{sample size.} \quad [31]$$

4.0 RESULTS AND DISCUSSION

The models used in this study can be considered site specific rather than universally applicable. Thus, the estimated model

Table 2: Observed Infiltration and Models Infiltration Rates for Different Months in the Study Area.

Observed Data	Month											
	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**July**	**Aug**	**Sept**	**Oct**	**Nov**	**Dec**
*Mean Monthly Rainfall, *P* (mm)	20.6	52.4	102.1	184.0	236.7	169.3	210.3	237.5	310.5	287.5	197.8	77.7
*Daily Cumulative Infiltration (mm)	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
**Infiltration Rate (mm h^{-1})	0.079	0.079	0.079	0.079	0.079	0.079	0.079	0.079	0.079	0.079	0.079	0.079
Model	**Monthly Infiltration Rate (mm h^{-1})**											
Smith-Parlange (Nonlinear)	0.227	0.227	0.227	0.075	0.005	0.106	0.015	0.005	0.041	0.015	0.013	0.227
Holtan	0.254	0.254	0.254	0.254	0.254	0.254	0.254	0.254	0.254	0.254	0.254	0.254
Singh-Yu	0.321	0.321	0.321	0.321	0.321	0.321	0.321	0.321	0.321	0.321	0.321	0.321
Morel-Seytoux	0.211	0.159	0.114	1.030	0.995	0.965	0.859	0.995	1.030	0.993	0.997	0.129
Horton	0.831	0.831	0.831	0.831	0.831	0.831	0.831	0.831	0.831	0.831	0.831	0.831
SCS	7.369	2.267	0.789	0.281	0.177	0.327	0.220	0.176	0.107	0.123	0.246	1.236
Green-Ampt	7.481	6.348	0.262	0.133	0.027	0.157	0.050	0.027	0.081	0.053	0.047	3.214
Philip	3.858	3.283	2.895	1.646	0.426	1.965	0.722	0.426	1.222	0.745	0.670	3.056
Mishra-Singh	5.504	5.504	5.504	5.504	5.504	5.504	5.504	5.504	5.504	5.504	5.504	5.504
Smith-Parlange (Linear)	11.534	5.125	53.777	10.833	0.016	17.095	0.657	0.016	3.596	0.769	0.493	6.088

**** Observed Data (Source: Muda Agricultural Development Authority, Alor Setar, Kedah, Malaysia)***
***** Calculated infiltration rate from observed daily cumulative infiltration of 1 mm***

parameter values as well as the model outputs may vary from location to location. In this study, the cumulative infiltration and the infiltration rates were estimated using the ten models mentioned above. Due to the scarcity of the observed data the models were not calibrated. Limited available observed data together with secondary information about the study area from literature were used in this study. Further descriptions are provided in the following sections.

4.1 Green-Ampt Model

In order to apply the Green-Ampt model (Eq (3)), the effective hydraulic conductivity K, the wetting front suction head Ψ, the porosity η and the effective porosity θ_e need to be measured or estimated. Generally, K varies along with Ψ, so the values of Ψ and K should be considered as typical representative values that may allow some variability in applications. Using the soil properties reported by Paramananthan (1989), the porosity, residual moisture content, the effective porosity and soil suction head were estimated.

The hydraulic conductivity of sandy loam soil decreases more rapidly with the decrease in the ponding depth than that of the clayey soil, such that at lower ponding depth values (or at higher suctions) the hydraulic conductivity of the clayey soil is higher. The soil in the study area is of heavy clay type (Chengai series) with the coefficient of permeability ranging from $1x10^{-7}$ to $8x10^{-6}$ cm s^{-1} (Kitamura, 1990). The coefficient of permeability is allowed to vary in different months as a function of moisture content in each month, considering the highest value (i.e., $8x10^{-6}$ cm s^{-1}) for dry soil during dry season and vice-versa. The area which is bare/outside of the canopy was assumed to be crusted and the effective hydraulic conductivity was considered equal to the saturated hydraulic conductivity K_s times a computed crust factor CRC (Eq (10)) of 0.71. The step-by-step procedure of calculating hourly cumulative infiltration and infiltration rate in each month using this model is shown in Table 1. The mean infiltration rate was found to be 1.490 mm h^{-1} by this model (Table 2).

4.2 Horton Model

By considering the USDA soil texture of type clay, an f_c value of 0.5mm h^{-1} was estimated using Eq (12). Following the recommendation of Singh (1992) to maintain the radio f_0/f_c of the order of 5 and using the value of the decay constant, k = 0.03 min^{-1} the mean value of f_t was estimated to be 0.831mm h^{-1} (Table 2). The variability of the infiltration parameters in the Horton model is due to the reasons that infiltration depends upon several factors that are not explicitly accounted for in finding f_c values for different soil types and vegetation covers. For example, the initial moisture content and organic content of the soil, vegetative cover, and season are not explicitly accounted for in the Horton model (Linsley *et al.,* 1982). Moreover, the USDA soil types do not match exactly with the soil types in the study area. Thus, an experimental study is required to estimate the values of f_0, f_c and k to find much more reliable infiltration rate in the study area.

4.3 SCS Model

The soil in the study area can typically be considered as a combination of fallow cover type and row cover type, comprising of *CN* for hydrologic soil group *D* (soils that swell significantly when wet, heavy plastic clay and saline soil). Therefore, the resulting *CN* was chosen based on the bare soil, crop residue cover and row crops (straight row with poor and good conditions only), and was considered to be the average of the curve numbers for these soil types. By referring to the CN Table of the SCS method the curve number was computed as: $CN = (94+93+90+91+89)/5 = 91.4$. Assuming an average rainfall intensity of 1-day duration, the probable maximum precipitation (PMP) was measured at an airport station closed to the study area (Al-Mamun and Hashim, 2004). The storage value was found to be 23.88mm. The infiltration rate (mm h^{-1}) in each month was estimated and the mean infiltration rate was found to be 1.110 mm h^{-1} (Table 2).

4.4 Philip Model

The inputs to this model are mainly the physio-chemical properties of soil and moisture content. In calculating K_s (Eq (18)) for different months, the effective saturation values for different months were taken from the Green-Ampt model. The value of K_s during December to February was considered to be equal to the maximum value obtained in March, because the computed values of K_s were found to be comparatively higher than the soil hydraulic conductivity during December to February. The hourly cumulative infiltration and infiltration rate were calculated, and the mean infiltration rate was found to be 1.743mm h^{-1} (Table 2).

4.5 Singh-Yu Model

Since the soil in the study area is heavy plastic clay, the minimum values of *a, m,* and *n* are taken from literature (Mishra *et al.,* 2003) as such values are not available in the study area, to achieve more realistic results. The hourly cumulative infiltration was taken from the Green-Ampt model (1911). The potential storage space available for moisture retention in soil column S_0 (Eq (22)) was considered equal to S_p of the SCS model (Eq (16)). Musgrave (1955) reported that the potential infiltration rate fc was varied from 0.0 to 0.13cm h^{-1} for the hydrologic soil group *D*. Accordingly, an average value of (0.0+0.13)/2, i.e., 0.065 (cm h^{-1}) was considered for the potential infiltration rate f_c. MADA (1977) and Teoh and Chua (1989) reported measured daily seepage and percolation loss of only 1mm for the Muda soil type. Following the recommendation of Singh (1992), the authors considered $f_0/f_c = 5$ and by further considering that the infiltration rate f must be higher than fc, the resulting hourly infiltration rate was estimated. The resulting mean infiltration rate from this model was estimated to be 0.321mm h^{-1} (Table 2).

4.6 Holtan Model

The storage S_a (mm) value (Eq (23)) was taken from the SCS model and converted to inches in order to comply with the system of units used in the Holtan model. The average value of the vegetation parameter *A* in the Holtan model for land use type 'fallow and row' crops with poor condition was considered as 0.1 (Frere *et al.,* 1975). Musgrave (1955) reported that the final infiltration rate fc varied from 0.0 to 0.13cm h^{-1} for the hydrologic soil group *D*. Accordingly, an average value of f_c = (0.0+0.13)/2, i.e., 0.065cm h-1 was considered in Eq (23). The authors refer once again to MADA (1977) and Teoh and Chua (1989) for the measured daily infiltration and percolation loss of only 1mm d^{-1} for the Muda soil type. Accordingly, it was assumed that the f_c value would be less than 1mm d^{-1}. Following the recommendation of Singh (1992) for f_0/f_c ratio to be about 5 and considering that the infiltration rate *f* must be higher than f_c, the infiltration rate was estimated considering the value of f_c at 0.5mm d^{-1} in absence of reliable data. Consequently, the mean infiltration rate from this model is found to be 0.254 mm h^{-1} (Table 2).

4.7 Mishra-Singh Model

In calculating the infiltration rate f using the Mishra-Singh model, the S_p, f_c and *k* values of Eq (24) were obtained from the SCS, Holtan, and Horton models, respectively. The estimated mean infiltration rate using this model was found to be 5.504 mm h^{-1} (Table 2). Considering typical values of S_p, f_c and *k* for the Narsinghpur Clay (NC) (Mishra *et al.,* 2003), the estimated infiltration rate was found to be very high (5.504mm h^{-1}) as shown in Table 2.

4.8 Smith-Parlange Linear and Nonlinear Models

In calculating f (mm h^{-1}) by Smith-Parlange nonlinear model (Eq 25), the following considerations were undertaken: (i) the values of the cumulative infiltration *F* were obtained from the Green-Ampt model, (ii) the K_s values were obtained from the Philip model, (iii) the value of K_i was assumed equal to *K* value, (iv) ψ value was taken from the Green-Ampt model to calculate $\psi_{avg.}$, (v) the initial water content θ_i for each month was assumed to be the respective moisture content θ in each month, and (vi) the θ values were considered 99.9% in May and August instead of 100% in order not to get zero value of *C* as zero value of *C* results FK_s/C infinity and produces an unrealistic result. The hourly infiltration rates and the corresponding cumulative infiltration amounts for each month were calculated, and the resulting mean infiltration rates from the linear and nonlinear Smith-Parlange models were found to be 9.167mm h^{-1} and 0.098mm h^{-1}, respectively (Table 2).

4.9 Morel-Seytoux Model

In calculating the values of hourly cumulative infiltration *F(t)* and the infiltration rate *f(t)* ((Eq 29)), the following considerations were made: (i) the value of the saturated hydraulic conductivity K_s was obtained from the Philip model, (ii) the ψ value was obtained from the Green-Ampt model and set identical to H_c value, (iii) the value of t_p was obtained from rainfall intensity *i*, which was assumed to be an average rainfall intensity of probable maximum precipitation of 1-day duration measured at Airport Station close to the study area, Kedah, Malaysia (Al-Mamun and Hashim, 2004), (iv) the initial water content θ_i was assumed to be the respective moisture content θ in each month, and (v) the value of β was fixed at 1.3. The resulting hourly cumulative infiltration *F(t)* and the corresponding infiltration rate *f(t)* were estimated, and the mean monthly infiltration rate using Eq. (3) was found to be 0.706mm h^{-1} (Table 2).

4.10 Evaluation of Models Performance

The models used in this study differ by their mathematical structures and therefore, their parameters are required to be calibrated, even when the input hydrologic data are from the same site. Predictions from uncalibrated models are uncertain.

The observed total existing water from 1991-97 in each infinitesimal area for dry (February-July) and wet (August-January) crop seasons were converted to moisture content and averaged, and used as an input to the models used in this study (Kitamura, 1987). The available observed data, together with secondary information about the study area literature were used to estimate the infiltration rate.

The observed cumulative infiltration *F(t)* of 1mm was converted to the infiltration rate *f(t)* (mm h^{-1}) for comparison purpose. Using the Green-Ampt Model (Eq (2)), the corresponding hydraulic conductivity *K* value was found to be 0.0705cm d^{-1} against an observed daily cumulative infiltration and percolation loss of 1mm, keeping the other parameters of Eq (2) the same as the parameters of the Green-Ampt model. Using Eq (3), the corresponding converted observed infiltration rate of 0.0786mm h^{-1} was achieved against the observed daily cumulative infiltration and percolation loss of 1mm in order to compare the models results with the observed infiltration rate, i.e., to fulfil the comparison requirement.

The performance of the models was evaluated based on the relative mean absolute error (RMAE) criterion. The RMAE values for the 10 models were computed and the models were ranked according to the increasing RMAE values to identify the most suitable model for the study area. Within the selected modelling framework, the performance evaluation of the models suggests that the Smith-Parlange nonlinear model performed the best compared to other models. The second best suited model was found to be the Holtan model. The suitability of the other models in descending order can be found according to the ascending order of RMAE values and ranks, given in Table 3.

Table 3: Performance of Different Models in Estimation of the Infiltration Rate.

Model	Mean Monthly Model Infiltration Rate (mm h^{-1})	RMAE	Rank
Smith-Parlange (Nonlinear)	0.098	0.253	1
Holtan	0.254	2.230	2
Singh-Yu	0.321	3.083	3
Morel-Seytoux	0.706	7.990	4
Horton	0.831	9.570	5
SCS	1.110	13.126	6
Green-Ampt	1.490	17.961	7
Philip	1.743	21.180	8
Mishra-Singh	5.504	69.040	9
Smith-Parlange (Linear)	9.167	115.652	10

However, the applications of the other available models and/or the use of individual storm events and much more precise field data might alter this suitability for the study area. Therefore, a detailed experimental study can be undertaken to run the models with more readily and to develop a new model for the heavy marine clay type soils in the study area.

The practical benefits of this study are to estimate the modelled cumulative infiltration quantity and infiltration rates in different months to allow irrigation scheduling and estimating water loss and in turn to compute the surface runoff from the scheme to design drainage channels.

5.0 CONCLUSIONS

Infiltration models were used in the present study to calculate cumulative infiltration and infiltration rates using uncalibrated models. Available observed data and information about the study area from literature were used in estimating infiltration rates. Using the soil properties reported by Paramananthan (1989), the porosity, residual moisture content, the effective porosity and soil suction head were computed. The soils in the study area is heavy clay type (Chengai series), with the coefficient of permeability ranging from to $1x10^{-7}$ to $8x10^{-6}$ cm s^{-1} (Kitamura, 1990). The coefficient of permeability was allowed to vary as a function of moisture content in different months, considering the highest value for the dry soil and vice-versa. The performance of the models was evaluated based on the RMAE criterion. Within the selected modelling framework, the evaluation of performance of the models is helpful in identifying models, which might be used to estimate the infiltration rate in the study area. It has been found that compared to other models the infiltration rate estimated by the physically based Smith-Parlange nonlinear model (1978) was closest to the observed rate. The suitability of the other models can respectively be regarded as the Holtan, Singh-Yu, Morel-Seytoux, Horton, SCS, Green-Ampt, Philip, Mishra-Singh, and Smith-Parlange (linear) model (Table 2).

The models used in this study are site specific and not as such universal. Thus, their performance may vary from site to site. The performance of the models is based on the results obtained using limited data and secondary information. Thus, a detailed experimental study can be undertaken to run these models satisfactorily in the near future. Finally, based on the available data and information, it can be concluded that the Smith-Parlange nonlinear model is considered to be the most suitable model for estimating infiltration in the study area. It is expected that this study will be used as a guideline in model selection and estimation of infiltration rates in the agricultural field of Malaysia, which in turn will help in irrigation water management, soil erosion control, irrigation efficiency improvement and agricultural pollution assessment.

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PROFILES

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EN 206 Conformity Testing for Concrete Strength in Compression

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ABSTRACT

The design of concrete structures in accordance with EN 1992-1-1 adopts the characteristic cylinder compressive strength in its equations. EN 206 provides for conformity testing for concrete strength in compression using 150mm diameter by 300mm length cylinders or 150mm cubes only. The complementary standard to EN 206 in UK, BS 8500 (SS 544 in Singapore, MS 523 in Malaysia) has added provisions (clause 12.2) for the use of 100mm cubes for conformity testing. The conformity criteria for 100mm cube specimens are to be the same as those for 150mm cubes. A series of tests based on 3 selected levels of compressive strength has been conducted to examine the relationship between these 3 types of test specimens for compressive strength of concrete. For each strength level, 100 batches of concrete were produced over a period of several months. The test results are presented with analysis based on the mean of 3 numbers for each type of test specimens prepared from the same batch at each time of preparation. The results of this study for the 3 strength levels support the relationship between standard cylinder compressive strength and standard cube compressive strength in EN 206. In addition, results also support the recommendation that standard 100mm cube compressive strength is equivalent to that of standard 150mm cube compressive strength in BS 8500. The use of the small size cubes and certification of designed concrete promote sustainability in concrete construction.

Keywords: Cube, Concrete, Conformity, Cylinder, Strength, Sustainability.

1.0 INTRODUCTION

BS EN 206 [1] adopts only 150mm diameter by 300mm length cylinders or 150mm cubes as standard specimens for determining compressive strength of concrete. Both BS EN 1992-1-1 [2] and BS EN 206 [1] provide for equivalent cube compressive strength corresponding to cylinder compressive strength. In general, up to strength class of C55/67, the ratio of 150mm cube compressive strength/150mm cylinder compressive strength is nominally 1.25 (with rounding to nearest 1 MPa). Above strength class of C55/67 up to C100/115, a constant difference of 15 MPa higher for cube compressive strength above that of cylinder compressive strength has been adopted. These relationships are examined at three strength class levels, i.e. C32/40, C50/60 and C65/80. In addition, the ratio of 100mm cube compressive strength/150mm cube compressive strength and the ratio of 100mm cube compressive strength/150mm cylinder strength at these strength levels are also determined for the same three strength class levels. The test results based on three specimens of each shape and size at the age of 28 days after standard curing are analysed to provide an assessment of their relationships. A brief summary of these results has been presented at the 40th OWICS Anniversary Conference in August 2015 by Tam *et. al*,[3]. A more in-depth analysis is reported in this paper. Although limited in scope, the analysis provides a reasonable indication of their implications in conformity assessment of the characteristic concrete compressive strength based on 100 batches of concrete for each of the three strength class levels. These were produced over a period of several months (145 to 206 days) in the same RMC plant and may be deemed to be representative of normal production in a local ready-mixed concrete (RMC) plant using constituent materials generally available in Singapore.

2.0 BACKGROUND

The topic on effect of shape and size of test specimens for determination of concrete compressive strength has been studied by various researchers as early as 1925 e.g. Gonnerman [4] Neville, [5] has reviewed research findings from extensive published literature and reported that approximately 100mm cubes to be 1.05 times of 150mm cubes and but from analysis of numerous data by Neville [6], proposed the relationship between concrete specimens of different shapes and sizes (f_c) relative to that of a 6 inch (150 mm) cube ($f_{cu,6}$) as follows:

$$f_c/f_{cu,6} = 0.56 + 0.697/(V/6hd + h/d)$$

where V = volume of specimen, (V/150hd + h/d for h and d in mm)

h = height (in inches), and

d = least lateral dimension (in inches)

Substituting dimensions of a 6 inch (150mm) cube into the above equation results in a value of 0.91 instead of the expected value of 1.0. Based on this relationship, the ratio of 4 inch (100mm) cube relative to that of a 6 inch (150mm) cube is 0.98 but adjusting with the factor of 0.91, the ratio becomes 1.08. A

study by Leung and Ho (1996) [7] in Hong Kong based on data from 8 projects for grade 20 to 50 (C16/20 to C40/50), for a total of 349 batches, the mean ratio of 100mm to 150mm cubes was found to be 1.05. However, it ranges from 0.79 to 1.23 with up to 34% of individual ratios (15/44) below 1.0 in one project. For the other 7 projects the percentage of individual ratios below 1.0 varies from 4% to 22%. The mean for the total of 349 individual ratios is 17%. Similar percentage of ratios below 1.0 was also reported by Tam *et. al,* [3]. A more recent study by Wong (2103) [8] in Hong Kong compared the ratio of 150mm x 300mm cylinders to 100mm cubes and found the mean ratio is 0.78 for up to grade 80 (C65/80) and 0.80 for above grade 80 (up to 119 MPa for 100mm cubes). Taking the strength ratio of 100mm cube to 150mm cube as 1.05, the ratio for 150mm x 300mm cylinder to 150mm cubes becomes 0.82 for up to grade 80 and 0.84 for above grade 80. These findings differ slightly from values in EN 206 [1]. Even after significant number of studies there is as yet no definitive finding on the effect of specimen size for determining concrete compressive strength. Moreover, most studies are based on limited number of samples tested and generally, from a selected number of strength levels with specimens from the same sample tested at several ages. Hence, the standard deviation, arising from variability arising from constituent materials, batching and sampling on characteristic value of concrete compressive strength has not been studied. The present study aims the provided information where these factors are included in the selected strength levels.

The first objective of the current study is to compare the specified relationship between characteristic values of cylinder and cube compressive strength as given in EN 206 [1] at three strength levels. The second objective is to assess if the characteristic compressive strength at the age of 28 days of 100mm cubes to that of 150mm cubes may be deemed to be the same as stated in BS 8500-2 [9]. Both of these factors are of special interest to countries where structural design is based on cylinder compressive strength in EN 1992-1-1 [2] and conformity criteria may be based on 100mm and/or 150mm cubes.

3.0 EXPERIMENTAL DETAILS

The constituent materials for casting of all the concrete specimens are those commonly used in current production in a local RMC plant. The three concretes were produced by the same plant using Portland cement to BS EN 197-1 [10] CEM I – 42.5 R, 20mm maximum size granite and natural sand together with a superplasticiser to produce a consistence class of S4 given in BS EN 206 [1] for designed concretes, over a period of several consecutive months. Silica fume (SF) to BS EN 13263-1 [11] had been added to C65/80 concrete only. Table 1 shows the composition of the three concretes. All the specimens were cast, then demolded after 24 hours and cured at 27 ± 5 °C as recommended for Singapore laboratories under SS 544-2 [12] Annex ZZA until age of 28 days when they were tested at the saturated condition. No unexpected performance of the constituent materials was noticed during the period of this test program which used the same constituent materials for normal production of concrete in the same plant. All the three types of specimens were cast from the same batch of concrete. End preparation for all cylinder specimens was by grinding.

Table 1: Composition of concretes.

Concrete	Composition (kg/m^3)				
	Cement (SF)	Water	Fine Aggregate	Coarse Aggregate	Admixture
C32/40	395(0)	175	776	990	5.39
C50/60	530(0)	175	660	990	7.23
C65/80	550(40)	150	590	990	11.31

4.0 TEST RESULTS

The test data of the three populations of concrete are analyzed in terms of the following:

a. For each concrete, the mean and standard deviation of all the test results (average of 3 specimens).
b. For each concrete, the ratio of 150mm cube/150mm cylinder specimens ($f_{c,150cu}/f_{c,150cyl}$) for each batch of concrete.
c. For each concrete, the ratio of 100mm cube/150mm cube specimens ($f_{c,100cu}/f_{c,150cu}$) for each batch of concrete
d. For each concrete, the mean of the ratio calculated in (b) and (c) above.
e. For each concrete, the mean of the ratio of 150mm cube/150mm cylinder specimens based on their characteristic values (f_{ck}), mean (f_m) and from (a) above.
f. For each concrete, (strength class C32/40 and C50/60), distribution of the ratio of 150mm cube/150mm cylinders ($f_{c,150cu}/f_{c,150cyl}$).
g. For each batch of concrete for each concrete, distribution of the difference between 150mm cube and 150mm cylinders ($f_{c,150cu} - f_{c,150cyl}$) for strength class C65/80.
h. For each concrete; distribution of the ratio of 100mm cube/150mm cube specimens ($f_{c,100cu}/f_{c,150cu}$) for each batch of concrete.

[Note: the results of item (h) indicate that the ratios for ($f_{c,100cu}/f_{c,150cu}$) does not deviate significantly from 1.0 and hence similar analysis for the case of 100mm cubes corresponding to item (f) as item (g) is omitted for which the finding will be similar to the case of 150mm cubes in item (f)].

Annex A is a summary of the data for items (a) to (d) stated above which has been presented recently by Tam *et. al,* [3]. Annex B shows the overlapping of the distribution of compressive strength of both 100mm cubes and 150mm cubes for each of the 3 strength levels reported by Tam *et. al,* [3]. The findings are summarized as follows:

(1) The ratio of 150mm cylinders/150mm cubes for C32/40 based on characteristic strengths, ($f_{ck,150cy}/f_{ck,150cu}$) = 0.79 and based on mean strengths, ($f_{cm,150cy}/f_{cm,150cu}$) = 0.80.
(2) The ratio of 150mm cylinders/150mm cubes for C50/60 based on characteristic strengths, ($f_{ck,150cy}/f_{ck,150cu}$) = 0.82 and based on mean strengths, ($f_{cm,150cy}/f_{cm,150cu}$) = 0.83.
(3) The difference between 150mm cubes and 150mm cylinders specimens for C65/80 based on characteristic strengths, ($f_{ck,150cu} - f_{ck,150cy}$) = 12.9 MPa and based on mean strengths, ($f_{cm,150cu} - f_{cm,150cy}$) = 12.7 MPa.
(4) The ratio of 100mm cube/150mm cube for C32/40 based on characteristic strengths, ($f_{ck,100cu}/f_{ck,150cu}$) = 1.01(2) and based on mean strengths, ($f_{cm,100cu}/f_{cm,150cu}$) = 1.01(2).

Annex A

Extracted from Reference [3], Tam et al (2015)

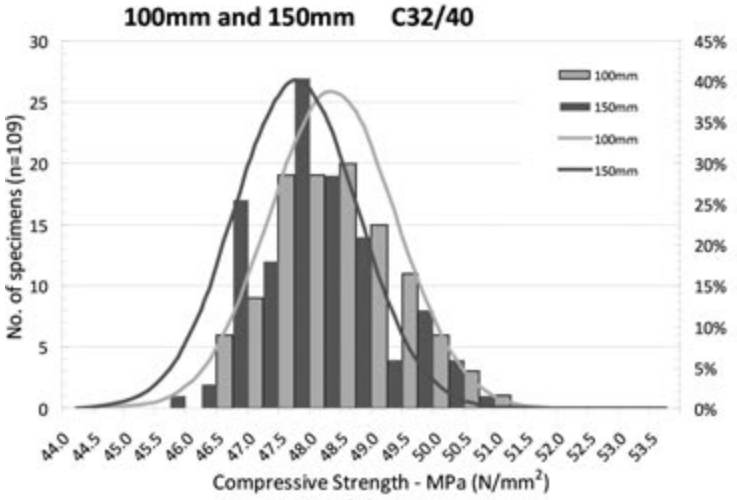
Table A.1 – Summary of test results

Strength class/property	C32/40			C50/60			C65/80		
Specimen	$f_{cu,100}$	$f_{cu,150}$	$f_{cyl,150}$	$f_{cu,100}$	$f_{cu,150}$	$f_{cyl,150}$	$f_{cu,100}$	$f_{cu,150}$	$f_{cy,150}$
Strength (3 specimens for each test)									
Mean, (f_m)	48.1	47.5	38.0	68.9	68.0	56.1	94.9	93.7	80.9
Maximum	52.5	51.3	41.6	75.4	73.9	60.6	111.1	104.3	96.3
Minimum	44.6	44.7	33.9	64.2	63.3	49.8	87.4	84.2	69.7
Range	7.9	6.6	7.7	11.2	10.6	10.8	23.4	20.1	26.6
Standard deviation (3 specimens for each test)									
Mean	0.91	0.90	0.94	1.23	1.19	1.12	1.74	1.77	1.86
Maximum	1.98	1.82	2.99	2.57	2.99	2.87	4.45	5.20	3.84
Minimum	0.17	0.16	0.21	0.13	0.21	0.21	0.29	0.29	0.54
Range	1.81	1.66	2.78	2.44	2.78	2.66	4.16	4.91	3.30
No. of data	109	109	105	109	109	109	105	105	105
Period of time (days)	151	151	145	206	206	206	155	155	155
Characteristic strength based on mean strength and mean standard deviation									
$f_{ck,cu}$	46.6	46.1		66.9	66.0		92.1	90.8	
$f_{ck,cyl}$			36.4			54.3			77.9
Ratio	Based on f_{ck}								
$(f_{ck,100cu})/(f_{ck,150cu})$	1.01 (1.012)			1.01 (1.013)			1.01 (1.014)		
$(f_{ck,150cyl})/(f_{ck,150cu})$			0.79 (0.791)			0.82 (0.822)			
$(f_{ck,150cu})-(f_{ck,150cyl})$									12.9
Ratio	Based on f_{cm}								
$(f_{cm,100cu})/(f_{cm,150cu})$	1.01 (1.012)			1.01 (1.013)			1.01 (1.013)		
$(f_{cm,150cyl})/(f_{cm,150cu})$			0.80 (0.799)			0.83 (0.825)			
$(f_{cm,150cu})-(f_{cm,150cyl})$									12.7
Ratio	Based on mean of individual ratios for each batch								
$(f_{ci,100cu})/(f_{ci,150cu})$	1.01 (1.013)			1.01 (1.014)			1.01 (1.013)		
$(f_{ci,150cyl})/(f_{ci,150cu})$			0.80 (0.800)			0.81 (0.814)			
$(f_{ci,150cu})-(f_{ci,150cyl})$									12.3

Annex B

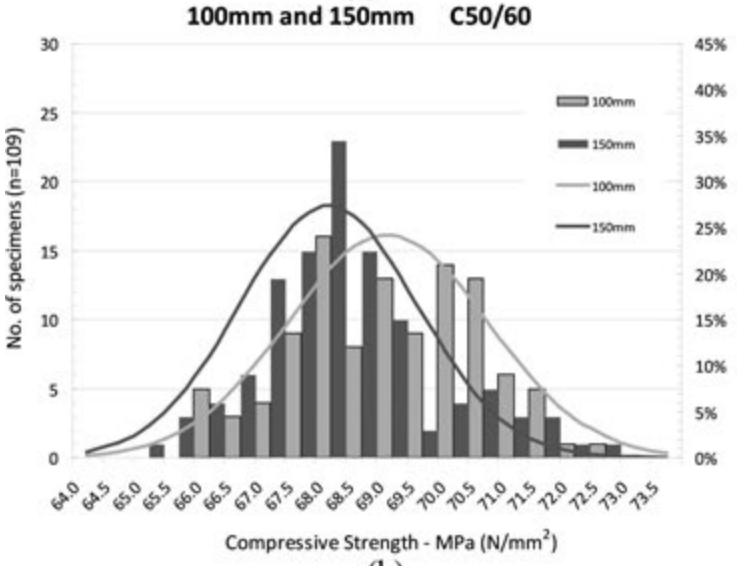
Extracted from Reference [3], Tam et al (2015)

Figure B.1 – Distribution of Cube Compressive Strength data 100 mm cubes and 150 mm cubes



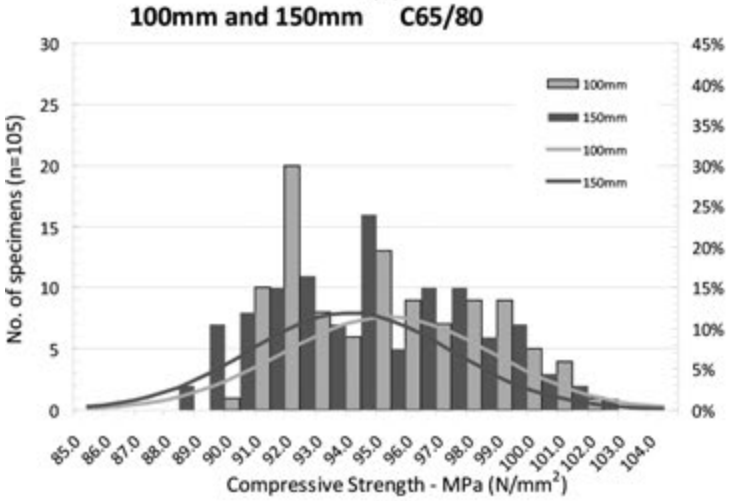
(a)

Cube	100 mm	150 mm
Mean	48.1 MPa	47.5 MPa
Std. Dev.	0.91 MPa	0.90 MPa
$f_{ck,cu}$	46.6 MPa	46.1 MPa
n	109	109



(b)

Cube	100 mm	150 mm
Mean	68.9 MPa	68.0 MPa
Std. Dev.	1.23 MPa	1.19 MPa
$f_{ck,cu}$	66.9 MPa	66.0 MPa
n	109	109



(c)

Cube	100 mm	150 mm
Mean	94.2 MPa	93.0 MPa
Std. Dev.	1.74 MPa	1.77 MPa
$f_{ck,cu}$	92.1 MPa	90.8 MPa
n	105	105

Figures B.1(a), 1(b) and 1 (c)

Table 2: Summary of Standard Deviations for Compressive Strength.

Specimen Type	Standard deviation – MPa											
	Mean			Maximum			Minimum			Range		
	$f_{cu,100}$	$f_{cu,150}$	$f_{cyl,150}$	$f_{cu,100}$	$f_{cu,150}$	$f_{cyl,150}$	$f_{cu,100}$	$f_{cu,150}$	$f_{cyl,150}$	$f_{cu,100}$	$f_{cu,150}$	$f_{cyl,150}$
C32/40	0.91	0.90	0.94	1.98	1.82	2.99	0.17	0.16	0.21	1.81	1.66	2.78
C50/60	1.23	1.19	1.12	2.57	2.99	2.87	0.13	0.21	0.21	2.44	2.78	2.66
C65/80	1.74	1.77	1.86	4.45	5.20	3.84	0.29	0.29	0.54	4.16	4.91	3.30

(5) The ratio of 100mm cube/150mm cube for C50/60 based on characteristic strengths, ($f_{ck,100cu}/f_{ck,150cu}$) = 1.01(3) and based on mean strengths ($f_{cm,100cu}/f_{cm,150cu}$) = 1.01(3).

(6) The ratio of 100mm cube/150mm cube for C65/80 based on characteristic strengths, ($f_{ck,100cu}/f_{ck,150cu}$) = 1.01(4) and based on mean strengths ($f_{cm,100cu}/f_{cm,150cu}$) = 1.01(3).

In addition, the test results in Annex A also provide the following findings:

(7) Standard deviations based on 3 specimens for each batch show generally an increasing trend with increase in compressive strength. The mean and the range of standard deviations for all the 3 types of specimens (100mm cube, 150mm cube and 150mm diameter by 300mm length cylinders) are as summarised above:

5.0 DISCUSSION

Based on the experimental test data obtained, they have indicated the following:

(1) The test results for the ratio of 150mm cylinders/150mm cubes are in agreement with those in Table 12 of BS EN 206 [1] with a nominal value of 0.80 for strength class up to C55/67. Above this strength class and up to C100/115, Table 12 of BS EN 206 [1] shows a constant difference of 15 MPa compared to 13 MPa based on test results for C65/80. Hence in designed concrete, the adoption of a difference of 15 MPa for cube compressive strength above that of cylinder compressive strength will be conservative.

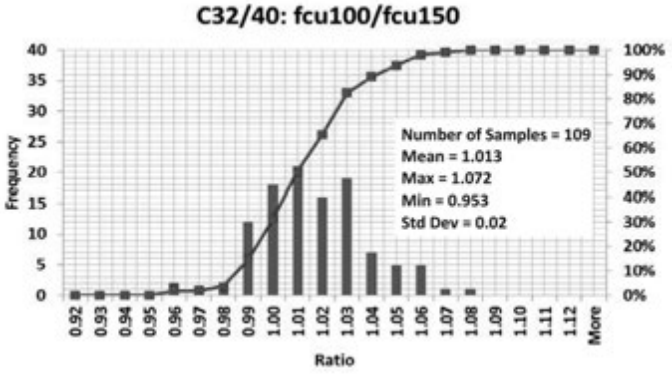
(2) The ratios of 100mm cube/150mm cube in all cases are only marginally above unity. This observation is based on 3 large populations of over 100 batches for each strength level studied. However, it may not be the case when comparison is based on small sample sizes of 2 or 3 specimens of each size generally adopted in conformity assessment. In order to illustrate the situation where the ratio may be higher or lower than unity, a more detailed analysis of the test results is presented in the following section.

(3) For each strength level, the 3 different types of test specimens show similar values of standard deviation for their mean, maximum and minimum standard deviations. However, all three values of standard deviations increase with increasing strength levels for all 3 types of specimens as well as their range of standard deviations.

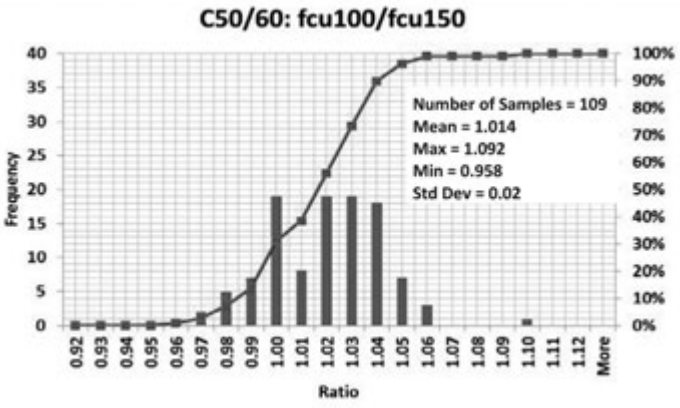
(4) For all 3 strength levels the coefficient of variation (standard deviation/mean) is approximately 2%. This is in agreement with the observation in relation to Figure 14.4 of Neville [13] that "for a constant degree of control, laboratory test data, as well as some results of actual site tests, have been shown to support the suggestion of a constant coefficient of variation for well-compacted concrete of different mix proportions with strengths higher than about 10 MPa". Although other data from construction sites, e.g. Figure 14.6 of Neville [14] show that "coefficient of variation is constant up to some limiting value of strength but, for higher strength, the standard deviation remains constant". Hence, the issue of constant standard deviation or constant coefficient of variation remains to be controversial.

5.1 Ratio of 100mm cube/150 mm cube

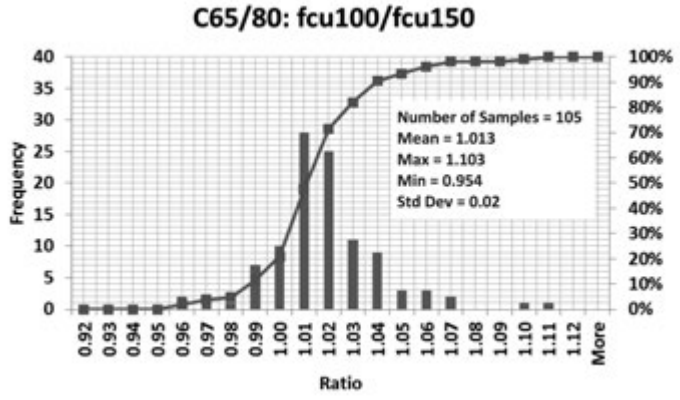
For each of the 3 strength levels, slightly over 100 batches were produced for which 3 specimens of 100mm cube and 3 specimens of 150mm cube were tested in each batch. The distribution of compressive strength at each strength level is presented in Annex B where the overlapping of the distribution of the two sizes of cubes is clearly shown in all the 3 strength levels tested. In order to provide a better understanding of the test data, the distribution of the ratio ($f_{ci,100cu}/f_{ci,150cu}$) for each batch of the slightly over 100 batches in each strength level is presented in Figure 1(a) for C32/40, Figure 1(b) for C50/60 and Figure 1(c) for C65/80.



(a) – C32/40



(b) – C50/60



(c) – C 65/80

Figure 1: Ratio of ($f_{ci,100cu}/f_{ci,150cu}$) at 3 different strength levels.

It can be seen from the Figure 1 that a wide range of the ratio is obtained for each strength level, from 0.95 to 1.07. Since around 100 batches (105 or 109) of each strength level were tested, the number of samples at a particular ratio represents approximately the percentage of test results that has been obtained. Firstly, the cumulative percentage of test results up to ratio of 1.0 in Figure 1(a) is 31%, for C32/40, in Figure 1(b) 31% for C50/60 and 21% in Figure 1(c) for C65/80. Hence, in general, 20% to 30% of cases in testing for cube compressive strength may show equal or higher strength for the 150mm cubes than corresponding 100mm cubes. This implies that test results for 1 in 5 to 1 in 6 batches, the expectation that the small size cube specimens should show a higher strength may not happen. Figure 12.17 of Neville [13], shown as Figure D.1 in Annex D, has suggested that the ratio for ($f_{ci,100cu}/f_{ci,150cu}$) is 1.04. For all the 3 strength levels, only about 10% of the ratio exceeded this ratio of 1.04. On the other hand, Figure 12.20 of Neville [13], representing data from several published studies leading to a generalized relationship between ratio of cube compressive strength of concrete specimens of a given set of dimensions, f_c, to cube compressive strength of a 6 inch (150 mm) cube, $f_{cu,6}$, as follows:

$f_c/f_{cu,6} = 0.56 + (0.697/[V/6hd + h/d)$, and for the case of 4 in. (100mm) cubes, the ratio = 0.98.

After adjusting for the case of a 6 inch (150 mm) cube giving a ratio of 0.91, the adjusted ratio = 1.07.

It can be expected that the experimental data from multiple sources for 100mm cubes may result in a range of values relative to 150mm cubes as shown in Figure 12.20 of Neville [13], but with variations in the ratio around (1.0±0.2). This may imply that the difference in measured compressive strength, if any, between 100mm and 150mm cubes is generally about 10% and of minor significance in practice. In particularly for small sample size adopted for assessment in site practice, the ratio can be either above or below unity as shown in the test data presented. The recommendation in clause 12.2 of BS 8500-2 [9] or its Singapore equivalent SS 544-2 [12], to consider the assessment of 100mm cubes with the same criteria for 150mm cubes is supported by the large populations of over 100 batches of test data for each of the 3 strength levels.

5.2 Relationship Between Cylinder and Cube Compressive Strength

The difference in measured compressive strength between 150mm diameter by 300mm length cylinder specimens and 150mm cubes is not due the difference in shape but the difference in aspect ratio (length/lateral dimension). The nominal aspect ratio (h/d) of a standard cylinder = 2, but that of a standard cube = 1. Due to this difference and hence the influence of end restraint effect on the concrete specimen has resulted in two different mode of failure. As described by Neville [13], a "complex system of stress is developed between the end surfaces of the concrete specimen and the adjacent steel platens of the testing machine". The induced bi-axial confining stress depends on the elastic properties (modulus of elasticity and Poisson's ratio) of both the steel platen and the strength level of concrete specimen and hence its modulus of elasticity. The modulus of elasticity of concrete increases with its compressive strength which results in a lower intensity of the induced bi-axial confining stress. The influence of end restraint may also extend to a shorter distance from the interface than the suggested value of $(\sqrt{3})d/2$ by Neville [13]. A constant difference of 15 MPa between cube compressive strength and cylinder compressive strength in Table 12 of BS EN 206: [1] for strength class of C60/75 and above leading to the ratio of ($f_{ck,150cu}/f_{ck,150cyl}$) decreasing from nominally 1.25 for strength class up to C55/67 to 1.15 for C100/115. From the experimental data, for C32/40, the mean ratio = 0.80 (51% of results), for C50/60, mean ratio = 0.83 (0.82 at 39%, 0.83 at 64%) and for C65/80, the mean ratio = 0.86 (50 at 0.85). There is a tendency for the ratio to increase with strength levels. In addition, both the minimum value and the maximum ratio also increased with strength level, 0.78 to 0.84 for C32/40, 0.79 to 0.87 for C50/60 and 0.81 to 0.95 for C65/80. For C65/80, the nominal difference of 15 MPa between 150mm cubes and 150mm diameter by 300mm length cylinders is indicated in Table 12 of BS EN 206 [1]. The range of this difference obtained ranges from 5 MPa to 19 MPa, mean = 13 MPa (47%) and 64% of results up to 15 MPa. However, unlike experience with strength classes up to C55/67, there is much less published data on the relationship between cube compressive strength and cylinder compressive strength for C60/75 and above. It is prudent to test of both types of specimens during the stage of initial tests in the development of high strength concrete to gain more data on this relationship, particularly with locally available materials for concrete production.

The study by Wong (2013) [8] in Hong Kong recommended a constant factor of 0.80 for cube strength of 80 MPa and above. Hence for cube strengths of 80, 90 and 100 MPa, the corresponding cylinder strengths are 64, 72 and 80 MPa showing

a difference of 16, 18 and 20 MPa respectively. These differences are higher than the nominal value of 15 MPa in EN 206 and may require design target strength based on cubes to be more conservative and hence higher economic impact in production.

6.0 SUSTAINABILITY IN CONCRETE STRENGTH ASSESSMENT

Significant volume of concrete is used in the assessment of compressive strength of concrete for initial/trial mix testing, production conformity during production as well as samples taken on site at the time of delivery for conformity assessment in relation to the project specification. After testing the specimens have to be disposed as waste, at best sent to aggregate recycling plants. Hence, a reduction in the volume of concrete involved in such testing is a step forward towards a more sustainable concrete industry.

6.1 Specimen size

The adoption of the smaller size cube specimens has significant effect on the sustainable use of concrete, as the volume of each 150mm cube is more than sufficient to make 3 numbers of 100mm cubes. This smaller volume of concrete needed for routine testing in conformity assessment conserves materials resources for concrete and reduces the volume of waste for storage and later disposal after testing. In addition, curing capacity of existing facilities is able to cater for 3 times more test samples and time for testing and energy for loading are also reduced to achieve the same failure stress, besides easier handling of test specimens leading to better productivity and/or resource savings in the test laboratory. This green practice has been successfully implemented by Singapore's Housing & Development Board (HDB) since 2007 and provides strong evidence for the local concrete industry to adopt this practice, a forward step towards a more sustainable concrete industry. It may be of interest to note that in the first edition of Properties of Concrete, Neville [14], on the section of "specimen size and aggregate size" the issue of recommended value for the ratio of the minimum dimension of the test specimen to the maximum aggregate size was stated as follows: "BS 1881: 1952 prescribes a test cube not smaller than 4 in. when ¾ in. aggregate is used, i.e. a ratio of 5-1/3, but 6 in. cubes may be used with 1½ in. aggregate. A.S.T.M. Standard C192-57 limits the ratio of the diameter of the cylinder to the maximum aggregate size to 3, and the U.S. Bureau of Reclamation to 4. A value of between 3 and 4 is generally accepted as satisfactory". Hence, it is convenient in practice for one size of cubes to cater for both 20mm and 40mm maximum aggregate size rather than due to technical requirement that 150mm cubes are commonly specified in evaluation of compressive strength.

6.2 Concrete Specification

Currently, it is a common practice to specify for 3 rounds of successful initial/trail mix testing of a given concrete strength for assessment before concrete is accepted for delivery to a project. This is routinely conducted for commonly specified concrete to strength classes of C25/30 to C40/50 at slump range of 75mm to 150mm even the RMC plant has been producing these strength classes on a regular basis for several years. This practice is still continued although in Singapore all structural grade concretes are now produced under certification (certificate of conformity) by accredited RMC certification bodies (CB's) of Singapore Accreditation Council (SAC), since 1 October 2010 mandated by BCA [15]. This move is in support of the new approach to replace the practice of initial/trail mix testing with "certified concretes" each with their performance guaranteed by the RMC producer for which the particular concretes are under regular production conformity evaluation within the plant and the data for which are subject to verification by a CB under the SAC's Certification Scheme for RMC. An example of an RMC production control data for C32/40 over a period of 12 consecutive calendar months is present by Tam, *et. al,* [3] and reproduced in Annex C, Figure C.1. In addition, trial mixes that were conducted for projects for this same concrete for which their data are also plotted for the dates in which production control date were also available. It can be noted that the initial/trail mix test results are always higher than the mean strength of the production control data.

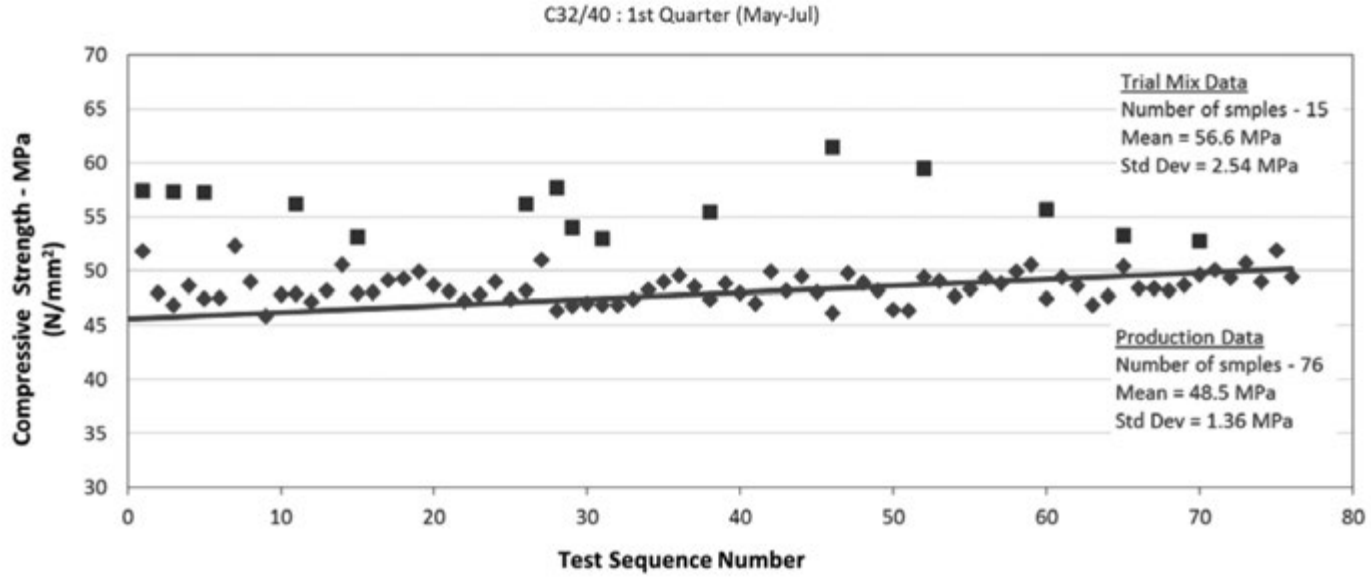


Figure 2: (a) Production control data - 1st Quarter.

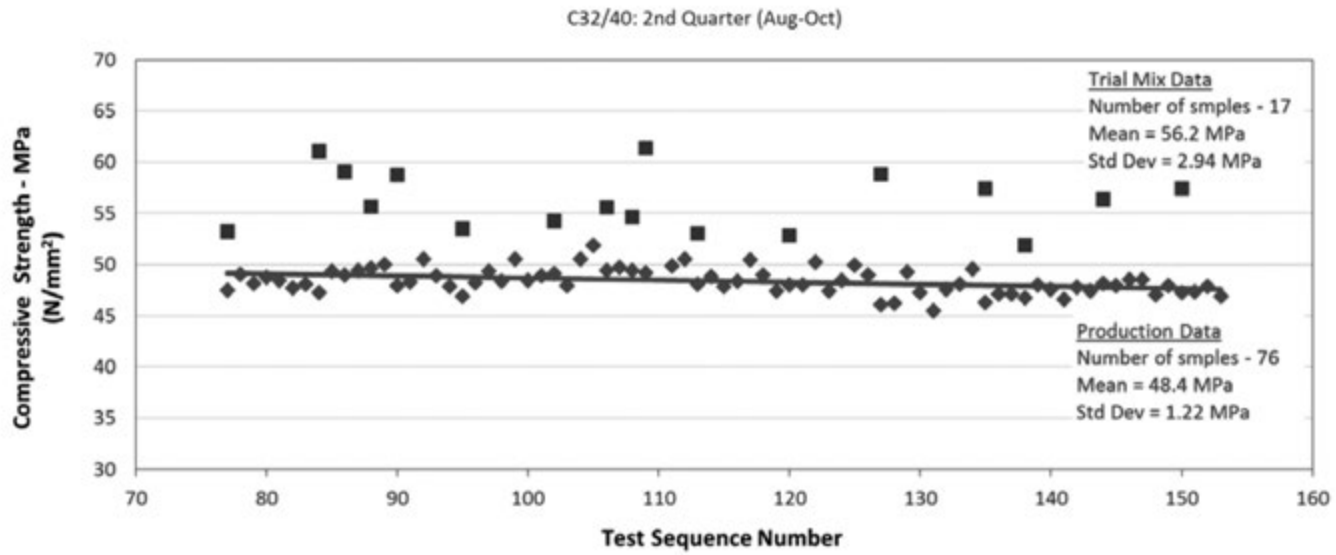


Figure 2: (b) Production control data - 2nd Quarter.

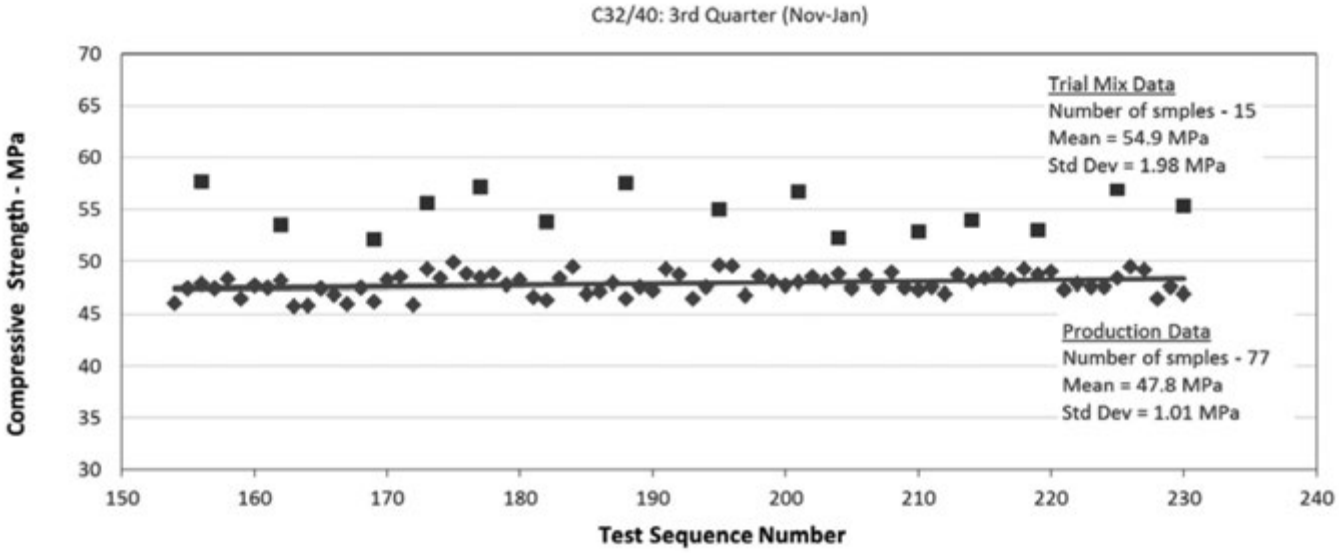


Figure 2: (c) Production control data - 3rd Quarter.

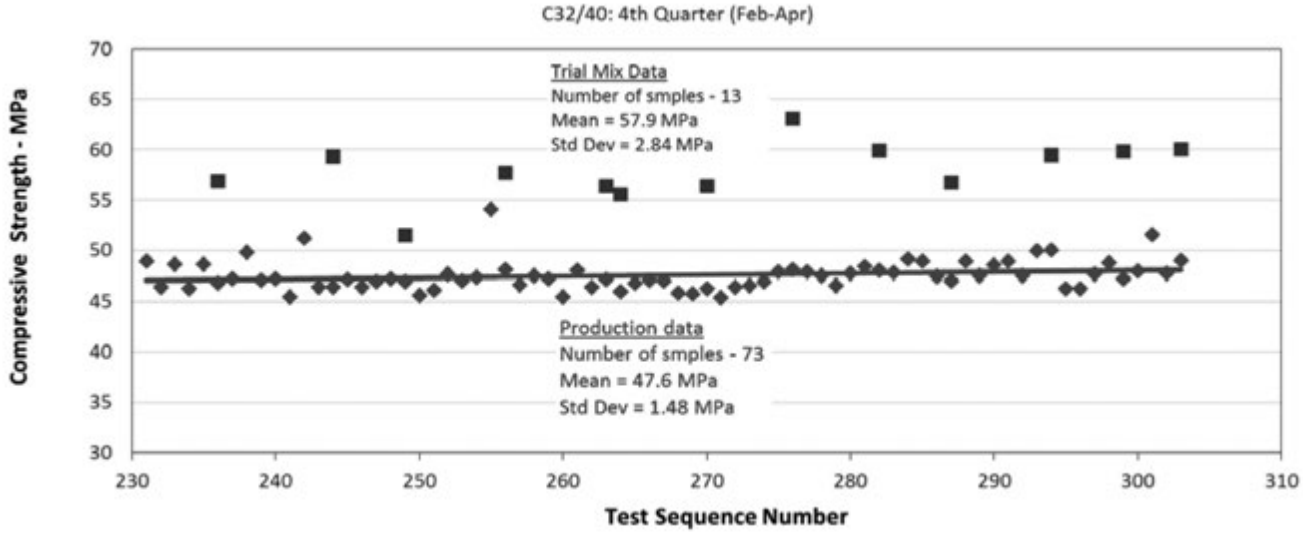


Figure 2: (d) Production control data - 4th Quarter.

Annex C
Extracted from Reference [3], Tam et. al, (2015).

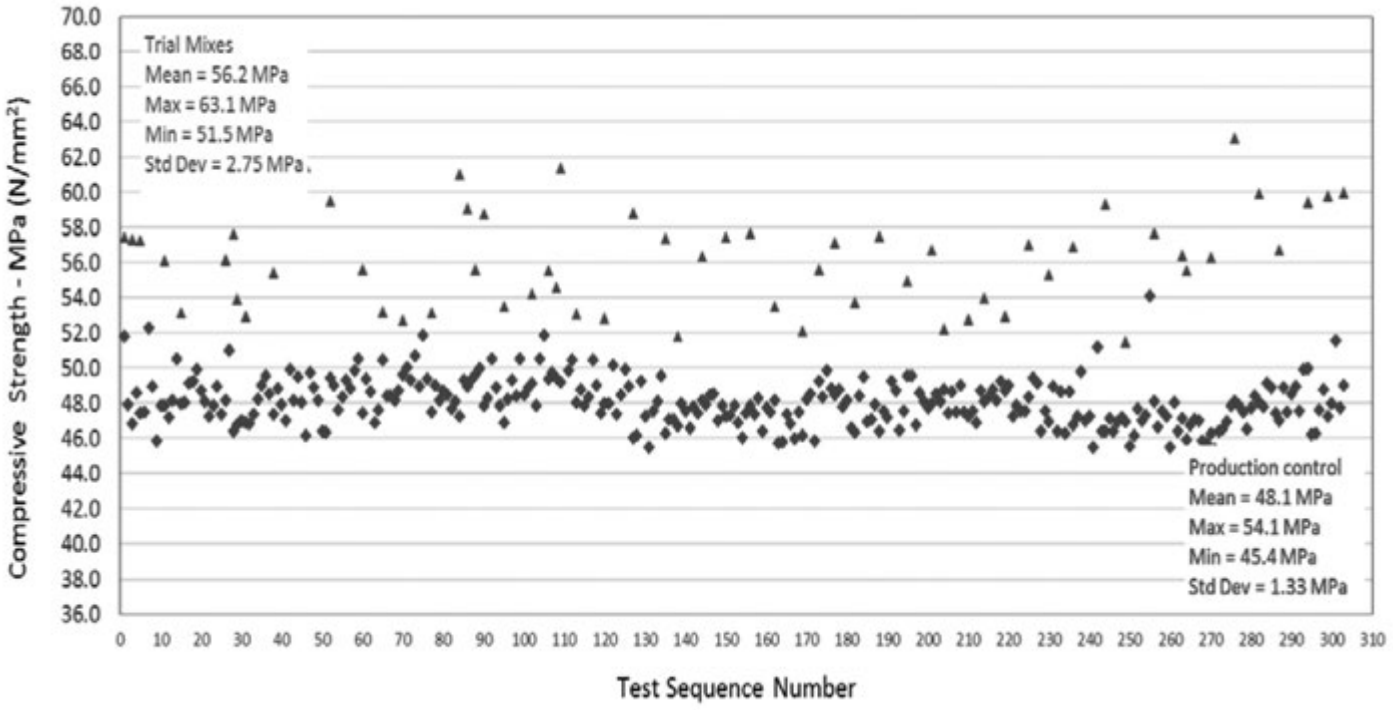


Figure C.1: RMC production control data for C32/40 over a period of 12 consecutive calendar months.

The Figures 2 (a) to 2 (d) suggest that a more reliable approach is to consider the recent production performance, e.g. latest 3 months production with at least 30 data sets, than conducting traditional trial mixes testing for potential future production quality. In order to illustrate this concept, the production control data shown in Figure C.1 (Annex C), is divided into each of the 4 consecutive calendar months as shown in Figure 2 (a), (b), (c) and (d) below. Although the design margin for C32/40 is 8 MPa, the production data is within the range of (40±4) MPa due to low variability of production (standard deviation < 2 MPa) in each.

The summary of number of samples, mean and standard deviations for each quarter is given in Table 3.

The Table 3 performance results indicate that the mean and standard deviation over a quarter of a year with at least 70 testing results can serve as a reliable basis for expecting acceptable characteristic strength for delivery of concrete in the following months. This is demonstrated by the performance in the succeeding quarters of production. The adoption of this alternate approach to conducting trial mixes of a certified concrete in production over several months enables a project to accept delivery of the selected concrete without delay time taken for confirmation by trial mixes. In addition, this leads to better productivity and/or savings in both manpower and materials resources at the initial stage of a project in concrete construction. Such situation is practiced in a precast concrete plant based on continuing production performance of a concrete without the need for any occasional trial mixes to verify the same concrete for continuing production.

6.3 Impact on sustainability

Based on the above findings, it can be seen that firstly the volume of test specimens needed for conformity testing can be reduced by 2/3 when 100mm cubes are adopted in place of 150mm cubes. The volume of test specimens to be disposed after testing is also reduced by this factor. Existing curing capacity is effectively increase by a factor of 3. The time required for loading of test specimens is also reduced besides ease of handling resulting in higher productivity in the testing laboratory.

By adopting certification of designed concrete, the practice of requiring satisfactory trial mixes to be conducted before concrete can be delivered to each project site for the same designed concrete produced by the same RMC plant can be limited only to new designed concrete for which initial tests

Table 3: Summary of analysis for 4 consecutive calendar months.

Duration	Number of samples	Mean (MPa)	Maximum (MPa)	Minimum (MPa)	Range (MPa)	Standard deviation (MPa)
1st Quarter (May-Jul)	76	48.5	52.3	45.9	6.4	1.36
2nd Quarter (Aug-Oct)	76	48.4	51.9	45.5	6.4	1.22
3rd Quarter (Nov-Jan)	77	47.8	49.9	45.8	4.1	1.01
4th Quarter (Feb-Apr)	73	47.6	54.1	45.4	8.7	1.48

are performed for conformity of specified requirements before production. Significant savings in cost in staff time and resources are achieved besides increasing productivity on site.

7.0 CONCLUDING REMARKS

Only 3 strength levels up to C65/80 have been tested and until data from more concretes above C60/75 are available, the findings lead to the following observations:

a. 100mm cubes provide similar measured compressive strength as 150mm with only a very small difference which is of little practical significance in conformity assessment, particularly when small sample size of two or three specimens taken on site at delivery.
b. Strength ratio in compression between 150mm cubes and 150mm diameter by 300mm length cylinders tends to decrease with increasing compressive strength levels supporting the commonly accepted value of 1.25 up to C50/60 to 1.15 at C100/115 based on a constant difference of 15 MPa as provided in BS EN 206 [1].
c. Difference in compressive strength between 150mm cubes and 150mm diameter by 300mm length cylinders for C65/80 is about the same as the nominal value of 15 MPa in BS EN 206 [1] for strength classes of C60/75 and above.
d. The use of 100mm cubes for production conformity testing instead of current practice of specifying only 150mm cubes as well as for identity testing of site samples is a significant reduction in concrete volume for preparing test samples as well as the amount of waste disposal of tested specimens to promote sustainable concrete construction.
e. The replacement of current practice of trial mixes for each new project with certification of RMC production reduces both time and resources before the start of delivery and hence overall productivity on construction sites.

The production performance data over a calendar year for C32/40 together with the occasional trial mixes carried out over the same period for various projects illustrate its reliability to provide the alternate approach to the need for trial mixes when a certified concrete is in good conformity control over a continuous period of months with at least 70 test results.

The adoption of 100mm cubes for conformity evaluation of compressive strength and the alternate approach of production performance data in place of trial mixes is recommended as a step towards promoting a more sustainable concrete construction. ■

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PROFILES

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Load Distribution Behaviour of Bored Pile in Various Soil Formation: Rock Socket in Limestone, Schist and Sandstone

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ABSTRACT

Rock socketed bored pile is a solution when the load from the structure is very high and/or accessible bearing surface has an inadequate bearing capacity. The study is based on instrumented bored pile socketing into different types of rock namely. limestone, schist and sandstone at three sites. The result for three (3) test piles namely PTP1, UTP-1 and TP2 shows most of the load are resisted by friction rather than end bearing at the pile working load. The load apportioned to end bearing at higher loads varies for the three test piles. Comparison of observed mobilised skin friction in the rocks with empirical methods indicates that prediction values from Williams and Pells [1] over design for two out of the three test piles and that by Hovarth [2] are under design for two out of the three test piles.

Keywords: Empirical Methods, Instrumented Bored Pile, Rock Socket, Shaft Resistance.

1.0 INTRODUCTION

Pile foundations are used to support heavily loaded structure such as high rise buildings and bridges. Bored piles are commonly used in Malaysia due to its low noise, low vibration and flexibility of sizes to suit different loading conditions and subsoil conditions.

Rock socketed bored pile is a solution when the load from the structure is very high and/or accessible bearing surface has an inadequate bearing capacity. It may be necessary to drill a shaft into the underlying rock and construct a socketed pile. The support provided by socketed bored pile comes from the shear strength around the shaft and the end bearing at the toe of the pile. Many researchers have investigated the behavior of rock socketed bored pile and relate the uniaxial compressive strength (UCS) of intact rock surrounding the pile to the shaft resistance of the pile without considering the rock mass quality (Rosenberg and Journeaux, 1976) [3].

Pile testing is a fundamental part of the pile foundation design. A pile load test is normally carried out to assess the geotechnical capacity of piles in the foundation system and as a tool to check the integrity of constructed pile and prediction of foundation settlements. In design, the concern is over what portion of the capacity is obtained at the pile toe and what is the shaft resistance in the specific soil layers. Therefore, when the purpose of the test is to provide data for design of a piled foundation then the pile must be instrumented in order to determine the load transfer (resistance distribution) such as shown in Figure 1.

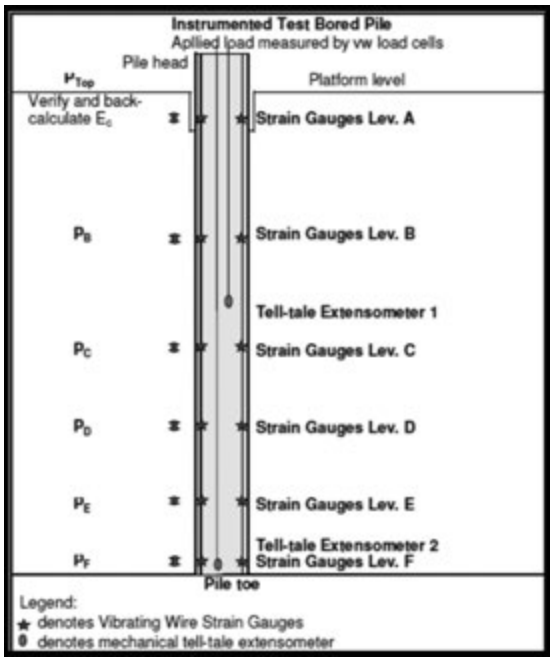

Figure 1: Schematic of Pile Instrumentation.

The objectives of this study are: -

- To study the behaviour of pile settlement under applied load.
- To determine the bearing capacity of pile and its apportionment into end bearing and shaft friction.
- To compare the behaviour of piles socketing into different type of rocks.

The study is based on case study of three (3) instrumented test bored piles at three actual developments. Data was collected to analyse and compare the behaviour of test pile socketing into different type of rocks in Malaysia. Vibrating wire strain gauges were installed in the test piles to reveal the load transfer behaviour along the pile. Extensometer was installed in test bored piles to observe the pile structural shortening but it is outside the scope of this paper.

1.1 Geotechnical Capacity of Bored Piles

The design of bored pile is normally based on the results of Standard Penetration Test (SPT-N) conducted in the borehole. In designing the pile, the empirical approach of unit skin resistance (f_s) and unit base resistance (f_b) is taken as:

$$f_s = K_s \text{ x SPT-N (in kPa)} \quad (1)$$

$$f_b = K_b \text{ x SPT-N (in kPa)} \quad (2)$$

Where K_s is shaft resistance coefficient and K_b is base resistance coefficient which varies according to soil type.

In current practice, these empirical formulas have been widely used for pile capacity calculation. Both the friction resistance and end bearing resistance are considered in design with an overall factor of safety 2.0 and 3.0 respectively. The design is an estimate thus it is important to understand the actual mobilisation of skin friction and end bearing with the pile movement. The data obtained from the instrumented static load test results can be used to verify the designed piled and the true load transfer behaviour of the bored piles can be observed.

Bored pile socketed in rocks can be expected to have higher pile capacity due to the higher unit friction resistance between the pile and the rock. Table 1 summarizes the typical design socket friction values for various rock formations in Malaysia.

Table 1: Summary of Rock Socket Unit Friction Design Values.

Rock Formation	Allowable Rock Socket Unit Friction	Source
Limestone	300 kPa for RQD < 25% 600 kPa for RQD = 25% to 70% 1000 kPa for RQD > 70% The above design values are subjected to 0.05 x minimum of (q_{uc}, f_{cu}) whichever is smaller.	Neoh [4]
Sandstone	0.10 x q_{uc}	Thorne [5]
Shale	0.05 x q_{uc}	Thorne [5]

Various other researchers have also developed more systematic approaches in rock socket design [1,3,6]. The following expression is used to compute the rock socket unit friction with consideration of the strength of intact rock and the rock mass effect due to the discontinuities.

$$F_s = \alpha \text{ x } \beta \text{ x } q_{uc} \quad (3)$$

Where q_{uc} is the unconfined compressive strength of intact rock

α is the reduction factor with respect to q_{uc} (Figure 2).

β is the reduction factor with respect to the rock mass effect (Figure 3).

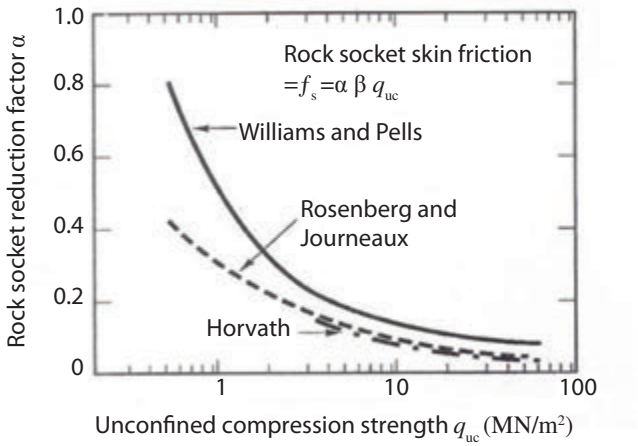

Figure 2: Rock Socket Reduction Factor, α versus Unconfined Compressive Strength. (after Tomlinson, [6]).

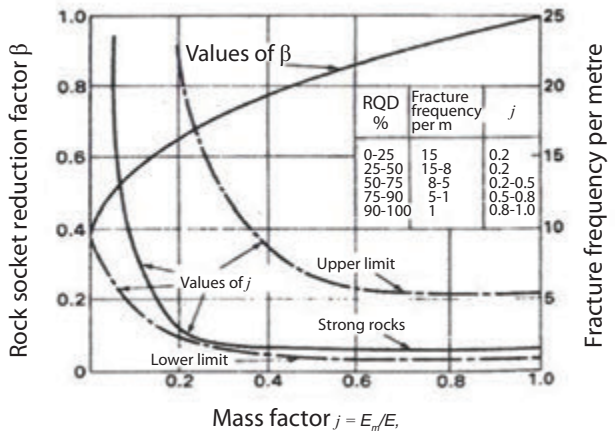

Figure 3: Rock Socket Reduction Factor, β, versus Rock Mass Discontinuity (after Tomlinson [6]).

2.0 SUBSOIL STRATA, PILE INSTALLATION AND INSTRUMENTATION

2.1 Site A

The site is located at Ipoh, Perak. The area is underlain by an extensive limestone bedrock formation namely the Kinta Limestone. The limestone bedrock rises above the alluvial plains to form limestone hills with steep to vertical slopes. The subsoil strata based on nearest borehole is shown in Figure 4.

PTP1 test pile of 1050mm diameter and 8.8m long is socketed into moderately strong limestone bedrock at depth 4.3m to 8.3m (4.0m length). Based on the nearest borehole data on site, the Rock Quality Designation (RQD) of the rock is between 54%

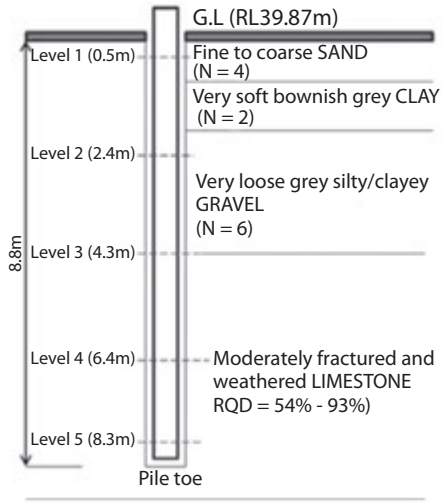

Figure 4: Subsoil Strata and Pile Instrumentation Levels for PTP1.

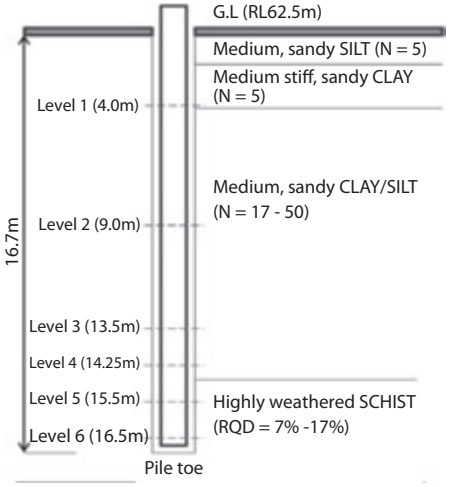

Figure 5: Subsoil Strata and Pile Instrumentation Levels for UTP-1.

to 93 % within Unconfined Compressive Strength (UCS) of 35 MPa.

Twenty (20) nos. of Geokon vibrating wire strain gauges (VWSGs) were installed in the test pile to measure strain at nominated locations From Level 1 to Level 5. Each level consists of four (4) nos. of VSWG. There were five (5) nos. of tell-tale extensometers installed at the five (5) levels (one for each level), corresponding to Level 1 to Level 5 from ground respectively. A polystyrene foam soft toe was installed at the base to eliminate end bearing contribution since end bearing was not considered in the design geotechnical capacity due to uncertainty of proper base cleaning during construction.

Maintain Load Test (MLT) was proposed to be carried out in three (3) cycles: first cycle with working load of 750tonnes, second cycle was twice working load of 1500tonnes and the third cycle was 2250tonnes. However, the third cycle was not completed as the pile failed during the step of loading from 1875tonnes (2.5 x working load) to 1950tonnes (2.6 x working load).

2.2 Site B

The proposed development is situated at Mukim Setapak, Daerah Gombak, Selangor where the geological formation consists of schist, phylite slate and sandstone. Soil profile based on nearest borehole is shown in Figure 5.

The test pile UTP-1 was a 1000mm diameter bored pile with embedded length of 16.7m below ground level. The pile was debonded by pre-augering the soil surrounding the the pile up to 13.5m depth. The debonding was conducted in order to observe the load distribution within the socketed depth when no friction resistance is provided by the upper soil.

At depth of 13.5m to 16.5m, the test pile UTP-1 was socketed 3.0m into schist rock. The nearest borehole data shows that RQD of the rock falls between 7% to 17% and the average UCS is 17 MPa.

Pile instrumentation consisted of twenty-four (24) nos. VWSG at six (6) different levels and three (3) nos. of telltale extensometers.

Loading were carried out in three (3) cycles: first cycle with working load of 650tonnes, during the second cycle the maximum load was 1300tonnes (2.0 x working load) and during the third cycle the maximum load was 1950tonnes (3.0 x working load).

2.3 Site C

The site is located at Kuala Lumpur and is underlain by Kenny Hill Formation which is a sequence of clastic sedimentary rocks consisting of interbedded shale, mudstone and sandstones The Kenny Hill material is basically a completely decomposed rock and generally sandy SILT soil. Based on the nearest borehole at the site, the ground profile is shown in Figure 6.

TP2 test pile (900mm diameter) is socketed into sandstone bedrock at depth 10.0m to 15.0m (5.0m length). Based on rock coring and compressive test results from nearest borehole, the Rock Quality Designation (RQD) of the rock falls between 29.3% to 44.6% with UCS of 20 MPa.

Pile instrumentation consisted of twenty-eight (28) nos. VWSG at seven (7) different levels and four (4) nos. of telltale extensometers.

The load test was carried out in four (4) cycles: first cycle with working load of 6000kN, second cycle with maximum load of 7500kN (1.25 x working load). During the third cycle the maximum load was 9000kN (1.5 x working load) and during the fourth cycle the maximum load was 15,000kN (2.5 x working load).

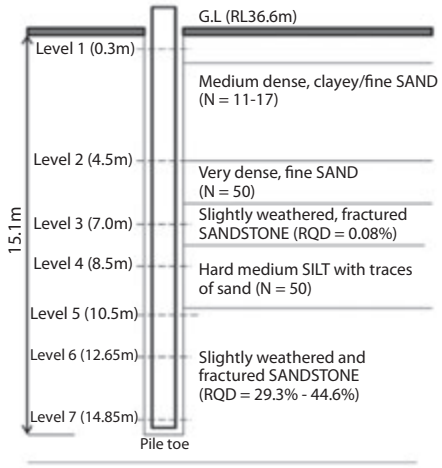

Figure 6: Subsoil Strata and Pile Instrumentation Levels for TP2.

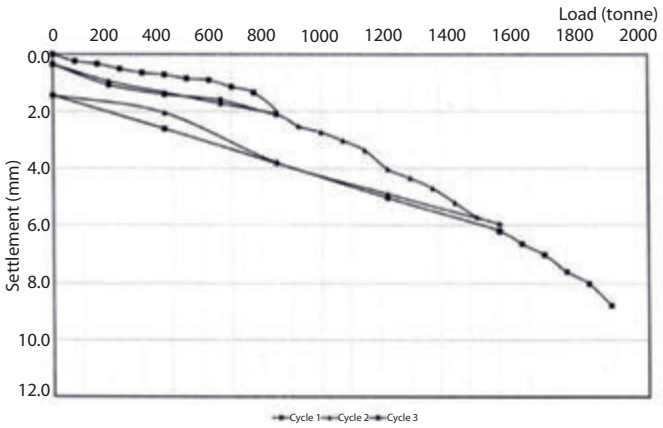

Figure 7: Load Settlement Curves for PTP1.

3.0 RESULTS AND DISCUSSION

3.1 Site A

Figure 7 shows the Load Settlement Behaviour of the Pile and Table 2 summarises the settlement behaviour.

Table 2: Settlement of Pile Top for PTP1.

Loading Cycle	Max Load (tons)	Max Settlement (mm)	Residual Settlement (mm)	Elastic Rebound (%)
1st	750.00	2.10	0.40	80.95
2nd	1500.00	6.00	1.40	76.67
3rd	1875.00	8.80	N/A	N/A

It can be seen that maximum pile top settlement was recorded at 8.80mm during the 3rd loading cycle when the maximum load of 1875tons was applied. It must be noted that the full program of loading steps for 3rd loading cycle could not be completed as the pile failed during the step of loading from 1875tons (2.5 x Working Load) to 1950tons (2.6 x Working load).

The results show that the settlement was 2.1mm (0.2% of the pile diameter) at pile working load and 6.0mm (0.57% of the pile diameter) at two times working load. It also shows that at working load the pile gives an elastic rebound of 80.95%.

Readings from the strain gauges were analysed to determine the load distribution behaviour and the mobilised unit friction and unit end bearing during the sequence of loading. The results are shown in Table 3 and Figure 8.

It is noted that the rock socket start from depth 4.3m to 8.3m. Table 3 tabulates the load distribution along the pile shaft and pile base. It shows that only about 3tons to 6tons (0.32% to 0.4%) of the applied load was carried by end bearing throughout the whole range of applied load. The small amount of load at the pile base is probably due to the installation of polystyrene foam soft toe. The soft toe was installed as to minimise the load interference from the pile base (the end bearing was neglected in design consideration).

Table 3: Summary of Load Distribution for PTP1.

Pile depth (m)	Load taken by surrounding soil (tons)		
	1.0 x Working Load (750 tons)	2.0 x Working Load (1500 tons)	2.5 x Working Load (1875 tons)
0-0.5	0.0	0.0	0.0
0.5-2.4	112.0	164.0	195.0
2.4-4.3	23.0	70.0	248.0
4.3-6.4	487.0	1123.0	1265.0
6.4-8.3	125.0	138.0	162.0
End Bearing	3.0	5.0	6.0

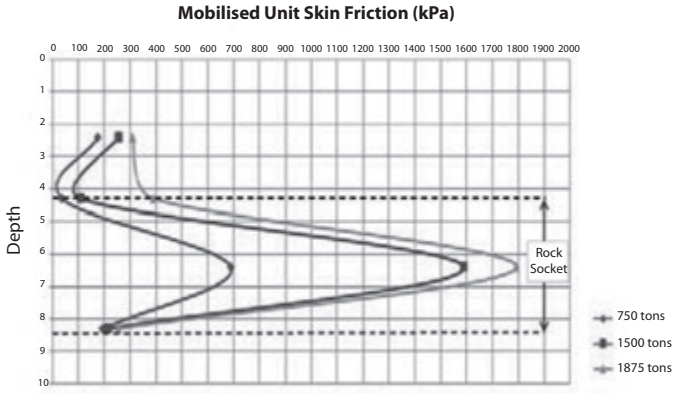

Figure 8: Mobilised Unit Skin Friction for PTP1.

Based on Figure 8, the chart shows that the maximum mobilised skin friction are at Level 3 to Level 4 with maximum value of 689kPa (1st loading cycle), 1590kPa (2nd loading cycle) and 1790kPa (3rd loading Cycle). Since, this pile was tested to fail, the maximum mobilised skin friction of 1790kPa is considered as ultimate value for the limestone of fair to good rock quality.

3.2 Site B

The Load Settlement Behaviour of the Test Pile is shown in Figure 9 and the settlement is summarised in Table 4.

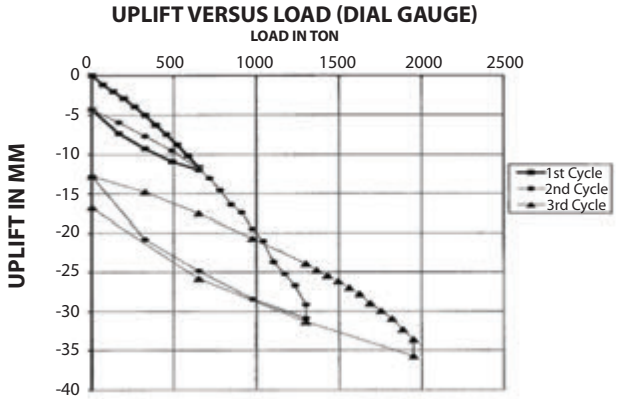

Figure 9: Load Movement Curves for UTP-1.

Table 4: Settlement of pile top for UTP-1.

Loading Cycle	Max Load (tons)	Max Settlement (mm)	Residual Settlement (mm)	Elastic Rebound (%)
1st	650.00	11.95	4.31	63.92
2nd	1300.00	30.88	12.75	58.71
3rd	1950.00	35.77	16.77	53.12

The test pile UTP-1 did not fail after loading up to three (3) times the working load. The pile top settlement was recorded at 11.95mm (1.2% of pile diameter), 30.88mm (3.1% of pile

diameter) and 35.77mm (3.6% of pile diameter) at test load of 650tons, 1300tons and 1950tons respectively.

Table 5: Summary of Load Distribution for UTP1.

Pile depth (m)	Load taken by surrounding soil (tons)		
	1.0 x Working Load (650 tons)	2.0 x Working Load (1300 tons)	3.0 x Working Load (1950 tons)
0-4.0	8.6	8.7	8.3
4.0-9.0	29.2	15.7	15.3
9.0-13.5	27.2	26.8	27.5
13.5-14.25	278.8	292.7	292.7
14.25-15.5	247.6	521.6	521.5
15.5-16.5	39.6	312.1	422.5
End Bearing	18.9	122.4	662.2

The load transfer distribution and mobilised skin friction and end bearing is shown in Table 5 and Figure 10 respectively.

It is noted that the rock socket start from depth 13.5m to 16.5m. Based on Table 5, it can be deduced that from 0m to 13.5m depth of pile, only a small amount of applied load which are 8.3tons to 27.5tons was distributed to the surrounding soil, due to the debonded section. Therefore, smaller load was recorded at depth up to 13.5m. At depth 13.5m and below, most of the load was taken by the rock socket.

It also shows that some percentage of loads was distributed to the pile base. The load distribution for end bearing was 18.9tons (2.9%) at normal working load, 122.4tons (9.2%) at two times working load and 662.2tons (34.0%) at three times working load. The trends of linearly increasing load transfer along the shaft and base resistance during maximum loading (three times of working load) indicates that ultimate shaft and base resistance were not fully mobilised at working load and that a settlement of 3.6% pile diameter was required to mobilised the end bearing to a significant value. This justify the practice of ignoring the end bearing in geotechnical capacity estimation.

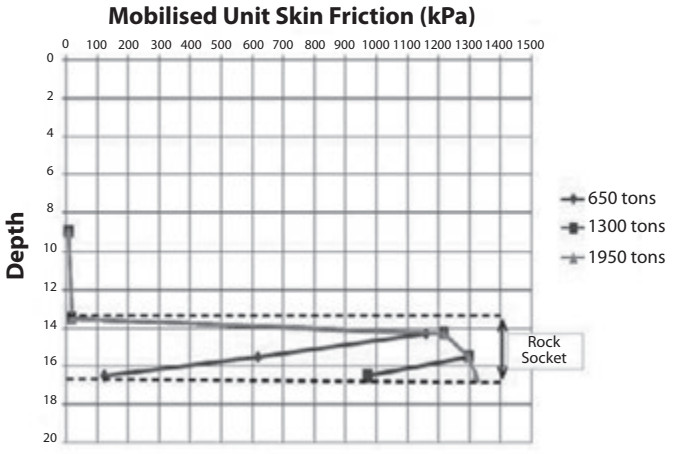

Figure 10: Mobilised Unit Skin Friction for UTP-1.

Figure 10 shows that the maximum mobilised skin friction is at 3rd loading cycle with maximum value of 1220kPa (Level 3 to Level 4), 1300kPa (Level 4 to Level 5) and 1320kPa (Level 5 to Level 6). It can be suggested that a value of 1300kPa may be considered as ultimate unit friction value for this very poor quality schist.

3.3 Site C

Readings from the strain gauges were analysed to determine the load distribution behavior and the mobilised unit friction and unit end bearing during the sequence of loading. The results are shown in Table 6 and Figure 11.

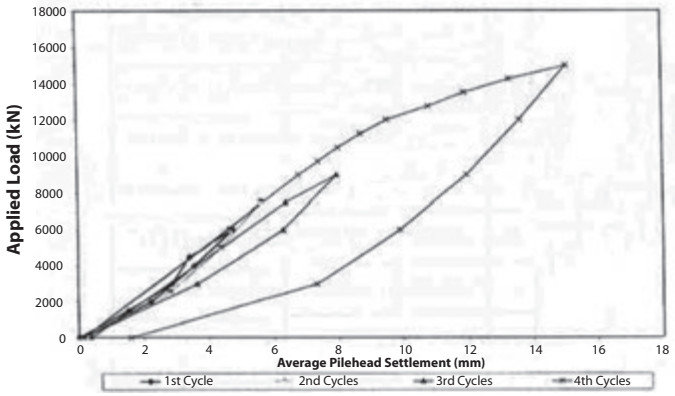

Figure 11: Load Movement Curves for TP2.

Table 6: Settlement of pile top for TP2.

Loading Cycle	Max Load (tons)	Max Settlement (mm)	Residual Settlement (mm)	Elastic Rebound (%)
1st	600.00	4.74	0.14	97.05
2nd	750.00	5.62	0.37	93.42
3rd	900.00	7.96	0.13	98.37
4th	1500.00	15.07	1.58	89.52

The pile top displacement (settlement) were recorded at 4.74mm (0.5% of diameter pile) at test load 600tons, 5.62mm (0.6% of pile diameter) at applied load 750 tons, 7.96mm (0.88% of pile diameter) and 15.07mm (16.7% of the pile diameter) at applied load of 900tons and 1500tons respectively. Table 6 also shows the higher percentage of elastic rebound is between 89.52% (applied load of 1500tons) to 97.05% at applied load of 600tons. The test pile TP2 was loaded up to 2.5 times working load and did not fail and the settlement was only 15.07mm. It indicates that the pile still can behave well if imposed load is more than that.

Pile Depth (m)	Load taken by surrounding soil (tons)			
	1.0 x Working Load (600 tons)	1.25 x Working Load (750 tons)	1.5 x Working Load (900 tons)	2.5 x Working Load (1500 tons)
0-0.3	0.0	0.0	0.0	0.0
0.3-4.5	68.2	28.0	39.2	99.8
4.5-7.0	100.3	185.3	206.4	356.3
7.0-8.5	71.7	108.6	125.9	199.3
8.5-10.5	118.3	185.4	217.0	330.5
10.5-12.65	205.2	227.0	264.5	435.9
12.65-14.85	17.8	12.4	13.7	28.6
End Bearing	18.5	28.5	33.3	49.6

It is noted that rock socket is from 10.5 to 14.85m depth. As shown in the table, only a small portion of applied loads about 18.5tons to 49.6tons (3.1% to 3.8%) were transferred to the pile base and most of the load was distributed to the surrounding soil and rock socket shaft. The ultimate shaft and base resistance were not fully mobilised at the pile working load as the load transfer along the shaft and the base still shows the trend of linearly increasing during maximum loading (2.5 times working load).

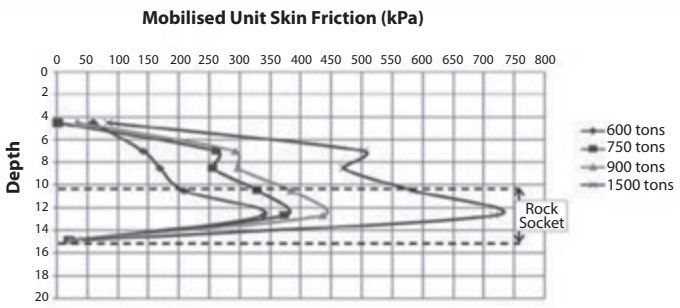

Figure 12: Distribution of Mobilised Unit Skin Friction for TP2.

Figure 12 shows that the maximum mobilised skin friction is Level 5 to Level 6 with maximum value of 338.0kPa (at 600tons), 373.0kPa (at 750tons), 435.0 kPa (at 900tons) and 717.0 kPa (at 1500tons). Based on these results it may be taken that the ultimate unit friction in the poor quality sandstone is around 700kPa.

3.4 Prediction of Ultimate Unit Skin Friction in Rock Socket

Various researchers have proposed numbers of empirical and semi-empirical design methods on rock socketed piles, most of them compute the ultimate unit skin friction based on average unconfined compressive strength (UCS) of the rock mass and applying reduction and correlation factors. In order to examine the applicability of these methods, their prediction values of ultimate unit skin friction in the rock socket are compared with the observed maximum unit skin friction values obtained from Site A (PTP1), Site B (UTP-1) and Site C (TP2).

Therefore, in order to determine the prediction value of each researchers noted in Table 8, the average Rock Quality Designation (RQD) and UCS from the nearest borehole data were used in the estimation. The value of Rock Socket Reduction Factor, α and Rock Socket Correlation Factor, β can be obtained from Figure 2 and Figure 3 respectively. Table 8 present the summary of the comparison between predictions with the observed maximum value of rock socket friction on site.

Table 8: Prediction of Ultimate Shaft Friction by Various Researchers.

Pile	RQD %	UCS (kPa)	Ultimate Unit Skin Friction, f_s (kPa)			Observed maximum unit shaft friction
			Rosenberg and Journeaux (1)	Horvath (2)	Williams and Pells (3)	
PTP1	54-93	35,000	1505.0	1204.0	2408.0	1790.0
UTP-1	7-17	17,000	884.0	773.5	1326.0	1320.0
TP2	29-45	20,000	1040.0	780.0	1430.0	717.0

(1), (2), (3) f_s is calculated from $f_s = \alpha \times \beta \times q_{uc}$ where α, β from respective charts

It can be deduced that for test pile PTP1, the observed maximum unit shaft friction of 1790 kPa was an ultimate resistance since the pile is loaded to failure. Rosenberg and Journeaux [3] method gives the nearest ultimate value of 1505 kPa.

With regard to UTP-1, method proposed by Williams and Pells [1] gave the nearest accurate ultimate skin friction of 1326.0kPa compared to observed value of 1320.0kPa. The other methods, gave quite lower value compared to the observed skin friction. Since the estimated skin friction is lower than the actual friction of the in situ rock, it can be assumed that those predictions by Rosenberg and Journeaux [3], and Horvath [2] methods are under design of skin friction, fs.

Whilst for TP2, Williams and Pells [1] method gave an ultimate value of 1430.0kPa, and Rosenberg and Journeaux [3] method gave a value of 1040.0kPa. Method by Horvath [2] gave the nearest ultimate skin friction (780kPa) compared to observed shaft friction value (717kPa).

It can be seen that for each test pile certain method over design, under design or predict closely the observed values.

It is noted that the ratio of ultimate mobilised unit friction for TP2 (sandstone) over maximum load is lower than those for UTP-1 (schist formation) even though the RQD and UCS is much better than those for the schist formation. It indicates that type of rock affects the friction at the shaft interface.

4.0 CONCLUSION

The performance of test pile PTP1, UTP-1 and TP2 shall be deemed to have satisfied the requirements of the JKR Standard Specification for pile head settlement where at design working load, the total settlement of the test piles did not exceed 12.5mm and when loaded to twice working load, the total settlement of the pile head did not exceed 38mm or 10% of the pile diameter whichever is lower. After removal of the designed working load, the residual settlement did not exceed 6.5mm and after removal of the test load at twice working load, the residual settlement did not exceed 20mm.

The test piles mainly utilised the frictional resistance to support the design capacity of pile with factor of safety at least 2.0. End bearing resistance is only mobilised from two to three times working load.

In most bored pile design, base resistance of bored pile is usually ignored due to uncertainties of base cleaning. The results in this study show that even if base cleaning were properly done very little end bearing resistance is utilised at pile working load. This could be a technical justification to disregard end bearing resistance for bored pile.

Comparison of rock skin friction from various methods with the observed values on site shows that lower value than actual skin friction is considered as under design. While the higher value than actual skin friction is considered as over design. This means that prediction of ultimate values from Horvath [2] is most conservative and that by William and Pells [1] is most liberal for those three (3) test piles.

The trend of mobilised skin friction and end bearing is similar for all test piles indicating that it is not affected by type of geological formation however the magnitude is dependent on the type of rock, strength and quality. ■

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PROFILES

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